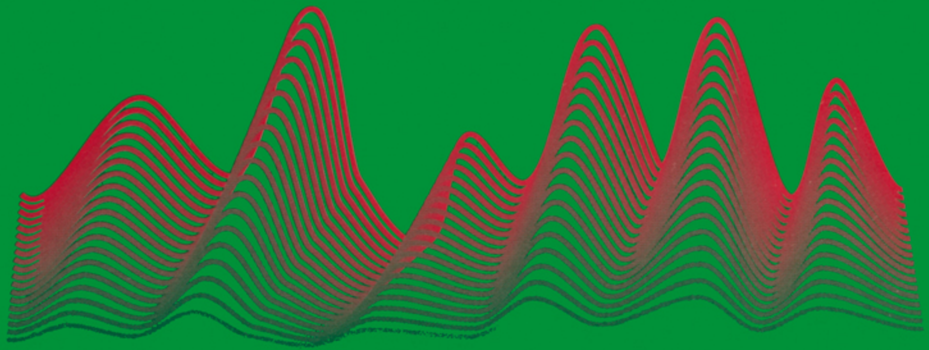
Post-Earthquake Rehabilitation and Reconstruction



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POST-EARTHQUAKE REHABILITATION AND RECONSTRUCTION

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CONTENTS

PREFACE

US and PRC have a long record of bilateral symposia/workshops on multiple hazard mitigation, particularly in earthquake engineering. As a result, a number of cooperative research projects have been successfully conducted by leading scholars in both countries. Based on the US/PRC Protocol Annex III Program, the Symposium/Workshop on Post-Earthquake Rehabilitation and Reconstruction was held May 24–27 in Kunming, Yunnan, China. Key features are listed below.

Objectives

- to discuss the lessons learned from recent seismic disasters in both countries
- to assess the current state of knowledge and practice in urban rebuilding after earthquakes and to identify frontal joint research opportunities for advancement of the current state of knowledge
- to stimulate future cooperative research on and development of subjects of common need and importance
- to evaluate the results of PRC's post-earthquake reconstruction program and their practical implications in the US
- to continue to build the long-term bilateral scientific relationship existing between academic and practicing communities

Topic areas

- damage assessment of structures after earthquakes
- lessons of post-earthquake recovery, rehabilitation and reconstruction, including public policy, land-use options, urban planning, and design
- issues in and examples of decision-making, and implementation of rehabilitation and reconstruction plans and policies
- repair, strengthening, retrofit and control of structures and lifeline systems
- post-earthquake socioeconomic problems covering issues of relief and recovery
- human and organizational behavior during emergency response, and strategies for improvement
- real-time monitoring of earthquake response and damage

US delegates

Professor Al Ang, University of California at Irvine
Dr Patricia Bolton, Battelle Human Affairs Research Center, Seattle
Dr Franklin Y. Cheng (Chairman), Curators' Professor of Civil Engineering, University of Missouri-Rolla
Professor Hal Cochrane, Director, Hazards Assessment Lab, Colorado State University
Dr Andrea Dargush, Assistant Director for Research and Education, National Center for Earthquake Engineering Research, State University of New York at Buffalo
Professor Barry Goodno, Georgia Institute of Technology

Professor Jim Jirsa, Janet S. Cockrell Centennial Chair in Engineering, University of Texas at Austin, Ferguson Structural Engineering Lab
Professor George Lee, Director, National Center for Earthquake Engineering Research, State University of New York at Buffalo, New York
Dr S.C. Liu, Program Director, Structural Systems, National Science Foundation, Arlington, Virginia
Professor Joanne Nigg, Director, Disaster Research Center, University of Delaware, Newark
Ms Maryann Phipps, Principal with H.J. Degenkolb Associates, San Francisco
Mr Li-Hong Sheng, Senior Bridge Engineer with California Department of Transportation
Professor M.P. Singh, Virginia Polytechnic Institute & State University
Professor Leon Wang, Old Dominion University

Chinese delegates

Deng Chong-Jian, Zhong Bei Harbour Engineering Supervision Corporation, Tianjin
Gou Fangming, Yunnan Construction Bureau
Hou Minzhong, Tangshan Earthquake Relief Office, Hebei Province
Hu Shu-sen, Anti-Seismic Office, Hebei Province
Lai Xianxiang, Office of Earthquake Resistance, Construction Committee of Sichuan Province
Li Jie, Zhengzhou Institute of Technology, Henan
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Liu Jiajun, Construction Bureau of Yunnan Province
Liu Wenyan, Earthquake Resistance & Vibration Research Center, SINOPEC, Beijing
Liu Zigang, Earthquake Resistance Office, Ministry of Construction, Beijing
Men Kai, Qingdao Institute of Architecture Engineering, Shandong
Shen Bin, Yunnan Urban-Rural Planning Design & Research Institute
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Zhang Mianzheng, Uighur Autonomous Regional Anti-Seismic Office, Xingjiang
Zhang Shuanglai, Anti-Seismic Office of Qinghuangdao, Hebei Province
Zhang Wanchang, Daqing Oil Field Architecture Design & Research Institute, Heilongjiang
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Zhao Zhilin, Earthquake Resistance Office, Construction Commission, Hebei Province
Zhou Fulin, South China Construction University, Guangzhou
Zhu Bolong, College of Structural Engineering, Tongji University, Shanghai

International guest observers

Professor F. Casciati, University of Pavia, Italy
Professor Chang-Koon Choi, Korea Advanced Institute of Technology
Mr James J. Gallo, Balvac Inc., Buffalo, New York
Professor Teizo Fujiwara, Disaster Prevention Research Institute, Kyoto University
Professor Yoshiaki Kawata, Disaster Prevention Research Institute, Kyoto University
Dr Akira Mita, Chief Research Engineer, Shimizu Corporation, Japan
Dr Miha Tomaževič, National Building & Civil Engineering Institute, Slovenia
Dr Makoto Watabe, Senior Managing Director, Shimizu Corporation, Japan
Dr David C.T. Yeh, Evasun Engineering Inc. and VIM Inc., Taiwan
Professor Chung-Bang Yun, Korea Advanced Institute of Technology
Instrumental in the success of this event were a number of individuals serving on committees or with organizations.

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Dr Franklin Y. Cheng (Chairman), Curators' Professor of Civil Engineering, University of Missouri-Rolla
Dr George C. Lee, Director, National Center for Earthquake Engineering Research, State University of New York at Buffalo
Dr S.C. Liu, Program Director, Structural Systems, National Science Foundation, Arlington, Virginia
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Zhang Zaiyong, Tongji University
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The symposium/workshop was jointly sponsored by the US National Foundation and the Chinese Ministry of Construction. Technical support includes Ms Elizabeth Farrell at the University of Missouri-Rolla, Ms Liu Wen and Mr An Dong at the Institute of Earthquake Engineering, China Academy of Building Research, and Dr James Milne, Publishing Editor, Engineering & Technology, Elsevier Science Ltd., Oxford, England. As organizers of the symposium/workshop and editors of the proceedings, we gratefully acknowledge their support.

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WANG YAYONG
Director
Institute of Earthquake Engineering
China Academy of Building Research

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Group picture after opening ceremony

Planning meeting in Beijing

Franklin Y. Cheng, U.S. Delegation Chair and Co-Editor

Wang Yayong, Co-Editor

Ding Shaoxiang, Chinese Delegation Chair

Welcome remarks at opening ceremony by Dr. S.C. Liu
National Science Foundation, USA

Welcome remarks at opening ceremony by Mr. Dou Yide
Ministry of Construction, PRC

Symposium/workshop in session

Colleagues old and new
(*left to right* Frank Cheng, Al Ang, David Yeh, Bolong Zu and Ye Yaoxian)

Dali cultural exhibition of performing arts

Tour of locale known for unique geology and earthquakes

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RESOLUTIONS OF SINO-US SYMPOSIUM/WORKSHOP ON POST-EARTHQUAKE REHABILITATION AND RECONSTRUCTION

The Sino-US Joint Symposium/Workshop on Post-Earthquake Rehabilitation and Reconstruction was held 24-27 May 1995 in Kunming, Yunnan, China. Participants conducted fruitful scientific exchanges in damage assessment of structures; lessons of post-earthquake recovery; repair, strengthening, retrofit and control of structures; socioeconomic problems; emergency response; and real-time monitoring of earthquake response. At the conclusion of the symposium/workshop, participants unanimously passed the following resolutions.

1. Collaborative research between Chinese and US researchers requires cross-flow of research results as published in journals and conference proceedings. While Chinese investigators have relatively easy access to international publications, Chinese research papers are often inaccessible to US researchers because they are published either in Chinese or in journals not widely circulated in the West. Therefore it is resolved that:

   a. Chinese researchers should be encouraged to publish in English and in international journals

   b. More Chinese-sponsored journals and conference proceedings should be published in the West

   c. Effective measures should be devised to disseminate research results to users

   d. Chinese researchers should be encouraged to serve on editorial boards and as editors of international journals

2. Based on the success of this event as acknowledged by all participants, it is resolved that a joint symposium/workshop be continued at intervals of two or three years.

3. Topics of mutual interest for further investigation, preferably as joint research activities, are listed by group as follows.

GROUP I DAMAGE ASSESSMENT OF STRUCTURES

It is proposed that further cooperative research be done on the following topics.

1. Vulnerability Assessment of Existing Cities and Localities

   a. Research on the development of realistic disaster scenarios

   b. Action guidelines for policy makers as to what options they are likely to have and what they should do

2. Development of Decision-making Process for Recovery and Reconstruction

3. Damage Level and Vulnerability Assessment in Emergency Scenarios (quick response rehabilitation guidelines)

4. Systematic Decision-making Process for Different Levels of Upgrading Deficient Structures

5. Code Upgrade

   a. Performance-based codes

b. Cost-based codes

c. Risk-based codes

d. Codes incorporating latest developments in fuzzy and expert systems

6. Continuation of Research

Basic topics:

a. Development and use of improved materials, structural systems, design procedures

b. Development of active protective systems (use of complementary strengths of China and USA and possibly other countries)

c. Design with protective systems. It is known that protective systems reduce structural response and make a structure safer. However, it is not quite clear how to obtain information such as how much and what type of protective system will reduce the response to a desired level.

Specialized topics:

d. Residual strength of existing and damaged structures

e. Summary of lessons learned from past earthquakes (features of various seismic events)

f. Study of secondary hazards caused by earthquakes

g. Seismic vulnerability of petrochemical facilities

h. Data on nonstructural system failures

Co-chairs: M.P. Singh and Yeh Yeoxian
Participants: F.Y. Cheng, M. Phipps, M.P. Singh, Liu Wenyan, Yeh Yeoxian

GROUP II REPAIR, STRENGTHENING, AND RETROFIT

The following topics were discussed. It was agreed that they represent areas of mutual interest and hold promise for cooperative research efforts that would benefit both countries.

1. Methods for post-earthquake repair and strengthening of structures

   a. Efficient, cost-effective construction techniques with emphasis on masonry and reinforced concrete structures

   b. Rehabilitation of nonstructural elements, utility and service systems as well as equipment inside buildings

   c. Development of demonstration projects which involve large-scale or field studies and innovative approaches, e.g. structural control techniques

2. Guidelines for post-earthquake evaluation, repair and strengthening

   a. Cost-effectiveness of different systems.

   b. Selection of most appropriate system including consideration of life-cycle costs

3. Development of procedures for nonlinear, multi-dimensional, time-history analyses of structures in the post-earthquake damage stage

   a. Pre-earthquake instrumentation/post-earthquake analysis of records and building response

   b. Experimental research or field studies designed to provide additional data since existing algorithms for behavior of structural elements may be inadequate

c. Use of Northridge experience (1994) and studies of building response to develop analytic procedures

Co-chairs: Zhu Bolong and James O. Jirsa
Participants: A-H.S. Ang, Ho Chang, Ding Shao-Xiang, Barry J. Goodno, Men Kai, Qiu Maode, Li-Hong Sheng, David C.T. Yeh, Yang Qing-Sheng

GROUP III RECOVERY AND RECONSTRUCTION

The following general topics were suggested as a basis for developing specific future cooperative research projects. Since these topics are considered equally important, no greater priority is given to one over any other.

Engineering, social and interdisciplinary studies of data-base development on strength/damage assessment, methodologies, standards, and cost-benefit for recovery and reconstruction of:

a. small to medium-size cities

b. petroleum facilities

c. port and harbor facilities

d. rural housing

Co-chairs: Leon R.L. Wang and Zhang Weiyue
Participants: Two US delegates, 15 Chinese delegates and one Japanese observer

GROUP IV SOCIO-ECONOMIC AND RESPONSE/RELIEF ISSUES

Criteria for project consideration

This group was asked to identify and prioritize a set of key research projects on the social aspects of response to and recovery from earthquake disasters that could be carried out in a collaborative manner by Chinese and American researchers. Since a long list of potential research interests was identified by group members, some method was needed to identify those that had the greatest likelihood of being successfully completed.

Recommended projects should meet the following three criteria.

1. The project must be "do-able." This criterion requires that pertinent data are available (or access to data is possible or data can be collected). Current methodologies and/or research strategies exist to allow the project to progress satisfactorily.

2. The project must be worth doing. Not only is this project attractive to potential researchers on its scientific merit but it can also yield results that would improve the understanding of recovery and the ability to reconstruct a safer environment. Potential "users" of research results are identifiable in both countries.

3. Collaborative research partners can be identified as interested in or at work on this problem. This criterion indicates that a favorable academic climate already exists for research on a given topic. This condition was identified as very important to the overall successful completion of collaborative projects.

Collaborative research project topics

The following four topics were identified as priority research projects related to the socio-economic aspects of response to and recovery from an earthquake disaster:

1. Comparison of methodologies to estimate losses from earthquakes of different sizes. There are currently several methodologies under development in both countries to try to project earthquake losses for different purposes: for example, rapid assessments of losses after an event; regional loss studies to provide a basis for national resources allocation; local area or community studies to guide local policy adoption for mitigation and preparedness. It would be informative and useful to see how these methodologies compare, both cross-societally and for various purposes.

2. Identification of methods to improve the performance of lifeline systems before an earthquake occurs. This topic was highly rated by the group given the function that lifeline systems play in limiting damage and loss in the immediate response period.

   While this topic may initially sound solely concerned with technical "fixes" for existing lifeline systems and facilities, it has a very important socio-economic aspect. Emphasis here would be on the adoption and implementation of technical solutions within various social and political contexts. Research concern would be to identify factors that facilitate or inhibit the use of various solutions, including technical competence required to retrofit a system, economic cost of the solution, socio-cultural acceptability of the solution, political feasibility of focusing on lifeline systems as opposed to other types of structures needing attention, etc.

3. Assessment of how well-prepared the public is to deal with earthquake-related problems. Concerns were raised about how the public would respond to actual events as well as to heightened probabilities that an earthquake would occur. This topic was divided into three sub-topic research problem areas.

   a. How prepared are publics in different seismic risk areas for an earthquake event? This includes an assessment of their knowledge of earthquake risk and hazards, preparedness and mitigation measures, and governmental action.

   b. What public education programs have been successful with respect to earthquake matters? This involves the identification and evaluation of public education programs.

   c. What social groups or organizations have been or can become involved in response and/or recovery activities? Here the focus is on the potential for involving social groups or formal organizations in response or recovery activities to enhance or augment local governmental response capabilities.

4. Evaluation of the role of insurance in mitigating earthquake hazards. Although China and the USA have taken different approaches to insuring private property/enterprises, there is a growing concern in both countries about the exposure of the insurance industry or the state to post-earthquake damage compensation. There is an hypothesis that insurance could be used to enhance private mitigation. However, this hypothesis needs rigorous empirical investigation to determine whether and under what conditions such enhancement might be possible.

Co-chairs: Joanne Nigg and Wang Ya'yong
Participants: Al H-S. Ang, Hal Cochrane, Andrea Dargush, Deng Xiao-Yun, Tang Langpeng, Zhang Mianming, Zhao Zhi-lin

BRIEFING ON REHABILITATION IN EARTHQUAKE DISASTER AREAS OF CHINA

LIU ZHIGANG

Earthquake Resistance Office, Ministry of Construction,
Beijing 100835 , PRC

ABSTRACT

Since the Xingtai earthquake in 1966, strong earthquakes have continuously occurred in Haicheng of Liaoning Province, Tangshan and Xingtai of Hebei Province, Daofu, Batang and Xiaojin of Sichuan Province, Lancang-Gengma of Yunnan Province and Mangai of Qinghai Province. Urban and rural construction were severely damaged. After the seismic shocks, under the leadership of our government, with the guidance of the administrative departments in charge at various levels, obvious achievements have been obtained in rehabilitation. In accordance with the practice of rehabilitation of earthquake disaster areas in our country, this article, for research and discussion, deals with policies, planning, model organization funds and experiences in reconstruction.

KEYWORDS

Earthquake; disaster; rehabilitation; reconstruction; planning.

POLICIES FOR REHABILITATION

After the Xingtai earthquake in 1966, our government, especially the late Premier Zhou Enlai, put forward a policy of "Self-reliance, arduous struggle, development of production and reconstruction of our homeland". Various earthquake disaster areas have formulated more detailed policies, in combination with actual conditions and earthquake disasters, politics, economy, history, nationalities, natural conditions, urban layout, function, weak points in earthquake resistance and urban planning, according to general policies. For example, the policy made after Xingtai earthquake in 1966, "Rebuilding at the original location, doing what is suitable to the location, benefiting production and facilitating people's lives"; the policy made after Tangshan earthquake in 1976, "Developing satellite cities and increasing small-scale cities and towns"; the policy made after the Lancang-Gengma earthquake in 1988, "Stressing the local point, doing things in order of urgency, adapting central policy to local situations (combination of systematic and regional management), and carrying out responsibilities of Assistant Director and County Director; achieving restoration first, then development according to one's ability". Practice reveals that these policies have played a guiding role in rehabilitation of earthquake disaster areas and made the work proceed smoothly.

PLANNING AND PRINCIPLES OF REHABILITATION

Rehabilitation should begin with formulation of general planning. Such planning should fully consider the conditions of resources and the prospects for development, and at the same time should make overall arrangements and implement them by stages. During implementation, principles of unified funds, unified design, unified construction, unified guidance and unified management should be upheld.

Cities and villages which already have a general plan should adopt the relevant parts of current planning, and should complement, adjust and revise their plan in accordance with the characteristics of seismic hazards to achieve a more comprehensive approach.

The scale, standard, method and stages for rehabilitation should take into consideration the limits of the state, the region, the collective body and the private person. Restoration of urban infrastructure should take the lead while restoration of urban lifelines closely related to the needs of people, houses and production facilities should also be given priority. The relationship between restoration and development, and between short-term and long-term goals should be carefully considered. Reconstruction of industries for building materials and energy as well as those key to the economy should take precedence in rehabilitation.

For ethnic minority areas, their customs should be respected in rehabilitation.

Reconstruction after earthquakes should be done at the original location; any change of location should be strictly controlled. Restoration at the original site should improve the current situation of higher population and higher building density, confused zoning of building function, and unfavorable seismic distribution of urban lifelines and land use. A matter of importance is location should be changed when it is superior to the original condition of water source, energy, transportation, telecommunication, seismic geology, engineering geology, hydrogeology and other natural disasters. Before a change of the location is considered, scientific proof that the new location is superior to the original one should be put forward.

Reconstruction should follow the procedures of capital construction, and all rehabilitation projects should have earthquake-resistant protection. Design strength is determined according to seismic design code of our country. When actual seismic intensity is higher than the intensity in the seismic zoning plan (1990) of our country, the seismic intensity of that area (or city) should be tested again. These test results would then serve as the basis for determining design strength in rehabilitation.

THREE MODELS FOR REHABILITATION

Rehabilitation at the Original Location

When earthquake damage to a city is not severe, there are still many houses and engineering facilities in need of strengthening, which could again be used after repair and maintenance. When the damage is so severe that repair is not possible or economical, the structures could be cleared and rebuilt at the original spot. This model is often used; for example, reconstruction after the Xingtai earthquake in 1966, Haicheng earthquake in 1975, Daofu earthquake in 1981, Lancang-Gengma earthquake in 1988 and Mangai earthquake in 1990 have all been at the original location.

Rehabilitation at Another Site

When the earthquake is quite serious, with the urban infrastructure basically destroyed and 90% of buildings severely damaged, reconstruction could be done at another site. Through scientific demonstration, it is first necessary to show that the new place is superior to the original one in the following: natural conditions (water source, energy, climate, ecological environment and land use) geological conditions (engineering geology, hydrogeology and seismic geology), development potential (available resources for development, transport, local materials, industrial foundation and infrastructure) and other conditions (politics, economy, science and technology, culture, social factors, psychological factors, investment effort, potential seismic hazard analysis). For example, reconstruction after the Wuqia earthquake followed this model.

Rehabilitation Partially at the Original Location and Partially at the New Site

When earthquake damage is rather severe, but urban infrastructures and buildings can still be used, a special policy may apply. This is to develop small urban centers, starting from actual conditions after an earthquake, when rehabilitation could partially be done at the original urban area, and partially at a new location, forming a satellite city. For example, after the Tangshan earthquake in 1976, rehabilitation followed this model. The newly built Tangshan was divided into three parts: the old urban area, the eastern mining area and the new Fengrun area, 25km apart from each other. In the Lubei District of the old urban area certain industries, like iron and steel, pottery and porcelain, electricity and machinery, as well as municipal agencies, remained as the political, economic and cultural centre. In the Lunan district of the old urban area, a scenic spot, was established as were some warehouses and small industries. In the eastern mining area, the Kailuan mine was the basis for rehabilitation. In the new Fengrun area (with newly-built living quarters to the east of Fengrun County) housing comprised the western part of the new Fengrun area and industry (including production of vehicles and textiles) on the eastern part.

ORGANIZATION AND FUNDS FOR REHABILITATION

Organization of rehabilitation should be handled by governments at various levels, corresponding to the situation such as the extent of earthquake damage and the size of cities. Detailed work should be done by Rehabilitation Headquarters which is in charge of organizational implementation and unified management. During implementation, it should periodically report to the people's government at the same level. Policy for solving major problems should be determined by the people's government at that level, after consultation with the experts and masses. rehabilitation Headquarters is headed by local administrative departments in charge of construction, and involves participation by departments of civil affairs, planning, materials, earthquake resistance and military region.

Funds for rehabilitation should take local finances as the main resource, supplemented by the State, should seek many sources, and should accept foreign and domestic donations. Management of funds should be coordinated to ensure a smooth rehabilitation process.

EXPERIENCE AND UNDERSTANDING

Through rehabilitation work after the Tangshan earthquake, Wuqia earthquake and Lancang-Gengma earthquake, our experience and understanding are as follows.

It is generally improper to carry out rehabilitation at another site due to increased time and expense. After the Lancang-Gengma earthquake, we faced the problem of whether to change locations for reconstruction. The Ministry of Construction assigned experts to carry out careful investigation and research on whether to move to Mengding County. They compared 18 factors, including conditions of geology, nature and development, in Lancang-Gengma versus Mengding County. As a result, Gengma County was rated 10 factors superior to Mengding County, 5 factors equal to each other and 3 factors inferior to Mengding County. This provided a rationale for not changing location. Scientific demonstration revealed that rehabilitation at the original place could save 240 million RMB, and reduce time by 2 years for the same scale of reconstruction. Finally the determination of reconstruction at the original place had been made by the provincial government. Practice has proved that this determination is correct.

In rehabilitation, damaged buildings should be salvaged whenever possible, i.e., repaired and strengthened. The eliminating of houses and engineering facilities should be strictly controlled. Before eliminating structures, a technical appraisal should be done and approval obtained from local departments in charge of construction.

Before rehabilitation, a feasible general plan should be compiled as the basis of details on functional classification, modification of built-up areas, environmental protection, disaster protection and short-term and long-term construction. In earthquake-struck areas, those enterprises, institutional agencies, organizations and private persons who engage in rehabilitation should be brought into a unified plan, unified management and unified implementation by Rehabilitation Headquarters.

In rehabilitation, various engineering design, strengthening and construction should take suitable measure of local conditions, draw on local resources, and make full use of local building material as well as salvageable material from destroyed houses, so as to decrease construction cost.

For rehabilitation, foreign funds should be absorbed as much as possible, funds raised from enterprises and institutions of various ownership and private persons should be encouraged, in order to alleviate the burden of the state.

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ASSESSMENT OF A BRIDGE COLLAPSE AND ITS DESIGN PARAMETERS FOR NORTHRIDGE EARTHQUAKE

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ABSTRACT

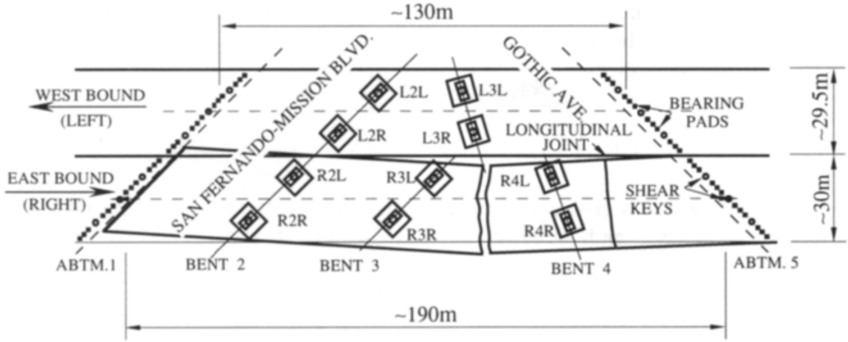
Collapse behavior of Mission-Gothic Undercrossing is investigated as a three-dimensional structural system with consideration of superstructure, column, and abutment conditions. Seismic input of vertical and horizontal ground motion for the study is modified from the record station at Sylmar County Hospital parking lot, about 18 kilometers from the epicenter, and Mission-Gothic Undercrossing, about 7.5 kilometers from the epicenter. Failure criteria of the bridge columns are developed by using nonlinear finite element analysis as well as biaxial interaction yield surface. Individual column's yield surface is then used in the bridge system study. Analytical results show bridge collapse behavior to be similar to that observed after the earthquake. The results indicate that, due to orientation of the columns connected to the bridge, force transmitted from bridge deck to columns is acting at a 45° from the strong axis of a cross-section which then causes a 13% reduction in column capacity. A circular flare for columns is recommended to avoid unequal column capacity under the lateral load. This study also indicates a need for additional stirrups to prevent spalling of concrete in the lower part of the flare. Unfavorable conditions in load combination for bridge columns will increase if the bridge structure is subjected to three-dimensional ground motion.

KEYWORDS

bridge structures; dynamic analysis; finite element; flared columns; ground motion; nonlinearity; Northridge earthquake; reinforced concrete; spiral; stiffness.

INTRODUCTION

During the January 17, 1994 Northridge earthquake, the Mission-Gothic Undercrossing bridge collapsed after severe damage. This structure features an unusual layout of bents, directional dependence of flared column stiffness, and an arrangement of stirrups in the lower part of the column's flare. These unique characteristics of the structure led to its damage and collapse under strong ground motion. Analysis was done with a computer program, IAI-NEABS (Imbsen & Associates, 1993) for which the bridge is modeled as a three-dimensional system. Superstructure of the bridge is divided into linear elements, while columns are modeled as nonlinear elements. Column yield surface includes interaction of biaxial bending for axial load and bending about x and y axis. Abutment conditions are also taken into account. Ground motion recorded at Sylmar County Hospital parking lot is modified to include the effect of distance between record station and bridge. Bridge columns are analyzed by using the computer program, NARCS (Lou and Cheng,



(a)

(b)

(c)

Fig. 1. Mission-Gothic Undercrossing (a) Dimensions of bridge structures; (b) Collapse of right bridge structure; (c) Failure of column L3L

1994) with eight-node isoparametric hexahedral nonlinear finite elements (Cheng and Lou, 1995; Cheng *et al.*, 1994). Comparisons between analytical results and observed phenomena are favorable. Recommendations to improve bridge performance are made.

DESCRIPTION OF STRUCTURE AND STRUCTURAL DAMAGES

Mission-Gothic Undercrossing is composed of two parallel bridge structures which carry the I-118 Simi Valley-San Fernando Freeway over the intersection of San Fernando-Mission Boulevard and Gothic Avenue in Los Angeles. It was designed in 1973 and built in 1976 (Moehle, 1994).

Two bridge structures, left and right, are separated by a longitudinal joint seal (Fig. 1a). The left bridge is approximately 130 m long and 29.5 m wide with three spans. The right one is a four-span bridge approximately 190 m long and 30 m wide. Both are supported by two-column bents and abutments.

A significant difference in length of the two bridges results from skews of the abutments, for they were designed to be parallel to San Fernando-Mission Boulevard and Gothic Avenue. Thus abutments 1 and 5 skew approximately 45° from the longitudinal direction of the bridges. On the left bridge are skews of 45° clockwise for bent 2, and 11° counter clockwise for bent 3. On the right bridge are skews of 45° clockwise for bent 2 and 3, and 11° counterclockwise for bent 4. Angles are measured from the east-west direction as shown in Fig. 1a.

Superstructure of both bridges is a cast-in-place post-tensioned box girder with a structural depth of 2.3 m, and sits on twelve elastomeric bearing pads (457 mm × 457 mm × 76 mm) at abutments. Each bridge end consists of three square shear keys (762 mm in length and width) which are embedded 380 mm into the abutment seat, allowing the superstructure to move 150 mm in the bridge's longitudinal direction.

Column height varies from 6.8 m to 7.5 m by measuring from the bottom soffit to the top of footing. At each bent the lower part of column is octagonal in 1,829 mm diameter with 45 #11 (35 mm diameter) longitudinal reinforcement and confined by a #5 (16 mm diameter) spiral with 90 mm pitch. A one-way flare in bent direction is over a 3.7 m transition length, and ends at the bottom soffit with a 1.829 m × 4.267 m rectangular cross-section. Twenty-two #11 (35 mm diameter) flare reinforcement starts at the onset of flare and extends 1.8 m into the cap beam. Over the top 2.4 m of the flare are rectangular stirrups #5 (16 mm diameter) at a spacing of 300 mm. But there is no stirrup to enclose reinforcement along the bottom 1.3 m of the flare. Column reinforcement details are shown in Fig. 2.

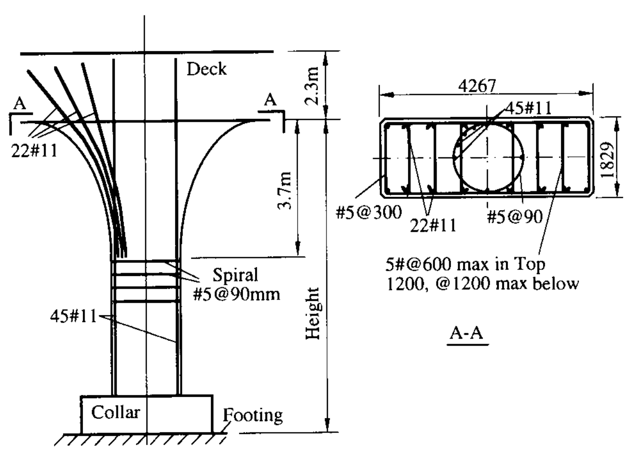


Fig. 2. Column reinforcement details

Footings are placed on 400 mm diameter 70-ton concrete piles. Dimensions of footing change from 6.1 m to 7.3 m in both directions and vary between 1.5 m and 2 m in depth.

As shown in Fig. 1b (Buckle, 1994), the right bridge collapsed completely between span 3 and 4. Spans 1 and 2 did not completely collapse but were severely damaged. Abutment 1 remained seated, having very little longitudinal movement; it had approximately 330 mm of transverse movement. At bent 2, the right column failed and slid on its footing due to damage of the reinforced concrete collar. The left column of bent 2 collapsed, dropping the flare to the side of column. At bents 3 and 4, columns failed at the lower part of flare. Since flares were not completely damaged, the bent was able to rest on them. At abutment 5, the end diaphragm was unseated.

Although it suffered major damage, the left bridge did not collapse. Most of the columns failed in the region of flare, but remained standing as shown in Fig. 1c (Buckle, 1994). The end diaphragm kept seated. At abutment 1, there was longitudinal movement of 76 mm and transverse movement of 51 mm. Transverse movement of 254 mm was observed at abutment 5.

STRUCTURAL MODEL FOR BRIDGE ANALYSIS

Since the left bridge structure did not collapse, analysis of collapse behavior is herein focused on the right bridge structure. Figure 3 is a sketch of the lumped mass model, which treats the right bridge as a three-dimensional structure.

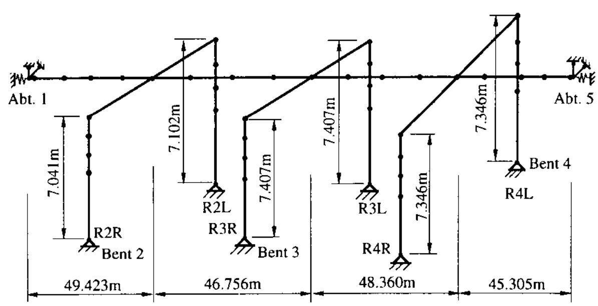


Fig. 3. Lumped mass model of right bridge structure

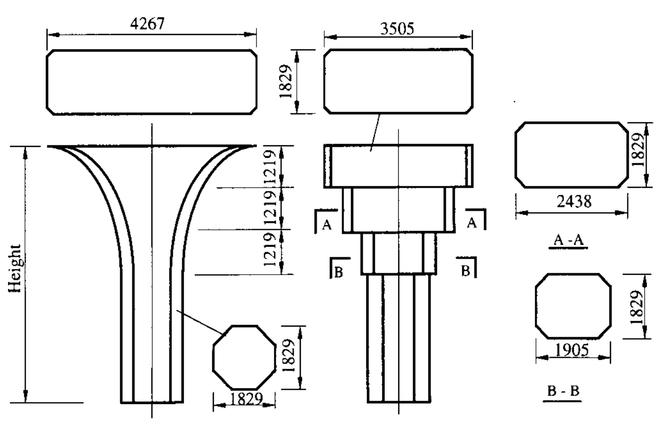


Fig. 4. Equivalent cross-section of column segments

A computer program called Nonlinear Earthquake Analysis of Bridge System, IAI-NEABS, is employed to perform time-history dynamic analysis (Imbson & Associates, 1993). Superstructures are considered as linear elements and columns are modeled as nonlinear elements.

To model the nonlinear column element, the upper part of column in the flare region is evenly divided into three elements with 1,219 mm in length. Due to heavy flare, an equivalent cross-section is introduced to treat each segment of the column as a straight-line element. Dimensions of equivalent cross-section are assumed at the segment's mid-point. The lower part of the column is considered as one element. Figure 4 describes the equivalent cross-section along with the column.

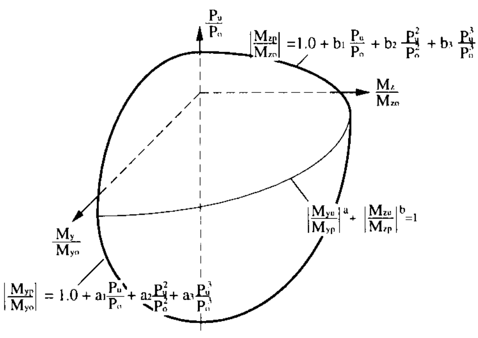


Fig. 5. Yield surface of reinforced concrete column

Before analyzing the bridge structure by IAI-NEABS (Imbsen and Penzien, 1984), a yield surface must be defined to predict the elastoplastic behavior of each reinforced concrete column element. Capacity of the reinforced concrete column can be determined by a combination of axial load and bending moments about two principal axes of the column cross-section. Figure 5 shows a sketch of the yield surface. If a combination of axial load and two bending moments is within the yield surface, the column has extra capacity to carry increasing load; otherwise, it yields. A computer program BIAX (Wallace, 1992) is used to determine capacities of each column segment. Results are summarized in Table 1. Symbols in Table 1 are consistent with those in Fig. 5. All the results are a part of data files for program IAI-NEABS.

Table 1. Capacity of each column segment

Size (mm x mm)	P_o (kN)	M_y (kN.m)	M_z (kN.m)	Parameters of yield surface in Fig. 5					
				a_1	a_2	a_3	b_1	b_2	b_3
1,829 × 1,829	109,725	14,959	14,959	-3.91992	-7.05966	-2.13974	-3.91992	-7.05966	-2.13974
1,905 × 1,829	114,142	15,305	15,081	-4.14611	-7.31107	-2.16496	-4.14732	-7.30555	-2.15822
2,438 × 1,829	158,853	28,583	22,891	-3.84528	-6.55112	-1.70584	-3.83488	-6.59450	-1.75961
3,505 × 1,829	227,312	45,305	24,605	-6.13207	-9.15266	-2.02059	-5.96119	-10.24710	-3.28590

A typical cross-section of superstructure is shown in Fig. 6. Properties of superstructure and column segments are shown in Table 2.

Figure 7 shows the sketch of elastomeric bearing pads and shear key at abutment. Elastic stiffness, K_F, of the bearing pad in longitudinal direction can be determined by the following expression

$$K_F = \frac{GA}{T} \tag{1}$$

where A and T are bearing pad area and thickness, respectively, G is shear modulus of bearing pad. Using G = 1.171 MPa, and pad dimensions in Fig. 7, the elastic stiffness in the longitudinal direction is 3,200 kN/m for each bearing pad.

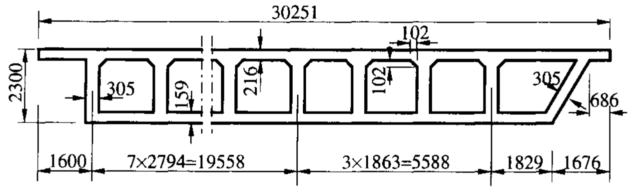


Fig. 6. Typical section of right bridge superstructure

Table 2. Properties of superstructure and column segments

		A (m^2)	I_x (m^4)	I_y (m^4)	I_z (m^4)
superstructure		17.628	38.615	1,472.881	13.634
column	1,829 × 1,829 (mm)	2.857	0.273	0.652	0.652
	1,905 × 1,829 (mm)	2.996	0.439	0.745	0.691
	2,438 × 1,829 (mm)	4.111	1.619	1.800	1.030
	3,505 × 1,829 (mm)	6.271	4.617	6.177	1.691

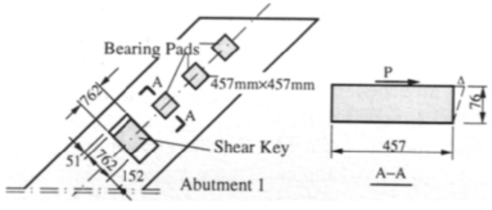


Fig. 7. Elastomeric bearing pads and shear key at abutment

Shear keys are used to prevent relative displacement between superstructure and abutment in the transverse direction. Columns are assumed to connect footings with hinges. Interaction between soil and foundation is not considered due to scant damage information.

NONLINEAR FINITE ELEMENT MODEL OF FLARED COLUMN

In order to have a thorough understanding of failure behavior of columns, nonlinear finite element analysis (NFEA) is employed to investigate the mechanics of columns with one-way flare under axial and lateral loads. Failure criterion for concrete and constitutive model by Ottosen for determination of secant modulus of concrete (Cheng and Lou, 1995) are incorporated into a nonlinear finite element model. Isoparametric hexahedral element with eight nodes is used for concrete, while reinforcement is modeled as elastoplastic material and uniformly distributed over concrete element. Perfect bond between reinforcement and concrete is assumed before concrete cracks, but a smeared cracking model, as soon as concrete cracks, is applied to deal with the subsequent behavior of concrete. A FORTRAN computer program named Nonlinear Analysis of Reinforced Concrete Structures (NARCS) is developed for UNIX system (Lou and cheng, 1994).

Figure 8 is the layout of finite element meshes for bridge column R4L. Column R4L is divided into 552 concrete elements and 312 reinforcement elements. Relative lateral displacement can be obtained assuming there is no movement at the top of the column. Lateral force is applied at the bottom of column, and dead load (13,148 kN) is uniformly distributed over nodes at the bottom surface of the column. Compressive strength of concrete is 31.66 N/mm^2. Steel yield strength is 414 N/mm^2 for main column reinforcement, and 303 N/mm^2 for stirrups and spirals.

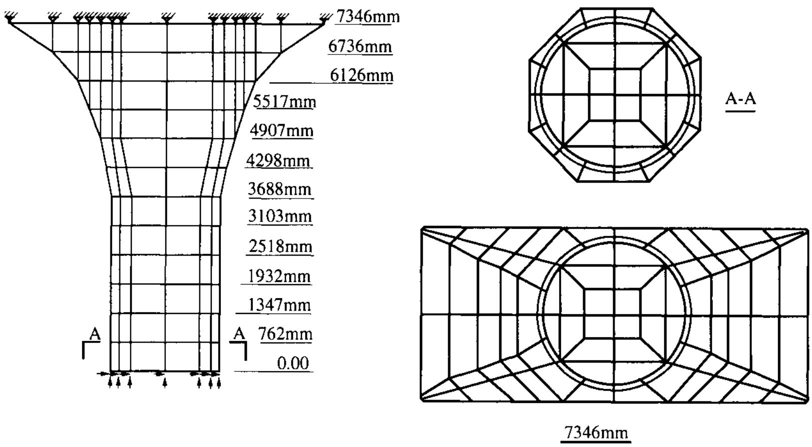


Fig. 8. Layout of finite element meshes for column R4L

DESCRIPTION OF LOADING

Seismic loading used for bridge collapse analysis was recorded at Sylmar County Hospital parking lot, which is approximately 18 km from the epicenter. This recording station provided acceleration, velocity and displacement in three directions with equal space at 0.02 second. Peak ground motion was 0.604 g at 4.08 seconds in east-west direction, 0.843 g at 4.20 seconds in north-south direction, and 0.535 g at 3.96 seconds in vertical direction.

Since there was no instrumentation in Mission-Gothic Undercrossing to record and collect data during Northridge earthquake, ground motion data recorded at Sylmar County Hospital parking lot had to be modified before bridge analysis. The following peak-horizontal-acceleration attenuation expression (Campbell, 1981) is used to compute acceleration a_1 at Mission-Gothic Undercrossing

$$a_1 = a \left\{ \frac{R_2 + 0.0606e^{(0.7M)}}{R_1 + 0.0606e^{(0.7M)}} \right\}^{1.09} \tag{2}$$

where a = peak acceleration in g at Sylmar County Hospital parking lot; M = magnitude of earthquake; R_1 = distance from epicenter to Mission-Gothic Undercrossing in kilometers; and R_2 = distance from epicenter to Sylmar County Hospital parking lot in kilometers. Distance from the epicenter to Mission-Gothic Undercrossing is approximately 7.5 km. Magnitude of the earthquake M is 6.7. Therefore the modifying coefficient is a_1/a = 1.16 for all components.

Accelerations in east-west direction and north-south direction are also modified to be consistent with longitudinal and transverse directions of Mission-Gothic Undercrossing. Figure 9 shows modified acceleration in longitudinal, transverse and vertical directions.

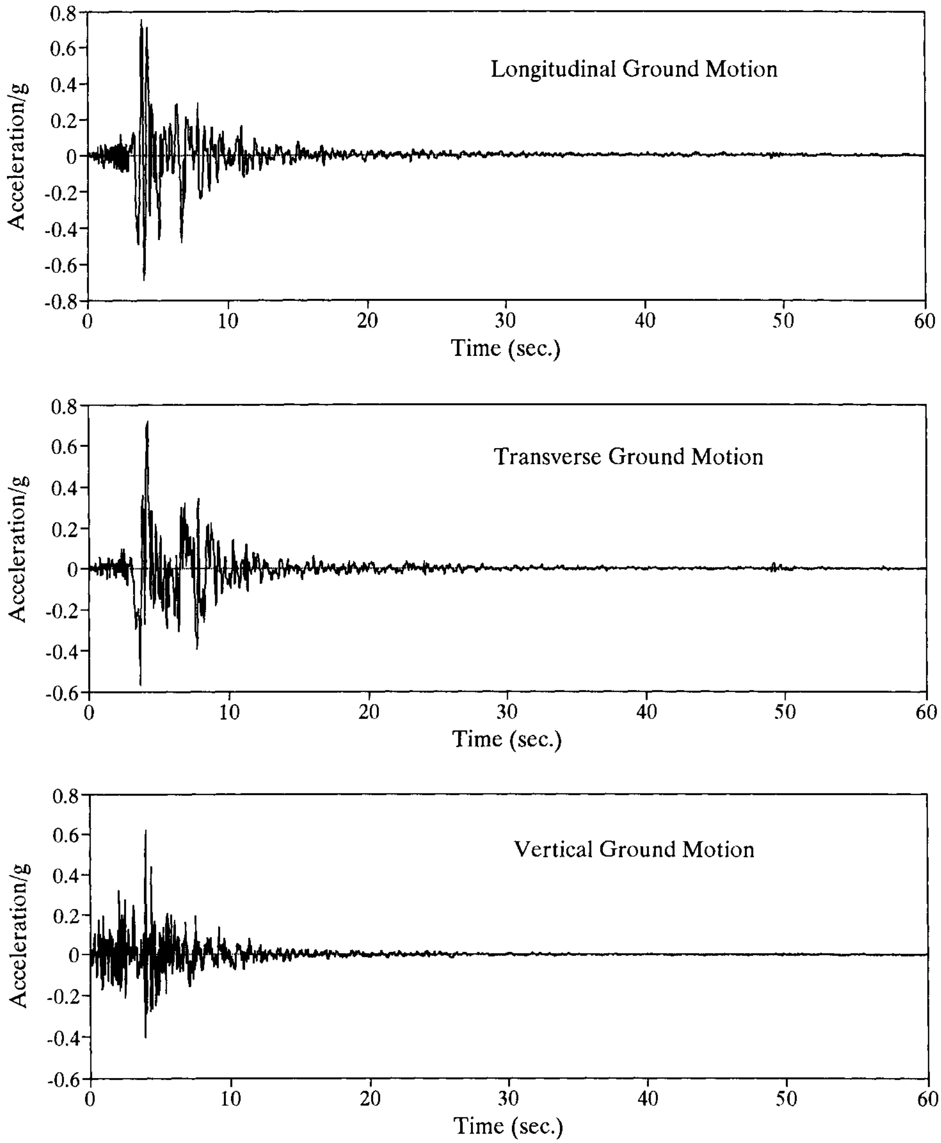


Fig. 9. Modified acceleration in longitudinal, transverse and vertical direction

NFEA RESULTS OF COLUMN R4L

Two cases of analysis are selected to describe the failure behavior of flared column R4L. One is a flared column subjected to lateral load in strong axis of cross-section with axial load. The other is subjected to lateral load at a 45° angle to strong axis with axial load. Both cases represent loaded columns of the right bridge at bent 4 and bent 2 or 3, respectively.

For a column with lateral load acting along strong axis, the increment of lateral displacement is proportional to the increment of load at the beginning of loading. When lateral loading increases to 1,352 kN, and correspondent lateral displacement is 7.61 mm, cracks perpendicular to strong direction of column cross-section appear at the bottom of the flare. Figures 10 and 11 represent the relationship of lateral load and displacement for column R4L. Although the occurrence of cracks causes reduction in a column's lateral stiffness, the stiffness reduction is small by comparison with uncracked column stiffness. Thus the relationship of lateral load and displacement stays approximately linear after the appearance of cracks. However, as lateral load increases, cracks propagate. As more cracks develop, a decrease in stiffness is responsible for deviation of load displacement curve. Consequently, nonlinear behavior of columns is clearly observed from load displacement curve. Longitudinal reinforcement begins to yield with lateral load of 2,288 kN. The crack pattern is shown in Fig. 12, which indicates that cracks develop mainly in the flare's lower part. Columns in this region are therefore easier to deform. Ultimate load reaches 3,640 kN with lateral displacement of 308 mm. Failure mechanics depicted above are similar to column failure seen in field observation after the earthquake (Priestly *et al.*, 1994), which indicates that concrete cracks and spiral rupture led to column collapse in bent 4.

For columns subjected to lateral load with a 45° angle to strong axis of cross-section, the load displacement curves for strong and weak axes are shown in Fig. 11 which is similar to Fig. 10 in the previous case. Cracks appear in the flare's lower part. These cracks are perpendicular to lateral load. In other words, cracks have a 45° angle to strong or weak axis. Cracking load is 1,144 kN with displacement of 4.56 mm in strong axis direction and 6.01 mm in weak direction; this load is lower than that acting in strong axis. Figure 11 shows that under the same load, a column has smaller displacement in strong axis because the column has more stiffness in strong axis than weak axis in the region of the flare.

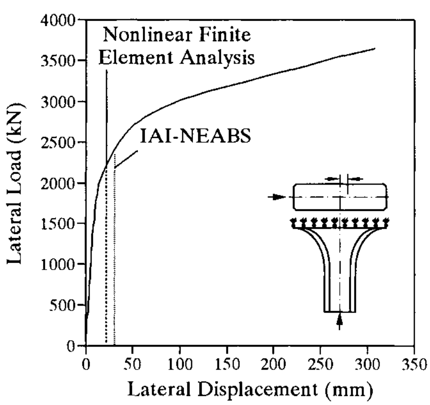


Fig. 10. R4L under strong direction load

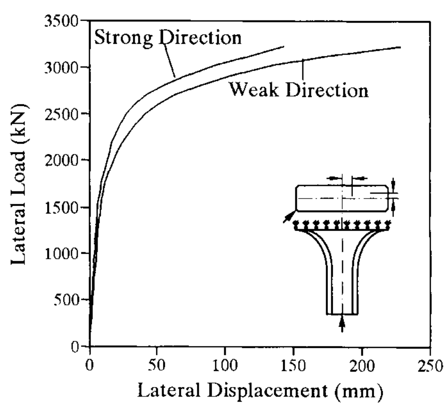


Fig. 11. R4L under 45°angled load

Consequently, the column fails in the weak axis plane. Ultimate load is 3,224 kN with a sudden increase of displacement from 174 mm to 229 mm in weak axis. Concrete cracks mainly appear in the flare's lower and upper part. Field observation (Priestly *et al.*, 1994) shows that flare damage began at the bottom of the flare where reinforcement was not enclosed by stirrups; concrete cracks developed upward with spalling of concrete over the region of the flare. Nonlinear finite element analysis reveals the same failure mechanics. Note that the ultimate lateral load in strong axis is higher than the same load in a 45° angle to strong axis. If all conditions are the same, and the direction of lateral load changes 45° from strong axis, ultimate load will decrease about 13%. Thus the unusual layout of skew bents results in different column capacities.

COLLAPSE BEHAVIOR OF RIGHT BRIDGE STRUCTURE

Four cases under different combinations of ground motion are performed by computer program IAI-NEABS on Sun Sparc Station for right bridge structure: 1) subjected to longitudinal ground motion; 2) subjected to transverse ground motion; 3) subjected to a combination of longitudinal and transverse ground motions; and 4) subjected to a combination of longitudinal, transverse, and vertical ground motion.

Figure 13 is longitudinal displacement of column R3L under longitudinal ground motion. At 4.04 seconds, the column begins to yield at the flare's lower part (i-end of element 26, Fig. 15). After 0.02 second, columns R2R, R2L and R3R also yield at the flare's lower part (i-end of Elements 11, 12 and 25). But yield does not develop in other parts of the columns during the response period; columns at bent 4 show no signs of yield during this period. Analytical results indicate that the right bridge will be damaged at the lower part of column flares at bents 2 and 3. This bridge will not collapse if subjected only to longitudinal ground motion. Maximum longitudinal displacement of column R3L is 29 mm at 4.24 seconds.

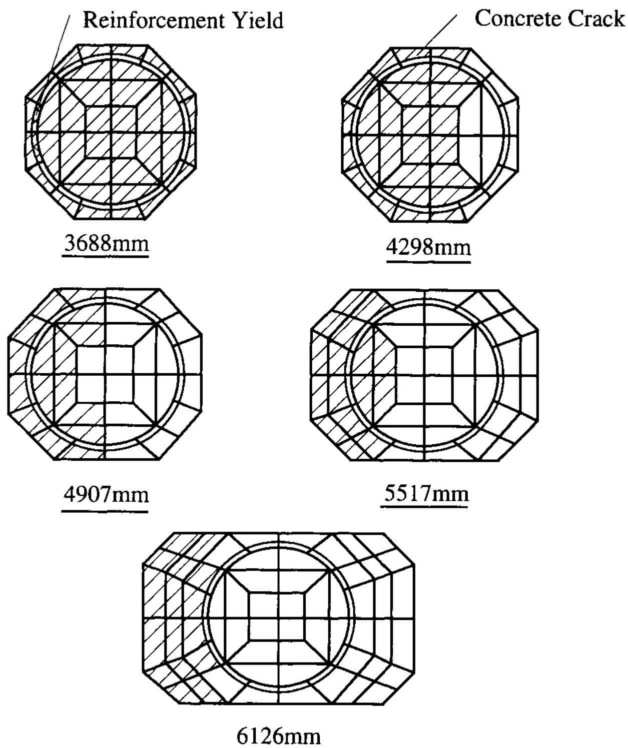


Fig. 12. Crack pattern of column R4L under lateral load in strong axis

Transverse displacement of column R3R subjected to transverse ground motion is shown in Fig. 14. With an increase in ground acceleration, the lower part of column R3R and R3L (elements 25 and 26, Fig. 15) appears to yield at 3.38 seconds. Corresponding displacement at column R3R is 31 mm. Maximum displacement of 75 mm occurs at 8.38 seconds. At 3.44 seconds, column R4L yields at flare i-end element 40, and then yields at j-end at 3.98 seconds. Element 40 is the lower part of the flare. Compare this with non-linear finite element solution of column R4L which starts to yield in longitudinal reinforcement at the bottom of the flare with lateral displacement of 25.12 mm (Fig. 10). Column R4L in the bridge structure represents 29 mm transverse displacement. Yield region of column R4L at 3.98 seconds is fairly close to crack pattern of nonlinear finite element method at first yield of longitudinal reinforcement (Fig. 12). Although these two approaches are based on different criteria for determining yield of element, or concrete crack and steel yield, results are similar.

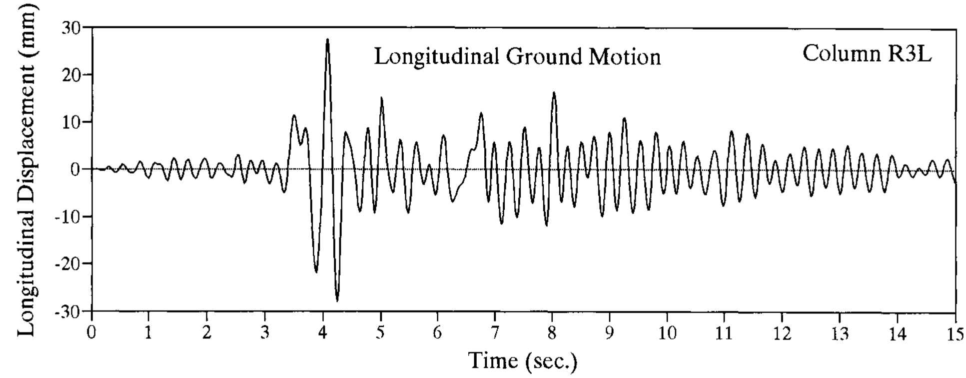


Fig. 13. Longitudinal displacement of column R3L under longitudinal acceleration

As noted above, nonlinear finite element analysis shows that capacity is reduced if lateral load has a 45° angle with strong axis of cross-section. Skew bents 2 and 3 result in lateral forces with an angle to strong axis acting on columns. Therefore columns R3L, R3R yield prior to columns at bent 4.

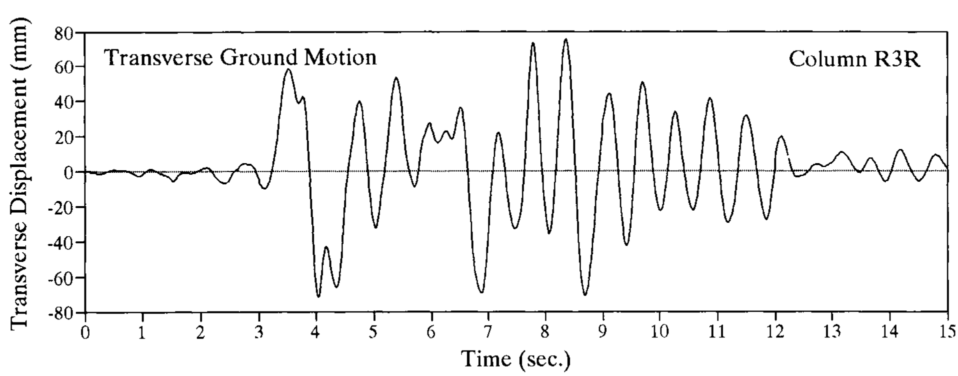


Fig. 14 Transverse displacement of column R3R subjected to transverse acceleration

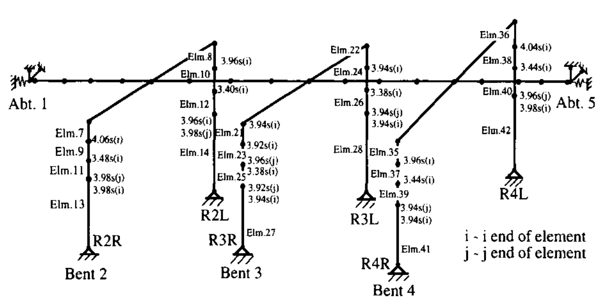


Fig. 15 Identification of elements

A circular flare is suggested herein to improve the capacity of columns under lateral load with an angle to strong axis. Due to the symmetry of circular flare in all directions, column capacity is independent of lateral load with an angle to strong axis. If circular flare is used to design the skew bents, all columns have the same capacity to resist applied loads.

Figure 15 is a summary of elements identification under three-dimensional ground motion. This structure yields at columns R3R and R3L in the lower part of flare at 3.38 seconds. After 0.06 second, column R4R then yields at 3.44 seconds. Column R2R also reaches yielding at 3.48 seconds. All the columns first yield in the lower part of the flare. Columns at bent 3 yield severely, while columns at bents 2 and 4 yield in the middle or lower part of the flare.

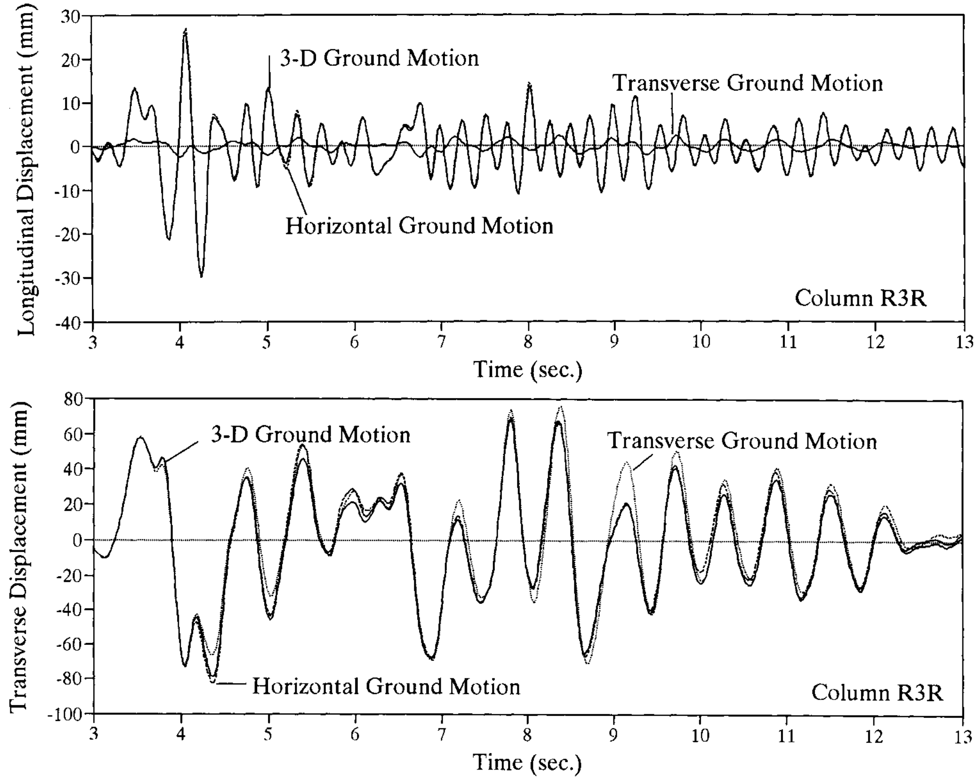


Fig. 16 Longitudinal and transverse displacements of column R3R for 1-D, 2-D, and 3-D ground motion

Failure of columns results in the collapse of the bridge superstructure. The yield pattern gives a view of collapse behavior which is similar to field observations after the earthquake. All columns yield within 0.7 second after 3.38 seconds. This means the bridge collapsed rather suddenly. Additional spirals are suggested for the lower part of the flare to enclose reinforcement and prevent propagation of concrete spalling. Comparison of transverse and longitudinal displacement of column R3R is made in Fig. 16 among transverse ground motion, horizontal ground motion and 3-D ground movements. If longitudinal and vertical ground motions are considered in analysis, it is apparent that longitudinal displacement of the structure greatly increases. Due to the skew of bents 2 and 3, columns in these bents are subjected to two-dimensional loads. An increase in longitudinal displacement leads to an increase of internal forces in columns.

Figure 17 represents a comparison of moment M_y (in strong axis) and M_z (in weak axis) at the bottom of the flare of column R4L (i-end of element 42). Difference of moments at the same time is observed due to different ground motion. At 4.58 seconds, moment M_z caused by three-dimensional ground motion is 14,513 kN-m. But the moment caused by transverse ground motion is 3,803 kN-m. The former is approximately 3.8 times the latter. In addition, moment M_y caused by three-dimensional ground motion (12,496 kN-m) is greater than that (11,760 kN-m) caused by transverse ground motion. Thus the increase of moments at two major axes of column cross-section leads to unfavorable conditions. Vertical ground motion should be considered in bridge column design. Bridge columns are usually subjected to compressive axial load and two-dimensional moments. To understand this unfavorable condition in load combination, vertical ground motion is an important factor in bridge structural analysis.

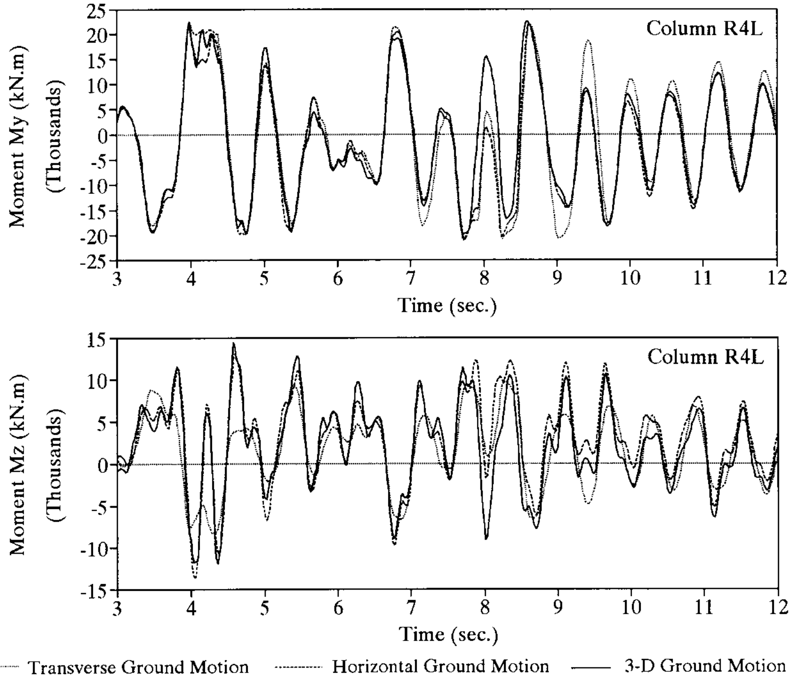


Fig. 17 Comparisons on moment M_y and M_z

CONCLUSIONS

1. A three-dimensional structure model is developed for analysis of the right bridge of Mission-Gothic Undercrossing. Computer results indicate that column failure led to collapse of the bridge's superstructure. Collapse behavior is similar to that observed in the field after the earthquake.

2. Nonlinear finite element method is employed to investigate failure behavior of the column subjected to lateral load with an angle to strong axis. For the column studied with lateral load at a 45° angle from strong axis of cross-section, column capacity is reduced about 13%.

3. A circular flare is suggested to improve column capacity under lateral load. If an unusual layout of skew bents is unavoidable, then circular flare for columns should be used since its capacity is independent of angled lateral load. Additional stirrups should be placed in the lower part of the flare to enclose reinforcement so that spalling of concrete in this region can be prevented.

4. Computer results show that the consideration of three-dimensional ground motion leads to an increase in longitudinal displacement of bridge structure and unfavorable loading conditions in bridge columns. Columns subjected to horizontal as well as vertical ground motion may be reduced in capacity.

ACKNOWLEDGMENT

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ACHIEVEMENTS AND EXPERIENCE OF HOMELAND REHABILITATION AFTER EARTHQUAKE IN LANCANG AND GENGMA

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ABSTRACT

Two successive strong earthquakes at the magnitude of 7.6 and 7.2 occurred in Lancang and Gengma counties on Nov. 6, 1988. Serious earthquake damage covered 30 cities and towns in 5 prefectures in southwest Yunnan. People of national ethnic minorities living in these areas suffered greatly from the disaster which caused injury and death of more than 7,000 and a direct economic loss of up to 2.05 billion yuan (RMB).

KEYWORDS

Earthquake resistance; rehabilitation; reconstruction; retrofit; disaster relief.

INTRODUCTION

Earthquakes are merciless. But the Chinese Communist Party and the government of China paid close attention to rehabilitation in earthquake-stricken areas. With the help of fraternal provinces, municipalities and prefectures, and aid given by compatriots in Hong Kong, Macao and Taiwan, overseas Chinese as well as friendly countries, organizations and persons, governments at different levels in the earthquake-stricken areas rallied. They could rely on the masses of people and engineers, striving to succeed in the rehabilitation of their homeland. After four years of hard work, 5,925 reconstruction items with a building area of 4.09 million square metres have been completed, including 82,000 rebuilt dwelling houses, the retrofit of 162,000 existing houses, 1,609 primary and middle schools, 267 hospitals and clinics, 612 public welfare and 204 social welfare facilities, the restoration of more than 400 industrial enterprises, the renovation and extension of 27 highways which are about 726 km and 11 large or mid-size bridges, the construction of 939 water conservancy facilities, 23 post offices and communication facilities, and 130 municipal engineering facilities in towns, as well as the new district in Gengma county. Some severely afflicted towns, such as Lancang and Cangyuan counties, began to have a new look.

REHABILITATION EXPERIENCE

We have summed up experience from the four-year rehabilitation as follows.

Formulate and Implement Correct Policy and Principle to Succeed at Rehabilitation.

It was not clear about what to do at the beginning in Lancang and Gengma. Some cadres and masses thought they could rely on the State absolutely. Some districts competed for projects of reconstruction and investment. A few departments and organizations even wanted to use this as an opportunity to gain additional profit. To counter this situation, we carried out conscientiously a policy "to rebuild the homeland through self-reliance, hard work and development of production" pointed out by Premier Li Peng when he inspected the earthquake-stricken area. According to this policy, we formulated the principles of "stressing the main points, putting the urgent items first, taking the work of local governments as dominant in combination with that of provincial departments concerned, giving top priority to rehabilitation and second order to new development according to the financial capability." With the "one policy and three principles", thoughts and actions of the people were united. The enthusiasm of all sides was sparked. We called meetings to mobilize the masses to take the interests of the totality into account. We told the severely afflicted areas that it was better to depend on themselves than the State, and the less afflicted areas to consider the difference in their circumstances and not scramble for funds. Through implementation of the "one policy and three principles", the cadres and masses realized that they should rehabilitate their homeland through self-reliance, and a plan for reconstruction was soon drawn up. Governments and organizations in earthquake-stricken areas raised more than 80 million yuan for rehabilitation. Most of the irrigation and water conservancy facilities were repaired by the people themselves; 170,000 houses were rebuilt or repaired by residents with money they raised, except a small appropriation from the state to each family. This is an important basis for the achievements in rehabilitation.

Strengthen Government Leadership to Ensure Implementation of Policies.

At first, we assigned most of the work to relevant departments of the provincial government. We found that this did not hearten the initiative of the authorities in the prefectures and counties. We therefore assigned responsibility to heads of the prefectures and counties. In this way, the leadership of local government was strengthened, and they divided themselves into two groups. One was responsible for routine duties and the other for rehabilitation. The second group provided headquarters for the work. Through local government at different levels, the "one policy and three principles" were carried out smoothly. Responsibility with heads of prefectures and counties did not mean that the participation of provincial departments concerned would be eliminated. The latter should send their functionaries to earthquake-stricken areas to join the effort and to serve the people there under the principle of "taking the work of local governments as dominant in combination with that of the provincial departments concerned". This is a dialectical unity. Only with the heartened initiative of all authorities and departments in disaster area and the excellent work of provincial departments can rehabilitation be carried out smoothly. Actual results show that the work of provincial departments was good and led to achievements. These departments helped the grassroots units and laid particular stress on grants towards investment in reconstruction of disaster areas.

In order to strengthen leadership, the provincial government established the Headquarters on Earthquake Resistance and Disaster Prevention. The governor and I are in charge of its work. Organizations assigned to the work have been set up in the departments and units concerned. This system of organizations is responsible for the everyday work of earthquake protection after rehabilitation. At present, there are more than 120 such organizations in prefectures, municipalities and counties, forming a network throughout the province.

Establish Systematic Management and Scientific Approach

Since this was the first time our province dealt with large-scale rehabilitation after a high-magnitude earthquake, there was no available experience to be used for reference. So we insisted on proceeding from reality, and effective management in a scientific way was gradually formed. The work of rehabilitation was carried out appropriately. During the period of the emergency, we drew up "Temporary Measures for Usage and Management of Disaster Relief Fund, and "Temporary Measures for Management of Disaster Relief Medicines and Medical Instruments". This limited relief was distributed reasonably so that waste was reduced and the basic need for the earthquake relief was met. During the period of reconstruction, we produced a series of documents, such as "Temporary Provisions on Tightening up Management for Rehabilitating Homeland in Earthquake-Stricken Areas", "Notice on Tightening up Management of Special Funds for Rehabilitating Homeland", "Notice on Exempting Taxes on Building Construction Investments in Earthquake-Stricken Areas", "Measures for Rebuilding Dwelling Places in Rural Disaster Areas in Yunnan", and "Notice Concerning Repairing and Rebuilding Houses Owned by Citizens". Under the guidance of these documents, concrete details in every phase were supervised to promote successful rehabilitation. In practical matters, we relied on thorough investigation of the realities and arranged each task accordingly. We summed up and exchanged our experience at various meetings including professional ones. Problems were solved immediately when they were found. Funds were used to meet real needs. Responsibilities were clearly assumed by definite persons, and all regulations and provisions were fully implemented.

Foundation of Rehabilitation

Preliminary work in earthquake-stricken areas is very important for the construction of prosperous and beautiful towns. When planning, we found the following: some villages were situated on slopes prone to landslides and had to be moved; part of the site of some county towns was not suitable for construction and a new district had to be developed; the construction of some towns was disorganized, without a plan, and had to be laid out and then rebuilt. With the great effort of vast numbers of engineers, plans to reconstruct four severely afflicted counties and 18 small towns were finished within a time span of a little more than one year. This allowed reconstruction to proceed under the guidance of a full plan with unified implementation and management. Provincial design organizations assisted the disaster region to complete 368 individual projects with an area of more than 430,000 square metres. Also this laid a favourable foundation for rehabilitation. In particular, planning for the town of Gengma county was completed excellently and swiftly. Now the main streets in the new district of the town have been constructed and the buildings along the streets are very attractive.

Qualified Engineers and Technicians Ensure Rehabilitation Quality

Lancang and Gengma are regions inhabited by minority nationalities with a scarcity of engineers. In order to facilitate reconstruction in the disaster areas, a qualified team including structural engineers, planning designers, architects, budget experts and quality supervisors was organized to work in the disaster areas. Simultaneously, construction teams with superior ability were sent to the disaster areas. This ensured the quality and schedule of the projects, and saved capital funds as well.

IMPROVEMENT OF EARTHQUAKE-RESISTANT CAPACITY

In recent years, we have been in charge of earthquake resistance and strengthening buildings and engineering facilities in the whole province as well as rehabilitation in the disaster areas of Lancang and Gengma. Through the actual work, we realized that the most important measure is to increase the earthquake-resistant capacity of buildings and engineering facilities. This has been proven by the cases of earthquake disasters both at home and aboard, and the examples of many engineering facilities which were damaged during Lancang-Gengma earthquake. Gengma Cinema, for instance, had been designed according to aseismic design code with its reasonable structure, firm connection, and good construction quality. It still stood after two shocks without any damage, even its glass framed in 30m long windows. But the buildings nearby which were designed without earthquake-resistant measures were seriously damaged. Some of them were leveled to the ground. It is evident that measures of earthquake prevention are effective to withstand and mitigate the earthquake hazard.

In order to safeguard the earthquake resistance of new engineering projects and retrofit of existing facilities, the provincial government has promulgated documents such as "Requirements on Earthquake Resistance of Recently Constructed Projects", and "Detailed Regulations for Management on Earthquake Resistance of Recently Constructed Projects in Yunnan". Approval for earthquake-resistant measures of new projects has been strictly controlled by the authorities concerned so that the projects can be designed and constructed according to state requirements. As to the improvement of existing structures, buildings of 3.5 million square meters and other municipal facilities (such as communication, pipework and bridges) have been strengthened through the effort of many years. As a result of these measures, the earthquake-resistant capacity of buildings and engineering facilities all over the province has been greatly improved.

Now more than four years have passed and we have accomplished the rehabilitation of our homeland. Buildings and engineering facilities in the disaster areas of Lancang and Gengma have been repaired or rebuilt. The economy there grows rapidly. Ethnic minority nationalities in the earthquake-stricken areas are united. Their living standard has improved. Because of social stability and economic growth, the people can live and work in peace and contentment. The border areas of Yunnan extend a scene of prosperity.

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LESSONS LEARNED FROM RECENT EARTHQUAKES AND THEIR IMPACT ON NCEER RESEARCH AND IMPLEMENTATION EFFORTS

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ABSTRACT

Damaging earthquakes provide opportunities to understand problematic effects on built and socioeconomic environments and to assess the validity of our past attempts to address areas of concern. Clearly, the impact of these natural disasters and the research needed to mitigate against them cross disciplinary as well as professional boundaries. This paper will review some significant lessons learned from past earthquake experiences and the influence these developments have had on the research and implementation activities of the US National Center for Earthquake Engineering Research.

INTRODUCTION

The National Earthquake Hazards Reduction Program (NEHRP) was established by the United States Congress in 1977, the product of growing national awareness and concern about seismic risk and safety. NEHRP was introduced to provide a comprehensive, integrated national program to reduce losses of life and property resulting from earthquakes. The Congress recognized that losses and disruption to the individual, the community, the State, and the Nation caused by earthquakes could be substantially reduced through the development and implementation of earthquake hazards reduction measures. The Federal Government was directed to provide a central focus for leading, coordinating, and conducting earthquake research, hazard mitigation, and disaster preparedness. The Federal Emergency Management Agency (FEMA), the United States Geological Survey (USGS), the National Institute for Standards and Technology (NIST) and the National Science Foundation (NSF) have been charged with carrying out the principal Federal responsibilities outlined within NEHRP for the reduction of losses caused by earthquakes.

As shown in Fig. 1, NSF supports earthquake research through various avenues. These include individual funded research, through coordinated sponsored programs such as TCCMAR (Technical Coordinating Committee on Masonry Research), R&R (Repair and Rehabilitation), and PRESSS (Precast Seismic Structural Systems), and focused research centers. With the USGS and FEMA, NSF supports the Southern California Earthquake Center (SCEC), a cooperative research program to model the regional seismic hazard in Southern California to allow scientists, engineers, and government officials to reduce risk. The National Center for Earthquake Engineering Research (NCEER) was established in 1986 by NSF through its Earthquake Hazard Mitigation Program (EHM) in the Division of Civil and Mechanical Systems. The EHM program is one of NSF's implementors of NEHRP and is intended to assist the Foundation in meeting its NEHRP-prescribed objectives summarized in Fig. 1.

<table>
<tr><th>ORGANIZATION</th><th>MECHANISM</th><th>GENERAL NEHRP OBJECTIVES</th></tr>
<tr><td>FEMA
NIST
USGS</td><td>- In-house projects
- Sponsored Projects</td><td rowspan="5">- Enhance knowledge and understanding of earthquakes and their effects
- Increased availability of Information
- Target outreach and Implementation efforts
- Transfer Technology
- Conduct problem-focused research
- Encourage community adoption of codes</td></tr>
<tr><td rowspan="3">NSF</td><td>- Individual Sponsored Projects</td></tr>
<tr><td>- Coordinated Sponsored programs (TCCMAR, R&R, etc.)</td></tr>
<tr><td>- NCEER: Systematically integrated Interdisciplinary approach</td></tr>
<tr><td>State and Local Government
Other Mission Agencies</td><td>- In-house Projects
- Sponsored Projects</td></tr>
</table>

Fig. 1. NCEER's role in NEHRP

NCEER offers NSF an additional, unique mechanism to address earthquake engineering problems by using a systematically-integrated and interdisciplinary approach. Teams of researchers brought together from across the country are charged with advancing knowledge to minimize loss of life and property caused by earthquakes, with particular attention to the lesser-understood earthquake risk to the Central and Eastern US. In addition to the Center's resource of technical expertise, the involvement of multiple institutions provides access to a diversity of experimental and testing facilities, including shaking table and other structural testing devices, soil testing laboratories and centrifuge facilities.

NCEER Mission and Management

Since its establishment, the mandate of NCEER has been to conduct mission-oriented research to advance the state-of-the-art in earthquake engineering through both curiosity-driven and problem-focused research, and a dedicated program to promote knowledge utilization. To most effectively achieve this goal, the NCEER research program recognizes and reflects the interdisciplinary nature of disaster research and management, assembling teams of specialists in many fields and professions to identify knowledge gaps, develop potential solutions, evaluate applications, and recommend improvements in seismic methodologies and policies. Experts are drawn from fields of seismology, geotechnical and structural engineering, as well as economics and the social sciences, and represent academia, industry and the public sector. Teams contribute to four primary research concentrations which focus on the seismic performance, retrofit and enhanced design of buildings, nonstructural systems, lifelines (including gas, oil, water, electric power and telecommunications), highways and bridges. Studies in these problem-focused areas incorporate integral fundamental knowledge on ground motion and the seismic hazard, geotechnical influences, risk assessment, social systems and economics provided by experts in these fields. Research on smart materials and intelligent systems for structural vibration control is also conducted, to develop innovative approaches to seismic design. The interaction between fundamental and applied research, the use of demonstration projects to validate research applications and the transferral of knowledge and technology is described graphically in Fig. 2.

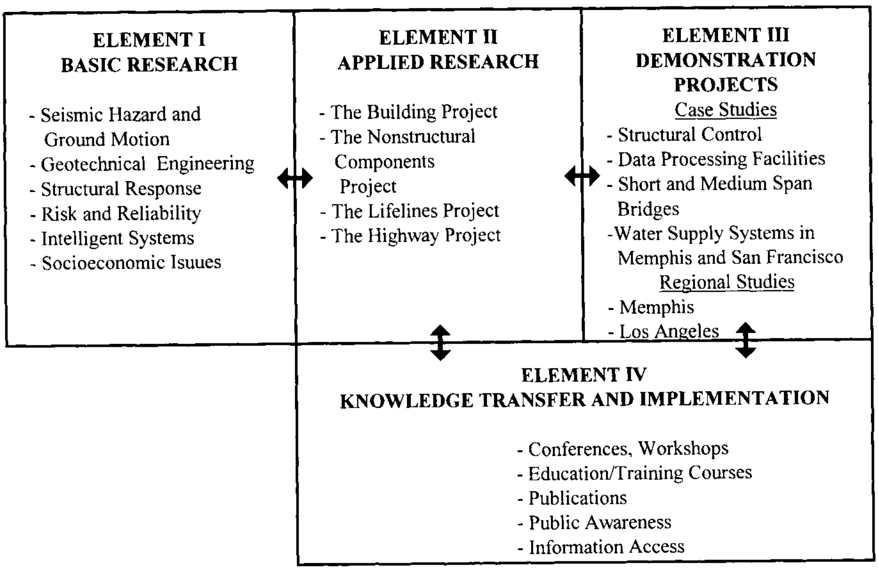

Fig. 2. NCEER Research and Implementation Plan

NCEER research funding by NSF requires that an additional and equal amount of matching funding be obtained from external non-federal sources. NCEER's leveraging of NSF research dollars to attract supplemental financial support has allowed the development of a cost-effective program, which can conduct both fundamental and problem-focused research while sustaining additional activities toward knowledge utilization and implementation.

LESSONS FROM RECENT EARTHQUAKES

The broad understanding of the causes and patterns of earthquake damage have been based on observed experiences and years of substantiating research. Nonetheless, each damaging earthquake is distinguished by its own unique signature and set of accompanying consequences which are constrained by geologic site conditions, configuration of the constructed environment and the fabric of existing sociopolitical systems. As a result, each destructive event offers lessons of particular value to earthquake mitigation research. Some significant observations from selected past earthquakes are summarized here as lessons learned.

Mexico City, Mexico - 1985

The general area of the epicenter for the September 19, 1985 Mexico City earthquake was located roughly 250 miles away from the heart of the city. The main shock of the event, M_S 8.1, was followed within 36 hours by a 7.5 aftershock. Older sections of the city were located on deep alluvium over an ancient lake bed experiencing significantly amplified ground shaking. This resulted in much damage to the structures located there. About 90% of the damage was caused by ground shaking, with liquefaction and fire contributing to the remaining 10%. Over 4,500 people were killed and 14,000 injured. Nearly 5,800 buildings were damaged, with 90 percent of the damage occurring on the area located on the site of the ancient lake bed. The water supply and distribution systems of the metropolitan area of Mexico City were severely damaged, leaving more than 5.3 million people without water, more than ever experienced by an urban area.

Main types of damage were caused by: brittle failure of columns, uncertainties associated with nonlinear response, pounding and soil-structure interaction, P-delta effects, overloading of buildings, unsatisfactory performance of waffle slabs, foundation failure, exacerbation of pre-existing damage and local soil strains. Dramatic variations in damage density were observed throughout the effected area (Rosenbleuth et al., 1988).

Housing for nearly half a million residents was lost, with severe damage also to many schools and hospitals. In the aftermath of the event, a building code was developed which incorporates seismic design considerations which account for the differing soils and site conditions throughout the city. The rebuilding process allowed for the improvement of the quality of residential housing and markedly improved seismic strengthening of school buildings. The Mexico City earthquake has provided the earthquake community with important insight about the consideration of site response issues in seismic design (Spangle and Associates, 1990).

Spitak, Armenia, USSR - 1988

On December 6, 1988, a M_S 6.2 earthquake occurred near the Armenian cities of Leninakan (population: 300,000), Kirovakan (population: 200,000) and Spitak (population: 30,000) causing severe damage over approximately 25% of the area of the Republic. Some damage was experienced in 60% of the region, with tremendous economic and social effects. More than 25,000 deaths resulted from the event and its magnitude 5.9 aftershock. Many modern engineered buildings constructed of precast concrete and stone masonry exhibited generally poor performance with many collapsing. Essentially unreinforced stone masonry buildings were all heavily damaged in Spitak, but to a much lesser extent in Leninakan. However, 72 modern, nine-story precast-panel buildings in Leninakan collapsed due to an absence of ductile details, poorly connected precast diaphragm elements and eccentricities in column bars at splices. Numerous industrial facilities collapsed or experienced damage severe enough to result in complete loss. Structural damage observed reflected the need for improved and upgraded building codes to insure seismic resistant design, as well as observation of more rigorous construction standards. Lifeline systems in Leninakan were severely disrupted for the entire emergency response period. Search and rescue teams were greatly hampered by the large number and condition of collapsed buildings, as well as the shortage at the outset of critical rescue equipment. Regional failure of vital railways and roadways also presented a major problem and significantly impeded emergency response (Wyllie, Jr. and Filson, 1989, Yegian and Ghahraman, 1992).

Four years after the earthquake, it was estimated that only 30% of needed housing had been restored, with more than a quarter of a million people living in substandard shelter conditions. Reasons for this include lack of organization and coordination in the reconstruction process, with policies and decisions made before adequate scientific assessments had been made. The reconstruction process initiated has not been cost effective, and has involved a work force from outside the region with little prior experience in a large-scale project, and little input from local residents in the planning stages. Many of these problems have been caused by the dissolution of the USSR and the jurisdictional obstacles which resulted from it. The Armenia experience highlights the need for coordination between jurisdictions for the effected region, and the importance of a well-conceived reconstruction plan (Ghahraman and Yegian, 1993).

Santa Cruz, California - 1989

On October 17, 1989, a M_S 7.1 earthquake occurred about 60 miles to the southeast of the Bay area cities of San Francisco and Oakland, California resulting in loss of life, and damage to the socioeconomic and civil infrastructure of the area. But for the civic attention of the community to baseball's World Series, the toll would have likely been greater. Damage occurred throughout an 8,000 square kilometer area, with the greatest loss of life more than 90 kilometers away from the epicenter at the Cypress Street Viaduct in Oakland. The earthquake, known popularly as Loma Prieta, killed 62 people, injured 3,757, destroyed numerous businesses and left over 12,000 people without homes. Amplification of ground motion by the soft fill of the Marina District resulted in damage to lifelines and numerous structures and caused displacement of hundreds of residents (Benuska, 1990).

Closer to the epicenter, cities of Santa Cruz and Watsonville each lost 60% of their downtown commercial districts, with further damage resulting in an 8% loss of the Watsonville housing stock. Most of the structural damage occurred in unreinforced masonry commercial buildings, older wood frame homes, and apartments which did not meet the current seismic design standards (Comerio, 1990).

Elevated double-decker freeways did not perform well, with major damage to sections of Interstate 480, 280 and 880 (Cypress Street). Compounded by additional structural damage to Interstate 80 and State Highway 17, the region experienced major traffic problems. All lifelines systems were impacted, with outages of gas supply and electric power for periods of days following the event, and in some areas a potable water supply was unavailable for months. The telecommunications systems suffered no significant damage but was overloaded in the first days following the event (US Army Corps of Engineers, 1990).

Damage caused by the Loma Prieta earthquake to unreinforced masonry buildings, bridges, pipelines and other residential and nonresidential structures clearly demonstrates the impact of low-level shaking amplified by soft soil conditions - an earthquake scenario that could be envisioned for other regions in United States.

Northridge, California - 1994

The M_W 6.7 Northridge, California earthquake of January 17, 1994 was revealing in many ways. While traditionally seismically vulnerable structures performed as expected, structures designed to more exacting standards and those with typically better performance histories suffered unexpected damage.

Seven highway bridges suffered partial collapses, with another 170 bridges sustaining damages ranging from minor cracking to slumping of abutment fills. While most of the bridges with collapsed spans had been retrofitted, none had been built to current seismic standards. Some bridges in the epicentral area had additional strengthening provided through the steel-jacketing of columns. No distress was observed in these columns even in areas of locally strong ground shaking (Buckle, 1990).

While general performance of lifelines was much improved over that exhibited during the 1971 San Fernando event, more specific vulnerabilities of lifelines were revealed, with additional attention to pipe materials and connection techniques. Furthermore, although lifeline networks have expanded as warranted by the rapid growth of Los Angeles, lifeline vulnerabilities may place unexpected burdens on the response and recovery mechanisms in place (O'Rourke et al., 1994).

Structural damage accounted for most of the losses in the Northridge earthquake, although in general, certain structures performed relatively well due to the enforcement of more stringent design standards. Damage was observed to be most severe in the following types of structures: poorly engineered wood frame structures; older reinforced concrete moment resisting frames with nonductile details; newer reinforced concrete moment resisting frames with adequate details but without sufficient seismic resisting systems; precast structures without suitable seismic resisting systems and lacking redundancy; unreinforced masonry systems not retrofitted to current required levels; and reinforced masonry structures that were not upgraded to the maximum credible level earthquake. A large number of two- to four- story wood frame apartment buildings collapsed due to lack of suitable lateral resisting systems. The damage, and in some cases, collapse of newer structures, especially parking garages, commercial buildings and apartment complexes suggest a needed reexamination of minimum code-mandated performance criteria. The disruption of nonstructural components and building contents in schools and particularly in hospitals during this earthquake was considered to be unacceptable (Goltz, 1994, Hall, 1994).

A notable discovery in weeks following the earthquake was the unexpected fracturing in welded beam-to-column connections in multistory steel moment resisting frame buildings, spurring numerous followup studies to examine this atypical performance (Hall, 1994).

The State of California moved quickly to restore transportation systems and utility capabilities to full functionality, but also presented the Los Angeles region with a number of additional challenges which will be studied and addressed for years to come.

Hyogo-ken Nambu, Japan - 1995

The Great Hanshin earthquake occurred January 17, 1995 and resulted in over 5,000 deaths and more than 26,000 injuries. A major impact of the M_W 6.8 earthquake was on transportation systems. Kobe's location is critically positioned on the main transportation corridor between northern and southern Japan. A 500 meter segment of the Hanshin expressway collapsed, as well as several spans of elevated structures supporting rail lines for the bullet train. Widespread liquefaction was evident on Kobe Port Island and Rokko Island. The port of Kobe, which handles nearly one-third of Japan's container freight traffic, suffered major damage, leading to subsequent cessation of port functions. Damage to residential structures was prevalent in those older wooden structures with heavy tile roofs that were not built to seismic standards. As many as 75,000 buildings have reportedly been destroyed by this earthquake. These include a number of midrise reinforced concrete buildings built during the 1960's and 1970's which appear to have suffered shear failures in columns at mid-building height, with subsequent pancaking of single upper stories. Future study is expected to assess the effectiveness of 1981 seismic building code enhancements in the performance of Kobe's structures.

While electric power and telecommunications systems were restored quickly, gas and water are expected to take many weeks to be fully restored. Loss from post-earthquake fire was very dramatic, with nearly 100 fires breaking out within minutes after the event. Fire predominated in densely populated built-up areas of the central city, consisting of mixed residential-industrial-commercial occupancies and primarily wood construction (NCEER, 1995).

IMPACT ON NCEER RESEARCH EFFORTS

Experiences from past earthquakes emphasize the need for continued research to improve preparedness and mitigation solutions. Using its multidisciplinary systems-integrated approach, NCEER uses these experiences to initiate or to redirect research for the development of timely and useful solutions to recurrent and emerging engineering problems.

Buildings

Earthquake damage to buildings and their nonstructural systems and components can lead to human casualties, as well as economic disruption as a result of property loss and business interruption. To address this problem the Center's **Building Project** examines two classes of buildings that are common to the US: unreinforced masonry and lightly reinforced concrete buildings, both vulnerable to moderate and even minimal shaking. Objectives are to develop rational methods for evaluating these structures and to devise cost-effective methods for retrofitting to provide adequate seismic resistance. To achieve this, integral studies define seismic and geotechnical hazards, assess structural fragility, evaluate and refine conventional strengthening methods, develop and apply smart materials and innovative protective systems, improve methods for estimating potential losses, examine relationships between building type, mortality and injury, and establish reliable inventory methods for existing building stock.

Analytical Tools

The need for efficient and reliable analytical tools to support experimental research on the seismic behavior of reinforced concrete structures has led to the development of the computer program IDARC (Reinhorn et al., 1994), which presently facilitates analysis of components, subassemblages and entire structures under static, quasistatic cyclic and seismic loads. This program has been used for the design evaluation of experiments - both quasistatic cyclic and shaking table tests of components, assemblages and structures; and the evaluation of seismic vulnerability of existing construction particularly in the Eastern US and regions of low to moderate seismicity.

The engineering professional is targeted as the end-user of the tools and techniques developed. The most recent release version of IDARC executes on personal computers and provides options for a host of simple monotonic analyses that relate directly to current efforts in new and improved seismic design procedures.

Calibrating the analytical models to observed test performance will pave the way for eventual simulation studies of the entire stock of concrete buildings in the Eastern and Central US. In addition to studies of damage prediction in the event of an earthquake, these tools can also assist in evaluating the performance of retrofit strategies.

NIST has adopted IDARC as the primary program for their research work and IDARC was also selected as the computational tool to carry out a detailed analysis of the Cypress Viaduct in Oakland, California in a USGS project.

Structural Control Applications

The US Court of Appeals Ninth District building in San Francisco was completed in 1905 and expanded in 1933. The historic building suffered damage during the 1989 Loma Prieta Earthquake. In the same year, NCEER and Earthquake Protection Systems, Inc., began collaborating on the shake table testing of large multi-story steel moment frame models with Friction Pendulum System (FPS) isolators. The FPS device is a form of spherically-shaped, articulated sliding bearing (Constantinou, 1994).

The research on the seismic isolation of buildings, together with research conducted in parallel on the seismic isolation of bridges, established FPS as a highly researched, well understood and effective seismic isolation system. As a result of this research, FPS was selected for the seismic isolation of the US Court of Appeals building. The selection of FPS resulted in considerable cost savings in the isolation system chosen and in related construction and scheduling costs (Mokha et al., 1991).

Nonstructural Components

The **Nonstructural Components Project** focuses on minimizing damage to building equipment and content by improving methods of analysis and design and development of innovative support and restraint systems for specialized items such as computer mainframes, hospital equipment and museum artifacts.

Observed nonstructural damage during major earthquakes has played an important role in the investigation of a wide range of research issues dealing with seismic behavior of nonstructural components. These include formulation of codes and provisions for nonstructural components which specify safety levels to which these components must be anchored or attached to a primary structure, and the development of seismic design guidelines for their anchorage details and restraints.

The Northridge earthquake is viewed as one of the most information-rich events for addressing issues related to the observed seismic behavior of nonstructural components. The objectives of this NCEER study are to assess the effectiveness of recently recommended 1994 NEHRP design criteria accounting for various dynamic effects on nonstructural components and to evaluate the performance of a selected group of seismic restraints used in the affected area. It is expected that these studies will lead to the development of safer and more rational design guidelines for operating these components in a seismic environment.

In another NCEER project, protection technology development employing passive, active, and hybrid control systems is used to minimize damage from seismic loads to sensitive equipment and critical subsystems. These activities take advantage of parallel advancements in passive/active/hybrid protective systems technology and involve experimental verification and close interaction with industry. In cooperation with IBM and other industrial participants, the focus of this research task has been on the development of innovative restraining devices (passive and active) for sensitive instruments and equipment. A series of experiments involving computer equipment using some of the innovative support systems as well as conventional systems have been conducted with some innovative support systems showing significant performance improvements over conventional systems. The types of innovative devices under research and development include: shape-memory alloy materials, electro-rheological fluid isolation systems, and isolation systems with active control.

Lifelines

Lifeline damage in recent earthquakes and the multimillion dollar losses that result from service disruption have reinforced the need for comprehensive studies on these elements of the built environment. Recognizing the dangerous potential for fire following earthquakes, NCEER's **Lifelines Project** has performed detailed studies of the water supply systems in San Francisco and Memphis. Studies were then expanded to oil and gas transmission lines in the Central US and more recently to include electric power and telecommunications systems. Supporting geotechnical studies have focused on liquefaction and other aspects of large ground deformation which have serious consequences for buried lifelines. Additional research including systems modeling, fragility assessments, the economic impacts of service disruption, and consequences of hazardous materials release have contributed to advanced understanding of lifeline performance. The 1989 Loma Prieta and 1994 Northridge earthquakes provided additional opportunities to validate some of NCEER's ongoing studies and to redirect some other relevant work to benefit from lessons arising from the two events.

The serviceability of water supply systems in San Francisco has been studied by NCEER since 1987, along with their ability to control fire following earthquakes. The objectives of the study (O'Rourke et al., 1990a) have been to promote advanced computer graphic techniques for lifeline systems and to validate these methods through application to a real system, while demonstrating potential improvements which can be made to existing systems to improve seismic performance. This cooperative study has involved NCEER researchers at Cornell and EQE, as well as end-users in the Water and Fire Departments of the City of San Francisco. A computer code developed at Cornell for the hydraulic analysis and seismic evaluation of water delivery systems, GISALLE (**G**raphical **I**nteractive **S**erviceability **A**nalysis for **L**ifeline **E**ngineering), has been structured to account for various flow patterns likely to occur in damaged systems. The code has been used to help improve the seismic capabilities of the auxiliary water supply system and the emergency plans of the San Francisco Fire Department. The strong working relationship established between NCEER researchers and officials of the City of San Francisco Water and Fire Departments as a result of this effort was instrumental in the passage of a $46.2 million bond issue to improve the seismic response of the city's water supply system prior to Loma Prieta.

The computer code was later validated against pressures and flows demonstrated by damaged systems in the Marina District after the Loma Prieta earthquake and was proven to be consistent with these measurements.

The effort has emphasized the contribution of site response in damage to lifeline systems and that areas of high likelihood of water main breakage during earthquakes can be identified with existing modeling techniques. Emergency response can then be augmented by the availability of portable water supply systems such as those which were put in place by the City of San Francisco prior to the 1989 event (O'Rourke et al., 1990b).

Loma Prieta also caused disruption to utility services in areas closer to the epicenter, such as the community of Watsonville, which experienced damage and disruption to its electric power and water supply systems, straining the emergency response capabilities of the city. NCEER researchers conducted a followup study to determine how a community provides essential services to its residents in spite of lifeline damage and to examine the mechanisms in place to restore those services (Isenberg et al, 1990). The most significant problem observed in Watsonville was the failure of electric power and the importance of availability of electric generators to enable continued operation of the local hospital, water well pumps and sanitary system lift stations. Loss of lifeline service, and, in particular, that of electrical power, was a primary source of damage and economic interruption to businesses in the area.

NCEER researchers are also working closely with Los Angeles Department of Water & Power (LADWP) to develop improved procedures that can then be used to better predict lifeline performance in future earthquakes for the LADWP and other lifeline systems. The same tools, methods and procedures will be used to evaluate the seismic performance of the LADWP systems since the 1971 San Fernando earthquake, to examine the effectiveness of retrofit measures which were implemented as a result of the 1971 event. Another ongoing effort involves NCEER researchers and colleagues at the Southern California Gas Company. The objective of the collaborative effort is to identify potential vulnerabilities of the Company's extensive gas system. Data on areas of critical gas line failure as a result of the 1994 Northridge event, as shown in Figures 3a and 3b, are expected to confirm some particular system vulnerabilities that may be important in subsequent earthquakes. The results of the studies are intended to assist management in the development of effective preparedness and response procedures and repair strategies (O'Rourke and Palmer, 1994).

Highways and Bridges

Transportation systems are major lifelines which form the focus of the **Highway Project**. All components of both new and existing highway systems are being studied, including short and long span bridges. Again, contributing research areas examine seismic and geotechnical hazards, develop both conventional and innovative retrofit methods by using smart materials and protective systems, apply risk assessment methodologies to highway networks, define importance and performance criteria, improve prioritization procedures, and estimate the highway stock. The development of manuals for the evaluation and retrofit of existing lifelines and the preparation of performance-based standards for all new lifelines are long term goals.

A multidisciplinary NCEER study being conducted as part of its Federal Highway Administration (FHWA) activities is using the Loma Prieta earthquake to examine the influence of the spatial variation in the ground motion on the performance of the southbound I-5/SR14 Separation and Overhead. It has been speculated that out-of-phase ground motions between bridge columns may have contributed to the collapse of this structure. In the first part of this project, ground motions at the site are estimated based on the aftershock records and evaluation of the seismology of the region. This is then followed by a series of structural analyses of increasing complexity to determine the consequences of spatial variation on structural performance. The results of this study will have application to the design of future long-span and multi-span structures as well as the retrofit of existing structures of this type.

Under the Highway Project, NCEER also participated in a seismic hazard assessment of the Queensboro Bridge in New York City, a historic bridge nearly a century old, which is a major river crossing linking the boroughs of Manhattan and Queens. The objective of the project was to establish site specific seismic response spectra and time histories at the foundation level for each of the major structures of the bridge, for (1) a low to moderate earthquake which may be expected to occur several times throughout the life of the bridge, and (2) a severe earthquake which may occur only once in its lifetime.

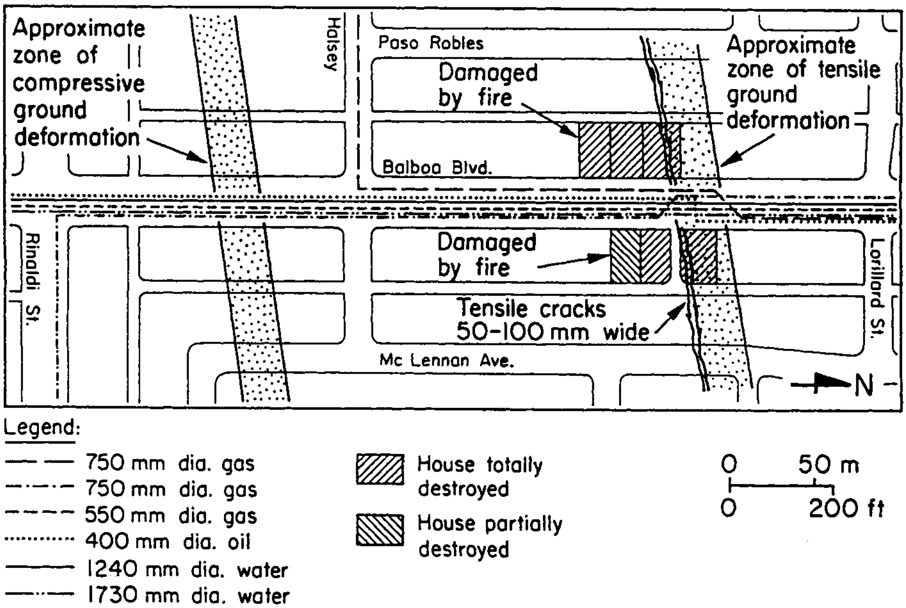

Fig. 3a. Map of major Pipelines, Fire Damage, and Ground Deformation on Balboa Boulevard

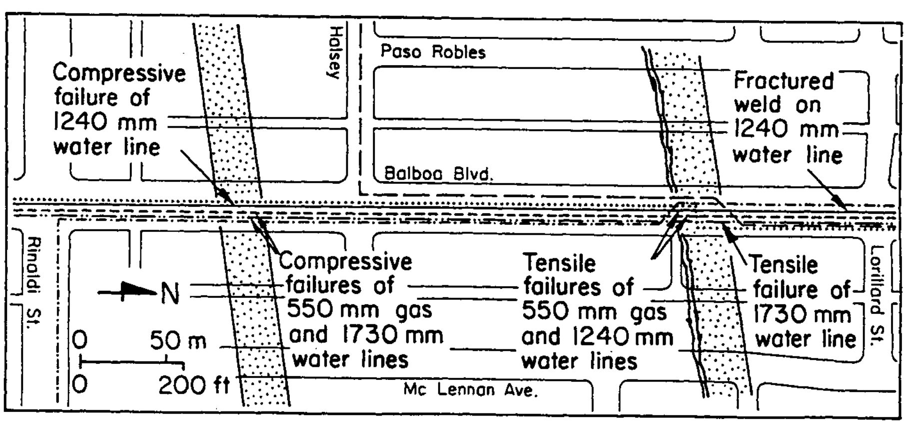

Fig. 3b. Map of major Pipelines, Ground Deformation Zones, and Locations of Pipeline Damage on Balboa Boulevard

The project included a review of past and current information on the seismicity, geology and fault patterns of the region around New York City, and a review of existing national, New York State, and New York City seismic design levels, in order to determine the seismic exposure of the site around the Queensboro Bridge. Consideration was given, but not limited, to the frequency with which earthquakes of varying size occur, their expected locations, the characteristics of ground motion generated at the site, and the geotechnical properties of the site (Friedland et al., 1994).

IMPACT ON NCEER'S IMPLEMENTATION EFFORTS

It is well recognized that the implementation of research findings and application of potential seismic solutions is critical to the effectiveness of natural hazards mitigation efforts. Knowledge transfer is a complex and dynamic process which is influenced by many external variables which in turn affect the ultimate utilization of research findings.

Successful development of implementable results requires earth scientists to interact with engineers, engineers to work more closely with social scientists, and researchers to more actively involve potential end users in the research process. In addition to recognizing the need and demand for research, and the development of practices and applications, users must also accept the feasibility and cost benefits of the technological solutions. Implementation of engineering solutions is further complicated by the need for a balanced approach to technical, societal, and political considerations and the complex processes which must be negotiated to implement innovations. The presence of well-informed advocates within the process is viewed as essential to effective implementation.

It is clear that an outstanding factor in overcoming these obstacles is the education of many constituencies from researcher to policymaker. Damaging earthquakes serve as powerful educational tools. But education of these varied groups can also be accomplished through a number of different mechanisms such as: involvement of researchers and policy makers in problem-focused research projects; public awareness outreach activities; information access and publications; conferences and seminars; the development of codes, standards and guidelines; and demonstration projects. Some specific examples of NCEER's activities to further knowledge transfer and implementation of research are summarized in the following paragraphs.

The ATC-34 project is a joint study by NCEER and the Applied Technology Council into response modification factors and other earthquake-resistant design code issues. The initial objectives of the effort were to review the formulation of the response modification factors used in seismic codes in the US, and the deficiencies and strengths of current seismic design codes, and to highlight issues requiring research or study to improve seismic design practice in the US. The results of these studies are included in the first ATC-34 report, "A Critical Review of Current Code Approaches to Earthquake-Resistant Design."

This report is significant to both the scientific community and engineers in private practice. For both the scientific community and design professional, the report provides a state-of-the-art overview of current practice; and, the results of recent research on response modification factors, displacement amplification factors, and risk and reliability in the seismic environment. The report should help academicians to better focus their current and future research, and provide design professionals with greater insight into the shortcomings of current seismic design practice. The draft report has already been used as reference and resource material by other teams involved in developing seismic design codes and guidelines (SEAOC Vision 2000; and FEMA/EERC Problem Focused Studies).

The focus of the project to date has been building structures. Other types of engineered civil structures such as industrial facilities, dams, lifelines and transportation structures have not been considered, although many observations are directly applicable to bridge structures.

Another major NCEER effort is devoted to the 1994 update of the NEHRP provisions for nonstructural components. The thrust of this work is to critically assess current seismic design formulas for nonstructural components and to identify some of their shortcomings. Improvements to these formulas are recommended which preserve the equivalent lateral force format for design and applicability and corrects some of their deficiencies based on analyses, experimental results and observation data from past earthquakes. The recommended revision takes into account both effects of nonstructural component anchorage detailing and its supporting structural characteristics. Three case studies show the relative conservativeness involved in different codes and the importance of the factors ignored in the current provisions. Necessity for deformation constraints in component design motivates the development of simple displacement equations. The products of this activity should serve as useful tools for design engineers and architects. This effort is carried out through a close working relationship with members of Task Subcommittee 8, Building Seismic Safety Council (BSSC).

In a full-scale implementation project, NCEER has recently negotiated with the Naval Facilities Engineering Command, Department of the Navy to design and implement viscoelastic dampers for a Navy-owned reinforced concrete structure to provide seismic hazard reduction. The work includes the design, construction and performance monitoring of viscoelastic dampers as passive energy dissipation devices to be installed in the building. The project will serve as a demonstration of the feasibility of this innovative technology for seismic strengthening of similar buildings located in high seismic risk areas.

This project represents the demonstration of extensive NCEER research over the last few years in the seismic applications of viscoelastic dampers. Through analysis, laboratory experiments, and full-scale structural tests, NCEER research provided information on the dynamic behavior of viscoelastic materials in the seismic environment, demonstrated the viability of incorporating viscoelastic dampers into new or existing structures for seismic hazard reduction, and developed design procedures for their use in steel-frame and concrete structures. While full-scale implementation of viscoelastic dampers to steel-frame structures has taken place, the Navy Project provides the first application of this innovative technology to a reinforced concrete structure (Soong, 1995).

In 1983, the Federal Highway Administration (FHWA) published a set of guidelines for seismic retrofitting of highway bridges. Since the 10 years of its publication, significant progress in assessing the seismic hazard, understanding the response of bridges and in the development of retrofit technologies has been made. Many of these advances are the result of an aggressive research program by Caltrans after the Loma Prieta earthquake. In response to these advances the FHWA initiated a project to update the 1983 guidelines, with the new **Seismic Retrofitting Manual for Highways and Bridges** completed by NCEER in the spring of 1994 (Buckle et al, 1994).

The new manual is based primarily on research conducted during the development of the 1983 guidelines by ATC, current Caltrans design aids, and recent research conducted at the University of California at San Diego and elsewhere. The manual offers procedures for evaluating and upgrading the seismic resistance of existing highways and bridges. Specifically it contains, (1) a preliminary screening process to identify and prioritize bridges that need to be evaluated for seismic retrofitting; (2) two alternative methodologies for quantitatively evaluating the seismic capacity of an existing bridge and determining the overall effectiveness of alternative seismic retrofitting measures; and (3) retrofit measures and design requirements for increasing the seismic resistance of existing bridges.

For seismic retrofitting, bridges must be classified according to their Seismic Performance Category (SPC) which is based on seismic hazard and structure importance. Two importance classifications are specified in the manual, "Essential" - bridges which must continue to function after an earthquake or which cross routes that must continue to operate immediately after an earthquake, and "Standard" - all other bridges. The Importance Classification is necessary to determine societal/survival and security/defense requirements. SPC's differ from the AASHTO Standard Specifications for Highway Bridges (new design) where no allowance for structure importance is made in seismic zones with acceleration coefficients less 0.29. It is particularly important to distinguish the difference between "essential" and "standard" structures in low to moderate seismic zones because of the high cost of retrofitting.

Seismic retrofitting is only one solution for minimizing hazard of existing bridges that are vulnerable to serious damage during an earthquake. Others include bridge closure, bridge replacement or acceptance of the risk of seismic damage. While bridge closure or replacement are generally only considered when other deficiencies exist, it is necessary to determine the cost and effectiveness of accepting or retrofitting for the seismic risk.

This assessment consists of preliminary screening, detailed evaluation, and design of retrofit measures. In the US, retrofit based on conventional strengthening to increase capacity of the structure to meet its demand is most commonly used. The decision to use a retrofitting scheme is based on its effectiveness in preventing unacceptable performance, the cost of retrofitting, and the remaining service life of the bridge.

CONCLUSIONS

Earthquakes continue to be the dynamic laboratory within which engineering and social solutions are tested and perfected. Successes are gauged by the number of lives saved, the ability of society to carry on in spite of disaster, and the quickness with which the normal functions of an impacted area can be restored. While considerable progress has been made to improve the earthquake resistance of our environment, organizations which work to mitigate the earthquake hazard are obliged to make the most of the lessons of earthquake history, which are destined to be repeated unless they are heeded.

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ORGANIZATION AND MANAGEMENT OF RECONSTRUCTION WORK IN LANCANG-GENGMA EARTHQUAKE-STRICKEN AREAS

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ABSTRACT

This paper looks at the earthquake which occurred on November 6th, 1988, in Lancang-Gengma area, Yunnan province, and introduces the effective reconstruction work organized and administered by Yunnan Construction Bureau in those earthquake-stricken areas.

KEYWORDS

Earthquake; repair and reinforcement; planning; reconstruction; damage.

INTRODUCTION

An earthquake, which registered 7.6 and 7.2 on the Richter scale, shook the Lancang-Gengma area in the southwest region of Yunnan province on November 6th, 1988. This was another disastrous shock following three earthquakes which registered greater than 7.0 on the Richter scale in Tonghai, Zhaotong and Longling in the 1970s. More than 30 towns and counties in 5 prefectures, including 2 national autonomous prefectures, with an area of 120,000km were affected. The direct economic loss was 2.75 billion yuan. More than 7,000 persons were injured or died. Losses caused by the disaster are second only to those of the 1976 earthquake in Tangshan, China. In 1988 the severely afflicted area covered seven counties among which six are national autonomous counties. Eight ethnic minority nationalities, such as Lahu, Wa and Dai people with a population of 1,852,800, suffered from the earthquake.

The earthquake-stricken area is located in the southern section of HengDuan Mountain in China, which is strategically situated. Access is difficult. Mountains comprise 90% of the earthquake-stricken area. Although natural resources are rich in that region, the standard of living is lower than that in the interior of the province. Among 30 damaged towns and counties, most are subsidized by the state.

Generally, it was difficult to carry large-scale reconstruction with the weak economic base, low level of productivity, poor transport facilities, and shortage of qualified personnel. In view of the practical situation in the disaster area, we organized and administered serious reconstruction work in order to resume production and normal life and to strengthen the unity of ethnic minorty nationalities for stability on the frontier.

ORGANIZATION AND MANAGEMENT OF RECONSTRUCTION

Thanks to the profound concern of the party central committee and the state council, Yunnan provincial government formulated a policy and three principles for reconstruction in the earthquake-stricken areas according to instructions given by Premier Li Peng when he inspected conditions after the disaster. In consideration of numerous facts, such as state economic capability, foreign aid, local geographic features, condition of local economic development and technical personnel, the government formulated a policy. It called the people in earthquake-stricken areas to rebuild their homeland through productive development, self-reliance and hard work. The three principles were: to give prominence to the projects closely related to mass life; to assign a head of a prefecture or county to direct reconstruction in a certain area; to carry out reconstruction by stages under overall planning.

Under the unified leadership of the Provincial Party Committee and government, the Construction Bureau played the role of governmental administrative department and organized reconstruction work with effective measures according to the characteristics of each stage and knowledge coming from experience. In this way, the measures taken for reconstruction work were practical, and remarkable success was achieved. The main measures are as follows.

Investigation as Guidance to Determine Repair and Reinforcement of Damaged Buildings

After an immediate and detailed investigation into the situation in earthquake-stricken areas, plans and criteria for repairs and reinforcement were made in order to guide technically the determination of a damaged building's risk level. Some buildings were repaired and reinforced instead of being demolished so that the investment which could be used to build new houses was saved.

After the earthquake occurred, eight groups consisting of heads, specialists and engineering technicians from YCB's subordinate units, as well as leaders from YCB and its sections, numbering more than 100, were sent by YCB to disaster areas. This was in accordance with the unified plan of the provincial government to investigate conditions of the earthquake disaster and to indicate the rush-repairs of important public buildings and key projects. A group from Yunnan Design Institute, for instance, made its suggestions on site for the repair and reinforcement of the main factory buildings at eight sugar refineries and cane mills in six counties after an on-the-spot check. Since the buildings were reinforced in time, the factories all went into operation on schedule. In order to ensure accurate statistical figures about the damage from the earthquake being reported to the leadership without delay, YCB instructed the Urban and Rural Construction Cadre's School to conduct short-term training courses in Lincang and Simao prefectures for the determination of dangerous buildings' earthquake-resistant capacity; 155 persons were trained. This mitigated the shortage of personnel for overall determination of damaged building's risk level; findings were reported to the leadership for elimination of dangerous buildings. At the same time, YCB submitted the "Report on the Determination of Building's Risk Level and Measures for Repair and Reinforcement of Damaged Buildings in Severely Afflicted Areas" to the provincial government, in which clear criteria for the determination of building's risk level and reinforcement were provided. Thirty-one buildings, nearly 60 thousand square meters, which might otherwise have been demolished, were repaired and reinforced so that the investment for rebuilding was saved.

Unified Plan for Reconstruction Work

Through comprehensive and rational planning, investigation and surveying, the reconstruction work in earthquake-stricken areas was carried out and controlled under a unified plan.

In order to guarantee smooth reconstruction, the units of planning, investigation, surveying and design in Yunnan province were organized by YCB. They worked upon a unified plan for administration of reconstruction in counties and towns in the severely afflicted area.

Before the earthquake, there was no proper arrangement for construction in some counties and towns so that the houses were built in a disorderly manner. Some buildings were located on a landslide. Some places were not suited for building sites. In view of this situation, the engineers and technicians worked hard and drew up reconstruction plans for 4 severely afflicted counties and 18 towns in only one year. Gengma county was a focal point of reconstruction in a severely afflicted area. As to the problem to rebuild a new county town on the old site or on a new site, an investigation group was sent by YCB to study the realities in depth. Then the "Report on Whether Gengma County Town Should Be Moved or Not" and the "Report on Reconstruction of Gengma County Town after the Earthquake" were submitted. In these two reports, it was proposed that the county town should be rebuilt on the old site. According to the overall plan, a landslide to the north of the county town should be avoided and the new district of the town should be developed toward the south. After thorough discussion in six meetings attended by specialists, and two meetings for decisionmaking attended by the leaders of the provincial government, the overall plan for reconstruction of Gengma county town was approved by the government. It has been proven by experience that it was wise to rebuild the new county town on the old site. Now the Gengma county town is a picture of prosperity with its new south district where crisscross streets are lined with buildings in national ethnic styles.

A Series of Documents and Regulations as Direction of Reconstruction

The function of administrative departments in supervision and control was strengthened to give full play for reconstruction through formulating a series of documents and regulations.

Earthquake magnitude was great and large mountainous districts on the frontier, inhabited by ethnic minority nationalities, were affected. It was the first time any province had to carry out such large-scale reconstruction work without prior experience. Taking one policy and three principles as a guiding ideology, a series of documents, such as "Method of Administration for Rebuilding of Rural Houses in Yunnan Earthquake Stricken Areas", "Temporary Provisions on Questions to Administer Engineering Earthquake-Resistant Technology", and "Provisions on Strengthening Construction Management in Earthquake-Stricken Areas", was produced by YCB. Thus building construction and technical management could proceed under properly developed rules and regulations. In-depth studies of the realities were made and various specialist meetings were called to solve problems promptly when they were discovered. During the period of reconstruction, persons who were heads of the units of planning, geotechnical investigation, design and construction were called together to Gengma county to attend a meeting at which five decisions were made to fix funds for rebuilding and personnel responsible for when each project should be started and finished. Strict regulations were also formulated at the meeting to administer the work of planning, design and construction. Consequently, design and construction that had been in confusion for a time in some districts were clarified.

Qualified Technicians Ensure the Quality of Reconstruction

A team consisting of qualified engineering technicians for construction supervision was organized to ensure the quality and schedule of reconstruction projects. Deputy heads in charge of science and technology in counties and engineering specialists, including planners, architects, structural engineers, preliminary estimate personnel and supervisors, were sent to the earthquake-stricken areas. This was conducive to planning and construction, control of engineering quality, and money-saving in the disaster areas.

Because of poor economic conditions and shortages of special technicians in Lancang-Gengma area, there were many difficulties in reconstruction. So measures were taken in this connection by YCB to administer and supervise the whole procedure in capital construction, involving site selection and geotechnical investigation, for reconstruction to proceed smoothly.

A cadre who had great ability to be the county's deputy head in charge of science and technology was sent to Gengma county to oversee reconstruction.

Based on damage to schools and hospital buildings, YCB, in cooperation with the Provincial Education Commission and the Provincial Public Health Bureau, sponsored a competition of **blueprints** for primary and middle schools and hospitals in counties and towns. Excellent plans were thus available for selection by users. This speeded up the rebuilding of schools and hospitals in the earthquake-stricken areas.

Eight groups consisting of specialists in planning, design, quality control and supervision, numbering 231 persons, were sent to severely afflicted areas to develop technical criteria, or to defer them unless their design and construction had been revised. Besides, more than 600 technicians from the provincial, prefectural and county units of design, construction, supervision and estimation were sent by YCB directly to administer and supervise some key projects on site for the purpose of quality control so that the new structures could sustain earthquakes in the future. In four years, the technicians in the groups had examined the drawings of 675 projects, checked the preliminary estimates of 1,046 items, and supervised 1,084 projects, saving 17.05 million yuan.

In cooperation with authorities concerned, on-the-spot offices of reconstruction were set up in Lancang, Gengma and Cangyuan, the three counties afflicted severely by the earthquake. They coordinated reconstruction work and dealt with problems that had cropped up in planning, design and construction. A detailed plan for a new district in Gengma county was examined and approved on site.

It has been proven by experience that good results were attained in organization and management of reconstruction in the earthquake-stricken areas by YCB with implementation of "one policy and three principles" under the leadership of the party committee and the government of Yunnan province. In four years, 5,925 reconstruction projects with an investment of 1.25 billion yuan and a building area of 4.09 million square metres were completed in the earthquake-stricken areas. Numbers of public buildings such as primary and middle schools, hospitals and clinics were repaired or rebuilt. Infrastructure and rural houses were also improved as much as possible. County towns of Gengma, Lancang and Cangyuan have a new appearance. Through the experience of many years, not only was reconstruction satisfactorily completed, but also a large number of technicians were trained. An exemplary management system with effective measures was formed and valuable knowledge has been accumulated for reconstruction after earthquakes in our province and other parts of the country.

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ANTICIPATED BUSINESS DISRUPTION EFFECTS DUE TO EARTHQUAKE-INDUCED LIFELINE INTERRUPTIONS[1]

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ABSTRACT

The importance of continuity in the provision of lifeline services has been long recognized with respect to facilitating emergency response to a major earthquake, but little attention has been directed toward the importance of lifeline continuity for minimizing economic disruption. This paper focuses on the potential indirect economic impacts of lifeline disruption on businesses. A study was conducted with a random sample of businesses in Memphis, Tennessee to determine their dependence on various lifeline systems and what level of economic impacts businesses would experience if those systems failed due to a damaging earthquake event. Findings point to the need to address this problem with a collective approach by integrating lifeline service providers into community recovery planning and by involving business associations in educational programs for their members.

KEYWORDS

Earthquake; Business; Lifeline; Economic Loss; Disaster Impacts.

INTRODUCTION

One of the major problems in anticipating the magnitude of economic losses that can be caused by a destructive earthquake is understanding the various complex ways in which the economic sector--including both large and small businesses--can be effected. This information is extremely important to communities that are trying to develop both emergency response plans as well as community recovery plans. While engineers have been working on loss estimation methodologies for the last 15 years (cf, NAS, 1989) the

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losses they have principally been concerned about were those due to direct (or primary) effects of earthquakes; that is, the structural and non-structural damage to buildings and their systems due to ground motion.

In contrast, the approach taken in this paper focuses on the indirect (or secondary) effects of an earthquake that prohibit businesses **without** primary damage from operating in a normal way. One of the primary ways in which undamaged businesses' activities could be restricted is through the loss of lifeline services. Tierney (1994), in a recent analysis of the impacts of the 1993 Midwest Floods on Des Moines, Iowa, concluded that damage to lifelines and resultant service interruptions were a more important cause of business disruption than was the physical flooding itself.

As recently illustrated by both the Northridge and Kobe earthquakes, the effects of lifeline failures go far beyond the immediate impact area, creating problems for the economic health of an entire metropolitan area and, perhaps, to a larger national region. However, almost no research has addressed this question of the relationship between lifeline failures due to an earthquake and economic consequences for the affected area. This paper begins to address this neglected area of research.

Literature On Business Disruption

As significant as the economic sector obviously is in the recovery process for any community, it has received almost no attention from social scientists involved in disaster research. In his extensive review of disaster research findings, Drabek (1986) does not even mention the economic sector. After an exhaustive review of the literature on business recovery following a disaster, Dahlhamer (1992) found only three studies (Durkin 1984; French et al. 1984; Nigg and Tierney 1990) that specifically addressed this issue.

However, sprinkled throughout the disaster literature are indications of the disruption of community life due to disaster impacts on the business community. For example, one of the first events that documented the impacts of disaster on business communities was the Xenia tornado of April 3, 1974. The entire downtown area, housing the city's business district, was devastated. Approximately 155 commercial and four industrial businesses in 121 structures were destroyed, including eight supermarkets. One hundred other businesses suffered major or minor damage (DRC 1976).

More recently, the downtown business district of Santa Cruz, California was devastated by the October 17, 1989 Loma Prieta earthquake. It was estimated that 60% (approximately 650) of the downtown businesses were destroyed or sufficiently damaged to require at least temporary closure (DRC 1993).

There are two compelling reasons why social scientists should study the business recovery process. First, businesses as units of analysis have many of the same characteristics as households: they vary in size, they have incomes, they age, they have socioeconomic locations in the social structure, they are physically housed in structures that are more or less vulnerable, they may be embedded in a network of community organizations, and the types and amount of resources they have access to varies. On the basis of these characteristics, some businesses are obviously going to be less vulnerable to a disaster agent and more capable of recovering from disaster impacts.

This differential in business vulnerability raises questions about the adequacy of programs available to businesses to assist them recover, and whether those programs have similar problems of availability for certain classes of businesses (as was found for certain classes of families). Dahlhamer (1992) did find evidence that some types of businesses have greater success in obtaining governmental loans. In general, he found that businesses with older owners, that were located in a building also owned by the business owner, and whose owner had good credit could get a federal loan more easily than other types of owners.

He also found that some business owners got more favorable loan terms than others. Dahlhamer concluded that the federal disaster loan program was systematically not assisting those businesses that needed the greatest amount of assistance to recover but was aiding those businesses that could have gotten loans from commercial sources.

Second, businesses play vital roles in communities by providing goods and services to specific client groups, as well as providing employment opportunities for community residents. If businesses must close due to structural damage, inventory losses, losses of employees, or losses of markets, what consequences does that have for both family and community recovery? Obviously, the longer businesses are closed, the greater the economic strain on families whose members were employed by those enterprises. Also, when businesses that provide basic goods and services (e.g., markets, clothing stores, gas stations, banks, utility companies) to community residents are not operational, the greater the temporal constraint--the length of time it takes household members to complete routine daily tasks--on family recovery (Trainer and Bolin 1976).

Beyond these obvious implications for household recovery, community recovery can be effected by business disruption in two important ways. First, the longer commercial enterprises are non-operational, the greater the impact on revenues for the local government. Local governments receive a great deal of their operational income by collecting fees and taxes on commercial transactions or from property taxes. Following a disaster, a community's revenues from these sources may drop dramatically, until property owners can repair commercial buildings and businesses can recover sufficiently to put employees back to work, providing goods and services. While there is some compensation for the decline in these revenues to local government from the infusion of external aid, this intervention is short-lived. If the business sector does not sufficiently recover, community-based services (public works maintenance, social and health services, schools, cultural and recreational programs, and planned economic development initiatives) will be cut back, delayed, or eliminated.

Second, some businesses serve the needs of particular neighborhoods and rely on local residents to use their establishments. When such businesses can not recover from a disaster, what consequences does this have for the neighborhood or the community that business serves? Some research has suggested that the character of the community may actually be changed if people have to leave their neighborhoods to market, shop, bank, and use recreational facilities, or if their children have to go to schools at a greater distance from their homes. Bondedness to the neighborhood may decline; and the businesses that remain may actually be hurt by the general decline in foot-traffic or normal transportation patterns that had supported them in pre-disaster times.

The Memphis Business Study: Research Design

This study was conducted with businesses in Memphis and Shelby County, Tennessee in June, 1993. Memphis resides in one of the highest Modified Mercalli Intensity zones projected from the 1811-1812 great earthquakes that occurred within the New Madrid Fault zone (see Figure 1). The area around Memphis has not had any significant, large magnitude earthquakes in the recent century; however, earth scientists are anticipating that this area could experience an earthquake of magnitude 6 within the next 20 years.

Until recently, neither the building code for the state of Tennessee nor the city of Memphis required seismic design in engineered structures; and none of the lifeline companies in the area had taken seismic ground forces into consideration in the design of their systems. Because of the seismic hazard and vulnerability and because Memphis plays such a prominent role in the economic health of the region (including northeastern Arkansas), Shelby County was an excellent candidate site for this study.

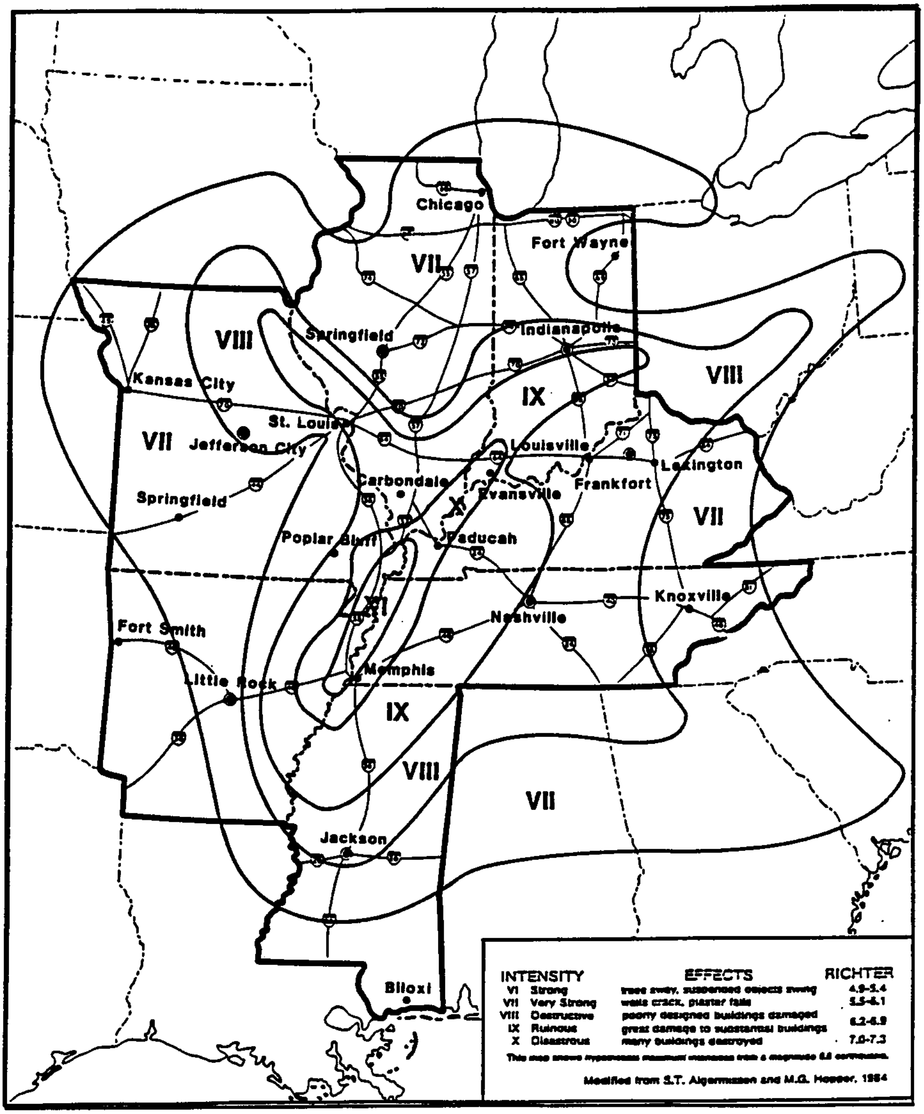

Fig. 1. Map of the New Madrid Fault System with Modified Mercalli Intensity Zones

Shelby County had 34,106 businesses, which was the sampling frame for this study. These businesses were stratified on two criteria. First, because it was hypothesized that different types of businesses might have different needs for lifeline services, businesses were classified into five sectors. Those sectors were: wholesale and retail trade; manufacturing and construction; business and professional services; financial, real estate, and insurance; and a residual category of other businesses (including agriculture, fishing, mining, forestry, and transportation).

Second, it was hypothesized that large businesses (that is, those that employ more workers), if they could not function, could have a potentially larger negative impact on the economics of the region. Therefore businesses were also stratified into two categories: small businesses having 19 or fewer employees, and large businesses with 20 or more employees.

We then sampled proportionately from each of the 10 subsamples, selecting 1840 businesses to be included in the study. This produced a sampling error of +/- 4%. Self-completion questionnaires were mailed to the businesses, with a cover letter requesting that the survey instrument be completed by the owner or manager of the business. Two weeks after the initial mailing, respondents were sent a reminder postcard; and four weeks later a second mailing was sent to all businesses that had not yet responded. A follow-up phone call was then made to each non-responding business to encourage their participation in the study. After four months, 735 questionnaires had been returned, yielding a response rate of 40%.

Characteristics Of Businesses In The Study

Before looking at how these businesses evaluated their reliance on lifelines, it is important to know something about them--how old they were, what type of ownership the business had, whether they owned the building that housed their business, and how many employees they had.

Businesses ranged from new enterprises, open for one year or less, to established companies, the oldest having been in business for 139 years. The average time they had been in operation was 22 years. Businesses in the service sector and smaller businesses were significantly newer than other types of businesses. Over two-thirds (69%) of the businesses were independent; that is, they were not a part of a franchise or a chain). Although the average number of employees was 60, the median number of employees was only 6, indicating the most businesses were small.

Only 37% of the sample owned the building which housed their business; others were renting or leasing their facility. In general, manufacturing operations were more likely to own their own buildings than were businesses in other sectors. Also, large businesses were more likely than small businesses to own their facilities.

Importance Of Lifelines To Business Operations

Since we are interested in the effects of lifeline system interruptions or failures on business, we first had to determine which systems are most important to the businesses' operations during normal times. Owners or managers were asked to indicate, generally, the degree of importance each type of lifeline service had for the business under normal conditions. In Table 1, we see that electrical and telephone services are "very important" (82% and 78%, respectively) for all types of the businesses. The availability of water and natural gas, and the ability to discharge wastewater were deemed to be much less important services on a day-to-day basis.

A similar pattern was seen when owners were asked how many hours their businesses could continue to operate if a particular lifeline service was not available (Table 2). Without electricity, the average

Table 1. Importance Of Lifeline Service To Business Operations Under Normal Conditions

IMPORTANCE	LIFELINE SERVICES				
	Electricity	Water	Natural Gas	Wastewater Treatment	Telephone
Very Important	82%	27%	18%	23%	78%
Important	14	34	29	32	17
Not Very Important	3	31	39	33	3
Not Important At All	1	8	13	13	2
TOTAL	100%	100%	99%[1]	101%[1]	100%

[1]Does not total 100% due to rounding.

Table 2. Median Number Of Hours Businesses Could Operate With Lifeline Loss

LIFELINE SERVICE	MEDIAN NUMBER OF HOURS
Electricity	0
Water	48
Natural Gas	120
Wastewater Treatment	48
Telephones	4

company would have to close immediately; without phone or telecommunications service they would last for about 1/2 day. Without water or wastewater services, they would close after two days. Of those companies who used natural gas, they expected they could stay open for about one work week before having to close.

Influence of Sector and Size on Business Operations

We were interested in whether the type of business or its size would have any effect on dependence on lifeline services. As can be seen in Table 3, if interruption to the water system were to occur, there is substantial variation across sectors and between large and small businesses in terms of the length of time they could continue to operate.

In order to investigate this variation empirically, the number of hours a business could remain open if a specific lifeline service were interrupted was used as the dependent variable. ANOVA techniques were employed to look for consistency across the sectors and t-tests were used to compare businesses on the basis of their size.

When electrical service was interrupted, 59% of the businesses reported they would have to shut down immediately; but 5% said they could operate indefinitely without the service. Neither sector nor size mattered (Table 4); that is, the availability of electricity was critical to the functioning of all businesses, irrespective of type or size. While fewer than one-fifth (18%) of the natural gas users would have to shut down immediately, neither economic sector nor size of the business had any effect.

Economic sector was found to be important when considering both the loss of wastewater disposal and telephone services. Although one in five businesses reported that they would have to close immediately if wastewater or sewerage services were lost, service-oriented businesses would have to close significantly sooner than other types of businesses. While 45% of all businesses reportedly would have to close immediately if telephone (or telecommunications) services were lost, those businesses in wholesale or retail trade could stay open significantly longer than businesses in other sectors.

Size of the business was found to be important only when water systems failed. Even though 1/4 of all business would have to cease operations immediately if water weren't available, small businesses could operate significantly longer than larger ones without water.

Effects of the Loss of Multiple Lifeline Systems

What has been learned from past earthquake disasters is that multiple lifeline systems are often disrupted in the same event. In the Northridge earthquake, for example, water, wastewater, and natural gas systems were disrupted over large parts of the San Fernando Valley for an extended period of time. Due to the Kobe earthquake, electrical service to large parts of the metropolitan area was not restored for nearly a week; and natural gas was not restored for several months to parts of the metropolitan area.

In order to investigate what the combined effects of the disruption of these systems would have on a business's ability to continue to operate, a composite measure of disruption was created by counting the number of times an owner indicated that a lifeline service was "very important" to the functioning of the business (as presented in Table 1).[3] As indicated in Table 5, almost 2/3 of the businesses (65%) would be "very disrupted" if two to four of these systems were interrupted by an earthquake. For only two percent of the businesses were no lifeline systems very important, subsequently, they would experience little disruption.

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[3]Because natural gas was not used by everyone, it was deleted from this measure. Disruption only refers to the loss of electricity, phone, water, and wastewater disposal services.

Table 3. Median Number of Hours Businesses Could Operate With Loss of Water By Type And Size Of Business

Type and Size of Business	Median Number of Hours
Wholesale and Retail Trade:	
Small[a] (N=124)	120.0
Large[b] (N=36)	24.0
Manufacturing and Construction:	
Small (N=64)	72.0
Large (N=26)	48.0
Business and Professional Services:	
Small (N=129)	24.0
Large (N=52)	23.5
Finance, Insurance, and Real Estate:	
Small (N=71)	120.0
Large (N=23)	8.0
Other:[c]	
Small (N=61)	168.0
Large (N=29)	72.0
All Businesses (N=627):	48.0

[a] Small businesses are those with 19 or less employees.
[b] Large businesses are those with 20 or more employees.
[c] "Other" consists of agricultural, fishing, forestry, mining, transportation, and public communications firms.

Table 4. Effects Of Sector And Size On Businesses' Ability To Carry On Normal Operations When A Lifeline Service Is Interrupted

Lost Lifeline	% Close Immediately	% Operate Indefinitely	Sector Effects (F)	Size Effects (t)
Electricity	59%	5%	1.53	-.84
Water	25%	12%	.88	-2.31*
Natural Gas*	18%	12%	.21	.56
Wastewater/ Sewerage	20%	a	3.02*	1.12
Phone/ Telecommunications	45%	4%	3.40**	1.11

* Significant at the .05 level
** Significant
[a] Less than 1%

Table 5. Index Of Total Expected Disruption Of Lifeline Systems On Ability To Carry On Business Activities

SEVERITY OF EXPECTED DISRUPTION	PERCENTAGE
Extreme Disruption	28%
Major Disruption	37
Modest Disruption	33
Little Disruption	2
TOTAL %	100%
TOTAL N	(704)

Implications Of Findings On Lifeline Importance

These findings give us an important, and overlooked, perspective on the criticality of lifelines for business continuity in the immediate post-earthquake impact period. Even businesses that could function because their facilities and buildings sustained no structural damage would be disrupted if lifeline systems fail. This study provides an indication of how vulnerable businesses, especially those of a certain size and in specific business sectors, are to economic disruption because of lifeline system failures. And, since we know that multiple lifelines are often severely interrupted during the same event, we have some assessment of the extent of possible business disruption due to the failure of multiple lifeline systems in the same event.

Even if business owners and managers had acted on the need to structurally reinforce their buildings or to lease only properties that were structurally sound, such investments may not protect them from economic disruption. If they lose lifeline services--especially electricity and phone services--they may have to close anyway until those services are restored.

This information on importance of lifeline services gives lifeline providers--both public and private--some indication how quickly they must be able to restore service before substantial secondary economic losses will begin to take place within the functional business community. We now have evidence to indicate that lifeline "hardening" and disaster response planning for service restoration are not only important in assisting the general community during the emergency response period, but are also crucial for reducing post-impact economic losses within the larger area and in speeding the community's recovery.

These findings demonstrate the need for communities to incorporate lifeline service providers into their disaster response and recovery planning in order to decrease the secondary effects of an earthquake event. Clearly, the emergency management officials and leaders in the business community must become more active in making the economic sector aware of the potential problems they could face and develop strategies to deal with these problems.

However, it should not be assumed that business owners can resolve these problems on an individual basis. The potential economic disruption due to lifeline losses is a collective problem, beyond the capability of any individual owner to address. This issue is one that needs to be addressed from a collective basis, including lifelines service providers, emergency management and community officials, and business organizations.

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DECISION-MAKING FOR RECOVERY AND RECONSTRUCTION FOLLOWING A STRONG EARTHQUAKE

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ABSTRACT

This paper presents decision-making methodologies for recovery and reconstruction following a strong earthquake, including decision-making for land use and priority of reconstruction for economic sectors. Reconstruction evaluation tools are also described in the paper.

KEYWORDS

Strong earthquake; recovery; reconstruction; decision-making; land use planning.

INTRODUCTION

The history of earthquake disasters repeatedly exhorts us to be careful of the following two pressures faced immediately after a damaging earthquake.

-- Economic, social, psychological and political pressures foster rehabilitation and reconstruction as rapidly as possible. The prevailing attitude is a desire to help those who have suffered injuries, disruption of their lives and property damage. The overriding concern is with immediate needs, not with future disasters.

-- Recent bitter experience and the concern for significant reduction of future risk foster improving safety in post-earthquake reconstruction. Survivors hope to build and to repair buildings and structures to much better withstand future earthquakes.

Correct and rational decision-making for post-earthquake reconstruction including land-use planning, emergency shelter construction, priority of recovery of economic sectors and financial resources for reconstruction, is the key to resolve the above-mentioned pressures.

LAND-USE DECISION-MAKING

The increase in safety from improved building and structural characteristics is very significant. However, little attention has been given to avoiding or restricting development or reconstruction in areas revealed by the earthquake as especially hazardous. History has shown that well-planned land-use changes following an earthquake can effectively reduce risk from future earthquakes. The first problem faced by the land-use planner is how to select the reconstruction site. One of the following three solutions can be selected by the land-use planner:

1. Rebuild at the original place;

2. Partially rebuild at the original place, partially move to a close neighboring place;

3. Renounce the original place and move to a new place.

The solution for relocation not only needs much more investment than that of rebuilding at the original place, but also needs to break through all kinds of obstructions.

SYSTEM DYNAMICS METHOD FOR RECONSTRUCTION DECISION-MAKING

System dynamics method was published by Professor Jay W. Forester from MIT in his book "Industry Dynamics" in 1960 and used in the project "China in 2000" by professors from Shanghai Jiaotong University in 1984. A series of data in the year of 2000 and 2030 in China was worked out. Future socio-economic development trends in China were analyzed based on these data.

System dynamics model can be applied to analysis of a complicated socio-economic system by computer simulation. In the phase of reconstruction, system dynamics model is suitable for the following four important and complex problems:

- Population recovery and growth;
- Infrastructure system reconstruction;
- Housing and other building construction;
- Priorities for rehabilitation and development of industrial sectors.

The following steps were taken to establish the system dynamics model:

1. Problem identification, including: a) Identifying the problem to be solved and its requirement. b) Determining the main variable. c) Collecting relevant data and analyzing the relationship among different factors and variables.

2. Making causality feedback drawings and setting up model frame, including: a) Determining modulars for the model or submodels and their modulars. b) Making causality feedback return circuit diagram.

3. Making system flow-chart by using system dynamics symbols.

4. Setting up DYNAMO equations, compiling system program by using DYNAMO language and testing on the computer.

5. Drawing up possible solutions for decision-making and doing simulation test on computer, analyzing results for different solutions and proposing recommendations for decision-makers.

In cases of industry development, the simulation test for investment and real estate growth of different industrial sectors in Tangshan city after the earthquake was conducted on the computer by the proposed model. Results have shown that the output value and real estate value given by the computer simulation test are close to the practical values. Thus the model and the selected parameters and equations conform to the practical situation. According to this model, computer simulation analysis for three solutions shown in Table 1 was performed and the results are shown in Table 2.

Table 1 Development solutions and percentage of investment for industrial sectors

Solution	Metal-lurgic	Elec-tric	Coal	Che-mical	Mecha-nical	Bldg. mater.	Forest	Food	Tex-tile	Paper making	Other
1	5.0	37.1	37.2	2.6	5.8	3.1	0.4	1.4	4.0	0.6	2.2
2	6.0	30.0	30.0	3.0	6.0	3.0	0.5	7.0	10.0	2.3	2.2
3	10.0	26.5	26.5	8.0	10.0	3.0	0.5	7.0	10.0	2.3	2.2

Note: Solution 1 represents the case similar to practical one after the earthquake;
Solution 2 the case of increasing investment in light industry and decreasing investment in heavy industry;
Solution 3 the case of increasing investment in light industry and properly increasing investment in some sectors, such as mechanical, metallurgic and chemical sectors.

During the period of rehabilitation in Tangshan city, priority was given to the recovery of the leading industrial sectors before the event. Among the investments, coal and electrical occupy first place; mechanical, metallurgic, textile, building material and chemical come second.

During the post-earthquake rehabilitation phase, it would be better to properly increase the proportion of investment for light industry in the industrial sector as a whole. If this decision had been adopted in the rehabilitation phase, the economic benefit for industry as a whole would have increased, jobs would have grown, tax and profit would have greatly increased. At the same time, the consumption of water, electricity and land increased slightly.

Table 2 Computer simulation results of development solutions for industrial sectors (1985)

Solution	Total output value (TOV)	Recovery percentage (RP)	Tax and profit (TP)	Ratio of TP and TOV	Ratio of TUV and real estate value	Water consumption	Electricity and land consumption
1	311.8e3	132.37	58.52e3	18.77	7.42	less	
2	409.8e3	173.97	72.36e3	17.66	9.76	Slightly more than 1.0	Near 1.0
3	469.2e3	199.20	81.12e3	18.14	10.54	Slightly more than 1.2	More than 1.2

During the post-earthquake rehabilitation phase, if the proportion of investment for heavy industry had been properly adjusted, such as if investment in mechanical, chemical and metallurgic sector had increased and investment in electrical and coal industry had decreased, the total output value, tax, profit and jobs would have increased. However, at the same time, the consumption of water, land and electricity would have greatly risen.

ANALYTICAL HIERARCHY PROCESS METHOD FOR DECISION-MAKING OF INDUSTRY SECTOR REHABILITATION PRIORITY FOLLOWING A DAMAGING EARTHQUAKE

The analytical hierarchy process method was proposed by American operational research professor A.L.Saaty in the early 1970s. It can be used to solve very complicated problems which are difficult to deal with fully by quantitative analysis. The method consists of the following steps.

Problem Identification

Dividing the factors involved in the problem to be solved into several hierarchies and indicating the relationship among the factors. For example, for the decision-making problem, the top hierarchy represents the objective; the middle hierarchy represents links to reach the objective, such as tactics, restrictions and criteria; the lowest hierarchy represents policies and measures to solve the problem.

Structuring Judgment Matrix

If element A_K in hierarchy A is related to element B_i (i=1,2...n) in the lower hierarchy, the judgment matrix B is:

A_k	B_1	B_2	...	B_n
B_1	B_{11}	B_{12}	...	B_{1n}
B_2	B_{21}	B_{22}	...	B_{2n}
			...	
B_n	B_{n1}	B_{n2}	...	B_{nn}

In the above matrix, B represents the relatively important value of B to B for A. Usually, 1, 2, 3, ..., 9 and their reciprocal are taken for B. For example: 1 indicates B is of same importance to B; 3 indicates B is of importance to B; 5 indicates B is of more importance to B; 7 indicates B is of much more importance to B; 9 indicates B is of great importance to B.

Arrange in Importance Order for Hierarchy

For an element in an above hierarchy, the weight W is arranged in importance order for the element in the present hierarchy and related to it can be calculated by the following formula:

$$BW = \lambda_{max} W \tag{1}$$

General Arrangement in Importance Order for Hierarchy

Assuming the weights of elements A_1, A_2, ..., A_m in hierarchy A are a_1, a_2, ..., a_m, respectively and the results for arrangement in importance order of elements B_1, B_2, ..., B_m corresponding to A_i in hierarchy B are b_1^i, b_2^i, . . ., b_m^i, respectively, then the general arrangement in importance order for hierarchy B can be calculated by following table.

Checking Consistency

When the ratio between random consistency index and consistency index for hierarchy general arrangement in importance order is less than or equals 0.10, it can be recognized as satisfactory consistency.

Hierarchy A	A_1 a_1	A_2 a_2	...	A_m a_m	General arrangement in importance order
Hierarchy B					
B_1	b^1_1	b^2_1	...	b^m_1	$\sum_{i=1}^{m} a_i \cdot b^i_1$
B_2	b^1_2	b^2_2	...	b^m_2	$\sum_{i=1}^{m} a_i \cdot b^i_2$
B_n	b^1_n	b^2_n	...	b^m_n	$\sum_{i=1}^{m} a_i \cdot b^i_n$

The hierarchy analysis model is established based on the following four principles:

-- Give priority to what is closely related to people's lives;
-- Give priority to what is closely related to national and regional economy development projects;
-- Give priority to local key enterprises;
-- Postpone or cancel the construction of projects with less benefit and with unreasonable distribution of industry.

According to the responses of leaders and experts from Tangshan municipal government, to reach the general objective of recovery from earthquake disaster (hierarchy 0), the arrangement in importance order and the weight for evaluation hierarchy (hierarchy C) are as follows :

Arrangement in Importance Order	Subhierarchy	Subhierarchy Description	Weight
1	C1	Contribute to recovery of resident's daily life	0.6506
2	C2	Contribute to development of national key projects	0.2225
3	C3	Contribute to recovery of local economy	0.1268

The arrangement in importance order for the measures hierarchy (Hierarchy p) is shown in Table 3. It follows that after a damaging earthquake:

-- Rehabilitation and reconstruction of the infrastructure, such as water supply, power supply and communication, should have top priority;
-- Secondly, more attention should be paid to job generation and housing construction;
-- At the same time, attention should be paid to recovery of the most effective industrial sectors.

POST-EARTHQUAKE ACTIVITY MODEL

Post-earthquake activity model is an effective tool for evaluation of the result of activities following a damaging earthquake. By this model, activities are divided into the following phases.

Table 3 Arrangement in importance order for recovering earthquake disaster in measures hierarchy

	For general objective	For C1	For C2	For C3
	(Hierarchy 0)	(Hierarchy C)	(Hierarchy 0)	(Hierarchy C)
Water supply	1	1	3	1
Power supply	2	2	1	2
Communication	3	3	6	3
Employment	4	5		
Grain & Grocery	5	4		
Fuel	6		2	12
Machinery	7		5	5
Road & Bridges	8	10	8	4
Housing	9	6		
Commerce	10	7		
Building Material	11		7	6
Metallurgy	12		4	13
Textile	13		9	7
Heating& Gas Supply	14	9		
Medical Treatment	15	8		
Petrochemistry	16		10	11
Food	17		12	8
Papermaking	18		11	10
Forest	19		13	9

Emergency Phase

Emergency measures are usually those which are taken immediately following disaster impact and are mainly directed towards saving lives and protecting property, and dealing with the immediate disruption, damage and other effects caused by the disaster. This phase applies to a fairly short period ranging from several days to 2- 3 weeks after impact. The end of this phase is characterized by completion of the following activities:

-- Search and rescue;
-- Provision of emergency food, shelter and medical assistance;
-- Clearance of ruins on the main roads.

Recovery Phase

Recovery phase is the process by which impacted areas are assisted in returning to their normal level of functioning following a disaster. The recovery process can be protracted, taking several months, or even more than one year. The following three categories of activity are usually regarded as coming within this phase:

-- Restoration of essential services, such as main urban services, public utilities, traffic and transportation, and of repairable buildings and structures;
-- Provision of temporary housing and adoption of measures to assist the physical and psychological rehabilitation of disaster victims;

-- Basic clearance of ruins caused by the disaster.

Recovery Reconstruction Phase (Reconstruction Phase I)

During this segment, the affected areas are assisted in returning to their level of functioning prior to disaster impact. Long-term measures of reconstruction, including the replacement of buildings and infrastructure which have been destroyed by the disaster are also taken in this segment.

Development Reconstruction Phase (Reconstruction Phase II)

In the modern world, countries are becoming increasingly inter-related and interdependent. Therefore, the development reconstruction phase provides the link between disaster-related activities and regional or national development. Since the results of disaster are effectively reflected in future policies and the interests of regional or national progress, the following activities should be undertaken in this segment in order to produce the best possible benefits and to ensure that regional or national development does not create further disaster problems nor exacerbate existing ones:

-- Introducing improved and advanced building systems and programs;
-- Applying experiences learned from the disaster to future research and development programs;
-- Utilizing international assistance to optimum effect.

A typical post-earthquake reconstruction model is shown in Fig. 1. Usually, of the first three phases, the duration of the latter phase is ten times more than the former.

CASE STUDIES

1975 Haicheng Earthquake

Search and rescue activities after the impact took six days and the last survivor was rescued from the disaster 110 hours after the impact. Shelters were provided to homeless victims within 10 days after the shock. As regards the recovery of industries, the main products and output reached their level prior to the quake on the 10th day after the event, and all of the sectors were restored two months after the impact. The entire reconstruction process lasted about 10 years. A post-earthquake reconstruction model is shown in Fig. 2.

Since ground motion caused damage to buildings and soil liquefaction caused damage to most of bridges, it was decided to rebuild in the original place without change of land use.

1976 Tangshan Earthquake

A post-earthquake reconstruction model following the 1976 Tangshan event is shown in Fig. 3.

Before the event, Tangshan city was divided into three districts, namely the eastern mining area, Lunan district and Lubei district. According to the reconstruction plan, the new Tangshan city is divided into three areas which are separated from each other at a distance of some 25 km. The Lubei district is reserved and its population is kept at a figure of 250,000. The main factories, enterprises and inhabitants of Lunan district have moved to the new district at the eastern part of Fengren County to form a satellite city with a population of 150,000. The eastern mining district is restored with a population of 300,000.

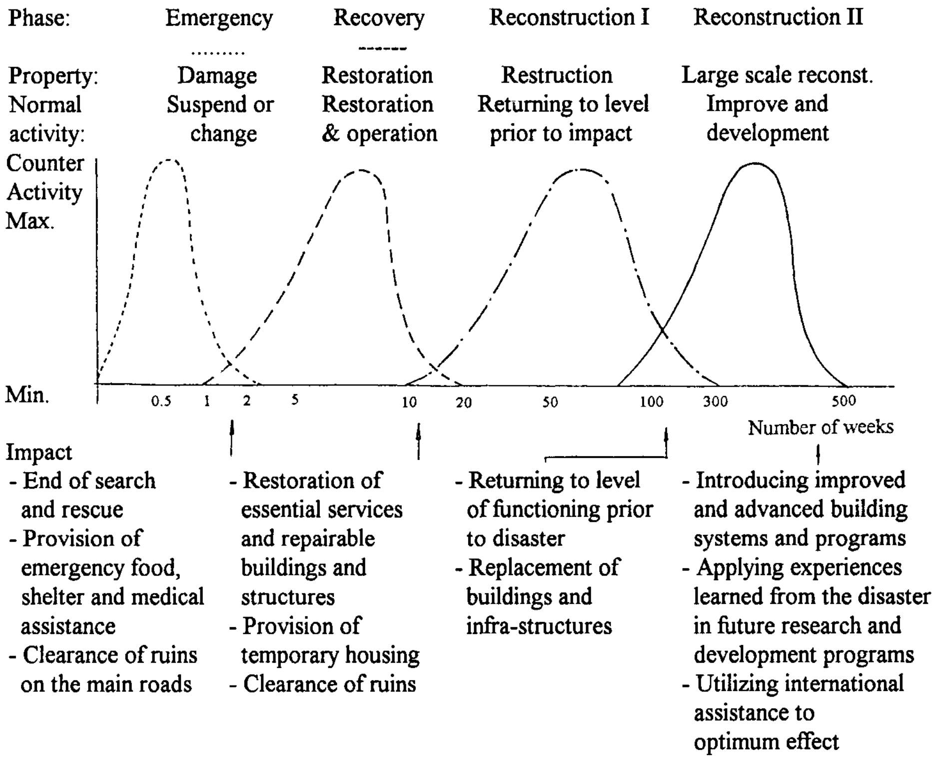


Fig. 1 Typical Post-earthquake Reconstruction Activity Model

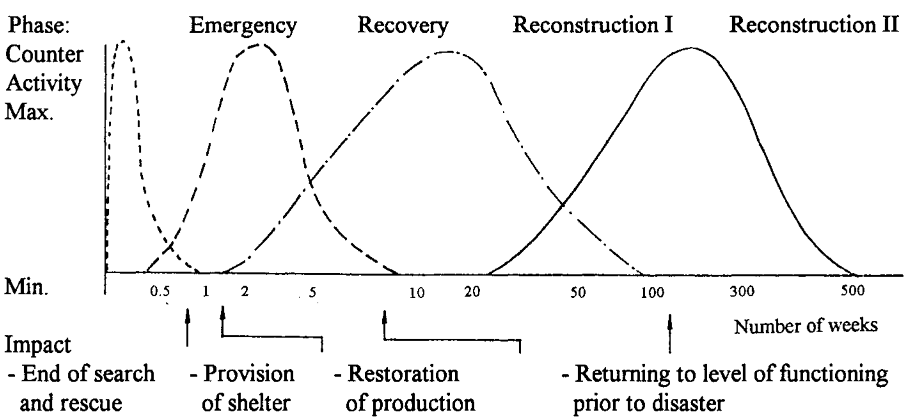
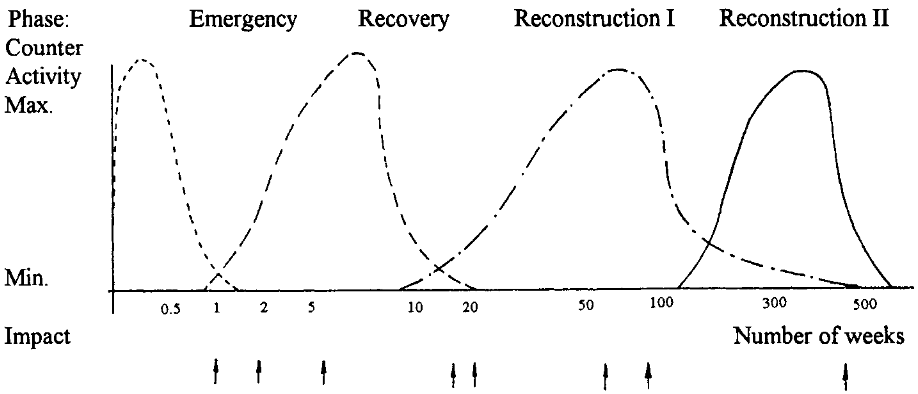


Fig. 2 Post-Earthquake Reconstruction Activity Model for 1975 Haicheng Earthquake



1. End of search and rescue and start of clearance of cadavers
2. Start to build temporary housing
3. Shops restored
4. Completion of temporary housing construction
5. End of epidemic prevention and clearance of cadavers
6. Total industrial output reached to the level prior to the event
7. Start of large scale housing construction projects
8. Completion of housing construction projects

Fig. 3 Post-Earthquake Reconstruction Activity Model for 1976 Tangshan Earthquake

CONCLUSION

Reconstruction following an earthquake is a complicated problem with social, economic and technological aspects. However, for an under-developed country or region there is a good opportunity to change its original economy development model, to push urban and rural renewal forward. Therefore, rational decision-making is the key to accelerate the reconstruction process and to improve the pattern of human settlement.

There are three choices for land-use planning in post-earthquake reconstruction. Rebuilding at the original place should be given first priority. Otherwise, partially rebuilding at the original place, partially moving to close neighboring place may be considered. Renouncing the original place and moving to a new place is a more expensive and difficult solution. It can be adopted under the following conditions:

-- Damage to buildings and structures from ground motion.
-- Willingness of inhabitants to relocate.
-- Difficulty of measures to mitigate future distress .
-- Economic feasibility.

It is necessary to identify priorities for recovery of economic sectors because financial resources are limited. Both system dynamics method and analytical hierarchy process method can be used for this purpose.

Post-earthquake reconstruction activity consists of four phases: Emergency, Recovery, Reconstruction I, and Reconstruction II. Post-earthquake reconstruction model based on these phases is an effective tool to evaluate the results of reconstruction activity and the effect of policies.

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BASIS FOR COST-EFFECTIVE DECISIONS ON UPGRADING EXISTING STRUCTURES FOR EARTHQUAKE PROTECTION

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ABSTRACT

A systematic approach is proposed for formulating cost-effective decisions on the specification of appropriate levels of upgrading existing structures for earthquake protection. The decisions involve the determination of the acceptable risks (or target reliabilities) for damage control and life safety. Optimal acceptable risks are determined on the basis of minimum expected life-cycle costs, and from which the corresponding criteria for upgrading are developed. The approach is illustrated for rehabilitating/strengthening a specific reinforced concrete building in Mexico City that was damaged during the 1985 earthquake.

KEYWORDS

Cost-Effective Design; Target Reliability; Seismic Criteria; Upgrading Criteria; Optimal Design.

INTRODUCTION

As in the design of new structures, the economics of upgrading existing structures, for earthquake protection is of fundamental concern and importance in engineering. Although this is generally recognized, the significance of economic factors has not been integrated properly or explicitly with the technical issues in the development of criteria for upgrading of structures. These issues have been examined, however, for the design of new structures (e.g. Liu *et al.*, 1972, 1976; Rosenblueth, 1976) and specific models have been suggested.

The purpose of upgrading criteria is to insure adequate level of safety or performance of a structure. In light of unavoidable uncertainties in the prediction of loadings and in the estimation of structural capacities, structural safety or performance may be measured in terms of probability or risk. The main issue, therefore, requires the determination of "what constitutes acceptable risks" for upgrading existing structures (or designing new structures), and on the basis of which associated criteria for upgrading may be formulated. Properly, the acceptable risk (or target reliability) ought to depend on the following:

(i) the seismicity or seismic hazard at the site of a structure;
(ii) the degree of structural damage caused by a given earthquake;
(iii) the cost of upgrading versus the potential losses from future damages or collapse.

Proposed is an approach to systematically integrate the above factors quantitatively to obtain the optimal acceptable risks (or target reliabilities) for damage control and collapse prevention against earthquake hazard, and from which the corresponding criteria for upgrading may be formulated. There have been considerable technical developments in structural reliability, seismic hazard analysis, and on structural response and damage analysis, all of which have on earthquake-resistant engineering . The potential benefits of these technological developments can be significantly enhanced through the integration with the relevant socio-economic factors.

Overview of Approach

A key decision in the upgrading of existing structures, or rehabilitating damaged structures, through strengthening or retrofitting, for protection against future earthquakes is the specification of the appropriate level of upgrading. As in the design of new structures, the level of upgrading may require a trade-off between the cost of upgrading and the desired level of protection against potential future losses caused by earthquakes. This involves the consideration of the expected damage costs from future earthquakes , besides the cost of upgrading. The decision problem, therefore, may be formulated on the basis of minimizing the expected life-cycle cost as a function of the underlying risk (probability of damage or collapse) or reliability.
The essence of the approach, therefore, is based on the minimization of the life-cycle cost as a function of structural reliability against earthquake damage. For completeness, the life-cycle cost must include the potential damage cost from all possible earthquakes that may occur in the future. Also, in order to formulate all the cost functions on a uniform cost basis (e.g. in terms of present worth), the times of occurrence of the future earthquakes must be specified or modeled. As the cost of upgrading, as well as the potential damage cost, will depend on the intensity of the earthquake ground motion and the times of occurrence of future earthquakes which are unpredictible, the expected life-cycle cost function may be formulated for specific future earthquake intensities.

The basic formulation, therefore, involves a multi-step process. The process assumes a given intensity of an earthquake, and on this basis the expected life-cycle cost can be formulated as a function of the probability of damage (or collapse). For a structure with a prescribed level of upgrading the associated upgrading cost may be estimated on the basis of an upgrading cost function; the probability of damage or reliability of the upgraded structure may then be calculated under the given earthquake intensity. By varying the upgrading level and corresponding cost, the reliability will also vary accordingly under the same earthquake intensity; thus, generating the upgrading cost as a function of the reliability of the upgraded structure. For each of the upgraded structures, the expected cost of damage (including repair cost and other losses) may also be developed as a function of the reliability under a given intensity of the earthquake. The expected cost of damage for a specific design may be obtained by integrating over all the possible intensities given the occurrence of an earthquake. For every design, therefore, there will be an expected damage cost; also, the expected reliability for each design may be obtained by integrating the conditional reliabilities (corresponding to given intensities) over the seismic hazard curve for the life of the structure, yielding a single curve relating the expected damage costs with the expected reliabilities for several alternative designs. Consistent with the upgrading cost, the expected cost of future damages must be discounted for its present value; for this purpose, the occurrences of future earthquakes may be modeled as a random point process. In this step, the potential occurrences of several earthquakes within the life of the structure is accounted for. The combination of the upgrading cost and the expected total damage cost, therefore, constitute the expected life-cycle cost of the upgraded structure as a function of reliability. There is a damage probability (or reliability) that will yield the minimum life-cycle cost; this is the expected optimal risk which may be defined as the *acceptable risk* for damage control or life safety.

The process requires extensive assessments of structural damage and reliability, as needed in the development of the upgrading cost and damage cost functions. Moreover, in order to fully use available damage and loss data, (e.g. from past earthquakes), pertinent damage costs as functions of the damage probabilities must be developed.

FORMULATION OF COST FUNCTIONS

In the formulation of the cost functions, the different cost items may be classified into three categories as follows:

1. Those that vary with risk or reliability; i.e. the cost will increase or decrease with the probability of damage (risk) underlying a structure- -e.g. the cost of upgrading.
2. Those that are consequences of damage or collapse of a structure- -e.g. repair cost and other damage losses.
3. Those that are independent of risk- -e.g., cost of finishings.

Cost items of the first category are directly functions of the underlying risk or reliability, whereas those of the second category depend on the level of damage and, therefore, are indirectly functions of risk. Cost items of the third category are constants and therefore will not influence the determination of the optimal risk.

Upgrading Cost Function

For a given method of strengthening, the cost of upgrading will naturally increase with the level of upgrading and, therefore, with the reliability of the upgraded structure relative to that of the original (unstrengthened) structure. With this consideration, the cost of upgrading may be formulated as follows:

$$\frac{C_u}{C_0} = e^{k_1[1-(\frac{p_f}{p_{f_0}})^{k_2}]} - 1 \tag{1}$$

where:

C_u = cost of upgrading;
C_0 = cost of original structure (with no upgrading);
k_1, k_2 = constants;
p_f = probability of damage of the upgraded structure;
p_{f_0} = probability of damage of the original structure.

To develop the upgrading cost function for an existing structure (which may represent also a class of structures) the structure may be assumed to be upgraded to several levels of improvement for which the respective upgrading costs may be estimated for the particular method of upgrading. The reliability of each of these upgraded structures can then be assessed (see method below) under a given site-specific earthquake of specified intensity, from which the upgrading cost as a function of reliability is obtained for the specified intensity using eq. (1). By varying the intensity of the same earthquake (or ensemble of earthquakes), a familiy of upgrading cost functions for the structure is generated, as shown schematically in Fig. 1.

Damage Cost Function

The cost associated with a structural damage or collapse must include the cost of repair as well as all the consequent losses caused by the damage; the latter would include the loss of contents, subsequent economic loss, and in the case of severe damage and collapse the cost of injury and life loss.

The estimation of losses caused by earthquakes or other natural hazards has recently attracted new interests (e.g. ATC-13, 1985; NRC, 1992; FEMA, 1994). However, based on limited available information and data the current methods for loss estimation are subject to high degree of uncertainty. Furthermore, the cost of human life has remained controversial and suggestions for estimating the impact of a life loss may vary widely. Nevertheless, in order to complete the total life-cycle cost, all potential losses must be included and expressed in common terms, namely in terms of present economic worth (see below).

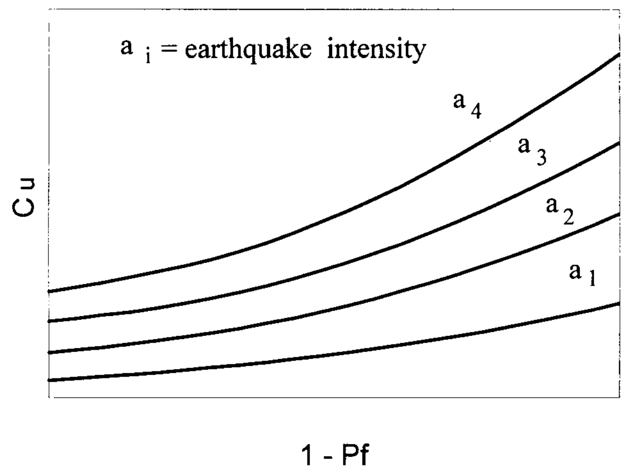

Fig. 1 Upgrading cost functions

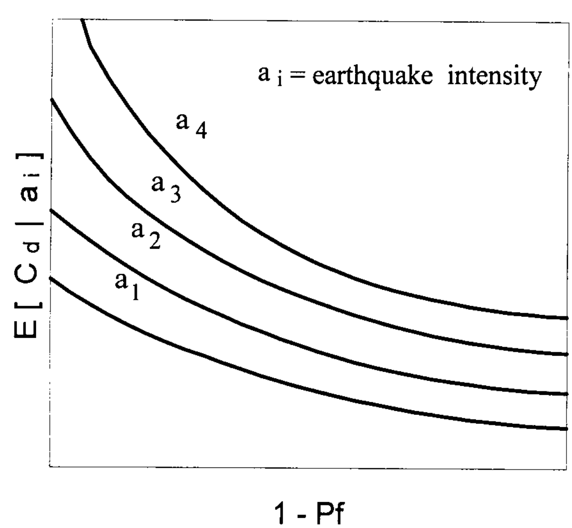

Fig. 2 Damage cost functions

Thus, the total damage cost would be (in the case of buildings)

$$C_d = C_r + C_c + C_e + C_{in} + C_f \tag{2}$$

where: C_r = the cost of repair or replacement; C_c = the loss of contents; C_e = the economic loss due to business interruption; C_{in} = the cost of injury; C_f = the cost of life loss.

Each of the damage cost components in eq. (2) will depend on the global damage level x, i.e.

$$C_j = C_j(x) \tag{3}$$

where $j = r, c, e, in, f$ of eq. (2).

As there are probabilities associated with damage levels and earthquake occurrences, it is the expected damage cost that is pertinent. Moreover, as this cost pertains to future occurrences of earthquakes, it must be discounted for present worth. Therefore, if the damage level X under a given earthquake intensity is defined with PDF, $f_X(x)$, each of the corresponding expected damage cost items in eq. (2) would be:

$$E[C_j] = \int_0^\infty c_j(x) f_X(x) dx \tag{4}$$

Conceptually, the expected damage cost function would be as shown in Fig. 2, in which each curve corresponds to a given earthquake intensity.

The upgrading cost, and the repair cost and loss of contents, involve largely technical issues; however, the other damage cost items in eq. (2) would involve socio-economic considerations.

Present Worth of Future Losses

For consistency, all the expected damage cost items must be expressed on a common basis; e.g. in terms of the present worth. The cost of upgrading would normally be in present value. All the expected damage costs are associated with structural damage or collapse caused by future earthquakes, and thus the present worth of the respective costs will depend on the times of occurrence of these earthquakes. As the occurrences of the earthquakes are unpredictible, the corresponding occurrence times may be described as random variables. In this regard, assuming that the occurrences of potentially damaging earthquakes at a site constitute a Poisson process, the occurrence time of each earthquake is defined by the gamma distribution (Ang and De Leon, 1995).

The present worth of a future loss associated with the damage caused by the kth earthquake, assuming a discount rate of q is:

$$C_d' = C_d \left(\frac{1}{1+q}\right)^{t_k} \tag{5}$$

in which C_d = current cost of damage for the kth earthquake.

Over the life L, the present worth of the expected damage cost caused by all future earthquakes can then be shown to be (Ang and De Leon, 1995):

$$E[C_d'] = E[C_d] \sum_{n=1}^{\infty} \left[\sum_{k=1}^{n} \frac{\Gamma(k, \alpha L)}{\Gamma(k, \nu L)} \left(\frac{\nu}{\alpha}\right)^k\right] \frac{(\nu L)^n}{n!} e^{-\nu L} \tag{6}$$

where:

ν = mean annual occurrence rate of significant earthquake intensities;

$\alpha = \nu + ln(1+q)$;

$\Gamma(k, \alpha L)$ = the incomplete gamma function, and

$$E[C_d] = \int_{a_{min}}^{a_{max}} E[C_d(a)] f_A(a) da \tag{7}$$

in which $f_A(a)$ = PDF of the intensity of one earthquake, which may be derived from the annual seismic hazard curve.
A plot of the present worth factor, $E[C_d']/E[C_d]$, as a function of the discount rate, is shown in Fig. 3 appropriate for Mexico City.

STRUCTURAL DAMAGE AND RELIABILITY ASSESSMENTS

Extensive assessments of structural damage and reliability are obviously needed in the formulation of the risk-based cost functions; i. e. the calculations for determining X and $f_X(x)$ in eq. (4). For these purposes, the required probabilities of damage for each of the upgraded structures may be obtained using a damage model and reliability method that are well established. The necessary analyses for reinforced concrete structures can be summarized as follows.

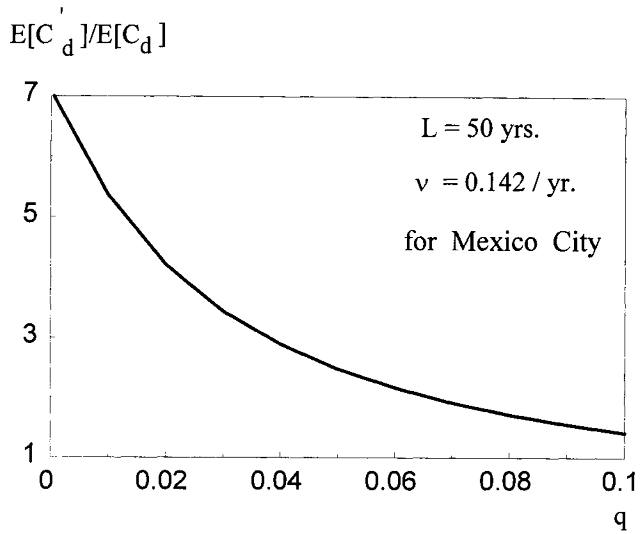

Fig. 3 Present worth factors

Damage of a reinforced concrete element is defined by an index (Park and Ang, 1985):

$$D = \frac{x_m}{x_u} + \frac{\beta_0\, E}{Q_y\, x_u} \tag{8}$$

where:
D = member damage index;
x_m = maximum response displacement;
E = dissipated hysteretic energy;
Q_y = yielding shear capacity;
x_u = ultimate displacement capacity;
β_0 = constant.

For a structure, its global damage is a function of the damages of its constituent elements or components, particularly of the critical components. Assuming that the damages of the critical elements are D_i, the global damage of the structure may be defined as:

$$(D_t > d) = \bigcup_i (D_i > d) \tag{9}$$

where: D_i = damage of critical element i;
D_t = global damage of the structure;
$\bigcup$ = union of events.

In assessing the global damage of a given structure, the damages of its constituent components are, therefore, required. For this purpose, the structure must be modeled and analyzed for its response to a given earthquake; the process involves nonlinear and hysteretic response analysis of the structure from which the response statistics (namely, the maximum deformation and dissipated hysteretic energy) of the constituent elements can be calculated and the respective damage indices, D_i, can be evaluated through eq. (8). The global damage of the structure and associated probabilities may then be evaluated on the basis of eq. (9). Monte Carlo techniques may be used to perform these assessments. Also, fast Monte Carlo simulations (Schueller, *et al*, 1991; Wang and Ang, 1993) may be applied effectively for this purpose.

OPTIMAL TARGET RELIABILITY FOR UPGRADING

Expected Life-Cycle Cost

Combining the upgrading cost with the expected damage cost should yield the expected life-cycle cost function; thus,

$$E[C_t] = C_u + E[C_d'] \tag{10}$$

in which $E[C_d']$ is obtained from eqs. (2) through (4) and (6) and (7).

This expected life-cycle cost is conditional on a given intensity of the earthquake. By varying the earthquake intensity, a family of expected life-cycle cost functions is generated, each of which corresponds to a specified intensity, as shown conceptually in Fig. 4.

Composite Cost Function

For each of the designs, the expected damage cost under a given earthquake can be obtained through eq. (7) in which $E[C_d(a)]$ is evaluated through eq. (4). Also, for each design, the corresponding expected reliability against the lifetime maximum earthquake intensity, defined by the lifetime seismic hazard curve, is evaluated. These results yield the composite expected life-cycle function as shown in Fig. 5.

Determination of Optimal Target Reliability

In Fig.4, for each of the earthquake intensities, there is a reliability that corresponds to the minimum expected life-cycle cost. Therefore , if a "design earthquake" is specified, the optimal target reliability would be defined as the reliability that corresponds to the minimum life-cycle cost for the specified design earthquake intensity. However, in order to take account of all possible earthquake intensities at the site of the structure, as defined by a seismic hazard curve, the expected optimal reliability may be adopted as the target reliability for upgrading, which can be determined as follows. From the composite cost function of Fig. 5, there is a reliability that corresponds to the minimum expected life-cycle cost. This would be the appropriate expected target reliability for upgrading.

ILLUSTRATIVE APPLICATION

The approach described above is illustrated with the development of criteria for the upgrading of a 7-story R/C building in Mexico City. The plan and elevation of the building (which is a regular

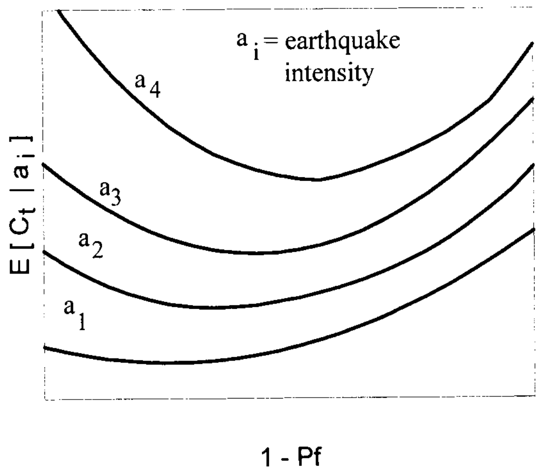

Fig. 4 Expected life-cycle cost functions

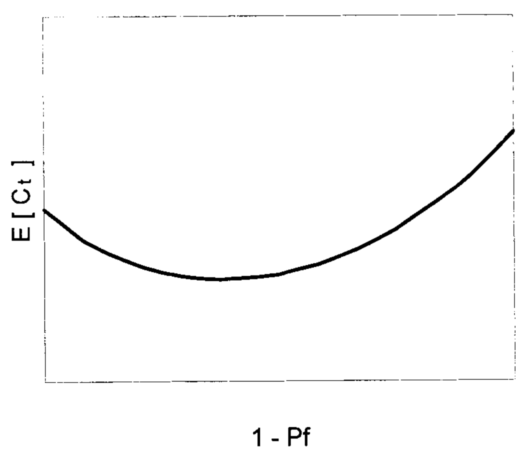

Fig. 5 Composite life-cycle cost function

R/C framed structure) is shown in Fig. 6. It is assumed that the building was originally designed with a base shear coefficient of 0.15.

Analysis of Structural Damage and Cost Data

Actual repair cost of buildings damaged by the 1985 Mexico City earthquake were reported in Guerrero (1990). These repair cost data may be used to determine the parameters for the upgrading cost function of eq. (1). Based on the column dimensions and reinforcements of both the original

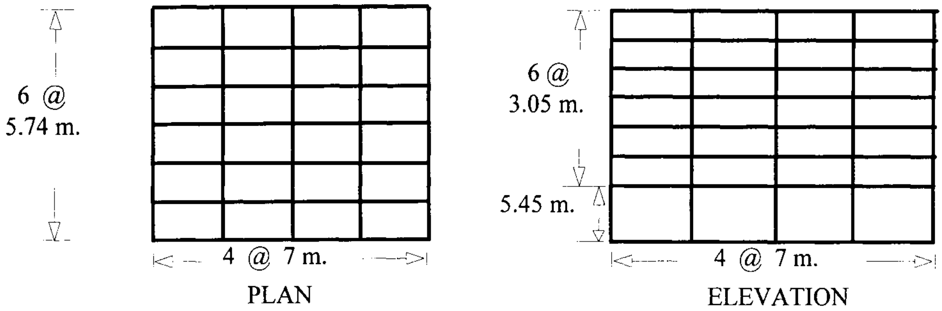

Fig. 6 Building plan and elevation

and the corresponding upgraded buildings, the respective lateral load capacities are estimated and the associated damage probabilities (p_f and p_{f_0}) are assessed under the intensity of the 1985 earthquake; namely, with PGA of 0.17g.

Then, through nonlinear regression, the coefficients k_1 and k_2 are evaluated, yielding the values presented in Table 1. The resulting curve and the actual strengthening cost data can be observed in Fig. 7.

Table 1. Constants k_1 and k_2 for Upgrading Costs (Three Limit States)

Limit state	k_1	k_2
$D > 0.2$	-0.32	-0.33
$D > 0.5$	-0.70	-0.10
$D > 1.0$	-0.29	-0.25

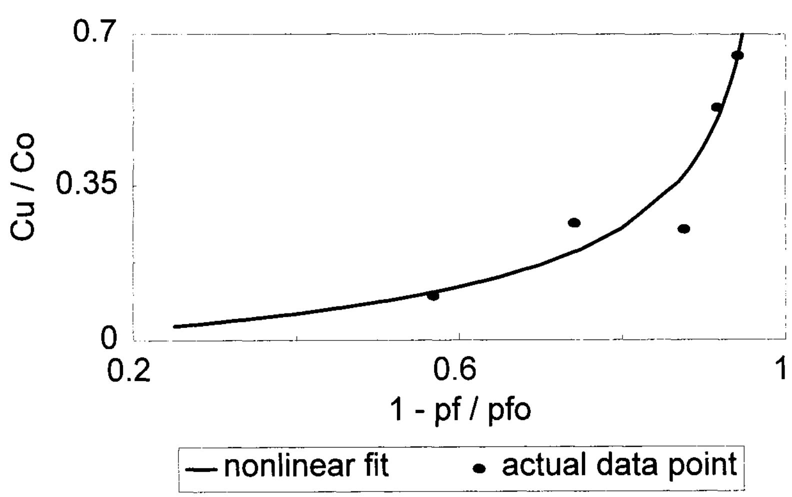

Fig. 7 Upgrading cost function for PGA = 0.17g

Development of Cost Functions

For this illustration, all the costs pertain to Mexico City.

Upgrading Cost To develop the strengthening cost function for the 7-story R/C building, the structural members are assumed to be strengthened to several levels of upgrading by the method of jacketing, which involves wrapping each of the columns by adding reinforced concrete with vertical and lateral (or spiral) reinforcements. On the basis of the respective static lateral load capacity of the upgraded building, the corresponding equivalent lateral load capacities may be evaluated. The probability of damage of each of the upgraded buildings is then assessed under a given intensity of an earthquake in Mexico City (presumed to have the same spectral shape as the 1985 event). From the calculated damage probabilities, including that of the original structure (unstrengthened), the corresponding cost of upgrading for each of the buildings is estimated with eq. (1); thus, establishing the upgrading cost function for the building when subjected to the given earthquake intensity. For other intensities of the same earthquake, similar cost functions are generated resulting in the family of upgrading cost functions for the building, as shown in Fig. 8.

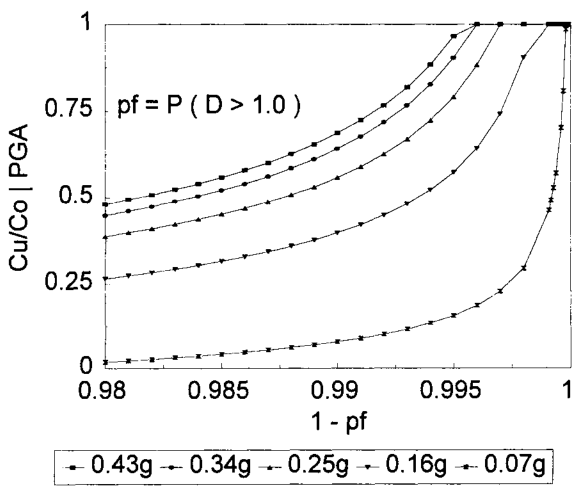

Fig. 8 Upgrading cost functions for several PGA

Expected Damage Cost Function. One component of the expected damage cost is the cost of repair; this cost is obviously dependent on the degree of damage and, therefore, will depend also on the earthquake intensity. As the limit of repairable damage in Mexico City is D=0.5 (De Leon and Ang, 1994), the repair cost function developed on the basis of available repair cost data is:

$$C_r = 1.64\, C_R\, D_m \ ; \ 0 \leq D_m < 0.5 \tag{11}$$

$$C_r = C_R \quad ; \ D_m > 0.5 \tag{12}$$

where C_R = replacement cost of the original structure = $1.15C_0$.

The total loss of contents is assumed to be 50% of the replacement cost (at collapse), and will vary linearly with D_m for intermediate damage; thus,

$$C_c = 0.5C_0D_m \ ; \ 0 \leq D_m \leq 1.0 \tag{13}$$

$$C_c = 0.5\ C_0\ ;\ D_m > 1.0 \tag{14}$$

The direct economic loss caused by structural damage (such as business interruption, loss of revenue, etc.) will depend on the usage of the structure and the severity of damage. It will be taken, in this study, as the loss of rentals (assuming the building is an apartment dwelling) and it may be determined on the basis of the following assumptions:

(i) the maximum period of reconstruction is two years;
(ii) the average monthly rental is 20 dollars per square meter of floor area.

Thus, the maximum possible economic loss for the apartment building is:

$$C_{emax} = 20(12)(2)A = 480A \tag{15}$$

where A = floor area of the building, in m^2.

Therefore, assuming that the economic loss varies as a quadratic function of the median damage:

$$C_e = 480A(D_m^2)\ ;\ 0 \leq D_m < 1.0 \tag{16}$$

$$C_e = 480A\quad ;\ D_m \geq 1.0 \tag{17}$$

where A = floor area of the building.

The cost of injuries may be estimated based on the following assumptions:

(i) the cost of each disabling injury is 117,000 dls., whereas for each non-disabling injury the cost is 1,667 dls.
(ii) the average number of injuries per unit area of collapsed buildings is 0.0168 per square meter of floor area; this is estimated from data reported in Refs. 5 and 13.
(iii) 90% of all injuries are non-disabling, and the remaining 10% are disabling.

With these assumptions, the maximum cost of injury associated with the collapse of buildings is:

$$C_{inmax} = 0.0168[\frac{1}{10}117,000 + \frac{9}{10}1,667]A = 222A \tag{18}$$

Therefore, assuming that the cost of injury varies quadratically with respect to the median damage:

$$C_{in} = 222A(D_m^2)\ ;\ 0 \leq D_m < 1.0 \tag{19}$$

$$C_{in} = 222A\quad ;\ D_m \geq 1.0 \tag{20}$$

The cost associated with the loss of human lives is controversial. However, in order to complete the life-cycle cost, the potential loss of life caused by the damage or collapse of a structure must be included and translated into economic terms. There have been alternative suggestions for this purpose. For example, Rosenblueth (1976) proposed that the "human capital" approach be used in which the average cost of a human life is calculated as the individual's contribution to the national GDP over his remaining productive life. For Mexico City, the following data is available:

(i) the average per capita annual income is 4,680 dls. (Europa, 1993);
(ii) the average casualty per unit area of the collapsed buildings during the 1985 earthquake is 0.0122 per square meter of floor area (based on data in Refs. 5 and 13).

On these bases, and assuming that the average remaining productive life of an individual is 25 years, the cost associated with the loss of human lives (following Rosenblueth, 1976) caused by a structural collapse is:

$$C_{fmax} = 0.0122A(4,680)(25) = 1427A \tag{21}$$

Therefore, assuming that the loss of life varies as the 4th power of the median damage:

$$C_f = 1427A(D_m^4) \quad ; \; 0 \leq D_m < 1.0 \tag{22}$$

$$C_f = 1427A \quad ; \; D_m \geq 1.0 \tag{23}$$

eqs. (12) through (24) are used (with $D_m = x$) in eq. (3) to obtain the respective expected damage costs.

All the cost are in terms of 1985 US dollars. With the present worth factor from Fig. 4 for $q = 8\%$, the expected damage costs for the 7-story building are generated as functions of the reliability $(1 - p_f)$ under varying intensities of the 1985 Mexico City earthquake. The results are summarized in Fig. 9, for five PGA's.

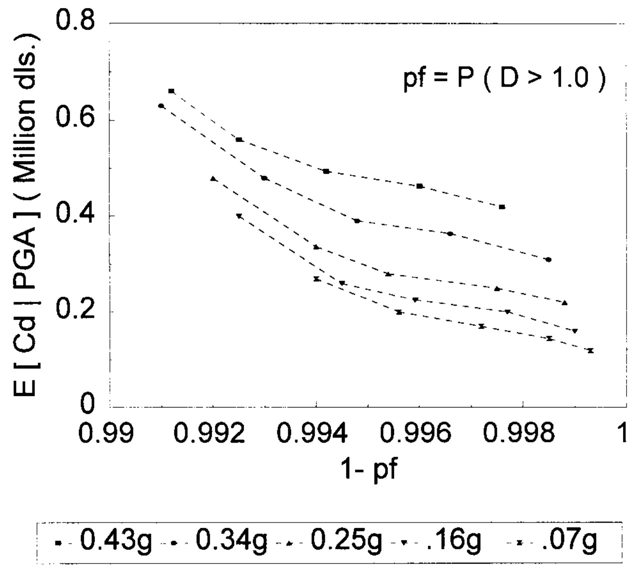

Fig. 9 Damage cost function

Expected Life-Cycle Cost Functions. Combining the upgrading cost and the expected damage cost under each of the various earthquake intensities yields the expected life-cycle cost as a function of the reliability $(1 - p_f)$ as shown in Fig. 10. The corresponding composite cost function is shown in Fig. 11.

Optimal Target Reliability and Upgrading Criteria

In Fig. 10 the optimal reliability corresponding to the minimum expected life-cycle cost is indicated for each earthquake intensity. Therefore, if a "design earthquake" is specified, the indicated optimal reliability can be used as the target reliability for design. For example if a PGA of 0.24g is designated as the design earthquake for Mexico City (as suggested by Rosenblueth, 1989) the optimal lifetime target reliability against collapse would be 0.014, and the corresponding optimal base shear coefficient for upgrading would be 0.36.

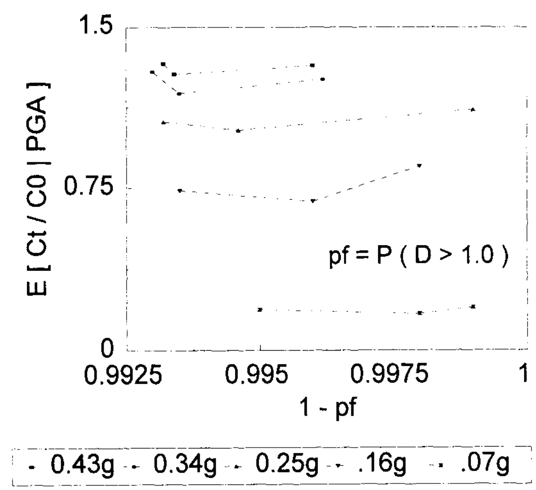

Fig. 10 Expected life-cycle cost functions

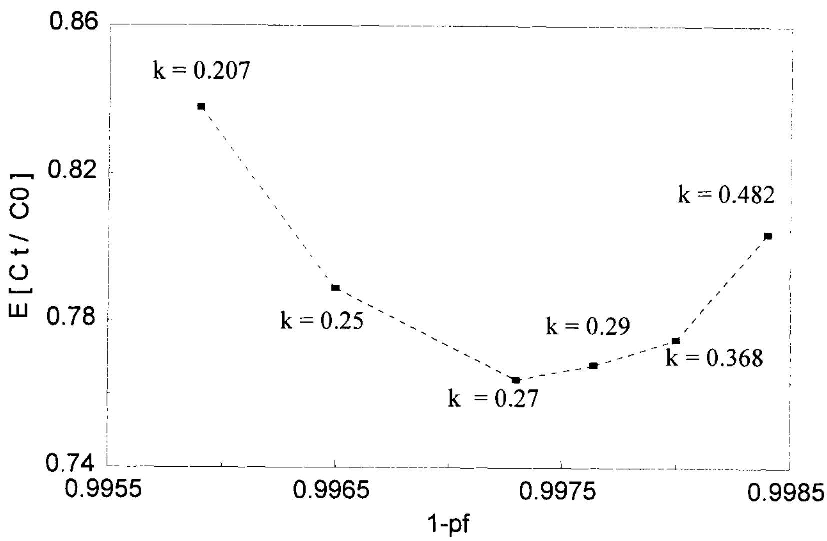

Fig. 11 Composite cost function

For protection against all significant earthquake intensities in Mexico City as defined by the 50-year hazard curve of Fig. 12, the expected optimal reliability is obtained from the composite life-cycle cost function of Fig. 11, giving a reliability of 0.9973 and the associated base shear coefficient is 0.27 for upgrading the building.
The above target reliabilities and associated base shear coefficients are also summarized in Table 2. In Table 2, the optimal upgrading costs are also indicated, either for all possible earthquakes, or for specified "design" earthquakes.

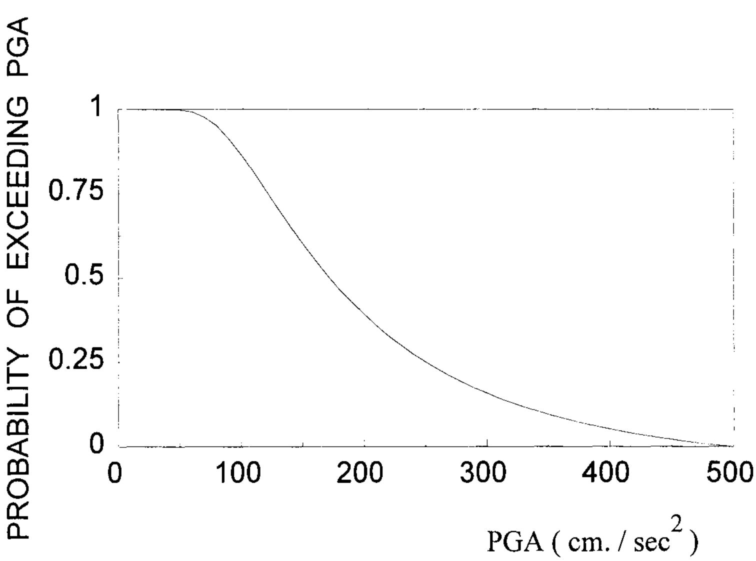

Fig. 12 Seismic hazard curve for México City
(50-year hazard; soft soil zone)

CONCLUSIONS

Decisions for upgrading existing structures for earthquake protection can be developed systematically by integrating all the important technical factors with the economics of earthquake hazard mitigation. The approach described herein offers an effective way to accomplish this objective. Optimal acceptable risks (or target reliabilities) are determined for damage control and collapse prevention of upgraded structures for earthquake protection, and on the basis of which the corresponding criteria (e.g. required base shear coefficients) for upgrading can be developed. The methodology for such developments will allow the systematic formulation of cost-effective criteria for structural upgrading.

Table 2. Summary of Results

Limit state	Optimal acceptable risk	Base shear coefficient	Optimal upgrade cost	Prescribed earthquake intensity
$D = 0.2$	0.15	0.27	0.35 C_0	All possible in 50 years
$D = 0.5$	0.06	0.27	0.36 C_0	"
$D = 1.0$	$2.7x10^{-3}$	0.27	0.37 C_0	"
$D = 0.2$	0.055	0.21	0.28 C_0	$0.07g^*$
$D = 1.0$	0.014	0.36	0.45 C_0	$0.24g^{**}$
$D = 1.0$	0.020	0.58	0.80 C_0	$0.36g^{***}$

* Corresponds to a 10% annual exceedance probability.

** As suggested by Rosenblueth *et al.* (1989) which has a return period of 143 years.

*** Corresponds to a 10% exceedance probability in 50 years (return period of 475 yrs.).

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APPLICATIONS OF ARTIFICIAL INTELLIGENCE IN GIS TO URBAN PLANNING FOR EARTHQUAKE DISASTER MITIGATION

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ABSTRACT

The paper presents a state-of-the art application of a logical combination of geographic information system (GIS) and artificial intelligence (AI) to urban planning for earthquake disaster mitigation. The evolution of GIS has provided scientists and engineers with powerful tools for the modeling of large amounts of spatially related geographic data (see Shah *et al.*, 1991). However, many applications require not only calling data from GIS but also more sophisticated inference and assessment mechanisms. First, this paper describes the artificial intelligence technology, such as fuzzy assessment and neural networks, in particular for site evaluation, forecasting the distribution of ground motion and the potential of liquefaction. Second, an application of AI+GIS to urban planning for earthquake disaster mitigation for Xin Xiang City, a mid-size city in Central China, is given as an example.

KEYWORD

Artificial intelligence; GIS; earthquake; disater mitigation; fuzzy; neural network.

INTRODUCTION

An urban plan for earthquake disaster mitigation under the general plan for a modern city has been drawn up based on the results of seismic hazard analysis and seismic vulnerability analysis. A huge amount of spatial data is required by such analyses that are heavily dependent on data management mechanisms by which data are stored, retrieved and manipulated. Also, the data input and output are very time-consuming and labour intensive for drawing up the plan and representing it in forms such as analog maps, tables or electronic data. GIS can be used not only because of its ability to deal with extremely large data sets of spatially referenced information, but also because of the ability to dynamically and automatically retrieve the data and manipulate it to derive new information.

At the Fourth International Conference on Seismic Zonation of 1991, Stanford, California, participants presented successful applications of GIS in the following fields:

1. Modeling and reduction of natural or man-made disasters that are a matter of deep concern by government at every level for the development of urban and rural planning (see Dangermond , 1991).

2. Environmal protection and assessment of geological disasters (see Driel N, 1991).

3. Seismic risk estimation, seismic zonation, identification of groups of hazardous buildings within a region, estimation of economic impact of various scenarios, evaluation of earthquake mitigation strategies and seismic insurance, etc. (see Shah *et al.*, 1991).

4. Policy and decision-making for rescue work and public safety in seismic regions (see Topping , 1991).

The authors of this paper have worked out urban plans for some cities in China since 1988. Based on the concept of GIS, management software with a graphical data base that is similar to commercial GIS has been developed, in particular for seismic zonation and disaster mitigation plans. Within the framework of this management system, some artificial intelligence knowledge processes, such as fuzzy assessment and neural networks, are involved. The combination of these two types of systems allows users to extend the functionality of either AI or GIS in new ways. It becomes possible to perform reasoning beyond simple spatial analysis of GIS data. For example, a fuzzy knowledge process is used to evaluate and classify a specified construction site according to the seismic-geological and topographic data sets as well as the seismic zonation data sets. Another example is the application of neural networks to program a large amount of input data, such as peak ground acceleration PGA, intensity and liquefaction indices which were collected from real events in the past, and to establish subsquently the weight and operation of functions of the network. Once a neural network has been programmed to a point where errors in output are below a given threshold value, the network is considered an intelligent inference procedure and can be adopted to evaluate new sets of inputs. From the AI perspective, data ordered by AI knowledge process can be managed with the capacities of the GIS. The spatial context of the data being analyzed can be preserved to help the presentation and classification of the results in the original geographic context managed by GIS.

FRAMEWORK OF GIS AND SPATIAL CONTEXT OF DATA FOR SEISMIC ZONATION OF XIN XIANG CITY

The information required and referenced in seismic zonation is usually stored and retrieved spatially with analog maps, tables and characters. When people treat them manually, there is limited capacity for compiling, updating and combining from multiple maps, and the process is time-consuming and labour intensive.

GIS can be used to nonmanually process the spatially referenced information. Capacities of GIS developed particularly for seismic zonation include the following.

Data Input.

Analog maps, and figures can be digitized manually or scanned automatically into the GIS. Necessary and appropriate attributes are then assigned to the digitized "map data". The map data can also be compiled before calling. Some types of data updated mechanisms are adopted in GIS instead of digitizing and scanning. Fig. 1 shows the seismo-geological map of Xin Xiang City which was scanned into the GIS (see Wang and Wang, 1993).

Data Call.

The digitized map data can be retrieved or called in a single-layered or manipulated formation. For seismic zonation task, the following maps are needed: existing municipal map; geographical map; topographic map; seismic-geological map; maps of distribution of the population, buildings and infrastructures, hazardous material storage, emergency rescue works, etc.; maps of seismic zonation, including site classification, liquefaction potential and ground strong motion distribution. Fig. 2 gives a geological map with the attribution data of soil profile of boreholes in the Xin Xiang . Users can call for any sub-zone of one square kilometer to find out the boreholes involed and relative attribution data of the soil profile and water level underground as well as the index of liquefaction potential.

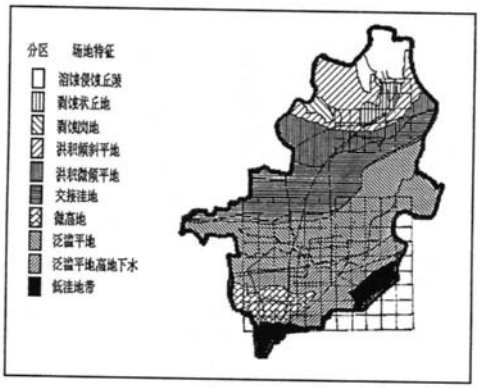

Fig.1 Seismo-geological map of Xin Xiang City

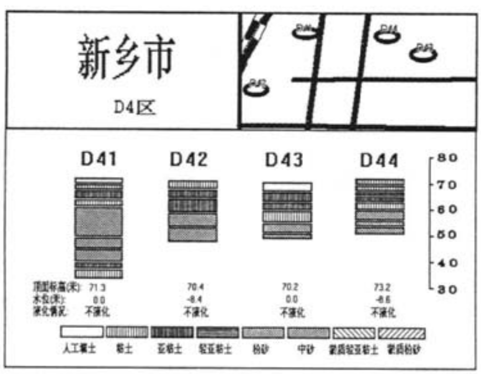

Fig.2 Soil profile of boreholes in Xin Xiang zonation

By calling the stored map data, users can go over the GIS for maps and related attribution data, such as earthquake focus distribution map, seismic fault distribution map, zonation maps of design earthquake strong motion (PGA, PGV, response spectra, etc.), seismic influence zonation map, zonation maps of economic impact and loss of life under various exceedance probabilities.

Data Analysis.

A comprehensive analysis and assessment process with a combination of GIS and AI can be implemented for a specified construction site in the city. A single factor, or more, can be selected to go through this process. For example, seismic evaluation for a given site can be done with consideration of three factors: class of site (hard, moderate hard, moderate soft, soft alluvium), seismicity, geological condition and liquefaction potential. The technique of overlap operations on different layers of data offers users new information on seismic zonation. Fig. 3 shows a comprehensive evaluation map for a specified construction site that indicates a so-called site index to let users know whether the site is a favorable one for engineering projects.

Based on essential map data and attribution data, optimum decisions can also be made for land use, relief and rescue work, evacuation path organization, strategies of rehabilitation and reconstruction, etc.

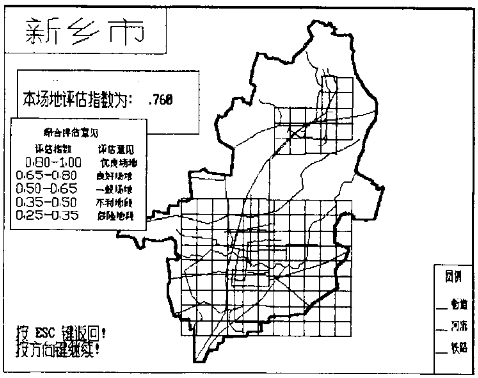

Fig.3 Comprehensive evaluation map

<u>Data Output.</u>

GIS can be used not only because of its ability to handle a vast extremely large amount of data on spatially referenced formation, but also because of its ability to dynamically and automatically retrieve the data and comprehensively manipulate it to derive new information. A multi-layer map derived by data call and data analysis can give users the solutions for their particular requirements. For example, the seismic zonation of Xin Xiang City can be seen in Fig. 4. This map displays the grid of zoning with earthquake strong motion parameters, such as PGA, response spectra, for each block of zoning. Such a map is very useful for the seismic design of engineering projects in a city.

APPLICATION OF ARTIFICIAL INTELLIGENCE TECHNOLOGY IN GIS FOR SEISMIC ZONATION OF XIN XIANG CITY

A combination of GIS and AI knowledge processes offers a powerful solution for the seismic zonation task. This paper presents some types of AI technology to be used with GIS for seismic evaluation of site, estimation of earthquake strong ground motion and liquefaction potential, and optimum decision-making for emergency response.

<u>Fuzzy Assessment of Site.</u>

Three main factors are involved in seismic evaluation of a specific construction site segment.

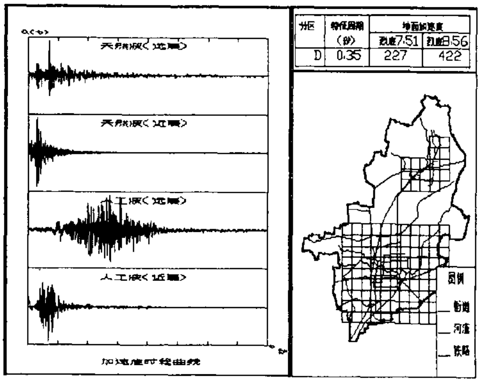

Fig.4 Earthquake strong motion parameters

1. Site class that is identified by considering the rigidity and depth of deposits at the site. Four classes of site are classified in Chinese Building Seismic Design Code that indicate sites of hard rock, moderately hard, moderately soft, and soft alluvium (see GBJ 11-89).

2. Seismicity that describes the features of sources and faults and the distribution of ground motion. An engineering object should be favorably located on a spot with lower potential of earthquake activities or far away from the faults.

3. Geological conditions that are the exposure of topography, seismo-geology, liquefaction potential, etc.

A fuzzy assessment approach has been adopted to evaluate the site. This process begins by indicating a desired point on the seismic zonation map. A fuzzy relationship between the site index set and the influence factors can then be set up as

$$\tilde{S} = \tilde{W} \cdot \tilde{F} \tag{1}$$

where $\tilde{w}$ is the fuzzy weighted vector that can be expressed as

$$\tilde{W} = \left[\frac{W_1}{\tilde{A}}, \frac{W_2}{\tilde{B}}, \frac{W_3}{\tilde{C}} \right] \tag{2}$$

where W_1, W_2, W_3, the weight factors, have been preset and made available in the software program for computation or, instead, are defined by users with consideration of the three influence factors. $\tilde{F}$ is the fuzzy relationship matrix given by

$$\tilde{F} = \begin{bmatrix} \tilde{A} \\ \tilde{B} \\ \tilde{C} \end{bmatrix} = \begin{bmatrix} a_1 & a_2 & a_3 \\ b_1 & b_2 & b_3 \\ c_1 & c_2 & c_3 \end{bmatrix} \tag{3}$$

Five classes of site segment can be identified as Fine, Good, Fair, Poor and Dangerous, with evaluation indices ranging, respectively, in 1.00-0.80, 0.80-0.60, 0.60-0.40, 0.40-0.20 and 0.20-0.00.

Neural Network Technology.

A neural network consists of a large number of process elements (artificial neurons) which are connected to each other to implement an information process. In the usual case, each artificial neuron takes some input, applies some predetermined node specific weight to the input value, and then generates output based on some internal non-linear transfer function.

This information process involves the storage of data and knowledge that is represented in a distributive manner among neurons, and the recognization of information that is related to the dynamic adjustment procedure of connective weights among neurons. A data call or knowledge retrieval is carried out in a parallel manner within a trained network (see Jiao L. C., 1990).

Neural networks are designed to solve problems that are related to a large number of sampling data sets, for example, seismic intensity, strong ground motion (PGA, PGV), liquefaction indices of a site, damage indices of various types of buildings, which can be obtained from historic records, experimental research or field observations. These problems are not easily represented by discrete pieces of knowledge and solved by normal statistics methodology.

A back-propagation (BP) neural network model with a hidden layer is given in Fig. 5. The input layer, named as S-layer, has M neurons; the output layer O employs L neurons, and the hidden layer h with N neurons functions as the input layer of O-layer and the output layer of S-layer. Input to the network can be represented as a vector X, consisting of components $X_0, X_1, \ldots X_{M-1}$. Each node in an internal layer takes as input the sum of the node output of the previous layers, and gives as output this value adjusted according to the weights and transfer functions of the node. This process can be expressed by the following equation for a given layer:

$$Y = F(\sum_{i=0}^{M-1} W_i \cdot X_i - \theta) \tag{4}$$

where Y is output of the node in this layer, W_i is the weight for input X_i, θ is the threshold value for the neuron element i; the transfer function of the node can be given as various types of function. The Sigmoid Function is used for this network.

$$F(x) = \frac{1}{1+e^{-x}} \tag{5}$$

Integration of GIS and neural networks has been a powerful tool for seismic risk evaluation. Various models of BP neural networks, combined with GIS have been employed to evaluate and forecast seismic intensity, PGA and liquefaction potential.

Evaluation and Forecast of Seismic Intensity (4-3-5 Layered Model).

Single or multiple factors, such as magnitude M, PGA, PGV and building damage index I_d are normally used to identify seismic intensity I for a quake-affected region. A data set of four factors of average building

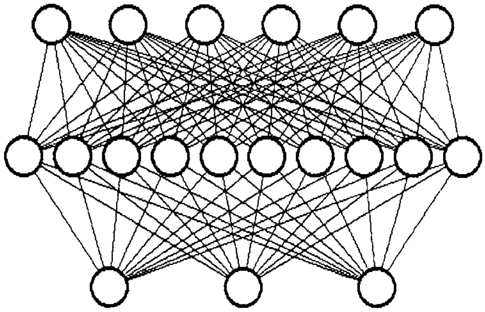

Fig.5 A back-propagation (BP) neural network

damage index I_d, peak accelerations in horizontal and vertical directions A_h and A_v peak velocity in horizontal V_h is brought into correspondence with a given intensity defined by the Chinese Seismic Intensity Scale (see Liu H. X. 1981). The data set has been introduced in a 4-3-5 layered BP model of neural network process for seismic intensity evaluation. Results are given in Table 1 where EXP. is the expectancy intensity and OUT. is the output value of O-Layer of the network, i.e. the identified intensity. A good match between EXP. and OUT. can be found in Table 1.

Table 1 Evaluation of Seismic Intensity by BP Neural Network

I	I_d	A_h (G)	A_v (G)	V_h (m/s)		output intensity(degree) 6	7	8	9	10
VI	0.05	0.063	0.031	0.05	EXP.	1	0	0	0	0
					OUT.	0.98	0.04	0	0	0
VII	0.20	0.125	0.063	0.10	EXP.	0	1	0	0	0
					OUT.	0.02	0.94	0.04	0	0
VIII	0.40	0.250	0.125	0.20	EXP.	0	0	1	0	0
					OUT.	0	0.04	0.95	0.03	0
IX	0.60	0.500	0.250	0.40	EXP.	0	0	0	1	0
					OUT.	0	0	0.04	0.97	0.01
X	0.80	1.000	0.500	0.60	EXP.	0	0	0	0	1
					OUT.	0	0	0	0.02	0.99

Weighted matrices and threshold value matrices between any two layers can be used to describe the input-output relationship for a trained neural network as follows:

$$W(s-h) = \begin{bmatrix} -15.951 & 0.215 & 8.129 \\ -8.231 & -6.717 & 10.800 \\ -3.909 & -3.607 & 5.039 \\ -7.117 & -4.896 & 8.642 \end{bmatrix} \tag{6}$$

$$\theta(s-h) = (4.709 \quad 8.479 \quad -8.691) \tag{7}$$

$$W(h-o)=\begin{bmatrix} 14.046 & -11.232 & -26.364 & -9.160 & -1.506 \\ -1.920 & -5.199 & 6.386 & 8.004 & -9.902 \\ -4.605 & -26.136 & -9.785 & 12.614 & 5.498 \end{bmatrix} \tag{8}$$

$$\theta(h-o)=\begin{pmatrix} -7.377 & 2.249 & 0.614 & -16.511 & -0.897 \end{pmatrix} \tag{9}$$

A comparison between two approaches of fuzzy mathematics and neural network is given as follows, assuming I_d=0.45, A_h=160 cm/s^2, A_v=90 cm/s^2, V_h=25 cm/s.

A fuzzy vector of seismic intensity can be obtained, using the fuzzy comprehensive assessment

$$B=\frac{0.00}{VI}+\frac{0.60}{VII}+\frac{0.80}{VIII}+\frac{0.40}{IX}+\frac{0.15}{X} \tag{10}$$

The corresponding assessment by the BP neural network is given as

$$B=\frac{0.00}{VI}+\frac{0.20}{VII}+\frac{0.97}{VIII}+\frac{0.01}{IX}+\frac{0.00}{X} \tag{11}$$

where the symbols VI-X express the intensities of 6-10 degrees respectively, and the numerators in the neural network are the subordinate of seismic intensity in fuzzy vector and the weight of seismic intensity. The identified intensity is 8 degrees, but note that the assessment done by the neural network is much better than that by the fuzzy approach. Let's put I_d=0.30, V_h=20cm/s; then different results can be provided, respectively, by the fuzzy analysis or by the neural network method.

$$B=\frac{0.00}{VI}+\frac{0.60}{VII}+\frac{0.60}{VIII}+\frac{0.37}{IX}+\frac{0.11}{X} \tag{12}$$

$$B=\frac{0.00}{VI}+\frac{0.99}{VII}+\frac{0.99}{VIII}+\frac{0.09}{IX}+\frac{0.00}{X} \tag{13}$$

It can be seen that the recognized intensity is 7-8 degrees and the neural network behaves better than the fuzzy assessment does.

Identification and Forecast of Seismic Liquefaction of Site (4-19-1 Layered Model)

Four parameters of seismic intensity, measured resistance value of standard penetration test (SPT), depth of underground water level and buried depth of liquefaction-potential soil (silt and sand) have been adopted by Chinese Code of Seismic Design of Buildings to identify the liquefaction potential of saturated soils at a site. The liquefaction identification formula was drawn up based on data sets collected from liquefaction failures during the recent earthquake events, home and abroad, and engineering surveys on the spot.

The authors took from the data sets 163 samples with intensities ranging 7-10 degrees to perform the identification of liquefaction potential in two ways: standard code-based method and neural network method. A comparison of identified or predicted liquefaction possibility with observed data was made at same time.

A data set of a 123 samples out of 163-sample package has been used to program a 4-19-1 layered BP model. Weights of the nodes in the network are adjusted during the process until errors in output are below an acceptable minimum. Once the network has been programmed and the operation of functions in the network has been established, the network is used to evaluate a new data set of the remaining 40 samples. Accuracy of identification and forecast is up to 94.5%. On the other hand, a code-based assessment method was employed to evaluate the liquefaction possibilities for the same data set. Its accuracy is only 85.3%.

Results of evaluation by both methods and data actually collected from previous events are shown in Table 2 and Table 3, respectively.

Identification of Peak Acceleration of Ground Motion (3-15-1 Layered Model)

Parameters of earthquake strong ground motion, such as peak acceleration (PGA), peak velocity (PGV) and response spectra, are considered as related to magnitude, epicenter distance and site condition. A 3-15-1 layered model of neural network has been set up to process a data set which is composed of 35 acceleration records obtained from the 1988 Lancang-Gengma Earthquake (see Wang , 1989). Table 4 shows the net output PGA of the net compared with actual PGA. It differs from the seismic intensity and the liquefaction index in that the value of PGA is a continuous decimal digit. Accuracy of process is under various acceptable error levels as shown in Figure 6.

CONCLUSION

Artificial intelligence tools combined with GIS can provide vast amounts of referenced data for use used in earthquake hazard analyses for microzonation. The technology of AI+GIS offers engineers and municipal officers a powerful tool to solve many problems that are difficult or even impossible to model by using traditional simulation or knowledge-based approaches. Satisfactory results can be obtained by using the AI+GIS technology for such factors as the estimation of damage distribution of buildings and infrastructures, the attenuation of strong ground motion, the economic impact and losses, and the evaluation of earthquake disaster mitigation strategies.

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Table 2 Evaluation of Liquefaction Possibilities (123 samples for proramming)

No	I_s	D_w	D_s	S_p	L_a	L_m	L_c	No	I_s	D_w	D_s	S_p	L_a	L_m	L_c
1	7	1.60	2.30	9	0	0.70	0	63	8	0.3	1.4	6.	1	1.0	1
2	7	1.04	2.05	9	0	0.71	0	64	8	0.66	1.1	18	0	0.00	0
3	7	1.05	4.30	12	0	0.01	0	65	8	1.4	.70	18	0	0.00	0
4	7	0.65	3.90	10	0	0.13	0	66	8	1.08	.60	14	0	0.00	0
5	7	2.50	4.30	9	0	0.07	0	67	8	.80	.36	14	0	0.00	0
6	7	2.0	6.0	9.5	0	0.09	0	68	8	1.63	1.04	7.	1	0.95	1
7	8	1.03	4.41	14.6	0	0.04	0	69	8	.52	.60	10	0	0.02	0
8	8	1.15	1.40	14	0	0.00	0	70	9	1.0	1.15	6.8	1	1.0	1
9	8	1.30	1.44	13	0	0.06	0	71	9	2.02	3.33	15.7	1	0.99	1
10	7	1.40	4.35	9	1	0.64	0	72	9	1.0	1.18	6.8	1	1.0	1
11	7	1.25	1.80	4	1	1.0	1	73	9	1.0	2.96	34.	0	0.00	0
12	7	1.25	4.30	7	1	1.0	1	74	9	0.0	.50	8.9	1	0.99	1
13	7	1.20	2.30	6	1	1.0	1	75	9	.74	1.6	9.3	1	1.0	1
14	7	0.60	1.80	2	1	1.0	1	76	9	.85	.80	3.9	1	1.0	1
15	7	0.70	2.30	1	1	1.0	1	77	9	.85	1.03	7.1	1	1.0	1
16	7	0.95	3.30	9	1	0.79	0	78	9	1.05	1.68	10	1	1.0	1
17	7	2.00	9.50	6	0	0.04	1	79	9	.40	1.80	13.6	1	0.97	1
18	7	3.15	6.38	12	0	0.00	0	80	9	1.0	2.10	16	1	0.94	1
19	7	1.59	6.65	23	0	0.00	0	81	9	2.35	5.40	28.6	0	0.00	0
20	7	2.00	8.65	10	0	0.02	0	82	9	1.08	4.38	50	0	0.00	0
21	7	1.20	14.95	21	0	0.00	0	83	7	0.85	1.80	2	1	1.0	1
22	8	2.25	2.30	4	1	1.0	1	84	7	1.09	4.15	5	1	1.0	1
23	8	2.30	5.30	10	1	0.72	1	85	7	1.20	2.45	8	1	0.81	0
24	8	2.50	2.30	4	1	1.0	1	86	7	0.80	1.35	6	1	1.0	0
25	8	2.90	5.30	9	1	1.0	1	87	7	0.50	1.7	3	1	1.0	1
26	8	0.60	1.30	8	1	1.0	1	88	7	1.60	2.1	8	1	0.67	0
27	8	3.00	3.30	12	1	1.0	0	89	7	0.76	3.9	5	1	1.0	1
28	8	1.35	4.30	1	1	1.0	1	90	7	1.10	3.3	7	1	1.0	0
29	8	1.10	5.30	6	1	1.0	1	91	7	1.40	2.3	2	1	1.0	1
30	8	3.85	9.10	17	0	0.00	0	92	7	1.10	6.3	9	1	1.0	0
31	8	1.53	11.9	26	0	0.00	0	93	7	0.76	4.3	16	0	0.00	0
32	8	1.10	1.30	15	0	0.00	0	94	7	1.37	3.6	19	0	0.00	0
33	8	2.80	3.30	16	0	0.00	0	95	7	1.05	8.8	14	0	0.00	0
34	8	2.00	3.46	8	0	0.74	1	96	7	1.05	3.3	19	0	0.00	0
35	9	1.15	4.50	22	1	0.00	0	97	7	1.00	4.8	14	0	0.00	0
36	9	3.50	4.45	13	1	0.98	1	98	7	0.65	14.4	16	0	0.00	0
37	9	1.37	5.12	4	1	1.0	1	99	8	1.60	8.7	8	1	1.0	1
38	9	1.05	5.05	10	1	1.0	1	100	8	3.30	5.8	5	1	0.99	1
39	9	2.44	3.35	11	1	0.64	1	101	8	1.12	9.22	12	1	0.98	1
40	9	1.05	5.95	9	1	1.0	1	102	8	3.00	5.1	9	1	0.99	1
41	9	1.25	6.80	10.5	1	1.0	1	103	8	3.20	7.2	8	1	0.96	1
42	9	1.98	8.62	7	1	1.0	1	104	8	2.30	3.3	9	1	0.68	1
43	9	0.85	3.30	9	1	1.0	1	105	8	0.75	9.4	11	1	0.99	1
44	9	0.61	3.80	11.5	1	0.99	1	106	8	0.65	8.45	11.5	1	0.97	1
45	9	5.60	20.00	16	0	0.00	1	107	8	1.00	2.9	7.5	1	1.0	1
46	9	5.30	14.80	73	0	0.00	0	108	8	0.65	5.3	1	1	1.0	1
47	9	5.00	13.52	64	0	0.00	0	109	8	1.50	2.65	6	1	1.0	1
48	9	4.90	9.38	61	0	0.00	0	110	8	2.30	12.3	13	0	0.00	1
49	9	5.90	6.43	14	0	0.00	1	111	8	3.30	13.80	17	0	0.00	1
50	9	2.30	6.40	18	0	0.01	1	112	8	3.10	4.30	15	0	0.05	0
51	7	1.5	7.0	6.0	1	1.0	1	113	8	3.10	9.30	51	0	0.00	0
52	8	1.5	10.5	11.0	1	0.97	1	114	9	3.65	5.85	16.8	1	0.94	1
53	7	2.0	3.0	6.0	1	0.94	1	115	9	1.00	2.30	7	1	1.0	1
54	7	1.0	12.0	15.5	0	0.00	0	116	9	1.01	1.80	14	1	1.0	1
55	7	2.0	13.0	14.5	0	0.00	0	117	9	1.30	3.60	4	1	1.0	1
56	6	1.64	0.8	3.6	1	0.97	1	118	9	1.3	3.80	17	0	0.01	1
57	7	2.5	1.9	12.5	0	0.00	0	119	9	1.05	2.80	17	0	0.04	1
58	7	2.0	2.0	5.0	1	0.93	1	120	9	1.45	5.40	16.5	1	0.94	1
59	7	0.1	2.0	6.1	1	0.96	1	121	9	1.03	4.41	14.6	1	0.99	1
60	7	0.1	2.0	4.3	1	1.0	1	122	9	1	2.85	12	1	0.88	1
61	7	0.0	1.0	2.5	1	1.0	1	123	9	1	1.50	13	1	1.00	1
62	7	0.0	0.9	2.1	0	1.0	1								

Table 3 Evaluation of Liquefaction Possibilities (40 samples for forecasting)

No	I_s	D_w	D_s	S_p	L_a	L_n	L_c	No	I_s	D_w	D_s	S_p	L_a	L_n	L_c
1	9	1.57	3.75	7.0	1	1.00	1	21	8	0.45	0.80	13.0	0	0.00	0
2	9	5.70	10.15	34.0	0	0.00	0	22	8	1.40	1.74	18.0	0	0.00	0
3	9	1.80	4.00	12.0	0	0.70	1	23	8	0.81	1.00	18.0	0	0.00	0
4	9	3.50	8.35	31.0	0	0.00	0	24	8	1.40	1.40	18.0	0	0.00	0
5	9	4.50	4.50	22.0	0	0.00	0	25	8	1.40	0.88	18.0	0	0.00	0
5	9	4.54	7.30	18.0	0	0.00	1	26	8	0.94	0.70	11.0	0	0.01	0
7	9	5.50	9.95	30.0	0	0.00	0	27	8	2.02	3.33	15.7	0	0.00	0
8	7	1.50	5.20	5.5	1	1.00	1	28	8	2.63	1.52	4.3	1	1.00	1
9	7	1.50	6.40	6.0	1	1.00	1	29	9	1.29	2.13	9.3	1	1.00	1
10	8	2.00	10.00	9.0	1	0.82	1	30	9	2.50	1.90	12.5	1	1.00	1
11	8	2.00	10.30	9.0	1	0.57	1	31	9	2.00	4.40	32.0	0	0.00	0
12	7	1.00	8.00	15.0	0	0.00	0	32	9	2.00	1.00	15.0	1	0.02	0
13	7	1.00	12.00	20.0	0	0.00	0	33	9	0.80	1.40	5.7	1	1.00	1
14	6	1.45	2.34	7.0	1	0.70	0	34	9	1.00	1.50	16.0	1	0.00	0
15	7	2.77	1.45	12.0	0	0.00	0	35	9	1.50	3.50	8.9	1	1.00	1
16	7	1.00	2.00	2.5	1	1.00	1	36	9	1.50	1.70	9.3	1	0.98	1
17	7	1.00	4.00	2.5	1	1.00	1	37	9	1.50	6.40	11.0	1	1.00	1
18	8	0.00	6.50	6.1	0	1.00	1	38	9	2.77	1.45	12.0	1	0.58	1
19	8	1.12	1.15	6.8	1	1.00	1	39	9	0.43	2.61	10.0	1	1.00	1
20	8	0.70	0.76	15.0	0	0.00	0	40	9	1.00	2.75	9.0	1	1.00	1

The physical meanings of symbols in Table 2 and Table 3 are as follows:

I_s -- seismic intensity; D_w -- depth of underground water level (M); D_s -- buried depth of liquefaction-potential soil (M); S_p -- numbers of SPT; L_a -- actual liquefaction index (1: liquefied site, 0: unliquefied site); L_n -- neuro-net defined liquefaction index (a digit of real type where > 0.5 means high potential of liquefaction and < 0.5 means low potential or unliquefiable); L_c -- code based identification index of liquefaction (1: liquefiable site, 0: unliquefiable site)

Table 4 Identification of PGA (35 samples for programming)

N_o	M	R	T_S	PGA_a	PGA_n	N_o	M	R	T_S	PGA_a	PGA_n
1	7.6	142.5	0.29	50.05	52.73	19	4.0	17.9	0.20	50.65	54.88
2	7.6	128.1	0.27	48.01	46.77	20	4.0	17.9	0.20	62.51	57.12
3	6.7	3.8	0.20	541.66	526.22	21	3.8	1.0	0.20	74.30	69.77
4	6.7	3.8	0.20	506.44	526.22	22	3.7	13.2	0.20	94.77	100.02
5	6.3	17.7	0.05	53.36	50.66	23	3.6	19.4	0.20	61.30	54.88
6	6.3	17.6	0.05	48.79	50.66	24	3.5	18.5	0.20	46.53	57.12
7	6.3	13.0	0.18	103.02	108.35	25	3.4	3.8	0.20	75.71	69.77
8	6.1	23.7	0.30	53.93	57.12	26	3.4	3.8	0.20	66.40	72.62
9	6.1	58.7	0.20	24.98	24.66	27	3.2	7.0	0.20	76.40	69.77
10	5.3	33.4	0.30	69.28	69.77	28	3.2	7.0	0.20	68.11	69.77
11	5.0	11.8	0.30	295.95	161.65	29	3.2	19.6	0.18	24.43	24.66
12	4.6	10.8	0.30	90.02	161.65	30	3.1	18.0	0.20	52.78	54.88
13	4.5	4.9	0.20	196.04	197.45	31	3.1	18.0	0.20	58.34	52.73
14	4.5	13.2	0.18	56.13	50.66	32	3.1	19.3	0.20	49.22	48.68
15	4.5	18.5	0.30	45.86	48.68	33	3.1	19.3	0.20	46.32	48.68
16	4.3	16.6	0.30	61.23	64.41	34	3.1	9.8	0.20	56.55	59.46
17	4.1	18.9	0.30	67.71	67.04	35	2.9	11.6	0.20	27.74	27.80
18	4.1	10.6	0.30	140.75	143.37						

Note: M -- magnitude; R -- distance from epicenter to site (Km);
T_s -- characteristic period of site (sec); PGA_a -- actual PGA;
PGA_n -- net-output PGA (cm/sec^2)

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EARTHQUAKE DAMAGE ESTIMATION AND REHABILITATION PRIORITIZATION OF BURIED LIFELINES

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ABSTRACT

Buried lifelines which include water, sewer and gas pipelines etc. are important to the health, supply and safety of the populace. Most buried pipelines under operation in urban areas were old and deteriorated. Furthermore, they have not been designed to resist earthquakes. Seismic rehabilitation or retrofit is a cost-effective way to prevent pipeline damage caused by future earthquakes. In general, it is very difficult, if not impossible, to rehabilitate all buried pipelines at the same time because of limited funds, time, and other available resources.

This paper presents a priority strategy for rehabilitation of buried pipelines considering several important factors such as pipeline damage probability, rehabilitation cost, rehabilitation rate (e.g. km/day), pipeline importance factor, and total funds available.

KEYWORDS

Buried pipelines; earthquake; earthquake damage; earthquake engineering; lifelines; lifeline earthquake engineering; pipelines; pipeline damage probability; prioritization; rehabilitation; rehabilitation prioritization.

INTRODUCTION

Buried water, sewer and gas pipelines have been damaged heavily by earthquakes, including the recent Northridge earthquake of January 17, 1994 (Hall, 1994) in the U. S. and the Hyogo-ken Nambu earthquake of January 17, 1995 (Comartin, et al., 1995, INCEDE, 1995) in Japan. Because of the importance of buried lifelines to the health, supply and safety of the populace, mitigation of earthquake damage of buried lifelines becomes a worldwide concern. The response behavior of buried pipelines has been studied due to recent earthquakes in China, Japan and the United States (Hou et al., 1994, Kubo, 1980, Technical Committee of TCLEE, 1983, Wang et al., 1985). Planning for post-earthquake recovery (Johnson, 1983, McCaffrey & O'Rourke, 1983), emergency operation (Katayama, 1983), and repair and rehabilitation of buried pipeline systems (Katayama, 1983, Shinozuka et al, 1991, Wang & Li, 1993) after earthquakes have also been studied by many researchers. Unfortunately, most of the existing literatures (Eguchi, 1983; Ramos, 1983, Shinozuka et al., 1991) concerning restoration and recovery of buried pipelines gave a qualitative rather than quantitative description.

Many existing water, sewer and gas pipelines have been built several decades ago without seismic considerations. They are unsafe even under moderate earthquakes. The improvement of earthquake resistance of buried pipelines is urgently needed in earthquake prone areas. With the proper rehabilitation, earthquake damage can be mitigated. Pipeline rehabilitation is costly. It is impossible to rehabilitate all pipelines in a system in a short time for insufficient manpower, equipment, materials and funds. The paper provides a scheme/guideline showing the priority for rehabilitation of pipelines of any existing system.

REHABILITATION SCHEME

A water, sewer or gas pipeline system can be divided into mains and branches as shown in Fig. 1. Pipelines in the main system are much more important than pipelines in the branch system. Before starting the rehabilitation of the branch system, pipelines in the main system must be rehabilitated completely. The rehabilitation of the main system is assumed to be performed pipe link by pipe link, while branch system is to be performed area by area. Once a pipe link or an area is rehabilitated, the performance of the rehabilitated pipe links or areas should be monitored to evaluate the effectiveness of the actions taken.

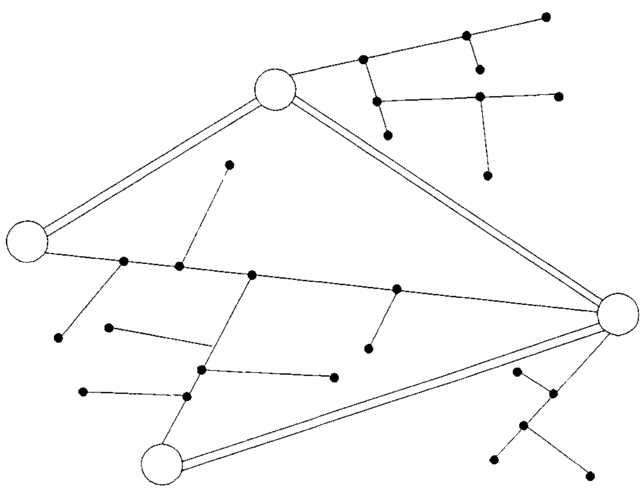


Fig. 1 Main and Branch of a Pipeline System

The strategy to establish priorities for rehabilitating buried pipelines requires the following tasks: (1) evaluating the damage probability of buried pipe links using a probability-based Earthquake Damage Estimation Model (**EDEM**) (Li, 1994, Wang & Li, 1993) which will include such important parameters as pipe diameter, soil condition, pipe material, joint type, buried depth, buried age and earthquake intensity, etc.; (2) calculating the node weights of the system which will represent the importance of the network nodes; (3) calculating the pipe link weights of the network, either based on a cost or a time factor, which will show the significance of the pipe links; (4) generalizing the source expansion tree which will enable the ranking of the damage probability of all pipe links of the pipeline system; and (5) setting the priorities based on the generalized link weight which is defined as the summation of node weights for the nodes served by the pipe link. The pipe link which has the largest link weight is the one to be rehabilitated first, and so on.

To assist the development of pipeline rehabilitation strategy (Wang and Li, 1993), this paper reports the statistical analysis of the pipeline damage data from 7 past earthquakes occurred in China, Japan and the United States. Pipeline damage probabilities serve as a basis for determining rehabilitation priority.

DESIRABLE DATABASE

For developing a comprehensive earthquake damage estimation model for buried pipelines, it is desirable that the following information items be included in the database for each pipe link of the pipeline system: (1) pipe diameter; (2) pipe material; (3) joint type; (4) pipe link length; (5) surrounding soil condition; (6) buried depth; (7) buried year; (8) number of customers served; (9) unit rehabilitation cost ($/km); (10) rehabilitation rate (km/day); (11) suitable rehabilitation method; (12) functional usage (i.e. residential or commercial); and (13) expected earthquake intensity (MMI), etc.. In this paper, it is assumed that most, if not all of the data on above information items are available for each pipe link of the system and have been stored in the computer database.

PARAMETERS AND ASSUMPTIONS

Parameters

In earthquake damage investigations, pipeline failures in term of breaks per unit length (bks/km) are often reported in a statistical manner without analysis/correlations to the influential parameters. To develop a workable damage probability model, it would be too difficult, if not impossible, to correlate the damage data to all influential parameters. In this paper, some factors such as internal pressure, fault movement, and landslide are not considered due to limited damage data obtained from past earthquakes. Since the damage rate (e.g. bks/km) of buried pipelines has been found to have strong relations with some independent influential parameters, these parameters are assumed to be the statistically independent for simplicity reasons. Although more parameters can be included in EDEM if sufficient data are available, only six statistically independent parameters shown below are used in the demonstration analysis discussed later: (1) Pipe diameter; (2) Soil condition (including soil liquefaction effects); (3) Pipe types (the combination of pipe material and joint type); (4) Earthquake intensity; (5) Buried age (including corrosion effects); (6) Buried depth.

Assumptions

The assumptions used for the probability damage model are:

(1) Pipeline system information is already in database;
(2) The influential parameters are considered statistically independent (Freund, 1971);
(3) Pipe link damage follows the Poisson's Law (Crovelli, 1973) along the length.

PIPE LINK DAMAGE PROBABILITY

According to above assumptions, the damage probability of a pipe link is the combined damage probabilities of all statistically independent parameters. Thus, the damage probability model for the pipeline can be written as:

$$P_j = 1 - [1 - P_j^d(i)] \cdot [1 - P_j^s(k)] \cdot [1 - P_j^m(y)] \cdot [1 - P_j^e(x)] \cdot [1 - P_j^a(l)] \cdot [1 - P_j^b(g)] \tag{1}$$

where P_j is the damage probability of a pipe link j. P_j^d, P_j^s, P_j^m, P_j^c, P_j^a, and P_j^b are damage probabilities with respect to pipe diameter, soil condition, pipe type, earthquake intensity, buried age and buried depth, respectively. i, k, y, x, l, and g are indices for parameter domains (**PD**) of pipe diameter, soil condition, pipe type, earthquake intensity, buried age and buried depth, respectively. The parameter domains verse domain indices are given in Table 1.

Table 1 Summary of Parameter Domains

Index	Parameter Domain					
	pipe diameter (mm)	soil condition SWV (m/s)	pipe type	buried age (year)	buried depth (m)	earthquake intensity (MMI)
1	0-74	0-149	CPCF	0-9	0-1.49	0-12
2	75-99	150-449	CPLF	10-19	1.5-1.79	
3	100-124	450-749	PVCP	20-29	1.8-2.09	
4	125-149	750-1799	CIGJ	30-39	2.1-2.39	
5	150-199	≥1800	DIMJ	40-49	2.4-2.99	
6	200-249		DIFJ	50-59	3.0-3.59	
7	250-299		CSP	60-69	≥3.6	
8	300-349			≥70		
9	350-399					
10	400-449					
11	450-499					
12	500-549					
13	550-599					
14	600-699					
15	≥700					

Table 1 show the **PD**s for various parameters. From Table 1, one can see that the **PD**s for pipe diameter, buried age, buried depth and earthquake intensity are in numerical forms which are self-explanatory. Shear wave velocity is used to classify the soil conditions. For liquefiable soil, the shear wave velocity is zero. It belongs to soil condition index (category) one. Pipe types including joint properties are classified into 7 pipe type indices (categories) listed below:

(1) Segmented pipe with asbestos cement or cement mortar filler which includes ACP, CP, CIP and WIP. For simplicity, this type is called Chain Pipe with Cement Filler (CPCF);
(2) Segmented pipe with lead filler which includes ACP, CP, CIP and WIP. This type is called Chain Pipe with Lead Filler (CPLF);
(3) Polyvinyl chloride pipe (PVCP) with rubber gasket joint;
(4) Cast Iron pipe with rubber Gasket Joint (CIGJ);
(5) Ductile Iron pipe with Mechanical Joint (DIMJ);
(6) Ductile Iron pipe with Flexible Joint (DIFJ);
(7) Continuous Steel Pipe which includes steel pipe with screw joint and steel pipe with welded joint (CSP).

According to the assumption that pipeline damage follows Poisson's Law along the pipe length, the damage probability with exactly c breaks for each parameter can be expressed as:

$$P_j^c(z) = \frac{[R_j(z)\ L_j]^c}{c!} e^{-R_j(z)\ L_j} \qquad (2)$$

where z is an index correlated to 6 parameters i, k, y, x, l or g shown in Eq 1. c is a random number and can be 0, 1, 2, and so on. When c equals zero, $P_j^0 = e^{-Rj(z)Lj}$ is the 0-break damage probability which means reliability. L_j is the pipe link length (km). R_j is the damage rate. The damage probability, Eq. 2, can also be expressed as:

$$P_j(z) = 1.0 - e^{-R_j(z)\,L_j} \tag{3}$$

Statistical Damage Rates of Pipelines

In order to analyze and compare pipeline damage for different **PD**s, a statistical damage rate, which is defined as breaks per unit length (bks/km) is used:

$$R_s(\, z\,) = \frac{N_b}{L_j} \tag{4}$$

where N_b is the number of breaks and L_j, the length (km) of pipe link j. $R_s(z)$ is the statistical damage rate. Subscript s means that the damage rate is calculated from statistical damage data.

Normalized Damage Rate of Pipelines

From statistical damage data, the pipelines with the same parameter have different damage rates from different earthquakes. Even in one earthquake event, the pipelines with the same parameter also have different damage rates in different cities or different areas. Note that pipeline damage assessment in this paper is for a comparative study. Relative values would be sufficient to serve the purpose. In order to compare damage rates of the same pipeline in different earthquakes or to compare damage rates of different pipelines in the same earthquake, we introduce the concept of a normalized damage rate defined as:

$$R_n(\, z\,) = \frac{R_s(\, z\,)}{\sum_{k=N^1}^{N^f} R_s(\, k\,)} \tag{5}$$

where $R_n(z)$ is the normalized damage rate. Subscript n means normalization. N^1 is the first index and N^f is the last index correlated to parameter z under the statistical analysis in an earthquake event.

Projected Damage Rate of Pipelines

In order to correlate pipe damage rate with all **PD**s for each parameter, we extend the statistical lines in both directions: one is toward smaller **PD** indices, another toward larger **PD** indices by the regression method (Crovelli, 1973; Freund, 1971). The values correlated to **PD** indices are called the projected damage rates.

Normalized Projected Damage Rate of Pipelines

From the projected damage rates, one can easily obtain the normalized projected damage rate by the following equation:

$$\begin{aligned} R_{np}(\, z\,) &= \frac{\textit{Projected Damage Rate } R_p(\, z\,)}{\textit{Total Projected Damage Rate}} \\ &= \frac{R_p(\, z\,)}{\sum_{k=1}^{N^f} R_p(\, k\,)} \end{aligned} \tag{6}$$

were $R_{np}(z)$ is the normalized projected damage rate. From Eq. 6 one can see that

$$\sum_{k=1}^{N^f} R_{np}(k) = 1 \tag{7}$$

Weighted Damage Rate of Pipelines

From the statistics, one may visualize that the normalized projected damage rate of each **PD** has different value for different earthquake. Now, a weighted damage rate of pipe link j is introduced by considering earthquake intensity as:

$$R_{jw}(z) = \frac{\sum_{k=1}^{N_e} E_k \, [R_{np}(z)]_k}{\sum_{k=1}^{N_e} E_k} \tag{8}$$

where $R_{jw}(z)$ is the weighted damage rate of pipe link j corresponding to **PD** index z. N_e is the number of earthquakes to be considered. In this study, $N_e = 7$ for data from seven earthquakes. E_k is (k)th earthquake energy (Dowrich, 1978 which is expressed by

$$Log\ E_k = 11.4 + 1.5\ M_k \tag{9}$$

in which M_k is the magnitude of earthquake k. The weighted damage rates for the six parameters using data from 7 earthquakes extracted from Li's dissertation (Li, 1994) are shown in Table 2.

Table 2 Summary of Normalized Damage Rate with Respect to Various Parameters

	Parameter Domain					
Index	**pipe diameter (mm)**	**soil condition SWV (m/s)**	**pipe type**	**buried age (year)**	**buried depth (m)**	**earthquake intensity (x)**
1	0.2622	0.4642	0.3137	0.0494	0.3722	for $0 \leq x \leq 9$
2	0.1542	0.2786	0.2278	0.0551	0.2096	$\frac{10^{1.717x-14.222}}{4.2741}$
3	0.1095	0.1253	0.1906	0.0626	0.1484	for $x > 9$
4	0.0856	0.0863	0.1075	0.0729	0.1104	0.8101
5	0.0709	0.0456	0.0654	0.0883	0.0757	
6	0.0537		0.0535	0.1141	0.0500	x is the
7	0.0439		0.0416	0.1684	0.0336	Modified
8	0.0377			0.3892		Mercalli Intensity
9	0.0333					
10	0.0300					
11	0.0274					
12	0.0254					
13	0.0238					
14	0.0223					
15	0.0201					
total	1.0000	1.0000	1.0000	1.0000	1.0000	

VERIFICATION OF DAMAGE PROBABILITY MODEL

The verification of the damage probability model discussed above was carried out using data of pipelines near the Upper Norman Reservoir during the 1971 San Fernando earthquake (McCaffrey and O'Rourke, 1983). Among the different pipelines, the **PD** values or indices for pipe diameter, buried age, buried depth can be obtained from Table 1. The **PD** for soil conditions are determined by the ground deformation after the earthquake (McCaffrey, 1983). The pipe type is assumed to be the same as category index 7 using the stated definitions. Earthquake intensity of 6.6 is also assumed to be the same over the entire area. The details of the verification has been given in the Li's dissertation (Li, 1994) and thus will not be repeated herein.

PRIORITIZATION FOR REHABILITATION OF PIPELINES

Node Weight

Node weight which is denoted by NW_i represents the importance and the reliability of the node i. It is defined as:

$$NW_i = \frac{1}{2} \sum_{j=1}^{N_i} \alpha_j P_j \tag{10}$$

where NW_i is the number of pipelines directly connected to node i. α_j is the importance factor of pipe j. The node which has the largest node weight in the system is the most important node.

Determination of Importance Factor

The importance factor α_j of pipe j is determined by:

$$\alpha_j = \alpha^1_j \, \alpha^2_j \, \alpha^3_j \tag{11}$$

where α_j^1 is the pipeline usage factor. For examples, pipelines used for emergencies, hospitals, schools and residents are much more importance than those for shops or factories. The more important the pipeline is, the larger value the pipeline usage factor should be.

α_j^2 is the system factor. It is known that pipelines in the main system are more important than in the branch system. The system factor for pipeline in the main system should be larger than in the branch system.

α_j^3 is the population factor. According to the population served by pipelines, the more population the pipeline serves, the more important the pipeline is. The population factor represents the importance of pipeline from the number of population served by the pipeline.

Obviously, the values of three factors should be decided by expert experience. The recommended values are: $\alpha_j^1 = 1$ for normal daily use; $\alpha_j^1 > 1$ for emergency and hospital use; $\alpha_j^2 = 1$ for branch pipeline; $\alpha_j^2 > 1$ for main pipeline; α_j^3 can be determined by:

$$\alpha^3_j = e^{\frac{M_j}{\sum_{k=1}^{N} M_k}} \tag{12}$$

where M_j is the population served by pipeline j; ΣM_k is the total number of population served by the pipeline system. Actually, α_j^3 can also be recommended as:

$$\alpha^3_j = e^{\frac{Q_j}{\sum_{k=1}^{N} Q_k}} \tag{13}$$

where Q_j is the normal water flow of pipeline j; ΣQ_k is the total water flow in the pipeline system.

Link Weight

Link weight which is denoted by LW_j represents the importance of pipe link j. It is defined as:

$$LW_j = NW_i + NW_k \tag{14}$$

where i and k are two nodes of pipe j. The more link weight the pipe link has, the more important the pipe link is.

Rehabilitation Time of each Pipeline

Rehabilitation time T_j of a pipe link j is defined as:

$$T_j = \frac{L_j}{\gamma_j} \tag{15}$$

where γ_j is the evaluated rehabilitation rate (km/day) of pipe link j. Note that the rehabilitation rate represents the suitability and the efficiency of rehabilitation of the pipe link j. Some pipe links may be difficult or impossible to be rehabilitated from the standpoint of present technology or funds available. From this viewpoint of rehabilitation rate, rehabilitation should start from pipelines which have less rehabilitation difficulty and less expense.

In general, the speed or rate of rehabilitation has relations with rehabilitation method, equipment, surrounding environment and manpower used for carrying out the rehabilitation work, etc. Before carrying out the prioritization scheme, the rehabilitation plan which includes rehabilitation method, rehabilitation efficiency and cost required for each pipeline in the system is supposed to be stored in the database.

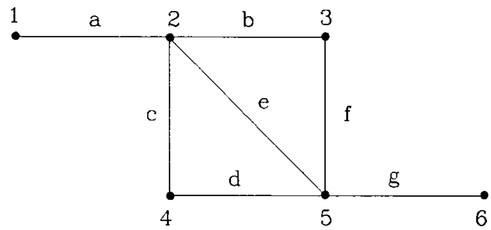
Link Weight Efficiency

Theoretically, a pipe link which has the largest link weight is the most important pipeline and needs to be rehabilitated first. But the pipe link may have very low rehabilitation rate. Link weight efficiency denoted by LWE_j represents the link weight per unit rehabilitation time, which is defined as:

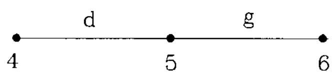
$$LWE_j = \frac{LW_j}{T_j} \tag{16}$$

Definition of a Sub-system

A composite sub-system (denoted by $S_{(a,b,c,...)}$, a, b, c are node numbers) is defined as a system that all nodes in the sub-system are connected to at one by pipe link which will not be rehabilitated. If a node is connected by pipelines that all will be rehabilitated, the node is called a node sub-system. As indicated earlier, if there are more than two nodes in a sub-system, any node will be connected to at least by one pipe link which will not be rehabilitated. For example, figure 2(a) shows a simple pipeline system. It is assumed that pipelines a, b, c and g will be rehabilitated. The system can be expressed by sub-systems as in Fig. 2(b). Sub-system $S_{(1)}$ is a node sub-system. Since there is only one node, the pipe link, a, connected to that node is to be rehabilitated. Sub-system $S_{(2,3,4,5)}$ is a composite subsystem with four nodes and five pipe links. Any node in this sub-system is connected to at least one pipe link which will not be rehabilitated.



(a) Pipeline System



(b) Sub-system

Fig. 2 An Example of Pipeline System

Prioritization Scheme for Main System

To decide the priorities of pipeline rehabilitation in a system with the objective of minimizing rehabilitation time, there are four items to be considered:

(1) Total funds available;
(2) Damage probability of each pipeline;
(3) Rehabilitation time of each pipeline; and
(4) Rehabilitation cost of each pipeline.

The procedure of prioritization for pipeline rehabilitation includes the following steps:

Step one is to make a list of all pipe links in the main system according to damage probabilities from the largest one to the smallest one.

Step two is to sum the rehabilitation costs required according to the list in step one to be equal or within the total funds available, i.e.,

$$C = \sum_{k=1}^{N1} C_k \leq C^T \tag{17}$$

$$\sum_{k=1}^{N1+1} C_k > C^T \tag{18}$$

where C_k is the cost required for the rehabilitation of pipeline k, C^T is the total funds available and N1 is
the number of pipe links to which the total cost for rehabilitation is under the total funds, C, available. P^c is the damage probability of pipeline No.N1.

Step three is to divide the N1 pipe links in the main system, which have damage probability larger than or equal to P^c into M sub-systems. If there are more than two sources, all sources in the system are assumed to be one sub-system.

Step four is to establish the optimization objective. The objective is to minimize the rehabilitation time T of the system to make water, sewer or gas reach all nodes in the system. As it is known that in a system with M sub-systems, the minimum requirement to connect M sub-systems is M-1 pipe links. The objective of minimizing the rehabilitation time can be expressed as:

$$T = \sum_{k=1}^{M-1} T_k \tag{19}$$

for all M sub-systems connected by M-1 pipe links;

Step five is to search for a Generalized Source Expansion Tree. The method is called Generalized Source Expansion Method. The expansion for a sub-system starts from the source connected to the pipe link which has the smallest rehabilitation time to an immediate node, then continues to expand to other sub-systems until all sub-systems are covered. Every expansion covers one sub-system and one pipe link. This method expanding from the source to other nodes is called Generalized Source Expansion Method. The final system which includes M sub-systems and M-1 pipe links like a tree is called the generalized source expansion tree. When an earthquake occurs, the pipeline system should be in operation without failure if these M-1 pipe links of the expansion tree have been properly rehabilitated.

Step six is to calculate sub-system weight. The sub-system weight SSW_i of the sub-system i is defined as:

$$SSW_i = \sum_{k=1}^{N_i} NW_k \tag{20}$$

where N_i is the number of nodes in the sub-system i.

Step seven is to calculate generalized link weight. The generalized link weight GLW_j of pipe link j is defined as the summation of all sub-system weights associated with pipelines served by the pipe link j, which is:

$$GLW_j = \sum_{k=1}^{N_s} SSW_k \tag{21}$$

where N_s is the number of sub-systems served by the pipe link j.

Step eight is to calculate link weight efficiency of other pipe links in the system which have damage probability larger than or equal to P^c and are not covered by generalized source expansion tree according to Eq. 16.

Step nine is to establish the priorities for pipe link rehabilitation. Pipe links covered in the generalized source expansion tree are listed according to generalized link weight from the largest one to the smallest one followed by other pipe links according to the link weight efficiency also from the largest one to the smallest one. No.1 in the list is the one on which the rehabilitation work will be carried out first. No.2 is the second and so on.

Prioritization Scheme for Branch System

According to the rehabilitation scheme, the rehabilitation work of branch system is assumed to be performed area by area. The procedure of pipeline prioritization for rehabilitation in branch system is as follows:

Step one and step two are the same above by changing main system to branch system.

Step three is to calculate average link weight efficiency which is defined as:

$$ALWE_i = \frac{\sum_{k=1}^{M_i} LW_k}{\sum_{k=1}^{M_i} T_k} \tag{22}$$

where $ALWE_i$ is the average link weight efficiency of area i and M_i is the number of pipelines with damage probability larger than or equal to P^c.

Step four is to make a list of all areas according to the average link weight efficiencies from the largest one to the smallest one. The area that has the largest link weight efficiency will be rehabilitated first.

REHABILITATION EXAMPLE

Figure 3(a) shows an example of main pipeline system. In this example, node 1 is a source. Table 3 shows the assumed domain parameter indices of pipe links in the system. Damage rates with respect to each system pipe links can be taken from Table 2 and are shown in Table 4. Damage probability of each pipe link parameter can be calculated by first finding damage probability along the length according to Poisson's Law using Eq. 2 and then calculating over all (including all parameters) pipe link the damage probability of the pipe link. The results are shown in Table 5. Rehabilitation rate, rehabilitation cost and importance factor of each pipe link are assumed also shown in Table 5.

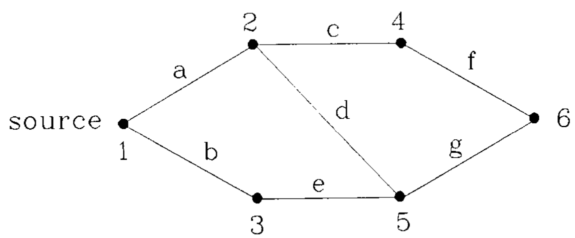


Fig. 3a Main Pipeline System

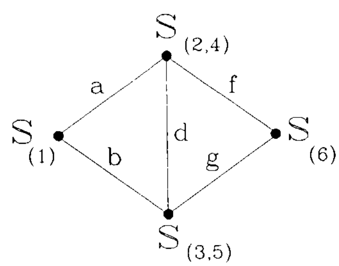


Fig. 3b Sub-system Expression

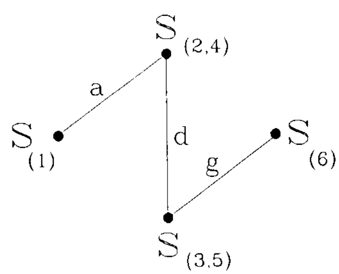


Fig. 3c Generalized Source Expansion Tree

Table 3 Domain Parameter Indices of Pipelines in Fig. 3

Pipeline	Domain Parameter Indices						Length (km)
	pipe diameter	soil condi-tion	pipe type	earthquake intensity	buried age	buried depth	
a	8	2	7	7	6	3	1
b	8	2	7	7	6	3	1
c	7	4	7	7	5	3	0.8
d	7	4	7	7	5	3	1.8
e	7	4	7	7	5	3	0.8
f	7	2	7	7	4	3	1
g	7	2	7	7	4	3	1

Table 4 Damage Rate correlated to each Index in Table 3

Pipeline	Domain Parameter Indices					
	pipe diameter	soil condition	pipe type	earthquake intensity	buried age	buried depth
a	0.0377	0.2786	0.0416	0.0015	0.1141	0.1484
b	0.0377	0.2786	0.0416	0.0015	0.1141	0.1484
c	0.0439	0.0863	0.0416	0.0015	0.0883	0.1484
d	0.0439	0.0863	0.0416	0.0015	0.0883	0.1484
e	0.0439	0.0863	0.0416	0.0015	0.0883	0.1484
f	0.0439	0.2786	0.0416	0.0015	0.0729	0.1484
g	0.0439	0.2786	0.0416	0.0015	0.0729	0.1484

Table 5 Rehabilitation Information of Pipelines in Fig. 3

Pipeline	Damage Probability (%)	Rehabilitation Rate (km/day)	Rehabilitation Cost ($)	Importance Factor
a	46.09	0.04	1000	1.5
b	46.09	0.03	2000	1.5
c	27.95	0.04	3000	1
d	38.27	0.06	2000	1
e	27.95	0.04	3000	1
f	44.40	0.03	1000	1
g	44.40	0.04	2500	1

Using the pipe link damage probabilities and the assigned importance factors given in Table 5, the node weights of the pipeline system can be calculated by using Eq. 10 and shown below:

node	1	2	3	4	5	6
NW	0.6914	0.6768	0.4854	0.3618	0.5531	0.4440

For this example, it is assumed that the total funds available for pipeline rehabilitation is $10,000. The prioritization procedures for rehabilitation by minimizing the rehabilitation time are:

Step one: The order of pipe link list according to the damage probability is a, b, d, g, f, c and e. When there are more than two pipelines that have the same damage probability, the one with the smallest rehabilitation time goes first on the list.

Step two: The cumulative costs on the rehabilitation of pipe links based on the probability order is shown below:

Pipe Link	Damage Probability	Rehab Rate	Cost($)	Cumulative Cost($)
a	0.4609	0.04 km/day	1000	1000
b	0.4609	0.03 km/day	2000	3000
g	0.4440	0.04 km/day	2500	5500
f	0.4440	0.03 km/day	1000	6500
d	0.3827	0.06 km/day	2000	8500
c	0.2795	0.04 km/day	3000	11500
e	0.2795	0.04 km/day	3000	14500

By comparing the total funds of $10,000 available, the reference damage probability, P^c is found to be 38.27%, and pipe links a, b, g, f, and d could be rehabilitated within the budget of the given year.

Step three: Since pipe links c and e will not be rehabilitated, four sub-systems as shown in Fig. 3(b) can be constructed. Note that the sub-systems $S_{(1)}$ and $S_{(6)}$ are node subsystems that all connecting pipe links will be rehabilitated. $S_{(2,4)}$ and $S_{(3,5)}$ are composite subsystems in which the nodes are connected to at least one pipe link that will not be rehabilitated.

Step four: For four sub-systems, the minimum requirement to connect these four sub-systems is 3 pipe links. The objective function is the rehabilitation time to be minimum as shown below:

$$T = \sum_{k=1}^{3} T_k \qquad (24)$$

Step five: Source expansion starts from $S_{(1)}$ along the pipe link a to $S_{(2,4)}$, then along the pipe link d to $S_{(3,5)}$, finally, along the pipe link g to $S_{(6)}$. The generalized source expansion tree is shown in Fig. 3(c). The minimum rehabilitation time is 80 days (1 km/0.04 km/day = 25 days for link a, 1.8 km/0.06 km/day = 30 days for link d, and 1 km/ 0.04 km/day = 25 days for link g). If an earthquake occurs after 80 rehabilitation days, the system should be safe and is under normal operation.

Step six: The sub-system weights are:

Sub-system	Sub-system Weight
$S_{(1)}$	0.6914
$S_{(2,4)}$	0.6768+0.3618=1.0386
$S_{(3,5)}$	0.4854+0.5531=1.0385
$S_{(6)}$	0.4440

Step seven: The generalized link weights are:

Pipe Link	Generalized Link Weight
a	1.0386+1.0385+0.4440=2.5211
d	1.0385+0.4440=1.48225
g	0.4440

Step eight: Link weight efficiencies of pipelines b and f are:

Pipe Link	Link Weight Efficiency (link weight/day)
b	(0.6914+0.4854)*0.03=0.0354
f	(0.3618+0.4440)*0.03=0.0242

Step nine: The list according to the generalized link weight and the link weight efficiency is a, d, g, b, and f. Pipe link a is the one to be rehabilitated first, then pipeline d and so on. Pipeline f is the last one to be rehabilitated with the funds available.

CONCLUSIONS

The prioritization scheme for rehabilitation of buried pipelines provides a rehabilitation planning. Pipelines no matter in main or branch system are classified according to damage probability evaluation. The rehabilitation work starts from pipe links with higher damage probability, more functional importance and higher rehabilitation rate. The scheme can minimize the earthquake damage to pipelines if earthquake occurs during or after the rehabilitation work.

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EVENT SYSTEM SIMULATION OF POST-EARTHQUAKE FIRE

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ABSTRACT

Based on statistical analysis of numerous data, the probability of post-earthquake fire's occurrence is determined, and the probability density function of time for each stage of fire rescue is put forward. A method of discrete event system simulation (EEDC) is then developed for experiment in post-earthquake fire's occurrence and rescue process. An example shows the advantages and realism of the suggested method.

KEYWORD

Post-earthquake fire; system simulation; discrete event; fire density; rescue process.

INTRODUCTION

Fire is the most dangerous of all the post-earthquake disasters. Sometimes it can kill far more people than the earthquake itself. A post-earthquake fire is usually induced by strong earthquake. In 1906, the post-earthquake fire in San Francisco kept burning for more than three days, and destroyed 508 districts out of a total of 521. Later analysis revealed that no more than 20 percent of the damage was caused by earthquake itself; most of the rest resulted from the post-earthquake fire. In Japan, the Kanto earthquake in 1923 destroyed no more than 130,000 buildings; on the other hand, the post-earthquake fire destroyed more than 450,000 buildings. In China, things are similar: in 1975, although the authorities had made the prediction, and some emergency measures were taken, the Haicheng earthquake caused 60 post-earthquake fires. According to partial statistics, 5 fires broke out after the Tangshan earthquake in 1976. There were 36 fires in the affected city Tianjin at same time. Post-earthquake fire has been a highlight of earthquake engineering because of its serious results and high frequency. It also becomes the key factor in the earthquake disaster control process.

A post-earthquake fire's occurrence probability is first determined in this paper. A method of discrete event system simulation (EEDC) is then developed for experiment in post-earthquake fire's occurrence and rescue process, which makes it possible to evaluate the hazard of post-earthquake fire and the configuration of the city's fire brigade (FB) system. In order to set up the simulation model , a vast amount of data is investigated, based on which the probability density function for duration of each stage of rescue work is determined. Finally, an example is given to show the advantages and realism of the simulation methodology.

PROBABILITY OF POST-EARTHQUAKE FIRE'S OCCURRENCE (PPFO)

Influential Factors

According to Mohammadi *et al*, the following factors may have some influence on PPFO: population density, earthquake intensity, and characteristics of soil, among others. According to a study under general conditions (see Li and Song, 1994), we have drawn this conclusion: because of the mutual action of all factors, there is weak linkage between the occurrence of fire and the density of population. This is true for other factors except the total area of buildings. The decisive factor is the total size of buildings in the studied district. According to the statistical analysis on a number of post-earthquake fires, with general factors of the earthquake being taken into account, the following conclusion is reached: PPFO has almost the same characteristics as normal fires. The appended feature is that PPFO has a direct relation to the total size of damage, which is called the “effective area of fire source" (EAFS). PPFO divided by the EAFS is called "fire density". Limited by contemporary technique, all earthquake-resistance design is based on the concept of "standard intensity". The results of PPFO have to be expressed in the form of "standard intensity".

Historical Specimens

Some historical seismic disaster specimens are chosen for the purpose of determining PPFO. Cases are shown in Table 1.

Table 1 Historical earthquake and post-earthquake fire

Earthquake	Date	Magnitude	Number of fires	EAFS	Fire density
Xingtai	1966	6.8	383	1980	0.1934
Haicheng	1975	7.3	60	334	0.1799
Kanto	1923	7.9	163	764	0.2135
Japan 1	1948	7.3	29	144	0.2014
Japan 2	1964	7.5	9	43	0.2093

Mean=0.1955, Variance=0.0134

The following conclusion comes from the above table. No matter where, no matter when, no matter how serious the earthquake is, fire density keeps rather steady in the scope of 430,000 to 19.8 million square meters. Relative deviation is no more than 6.7%.

PPFO

When the total size of buildings in a city is taken into consideration, the fire density just discussed is changed into PPFO. Based on comprehensive statistical analysis of earthquake-destroyed specimens of civil buildings, taking Feng's outcome, Table 2 is given as follows.

Table 2 Intensity and EAFS

Earthquake Intensity	6	7	8	9	10
EAFS(%)	0.6	9.3	32.9	60.9	87.1

Using the method of linear regression, the relationship between seismic intensity and the size of EAFS can be expressed as:

$$\psi = 22.46 \times I - 141.52 \tag{1}$$

where ψ is the ratio of EAFS and I is the intensity of the earthquake. The conference factor of the female is 0.9865.

PPFO can be found using the equations:

$$\lambda = 0.0448 \times I - 0.2823 \qquad (I \geq 7) \tag{2}$$

where λ is the probability of post-earthquake fire's occurrence and I is the intensity of the earthquake.

When the intensity of the earthquake is less than 7, PPFO can be found using normal fire probability. Usually, the first five days are regarded as the period of post-earthquake fire occurrence.

DISCRETE EVENT SYSTEM SIMULATION

Main Features of System Simulation

A system is a group of active entities which have mutual relationship and interaction, classified by these characteristics. There are two kinds of systems: one is a continuous system whose mechanism can be expressed by differential equations; the other is a discrete system which can be expressed in the form of process graphics and logical relationships. Engineering systems are usually discrete systems. A discrete event system is the kind of system whose state changes only at a series of discrete time points and the state remains steady between any two time points. This kind of system does not usually obey any basic rules. The relationship of its parameters cannot be expressed in the form of analytic equation. Moreover, the main parameters of the system usually involve a stochastic process or complicated probability distribution. It is more difficult to model a discrete event system, sometimes even impossible. A post-earthquake disaster system is just this kind of question.

System simulation is a method for system evaluation by experiment with the model of a system which already exists or is being designed. In other words, this method uses an active mathematical model instead of a real system to do experiments for the system's analysing, designing and decision-making. It is a boundary science based on system science, computer science, probability theory and mathematical statistics. At the same time, it is a comprehensive testing science as well (see Cheng Hongde, 1982).

Characteristics of discrete event simulation system are as follows.

Modelling Character. It is a difficult task to set up the mathematical model for a complicated system whose parameters are usually a stochastic process. Even after the mathematical model is set up, it is tedious to find the solution, sometimes even impossible. On the other hand, simulation can model the system discussed, based on internal logical and mathematical relationships. The modelling process goes according to the behaviour of the system, and is much easier than analytic methods. This method is almost the only one that can perfectly depict a post-earthquake disaster system.

Experiment Capability For some systems, such as one which is designed or one which is non-existent (potential system), or, for example, the post-earthquake disaster system, the discrete event system simulation offers advantage of its "modelling" character. As far as development of contemporary techniques, the discrete event system simulation is the most efficient method for evaluation of a post-earthquake system.

Simulation Model

There are several kinds of methods for the modelling of DESS. Features of the model can be stated using process-oriented modelling.

All entities in a system can be divided into two parts: flowing entities and resources. The former refers to entities which go through the system, then go away, such as a customer in a shopping centre and a fire in the system discussed. The latter refers to entities which offer a service. The former are created during a particular length of time based on a determined probability model of "creation", then travel in the system and receive the system's services until they depart from the system. The latter offer some kind of service in a random length of time which is determined by a probability model of "service", and advance the system to the next stage. Usually, the rules in a system can be divided into two types: one is sequence-related rules, the other is direction-related rules. The former determine the way in which the resources choose the flowing entities. The latter determine the way in which the flowing entities go through the system. A system model runs in a particular length of time which is determined by end-status condition or the stabilisation-status condition. Then, through statistical survey, the characteristics of the main parameters of the system are determined. After that, an evaluation of the system's function can be done. All the above are used to alter the system for the purpose of improved function. This is the essence of discrete event system simulation. Characteristics of the system can be determined only after many independent cycles of simulations have been done. This is because the result of any given process of simulation is just a sample of the system's characteristics. Only after a sufficient number of independent simulations are performed can a series of responses by the system, such as the mean, variance, minimum and maximum value, be determined. However, no matter how many simulations there are, they remain samples of the given system parameters. Statistical inference is needed to find the useful characteristics of a system. In order to design or alter the system to get an optimal one, a set of rules should be made on which basis the parameters of the system can be revised in a specific way. Thus characteristics of system function can be seen and a system can be optimised through repitition of the above process.

SIMULATION MODEL FOR FIRE RESCUE PROCESS

According to the government statistical standard, for simplicity, the fire rescue process is divided into four stages as follows: (a) occurrence till fire bureau (FB) gets news; (b) FB gets the news till dispatch; (c) dispatch till arrival at site; (d) arrival at site till completion of rescue work. Several hundreds of examples are taken into consideration which are kept on record in 17 cities of Henan province 1992-1993.

Occurrence till FB Gets News: T1.

Total number of samples taken in this stage is 436. The shortest is 9 minutes, the longest is 40 minutes. Total length of time, 40 minute is divided into 8 segments, as shown in Fig. 1. Gamma function is adopted to fit the probability density function of T1.

FB Gets News till Dispatch: T2

Total number of samples taken in this stage is 645. The shortest is 0 minutes, the longest is 5 minutes. There are 619 samples whose time is under 1 minute; they comprise 96% of the total number. For simplicity of usage, the practical parameter is adopted as a constant.

Dispatch till Arrival at Site

For the purpose of setting up the rescue model, time spent on the road from FB to fire site is assumed to stay in proportion to the distance. Time can be calculated by a process of simulation. Taking the post-earthquake situation into consideration, the speed of the FB can be assumed to be 350m/min. Distance from FB to fire site is called "necessary distance" (ND). It can be determined by following rules: (a) Minimum distance from FB to each node should first be determined, provided these nodes (a, b, c...) are on the boundary of district i; that is: min(da), min(db), min(dc)...; (b) ND is the largest of the minimum distances. That is:

$$ND=\max\{\min(da), \min(db), \min(dc)...\} \tag{3}$$

Arrival till Completion of Rescue Work: T4

Some 560 samples are taken into consideration at this stage. The span of time is 26 minutes to 294 minutes. Total length of time, 300 minutes, is divided into 10 segments, as shown in Fig. 2. Gamma function is again adopted to fit the probability density function of T4.

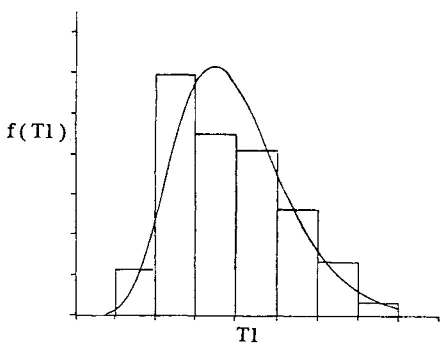

Fig. 1 Probabilistic distribution of alarm time T1

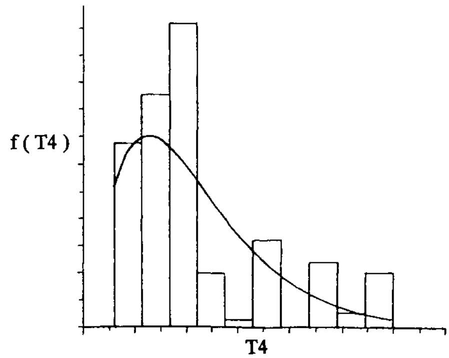

Fig. 2 Probabilistic distribution of rescue time T4

EXAMPLE OF SIMULATION ANALYSIS

In order to show the advantages and practical characteristics of the suggested method, a city is chosen from Henan province, whose simulation analysis results generally represent the characteristics of this method.

Main Parameters

Of the streets, 8 are longitudinal and 7 transverse. The city is divided by these streets into 33 districts. The total grid of streets has 47 nodes. There is only one FB in this city, located on the 22nd node. Total building area is 2,981,100 square meters. The areas of building and the necessary distance from the FB to each district are shown in Table 3.

Simulation Models

Post-earthquake fire time frame is 5 days. The Possion process is used for the occurrence model. Gamma density distribution is used for the model of alarm and rescue time.

Results

After extensive analysis, the main parameters of the city in an earthquake of intensity 8 and 9 are shown in Tables 4 and 5.

Table 3 Necessary Distance and Building Size

District	ND	Building Size	District	ND	Building Size
1	4970	43.75	2	3950	17.50
3	3400	8.75	4	4520	2.00
5	3550	1.50	6	3000	2.00
7	4090	2.00	8	3100	9.55
9	2570	12.20	10	2050	5.51
11	3890	5.20	12	3340	5.15
13	1830	8.97	14	2380	9.07
15	1830	14.01	16	1250	9.55
17	2570	1.50	18	2030	4.82
19	2610	8.95	20	2060	9.23
21	1520	9.64	22	950	11.85
23	3900	3.85	24	3500	10.74
25	3070	20.95	26	2520	7.81
27	2000	3.36	28	1400	7.50
29	4900	1.50	30	4050	2.20
31	3500	9.20	32	3000	3.50

Table 4 Parameters in earthquake of intensity 8

Number of post-earthquake fires	Duration of wait	Efficiency of FB	Rescue time
29	0.2145	6.8%	82.5 min

Table 5 Parameters in earthquake of intensity 9

Number of post-earthquake fires	Duration of wait	Efficiency of FB	Rescue time
35	0.2665	8.3%	83.2 min

Results show that some of the post-earthquake fires cannot be handled immediately after they occur. This is shown by the waiting time. The longer of wait is , the more danger to the system.

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Damage Estimation of Structures Incorporating Structural Identification

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ABSTRACT

The problem of the structural identification becomes important, particularly with relation to the rapid increase of the number of the damaged or deteriorated structures, such as highway bridges, buildings, and industrial facilities. This paper summarizes the recent studies related to those problems by the present authors. The system identfication methods are generally classified as the time domain and the frequency domain methods. As time doamin methods, the sequential algorithms such as the extended Kalman filter and the sequential prediction error method are studied. Several techniques for improving the convergences are incorporated. As frequency domain methods, a new frequency response function estimator is introduced. For damage estimation of existing structures, the modal perturbation and the sensitivity matrix methods are studied. From the example analysis, it has been found that the combined utilization of the measurement data for the static response and the dynamic (modal) properties are very effictive for the damage estimation.

KEYWORDS

Damage Assessment, Structural Identification, Extended Kalman Filtering, Sequential Prediction Error Method, Modal Perturbation Method, Sensitivity Matrix Method, Static Deflection

TIME DOMAIN IDENTIFICATION METHODS

Time domain identification is divided into two categories. One refers to the batch algorithm, and the other the sequential algorithm.The batch algorithm is generally known to be very time-consuming, especially when nonlinear optimization techniques should be used. In this paper, the extended Kalman filter and the sequential prediction error methods are discussed, which are sequential algorithms.

Extended Kalman Filetering Techniques

The Kalman filtering technique (Yun et al, 1982 and Imai et al ,1989) is the method of the state etsimation for a linear system based on the measurement squence of the system output which is usually contaminated with noises. It is accomplished by way of minimizing the mean square estimation error. Its extension to the nonlinear system is called as the extended Kalman filtering(EKF), which consists of linearization of the nonlinear state equation and state estimation using the Kalman filter at each time step. By introducing an augmented state vector composed of the state variables and the unknown parameters to be identified, the EKF technique can be used for the parameter estimation of the linear as well as the nonlinear systems. Since the EKF is basically for nonlinear systems, other methods such as the sequential prediction error methods may be more efficient for linear systems.

Sequential Prediction Error Method (Goodwin et al, 1984, Lee et al, 1991 and Yun et al, 1992)

In this parameter estimation process, an ARMAX model was used to process the observation data on the substructure, which are generally subjected to random noises such as modeling errors, unmeasurable disturbances and measurements errors. Eqaution of motion can be transformed into ARMA model by the method described in Reference , called as OCF(observable cannonical transform). For the response $y(t)$, which is displacement or velocity or acceleration values according to measurement type, and the input excitation $u(t)$. Then, including the random noise effect, the following ARMAX model can be derived as

$$y(k+1) = P_1\, y(k) + P_2\, y(k-1) + Q_0\, u(k+1) + Q_1\, u(k) + Q_2 u(k-1) + e(k+1) + H_1\, e(k) + H_2 e(k-1) \quad (1)$$

where $y(k)$ and $u(k)$ are the measurement vectors of response and input at time k, $e(k)$ is the estimation error vector, and P_i , Q_i and H_i are the ARMAX parameter matrices to be identified. The ARMAX model is known to be efficient for data with correlated observational noise owing to the inclusion of the estimation error process($e(k+1) + H_1\, e(k) + H_2 e(k-1)$) in the model. The ARMAX parameters (P_i , Q_i and H_i) can be estimated by using the sequential prediction error method, then the system matrices (J_{mm}, K_{mm} and L_{mm}) are evaluated thereafter using a similar scheme in the reference by (Lee, 1991).
For the parameter estimation of the a linear structure, the equation of motion may be transformed into an auto-regressive and moving average with auxiliary input(ARMAX) model (Goodwin et al, 1984 and Lee et al, 1991). It is in terms of the measured time histories of the response and the excitation. Then the prediction error model can be constructed as

$$y(k) = W(k)^T \theta + e(k,\theta) \quad (2)$$

where $y(k)$ is the response measurement vector at $t = k\Delta t$, θ is the vector of the unknown parameters, $W(k)$ is the regression matrix composed of the measurement data up to $t = k\Delta t$, and $e(k,\theta)$ is the error in the response prediction.

The unknown parameters can be determined by minimizing the summation of the prediction error as

$$V_{k+1}(\theta) = V_k(\theta) + \frac{1}{2} e(k+1,\theta)^T e(k+1,\theta) \quad (3)$$

which results in the following sequential algorithm for θ

$$\hat{\theta}(k+1) = \hat{\theta}(k) + F(k+1)\Psi(k)\{e(k+1,\hat{\theta}(k))\} \quad (4)$$

$$F(k+1)^{-1} = F(k)^{-1} + \Psi(k)\Psi(k)^T \quad with \quad F(0) = \beta \cdot I \;\; (\beta > 0) \tag{5}$$

$$\Psi(k) = -\left.\frac{de(k+1,\theta)}{d\theta}\right|_{\theta=\hat{\theta}(k)} \tag{6}$$

where $\hat{\theta}(k)$ denotes the estimates of θ at $t = k\Delta t$; $\Psi(k)$ is the matrix of the negative gradient vectors of the prediction error $e(k)$; $F(k)$ is the adaptation gain matrix which is an inverse of the second derivative matrix of the criterion function $V_k(\theta)$.

Techniques for Improving Convergences (Lee et al, 1991 and Yun et al, 1992)

Many techniques were developed to improve the convergence and the robustness of the sequential estimation methods. Among them the exponential data weighting and square root estimation of adaptation gain matrix are found to be very effective.

Exponential Data weighting It is generally true that the prediction error associated with the recent estimate is more informative for the next estimation than the past ones. The concept can be incorporated into the criteria function by introducing a weighting $a(k)$ as

$$V_{k+1}(\theta) = a(k)\, V_k(\theta) + \frac{1}{2} e(k+1,\hat{\theta})^T e(k+1,\hat{\theta}) \tag{7}$$

where $a(k) < 1$. Using the above criteria function, an enhanced algorithm for the adaptation gain matrix can be obtained as

$$F(k+1)^{-1} = a(k)\, F(k)^{-1} + \Psi(k)\, \Psi(k)^T \tag{8}$$

Global Data weighting The data weighting technique can be also applied globally,then a modified algorithm for $F(k)$ is obtained as

$$F(N_0+1)^{-1} = wF(N_0)^{-1} + \Psi(N_0)\Psi(N_0)^T \tag{9}$$

where N_0 is the number of the measured time data points in a record, and w is the weighting parameter $(w < 1)$.

Square Root Estimation of Adaptation Gain Matrix With the sequential algorithm, inversions of the matrices with the dimension of the output are required as in Eq. (5). Series of the matrix inversions may cause the adaptation gain matrix to lose the positive definiteness, especially when the quality of the measurement data and/or the concurrent estimates for the unknown parameters are poor. If the positive definiteness of the matrix is lost, the sequential prediction error algorithm could diverge to the direction of increasing the prediction error. In this study, the square root estimation algorithm is employed to insure the positive definiteness of the adaptation gain matrix. In this technique, the matrix inversion is carried out by using the triangular factorization, in which the diagonal elements are estimated by taking square roots of the positive qualities. Details of the technique can be found in *References* 7,9 and10 .

Example Analysis

In order to investigate the effectiveness of the techniques presented in this study, example studies are carried out for an idealized case with 2 degrees of freedom. Artificial time histories of two excitations and two acceleration responses are generated based on the assumed exact ARMAX parameters. The parameters are estimated by the sequential algorithm incorporating the techniques described in the previous sections, and the results are compared with the assumed exact values. The initial guesses for the parameters and the adaptation gain matrix are taken as ; $\hat{\theta}(0) = 0$ and $F(0) = 100I$. *Figures* 1 and 2 show the processes of the sequential parameter estimations by incorporting the techniques above. From the estimated parameters, it can be observed that, by using the exponential data weighting technique, the better estimates can be obtained. It is noted that divergence problem has been experienced as shown in *Figure 2*, if the square root estimation technique is not utilized. The experimental study is also performed for a 3 story shear building model as shown in *Figure 3*. The estimated modal parameters from different sets of measurement data are found to be very consistent as shown in *Table 1*.

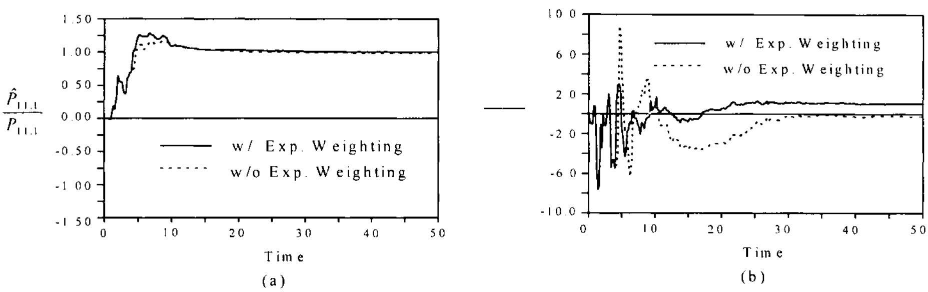


Fig. 1 Effect of Exponential Data Weighting on ARMAX parameters for a 2-DOF Case

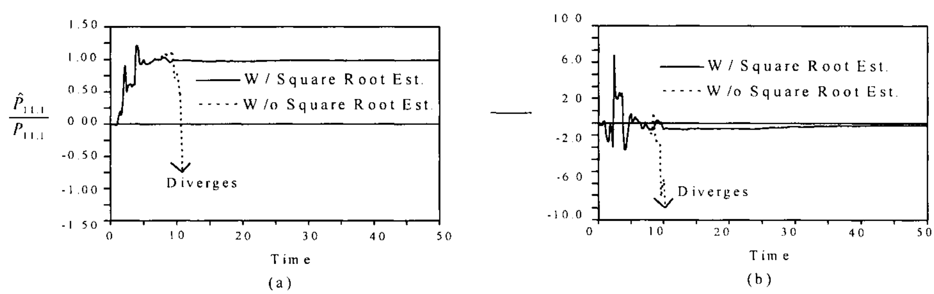


Fig. 2 Effect of Square Root Estimation on ARMAX parameters for a 2-DOF Case

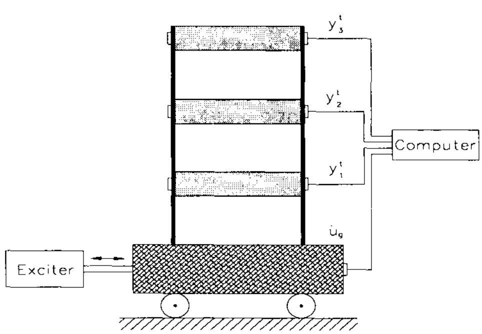


Fig. 3 Experimental Set-up of a Building Model

Table 1 Estimated Modal Parameters of Building Model

Modal Properties		Experiment 1		Experiment 2		Experiment 3		Analysis Using Shear Building Model	
f_1	ξ_1	3.62	0.0059	3.59	0.0051	3.56	0.0021	3.58	—
f_2	ξ_2	9.98	0.0015	9.99	0.0010	10.18	0.0021	11.32	—
f_3	ξ_3	14.78	0.0075	14.79	0.0032	15.06	0.0066	16.49	—
Ψ_1	3F	1.	(0)	1.	(0)	1.	(0)	1.	(0)
	2F	0.805	(2)	0.807	(2)	0.813	(1)	0.802	(0)
	1F	0.447	(5)	0.448	(4)	0.445	(3)	0.445	(0)
Ψ_2	3F	1.	(0)	1.	(0)	1.	(0)	1.	(0)
	2F	0.426	(176)	0.395	(175)	0.470	(179)	0.554	(180)
	1F	1.154	(177)	1.145	(175)	1.238	(176)	1.250	(180)
Ψ_3	3F	1.	(0)	1.	(0)	1.	(0)	1.	(0)
	2F	2.284	(177)	2.531	(177)	2.381	(168)	2.242	(180)
	1F	1.557	(2)	1.598	(3)	1.096	(13)	1.801	(0)
Input Types		Implusive		Implusive		Long Duration			

Note : 1. The unit of f_i is Hz
2. Values in parentheses denote the phase angles of the complex modes in degrees

FREQUENCY DOMAIN IDENTIFICATION METHODS

Frequency Response Function Estimators

Using the measurement data, the frequency response functions are usually calculated as

$$H_1(f) = S_{xy}(f)/S_{xx}(f) = H(f)/(1 + S_{mm}(f)/S_{uu}(f)) \tag{10}$$
$$H_2(f) = S_{yy}(f)/S_{yx}(f) = H(f)\,(1 + S_{nn}(f)/S_{vv}(f)) \tag{11}$$

where $H(f)$ is the true frequency response function; u and v denote the true input and output; x and y denote the measured excitation and response; m and n denote the measurement noise of input and output; $S_{uu}(f)$, $S_{vv}(f)$, $S_{xx}(f)$, $S_{yy}(f)$, $S_{mm}(f)$ and $S_{nn}(f)$ are the power spectral density(PSD) functions; $S_{xy}(f)$ and $S_{yx}(f)$ are the cross-spectral density functions.

During experiments on real structural systems, it is frequently observed that the input PSD function, $S_{uu}(f)$, drops drastically near resonance, particularly for the case with random excitations(Mitchell, 1982). The low input power spectrum in the resonance region causes $H_1(f)$ much less than the true FRF. On the other hand, $H_2(f)$ gives an excellent estimate of $H(f)$ in the resonance region, since the response PSD function, $S_{vv}(f)$, at resonance is considerably larger than that of output noise, $S_{nn}(f)$. However, $H_2(f)$ is a much poorer estimator in the antiresonance region than $H_1(f)$, since the true output power spectrum at antiresonance drops drastically to the level of the output noise spectrum.

Noise is not the only source of error in the FRF estimation. The resolution bias error causes underestimation of spectral peaks and overestimation of spectral troughs. Cawley(1984) investigated the effects of the resolution bias error on the FRF estimators, $H_1(f)$ and $H_2(f)$. Schmidt (1985) analyzed the resolution bias errors in spectral density, frequency response and coherence function measurements. It has been shown that, due to the resolution bias error, $H_1(f)$ produces a result of underestimation of the FRF at resonances, but it gives good results at antiresonances. On the other hand, $H_2(f)$ yields more accurate estimates at resonances, but gives poor estimates at antiresonances.

In a recent paper by Fabunmi and Tasker (1988), a new estimator for the FRF was proposed, which was based on the joint minimization of the measurement input and output noises as

$$H_3(f) = \frac{H_2(f)[\gamma^2_{xy}(f)F(f)/|H_1(f)|^2+1]}{F(f)/|H_1(f)|^2+1} \tag{12}$$

where $\gamma_{xy}(f)$ is coherence function and $F(f)$ is a normalizing function. $H_3(f)$ of Eq. (12) was intended to approach $H_1(f)$ at antiresonances and $H_2(f)$ at resonances. For that purpose, $F(f)/|H_1(f)|^2$ should be significantly smaller than unity at resonance, whereas $\gamma^2_{xy}(f)F(f)/|H_1(f)|^2$ should be larger than unity at antiresonances. For displacement and acceleration FRF's, the normalizing function was obtained as

$$F(f) = -((S_{xy}(f)-S_{yx}(f))/2S_{yy}(f)^2 \tag{13}$$

and for the velocity FRF, it was obtained as

$$F(f) = ((S_{xy}(f)+S_{yx}(f))/2S_{yy}(f)^2 \tag{14}$$

Since the normalizing function, $F(f)$, depends on the power and the cross spectra of the measured records, it may not be possible that the intended characteristics for the normalizing function are achieved; namely, depending on the measurement data, $F(f)/|H_1(f)|^2$ may not be significantly smaller than unity at resonance, and $\gamma^2_{xy}(f)F(f)/|H_1(f)|^2$ may not be much larger than unity at antiresonances. Therefore, $H_3(f)$ may give undesirable results over some frequency ranges, as shown in the first example analysis.

Frequency response functions (FRF) are the most fundamental data for the frequency domain identifications of structural systems. Several techniques have emerged for the estimation of FRF's in such a way to minimize the effect of measurement noise. By converting many samples of measured time history records into the frequency domain, the conventional FRF's, which usually designated as $H_1(f)$ and $H_2(f)$(Bendat et al, 1971), are obtained from the relations between the averaged power spectra and the cross spectra of the excitation and response measurement records. In a recent paper, Fabunmi and Tasker (Fabunmi et al, 1988) introduced a new estimator $H_3(f)$, which is based on the minimization of the measurement noises. More recently, an improved FRF estimator $H_4(f)$ was proposed by the current author (Hong at al, 1993) as

$$H_4(f) = (1-W(f))\,H_1(f) + W(f)\,H_2(f) \tag{15}$$

where $W(f)$ is the weighting function. The new FRF estimator in Eq. (15) is designed to have the characteristics that it approaches $H_1(f)$ at antiresonances and $H_2(f)$ at resonances. Thus, the weighting function, $W(f)$, is to have unity at resonances and zero or near zero at antiresonances. An exponential function is selected for the weighting function, which is given as

$$W(f) = \exp(-(\frac{f/f_0-1}{\alpha})^2) \tag{16}$$

where f_0 is the natural frequency, and α is the parameter of the exponential weighting function. The weighting function, $W(f)$, has unity correctly at the resonance and approaches zero as the frequency being far apart from the resonance frequency. The shape of the weighting function depends on the parameter α, which may be determined by way of minimizing the resolution bias error.

Example Analysis

Numerical investigation is performed for a 3-span bridge model as shown in *Figure* 4. The results in Figure 5 indicate that $H_4(f)$ is more accurate than the other FRF estimators.

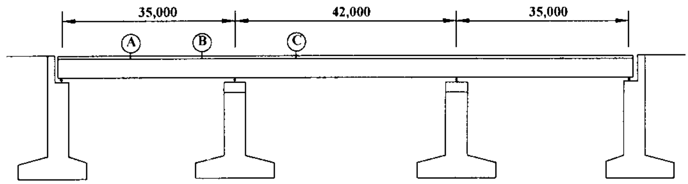
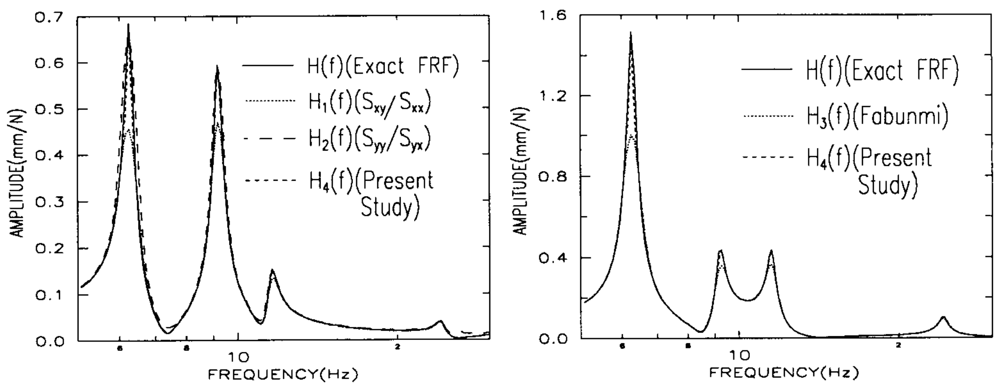


Fig 4. Sketch of a continuous 3-span bridge model (in mm)



(a) FRF's at Point B (b) FRF's at Point C

Fig 5 . Frequency response functions of a 3-span bridge model at Points B and C for an exciation at Point A (using Hanning Window)

DAMAGE ESTIMATION OF STRUCTURES

For the safety evaulation of the existing structures, it is very important to estimate the degree of damage or deterioration. Advanced system identification techniques were extensively applied to the aero-space structures. However, for the civil engineering structures, the safety evaluation has relied upon conventional methods, such as visual inspection, non-destructive material tests on critical locations, monitoring of static strain and deflection. The information on the changes of dynamic characteristics of the structures may be incorporated for the improvement of the safety evaluation. In this paper, the inverse modal perturbation and the sensitivity methods are discussed.

Inverse Modal Perturbation Method (Hong, K. S. And Yun, C. B. ,1993)

The properties of a structure which has suffered damage can be defined by introducing perturbations to the original values as

$$K' = K + \Delta K, \quad M' = M + \Delta M$$
$$\Lambda' = \Lambda + \Delta\Lambda, \quad \Phi' = \Phi + \Delta\Phi \tag{17}$$

where K, M, Λ and Φ are the matrices of the stiffness, mass, eigenvalues (or squares of natural frequencies), and eigenvectors (or mode shapes) of the original structure, respectively, and K', M', Λ', and Φ' are those of the damaged structure. Δ denotes the corresponding perturbation due to damage. The second order perturbation equation for the eigenvalue problem can be derived as

$$\Phi^T \Delta K \Phi' = \mathcal{M}(C^T \Lambda' - \Lambda' C^T + \Delta\Lambda) \tag{18}$$

where $\mathcal{M} = \Phi^T M \Phi$ is the generalized mass matrix of the original structure, the term C_{ij} relates the participation of the j-th mode of the original structure to the change in the i-th mode i.e. $\Delta\phi_i = \sum_{j=1}^{n} C_{ij}\,\phi_j$. If only small perturbations which are of order Δ are considered, the first order perturbation equation can be obtained from equation (18) as

$$\Phi^T \Delta K \Phi = \mathcal{M}(C^T \Lambda - \Lambda C^T + \Delta\Lambda) \tag{19}$$

To deal with the damage estimation problem effectively, a practical interpretation must be given to the structural changes due to the structural damage, ΔK. The stiffness changes may be decomposed into l-element stiffness changes, where l is the total number of the element stiffness components which are suspected to get damage. Then, ΔK becomes

$$\Delta K = \sum_{e=1}^{l} K_e\,\alpha_e \tag{20}$$

where α_e represents the fractional change(damage coefficient) in the e-th element stiffness component : $-1 \le \alpha_e \le 0$.

This perturbation equation can be solved by way of minimizing the estimation error with respect to the damage coefficient α_e's. The objective function may be taken as the sum of squared errors among the modal perturbation matrix equations, which are related to the changes of the natural frequencies in the perturbation equation

$$\min.\ J = \sum_{k=1}^{n_k}\left\{\sum_{e=1}^{l}\phi_k^T K_e\,\phi'_k\,\alpha_e - \mathcal{M}_k(1 + C_{kk})\Delta\lambda_k\right\}^2 \tag{21}$$

where n_k is the number of the measured natural frequencies. On the other hand, the constraint equations for the above quadratic optimization problems may consist of the equations related to the mode shape changes in the perturbation equation.

Sensitivity Method

This method uses the sensitivity matrices which relate the changes of structural properties of interest to the changes of structural parameters related to damage. For the case of natural frequencies, mode shapes and static deflection, the relations can be written as

$$\{\delta\omega^2\} = [\bar{S}]\{\delta p\} \tag{22}$$
$$\{\delta\Phi_k\} \cong [\tilde{S}]_k\,\{\delta p\} \tag{23}$$
$$\{\delta u\} \cong [\hat{S}]\{\delta p\} \tag{24}$$

where $\{\delta p\}$ is the small variation of the parameters, $[\bar{S}]$, $[\tilde{S}]$ and $[\hat{S}]$ are the sensitivity matrices as follow

$$\bar{S}_{ij} = \frac{\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{li}\phi_{mi}\frac{\partial K^e_{lm}}{\partial p_j}\right) - \omega_i^2\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{li}\phi_{mi}\frac{\partial M^e_{lm}}{\partial p_j}\right)}{\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{li}M^e_{lm}\phi_{mi}\right)} \tag{25}$$

$$\begin{aligned}\tilde{S}_{nkj} = &\sum_{q=1}^{N_{sps}}\left(\frac{\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{lq}\phi_{mk}\phi_{nq}\frac{\partial K^e_{lm}}{\partial p_j}\right)}{(\omega_k^2-\omega_q^2)\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{lq}M^e_{lm}\phi_{mq}\right)}(1-\delta_{qk})\right)\\ &-\sum_{q=1}^{N_{sps}}\left(\frac{\omega_k^2\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{lq}\phi_{mk}\phi_{nq}\frac{\partial M^e_{lm}}{\partial p_j}\right)(1-\delta_{qk})}{(\omega_k^2-\omega_q^2)\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{lq}M^e_{lm}\phi_{mq}\right)} + \frac{\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{lk}\phi_{mk}\phi_{nq}\frac{\partial M^e_{lm}}{\partial p_j}\right)(\delta_{qk})}{2\sum_{l=1}^{N_{dof}}\sum_{m=1}^{N_{dof}}\sum_{e=1}^{N_e}\left(\phi_{lk}M^e_{lm}\phi_{mk}\right)}\right)\end{aligned} \tag{26}$$

$$[\hat{S}] = -[K]^{-1}\left[\frac{\partial K}{\partial p}\right]\{u\} \tag{27}$$

Using a similar procedure to those in the perturbation method, the objective function and constraint conditions can be constructed. Then the parameters can be determined by way of minimizing the objective function.

Example Analysis

Numerical investigations were carried out for a truss shown in *Figure 6*. The damage coefficients estimated by the first and second order perturbation equations indicated that the results by the first order perturbation do not improve significantly, though the number of the natural frequencies included in the analysis increases. On the other hand, the results by the second order perturbation get improved significantly even in the case using only a few natural frequency measurements. *Figure 7* shows the estimated element damages of a case with 5 damaged elements. The results were obtained using the sensitivity methods iteratively. In general, the estimates are found to improve significantly, if the static deflection measurements and the modal property changes are incorporated.

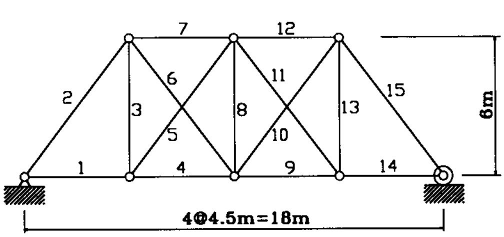


Figure 6 Truss model

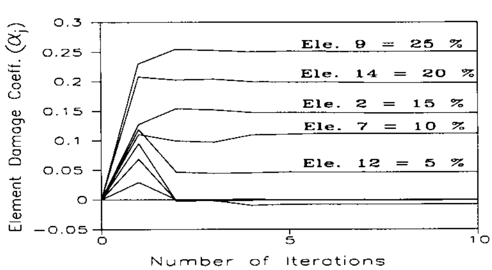


Figure 7 Estimates of Damage Parameters

CONCLUDING REMARKS

In this paper, time and frequency domain identification methods of structural systems and their application to the damage estimation of the existing structures are discussed. Several techniques for improving the estimation algorithm are also investigated. From the numerical example studies, it has been found that a combined utilization of the static and the dynamic response measurement data is very effective for the estimation of structural damage.

Table 2. Estimated Damages by Different Methods

Methods		Perturbation		Sensitivity		
		1st Order	2nd Order	Dynamic	Dynamic + Static	
Element	**Exact**	**(5)***	**(5)***	**(5)***	**(2)***	**(5)***
1	0.00	0.00	0.00	0.01	0.00	0.00
2	**0.20**	**0.00**	**0.20**	**0.20**	**0.19**	**0.20**
3	0.00	0.00	0.00	0.01	0.00	0.00
4	0.00	0.00	0.05	-0.03	0.00	0.00
5	**0.10**	**0.00**	**0.10**	**0.13**	**0.10**	**0.10**
6	0.00	0.00	0.00	-0.01	-0.02	0.02
7	**0.30**	**0.00**	**0.30**	**0.26**	**0.31**	**0.30**
8	**0.10**	**0.10**	**0.10**	**0.10**	**0.11**	**0.09**
9	0.00	1.00	0.07	0.02	0.00	0.00
10	0.00	0.06	0.00	-0.01	-0.02	-0.01
11	**0.20**	**0.17**	**0.20**	**0.21**	**0.20**	**0.21**
12	0.00	0.00	0.00	0.08	0.06	0.00
13	0.00	0.00	0.00	-0.01	0.00	0.00
14	0.00	0.00	0.00	0.01	0.05	-0.02
15	0.00	0.01	0.00	-0.04	-0.05	0.00
Error(%)	**0.00**	**602.42**	**3.89**	**6.58**	**5.10**	**0.58**

Note : (*) denote the number of mode shape used for damage estimation

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RISK MANAGEMENT FOR URBAN PLANNING AGAINST STRONG EARTHQUAKES

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ABSTRACT

Typical types of damage to buildings and the infrastructures subjected to the Hyogoken Nanbu also called the Hanshin-Awaji Earthquake are introduced and statistics on the extent of damage and characteristics of the structures obtained from Ashiya, a city next to Kobe, were analyzed. The degree of damage was shown to be closely related to the period when buildings were constructed, as evidenced by changed building codes and the material deterioration of wooden houses. The anti-seismic strength of 263 existing reinforced concrete structures were compared with the extent of damage level in terms of when they were constructed. A method of damage estimation in an urban area Kyoto City, subjected to a scenario earthquake is presented that is based on the strength distribution and soil conditions in that area. A method of integrated risk management for future urban planning against strong earthquake damage based on the experience of the Hyogoken nanbu earthquake also is discussed.

KEYWORDS

Hyogoken Nanbu Earthquake, Great Hanshin Earthquake, Wooden Houses, RC Building, Damage Distribution, Urban Disaster Mitigation, Risk Management

INTRODUCTION

The 1995 Hyogoken nanbu earthquake wreaked havoc throughout the Kobe area, destroying or damaging a large percentage of the infrastructure. 5,502 persons were confirmed dead, 38,495 injured, and more than 100,000 buildings and homes destroyed or damaged beyond repair. The Hyogo Prefecture Police reported that nearly 90% of the people killed in the earthquake were crushed or suffocated due to the collapse of homes, most of which were built of wood. This was a consequence of the earthquake occurring at 5:46 am. The estimate of the direct cost of damage by the Hyogo Prefecture government is at least 10 trillion Japanese yen. Damage to houses and buildings is reported to account for 60% of the total losses.

Fig.1 Areas of severe building damage

Typical types of damage to buildings and the infrastructure, as well as the damage distribution for reinforced concrete structures and wooden houses subjected to the earthquake are discussed and reasons suggested for the damage done. Methods for estimating the average anti-seismic strength of buildings and damage in typical urban areas are proposed that are based on the local distribution of various types of buildings and different periods of construction. Integrated risk management for future urban planning against strong earthquake damage also is discussed.

CHARACTERISTICS OF THE EARTHQUAKE

The 1995 Hyogoken nanbu earthquake which measured 7.2 on the Richter scale took place off the City of Kobe on the 17th of January. The main shock was located under the Akashi Straight, several kilometers north of Awaji Island at a focal depth estimated to be 17 km. Photo 1 shows the Nojima fault in the northern part of Awaji Island. Movement continued 50 km northeast to the Rokko fault on the northern border of Kobe City. Epicenters of the after shocks were located along the mountains skirting Kobe. The areas that suffered a local intensity of VII on the Japan Meteorological Agency (JMA) scale, which means that more than 30 % of the homes completely collapsed, are located along the coast line of the Inland Sea in a strip about 1 km wide, as shown in Fig.1 (JMA1995). In November 1995, the JMA revised the definition of local intensity as the 10 stages 0, 1, 2, 3, 4, 5(weak, strong), 6(weak, strong) and 7 automatically determined by the intensity meter. As shown later, as the strength of residential houses has increased under improved construction codes, it is no longer suitable as a measure of local intensity. In many places in the affected area, maximum accelerations exceeded 500cm/s^2, in some places reaching 817 cm/s^2 (JMA). The maximum velocity was estimated to be 150 cm/s in central Kobe. High peak vertical accelerations also have been reported.

Figure 2 shows the response spectra of earthquakes recorded at the Kobe branch of JMA together with those of other famous earthquakes including the Northridge California event (Building Research Inst. 1995) .

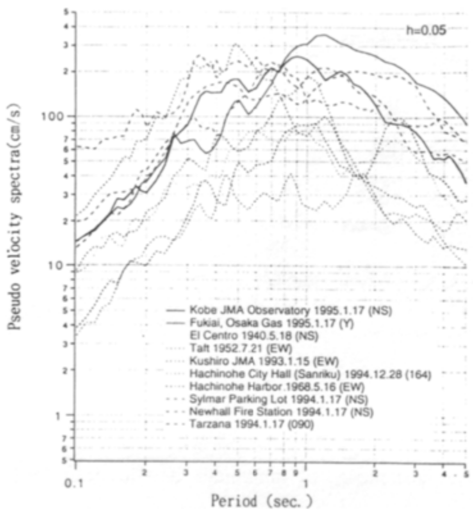


Fig.2 Response spectra of strong earthquakes

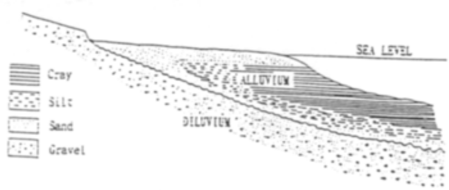


Fig.3 Typical NS cross section of Kobe

The predominant periods of the Hyogoken nanbu earthquake are 0.35-1.0 sec. acceleration and 0.7-1.5 sec. velocity.

GROUND MOTION OF STRICKEN AREA

The northern part of Kobe City is located in the mountainous Rokko area. The soil condition from north to south in Kobe changes shallow alluvial soil to deep one. A typical cross section indicating the change from dilluvial to alluvial soil in Kobe (Kobe city, 1980) is shown in Fig.3. The relation between damaged areas and the dynamic characteristics of the soil ground is analyzed by using the finite element method and ground motion recorded at the JMA (Kawase et al., 1995). They estimated that the maximum velocity at the surface layer near the Japan Railway's San'nomiya station, 1km from the fault zone was nearly 150 cm/s. In this area most of the modern buildings suffered severe damage as shown later. On Port Island, an artificial island, the ground acceleration measured at GL-88m, was 500 cm/s^2 for the NS component, but surface motion was less than this value due to liquefaction.

DAMAGE TO CIVIL STRUCTURES AND COASTAL STRUCTURES

The national and private railways between Osaka and Kobe were completely disrupted due to severe damage. Photo 2 shows the collapse of the Sanyo Shinkansen railroad tracks at Nishinomiya, a satellite city betwwen Osaka and Kobe. The reason for this may have been shear failure of the RC columns. The elevated Hanshin Highway, supported by reinforced concrete columns also was overturned for about 500m (Photo 3). The reason for this damage was marked deformation of the columns due to the severe power of the earthquake and the shear failure of the columns (Photo 4).

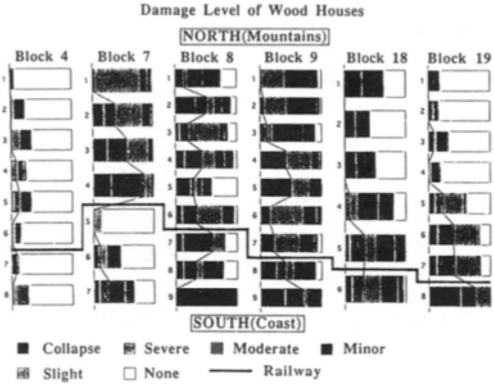


Fig.4 Collapse ratio distribution of wooden houses in downtown Kobe

In some places close to the mountains, buildings were severely damaged by landslides, as shown in Photo 5. Coastal facilities in Japan have been frequently damaged by moderate or severe earthquakes. Bay area of Kobe is an area of reclamation on which modern residential and business structures as well as coastal facilities have been built. In the **reclaimed** area liquefaction occurred, as shown in Photo 6. Lifeline systems, **in particular** the water supply and gas systems were damaged severely in many places. Three months were required to restore the gas system, a major reason being delay of sweeping the collapsed buildings and the debris of buildings.

DAMAGE TO BUILDING STRUCTURES

Damage Distribution of Building Structures

Many types of residential buildings are found in modern Japanese cities: traditional wooden houses with heavy ceramic tile roofing that provides excellent resistance to strong gusts of wind, new types of prefabricated houses with light panel roofing, two-story wooden apartment houses built 3 decades ago, modern high-rise apartments built of reinforced concrete and so on. Anti-seismic capacity of residential buildings depends on the weight of roofing, the degree of deterioration of wood, eccentricity of rigidity due to narrow front of a small house.

Figure 4 shows the damage distribution for several blocks of wooden houses in Chuo-ku (downtown Kobe), where many buildings and houses are clustered. In this figure, the solid line represents the Japan National Railway. The attenuation of damage, from north to south, that is, from the mountains to the coast, is clear. This reduction correlates well with changes in soil conditions, from the progressively more shallow and dense soils inland to the soft soils along the coast; however, the damage distribution also correlates equally well with the age of the structures, as the city developed first along the railway, then toward the coast, finally spreading out to the mountains. Damage distribution for different types of construction and different periods of construction in Ashiya City, abutting the east ward of Kobe, is shown in Table 1. Buildings constructed many years ago are severely damaged, one reason being the early Japanese building code. Another is the material deterioration of wooden houses.

Wooden Houses

Totally, 86,732 houses in Kobe City and more than 200,000 nearby areas collapsed completely or were severely damaged. Extensive surveys showed that, in general, older constructions suffered the most severe damage. A number of construction features combined to make many of these older houses (Photo 7) prone to collapse. Heavy ceramic tile roofs are widely used in Japan; but as seismic forces are simply inertial forces induced by ground shaking, such heavy construction incurs greater seismically-induced forces. The resistance of wooden houses deteriorates with the age of construction and damage due to termites (Photo 8), as shown in Fig.5 and Table 1(a). Houses with light roof systems, made of asphalt tiles or light metal sheets, generally performed better during this earthquake (Photo 9 and Table 1(b)).

Table 1 Damage to buildings by the period of construction in Ashiya City

(a) Damage ratio for traditional wooden houses with heavy roofs

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	2155	72.6	20.7	4.9	1.6	0.1
1946~1961	1848	60.8	26.5	10.2	2.2	0.3
1962~1971	1698	46.8	34.9	15.6	2.7	0
1972~1981	1665	31.2	38.1	26.0	4.7	0.1
1982~1991	699	9.4	45.1	38.6	6.9	0
1992~1995	47	6.4	21.3	57.5	12.8	2.1

(b) Damage ratio for traditional wooden houses with light roofs

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	68	52.9	20.6	22.1	4.4	0
1946~1961	352	42.1	27.8	19.3	9.4	1.4
1962~1971	618	25.9	29.1	33.3	10.5	1.1
1972~1981	521	13.2	25.0	46.5	14.8	0.6
1982~1991	623	4.3	25.2	49.4	20.7	0.3
1992~1995	200	5.0	9.0	58.5	27.0	0.5

(c) Damage ratio for reinforced concrete structures

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	27	3.7	22.2	44.4	29.6	0
1946~1961	138	4.4	21.0	53.6	16.7	4.4
1962~1971	522	9.4	16.9	45.8	24.9	3.1
1972~1981	809	1.9	20.9	52.5	24.0	0.6
1982~1991	1107	0.5	18.4	60.0	20.4	0.8
1992~1995	120	0	7.5	49.2	42.6	0.8

(d) Damage ratio for steel structures

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	0	0	0	0	0	0
1946~1961	5	20.0	40.0	20.0	20.0	0
1962~1971	105	41.9	24.8	22.9	9.5	1.0
1972~1981	156	17.3	23.1	47.4	10.9	1.3
1982~1991	225	6.7	24.4	49.3	19.1	0.4
1992~1995	59	3.4	18.6	57.6	20.3	0

(e) Damage ratio for prefabricated wooden houses with heavy roofs

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	0	0	0	0	0	0
1946~1961	0	0	0	0	0	0
1962~1971	0	0	0	0	0	0
1972~1981	18	0	16.6	44.4	38.9	0
1982~1991	39	2.6	41.3	43.6	12.8	0
1992~1995	4	0	0	50.0	50.0	0

(f) Damage ratio for prefabricated wooden houses with light roofs

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	0	0	0	0	0	0
1946~1961	0	0	0	0	0	0
1962~1971	4	0	50.0	50.0	0	0
1972~1981	119	5.0	5.9	51.3	36.1	1.7
1982~1991	574	0.7	38.9	38.7	21.8	0
1992~1995	243	3.2	4.8	49.1	41.9	0.8

(g) Damage ratio for light gauge steel structures

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	0	0	0	0	0	0
1946~1961	4	25.0	0	50.0	25.0	0
1962~1971	75	42.7	20.0	24.0	9.3	4.0
1972~1981	19	10.5	26.3	47.4	10.5	5.3
1982~1991	5	0	0	40.0	60.0	0
1992~1995	9	11.1	11.1	22.2	55.6	0

(h) Damage ratio for prefabricated steel houses

Period of Construction	No. of houses	Damage (%)				
		collapse	half	partial	standing	other
~1945	0	0	0	0	0	0
1946~1961	1	0	100.0	0	0	0
1962~1971	43	7.0	18.6	34.8	37.2	2.3
1972~1981	222	4.5	14.0	52.7	28.4	0.5
1982~1991	462	1.3	35.3	42.6	20.4	0.4
1992~1995	105	5.7	9.5	51.4	33.3	0

Photo 1 Nojima Fault in northern Awaji Island.

Photo 2 Damage to the Sanyo Shinkansen Railway near Nishinomiya

Photo 3 Cantilevered columns of the Hanshin Highway at Ashiya City

Photo 4 Shear failure of a reinforced concrete column of the Hanshin Highway

Photo 5 Damage done by a land slide at Yurinocho, Nishinomiya

Photo 6 Damage caused by liquefaction on Port Island

Photo 7 A collapsed old house

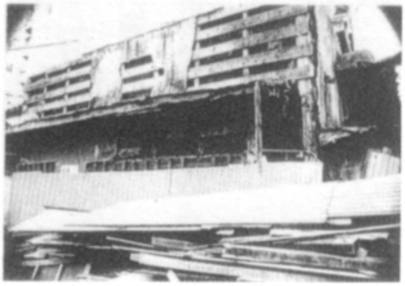


Photo 8 Deterioration of wooden building

Photo 9 Houses with light roofs that survived in a severely damaged area

Photo 10 Coumn pulled out from its connection to the Foundation

Photo 11 House with a narrow front damaged by torsional vibration

Photo 12 More than 7,000 houses were destroyed by fire

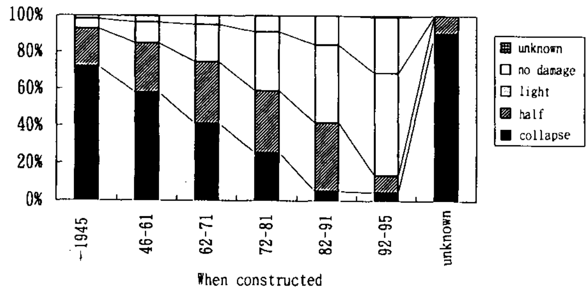


Fig.5 Damage contrast by period of construction of wooden houses in Ashiya City

One of the major causes of residential house collapse is the absence of structural integrity. It has been the Japanese practice to construct wooden houses by the use of interlocking wood parts (**tendons** and mortises); that is, without nails or other positive non-wood connectors. At best, only few nails were found in the members of collapsed structures. During severe horizontal as well as vertical shaking, columns frequently were pulled out of their socket supports in the beams (Photo 10). Another major cause of collapse is the way the narrow fronts of houses were designed. Many houses collapsed (Photo 11) due to torsional vibration. **mainly because they were built without** an adequate set of resisting walls. Fire protection is also an important factor for wooden houses. More than 7,000 houses were burned down in Kobe (Photo 12).

It is noteworthy that in many instances new houses, which were undamaged or had sustained at most minor damage, were found adjacent to totally collapsed dwellings. Some of these new houses were pre-fabricated structures (Table 1(e)-(h)) constructed with light-gage steel beams and columns, bar braces, and light asphalt sheet roofing. Furthermore, although traditional wooden houses are still being constructed, they must comply with specifications that require extensive nailing, the use of steel connections, and prescribe that walls must constitute a percentage of the total floor area. The satisfactory seismic performance of such houses built with these construction techniques is encouraging.

Reinforced Concrete Structures

Loss of a story is typically the consequence of excessive damage to that story, as opposed to the uniform sharing of damage and ductility by all of a structure's stories which is implicit in modern building design codes. During the Hyogoken nanbu earthquake, single story failure was frequent for the first story of RC buildings. Experience gained from past earthquakes has shown that contemporary architectural designs that emphasize a maximum amount of clear open space in the first story, combined with the higher story heights typical of these buildings, are responsible for greater swaying and thereby the accumulation of damage to the first story.

Loss of a higher story is attributed to a number of causes (Photo 13). Setback portions and adjacent lower building portions frequently provided additional strength and rigidity to the lower stories, so that damage was concentrated in the first story above these lower stories. At other times, the story lost was the first one above a structural discontinuity, such as the end of a shear wall, a change in column cross-section, or even the transition story from a composite steel-reinforced-concrete structural system to a pure reinforced-concrete system. In the last, it must be noted that encasement of a steel frame in a reinforced-concrete frame has been a popular, uniquely Japanese approach to enhance the seismic resistance of buildings. All the story-collapses that took place occurred in buildings designed prior to the introduction of ductile material requirements to Japanese building codes. For example, a close view of typical reinforcement detailing shows the gross inadequacy of transverse reinforcement when compared with current requirements. It was standard practice prior to 1971 to space this transverse reinforcement at 300 mm intervals center-to-center (Photo 14). Code changes introduced in 1971 reduced this spacing to 150 mm, further reducing it to 100 mm at the ends of structural members, thereby providing more appropriate shear resistance and concrete confinement.

The latest building code changes, made in 1981, introduced other stringent ductile-design requirements as well as a two-level design procedure which effectively requires that the ultimate seismic resistance of buildings be verified. As a result of this evolution of design requirements, a striking contrast between the life-safety protection afforded by adjacent buildings constructed in different eras was clearly seen after the earthquake. Table 1(c) clearly shows the contrast in the damage done versus age of construction of RC buildings. Whereas the newer buildings sometimes suffered internal structural damage, few of them totally or partially collapsed (Photo 15). One severe type of damage to the newer buildings is to the 1st story open style, one reason for which is the lack of strength and stiffness in the columns of the 1st story.

Data on 263 low- and middle-rise RC buildings were analyzed to clarify the relation between earthquake resistance and the extent of damage. Figure 6 shows the distribution of ultimate shear force coefficient by age of construction. Shear strength in one direction is assumed to be 25Aw+7Ac, where Aw and Ac are the sums of the cross section of the wall and columns (cm^2) of the 1st story. The shear coefficients of the weak axis of older buildings mainly are less than 0.5; whereas those of the newer buildings show wider distribution and larger values. The deformability of the newer RC buildings must be large because of the **requirements** of the 1981 building code. **As shown in Fig. 7,** extent of damage done to such buildings depends on the shear stress, wall ratio, and construction age, the wall ratio being defined as the sum of the cross section of the bearing wall divided by the total floor area of the building, Aw/A,and shear stress is being calculated by W/(Aw+Ac)

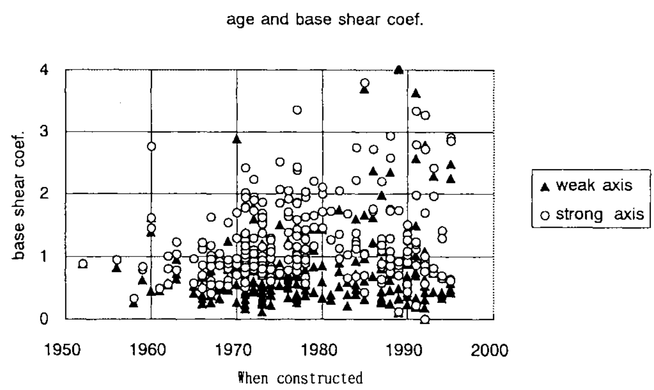


Fig.6 Distribution of the ultimate base shear coefficients of RC buildings

Photo 13 Collapsed fourth story of an RC building

Photo 14 Poor transverse reinforcement

Photo 15 Building designed under new Code was not severely damaged

Photo 16 Severely damaged steel-braced Frame

Photo 17 High rise steel apartment building damaged

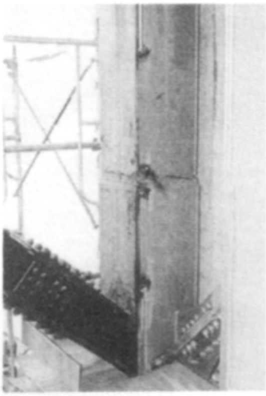
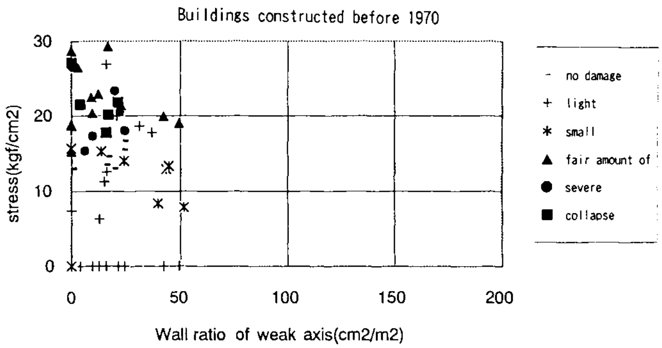
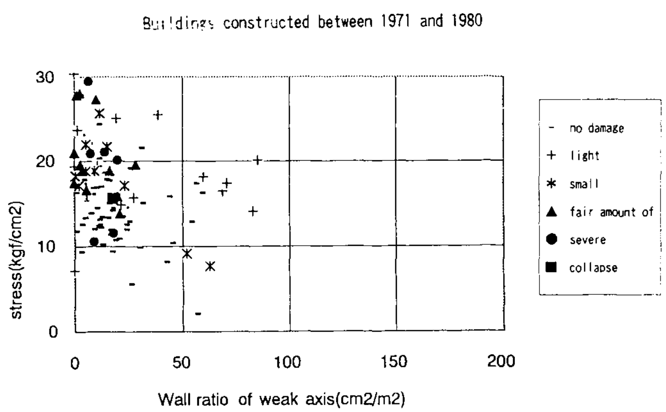


Photo 18 Brittle failure of a column member in the building in Photo 17





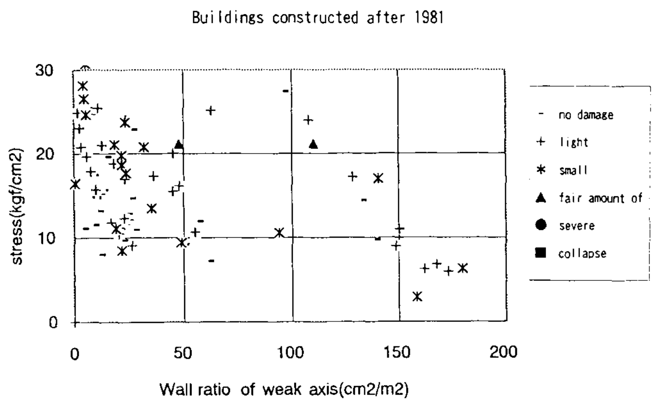
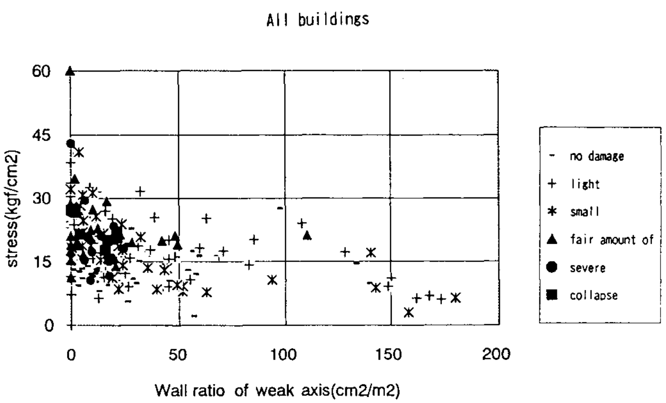


Fig. 7 Extent of damage and the shear stress

corresponding to the inertial force of the upper part of the 1st story subjected to 1g acceleration, and W being the total weight. The extent of damage to the weak axis is very high in older buildings, and the smaller the wall ratio the greater the damage, as Dr. T. Shiga suggested after the 1968 Tokachi-oki earthquake (Shiga, 1970).

Steel Structures

A large number of braced-frame structures were damaged (Photo 16), but few collapsed, even those for which insufficient workmanship was uncovered. In general, frames with extremely slender braces, such as rods or thin plates, showed poor seismic performance, the braces frequently rupturing under tension. Many moment frames were found to have damage to their beam-to-column connections. In many cases, damage could be attributed to the failure of the welds, as found in frames subjected to the Northridge California earthquake.



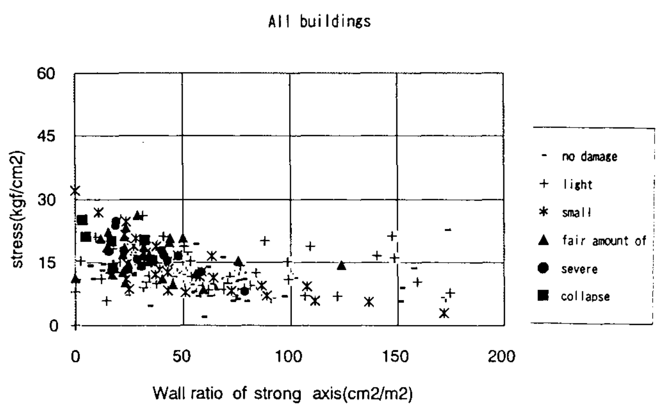


Fig. 7 Extent of damage and the shear stress (continued)

Because steel structures frequently are hidden behind fire-proofing coatings and architectural finishes, it will take some time before the full extent of damage to steel buildings is known. Some new steel buildings, however, are known to have suffered important damage. Although most of the damage seen in new steel buildings has been ductile deformation to be expected given the design philosophy, some brittle failures of steel sections and welds have been observed on high-rise braced-frame structures (Photos 17 and 18). In addition that input motion exceeded the design ground motion, amplification was large in the 19th-story and 24th-story buildings due to fundamental frequency of structure and frequency characteristics of ground motion,and large axial forces were introduced in the columns and braces due to overturning moment and vertical responses. More detailed studies are underway to provide definitive information on these matters.

SEISMIC RISK MANAGEMENT FOR FUTURE URBAN PLANNING

Statistics on the anti-seismic strength of wooden houses were gathered in Kushiro City, which was struck by the 1993 Kushiro-oki earthquake (Fujiwara et al., 1994). The average strength of wooden houses in Kushiro is 2-3 times the value prescribed in the building code, whereas the values for houses in Kobe and surrounding areas are very small (Suzuki T., 1995). The strength of wooden houses may vary with the climate or construction method. We surveyed statistics for wooden houses in Kyoto, in which the number of houses built before 1973 in each mesh (1km x 1km) on 1988 exceeded 3,000 houses in 5 blocks (Kitahara A. et al., 1990). The damage estimation for the real distribution of existing houses subjected to an assumed earthquake was compared with an idealized distribution of houses built under a new code incorporating the dynamic characteristics of houses and soils corresponding to age and form (Fig.8).

(a) Collapse ratios for existing houses (b) Collapse ratios for houses assumed constructed after 1981

Fig.8 Collapse ratios for houses in a scenario earthquake for Kyoto City, Japan

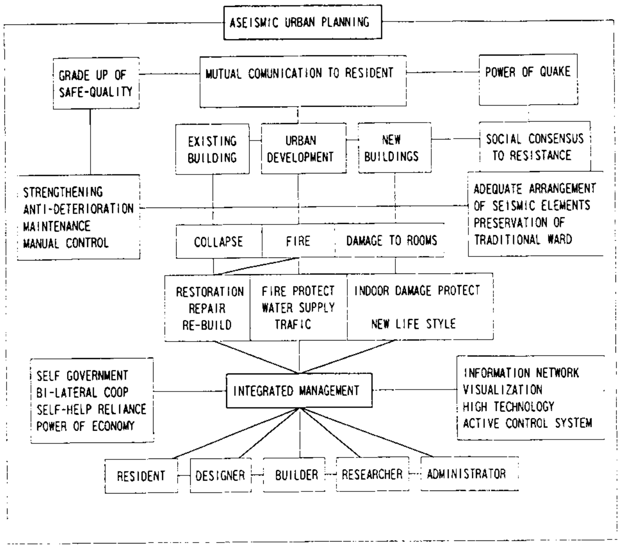


Fig.9 Integrated management for urban seismic planning

As stated, the Hyogoken nanbu earthquake brought to light a wide range of problems related to seismic disaster. Figure 9 shows the flow of Integrated Management for Urban Seismic Planning. The following problems must be studied in order to mitigate earthquake disasters and to enable future urban planning against strong earthquakes.

1) Most of the buildings that collapsed or were severely damaged were built before the new building code, 1981. To estimate seismic risk in urban areas, the strength of existing buildings and houses must be surveyed. Some information, for example, that obtained for Kushiro and Kyoto City may be useful. This method of risk estimation should be verified based on the damage done by the Hyogoken nanbu earthquake.

2) The strengthening of existing wooden houses is one of the most urgent problems to be solved. Easily understandable manuals edited by engineers and researchers must be provided for resident and owners. People should be able to judge the safety of their own houses and to maintain them in a normal state because the resistance of houses generally deteriorates with age.

3) Some buildings with garages or stores in their first stories, particularly small houses with narrow fronts may be severely damaged. In the last case, it may be difficult to strengthen such houses individually. Reform of urban planning must be advanced by planners, administrators and residents.

4) Many kinds of serious damage occurred simultaneously: the collapse of buildings and the disruption of traffic and lifeline systems (water, gas, electricity, telephone service) and widespread fires. It is very difficult for individual organizations to respond to such problems. Integrated management that **incorporates** the expertise of persons with different specialties is necessary to create well-matched urban society between preservation of comfortable tradition and development, that is able to cope with the aftermatch of a severe earthquake.

5) Nearly 10% of the people killed by the earthquake were crushed by the overturning of furniture.

Selection of living space for older persons and prevention of overturning of furnitures are also important for mitigation of the death and the injured.

ACKNOWLEDGMENTS

We thank the Kansai-branch of the Japan Structural Consultants Association for the data on RC structures, and the city of Ashiya for the damage and fundamental database information, which were very useful for the damage estimations. We thank the many people who helped with this manuscript, in particular Dr. Masayoshi Nakashima, Professor, the Disast. Prev. Res. Inst., Kyoto University, for his wholehearted cooperation.

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EVALUATION OF LOSSES FROM EARTHQUAKE DAMAGE TO EQUIPMENT IN PETROCHEMICAL ENTERPRISES

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ABSTRACT

Seismic risk, earthquake damage, and losses from the damage have been studied primarily in relation to structures and nuclear power plants. Recently, however, attention has been focused on other large industrial plants. This paper presents the methodology for analysis of earthquake to petrochemical equipment, describes the standard of grade classification of earthquake damage to equipment, and gives the regulations on evaluation of losses from earthquakes to equipment.

KEYWORDS

Earthquake damage; equipment; petrochemical; enterprises; seismic risk; economic loss.

INTRODUCTION

Earthquakes are destructive natural disasters. The risks from earthquakes to petrochemical plants are not only direct damage to equipment, but also serious secondary disasters. Public concern for the consequences of earthquake damage is growing, especially in highly populated areas near these plants. The need for a comprehensive methodology to evaluate risks and losses from the failure of major petrochemical facilities has recently been recognized. Some regulations on losses from earthquake damage to equipment appear in Standard of Grade Classification of Earthquake Damage for Petrochemical Industrial Equipment, which was worked out by China Petrochemical Corporation (SINOPEC).

This paper describes the analysis of earthquake risks to petrochemical equipment, the grade classification of earthquake damage to equipment, and the losses from such damage.

SEISMIC RISK TO EQUIPMENT

Major petrochemical epuipment is often a conglomerate of interconnected structural, mechanical, and electrical subsystems. These subsystems are functionally and physically dependent. Seismic risk analysis techniques for conventional buildings, which generally are single structures, cannot be applied directly to major petrochemical equipment because of the complexity of such equipment. Petrochemical facilities may differ considerably from nuclear power plants. It is not worthwhile applying analysis methods for nuclear power plants to petrochemical equipment.

A general risk methodology suitable for application to major petrochemical facilities must treat the facility as a system. Physical and functional dependencies must be considered. The main steps in the methodology include:

(1) evaluation of the site hazard;

(2) evaluation of component perfomance;

(3) assesment of overall plant perfomance;

(4) computation of losses.

Aspects of methodology are further subdivided as shown in Fig. 1.

Site hazard analysis seeks the probability of danger to a structure under given circumstances. At present, numerous methods are available for the evaluation of potential ground motion at a site. The advanced first-order-second-moment method can be used in analyzing component performance. The correlation between components should be considered in the analysis of overall plant perfomance. A major step prior to overall system reliability is the identification of all components, their correlation and their function within the system. For the analysis of major petrochemical facilities, there are several risk criteria to select, such as capital loss, production loss, fire, and release of hazardous materials.

GRADE CLASSIFICATION OF EARTHQUAKE DAMAGE TO EQUIPMENT

In order to know the degree of earthquake damage and to evaluate the economic losses from it in petrochemical enterprises, SINOPEC has worked out "Standard of Grade Classification of Earthquake Damage for Petrochemical Industrial Equipment". The standard is suitable for grade classification of damage to equipment in use, lying idle, or in storage in petrochemical enterprises. Damage was caused by earthquakes or secondary disasters from earthquakes. Earthquake damage to equipment is classified into

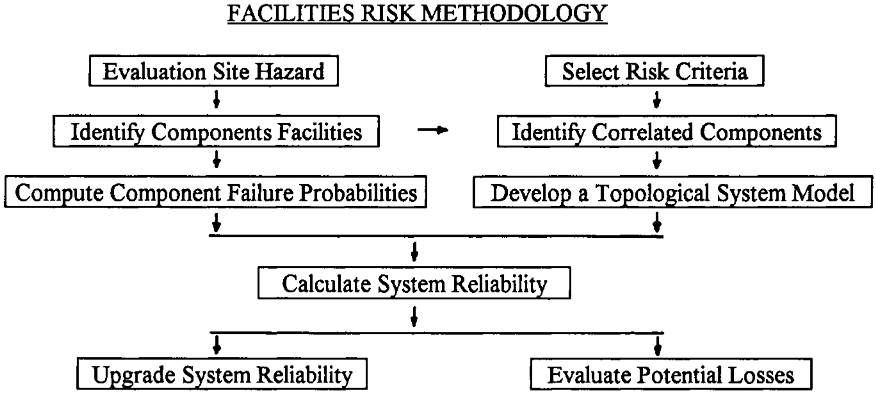

Fig. 1. Steps in seismic risk analysis of major petrochemical facilities

four grades: basically good condition, minor damage, moderate damage, and serious damage.

Basically good condition means that the main body and main parts of the equipment are undamaged; attachments may be damaged slightly; the foundation of the equipment and supporting components are intact; it generally stays in operation without repair.

Minor damage means that the supporting components, the body of the equipment, and/or the main parts are damaged slightly; attachments are damaged to various extents; minor cracks occur in the foundation of the equipment; it stays in operation with ordinary repairs.

Moderate damage means that the main body of the equipment is damaged to a certain extent; some supporting components and main parts of the equipment are obviously damaged; differential settlement and cracks occur in the foundation of the equipment; several attachments are damaged seriously; it may stay in operation by repairing or exchanging some parts.

Serious damage means that most supporting components, the body of the equipment, and/or main parts are obviously damaged; serious differential settlement and cracks occur in the foundation of the equipment; it needs extensive repairing or exchanging, or it is not worth repairing at all.

Different regulations are made for the gradation of damage, according to structural characterisics and function of various kinds of equipment. Processing equipment, which includes furnaces, towers, reactors, tanks, heat exchangers, horizontal vessels, air coolers, chemical fiber equipment, power equipment, instrument-automatic equipment, pipe-ways, and communication equipment is involved in the standard. The principles followed during grade classification of damage to equipment are:

(1) Grade classification of damage is based on the type of equipment;

(2) Degree of damage to equipment is determined on the basis of degree of damage to body of equipment;

(3) A corresponding concept of amount is introduced into the judgement of the degree of damage to equipment;

(4) Degree of difficulty in repairing, service in continuing or not, and direct economic loss should be considered in grade classification of damage to equipment;

(5) Damage, caused by other reasons before the earthquake, should not be considered in grade classification of damage to equipment.

After a destructive earthquake occurs, the investigation and grade clssification of damage to equipment should be conducted under the above principles.

EVALUATION OF DIRECT ECONOMIC LOSS FROM EARRTHQUAKE DAMAGE TO EQUIPMENT

An evaluation of direct economic loss from earthquake damage to equipment is based on the grade classification of damage to equipment. Principles of evaluation by the above-mentioned standards are:

(1) If the equipment is basically in good condition, it should be calculated according to actual maintenance cost in eliminating the minor damage;

(2) If the equipment has minor or moderate damage, it should be calculated according to actual cost of repairing and exchanging parts ;

(3) If the equipment has serious damage, and is not worth of repairing, it should be evaluated according to the current price multiplied by a depletion factor reflecting the degree of new and old equipment; for equipment that needs to be extensively repaired or exchanged, it should be calculated according to the actual cost of repairing or exchanging;

(4) For equipment in storage which was damage to the extent that it cannot be put into operation, it should be calculated according to the purchase price, eliminating the residual value.

Evaluation of economic loss is a complicated task. For convenience in calculation, some formula is useful, especially when the formula is related to the grade of earthquake damage. The work was done by the standard of conventional buildings, but not by the above-mentioned standard due to the characteristics of petrochemical equipment.

CONCLUDING REMARKS

The evaluation of seismic risks to equipment and the losses from damage to equipment are discussed in this paper. A practical program for the computation of losses from earthquake damage should be developed on the basis of future research. More extensive evaluation of losses, such as the assessment of secondary disasters caused by earthquakes, is important to include.

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RESTORATION AND RECONSTRUCTION OF BATANG EARTHQUAKE-STRICKEN AREAS

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ABSTRACT

Batang county is located in the western border of Sichuan province, China. Basic seismic intensity is degree-8. A succession of over-6-magnitude earthquakes shocked these areas on April 16, 1989. This paper reviews the earthquake and the process of post-earthquake rehabilitation and reconstruction. First is a brief description of the earthquake and its disastrous consequences. Then details of work in rehabilitation and reconstruction after the earthquake are presented. Finally, the lessons and inspiration of reconstruction are noted.

KEYWORDS

Earthquake; restoration; reconstruction; planning; investment efficiency.

THE EARTHQUAKE AND ITS DISASTROUS CONSEQUENCES

During the days from April 16 to May 3, 1989, a succession of over-6-magnitude earthquakes shocked Batang county 4 times. Among these, two were of 6.7 magnitude, one was 6.4 and the other 6.2. Including more than 20,000 medium and slight shocks, the disaster was actually a rare "mass shock earthquake" in the history of seismology. At the same time, it spread to the neighbouring counties of Li Tang, Bai Yu, De Rong, Xiang Cheng, Dao Cheng and some areas in Tibet Autonomous Region and Yunnan province. Loss from the earthquake was heavy. In Batang, the shock hit 9,064 houses and 46,524 persons in 5 towns and 23 villages, causing 8 people to die, injuring 97 and killing 7,800 domestic animals. All buildings and engineering installations in the county were destroyed in varying degrees, causing an economic loss of 296 million yuan (RMB).

RESTORATION AND RECONSTRUCTION

Immediately after the occurrence of the earthquake, the party committees and government at different levels showed their concern for the stricken areas, and quickly organized the work of disaster relief and social help. Their advocacy gained support and aid at home and from international societies. Therefore, relief work was victorious. In spite of the financial difficulty of the state, 100 million yuan were allotted with some supplies based on the amount of the fund, such as steel and cement for reconstruction, and 80 million yuan from the total allocation were allotted to the heavily stricken Batang county.

It was not easy to do a lot for the people in need even with the fund. Reconstruction faced great and unexpected difficulties. One was long-distance transportation and bad highway conditions. More than 40 million kilograms of building materials were needed for delivery to the work site by 8000 transports; each day 13 loads of materials had to be transported over 800 kilometer to the site. Frequent landslides also caused trouble for construction. Another problem involved time limitation (only 3 years), large-scale projects (including towns, villages and economic development), and extensive travel schedule (the farthest village is above 200 kilometers away from the county centre). A third problem is attributed to the adverse natural environment: a wide range of temperature between day and night and a changeable climate seriously hindered transportation and work . A final difficulty was that projects in high-intensity earthquake areas must be rigorously constructed, but the region had a shortage of such professionals as engineers and technicians. People in the county, however, led by the county's 'four leading groups' (Party Committee, government, Congress and the Political Consultative Conference), did not flee from difficulty; they joined forces for a battle of reconstruction.

Establishment of Organizations, Planning for Reconstruction, Preparation for Preliminary Work

Directed by provincial, prefectural and county's leading group for reconstruction, the county's reconstruction headquarters and its office were set up in November 1989. Many professional teams such as finance, engineering, resettlement, economic potential, local building materials and information were put in charge of the matters concerned. No. 175 document issued by the People's Government of Sichuan Province specified resistance and good reconstruction of the county with each building earthquake resistance and their native characteristics. Instructive material, views on planning of reconstruction for Batang county was also produced, based on site investigation and analytical results conducted by the Provincial Reconstruction Group in disaster-stricken areas. These instructions placed the key work in the respective county districts and coordinated it with reconstruction of civic installations.

From February to March of the next year, a second reconstruction plan drawn by the Institute for

Architectural Design of Sichuan was finally chosen after many review meetings held by the Construction Committee of Sichuan Province (CCSP) and the Provincial Reconstruction Leading Group. Thus, "Reconstruction Planning for Batang County" was ratified formally in the No. 94 document (CCSP,1990). By July, all preparations were complete.

Implementation of Projects and Allocation of Funds

"Reconstruction Planning for Batang County" was an overall plan divided into two stages, the "Overall Planning Period (1990-2010)" and the "Short Term Reconstruction Period (1990-1994)". The affected district of the county was 0.6 square kilometres all together. Among the total investment, 84.1455 million yuan were for short-term reconstruction, including respectively 42.0856 million yuan for civic housing, 10.7099 million yuan for civic auxiliary installation, 5 million yuan for the peasant housing subsidy, 1 million yuan for temple maintenance, 8 million yuan for reconstruction of townships, 15 million yuan for economical development projects and 2.35 million yuan for other investment. These successive funds ensured the smooth implementation of reconstruction.

Meticulous Design and Construction

Reconstruction of Batang was an extremely arduous but glorious task. It is politically and economically significant as well as decisive for the stability of the disaster-stricken areas and even for the union of the whole entire Zang nationality area. Led by the 'four leading groups' and the 'county's headquarters for reconstruction', 5 design institutes, including the Institute for Architectural Design of Sichuan, over 21 construction teams from Zigong, Jiangyou and Qionglai, and a military transport team from Chengdu were involved in the painstaking work. Everyone in the effort was duty-bound to go all out for the victims of the earthquake. Having the needs of the people in Batang at heart, the troops for reconstruction successfully accomplished their strenuous task even under adverse conditions of continuous traffic jam, and an additional 10-thousand square metres of building acreage and civic auxiliary installations exceeding the original plan.

Reconstruction of Batang was completed in September 1992. Its investment (86.5046 million yuan) accounts for 100.5% of the planned total investment. Of the total financial input, 49.7464 million yuan were for 113.3 thousand square metres of reconstruction, 600 thousand yuan for 3,200 square metres of housing rehabilitation, 11.6033 million yuan for 7 civic installations in the town district, 13.12 million yuan for reconstruction of economic development sites, 5 million yuan for peasant housing subsidy, 1.03 million yuan for religious temples and 4.1537 million yuan for other investments.

Guarantee of Strict Anti-Seismic Structure and Quality

Anti-seismic structure is regarded as the most effective measure to ease the suffering; such a system is of vital importance for reconstruction. Therefore, the drafting of anti-seismic design and budgetary estimates were strictly examined and approved after studies on new earthquake-proof technology. Some buildings were constructed by adopting a supporting system of special concrete blocks. In the meantime, 24 principles in 'Items for Work Regulation of the Reconstruction of Batang' defined standards of the work teams, qualification certificates, bidding, site management, organization, quality check, safety, pre-acceptance check and rewards-punishments. In addition, high quality and low cost of building materials was ensured as a result of unified distribution of large quantities and purchase of small amounts by the work teams. Measures such as training and assessing inspectors,on-the-spot guidance and cross-checks were taken to strengthen the management of the construction teams. Quality check departments inspected all construction and public installations, making sure the 64-thousand square metre reinforced-concrete houses were fully in accordance with the current standard of aseismic design intensity (degree 8) and the degree 9 intensity specified in the Code for Aseismic Design of Industrial and Civil Building Structures (TJ11-78). Buildings like the 3.6-thousand square metre brick-concrete structures and 23-thousand square metre special concrete block houses in the town district, together with the 800 square metre reinforced-concrete buildings, 7,700 square metre brick-concrete structures and the more than 14,000 square metre wooden houses in rural areas were all up to quality standards,and some reached even higher standards.

Raising Investment Efficiency by Economical Use of Money

For the purpose of saving money, factories of sand-stone-brick were set up, wholesale building materials were delivered, final budgets were strengthened, and construction volunteers were organized. All these resulted in a 8.447 million yuan surplus, which made possible an additional 10,300 square metres of housing, concrete roads, 150KW of electrical installations, 3 repaired power plants (1390KW installed capacity), 9.3 kilometres of canals, 0.57 square kilometres of irrigated area, 1,285 metres of converted irrigation coverage, 0.505 square kilometres of farmland, 0.637 square kilometres of forestland and orchards, and 16 pieces of equipment of various kinds.

Importance of Pre-Acceptance Approval for Quality of Work

In July 1992, the Provincial Reconstruction Office checked the work and made some revisions after inspection. In October, led by the CCSP, 85 representatives from the departments concerned checked and approved the work. They spoke highly of the reconstruction whose quota was met 9 months ahead of time. Now an entirely new county district stands on the Sichuan-Tibet plateau with its houses, public installations, design and construction quality much better than the original level before the earthquake. It is undoubtedly conducive to promotion of the economy and culture of Batang and its stability as well as that of the entire Zang nationality area.

LESSONS AND INSPIRATION OF RECONSTRUCTION

Comprehensive Direction by Organizations for Good Results

Reflecting the concern of the Party Central Committee, leaders from the province, autonomous region and county set up the reconstruction's leading groups and headquarters; these organizations undertook the work of logistics, decision-making and simplification of formalities as well as conducting analysis of the guiding principles, investment items, overall planning, design and construction. They created favourable climate for reconstruction and expedited as a result.

Importance of Planning for Reconstruction

Planning, which is the basis for restoration and reconstruction, was worked out according to state standards of the earthquake disaster scale. Two investigating teams were sent by CCSP to collect details of the disaster. Thus, the document "Opinions on Planning Batang's Reconstruction" was produced. After several approaches were analyzed and evaluated, a plan was chosen. Unified planning and design, implemented by stages, not only guaranteed the rational use of money and the quality of work but also quick completion 9 months ahead of time.

Practical Measures Decisive for Success

Methods like centralized control of funds and strict budgeting had to be adopted due to conditions: the county is situated in Sichuan's border areas of high earthquake intensity with limited funds and an urgent need of technology. Measures such as the unified purchase of building materials, the application of new technology for anti-seismic structures, the choice of good design and construction teams, the use of good principles of reconstruction are all decisive factors in smooth construction. Success is shown by the budget surplus and the completion of work 9 months before the deadline.

Integration of Reconstruction and Economic Development in Batang County

Considering the county's situation in a poor, mountainous district with an underdeveloped economy, more than 12 million yuan were shifted to economic development, such as transforming (including repair) of over 30 kilometres of canals, setting up 5 power plants (2300KW installed capacity), renovating villages' small power stations (with a capacity of 1550KW), and building more than 20 kilometres of highway and a bridge. All these projects have the goal of tapping Batang's potential, as resolved by the People's Government of Sichuan Province. Clearly, a solid foundation for Batang's further economic development has been laid.

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DA-YANG EARTHQUAKE AND POST-QUAKE RELIEF WORK AND RECONSTRUCTION IN YANG-YUAN COUNTY

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ABSTRACT

This article gives a survey of emergency measures, anti-seismic and relief work, utilization of foreign loans, and reconstruction in Yang-Yuan County after the earthquake of 1989, measuring 6.1 on the Richter scale, in the area of Da-tong City and Yang-yuan County in North China. The article makes observations on the experiences and lessons about farmhouse building, choice of sites, types of architectural structure and post-quake reconstruction. It also expounds the necessity to establish countermeasures of pre-quake precautions and post-quake relief work, to publicize anti-seismic knowledge and to enhance people's consciousness of reducing earthquake effects.

KEYWORDS

Earthquake; damage; relief work; reconstruction; precautions.

INTRODUCTION

No matter where an earthquake occurs, in the city or the countryside, it does damage in varying degrees to the area, and causes injury and death as well as economic losses. For earthquake areas, some research should be done, such as summarizing the experiences and lessons of anti-seismic and relief work, homeland reconstruction after an earthquake, establishing countermeasures in anti-seismic and relief work, and increasing the ability of unified protection, to reduce earthquake damage.

BRIEF ACCOUNT OF DA-YANG EARTHQUAKE AND ANALYSIS OF EARTHQUAKE DAMAGE

On October 19, 1989, a destructive earthquake occurred, measuring 6.1 on the Richter scale, in the area of Da-tong City and Yang-yuan County, a juncture of Hebei and Shanxi Provinces and Inner-Mongolia Autonomous Region, North China. Next to the 1976 Tangshan earthquake, it was the strongest, farthest-reaching earthquake in North China, causing severe damage. The disaster area was distributed over Yang-yuan and Yu-xian Counties, Zhang-jia-kou, Hebei Province, and Datong and Yang-gao counties, Datong city, Shanxi Province. In Yang-yuan county, 11 towns and 134 villages suffered different degrees of destruction and casualties: 49,000 houses were destroyed, 8500 houses toppled over; more then 80,000 people suffered from the casualties, 388 people were injured and 3 were killed; over 500 domestic animals were killed; some irrigation and water conservancy installations were destroyed. Economic losses reached 75 million RMB yuan.

More than ten villages were severely afflicted in Yang-yuan County, such as Shi-jia-hui, Xi-fu-di and Dong-jing-ji. Main causes of the destruction and casualties are as follows.

1. The tectonic structure of the earthquake was the Sang-gan-he River fault. The seismic focus was about 10 kilometers deep. Over several days, 6 after-shocks of above 5 on the Richter scale occurred in succession. While the seismic focus was shallow, the intensity was high. The county seat was hit by the earthquake with an intensity of 7. The quake had very strong effects on farmhouses.

2. Some villages in the severely afflicted area lie just on the old course or floodplain of the Sang-gan-he River, which is not a favorable anti-seismic construction sit e.

3. Most houses in the earthquake-stricken area were built mainly of earth and wood; some houses had walls of immature soil and vault roofs of sun-dried mud bricks. Neither type of house had much anti-seismic strength.

4. Building materials for the farmhouses were mainly immature soil, or products of immature soil, which were adhered by clay. Quality of construction was very poor.

5. People there used to build flat-roof houses. The front wall of the house was 1.7 meters lower than the back wall. This is not a favorable anti-seismic structure.

6. Villagers were indifferent to precautions against earthquakes and had little consciousness of self-protection. They bungled the chance of anti-seismic and relief work so that conditions became worse.

POST-QUAKE RELIEF WORK AND RECONSTRUCTION

As soon as the earthquake took place, the "Emergency Plan Against the Coming Earthquake in Yang-Yuan County" went into effect. The municipal government of Yang-yuan County established headquarters in charge of anti-seismic and relief work. Over 300 technical and managerial personnel were divided into more than ten groups. They were sent to the earthquake-stricken area to help people there. Every town and village, every productive and administrative unit set up anti-seismic groups and teams, formed mainly by young people. Amid self-help and mutual aid, they organized the villagers and helped them leave the toppled houses and the dangerous houses with big fissures. They also rescued and transported the wounded. They evacuated old people, women and children to safe places to avoid more injury from the strong aftershocks.

After anti-seismic and relief work, we took steps to rebuild our homeland. Four jobs have been done as follows.

Appraise the Houses

In order to avoid blind demolition of houses after the earthquake, we appraised them to let people know which houses were safe enough to live in, which houses were dangerous and should be dismantled right away, and which houses needed to be repaired. We divided them into three kinds: fairly good houses, houses in need of repair and reinforcement, and houses which should be dismantled and rebuilt.

Fairly Good Houses. Load-bearing structures were mostly intact. There was slight damage to some parts of the house. People could go on living in them with minor repairs or none at all .

Houses in Need of Repair and Reinforcement. Load-bearing structures were slightly damaged. The damaged area is no more than 30% of the entire structure. Such houses belong in this category. We gave the villagers some technical guidance in repairing and reinforcing. Two points should be noted: 1) In reconstruction we do not copy the old building patterns nor neglect anti-seismic measures; 2) In reinforcement we do not consider only part of the building, but rather whole anti-seismic function.

Houses be Dismantled and Rebuilt. Load-bearing structures, or most of the building had been severely damaged, or even toppled. They were either beyond repair or the cost would have been prohibitive.

Formulate a Good Reconstruction Plan

In reconstruction, a good plan is essential. It should be long-term and coordinated with the development plans of the towns and villages. Distribution of resources should be rational. We should choose a favorable anti-seismic position. Floodplains and old courses of rivers with high-water tables should be avoided. According to anti-seismic requirements, enough passageway should be left for people to disperse in time of danger.

Establish Unified Designs and Reform Farmhouse Construction

Reconstruction should be unified and efficient. According to the requirements against earthquake with an intensity of 7, newly-designed construction has three different types: bricks and concrete, bricks and wood, and wood and sun-dried mud bricks. There was a standard blueprint for each type. It was required that all newly-built houses have gable roofs, which have a better anti-seismic function. There were detailed specifications for anti-seismic structures and measures.

Strengthen Management of Reconstruction

In order to guarantee the engineering quality of reconstruction, we conducted a technical training course which engineers, technicians and assistant cadres of the towns attended. In the tasks of the repairing, reinforcing and rebuilding, these technical experts conducted and supervised the work so that engineering quality was assured.

IMPLEMENTATION OF RECONSTRUCTION

Yang-yuan County lies in the northwest of Hebei Province. Natural conditions there are poor. It is an arid and backward area, one of the poorest counties in Hebei Province. Its financial subsidy each year comes to RMB three million yuan. The earthquake caused the county economic losses of 75 million RMB yuan. Cost of the reconstruction is expected to be 50 million RMB yuan.

State government, social organizations and international organizations have paid close attention and attached great importance to the earthquake. Ministry of Construction, Ministry of Finance and World Bank sent officials several times to the area. They learned about the post-disaster situation, and put forward the proposals concerning reconstruction. Officials from the Chinese government and World Bank have done numerous investigations, demonstrations, evaluations and negotiations. World Bank offered a long-term interest-free loan of 6 million US dollars, effective in April 1990 after the necessary procedures.

In order to make good use of the loan, an Office of Foreign Loan was set up in Yang-yuan County, which was responsible for all matters concerned. The office examined plan and ratified applications from working units and farmers, loan repayment plans and drawings for house-building. Also the office was in charge of the management, coordination, supervision and implementation. In decisions on projects, we adhered to the principle that schools, hospitals and hardest-hit areas should be given first priority.

SEVERAL LESSONS

In two years of reconstruction, 30,000 houses have been built, repaired or reinforced in Yang-yuan County. More than 50,000 people with housing problems have been resettled. Over 1,300 classrooms in 61 schools have been rebuilt or repaired. A new impatient facility and Dong-jing-ji Hospital have been built to ensure epidemic prevention and medical treatment of 300,000 people in the county and neighboring areas. A new flour mill has been built in the grain and oil processing factory. Seventy motor-pump wells have been finished. New irrigation canals, a total of 185,000 meters long, have been finished. Thus the water conservancy facilities of 58 towns and villages have been improved. Construction tasks have been completed three years ahead of schedule, so that the farmer's burden both in spirit and materials has been lightened, and the state has invested less. Another earthquake of M 5.8 degrees occurred on March 26, 1991; the reconstruction and major projects there did not suffer damage. We made the most of the World Bank loan, which has brought both economic and social improvement.

There are four lessons from the people's anti-seismic and relief work as well as reconstruction in Yang-yuan County.

Take Precautions against Earthquakes

In recent years, our country has monitored this region as a critical earthquake area. Some precautionary work had been done before the 1989 earthquake. This earthquake caused severe structural damage and economic loss but fewer casualties due to prior dissemination of an "Emergency Plan Against the Coming Earthquake in Yang-yuan County". As soon as disaster struck, anti-seismic organizations were founded immediately. Their anti-seismic and relief work made possible a decrease in casualties. The “emergency plan” thus played a key role.

Be Practical and Realistic in Reconstruction

The earthquake caused heavy losses in a large area in Yang-yuan County. Farmers there are still poor and it was difficult to raise enough money for reconstruction. On one hand, we put an emphasis on our own efforts; on the other hand, we tried to get outside support whenever practical and realistic. With supporters from all walks of life, we strengthened management and made good use of the loan. Construction has been brought to the same level as it was before the earthquake, three years ahead of schedule.

Combine Construction Plans with Precautions against Earthquakes

We made sure the construction plans of towns and villages included precautions against earthquake. All aspects such as site choice of towns and villages, scale of construction, rational distribution, and type of reconstruction were considered. During the process of site choice, design, construction, inspection and approval, the "anti-seismic" has been considered. From analysis of the two destructive earthquakes, measuring 6.1 in 1989 and 5.8 in 1991, it is clear that only with anti-seismic construction can we reduce the earthquakes calamities.

Disseminate Knowledge to Enhance People's "Anti-Seismic" Consciousness

An important factor in reducing earthquake effects is to enhance the anti-seismic consciousness of people in earthquake areas and to develop their ability to deal with emergencies. This is part of the experience of Yang-yuan County. In order to enhance people's anti-seismic consciousness, a lot of information must be disseminated. Before the Da-yang Earthquake, we let people in the seismic area know: What is an earthquake? Why do houses have cracks in the walls after a quake? Why do houses with cracks collapse when a strong aftershock occurs? What preventive measures can we take when an earthquake occurs? How can we increase the anti-seismic strength of houses? What scientific measures are there against an earthquake? People's anti-seismic consciousness has been enhanced greatly after dissemination of information. When another earthquake of 5.8 came to the same area in 1991, the people were not in turmoil; they took the initiative in anti-seismic measures.

THE INTEGRATION OF HOUSING RECOVERY INTO RECONSTRUCTION PLANNING

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ABSTRACT

Many cities in areas of seismic activity will face extensive reconstruction and housing recovery challenges in the future. Past case studies of post-earthquake reconstruction processes indicate that during reconstruction planning, trade-offs are made among four general reconstruction goals: rapid recovery of infrastructure; familiar character of the city; enhanced livability and urban amenities; and reduced vulnerability to seismic events. The decision about which outcomes to emphasize is not a technical decision. This decision represents a policy choice based on community values about priorities for reconstruction. For example, reduced vulnerability typically is not achieved where rapid reconstruction is emphasized. The decision about reconstruction in general will affect the outcome of housing recovery efforts. A matrix of housing recovery outcomes in relation to overall city reconstruction goals is set forth. This matrix permits the development of propositions about the integration of housing recovery into broader reconstruction programming. These propositions are elaborated, and brief descriptions of past reconstruction programs are presented to illustrate these relationships.

KEYWORDS

Urban earthquake; reconstruction planning; housing recovery; value choice; policy.

INTRODUCTION

Large earthquakes in urban areas can bring about considerable disruption to the built environment (buildings and lifelines), to the economy of the area, and to the physical and psychological well-being of the population. Structural engineers familiar with earthquake damage are well aware that earthquake damage to an urban area is not random, and much of it is even predictable. The earthquake forces seek out specific weaknesses in several types of systems. The extent of the damage is related to many seismic, geophysical, engineering, and social elements.

During this century our understanding of the relationships among these variables has increased with each earthquake disaster. Many professional groups and agencies in the United States and elsewhere visit damaged areas to observe the damage, analyze findings from monitoring equipment, and present reports following earthquakes, in order to present information on the seismic aspects and the types of damage.

Social factors also play an important role since human decisions about where to build, what to build, and how to build set the stage for disasters. Human decisions about where to build and what building technology to use are guided by social and economic pressures, as well as cultural beliefs and norms.

Increasingly, engineers are saying after major earthquakes -- "there were few real surprises" with respect to what was damaged and what was not. Urban planners and social scientists who study post-earthquake processes recognize that urban problems and social inequities that existed before the earthquake will more than likely be replicated or worsened in the course of reconstruction. If this is to be prevented, public officials must chose to have it be otherwise, and actively govern the process.

The implication of these observations is that many aspects of seismic risk, structural design, building standards, and housing policy have the potential to be addressed before an earthquake strikes. As seismic and engineering knowledge increases there is the potential to apply it to new and existing development in order to reduce losses in future earthquakes. However, for most major cities in the world a very large proportion of the structures in the city were built before specific knowledge was available or before it was applied through land use planning and construction practices or codes. Regrettably, it is not truly feasible to reverse this condition of already existing development vulnerable to earthquakes, and it is inevitable that many cities in seismically active areas will face extensive reconstruction challenges sometime in the future.

Human need, governmental responsibility, and world opinion typically focus immediate and often massive efforts on seeing to the short-term well-being of the disaster-stricken population. However, much less attention has been focused on the human consequences of longer term reconstruction policies. Many of these impacts lie with the way in which the rehousing of the displaced families is approached in the post-earthquake period. This paper will present a set of propositions about the way in which the broader goals for reconstruction can affect the housing recovery outcomes for the displaced households. It will review some specific examples of how housing recovery has been addressed in several past or ongoing earthquake disaster reconstruction efforts. It will identify some of the central reconstruction decisions and programming that can be prepared for in advance. It will argue that it is necessary to focus on specific housing problems that existed before the earthquake when shaping the course of programming for post-earthquake housing reconstruction.

VALUE CHOICES IN RECONSTRUCTION

One of the earliest systematic comparative studies of post-disaster reconstruction examined four reconstruction efforts in depth, three of which were related to earthquake disasters in San Francisco, California (1906), Anchorage, Alaska (1964), and Managua, Nicaragua (1972). Hundreds of documents were examined that described changes in the cities, and the course of the reconstruction effort. For two of the cities, hundreds of interviews were conducted with officials in charge of reconstruction, and with household members who had been displaced by the damage. The authors concluded that there are seven basic issues that arise after a large-scale disaster (Haas, *et al.*, 1977):

- Should normal, as contrasted to extraordinary, decision-making mechanisms be used in deciding how, when, and where to rebuild the heavily damaged city?
- Should there be changes in land use?
- Should there be changes in the building code?
- Should a concerted effort be made to make the city more efficient and more attractive?
- Should there be compensation or special financial assistance for private property losses?
- How should disaster-produced personal and family problems be handled?
- How should increased local public expenditures be financed?

Another notable effort to collect information about reconstruction planning across several cities involved a workshop that was held in order to compare the problems and solutions identified in post-earthquake situations for ten selected urban earthquakes. Officials who had participated in the planning and reconstruction were invited to tell their stories, from which typical reconstruction issues were extracted (Mader and Tyler, 1991). These case studies indicated the various ways in which issues such as the above were dealt with in the post-earthquake setting.

Reconstruction Planning Challenges

Each of the issues above calls for a decision to be made among options. Technical knowledge -- seismological, engineering, financial, social -- is critical to understanding the options available. However, the most important point here is that the decision about how to address each of these issues is a value choice. The comparative studies indicate that typical primary objectives for reconstruction programs include early return of the city to its normal activities, reduction of future vulnerability to earthquakes, and improved efficiency and amenity in the affected city (Kates, 1977).

Studies that addressed the opinions of the residents about the course of reconstruction indicate that ambitious plans for making the city more efficient and beautiful, or more modern in appearance, may result in a city that no longer has a familiar feel for many of its residents (Haas, *et al.*, 1977). Taking this into account, we propose four important goals for reconstruction planning.

- to achieve rapid recovery of homes, businesses, and urban lifelines;
- to retain the familiar character of the city;
- to provide enhanced livability and urban amenity;
- to have a city with reduced vulnerability to future earthquakes.

Case studies of reconstruction (Haas, *et al.*, 1977; Mader and Tyler, 1991) indicate that it will not be possible to achieve all of these equally. Value trade-offs will have to be considered. The decision-makers will have to answer such questions as: Is some of the speed of rebuilding to be traded for making the infrastructure more efficient? Is some of the former character of the city and its trademark style or neighborhoods to be traded for a city that is less congested, or which better protects its residents against future earthquake losses?

Viable and thriving cities are constantly faced with decisions about such things as replacing dilapidated buildings, making the city more livable (e.g., more open space; better transportation) for its residents, redirecting residential building to redistribute population for some valued end, or permitting a major development that is envisioned to, for example, replace some blighted area with a showplace development like a civic center. A major earthquake disaster can force the need and the urgency to decide about many similar things at once.

In a major disaster, all aspects of recovery often need to be addressed, including business and industry recovery, lifeline recovery (water, power, transportation routes), and housing recovery. In major earthquakes that occur under or in close proximity to urban concentrations, commerce, lifelines, and housing may all suffer extensive damage, such as was seen in the 1972 earthquake in Managua, Nicaragua, and the Japan Kansai region earthquake in early 1995 that centered its damage in the city of Kobe. Sometimes one of these elements suffers more damage than others. For example, in many California earthquakes affecting smaller communities (e.g. Coalinga, Whittier, Santa Cruz), business recovery was the major challenge because most of the damage was sustained by the older commercial district buildings. In the Loma Prieta earthquake of 1989, and the Northridge earthquake of 1994, repair of damage to bridges and highways linking the urban centers and enabling residents to move about the sprawling metropolitan area received a high priority.

While it might seem that everyone would have the same post-earthquake goal -- get the city back on its feet -- there actually are many development "agendas." Some of these agendas are new, provoked by the opportunities created by the damage to seriously consider changes in land use or building style.

Others are pre-quake agendas held by various stakeholders who now see the opportunity to re-negotiate for where on the list of post-earthquake priorities their agenda will fall. Also, the scientific and engineering community will bring their technical values to the process; the urban planners will bring their spatial and aesthetic values; the agencies their social welfare responsibilities; and, the developers their economic goals. Private citizens will bring their life style values. All of these elements will need to be integrated, or their exclusion negotiated, during the process of establishing a policy for reconstruction programming.

Housing Recovery Challenges

Whatever the extent of losses in commerce and lifelines, when the residential building stock sustains considerable damage and destruction, attention to housing recovery is likely to become a central concern. Even in large cities, where only a small percentage of total housing is lost, the absolute numbers of housing units that can't be reoccupied after an earthquake can be staggering: e.g., Skopje (1963), 20,000; Managua (1972), 53,000; Mexico City (1985), 95,000; Kobe (1995), 20,000+ (Mader and Tyler, 1991; Earthquake Engineering Research Institute, 1995). Besides addressing basic humanitarian considerations, housing recovery is important for enabling families to achieve some degree of normalcy in their daily routines in order to effectively carry out their functions as workers and household caretakers.

As with earthquake damage in general, damage to residential structures is not random. Severe damage is typically found in relation to soil or building characteristics that are vulnerable to earthquake effects. Furthermore, there often is some relationship between physical aspects such as the materials, design, or location of the housing and some social or cultural variable. The criteria by which residents will judge the desirability of the recovery outcome include social and cultural variables such as housing cost, housing style, neighborhood layout, neighborhood social networks, and access to workplace. Studies of family recovery following earthquakes and other disasters. (Haas, *et al.*, 1977; Bolin and Bolton, 1978; Bolin and Bolton, 1986) find that urban families are typically very anxious to return to their normal daily routines, within the context and privacy of their own family and dwelling. At the same time, those displaced from their homes by the damage often do not feel that the permanent housing type and location they eventually obtain meets their social needs. Most families feel strongly that their housing be appropriate to their social status, be amenable to maintaining their preferred social identity and networks (such as is true of ethnic neighborhoods), and permits them to sustain their life style without having to put a higher proportion of their income into housing rent or payment and maintenance than was the case for them before the earthquake. When households do not perceived that these elements have been achieved, this contributes substantially to the prolongation of the stresses brought to their lives by the disaster.

Thus, we find that the goals for "good" housing recovery are fairly parallel to those for city reconstruction in general. The damaged city's residents will want their newly repaired or replaced housing to meet the following criteria:

- be <u>quickly available</u> to them;
- be <u>socially habitable</u>, that is, be consistent in type and location with their social and cultural identity, and permit them to maintain prior social interaction patterns;
- be <u>sustainable</u> to them, in terms of costs to live in it and maintain in a habitable condition;
- be <u>safer</u> in future earthquakes.

Because of the lack of unlimited resources, knowledge, and expertise, it will not be feasible to achieve each of these outcomes to the same extent as an outcome of the reconstruction process. A high level of achievement of one or two of them may diminish the likelihood of a high level of achievement with one of the others.

An important avenue to quickly rehousing part of those displaced is to allow homeowners to repair their homes as best they can so they can quickly reoccupy them, but it may be done at the expense of seismic safety. The quickness of the repair effort will be compromised by taking time to establish and get consensus on appropriate repair standards, and carry out a process to enforce these. When the replacement housing being provided differs from pre quake housing with respect to location and type of housing, then the decision must be made as to whether to move quickly to get new housing built, without resident involvement in the design of the program, or to enable the involvement of the future occupants in designing the program so as to achieve a high degree of social habitability, but probably slowing down the progress toward reconstruction.

Often excess housing that is available following an earthquake is not truly affordable to low income families who have been occupying older, less valuable dwellings which are now gone. Providing a temporary subsidy program so displaced families can be quickly accommodated in the available housing can serve to create a second phase of displacement when the relief period ends, unless affordable replacement housing has been developed in the meantime. This was true in the aftermath of the Coalinga earthquake in California in 1983, because the area was in economic decline and little new building of any kind was occurring. Post-earthquake housing within the price or rental range of pre-quake housing is also difficult to achieve in rapidly growing areas like Los Angeles and San Francisco. In many parts of these metropolitan areas, land, building, and rental costs have greatly increased in the last fifty years, resulting in the inability to provide new replacement housing at a market price anywhere near the rental cost of the older buildings that often are "removed" by the earthquake. Even in normal times in such areas, new housing for low income families must be subsidized.

In the early months after an earthquake, housing loss victims may also indicate a concern for having a dwelling less vulnerable to earthquakes so they don't have to suffer such loss and disruption again. However, the element of safety does not carry nearly the weight as do the social aspects of their recovered housing situation. Many people will readily occupy a dwelling that meets their present status identification and is affordable, even if it is not safe from some unspecified future earthquake event. Thus private builders and developers are not likely to voluntarily provide seismic resistance. In cultures where individual home owners repair or build their own homes, there may be a lack of understanding or materials necessary to enhance seismic resistance of the traditional house design. Thus the burden of emphasizing seismic safety in both housing repair and housing replacement typically falls to the government. Guidance for how to do it and regulation to ensure that certain building standards are met, call for time, expertise, and the political will to develop minimum standards and to implement the education or enforcement program.

Another dimension of the housing replacement is the common dilemma of whether to provide a progression of housing solutions, from emergency shelter, to temporary housing, to permanent housing for those households that are unable to obtain replacement housing using their own resources or insurance (Mader and Tyler, 1991; Greene and Pantelic, 1992). Permanent housing can take a long time to plan and construct in large enough quantities to meet the needs of the victims of a major urban disaster. If the vacancy rate in existing housing in the surrounding urban area or nearby cities is high enough, it might be utilized to absorb displaced families in the short term. The other common solution is to quickly site and build large quantities of uniform simple dwellings with the intention that they be removed once better permanent housing is available. However, in many instances these dwellings are never removed and become a critical, but substandard and maybe unsafe, solution to longer term housing needs. Some planners now subscribe to the approach of avoiding the provision of extensive temporary housing programs, preferring to concentrate resources and efforts on developing new permanent housing. (Greene and Pantelic, 1992) This creates short term hardship and inconvenience on the families displaced by the damage, but may result in better and more quickly implemented permanent housing programs, such as occurred following the 1985 Mexico City earthquake disaster (Mader and Tyler, 1991; Applied Technology Council, 1991).

Integration of Reconstruction Planning and Housing Recovery

Unless housing replacement is the predominant problem created by the earthquake, the approach to housing recovery will be embedded in the goals and priorities for overall reconstruction planning and programming in that community. Governance mechanisms, national and local decision-making and implementation responsibilities, resource availability, and the salience of public or private sector mechanisms for development will vary from society to society, and from earthquake to earthquake.

There probably is no "right" way to do reconstruction planning. But case studies make it evident that there are different consequences, depending on the approach taken to planning, and the factors and stakeholders taken into account when the value choices are being made to emphasize one aspect of recovery over another. The complexity involved with large scale reconstruction programming will necessitate considerable cooperation and forbearance on the part of citizens and other investors in the process.

However, it appears that this cooperation and patience in the community will be difficult to sustain if the designers and implementors of the reconstruction process are unable to ensure a high degree of certainty that repairs or development of infrastructure, commercial activity, or housing will be accomplished in the specified locations, and more or less within the expected time frame. If reconstruction milestones are not specified, or implementation does not seem to adhere to publicized plans and policies, individuals and corporate developers will resolve lingering uncertainties in their future by making their own informal decisions about how to repair and where and how to re-build. Residential and businesses districts will emerge, but most likely without any coherent framework and with limited improvement in seismic safety. The Managua reconstruction provides numerous examples of these types of actions (Haas, *et al.*, 1977).

RECONSTRUCTION GOALS AND HOUSING RECOVERY

The matrix set forth in Table 1 presents propositions about the way in which the general reconstruction goals are likely to impact on the housing recovery outcomes. Each cell represents a point for considering the trade-offs in housing recovery that are likely to be made, depending on the primary goal or goals driving the overall reconstruction process. These are briefly discussed, and examples provided of reconstruction efforts that illustrate the types of trade-offs that have been made in recent cases, and some of the variables that affected the integration of housing recovery into reconstruction.

Implications of Reconstructing the City as Fast as Possible

When the policy is to make decisions based on the goal of rapid replacement of structures and rapid recovery of functions, the propositions outlined in Table 1 suggest the consequences to housing recovery, in terms of the four criteria outlined above, will be generally as follows. If the displaced population cannot be absorbed by the surviving housing in the surrounding area, it is likely that the policy will place a priority on addressing residential needs in the short-term. This commonly is done by engaging in rapid construction of extensive amounts of temporary housing, which are not likely to ever be removed. These solutions seldom attend to the social meaning of housing, focusing mainly on the basic physical aspects such as walls, roof, and water. Since programming focuses on the number of houses rather than the quality, it will not result in a major upgrading of housing quality or neighborhood amenities, and thus households are likely to be able to bear the costs of the replacement housing in the long run. It is not likely that the resources and time will be put into identifying safe locations, or developing better repair and building standards and practices, and thus it is unlikely that housing, especially repaired housing, will be safer. The disaster may thus be repeated again within a few years or decades. This pattern was particularly true prior to the mid-twentieth century, and is still true for towns and cities in rural and less developed parts of the world. However, in recent decades, those officials or

Table 1. Implications of Selected Overall Reconstruction Goals for Various Housing Recovery Outcomes: Propositions

OVERALL CITY RECONSTRUCTION GOALS:	HOUSING RECOVERY OUTCOMES: *Quickly available*	*Socially habitable*	*Sustainable/ affordable*	*Safer in future earthquakes*
Rapid recovery of facilities and functions	Substantial proportion of displaced families rehoused within first year	Much uniform and simple replacement housing with low emphasis on resident preferences	Replacement housing likely to be affordable	Low emphasis on earthquake resistance of housing
Familiar character and layout of the rebuilt city	Substantial proportion of displaced families rehoused within first two years	Housing type and location likely to be similar to pre-quake and acceptable to occupants	Pre-quake housing issues and problems likely to be replicated or exacerbated	Low emphasis on earthquake resistance of housing
Enhanced livability and urban amenities	Recovery of permanent housing likely delayed by time-consuming master planning for reconstruction	Many older neighborhoods removed and residents relocated to less convenient locations.	Replacement housing likely to be less affordable unless subsidized	Replacement housing likely to be less vulnerable to earthquake damage.
Rebuilt city with reduced vulnerability to future earthquakes	Housing recovery likely to be delayed while hazards better identified	Many areas of city may be rezoned, affecting residential choices	Replacement housing likely to be less affordable unless subsidized	Repaired houses and new residential construction likely to be less vulnerable to earthquake damage

professionals placed in charge of reconstruction planning are less likely to ignore the obvious earthquake risk.

<u>Anchorage</u>. Anchorage, Alaska, is an example of a location in which the speed of recovery was the central priority following the 1964 Great Alaska Earthquake. The inhospitable climate of this Alaskan city meant that restoration of functions, facilities, and housing needed to be accomplished before the next winter. This turned out to be more than feasible, mainly due to an unprecedented amount of federal government resources that were readily committed to reconstruction. Housing recovery was necessary in only two areas, a low income area in the central business district, and an upper income neighborhood on a sloping, highly unstable location. At that time there was an excess capacity of vacant rental units for lower income households in the city, and thus there was no urgency, or even reason, to replace the older rental buildings that were badly damaged. On the other hand, the upper income neighborhood recovery, which experience shows usually precedes other housing recovery in post-disaster settings due to the availability of private resources to accomplish it, was then slowed while the wisdom and approach

for rebuilding in this landslide location was debated. In this remote part of the United States, feelings run strong about the role of the government in regulating private property, and programs to reduce vulnerability that constrained choice were difficult to implement. Thus, the housing needs were quickly resolved (within the first year), but usually with little emphasis on reduced vulnerability to earthquakes. (Kates, 1977; Selkregg and Preuss, 1984).

Leninakan. A moderate-sized earthquake in early December, 1988, heavily damaged many parts of the Republic of Armenia in the Soviet Union. The largest city, called Leninakan at that time, lost 39% of its housing, and over 85% of its industry, medical facilities, and schools (Mader and Tyler, 1991). The then president of the Soviet Union (Mikhail Gorbachev) pledged to rebuild the city within two years. A Vice Prime Minister coordinated the recovery process for the first seven months. He added to the priority of rapid reconstruction the development of a safer city. Local authorities rejected the standard plan for urban development in the Soviet Union, arguing that it was inappropriate to the cultural preferences in this remote region. Several commissions were quickly established to simultaneously accomplish a variety of tasks, such as: deciding what buildings could be repaired; producing seismic zonation maps to guide the siting of new construction; designating appropriate sites for temporary housing; determining building standards for rebuilding; and creating a general land use plan for the rebuilding and expansion of the city. Much of this was accomplished within seven months, after which coordination was turned back to the republic's Council of Ministers. At that time, 50,000 workers from other republics began to clear construction sites and initiate some rebuilding.

However, the formal reconstruction effort was not sustained even throughout the rest of that first year (Mader and Tyler, 1991), in the face of hostilities with Azerbaijan, and a variety of other barriers to implementing the plans, such as: delay in converting the agricultural lands that were to be used for urban expansion, delays in engineering the construction sites; a lack of building materials; a lack of technical background in the necessary construction techniques; and the inability to develop an efficient way to organize construction. Thus, the capability to plan and design far exceeded the capacity of the central and local governments to launch and sustain an effort to implement plans to quickly provide new permanent housing. And it seems likely that, in the face of severe political and economic problems, the objective of seismic safety receded for whatever reconstruction has been possible in the past five years.

Implications of Reconstructing the City to be as Familiar as Possible

When the policy is to make decisions based on the goal of retaining the layout and character of much of the city, the propositions outlined in Table 1 suggest the consequences to housing recovery, in terms of the four criteria outlined above, will be generally as follows. Housing recovery will likely be relatively fast unless there is considerable controversy over what can or ought to be saved and what cannot. Since the layout of the city, and the style of buildings is going to be very similar to that before the earthquake, little data collection and planning is necessary. The extent of choice of types of housing may basically be the same as that prior to the earthquake. Most of the displaced will find themselves eventually back in their previous housing and neighborhood, for better or for worse. If portions of the population had come to think of their pre-quake housing as substandard and outdated, then controversy may arise about how best to use this sudden demolition of housing to engage in extensive urban renewal or at least housing modernization. If much of the housing must be totally rebuilt, and inflation has affected the costs of land and construction in recent years, many owners won't be able to afford to replace their houses without substantial financial relief. If restrictions are placed on how much rents can be raised, then developers will have to be given incentives to supply rental housing for lower income families. If the city is pushing for retention of its former layout and building style, seismic safety is the most likely criterion to get ignored in the process. A familiar city means that hazardous land uses may not be avoided, and knowledge about more seismic resistant housing styles and materials will be ignored.

Many examples probably can be found in seismically active areas like Latin America and Italy where destroyed cities were rebuilt with much the same form and housing as before the earthquake. However, instances of badly damaged cities basically being rebuilt as they were are probably less common now than they were prior to the mid-twentieth century, when data began to accumulate on earthquake patterns and on how to design and build more seismically resistant structures.

Friuli. The 1976 earthquakes in the Friuli region provide an example of an area in which there was a high value placed on retaining the character of the towns (Mader and Tyler, 1991). These towns had suffered extensive damage in 1928, and been rebuilt. In 1975 the Italian government had approved a law for anti-seismic construction, and information had been prepared on construction techniques. When the May, 1976, earthquake occurred, the associations for design professionals advocated the use of construction that would reduce earthquake risks. Financial assistance was made available for small repairs, and the less damaged towns quickly moved to make their damaged housing habitable. The provision of financial assistance had been expanded to help repair the infrastructure and industries of the more badly damaged areas but the September, 1976, earthquake displaced several thousand more people and the need for replacement of housing became even more urgent. Controversy arose over the probable consequences to the traditional urban form of a May 1976 zoning law, and a mechanism was established for providing a high degree of participation of the residents in the design of the replacement housing projects. The outcome in several of the towns suggests that the criteria for good housing recovery were retention of historic land use patterns and neighborhood character, but internally modernized homes. Seismic safety had nearly equal footing in the outcome, although this was most likely more of a criterion for the government and the professionals than it was for the home owners.

Implications of Reconstruction to Improve the City's Amenities

When the policy is to make decisions based on the goal of making the city more livable and enhancing urban amenities, the propositions outlined in Table 1 suggest the consequences to housing recovery, in terms of the four criteria outlined above, will be generally as follows. The development of permanent replacement housing will likely be delayed while a master plan is prepared for the future new and improved city. If housing loss has been substantial and there is little capacity in the region to absorb the displaced into other housing, temporary housing projects are likely to undertaken, which if they become permanent will not be consistent with the intent of the master planning. For those neighborhoods that are displaced or greatly altered in exchange for such amenities as wider streets, more open spaces, single use zones, and grand public use facilities, the former residents are very likely to be displeased with the alternatives offered them, at least until they establish new social networks and mobility patterns and become used to their new surroundings. If the replacement of housing in this enhanced city is totally provided by private developers and driven by market forces, many households are likely to find that the costs of housing have increased considerably. It will probably be necessary for various types of subsidies to be applied if the new replacement housing, rental or otherwise, is to be affordable for those in the lower income groups, especially in the short-run. If the policy is to emphasize the achievement of a better, more livable city, it is likely that better seismic safety will also be included as part of the criteria for good redevelopment, including residential areas. If a large proportion of the city was relatively undamaged, planning controversies may emerge about how to meld the old with the new, and what to do about the less seismically resistant buildings that survived this earthquake, but may not survive a larger or different type earthquake.

Skopje, Yugoslavia, and Managua, Nicaragua, are examples of cities where the government decided that the opportunity created by the destroyed city should be seized in order to upgrade and enhance the city's infrastructure and amenities. Skopje's relatively clear-cut reconstruction policy placed a high value on rapid housing replacement. Managua's central policy included plans to redesign the urban center and relocate many commercial functions, although for the most part a truly clear picture of the policy was obscured by local politics; in general it did not seem to emphasize the immediate needs of the majority of the residents, and in particular, those of limited means.

Managua. Nicaragua's capital city of Managua is an example of a reconstruction process in which master planning for the city to be reconstructed was integrated with regional planning in order to decentralize the city's population and reduce the densities of population and facilities vulnerable to earthquakes (Mader and Tyler, 1991). The three-phased planning process was designed and carried out with considerable assistance of foreign experts, initially unfamiliar with Managua, because of the lack of trained planning specialists in Nicaragua. The most timely aspect of this assistance was that provided by Mexican planners in quickly setting forth standards for repair and rebuilding. This allowed legal building to begin quickly, but resources and local expertise to aggressively enforce the standards were lacking, so the standards may not bring about the degree of improved seismic safety hoped for. Other reconstruction assistance flowed in from the international community, but extensive corruption with respect to its distribution limited its effectiveness in the reconstruction effort.

The moderate earthquake in December of 1972 had destroyed about 75% of the housing units of Managua, as well as over 80% of the smaller manufacturing and retail establishments, and a large amount of public facilities and infrastructure. Although 10,000 units of simple temporary housing for low income families were built out on the eastern periphery within months with international assistance funds, an estimated 53,000 units of housing had been lost in the earthquake. The huge dreary plot of wooden shed-like structures lacked infrastructure, including bus routes that would enable the residents to go to their shopping and go to work in other parts of the city. This temporary housing remained mostly unused for several years, until programs were carried out to upgrade it into more attractive permanent housing for the working class families of Managua (Haas, *et al.*, 1977). In lieu of timely governmental planning decisions about the lay out of the city, individual residents and private investors proceeded to re-establish homes and businesses as best they could, on land beyond the original central city boundary where a moratorium had been placed on re-building and outside the adjoining band of land largely controlled by the ruling Somoza family. While laissez faire efforts got the city back on its feet within about three years, the result was not the orderly and beautiful city envisioned in the master planning effort; many families remained dissatisfied with the unwieldy city, and in particular with their post-earthquake housing and location (Haas, *et al.*, 1977). After the 1979 revolution, which brought about further damage and disruption to the city, the formal plans to decentralize and rebuild the central city finally began to be implemented.

Skopje. Compared to the planning outcomes for Managua, the early decisions in Skopje about the priority to be placed on housing replacement, and the high level of assistance available did permit Skopje to achieve relatively rapid and safer results for housing replacement (Mader and Tyler, 1991). The moderate, but shallow focus, earthquake of July 1963 that struck very close to the city seriously damaged or destroyed about 50% of the buildings in Skopje. Seventy-five percent of the housing, approximately 20,000 units, were destroyed or badly damaged. A 1948 building code that might have prevented much of the damage had been mostly ignored in local practice. President Tito saw the earthquake as an opportunity to make Skopje a showcase city for his socialist regime. An uncommonly high amount of assistance was available for the reconstruction, as east and west block nations vied with each other to contribute. Yugoslavian brotherhood was invoked and labor and financial assistance were called in from each of the other provinces. The traditional master plan for gradual change that had been revised in 1962 was discarded and planning more appropriate to extensive rebuilding was undertaken.

Because of the magnitude of the housing problem, housing recovery took a central place in the overall reconstruction program, and thus did not get ignored by planners intent on creating a totally new city. The city authorized the repair of the less serious damage to take place without obtaining a permit, and tens of thousands of units were quickly repaired and reoccupied. This helped with rapid re-housing, but may have compromised safety to some extent. Building efforts in the first two years following the earthquake were focused on replacing the housing lost in the earthquake. An important element in being able to work quickly on housing recovery was that this housing replacement stage did not involve extensive formal planning; the formal planning was to focus on the re-design of the central city and was completed in about four years after the earthquake. Although the city did not extensively plan the layout of the new housing, it did quickly complete seismic zonation studies and use these as a basis to limit building in some areas, and guide the type of building appropriate to certain sites, based on the soils information.

In the first year several areas of low density temporary housing, often of a pre-fab design, were put in place out beyond the central city. This housing not only became permanent, but in some instances, is the preferred housing for privileged families. Much of the new permanent housing that was provided within the first year represented a style not previously used in the city, that of block-style multi-story apartments, but spaced so as to maintain low urban density in this known hazardous location. There was no requirement that the families occupying this new housing had to have lost their previous housing to the earthquake, making this more a type of housing renewal program for the city, rather than strictly disaster reconstruction. The population more than tripled in the next 25 years, and the more "modernized" city, unfortunately, is also characterized by urban sprawl and traffic congestion.

Implications of Reconstruction Focused on Reducing Vulnerability to Earthquakes

When the policy is to make decisions based on the goal of reducing vulnerability to earthquakes, the propositions outlined in Table 1 suggest that the consequences to housing recovery, in terms of the four criteria outlined above, will be generally as follows. The re-establishment of adequate amounts of permanent housing is likely to be delayed if a heavy emphasis is being placed on future seismic safety for the destroyed city. Housing safety rests both with housing design and construction practices, and with the location of the housing. The type of structural weaknesses that lent themselves to the extensive damage can be more quickly identified and understood than often is the case with the seismicity or geology of the region. Extensive studies may be necessary in order to understand the tectonic and sub-soil environment of the region and then apply it to land use policy. On the other hand, arriving at agreement among engineers, developers, and city officials on the minimum construction standards that will be prudent to adopt for repair and rebuilding can also be time-consuming. It is possible that some types of housing construction, or specific housing locations that were preferred by or familiar to many of the residents may be prohibited when new building policies are established based on study findings about the vulnerabilities exposed in the past earthquake. The residents of the city may not like the unfamiliar new options provided them in the reconstruction unless they have been provided some involvement in the design of new neighborhoods and replacement housing. The most common reason for post-earthquake housing to exceed the cost of former housing, especially in housing for low income families, is the fact that earthquakes often damage old, already depreciated, housing which must be replaced at current costs. Whether or not that is the case for the housing destroyed, altering construction design, or changing the locations or site preparations of residential areas may add some cost to residential reconstruction, compared to traditional pre-quake practices. Getting developers to replace multi-unit rental buildings may entail some type of subsidy if sustainable rents are to be retained for lower income residents. Housing also will be safer. When the reconstruction planning is emphasizing seismic safety, it is likely that both unsafe areas and inappropriate building design also will be identified with respect to housing replacement. It is likely that repair and building standards will be adopted that will provide better seismic resistance for permanent housing. However, if the planning and study take too long, individuals are apt to go forward with their own repairs or rebuilding, without attention to seismic concerns.

El Asnam. The earthquake in October of 1980 that heavily damaged the Algerian city of El Asnam (since renamed Ech Chelif) occurred just 26 years after another earthquake had also destroyed the city. This history and the four other earthquakes in the 100 years before that prompted officials to emphasize seismic safety for the city to be reconstructed following the 1980 earthquake (Mader and Tyler, 1991). When the city was rebuilt in 1954 the geological characteristics of the area were generally ignored, but new building code standards with anti-seismic features were adopted. The 1980 earthquake made it plain that both better land use planning and even better building standards should to be applied if the city was to be rebuilt at this location.

A conference of international experts quickly convened by the Algerian government set forth several recommendations for what ought to be done in the short run, such as get temporary housing built, and in the long run, such as improve building standards and prepare a new urban plan based on geological and seismic studies. The entire population was evacuated to a tent city, sites were prepared, and construction of pre-fab temporary housing begun about 8 months after the earthquake was completed within about 15 months (i.e., two years after the earthquake). Geologic and seismic studies began in 1983 to provide an assessment of the earthquake potential and provide detailed evaluation and mapping of earthquake hazards. Local authorities prohibited any rebuilding in the damaged area until these studies were completed. In 1987, the national government completed plans for the reconstruction of El Asnam and reconstruction began, scheduled to be completed in 1992. Repair of the repairable houses was completed in 1990, and most other displaced families had housing by that time, by virtue of having purchased their prefab "temporary" house from the government. Fortunately, the seismic studies indicated that the locations of the temporary housing projects selected right after the earthquake were suitable for housing. Had they not been, despite 10 years of study and planning, the city may still have had a high proportion of its housing at risk to extensive damage.

PRE-EARTHQUAKE RECONSTRUCTION PLANNING

It is not possible to walk around an earthquake ravaged city such as Managua, or Mexico City, or Kobe City, where thousands of people have been killed while in their homes, and tens of thousands of families have been displaced from their residences, to reflect on how such catastrophes can be avoided. We have grown to recognize that the technology for providing seismic safety to individual houses is not difficult to implement, and although there have been some hard lessons with respect to large apartment buildings, the melding of existing information on the behavior of soils and buildings during earthquakes should be able to reduce the number of future failures.

Unfortunately, earthquake loss reduction is not simply a technical issue. Applications of our new knowledge have passed the test of some major earthquakes in recent decades. However, many political, economic, social, and cultural factors interact even in today's world, to prevent the application of our knowledge to construction being carried out in cities, towns, and villages around the world. Furthermore, and most seriously, even if from here on out, every new urban structure incorporated the lessons to date about siting and engineering design, disasters like that in Kobe, Japan, are waiting to happen. Large proportions of urban areas have already been built in the decades and even centuries before some of the variables were understood, or before there was much emphasis on applying what was known even at that time. Or successful (even if inadvertent), techniques for mitigating damage or loss of life have been given up for new building styles emphasizing some other value.

It is not plausible to think that very many cities, whatever their governmental philosophy, are likely to commit the resources necessary to systematically replace or strengthen large inventories of vulnerable older buildings in which many of the city's families and economic activities are housed. Regrettably, this means that we can expect to have to address the issues of large scale post-earthquake reconstruction efforts many more times.

Preparing for Reconstruction

Emergency response professionals have long understood that identifying and characterizing likely disrupting events and preparing to respond to them makes good sense. A well designed and implemented emergency management system that integrates the functions and capabilities of many agencies and organizations can save lives and shorten the time that a city exists in a highly disrupted state following a damaging earthquake. Financial, regulatory, and social welfare officials in cities acknowledged to be at major risk from earthquakes could well benefit by adhering to a similar philosophy with respect to long term recovery and reconstruction. As indicated above, studies of recovery and reconstruction have identified the types of problems that must be solved and the issues that must be resolved in the post-disaster setting. The city of Los Angeles, California, is an example of professionals and government officials deciding to engage in pre-earthquake planning for post-earthquake recovery (Spangle, 1987). This sprawling urban area has not yet suffered a truly catastrophic earthquake but the Northridge earthquake in January 1994 presented serious recovery challenges and was able to immediately implement the existing recovery plan for how agencies will work together (Earthquake Engineering Research Institute, 1994). An aggressive program is ongoing to combine both regular and disaster housing financing programs, with a major goal of development of replacement housing that is affordable for these families (Los Angeles Housing Department, 1995).

The examples provided here and in other case studies make it apparent that reconstruction also is not just a construction problem. The speed and type of reconstruction rests on the way in which the political, economic, and social variables are configured in the post-disaster setting. Choices have to be made that are value-based, and resources allotted. These choices may have to be re-negotiated at some point in the process, if some segment of the population believes the process is going to leave them worse off than before. Organizational and financial as well as technology mechanisms have to be put in place that make it possible to meet the goals. Slow resolution of housing losses prolongs the human suffering and stresses created by the earthquake, and can inhibit the productive capacity of the population for a long period. The frequently noted tendency to place band-aids on the housing problems rather than to work out innovative ways to use the opportunity as a development as well as a reconstruction activity needs to be addressed.

The Mexico City program for replacing housing in the central district indicates what can be done if the will exists to act aggressively and courageously to take major short term risks in policy and financing in order to ensure a better and less costly long term future (Mader and Tyler, 1991; Applied Technology Council, 1991). In that case, a very densely settled, but also somewhat dilapidated part of the city suffered most of the damage. The residents strongly opposed a general program to simply relocate them to a new district beyond the city. The leaders decided that it would be possible, and even preferable, to use this as an opportunity to revitalize this historic but run-down section of Mexico City. The World Bank helped them design a program that would not only replace the housing, but incorporate the ideas of the displaced residents, and result in the new units being owned by the residents rather than be rental units. The Bank also insisted on attention to seismic safety. The process involved aggressive actions including expropriation of the land with the damaged housing, extensive subsidies to assist the families in buying the units, and a new philosophy and complete re-design and streamlining of administrative procedures that would permit building permits to be processed rapidly, and for payments to suppliers and various reimbursements to be handled quickly. The displaced residents cooperated in the process by suffering many months of hardship living in minimal shacks and sanitary facilities erected in the medians and parks of the area being redeveloped. This enabled the residents to stay in their neighborhood, close to their businesses and schools, and this temporary housing was happily torn down.

Case descriptions of reconstruction efforts identify both innovative solutions and several types of general problems that will have to be solved during the reconstruction period that also lend themselves to pre-earthquake planning in cities with a major earthquake risk. It is easier to achieve a necessary degree of consensus about solutions to specific issues when the possibility of having to apply the mechanism seems remote. When certain types of policies about the use of resources and appropriate standards are developed and set prior to the earthquake, decision making following the disaster probably can be shortened since reconstruction becomes more of an implementation activity than a policy battleground. Many other problems also have been identified (Haas, *et al.*, 1977, Mader and Tyler, 1991; Applied Technology Council, 1991; Spangle, 1987), but five fairly general ones can be suggested for attention in planning and preparing for reconstruction prior to an inevitable major earthquake disaster.

Developing Land Use Mechanisms. Cities tend to at least double, if not more, the amount of land seemingly required to replace the equivalent pre-earthquake functions. A strategy for altering land uses needs to be developed, with such elements as streamlined land acquisition, withdrawal, and compensation mechanisms, safe storage of the inventory of information necessary to take action on land use decisions, designation of institutional responsibility for carrying out the actions, with these things developed with adequate involvement of citizens and agencies to legitimate the use of the mechanisms without further discussion after the earthquake.

Identifying Financing Options. The increasing density of urban settlements has lead to each major earthquake disaster representing a far greater amount of economic loss to the community than the last. Options for financing reconstruction, including housing recovery, need to be considered and analyzed. It has become increasingly necessary to reach beyond the city's and often even the country's resources to get the necessary level of financial assistance to permit rapid expenditures for demolition, debris disposal, repair, planning, and construction of the infrastructure, and to meet housing needs. The city needs to have a good understanding of all local, national, and international options, pre-established authority for committing to options, and an agency or group prepared to consider innovative ways to combine traditional emergency response and traditional development mechanisms.

Establishing Repair Standards. Often there are large amounts of dwellings and other structures that could be quickly put back in service if repairs can proceed swiftly. However, it is also important to regulate this activity in order that it also achieve the longer term goal of seismic safety as well as rapid recovery. The development of building standards, and likewise, of repair standards, typically necessitate a fairly prolonged exchange of information and opinions by experts. If this issue cannot be resolved quickly and repair work is prohibited in the meantime, clandestine and often inappropriate repairs will far outstrip those done once the program is finally established. The type of risks can to be delineated based on experience from other earthquakes and knowledge about the local hazards, policy decisions need to be made for how strict standards need to be and what sanctions will be applied for not following them, and the authority and tools for implementing the repair program established ahead of time, so it can begin as soon as possible after the earthquake.

Streamlining Administrative Procedures. Even when policies for what to build, where to build, how to pay for it, and who to have do it have been taken care of, it then becomes necessary for the city's operational and regulatory agencies to implement the decisions. The normal procedures for activities such as processing building permits or dispersing payments often have not been assessed and updated in recent decades and long delays in accomplishing such tasks has come to be accepted, even if not appreciated, by those served by the various agencies. After a disaster, not only will many more simultaneous demands be placed on most city agencies, all will be more viewed as much more urgent than normal. The notion of changing the "business as usual" mentality, can well be addressed at any time in most cities, and greater efficiencies achieved both before and after the next disaster through thoughtful analysis of requirements, procedures, and organizational paths. These procedures can then be even further expedited in the post-earthquake setting, by extending the normal hours of service.

Delineating Long Term Housing Needs. Pre-earthquake preparedness for implementing reconstruction decisions and processes such as the above all will contribute significantly to the recovery of housing, by making it possible to move swiftly to repair damaged housing, or site and build replacement housing. However, the urgency associated with getting displaced families rehoused in the short run has been shown to have disappointing or hazardous long term consequences. Consideration of how to move as quickly and effectively toward providing permanent housing for the long term can be done within the context of the larger housing issues facing the city even without an earthquake. Supplying healthful and affordable housing for low income populations is a problem in most cities of the world that are experiencing significant growth. Since this problem is often managed through the continuing use of older, substandard, and often not seismically resistant, structures, the problem of housing for low income families is greatly exacerbated by earthquakes and the most difficult one to solve in the aftermath. The earthquake often will resolve the problem of what to do with the substandard housing, and may help bring expanded resources to bear on the problem of housing. Cities that have specific planning goals for redeveloping their substandard low income housing and improving housing opportunities for lower income families, can move quickly to accomplish much toward this goal by focusing attention and resources on permanent housing programs for this population. Given a high degree of certainty of what type of housing will be built, where it will be built, and when it will be available makes it possible for families to abide considerable inconvenience in the short term in order to have a better long term situation.

Conclusion

Certainly pre-earthquake planning and preparedness for reconstruction won't necessarily result in the ability to meet all the problems that emerge, nor will it make it possible to eliminate large amounts of disruption and stress in the lives of the families most affected by the earthquake. Political factors can affect the will to execute seemingly rational policies, economic factors can affect the capability to totally address problems, and social factors, such as class or ethnic antagonism, may make it impossible for leaders to get "permission" to move forward with policies and procedures that seem best suited for achieving a functioning and livable city within a reasonable amount of time after a major earthquake. However, it is important to learn from past reconstruction efforts, to think clearly about the vision for the city, its people, and its character, and to understand what must be done to minimize the consequences of valuing one option for reconstruction over another. It also is important that the increasingly urgent problem in major urban areas, that of adequate housing for families with modest or low incomes, not be ignored when housing recovery programs are adopted following major earthquakes. Post-earthquake housing reconstruction goals and routine housing development and re-development goals also can be integrated in reconstruction planning.

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INQUIRY INTO RECONSTRUCTION OF CHANGED PLACE IN WUQIA COUNTY TOWN

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ABSTRACT

In this paper, the decision to rebuild an earthquake-damaged town in a new location is examined. Factors influencing this major decision are examined. It is suggested that some important considerations were ignored.

KEYWORDS

Earthquake; rehabilitation; reconstruction; location; engineering; Wuqia.

INTRODUCTION

At 20:41 minutes on August 23rd of 1985, a strong earthquake measuring 7.4 on the Richter scale struck Wuqia county in Xinjiang with its epicenter being located at north latitude 39°33' and east longitude 75°06'. The quake, whose seismic focus was 18 -- 20 kilometers deep, left 20 people dead, 100 or more injured, 70% of the town's houses collapsed and 2,317 families victimized, with economic loss totaling up to 62,680,000 RMB yuan.

After scientific inquiry, it was decided to set up a new town in another place because the old county seat was located at the confluence of fault zones. A "safety island" which is 6 kilometers away from the old town was chosen as the location of the new one. A total sum of 55,362,800 RMB yuan has been put into its rehabilitation and reconstruction. After three years' effort, the completed projects in the new Wuqia county seat were inspected and approved by the government with their relatively full set of functions. Our inquiry into the experience of reconstruction led us to certain observations.

First, the new county seat lies only 6 kilometers away from the old one. There was a question of the coefficient for safety in stronger earthquakes.

Second, afforestation belts, roads, water supply system and other civic facilities in the old town could no longer be utilized.

Third, the soil is thin, which makes afforestation difficult.

Fourth, after making a further geological survey of the site for the new town seat, we found that a belt of Kata-rock with a width of 50--80 meters goes through the foundation, and a soft intercalary strata with a width of 150 --200 meters lies just beneath surface soil near the center of town. In a word, we should proceed with caution and make a comparative study for a scientific basis to do post-earthquake reconstruction in another place.

ASPECTS OF RECONSTRUCTION

As noted, at 20:41 minutes on August 23rd of 1985, a strong earthquake measuring 7.4 on the Richter scale struck Wuqia county in Xinjiang. Its epicenter was located at latitude 39.33° north and longitude 75.6° east, the depth of seismic focus was 18 or 20 kilometers and aftershocks lasted 129 days. At 4:45 minutes, on the morning of September 12th, another strong aftershock occurred at 6.8 on the Richter scale. It was a shallow-focus tectonic earthquake with a sequence of fore-shock, principal earthquake and after-shock. Wuqia earthquake is one of the largest quakes since the 1976 Tangshan earthquake. Before the quake, no prediction was available.

Since Wuqia county is located in a high-intensity zone as well as the intersection of a fracture, heavy damage was caused by the disaster. All the adobe houses fell down, which make up 70% of total housing in the county; brick and concrete houses suffered damage to various degrees except for three which suffered only minor damage. At the county seat, 20 people died and 100 or more were injured; disaster-stricken households totaled 2,317 and economic loss amounted to 62,680,000 RMB yuan.

After the event, we received generous aid from all parts of the country. Government at top levels made careful arrangements for anti-quake measures and disaster prevention in Wuqia county. A new town was built in another place so that the locals might live and work in peace as soon as possible. Total funds for construction reached 55,362,800 RMB yuan, of which 49,350,000 yuan were allocated by the state. Under the leadership and concern of government at all levels in the disaster area, people from different ethnic groups, in the past three years, worked hard in a joint efforts. They overcame one difficulty after another and completed 10 large projects and 79 sub-projects with floor space of more than 120,000 square meters, namely: one downtown area, inclusive of administration, commerce, culture and other public works; three relatively concentrated sub-central areas, inclusive of three schools, one hospital with 100 beds, water works, boilers' house and other civil engineering projects. In addition to those mentioned above, the list includes: 7 kms of trunk highway and secondary highway, 30.1 kms of water-supply pipeline, 25.66 kms of water-discharge pipe, 6.24 kms of trunk or sub-pipe for heating, 3 wells as a water source, 1 clear-water pool, septic tank, 8 kms of electrical line with 10 kw, a 1,000 kw central power transformation, 500 program-controlled telephone switchboards, 760,000 square meters of shelter forest belt (one times more than planned). The town green area is 53,300 square meters. Places near the intersections of downtown roads are decorated with shrubs, flowers, lawns, monuments and sculptures, which look splendid and colorful. Buildings in the downtown area exemplify local style.

The new town of Wuqia better reflects the intention of a general plan with its relatively full set of functions as well as beautiful surroundings which accommodate the people well in both work and recreation. This town, with not only national style but also construction standards of the 1980s, has come into being. Situated in our most western prefecture, Kezilesu Autonomous Prefecture, it will benefit the stability and unity of this multi-ethnic region as well as the economy.

In reconstruction work for Wuqia county seat, we have made an achievement and gained experience. However, we still have lessons to learn. As for reconstruction, the authors advance their perspective as follows for others to probe.

After the quake struck Wuqia, there arose a controversial, comprehensive, scientific and technical problem as to relocation. Based on the data at hand, one of six sites would be chosen as the site of a new town.

No. 1 is more than one kilometer east of location No. 2 and will face the danger of flood, for the Kuzigun River is runs through it.

No. 2 lies 6 kilometers away from the old county seat. Yet its geological formation is favorable since it is surrounded by the Kezile Fault, the Canergan Fault and the Wuqia Fault which form a relatively stable down-warping plate ("safety island") with an area of 6 by 8 square kilometers. As part of the Gobi Desert, this location is good in terms of engineering geology. Bearing capacity of the ground is 30 T/m^2. There is less threat from flood. Underground water level stays at some 70 meters with the characteristics of larger yield, stable flow and good quality. The site occupies no cropland and stands close to the highways with an incline of 2 -- 2.3% for easy access to transportation.

No. 3 and No.5 are located in a fault zone, thus facing the danger of a quake; furthermore, location No. 5 has traffic interference, for it straddles a highway.

No.4 is quite close to a fault zone, which makes the location vulnerable to earthquakes. Even worse, it is near a cement factory and a limestone pit, with heavily polluted air and loud noise.

No. 6 is the original place. Where the old town stands is at the juncture of several fault zones. Besides this, other factors adversely affect this location, such as poor conditions in both geology and engineering (10 T/m^2), higher subterranean water level, quicksand and shock liquefication phenomenon. All these make the place subject to more danger from a quake. Furthermore, it suffers every year from varying degrees of floods because of lowlands.

After close comparison, location No. 2 became the site for the new town. In the opinion of the authors, there are drawbacks to the new location.

Fault Zone

Wuqia, an active earthquake zone in Xinjiang whose seismic geological formation is complex, has witnessed quakes measuring more than 5 on the Richter scale some 36 times since 1900. Of these, an intensity of 7 -- 7.4 on the Richter scale happened three times, and an intensity of 6-- 6.9 on the Richter scale happened 6 times. Earthquakes used to strike the area at latitude 38 30' -- 40 30' north and longitude 74 30' -- 76 30' east, which feature a higher rate of earthquakes and stronger intensity. The site for the new town has not avoided this range.

In 1991, the sub-institute of chemistry and physics under the first survey institute of Xinjiang Uighur Autonomous Regional Geology and Mineral Resource Bureau was entrusted by Kezilesu Autonomous Prefectural Construction Bureau and the Construction Bureau in Wuqia County to make a key-point exploration to a shallow strata earthquake in the area planned for the new town. They discovered a hidden fault zone in the southern part of the town with its trend approaching east-west and inclining south. This crushed fault zone with a width of 50 --80 meters cuts through a quaternary geological formation, which belongs to an active fault. In addition to that, they found a soft interbed in shallow strata below the surface near the downtown area, which runs northwest to southeast with a width of 150 -- 200 meters. Furthermore, properties of the rocky soil are quite uneven in the sectional subterranean shallow strata at the new site. Thus the so-called "safety island" is not quite safe.

Geological Engineering Matters

Brick and concrete buildings with anti-seismic features in the old town include: cinema, office building of grain bureau, hotel, commercial and industrial bank office building and other houses which suffered little or no damage. Adobe houses which collapsed were mostly built in the 1950s or 1960s with poor-quality construction and no anti-seismic features. Geological engineering conditions in the original location are a bit worse than in the new town. This matter would have been dealt with if we had paid enough attention to anti-seismic features and improved the quality of construction accordingly to design the main structures.

Threat of Flood

Land formation of Wuqia county is characterized by tilt from north to south and the Kuzigun River from the northern valley flowing south. Location No. 2 and No. 6 are both located on the western side of the river. When flood water tops its discharge capacity, both locations face danger. Anti-flood works are now installed in the new town while there were none in the old town. Prevention of flood simply means building the necessary facilities in the old town.

Overall, we think the new town should have been built on the original site.

(1) The old town lies 6 kilometers away from the new one. In the event of stronger earthquakes, what distance is the safety co-efficient?

(2) Afforestation, roads, water supply and other civic facilities in the old town could still be utilized and thus reduce construction expense on civic works and lighten the burden on the state. (Ten civic works projects cost 10,570,100 RMB yuan for the new town).

(3) Vacant houses, which are neglected nowadays, can still be used in the old town. It is sad to desert those houses and do nothing with them.

(4) The new town is located on the alluvial fanlike Gobi Desert formed by the Kuzigun River. There the soil is thin and conditions for afforestation are poor. But the old town is located in a place where woods are green and lush, natural scenery is wonderful, and sandstorms are fewer.

Mr. Wang Enmao, a former vice-chairman of CPPCC once pointed out: "As for the choice of location for the new town of Wuqia, we must listen to the ideas and opinions of planners and surveyors as well as the inhabitants of Wuqia county. If the old town seat has passed the tests, a new town will be established in the same place; if another better place is found, a new town will be built there ". Choice of location for the new town bears much importance. We must have basic reliable first-hand data. However, no thorough geological exploration was made at the same time. So the decision to relocate for reconstruction was blind, coming from scant scientific evidence and rough economic analysis. Generally speaking, on a piece of white paper we can paint a beautiful picture, which just reflects our train of thought on reconstruction of a new town in a different place. Consider an analogy. If we want to be handsome, we'd rather buy the latest style than patch an old coat. Reconstruction in this case did not adapt to the circumstances of the state, of our autonomous region or of our county.

Our summary of the experience and lessons from reconstruction of a new Wuqia County town may leave the reader with something to consider.

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DISCUSSION OF A SYSTEMATIC ENGINEERING STUDY FOR PLANNING AND REBUILDING AFTER AN EARTHQUAKE DISASTER IN DAQING OIL FIELD

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ABSTRACT

In this paper a study on the plan to recover and rebuild after an earthquake disaster in Daqing oil field is discussed from the angle of system science and whole analysis. Important points in the study are given and constructive suggestions made. It is pointed out that recovering and rebuilding follows a disaster while planning should precede a disaster.

KEYWORDS

Earthquake disaster; recovery; rebuild; system science; oilfield; expert system; probability; Daqing oil field.

INTRODUCTION

Daqing oil field is the most important base of petroleum and natural gas in our country and in a vital position in the development of the national economy. Constructed thirty years ago, 25 oil fields have been explored. Crude oil production is $5{,}535 \times 10^4$t / year; natural gas production is 25.4×10^8 m^3 / year. Daqing is an industrial city which combines traffic, electric and water power, communication, chemical industry, machinery, business and trade and about a million inhabitants. Although the zoning seismic intensity of Daqing area is 6, it is probable that a violent earthquake would occur in the area with a zoning intensity of 6. Tangshan city is an evident example. Considering randomization and damage from a large earthquake, a study on the planning of recovering and rebuilding for Daqing oil field is a problem with historic and current significance, involving the state plan and people's lives.

STUDY ON PLANNING WITH HELP OF SYSTEM SCIENCE

Earthquake disaster recovering and rebuilding is a complex system of engineering concerned with politics, economics, inhabitants' life and safety. However, Daqing oil field is a system composed of its geographical position, population, traffic, communication, electric and water power, transportation of oil and gas, business and trade, medical treatment, hygiene, fire safety, inhabited buildings and various other oil installations. From the view of post-earthquake disaster recovering and rebuilding, it is a whole and also a system which is combined with many factors and elements depending on each other for existence, holding each other back. For analyzing relations between the internal and external factors, we ought to study the problem from the view of system science, seek and optimize the solution for this problem by use of scientific and technological methods. Utilizing rational manpower and material resources and optimum managing means are to achieve the goal. For this reason, we make the following considerations:

(1) Tactics for recovering and rebuilding ought to pay attention to the whole and analyze relationships between different elements.
(2) A study has to depend on science, use scientific data to illustrate reasons, have a definite objective in view and guard against possible trouble.
(3) Due to many stages, many levels, many factors, the decision must involve many goals.
(4) Planning for recovering and rebuilding is part of aseismic and disaster-resistant system engineering. The study must be combined with the planning of aseismic and disaster resistance.
(5) Recovering and rebuilding follows a disaster, and study on planning should precede a disaster.

MAIN STUDY

Study on recovering and rebuilding plan for earthquake disaster in Daqing oil field takes Daqing oil field as a system so that after an earthquake, for the capacity of economy and rescue, utilizing system science methods and optimum decisions become the goal. Thus the main study must include several aspects.

Study on methods of base material gathering and analyzing: Its purpose is to research the methods of earthquake disaster investigating and statistics, hardware measuring, dynamic state disaster forecasting, disaster zonation and levels of dynamic state division.

Study on modeling-truth system: Its goal is to study recovering and rebuilding active modeling-truth model according to policies, to forecast disaster damage level, to probe different effects on the recovering and rebuilding speed under different policies.

Study on economic policies: Its purpose is to study economic problems for recovering and rebuilding once earthquake damage occurs in Daqing oil field: for example, selection of rebuilding projects, demonstration of rebuilding programs, decision of rebuilding orders.

Study on rescue technology: Its purpose is to research rescue orders and management and methods.

Study on probability and maintainability for lifeline systems and special equipment: Its aim is to study maintainable tactics of special equipment and run probability study for lifeline engineering suffering earthquake disaster damages in Daqing oil field.

DISCUSSION OF ESTABLISHING EXPERT SYSTEM

Planning for recovering and rebuilding is system engineering which has different intentions at different times with the development of science and technology. However, combined with modern computer technology, to establish an expert system for recovering and rebuilding under earthquake disaster in the oil field is necessary. Thus, we not only get an information system for recovering and rebuilding disaster damage but also renew and increase knowledge of the system at any time to ensure continuity of the study with the development of science and technology.

Expert system frame to be considered must have the following content:

(1) Dynamic prediction of earthquake disaster losses.
(2) Disaster statistics and analysis.
(3) Expert knowledge and experience for recovering and rebuilding.
(4) Economic policy.
(5) Modeling -truth sub-system.

Central work for building an expert system of oil field recovering and rebuilding is to establish data and interrelationship library on the computer. For the expert knowledge and experience, element knowledge will be abstracted by means of summing up past criteria for recovering and rebuilding.

NECESSITY OF RUNNING PROBABILITY STUDY FOR OIL FIELD LIFELINE ENGINEERING

Power, traffic, electric and water power, communication, oil-gas gathering and transporting are among the key links to ensure oil field production and inhabitants' daily life. After an earthquake, running a probability study of lifeline systems deals with personnel for recovering and rebuilding, decisions for rescuing and society undertaking risk. Therefore, running a probability study for lifeline systems suffering earthquake damage as predicted is necessary.

The entire running probability study for lifeline systems under earthquake disaster can be expressed as:

$$R = f(x_1, x_2 \cdots x_n) \tag{1}$$

where x_i -- sub-system probability ($i = 1, 2, 3, \ldots$)

Every sub-system K can be analyzed according to series, parallel, series-parallel, parallel-series model.

$$X_k = \prod_{i=1}^{n} r_i \tag{2}$$

series:

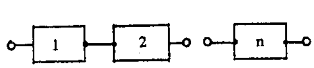

Fig. 1 Series logic frame

parallel:

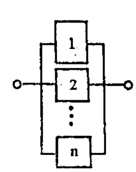

Fig. 2 Parallel logic frame

$$X_k = 1 - \prod_{i=1}^{n} (1 - r_i) \tag{3}$$

series-parallel:

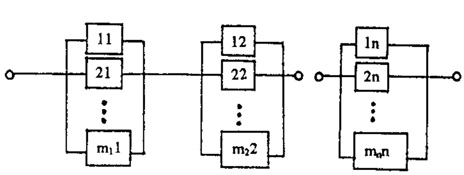

Fig. 3 Series-parallel logic frame

parallel-series:

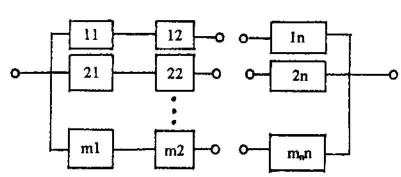

Fig. 4 Parallel-series frame

$$X_k = \prod_{j=1}^{n} [\, 1- \prod_{i=1}^{mj} (\, 1 - r_{ij}\,) \tag{4}$$

$$X_k = 1 - \prod_{i=1}^{m} [\, 1- \prod_{j=1}^{n} r_{ij}\,] \tag{5}$$

where r_i, r_{ij} -- element probability.

The probability-optimum of recovering and rebuilding for lifeline systems can be expressed as the following mode:

$$\begin{aligned} &\min \quad C_S = \sum_{j=1}^{n} C_j\,(X_j) \\ &R = f\,(\,X_1, X_2, \cdots X_n\,) \geq R_r \end{aligned} \tag{6}$$

where C_S -- total investments for lifetime systems to resume and reconstruct,
C_j -- sub-system investment for recovering and rebuilding,
R -- system probability,
R_r -- set probability for systems.

DISCUSSION OF MAINTAINING STANDARDS

If the oil field is damaged by an earthquake, the need to maintain equipment is unavoidable. For this reason, it is important that a study on the optimum policy for maintaining equipment after earthquake damage is made before an earthquake occurs. Although this study is a complex problem which is involved with the oil field production technological process, we consider that maintaining standards must be dividing in accordance with major, sub-major, and common maintenance based on necessity.

CONCLUDING REMARKS

This paper is only a discussion on planning for the recovery and rebuilding of Daqing oil field after an earthquake. In case of inadequacies, we hope experts won't spare their comments.

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1995 JANUARY 17
DEVASTATING HANSHIN- AWAJI EARTHQUAKE

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ABSTRACT

At the beginning, ground motions due to Hanshin Earthquake were explained how severe they are comparing the other ground motions due to the other recent earthquakes and the standard earthquakes such as El Centro Earthquake, in linear and non-linear response. Then the damage features of R.C. and S.R.C building structures were explained. The damage features of steel buildings were next explained. Finally the severe damage of wooden houses were explained suggesting what kind of wooden houses subjected to the serious damage.

KEY VORDS

Active fault, near field earthquake, Doppler effect, response spectrum, linear and non-linear, soft first story, damage of connection, fracture of welding at beam-column connections, heavy roof tiles with clay.

1. Introduction

The Hanshin Earthquake caused severe damage to the southern portion of the Hyogo Prefecture, particularly to the city of Kobe. The severely damaged areas are located in a narrow band extending from the northern part of the Awaji island to Takarazuka city, and which includes the cities of Kobe, Ashiya, and Nishinomiya.

The earthquake occurred at 5:46 a.m. local time on 17 January 1995. The Richter Magnitude of the earthquake was estimated to be 7.2; the focal depth was approximately 14.3 km. The epicenter was located at 34.0364 degree, North latitude 135.026 degree East longitude. Death casualties were 5502 at present(July 10 in 1995) and about 30,000 people were injured, 63245 houses and buildings were collapsed and in 498 places, fire broke out and 265 roads were closed. Financially, about $120 billions were lost.

2. Earthquake Ground Motions

(1) Map of seismic intensity

In Japan, ground motion intensities are generally assigned a numerical value according to the Japanese Shindo scale by the Japanese Meteorological Agency (JMA). For this earthquake, values of the Shindo are shown in *Figure 1*. The cities of Kobe and summits of near Mountain of Rokko were assigned a Shindo 6 intensity; the cities of Kyoto, Hikone, and Toyoka were assigned a Shindo 5 intensity. Later, the JMA reassigned the Shindo 6 to 7 for some portions of the cities of Kobe, Ashiya, Nishinomiya, and Takarazuka, and portions of the northern part of Awaji island, considering the damage sustained by these areas *(Figure 2)*

(2) Fault parameters

The solution based on the distribution of compression and extension of the initial motion became clear. It can be inferred that the local mechanism is characterized by a N229E strike, dip equal to 77 degrees, and slip of 173.

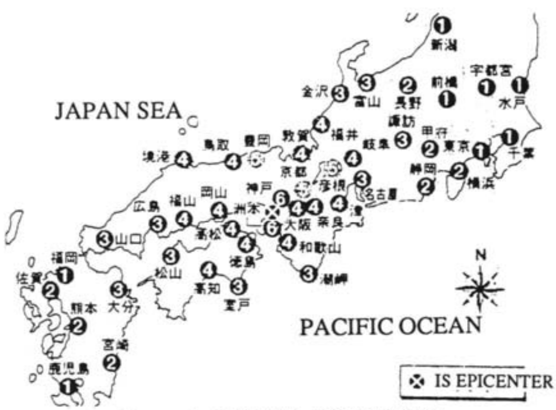

Figure.1 INITIAL INTENSITY

Prof. Kikuchi of Yokohama City University reported that a bilateral rupture had occurred with the following characteristics:

Focal mechanism: strike=233 degrees, dip=85 degrees, slip=165 Seismic moment: $M_o = 2.5 \times 10^{26}$ dyne cm

Fault area: $S = 40 \times 10 km^2$

Relative displacement: U=2.1 m

Stress drop: D=100 - 200 bar

Duration of main rupture: T=11 sec

This earthquake was supposed to consist of three sub-events; the parameters for these sub-events are shown in *Figure 3*.

Figure.2 REVISED INTENSITY

Activity of aftershocks

The distribution of aftershocks as of 9:00 a.m. local time on Jan. 27 was occurred. The surface projection of aftershock activity was along a line approximately 50 km long; this length is slightly longer than the length of 40 km estimated by Prof. Kikuchi for the main shock. Aftershocks occurred with decreasing frequency after the main shock. As of 9:00 a.m. local time on Jan. 27, eight aftershocks occurred having maximum intensities of Shindo 4 or greater.

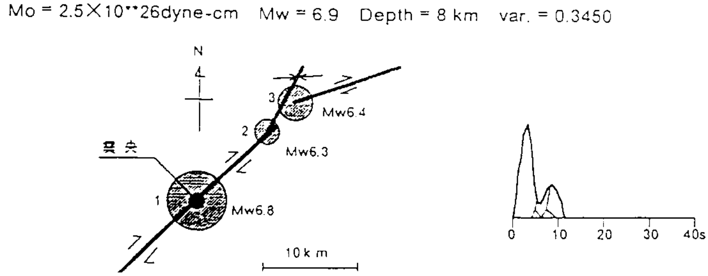

Figure 3
Fracture
of Active
Faults.

(3) Time Histories and Response Spectrum (linear and non-linear response) of the Representative Recorded Ground Motions.

In *figure 4,* recorded time histories of ground motions of three dimensions (North-south, East-west and vertical directions) at Kobe Oceanic Meteorological Agency were indicated. Although, the site is not so flat but some about 20 meters high hill, these were the one of the representative earthquake ground motions of Hanshin Earthquake. As they were indicated, peak acceleration in the north-south directions was 818 gals, 617 gals in the east-west directions and 332 gals in the vertical direction. The duration of the ground motions was quite short,of less than 15 seconds,as indicated in these time histories. The linear response spectrum of recorded ground motions at Kobe Oceanic Meteorological Agency were shown in solid line in *Figure 5* , in the north-south direction. As were shown in the same figure, the response spectrum of Tazana record-due to Northridge Earthquake at the suburbs of Los Angeles 17 Jan. 1994 and the one in the east west direction of Kushiro Meteorological Agency due to Kushiro Earthquake 15 jan. 1993. Comparing the response spectrum of Hanshin Earthquake with these spectra, Hanshin Earthquake ground motions are not so severe in linear response spectrum. At the same scale, time histories of records at Kushiro Meteorological Agency were shown in *Figure 6.* Koshiro Earthquake seems much more severe earthquake

than Hanshin Earthquake. But actually, due to Kushiro Earthquake, two casualties and quite limited damages were observed, while the peak acceleration was 910 gals greater than that of Hanshin Earthquake.

As for the vertical ground motions, the spectrum of which was shown in *Figure 7* in also solid line. Vertically, longer periods (from about 0.8 seconds to 2.0 seconds) of Hanshin Earthquake ground motions were some exceeding than the other ground motions. In a sense, intensities of vertical ground motions were about half of those in horizontal ground motions. Although the intensities of ground motions due to Hanshin Earthquake in linear response were not so severe comparing the other ones due to Kushiro Earthquake and foreign Northridge Earthquake, non-linear (Degrading Tri-linear Model suitable for R.C. structures shown in *Figures 8 and 9*) response spectra of ground motions, for instance, recorded at Kobe Oceanic Meteorological Agency due to Hanshin Earthquake were extremely severe comparing the other ones mentioned the above as is shown in *Figure 10.*

This research was performed by Professor Tadao Minami, University of Tokyo.

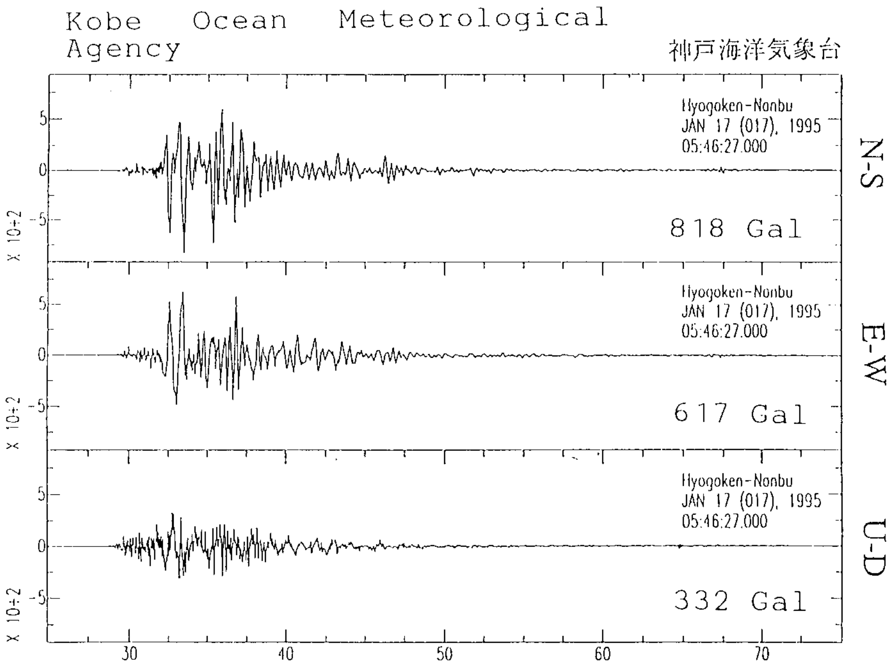

Figure 4. RECORDED TIME HISTORIES DUE TO HANSHIN EARTHQUAKE.

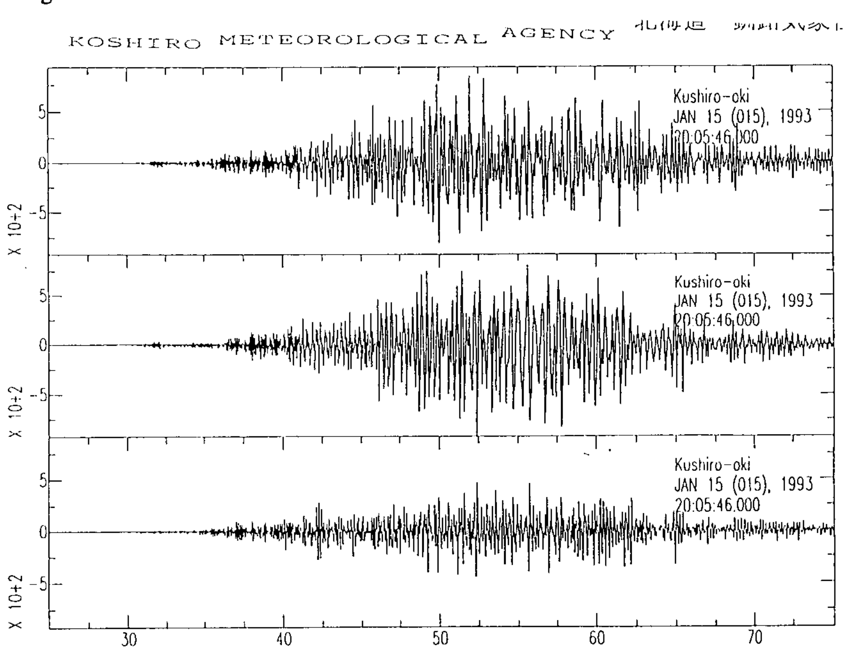

Figure 6 RECORDED TIME HISTORIES DUE TO KUSHIRO-OKI EARTHQUAKE

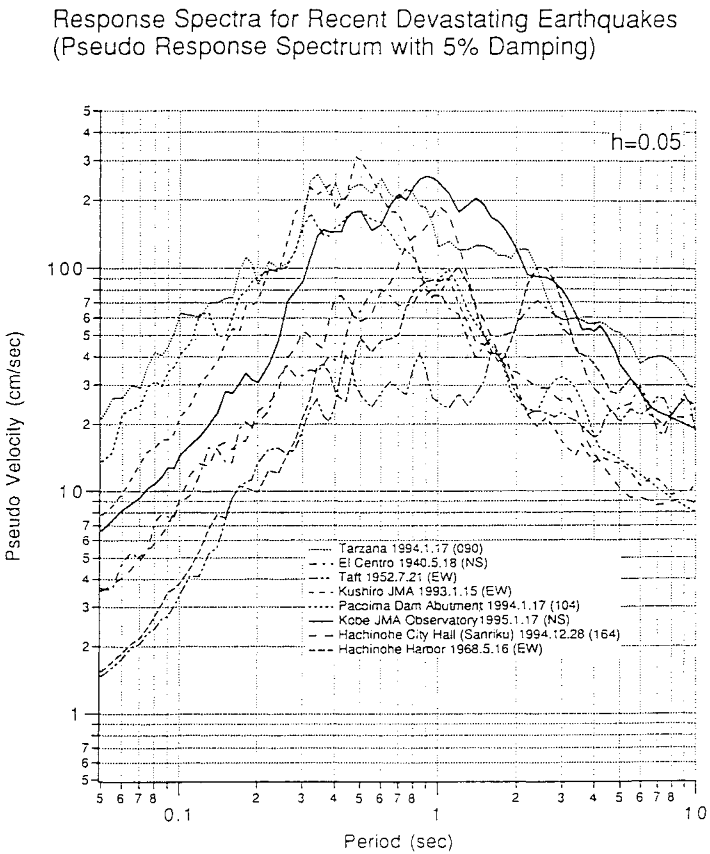

Figure 5.
RESPONSE SPECTRA of KOBE OCEANIC METEOROLOGICAL AGENCY(HORIZONTAL)and OTHERS

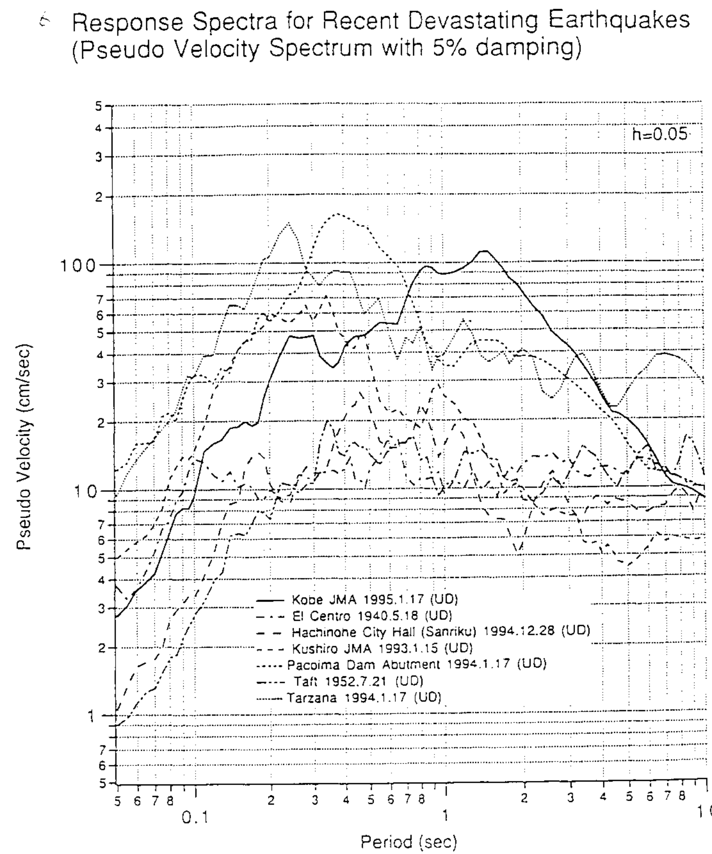

Figure 7.
RESPONSE SPECTRA of KOBE OCEANIC METEOROLOGICAL AGENCY(VERTICAL) and OTHERS.

If the base shear coefficient were to be reduced in one third, as is shown in *Figure 11,* rather small peak acceleration records at STC in Mexico City due to Mexican Earthquake 1985 became rather severer than Hanshin Earthquake ground motions.

The above facts indicate that ground motions due to Hanshin Earthquake were quite severe motions against rather strong structures to statically lateral force such as shown in *Figure 10,* for instance, base shear coefficient of 10 stories high buildings is 0.5, while those due to Mexican Earthquake were severe ones against weak structures with statically small lateral base shear coefficient. which is almost one-third of the average coefficients in Japan.

(4) Some Features of Earthquake Ground Motions As Engineering Point of View

Earthquake ground motions of Hanshin earthquake as view from buildings engineering have the following features:

(i) Quite large peak accelerations and peak velocities were recorded in wide districts as near field earthquake.

(ii) Periods between 0.8 to 2.0 seconds were especially exceeded.

(iii) Large vertical ground motions were recorded.

(iv) Duration of ground motions was quite short.

(v) Large shakings toward to north direction were observed.

(vi) Non -linear response of vibrations in soft soil areas were observed.

Also, effect of soil-structure interactions were in various districts observed. Approximately,. peak accelerations of recorded ground motions near the epicenter(within 40-50km), were about ranged 300 gals to 800 gals, and peak velocities of those were very large as more than 80 cm/sec.

3. Damage of Reinforced Concrete Structures

(1) Features of the damage

The features of the damage of reinforced concrete (R.C.) and steel-reinforced concrete (S.R.C.) structures are:

(i) The damage ratio of the structures constructed before 1971 when the requirement of hoop interval of column was revised to the half, were quite high.

(ii) The damage ratio of the structures constructed after 1971, in reverse, were quite low, especially buildings constructed after 1981 when the present seismic design codes and regulations for buildings were revised and performed, severe damage of the buildings could not be found except special structural types such as soft first story named in japan PIROTY.

The detailed damage features are:

(i) Collapse or big damage of soft first story structures.

(ii) Collapse or big damage of the first story of the other than soft first story type.

(iii) Complete collapse or big damage of one of the story of inter stories of the structures (*Figure 12*).

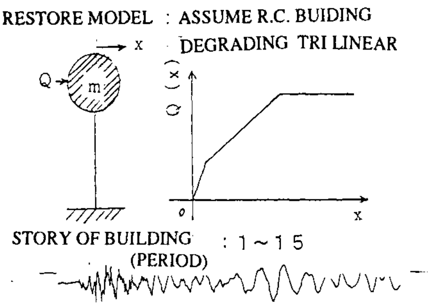

Figure 8
DEGRADING TRI-LINEAR MODEL

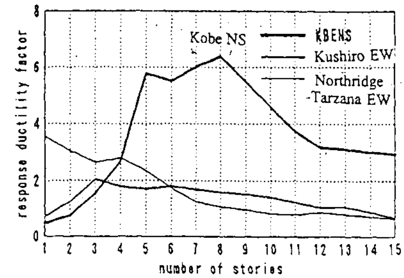

Figure 10
NON-LINEAR RESPONSE
FOR JAPANESE AVERAGE
BASE SHEAR COEFFICIENT

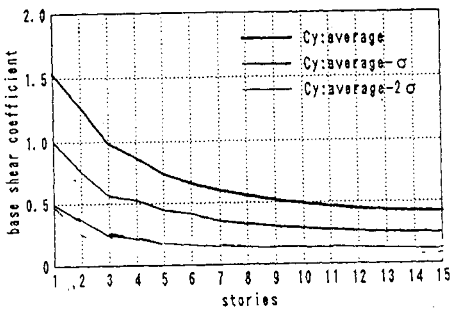

Figure 9
AVERAGE BASE SHEAR
COEFFICIENT of EXISTING
BUILDINGS in SIZUOKA JAPAN

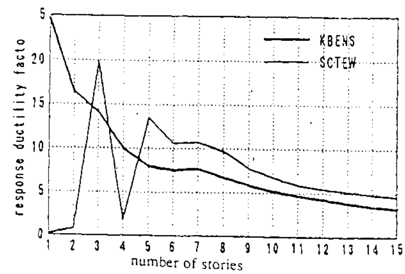

Figure 11
NON-LINEAR RESPONSE
FOR MEXICAN AVERAGE
BASE SHEAR COEFFICIENT

Figure 12
COLLAPSE of INTER-STORIES
of R.C.Structure.

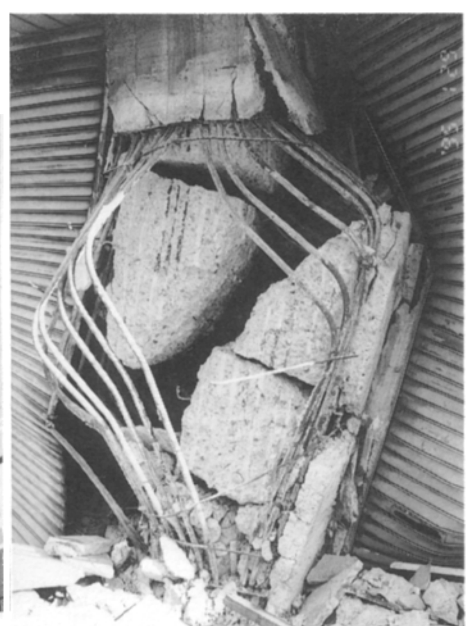

Figure13
SEVERE SHEAR FRACTURE
of R.C.COLUMN.

(iv) Damage at the boundary of steel R.C. composite and R.C. structures.
(v) Damage at connections of steel structure part and anchor connections at the bottom of columns
(vi) Damage at the anchor connections of reinforcing bars of the structural walls of S.R.C.
(vii) Damage at the beam-column connections.

(viii) Shear collapse or severe damage of R.C. columns due to fractures of hoops of the columns (*Figure 13*).
(ix) damage due to butting of neighbor structures.
(x) Fall-down damage of pre-cast boards of roofs.

Wall type R.C. structures had almost no damage even though they were constructed before 1981.
The one of the main reasons of the above damages would be as follows:
(i) Collapse or big damage of soft first story structures.
As the name of these type of structure says, the first story is considerably weak and soft comparing with the other stories . In Japan these type of structures are called "PIROTY". Essentially, earthquake shaking energy concentrates to the soft deformable portions just as piroty type of structures. So the energy of ground motions concentrated to the top and the bottom of the first story's columns. Then the these portions became more flexible due to some damage, so again the energy of ground motions concentrate more to the same parts of the structure. This phenomenon proceeds in a rather short time and finally destroyed the first story of the soft first story, where the most severe shear force was loaded.
(ii) Collapse or big damage of the first story of the other than soft first story type.
Damage of this kind happened to the building structures constructed before 1981 according to the old seismic design codes. The first story usually loaded the largest shear force. Essentially, because of the lack of the final shear strength comparing to the actual shear force due to Hanshin Earthquake ground motions, brittle shear failures were occurred in the weakest first story.
(iii) Complete collapse or big damage of one of the story of inter stories of the structures. According to the old seismic design code, the lateral shear coefficient was 0.2 constantly from the ground level up to 16m high. It was different from real shaking mode, while the present seismic design codes require Ai distribution mode similar to the real shaking mode. So around 16m high story, the design lateral shear force and real one became much different; the former was rather less than the real response force. So the inter story were completely collapsed or were severely damaged. Some other concept was proposed by Professor Iwan of Cal. Tech. According to him, the deformation of ground motions propagated up to the top of the building and reflected back to the inter story; meanwhile the next big deformation of the

ground motions also propagated up to the same inter story. If the phase were the same direction, deformation became big enough to destroy the whole inter story .

(iii) Shear collapse or severe damage of R.C. columns due to fractures of hoops of the columns.

Due to 1968 Tokachi-oki Earthquake, various R.C columns were failured by lateral shear force because the intervals of hoops (in those days intervals were less than 30 cm) were not enough to resist against the actual shear force.

So after the Earthquake, seismic design codes were changed in increasing the strength of hoop as to decrease the intervals of hoop less than 15 cm and near the beam-to -column connection less than 10 cm intervals were requested. Since then,shear strength of R.C. column became at least twice as strong as the old one.

4. Damage of Steel Structures.

(1) The features of damage

The features of damage of steel structures were:

buildings constructed not only before 1981, but also constructed after 1981 were both damaged. Severe damage were mainly in welding portions, high-tension bolts connections parts and base and anchor parts of the columns.

(i) In case of box type columns were used, damage were around the base of columns, fracture of welding at beam-column connections and fracture of column to column connections.

(ii) In case of diagonal bracing type structures, fractures of bracings and at the bolt connections, essentially by tension forces. But in some cases buckling of bracings were observed by compression forces. And gasset plates for bracing were fractured, as well as fractures of beams due to lack of bending strength.

(iii) The typical damage due to the long duration stains of thin plate steel structures.

(iv) Brittle fractures of columns composed by thick (more than 50 mm thickness) plate with confined shape -like box shape or tube shape- of rather high-rise residential buildings

(v) Butting of two neighbor steel buildings.

The one of the main reasons of the above damages would be as follows:

(i) Damage around the base of the columns.

The damage were concentrated to the bare type columns, and fractures of base concrete and anchor bolts were much observed. Generally the base of the bare columns were assumed to be pin connection but the actual base of columns were not complete pin but rather limited deformabilities with fixed conditions. So some bending moments were actually worked where strength against bending moment were not considered at the design. 16% of the steel buildings constructed before 1981 were damaged at bases of columns but after 1981 when the present seismic design codes were started to be practical, 4% of them were damaged.

(ii) Fracture of welding at beam-column connections.

These type of damages mainly happened on the structures of box type columns and H type beams connected with simple diaphragm which had almost no plastic deformations. So damage were concentrated to welding portions, such as box column and diaphragm, beam-to-column panel and diaphragm, end of H beam at under parts of flanges and webs as well.

(iii) Brittle fractures of columns composed by thick (more than 50 mm thickness) plate with confined shape -like box shape or tube shape- of rather high-rise residential buildings

Quite brittle tension fractures occurred at the near welding portions of column-to column, as were shown in *Figure 14*. The reasons of these brittle tension fractures were not so clear, but almost similar fractures occurred at the columns of railway bridges of infra-structures. Some researcher suggested that these kind of brittle tension fractures occur when the following 3 S conditions came identically ; a) Size (thickness is more than 50 mm, and width of column is more than 50 cm), b) Shape (close shape such as box shape or tube shape, not like H shape) and

c) Speed (speed of strain of the material must be quite high speed of strain).

Figure 14
QUITE BRITTLE TENSION FRACTURES AT STEEL LARGE SIZE BOX COLUMN.

5. Damage of Wooden Houses

(1) Non damaged or slight damaged wooden houses.

The following types of wooden houses had a little damage even in the districts of Intensity 7 of JMA.

(i) Houses could meet with the present seismic design codes, and proper construction check were performed for instance, such as houses built by the public residential funds which were usually checked on the construction qualities during constructions; among those types of 529 houses in Kobe areas, only 1.7% of them were collapsed.

Figure 15
COMPLETE COLLAPSE of THE FIRST STORY of WOODEN HOUSE CONSTRUCTED TWO MONTHS BEFORE THE EARTHQUAKE

(ii) The traditional type of the wooden houses with proper amount of shear walls located properly.
(iii) Two by four frame boards walls type wooden houses.
(iv) Pre-fabricated wooden houses.
(v) Wooden houses of 3 stories, structural calculation were made.
The one of the main reasons of the damages would be as follows:
(i) The most of the collapsed or failured wooden houses had the following similar conditions:
(a) Old houses since the construction, so columns and foundations were in some portions rotten or termites effected the strengths of columns. (b) Very heavy Roof tiles with clays beneath the roof tiles for the protection against typhoons. And (c) Did not meet the present seismic design codes.
(ii) Improper amount of walls and improper locations of them.
Essentially, walls could not be located in south direction, because generally the south direction would like to be opened without walls. This natural request made the wooden houses to be rather weak against earthquake ground motions. *Figure 15* shows typical collapsed first story due to lacks of walls and improper locations of them only after two months of construction.
So many numbers of death casualties due to the collapses of wooden houses as almost 5000 people. However, numbers of collapsed wooden houses was about 60,000. So as average every 12 collapsed wooden houses killed one person.

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EXPERIENCE AND LESSONS OF POST-EARTHQUAKE RECONSTRUCTION IN LUHUO COUNTY

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ABSTRACT

This paper examines post-earthquake reconstruction and seismic hazard forecast in earthquake resistance and disaster prevention planning in Luhuo County. Experiences and lessons are summed up for future reference.

KEYWORDS

Post-earthquake; reconstruction; seismic hazard; forecast; disaster prevention; Luhuo earthquake.

INTRODUCTION

Xianshui River fault in China and San Andreas fault in America are both large, famous fault zones. They are well-known at home and abroad for their high seismic frequency, strong intensity, clear new tectonic activity and speed of fault sliding. Seismic research in China and America carried out much scientific study and academic exchange for this reason.

Luhuo county is located at the middle-upper section of Xianshui River fault zone, with much earthquake activity. According to historical data, strong earthquakes of greater than M6.0 occurred seven times at Luhuo county and nearby regions during the 157 years between 1816 and 1973. Among them were an earthquake of M7.5 in 1816, an earthquake of M7.25 in 1923 and an earthquake of M7. 6 in 1973. Local people of various ethnic nationalities suffered extremely serious damage. Note that two earthquakes of greater than M7.0 in 1923 and 1973 happened at the same place in a span of 50 years.This is seldom seen in the earthquake history of China and the rest of the world. Judging from this, the danger of earthquake in Luhuo county is out of the ordinary.

WORK OF POST-EARTHQUAKE RECONSTRUCTION

On February 6, 1973, a strong earthquake of M7.6 occurred at Luhuo. The quake center was about 10 kilometers away from Luhuo town, and quake intensity there was 10 degree. Houses and equipment throughout town lay in ruins. The earthquake resulted in serious calamities to human life and property. After this earthquake, Sichuan Province government paid great attention to the area and organized earthquake resistance and disaster relief. Southwest Architectural Design & Research Institute of China was assigned to be in charge of post-earthquake reconstruction work in Luhuo county. The author took part in and was responsible for work in Luhuo county. Reconstruction in the county town adhered to 9 degree of zoning intensity.

Reconstruction work was a central task of local government after the strong quake. Through headquarters of reconstruction, unified control was established for the subsidy of earthquake resistance and disaster relief by central authorities, local government and indivdual donations. These organizations established unity in the applicatiom of earthquake resistance planning and earthquake protection to city buildings and structures. Routine work in Luhuo county was appointed to the reconstruction office. Measures for earthquake resistance in the design of major department houses and public buildings was assigned to the design institute. Reconstruction of department buildings under county government and houses owned by citizens of the county town was appointed to the reconstruction office. Reconstruction was a task of top priority. Bringing enthusiasm into full play was a key factor in accelerating reconstruction speed.

Tailoring measures to local conditions and using locally available materials is also a cardinal principle in accelerating construction. Luhuo county is located in an exceedingly cold outlying district. The locale has only timber and natural flake stone for building materials. A masonry flake stone wall with low adhesion mortar hasn't the capacity of earthquake resistance. Architectural materials such as steel, cement and brick must be transported from other places and are very expensive. So timber becomes the only building material with earthquake resistance capacity. But its fire protection capacity and cold protection capacity are poor.

Locally the earthquake resistance capacity of traditional wood structures is also poor. Improvement of timber structures can be accomplished by the scientific method. For this purpose, designers studied the situation in depth. Their approach was to integrate theory and practice. They combined existing theory of earthquake resistance with local construction practice. Also they gave consideration to the habits and characteristics of Tibetan houses. Thus an earthquake resistance calculation method and construction details for houses made of wood were assembled through analysis and study .

Management of reconstruction work in the county town proceeded as follows .

a. Earthquake resistance planning is assigned to the design institute for unity of approach and implementation by stages .

b. Reconstruction department makes overall arrangements for funds throughout the county and allots a quota to every unit .

c. Anti-seismic construction measures for buildings were handled in a unified way by the county office of reconstruction while actual construction of buildings was arranged by each unit itself.

d. Construction teams for reconstruction work are left to each unit itself to handle .

Note that Luhuo county is located in an exceedingly cold outlying district, which is deficient in building materials, difficult in transport, and lacking in qualified technical personnel. Moreover, handling accomodations for the disaster-stricken people as quickly as possible is necessary. With these unfavorable conditions, it can be hard to avoid the danger of seismic destruction for the county town.

FORECAST ANALYSIS OF SEISMIC HAZARD FOR HOUSING

In October 1992, the planning of earthquake resistance and disaster protection in Luhuo county was adopted by provincial appraisal. In June 1991, the forecast work of seismic hazard was carried out by guidance of scientific research and design unit whose staff is familiar with forecast technique in Sichuan Province. The author took part in their work as it applied to houses. Seismic hazard forecast of county houses adheres to a basic 9 degree intensity which is shown by the map of China Seismic Intensity Zoning.

The forecast method used in various kinds of houses is the so-called two-stage evaluation method which is used in multistory brick houses based on checking computations of aseismic strength, in reinforced concrete frame houses based on checking computations of elastoplastic deformation. A vague combined judgment method is used in R.C. bent frame houses and older houses. These forecast methods have been translated into a computer program which can be used directly by forecast personnel. Seismic hazard degree forecast of different kinds of houses of Luhuo county can be seen in Table 1.

As shown in Table 1, county houses which suffered middle and more than middle-degree destruction occupied 76 percent of the total architectural area of planning region houses. That region comprises county houses after the earthquake of M7.6 in 1973 at Luhuo; three-fourths of the houses did not conform to the requirements for earthquake resistance protection. Seismic hazard forecast personnel found that earthquake resistance protection which did not meet the requirements of 9 degree intensity characterized the majority of houses. This is a universal phenomenon, along with poor quality construction of houses, rotten wood structural members, and cracking. So the statistical results of seismic hazard forecast in Table 1 tally with the damages.

Along with development of the economy and improvement of the living standard, many reinforced concrere frame buildings and multistory brick houses have been built in Luhuo county. Compared to past houses made only of wood, last, current standards have been improved. There was also a phenomenon of blind competition in decoration of houses which had been built in recent years. And question of quality in earthquake resistance protection of houses was ignored too much. Another phenomenon of building not according to earthquake resistance protection has arisen in construction.

Table 1 Seismic Hazard Analysis of Luhuo County Housing

structure type	intact area	intact %	light des. area	light des. %	middle des. area	middle des. %	serious des. area	serious des. %	collapse area	collapse %	total area (m^2)
multistory brick house					2399	100					2399
old style house	2388	10	38938	16.3	64976	27.2	71044	29.7	40037	16.8	238882
R.C. frame structure	109	0.4	2965	12	11371	46	4792	19	5547	22.6	24784
R.C. bent frame							316	100			316
total	23997	9	41903	15	78746	30	76152	29	45584	17	266382

CONCLUDING REMARKS

From the experience of post-earthquake reconstruction, it is concluded that: close attention by govermnent is a precondition of the work of earthquake resistance and reconstruction. Bringing enthusiasm into full play is a key factor in accelerating reconstruction speed. Tailoring measures to local condition and using locally available materials is a cardinal principle of reconstruction work. Combining theory with practice is a channel for solving various problems of reconstruction work. Using advanced forecast and calculation methods can ensure that forecast results are believable .

Lessons of post-earthquake reconstruction are: when everyone does things in his own way without restrictive measures, earthquake resistance protection quality for the majority of houses cannot be pledged, and the danger of seismic destruction remains for the county town. In construction, emulating blindly and not paying attention to the quality of earthquake resistance protection makes new buildings fail to meet the requirements of earthquake resistance protection .

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MEASURES TO RESUME AND REBUILD TANGSHAN

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ABSTRACT

This article introduces the earthquake disaster of Tangshan in 1976. Furthermore, it lays special emphasis on expounding the plan for Tangshan's resumption, rebuilding and anti-quake measures during implementation of the plan. Finally, several pieces of valuable experience aimed at resumption and rebuilding the city of the earthquake-stricken area are noted for future consideration .

KEYWORDS

Earthquake disaster; construction plan; rebuilding; anti-earthquake design; investment; Tangshan earthquake.

GENERAL SITUATION OF EARTHQUAKE DISASTER

At 3:42 in the morning on July 28th, 1976, a devastating earthquake (M=7.8) took place from Tangshan city to Fengnan county. The earthquake spread to 14 provinces and cities with an area adding up to 2,170,000 square kilometers, and it caused destruction within 30,000 square kilometers. Its intensity level reached 11 in the epicentral area. Its macroscopic epicentre lay in the Lunan District. Buildings in an area of 18,680,000 square meters collapsed only in the Lunan District of Tangshan, covering 87 percent of the total construction in the whole city; 94 percent of diverse civil construction in the urban district was damaged by the earthquake. Traffic, communication, water supply and power supply were interrupted; public facilities machinery and equipment for agriculture and industry as well as construction of water conservancy works were all badly damaged. Economic loss caused directly by the earthquake added up to 100 billion yuan. Altogether 242,469 people died from the earthquake; among them 143,022 people died in the city of Tangshan (including 12,103 nonlocal transient population); of 167,539 people badly injured, 81,630 were in the city of Tangshan; 7, 216 entire families died from the earthquake, nearly 10,000 families disintegrated, 2,562 children became orphans, and 895 people lost their spouses. Loss of limbs afflicted 1,814 people. Before the earthquake, there were 123,578 people in the Lunan District where the epicenter was located; of these, 35,574 people or 28.79 percent of the population in the urban district, died from the earthquake. Such massive fatalities are rare in the history of earthquakes.

GENERAL FEATURES OF CONSTRUCTION PLAN FOR NEW TANGSHAN

Overall Layout

Before 1950's, Tangshan was divided into two districts, namely the eastern mining area and the old urban district. The latter was subdivided into two administrative districts. One was the Lunan District and the other, Lubei District (lying to the north of the Beijing-Shanhaiguan Railway).

In accordance with the guiding principles for urban construction and in the light of specific post-earthquake conditions, the development of small cities took precedence. Therefore, the construction plan for the old urban district was subject to considerable revision. Under the revised plan, the old urban district was restored on the basis of the original Lubei District. Existing steel plants and power plants were preserved. Local and municipal party and government organizations were set up here because it is the political, economic and cultural center. The area covers 40 square kilometers and the population remains at a figure of 40 thousand.

In the old Lunan District priority went to the development of scenic spots as well as a small number of warehouses and small industries which present no environmental problems. Subsidence pits due to mining were transformed into lakes. Some representative earthquake relics were preserved. Reference rooms concerned with the earthquake were built for the scientific workers both here and abroad to do studies, go sightseeing or pay visits. Factories, enterprises and local inhabitants resettled in a new district at the eastern part of Fengren County, forming a satellite city. This area covers 7.34 square kilometers, and the population remains at 60,000. The eastern mining district was restored on the basis of individual Kailuan collieries with a population of 30,000; this area covers 25 square kilometers.

In this way, the city was divided into three parts which are separated from each other by a distance of some 25 km. All three parts are linked by the Beijing-Shanhaiguan, Tongxian-Tuozitou and Tangshan-Zunhua Railways as well as three highways, i.e. Tangshan-Fengnan, Tangshan-Guye and Fengnan-Guye.

Furthermore, plants and mines (such as Majiagou Colliery, Jinggezhuang Colliery and Douhe Power Plant) which were originally scattered around the periphery of the old urban district established their own small towns. Chemical engineering plants relocated.

Functional Division of New Urban District

The new construction plan changed the former unreasonable layout of functional divisions as well as the irrational situations manifested in a criss-cross of factories with residential quarters. In light of the practical situation of the three parts, and adhering to the principle of facilitating both production and daily life, a rational overall layout was arranged, allowing the industrial residential and storage areas to be clearly demarcated. Each one is characterized by its own distinguishing features and style.

Dacheng Hill is the only scenic spot in the urban district. The Douhe River meanders from the north toward the east and then runs southward at the foot of the hill. The river bank is lined with green trees giving welcome shade. It is a picturesque hilly area unmatched for its scenic beauty. In the new plan, full use was made of such natural conditions. Steel plants and porcelain factories were set up to the east of the river, while to the north of the river, an automobile factory and engineering industry were established. Allocated to the west of the river were residential quarters with Dacheng Hill serving as a natural segregation zone. At the western part of the urban district a small industrial area was put on the border of the residential area where industries not harmful such as electronics and food as well as other light industry were set up. Such an arrangement of industries by type is beneficial both to production and mutual cooperation.

The central part of the urban district is located at the geometrical center of the western residential quarters. Being near the trunk lines, it has easy access. Administrative, cultural, business and sports centers are likewise here, forming an integral architectural complex. In the administrative centre are Party, government and mass organizations. Being arranged centrally, it' s convenient for them to get in touch with each other. Included in the cultural and business centers are general bazaars, specialized stores, cinemas, banks, post offices, hotels, bookstores and scientific centers. Provided in the sports center are various sports facilities such as stadiums, gyms and swimming pools. The residential area, being close to the center of the urban district and to the industrial district makes it convenient for people and helpful in reducing the burden of traffic. Warehouses are arranged near the railway freight yard or on the residential area based on their use.

The new urban district develops eastward from the Fengren County. Behind it is the Huanxiang River and in front is the Tongtuo Railway. The western part of the district is allocated to living quarters while the eastern part is designated as industry and storage. Large-scale plants such as the rolling stock gears, light machinery, and Huaxin Textile Mill are set up here. In addition, a large park was established by taking advantage of a bend in the Huanxiang River.

In the eastern mining district are small mining towns composed of five widely dispersed Kailuan Collieries. The Chaogezhuang region borders on the Baiyun Hill and 3 km southwards lies the Tangjiazhuang region which stands opposite the Linxi region. These five regions, each having an independent layout, are linked by highways forming a comprehensive network.

Planning of Residential District

There are 30,000-50,000 inhabitants in each residential area which is composed of five smaller sectors. Each residential area covers 70-100 hectares, within which business, cultural, educational and sanitary facilities are provided. In each sector is a residents committee, middle school, primary school, youth club kindergarten, nursery, food market, grainshop, snack bars, barbers, savings bank and bicycle parks, all occupying 10-12% of the residential area and averaging 1.2- 1.4 square meters per capita. In each residential area is a street office, local police station, hardware stores, department stores, theaters, children's center, restaurant, drugstores, neighborhood service center, post office, clinic, bookstore, gas tank and central heating system with a construction area of 0.5 square meters per capita.

Dwellings are usually four or five-story houses with an area of 40-50 square meters for each household. In order to make the arrangement diverse, some six-story houses are also added locally. Each household in the newly-built dwelling shares 1-3 rooms and enjoys the use of a private kitchen, bathroom, gas valve and heating system. Each household is provided with a kilowatt-hour meter, water meter and gas meter. For environmental improvement and beautifying the city, afforestation is arranged at three levels, i. e. municipal administrative and residential with an area averaging 6 square meters per capita. There are 26 parks, eight at the municipal level and eighteen at the district level. Along the seashore, riverbanks and roadsides, embellishment is well underway. The squares between houses are also adorned with flowers and grass. Thus a comprehensive beautification system has been formed.

Traffic Planning in Urban District

Before the earthquake, the streets in Tangshan were narrow and crooked, with many T-shaped road junctions. Traffic was impeded for lack of connecting roads. To tackle this problem, measures were taken in the new plan to widen the roads and open up new trunk lines. Apart from this, all T-shaped road junctions have been cut through to form a good transport and communication network. Trunk lines have been widened from the former 30m to the present 40-50 m, sub-trunk lines from 20m to 30-35m, and branch lines from several meters to 20-25 m. We have put into service two grade crossings, opened up 27 trunk and sub-trunk lines and seven intersections with safety islands. Eight trunk lines, each more than 40m long, are built with three-slab pavements. Lane lines are provided to ensure safe driving. Moreover, a special lane leading from the residential district directly to the industrial region was opened for bicycle riders.

Traffic conditions were diversely affected by the Beijing-Shanhaiguan Railway, with much coal being mined underneath it. Since the railway runs through the city, it was therefore decided to change its route by moving it to the western part of the urban district. A new railway station was set up at the westernmost part. The original Beijing-Shanhainguan Railway was changed over to a special industry line. At the same time, these special trains were opened to traffic. With the change-over of the Beijing-Shanhaiguan Railway, readjustment of the special industry lines can be made if traffic problems arise.

EARTHQUAKE RESISTANCE CONSIDERATIONS IN PLANNING PROCESS

At a nationwide anti-seismic meeting, Tangshan was recognized as one of the cities possessing earthquake hazards. During reconstruction, earthquake-resistant performance was taken into account from every aspect for selection of construction sites, architectural design and construction details.

Selection of Proposed Construction Sites

Extensive work was undertaken during and after planning stage to reduce the hazards of future earthquake. This work included geological exploration, general survey of water sources, division of areas of high seismic activity, analysis of vulnerability and topographic survey. By considering this information along with the effects of various factories and through rational selection of construction sites, the irrational layout of old Tangshan city was changed and a new city constructed on the basis of modern technology. The Lunan district was located in an active fracture zone. During the July 28 earthquake, almost all industrial and civil structures collapsed, resulting in severe damage. Furthermore, coal seams are mined underground. In some districts, coal seams are left unmined but in the long run, mining will resume and underground tunnels dug. Hence priority has been given to this district in the new plan. Key plants, enterprises and inhabitants moved to a new district in the eastern part of Fengren County with favorable geological and hydrological conditions.

Tangshan's old urban district was developed on the basis of the old Lubei district. The construction site was situated to the north of the subsidence caused by mining. This area possesses favorable geological conditions, with ground mostly in categories 1 and 2 and no mining subsidence.

The eastern mining district was developed on the basis of the above-mentioned layout. Compared with large cities, this layout is favorable for earthquake resistance and relief work. It offers further merits of good transport services, convenience for people to evacuate, for firefighting and rescue activities. A city of this size not only meets actual demands but also can reduce the load on unit area and strengthen structural components. To mitigate the adverse effects of landslip, an area with a width of 60-80m along the river was kept as a shelter belt. Foundations of buildings near the shelter belt were treated with intense ramming by 10 ton rammers to increase their load bearing capacities. In the new urban district in Fengren County, a rift runs in the east-north direction. Under the new plan, an area with a width of 100 meters became an afforestation zone to beautify the city and reduce seismic hazard. Tangshan Mining Institute was located in an area with a bed-rock ridge and an abrupt change of thickness in the quaternary system. Due to this ridge and the magnification effect of the stratum in the quaternary system, the institute suffered severe damage. This area being in the vicinity of the urban center, it was decided to reconstruct the institute on its former site. To reduce damage in another seismic event, technical measures were taken to reduce the height of the buildings and strengthen their structures. In the No. 25 residential area, a subsidence cave appeared after the earthquake. Exploration showed that the cause was underground limestone caves. Due to the effect of surface water, soil erosion occurred above the bed rock, thus resulting in the subsidence. To tackle this problem, a method was used to locate the peripherally stable zone by means of the angle of internal friction of fine sand in water (26 c). By comparison, in areas with shallow bed rock and high bearing capacity, tall buildings and important service facilities were built. These include the Workers Hospital, No. 2 Hospital, fourteen-story Local Guest House, twelve-story No. 2 City Guest House and sixteen-story Xinhua Hotel. In order to facilitate evacuation and avoid injuries due to collapse of structures, ample space was left between the buildings. Based on the lessons of the Tangshan earthquake, a three to six-story building generally collapsed to an extent of 2h/3 (h = height of the building).

During a devastating earthquake, buildings on both sides may collapse. Therefore, it is appropriate to have a thoroughfare with a width of 5-6 meters between both buildings for easy evacuation. This thoroughfare should be flat and straight. Open space should also be retained and water supply facilities installed to make this space suitable for refuge during the earthquake.

According to national seismic zonation, Tangshan is a city subject to an intensity level of 8 degree. Hence, in their design, except for lifeline structures (which have to be fortified against an intensity level of 8, yet checked and accepted according to 9), all ordinary industrial and civil buildings are fortified against 8. Proceeding from actual conditions, three types of construction configurations were adopted, namely, "concrete poured internally, brick laid externally or poured internally, clad externally", "unreinforced masonry construction with structural columns", "frame and lightweight panel construction".

It should be emphasized that planning for earthquake-prone cities must be an ongoing process with strategic significance rather than a sporadic response to temporary concerns over earthquake resistance performance of individual structures.

During the Tangshan earthquake of 1976, all structures ceased to perform necessary services because of the irrational layout of the city and the deficiency in earthquake resistance measures. Therefore rescue and relief work required massive nation-wide assistance. Of the 148,022 people killed during the event, a great number died on the way to other places for lack of timely treatment. In a sense, severe damage to lifeline structures aggravated the disaster. This issues a serious warning to countries of high seismic activity. Attention has been paid to the problem in the new Tangshan plan and appropriate procedures have been taken as follows based on national conditions.

Precautions against Earthquake Hazards

Urban Traffic. In order to solve the traffic problems encountered after the earthquake, more arterial roads were provided for closer contact with neighboring cities, i.e., Beijing, Tientsin and Qinhuangdao.

Water Supply. Measures were taken to establish a decentralized, ringlike water supply with multiple waterworks. Mines and other large users must provide their own water. Wells in rural area should be preserved for urgent needs. In town squares or open fields, hydrants were installed for drinking water and firefighting.

Power Supply. Measures were also taken to adopt a ringlike power supply system. Four power stations located at Beijing, Tientsin and Tangshan were connected with a 220,000V high-tension transmission line to avoid a power failure in case one of the four stations is destroyed.

Urban Communications. A wireless communication system was combined with a wired system and operating centers were set in different places. Cables are mostly buried underground and linkage is arranged to maintain operations following a seismic event.

Residential districts should be built at a distance from ground exhibiting liquefaction and from areas with fractured zones, landslides and limestone caves. Otherwise, technical measures should be taken. For example, in the course of implementation of the new plan, some dwellings were built at the bend of Douhe River with unfavorable geological conditions (showing high potential of liquefaction) because the task was pressing and time was short. Difficulties in moving earthquake-proof sheds and in taking over the land also contributed to the adoption of such an expedient measure. In order to keep buildings from cracking due to uneven settlement of the ground, measures were taken to enlarge the cross section of engineering enterprises as well as to relock warehouses storing flammables, explosives and poison. Reservoir dams at the upper reaches of the Douhe River have been strengthened to comply with requirements for confronting what may be the biggest flood of the century and great demands on its seismic resistance performance. The river itself has been widened and straightened to increase its flood discharge capacity and protect the city.

Formulation of Plan against Earthquake

The new Tangshan is fortified against earthquakes even beyond national stipulations, with key parts of the lifelines at a degree while ordinary industrial and civil constructions are fortified at the basic level of 8. Tangshan is probably the most strictly fortified in the whole country. However, it is never enough for the safety and normal function of city to rely on anti-earthquake fortification. An earthquake plan suited to local realities should be formulated and implemented. Only through such measures can we reduce earthquake hazards to a minimum. Tangshan is no exception. From 1989 on, the task of formulating a plan has been done with the guidance of the construction department. This plan is basically completed pending approval by higher authorities. Large and mid-scale factories in urban districts such as Kailuan Mining Office, Tangshan Steel and Iron Company, are starting to draw up anti- seismic plans against earthquake. A transformed city now stands East in Heibei Province .

ACHIEVEMENTS OF RECONSTRUCTION

Total investment in reconstructing Tangshan was 43.57 billion yuan. By the end of June 1986, 41.51 billion yuan (about 95 percent of the total) had been used. Buildings encompassed 18 million square meters, including 11.2 million square meters of residential buildings; 225 thousand families moved into new houses (81.3 thousand people). By July 28th, 1986, the 10th anniversary of the earthquake event, it was announced that reconstruction had been basically completed. In recent years, rent reform in Tangshan has brought vitality to housing construction. By the end of 1991, dwellings in the city added up to 15.6458 million square meters with 11.39 million square meters in use. Average living area was 7.4 square meters, average usable floor area was 11.5 square meters; 236,943 families (7. 82 million people or 75.2 percent of the population) use gas. Area heated is 10.29 million square meters, with central heating used by 57.12 percent of the population. Average water used is 133 L per day, by 100 percent of the population. Vegetation in the city covers 2,027 hectares, plant 18 percent of the area.

As reconstruction progressed, the local economy developed. By the end of 1977, the production of industry and agriculture had all recovered. Gross output of industry and agriculture was 24.1 billion yuan by the end of 1978, the level before the earthquake. People began massive rebuilding in 1979. After seven years, most of the work was completed. By 1985, gross output value of industry and agriculture reached 41.08 billion yuan, 66 percent more than in 1975; with a 5.2 percent increase per year, 65.5 billion yuan in taxes was generated.

Pride in the new Tangshan is expressed by praising the political, economic and social order.

LESSONS OF EXPERIENCE

The project to rebuild Tangshan was so vast, the scope is so wide, the difficulty so great that it is rare in the annals of architecture in China or elsewhere. Here is a summary of the experience.

Lessen the Burden on the Nation through Self-Reliance

"Self-reliance and arduous struggle" is part of the legacy of the Chinese revolution. To strengthen the nation, we should still advocate the spirit of "self-reliance and arduous struggle" in all causes. By relying on this spirit, the people of Tangshan overcame difficulties and rebuilt their homeland from ruins. However, it was a mistake to have refused foreign aid. This approach has changed with an information policy .

Make factories be responsible for their production and profit. Tangshan adopted this rebuilding its factories. This method not only promoted the local production but also eased the burden on the nation. It remains valuable for the nation.

Unify Support from all over the Country

A work force from 15 provinces and cities as well as the People's Liberation Army was assembled to rebuild Tangshan. They helped with all aspects of the project including a survey of the topography. We amassed a large technical crew to form Tangshan Construction Command Post in time to coordinate the work under the direction of the Hebei Provincial Party Committee. A "Six-unity" method of unified plan, unified design, unified construction, unified fund, unified assignment, and unified management was put into practice, and it proved to be successful.

Standardize Design and Industrialize Construction

In design, use of standard structural members is a goal. This makes production of components as well as procedures in construction more efficient.

Construct the Periphery First then the City's Former Site

Given the ruins and shockproof sheds all over the city after the earthquake, there was nowhere for the residents to stay. It is practical to first build dwellings at some distance and gradually move farther outside of the city, then return to construct the former site. Along with mobilization, certain laws and decrees are necessary to ensure that the mass construction proceeds.

Build Network of Highways in Advance.

Since the construction of dwellings is urgent, some began this task before the city's network of highways was completed. As a result, drainage, heating and coal gas systems cannot be put together, and there is the prospect of frequently covering up various pipelines. Without a highway network, traffic is blocked which can delay construction. People became dissatisfied. Residents have to cook by burning coal because residential areas cannot provide heating and coal gas soon enough. This situation of "fire and smoke everywhere" causes fumes indoors and pollutes the environment .

Develop Satellite Cities and Open up Small Towns

This approach is important. Drawing up such a program can reduce earthquake hazards to a minimum.

Select an Appropriate Site for Construction of a New Town

This selection should be regulated. Factors to consider are natural resources and economic potential as well as availability of water and flood control.

Adhere strictly to national standards for subsidence areas due to mining. Major construction where subsidence exists cannot be permitted.

Construct residential areas according to a general urban plan. Living quarters should be located where a shockproof and anti-accident program has been implemented. In construction, to "stick in a pin wherever there is room" should be strictly forbidden. Such a program helps ensure protection of residential areas.

AN ACCOUNT OF RESTORING OPERATIONS AFTER TANGSHAN EARTHQUAKE

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ABSTRACT

This paper discusses the operations, methods and measures for restoring production, livelihood and normal order in a short period of time after the Tangshan earthquake, July 28, 1976. At the same time, it gives a description of work to prevent epidemics and eliminate disease.

KEYWORDS

Post-earthquake; restoring; life-line system; epidemic prevention; disease elimination; production; Tangshan earthquake.

INTRODUCTION

Restoring normal operations after an earthquake begins with meeting an emergency and rushing to the rescue. Then comes the long process of restoring the homeland. Proper handling of restoration is important in reassuring the public, saving funds and laying a good foundation for rebuilding. What happened after the earthquake of Tangshan provides a case study.

RESTORING LIFE-LINE SYSTEMS

Postal and Telecommunication

The Ministry of Post and Telecommunications quickly set up an office for earthquake resistance and rescue. A contingent left for Tangshan immediately. More than 2,000 people, over 200 cars, a large quantity of communication devices, and materials to repair telecommunication came from 9 provinces and cities under the central government and autonomous regions as well as the other 8 prefectures of Hebei Province. They reached the disaster area by traveling day and night in a steady stream. At the same time, vast numbers of postal and telecommunication staff from the Post and Telecommunications Bureau of Hebei Province, Tangshan City and Tangshan Prefecture went into action quickly. They used underground cables not damaged by the earthquake and put the first trunk line through to Beijing the morning after the earthquake. In the afternoon, they put through a contact line with Shijiazhuang, the capital of Hebei Province. The next day direct circuits to Beijing, Tianjin, Shijiazhuang, Shenyang were put through from Tangshan. Shortly after that, emergency measures were adopted in the province. By July 31, original circuits were basically restored. This gave the earthquake resistance and rescue headquarters the ability to get through to the major disaster locations. Once the main communication lines were restored, the contingent from 9 provinces and cities coordinated work with the postal and telecommunications staff of Tangshan City and Tangshan Prefecture. By the end of August, 64 sets of various communication devices had been installed. Carrier devices approximated the pre-earthquake level. There were 116 trunk circuits, 13 more than before the earthquake. Telephones from Tangshan City to all the counties were restored. In the city, 11 sets of 100-channel magnet exchanges and telephones for 230 customers were installed. As for radio communications, except the Tangshan to Shijiazhuang and Tangshan to Douhe Reservoir, 14 circuits from Tangshan to all the counties were added to be opened up. Telex circuits from Tangshan to Beijing and Shijiazhuang were also installed. By September 1, cable business from Tangshan to other parts of the country had been reopened to the public, and 400 communication circuits from counties to communes had been restored. As for postal operations, all except package and cabled remittance, were restored to the pre-earthquake level.

Communications and Transportation

This job was conducted in two stages. In the first stage, temporary measures and simple transport service were targeted. Headquarters had quickly transferred more than 4,500 professionals, more than 28,000 People's Liberation Army (PLA) commanders and soldiers, and more than 15,000 local people to reinforce damaged highways and erect makeshift bridges. Use of ferries and wading through shallow water were adopted to temporarily maintain transport service. This stage basically ended on August 15. In the second stage, restoration was the goal. A strategy was adopted to give priority to trunk lines and major damage. Highway transport was restored to the pre-earthquake level in less than 20 days.

At the same time, restoration of railways was in full swing. Over 42,300 people from more than 28 units had been mobilized. Jingshan and Tongtuo were top priority for repair in a concentrated effort to open the railways to traffic first and fully restore them second as well as repair trunk lines first, branch and special lines second. On August 3, Tongtuo line reopened and on August 7, a single track of Jingshan railway reopened. On August 10, multiple tracks of Jingshan reopened. Shortly after this, full restoration of the railways commenced. By September, passenger trains of all the railroad lines reopened. By the end of October, railway transport capacity basically reached the pre-earthquake level.

Power Supply

Restoration of the power supply system took place in 4 stages. In the first stage, power from Beijing was transmitted to Tangshan. The Ministry of Power and Water Conservancy and the Beijing Power Administration Bureau organized a work forces to restore operation of the power lines from Yutian to Tangshan by 6 PM the day after the earthquake and to transmit power from Beijing to Tangshan. On the third day after the earthquake, power was restored to water resource locations of the city, airport and Kailuan Coal Mine. In the second stage, incoming power from one circuit was expanded to four directions from Beijing, Tianjin, Chengde and Qinhuangdao with four 110kw circuits. Eight 110kw substations and thirteen 35kw substations were restored in Tangshan Prefecture. On August 7, the lamps on more than 20 streets in downtown were relit. On August 10, the 10 counties in the suburbs of Tangshan regained their power supply. In the third stage, power transmitting capacity was increased. From August 15 to August 21, work on the 220 kw Douhan and Doulu circuits and the 120,000 kw main transformer of Lujiatuo Substation restored 220 kw power to Tangshan and increased the power supply to 100,000 kw. At the same time, 33 substations in the city were returned to operation and by August 19, all substations were restored. On August 27, the power load in Tangshan Prefecture was increased to 138,000 kw; power consumption in coal mines reached 40,000 kw and in factories and mines, it reached 183,000kw. In this stage, the removal and reinforcement of dangerous buildings at other generating units of Tangshan Power Plant allowed quick repair of the generating units. In the fourth stage, restoration of the power plants took place. At the end of September, units No.0 and No.3-6 in Tangshan Power Plant all began to generate power. In Douhe Power Plant, power was generated in Units No. 2, 3, 4, 1 on November 6, 1977, and Feb. 7, August. 10, 1978 respectively.

Water Supply

In accordance with the principle of restoring water resource first and pipe network second, as well as main pipelines first and branch pipelines second, water resources took precedence. Each well was put into operation after repair. In the pipe network, branch lines were turned off to restore the main lines. A trial water supply with strict inspections ensured effective repair. After the main lines were in operation, the branch lines were restored. Restoration proceeded one by one for the local network, forming a small ring. In most parts of the city proper, well water was supplied five days after the earthquake. Prior to this, the city proper was divided into several districts for temporary water distribution.

When the main pipelines were in place, they were connected to the network as part of the water management plan. By October 10, water supply reached its pre-earthquake level.

Medical System

Through the central government, over 8,900 medical workers were transferred from 25 provinces and cities such as Beijing, Tianjin, Shanghai, Jilin, Shandong, Henan, Hubei, Jiangsu, Jiangxi, Gansu, Heilongjiang, Guangdong, Ningxia and the non-disaster areas of Hebei Province to organize a medical and epidemic prevention contingent. According to preliminary statistics, there were 134 medical teams in the city proper. Transported to the scene were 100,000 sets of 365 different medical instruments, over 50,000 kinds of remedies, nearly 1,000 tons of epidemic prevention medicine and vaccine for 1 million people. In less than half a year, clinics, hospital beds and medical personnel reached 80% of pre-earthquake capacity. Commune-level hospitals were generally larger than those before the earthquake. Thus medical and disease prevention standards were guaranteed.

Public Safety and Fire-Fighting

To restore order quickly, the Headquarters of Earthquake Resistance and Rescue in Hebei Province held a conference on July 31. Law enforcement officials of the city and prefecture met and proceeded to restore operations. Fire-fighting brigades likewise restored operations quickly. They disseminated information on fire safety, and helped to prevent any major fires.

Commerce and Grain

People's lives were disrupted, especially in Tangshan City, the epicenter. Grain was buried and cooking utensils were smashed. Since the earthquake took place before down, most people were asleep and thus had no street clothes. During the first few days after the earthquake, a supply system was implemented. Groups were organized to restore the availability of grain, and other staples. Over 7.5 million kg of grain were soon transferred to the disaster areas. By August 14, 108 grain supply shops were back in operation. By the end of October, 23 million kg of grain were available. This came close to the pre-earthquake level of 25.5 million kg. By August 16, the number of commercial services reached 124. Banks began to reopen on August 5. Overall requirements were to salvage goods and restore warehouses. By the end of December, over 520 shops had reopened throughout the city, 74% of those in business before the earthquake. Other outlets were also reestablished, and the supply of daily essentials was basically guaranteed.

PREVENTION OF EPIDEMICS AND ELIMINATION OF DISEASE

Since the death toll was high and the debris was extensive from fallen buildings, it was an arduous task to remove corpses. Headquarters of Earthquake Resistance and Rescue in Hebei Province decided after investigation that the dead should be buried in 8 cemeteries over 5 km away from the city proper at a depth of over 1m. Epidemic prevention personnel and medical teams cooperated closely with the contingents removing corpses and disinfected the places where people had died. In order to eliminate the people's fear that the presence of death would cause disease, Hebei Medical College, the Public Health and Epidemic Prevention Station of the Province, Research Institute of Medical Science, the Research Institute of Soil and Fertilizer Maintenance for the province, and the cities of Tangshan and Qinhuangdao organized a research team. This team collected and analyzed 394 samples of air, ground water, and running water from appropriate locations including fields of cabbages and turnips near the graves. Results proved that the absence of pathogenic bacteria for typhoid, dysentery and cholera. Care was taken to protect water resources and sterilize drinking water. Water resources were guarded by the army or militia. Public health personnel accompanied water delivery wagons to sterilize the water wagon by wagon. The water in vats and barrels for each household or collective unit in tents was also sterilized. As for the countryside, the main task was to repair damaged water taps and motor-pumped wells and to dig out the larger wells. Careful water treatment prevented the spread of enteritis, ensured the control of dysentery and stopped the occurrence of other diseases.

Flies and mosquitoes were a problem in the disaster area. The central government dispatched four airplanes to spray pesticide for Tangshan City, suburbs, eastern mine district and county town of Fengnan on August 9, 16, 23, and September 5, respectively. Airplanes returned on July 28 and August 18, 1977, to spray. Also in the disaster area, 31 fire trucks, over 1,900 various sprayers and more than 50,000 small household dispensers were used for this purpose. Diseases spread by flies and mosquitoes were thus eliminated.

Those with contagious diseases were treated by medical teams and kept in isolation to avoid the spread of infection.

In order to prevent encephalitis, influenza, dysentery, typhoid and cholera, a program o innoculation was implemented the first winter after the earthquake. Adults (4 million) were universally innoculated against influenza. Children (2 million) were universally inoculated against encephalitis (2 million person-times). Thus the incidence of these two diseases dropped by 95% and 71% respectively. When summer came, inoculations were given for the prevention of dysentery, encephalitis B, typhoid and cholera. For dysentery, special emphasis was put on cleaning up the debris, dirt, garbage and contaminants, restoring public health facilities, eradicating pests, paying attention to hygiene and preventing diseases.

Fairly good results were achieved. It was estimated that 1 million people might succumb to diseases the year after the earthquake. Instead, as of August 1977, the number was less than 83,000 and less than 100,000 through December. The inoculation program played an important role in controlling disease. It was viewed as miraculous in our country that no epidemics occurred after the Tangshan earthquake.

RESTORING PRODUCTION

Industrial Production

Principles for restoring production involved giving higher priority to industries with a greater effect on the national economy. Methods for restoring production involved a plan as follows: to organize and finish the jobs suited to one's special training; to remove any dangers and ensure the safety of the tasks at hand; to remove debris and salvage usable material and equipment; to persevere in restoration of local production and stimulate progress.

Coal Mines

Kailuan Coal Mine played an important role in Hebei Province as well as the entire country. The central government paid special attention to Kailuan, emphasizing the resumption of coal mining. First the accumulated water in the pits had to be discharged. It was a tremendous effort. In mid-August, about two-thirds of the underground mine was flooded. More than half of the water-discharge pump rooms and pumps were also flooded. Through the hard work of a national contingent of engineers and personnel from 10 national design institutes, various methods were used. A series of procedures were implemented simultaneously. As a result, the flood receded. Repair and reinforcement could then be done for the tunnels, shafts and chambers. Restoration of above-ground buildings was done at the same time. Damaged machinery, electrical equipment and other components of production were repaired. With the assistance of the army and civilians from all over the country, great results were achieved in Tangshan. Three days after the earthquake, power was restored to the mining district. Ten days after the earthquake, the first batch of coal was produced at Majiagou Mine. Four months after the earthquake, Lujiatuo Mine was totally restored. In one year and a half, 160 million tons of accumulated water in the mines had been discharged, 370 km of tunnels and over 100 coal faces were restored, some 33,000 pieces of motorized equipment were repaired. Over 1 million sq. m of ground surface buildings were restored. As of December 1977, raw coal output reached the pre-earthquake level.

Iron and Steel

Personnel at Tangshan Iron and Steel Corp. joined forces with workers, engineers and technicians as well as a contingent from Anshan Iron and Steel Corp., Baotou Iron and Steel Corp., Wuhan Iron and Steel Corp., Taiyuan Iron and Steel Corp. Also assisting were capital construction engineers from the engineering troops. They concentrated their immediate effort on No. 1 Steelmaking Mill. Less than one month after the earthquake, the first furnace of steel was made. Subsequently, the Medium Steel Rolling Mill, No. 3 Steel

Rolling Mill and Small Steel Rolling Mill were returned to production on September 30, October 1, and November 1, 1977, respectively.

Light Industry

Light industry was damaged by the earthquake with serious consequences for people's livelihood. Thus the effort of industrial restoration made light industry a focal point. This process was characterized by a spirit of self-reliance and arduous struggle. It involved removing the danger actively, doing things simply and thriftily, using the local resources, doing easy things first and difficult things second. All the enterprises of factories and mines restored any production line that could be restored and restored any workshop that could be restored. Through this vast cooperative effort, industrial production was successfully restored. Of 692 disaster-stricken enterprises, 666 enterprises were returned to production; they accounted for 96% of the total affected by the disaster. Of the reactivated enterprises, 514 of them returned to full-scale production. By May 1997, 55% of industrial enterprises returned to production. Those restored to their pre-earthquake level accounted for 50% of the total.

Agriculture

Irrigation and water conservancy were seriously damaged by the earthquake. Serious leakage of large reservoirs occurred. Small and the medium reservoirs were also damaged. Big dams for flood diversion of rivers were cracked by the earthquake and the river beds altered. Most of the motor-pumped wells were tilted and their pipelines were choked with silt. Most of the drainage systems were impaired. Water accumulated on the ground and crops were flooded. Toward the restoration of the agriculture, the State Council called for a bumper harvest in current year. Party organizations at various levels in the countryside handled full mobilization of the masses and comprehensive management of the autumn crops. Two groups of people were organized in each village. Some coordinated the work of construction and living arrangements as well as care of the injured; others coordinated the work of immediate discharge of accumulated water, as well as the weeding and clean up of the wasteland. In Tangshan Prefecture, 2 million laborers took part everyday in disaster relief and autumn harvest efforts.

To help restore agricultural production in the disaster areas, before wheat planting, 3,300 tractors as well as 1600 sets of implements were allocated to Tangshan Prefecture. A large quantity of production items and farm implements were transported to villages for future use to ensure continuity of production. Wheat planted in the prefecture covered an area of 4.1 million mu (Chinese acres), and the task was completed ahead of schedule.

In the latter part of September, a river-harnessing contingent of 200,000 people from 8 prefectures rushed to repair the three reservoirs of Dohe, Yanghe and Qiuzhuang and the four dams of Huanxiang River, Duohe River, Shahe River and Baigezhuang main water canal. Working regardless of the weather, rain or shine, they also completed projects where the Luanhe River enters the sea as well as diverting and draining Luandong. By March, 1977, they were basically finished. At the same time, over 30,000 people from 8 prefectures of the province assisted in well drilling. They were equipped with more than 1,300 sets of drillers, over 200 air compressors, electrical measuring meters and over 800 sets of dies for making well pipes. Their speedy transfer to the disaster areas and cooperation with the masses there ensured the repair of old wells and drilling of new ones. By the end of March 1977, 37,220 new wells had been drilled and 26,410 old wells repaired. This operation returned small irrigation and water conservancy projects to pre-earthquake levels and laid the foundation for an output increase in 1977.

RESTORING NORMAL LIFE

On August 6, 1976, the Headquarters of Tangshan Earthquake Resistance and Rescue of Hebei Province held a meeting on home construction. Earthquake resistance tents and simple houses were standing. Guiding principles were put forward as mobilizing the masses, relying on the collective, using local materials, doing things simply and thriftily, and making gradual improvements. A home construction movement with the general mobilization of leaders and people, and joint participation of military and civilians, soon formed. By the end of October, temporary buildings comprising 451,000 rooms were built in Tangshan City and temporary buildings comprising 1.57 million rooms in the affected counties were built. Each household generally had one or two rooms for living quarters. Before the onset of winter, households which were short of clothes and cotton quilts received those items from the state through a special relief fund. At the same time, distribution of coal for heat was also arranged. In September, the government continued to pay salaries, and paid earnings for August retroactively.

An important focal point of reconstruction was the school system. Workers and staff members in the field of education and culture used their own hands to, first, resume classes and, second, build schools. By the middle of August, nearly 100 primary and middle schools had resumed classes. By September 1, more than 400 primary and middle schools in Tangshan City had done so. Tangshan Mining and Metallurgical Institute which suffered severe damage also resumed classes in the middle of September. Another focal point was loss of family. More than 8,000 elderly and children found themselves alone after the earthquake. Special arrangements were made for widows, widowers and orphans.

Tangshan has been an industrial city for 100 years. Its coal, iron, steel, power plants, and cement production played a key role in the national economy. Tangshan porcelain was used daily. The earthquake caused unprecedented damage to industry, agriculture, everyday live and overall production. To meet the emergency, the party and government transferred manpower and material to help, along with the PLA. Statistics show that in the year after the Tangshan earthquake, 85% of enterprises under Tangshan

Prefecture, Tangshan City and counties restored production totally or partially. Grain and other agricultural output reached the level of the 1974 bumper harvest. By the end of 1977, total industrial output in Tangshan City reached the level of 1975. Normal life for people in the disaster areas was likewise restored.

CONCLUSION

The post-earthquake experience in Tangshan showed that a mass compaign was able to achieve successful restoration.

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EXPLORATION OF RESTORATION AND RECONSTRUCTION IN AN EARTHQUAKE-STRICKEN AREA

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ABSTRACT

This article describes some measures and methods to restore and reconstruct an earthquake-stricken area in light of the experience and lessons drawn from Tangshan and Xingtai earthquakes. Observations may serve for future reference.

KEYWORDS

Earthquake disaster; countermeasure; organization; restoration; reconstruction; fund; management; supervision; inspection; Tangshan earthquake.

INTRODUCTION

An earthquake is a natural disaster, and a strong earthquake causes loss and destruction to a larger extent. Through such disasters, human beings have accumulated a lot of experience in dealing with earthquakes. Anti-seismic prevention is the basis for dealing with earthquakes; earthquake relief is the premise for minimizing loss, restoring production and normal life, and for reconstruction. And restoration and reconstruction are the culmination of disaster relief. The following is a brief exploration of how to restore and reconstruct an earthquake-stricken area based on experience from strong earthquakes in Hebei Province.

RESTORATION AND RECONSTRUCTION

Restoration and reconstruction work are linked to earthquake magnitude and the damage conditions. Governments at different levels in a stricken area should establish earthquake relief headquarters to take charge of relief work and reconstruction. There are several phases of readiness.

Classification of Destructive Earthquakes

Earthquakes of 7 or more than 7 magnitude which occur in densely populated area and cause very serious disasters are severe destructive earthquakes.

Earthquakes of 6 -- 6.9 magnitude which occur in large or mediumsize cities and cause great losses are strong destructive earthquakes .

Earthquakes below 5.9 magnitude which occur in small towns or in the countryside and cause ordinary losses are destructive earthquakes .

Classification of Countermeasures

When an earthquake of more than 7 magnitude occurs and causes a severe disaster, the provincial government organizes anti-earthquake relief, restoration and reconstruction. Departments directly under the provincial government act in coordination. The municipal or county government of the disaster area implements decisions on relief and reconstruction .

When an earthquake from 6 to 6.9 magnitude occurs and causes great losses, the prefectural or municipal government of the stricken area organizes the relief work with the help and direction of the provincial government .

When an earthquake below 5.9 magnitude occurs and causes ordinary losses, the county government organizes earthquake relief work with the help of higher authorities.

Organization of Restoration and Reconstruction

Organization to restore and reconstruct a stricken area continues and refines earthquake relief. Based on classification of destructive earthquake and countermeasures, prefectural and county earthquake relief headquarters should be established. Their commander-in-chief should be the top leader of the Party and government, and deputy commanders-in-chief should be the military leader and head of the army base. Staff of the headquarters should represent such departments as organization, public information, finance, planning, civil administration, construction, seismology, trade, goods and materials, food, communication, Red Cross, telecommunication, public health, agriculture, water conservancy, public security, insurance and banking. An office whose director is the chief secretary of the government should be attached to headquarters. The job of this office is to organize and coordinate public information emergency removal, public order, fire control, materials supply, communication and transportation, financial planning, capital construction and the work of other specialized groups. At the head of each specialized group should be the leader of a given department who implements decisions of the headquarters.

Principles of Restoration and Reconstruction

Restoration and reconstruction work should be done under the leadership of local government and in accordance with the policy of "self-reliance, vigorous struggle, developing production and rebuilding homeland", the principle of "taking the locality as dominant, state subsidy as supplemental and insurance as compensation", and the principle of "being beneficial to production and convenient to the life of the people".

When a particularly severe earthquake occurs, and the task of reconstruction is heavy, the provincial government should appeal to the state for help or to the world for international aid. Meanwhile, all possible social forces should be mobilized to support the restoration and reconstruction.

Lifeline projects, residential houses and productive installations which have an impact on daily life and production, take priority. The relationship between short-term and long-term goals as well as between reconstruction and regional development should be well-coordinated.

Reconstruction should be on the original site. If it is necessary to move, it should be rationally evaluated and authorized by the provincial government or State Council.

Restoration and reconstruction should be in accordance with the principle of "unifying plans and arrangements, stressing the focal points and implementing by stages". Regional resources, social benefits and economic effects should be considered.

If the actual destruction is greater than that figured by a given zonning intensity, the department responsible for that test should redetermine the earthquake intensity.

A principle of restoration and reconstruction is "taking repair as dominant and building new ones as supplemental". After appraisal and review, the design and construction should be integrated. The department in charge of construction makes the final inspection.

Shelters and simple houses during the earthquake relief phase should be built according to the anti-seismic plan and emergency evacuation rules of the city. They should not be in or near a reconstruction site in order to avoid possible demolition.

The interrelated functions of the city should be optimized. Commercial, residential, industrial and entertainment zones as well as sightseeing spots and the natural environment should be well-planned.

Streets, highways, bridges, hospitals and firefighting installations as well as power, water, food, gas and steam supplies must be well-planned in order to improve overall anti-seismic effectiveness.

Anti-seismic and anti-flood functions as well as fire prevention in a city or town should be considered in order to strengthen the ability to withstand natural disasters.

The layout of a city may be improved. Enterprises which cause pollution, use a large amount of water and energy, or need major transportation should be outside the urban area and separated from residential or commercial zones. Industry may be relocated on the basis of advantageous resources, and a better development plan formulated.

During restoration and reconstruction, to avoid waste, buildings which can be repaired or provide material for future use must not be demolished.

Restoration and reconstruction need optimal planning and advanced technology and equipment. It is important to follow the principle of "suiting measures to local conditions, drawing on local resources, conserving investment and lowering construction cost".

Source and Management of Funds .

In restoration and reconstruction, economic problems appear, such as the selection and sequence of the plans and projects, which will all involve expense. But the basic problem is the source of funds. Collecting funds for restoration and reconstruction should include the following.

Work should be based on the principles of self-reliance, vigorous struggle, suiting measures to local conditions, using local resources, restoration on the original place, reusing existing material, taking repair as dominant and rebuilding as supplemental. These are important measures to conserve investment, hasten progress, shorten the recycle period and develop production.

State subsidy and local fundraising facilitate the tasks of restoration and reconstruction. These two sources of funds are important to the earthquake-stricken area.

International aid and loans from world monetary establishments are good sources of funds for reconstruction. For example, a loan from World Bank played an important role in restoration and reconstruction after the Datong-Yangyuan Earthquake.

To mobilize society as a whole for support, various enterprises, public institutions and individuals may raise funds and channel these funds to restoration and reconstruction.

Issuance of construction bonds and special public bonds in domestic financial markets as well as the compensatory money from disaster insurance helps restoration and reconstruction.

Enterprises and companies with strong profits can raise funds by issuing stocks.

In order to use funds rationally and effectively, these funds must be clearly earmarked for each project at the outset. Embezzlement in any department or office at any level is strictly forbidden.

An audit department should supervise the funds to prevent any diversion, waste or corruption.

Construction Management.

Restoration and reconstruction always involve urgent tasks and a diverse staff. Headquarters and administrative departments should not only limit the price of building materials but also supervise their markets.

Damaged structures must be evaluated in terms of anti-seismic functions. A system of authorization for such work as demolishing and rebuilding is necessary.

A careful plan for demolition and relocation must be made, along with arrangements for citizens, public institutions and enterprises in affected areas. Restoration and reconstruction can then proceed more smoothly.

Fundamental installations in a reconstruction area should have unified plan, funds, design, construction and management to assure their quality and suitability.

Reconstruction work by anyone in the affected area must be brought into the unified plan and include effective management and final inspection.

Every project in the reconstruction area must be implemented in accordance with procedures for capital construction and strictly checked for anti-seismic features.

Units or companies which undertake design or reconstruction must be screened for their qualifications. Projects cannot exceed certain limits. Certification is mandatory for designers and builders. Construction to rebuild towns or villages should be done by trained personnel who are technically directed.

An inspection department for construction quality should supervise the entire course of reconstruction and judge it for quality.

During restoration and reconstruction, the government should encourage enterprises, public institutions, social associations and individuals to build residential or commercial installations by collecting money themselves. But the government should offer help by supplying materials, equipment and technology.

Legal Matters.

Government at all levels should praise and award teams or individuals who make outstanding contributions to restoration and reconstruction.

Any teams or individuals who sabotage or delay reconstruction must be punished.

The following should incur legal liability:

a) Those who build houses in violation of the reconstruction plan and cause an accident.

b) Those do construction in violation of the anti-seismic design code and cause an accident.

c) Those who divert or waste construction funds or are otherwise corrupt in handling these funds.

d) Those who use unapproved technology, materials or structure systems and cause an accident.

During restoration and reconstruction period, any criminals, those derelict in their duty or other law-breakers should be prosecuted according to law.

CHARACTERISTICS AND METHODS OF RECOVERY AND RECONSTRUCTION IN EARTHQUAKE-STRICKEN AREA INCLUDING RECONSTRUCTION OF KEPING COUNTY

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ABSTRACT

On February 25th, 1991, a strong earthquake of 6.5 on the Richter scale hit Keping County, Xinjiang, China. Its epicentre was located at east longitude 79°06', north latitude 40°18', a distance of 17 kilometers from Keping county seat. In this quake, three people suffered slight injuries and no one died. However, the quake did heavy damage to Keping county with direct economic loss of 1,500,000 RMB yuan. Since then, local inhabitants have built new houses comprising 50,000 square meters during three years' rehabilitation and reconstruction. Of these, 40,000 square meters are in town (county seat) and 10,000 square meters in villages. Apartments accounted for 80% of newly built projects.

KEYWORDS

Earthquake; recovery; reconstruction; fund; planning; Keping earthquake.

INTRODUCTION

The process of rehabilitation and reconstruction in Keping county can be outlined as follows:

Taking Steps and Countermeasures

a) Making overall arrangements for construction projects and funds
b) Planning in advance for other items

c) Setting up defences against earthqakes

d) Unifying project administration

Learning from Experience

a) Unified management and integral coordination

b) Guiding quake victims and assisting them in rehabilitation and reconstruction

c) Taking a serious attitude toward post-quake rehabilitation and reconstruction of homesteads on a different site

d) Coordinating reconstruction and reformation

PROFILE OF KEPING COUNTY

Keping, a western county under the jurisdiction of Aksu prefecture, lies in the southwestern part of Xinjiang. Its land forms three parts, namely: (1) mountainous areas in the northeast and southwest, (2) a basin and (3) the Aqia plain. The terrain slants from northwest to southeast with elevation of generally from 1,000 to 3,000 meters above sea level. Its maximum length covers a distance of 163.5 kilometers from east to west and its widest point from north to south is 105 kilometers. Keping covers an area of 11,260 square kilometers of which alpine zones comprise 72.4%, plains and basins 27.6%.

Keping is an agricultural county, poor with a population of 35,498, of which 7,100 is urban. It has 3 townships, 1 town, 147 communities and 7,329 households.

Keping is also a frequent locale for earthquakes. From 1904 to 1987, earthquakes measuring 4.75 on the Richter scale struck the county 31 times. Since then, seismic activity has continued. In a time span of only two years (1989 to 1990) earthquakes measuring 3.8--5.4 on the Richter scale struck 10 times. Seismic intensity zones in Keping are 7-8 degrees, and 7 degrees for the county seat. The area of 8 degrees is 440 square kilometers while that of 7 degrees is 1,424 square meters in the isoseismal. On February 25th, 1991, an earthquake measuring 6.5 on the Richter scale struck 17 kilometers from southern Keping county.

Buildings in the county totaled 861,000 square meters. A total of 183,000 square meters suffered damage to various extents in the earthquake, of which 5,000 square meters collapsed, 52,000 square meters were heavily damaged, 61, 000 square meters were moderately damaged, 65,000 square were slishtly damaged. There were 2,102 households affected by this disaster with a population of 10,600. As noted, there were three injuries and no deaths. Total direct economic losses were 15,000,000 RMB yuan.

After the quake, every level of government and its departments were concerned. People from all walks of life were mobilized for anti-seismic efforts and disaster relief. New buildings totaling 50,000 square meters have been completed. Of these, 40,000 square meters are in the county seat and 10,000 square meters in the rural area. Some 80% of newly built projects are dwellings and 480 households in the disaster area have moved into new homes. After the quake, an appraisal was made of older homes, and 7,800 square meters of housing in the county seat were reinforced. Civic facilities returned to normal operation after being repaired. Three new villages have been set up in rural Keping, and households hit severely by the disaster moved into new homes. Essential reinforcements were made to house damaged moderately or slightly by the quake.

Keping County's appearance has improved since reconstruction of townships and villages. Valuable experience was gained as a result of reconstruction work.

RECOVERY AND RECONSTRUCTION

Post-Earthquake Measures

After the quake, Keping implemented the guideline "Self-reliance, developing production, assisting and relieving each other, reconstructing homesteads". Furthermore, Keping abided by the construction principle, "overall planning, due consideration to both short and long-range targets, adaptation to local conditions, and implementation step by step". It took Keping a little more than 3 years to complete reconstruction with better quality and fewer capital funds.

Leadership for recovery and reconstruction of homesteads was set up with the government leaders as its primary members. A special office for recovery and reconstruction was established to take charge of overall planning, site choice, architectural design, building material production and supply, fundraising and expenditure, and construction management as well as quality control. In the meantime, the masses should be encouraged to go forward with an indomitable spirit, build confidence, work in unity and help one another, rely on collective wisdom and work with collective ideas, and make a concerted effort to reconstruct homesteads well by means of publicity and education.

Overall Arrangements of Construction Projects and Funds

After an analysis of projects, they were put in order of importance. Projects concerning the life of the people took priority, being followed by production. Priority also went to projects closely related to households in heavily damaged areas and to emergencies. A budget allocated funds in advance for key construction projects. Thus the situation of 80% of local victims was handled in the same year, and post office buildings and barns went into operation after construction was completed.

Planning Stage of Construction

In order to meet the need for development in the county seat, overall planning was aimed at improvement of public facilities and construction of permanent houses while curbing random construction of houses here and there. The process of rural reconstruction worked as follows. First, three new villages were built with aid from Italy. Second, planning staff conducted a study-tour in the hinterland. Third, in line with local conditions and national characteristics, projects were selected for implementation.

Proper-Seismic Prevention and Reinforcement

Stricter inspection must be made for reconstructed buildings so that they conform to anti-seismic requirements. An appraisal of the existing buildings and public facilities showed where reinforcement was necessary. Before the quake, a pilot group of anti-seismic houses had been built in Keping county. After the quake, to our delight, houses with that layout and structure remained safe and sound. Those without such features would collapse, which not only offers a vivid textbook in anti-seismic rural housing but also serves as an example for recovery and reconstruction.

Unified Management of Reconstruction

Owing to the urgency of reconstruction as well as the onerous task, special measures were taken. Housing for those who suffered most during the earthquake had to be completed before winter. There was a shortage of local construction staff but personnel were available from other places. Thus a group of qualified staff was selected, and their credentials checked. We opened a competitive bidding process and prohibited subcontracting for projects. Unified management was part of the process. Budget monitoring and approval were firmly controlled.

Reflections on Experience

Recovery and reconstruction need unified management and coordinated procedures. Government leaders from the quake-stricken area should take command personally. A group should then be formed to coordinate all projects. Members of this group should come from different departments, including finance, civil affairs, construction, materials and planning commission. At the same, time all phases of work should follow the prorer sequence in order to stay on schedule and ensure the safety of projects.

People in a disaster area need proper guidance for recovery and reconstruction. Keping is a poor county where the masses, after the earthquake, had a "dependent mentality" to seek relief from the nation. People can be mobilized to base their goals on self-reliance, to wit, trying to raise funds on their own and generating enthusiasm through productivity. Reconstruction money comes from three sources: (1) insurance companies, (2) the state, and (3) local production of native goods and some raw material. Reconstruction projects cost 8 million RMB yuan, of which one third came from state subsidies or outside sources, one third from mutual assistance funds of various departments, and one third from local self-reliance.

Relocation had to be dealt with in a serious manner after the quake. A new village was required by the government to built in the outskirts of the county for 100 hard-hit households with aid from Italy. But the construction plan changed to establish three villages composed of 16,40 and 44 households. Leaders at all levels had studied the situation with engineers and technicians, and solicited their advice. Villagers had discussed the situation with unit leaders. Several factors emerged. Farmers were unwilling to leave their homeland. Less capital was needed for infrastructure and public works in dispersed than in centralized construction. Basic demands for daily life and production could be met sooner. After a geological survey of the area intended for the new village of 100 households, the site proved unsuitable due to crushed strata and a rift zone. Finding the facts led to better results.

Transformation should go hand in hand with recovery. Restoration cannot restore the past .The effort should be combined with basic design changes as well as social and economic development. A general plan for Keping county seat and its townships had been drawn up and put into operation by the end of 1980s. In the 1991 quake, buildings and infrastructure suffered heavy damage. Before rebuilding, any pre-earthquake plan should be modified and improved. To strengthen the town's overall anti-seismic capability, a set of infrastructures should be completed and public welfare establishments should function with higher standards. Before a new project is launched, consideration should be given to its grace as well as its harmony with other structures. There is an opportunity for beautification along with reconstruction.

Keping is a key county in the anti-seismic efforts of Xinjiang. Local government pays close attention to this work. In 1987, a primary group was formed to oversee these efforts. In 1989, plans for earthquake protection and disaster reduction were drawn up. An annual event is scheduled around July 28th when a major earthquake struck Tangshan, Hebei province, in 1976. The occasion is designated "Publicity Week" for earthquake protection and disaster reduction. In 1990, a knowledge contest, "Anti-quake and Prevention of Disaster" was held nationwide for the first time. Before the earthquake, public buildings and structures closely related to people's lives were reinforced. In the outlying area, a pilot group of anti-seismic houses was built. Before the quake, many preventive measures were in effect so losses were reduced. During the quake, people remained calm, society maintained normalcy and relief work proceeded in an orderly manner. After the quake, the experience was summarized as "Six Differences". It makes a difference whether or not there are plans for anti-quake and prevention of disaster, whether or not these plans are put into action, whether or not old buildings and houses are reinforced, whether or not publicity drives for anti-quake and prevention of disaster have ever been done, whether or not the location of buildings is all right, whether or not project designs include defence against quakes, whether or not the quality of a project is guaranteed. In the past three years, Keping county has completed its recovery and reconstruction work as well as improved its economy. There is a new outlook characterized by harmonious relationships between different ethnic groups, a stable society and a better life for the people.

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MANAGING A SEISMIC VULNERABILITY DATABASE

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ABSTRACT

The paper discusses the possibility of exporting, to different national situations, the Italian experience in assembling and exploiting the level-II vulnerability database made available for masonry buildings. Attention is focused on the collection of the data which are required to assemble the database and on the procedure for consulting it. An expert system can drives such operations: it must be able to detect operator mistakes and to check the global consistency of the data. The benefit of consulting the database for optimizing the rehabilitation design of existing masonry buildings is emphasized. The aftershock update of the database via causal probabilistic networks is eventually discussed.

KEYWORDS

Causal Probabilistic Network; Database; Expert System; Masonry; Rehabilitation Design; Strengthening; Vulnerability.

INTRODUCTION

The authors have recently published (Casciati *et al.*, 1994) an automatic procedure of consultation of the Italian vulnerability database for masonry buildings. It is made of 100,000 buildings, that were inspected to fill a level-I form (GNDT, 1989), and 1,500 buildings for which the level-II vulnerability form was compiled. Local governments and the Italy Group of Earthquake Mitigation (GNDT) of the National Research Council (CNR) sponsored these surveys in the field. Immediate help toward structural decisions can only be found within the second, smaller set of buildings. These data cover the building type and the material characterization in the different administrative areas, the result being a statistical description of the vulnerability index for a given building type. The evaluation of the vulnerability index is pursued through a heuristic approach that exploits the available expertise and simple structural mechanics schemes (Benedetti and Petrini, 1984; Bernardini *et al.*, 1990).

DATABASE AND VULNERABILITY INDEX

The masonry database assembled by Baratta and Zuccaro (1993) contains 1,745 GNDT vulnerability forms of level-II. Only 1,595 of these files appear to be undefective.

Table 1 - Table of contents for a file of the vulnerability data base.

Field	Type	Description	Field	Type	Description
1	character	province code	30	real	total area of the building (it. 3)
2	integer	county code	31	real	area of the walls along dir. x (it. 3)
3	integer	sequential form number	32	real	area of the walls along dir. y (it. 3)
4	character	number of the form-group	33	real	shear strength of the building (it. 3)
5	character	item 1 classification	34	real	interstory height (it. 3)
6	character	inform. quality for item 1	35	real	dead load (it. 3)
7	character	item 2 classification	36	real	permanent load (it. 3)
8	character	inform. quality for item 2	37	real	land slope (%; item 4)
9	character	item 3 classification	38	real	site condition (it. 4)
10	character	inform. quality for item 3	39	real	difference in level (it. 4)
11	character	item 4 classification	40	integer	presence of leapt stories (it. 5)
12	character	inform. quality for item 4	41	integer	hor. element connection (it. 5)
13	character	item 5 classification	42	real	connected elements (%; item 5)
14	character	inform. quality for item 5	43	character	main plan parameter (%; item 6)
15	character	item 6 classification	44	character	secondary plan parameter (%; item 6)
16	character	inform. quality for item 6	45	real	mass vertical variation (%; item 7)
17	character	item 7 classification	46	real	vertical conf. parameter (%; item 7)
18	character	inform. quality for item 7	47	real	portico area (%; item 7)
19	character	item 8 classification	48	integer	gr. floor portico presence (it. 7)
20	character	inform. quality for item 8	49	real	wall distance over thickness (it. 8)
21	character	item 9 classification	50	character	roof type (it. 9)
22	character	inform. quality for item 9	51	integer	ext. beam presence (it. 9)
23	character	item 10 classification	52	integer	chain presence (it. 9)
24	character	inform. quality for item 10	53	real	roof permanent load (it. 9)
25	character	item 11 classification	54	real	roof length (it. 9)
26	character	inform. quality for item 11	55	real	roof perimeter (it. 9)
27	real	structural organization (it. 1)	56	integer 1	property (from level 1)
28	character	resistant system quality (it. 2)	57	integer	condition of use (from level 1)
29	integer	number of the stories (it. 3)	58	integer	occupancy (from level 1)

Their information was extended by some items coming from the corresponding level-I forms. The structure of a single file of the data bank consists of 58 items (see Table 1): 55 items correspond to assessment elements in the level-II vulnerability form; 3 items come from the GNDT level-I form: property of the building, condition of use, and occupancy.

The resulting data bank of 1,595 buildings covers 11 provinces of Italy (indeed, only 1,194 files are complete and can be used within a statistical analysis). It includes 116 one-story buildings (111 with a complete set of data), 454 two-story buildings (431), 600 three-story buildings (464), 346 four-story buildings (153), 73 five-story buildings (31), 5 six-story buildings (4) and 1 seven-story building (0).

Data Base Organization

The data base was stored into two files of different formats. One has the suffix DBF; it is a DBASE-file and must be managed by the general purpose database manager DBASE (1989; DBIT, 1992). The second file has the suffix "TXT" and is an ASCII file to be used by the consultation code BUILD. This code acts as a data selector intended to facilitate the use of the vulnerability analysis program (Casciati *et al.*, 1994). To run the code BUILD involves the following steps:

- at start, the list of the data fields, among which the selection can be operated, appears on the screen;

Table 2 - Class values and weights for the GNDT vulnerability index.

Item i	Class value S_i				Weight W_i
	A	B	C	D	
1	0	5	20	45	1.00
2	0	5	25	45	0.25
3	0	5	25	45	1.50
4	0	5	25	45	0.75
5	0	5	15	45	1.00[a]
6	0	5	25	45	0.50
7	0	5	25	45	1.00[b]
8	0	5	25	45	0.25
9	0	15	25	45	1.00[c]
10	0	0	25	45	0.25
11	0	5	25	45	1.00

[a] if the horizontal elements have very good connections the weight takes the value 1; 0.5 is considered for bad connections and a linear interpolation in the intermediate case.

[b] if there is the portico in the building $W_7 = 1$; else $W_7 = 0.5$.

[c] one considers two events: high values of the dead load and low percentage of actually supported roof: for both events true $W_9 = 1.5$; for one of them true $W_9 = 0.75$; otherwise $W_9 = 0.5$.

- the user selects the search conditions;
- as a result some files are created. In particular the information required by the GNDT level-II form is saved in the file ARCHGNDT.

If some information needs to be updated, it will be done by running the code DBASE. Then, one transfers the updated DBF-file to the ASCII file.

Vulnerability Assessment

The method of classifying masonry buildings in seismic area proposed by Benedetti and Petrini (1984) makes use of a numerical value, called the "vulnerability index". It represents the seismic quality of each building and is obtained as a weighted sum of the numerical values expressing the quality of the structural and non-structural elements of interest collected in the level-II vulnerability form (Benedetti *et al.*, 1988; Casciati and Faravelli, 1991a and 1991b). This form has to be filled for each building of interest. It makes use of the qualitative and quantitative answers about some assessment elements. These answers are used to classify 11 decisional items into one of four classes (A; B; C and D). A vulnerability index is then reached by assigning, by further expertise, a value S_i to each class and a weight W_i to each of the classified items:

$$VI_g = \sum_{i=1}^{11} W_i S_i \tag{1}$$

The values of W_i and S_i are given in Table 2 (Benedetti and Petrini, 1984; Benedetti *et al.*, 1988; Casciati and Faravelli, 1992).

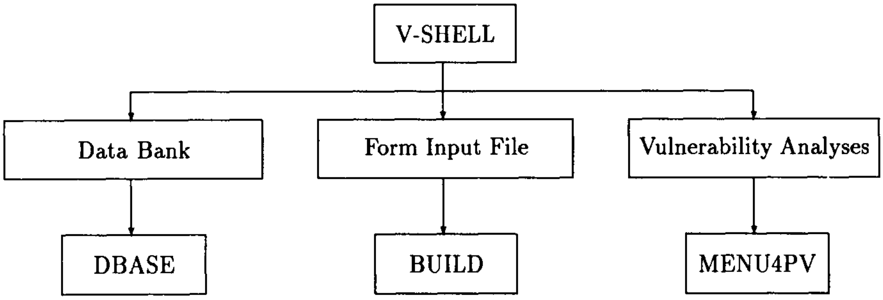

Figure 1: Flow-chart of the vulnerability shell (V-SHELL)

Table 3 - Sample size, means, standard deviations and coefficients of variation of VI_g arising from vulnerability database consultations for different provinces (from Casciati *et al.*, 1994); the range of definition is (0, 405).

Province	Sample Size	Mean Value	Stand. Dev.	Coef. of Variation
76	271	146	36	25%
46	469	92	48	52%
35	120	81	41	50%
45	95	74	37	50%

VULNERABILITY SHELL AND ITS USE

The code V-SHELL is a window environment under which the data bank can be consulted, the vulnerability analysis developed and statistics of the results inferred. It includes the following functions (see Figure 1):

1. the option (code) DBASE manages the seismic vulnerability data bank;
2. the option (code) BUILD extracts the data from the data bank and saves them in three appropriate files for the successive vulnerability analysis;
3. the code MENU4PV performs the vulnerability analysis and computes the statistics of the results over the sample extracted by the option BUILD.

Assume one must design the rehabilitation scheme of a masonry existing building. A level-II vulnerability form for it can easily be filled, so that a preliminary understanding of its behaviour is achieved. The building is located in a certain province: a consultation of the data bank with this input provides information of the type summarized in Table 3. Similarly, if the number of stories in the building is known, the data bank can be consulted with this input and examples of results are given in Table 4. These tables just show how the mean vulnerability index changes from province to province (the largest value is two times the smallest one in Table 3) and how it increases as the number of stories increases. Moreover, as a general rule the coefficient of variation decreases as the mean increases.

Bivariate input selections can also be operated. Table 5 illustrates the interaction between province and number of stories.

Table 4 - Sample size, means, standard deviations and coefficients of variation of VI_g arising from vulnerability database consultation for different values of the number of stories (from Casciati *et al.*, 1994); the range of definition is (0, 405).

Story number	Sample Size	Mean Value	Stand. Dev.	Coef. of Variation
1	111	90	49	55%
2	431	101	51	50%
3	464	108	50	46%
4	153	118	44	37%

EXPORTING THE EXPERTISE

Unlimited financial resources could permit each local governement to repair and strengthen the whole historical core of the municipality. Obvioulsy, this is not the general case. Before strengthening monumental sites, it is necessary to assess the actual structural vulnerability of their components. The knowledge of the liability of the different buildings to suffer damage, in fact, makes possible to establish the priorities and to prepare a long-term economical plan of prevention and mitigation. The limited amount of financial resources which precludes one from repairing and strengthening all the existing (masonry) buildings, also prevents from a large scale survey. The results of the previous investigations on masonry buildings are therefore especially valuable since their statistics can be used to have a preliminary diagnosis (Casciati and Faravelli, 1992). Some results of this statistical analysis can be regarded as an actual support for assessing the state of health of the building.

The basic quantities that characterize a given building can be collected by consulting a suitable expert system which summarizes the available expertise. The expert system is built by writing the governing sequence of rules within any selected commercial "shell".

Implementing the Data Collection Expert System

A shell already includes:

the *inference engine* which processes the answers to some questions; they are automatically posed in order to reach a conclusion over the topic for which it is consulted;

the *text editor* by which a text is compiled and, hence, the knowledge base is made available to the inference engine; the text is a sequence of rules, prepared by an expert pool;

the *consultant* the activation of which (by an operator) provides the answers that the inference engine needs during its deductive path (either backward or forward chaining).

An early efficient shell was INSIGHT 2+(1984) . It was used to implement a "Vulnerability Assistant" expert system in (Casciati and Faravelli, 1988). A "confidence measure" is also available in the shell. When the operator is required to provide an answer to a question, YES or NO (TRUE or FALSE) are generally two extreme situations. It is much more likely that the answer the operator can provide is "almost surely true (or false)" or "may be true (or false)". The shell permits the operator to cover all the range between 0 (FALSE) and 100 (TRUE). The consultation of such an expert system provides two kinds of results: the resulting classification for the item under investigation and the quality of the information which led to this classification. This quality is expressed by the resulting confidence measure. A number in the range (0,100) substitutes therefore the naive concept codified in the original GNDT form of four different degrees of confidence. They are: E (high quality), M (average), B (low quality) and A (operator's guess).

However, this confidence measure is also the weak point of the shell INSIGHT2+. The reason is that the confidence calculations are driven directly by the inference engine. In other words the expert

Table 5 - Sample size, means and standard deviations of VI_g arising from vulnerability database consultations for different values of province and number of stories (from Casciati *et al.*, 1994).

Province	Number of Stories	Sample Size	Mean Value	Standard Deviation
76	2	93	150	36
76	3	124	147	34
76	4	39	142	33
46	1	46	83	52
46	2	184	89	45
46	3	188	90	50
46	4	48	116	40

who builds the knowledge base is unable to dominate these calculations. For instance, a conclusion is reached when the confidence on it is greater than a value fixed by the expert, but no mention is made on the alternative events. Even with a low confidence in their occurrence, in fact, these events can significantly influence the deductive process. Moreover, the limit value fixed by the expert cannot be too low or too high, otherwise the conclusion comes out to be too optimistic or too pessimistic, respectively. The authors prepared therefore a more sophisticated algorithm within the "working environment" NEXPERT (1989). It is obtained by building inside the knowledge base a logic treatment of uncertainty accounting for the elements listed above and other minor aspects. This, of course, is made by additional rules which condition the influence engine process (Casciati and Faravelli, 1991b). At the end of the consultation one also obtains the probability mass function of the vulnerability index and some central measures (Casciati and Faravelli, 1992).

Exploiting the Expert System and the Database

Both these expert systems were made available to local gouvernments and/or authorities interested in exploiting their potentialities. Once the properties of the single building under investigation have been assessed through the expert system, the vulnerability index is estimated. Moreover, by consulting the available data base, reliable ranges are obtained for the vulnerability one can achieve by an adequate strengthening of the building.

A DIAGNOSIS APPLICATION

Figure 2 shows the oriented acyclic graph for the structural diagnosis example developed in (Casciati and Faravelli, 1995). There is a spectrum of six limit states: each of them has links with seven quantities which can be measured, observed and/or monitored. The total number of nodes is 14. The limit-state nodes express, by means of two states, yes, Y, and no, N, the presence of:

1. external subsidence (node 2) influenced by the geotechnical report (node 8);
2. excessive stress (node 3) influenced by the results of tests executed by flat jacks (node 9), dilatometers (node 10) and material specimens (node 11);
3. internal subsidence (node 4) influenced by the readings of the dilatometers (node 10) and the readings of the transducers at the cracks (node 12);
4. material deterioration (node 5) influenced by the results on the material specimens (nodo 11) and by the readings of the crack transducers (node 12);

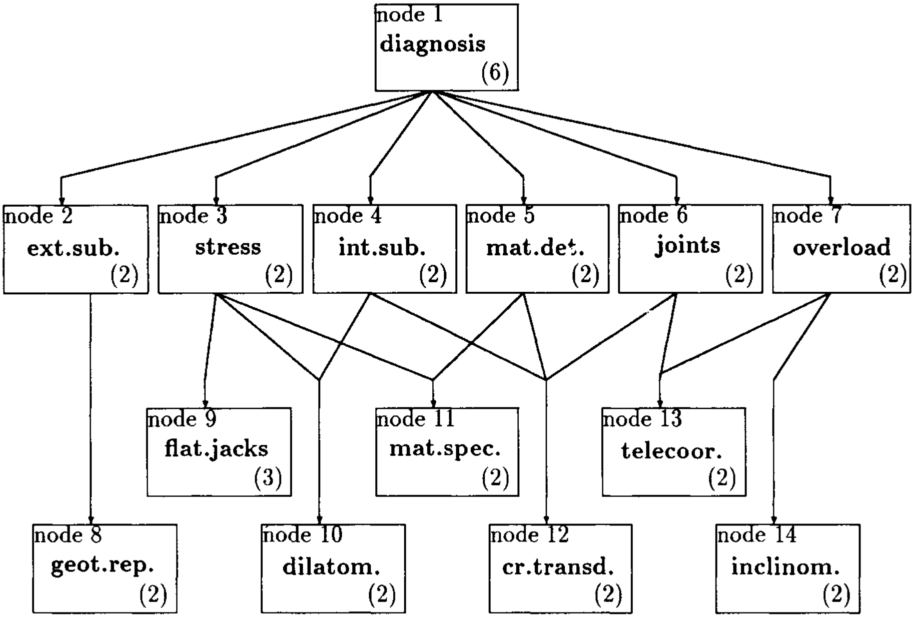

Figure 2 - Oriented acyclic graph for the numerical experiment: the number between brackets is the number of states of the discrete variable associated with each node.

5. inadequate joints between vertical and horizontal elements (node 6) influenced by the measures of the crack transducers (node 12) and of the telecoordinometer (node 13);

6. overload (node 7) detected by the telecoordinometer (node 13) and the top inclinometer (node 14).

Table 6 - Subjective evaluation of the conditional probability associated with the links (node 1 $\rightarrow$ node 2) and (node 2 $\rightarrow$ node 8). The diagnosis concerns with an anomalous state of `stress`, `external subsidence`, `internal subsidence`, `material deterioration`, `overload` and `joints`.

	node 2: ext. subsidence ?	
node 1: limite state ?	Yes	No
`stress`	0.10	0.90
`ex. subsidence`	0.90	0.10
`in. subsidence`	0.05	0.95
`material det.`	0.05	0.95
`overload`	0.10	0.90
`joints`	0.10	0.90

	node 8: geotech. rep. ?	
node 2: ext. sub. ?	Yes	No
Yes	0.90	0.10
No	0.05	0.95

Each of the six intermediate nodes corresponds to a load effect to be compared with the corresponding strength: the diagnosis consists in classifying the state of health of the structure. The graph transformation leads one to build the joint tree where the qualitative aspects till now considered must then be replaced by quantitative estimates in view of the assessment of the relevant probabilistic structure. An example of subjective evaluation of the conditional probability table is provided in Table 6.

Evidence propagation

Once the joint tree has been obtained and one incorporates the evidence that one of the bottom item is true, the probabilistic structure of the network is modified; these changes are then transferred into a modified degree of confidence for the single diagnosis states.

Learning

When the conditional probabilities are estimated on a subjective basis, a range for them can also be specified: this makes explicit the uncertainty which characterizes these values. An example of ranges for these probability is given in Table 7, together with the corresponding values of the variables ϑ_i of the theoretical model introduced in (Casciati and Faravelli, 1995); the values of Table 6 become in this case the a priori means of the probability parameter. An updatings of the a priori distribution for different cases of evidence via a Bayesian deductive scheme was then implemented in (Casciati and Faravelli, 1995).

Table 7 - Subjective estimation of the conditional probabilities associated with the links (node 1 $\rightarrow$ node 2) and (node 2 $\rightarrow$ 8). The corresponding ranges are provided together with their translation into values of the parameters ϑ_1 and ϑ_2 of the beta distribution.

node 1: limit state ?	nodo 2: Yes		ext. sub. ? No	
	range	ϑ_1	range	ϑ_2
stress	.05-.20	0.779	.70-.99	5.260
ex. subsidence	.70-.99	5.260	.05-.20	0.779
in. subsidence	.02-.08	1.280	.90-.99	24.30
material det.	.02-.08	1.280	.90-.99	24.3
overload	.05-.20	0.779	.70-.99	5.260
joints	.05-.20	0.779	.70-.99	5.260
node 2: subsidence ?	node 8: Yes		geotech. rep. ? No	
	range	ϑ_1	range	ϑ_2
Yes	.80-.99	9.320	.02-.15	0.890
No	.01-.10	1.410	.90-.99	24.26

The network of Figure 2 can easily be replaced by substituting disgnosis with vulnerability and the bottom items with the 11 ones which characterize the GNDT level-II vulnerability form. Examples can be found in (Faravelli and Gherardini, 1992a and 1992b).

CONCLUSIONS

This paper illustrates the developments in progress in Italy in the field of vulnerability assessment. The adoption of expert system shells in particular, opens the way to a large variety of improvements.

Many of the parameters to be estimated and/or investigated, are uncertain quantities due to the building technology which characterized the past centuries. A scheme of uncertainty treatment was implemented in (Casciati and Faravelli, 1989, 1991b and 1992).

This paper emphasizes the help to the rehabilitation design of masonry buildings that can be provided by a statistical use of seismic vulnerability database Further developments are required toward a diffusion of such a preliminary design approach and, in particular, a comparison between the databases of different countries could avoid the bias associated with the basic homogeneity of the Italian data which form the database in its present form.

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THE CRITERIA FOR ASSESSMENT OF SEISMICALLY DAMAGED DEGREE OF BUILDINGS IN JAPAN

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ABSTRACT

The damage classifications for the seismically damaged buildings (Reinforced concrete structures, steel structures and wood structures) are presented along the guidelines published in Japan. The guidelines, in general, provide two levels of damage evaluation; Rapid Evaluation and Overall Damage Classification, including repairing techniques. Rapid evaluation begins with a quick visual inspection for the general level of damage, and then within a day or two days, determines the building usability to avoid the potential damage due to aftershocks. The second level classification as a next step is the more detailed overall damage classification to be done during at least about a week to about a month decide whether repairing and/or strengthening are needed for the long-term continued use of buildings, or whether the building should be demolished or removed. Based on the overall damage classification, the appropriate action or retrofitting is applied to the buildings. These classifications , even if already determined, may be changed by reinspection due to new found damage or aftershock, or reinspection after emergency repairs.

KEYWORDS

Building safety evaluation; rapid damage evaluation; overall damage clasification; building usability; retrofitting; repairing; strengthening; reinforced concrete structures; steel structures; wood structures.

INTRODUCTION

As is well-known, Japan is quite seismically active zone. During the past half a century in Japan, however, respective occasional major earthquakes have resulted in a loss of less than 50 human lives due to the collapse of buildings and slope instabilities, unfortunately except due to tsunami disaster. Tsunamis, tidal waves, accompanied with two major earthquakes, Nihonkai-Chubuoki Earthquake in 1983 and Hokkaido-Nanseioki Earthquake in 1993, deprived about 300 people of their lives.

Until 1980, earthquake resistant regulations in the three major seismically active countries such as Japan, Greece and Italy, consisted of statically oriented concepts ignoring some key dynamic factors of structures. In 1981 the Japanese earthquake resistant regulations were drastically revised, taking account of key dynamic characteristics of structures. In the meantime, some of the existing buildings could not conform to the new regulations any more. So the guidelines for reinforced concrete (RC) structures and steel structures were published, instructing how to retrofit structures. These guidelines provide procedures to estimate the seismic resistant capacities of structures, earthquake input forces, and strengthening techniques. For estimation of seismic resistant capacity two steps of approaches are demonstrated: a quick review for the buildings in problem, and a detailed diagnosis to provide the data bases for strengthening of the buildings in problem.

In the late 1980's, along the line of the guidelines introduced above, the technical committee for "The criteria for assessment of seismically damaged degree of buildings and technologies to repair them," chaired by Professor Okada, were organized. The committee, based on the established technologies of a quick review for earthquake resistant capacities, completed the guidelines of the three separate text books dealing with RC structures, steel structures, and wood structures respectively, which were unfortunately written in Japanese.

Although contents of the guidelines seem to be satisfactory, one of the serious problems at the occasion of the real major earthquake disaster is how to realize or perform the instructions in these guidelines. If volunteer engineers or well-trained, experienced engineers to estimate degrees of damage to buildings would be in short supply, the guidelines might be left as unrealistic documents. To avoid such miserable results and to realize the instructions in the guidelines, every effort must be made.for instance,by training engineers, or by seeking additional manpower. And further cooperation and consultation with relevant researchers and local governmental staff will be urgently needed.

This report pays more attention to the damage classifications for three different types of structures rather than to techniques for repairs, and along the line of the guidelines describes the general procedures for building inspection, followed by the respective damage evaluation procedures for RC structures, steel structures, and wood structures.

GENERAL PROCEDURES FOR BUILDING INSPECTION

Immediately after a damaging earthquake, building safety evaluation must be done by the local well-trained, or qualified structural engineers. Figure 1. illustrates the general damage evaluation procedure for ordinary buildings, such as RC structures, steel structures, and wood structures. Regardless of the structural systems, the damage evaluation is based on the similar concept. The only difference is the criteria for damage rating and building usability. The guidelines, in general, provide two levels of damage evaluation: Rapid Evaluation and Overall Damage Classification, and also techniques for repairs.

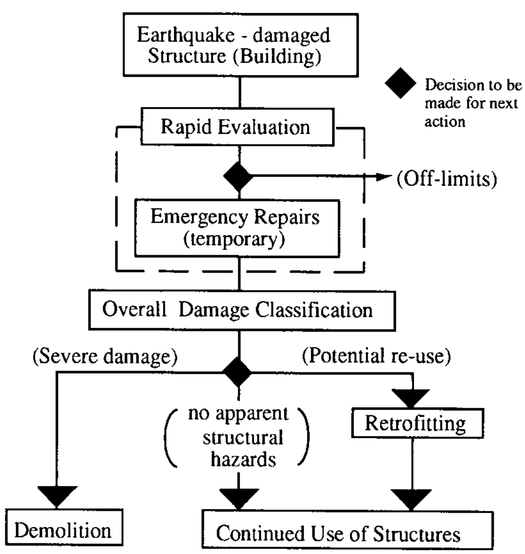

Fig. 1. General procedures for building safety evaluation

Rapid evaluation begins with a quick visual inspection for the general level of damage, and then within a day or two days, determines the building usability to avoid the potential damage due to aftershocks.

Some emergency repairs may be applied for temporary use of buildings until the overall retrofitting begins, depending on the damage classification. When applying emergency repairs, first of all, the vertical load system should be a major consideration, and secondly the lateral load system.

The second level classification as a next step is the more detailed overall damage classification to be done during at least about a week to about a month decide whether repairing and/or strengthening are needed for the long-term continued use of buildings, or whether the building should be demolished or removed. Based on the overall damage classification, the appropriate action or retrofitting is applied to the buildings. These classifications , even if already determined, may be changed by reinspection due to new found damage or aftershock, or reinspection after emergency repairs.

REINFORCED CONCRETE STRUCTURES

Structures dealt with in this section are RC structures: cast-in-place concrete frame structures, and wall structures, including other structural systems such as steel reinforce concrete (SRC: concrete reinforced with structural steel core) structures, and reinforced concrete block structures. The damage evaluation can be applied to precast concrete structures; however, more versatile consideration must be required, especially for the connections of precast concrete members. The buildings of 31 meters or higher, or ten or more stories require another detailed damage evaluation procedure, which is not available at the moment.

Rapid Evaluation

Inside/Outside Inspection. Figure 2 illustrates the flowchart showing Rapid Evaluation for RC structures. Firstly the outside of the building is inspected in terms of building settlement, leaning, structural element damage, falling hazards, and overturning hazards. Secondly the inside of the building is inspected in terms of structural element damage, falling hazards, and overturning hazards. If the outside inspection rates the building Dangerous, the inside inspection is not necessarily required. Unless otherwise, the inside inspection is required.

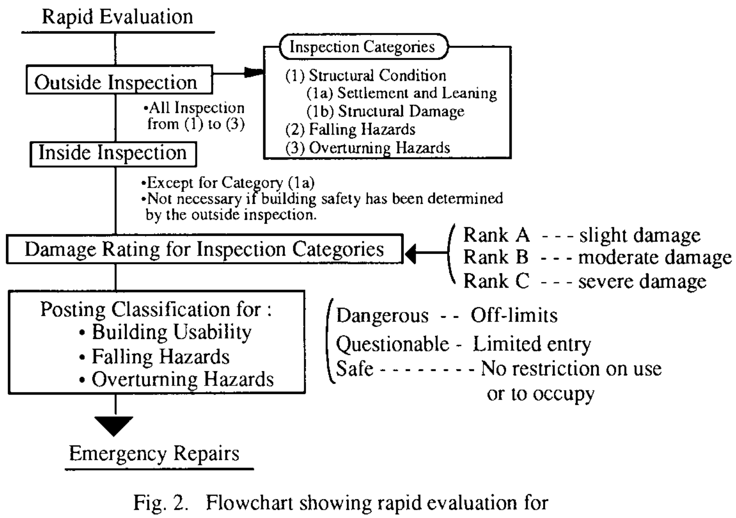

Fig. 2. Flowchart showing rapid evaluation for reinforced concrete structures

Damage Rating. Damage is rated for the three inspection categories; building settlement, leaning, and structural element damage as shown in Tables 1 , 2 and 3 respectively. Building settlement is rated from Rank A (slight) to Rank C (severe) based on the amount of settlement. Building leaning is also rated likewise. Damage rating for structural elements is based on a damage ratio, defined as a ratio of the numbers or lengths of damaged structural elements and the interior/exterior structural elements inspected. For detailed rating of degree of damage to structural elements, refer to Table 4.

Table 1. Building settlement rating (RC structures)

Ratings	Criteria S (m) : Settlement
Rank A	$S \leq 0.2$ m
Rank B	$0.2 \text{ m} < S \leq 1.0$ m
Rank C	$S > 1.0$ m

Table 2. Building leaning rating (RC structures)

Ratings	Criteria θ : Leaning Angle
Rank A	$\theta \leq 1/60$
Rank B	$1/60 < \theta \leq 1/30$
Rank C	$\theta > 1/30$

Table 3. Damage rating for structural elements (inside / outside inspection : RC structures)

Ratings	Criteria
Rank A	$N_{IV}/N < 10$ %, or $N_V/N < 1$ %
Rank B	$10 \% \leq N_{IV}/N < 20$ %, or $1 \% \leq N_V/N < 10$ %
Rank C	$N_{IV}/N \geq 20$ %, $N_V/N \geq 10$ %, or apparently totally damaged

Note N_{IV} : The number of interior or exterior columns rated IV of damage degree,
N_V : The number of interior or exterior columns rated V of damage degree,
N : The Total number of interior or exterior columns inspected.

Table 4. Rating of degree of damage to structural elements - columns / beams / structural walls - (RC structures)

Ratings	Description
I	Invisible cracks of about 0.2 mm wide or less unless closely observed.
II	Visible cracks of about 0.2 - 1 mm wide.
III	Large cracks of about 1 - 2 mm wide ; a few concrete spalling.
IV	A lot of large cracks of about 2 mm wide or more ; concrete severely spalling ; and reinforcement exposed.
V	Core concrete severely crushed ; reinforcement exposed and severely buckled, or broken ; columns extremely vertically deformed and leaning.

Note structural damage rating is done to the most severely damaged story of a building.

Building Usability Classification. Using these classifications for building usability and other hazards as indicated in Tables 5. and 6. respectively, posting classification for the building can be determined as "Off-limits" if either of the three categories rated Dangerous, "Limited entry" if either of the three categories rated Questionable, or "No restriction on use or to occupy" if all of the three categories rated Safe. Table 7. shows the criteria for the posting classification.

Table 5. Building usability classification (Rapid Evaluation : RC structures)

Building Usability Classification	Criteria (Number of Ranks)	Inspection Categories
Dangerous	Rank C ≥ 1 or Rank B ≥ 2	-Leaning and Settlement -Structural Damage
Questionable	Rank B ≥ 1	-Leaning and Settlement -Structural Damage
	Degree of damage III or more	-Structural Damage
Safe	None of the above criteria	

* Damage to structural elements is rated on a scale from I (slight) to V (severe)

Table 6. Posting safety classification for falling and overturning hazards (Rapid evaluation : RC structures)

Posting Classification	Criteria (Number of Ranks)
Dangerous	Rank C ≥ 1 or Rank B ≥ 2
Questionable	Rank B ≥ 1
Safe	None of the above criteria

Table 7. Posting classification based on rapid evaluation
(RC structures)

Categories / Posting Classification	Building Usability	Falling Hazards	Overturning Hazards
Off-limits	Dangerous	- any -	- any -
	- any -	Dangerous	- any -
	- any -	- any -	Dangerous
Limited entry	Questionable	- any -	- any -
	- any -	Questionable	- any -
	- any -	- any -	Questionable
No restriction on use or to occupy	Safe	Safe	Safe

Emergency Repairs. Emergency repairs are temporarily applied to the building classified as Dangerous or Questionable until the overall retrofitting begins. As shown in Fig. 3 , prior to applying the emergency repairs, the structural elements rated III or greater in degree of damage must be inspected first, and then the appropriate repairs are applied to the vertical load system and lateral load system. Until the overall retrofitting is done, the building movement must be monitored continuously.

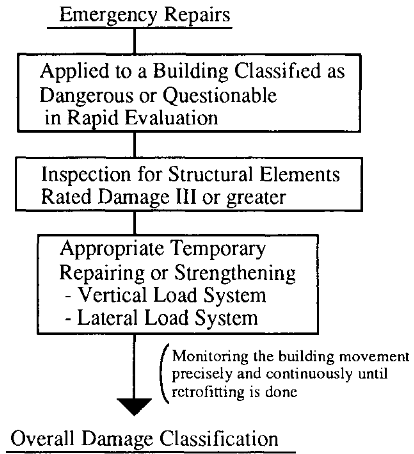

Fig. 3. Flowchart of emergency repairs
(RC Structures)

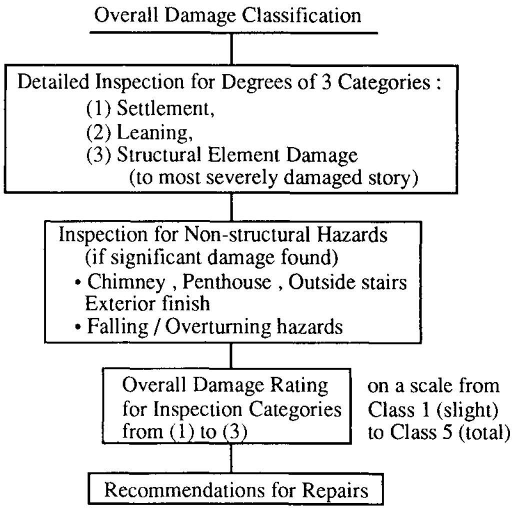

Fig. 4. Flowchart of overall damage classification
(RC structures)

Overall Damage Classification

Figure 4 shows the Overall Damage Classification for RC structures. The structural engineers inspect the building in terms of building settlement, leaning, and structural element damage at the most severely damaged story. The inspection ratio, defined as a ratio of the number of structural elements inspected and the total number of the structural elements, must be more than 50% or more to make the classifications more reliable. If serious damage is found to non-structural elements such as chimney, penthouse, outside stairs, and exterior finishes, as well as falling or overturning hazards, further inspection is needed.

Damage Rating. Tables 8 , 9 and 10 indicate the overall damage ratings for building settlement, leaning, and structural element damage respectively. The building may be classified as Class 1 (slight) to Class 5 (total/collapse). Criteria for the building leaning as shown in Table 9 are slightly different from those in Table 2. Overall structural damage is rated with respect to "Approximate Story Damage Ratio", defined by summarizing the degree of damage at a given damage level.

Table 8. Overall damage rating (1)
- settlement - (RC structures)

Ratings	Criteria
Class 2	$S \le 0.2$ m
Class 3	$0.2 \text{ m} < S \le 1.0$ m
Class 4	$S > 1.0$ m

S (m) : maximum settlement of a building

Table 9. Overall damage rating (2)
- leaning - (RC structures)

Ratings	Criteria
Class 2	$\theta \le 1/100$
Class 3	$1/100 < \theta \le 3/100$
Class 4	$3/100 < \theta \le 6/100$
Class 5	$\theta > 6/100$

θ (radian) : Leaning Angle of a building
$= \sqrt{\theta_x^2 + \theta_y^2}$

Table 10. Overall damage rating (3)
--- structural element damage ---
(RC structures)

Ratings	Criteria
Class 1	$D \le 5$
Class 2	$5 < D \le 10$
Class 3	$10 < D \le 50$
Class 4	$D > 50$
Class 5	$D_V = 50$

D : Approximate story damage ratio
($D = D_I + D_{II} + D_{III} + D_{IV} + D_V$)

D_I - D_V : degree of damage at a given damage level for Structural elements rated from I to V respectively at the most severely damaged story ; defined as

$D_I = 10 \cdot B_I/A \le 5$,
$D_{II} = 26 \cdot B_{II}/A \le 13$,
$D_{III} = 60 \cdot B_{III}/A \le 30$,
$D_{IV} = 100 \cdot B_{IV}/A \le 50$, and
$D_V = (1000/7) \cdot B_V/A \le 50$.

If $B_V/A > 0.5$, then overall structural damage is rated Class 5 , where B_I - B_V are the numbers of damaged elements rated from I to V respectively, and A is the number of elements available for inspection.

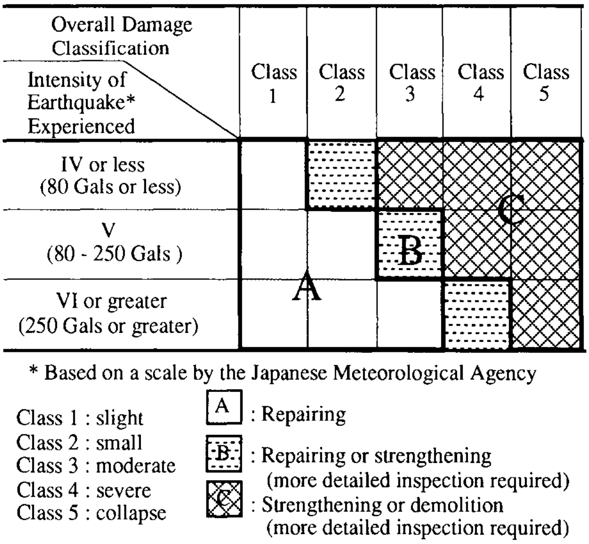

Fig. 5. Recommended actions to RC structures based on overall damage classification

Determination of Building Damage Classification. The severest overall damage rating among the three categories is taken as the overall building damage classification. Fig. 5. indicates that based on the damage classification and intensity of earthquake experienced, appropriate actions to the damaged building are determined as "Repairing", "Repairing or Strengthening", or "Demolition". If the overall classification falls onto a borderline between Class 2 (small) and Class 3 (moderate), or between Class 3 (moderate) and Class 4 (severe), more detailed inspections of foundation and structural elements is desirable.

Guidelines for Repairing and Strengthening

Classified as more detailed inspection required, or strengthening required, appropriate repairing or strengthening is applied to the buildings. Prior to retrofitting, detailed planning is needed for the earthquake resistant capacities, costs, and building usability. Especially for the earthquake resistant capacities, a damage index is computed from a ratio of the structure's seismic index after and before an earthquake. To determine the seismic index, several other indexes are computed, including a basic structural index, local geological index, structural design index, and time index.

STEEL STRUCTURES

The damage evaluation procedure described in this section is applied to steel frame structures of not higher than 45 meters. However, long-spanned structures, spatial truss structures, or suspended structures are not dealt with.

Rapid Evaluation

Rapid Evaluation for steel structures, as shown in Fig. 6 , begins with a quick visual inspection in terms of building settlement, leaning, structural element damage, non-structural damage (including damage to architectural interior or exterior panels), and falling/overturning hazards. Degree of damage is then rated Rank A (slight), Rank B (moderate), or Rank C (severe) based on the criteria for each category.

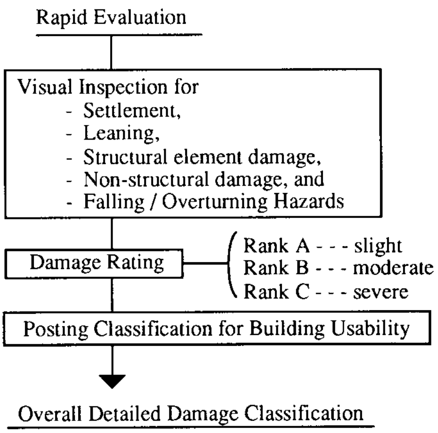

Fig. 6. Flowchart of rapid evaluation
(Steel structures)

Table 11. Structural damage rating
(Steel structure)

Ratings	(Settlement)	Frame system (Leaning)	Bracing system	Truss system
Is	$\theta \leq 1/150$	$\phi \leq 1/150$ - elements yielded - cracking at bottom of column concrete	- compression brace slightly buckled - cracking at bottom of column concrete	- horizontal bracing on ceiling locally buckled
(Rank A)				
IIs	$1/150 < \theta < 1/100$	$1/150 < \phi < 1/100$ - panel zone yielded - anchor bolt stretched	- high tension bolt shear slipped - anchor bolt stretched - tension brace yielded	- truss member slightly buckled, out of plane
IIIs	$1/100 < \theta < 1/50$	$1/100 < \phi < 1/50$ - slightly locally buckled	$\phi \leq 1/50$ - brace broken - connection broken	- truss member greatly buckled, out of plane
(Rank B)				
IVs	$1/50 < \theta \leq 1/30$	$1/50 < \phi \leq 1/30$ - moderately locally buckled	$1/50 < \phi \leq 1/30$	- chord or collector slightly locally buckled
Vs (Rank C)	$\theta > 1/30$	$\phi > 1/30$ - connection broken - greatly locally buckled	$\phi > 1/30$	- chord or collector slightly locally buckled - connection broken
VIs		collapsed	collapsed	collapsed

θ : Inclined angle due to unbalanced settlement
ϕ : Leaning angle of column
Note the severest damage rating is regarded as the overall structural damage rating.

Table 12. Non - structural damage rating
(Steel structure)

Ratings	Interior/Exterior wall	Ceiling	Opening	Equivalent Leaning
Iw (Rank A)	- small cracks at corners	- separation	- a few cracks at corner	about 1/150 or less
IIw	- shear slippage - slightly falling	- partially falling	- a lot of cracks at corner - hard to open doors	about 1/150 - 1/50
(Rank B)				
IIIw	- overall cracking - partially falling - bowed	- totally falling	- most of corners broken - impossible to open doors	about 1/50 - 1/30
IVw (Rank C)	- entirely falling	- entirely falling	- severely damaged	about 1/30 or more

Note the severest damage rating in this table is regarded as the overall non-structural damage rating.

Damage Rating. Structural damage is rated in terms of building settlement, leaning, and conditions of frame system, bracing system and truss system. Table 11 , even though showing the criteria of the overall damage rating for the structural damage, can be applied to the rapid damage evaluation. Rank A, at large, may correspond to Is and IIs rated in Table 11 , Rank B to IIIs and IVs, and Rank C to Vs respectively.
Non-structural damage is also rated from Rank A to Rank C based on the criteria as shown in Table 12. This table, although showing the criteria of the overall damage rating for the non-structural damage, can be applied to the rapid damage evaluation as well. Rank A may correspond to Iw rated in Table 12., Rank B to IIw and IIIw, and Rank C to IVw respectively.

Building Usability Classification. Based on the rapid ratings for structural and non-structural damages, the building usability is classified as "Dangerous", "Questionable", or "Safe". As shown in Table 13 , "Dangerous" is assigned to "Off-limits" as a posting class, and "Questionable" to "Limited entry", and "Safe" to "No restriction on use or to occupy" respectively.

Table 13. Building usability classification (Steel structures)

Classification	Criteria (Number of Ranks)	Posting
Dangerous	Rank C ≥ 1 or Rank B ≥ Half of the total number	Off - limits
Questionable	Rank C = 0 and Rank B < Half of the total number	Limited entry
Safe	All Rank A's	No restriction on use or to occupy

Note Rank A : may correspond to Is and IIs for structural damage ratings , and to Iw for non - structural damage rating .
Rank B : may correspond to IIIs and IVs , and to IIw and IIIw .
Rank C : may correspond to Vs , and to IVw .

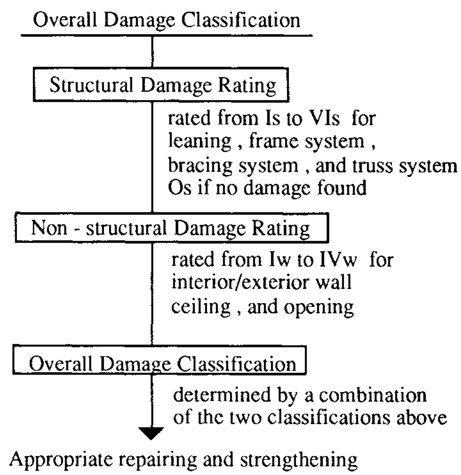

Fig. 7. Flowchart of overall damage classification (Steel structures)

Overall Damage Classification

Damage Rating. Figure 7 shows the flowchart of the overall damage classification for steel structures. Detailed structural damage is rated from Is to VIs using the criteria shown in Table 11 , and if no structural damage is found, rated Os (zero-s). In addition, non-structural damage to interior or exterior walls, ceiling, and openings is rated from Iw to IVw based on the criteria shown in Table 12.

Determination of Building Damage Classification. As shown in Fig. 8 , the overall damage classification is determined by a combination of the respective severest structural and non-structural damage ratings. Appropriate actions based on the overall building damage classification are described in Fig. 9. Some differences from RC structures for recommended actions are as follows;

- "No action taken" for Class 1 (slight), compared to "Repairing required" in RC structures.
- Class 5 (total/collapse) included to Class 4 (severe), compared to separately classified.

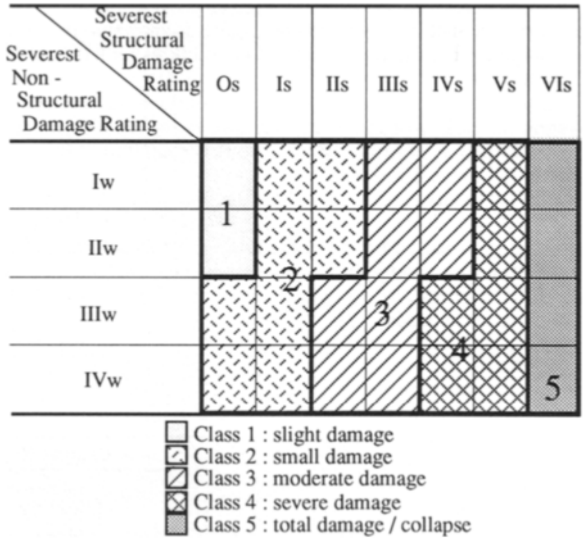

Fig. 8. Determination of overall damage classification
(Steel structure)

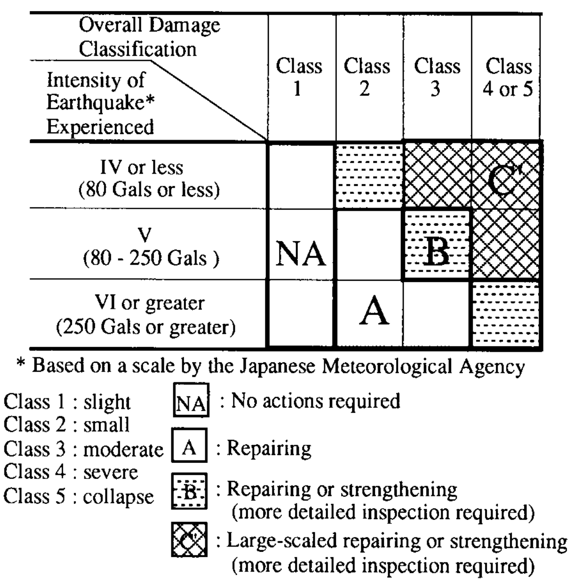

Fig. 9. Recommended actions to steel structures
based on overall damage classification

WOOD STRUCTURES

Wood structures dealt with in this section are dwellings, commercial buildings, apartments, and hospitals, of one or two stories in height. However, prefabricated wood structures, and the Japanese traditional wood structures are not dealt with.

Rapid Evaluation

Damage Rating. Figure 10 illustrates the flowchart of Rapid Evaluation for wood structures. By a quick visual inspection, degree of damage is rated Rank A (slight) to Rank C (severe) for building settlement, leaning of the first story, interior/exterior finishes, and falling/overturning hazards. This is based on the criteria shown in Tables 14, 15 and 16 for each category.

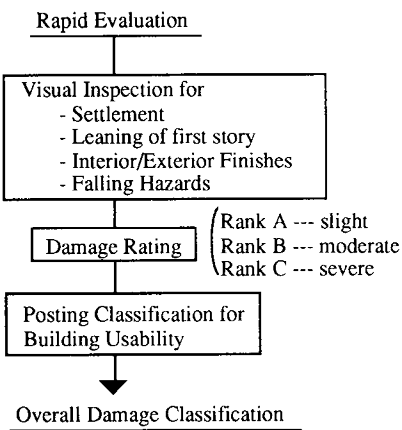

Fig. 10. Flowchart of rapid evaluation
(Wood structures)

Table 14. Building settlement rating
(Rapid evaluation : Wood structures)

Ratings	Criteria
Rank A	No serious settlement
Rank B	Roof or floor greatly displaced up and down
Rank C	Roof truss broken and Serious settlement

Table 15. Building leaning rating at first story
(Rapid evaluation : Wood structures)

Ratings	Criteria θ : Leaning Angle
Rank A	$\theta \leq 1/60$
Rank B	$1/60 < \theta \leq 1/20$
Rank C	$\theta > 1/20$

Table 17. Building usability classification
(Wood structures)

Classification	Criteria (Number of ranks)	Posting
Dangerous	Rank C $\geq$ 1	Off -limits
Questionable	Rank C > 0	Limits entry
Safe	All rank A's	No restriction on use or to occupy

Table 16. Rating for interior / exterior finishes
(Rapid evaluation : Wood structures)

Ratings	Criteria
Rank A	Small cracks
Rank B	Large cracks, and slightly spalling
Rank C	Large cracks, and falling

Building Usability Classification. Using the damage rating for each category, the building usability is classified likewise as in "Steel Structures". Table 17 shows the criteria for the building usability.

Overall Damage Classification

Damage Rating. Figure 11 shows the flowchart of the overall damage classification for wood structures. Wood structures are rated for foundation, floor framing, columns, structural walls, exterior wall finish, and roof framing and its finish. Damage rating for each category is expressed by the damage ratio, defined as a ratio of the number or length of the damaged elements, and the total number or length of the inspected elements. The criteria of the overall damage classification based on the damage ratio is shown in Table 18. In addition, other than this damage rating, conditions of damage are also taken into account as the criteria for degree of damage.

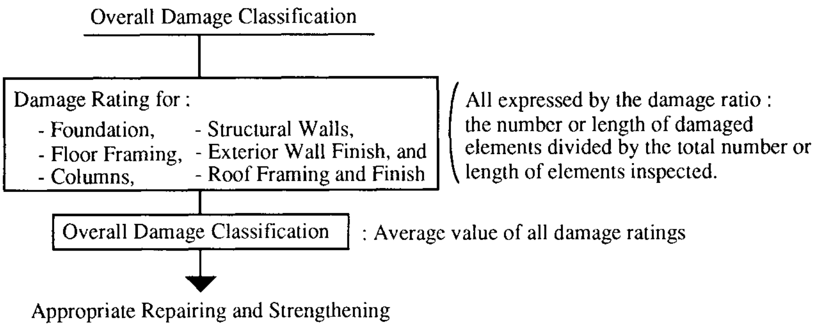

Fig. 11. Flowchart of overall damage classification
(Wood structures)

Table 18. Criteria of overall damage classification
based on damage ratio (Wood structures)

Inspection Category / Damage Classification	Foundation	- Floor framing - Columns - Structural walls	- Exterior wall finish - Roof framing and finish
I	15% or less	10% or less	15% or less
II	15-30%	10-30%	15-40%
III	30-65%	30-60%	40-65%
IV	65-85%	60-85%	65-85%
V	85% or more	85% or more	85% or more

Determination of Building Damage Classification. Based on the overall damage rating, the building damage classification is determined by averaging all the damage ranks and then rounding the value; however, if either of foundation, framing, or structural walls is rated Rank IV (severe) or more, this rating is regarded as the overall damage classification.

CONCLUDING REMARKS

Since the guidelines for the damage evaluation are to be applied at the emergency, it is urgently necessary to make the guidelines practical, and to train the structural engineers for the immediate and effective damage evaluation before disastrous incidents would occur.

As for the rapid evaluation, especially, the local government will be expected to prepare for the case in advance, for instance, by organizing the overall systems for the damage evaluation or, for finding volunteer engineers, or additional well-trained, experienced manpower to do inspections, so that quick and effective actions can be taken.

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A STUDY OF RAPID REPAIR IN SEISMIC AREA

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ABSTRACT

After a strong earthquake, how to restore normal life and work is an important problem. Thus rapid repair of damage concrete and masonry builing structures is valuable. Aspects of such repair work discussed in this paper.

KEYWORDS

Earthquake; rapid repair; wall; crack; concrete beam; joint; shaking table test.

INTRODUCTION

During a strong earthquake, in general, about 15% of builings may collapse and the other 85% have damage that can be repaired. If people can repair all the damaged builings within 1 or 2 days, it is a significant contribution to the mitigation of disaster.

TEST RESULTS OF CRACKED WALLS

In order to decrease the amount of work , the repair range of a damaged structure should be detemined. We defined an allowable crack width for damaged walls or columns as $[W]_{min}$ and $[W]_{max}$.

When the existing crack width of damaged walls is less than $[W]_{min}$, the vertical loading capacity only decreases by 15% of the original capacity. Such a wall can still be used.

When crack width is larger than $[W]_{max}$, the loading capacity is difficult to restore to the 85% of original capacity by means of repair.

Figure. 1 shows the relationship between exising crack width and capacity of a concrete wall which has a cross section 150×450mm and a height of 1000mm. In Fig. 1, the repaired wall is injected with the epoxy (mark 2101#).

Figure. 2 shows the same relationship for masonry walls. Dimensions of the masonry walls are 240 x 990 x 960mm. On the basis of test results, allowable crack width is given in Table 1.

Table 1 Allowable crack width (mm)

	Concrete wall	Masonry wall
$[W]_{min}$	0.5	3
$[W]_{max}$	1	6

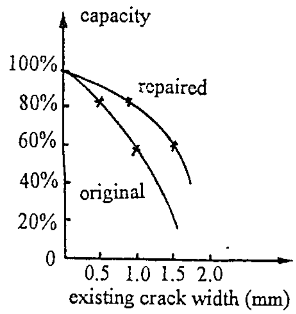

Fig. 1 Existing crack width and capacity of concrete walls

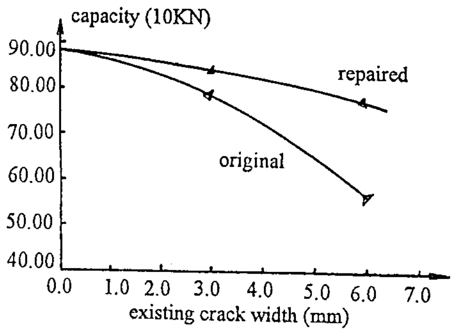

Fig. 2 Existing crack width and capacity of masonry walls

TEST OF REPAIRED CONCRETE BEAMS AND JOINTS

Test Specimens of Concrete Beams

Figure 3 shows the concrete beam specimens. Two concrete beams subjected to static horizontal loading up to the diagonal crack initially reached 2mm. Cracks in these two beams were then injected with cement-epoxy, and one was used for reloading while the other was subjected to the fatigue loading.

Test Results of Beams

Static reloading test results are shown in Fig. 4. Behavior of repaired beam is close to that of the original.

In the fatigue test, the repaired beam was subjected to cyclic loading two million times. The last static loading test shows that the repaired beam is improved about 28%.

Test Specimens of Concrete Beam-Column Joints.

Figure. 5 shows the test beam-column joints specimens. The joints specimen was subjected to horizontal cyclic loading to start with, and then the diagonal cracks of the beam specimen were injected with cement epoxy . Test results of the repaired joint specimen are shown in Fig.6.

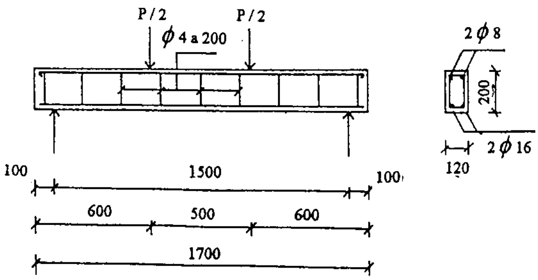

Fig. 3 Beam specimen

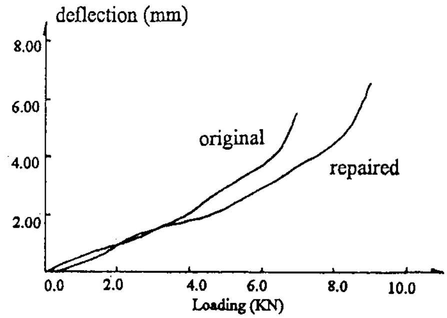

Fig. 4 Deflection and loading

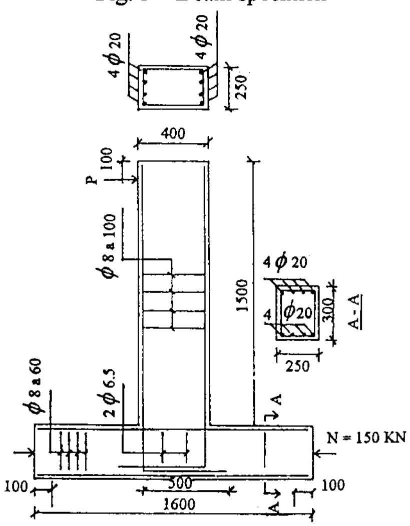

Fig.5 Test specimen of joint

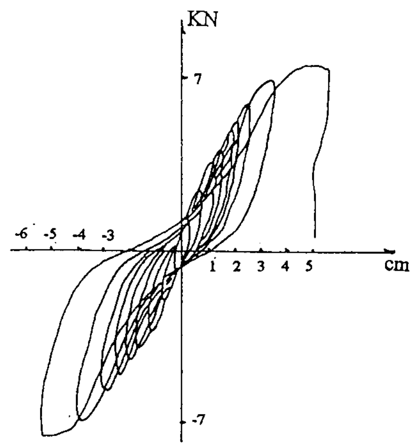

Fig. 6 Hysteresis loops of repaired joint specimens

SHAKING TABLE TEST OF REPAIRED BUILDING MODEL

A Hainan tall building model was tested on shaking table in Tongji University up to the point of serious amage, and repaired with cement-epoxy. After repair, the model increased its aseismic capacity from intensity 7 to 8.

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STRUCTURAL ASSESSMENT BY LOAD TEST PROVING for IN-SITU STRUCTURAL CRACK REPAIR PROJECT USING VACUUM TECHNOLOGY

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ABSTRACT

This paper presents an analysis of the procedures and results concluded from load testing beams and columns repaired by vacuum injection at the Student Activities Center Building. The building is located on the campus of National Taiwan Ocean University. During and after the repair of the cracks by the patented vacuum injection method, a number of evaluation stages were performed to affirm structural restoration and safe occupancy of the building.

Prior to the employment of the repairs, an evaluation of the cracking included review of design and construction data. After evaluation, a description accurately locating and exactly depicting the existing cracks was recorded on drawings and accompanied with an overall evaluation of the existing concrete condition. The review conclusion and drawings were supplemented with photographs recording the condition of the individual members.

Considering the careful evaluation of the extent of cracking in the members, a method of repair was required that would restore strength, restore flexural stiffness and restore overall functional performance of the individual members. Because the vacuum injection/impregnation processes were fundamentally superior to conventional epoxy injection means, this method was selected for the repairs. Vacuum injection was performed by the patent holder Balvac, Inc., Buffalo, New York.

Upon conclusion of the repairs by Balvac, Inc., an investigation of the extent of depth and penetration of the repair resin materials in the cracks was performed. To validate the penetration of the repair materials into the depths of the fracture, core drilled samples were visually examined utilizing 7X scope. The scope was additionally utilized to visually examine the complete filling of the cavity and penetration of the repair materials in the fracture walls. Final load testing of the individual members and the structure was performed in accordance with applicable codes and accepted practices. The results of the testing concluded the repairs performed by vacuum injection completely restored the structural integrity of the members and also revealed promise of mitigating reinforcement corrosion within concrete members with the repair process.

KEYWORDS

Beams; Concrete; Epoxy Pressure Injection; Impregnation; Methyl-Methacrylate (MMA); Repairs; Structural; Vacuum Injection

INTRODUCTION

Repairs to structural cracks were performed on a building situated on the campus of National Taiwan Ocean University. This three story structure, constructed in the mid 1960s' and known as the Student Activities Center Building, is located on the northeast coast of Taiwan, some thirty miles from Taipei. The building is constructed of concrete frame construction, composed of individual bays, each with two columns and haunched beams.

The building is located in the coastal area and suffers from many of the inherent problems. In particular, the area experiences many tremors each year. After a rather strong seismic disturbance, large cracks in the columns and beams were repaired by conventional pressure injection methods. Upon completion, the fractured members were examined and found to remain in a severely broken condition. This event prompted a complete structural safety investigation by the Chinese Institute of Civil and Hydraulic Engineering. This structural safety investigation concluded the building was, because of the cracked beams and columns, creating an eminently dangerous condition. The building was deemed off-limits and recommendations were to perform a more substantial and comprehensive evaluation and repair program. Evasun Engineering, Inc. was especially commissioned by the directors of the University to design and perform the repairs on a turn-key basis. A durable and effective method of restoring structural integrity and ensuring student safety was sought by Evasun.

After initial evaluation, it was concluded the only two options were to: (a.) remove the defective concrete and re-pour the member or, (b.) utilize a method of in-situ repair. The removal and replacement of the damaged and deteriorated concrete was concluded to be a prohibitively expensive preference, not to mention a lengthy intrusion on the student body and faculty. Therefore, attention was devoted by Evasun to employing a suitable in-situ repair.

Following a number of discussions with colleagues and structural repair specialist across Asia, the Middle East and the United States, Evasun learned of an acclaimed method of repair; a method of repair touted vastly successful under a wide range of applications and conditions. By creating a partial vacuum in the concrete, repair resins could be introduced into the fracture resulting in a more complete and permanent repair of the member. This patented method was only available from the US company, Balvac, Inc..

The repairs were performed and completed by Balvac, Inc., under the participating direction of Evasun Engineering using the Balvac® Injection/Impregnation Process of vacuum injection.

EVALUATION, REPAIR AND TESTING OF DAMAGED CONCRETE MEMBERS

The objectives of the evaluation of the Student Activities Center Building were to: (a.) document the current condition of the individual members and estimate the influence of fracture conditions on structural integrity; (b.) determine the extent of the fractures; (c.) develop a method of repair. A cursory review of the as-built drawings was conducted. However, a tacit assumption was made by Evasun that the structure was designed and built adequately. Investigation of distress related to design or construction deficiency is beyond the scope of this paper.

Depiction of Existing Cracks.

The safety investigation report of the cracked beams and columns of the Student Activities Center was comprehensively completed by the Chinese Institute of Civil and Hydraulic Engineering and reviewed by Evasun. A visual examination of the concrete beams and columns was conducted and all areas of cracks were observed for leaking, mineral deposits, rusting, spalling scaling and deterioration. The visual examination was somewhat hindered by the existing cementicious coating applied to the members. Selected areas of this material were removed and an electromagnetic device, known as a pacometer, was used to determine the depth of cover over the reinforcement (Ref 1.). Areas of thinner concrete cover were investigated by exploratory chipping and sounding.

Determination of the Severity

This closer visual examination by Evasun engineers revealed even more extensive and severe cracking than was reported by the findings of the CICHE report. Core samples were extracted from the more severely fractured members. It was discovered through the investigative core drilling that the interiors of the cracks were much more serious than observed from the outside surface area examination. Field examinations of the core samples by 7X scope revealed smaller fractures radiating from the main fracture line and even finer cracks radiating from the smaller connected cracks. In some instances, the interior of the fracture was in excess of 50mm in width, with total debonding of the cement paste from the reinforcing steel. For this reason, some of the outer material was removed to reveal and access the inner cracking for closer examination. It was also discovered that previous attempts to repair the fractures with conventional epoxy injection methods were totally unsuccessful. The exact location and classified severity of each crack was then recorded and verified as follows:

a. On the first floor level, 16 out of 25 (64%) columns contained severe cracking in the middle/center section of the columns and 4 out of 12 (33%) beams were severely cracked near the haunches.

b. On the second floor level, 7 out of 25 (28%) columns contained severe cracking in the middle/center section of the columns.

c. On the third floor level, 16 out of 25 (64%) columns contained severe cracking in the middle/center section of the columns.

The Repair Selection Process

The repair process to be employed by Evasun would necessitate the restoration of structural integrity to the members and, if economically feasible, the method would also provide extended serviceability to the individual members and the entire structure.

Selecting the repair system involved the consideration of the following criteria: (a.) economics; (b.) disruption of the repairs to the faculty and student body; (c.) ability of the method to be applied to the existing structural member conditions, and; (d.) consideration of the remaining structural service life.

The University requirements dictated the economics and disruption elements of the selection process. Their budget to perform the repairs were limited; the repairs must impose as little disruption to the facilities as possible, and; the repairs must be conducted at an expeditious pace to accommodate the late summer resumption of the university's scheduled opening. Either of the requirements imposed by the University eliminated the option of tearing out and re-pouring the individual damaged members so an in-situ form of repair was sought.

Pressure injection with epoxy material was first considered. However, the previous attempts at repairing the fractures by means of conventional pressure injection of epoxy were totally unsuccessful. The viscosity of the epoxy repair material prohibited penetration in all but the largest part of the fracture to a depth only just below the surface of the member. The fractures radiating from the main fractures were totally void of any repair material at all. Where the epoxy had penetrated to any degree, apparent moisture within the crack had totally prohibited any form of bond to the side wall areas. More than a few of the previous attempts resulted in the epoxy material remaining just below the surface, probably due to improper application procedures. Notwithstanding, fresh epoxies will not adhere to cured epoxies. It was determined where the previous repairs were partially filled with epoxy and unbonded to the fracture wall, this existing material would block the flow of new repair material and form a relief at the existing epoxy/new epoxy joining line. The entire repair would be rendered ineffective. Therefore, an alternative means of in-situ repair was preferred and explored.

After numerous conversations with colleagues, Evasun was informed of an in-situ method of repair that utilized vacuum technology in the repair of concrete, masonry and stone. Balvac, Inc., located in Buffalo, New York, held the patent to the unique process and had a ten year track record of successful repairs in the United States and Europe under a wide and extensive range of conditions.

The method is fundamentally based upon first creating a partial vacuum within the concrete and then introducing a repair resin into the concrete matrix. Balvac treats four basic conditions with four basic repairs: (a.) Vacuum installed plate bonding to strengthen and increase structural load capacity. (b.) Vacuum injection/impregnation of individual and discreet cracks; (c.) Impregnation flushing to impregnate surfaces and completely fill multiple and intimately spaced cracks in a wide area, and; (d.) Vacuum injection/impregnation of delaminated surfaces that eliminates the requirement of material removal. The vacuum injection/impregnation of individual and discreet cracks was used on this project.

The resin fills the cracks, including microcracks down to a width of 5 microns (see Reference 6). Upon curing, the repair resin bonds the fractured and fissured matrix into a monolithic structural member of exceedingly high strength. Partial vacuum creation and repair resin introduction are achieved by adhering vacuum and introduction porting devices onto the fracture being repaired. By means of special tubing, the porting devices are connected to the vacuum source and the partial vacuum pressures are applied to the enclosed system. The repair resins are introduced, while maintaining the negative pressures, to fill the major cracks, interconnected cracks and voids and micro cracks. The concrete matrix of the walls within the fracture is impregnated with the repair resin materials.

Excessive moisture is evacuated from the concrete matrix of the fracture wall surfaces along with any deleterious gases and/or materials. The concrete drying process can be monitored by using in-line hydrometers installed in the special vacuum tubing.

In terms of meeting the objectives of Evasun in completing the repairs to the damaged members of the Student Activities Center, the Balvac® Injection/Impregnation Process offered the following advantages over pressure injection methods:

- Repairs could be completed in a relatively short period of time with no sacrifice to the quality of the repair.
- Repairs could be completed within the price perimeters of the University budget and were competitive with conventional pressure methods of repair.
- Efficient and complete filling of existing fractures, interconnected fractures and voids and the complete filling of micro fractures.
- Total absence of pressure pockets would ensure and facilitate deeper fill of repair resin.
- Evacuation of moisture from the interior concrete matrix of the fracture.
- No possible extenuation of the fracture due to absence of applied pressures.
- Ability to introduce ultra-low viscosity materials into the fracture areas that would pass by and bond to the repair materials existing in the fractures.
- Improved bonding due to the lack of bubbling normally associated with low viscosity, low specific gravity repair resins.

- Susceptibility of continued reinforcing corrosion would be significantly diminished because of the evacuation of, and sealing out of, moisture from the treated concrete matrix.

Repairs to the Columns and Beams by Vacuum Injection/Impregnation

When the exact location and classification of severity were ascertained, technicians and special vacuum equipment were mobilized to the site of the work from Balvac, Inc.'s office in Washington, DC. After condition evaluation and review with Balvac, Inc., a uniquely blended acrylic polymer material, methyl methacrylate (MMA), was selected as the material to be used for the repairs (Ref 2. & 3.). This low viscosity, high strength material was particularly developed with special modifiers for the Balvac Injection/Impregnation Process™ of vacuum injection. It is noted for its ultra low viscosity (5-15 cps) highly rated physical properties, flexibility, and its superior bonding and wetting properties. The material is not temperature sensitive and is easily mixed and modified for specific field conditions. Unlike epoxies, MMA is favorably forgiving when not mixed "just right" and will easily bond to previously cured MMA or epoxy.

The odor of the monomers, and toxicity commonly associated with all polymer components, required the usual precautions for handling. Foremost with the use of MMA was the proper and adequate ventilation of the emitted vapors (Ref 4.). Balvac employed high volume air moving exhaust equipment that totally exhausted the vapors from the work area to the outside.

Materials - The basic monomer of the repair material was methylmethacrylate (MMA). MMA is a slightly amber liquid that looks like colored water and is about the same viscosity. It has a sharp odor that can be detected by smell in as little as one part per million. An inhibitor is added for longer storage times.

Inhibitors are additives used in MMA to prevent premature polymerization, that can be caused from excessive temperatures, contaminates, etc.. The two most common inhibitors are methyl ester of hydroquinone (MEHQ) and hydroquinone (HQ). These inhibitors and promoters are normally added to the monomer by the acrylic manufacturer and require no field mixing or attendance to whatsoever.

Promoters are used in very small quantities to increase the decomposition rate of the initiator, which will result in faster curing of the polymer. The generally preferred promoter used is dimethyl-para-toluidine (DPT), a liquid with the same viscosity of the MMA.

The initiator is added to this mixture of MMA, MEHQ and DPT that initiates the polymerization process, or curing of the repairing resin. Benzoyl Peroxide is a white powder or liquid that readily dissolves in the MMA. The amount of initiator added to the MMA is directly related to the time desired for curing the polymer. Increased amounts of initiator will result in more rapid polymerization of the monomers.

Physical properties of the cured materials generally range in the area of 10,000 psi compressive strength, 7,000 psi tensile strength, 4,000 psi flexural strength and 7000,000 psi modulus of elasticity. (Ref 5.)

Repairs- The structural repairs were performed by senior repair technicians from Balvac, Inc. and Evasun Engineering, Inc. in two required phases. Each crack face on each column and beam was carefully fitted with access port devices attached to special hoses to allow the evacuation process and the induction of repair resin materials. To prevent the repair polymers from draining from the fracture during the repair process and insure vacuum pressure maintenance, the fracture line was sealed with a cementicious material heavily bodied and concentrated with latex additive. The entire fracture system was then tested for negative pressure breaches. After sealing the access ports and the entire fracture line, the closed system was evacuated down to a pressure of 0.8 to 0.14 in Hg absolute or less and maintained during the introduction of the repair resins.

Investigation of depth and penetration of repair resin materials in the cracks.

After the repairs were completed, five individual cracks, previously noted as moderate to severely cracked, were selected to extract core samples for examination. These cores were thoroughly examined by microscopic method. Microscopic examination revealed the MMA resin material had not only filled the subject crack completely, but had also completely filled the previously undetected and connecting adjacent cracks. The bonding of the MMA was good in each case with concrete failure occurring first when testing stress was applied.

In one of the extracted cores where reinforcing was passed through, it is worthy to note it was found the MMA completely filled the area around the corroded reinforcement bar. This would indicate the mitigation of the corrosion process is performed with The Balvac® Injection/Impregnation Process methods of repair and warrants separate further study intended by the authors in 1995.

Load Testing of Selected Repair Members

To further substantiate and confirm, (1) the effectiveness of the repaired members, (2)the ability of the repaired members to meet the requirements of the code specifications and (3)the safe occupation of the structure, load testing was performed on an area previously noted to be the most seriously damaged in the structural frame of the Student Activities Center Building. The selected area included both columns and beams that were considered most serious. The details of the loading test processes are described as follows:

a. Test Criteria

1. Code §337, Technical Specification of Architecture (ROC)
2. ACI Code 318-89, Chapter 20.

b. Period of Testing

1. Three days, September 23-25, 1994.

c. Loading Method

1. With consideration given to time, feasibility, and safety, water was used to load the testing area.

Load Testing Procedures

In accordance with the code specifications referenced above, the criteria for the load testing of the flexural members were delineated as follows:

1. Base readings (the reference for deflection measurements) shall be made immediately prior to the application of the test load.

2. That portion of the structure selected for loading shall be subjected to a total load, including dead loads already acting, equivalent to 0.85(1.4D+1.7L). Determination of L shall include live load reductions as permitted by the general building code of which this code forms a part.

3. Test load shall be applied in not less than four approximately equal increments without shock to the structure and in such a manner as to avoid arching of loading materials.

4. Test load shall be removed immediately after initial deflection reading, and final deflection reading shall be taken 24 hours after removal of the test load.

5. If the portion of the structure tested shows visible evidence of failure, the portion tested shall be considered to have failed the test, and no re-testing of the previously tested portion shall be permitted.

6. If the portion of the structure tested shows no visible evidence of failure, the following criteria shall be taken as indication of satisfactory behavior:

a. If measurement maximum deflection δ of a beam, floor or roof is less than $l_t^2/20000h$.(cm).

b. If measurement maximum deflection δ of a beam, floor or roof is exceeding $l_t^2/2000h$.(cm) deflection recovery within 24 hours after removal of the test load shall be at least 75 percent of the maximum deflection for non-prestressed concrete.

7. Non-prestressed concrete construction failing to show 75 percent recovery of deflection may be re-tested not earlier than 72 hours after removal of the first test load. The portion of the structure re-tested shall be considered satisfactory if:

a. The portion of the structure tested shows no evidence of failure in the re-test, and;

b. Deflection recovery caused by the second test load is at least 80 percent of the maximum deflection in the second test.

Calculation of Test Load and Deflection.

a. Calculation of Test Load:

The dead load is not considered since the structure was constructed a number of years ago. The live load considered is 300kg/m2, and the factor for live load is 1.7. With no reduction in live load, the test load is calculated to be 51kg/m2. This is equivalent to 51cm depth of water loaded to the members.

b. Calculation of maximum allowable deflection:

l_t=1143cm
h=70cm
$\delta=l_t^2/20000h$=0.93cm=9.3mm

Measurement of Deflection

a. Set five dial gages with the accuracy of 0.01mm to measure the deflections in different positions.

b. Visually observe with microscope to detect any possible cracks occurring during testing.

Loading Test Procedure

The test load was applied in four approximately equal increments. The deflections were read from the dial gages immediately after each test load was in position, all the while observing the members with microscope for the development of cracks. The test load was removed and the final deflection readings were taken 24 hours after the removal. The recovery of deflection was also observed.

Results of Loading Test

From the results of the deflection readings, the maximum deflection with 51cm depth of water loaded is 2.6mm. This deflection is smaller than the maximum allowable deflection of 9.3mm calculated from the code specification. Therefore, the deflection is satisfactory. During the test, the deflection displayed linear elasticity and no visible cracking was detected from the examination. After the load was sustained for 24 hours, the deflection was increased by only 0.12mm and there was no phenomenon of residue. The recovery is calculated to be 99 percent for regenerance.

SUMMARY AND CONCLUSIONS

From the results of the load testing it is concluded the strength and stiffness of the structure after the crack repairs utilizing the Balvac® Injection/Impregnation Process displays superior behavior and totally satisfied the requirements of the code specifications. The vacuum technology proved to provide superior abilities when compared with pressure injection methods. The repairs were totally completed in half the estimated time of conventional repairs at a comparable cost. Examination of cores extracted from the repaired areas with 7X scope revealed material 355mm deep at widths less than .025mm. Radiated fractures connected to the main fracture were completely filled and field shear testing failed within the concrete and not at the adhesion line of the fracture.

The repairs performed on the columns and beams of the Student Activities Center demonstrated the ability of MMA polymer systems to fully restore structural integrity to damaged concrete via in-situ vacuum repairs. While MMA is a liquid with varying degrees of volatility, toxicity and flammability, the repair resin has proven to be very fluid and will soak into dry concrete, filling cement matrix cracks and voids much the same as water. The merits of the low viscosity MMA materials, in concert with the vacuum processes, are evidenced by complete and total penetration of repair resin into the interior fracture walls of the repaired beams and columns of the reconditioned structure.

The safe occupancy of the structure has been totally restored and is evidenced by the exceptional results of the loading test performed in accordance with criteria imposed by the code specification requirements.

ACKNOWLEDGMENTS

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BRIDGES AND SEISMIC RETROFIT IN CALIFORNIA

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ABSTRACT

There are more than 24,000 publicly owned bridges in the State of California. In the last thirty years, a number of these bridges had been damaged by several strong earthquakes. The California Department of Transportation (CALTRANS) started its first bridge retrofit program after the 1971 San Fernando earthquake. After the 1989 Loma Prieta earthquake, a bridge retrofit program was established for all publicly owned bridges in the State of California. CALTRANS has developed and implemented a number of structure retrofit techniques for these retrofit programs. These retrofit techniques are used at expansion joints, columns, footings, bent caps, and abutments.

KEYWORDS

Seismic retrofit; bridges

INTRODUCTION

The majority of bridges in the State of California are build before 1971. Bridges build at that time were design to resist a maximum of 0.06g lateral force. In addition, early knowledge in the structure detail did not consider a ductile member behavior.

Earthquakes have occurred throughout most of California in the past and are to be expected in the future. Studying the damage from earthquake is valuable. The knowledge gained by studying the damage gives an insight into how structures react to seismic shaking and what can be done to mitigated the damage expected from the larger earthquakes that are certain to occur in the future.

The San Fernando earthquake on February 9, 1971, was the first event to cause significant amount of damage to the bridges in the State of California. The total amount of earthquake damage to bridges experienced before that time was minor and generally ignored. Two major earthquakes since 1971, the 1989 Loma Prieta earthquake and the 1994 Northridge earthquake, have also cause significant damage to the bridges. Two major impacts are resulted from the 1971 earthquake. The first is the start of the Phase I retrofit program with hinge restrainers. The second is the adoption of the first state-of-practice seismic design specification based on the response spectrum. The 1989 earthquake has accelerated the column retrofit program.

SEISMIC RETROFIT PROGRAM

Phase I Retrofit Program

The Phase I hinge retrofit program began right after 1971 San Fernando Earthquake. This consists of tie superstructure together, tie superstructure with their supporting components, provide additional supports, and limit superstructure's displacement. From 1971 to 1986 CALTRANS had retrofitted more than 1000 bridges with a total budget of $56 millions.

Seismic Safety Retrofit program

After the completion of Phase I retrofit program, CALTRANS starts the second phase of bridge retrofit in 1986 to improve the column ductility. After 1989 Loma Prieta earthquake, the column retrofit program was expanded to the bridge seismic safety retrofit program. CALTRANS was directed by the SB36X to retrofit all publicly owned bridges. All bridges are to be reviewed and screened to determined if any seismic retrofit is needed to prevent structures from collapse.

Retrofit Prioritization

Due to the massive number of bridges, there is a need for a procedure to assess the relative vulnerability of each bridge and prepare a priority list for retrofit work. The most critical bridges must reinforce and retrofit first. The prioritization and risk analysis algorithm have been modified several times since we began phase I retrofit program. Earlier procedures were primarily emphisized on engineer's experience by weight factors. The latest algorithm has incorporated risk analysis theory, site conditions, seismicity and engineer's experiences. The following characteristics and relative weights are used in the current algorithm:

Activity	
Seismic activity attribute	(0-1)
Seismic Hazard	
Peak rock acceleration	(0-0.38)
Soil condition	(0-0.33)
Seismic duration	(0-0.29)
Impact	
Detour distance	(0-0.14)
Average daily traffic	(0-0.28)
ADT under/over structure	(0-0.12)
Leased space (Office)	(0-0.15)
Leased space (Parking, storage)	(0-0.07)
Route type	(0-0.07)
Utility	(0-0.10)
Facility crossed	(0-0.07)
Vulnerability	
Year designed(constructed)	(0-0.25)
Hinge	(0-0.165)
Outrigger, shared column	(0-0.22)
Redundancy	(0-0.165)
Skew	(0-0.12)
Abutment type	(0-0.08)

$$Risk=Activity*\Sigma Hazard*(0.60*\Sigma Impact+0.40*\Sigma Vulnerability)$$

TRADITIONAL RETROFIT METHODS

Expansion Joint

Expansion joints are used to reduce the substructure stresses caused by superstructure temperature, creep, relaxation,.. etc. effect. It is also used for construction convenience. Except simply supported structures, the distance between two expansion joints is approximately 300 feet (100 m). In recent years, this distance has been increased to 1000 feet (330 m). Restrainers are installed at hinges and bearings to limit the relative superstructure longitudinal movement and keep the structure tied together during earthquakes. The major purpose of restrainers is to prevent spans falling off from their supports. Because of the geometric nonlinear nature, the restrainers' design is based on base shear coefficient or equivalent static push-over analysis.

unseated condition can not be prevented by using restrainers along (Figure 3). Catch blocks are added at the abutments and top of piers (Figure 4).

Column

Bridges built before 1971 had a typical column shear reinforcement of #4 at 12 inches (30 cm) spacing (Figure 5). At the bottom of columns, rebar splices are common. Columns with this type of reinforcement detail can not provide enough confinement stress to core concrete. In addition, longitudinal reinforcements can not develop their full strength if lap splices were used. These columns can not sustain repeated cyclic loading and will fail in a brittle manner. Seismic design specification based on ductile requirement after 1971 has dramatically change the reinforcement detail in columns for better cyclic behavior. To retrofit existing columns to improve their seismic resistant capability, steel jackets can be added to the outside of the column (Figure 6). For non-circular section, an ellipse section is preferred (Figure 7). An infill concrete shear panel also can be added between columns if a change from flexure behavior to shear wall behavior is feasible (Figure 8).

Bent Cap

There are a number of bridges that are I or T shape girders simply supported by drop bent caps. There are others that have outrigger bent caps. Both of these bent caps have poor reinforcement detail at joint regions. Column longitudinal bars not have hoops at their ends and terminated 6 to 12 inches from the top. At the joints, rebars can not transfer beam stresses efficiently into the columns. Once a diagonal crack develops at the joint, column rebars gradually loss their anchorage. The exterior and the outrigger knee joint will disintegrate and consequently the bent cap will lose its supports. The typical way to retrofit bent cap is to add concrete or steel jackets to boost the cap capacity and improve joint detail. Prestressing also used to increase the allowable joint shear stress if needed (Figure 9, 10).

Foundation

Spread footing and pile cap built before 1971 usually did not have a top mat of steel. In addition, the earlier pile-cap connection detail can not develop full pile tension strength and can not be treated as moment connection either. Foundation design practice at CALTRANS has been dramatically changed in late 70's. The new foundations design requires that any structural failure shall be above the ground level. To achieve this goal, footing and pile cap have to be able to resist a higher force than the force at the column. Footing and pile cap can be retrofitted by enlarging, thickening, and adding new piles. Top mat of steel and shear stirrup are the must (Figure 11).

Abutment

Wingwall failures are common during earthquakes in addition to girder unseated at abutment. Restrainers can be added at abutment to limit relative displacement at abutment expansion joint and prevent unseated failure. In addition, catch block can be used to lengthen the supports. External piles can be used to reduce the lateral movement of the abutments and prevent wingwall failure. (Figure 12)

Others

Other than improving structure member behavior by increasing strength and ductility, sometime additional structure members can be added to the system to change the system dynamic behavior of bridges. The Santa Monica Viaduct in the Los Angeles area used the link beam concept to increase the column stiffness by adding beam between columns. Other examples include add additional frame at abutment, add new bent cap or extend existing bent cap (Figure 13).

NEW TECHNOLOGY IN RETROFIT

Base Isolation

reduce the strength and ductility demand. Base isolators can be designed to limit the maximum strength demand from an earthquake to a lower level than the column can resist (Figure 14). Designer should completely understand the dynamic behavior of isolators before any application can take place.

Advanced Fiber Composite

Advanced fiber composite have been used in the consumer market and airspace structure for a long time. It has the advantage of high stiffness and weight ratio. There are million combinations of fibers and resins mix. Because the custom design nature of this material, cost can be higher than traditional retrofit material. Durability should be emphasize during the composite selection process (Figure 15).

CONCLUSION

This paper gives an overview of CALTRANS' bridge retrofit from 1971 to date. Experiences from the last several earthquakes such as Northridge in 1994, Landers in 1992, Cape Mendicino in 1992, Loma Prieta in 1989, and Palm Springs in 1986 demonstrated that both hinge retrofits and column retrofits perform well. Restrainers can limit earthquake damage to bridges. They can keep superstructures together when properly designed. Column retrofit can increase column displacement capacity. They can also resist higher lateral forces if proper design procedure is followed. Current knowledge in the structural behavior of retrofit bridges is limited. Before a full structure replacement decision can be made, bridges need to be retrofitted to the current seismic design standards to extend a structure's life span. Recent experience at Kobe, Japan, again demonstrates this urgent need to retrofit existing structures with non-ductile detail. By increasing a bridge's seismic resistance capacity, the traveling public can be protected from possible bridge collapse and emergency relief can be ensured after a major earthquake.

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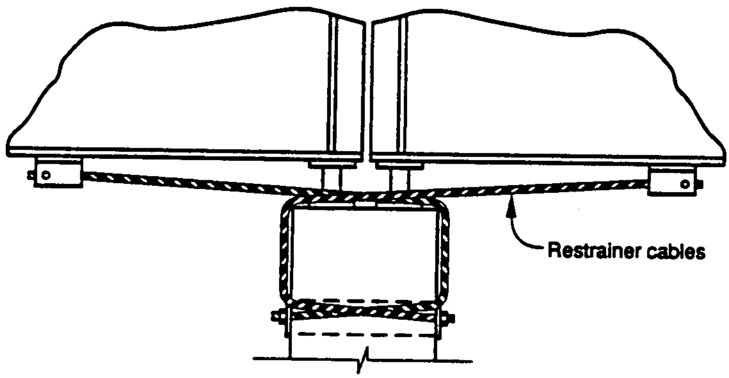
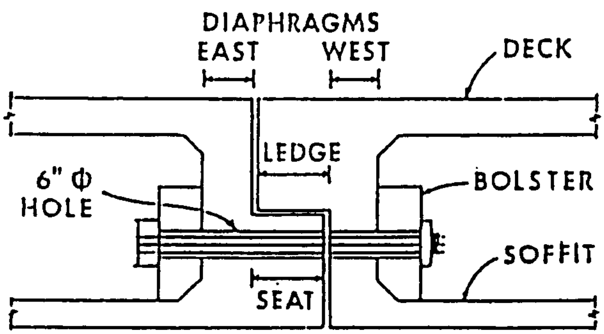


Figure 1



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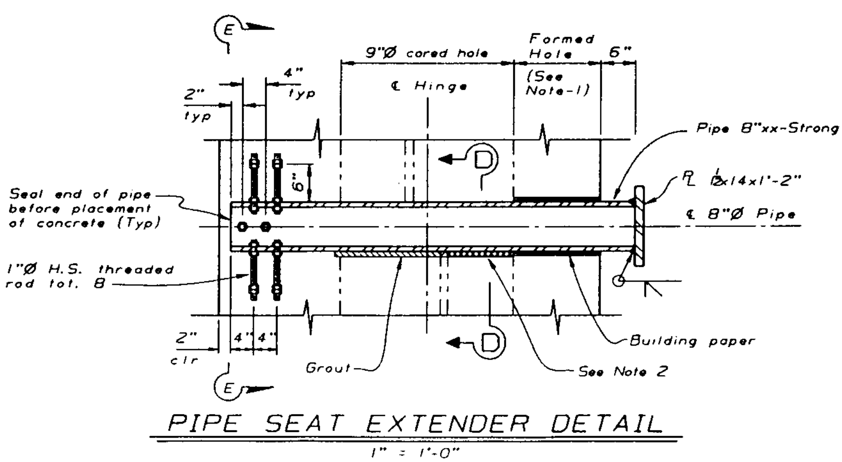

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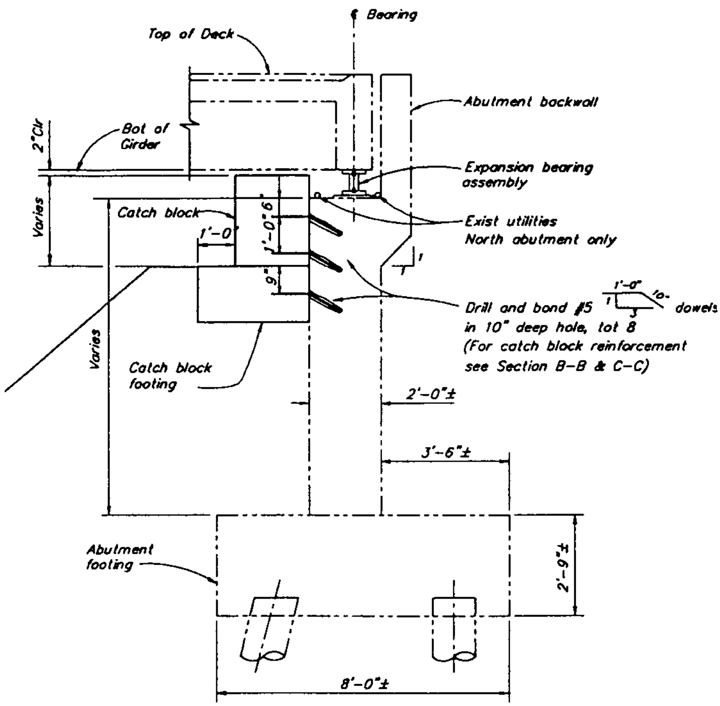

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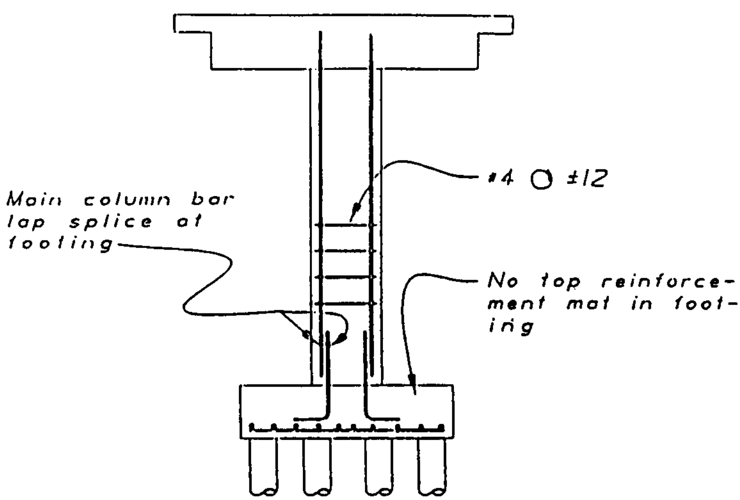

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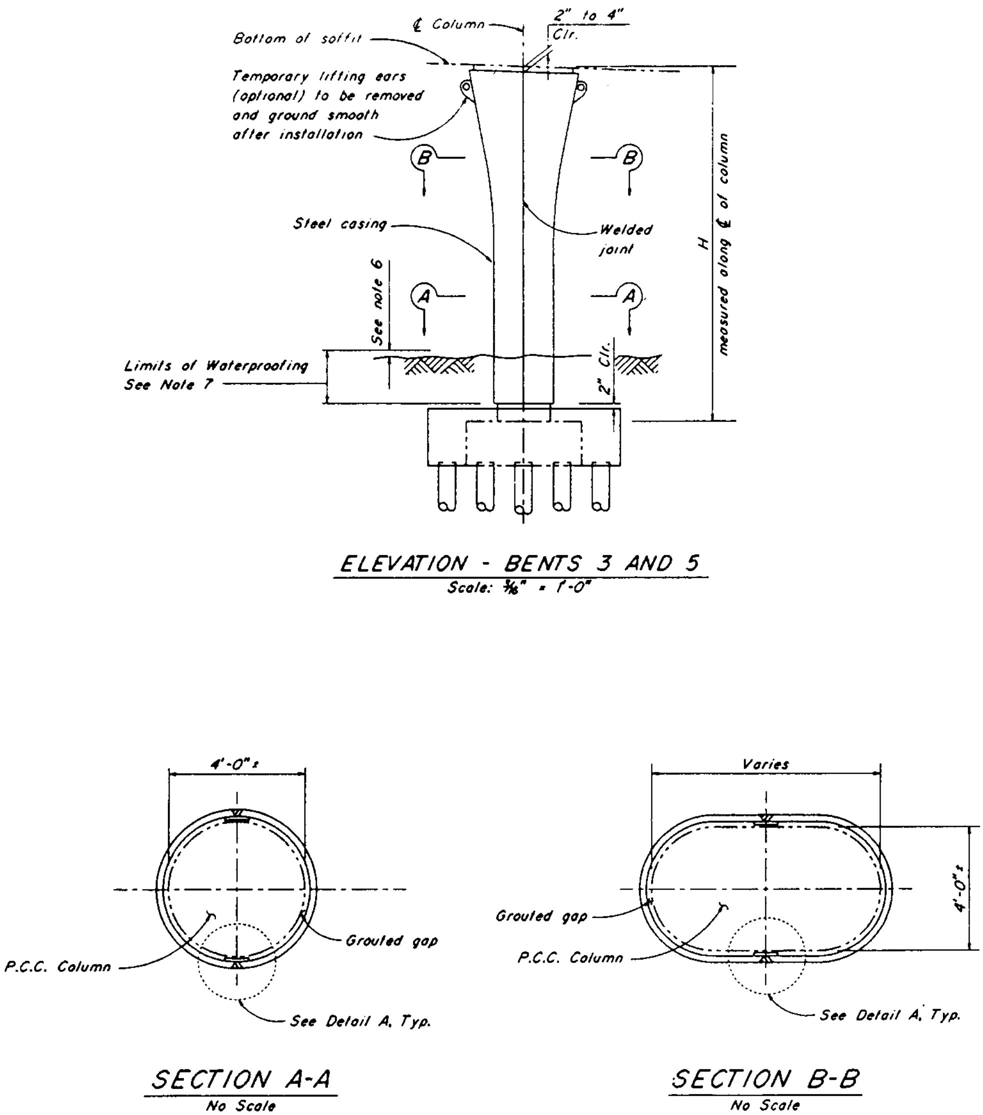
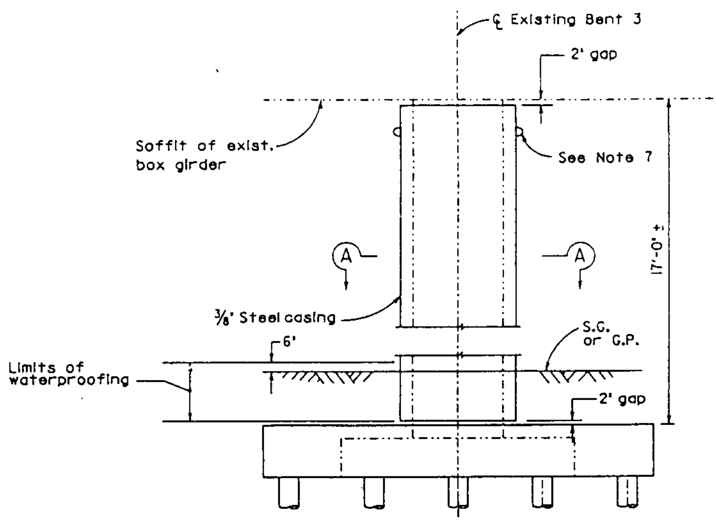
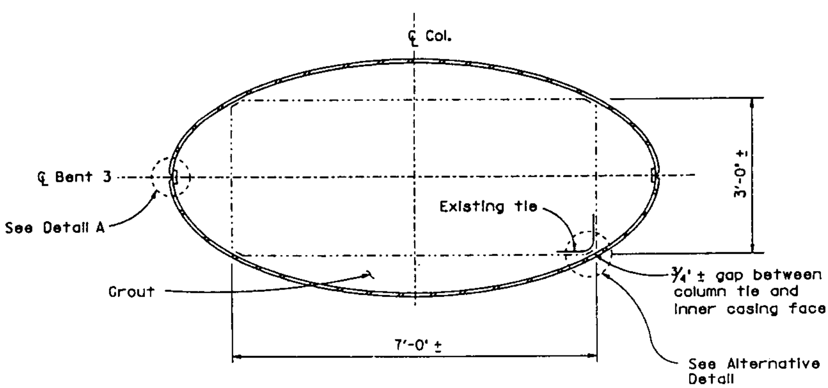


℄ Column
2" to 4" Clr.
Bottom of soffit
Temporary lifting ears (optional) to be removed and ground smooth after installation
B
B
Steel casing
Welded joint
H measured along ℄ of column
See note 6
A
A
Limits of Waterproofing See Note 7
2" Clr.
ELEVATION - BENTS 3 AND 5
Scale: 3/16" = 1'-0"
4'-0"±
Grouted gap
P.C.C. Column
See Detail A, Typ.
SECTION A-A
No Scale
Varies
4'-0"±
Grouted gap
P.C.C. Column
See Detail A, Typ.
SECTION B-B
No Scale

Figure 6



ELEVATION-BENT 3
Scale: 3/16" = 1'-0"



SECTION A-A
Scale: 3/4 " = 1'-0"

Figure 7

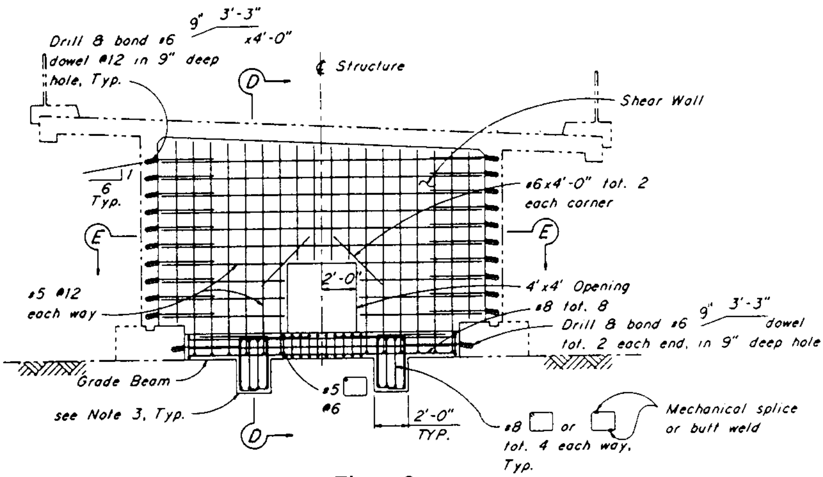
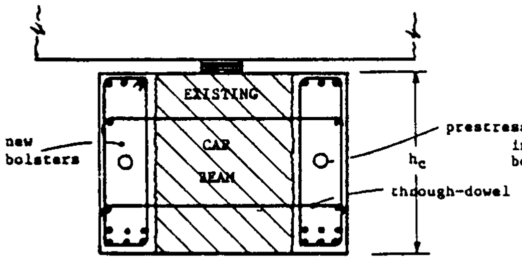


Figure 8



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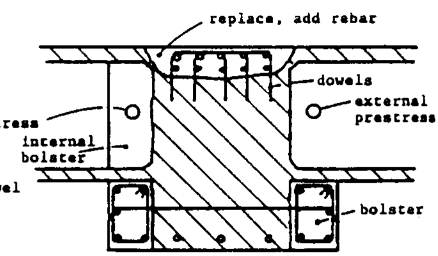

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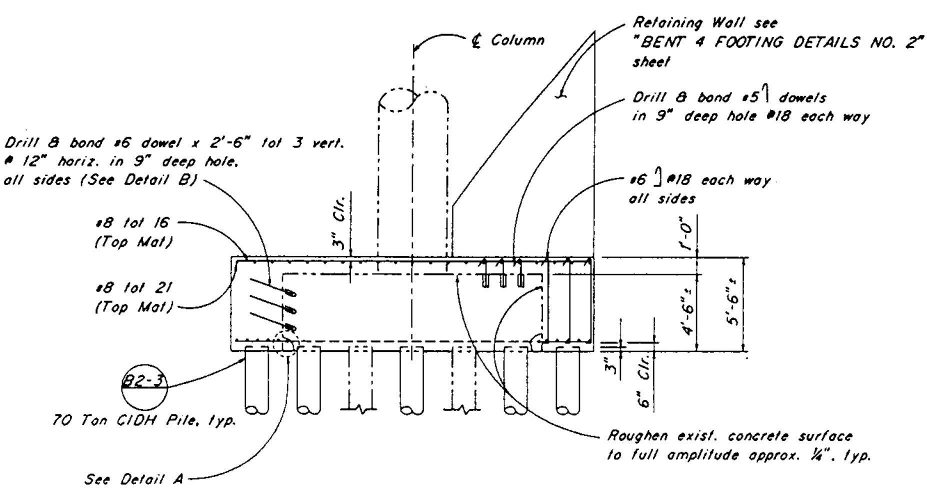

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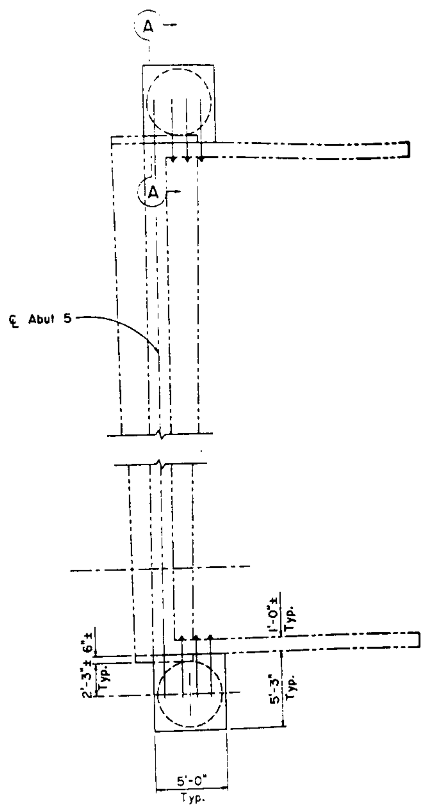

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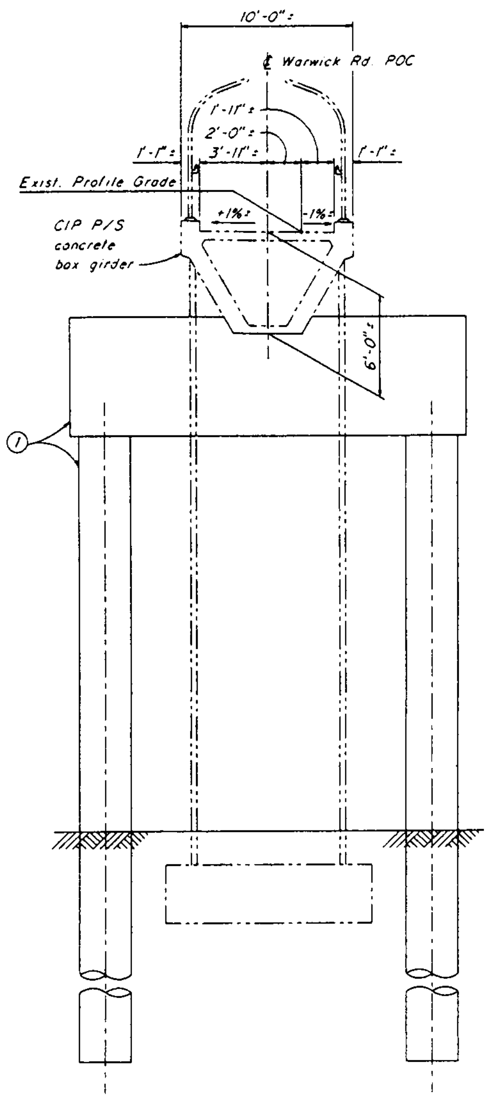

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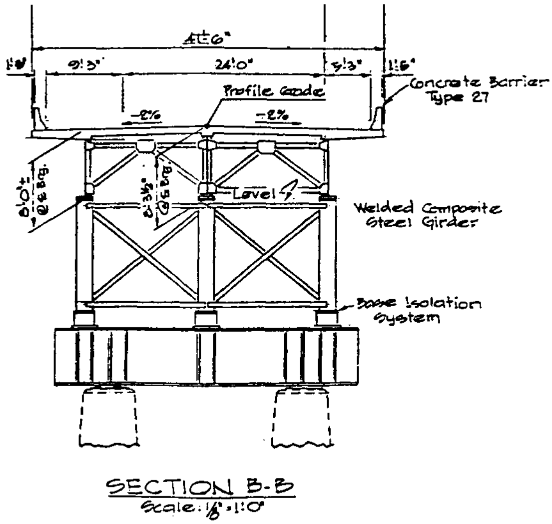

Figure 14

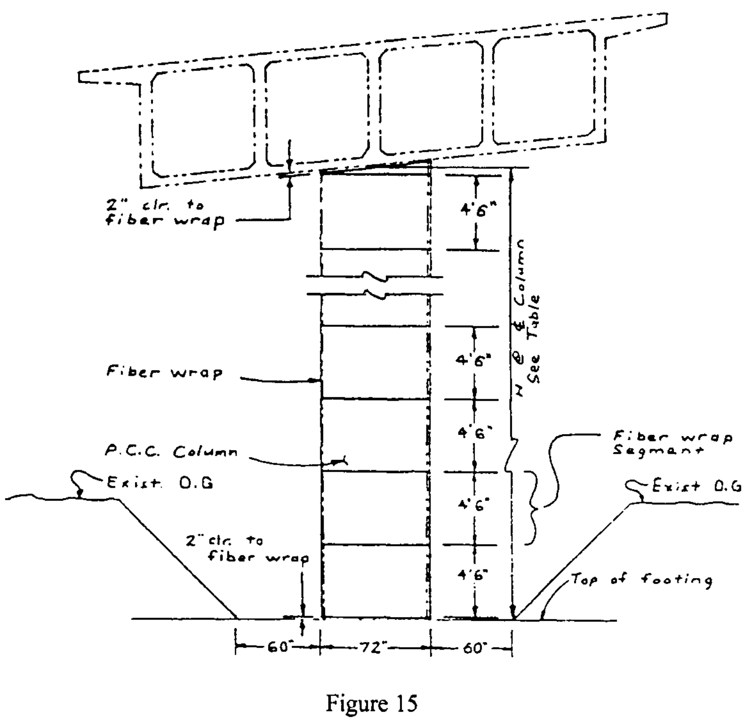

Figure 15

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INTRODUCTION TO TECHNOLOGICAL RESEARCH USING THIN REINFORCING STEEL PRESTRESSED CONCRETE IN ANTI-SEISMIC REINFORCEMENT

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ABSTRACT

This article introduces the rationale for technological research using thin reinforcing steel and prestressed concrete in anti-seismic reinforcement. In view of the basis of this technology in the course of construction and examples of actual construction using experimental engineering of different types, the writer holds that nonanti-seismic structures built before 1989 were surveyed, appraised and reinforced, but still carry heavy responsibilities due to large scale and quantity. To change traditional ways of reinforcement and speed up the course of anti-seismic reinforcement, there is a higher reference value and a bright future in the technology of using thin reinforcing steel prestressed concrete in anti-seismic reinforcement.

KEYWORDS

Seismic reinforcement; prestressed concrete; thin reinforcing steel; material; construction.

INTRODUCTION

It is one of the major anti-seismic missions of our country at present to reinforce buildings now available. The technology of using thin reinforcing steel prestressed concrete in anti-seismic reinforcement (called "thin reinforcement technology" for short) is a new method in anti-seismic reinforcement advanced by the writer and research group. This method is based on studying the present condition of anti-seismic reinforcement at home and abroad after many years' practice in anti-seismic reinforcement. This new technology aims at innovation to bring anti-seismic reinforcement of our country to a new stage.

RATIONALE FOR THIN REINFORCEMENT

This method of reinforcing accessories aims at increasing the strength and soundness of buildings as well as their ability to deform. Experience makes clear that the new method is feasible in terms of theory and practice. Former ways of reinforcing roof beams and pillars had deficiencies as follows .

1) Sizable consumption of material and money.

2) Large external form and heavy reinforcement body. This not only wastes material and money but also impairs the original style and architectural features of buildings.

3) In terms of technology, the component of reinforcement is nonprestressed. Its material is brittle and thus has good rigidity but poor pliability. End results are less than ideal. Costs are too high with the method of strengthening rigidity in reinforcement.

Thin reinforcement technology makes a vital improvement in traditional methods of reinforcement. It starts by increasing the ductility of structures, reinforcing the weak points to prevent collapse despite cracking.

The Rationale for This Method

1) Thin reinforced steel replaces thick reinforced steel.

2) Prestressing force is brought to bear on thin reinforced steel. This enhances structural soundness and raises the ductility and shear strength of buildings and structures.

3) Thin reinforced steel is anchored with proper density.

4) Surfaces are coated with thin concrete.

Major Advantages

1) By using this method of reinforcement, anti-seismic ability should meet local intensity requirements.

2) Use of thin reinforced steel, along with lighter weight reinforcement body, costs one-third less than the usual practice. Ease of operation cuts nearly in half the amount of time required.

3) The original style and architectural features of buildings can basically be retained which helps in urban beautification and historic preservation.

BASIS OF THIN REINFORCEMENT

Thin Reinforcing Steel

Many pieces of thin reinforcing steel replace thick reinforcing steel. Size and strength of thin reinforcing steel should be calculated on the principle that its strength cannot be lower than that of thick reinforcing steel. This conversion value can be obtained from Table 1.

Table 1. Conversion Table for Standard and Thin Reinforcing Steel

Item	Reinforcing method	Diameter (mm)	Stress (KN)	Weight (kg)
Girth	Standard	4Φ12	108.6	3.552
	Thin	12Φ65	122.3	1.848
4 Corner posts	Standard	12Φ12	325.7	10.656
	Thin	36Φ65	366.9	5.544
Draw bar	Standard	2Φ12	54.3	1.776
	Thin	6Φ65	61.2	0.924
Buttress posts	Standard	6Φ12	162.9	5.328
	Thin	18Φ65	183.5	2.772

Thinness

In reinforcement, the major action of concrete is not to withstand pressure, but to protect reinforcing steel and buildings to which it is adhered. Therefore, overly thick concrete causes not only diffiult construction and higher cost but also more seismic force as an end result due to increased dead weight. Thin reinforcing steel changes only minimally the section measurement of roof beams, pillars and walls. Thin reinforcement can also protect reinforcing steel and buildings by reducing force due to decreased dead weight. Side effects of reinforcement are thus lessened significantly. As noted, the orginal style and features of buildings can be preserved.

Prestressing Force

To use thin reinforcing steel, it is fastened tightly to the reinforcement body with anchorage at both ends of the body, pressed against the original building, bring the right amount of prestressing force to bear on it. Buildings can thus be tightly bound which surpasses the usual practice in improving structural soundness.

EXAMPLES OF ACTUAL CONSTRUCTION WITH TECHNOLOGY OF THIN REINFORCEMENT

Practical experience in using thin reinforcing steel prestressed concrete in anti-seismic reinforcement was derived from variours structures such as the 1st Middle School of Huaiyin (a two-storey brick-concrete building), housing at Civil Air Defence of Nantong (one-storey underground and five-storey above ground brick concrete structure), and the 35-meter-high brick chimney of Nantong Pesticide Factory. Housing in Nantong serves as our example.

Features of Project

Housing at Civil Air Defence Company in Nantong (No.68, Damatou) was built in December 1982. Its area is 2300 square meters. Its structure is as follows: one-storey underground basement, five-storey above ground brick-concrete building, first three storeys of clay brick, other two storeys with aerated cement, compound slab roof and no structural pillars. After a number of years, many slanting, level and vertical crevices appear on inside and outside walls. Width of the crevices is about 8mm. Roof boards are compound slab, so the thermal effect is poor, deformation due to temperature is large, and it is unsafe for occupancy.

In August 1991, the Civil Air Defence Office of Nantong invited experts from the design department of the city's Construction Commission and Appraisal Group for Anti-Seismic Reinforcement to evaluate this residential building. They shared the same view on the need for anti-seismic reinforcement. Details of the project are as follows.

1) Structural pillars, girths, and draw bars should be added to strengthen the entire building in accordance with the seismic intensity of 6 degrees.

2) To avoid deformation of the building due to temperature, the insulating layer of roofing should be reinforced .

3) In the process of reinforcement, wall crevices should be dealt with, especially the walls on the 4th and 5th floor which should be reinforced .

4) In the design phase, check the strength of the building's foundation on the basis of the load after being reinforced and then determine whether or not to reinforce the foundation .

Considering the evidence of a greatly increased load after reinforcement, as well as the reinforcing method in operation, and for the sake of using material and financial resources sparingly, the Civil Air Defence Office of city of Nantong made a decision. They decided to entrust Anti-Seismic Office of Construction

Commission of city of Huaiyin to design and construct anti-seismic reinforcement by using the new technology of thin reinforcing steel prestressed concrete .

Machines and Special-Purpose Materials

There are two key links for tensility of thin reinforcing steel and anchorage in the construction of thin reinforcement. To ensure that both links meet quality requirements, special-purpose thin reinforcing steel, special-purpose anchor for fixing this steel and walls, in addition to concrete, sand and other building materials, were used. Besides general building machines, we developed special-purpose on-the-spot thin reinforcing steel prestressing force tensiler. This machine is small and easy to operate. It can put the appropriate prestressed force numeral value from 0 to 1,000kg on thin reinforcing steel, and satisfy completely the needs of construction technology for thin prestressed force .

Construction Technology

Construction steps are: a) Eradicate whitewash layer and clean walls in accordance with design width of positions reinforced. b) Fix anchorage at both ends of positions reinforced and fasten both ends of reinforcing steel. c) Reinforcing force tensiler tensiles in accordance with design drawing. d) Install reinforcing steel at fixed intervals along the direction of the steel's tensility and attach cramps to the walls at these intervals to make the steel flush with the walls. e) Smear the walls with cement mortar and cement mortar surface layer.

Experience

Thin reinforcing steel, thinness and prestressing force are three key problems in this technology. We know from experience that the following aspects of construction affect whether or not engineering quality meets design requirements in anti-seismic reinforcement.

1) In the process of bearing, thin reinforcing steel loses fixed stress. Select anchoring foot of proper density to anchor with bearing thin reinforcing steel and wall unit. In this way, stress losses can be decreased effectively. Also thin reinforcing steel can be pressed closer to the wall unit, be poured with the wall unit and help parts newly reinforced function together with the unit as a whole.

2) Stipulate stress value for hoop reinforcing steel in construction . a) Satisfy the requirement of thin reinforcing steel flush with the wall. b) Bearing thin reinforcing steel should bind the wall only, and have no extra pressure on the wall unit. c) Bearing thin reinforcing steel basically cannot be flawed in physical and chemical properties or strength of steel. On the basis of the above-mentioned requirements, prestress is 5 to 10 percent of permissible strength of its material. d) For reinforcing steel bearing corners and posts of

vertical walls, the prestress value can be increased appropriately (within 40 percent of permissible strength) as vertical prestress value and frictional coefficient in the building's storeys are increased, and shear strength of wall body is strengthened. Furthermore, load to the foundation is not increased.

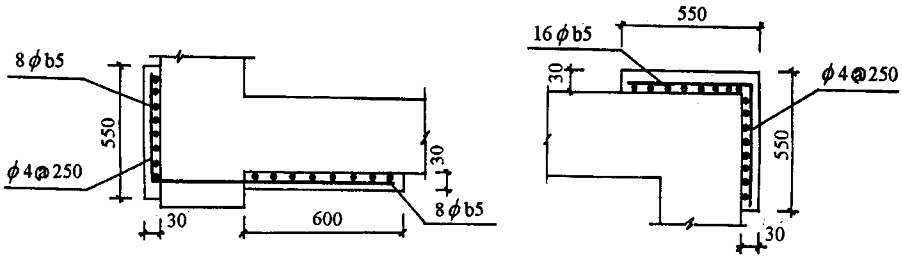

Fig. 1 Level reinforcing steel installed on wall body

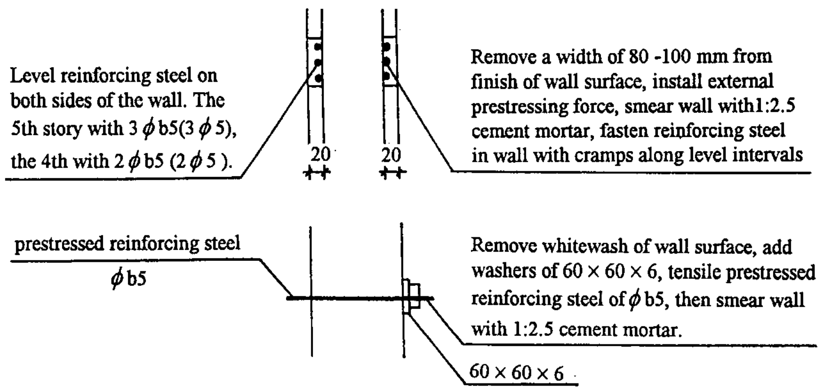

Fig. 2 Prestressing force draw bar anchoring on the wall surface

Adopt the method of symmetrical force augmentation for overcoming the eccentric stress effect probably caused by vertical prestress force in construction.

CONCLUSIONS

Thin reinforcing steel used in the technology of anti-seismic reinforcement with thin reinforcing steel prestress concrete is calculated on the basis of equistrength conversion, and the strength of it is higher than that of thick reinforcing steel.

Technology of thin reinforcement bears prestress force, the integral performance of building is enhanced crosswise and frictional coefficient in the building's storeys is increased vertically. It generally enhances the shear strength and ductility of a building .

Technology of thin reinforcement is simple and easy to construct, does not disrupt the building's original structure, appearance, style and features. It reduces pollution and shorters the time project takes.

Funds and material resources can be used sparingly with this technology .

Judging from the statistics of anti-seismic reinforcement for the whole nation, after the utmost effort of many years, anti-seismic reinforcement work is only a little more than one-third done. Our responsibilities are still heavy. Therefore, the new technology has a bright future .

For mastering the design and calculation of anti-seismic reinforcement, we suggest conducting single wall and integral model tests to understand the deformation and bearing capacity under repeated load and characteristics of integral statics and dynamics.

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APPRAISAL AND RECONSTRUCTION-STRENGTHENING DESIGN OF REINFORCED CONCRETE FRAME STRUCTURE AFTER AN EARTHQUAKE

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ABSTRACT

This paper is based on investigation of 198 R.C. frame buildings after the 1976 Tangshan earthquake and application of the Code for Seismic Design of Buildings. Appraising methods and foundations for R.C. frame structures after an earthquake are discussed. A series of recovery reinforcement methods and suitable conditions is offered. A three-level, three-stage anti-seismic method for R.C. frame structure design and reinforcement is presented.

KEYWORDS

Reinforced concrete; frame structure; seismic design; appraisal method; recovery; time history analysis; design standard.

GENERAL SURVEY

Earthquake data from Tangshan (see Liu Huixian, 1986) shows that among the 198 factory buildings with multistory R.C. frame structures, 8% collapsed and 27.8% were damaged severely (see Tab. 1). According to the principles of post-earthquake recovery, buildings should be repaired and reused if possible. Thus factory buildings damaged severely were repaired and reinforced, and production resumed generally within one year. In this way, a large amount of money was saved. Moreover, economic benefit was gained due to rapid production recovery.

Earthquake engineering in China developed quickly after Tangshan major earthquake. The Code for Seismic

Design of Buildings, GBJ11-89, and The Specification of the Seismic Intensity Zoning Map of China (1990), among others, were implemented. Other state standards such as The Code of Building Resistance Earthquake Appraisement and Reinforcement Design, The Administrative Regulations of Recovery and Rebuilding after Earthquake in Earthquake Area, among others, will be issued soon. In-depth studies of earthquake resistance of R.C. frame structures have been conducted. These results have laid a good foundation for the technology of structure appraisal and recovery reinforcement. Structural seismic effect and resistance remain uncertain. Post-earthquake appraisal and reinforcement of structures differs from seismic-resistant design of new buildings and from pre-earthquake appraisal and reinforcement of buildings. Many questions need to be discussed but there are few disscusions. Of special concern in China are the R.C. frame structural buildings comprising tens of millions of square meters. Seismic activity covers a vast area and inflicts terrible damage, so it is of great importance in engineering and the economy to discuss appraisal and recovery reinforcement.

Table 1 Earthquake Calamities of R.C. Frame Buildings

Intensity	Place	No.	In good condition		Slight damage		Moderate damage		Severe damage		Collapsed		Remarks
			No.	%	No.	%	No.	%	No.	%	No.	%	
6	Beijing	8	4	50.0	4	50.0							civil
7	Dafu in Tianjin Qianan in Tangshan	8	7	87.5	1	12.5							chemical industry
8	Tianjin	56	31	55.4	4	7.1	14	25.0	5	8.9	2(1)	3.6	light industry buildings
9	Tangshan Hangu in Tianjin	74	8(2)	10.8	13	17.6	14	18.9	33	44.6	6	8.1	
10	Tangshan	42	4(3)	9.5	6(3)	38.1	4	9.5	11	26.2	7	16.7	
11	Tangshan	10	1(4)	10.0			2	20.0	6	60.0	1	10.0	
	Total	198	55	27.8	38	19.2	34	17.2	55	27.8	16	8.0	

Remarks: (1) 13-storey frame in Tianjin alkali factory, and south-central building in second wool spinning factory in Tianjin.

(2) building for boiler room in Tianjin chemical factory.

(3) workshops of fireproof material factory, coking factory and cement factory.

(4) corridor of Kailuan coal mine.

APPRAISAL OF R.C. FRAME STRUCTURE AFTER EARTHQUAKE

Structure Appraisal after an Earthquake

Steps in this process are:

(1) Primary design, computing, construction maps and budget estimate.

(2) Construction notes on structure (to include changing design, quality of material, etc.).

(3) Maintenance conditions (such as concrete cracking, reinforced bar rusting).

(4) Study of materials in earthquake calamity.

Main factors influencing earthquake calamity of an R.C. frame structure are listed below.

External actions:

Strong motion-peak acceleration, earthquake intensity
Site conditions and change of foundation
Structural action-live load
Structural distribution of rigidity and quality
Type of building-elevation, plane figure

Structural supporting ability:

Structural system-spaced properly
Foundation plan
Size of section or member
Strength of materials
Volume of reinforcing bar
Construction measure

It is obvious that a stronger earthquake induces more serious calamity. Two examples of collapse took place in an 8-degree district, one the south-central building in the second wool spinning factory in Tianjin (see Wei Lian, 1981), and the other a 13-storey frame in Tianjin alkali factory. Three positions in floor columns, 26 positions in the second-storey columns and 25 positions in upper columns were destroyed in an earthquake of magnitude 7.8 in Tangshan, and collapsed in an earthquake of magnitude 6.9 (8 degrees) in Lutainan (see Fig. 1). Main causes may be serious liquefaction in foundation, cumulative seismic effect, small section of column (350×350), strong beam and weak column style.

Damaged forms and damaged degree at the ends of beams and columns determine the recovery reinforcement, that means appearance and development of plastic hinges.

Appraisal Methods for R.C. Frame Structure after Earthquake

Checking Anti-Seismic Computations for the Original Structure. Check the load-bearing capacity, elastic lateral distance and elastoplastic distance of the original structure according to real vibration parameters, and to compare them with the actual seismic damage. This sheds light on the reasons for the damage.

Response spectrum method cannot give an accurate illustration because the structure is in an elastoplastic state during a strong earthquake. Therefore time-history analysis may be used to appraise R.C. frame structure after an earthquake. At the outset, it should be made clear whether the structure is strong-column shape, strong-beam shape, or mixed shape.

Anti-seismic Structural Checking after an Earthquake. According to real vibrating parameters of floor and site, establish a suitable mechanical model. Next analyse structural load-bearing capacity and distance in the light of elastoplastic time-history analysis method. Then take out the thin storeys in the structure and thin sections of members.

Using Anti-Seismic Measures of Current Codes. Using these measures, pick out the places which contradict construction rules (including axis--pressing ratio of columns, closed stirrup at the ends of beam and column, anchoring of reinforced bars, stirrups at joint cores).

FRAME STRUCTURE AFTER EARTHQUAKE

Primary Method and Suitable Conditions (see Wei Lian, 1981.5.)

Steps to carry out this procedure are as follows.

(1) Reinforcing frame joints. A thickness of 70 --120 mm R.C. may be wrapped around joints when load-bearing capacity and ductility of joints are insufficient.

(2) Improving deformation capacity of columns with a spiral stirrup around circular column and a steel cover using flat or angle shape about rectangular column.

(3) Improving load-bearing capacity and deformation capacity of rectangular column by reinforcing four corners with L-type steel.

(4) Improving load-bearing capacity and deformation capacity of columns by reinforcing a thickness of 80 -- 100 mm R.C. around the column.

(5) Increasing load-bearing capacity and deformation capacity of entire frames by steel supports in some cases, or vibration-reducing device. This is suitable for hospitals, schools, and other important buildings.

(6) Increasing anti-seismic brick walls, and combining with partition walls and enclosing walls.

(7) Increasing anti-seismic R.C. walls when the load-bearing capacity and deformation capacity of frame are too low.

It is obvious that above three reinforcing methods can increase the load-bearing capacity and deformation capacity of a frame structure, and make it a frame shear structure.

Controlling Design Method of R.C. Frame Based on Three Levels and Three Stages

Consideration of Standards for Engineering to Reduce and Prevent Earthquake Damage. In general, this means traditional seismic resistance and reduction (including seismic isolation).

Japan requires 2nd and 3rd type buildings to be designed as the 1st level, and k = 0.2. Their code stipulates that the building should not be damaged after a small earthquake. Designed as 2nd level, and K = 1.0, it also requires that the building not collapse during a large earthquake. For a building over 60m, an elastoplastic time-history analysis should be made and approved by construction department. Recently the Kobe-Osaka earthquake caused more than 5000 deaths and $99.60 billion in losses. Damage caused by earthquakes is heavy. China's standard of resistance earthquake is lower than that of Japan's. This was reasonable considering the nation's economic condition in the 1970s and 1980s. With the development of economic construction, establishment of a market economy and heavy lessons from earthquakes, China joined with international standards. Our nation took economic construction and disaster reduction into account at the same time. At a meeting on reducing disaster and preventing earthquakes on 25th November 1994, a national goal was established. This goal is that the buildings of large and mid-size cities should be able to resist an earthquake of magnitude 6. So it should raise the level of structural resistance to earthquake and preventive measures, especially for the repair and reinforcement of buildings after an earthquake. We should consider large-scale repair work and adopt a higher-standard long-term plan.

Consideration of Design Standards for Frames Based on 3-Level and 3-Stage Earthquake Resistance. GBJ11-89 specifies standards and measures of resistance to earthquakes, that is " no damage after small earthquake, reparable after a moderate earthquake, no collapes after a major earthquake". For some frames, the relative drift angle of stories should be examined under the action of earthquake. The code does not specify the demands of repair after a moderate quake or the requirements of failure state and failure level after elastoplastic deformation occurs. According to structural mechanism theory (see Wang Chongchang, 1986), the structural failure state and failure level should have some elastoplastic deformation to form an absorbing energy mechanism to meet requirements for durability and for reparable after a moderate quake, no collapse after a large quake (see Fig. 2). It can be expressed as follows:

$$[\,Q_i\,] \le Q_i \le [\,Q'_i\,] \tag{1}$$

where i means 2nd or 3rd level, Q_i $[Q_i]$ $[Q'_i]$ means the relative drift angle of stories, the relative angle based on limit durabililty and the relative drift angle of stories based on preventive measures under 2nd and 3rd level seismic action. Consider the structure to be middle frequency before yielding which conforms to the principle of "absorbing energy equally". After yielding, it becomes a flexible structure and conforms to the principle of equal displacement. An example is a wall filled with bricks to resist lateral force. Its elastic limit relative drift angle of stories is 1/550; through calculating the durable limit relative angle of stories under the action of earthquake of magnitude of 7, 8, 9, the relative angles are 1/130, 1/70, 1/75, and 1/90, while preventive limit relative angle of stories are 1/100 and 1/30 (see Fig. 3).

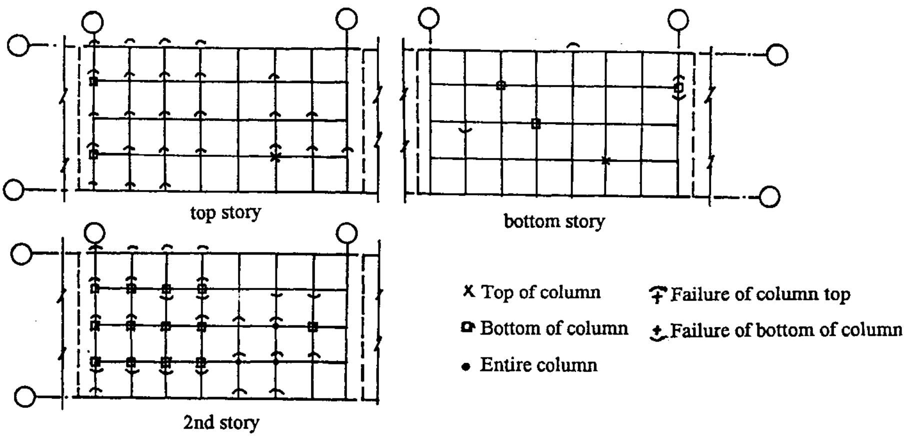

Fig. 1 The figure of failure and reinforced column after earthquake

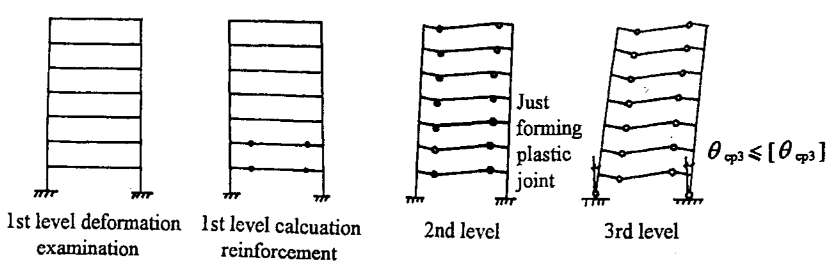

Fig. 2 The 3-level design state of R.C. Frame

Response Spectrum Method of Decomposing Vibrating Pattern. In chemical, electrical, light industry and civil buildings, the frame design is complex due to storage, equipment and partitions. It is common to have a floor with a large hole, alternating stories, composite frame. So the model for calculating structural resistance to earthquakes should take spatial behaviour, deformation of floor and torsional vibration pattern into account. In general, we adopt CQC method of composing vibration pattern to determine the real mechanical character, to find seismically weak parts and weak sections of elements. When investigating seismic resistance of repaired frames, perhaps we should consider the decline of resistance. For example, in a 50-year-old building the coefficient of decline should be 0.8.

Analysis Method of Time History. To compare the calculation results for composing vibration pattern, we should process the time history of elastic structure with much wave input.

To calculate the elastoplastic deformation of a structure under the action of a moderate or major earthquake, we ought to process elastoplastic time history of a structure in such earthquakes. Because the acceleration curve of floor vibration that we input is determined through risk analysis of an earthquake, we can use extended duration method to consider seismic influence and cumulative seismic damage (see Men Kai, 1995). Also, through elastoplastic time history, we can determine the joints appearance order and position. This process helps to achieve controlling design of structure mechanisms. For handling the results of time history, among correlative methods tried, the method of average response value and method of maxium envelope value are used. In general, the method of maxium envelope value is adopted, especially for repair and reinforcement design.

Choice of CAD Software. Computer techniques promote the development of earthquake engineering; calculating structural mechanics creates the conditions for accurate calculation of complex structures. This is important in developing countries and in the design of repair and reconstruction engineering. Economic benefits also derive from scientific techniques and careful planning.

At present, elastic structural CAD software applied to engineering includes TBSACAD, TAT, CATAR (Institute of Structural Engineering of China Academy of Building Research), BSSA, BSSD (Zhejiang Institute of Architectural Design), DASTAB (Dalian University of Science and Engineering), BGDG BSAD (Hunan University). Elastic time history analysis software includes TBDYNA, TAD-D (Institute of Structural Engineering of China Academy of Building Research). Elastoplastic time history analysis software includes PERS SPCB (Institute of Structural Engineering of China Academy of Building Research), NDCAD (Hunan University), and TAMSC (Tsinghua University). Software for elastoplastic time history analysis of spatial structural models has yet to be developed.

Estimate of Effects Repair and Reinforcement of R.C. Frame. Compare the original structure, using curve of story shear-lateral deformation, to a repaired and reinforced typical structure after seismic damage (see Fig. 4). Then the effect of repair and reinforcement of the frame can be estimated. Calculate the technical index of repaired and reinforced structures to summarize the experience of repair and reinforcement engineering.

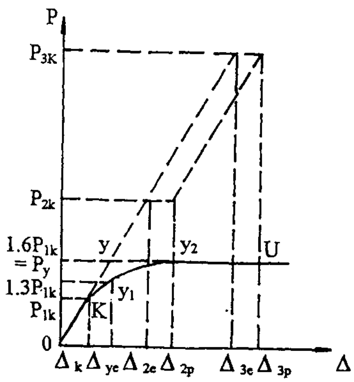

Fig. 3 P- Δ Curve of Frame Structure

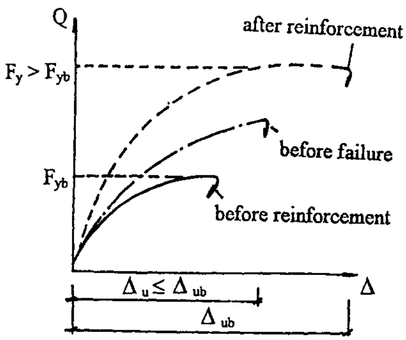

Fig. 4 Curve of Q- Δ

CONCLUSIONS

Through an investigation of 198 R.C. frames in the areas of magnitude 6-11 during Tangshan earthquake, less than 10% of them collapsed and 40% suffered little or no damage. This shows that R.C. frame has a good capacity for earthquake resistance. Also more than 90% of frame structures, including some damaged badly, were used after repair and reinforcement with good economic and social benefits. Reliable determination methods should be adopted after an earthquake. This allows for fully determining the causes of failure, seismically weak story of structure and weak section of element. Proper repair and reinforcement patterns should be identified to avoid creating a new weak section after repaired and reinforcement. Higher standards of seismic resistance along with careful repair and reinforcement of buildings are suggested. Three-level, three-stage controlling design method should be adopted for seismic resistance. As the chief determination method of R.C. frame for resisting earthquake, elastoplastic time history analysis is used.

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MEGA-SUB CONTROL SYSTEM FOR REDUCING SEISMIC RESPONSE OF TALL BUILDINGS

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ABSTRACT

In the present study, the performance of s mega-sub control system for tall buildings which was proposed recently is evaluated against near-field ground motions recorded from the January 17, 1994 Northridge Earthquake and the January 17, 1995 South Hyogo Earthquake. The mega-sub control system is a passive vibration control system which takes the advantage of the structural configuration of a mega-structure. In the system, the sub-structures contained in the mega-structure are tuned to suppress vibration response induced by earthquake or wind load. Numerical simulations for a 200m-tall steel building are conducted to show its feasibility and effectiveness. The results were compared with those for the standard El Centro 1940 NS ground motion.

KEYWORDS

Mega-structure; structural control; tall building; damper; near-field motion.

INTRODUCTION

It was a surprising incident for our engineering society that many tall steel buildings were damaged in the January 17, 1994 Northridge Earthquake and the January 17, 1995 South Hyogo Earthquake. In some cases, the steel columns were horizontally cut at their feet. In most cases, cracks were found in the vicinity of column and beam joints. Even in the undamaged steel buildings, the large deformation and drift mainly due to their low inherent dampings compared to reinforced concrete buildings caused collapse of walls and equipments inside. It is therefore an urgent task for structural engineers to limit or reduce the deformation and drift in lightly-damped buildings under seismic load. The simplest resolution for it is to augment damping devices into structural systems. However, structural characteristics common to most tall buildings, such as high rigidity of the structural system and the dominant bending deformation, result in technical difficulties which prevent the application of conventional damping devices [Watabe & Mita, 1993]. The high rigidity requires an unrealistically large number of damping devices in order to achieve a certain desirable damping ratio. A major component of the deformation associated with the fundamental mode is the bending deformation so that damping devices utilizing shear deformation would be no longer effective.

The mega-sub control system taking the advantage of the mega-structure configuration has been proposed by [Mita & Feng, 1994, Feng & Mita, 1995] to overcome these difficulties. The principle of the system for reducing vibration is the same with that for conventional mass damper systems. However, the system requires no additional mass. Mass damper systems such as a tuned mass damper and a hybrid mass damper have been applied to several actual buildings to suppress vibration dynamically. Unfortunately, they can only be used to ensure human comfort, but not to protect the building under earthquakes because of the power limitation of actuators as well as the stroke limitation. An attractive strategy for resolving the above mentioned issues is to use a portion of the building itself as a mass damper. Because a large mass ratio can be achieved, such a system has a potential to be used for safety purpose. Suspended floors [Nishiyama et al., 1991] and upper portions of a building [Ishimaru et al., 1991] were suggested to be used as mass dampers. In this paper, our interest is focused on the performance of the mega-sub control system against near-field ground motions which contain destructive energies in the high-frequency range.

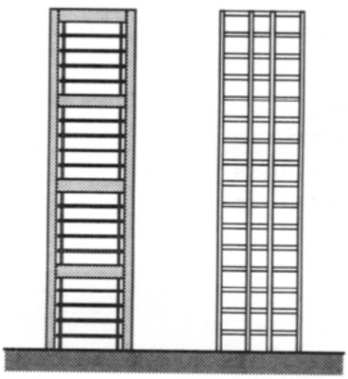

Figure 1. Mega-structure and rigid-frame.

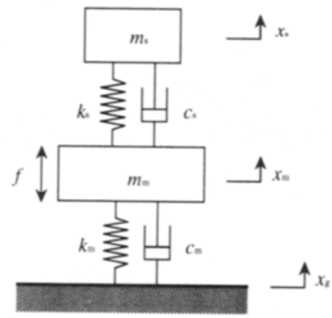

Figure 2. Simplified model.

MEGA-SUB CONTROL SYSTEM

The mega-structure is schematically shown in Fig. 1 in which a conventional rigid frame system is also shown for comparison. The mega-structure configuration is popular structural systems for super tall buildings because this system exhibits structural efficiency by allowing high rigidity of the structure while keeping minimum amount of structural materials to be used. The additional advantage of the mega-structure configuration is its planning flexibility for sub-structures consisting of several floors in the mega-structure. The structural constraints for the floor plans in the sub-structures are minimum. A response control strategy utilizing this mega-structure configuration is established as follows: First, the vibration energy (kinetic energy) of the mega-structure due to wind or seismic loads is transferred into sub-structures. Second, the transferred energy is dissipated in the sub-structures. The first step can be achieved by tuning the dynamic characteristics of sub-structures so that most kinetic energy naturally flows into the sub-structures. The second step can be easily established by designing the sub-structures to vibrate in shear modes so that any conventional damping devices could be used for energy dissipation in the sub-structures.

In the system, the sub-structures themselves serve as absorbers and thus the mass ratio achieved by the system can be significantly larger than the mass ratio of the conventional mass damper system which is typically of the order of one percent. The large mass ratio implies that extremely high level of response reduction in not only the mega- but also the sub-structures can be achieved. In addition, the system does not require any additional mass as seen in a conventional mass damper system. In the design of a conventional mass damper system, the response of the tuned mass itself is not the major concern except for the amount of its stroke. In the case of the mega-sub control system, however, the vibration of the sub-structures, which actually corresponds to the tuned mass in the mass damper system, also needs to be controlled to a low level for the purpose of improving the comfort of occupants and protecting vibration-sensitive equipment and other non structural components housed in the sub-structures. Therefore, the control objective here is not only to suppress the vibration of the mega-structure for structural safety purpose but also to confine the vibration of the sub-structures in an acceptable range.

The equation of motion for a simplified two-degrees-of-freedom system depicted in Fig. 2 and subject to wind and seismic loads can be written in the form

$$\begin{bmatrix} m_m & 0 \\ 0 & m_s \end{bmatrix} \begin{Bmatrix} \ddot{x}_m + \ddot{x}_g \\ \ddot{x}_s + \ddot{x}_g \end{Bmatrix} + \begin{bmatrix} c_m + c_s & -c_s \\ -c_s & c_s \end{bmatrix} \begin{Bmatrix} \dot{x}_m \\ \dot{x}_s \end{Bmatrix} + \begin{bmatrix} k_m + k_s & -k_s \\ -k_s & k_s \end{bmatrix} \begin{Bmatrix} x_m \\ x_s \end{Bmatrix} = \begin{Bmatrix} f \\ 0 \end{Bmatrix} \tag{1}$$

where x_m and x_s are the relative displacement of the mega- and sub-structures with respect to the ground. The natural frequencies and damping ratios are defined as follows.

$$\omega_m = \sqrt{\frac{k_m}{m_m}}, \quad \omega_s = \sqrt{\frac{k_s}{m_s}}, \quad h_m = \frac{c_m}{2m_m\omega_m}, \quad h_s = \frac{c_s}{2m_s\omega_m} \tag{2}$$

It is noted that the damping ratio of the sub-structure is defined with reference to the natural frequency of the mega-structure ω_m instead of the natural frequency of the sub-structure ω_s. The mass ratio and the frequency ratio are represented by the following parameters.

$$\mu = \frac{m_s}{m_m}, \quad \beta = \frac{\omega_s}{\omega_m} \tag{3}$$

The optimum parameters are determined to minimize the mean square values of the target response when subject to white noise as wind or seismic loads. The method to obtain these parameters can be found elsewhere [e.g. Grandall and Mark, 1963]. In the process, the damping ratio for the mega-structure is assumed to be zero to decouple the equations. The optimum parameters listed in Table 1 can be extended to multi-degree-of-freedom systems with the help of modal decomposition [Mita and Kaneko, 1994].

Table 1. Optimum parameters.

Excitation	Target response	β_{opt}	h_{sopt}
Wind f	x_m	$\frac{\sqrt{1+\mu/2}}{1+\mu}$	$\frac{1}{2}\sqrt{(1+\mu)\beta^4 - \frac{2+\mu}{1+\mu}\beta^2 + \frac{1}{1+\mu}}$
	$\ddot{x}_s$	0	$\frac{\beta^2}{2}$
Earthquake $\ddot{x}_g$	x_m	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	$\frac{1}{2}\sqrt{(1+\mu)\beta^4 + \beta^2 + \frac{1-3(1+\mu)^2\beta^2}{(1+\mu)^3}}$
	$\ddot{x}_s + \ddot{x}_g$	0	$\frac{\beta}{2}\sqrt{(1+\mu)\beta^2 + 1 - \frac{1}{1+\mu}}$

RESPONSE CHARACTERISTICS FOR NEAR-FIELD MOTIONS

Many accelerograms were recently recorded in the vicinity of the fault in the U. S. and Japan. Those records revealed the significantly large peak values in acceleration, velocity and displacement. Such large peak values have no been considered in the seismic design in most countries. It is a commonly urged subject to evaluate the impact of the large near-field ground motions on engineered buildings. In the following, performance of the mega-sub control system is examined for such near-field ground motions compared with a conventional building.

Table 2. Peak acceleration, velocity and displacement of selected near-field data.

Accelerogram	PGA (cm/s/s)	PGV (cm/s)	PGD (cm)
Kobe 1995 NS	818	90	21
Sylmar 1994 NS	827	139	57
El Centro 1940 NS	342	33	11

Selected Near-Field Data

Three important accelerograms are selected. The Kobe 1995 NS is the ground motion of the January 17, 1995 South Hyogo Earthquake in the NS direction recorded at Kobe by the Japan Meteorological Agency. The Sylmar 1994 NS is from the January 17, 1994 Northridge Earthquake at Sylmar recorded by the California Division of Mines and Geology. The El Centro 1940 NS was included as a reference. The peak ground acceleration (PGA), peak ground velocity (PGV) and peak ground displacement (PGV) obtained from the accelerograms are summarized in Table 2. Figure 3 represents the time histories of the ground motions used here. The distinctive importance of these near-field ground motions is the presence of large pulses resulting in the large velocities and displacements. The Kobe 1995 NS and the Sylmar 1994 NS show relatively large values in acceleration, velocity and displacement compared to the standard El Centro 1940 NS. Two percent damped response spectra for the near-field accelerograms are shown in Figure 4. The figure provides the frequency content of the accelerograms. It is noted that the shapes of the response spectra are almost identical when the PGA is properly adjusted.

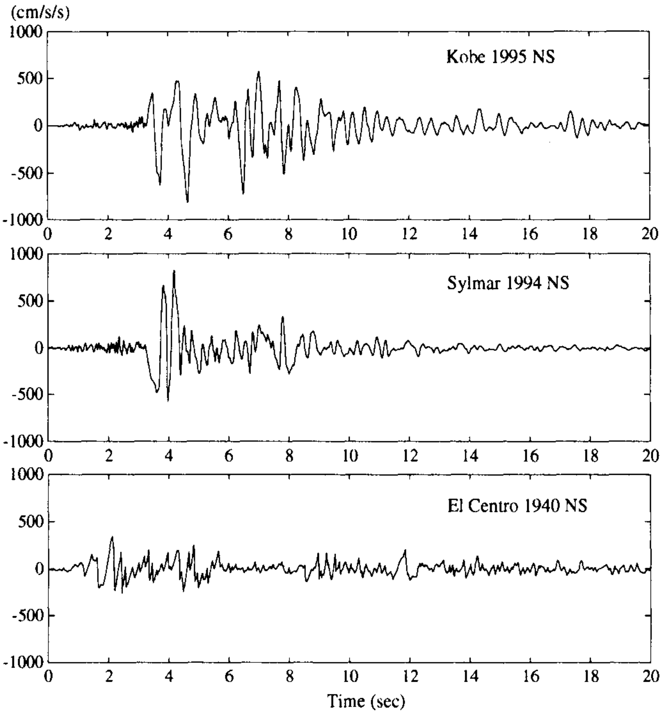

Figure 3. Acceleration time histories of ground motions.

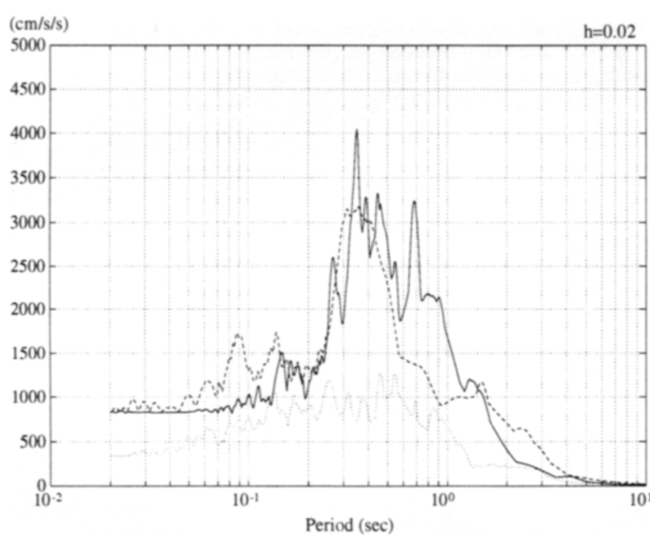

Figure 4. Acceleration response spectra for Kobe 1995 NS (solid line), Sylmar 1994 NS (broken line) and El Centro 1940 NS (dashed line).

Analysis Models

A 200m-tall steel building is considered. The total mass of the building is 70,000Mg. The spatial dimension of the building is 40m×40m×200m. Three models are employed for representing a rigid frame system and two mega-structure systems. The RF model is a building of rigid frame system. The damping ratio for the RF model is 0.02 for all modes. The first mega-structure model MS1 is equipped with a mega-sub control system in which the frequency ratio β of the sub-structures with respect to the first natural frequency of the mega-structure is set to be 0.5 as indicated in Figure 5. The second mega-structure model MS2 is equivalent to the MS1 except that the frequency ratio β=0.5 is defined in the reference to the second natural frequency of the mega-structure. The damping ratio in the mega-structure is 0.02 for all modes of vibration. It is noted that the mega-sub control system is applied only for top three blocks of the mega-structure due to architectural reasons. However, contribution of the first block for reducing the response is minimal. Therefore, limiting application of the mega-sub control system to the top three mega-blocks has little effects on the performance of the system.

The amount of dampers required to be implemented in the sub-structures is determined from the damping ratio h_s defined by Eq. 2. The value for the h_s is set to be 0.2 for both MS1 and MS2. It is noted that the natural frequency of the mega-structure needed for evaluating the damping ratio is the first natural frequency for both models. Therefore, the amount of dampers required for the MS1 is the same with that for the MS2. The amount assumed here is reasonable since it is in the same order of dampers typically used for a base-isolated building.

The amplitude of the acceleration transfer function between the ground motion and the acceleration response of the top mega-mass is shown in Figure 6. This figure clearly indicates that the MS1 is effective for response reduction on all modes, while the MS2 on modes higher than the second mode.

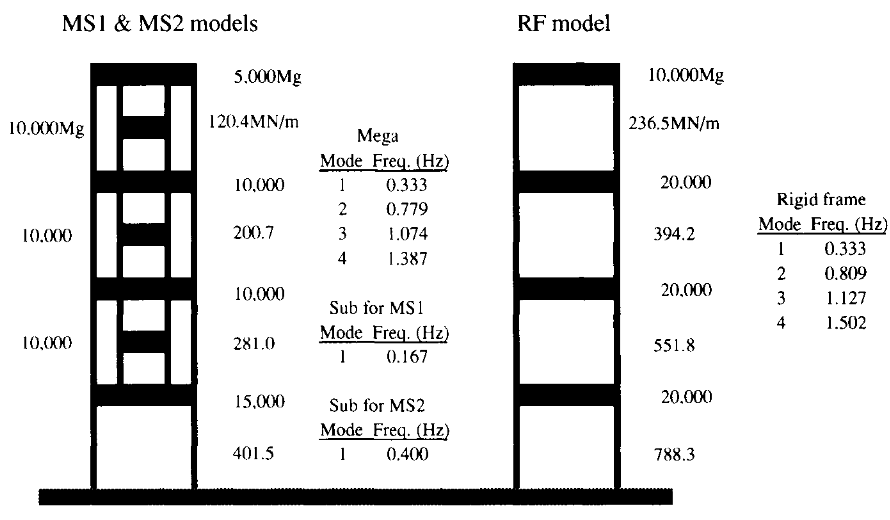

Figure 5. Analysis models for a 200m-tall building.

Response to Near-Field Motions

The maximum values of the base shear coefficients, the relative displacement of the top mega-mass, the absolute acceleration of the top mega-mass, the acceleration of the top sub-mass and the stroke between a mega-mass and a sub-mass subject to the Kobe 1995 NS, the Sylmar 1994 NS and the El Centro 1940 NS are listed in Table 3. The stroke listed in the table represents the largest value among the peak relative displacement responses between a mega-mass and a sub-mass at six connection points. The maximum stroke value is not confined at the top connection point because of the effects of higher vibration modes.

The values listed in Table 3 indicate the excellent performance of the mega-sub control systems employed for the MS1 and MS2. Reduction in the base shear coefficients is especially significant. The time histories of the base shear coefficients are plotted in Figure 7. As is observed generally in a heavily-damped structure, the effectiveness of the mega-sub control system becomes large in the latter phase of response. In the initial transient phase, however, the effectiveness is limited. Although the MS2 controls only modes higher than the second mode, there is no significant difference in the maximum response values in the MS1 and MS2. Only the acceleration response of the sub-mass of the MS2 becomes larger than the corresponding value of the MS1 but is still much smaller than that of the mega-mass. This is due to the frequency content of the near-field motions observed in the response spectra shown in Figure 4. All three ground motion records have large energy concentration in the range of 0.5Hz - 10Hz. At the first natural frequency of the mega-structure, 0.333Hz, the power of the ground motion is very small. To achieve the same performance, the stroke required between the mega- and sub-structure in the MS2 can be smaller than in the MS1. In an actual sub-structure system consisting of several floors, the drift demands for the MS2 would be smaller than those for the MS1. In addition, design and construction of such a stiff sub-structure is easier. Therefore, the MS2 is more feasible than the MS1 when considering near-field ground motions. For far-field ground motions, however, the contribution of the first mode becomes dominant. Therefore, if the design load for far-field ground motions is critical, the MS1 system should be preferred to the MS2.

Table 3. Maximum response of buildings of three types subject to near-field ground motions.

Accelerogram	Model	Base shear (%)	Top Disp. (cm)	Top Acc. (cm/s/s)	Sub Acc. (cm/s/s)	Stroke (cm)
Kobe 1995 NS	RF	27	72	1,105	-	-
	MS1	10	47	735	89	46
	MS2	10	50	672	175	31
Sylmar 1994 NS	RF	37	126	1,103	-	-
	MS1	14	78	770	134	72
	MS2	14	63	452	252	42
El Centro 1940 NS	RF	16	55	432	-	-
	MS1	7	20	243	34	18
	MS2	4	19	205	76	12

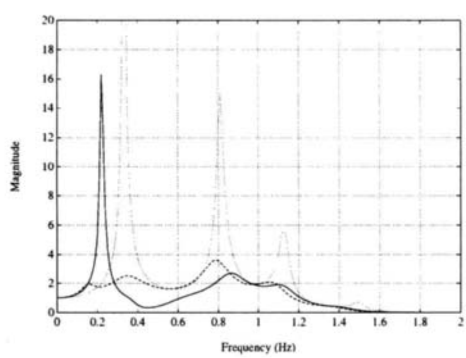

Figure 6. Acceleration transfer functions for MS2 (solid line), MS1(broken line), RF (dotted line).

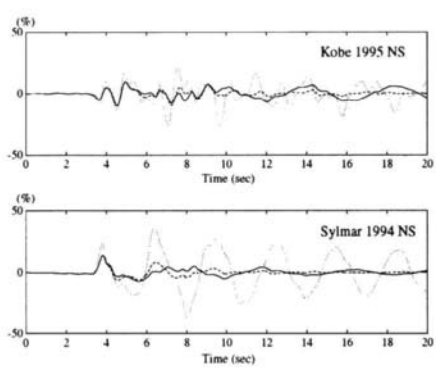

Figure 7. Time histories of base shear coefficients for MS2(solid line), MS1(broken line), RF(dotted line).

CONCLUDING REMARKS

The mega-sub control system examined here is a passive vibration control system which takes the advantage of the structural configuration of a mega-structure. In the system, the sub-structures contained in the mega-structure are tuned to suppress vibration energies induced by earthquake or wind load. The performance of the mega-sub control system was evaluated against near-field ground motions obtained from the January 17, 1994 Northridge Earthquake and the January 17, 1995 South Hyogo Earthquake for a 200m-tall steel building. The simulation results were compared with those for the standard El Centro 1940 NS ground motion. The simulation results exhibited the feasibility and effectiveness of the mega-sub control system against near-field ground motions which had major frequency content of 0.5Hz - 10Hz. The mega-sub control system which controls vibration modes excluding the first mode seems superior than the system which controls all the modes because of its smaller strokes and the feasibility of structural design for the portion of sub-structures. The amount of dampers required for the sub-structures is feasible and in the same order with that of dampers normally augmented in a base-isolated building. The simplicity and effectiveness of the system make this control strategy extremely attractive for its implementation in tall and super tall buildings against seismic load.

ACKNOWLEDGMENT

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STRENGTHENING OF OLD MASONRY HOUSES FOR SEISMIC LOADS

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ABSTRACT

Technical criteria for strengthening of old, traditionally built stone- and brick-masonry houses for seismic loads are discussed. Mechanical characteristics of existing masonry materials have been determined and efficiency of technical measures for stregthening the existing masonry walls have been tested in the laboratory and in-situ. By subjecting a series of models of simple two-storey masonry houses with identical structural configuration to simulated seismic ground motion, the influence of different types of floors and tying of walls with steel ties on seismic behaviour has been investigated. It has been found that existing urban and rural masonry houses can be strengthened so as to meet the up-to-date requirements of seismic resistant construction.

KEYWORDS

Existing buildings, stone-masonry, brick-masonry, strengthening, grouting, jacketing, steel ties.

INTRODUCTION

As has been learned from all recent earthquakes, modern buildings, designed according to requirements of state-of-the-art earthquake engineering, successfully resisted strong ground motion, whereas many old, non-engineered as well as engineered buildings suffered severe damage or collapsed. Specifically, the Great Hanshin Earthquake of 1995 pointed out the severity of problem of reduction of seismic vulnerability of existing buildings, "old" by the standards of earthquake-resistant design, and not merely by the age of construction.

The number of seismically vulnerable existing buildings in earthquake-prone countries is still large. Not only in Asia and Latin America, but also in Central Europe, Mediterranean region, as well as in the U.S.A., old masonry buildings of all types of construction, among them historical monuments, represent an important part of existing building stock. Since old masonry buildings are typical representatives of traditional non-engineered construction, their seismic vulnerability is, generally speaking, relatively high. Indeed, most earthquake damage and loss of life in those regions has taken place because of inadequate seismic behaviour of existing masonry buildings, in most cases residential houses of traditional type of construction (Fig. 1). On the other hand, there were cases where adequately constructed masonry buildings survived even the most devastating earthquakes (Fig. 2).

Fig. 1. Killari, India, 1993: completely destroyed village.

Fig. 2. Tangshan, China, 1976: survived building.

In order to avoid loss of life and excessive damage to buildings during future earthquakes, adequate measures to reduce the seismic vulnerability of old masonry buildings should be provided in time. This indicates the need for the development and implementation of adequate technical solutions that would reduce the seismic risk of this category of buildings after renewal and rehabilitation.

In this contribution, experience regarding the strengthening of existing masonry buildings for seismic loads, obtained at the National Building and Civil Engineering Institute (ZAG) in Ljubljana, Slovenia, will be briefly presented.

GENERAL CRITERIA

Despite different structural materials and type of construction of existing masonry buildings, their earthquake-damage can be classified in a uniform way. Generally, damage is a result of insufficient load-bearing capacity of the walls (Fig. 3) and/or inadequate connection between the walls (Fig. 4). An analysis of damage patterns can clearly define the weak and the good points of different structural systems. On the basis of the analysis of damage, the mechanisms of the behaviour of the walls and buildings can be defined. However, additional investigations to establish the structural characteristics and material properties need to be carried out in order to assess the seismic resistance of buildings and answer the question, why some buildings survided the earthquake whereas the others, although standing nearby, did not.

The investigations in seismic behaviour of existing masonry buildings have shown that their seismic resistance depends:

- On the quality of masonry.
- On the connection between walls and floors.
- On the structural layout, i.e. on distribution of structural walls in plan and elevation of the building.

Fig. 3. Cracks in the walls.

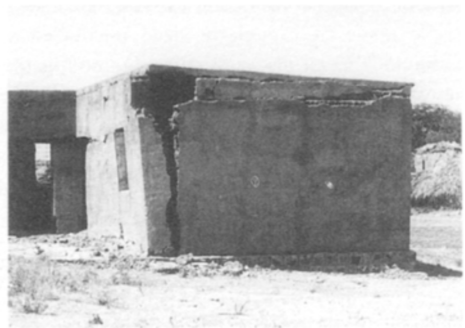

Fig. 4. Separation of walls.

Consequently, technical measures for repair and strengthening of masonry buildings can be classified into:

- Technical measures for strengthening of masonry walls. The structural walls should be uniformly distributed in both orthogonal directions of the building. They should be sufficient in number and strength to resist the expected seismic loads.
- Technical measures for tying the walls, anchoring and stiffening of floors. The walls should be adequately tied and connected. Rigid floor diaphragm action should be provided, and the floors should be well anchored into the walls to prevent out-of-plane vibration of the walls.
- Besides, the foundation system must be capable to transfer the increased ultimate loads from the strengthened upper structure into the soil.

The following criteria should be considered when implementing these measures to buildings:

- They must provide the required degree of seismic resistance of buildings. In this regard, experimental verification of effectiveness of the proposed technical solutions is necessary.
- The chosen technical solutions should be simple and economical to carry out.
- In the case of historical monuments, the proposed methods must also fulfil the requirements of restoration and conservation of cultural monuments.

METHODS OF STRENGTHENING OF MASONRY WALLS

On the basis of the analysis of damage and subsequent experimental investigations, different technical measures for repair and strengthening of masonry walls and buildings have been already developed. However, in order that the application of a specific measure will successfully improve the seismic behaviour of the building under consideration, the basic characteristics of existing structural materials and potential deficiencies of the building under seismic loads should be assessed. On the basis of this study, either suitable existing method of strengthening can be chosen, or new technical solution can be developed. However, the efficiency of any new or improved technique should be experimentally verified before the application on the building.

Mechanical Properties of Existing Masonry Materials

As regards masonry materials and structural typology, a great variety of masonry walls exists in the region. In Slovenia, stone-masonry, sometimes replaced by mixed stone-and-brick-masonry, has been used as the main construction material for centuries. The buildings, which are sometimes four to five storeys high (Fig. 5), are generally made of irregularly sized uncoursed stone (two-layered, rubble stone masonry with an infill consisting of smaller pieces of stone, bound together with lime mortar). Cut stone, or partly cut stone is used in the case of public and monumental buildings. Locally available material, such as limestone and slate, can be found. As most of the old urban nuclei are located in seismic zones, the load-bearing layers of stone-masonry walls are sometimes connected at uniform intervals with connecting stones, or regularly cut stones are used for better connections of walls at the corners and wall intersections. Also, the use of iron ties, connecting the walls horizontally at floor levels, is evident (Fig. 6).

Fig. 5. Typical historical buildings in the city of Ljubljana.

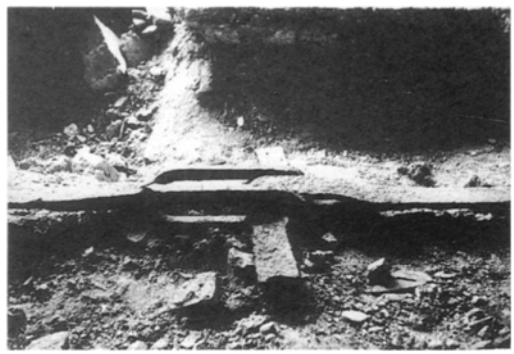

Fig. 6. Old iron wall tie in an historical building.

In Slovenia, brick has replaced stone in the XIXth century. Old, Austrian format brick (30x15x6.5 cm) and normal format brick (25x12x6.5 cm), laid in mortar of poor to good quality has been used for the construction of buildings before and after World War I, respectively. Perforated bricks and hollow concrete blocks have been used in the fifties and sixties.

Since masonry is non-elastic, non-homogeneous and unisotropic material, it is not possible to determine mechanical properties of walls of historic stone-masonry buildings by testing their constituent materials in the laboratory. Therefore, mechanical properties of existing masonry are determined by testing the walls either in the laboratory (Fig. 7) or in-situ (Fig. 8). It is, however, difficult to reliably reproduce the existing masonry walls in the laboratory, eventhough very thorough chemical and mechanical tests of the mortar and stone and/or brick may have been carried out. The only really reliable method of determining the load-carrying capacity of existing old masonry walls involves the carrying out of tests in situ, or cutting out specimens from these walls and testing them in the laboratory. Sometimes, flat-jacks are used in-situ, either to determine the compressive strength of the masonry or simply to estimate the stress state in the wall.

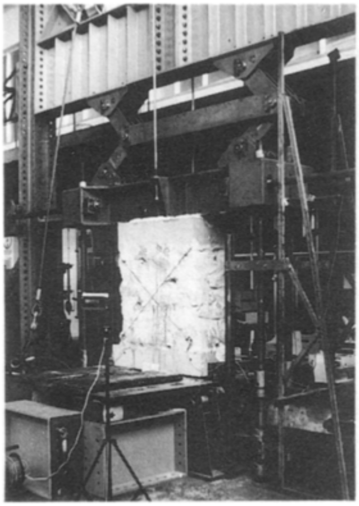

Fig. 7. Laboratory test of stone-masonry wall.

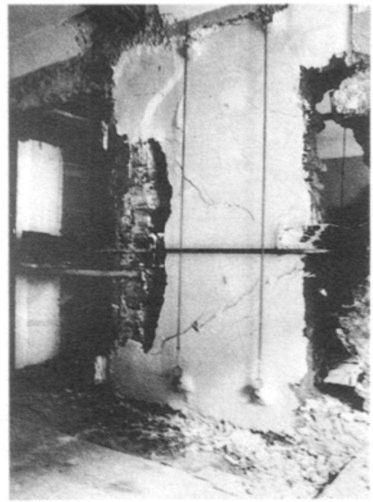

Fig. 8. In-situ test of stone-masonry wall.

Some indicative data regarding mechanical properties of existing masonry materials in this region are given below (Turnšek et al., 1978; Tomaževič and Sheppard, 1982; Sheppard, 1985; Sheppard and Terčelj, 1985; Sheppard and Tomaževič, 1986; Magenes, 1992; Bettio et al., 1993).

	Stone-masonry	Brick-masonry
Compressive strength f_c:	0.3 - 0.9 MPa	1.5 - 10.0 MPa
Tensile strength f_t:	0.02 - 0.19 MPa	0.10 - 0.70 MPa
Modulus of elasticity E:	200 - 1000 MPa	1500 - 3800 MPa
Shear modulus G:	70 - 90 MPa	60 - 165 MPa

Methods of Strengthening

Different methods can be used for stregthening of different types of masonry walls. The choice of the most suitable technical solution depends not only on the required degree of improving the resistance of the wall but also on the type and quality of masonry.

Stone masonry. Because of the method of construction of stone and mixed stone-and-brick masonry walls, specifically uncoursed rubble stone-masonry walls, many voids exist in the walls, uniformly distributed over their entire volume. The systematic filling these voids with injected cementitious grout represents an obvious and efficient method of strengthening. The simple idea that, after hardening, the injected grout will bond the loose parts of the wall together into a solid structure, is followed.

Originally, the grout mix consisting of 90 % of Portland cement PC-35 and 10 % of pozzolana, which is added to the mix in order to ensure the plasticity during the grouting procedure, has been proposed. Water is added to the mix (volumetric ratio between the dry part part of the mix and water varies from 1:1 to 1:0.9), and the mix is injected into the wall through injection tubes (nozzles), built into the joints between the stones over the entire surface of walls at uniform intervals (at 0.5 m to 1.0 m distance, depending on the structure of the wall). Holes are drilled between the stones deep into the walll (at least half of the thickness of the wall deep), and metallic or plastic injection tubes (nozzles) are put into the holes several centimetres deep and fixed by means of fast binding mortar. The walls are first moistened with mortar and then grouted. The procedure of grouting begins from the bottom of the wall and proceeds to the top. After the penetration of the grout at neighbouring tube, the nozzle is corked, the injection tube is fixed to the next (dry) nozzle and the grouting is continued. During the grouting, the posible flow of the grout out of the cracks and joints between the stones is prevented by application of dry fast binding cement. Because of the voids in the rubble stone-masonry walls, low pressure is needed to inject the grout (up to 2 bar). In lack of grouting equipment, the effect of gravity can be sometimes helpful: the grout is poured into the injection nozzle from the container placed in one of the upper storeys of the building. As the experiences show, the quantity of the dry part of the grout necessary to systematically fill the voids in stone-masonry walls does not exceed 50-150 kg per m^3 of the wall.

Typical experimental diagrams, which show the relationship between the average shear stress in the wall and storey rotation angle before and after cement grouting of typical mixed stone-and-brick-masonry walls, are shown in Fig. 9.

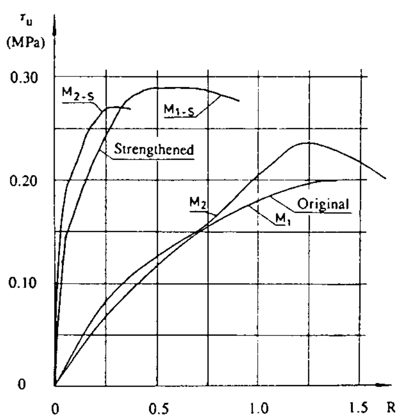

Fig. 9. Effect of cement-grouting of mixed stone-and-brick-masonry wall. (Sheppard and Tomaževič, 1986.)

A number of laboratory and in-situ seismic resistance tests have been already carried out to investigate the effect of grouting (Turnšek *et al.*, 1978; Tomaževič and Sheppard, 1982; Sheppard, 1985; Sheppard and Terčelj, 1985; Sheppard and Tomaževič, 1986; Bettio *et al.*, 1993). Recently, effectiveness of cement-based grouts for vertical and diagonal compression has been also studied by Vintzileou and Tassios (1995).

Systematic laboratory investigations have been also carried to study mechanical and technological properties of injection grout mixes in order to improve the injectability of the grout to be used for stone- and brick-masonry walls (Binda *et al.*, 1990; Binda and Baronio, 1992; Paillere *et al.*, 1992; Atkinson and Schuller, 1992).

It has been found that the level of improvement depends on the quality of original masonry. Similar values of tensile strength of cement-grouted stone- and mixed stone-and-brick-masonry have been obtained in most cases, regardless to the type and quality of original masonry. Some indicative values of tensile strength, obtained by testing, are given below.

	Original	Cement-grouted
Rubble stone, two layers, muddy sand	0.02 MPa	0.12 MPa
Rubble stone, two layers, clean sand	0.10 MPa	0.25 MPa
Rubble stone, mixed, clean sand	0.14 MPa	0.19 MPa

Injecting the grout into the stone-masonry walls has one major advantage: the intervention is invisible and is therefore ideal for structural strengthening of historic monuments. Unfortunately, the use of cement has serious deficiencies. By cement, foreign materials that change the original texture of the walls are introduced. Impurities in cement dissolved in the water may damage frescoes and other decorations, which can be frequently found on the surface of historic stone-masonry walls. Last, but not least, dampness may appear after injecting the cement grout into the walls.

These inconveniences are mainly due to capillary activity of the hardened cement grout. Therefore, the capillary active (hydrophilic) cement grout fabric should be replaced by a water repellent (hydrophobic) one. In order to achieve this, water repellent additives are usually added into the mix, and part of cement is replaced by inert aggregates in the form of fine sands (silica or lime-stone sands - Tomaževič and Apih, 1993). This, however, significantly reduces the strength of the original cement mix.

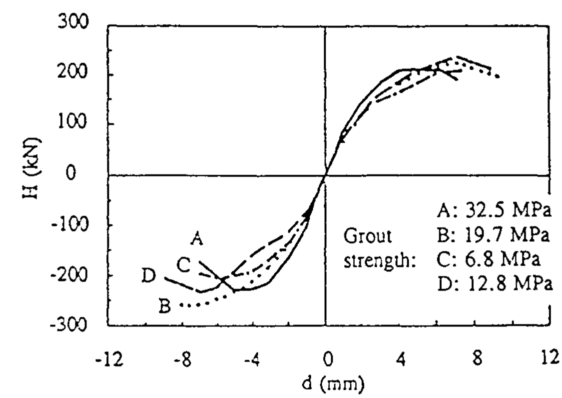

Fig. 10. Comparison of hysteresis envelopes obtained by lateral resistance tests of grouted stone-masonry walls. (Tomaževič and Apih, 1993.)

The influence of the strength of the grout on the lateral resistance of walls has been experimentally investigated (Tomaževič *et al.*, 1991). Although the compressive strength of different grout mixes varied from 7 to 32 MPa, the variation had no influence on the lateral resistance of the grouted walls (Fig. 10). The average value of the masonry's tensile strength of a set of 8 specimens was 0.33 MPa, with standard deviation 0.07 MPa and coefficient of variation equal to 21 %, what represents the usual variation of values of mechanical properties of the tested type of masonry walls.

One of the possible reasons why the strength of the grout did not influence the lateral resistance of the grouted rubble stone-masonry walls can be explained as follows. The potential vertical and lateral resistance of the wall is determined by the strength of original mortar, which transfers the external loads acting on the wall, from stone to stone. As was confirmed by visual inspection of the crushed walls after the tests, the grout did not penetrate into the original mortar. Consequently, the strength of the original mortar was not improved and, hence, the potential resistance of the wall did not change, regardless to the strength of the grout. What is changed is the lateral connection between the stones. By preventing the separation, splitting and buckling of the load-bearing layers and parts of the wall, the grout makes possible the activation of the full load-bearing capacity of the original masonry. The analogy with a masonry building, where the tying of the walls with steel ties prevents the separation and activates the potential seismic resistance of the whole building, is obvious.

Hence, it can be concluded that appropriate composition of the grout mix can be designed for each particular type of masonry and for each particular problem to be solved in a historic stone-masonry building. Locally available materials compatible with the original texture of historic walls can be used as a replacement of part of cement in the grout in order to reduce the expected side effects to an acceptable level.

Brick masonry. Whereas efficient interventions in stone-masonry are more or less limited to injecting the grout into the void parts of the walls, different possibilities are available for the repair and stregthening of brick- and block-masonry walls. The procedures can be classified into the following main groups:

- Sealing the cracked parts of the earthquake-damaged walls by injecting the cracks with cement or epoxy grout.
- Application of reinforced cement or epoxy coatings on one or both sides of the walls. The effects of usual reinforcing steel and cement mortar/concrete coating, ferro-cement, and, most recently, carbon fibre coating have been experimentally studied.
- Prestressing the walls in horizontal or vertical direction.

Obviously, by sealing the cracks with cement or epoxy grout, the original load-bearing capacity of the walls is recovered (sometimes, in the case of the masonry of poor quality, even improved), but the rigidity in most cases is not. Therefore, the grouting the cracks in the brick- or block-masonry walls represents a typical method of structural repair.

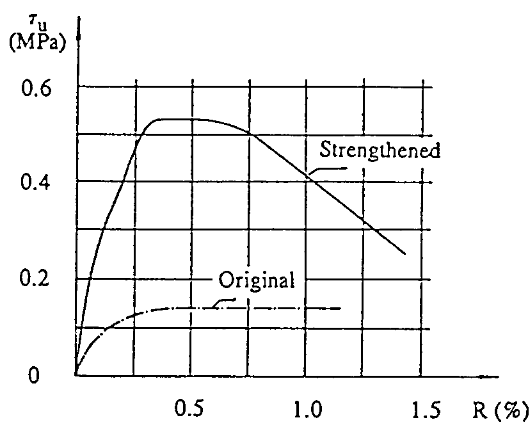

Fig. 11. The effect of strengthening a brick-masonry wall with reinforced-cement coating. (Sheppard and Tomaževič, 1986.)

The application of reinforced-cement coating (jacket) on one or both sides of the wall is the most widely used method for strengthening the existing brick- or block-masonry walls. Plaster is first removed from the wall.

Mortar is removed from the joints between the bricks or blocks, 10 - 15 mm deep, and the cracks in the wall are grouted. The wall surface is cleaned, moistered with water and spattered with cement mix. The first layer of cement coating (20 - 30 MPa compressive strength), 10 - 15 mm thick is applied, the reinforcing mesh is placed on both sides of the wall (4 - 6 mm diameter bars at 100 - 150 mm intervals in vertical and horizontal direction), connected together with steel anchors (6 mm bars, placed in the pre-drilled holes, 4 - 6 pieces per m^2 of the wall's surface). After the reinforcing mesh is connected to the anchors, the second layer of cement coating is applied, the total thickness of coating being 30 mm. Other systems of application of cement coating can be also used (e.g. shot-creting), or the coating is simply applied by concreting. In the latter case, however, the thickness of coating is increased to 80 - 100 mm.

Figure 11 shows the results of a typical lateral resistance test of original and strengthened wall (Sheppard and Tomaževič, 1986). The tested wall was originally 280 mm thick and was made of bricks (compressive strength 10 MPa), laid in lime mortar (compressive strength 0.3 MPa).

TYING OF WALLS WITH STEEL TIES

Where timber joists are not anchored and the walls are not tied, vertical cracks develop along the joints between walls, and transverse walls collapse due to out-of-plane forces. In order to ensure integrity of masonry structure during earthquakes, wooden floors are often replaced by r.c. slabs, anchored to supporting walls, or the walls are tied with steel ties and wooden floors anchored to the walls and/or braced with diagonal ties. In the usual seismic retrofit and rehabilitation practice, the walls are tied at each floor level with steel tie-bars placed on either side of walls and anchored at wall ends on steel plates.

Usually, the steel ties are placed below the floors in horizontal channels, which are cut in the plaster to the wall surface. It is not necessary to cut walls for placing the ties. If long, the bars are kept in their position by means of stirrups, placed through predrilled holes in the wall. Tie-bars are anchored at the ends of walls withs screws and nuts on steel anchor plates, 10-15 mm thick). 16 mm diameter, mild steel reinforcing bars (yield stress 240 MPa) are used in the case of old masonry buildings of normal dimensions (i.e. residential houses, 2-3 storeys high). After placing and fixing the nuts, the nuts are welded on the anchor plate. All steel parts are protected against corrosion by paint and then plastered, so that the ties are not visible.

The effects of different types of interventions in the floors of historical stone-masonry urban houses have already been subject of experimental studies, which made possible successful application of some technical solutions in practice (Boštjančič *et al.*, 1976, Benedetti and Castoldi, 1982). Recently, a coordinated research project has been carried out in U.S.A. and Italy (Ballio *et al.*, 1993).

In order to correlate the effects of different interventions in floor structures on seismic resistance, as well as to quantify the mechanism of the behaviour, two groups of four models of old stone- and brick-masonry urban houses, respectively, have been recently tested on a shaking table in Ljubljana. The models, built at 1:4 scale, represented simple two-storey urban houses with different types of floors, with tied and not tied walls have been tested (Tomaževič *et al.*, 1993, 1995). Four models have been built in stone- and four in brick-masonry, with similar structural configuration in each series. They have been tested by subjecting them to similar seismic excitation with increased intensity of motion in each successive test run. As regards the structural layout, models A_S and A_B (index "$_S$" denoting stone and "$_B$" brick-masonry) were referential models representing houses with wooden floors with freely supported timber joists. Models B_S and B_B had rigid r.c. slabs instead of wooden floors. The floors of models C_S and C_B were exactly the same as the floors of models A_S and A_B. However, the walls have been tied with steel ties, placed at both sides of the walls at each floor level, anchored by steel plates at the corners and prestressed. Models D_S and D_B were similar to models C_S and C_B. However, brick vault replaced the wooden floor in the case of stone-masonry building model D_S, and diagonal ties have been added to peripheral ties in the case of brick-masonry building model D_B. In order to observe the difference, steel ties have not been prestressed in the case of brick-masonry building model D_B.

The following main observations could be made when correlating the seismic behaviour of the tested models:

- Freely supported wooden floors of referential models without ties did not prevent separation of the walls. Consequently, the upper storey of the models disintegrated and partially collapsed before the models' final collapse. Severe out-of-plane vibration of transverse walls has been observed before the disintegration of the upper storey.

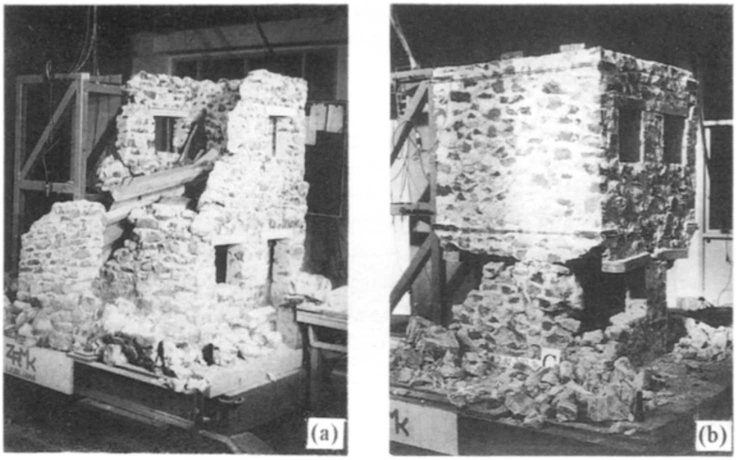

Fig. 12. Mechanism of collapse of stone-masonry model without (a) and with steel ties at floor levels (b).

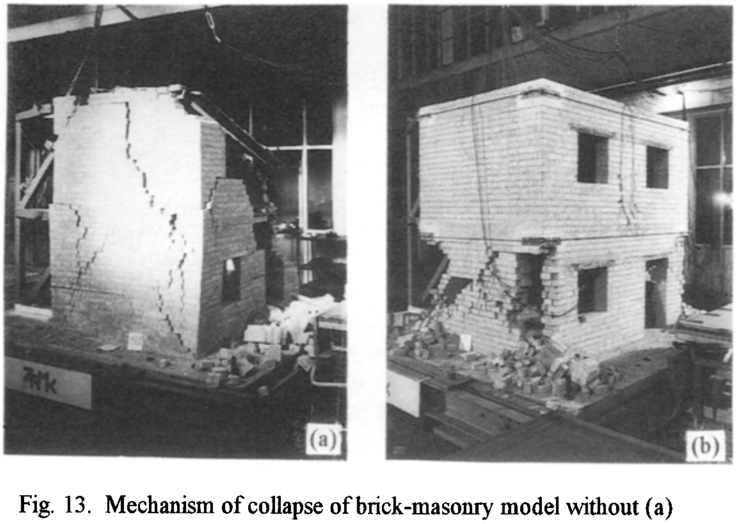

Fig. 13. Mechanism of collapse of brick-masonry model without (a) and with steel ties at floor levels (b).

- Horizontal steel ties prevented separation and disintegration of the walls. Models collapsed because of the shear failure of load-bearing walls in the first storey. However, although parts of the walls started falling off the models, the well connected wooden floors retained integrity until they fell down on the ground. No significant difference in ultimate behaviour of models with prestressed or simply placed ties has been observed.
- In the case where r.c. slabs replaced the wooden floors, models collapsed because severe damage occurred to the walls of the first storey. Walls of the second storey, confined with r.c. slabs at the bottom and top, vibrated as a rigid box placed on the top of the first storey. Sliding and rocking of the upper box caused pushing out and out-of-plane vibration of transverse walls. Despite falling out of corners and overturning of transverse walls in the first storey, the undamaged upper storey did not collapse.

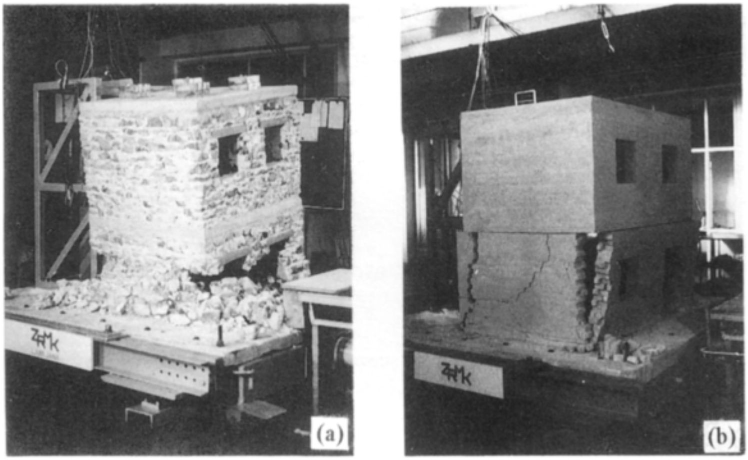

Fig. 14. Mechanism of collapse of models with r.c. slabs.
(a) Stone-masonry model. (b) Brick-masonry model.

In order to correlate the observed behaviour quantitatively, the relationship between the base shear coefficient BSC and first storey drift angle R has been evaluated on the basis of effective acceleration and displacement response of models during individual testing phases. The results are presented in Fig. 15 separately for each group of tested models.

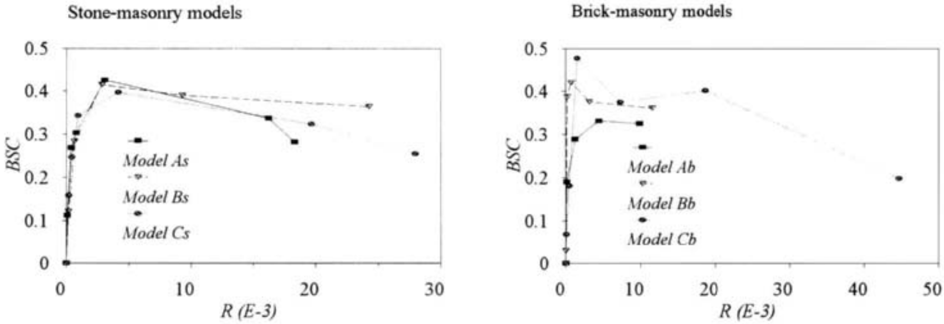

Fig. 15. Base shear coefficient BSC - first storey drift angle R hysteresis envelopes.

In Fig. 16, damage to both groups of models, expressed in terms of damage index I_d with regard to collapse is correlated with cumulative input energy of the shaking-table motion, defined as a sum of cumulative work done by the shaking-table in order to move the models, idealized as rigid bodies fixed to the platform, from the beginning of test to the end of the test run under consideration. This way, besides lateral resistance and displacement characteristics of the models, the intensity and duration of the shaking-table motion is also taken into account. The following definition of damage index I_d has been used: first damage to the walls: $I_d = 0.25$, shear cracks in the walls, crushing of corners: $I_d = 0.50$, severe cracks, falling out of corners: $I_d = 0.75$, collapse: $I_d = 1.00$.

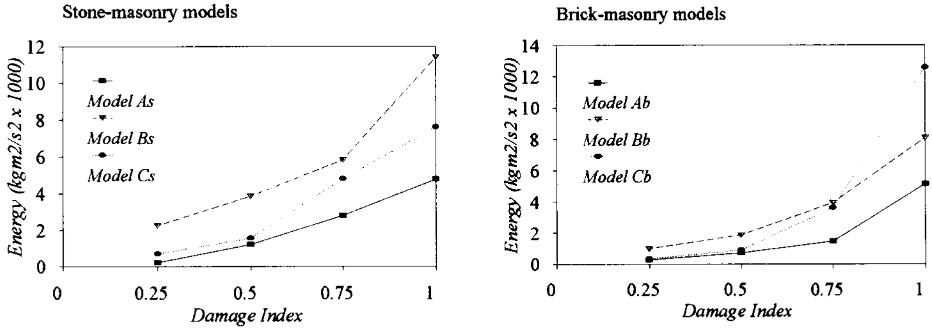

Fig. 16. Damage to models in dependence on cumulative input energy.

As can be seen in Figs. 15 and 16, the difference in seismic resistance between the models with rigid floors and/or tied walls and models without the ties is obvious. Significant difference can be seen as regards the deformability: models with rigid slabs and/or tied walls exhibited much larger deformations before collapse than models with freely supported wooden floors without ties. Quantitatively, the difference in the behaviour is especially evident if the observed degree of damage occurred to the models is compared with the input energy. More than 2-times greater amount of input energy was needed to cause damage and collapse of models with good connection of structural walls than in the case of referential models without ties.

The experiments have shown that seismic behaviour of masonry houses strongly depends on the in-plane rigidity of floors and connection of floors to walls. Although the actual mechanism of seismic behaviour in the case of models with r.c. slabs was different than in the case of models with tied walls, no significant difference in the values of parameters of seismic resistance has been observed when good connection of the walls has been provided by either the r.c. slabs or tying the walls with steel ties, in the particular case studied.

The analysis of changes in strain in the wall ties has indicated that, at ultimate state, the tension forces measured in the ties in the direction of seismic motion were of the same order of magnitude as the storey shear developed at respective floor level due to inertia forces. In transverse direction, however, steel ties acted as if they were reinforcing steel placed in imaginary tie-beams, i.e. band of masonry wall between the ties at floor levels. When subjected to out-of-plane vibration and bending of walls between stiffening walls, the tension in the bars increased in the case when they were on the tensioned side of the wall.

In order to ensure adequate tying of walls, the ties should resist the forces which may develop in the bars during earthquakes. Since the tension in the ties due to ultimate shear forces is generally greater than tension due to bending, it is suggested that minimum dimensions and number of ties is calculated on the basis of the ultimate resistance of the so-called typical segment of the building under consideration. Typical segment is part of a building, composed of structural walls, which limit the typical floor area or part of a building, defining - in a way - its seismic resistance (Fig. 17). Basically, coefficient of seismic resistance of such segment (ratio between lateral resistance and weight of the segment) should be the same as in the case of the whole structure. Therefore, such segment can be also called "critical segment".

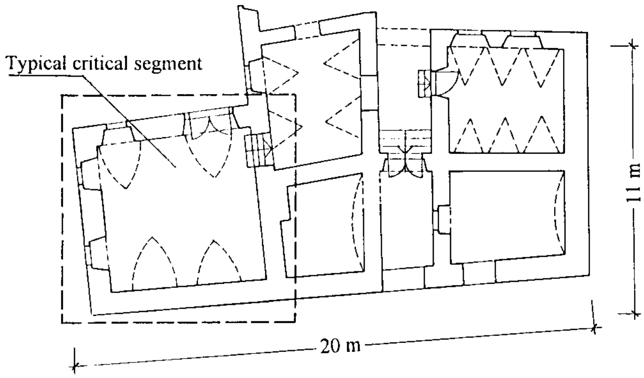

Fig. 17. Definition of critical segment of typical urban building.

Following the above considerations, the minimum diameter of a single tie-bar can be determined by:

$$D_{min} = \sqrt{(H_{u,seg} / n)(4 / \pi)(1 / f_y)}, \qquad (1)$$

where $H_{u,seg}$ - ultimate seismic resistance of critical segment of a storey, n - number of bars of steel ties, f_y - yield stress of reinforcing steel.

CONCLUSIONS

Criteria and possibilities for strengthening of old masonry houses for seismic loads have been discussed. Since the quality of existing masonry materials and connection between the walls and floors represent decisive parameters of seismic resistance of this type of construction, efforts should be oriented towards finding efficient and economical, easy to apply, technical solutions to strengthen the walls and to achieve rigid horizontal diaphragm action of floors and good connection between walls and floors.

Technological solutions for strengthening existing, traditionally constructed masonry walls depend on the type and quality of masonry materials. Injecting of cementitious grouts into the voids can be efficiently applied in the case of stone-masonry walls. By grouting the walls, compressive and tensile (shear) strength of stone-masonry is significantly improved. Since the degree of improvement does not significantly depend on the strength of the grout, grout mixes can be almost arbitrarily designed by replacing cement with additives and inert materials in order to meet different criteria related with conservation and restoration of historical monuments, if necessary. As has been shown by experiments, brick masonry walls can be efficiently strengthened by applying reinforced cement plaster layers on both sides of the wall.

In order to provide good connection of walls of buildings with wooden floors, wooden floors are many times replaced with rigid r.c. slabs. As has been shown in a series of model shaking-table tests, the replacement of wooden floors with rigid slabs, what represents a relatively costly and time-consuming procedure, and also a structural intervention, which is not compatible with traditional construction methods, is not always necessary. Any intervention in the floor structural system that results into a rigid horizontal diaphragm action of existing floor structure, is recommended. By preventing separation of walls with simple tying the walls with steel ties, similar degree of improvement in seismic resistance has been obtained as in the case of replacement of wooden floors with r.c. slabs.

The size and quantity of the ties can be determined by calculation. However, the experiments have shown, that the use of 20 - 22 mm diameter, mild steel reinforcing bars (yield stress 240 MPa) will be adequate in most cases. As indicated by experiments, the tension capacity of 16 mm diameter bars of the same mild reinforcing steel, which are most frequently used for tying the walls in the current renewal practice, is hardly sufficient to carry the expected ultimate forces even in the case of normal size masonry buildings.

After intervening in the floor structural system, rocking motion of the building can sometimes prevail during earthquake, what might cause substantial damage to corners. Therefore, when taking care for adequate rigid floor diaphragm action by means of stiffening the existing floor, or replacing the existing wooden floors with r.c. slabs, attention should be paid to corner zones. In order to prevent damage, crushing of masonry units and disintegration of masonry at those zones, strengthening of corners with steel anchors, placed at regular intervals along the height of the buildings, is sometimes necessary.

Almost 20 years ago, after being severely damaged by the Friuli earthquake of May of 1976, a three-storey stone masonry rural house has been strengthened by cement-grouting and tying of walls with steel ties. The building successfully resisted the subsequent September shocks of the same intensity, i.e. degree IX by MSK seismic intensity scale. The good behaviour of the strengthened building proved the correctness of idea of strengthening traditionally built, non-engineered masonry houses. Twenty years later, the methods, originally proposed at that time, have been improved and experimentally verified. As has been shown in this contribution, experimental data have been obtained regarding the mechanical properties of existing and strengthened masonry materials as well as regarding efficiency of strengthening methods. In addition, mathematical procedures have been developed, which make possible the verification of seismic resistance of strengthened masonry buildings by calculation.

In order to reduce the seismic risk of a large number of existing, traditionally built masonry buildings in many parts of the world, ways and means for implementing available knowledge into practice should be found in the nearest future. In many countries, including Slovenia, adequate legislation is yet to be prepared on the basis of balance between economical input needed for improving the resistance and possible losses in the case of expected earthquake. International Decade for Natural Disaster Reduction, a United Nations initiative, is a proper time for an internationally coordinated action. Every earthquake, occurred in the near past, has shown that adequately designed new buildings can resist strong earthquakes. There are ways, however, to seismically upgrade the existing old masonry buildings to meet a similar degree of safety. Some of these possibilities have been discussed in this contribution.

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ANTI-SEISMIC AND DISASTER CONTROL POLICY FOR RECONSTRUCTION OF GENGMA

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ABSTRACT

This paper proceeds from post-earthquake reconstruction to expound the city's plan against earthquakes and subsequent disaster. It is proposed that the city's plan combine with anti-seismic and disaster control measures subject to the city's size, function and property. However, the plan should be independent, professional, and comprehensive. In addition, this paper presents in detail the anti-seismic and control disaster policy of Gengma reconstruction, including use of land, life-line projects, relocation.

KEYWORDS

Post-earthquake; disaster control; reconstruction; city planning; Gengma earthquake.

INTRODUCTION

On November 6th, 1988, a strong earthquake struck Lancang, Gengma town in Yunnan province. It was a "double main shock" earthquake with high magnitude, shallow epicenter and repeatedly strong aftershocks. It was another major disaster in the wake of the Tangshan earthquake, causing destruction and loss. More than 95 percent of the houses in Gengma became dangerous and about 53 percent of them collapsed. Loss of housing came to 870,387 sq. m., and the town's infrastructure such as roads, water supply and communication was also destroyed.

The grave cataclysm Gengma suffered in this shock led to close attention to leadership and impelled people to ponder and record the experience and lessons of the disaster.

IMPLEMENTATION OF DISASTER-PREVENTION TOWN PLAN

The actual earthquake intensity of Gengma was equivalent to what had been designated by the state. But the town was damaged so extensively because it never took any anti-seismic measures. Appeals and warnings, issued by many experts in connection with the region's hidden dangers in construction, were confirmed by the shock. The main factors include: Leaders at different levels lacked anti-seismic consciousness. They had not done any systematic work to prevent shock and disaster in the town and construction projects. As the town was built, there universally existed the phenomena of irrational investment in terms of structure, underestimating infrastructure needs, and neglecting the risk factors in housing. Many new buildings were inferior in quality due to a shortage of technicians, chaos in managing construction, laxity in supervising, and weakness of measures.

Obviously, the serious problem was not confusion in what to do but the long delay in taking action. Had the bitter lessons of macroseismism at Tonghai, Longling in Yunnan been recorded conscientiously, urban anti-seismic plans could have been drawn up after the state issued the earthquake intensity in TOS. The possibility of preventing cataclysmic effects should have been forecast, so that counter-measures could be planned and the effect of a cataclysm reduced to the lowest degree. In fact, Gengma did not do so, and the city was destroyed. Events at Gengma showed potentially that to develop and implement a plan against earthquake and subsequent disaster has great importance.

In restoration, Gengma summarized the lessons and experience, emphasizing preventive measures. It is a small frontier town. By state stipulation, it only needs the third standard to plan countermeasures. But in view of the circumstances, the leaders earnestly analyzed the shock danger after the earthquake and fixed the intensity as 8 degrees.

To determine the small earthquake-influenced divisions, geological study was emphasized in the plan. In a range of 3 sq. km for expanding Gengma, 89 drillholes were constructed. By calculating and analyzing seven factors of stratum thickness, shear wave velocity of stratum, historic cataclysm record, fault distribution, slips, subterranean water level, a comprehensive evaluation was done and division of the new Gengma town on a seismic basis was fulfilled.

COMBINATION OF TOWN PLAN AND DISASTER CONTROL MEASURES

The general plan for the city is the prototype of its development while the plan for earthquake and disaster control relates to the city's nature, size and function. However, its scope and details are affected by the general plan of the city (G.P.C.). Their arrangements may not work simultaneously because of management and system. Some contradictions inevitably arise which reflect the utility of the urban land and reform of the old city. These contradictions seriously affect recovery from the general earthquake and disaster control in the city.

The first challenge of reconstruction in Gengma is to arrange both plans by the same institute under unified leadership. They search for the best plans to rapidly improve on city's earthquake and disaster control ability from their respective sides. The general plan of Gengma emphasizes analyses of the earthquake and geological materials. They try their best to shun the areas in which bad geological conditions exist and an earthquake may struck any time. Goals are to develop the new districts from Gengma street, discard part of the old city in a dangerous slope area, situate important buildings regardless of geological conditions, and build parks and greenbelt in order to meet the relocation needs of the citizens. On the other hand, an earthquake and disaster control plan has its countermeasures. For example, they deplore government building in an area where geological conditions are not the best but other factors may be suitable; while improving the ability of the general earthquake and disaster control may conflict with functional and scenic areas of the general plan, measures to improve structural integrity and divide the structure into systematic units with earthquake control are practical.

NEW ANTI-EARTHQUAKE AND ANTI-NATURAL-DISASTER FEATURES

To restore and reconstruct Gengma, the government spent millions of dollars. The program of restoration and reconstruction has now made great progress. A completely new anti-earthquake city has taken shape.

Capabilities of the new Gengma to resist earthquake and natural disaster are shown in the following items.

Different structures and different number of floors for buildings in different areas are considered and distinguished. This is done according to the degree of earthquake danger and intensity. Also any area not fit for construction is identified. These measures put the anti-earthquake task in an advantageous position. Because requirements and approval for land vary, development and construction should be in agreement with a series of standards.

To protect life-line projects when an earthquake takes place and ensure a quick recovery, the program sets up two ground-to-air exits which connect traffic to Lingchang and Kunming. This is done together with developing new areas, a network of new highways, and more exits for city streets. Electricity takes the form of a circular power supply, and water supply comes from different regions, with preservation of remaining sources of rivers, lakes, and wells. Also there is a supply of water in anti-earthquake shelters. Wire and wireless communication are used in the city. Wire circuits consist of underground cables as much as possible. More exits are set up to connect cables with each other in many detours. Thus the cables radiate into a large network. Medical establishments should set up rescue teams and make first-aid plans. Epidemic prevention establishments must pay attention to plague, distribute medicine in time, and train more medical staff. There are more fire hydrants built along roads, and a voluntary fire brigade has been organized. A fire control plan has been made. Agricultural branch of government oversees the measures for emergency food supplies and medical care.

To extend protection between potential earthquake spots, more prevention was planned. Dangerous containers are fixed. If there is an emergency in a flammable factory, power and production are cut off as a safety measure. The power supply is constantly checked, landslides are controlled, and residents receive fire-fighting training.

All new buildings are constructed to prevent earthquake damage according to degree of danger and use of land. The most advantageous form is chosen, and the wholeness of the structure is protected. All construction meets certain requirements and standards. Important life-line projects must get approval from the authorities so first-degree prevention is implemented.

The project uses parks, greenbelt, land, athletic fields, squares, parking lots, agricultural markets, schools and factory recreation areas for evacuation. Its radius is 0.15-0.8 km, 7 sq. m for one person. There are 28 evacuation and earthquake shelters. Evacuation roads are divided into first, second and third degree. Some citizens have access to shelter by making use of stadium in a good location, and approval from authorities to raise the standards to become an earthquake prevention centre.

CONCLUSION

Although the capability of Gengma to survive earthquakes and natural disasters is not perfect for historical reasons and difficulties in reconstruction, it has improved. As long as all anti-seismic and disaster measures are strictly carried out, the area will be safer when a severe earthquake occurs. There will no longer be collapses of buildings in such large numbers, and so many casualties among people. The city will avoid a state of malfunction, and improve its capability of survival.

EARTHQUAKE LOSS ASSESSMENT OF XIAMEN AND EARTHQUAKE INSURANCE

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ABSTRACT

This paper, based on the analysis of potential earthquake hazard in Xiamen, gives an earthquake loss estimation in contrast with insurance reserve and presents some suggestions of countermeasures for the insurance company in Xiamen.

KEYWORDS

Earthquake loss; insurance; hazard; Xiamen City; **general assets; probable liability.**

ANALYSIS OF POTENTIAL EARTHQUAKE HAZARD IN XIAMEN

Introduction

Xiamen, situated at longitude 118°04' 04" E, by latitude 24°26' 46" N, is located on the central part of Changle-Shaoan fault zone, eastward facing Taiwan, the most active seismic zone in China. Changle-Shaoan fault zone is one of the seismic belts of the highest intensity and frequency in the southeastern coastal area of China. Fault ruptures on Xiamen peninsula and nearby areas have fully developed. In particular, the three ruptures running NNE-NE, NW and approximately E-W have shown strong activity since the post-Pleistocene Epoch. Therefore, Xiamen has an earthquake-prone tectonic background.

Although Xiamen has never been the epicentre of a destructive earthquake, it has been influenced by six earthquakes measuring above 6 on Richter Scale and many times been involved in Taiwan earthquakes, according to historic records provided by Xiamen Earthquake Resistance Office. The year 1604 witnessed Quanzhou overseas exterminatory earthquake of Ms 8 and the intensity in Xiamen reached 8, hitting an all-time high and resulting in serious building damage, casualties and property losses. By various scientific methods, "Fujian and Jiangxi Earthquake Intensity District Report" reveals that the possible maximum earthquake magnitude in this area will be 7--7.25 in the next 100 years. However, Fujian Seismological Bureau holds that the possible maximum magnitude in Xiamen will be 5.75--6.5 with a possible maximum intensity of 7 on the Chinese scale.

A recent study after the Ms 7.3 earthquake in Taiwan Strait on Sept. 16, 1994, based on expert system and probability analysis, indicates that there is a probability of an Ms 5--6 after-shock in the coastal boundary area of Fujian and Guangdong Provinces in 1995 as warning signs of earthquake and seismological abnormality still exist.

Conclusions from Earthquake Hazard Analysis

According to a report provided by Xiamen Seismological Bureau, 50-year probabilities of exceeding intensity VI, VII, VIII are 63.2%, 11% and 2.5% respectively. Annual probabilities of exceeding and annual probability of occurrence can be deduced from the following formulas:

$$P(I > I_m) = 1 - [1 - Pt(I, I_m)]^{1/t} \quad (1)$$

$$P(I_m) = P1(I > I_{m-1}) - P1(I > I_m) \quad (2)$$

Results are shown in the Table 1.

Table 1. Probabilities of Earthquake Occurrence and Intensity

50-Year Probability of Exceeding	Seismic Intensity	Annual Probability of Exceeding	Annual Probability of Occurrence
0.632	VI	0.01979	
0.11	VII	0.00233	0.01746
0.025	VIII	0.00051	0.00182

Basic intensity of a city is defined as the intensity whose probability of exceeding for 50 years is 10%, and the maximum intensity as the intensity whose probability of exceeding for 50 years is 2%. From the above table, we know that the basic intensity and maximum intensity of Xiamen are VII and VIII respectively.

EARTHQUAKE LOSS ESTIMATION

Average Loss Ratio

Direct and indirect losses due to earthquake damage that are insurable include:

-- repair, fortification and reinstatement expenses of buildings due to direct physical damage
-- losses due to physical damage induced by fire or flood following an earthquake
-- economic losses due to business interruption of industrial and commercial enterprises by earthquake damage.

In consideration that the amount insured for consequential losses only accounts for about 2.6% of total amount insured, and losses are hard to assess, we may omit this part in calculation of the liability our company will assume as the consequence of an earthquake.

Direct economic losses from earthquakes generally include the following two parts:

-- economic losses of damaged buildings
-- economic losses of the contents of buildings

The average degree of loss can be indicated by average loss ratio. Loss ratio of property inside the building is generally slightly lower than that of the building itself, but as a simplified model, the two ratios may be regarded as approximately equal.

A distinction should be made between damage and loss. Damage is defined as the physical impact of an earthquake on a facility, while loss is a measure of the amount of money necessary to repair the damage to a facility. The translation of damage to loss depends on a number of factors such as: type of construction, perception of risk, requirement of building code enforcement agency, owner's objective in implementing repairs, and the post-earthquake assessment of the building. Seismic damage of buildings can be classified as "collapsed","destroyed", "seriously damaged", "moderately damaged", "slightly damaged or no damage", etc . For insurance purposes, loss ratio of a building can be defined as the ratio of cost of repairs to reinstatement value of a particular structure or contents. Different loss ratio curves can be found in earthquake or earthquake engineering documents.

Before we apply a proper loss ratio to Xiamen, let's first acquaint ourselves with the general state of buildings in Xiamen. An earthquake damage prediction sample study of general building stock includes all kinds of residential, administrative, commercial, manufacturing, light industrial as well as public buildings. In Xiamen in 1988 an expert system developed by YangYucheng shows that 21.3% of the built -up area fails to meet national aseismic requirements and the remaining 78.7% meets or barely meets these requirements. The following facts will help to explain the above conclusion. Many buildings in old urban districts such as Siming and Kaiyuan were constructed during the 1920s or 1930s and are in poor condition now. Not until

1974 did the first national "Seismic Design Code for Industrial and Civil Buildings" come into being. Buildings constructed before 1982 were not required to be built aseismically. Only buildings constructed after that time were required to be designed according to technical codes in accordance with a seismic defensive intensity of VII. As investment in recent years has been surging, with a boom in high-rise and multi-story building, the percentage of buildings up to aseismic standards should be higher.

Based on data collected in Tangshan earthquake, the average loss ratio curve is higher than that of Xiamen because the buildings at the time of earthquake were not designed or fortified to meet aseismic requirements. The average loss ratio curve recommended by Munich Reinsurance Company, however, would be lower than that of Xiamen, considering that the earthquake engineering standards of developed countries are higher . Therefore, in this paper we'll adopt the PICC curve developed by PICC Insurance Research Institute, a compromise between the above two, as the basis in calculation of Xiamen's maximum probable loss. From the curve, we know the average loss ratios of intensity 7 and 8 are 4% and 14%.

Assessment of Probable Maximum Earthquake Loss

For the purpose of earthquake insurance, a correct estimation of probable maximum loss in a certain period is necessary. The insurance company must maintain an earthquake insurance fund and increase it at an appropriate speed to ensure the ability to cope with the impact of a tremendous earthquake disaster.

The economy in Xiamen has enjoyed sustained and rapid development since its establishment as a special economic zone, especially in the '90s. Compared with 1981, the total GDP of the city in 1993 increased by 7.08 times; the value of primary, secondary and tertiary industries increased by 50.41%, 9.66 times and 8.75 times. Real assets investment in 1991, 1992, 1993 and 1994 were RMB 2.218 billion, 3.316 billion. 6.446 billion and 9.5 billion yuan, respectively. Annual growth rate of general assets has been well above 30% in the past 3 years. Consider that Xiamen is at the initial stage of reform and has the prospect of opening up trade with Taiwan. While its economic base is relatively small, we deem it appropriate to assume an annual growth rate of 30% before the year 2000.

General assets of a city are its total fixed assets plus liquid assets plus residents' property. From Xiamen Bureau of Finance and Xiamen Statistical Bureau, we know that total fixed and liquid assets of domestic and foreign enterprises and institutions were RMB 26.81 billion yuan. As for total residents' property, we can make a conservative estimate. The population of Xiamen is 1.1749 million. If we reckon property per person worth RMB 5,000 yuan, then the total residents' property is RMB 5.59 billion yuan. General assets of Xiamen in 1993 derived from the above figures is about RMB 32.4 billion yuan. Take the general assets in 1993 as the basic point and we can make the following calculation.

Maximum probable loss for Xiamen city in a certain year if an earthquake actually takes place can be calculated by the formula:

$$LM(t) = V_o(1+r)^t \cdot P(DR \mid I_m) \tag{3}$$

where V_o is general assets in 1993, r is annual growth rate of general assets, t is number of years since 1993, $P(DR \mid I_m)$ is average loss ratio.

Results are shown in the Table 2.

Table 2. Maximum Probable Loss (Unit: RMB million yuan)

Intensity	Year 1995	Year 2000
VII	2190	10572
VIII	7665	37002

As far as economic strategy is concerned, calculation of expected maximum probable loss appears more meaningful because it contains the probability of such an event. Assuming the occurrence of an earthquake is geometrically distributed, expected maximum probable loss in a certain year can be calculated by the following formula:

$$LM(t) = V_o(1+r)^t \cdot [1 - P(I_m)]^{t-1} P(I_m) \cdot P(DR \mid I_m) \tag{4}$$

where V_o is general assets in 1993, r is annual growth rate of general assets, $P(I_m)$ is annual probability of earthquake, $P(DR \mid I_m)$ is average loss ratio.

Accumulated expected maximum probable loss by a certain year will be:

$$LE = \sum_{t=1}^{T} LE(t) \tag{5}$$

Results are shown in the Table 3.

Table 3. Expected Maximum Probable Loss (Unit: RMB million yuan)

Intensity	Year 1995	Year 2000
VII	66.990	482.348
VIII	24.659	187.335

CHOICE OF EARTHQUAKE INSURANCE

Earthquake insurance throughout the world can be classified into three types: 1. earthquake included in basic coverage of fire and natural disasters; 2. earthquake as legal insurance; 3. earthquake as an additional coverage of fire insurance.

In view of an earthquake being particularly catastrophic, many insurers (especially earthquake-prone countries)refuse to underwrite this risk. Insurers in some countries exclude the earthquake risk in their basic insurance policy. However, this risk can be written in as additional coverage. In contrast, earthquake insurance in China is basic to most property and personal accident policies. Therefore, insurers have assumed enormous liability.

Take the People's Insurance Company of China, Xiamen Branch, for instance, up to the end of 1994. It has written domestic property insurance in the amount of RMB 10.5 billion yuan and foreign property insurance in the amount of RMB 8.4 billion yuan. The property underwriting proportion is approximately 45%. Based on the current proportion and assuming that it will increase by 5% annually before the year 2000, we can find the maximum probable liability and expected maximum probable liability. Results are shown in Table 4 and 5.

Table 4. Maximum Probable Liability (Unit: RMB million yuan)

Intensity	Year 1995	Year 2000
VII	1089	6692
VIII	3802	234322

As calculated, PICC Xiamen would assume maximum liability of RMB 6.7 billion yuan if an earthquake of intensity 7 actually happened in the year 2000.

Table 5. Expected Maximum Probable Liability (Unit: RMB million yuan)

Intensity	Year 1995	Year 2000
VII	31.653	290.877
VIII	11.651	112.971

This means that by the years 1995 and 2000, PICC Xiamen should have a reserve of RMB31.653 million and 290.877 million, respectively, to match the expected maximum probable liability for an earthquake of intensity 7.

Theoretically, under normal conditions for an insurance company, maximum indemnity capacity for earthquake losses will be the sum of premium surplus plus general reserve plus capital. The said premium surplus means, with a low claims ratio level, the difference between the acceptable indemnity level and actual indemnity level, with no operational loss suffered by an insurance company. Since PICC headquarters is the sole legal entity from all its branches, its maximum indemnity capacity represents that of its branches. There appears to be no problem of solvency for any of its branches.

However, if regarded as an independent business account unit responsible for its own profit and loss, PICC, Xiamen would face problems in case of an earthquake. Since restoration of the insurance business in 1980, the company has never been stricken by natural disaster and the threat of insolvency might not felt by some people. If an earthquake actually happens, the outstanding claims and outstanding liability reserves drawn solely from premiums of the prior year will be insufficient to cover the staggering figure of maximum probable loss. An effective way to arrange for earthquake insurance must be sought.

As a fundamental risk, earthquake is impersonal in origin and widespread in effect, which is essentially different from other risks such as fire. Since earthquake can affect other risks, a city hit by an earthquake, may be regarded as one big risk unit. The underwriting principal and practice should be different from other risks.

If earthquakes continue to be included in basic coverage, a disaster reserve should be established. This reserve should be drawn from premiums instead of from net profit to ensure timely accumulation. In 1994, PICC Xiamen earned total premiums of RMB272.076 million yuan, of which property insurance premiums reached RMB97 million yuan. With ever- increasing competition and a tendency toward market saturation, we estimate the annual growth rate to be at least 20% before the year 2000. Premium accumulation from 1995 to 2000 could be RMB3242.031 million yuan. If 8.97% is drawn from annual preimums, the reserve will barely match the expected maximum probable loss. Statistical material collected by PICC Insurance Research Institute in 1985 showed that earthquake loss accounts for 70 -- 80% of loss from natural disasters. Therefore 12.81% of annual premiums for natural disaster reserve seems minimal. These calculations are based on the assumption that the probability of liability exceeding assets is geometrically distributed over 50 years. If a severe earthquake occurred in the next five year before the reserve was big enough, it would take the insurance company decades to mend its wounds.

Under the financial system before 1994, new increases for the general reserve were drawn from profits after taxes. Since our country has a high tax rate on the insurance industry (55% for PICC), the growth of the reserve is relatively slow. Under this system, an operation loss incurred by the company might cause a deficit in the general reserve.

From the standpoint of the insurance company, the best way to diminish this threat is to separate earthquake from basic coverage. Different premium rates should be set for earthquake insurance in regions of different seismic intensity and for construction with different earthquake engineering standards. Implementation of a national earthquake premium system would be a good way to manage the problem.

Premium income and investment income (if any) for the underwriting of earthquake risk should be tax-free. A disaster reserve based on these sources of income should be encouraged to grow.

Better reinsurance arrangements for earthquake risk are absolutely necessary, especially the reinsurance of

RMB business. A smaller portion of this fundamental risk should be retained than for other risks whose maximum probable loss is more limited. Up till now, all domestic business transacted in RMB yuan has a 100% retention rate within China due to restrictions on RMB currency. This problem will not be solved until the day when RMB becomes a fully convertible and exchangeable currency. Merger of the dual exchange rate and abolition of the official exchange rate in January 1994 raised hopes of placing RMB business in the international reinsurance market. If a large portion of this risk is transferred to other companies or the international insurance market, financial conditions will be safer and more stable.

Finally, earthquake coverage should be given legal status to be specified in "Earthquake Disaster Law". Only in this way can an earthquake reserve be accumulated on an appropriate scale and at an appropriate speed. Its indemnification function would also be strengthened in the event of an earthquake. Meanwhile, this could bring into balance the underwriting of policies for high-risk vs. low-risk exposure.

CONCLUSION

If properly handled, earthquake insurance can help to mitigate the economic impact of a major earthquake without harming the insurance industry.

Statistics presented in this paper are unofficial and its treatment of the subject is incomplete. It merely expresses some personal views and serves as a reference point. As many problems remain unsolved, concerted efforts by government, scientists and experts in different fields are needed to find a rational and effective way to provide adequate earthquake insurance.

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HYBRID CONTROL OF BUILDINGS WITH HYSTERETIC CLADDING CONNECTION ELEMENTS

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ABSTRACT

Hybrid passive-active systems for controlling building seismic response offer the promise of combining the best features of each to achieve superior performance. Analytical investigations of hybrid control systems for buildings designed to combine passive damping provided by cladding-structure interaction with robust active control systems are described. The passive control forces were introduced through hysteretic interaction between heavy architectural cladding and the supporting structure, while the active control forces were assumed to be generated by an active tendon system. Results presented for a case study structure show that the hybrid control system was more effective in reducing interstory drift compared with the cases of either the passive or active systems acting alone. At the same time, the peak force requirements for the active system were reduced compared to the situation without passive damping augmentation.

KEYWORDS

Active control; robust control; mu synthesis; passive damping; passive control; structural control

INTRODUCTION

Designs for high-rise buildings located near seismic zones may include provisions for controlling the inelastic action of the structure to prevent catastrophic damage to the primary structural system. Some additional energy dissipation can be achieved with the use of discrete damping elements, but heavy-weight cladding systems together with "advanced" cladding panel connections (Pinelli *et al.*, 1992; Pinelli *et al.*, 1993; Pinelli *et al.*, 1995;) can also be utilized to provide stiffness and damping. These connections are specifically designed to dissipate energy, thereby reducing the probability of significant damage or even collapse.

The development of "advanced" connections between the precast panels and the supporting members of a structure represents a new approach to cladding design. These connections exhibit good damping characteristics and superior ductility under cyclic loading situations. The contribution from such passive cladding damping may help to prevent failure during moderate or strong earthquakes. However, this passive technique for system control must be tailored for each building structure and its unique structural dynamics, thus the overall performance of passively controlled structural systems becomes dependent on the spectral and temporal characteristics of any given seismic event.

When an active control system is used, the approach is to sense the structural motion during an actual earthquake and then to generate the necessary corrective actions in real time. The use of such feedback control has proven to be effective in rejecting external disturbances for many engineering systems. However, there are many uncertainties in the structural parameters which may affect the reliability of this control system. If an active control system is introduced into a structure which has been fitted with passive devices, uncertain nonlinearities during a seismic event may result in an unstable response. The control system must consider the presence of these passive elements. It must also account for both modeling errors and the non-stationary characteristics of the excitation, as well as for damage that may occur during an earthquake.

The purpose of this study is to optimize the control of a building structure by implementing both the passive cladding damping approach and the active tendon control system. The resulting "hybrid" control system produces reduced building response during a seismic loading event. This design utilizes passive elements to provide response attenuation for low levels of excitation where an active control system might be impractical to activate. For higher excitation levels, the performance demands on either component of the hybrid control system are reduced. In the present study, robust performance is optimized using H_∞ and μ-synthesis methods. Only sensed acceleration is used for feedback. Simulation studies were carried out using a two-dimensional DRAIN-2D (Powell, 1973) model of a quarter scale six-story frame structure previously constructed and tested at the National Center for Earthquake Engineering Research (NCEER) (Reinhorn *et al.*, 1989). Details of the implementation and analysis results are presented below and in (Craig *et al.*, 1993; Goodno *et al.*, 1992; Hsu *et al.*, 1994a; Hsu *et al.*, 1994b).

CLADDING FOR ENERGY DISSIPATION

It is common practice to ignore the structural functions that can be provided by a heavy cladding system in the lateral response design of high-rise buildings. However, results of past research (Pinelli *et al.*, 1993) have shown that the conventional design may not be conservative. The cladding can interact with its supporting structure during lateral response such as that produced by an earthquake. This unexpected interaction can cause damage or failure of the cladding. Alternatively, cladding can be used intentionally to fulfill a limited structural role. However, the stiffness and damping contribution which can be provided by the cladding system is most dependent on the cladding connections.

There are two ways to provide additional lateral resistance in structures using cladding. The use of purely elastic tie-back connections converts the architectural cladding into a stiffening wall, thus increasing the shear stiffness of the overall structure. However, the stiffer the connection, the greater the force that must be transmitted between the floors through the cladding panels. Since the capacity of the cladding panels is limited, too stiff a connection may result in panel failure.

A second approach uses "advanced" connections to attach the cladding system to the structure. Advanced connections have superior ductility and energy dissipative capacities. They are designed to yield before reaching the capacity of the cladding panel, thus limiting the forces transmitted into/through the panel. By using advanced connections, the cladding can provide additional lateral stiffness to the structure for small displacement levels, while developing significant energy dissipation under large interstory motion levels. This passive cladding control system provides an attractive mechanism for dissipating energy and thereby increasing the ability of the structure to survive a strong earthquake.

The complete panel attachment system consists of the connector elements and the associated panel and structural inserts. Previous results (Pinelli *et al.*, 1992) suggest that the inserts themselves are not capable of providing significant energy dissipation without loss of strength and damage to the integrity of panel and/or structure. Therefore, in the present study only the connector elements themselves are expected to provide the levels of ductility and damping required from advanced connections.

The hysteretic behavior of the connector elements determines the performance of the passive cladding system. Results of an experimental test program (Pinelli *et al.*, 1992) provided preliminary information on the design and the cyclic behavior of a particular type of energy dissipative connector (Fig. 1). These flexural connectors consisted of a section of square steel tube, partially cut away to create two narrow flexural elements. The width of the flexural elements was tapered to ensure maximum plastification throughout the material. Results of cyclic testing of these connectors showed that they provide attractive damping and ductility, and may be used as advanced connections. Figure 2 shows the panel attachment system with the tapered flexural connector, panel insert and the supporting anchor into the structure. Based on laboratory test data, a nonlinear composite connection model (Craig *et al.*, 1992; Pinelli *et al.*, 1995) shown in Fig. 3 is used to represent the behavior of the complete attachment system.

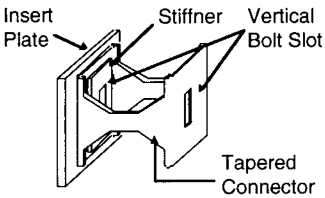


Fig. 1. Geometry of tapered flexural connector

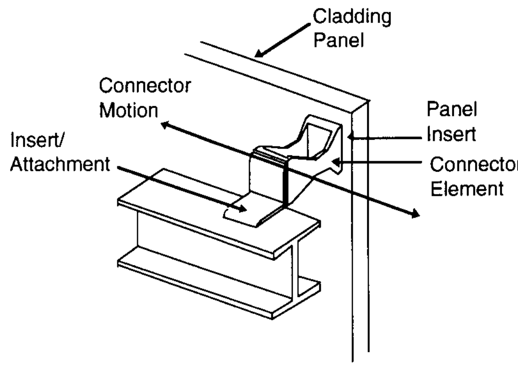


Fig. 2. Cladding attachment system with tapered energy dissipative connector.

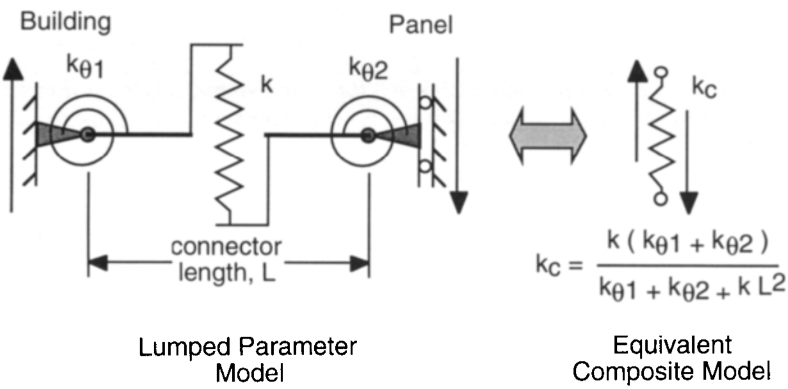


Fig. 3. Schematic of the composite connection model.

ROBUST BUILDING STRUCTURAL CONTROL

The problem of building structural control is posed as a robust performance problem. Robust performance means having a controller which maintains stability and performance specifications (disturbance attenuation) in spite of uncertainties in the dynamic system due to modeling errors and parameter variations. Because robust controllers can tolerate uncertainties, control of large complex building structures during an earthquake is an ideal application for them. For example, nonlinear element behavior can be treated as uncertainty and integrated into the robust control design. Also, the design is optimized for a worst case disturbance (a unit energy bounded disturbance that maximizes the energy in the response metric). This section briefly describes the H_∞ and μ-synthesis methods. Refer to (Doyle *et al.*, 1989) for details about the theory, and to (Maciejowski, 1989; Morari *et al.*, 1989) for its general application. Application to control of building structures is treated in more depth in (Calise *et al.*, 1993; Calise *et al.*, 1994; Soong, 1990).

H_∞ Control and μ-synthesis

The key concept of the H_∞ design is the infinity norm of a transfer function matrix. For a stable Laplace transform matrix T(s), the infinity norm is defined as the least upper bound over all frequencies of the maximum frequency-dependent singular values $\bar{\sigma}(j\omega)$:

$$\|T\|_\infty = \sup_{\omega}\left\{\bar{\sigma}\left(T(j\omega)\right)\right\} \tag{1}$$

The infinity norm can be interpreted as the gain of the system. It is the worst-case amplification of system outputs over all unit energy (L_2 norm) bounded inputs. The uncertainty in the system itself can also be formulated in terms of the infinity norm. In this formulation, uncertainty is represented by an infinity-norm bounded stable transfer function matrix $\Delta(s)$, for which $\|\Delta\|_\infty < \delta$. The set of all $\Delta(s)$ satisfying these conditions is denoted by $\boldsymbol{\Delta}_\delta$.

The robust control problem can be described as follows. Consider a generalized plant P(s) as shown in Fig. 4. P(s) is an augmented plant which is comprised of the nominal plant dynamics plus additional states to represent frequency weighting functions that are introduced as part of the design process. The state-space representation of P(s) is given by

$$\dot{x} = \mathrm{A}x + \mathrm{B}_1 \begin{bmatrix} w_1 \\ w_2 \end{bmatrix} + \mathrm{B}_2 u \tag{2}$$

$$\begin{bmatrix} z_1 \\ z_2 \end{bmatrix} = \mathrm{C}_1 x + \mathrm{D}_{11} \begin{bmatrix} w_1 \\ w_2 \end{bmatrix} + \mathrm{D}_{12} u \tag{3}$$

$$y = \mathrm{C}_2 x + \mathrm{D}_{21} \begin{bmatrix} w_1 \\ w_2 \end{bmatrix} + \mathrm{D}_{22} u \tag{4}$$

where x is the state vector, y is the observation vector, u is the control vector; z_2 is the performance vector; w_2 is the disturbance vector; and z_1 and w_1 are signals at points in the loop where uncertainties occur.

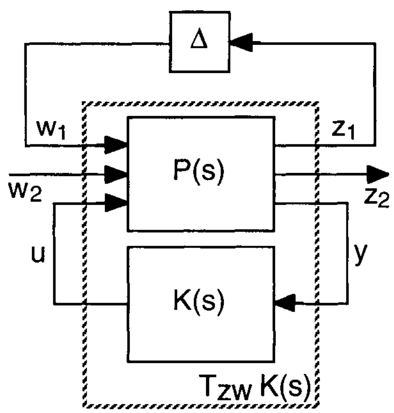


Fig. 4. Robust performance problem.

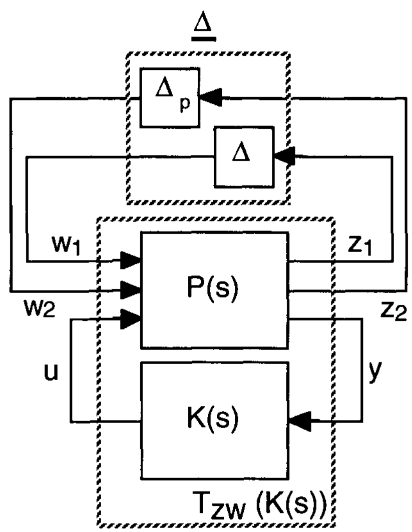


Fig. 5. Robust performance problem re-cast as a robust stability problem.

For a given stabilizing controller, K(s), let the nominal ($\Delta = 0$) closed loop system be denoted as $\mathrm{T}_{zw}(\mathrm{K(s)})$. This system has w_1 and w_2 as its inputs, and z_1 and z_2 as its outputs. Furthermore, we assume that the performance output variables in z_2 have been appropriately weighted so that the following index provides a reliable measure of good nominal ($\Delta = 0$) performance (NP)

$$\text{NP: } \|\mathrm{T}_{22}(\mathrm{K})\|_\infty < \varepsilon \tag{5}$$

where $\mathrm{T}_{ij}(\mathrm{K})$ is the closed loop transfer function from w_j to z_i. With this notation established, then it can be shown (Ref. 10) that a necessary and sufficient condition for robust stability (RS) is

$$\text{RS: } \|T_{11}(K)\|_\infty < 1/\delta \tag{6}$$

Note that by further appropriate scaling of z_1 and z_2, conditions (5) and (6) can be written as

$$\text{NP: } \|T_{22}(K)\|_\infty < 1 \tag{7a}$$

$$\text{RS: } \|T_{11}(K)\|_\infty < 1 \tag{7b}$$

since $\|aT(s)\|_\infty = |a|\|T(s)\|_\infty$.

Next consider the closure of both the upper and lower loops in Fig. 4, and denote the resulting system as M(K,Δ). Note that M(K,Δ) has only w_2 (the disturbance vector) as input and z_2 (the performance vector) as output. The definition of robust performance (RP) is that the RS condition in (7b) is satisfied, and that

$$\|M(K,\Delta)\|_\infty < 1 \tag{8}$$

is also met. Condition (8) means that the NP condition (7a) is also satisfied in the presence of uncertainty. That is, the upper loop is now closed, and the infinity norm of the transfer function from w_2 to z_2 remains less than 1 for all $\Delta \in \mathbf{\Delta}_1$. A sufficient condition for RP is (Pinelli *et al.*, 1995) that

$$\text{RP} \Leftarrow \|T_{zw}(K)\|_\infty < 1 \tag{9}$$

Therefore, the objective in H∞ design is to choose K(s) in such a way so as to satisfy condition (9). Note that the RP condition in (9) is similar to the RS condition in (7b). Therefore, the RP condition in (9) may be viewed as a RS condition, with the $w_2 \rightarrow z_2$ path closed through a fictitious uncertainty block Δp, as depicted in Fig. 5. Note that the resulting uncertainty block, $\underline{\Delta}$, has structure. This is the reason why condition (9) is only sufficient (and therefore conservative). It is also most often the case that the sub-block Δ itself also has a block diagonal structure, due to the presence of multiple uncertainty sources. In this situation, the RS condition in (7b) is also only sufficient, and therefore conservative.

To remove these sources of conservatism in H_∞ design, it is necessary to define the so-called "structured singular value" (Doyle, *et al.*, 1989), $\mu(T_{zw}(K))$, which takes into account the structure present in $\underline{\Delta}$. The corresponding "mu-measure" is defined as

$$\|T_{zw}(K)\|_\mu = \sup_\omega \left\{\mu\left(T_{zw}(j\omega)\right)\right\} \tag{10}$$

Computation of $\mu(T_{zw})$ entails computing the singular value of an appropriately scaled system of the form (Packard *et al.*, 1993)

$$\overline{T}_{zw}(K) = D(s)T_{zw}(K)D^{-1}(s) \tag{11}$$

Thus, the objective in μ-synthesis is to choose K(s) so that the closed loop system is stable (for Δ = 0) and that

$$\|D(s)T_{zw}(K)D^{-1}(s)\|_\infty < 1 \tag{12}$$

The computations required to achieve condition (12) (if possible) entail a procedure known as D-K iteration. At each iteration, the D scales are optimized so as to reduced the gap between $\mu(T_{zw})$ and its upper bound $\|D(s)T_{zw}(K)D^{-1}(s)\|_\infty$. Since D(s) depends on K(s), an iteration is required, in which $K_{i+1}(s)$ is designed to reduce $\left\| D_i(s) T_{zw}(K_{i+1}) D_i^{-1}(s) \right\|_\infty$, and then $D_{i+1}(s)$ is re-optimized for $K_{i+1}(s)$ (Balas *et al.*, 1993). The procedure is initiated with $D_0(s) = I$. This operation considerably increases the dimension of the resulting controller. In general, if the state space realizations for $T_{zw}(s)$ and D(s) are n_t and n_d respectively, the dimension of the resulting controller is $n_c = n_t + 2\,n_d$.

CASE STUDY

In order to illustrate the role of hybrid control applications in the seismic response of building structures, simulation studies were performed on a 1/4 scale 6 story steel space frame. The frame was used in recent analytical and experimental studies at the National Center for Earthquake Engineering Research (NCEER) (Reinhorn, *et al.,* 1989). The dimensions of the frame are 216" in height and 144" in width and there are 3 bays per story. The weight of the complete structure is 42.0 kips. The basic building analytical model was modified to include both cladding panels and an active tendon system. There are 2 precast cladding panels per bay supported by the frame. Each panel was assumed to weigh 150 lb., bringing the total building weight to 52.8 kips. The model is shown in Fig. 6. Panels were assumed to be rigid in plane and to be attached to the main frame with panel connections at the four corners. The two lower panel connections were assumed to be stiff bearing connections, and only the upper connections were considered flexible and ductile to provide hysteretic damping during seismic excitation.

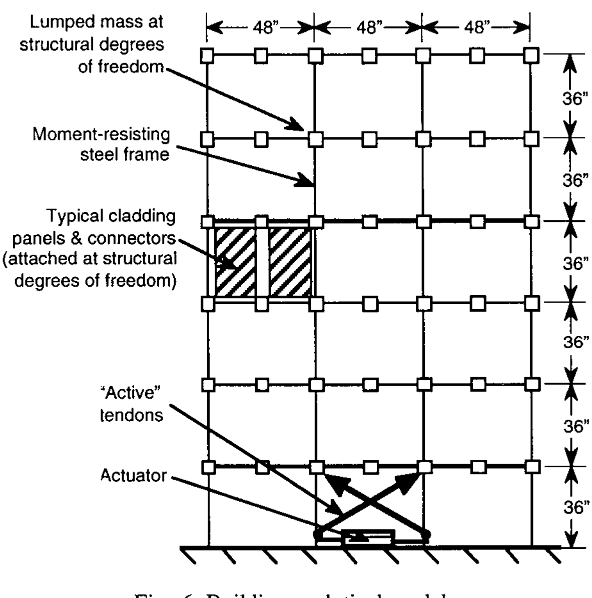


Fig. 6. Building analytical model.

The active tendon control system was assumed to use acceleration feedback, since acceleration response can be measured easily and accurately by acceleration sensors. The active control device was assumed to consist of a set of pretensioned tendons connected between a hydraulic actuator and the first floor of the structure, allowing control action to be transmitted to the frame between the ground and second floor.

Control Design Model

The control design was based on a linear elastic building model with 6 translational degrees-of-freedom (one per floor). Figure 7 shows the problem setup and the generalized plant. The transfer function of the nominal building is given by $(sI\text{-}A)^{-1}$. The blocks B and E represent the control effect matrix and the earthquake participation matrix, respectively. Variable y is the measurement vector (inertial accelerations at each floor). The elements of the performance vector, z_2, are the floor displacement vector and the actuator displacement, u.

Two exogenous disturbances, w_{21} and w_{22}, were considered in the design. Variable w_{21} represents the input earthquake excitation, and variable w_{22} is the sensor noise. The effects of actuator dynamics and time delay were represented by the inverse multiplicative uncertainty Δi. The cladding stiffness and damping properties were treated as an uncertainty in the structural stiffness, and were represented by the transfer function matrix Δk(s). The corresponding inputs and outputs to these uncertainty blocks are denoted as z_{11}, z_{12} and w_{11}, w_{12} in Fig. 7.

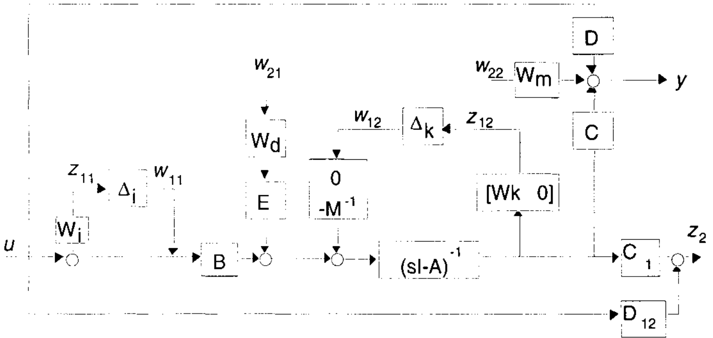
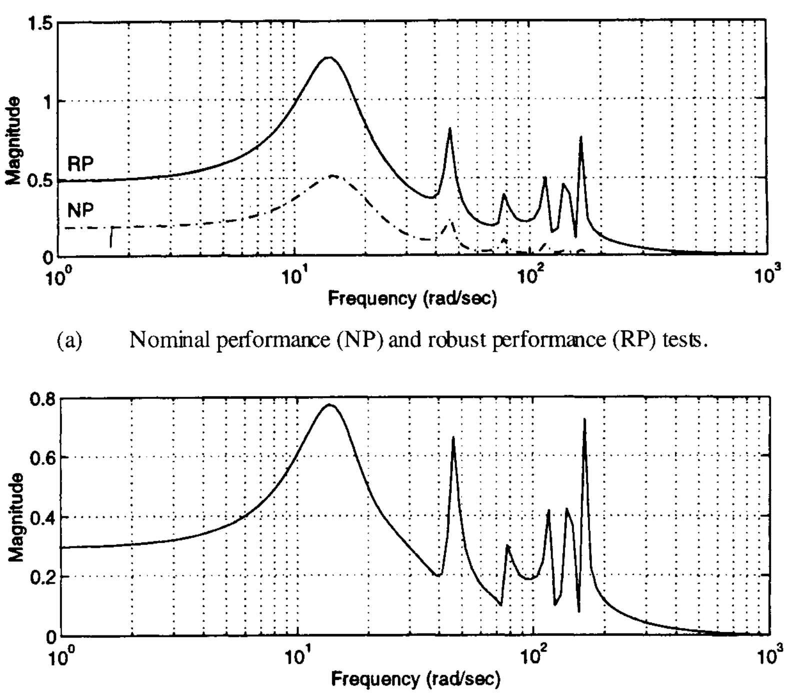


Fig. 7. Block diagram description of the generalized building control model.

Controllers were designed using the μ-synthesis method (H∞ design + D-K iterations). In this case, controllers were designed to minimize the relative floor displacements, while keeping the actuator displacement small. Results of the NP test, the RS test, and the RP test for the resulting μ controller are shown in Fig. 8. It can be observed that the closed-loop system satisfies RS test in Eq. (7b), but fails the RP test in Eq. (12). The interpretation here is that performance (as measured by the H∞ norm) is degraded by a factor of 2.5 for the worst case bounded uncertainty. In general, the level uncertainty limits the level of achievable performance, and the sensitivity of achievable performance is measured in RP test.



(a) Nominal performance (NP) and robust performance (RP) tests.

(b) Robust stability (RS) test

Fig. 8. Nominal performance, robust performance and robust stability test results.

Evaluation of Building Seismic Performance

A nonlinear dynamic analysis program, DRAIN-2D (Powell, 1973), was modified extensively and used to compute the responses of the case study structure to the 1940 El Centro earthquake NS record. New program modules, including passive cladding connection elements, active controllers, and dynamic compensators, were added to DRAIN-2D in order to represent the passive cladding and active tendon control in the study building. The cyclic force-displacement behavior of the upper panel connections was modeled with the elastic-pure-plastic curve. The skeleton curve was defined by the yield displacement δy and the yield force fy, which together characterize the passive cladding system.

The basis for comparison is the nominal open-loop system which refers to the case study NCEER frame with the cladding installed but not interacting with the frame (i.e., only the cladding mass considered). No cladding stiffness and damping were considered, and the active tendon system was not activated. Figure 9 shows the top floor displacement-time history. The maximum and rms displacements were computed to be 2.27" and 0.60", respectively, for the first 30 seconds of the input. Figure 10 shows the closed-loop system response using the active tendon system with the μ -synthesis controller. In this case, the maximum and the rms top floor displacements were reduced to 1.06" and 0.19", respectively. These correspond to 53% and 68% reductions, respectively. Figure 11 shows the closed-loop response with the passive cladding added to produce a hybrid system. The yield displacement and yield forces, δy and fy, in the upper cladding connections were specified as 0.01" and 1.25 kips respectively. The maximum and rms displacements were computed to be 0.83" and 0.12", respectively. For this particular case, additional 22% and 37% reductions on maximum and rms responses, respectively, were achieved. Meanwhile, the rms demand on the control force was reduced from 2.7 kips for the active control case to 2.1 kips for the hybrid active/passive configuration (22% reduction).

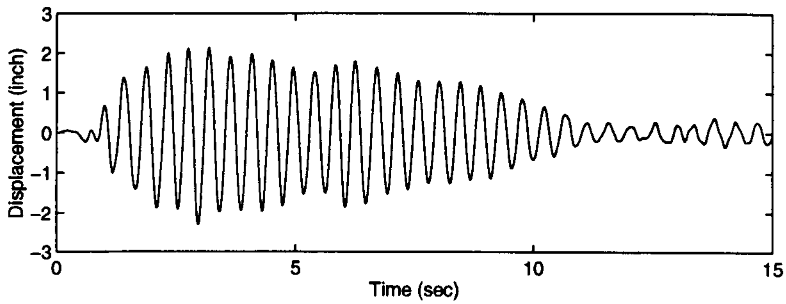


Fig. 9. Top floor displacement time-history for the open-loop system.

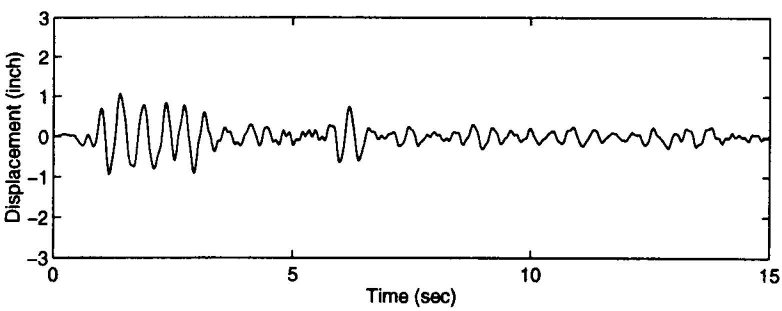


Fig. 10. Top floor displacement time-history for the active control system.

To broaden the understanding of the behavior of the hybrid system, simulations were carried out for 130 different cases. In each case, the same μ -synthesis controller was used, but the properties of the passive cladding system were varied. Results are summarized in Fig. 12, which shows a contour plot of the maximum interstory drift response as a function of associated connector properties, δy and fy. Each intersection point was defined by the associated δy and fy. The corresponding maximum drift response for the nominal open-loop system was computed to be 0.505".

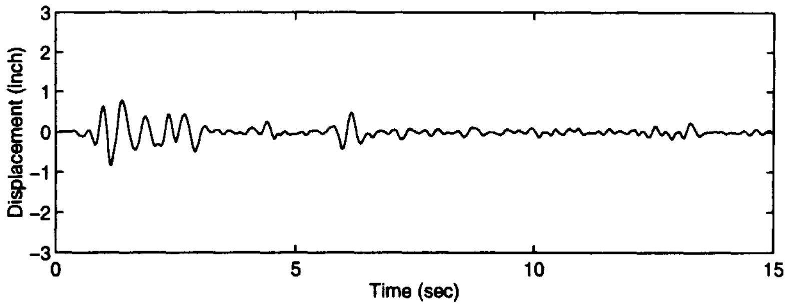


Fig. 11. Top floor displacement time-history for the hybrid control system.

Finally, Fig. 13 shows a similar contour plot for the required control force demand in the hybrid system. The maximum control demand of 13.6 kips is relaxed with the addition of passive cladding damping.

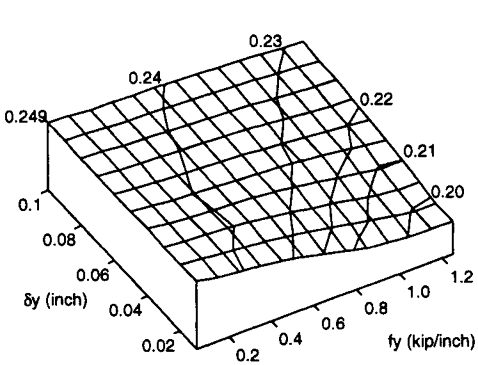


Fig. 12. Contour plot of the maximum interstory drift (inches)

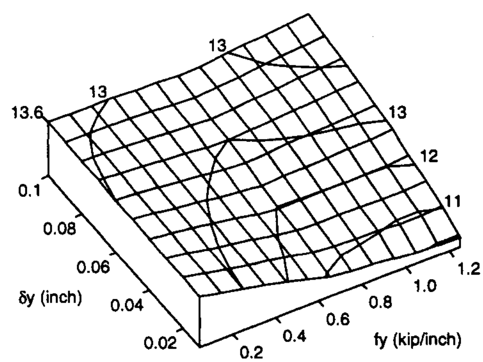


Fig. 13. Contour plot of the maximum control force (kips)

CONCLUSIONS

A hybrid control system, consisting of passive cladding and active tendon systems, was shown to be an effective means of reducing building seismic response for a broad range of input motion levels. The passive system consisted of heavy cladding combined with specially designed advanced connector elements which dissipate energy at low excitation levels and yet retain structural integrity under larger motion. The active tendon system was designed to operate at higher excitation levels. When used together, the resulting hybrid system was shown to yield reduced demands on either the passive or active systems acting alone, while at the same time lowering building interstory drift response. Although the performance was degraded in the presence of modeling uncertainties and for the worst case bounded disturbance, the resultant μ controller met the nominal performance and robust stability requirements. The main benefit of active control is that it continues to provide a margin of response suppression beyond the point where the advanced passive connectors may fail. It remains to verify analytical results with laboratory experiments on the case study structure in order to complete the validation of the concept of heavy cladding systems as part of an economical hybrid control system for buildings in seismic regions.

ACKNOWLEDGMENTS

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ACTIVE STRUCTURAL CONTROL OF SEISMIC EFFECTS: LIMITATIONS AND POTENTIAL

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ABSTRACT

Passive and active control methods are being examined currently for seismic protection of civil structures. Passive methods have already found application in practice. Active control method, on the other hand, are being studied vigorously for practical applications. This paper describes an active control method, known as the sliding control method, for seismic response control of civil structures. This method has been used in mechanical and electrical control applications, but its application to civil structures has not been explored much until very recently. The main characteristics of the method is its robustness under parametric uncertainties. As a result, it can be applied to linear as well as nonlinear structures. The paper describes the basic formulation of the method without going into the details of advanced mathematical subtleties. The approach is applied to a realistic 10-story building to evaluate the effectiveness of active control and also to evaluate the feasibility of its application in real buildings. The numerical results clearly indicate that active control can, indeed, be used to reduce seismic structural response. However, the magnitude of the control force and especially the power required for a reasonable reduction in the response seem to be very large for practical application. Alternative ways of applying large forces without using a large power must be explored for civil applications.

INTRODUCTION

Continued research efforts in the area of dynamics of structures subjected to earthquake induced ground motions have led to better understanding of structural behavior under seismic loads and to improved designs of earthquake resistant structures. The majority of current structural designs are made to withstand anticipated forces and deformations caused by a design level earthquake. Lately, however, increasing attention is also being given to reduce the design forces, deformations, and accelerations induced in structural systems by seismic motions.

The dynamic response reduction or control methods that are being considered can be classified as (1) passive and (2) active methods. Passive control approaches are basically of two types: Base isolators and energy dissipators. In both these approaches the control mechanism is activated by the motion of structure and no other external force or energy is applied from outside.

The active control methods, on the other hand, apply forces to the structure or activate internal forces or reactions to counteract the dynamic forces to reduce structural response. The control approaches which were primarily developed for electrical, mechanical and aerospace systems are now being extended for application to civil structures. This movement started about 1970, and among other leading researchers, Yao (1972) deserves significant credit for introducing civil engineering community to the concept of structural control. Soong (1988) and Yang and Soong (1988) provide an excellent background and historical development of control in civil structural systems.

One of the most commonly used control approach is based on the linear optimal control theory. In this approach one optimizes a non-negative quadratic performance function which depends upon the state vector of the system and the applied control effort. Several variations of this approach has been considered. The approach is valid for linearly behaving structural systems. It involves the solution of a Riccati equation, for which now several standard computing packages are available.

Other approaches such as independent modal control (Meirovitch and Oz, 1980), instantaneous optimal control (Yang and Akbarpour, 1987), acceleration feedback control (Spencer *et al.* 1993), and robust control techniques (Schmitendorf *et al.* 1994) have also been developed for structural control applications.

An ideal situation for a feedback controller is that the state-vector response be known in its entirety. Also, the application of the control action is assumed to be instantaneous as soon as the state measurement are fed back to the system. These are, of course, idealized conditions which can not be realized in practice. Research to counteract the effect of using incomplete state information and of the time delay involved in the application of control actions is continuing to provide practical answers.

One basic limitation of the majority of control method being investigated for civil applications is that they are applicable to only linearly behaving systems, although in many structural designs for seismic loads, nonlinear behavior is implicit. Also because of reliability issues, active control scheme are most likely to be considered only to supplement other protective passive devices such as base isolation or energy dissipation devices. Since most passive devices introduce nonlinear behavior in a structure, the active control scheme to be used with such passive devices must be able to incorporate such nonlinearities. The design of a control system for a nonlinear structure, however, is not as well established as for linear structures. Specialized nonlinear approaches which suit special applications have been developed. Examples of these are: trial-and-error procedures, feedback linearization, adaptive control and sliding mode control.

The sliding mode control has a well established theory (Itkis, 1976; Utkin, 1972, 1978, 1983, 1992; Slotine and Li, 1991; Zhou and Fisher, 1992; Utkin and Yang, 1978), although using it with a strictly nonlinear formulation sometimes may not be feasible. However, being a very robust approach it can still be effectively used to solve nonlinear control problems by simply treating the nonlinearities as variations of the nominal linear models. This paper deals with the application of the sliding mode control approach to earthquake engineering control problems. Such applications have also been studied earlier by Yang *et al* (1994) and in a preliminary form by Ramu, *et al* (1994). Here only the basic formulation of the approach is presented without devoting much time to advanced mathematical concepts and subtleties; these are provided in the literature cited earlier.

SLIDING MODE CONTROL

Basic Formulation:

The equations of motion for an n_f degrees-of-freedom structural system subjected to m-independent control actions $\mathbf{u}$, and earthquake induced base excitation $\ddot{x}_g(t)$, can be written in the following first order state form:

$$\dot{\mathbf{x}} = \mathbf{f}_s(\mathbf{x}) + \mathbf{B}\mathbf{u} + \mathbf{E}\ddot{\mathbf{x}}_\mathbf{g}(\mathbf{t}) \tag{1}$$

where $\mathbf{x}$ is a $n = 2n_f$ dimension state vector consisting of velocity and displacement responses of the system. The n dimension vector $\mathbf{f}_s(\mathbf{x})$ contains the nonlinear restoring and dissipative forces. It may be a nonlinear function of $\mathbf{x}$. $\mathbf{B}$ is a $n \times m$ input matrix of rank m. It depends upon where and how the control forces are applied. In most structural applications, except in the case of the parametric control, this matrix will be independent of the state vector. $\mathbf{E}$ is the disturbance input vector of dimension n for base excitation applied in one direction.

For a linear structural system acted upon by control actions $\mathbf{u}$ with the following equations of motion:

$$\mathbf{M}\ddot{\mathbf{z}} + \mathbf{C}\dot{\mathbf{z}} + \mathbf{K}\mathbf{z} = -\mathbf{M}\mathbf{r}\ddot{x}_g(t) + \mathbf{D}\mathbf{u}(t) \tag{2}$$

the state vector form of the equation can be written as:

$$\dot{\mathbf{x}} = \frac{d}{dt}\begin{Bmatrix} \dot{\mathbf{z}} \\ \mathbf{z} \end{Bmatrix} = \mathbf{Ax} + \mathbf{Bu} + \mathbf{E}\ddot{\mathbf{x}}_g(t) \tag{3}$$

where for this linear case,

$$\mathbf{A} = \begin{bmatrix} \mathbf{0} & \mathbf{I} \\ -\mathbf{M}^{-1}\mathbf{K} & -\mathbf{M}^{-1}\mathbf{C} \end{bmatrix}, \ \mathbf{B} = \begin{bmatrix} \mathbf{0} \\ \mathbf{M}^{-1}\mathbf{D} \end{bmatrix}, \ \mathbf{E} = \begin{bmatrix} 0 \\ -\mathbf{Mr} \end{bmatrix}. \tag{4}$$

In equation (2) $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$, respectively, are the mass, damping and stiffness matrices of the structure; $\mathbf{D}$ = location matrix; and $\mathbf{r}$ = influence vector.

The central objective of the sliding mode approach is to obtain m control actions, or design m controllers, to force the system state to reach a predefined manifold, called sliding surface, and then maintain it there. The sliding surface is chosen such that the motion when constrained to remain on the surface shows some desirable characteristics including the asymptotic stability of the system. In general, a sliding surface can be defined as a $(n-m)$ dimensional manifold as

$$\mathbf{s}(\mathbf{x}) = \begin{Bmatrix} s_1(\mathbf{x}) \\ s_2(\mathbf{x}) \\ \vdots \\ s_m(\mathbf{x}) \end{Bmatrix} = 0. \tag{5}$$

For a linear case, the sliding surface can be defined by hyperplanes as:

$$\mathbf{s}(\mathbf{x}) = \mathbf{Cx} = 0 \tag{6}$$

The control actions should be such that the motion approaches and stays on the sliding surface which implies that the origin of the m-dimensional space generated by $s_1, s_2, \ldots, s_m$ is an asymptotically stable equilibrium point (Ref. 10).

To maintain the system on the sliding surface, the required control actions must satisfy the following condition

$$\dot{\mathbf{s}}(t) = \frac{\partial \mathbf{s}}{\partial \mathbf{x}} \cdot \dot{\mathbf{x}} = \mathbf{G}\dot{\mathbf{x}} = 0 \tag{7}$$

at the sliding surface. In equation (7), the matrix of gradients $\mathbf{G}$ is a $m \times n$ matrix as follows

$$\mathbf{G}(x) = \begin{bmatrix} \frac{\partial s_1}{\partial x_1} & \frac{\partial s_1}{\partial x_2} & \cdots & \frac{\partial s_1}{\partial x_n} \\ \frac{\partial s_2}{\partial x_1} & \frac{\partial s_2}{\partial x_2} & \cdots & \frac{\partial s_2}{\partial x_n} \\ \vdots & \vdots & \ddots & \vdots \\ \frac{\partial s_m}{\partial x_1} & \frac{\partial s_m}{\partial x_2} & \cdots & \frac{\partial s_m}{\partial x_n} \end{bmatrix} \tag{8}$$

This matrix may depend upon the state vector for a nonlinear sliding surface. For a linear sliding surface, $\mathbf{G}$ is the same as the coefficient matrix $\mathbf{C}$ in equation (6). Substituting the state equation (1) in equation (7) and solving for the control actions $\hat{\mathbf{u}}$ which satisfy equation (7) we obtain

$$\hat{\mathbf{u}} = \mathbf{u}_{eq} + \mathbf{u}_h \tag{9}$$

where $\mathbf{u}_{eq}$ is the so-called equivalent control (Ref. 8) and $\mathbf{u}_h$ is the feed forward term of the control actions. These control terms are defined as:

$$\mathbf{u}_{eq} = -\left[\mathbf{GB}\right]^{-1}\mathbf{Gf}_s \ ; \ \mathbf{u}_h = -\left[\mathbf{GB}\right]^{-1}\mathbf{GE}\ddot{x}_g(t) \tag{10}$$

The sliding surface must be such that $[\mathbf{GB}]$ is nonsingular to permit inversion indicated in equation (10). That is, the rank of $\mathbf{G}$ must be m. With the control actions of equation (10), the state equation of motion (1) can be written as:

$$\dot{\mathbf{x}} = \mathbf{Q}\left\{\mathbf{f}_s(\mathbf{x}) + \mathbf{E}\ddot{x}_g\right\} \tag{11}$$

where matrix $\mathbf{Q}$ is defined as

$$\mathbf{Q} = \mathbf{I} - \mathbf{B}[\mathbf{G}\mathbf{B}]^{-1}\mathbf{G} \tag{12}$$

It is noted that in a general case, matrix $\mathbf{Q}$ is a function of state vector $\mathbf{x}$. One can reduce the order of the system of equations (11) by utilizing the static relationship represented by the sliding surface. In principle, we can solve equations (5) for m state variables $\mathbf{x}_2$ in terms of remaining $n-m$ state variables $\mathbf{x}_1$ as

$$\mathbf{x}_2 = \mathbf{g}(\mathbf{x}_1) \tag{13}$$

Substituting for $\mathbf{x}_2$ in terms of $\mathbf{x}_1$ in the state equation of motion (11), we obtain a reduced system as follows:

$$\dot{\mathbf{x}}_1 = \mathbf{f}_1(\mathbf{x}_1, \mathbf{g}(\mathbf{x}_1), \mathbf{G}) + \mathbf{Q}_1\mathbf{E}\ddot{x}_g \tag{14}$$

where $\mathbf{Q}_1$ is the appropriate submatrix of $\mathbf{Q}$ and $\mathbf{f}_1(\mathbf{x}_1, \mathbf{g}(\mathbf{x}_1), \mathbf{G}) = \mathbf{Q}_1\mathbf{f}_s(\mathbf{x})$.

For linear systems, the control actions which keep the state on the sliding surfaces are still defined by equations (10), except that the gradient matrix $\mathbf{G}$ is now replaced by the coefficient matrix $\mathbf{C}$. With these control actions, the equation of motion, separated into vectors $\mathbf{x}_1$ and $\mathbf{x}_2$, can be written as:

$$\begin{Bmatrix} \dot{\mathbf{x}}_1 \\ \dot{\mathbf{x}}_2 \end{Bmatrix} = \begin{bmatrix} \mathbf{Q}_1 \\ \mathbf{Q}_2 \end{bmatrix} \left\{ \begin{bmatrix} \mathbf{A}_{11} & \mathbf{A}_{12} \\ \mathbf{A}_{21} & \mathbf{A}_{22} \end{bmatrix} \begin{Bmatrix} \mathbf{x}_1 \\ \mathbf{x}_2 \end{Bmatrix} + \begin{Bmatrix} \mathbf{E}_1 \\ \mathbf{E}_2 \end{Bmatrix} \ddot{x}_g \right\} \tag{15}$$

where $\mathbf{Q}_1$ and $\mathbf{Q}_2$ are parts of matrix $\mathbf{Q}$ which is still defined by equation (12) except that $\mathbf{G}$ is now replaced by the coefficient matrix $\mathbf{C}$. We can eliminate $\mathbf{x}_2$ from these equations by using the linear sliding constraint. From equations (6) we obtain

$$\mathbf{C}_1\mathbf{x} = [\mathbf{C}_1 \ \mathbf{C}_2] \begin{Bmatrix} \mathbf{x}_1 \\ \mathbf{x}_2 \end{Bmatrix} = 0 \rightarrow \mathbf{x}_2 = -\mathbf{C}_2^{-1}\mathbf{C}_1\mathbf{x}_1. \tag{16}$$

Substituting for $\mathbf{x}_2$ into the first equation of (15) we obtain the following for a linear system,

$$\dot{\mathbf{x}}_1 = \mathbf{Q}_1 \left\{ \left[\mathbf{A}_{11} - \mathbf{A}_{12}\mathbf{C}_2^{-1}\mathbf{C}_1 \right] \mathbf{x}_1 + \mathbf{E}_1\ddot{x}_g \right\}. \tag{17}$$

Regular Form of Equations

We will attempt to use equation (14) for the nonlinear case and equation (17) for the linear case to define the sliding surface such that the motion defined by these equations has desirable characteristics. The dependence of matrix $\mathbf{Q}_1$ on the sliding surface (or its gradient matrix) however, complicates the use of these two equations directly, and thus this dependence must be removed. This can be achieved by transforming the state equations of motions into the so-called "regular form." The special characteristics of this form is that the first $n-m$ equations do not have any control action terms. This will happen when the first $n-m$ rows of matrix $\mathbf{B}$ are zero. For such a $\mathbf{B}$ matrix (see equation (12)) $\mathbf{Q}_1$ matrix will only contain first $(n-m)$ rows of the identity matrix $\mathbf{I}$ and thus rendering equations (14) and (17) independent of the gradient matrix.

In many structural applications, the control force applied to a particular degree of freedom may not induce forces in other degrees of freedom. In such a case, a simple re-ordering of the equations of motion will directly render them in the regular form. In other cases, however, a special transformation of coordinates may be needed.

When matrix $\mathbf{B}$ is independent of the state vector, it is possible to subdivide it into $\mathbf{B}_1$, and $\mathbf{B}_2$ as follows:

$$[\mathbf{B}] = \begin{bmatrix} \mathbf{B}_1 \\ \mathbf{B}_2 \end{bmatrix}. \tag{18}$$

with $\mathbf{B}_2$ being a $m \times m$ nonsingular matrix. The following transformation of variables, defined in terms of $\mathbf{B}_1$ and $\mathbf{B}_2$,

$$\mathbf{y} = \begin{Bmatrix} \mathbf{y}_1 \\ \mathbf{y}_2 \end{Bmatrix} = \mathbf{T}\mathbf{x} = \begin{bmatrix} \mathbf{I}_{n-m} & -\mathbf{B}_1\mathbf{B}_2^{-1} \\ \mathbf{0} & \mathbf{I}_m \end{bmatrix} \begin{Bmatrix} \mathbf{x}_1 \\ \mathbf{x}_2 \end{Bmatrix} \tag{19}$$

will then convert the original state equation to a regular form as follows.

$$\dot{\mathbf{y}} = \bar{\mathbf{f}}_s(\mathbf{y}) + \begin{bmatrix} \mathbf{0} \\ \mathbf{B}_2 \end{bmatrix} \mathbf{u} + \bar{\mathbf{E}}\ddot{x}_g. \tag{20}$$

where $\bar{\mathbf{f}}_s = \mathbf{T}\mathbf{f}_s(\mathbf{T}^{-1}\mathbf{y}), \bar{\mathbf{E}} = \mathbf{T}\mathbf{E}$ and

$$\mathbf{T}\mathbf{B} = \begin{bmatrix} \mathbf{0} \\ \mathbf{B}_2 \end{bmatrix} \tag{21}$$

Eq. (20) can be partitioned as

$$\begin{Bmatrix} \dot{\mathbf{y}}_1 \\ \dot{\mathbf{y}}_2 \end{Bmatrix} = \begin{Bmatrix} \bar{\mathbf{f}}_{s_1}(\bar{\mathbf{y}}_1, \bar{\mathbf{y}}_2) \\ \bar{\mathbf{f}}_{s_2}(\bar{\mathbf{y}}_1, \bar{\mathbf{y}}_2) \end{Bmatrix} + \begin{bmatrix} \mathbf{0} \\ \mathbf{B}_2 \end{bmatrix} \mathbf{u} + \begin{Bmatrix} \bar{\mathbf{E}}_1 \\ \bar{\mathbf{E}}_2 \end{Bmatrix} \ddot{x}_g \tag{22}$$

To reduce the order of the system of equation in (22), we now use the sliding surface equations in the transformed coordinates to solve for $\mathbf{y}_2$ in terms $\mathbf{y}_1$ as,

$$\mathbf{y}_2 = \bar{\mathbf{g}}(\mathbf{y}_1) \tag{23}$$

Substituting for $\mathbf{y}_2$ in the first equation of (22), we obtain the equation of motion for $\mathbf{y}_1$ as

$$\dot{\mathbf{y}}_1 = \bar{\mathbf{f}}_{s_1}(\mathbf{y}_1, \bar{\mathbf{g}}(\mathbf{y}_1)) + \bar{\mathbf{E}}_1\ddot{x}_g \tag{24}$$

In the case of linear systems, one similarly obtains the following regular form for the transformed variables

$$\begin{Bmatrix} \dot{\mathbf{y}}_1 \\ \dot{\mathbf{y}}_2 \end{Bmatrix} = \begin{bmatrix} \bar{\mathbf{A}}_{11} & \bar{\mathbf{A}}_{12} \\ \bar{\mathbf{A}}_{21} & \bar{\mathbf{A}}_{22} \end{bmatrix} \begin{Bmatrix} \mathbf{y}_1 \\ \mathbf{y}_2 \end{Bmatrix} + \begin{bmatrix} \mathbf{0} \\ \mathbf{B}_2 \end{bmatrix} \mathbf{u} + \begin{Bmatrix} \bar{\mathbf{E}}_1 \\ \bar{\mathbf{E}}_2 \end{Bmatrix} \ddot{x}_g \tag{25}$$

The corresponding sliding surfaces in term of variables $\mathbf{y}$ are

$$\mathbf{C}\mathbf{x} = \mathbf{C}\mathbf{T}^{-1}\mathbf{y} = \begin{bmatrix} \bar{\mathbf{C}}_1 & \bar{\mathbf{C}}_2 \end{bmatrix} \begin{Bmatrix} \mathbf{y}_1 \\ \mathbf{y}_2 \end{Bmatrix} = \mathbf{0} \tag{26}$$

where $\bar{\mathbf{C}}_1$ and $\bar{\mathbf{C}}_2$ are the submatrices of $\left[\mathbf{C}\mathbf{T}^{-1}\right]$. Substituting for $\mathbf{y}_2$ in terms of $\mathbf{y}_1$ from equation (26), we obtain the equations of motion for $\mathbf{y}_1$ as follows:

$$\dot{\mathbf{y}}_1 = \left[\bar{\mathbf{A}}_{11} - \bar{\mathbf{A}}_{12}\bar{\mathbf{C}}_2^{-1}\bar{\mathbf{C}}_1\right]\mathbf{y}_1 + \bar{\mathbf{E}}_1\ddot{\mathbf{x}}_g. \tag{27}$$

Calculation of Sliding Surface

The definition of the sliding surface now reduces to a proper selection of m functions $\bar{\mathbf{g}}(\mathbf{y}_1)$ in the nonlinear case or the prescription of matrix $(\bar{\mathbf{C}}_2^{-1}\bar{\mathbf{C}}_1)$ in the linear case. This selection should be such that the part of motion determined by these equations is asymptotically stable. This will ensure stability of the complete system, including the motion on the sliding surfaces. The vector $\mathbf{y}_2 = \bar{\mathbf{g}}(\mathbf{y}_1)$ in (24) and $\mathbf{y}_2 = -\mathbf{C}_2^{-1}\mathbf{C}_1\mathbf{y}_1$ in (27) represent feedback control actions. The sliding surface design problem, therefore, reduces to the specification of this control. In the selection of these feedback actions, usually the effect of external disturbance $\bar{\mathbf{E}}_1\ddot{x}_g$ is neglected.

There is no universal systematic approach to define nonlinear sliding surfaces for a nonlinear system corresponding to equation (24). Methods have been developed to suit the form of nonlinearity encountered in the problem. Lyapunov's direct method is used in a trial and error fashion to arrive at a feasible sliding surface which provides asymptotic stability. Cancellation of nonlinear terms by control actions to effectively reduce the problem to a linear case have also been successfully implemented (Slotine and Li, 1991; Yang, *et al*, 1994).

For a linear system, the job of establishing the sliding surface can be done quite systematically using some well established procedures. Without any loss of generality, it is convenient to choose $\bar{\mathbf{C}}_2$ to be an identity matrix. Some of the methods that are available to obtain $\bar{\mathbf{C}}_1$ corresponding to equation (27) are: (1) linear quadratic regulator (2) eigenstructure assignment and (3) parametric optimization techniques. All these approaches ensure that the part of the motion described by equation (27) is stable.

The linear quadratic regulator approach can be used with the minimization of a positive quadratic function of the original state vector $\mathbf{x}$. If desired equivalent control effort $\mathbf{u}_{eq}$ can also be included in the quadratic function. The problem then becomes a parametric optimization problem (Utkin and Yang, 1978).

Having established the sliding surface, we will now define the control actions which would bring the system to the sliding surface and then try to keep it there, while maintaining the stability of the system at all times. This is described in the following section.

CONTROLLER DESIGN

There are many different ways in which the desired control actions can be prescribed to force the system towards the sliding surface, bring it back to the surface if it strays away from it and maintain asymptotic stability of the system. In the prescription of control actions, it is a common practice to compensate for the systems actions through the equivalent control and for external disturbance through the feed forward control terms. Together these two control actions were denoted by $\hat{\mathbf{u}}$ in equation (9), and are re-written here as:

$$\hat{\mathbf{u}}(\mathbf{x}, \ddot{x}_g) = -\left[\mathbf{GB}\right]^{-1}\left[\mathbf{f}_s(\mathbf{x}) + \mathbf{E}\ddot{x}_g\right] \tag{28}$$

These control actions only ensure that $\dot{\mathbf{s}} = 0$. Further control actions are necessary to bring the trajectory to the sliding surface $\mathbf{s} = 0$. Denoting these additional control actions by $\mathbf{u}_d(\mathbf{x})$, the total control action can then be defined as

$$\mathbf{u}(\mathbf{x}) = \hat{\mathbf{u}}(\mathbf{x}, \ddot{x}_g) + \mathbf{u}_d(\mathbf{x}). \tag{29}$$

One can establish $\mathbf{u}_d(\mathbf{x})$ in a general way by the application of Lyapunov's direct method. In the following we give the expressions for three possible controllers, with different $\mathbf{u}_d(x)$ components. They satisfy the asymptotic stability conditions for the Lyapunov's function indicated by V

Controller I:

$$\mathbf{u}(x) = \hat{\mathbf{u}} - \left[\mathbf{GB}\right]^{-1} \Delta\ sign(\mathbf{s})$$
$$V = \frac{1}{2}\mathbf{s}^T\mathbf{s},$$
$$\frac{dV}{dt} = -\sum_{i=1}^{m}\delta_i|s_i|$$

Controller II:

$$\mathbf{u}(x) = \hat{\mathbf{u}} - \Delta\left[\mathbf{GB}\right]^T sign(\mathbf{s})$$
$$V = \frac{1}{2}\mathbf{s}^T\left[\mathbf{GB}\right]\Delta\left[\mathbf{GB}\right]^T\mathbf{s}$$
$$\frac{dV}{dt} = -\sum_{i=1}^{m}|s_i|$$

Controller III:

$$\mathbf{u}(x) = \hat{\mathbf{u}} - \left[\mathbf{GB}\right]^{-1}\Delta\frac{\mathbf{s}}{||\mathbf{s}||_2}$$
$$V = \frac{1}{2}\mathbf{s}^T\mathbf{s}$$
$$\frac{dV}{dt} = -\frac{\mathbf{s}^{\mathbf{T}}\Delta\ \mathbf{s}}{||\mathbf{s}||_2}$$

where Δ is diagonal matrix of design parameters with all positive elements, and $||\mathbf{s}||_2$ is the 2-norm or Euclidean length of vector $\mathbf{s}$.

In the first controller, each control action pertains to a switching surface $\mathbf{s}_i$. Whenever a surface is crossed, a reverse action to bring the trajectory back to the surface occurs. In the second controller, i^{th} control action is affected by the crossings of any or all switching surfaces. Thus if any one of the surfaces are crossed, a discontinuity in the control actions occur. The third controller is a continuous controller, except at the origin, $\mathbf{s} = 0$. For a single control action, that is for $m = 1$, all these controllers are the same.

The three controllers presented above are discontinuous controllers. As such they are likely to induce chattering in the system. To avoid chattering, one can resort to a boundary layer technique (Slotine and Li, 1991) or use continuous controllers. Continuous controllers, where the control action do not change abruptly, have been presented by Zhou and Fisher (1992) based on Lyapunov's direct approach. An extension to this approach to multiple control actions is presented by Yang et al. (Yang, *et al*, 1994). The controller obtained in this reference is the same as in the third controller described above except that the $\mathbf{u}_d(x)$ term does not contain the 2-norm of $\mathbf{s}$.

Controller without feedforward

It is also possible to obtain a controller without a complete feed forward contribution and yet insure attraction to the sliding surface within an acceptable domain. One need to consider only the extreme value of the external disturbance, as is shown in the following.

Consider a control action vector which consists of the equivalent control term, compensating the system's actions, and additional actions defined as follows:

$$\mathbf{u}(\mathbf{x}) = -[\mathbf{GB}]^{-1}\mathbf{Gf}_s - \Delta_1[\mathbf{GB}]^T\mathbf{s} \tag{30}$$

where Δ_1 is a positive definite diagonal matrix. Also consider the following Lyapunov function

$$V = \frac{1}{2}\mathbf{s}^T\mathbf{s} \tag{31}$$

The time derivative of this function can be shown to be

$$\frac{dV}{dt} = -\mathbf{s}^T\Theta\mathbf{s} + \mathbf{s}^T\mathbf{h} \tag{32}$$

where

$$\begin{aligned} \mathbf{h} &= [\mathbf{GB}]\,\mathbf{E}\ddot{x}_g(t) \\ \Theta &= [\mathbf{GB}]\,[\Delta_1]\,[\mathbf{GB}]^T \end{aligned} \tag{33}$$

It is noted that Θ is a positive definite symmetric matrix and vector $\mathbf{h}$ is directly proportional to the intensity of the ground motion. It can be shown that the first term of equation (31) is less than

$$-\mathbf{s}^T\Theta\mathbf{s} \le -\lambda_{min}||\mathbf{s}||_2^2 \tag{34}$$

where λ_{min} is the smallest eigenvalue of matrix Θ. It is also known that the second term is less than the average of the squared 2-norms as

$$\mathbf{s}^T\mathbf{h} \le \frac{1}{2}\left(||\mathbf{s}||_2^2 + ||\mathbf{h}||_2^2\right) \le \frac{1}{2}\left(||\mathbf{s}||_2^2 + e_{max}^2\right) \tag{35}$$

where e_{max} is the 2-norm of $\mathbf{h}$, calculated for $\ddot{x}_g$ being equal to the maximum ground acceleration level of interest. In view of equations (34-35), the time derivative of the Lyapunov function can now be shown to satisfy the following inequality

$$\frac{dV}{dt} \le -\lambda_{min}||\mathbf{s}||_2^2 + \frac{1}{2}\left(||\mathbf{s}||_2^2 + e_{max}^2\right) \tag{36}$$

Depending upon the choice of Δ_1 which defines λ_{min}, equation (36) establishes a region determined by

$$||\mathbf{s}||_2^2 > \frac{e_{max}}{\lambda_{min} - \frac{1}{2}}\ ;\ \lambda_{min} > \frac{1}{2} \tag{37}$$

where the attraction to the sliding surface is guaranteed. Actual region of attraction will in fact be larger than the one defined by equation (37). For a given e_{max} and predecided $||s||_2$, the magnitude of the smallest eigenvalue required can be calculated. This determines the matrix Δ_1 to be chosen to define the required control action. This particular approach has been used in the numerical examples.

NUMERICAL EXAMPLES

To examine the applicability of the sliding mode control, and also to investigate the feasibility of active control in practice, an example problem of a typical ten story building, representing a medium size office building with about 400 sq.m. of area per floor, was considered. The mass of the first floor was 420×10^3 kg, whereas the remaining floors had their mass equal to 350×10^3 kg each. The story stiffnesses were also distributed like the mass. That is, the first story stiffness was 700 MN/m and those of the remaining higher stories were 630 MN/m each. The structure was assumed to have a modal damping ratio of 3% in each mode. The frequencies of this system in Hz are 1.02, 3.02, 4.94, 6.71, 8.32, 9.77, 11.04, 12.08, 12.86 and 13.34.

Only one control action was applied to the building. Therefore, $m = 1$. To apply the control force, four modes of application were considered. Intuitively, the application of a control force on the roof level appears to be the most effective. Such a force can be applied either through to tuned mass damper, or through a pair of tendons operated by a winch at the ground level, or through inertial reaction of an accelerating mass or reaction of an exiting high pressure airblast. In the example considered here, the first three methods, that is, mass damper, tendons and a sliding mass have been examined. The drawback of a tuned mass damper is that it will require installation, and subsequent maintenance, of an oscillator with a rather heavy mass, damping and spring mechanisms. The disadvantage of using tendons is that horizontal force applied at the roof level will also be accompanied by a vertical force. We can reduce the vertical load by applying the tendon force at lower floor levels. In this case, however, the applied force may not be as effective as if it were applied at the top. Numerical results have also been obtained for the tendon force applied at the first floor levels for a comparison of the results.

To examine the effectiveness of the active control for different disturbances, the numerical results have been obtained for four ground motion inputs recorded in four different events: (1) El Centro, 1941, (2) San Fernando, 1971, (3) Loma Prieta, 1989, and (4) Parkfield, 1966. All four ground motions were normalized to a maximum ground acceleration level of 0.3g. The acceleration response spectra for these four inputs are plotted in Figure 1. The differences in the frequency characteristics of these inputs can be noted from this figure.

In the application of the force through a tuned mass damper (TMD), the TMD was nearly tuned to the first modal frequency. The TMD frequency was .93 cps and ratio of its frequency to the fundamental frequency of the building was 0.912. The mass of the damper was about 3% of the total building mass or about 30% of the top floor mass. The damping ratio of the TMD was chosen to be 11% of the critical value.

In Table 1, we show the displacement and acceleration responses of the building subjected to four different ground motion inputs. For each input, the first column shows the response of the unprotected building when no TMD or control actions are applied; the second column shows the response with a TMD, normalized by the unprotected building response; the third column shows the response with TMD and active control, again normalized by the unprotected building response; the fourth column shows the ratio of the TMD and active control response to the response with TMD alone. Thus this fourth column represents the effectiveness of the control action alone. The normalized response values less than 1.00 indicate the effectiveness of the response reduction scheme; the smaller this value, the more effective is the scheme in reducing the response.

From these results we observe that a TMD need not be equally effective with all inputs, and all response quantities. For example, a comparison of response ratios in columns (3), (7), (11) and (15) indicates that the TMD we used was most effective in reducing the displacement response for El Centro input and least effective for Loma Prieta input. Also comparing the ratios under accelerations response with the ratios in displacement response in the same columns, we observe that the TMD is less effective in reducing floor accelerations. In fact, if we compare, TMD acceleration response ratio values in columns

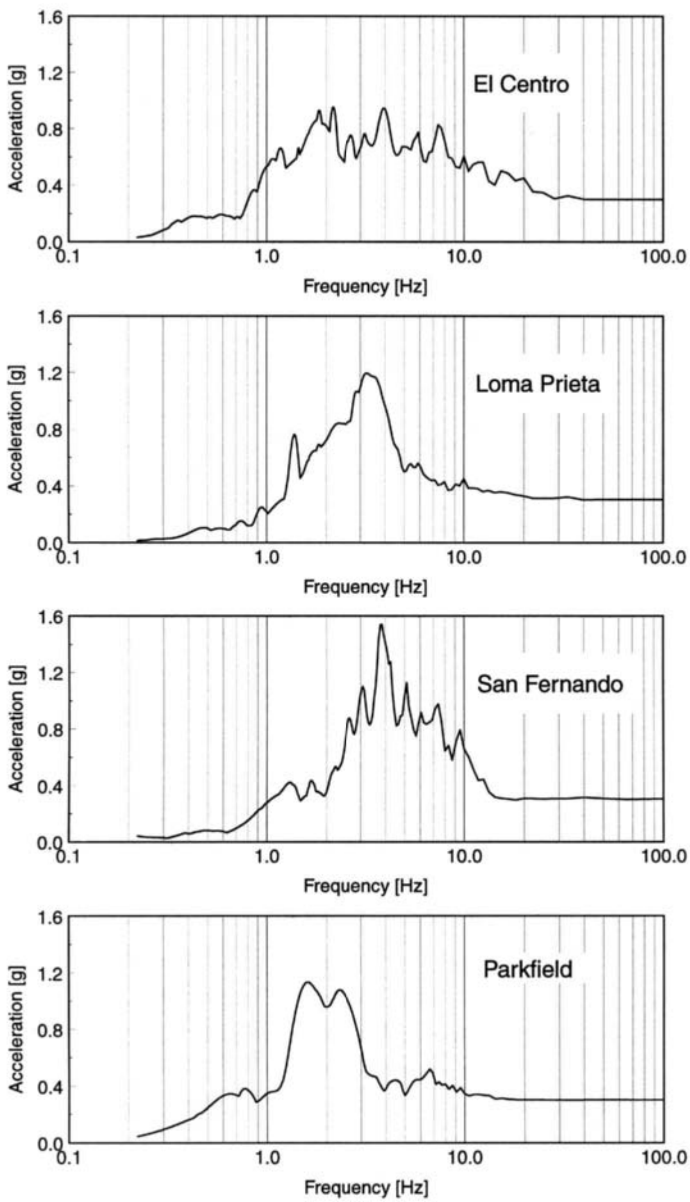

FIG. 1. Acceleration response spectra for El Centro, Loma Prieta, San Fernando and Parkfield ground acceleration records. (All normalized to 0.3g).

Table 1. Displacement and acceleration responses with and without active tuned mass damper for different earthquakes.

(a) Relative Displacements

	El Centro				San Fernando				Loma Prieta				Parkfield			
Floor	Response	Response Ratio			Response	Response Ratio			Response	Response Ratio			Response	Response Ratio		
	[cm]	TMD	CSMC	[4]/[3]	[cm]	TMD	CSMC	[8]/[7]	[cm]	TMD	CSMC	[12]/[11]	[cm]	TMD	CSMC	[16]/[15]
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]	[15]	[16]	[17]
1	**2.43**	0.66	0.27	0.41	**1.47**	0.77	0.45	0.58	**1.16**	0.96	0.63	0.65	**1.52**	0.92	0.62	0.68
2	**5.00**	0.65	0.27	0.41	**2.92**	0.80	0.47	0.59	**2.44**	0.96	0.61	0.64	**3.15**	0.91	0.62	0.68
3	**7.37**	0.63	0.26	0.41	**4.24**	0.79	0.48	0.61	**3.58**	0.95	0.61	0.63	**4.70**	0.91	0.62	0.68
4	**9.48**	0.62	0.26	0.42	**5.22**	0.78	0.50	0.64	**4.49**	0.95	0.62	0.65	**6.12**	0.89	0.61	0.69
5	**11.32**	0.61	0.26	0.43	**6.05**	0.73	0.50	0.68	**5.11**	0.94	0.64	0.68	**7.39**	0.88	0.61	0.69
6	**12.90**	0.61	0.26	0.44	**6.88**	0.73	0.48	0.66	**5.43**	0.94	0.67	0.71	**8.46**	0.87	0.61	0.70
7	**14.17**	0.62	0.27	0.43	**7.59**	0.73	0.45	0.62	**5.56**	0.93	0.70	0.75	**9.33**	0.86	0.60	0.70
8	**15.22**	0.63	0.27	0.42	**8.07**	0.75	0.43	0.58	**5.68**	0.98	0.72	0.73	**9.99**	0.86	0.61	0.72
9	**16.01**	0.64	0.27	0.42	**8.35**	0.77	0.42	0.54	**6.09**	0.98	0.68	0.70	**10.42**	0.86	0.63	0.73
10	**16.40**	0.65	0.27	0.42	**8.49**	0.79	0.41	0.52	**6.33**	0.98	0.66	0.68	**10.64**	0.87	0.64	0.74

(b) Absolute Accelerations

	El Centro				San Fernando				Loma Prieta				Parkfield			
Floor	Response	Response Ratio			Response	Response Ratio			Response	Response Ratio			Response	Response Ratio		
	[m/sec2]	TMD	CSMC	[4]/[3]	[m/sec2]	TMD	CSMC	[8]/[7]	[m/sec2]	TMD	CSMC	[12]/[11]	[m/sec2]	TMD	CSMC	[16]/[15]
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]	[15]	[16]	[17]
1	**3.17**	0.83	0.92	1.11	**4.02**	1.01	0.69	0.68	**3.04**	1.00	0.99	0.99	**2.88**	1.00	1.04	1.03
2	**4.09**	0.99	0.71	0.71	**4.82**	1.01	0.51	0.51	**4.30**	0.95	0.70	0.73	**2.88**	1.01	1.09	1.08
3	**4.73**	0.95	0.62	0.66	**6.19**	0.94	0.48	0.52	**5.30**	0.96	0.59	0.62	**2.97**	0.98	1.12	1.14
4	**5.00**	0.77	0.63	0.81	**7.39**	0.93	0.42	0.45	**5.72**	0.98	0.61	0.62	**3.35**	0.97	1.05	1.09
5	**6.00**	0.81	0.54	0.67	**6.50**	0.90	0.50	0.55	**5.86**	0.98	0.61	0.62	**3.81**	0.98	0.97	1.00
6	**6.78**	0.71	0.48	0.67	**4.27**	0.85	0.71	0.84	**4.71**	0.97	0.72	0.74	**3.97**	0.98	0.93	0.96
7	**6.31**	0.81	0.47	0.58	**4.66**	0.81	0.63	0.77	**2.88**	0.95	1.03	1.09	**3.92**	0.92	0.90	0.98
8	**6.68**	0.88	0.42	0.48	**4.61**	0.98	0.56	0.58	**3.49**	0.98	0.79	0.81	**4.16**	1.08	0.81	0.75
9	**7.78**	0.72	0.39	0.53	**5.60**	0.98	0.46	0.47	**5.30**	0.93	0.53	0.57	**4.67**	1.07	0.69	0.64
10	**8.99**	0.74	0.34	0.46	**7.63**	0.92	0.32	0.35	**6.78**	0.94	0.47	0.50	**5.07**	1.06	0.58	0.55

(3), (7), (11) and (15) across different earthquakes we observe that the presence of TMD may, in fact, increase floor accelerations for some input (in our case, column (15) under the Parkfield earthquake).

Comparison of normalized response in columns (4), (8), (12) and (16) which are all less than 1.0, we note that the active control, indeed, can reduce both acceleration and displacement responses. The effect of applying active control alone can be seen from the values presented in columns (5), (9), (13) and (17). Displacement response are reduced for all motions; again the most reduction is realized for the El Centro motion and least for the Parkfield motion. The control effectiveness with respect to the acceleration response is not uniform across various floors as well as different motions. In fact for some lower and middle floors, the acceleration responses have in fact been slightly increased. Perhaps inclusion of the acceleration responses in the objective function, which was minimized to define the sliding surface, may improve the effectiveness of the active control for reducing floor accelerations. Results similar to the one presented in Table 1 were also obtained for a sliding mass, but without any spring and damping elements. For the same mass as in the TMD, the response values were the same as in Table 1. However, the control effort, energy and power required in the two cases were different; the results for these quantities are presented in Table 3.

Table 2. Displacement and acceleration responses with active tendon control for different earthquakes.

(a) Relative Displacements

Floor	El Centro			San Fernando			Loma Prieta			Parkfield		
	Response [cm]	Response Fl. 10	Ratio Fl. 1	Response [cm]	Response Fl. 10	Ratio Fl. 1	Response [cm]	Response Fl. 10	Ratio Fl. 1	Response [cm]	Response Fl. 10	Ratio Fl. 1
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]
1	**2.43**	0.28	0.88	**1.47**	0.34	0.83	**1.16**	0.59	1.23	**1.52**	0.70	1.97
2	**5.00**	0.27	0.47	**2.92**	0.35	0.45	**2.44**	0.57	0.54	**3.15**	0.69	1.02
3	**7.37**	0.27	0.38	**4.24**	0.35	0.35	**3.58**	0.57	0.45	**4.70**	0.69	0.81
4	**9.48**	0.27	0.34	**5.22**	0.37	0.34	**4.49**	0.59	0.50	**6.12**	0.69	0.75
5	**11.32**	0.27	0.32	**6.05**	0.37	0.37	**5.11**	0.62	0.56	**7.39**	0.70	0.73
6	**12.90**	0.27	0.30	**6.88**	0.36	0.38	**5.43**	0.66	0.64	**8.46**	0.71	0.72
7	**14.17**	0.28	0.29	**7.59**	0.35	0.38	**5.56**	0.70	0.73	**9.33**	0.73	0.72
8	**15.22**	0.28	0.29	**8.07**	0.34	0.38	**5.68**	0.73	0.79	**9.99**	0.75	0.72
9	**16.01**	0.29	0.29	**8.35**	0.34	0.40	**6.09**	0.71	0.79	**10.42**	0.78	0.72
10	**16.40**	0.29	0.30	**8.49**	0.34	0.41	**6.33**	0.70	0.79	**10.64**	0.82	0.71

(b) Absolute Accelerations

Floor	El Centro			San Fernando			Loma Prieta			Parkfield		
	Response [m/sec2]	Response Fl. 10	Ratio Fl. 1	Response [cm]	Response Fl. 10	Ratio Fl. 1	Response [cm]	Response Fl. 10	Ratio Fl. 1	Response [cm]	Response Fl. 10	Ratio Fl. 1
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]
1	**3.17**	0.92	1.04	**4.02**	0.72	0.64	**3.04**	1.00	1.06	**2.88**	1.03	1.31
2	**4.09**	0.72	0.57	**4.82**	0.58	0.50	**4.30**	0.70	0.61	**2.88**	1.06	1.11
3	**4.73**	0.63	0.50	**6.19**	0.46	0.40	**5.30**	0.57	0.45	**2.97**	1.05	0.83
4	**5.00**	0.61	0.47	**7.39**	0.40	0.34	**5.72**	0.55	0.50	**3.35**	0.96	0.67
5	**6.00**	0.52	0.38	**6.50**	0.46	0.38	**5.86**	0.54	0.51	**3.81**	0.86	0.62
6	**6.78**	0.45	0.35	**4.27**	0.62	0.58	**4.71**	0.64	0.56	**3.97**	0.83	0.65
7	**6.31**	0.44	0.44	**4.66**	0.55	0.44	**2.88**	1.00	0.69	**3.92**	0.82	0.73
8	**6.68**	0.42	0.49	**4.61**	0.59	0.46	**3.49**	0.84	0.73	**4.16**	0.76	0.81
9	**7.78**	0.38	0.50	**5.60**	0.48	0.51	**5.30**	0.57	0.62	**4.67**	0.66	0.83
10	**8.99**	0.31	0.47	**7.63**	0.31	0.44	**6.78**	0.42	0.59	**5.07**	0.53	0.83

In Table 2 we present the results obtained for active tendon control forces applied at (1) the top floor and (2) the first floor. Both displacement and acceleration response are obtained for the four different input motions considered before. The responses with control actions are normalized by the response of the uncontrolled building. Again a ratio less than 1.0 is a measure of the effectiveness of the control action. It is interesting to note that tendon control is quite effective in reducing the displacement and acceleration response in most cases, although this effectiveness is not uniform for different inputs. In fact some acceleration response values may even be aggravated by the application of control action (values greater than 1.00 for the Parkfield motion, Column (12)).

Comparison of the results for the tendons as the top floor with those for tendons on the first floor indicates that the tendon on the top floor is clearly more effective. The response reducing effectiveness of the tendon on the first floor is also quite uneven over the height of the building.

Tables 1 and 2 only show that the active control can be effective in reducing the response. However, for practical applications it is more important to know what it will take to achieve this level of reduction in the response. For this, we present in Table 3, the maximum values of the mass drift, control force applied by the actuator and peak power required, both for tuned mass damper and the active mass (slider mass). Again, the results for the four input motions considered before are presented. First, we observe that the drift values are reasonable. The control force values are normalized by the weight of the top floor. A closer inspection of results showed that the force in active slider mass was nothing but the algebraic sum of the forces in the tuned mass damper spring, damping element and active control force applied by the actuator. We observe that the actuator will have to supply a larger force to a slider mass than to a tuned mass damper, except for the Parkfield motion where the two forces are nearly equal. Also the peak power requirement for a slider mass is significantly larger than that for a tuned mass. In this respect, it appears that it is more efficient to apply the force through a tuned mass damper than through a slider mass.

Table 3. Drift, maximum force and peak power requirements of a tuned mass damper and a sliding mass.

	Tuned Mass Damper			**Sliding Mass**		
	Drift [cm]	Control Force [ratio]	Max. Power [kW]	Drift [cm]	Control Force [ratio]	Max. Power [kW]
El Centro	60.32	0.63	3229.60	60.40	1.03	5794.90
San Fernando	33.70	0.51	2703.40	33.72	0.85	4449.40
Loma Prieta	40.66	0.64	4389.80	40.70	0.99	5091.20
Parkfield	90.14	1.18	6866.30	90.27	1.12	11073.00

In Table 4, a similar comparison is made between the active tendon force applied at the top and at the first floor levels. As one would expect intuitively, the tendon force applied at the top is much more efficient than the force applied at bottom as the maximum force, and peak power values are much smaller for the top floor than for the first floor.

Besides these comparisons of various schemes of applying forces, we must also closely examine the feasibility of providing these high forces and peak power demands. Even the smallest force value (51% of the top floor weight, in Table 3) seems to be too high for practical application. Some peak power values indicated in Table 3 and 4 also seem to be too high to be realistically available during a strong seismic event. The direct application of control schemes, therefore, seems impractical for any reasonable size building. For a building larger than the example problem considered here, the force and power requirement are likely to be even more impractical.

This, however, does not mean that all structural control schemes are impractical. It appears that the schemes which could utilize the strain energy of the structure to impart these large counteracting

Table 4. Maximum force and peak power requirements in tendons attached to the top and first floors.

	Active Tendons: Top Floor		**Active Tendons: First Floor**	
	Control Force [ratio]	Max. Power [kW]	Control Force [ratio]	Max. Power [kW]
El Centro	0.98	1240.10	3.60	1769.00
San Fernando	0.79	520.40	2.22	820.43
Loma Prieta	1.04	1108.00	2.76	1294.50
Parkfield	1.53	2373.30	5.47	2623.00

forces or which could dynamically change the distribution of the damping forces are likely to be more successful for practical applications. These schemes fall in the domain of parametric control where one would change the stiffness of the system dynamically by locking and unlocking bracings and change the damping characteristics by altering damping parameters.

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RETROFIT FOR EXISTING BUILDINGS WITH SEISMIC CONTROL OF STRUCTURES

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ABSTRACT

The technique of seismic control of structures can be used not only for new buildings to prevent damage from strong earthquakes, but also for retrofit of existing buildings very effectively to restore and improve a building's anti-earthquake ability. This paper describes the present status of research and application on seismic structural response control in China. Also the paper introduces some research results from the work of others in these fields, including seismic isolation systems, energy dissipation systems, TMD passive control systems, active and hybrid control systems. Future development of seismic structural response control in China is discussed.

KEYWORDS

Seismic control; retrofit; existing building; base isolation; sliding layer; dry friction isolation; energy dissipation.

INTRODUCTION

During earthquakes a structure which is fixed on the ground will respond gradually, increasing from the building's bottom (ground) to the building's top, like an "amplifier". This will result in damage or collapse of the structure or in damage of nonstructural components, decoration and facilities in the building due to large response of the structure. In order to reduce this response and avoid damage to the structure, traditional methods are to increase structural ductility and allow structural elements or joints to work in the inelastic range to dissipate the energy of a structure under earthquakes, thereby reducing the structural response, by forming an "inelastic structure system". This is the general structural system for earthquake resistance in many countries, including China at present.

But the usefulness of this traditional system is limited or in some cases, it is not very safe. First, it is difficult to control the structural damage level due to inelastic deformation, and it may be quite dangerous in a severe, unpredicted earthquake. Second, it can not be used in some important structures whose elements are not allowed to work in the inelastic range, such as some buildings whose decoration is very expensive, nuclear power plants, museums, and so on. Third, it is not possible to be used for buildings which contain precise instruments.

China is very frequently subjected to severe earthquakes every year and may have several very strong earthquakes in coming years depending on seismic prediction. Because of the huge population and speedy development of the economy in China, about 800 million square meters of civil buildings are built every year in seismic zones in mainland China. To find a more safe and suitable way for earthquake-resistant structure systems in buildings is an urgent goal. Structural control systems provide an advanced means toward this end.

Because the traditional method is not satisfactory in many cases for earthquake resistance, it is limited in the future although it is very common at present. China has paid more attention to some potential new systems for earthquake-resistant structures: base isolation, energy dissipation and structural control system. Some research results have been obtained, and some systems have been used successfully in engineering application in China (see Zhou, 1989, 1990a).

BASE ISOLATION SYSTEM

A history of the application of seismic isolation in China proves that the isolation system is more safe, economic and simple than the traditional structure system, specially suitable for use in cases where earthquakes are unpredictable. In mainland China, about 40 existing ancient buildings, towers and temples built thousands years ago with the isolation concept have successfully withstood several strong earthquakes. Two masonry buildings with sliding layer of asphalt surprisingly avoided damage in the 1976 Tangshan earthquake which ruined almost all buildings in Tangshan City and killed about 400,000 people. In the past 20 years, there were 15 buildings and 21 bridges built with base isolation, in which isolators were rubber bearings, sand sliding layer or graphite layer. Some different kinds of isolation buildings are under construction now.

In order to solve some problems for a wider application of isolation in China, two 8--story R.C. frame dwellings are to be built for full-scale tests. Both structures are the same, but one was base isolated with reinforced rubber pads, the other was base fixed. To coordinate the tests of these two buildings, a great number of low-cycle fatigue tests for isolators and shaking table tests for building models were conducted.

Based on the applications, testing results and analysis, a set of optimal design methods, computer programs and design rule were raised in China.

Recent Design and Application of Base Isolation System in China

In general, the base isolation device combining with energy dissipation device is required to possess three basic characteristics (see Zhou 1990b).

(1) Soft sliding: The structure can softly slide on the base in severe earthquakes. This characteristic can isolate the horizontal vibration from ground motion to structure, making the natural period of the structure long, and then reduce the acceleration response of the structure effectively.

(2) A certain amount of damping: This will dissipate the energy input to the structure, and then attenuate the response of the structure in an earthquake.

(3) Suitable Horizontal Stiffness: This will provide the primary stiffness in a wind load or minor earthquake.

There are four kinds of base isolation and energy dissipation systems for application:

(1) Rubber pad (steel plates reinforced) as isolator.

(2) Sliding pad as isolator, combined with steel elements as energy dissipater.

(3) Roller as isolator combines with steel elements as energy dissipater.

(4) Friction layer or sand (or other material) sliding layer as isolator and also energy dissipater.

One 8--story concrete frame house with base isolation using rubber bearings was constructed in Shantou City in south China (Fig. 1). Another 3 buildings with rubber bearings have been built and 8 buildings with rubber bearings will be built in south and north China. Most of isolation buildings built now have used the rubber bearings made by Shantou Industry Company of Vibration Isolation Product in China which are helped by UNIDO, MPRPA and EERC. Using these types of rubber bearings can assure the building safe in any strong earthquake, and it can reduce the cost of buildings about 5--20 % compared with the traditional fixed base building. So more and more buildings in all China will use these types of rubber bearings (see Zhou 1994).

Four 4--7 story brick buildings with sliding layer as isolator and steel elements as energy dissipater have been built in west China (Fig. 2, 3). Five low-rise brick buildings with sand sliding isolation have been built in north China (Fig. 4).

Fig. 1 An 8-story RC frame building with reinforced rubber pads

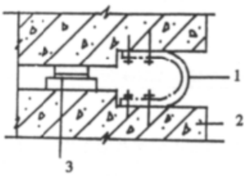

Fig. 2 Sliding layer

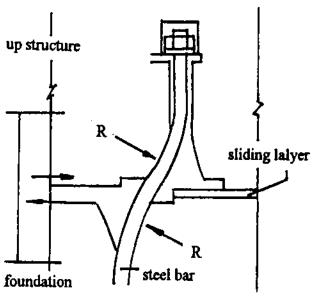

Fig. 3 Dry friction isolation

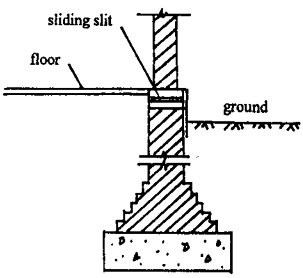

Fig. 4 Sliding slit at bottom of building

Testing and Analysis for Base Isolation System

Many tests have been conducted and some calculations for a theory of a seismic isolation system have now been established in China. The tests include two kinds of work.

(1) Tests of isolators and energy dissipaters: Static tests of full-scale elements, pseudo-static tests, low-cycle fatigue failure tests. A series of pseudo-tests were finished for rubber bearings, sliding layer, curved plates and friction layer.

(2) Shaking table tests for large-scale structural model: One 6-story steel frame model is tested on the shaking table. Test results are shown in Fig. 5. Another series of shaking table tests was also carried out in China: a 1/5-scale model structure with sliding layer and steel curve plate and some brick-building models with a layer.

Test results show that acceleration responses on each story of structure model are nearly the same. Thus the elements and joints of a structure with base isolation work within the elastic range. The acceleration response of a structure with base isolation is only 1/2 -- 1/10 the response of a structure fixed on the shaking table. This means base isolation is more effective to attenuate the structural response in an earthquake than any other method.

In order to investigate the effect of a base isolating system for high-rise buildings, a 1/15 scale model of a 9-story frame structure was tested on shaking table. Figure 6 shows the test records which relate to shaking table vibration with Freq.w =15.5 HZ and maximum acceleration of shaking table $\ddot{X}$ = 0.49g. Analyzing these results indicates that: (1) acceleration response at the top story of the structure model with base isolation is only 0.05g while the acceleration response at the same story with fixed base reaches 1.13g. The ratio of $\ddot{X}$ (isolation) / $\ddot{X}$ (fixed base) = 0.05g / 1.13g = 1 / 22. Thus base isolation is significantly effective to attenuate the structure response for high-rise buildings during severe earthquakes; (2) acceleration responses at each story of the structure model with base isolation are nearly the same. Thus the elements and joints of a structure with base isolation will work within the elastic range during an earthquake.

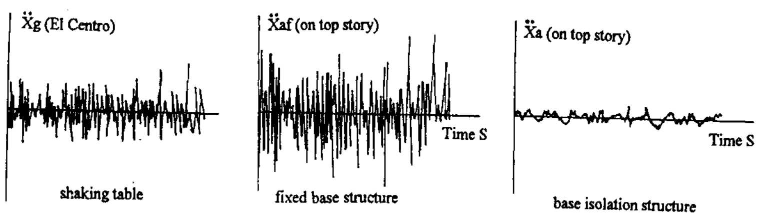

Fig. 5 Test results of shaking table with base isolation and fixed base model

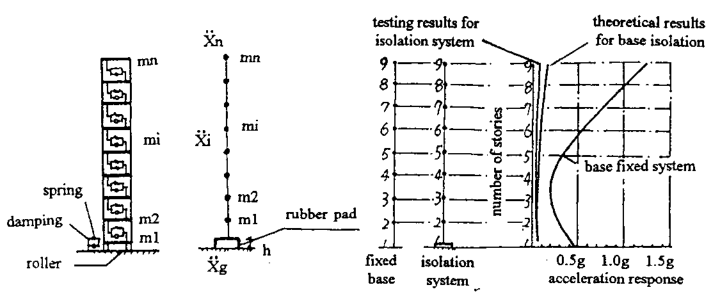

Fig. 6 Shaking table tests for 9-story building model

Calculation Theory of Base Isolation System

From a mathematical model, the basic differential equation of motion is given as:

$$M\ddot{X}_a + C_e\dot{X}_a + K X_a = C_e\dot{X}_g + K X_g \tag{1}$$

where M is the structural mass. C_e is the equivalent viscous damping of isolation and energy dissipation systems. K is the elastic stiffness of isolation and energy dissipation systems. X_g, $\dot{X}_g$, $\ddot{X}_g$ are the ground response of displacement, velocity and acceleration, respectively, in an earthquake. X_a, $\dot{X}_a$, $\ddot{X}_a$ are the structure response of displacement, velocity and acceleration, respectively, in an earthquake.

Another basic differential equation of motion can be written:

$$M \ddot{D}_u + C_e \dot{D}_u + K D_u = - M \ddot{X}_g \qquad (2)$$

where D_u, $\dot{D}_u$, $\ddot{D}_u$ are the maximum relative displacement, velocity and acceleration, respectively, between structure and ground.

Solve this equation with finding transfer function and get:

$$Du = X_g \sqrt{(1 - AR^2) (W/ Wn)^2 / (W / Wn)^2 - 2} \; /W^2 = X_g \cdot r \qquad (3)$$

$$AR = \ddot{X}_a / \ddot{X}_g = \sqrt{(1 + 4B^2) / \{4B^2 + [1 - (W / Wn)^2]^2 \}} \; \sqrt{W / Wn} \qquad (4)$$

Define W_n and W as the natural frequency of base isolation system and ground motion. Define $B = 2 (1 - 1/ U) / U\pi$ as energy dissipation damping ratio, $U = D_u / D_y$ as ductility factor, where D_u and D_y are relative displacements at ultimate point and yield point where $AR = \ddot{X}_a / \ddot{X}_g$ is called acceleration attenuation ratio of system. Comparing the theoretical values D_u and AR from Eq. (3) and (4) with the measured values approaches 1.0. Thus Eq. (3) and Eq. (4) give a reasonable estimation. The author has also compiled a program EBIS-1 for design in engineering application.

ENERGY DISSIPATION SYSTEM

Design and Engineering Application

There are two kinds of energy dissipation systems to be used.

(1) Energy dissipation components: These include energy dissipating bracing or energy dissipating shear wall.

(2) Energy dissipating damper: These dampers may be fixed in a certain position (such as on bracing, shear wall, joints).

An industrial structure with energy dissipating bracing and another 28-story high-rise building with energy dissipating bracing using friction layer were built in Guangzhou City in south China (Fig. 7).

Tests and Research on Energy Dissipation System

In order to evaluate the behavior of this new bracing system, 5 sets of full-scale models of this bracing were tested (Fig. 8). Base on the test results, the authors suggest some calculations for design (see Zhou, 1989).

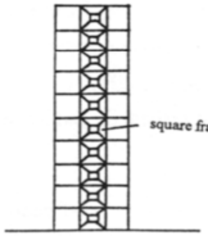

Fig. 7 Energy dissipation bracing for high-rise building

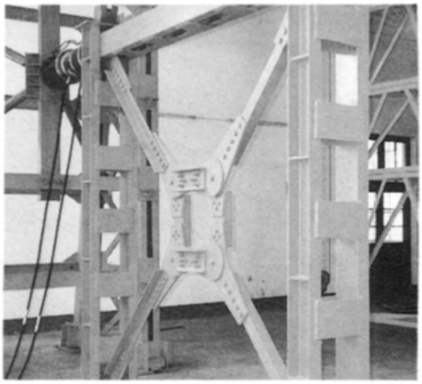

Fig. 8 Tests for full-scale model of friction bracing

Because steel elements were used as energy dissipaters, these may deform into plastic regions withstanding a large number of cyclic loads, and may fail due to the low cyclic fatigue in an earthquake. The permanence of resisting low-cycle fatigue failure can be represented by a parameter, number of loading cycles to fail, N. Many works indicate that N mainly depends on the absolute maximum strain value ε on the surface of mild steel elements in cyclic loads. So the relationship between ε and N is the key to predict low-cycle fatigue failure for mild steel elements.

In order to get accurate results, a series of X - shaped plates made of mild steel was chosen as the main specimens to be tested because vertical strain of the section is nearly uniformly distributed during horizontal bending under horizontal loads at the top or bottom of the specimen.

From statistical analysis (see Zhou, 1988, 1992), a theoretical curve to represent the relationship between ε and N was expressed by:

$$\varepsilon = 0.22 / N \tag{5}$$

where ε is maximum strain on surface of steel elements and N is number of loading cycles up to fail.

Results of tests and analysis indicate that the theoretical curve is very close to the test records, especially in the strain range ε = (0.5 - 3.0) % which covers general cases. Theoretical N values from the theoretical curve are always smaller than test values. Thus conservative results appropriate for design can be obtained from Eq. (5).

Design and Calculation of Energy Dissipation System

For high-rise structures with energy dissipating bracing, the energy balance equation any instant of time during an earthquake is:

$$E_{in} = E_p + E_k + E_p + E_b \tag{6}$$

where E_{in} is energy input to the structure, E_p is potential energy in structural vibration, E_k is kinetic energy in structural vibration, E_d is energy dissipated by viscous damping of structure, E_b is energy dissipated by bracing system (energy dissipater). Research indicates that the energy dissipater can dissipate about 90% of the total energy input at the end of an earthquake. So some items whose effect is relatively small can be neglected in Eq. (6). Then the energy dissipating design for earthquake resistance can be satisfied with:

$$E_{in} < E_b \tag{7}$$

For calculating energy input E_{in}, the system can be considered as a multi-degree-of-freedom system. Energy dissipated by bracing system E_b depends on the area enclosed by load-displacement loop curve (see Zhou, 1992).

TMD PASSIVE CONTROL SYSTEM

This system, in a certain position (such as on the roof), adds a filial structure (which possesses mass M, stiffness K and damping C) to change the dynamic characteristics of original structure. During an earthquake, the filial structure moves against the direction of the original structural vibration and reduces the response of the original structure. This filial structure is called TMD (tuned mass damper). Since this system operates without any outside energy sources, it is also called a passive control system.

This system can effectively reduce the response of a structure. It is safe and economic. It has been used in earthquake (or wind) resistant high-rise buildings, tower structures and some large span structures. One 192m-high TV tower with TMD was designed in south China. There are now three kinds of TMD under consideration in China.

(1) Supporting TMD (Fig. 9.a): The filial structure is supported on the roof or other places in the original structure. The mass can move in bilateral direction.

(2) Hanging TMD (Fig. 9.b): The filial structure is hung at top of the original structure. The mass can sway in any direction. In some cases, the water tank on top of the building may be designed as a filial structure hung at the top of the building.

(3) Impacting TMD (Fig. 9.c): The filial structure is a weight hung at the top of a structure. During an earthquake, the hanging weight impacts the original structure and reduces the response of the structure.

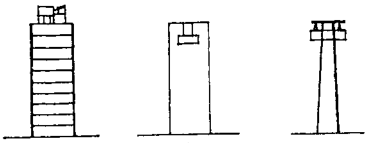

Fig. 9 TMD passive control system

ACTIVE AND HYBRID CONTROL SYSTEMS

The system, in a certain position, places an additional filial structure which operates by computer control and immediately changes the dynamic characteristics of the structure to reduce its response during an earthquake. Since this system is controlled by outside energy sources, it is called an active control system.

Three kinds of system are being studied in China: (1) A filial structure, which possesses mass, stiffness and damping, called AMD (active mass damper). (2) A filial structure, which is a bracing, called active control bracing. (3) Hybrid control system.

A hybrid of active control bracing with passive energy dissipation system was designed for a 68-story high-rise building in south China. Some tests are now being done for active control and hybrid control systems in China.

FUTURE DEVELOPMENT OF EARTHQUAKE-RESISTANT STRUCTURES IN CHINA

Traditional earthquake-resistant structure systems and design methods have resulted in many problems in application. Since the intensity of earthquakes is always uncertain, ground motion and structural response are complicated, and the requirements for modern structures are more and more strict. Traditional design methods for earthquake-resistant structures are not satisfactory for current use. A new system will improve structures in seismic zones. This new system is safer, more effective, simpler and more economical. Many earthquake engineers and scientists predict: " The coming decade will mean rapid development in base isolation, energy dissipation and control systems in China!"

ACKNOWLEDGMENTS

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DISCUSSION ON RECONSTRUCTION AND RESTORATION OF PETROLEUM ENTERPRISES IN EARTHQUAKE DISASTER AREA

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ABSTRACT

Damage suffered by petroleum enterprises after Haicheng and Tangshan earthquakes is reviewed. In order not to repeat such a disaster, comprehensive anti-seismic measures should be taken. Restoration and reconstruction of petroleum enterprises discussed herein were designed to strengthen their anti-seismic capability.

KEYWORDS

Earthquake disaster; reconstruction; restoration; petroleum enterprise; damage investigation; oil-gas pipeline; oil tank; lifeline system.

INTRODUCTION

Natural disasters are a great threat to mankind's existence. An earthquake disaster is more sudden and widespread than other disasters. China is a country of frequent earthquakes. A major earthquake not only causes instant damage to many buildings, but also causes a series of secondary disasters such as fire, flood, pestilence and environmental pollution. Also influenced are such factors as social psychology, economics, and public security. Although earthquakes cannot be controlled, the extent of their disaster may be reduced by various measures as part of a comprehensive approach.

Petroleum is a major industry in the national economy. Its characteristics include broad distribution, long assembly line, self-generated community and population concentration. It involves large-scale engineering systems such as oil-gas transportation and waste water disposal. These facilities are concentrated, and pipelines are crossed in length and breadth. They are continually operated in an environment with hazards of fire, explosion and solidification. Once enterprises are struck by a major earthquake, secondary disasters, as noted above, occur and production stops.

Other characteristics of petroleum enterprises are lifeline engineering (including water, power and heating supply, communication, traffic and medical facilities) located in various parts of oil field production and its community. When an earthquake occurs, obstacles are presented at certain points. If seismic intensity is lower, oil production is affected; if higher, the lives of people as well as production cease to be normal. Because a number of buildings collapsed, many people were killed or injured. It is even more serious that a strong earthquake causes secondary disasters in petroleum enterprises. Haicheng earthquake (Ms = 7.3, 1975) and Tangshan earthquake (Ms = 7.8, 1976) disrupted the petroleum industry. After Haicheng earthquake occurred, there were twenty-nine breaks in ten oil-gas pipelines in Liaohe oil field. Without oil-gas transport, Anshan steel corporation had to stop production. Due to oil wells closing and pipelines breaking, the quantity of Liaohe oil field production fell that year. Tangshan earthquake damaged a lot of petroleum installations as follows: 14 oil tanks along with well equipment were destroyed in Liaohe oil field; two places in the Qinhuangdao-Beijing oil transport pipeline ruptured; pipeline across Liaohe was also destroyed so that oil transport stopped for 11 days; flow loss of crude oil was 10 thousand tons, which caused serious pollution of the environment and brought vast economic loss.

It is a common desire of people in a disaster-stricken area that production be restored and homes reconstructed as fast as possible. Restoration refers to local repair and modification of focal points on the original plan. Reconstruction refers to renewal of construction after a disaster.

POST-EARTHQUAKE RESTORATION OF PETROLEUM ENTERPRISES

Restoration of Basic Needs

After an earthquake, such items as food, clothing, shelter, transportation and medicine must be restored so that people can function. Some measures are as follows.

It is necessary to strengthen leadership and unify direction. Departments such as planning, financial affairs, management, supplies, capital construction, medical facilities, transportation, oil construction, water and electricity, and communication should be organized by a director, forming groups to solve various problems by division of labor and regional as well as individual responsibility.

Construction brigades should be organized by departments of oil production, water and electricity supply, and communication system to construct temporary sheds and repair partly damaged buildings.

Vehicles and vessels should be repaired as fast as possible to restore loading capacity and ensure transport of goods and materials.

Temporary supply centers and medical facilities should be established.

Earthquake Damage Investigation

In order to accumulate data, it is necessary to investigate damage soon after an earthquake. An investigator group makes a video record of earthquake damage in disaster area, which is organized by the informational department of enterprise. Principle of the investigation is that the most important damage is investigated first, then the more general. Procedure of the investigation is as follows.

Isoseismic lines are plotted and earthquake damage distribution is mapped.

Damage to oil store-transportation systems, important oil tanks, reservoirs, and lifeline engineering systems is investigated, and secondary disaster potentials are reviewed.

Damage to construction is investigated. According to characteristics of the structure and type of construction, earthquake damage is described. This provides a basis for reconstruction, reinforcement, and anti-seismic design of new construction. Based on characteristics of structures, they are classified into masonry, masonry mixed, reinforced concrete and steel structures. Types of construction are divided into single story, multistory brick, reinforced concrete frame, complete-frame base-story, multistory interior frame, single story plant buildings, chimney and water tower, etc.. Degree of construction damage is divided into five grades, that is, basically no damage, slight damage, moderate damage, severe damage and collapse.

Restoration of Production

A unified management system is set up in the field. In order to ensure orderly development of post-earthquake restoration works, it is necessary to exert unified leadership.

Overall restoration work is led by the director of enterprises, the management organization of restoration production consisting of the departments of planning, financial affairs, capital construction and anti-earthquake. It is required that personnel in a specific field divide up the work and assign a portion to an individual or group. It is also essential to make sure that the approach is practical and realistic, to set up projects for restoration, to plan overall investment, and to distribute limited manpower and material. This distribution follows the principles of urgent need, to handle the easy first and then the difficult, to do quick repair while clearing and restore while producing.

In order to repair and strengthen construction, industrial equipment and facilities, the design department must develop appropriate plans.

Equipment department organizes the supply of facilities, and raw material should be fully utilized.

Construction teams are organized by departments of oil construction, water and electricity, and communication to perform the above-mentioned tasks.

Water supply and electricity should be recovered first, because they are necessary conditions for production. Restoration of water supply and electricity depends on pipeline material and degree of damage with the corresponding method of construction. If a pipe is broken, the damaged segment should be cut off, then added short pipe and casing using quick-drying cement to fill joints. Thus the water supply is restored rapidly. Electrical lines should be quickly repaired.

Storage and transportation systems for oil-gas, vital stations and reservoirs, and lifeline systems should be restored so that production can resume as soon as possible.

Residential construction should be restored according to the principle that easier repairs are made first, and then further improvements.

POST-EARTHQUAKE RECONSTRUCTION OF PETROLEUM ENTERPRISE

When a petroleum enterprise is reconstructed after an earthquake, it is necessary to consider full measures against disaster to improve the comprehensive anti-earthquake capability of enterprises.

When a petroleum enterprise is reconstructed, a plan should be established first. It should include the following: overall plan of mine region reconstruction, reconstruction subplan for respective systems, budgetary estimate included materials required, and a schedule.

Reconstruction should be advantageous to the economy, daily life, environment, and earthquake protection. It should also make rational distribution of resources and facilities.

In order to improve comprehensive anti-seismic capability of petroleum enterprise, a post-earthquake reconstruction plan should include the following items.

Reassessment of Basic Seismic Intensity

It is possible that the real seismic intensity of an earthquake differs from the zoning figure. The latter is the basis of anti-earthquake fortification standards during reconstruction. Basic intensity directly affects reconstruction scale and the development of new construction engineering. Thus it must be checked.

Overall Plan of Reconstruction

Layout of mine regions is regulated and scale of construction is strictly controlled. As noted, secondary disasters may occur after an earthquake, such as fire or explosion. As a counter-measure, warehouses for hazardous material are built leeward of residence districts, and far away from the latter.

These warehouses should not be located on an active fault, flood plain, potential liquefaction soil or other unfavorable sites.

Construction density and height should be in accordance with the building design code. When residences are constructed near tall buildings, a safety separation zone should be created, and narrow roads widened for emergency evacuation in case of an earthquake.

Planting and beautification should be a priority. Greenery not only can purify the air and beautify the environment, but also help evacuation during an earthquake. Open fields can be safe places for escape routes. The amount of land for fields to escape earthquakes should follow a per capita standard. Natural setting should be considered as much as possible to avoid unnecessary construction in green areas.

Medical facilities and fire stations should be strengthened. The goal is to treat the injured and control fire in time to save lives and property.

Anti-earthquake districts for self-help should be drawn. People may then be able to rescue each other more effectively.

The following departments and organizations should be set up in self-help districts:

a. Leadership (unified): The Party and administrators of an enterprise should be in charge of rescue efforts.

b. Emergency medical facilities: Treatment should be properly distributed and division of labour with individual responsibility for rescue areas distinctly made.

c. Fire defense team: All the objectives of firefighting should be incorporated into a fire defense plan.

d. Places for avoidance of earthquakes should be arranged as necessary.

e. The storage material: Restaurants, shops and warehouses of each self-help district should be used.

f. Rapid-repair engineering team: This team covers house management, repair, and capital construction. Its main task is rapid repair of oil, gas, water, electricity, communication, roads and bridges.

Reliable Preventive Measures for Oil-Gas Storage and Transportation System

Oil-Gas Pipelines. In general, pipelines are underground and easily corroded. Because oil is widely distributed, pipelines pass through sites under complicated conditions. When pipeline paths are selected, districts of soil liquefaction, landslide and active faults should be avoided. If a liquefaction district is not avoided, floating resistance measures should be taken for pipelines. If pipelines have to transverse a landslide area, a stability should be assessed. Measures are to be adopted in line with local conditions such as draining water, adding retaining walls and piers, anchoring staff, reducing slope for landslide, improving property of rock and soil, and unloading. When pipelines go under rivers and lakes, they are best laid by reducing slope, to dip generally no more than 30 degree. Entry and exit points for gas as well as the starting of point of lines should be exposed in high and low areas; collection points of oil pipelines in an earthquake area should be buried. If pipelines cross through the base of construction, a certain distance from the foundation should be maintained. There a trench or platform should be used. Soft metal pipe or flexible connectors to compensate for bending should be used where pipes go above or below the surface, where there is a bypass value or connection of facilities, and where soft and hard soils intersect.

Oil Tanks. Ground parameters provided by microzoning of earthquakes should determine the design and calculation of earthquake resistance for tanks. Analysis of dynamic reliability is proposed for them. If an oil tank is utilized for five years, thickness of the tank wall should be measured to discover corrosion and to take anti-seismic measures.

Soft metal pipe or bend compensators are installed in the inlet and outlet segments of an oil tank to prevent rupture from no synchronous deformation. The valve housing of an oil tank should be a light flexible structure, and installation of valve housing separate from the outer wall of the oil tank should be part of reconstruction. When light oil flows into the tank, leach water or other temperature reduction methods should be used during the summer, in hot regions.

Districts for Oil Tanks. Design of embankment for oil protection in a tank district should prevent any leakage after oil flow. It is required for this embankment to be fire-proof. It should be entirely made of soil or rubble or brick. Each type of embankment has to be sound, airtight and solid. Any water wells and sluice plates should be put in drain facilities of an oil tank district. Measures for recovery of oil should be strengthened so that oil cannot leak and cause fires. Tank districts should be constructed in leeward areas, not on unstable ground or steep slopes. Zones to separate tank and production facilities should be set up according to the fire safety code. No residential areas should be put in this zone. Roads should encircle tank districts to allow fire trucks free access.

Reconstruction of Lifeline Systems for Petroleum Enterprises

Water Supply. Resources of water from various regions are used to maintain an adequate supply. A flexible connector is used to fill the pipes. In a district of high demand for water, a self-sufficient source is used. Hardware for drawing water or underground firefighting are important to have.

Drainage. A method for separating rain from polluted water should be used. Let rain-water drain into a natural basin along a drainage system, channeling polluted water into a treatment site for the sake of the environment.

Electricity. Multiple power supplies and circular circuitry are used to supply electricity. Basic construction such as transformers, distributing substations, control and regulatory buildings, should follow criteria designed strengthen anti-earthquake capacity. A safe supply of electricity should be provided for economic productivity and public construction as well as organization of enterprises.

Communication. By means of wires with radio communication, people can contact each other. The apparatus should be set up in two houses. Underground cable is used for wired circuits, with multiple exits and diverse connections which become a radiating network. This network should be cleared during an earthquake.

Heating. A concentrated supply of heat can be achieved by extending and reconstructing the original boiler room.

Roads. In enterprise areas main roads should be widened, trunk roads increased, crooked portions straightened, T-shape roads made into four-way crossings, exits added, and a road network is set up to cover all directions. Furthermore, the anti-seismic capability of bridges should be ensured.

Reconstruction of Buildings

Construction System. Selection of a construction system is closely related to implementation of the reconstruction plan. Criteria involve local character and specific conditions. The current code of national anti-earthquake design is strictly applied. This code specifies no damage during a small earthquake and no collapse during a large earthquake. Plane layout should strive for homogeneity to avoid torsion.

Anti-earthquake design should be combined with standards of national construction. Note that building material should be easily obtained and conveniently constructed. A contradiction between anti-earthquake safety and economic benefit can be resolved by the future importance of this investment.

Emergency Shelter. Provision of sufficient heat and fire safety should be made for this kind of housing. Note that the proper site should be selected to prevent temporary shelter from becoming permanent and from obstructing implementation of the overall enterprise plan.

CONCLUSIONS

Earthquakes are difficult to forecast with precision due to the current level of science and technique. Nevertheless an earthquake disaster can be mitigated by a comprehensive disaster plan for oil enterprises. Anti-earthquake capability can be improved and the magnitude of secondary disaster can be controlled. We must coordinate a policy of anti-earthquake work covering all aspects of disaster.

REPAIR AND ITS EFFECT ON EARTHQUAKE DAMAGED PIER IN HARBOUR

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ABSTRACT

Tangshan earthquake affected Tianjin and Beijing regions. Tianjin port incurred damage and loss to different degrees, in particular the berth facilities. This paper describes the setting and structure of the piers, analyses the main earthquake damage and introduces repair measures and their effects.

Two site investigations were conducted. After the earthquake, the piers were repaired and continue to operate some 18 years later. The piers remain in normal use and have not been damaged again. Structural resistance of piers in seismic regions is noted.

KEYWORDS

Earthquake-damaged pier; shore slope; harbour; pier; repair and strengthening; Tianjin port; platform.

INTRODUCTION

A severe earthquake which reached 7.8 magnitude on July 28, 1976, in the district of Tangshan, Fengnan affected Tianjin and Beijing regions.

Tianjin port, the communication pilot of water and land in the northern part of China, incurred some degree of loss, particularly to berth facilities. Damage was heavy because the piers had not been designed as earthquake-proof structures.

Under the leadership of the Ministry of Communications, the relevant departments of engineering, design, and research cooperated to investigate the earthquake damage of Tianjin port. A full-scale plan for repair and reinforcement was implemented.

Setting and Structure of Piers in Tianjin Port

Tianjin port is situated in an alluvial plain at the mouth of a river. Its geological age is the fourth complete generation. The top 20m is recent sedimentation and the nature of the soil is poor. Under the surface, a 14m soil layer is saturated muddy soft clay and underneath this layer is clay-like loam, with sand under -20m. The muddy soil has such engineering characteristics as moisture content higher than liquid limit, low strength, high compressibility and poor permeability so it must be treated before it is used. Owing to lack of sand and stone in this area, most piers were made as unique high-pile beam-plate structure.

Since 1949, numerous deep-water piers have been built in two types, according to the shore-connecting structure of the piers. One is "rear sheet-pile" type and the other is "rubble mound" type. Shore slope is usually 1:3 ~ 1:4, as required by the stability of the river bank and there is a 45m horizontal distance. Hence, a wide-platform structure is always selected which is divided into front and rear platforms.

Front platforms are approximately 14m wide with assembled integral superstructures. Underneath the deck is pre-stressed pile. Rear platforms are 30m wide with simply supported structures underneath which are also pre-stressed pile. Construction joints are made between the front and the rear platform as well as between the rear platform and shore-connecting structure. Front platforms are essentially used for berthing, loading and unloading. Rear platforms are used for cargo transfer handling and temporary storage.

Analysis of Earthquake Damage to Piers

For a general view of the piers, total length is 4089m, of which 27.2% was seriously damaged but without devious deflection of the integral structure, 32.3% was moderately damaged but without failure in the superstructure, 40.5% was in fair or good condition.

Slippage of the slope bank under the deck plate was not obvious, but partial deflection existed. Deflection in pier structures was minor and water depth in front of piers had not obviously changed. Some retaining wall of rubble mound type shore-connecting was lower and elevation difference from the platform is about 40-50cm. With rear sheet-pile type shore-connecting structures, deflection of the sheet pile increased and concrete in the landward side was compressed. Fractures were generated and enforcement was also exposed. Land surface behind the pier was lower in general by about 15-20cm and in some places the subsidence was about 50cm.

Damage to super beam-plate structure on pier was caused on either side of construction gap by impact on the concrete of beam and plate. Front and rear platforms were fractured by compression. Adjacent platforms shifted in relation to each other, causing the railway which lay on the plate to deflect in an 's' shape.

Pile damage was heavy. Some cracks appeared (particularly in the ring) ranging from the pile head to 1.5m below it. Breadth of cracks was between 0.1~0.5mm. At the landward-side pile, the inside concrete was compressed to fracture and outside concrete was tensioned to fracture. Raking pile caps were generally damaged at the upper middle part of the cap. Fractures ran from the top of the compressible pile downward in a diagonal direction to the tensile pile. In the joint of some pile caps and super beam-plate, a gap was left.

Damage to vertical piles under the deck plate was slight. For a few berths, overloading the plate with stones caused cracks at the top of the pile within a 1m radius. Damage to reinforcement concrete raking cap and pre-stress concrete inclined piles is listed in Table 1.

Table 1. Damage to Concrete Raking and Cap

Berth	**Raking**	**Cracked pile**		**Broken pile**		**Cracked Raking pile**		**Broken Raking pile**	
No.	**No. pair**	**No.**	**%**	**No.**	**%**	**No.**	**%**	**No.**	**%**
5	50	30	30	20	20	33	65	12	24
7	34	2	3			3	9		
8	27	4	7			22	81		
9	32	25	39			4		31	97
10	27	26	48			7	26	20	74
11	27	38	70			14	52	11	41
12	36	17	24	6	8	5	14	8	23
13	39	46	50	2	3	2	51		
14	28	18	32	15	23	6	21	19	68
15	25	6	8	6	8	1	3	14	39
16	54	47	44	1	1	7	13	1	2
17	54	26	24	27	25	2	4	21	39
18	56	47	42	19	17	8	14	20	36
19	27	7	13	2	7	16	59		
20	32	11	17	6	9	11	34	4	13
21	59	30	25	3	3	28	47	5	8
22	27	32	59			20	74		
23	27	14	26			12	44	1	4
24	28	26	46			19	68		

Since the port was not designed as an earthquake-proof structure, only the forces of mooring, berthing and inspecting were computed during construction. Another factor in damage is the soil of the port area which is third-class ground. Many high-pile structures had been built on that soil. Therefore, vibration occurred during the earthquake which damaged the structure. Note that horizontal inertia force can destroy the vertical member of the structure. As stated herein, damage always appears on the raking pile and its cap joint which carry the horizontal forces. If ground liquefaction occurs during an earthquake, the transverse members (as beam or plate) would be destroyed by uneven settlement of piles. In the port area, liquefaction was slight, and damage to superstructure beam as well as plate was insignificant. During construction of the port, backfill of the pier was below standard. This caused the raking pile to crack during the earthquake. (Research report 1975)

Repair of Earthquake-Damaged Pier

Since 1949, concrete and reinforced concrete have been generally used in port construction. These materials are used at Tianjin port. Thus technical repair measures for damaged reinforced concrete piers have general significance. They are outlined below.

Method of Repairing Damaged Pile and Pile Cap. a. Crack breadth less than 0.1mm need not be treated; b. When crack breadth is between 0.1 and 0.5mm, chip out the gap in a triangular trough to a depth of about 10mm and spread it with mortar of epoxy resin; c. When crack breadth is between 0.5 and 2.0mm, chip out the gap in a triangular trough to a depth of about 10mm, and seal it tight with mortar of epoxy resin asphalt pitch. Then grout with starch of epoxy resin; d. When crack breadth is more than 2mm, wrap it outside with C35-strength concrete.

In damaged piles where some crack breadth on the pile body is more than 2mm, but most crack breadth is less than 0.5mm, and the cracks are close together with spacing less than 300mm, wrap the outside with concrete. Length of the concrete must exceed both the upper and lower crack by 200mm and its depth must be 150mm. The surface should be rough before grouting concrete on the pile body.

When using concrete to wrap the pile cap, the wrapped portion should not be less than 250--350 mm. With a properly rough surface, use the ring reinforcement and pour C35-strength concrete. Depth on each side is 250mm--350mm and bottom plate depth is 350--400mm. The major crack or torn portion must be cleaned first before grouting with concrete and cement put in that portion.

Method of Repairing Damaged Beam and Plate. a. Repair cracks on damaged beam and plate using the same method as that for piles. b. If a gap goes through the length of the plate, and its breadth is large, it can be grouted with thin starch of epoxy resin. Another method is to widen a piece of trough. Use hanging forms to pour concrete whose strength is no less than C35.

Method of Repairing Broken Pile. a. If the raking pile broke, which was situated near the damaged pile cap, concrete can be used to wrap the outside pile cap instead of connecting pile. Pay attention to avoid misalignment of the pile's body along the axis which would cause unfavorable force bearing; b. No matter which vertical or inclined pile is broken, attach reinforcement with electrical welding or bounding and chip roughness and clean the surface of the pile body, when it is connecting; c. When the longitudinal inclined pile under the mooring post is broken, pour a large cap ring around the broken pile to form a monolith with the side vertical pile after they are cleaned.

Other Repair Measures to Deal with Earthquake Damage. a. If the joint of upper beam-plate and pile cap is pulled or cracked, in order to restore the bearing action, a piece of iron should be placed as a cushion before putting the mortar cement into the torn portion; b. For a damaged vertical pile cap under a simple beam in the rear platform, brace the simple beam, tear down the pile cap, according to the reinforcement drawing, and pour concrete once more; c. When the pile, pile cap, beam and plate are damaged at their corner or pact of the surface, apply mortar cement. Strength of the mortar is no less than M30 and that of concrete is no less than C35.

Condition of Pier after Repair and Strengthening

Since the damaged pier was repaired in 1976, and put back in production, 18 years have passed. In view of its operation, requirements have been entirely met. No additional repair was necessary. Quotations from two site investigations follow.

The First Investigation Reported Its Findings on Damage at Xingang Harbor (1985).

In order to use the piers properly and understand the actual role of the piers, as well as to maintain safety in time and increase utility, the Port of Tianjin Authority has entrusted the investigation for the twenty-three piers and seven piers in three districts in the harbour of Tianjin Institute of Water Transport Engineering.

During the investigation of the piers, a survey was conducted of a boat entering the pier interior. Each pier and each pile was numbered by use of torch light to inspect them one by one.

Analysis based on the investigation of damaged piers in Tianjin port indicates that there is a great difference in the operation life of the plate and the beam pile of piers. It is estimated that the life of the lower structure (i.e. piles) would be twice that of the superstructures (i.e. beam and plate). This indicates that the condition of pier piles is better than that of the super beam-plate system.

The No.5 berth of Xingang has no record of damage in the foundation pile. In the No. 14 & No.15 berths of Xingang, the damage has been repaired. At the No. 18 berth of Xingang, the condition of pile and pile cap was good, but in the No.10 district some pile cap was damaged in the concrete by pressure from overloading.

Based on the entire investigation, it is expected that the berth built in 1957--1961 and used for a long time will incur serious damage on structure members due to sea erosion. Other piers, built in recent years, are basically sound. The phenomenon of damage always appears on some individual members. Hence it can be said that in recent decades the construction at Tianjin port was generally successful. With respect to design, construction and management, experience has been gained which can contribute to the construction of other piers in our country.

The Second Investigation Reported Its Findings on Damage at No.14--N0.15 Berths in Tianjin Port (1988).

During the general investigation in 1985 of Tianjin Port, it was discovered that damage to No.14 and No.15 berths were serious.

In order to save the two berths and to minimize losses, Port of Tianjin Authority entrusted the investigation to the research institute for data to repair the piers. No.14 and No.15 berths were built in 1961. Their length is 380m. Platform of the pier structure consists of pre-stress pile, upper position beam, pre-stress type plate and an impact member for berthing ships.

Piles there are basically sound but 18 were broken in two districts. According to the investigation, the damage appeared at the same time the pier was built. Backfill for the pier produced so much earth pressure that the pile was subjected to shear stress, resulting in damage to the joint of pile and its cap.

Pile caps are also basically sound. Damage to some pile caps was not caused by the earthquake but by wave impact and swift current against the concrete.

Based on the two investigations, a conclusion can be drawn. As noted, damaged piers were repaired after the earthquake and have been operational for 18 years. The piers remain in use and have not been damaged again.

It can also be concluded that the quality of repair by the construction engineering department is reliable and results were good. It should also be noted that in the past 18 years, the engineering facilities have not been subject to a violent earthquake.

Earthquake Resistance Measures for Pier Design in Seismic Regions

Pier Structure ought to be Simplified and Regularized. The earthquake damage survey showed that, during the architectural composition of piers, as lines were simplified and regularized, seismic resistance would improve. When the style of structure is more complicated, the method of partition should be used to divide them (such as to set seismic resistance joint, extension joint) into architectural, i.e. independent, parts to prevent torsion and stress concentration, which may cause even greater damage.

Superstructure Should be a Slender, High-Strength Sound Structure. A slender type and high-strength superstructure reduces the weight of the structure and decreases earthquake action. A currently used pre-stressed hollow plate is lightweight and has higher strength as well as better stiffness. Also in current use is pre-stressed super position structure, an ideal structural member. It can save form work, making it convenient to pour concrete for the superstructure, and can satisfy the requirement of a sound structure.

Piers under the Deck Should be Arranged as Symmetrically as Possible. It is difficult not only to bring the pile capacity into full play but also to arrange the piles symmetrically, particularly raking piles. For reasons of construction and operation, the raking piles often rotate at an angle corresponding to the axis of the frame bench. In order to decrease rotation of the platform in case of an earthquake, the raking piles thereupon should be arranged symmetrically.

Pre-Stressed Concrete or Steel Piles are Preferable for Piers. With regard to stiffness and capacity of deflection resistance, a pre-stressed concrete pile is better than a common concrete pile with the same cross-sectional area. In operation or during handling, storing and driving, the former can withstand unfavorable sets of forces including an earthquake. Then the piles will be subjected ont only to vertical load, but also to bending-compression hanging-tension and shear beyond the capacity of the common concrete pile. Whenever possible, steel piles should be used.

A Transitional Plate Used to Connect with the Shore. Test on a model shows that between platforms as well as between the platform and shore connection, structure impact occurs. Inertial force would be transferred to a simple transitional plate connecting with shore and pier. Thus earthquake inertia force which comes from the shore structures might be decreased.

Using Insulating and Buffering Materials between Platforms of Piers. When earthquake is severe, front and rear platforms of a pier impact each other, causing damage to the platform. Hence some elastic materials with a certain strength should be put in the joint between the platforms to buffer the impact and decrease earthquake damage.

Lay-out of Pier Structure should Facilitate Inspection And Maintenance. With a continuous plate member for touching against ships, it is better to provide a passage at both ends of the pier to ease the coming and going of sampan. The rear platform must have a square hole with an area no less than 1 sq. meter to give access for inspection and maintenance in urgent situations.

Adopt the Following Measures to Improve Anti-Seismic and Anti-Erosion Capacity of Pier Structures. a. Increase the protection course in reinforced concrete; b. Increase the compactness of concrete; c. Improve the quality of reinforcement before putting it into forms, and do not use rusted reinforcement material.

In order to increase seismic resistance and durability of concrete, it is better to improve the management and operation of the pier. No overloading should be allowed. Inspection and maintenance should be conducted at appropriate times.

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CATASTROPHIC EARTHQUAKES
AND
THE PROSPECT OF INDIRECT LOSS

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ABSTRACT

This paper discusses the design and application of a new regional economic model for rapidly assessing the indirect economic consequences of damage to residential structures, manufacturing facilities, and lifelines (particularly power generation and distribution, water supply, telecommunications, and transportation). The model is based on a computational algorithm which tracks indirect economic dislocations to economic sectors that suddenly can't find outlets for their products or adequate supplies of critical inputs. Surviving productive capacity is reallocated and augmented with imports to derive a realistic measure of lost employment and income. The model is used to illustrate: 1) how potentially large indirect losses could result from relatively modest earthquakes and 2) the futility of attempting to measure indirect losses using post event statistics (which reflect the effects of economic dislocation and the positive stimulus of rebuilding). The approach is then used to assess the economic fallout from the Kobe disaster.

KEYWORDS

Earthquake; damage, indirect; secondary; economic; input-output

INTRODUCTION

Natural disasters conjure up images of physical destruction -- buildings that have either collapsed due to ground motion or have been swept/blown away by waves and wind. There are, however, more subtle losses which spring from this destruction; loss of critical facilities, both private and public, can produce economic dislocations inducing unemployment in sectors not directly damaged by the event. The magnitude of such dislocations has been the subject of considerable debate. It is commonly believed that earthquakes threaten more than life and property; the viability regional economies and even the stability of nation's stock and bond markets are thought to be jeopardized. The following quotes taken from a recent National Committee on Property Insurance publication are illustrative of the concerns which have been voiced.

> "An 8.0 on the Richter Scale could do more economic damage than a 500-point drop in the Dow." Senator Albert Gore, Chairman, Subcommittee on Science, Technology and

Space. (National Committee on Property Insurance, 1989, 65)

"[p]rojections for the drop in US. GNP from a large earthquake in southern California are placed at 5 percent. Representative George Brown (D. Calif.) (National Committee on Property Insurance, 1989, 59)

Clearly, these projections are quite large. A 500 point drop in the Dow is equivalent to a 10 percent loss of the market's net worth, or approximately $2 trillion. A 5 percent reduction in GNP translates to a loss of $250 billion. Yet, when one looks at the evidence, these projections seem overstated. The economic repercussions following the Loma Prieta Earthquake (San Francisco, 1989), the Northridge Earthquake (Los Angeles, 1994) and, most recently, the Great Hanshin disaster (Kobe, 1995) are considerably lower than anticipated. Why? Are these not truly catastrophic events? Would larger events such as a Richter 8.2 in Tokyo or Los Angeles produce the dire projections Gore and Brown allude to? Or, is it possible that modern economies are more resilient than we have been led to believe?

The purpose of this paper is to provide an analysis of indirect loss, the so called economic ripple effects which stem from shocks to a nation's stock of productive capital. The anatomy of loss accounting is briefly discussed, followed by a brief description of a recently developed model for estimating indirect loss. The model is utilized to show the effects of rebuilding expenditures, damage patterns, economic structure, and capacity constraints on post disaster gross regional product. Lastly, A dynamic version of the model is applied to the Great Hanshin Earthquake to show how the Kobe economy is likely to behave during the reconstruction period.

DISASTER: THE ANATOMY OF ECONOMIC DISRUPTION

Most disasters produce some form of supply disturbances which ripple forward to demanders of critical products, and backward to the suppliers of raw and semi-finished ingredients. Input-output methods, the most commonly utilized tool for tracing interindustry linkages, provide little guidance as to how an economy unbalanced by disaster will restabilize. The approach presented below addresses this critical point. It is based on a newly developed methodology embodied in the National Institute for Buildings Standards, NIBS, Standardized Earthquake Loss Estimation Procedure, hereafter referred to as the **NIBS Indirect Loss Model**.

Forward and Backward Linkages

Earthquakes may produce dislocations in economic sectors not sustaining direct damage. Activities that are either forward-linked (rely on regional markets for their output) or backward-linked (rely on a regional source of supply) could experience interruptions in their operations. Such interruptions are called **indirect** damage. The extent of these losses depends upon such factors as the availability of alternative sources of supply and markets for products, the length of the production disturbance, and deferability of production. Figure 1 provides a highly simplified depiction of the how direct damages induce indirect losses.

In this economy A ships to B, and B to C. C supplies households with a final product and is also a supplier of inputs to A and B. There are two factories producing product B, one of which is destroyed in the earthquake. Indirect damages occur because: 1) direct damage to production facilities and inventories cause supply shortages for firms needing these supplies (forward linkage indirect damages); 2) because damaged production facilities reduce their demand for inputs from other producers (backward linkage indirect damage); or 3) because of reductions in government, investment, or export demands for goods and services caused by an earthquake.

Supply Shortages and Forward Linked Losses

The supply shortages caused as a result of losing B could cripple C, providing C is unable to locate alternative sources. Three options are possible: 1) it might secure additional supplies from outside the region (imports); 2) additional supplies might be obtained from the undamaged factory (excess

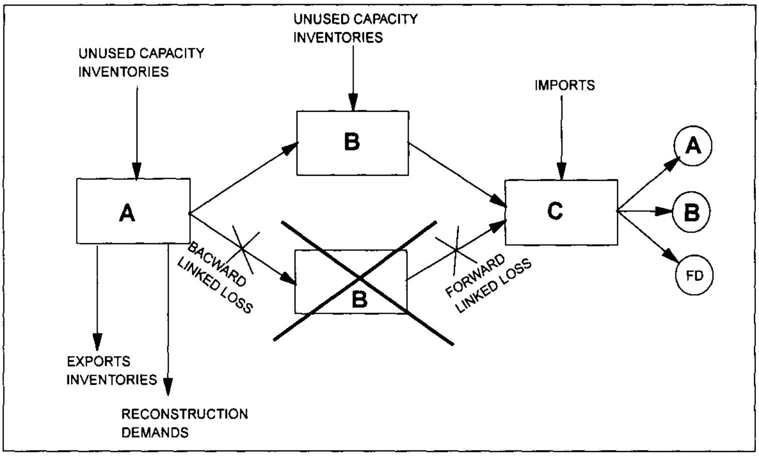

Fig. 1. Indirect losses and adjustments to lessen them

capacity); and 3) it could draw from B's inventories. The net effect of diminished supplies are referred to as forwardly linked losses, the term forward implying that the impact of direct damages is shifted to the next stage of the production process.

Demand Effects and Backward Linked Losses

Disasters can also produce indirect damages if consumer and producer demands for goods and services are reduced. If, in the example provided in Figure 1, firm B no longer requires inputs from A, then A may be forced to scale back operations. As in the case of forwardly linked losses, the affected firms may be able to circumvent a weakened market by either finding alternative outlets (exports), or building to inventory.

The higher rate of unemployment caused by direct damages and subsequent indirect factory closures could cause normal household demands to erode. However, it is more likely that the receipt of disaster assistance, unemployment compensation, or borrowing, would buoy household spending throughout the reconstruction period. Evidence from recent events (Hurricanes Andrew and Hugo, the Loma Prieta Earthquake and the Northridge Earthquake) confirms that normal household demands are only slightly altered by disaster.

Distinction Between Direct and Indirect Loss

Indirect losses stem solely from forward and backward linked costs. In contrast, direct loss is the sum of damages sustained by plant and equipment plus associated lost wages and other incomes. So, unemployment tied directly to the destruction of a facility is considered a direct loss, whereas unemployment stemming from supply shortages or loss of markets in termed indirect. As will be shown below, indirect loss is a very elusive concept. It can be quite large or diminish to zero, regardless of the severity of the event.

Indirect Loss: Winners and Losers

The severity and scope of post event economic dislocations hinge on how the disaster stricken region is circumscribed. The "target" area may sustain net losses or gains, depending upon the pattern of damage, and most important, the amount of external assistance received. See Figure 2. The larger region may suffer dislocations, or benefit from its trading relationships with the damaged region. Losses may be transmitted through import export channels, as would be the case when the destruction of critical port facilities interrupts the flow of strategic raw and semi-finished materials. In some instances gains might also be recorded. This would occur if production and commercial ventures are shifted to neighboring regions, or reconstruction spending served as an economic stimulant.

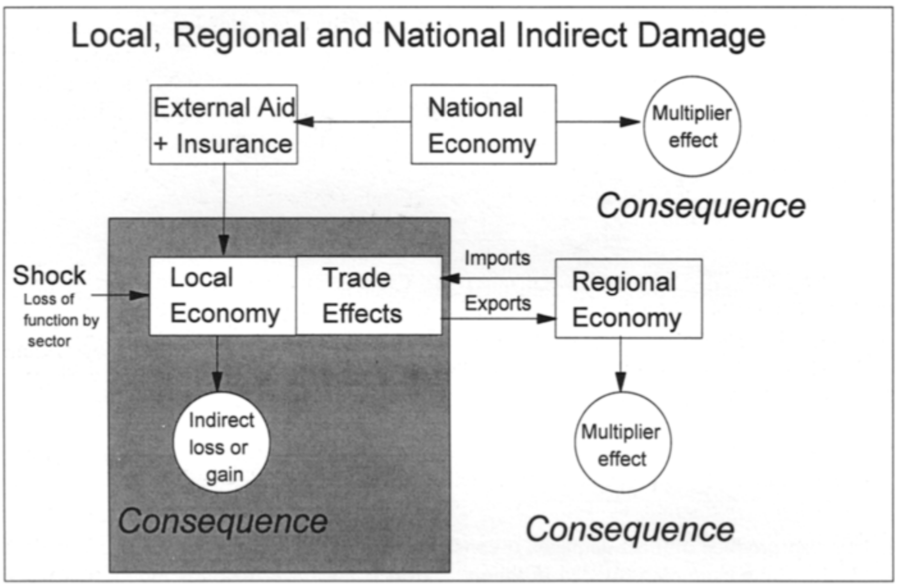

Fig. 2. Regional and national consequences

One might get the impression that disasters could be good for economies, but this only holds for the region. At the national level losses will be observed. Whatever the region gains the nation loses. Outside federal assistance has to be financed, either through tax increases, reduced government programs or an increase in the debt ceiling (implying a greater tax liability).

THE NIBS INDIRECT LOSS MODEL

The NIBS Indirect Loss Model is a computational algorithm which accounts for earthquake induced supply shortages (forward linkages) and demand reductions (backward linkages). The model is a version of a dynamic computable general equilibrium system designed to rebalance a region's interindustry trade flows based on discrepancies between sector supplies and demands. A complete description of the computational procedure for rebalancing the economy can be found in Cochrane and Steenson (1994).

A direct shock is introduced into the indirect loss model by adjusting the outputs and purchases in proportion to a sector's loss of function. Restrictions on shipments (forward linkages) and purchases (backward linkages) are computed and the resultant excess demands or supplies are derived. The first round effects are simply the direct loss of function times the inputs to that sector (backward links) and shipments from that sector (forward links). These first round effects produce excess demands and supplies which trigger a search for markets and alternative supply sources.

In building the model several critical choices had to be made regarding post event household spending patterns, labor mobility, elasticity of supplies from the construction industry, and the potential for product substitutions due to relative price changes. Evidence from previous disasters suggest that: 1) normal spending patterns are not significantly altered; 2) the workforce is highly mobile, particularly in the construction sector; and 3) relative prices do not change appreciably. Therefore, labor and construction sales are not constrained, and normal household spending is fixed and independent of current income. Given these conditions, the model assesses the net excess supplies (output less the sum of intermediate and final demands). A positive net value implies an excess supply; a negative indicates excess demand. It then attempts to resolve sectoral imbalances through a series of adjustments. If excess demand is detected, the algorithm checks to see if sufficient capacity exists in a sector. Excess capacities are a function of a user defined level of unemployment. Excess demands are met by first utilizing surviving productive capacity (including plant and equipment found to be idle prior to the earthquake). If surplus capacity is insufficient, the model then explores the potential of importing and/or drawing down inventories. These options are also provided by the user and are expressed as a percent of pre-event capacities.

Disposal of excess supplies is logically similar. Three options, inventory accumulation, exports, and production to meet reconstruction demands are explored. As in the case of the previous options, all are expressed as a percentage, and are determined by the user. In most cases excess supplies are not critical to the model's operation, particularly when reconstruction spending looms large. Much of the excesses are drawn into the rebuilding process.

After completing the first round of sectoral adjustments (changes in output to meet net excess demands), the algorithm recalculates the required shipments from and to each of the economy's sectors. Production, imports, and exports are again adjusted to bring markets back into balance. The process of reapportioning production diminishes the excesses detected in the previous round of calculations, but introduces new, albeit smaller, imbalances. Each alteration of output signals a new set of forwardly and backwardly linked requirements. The adjustment process continues until the economy is rebalanced.

The Sensitivity of Indirect Loss to Capacity, Damage and Reconstruction

A series of experiments were conducted to determine how indirect losses respond to direct damages (both magnitude and distribution), reconstruction spending, and the availability excess internal (locally unemployed resources) and external (imports) capacity. A ten sector economy similar in structure to that of Kobe's was used as a basis for the experiments which follow.

Sensitivity -- One Factor at a Time

Figure 3 (left) shows the effect of altering direct damage to manufacturing and transportation. Each is analyzed separately, that is, damage is assumed to occur in one of the two sectors, while all other sectors are assumed to suffer no direct losses. Some excess capacity was permitted (7.8 percent internal and 10 percent external). According to these results, indirect loss is highly sensitive to transportation disruptions, and less so to manufacturing. The reason for this lies in the relative size of the two sectors and the nature of indirect loss. The economy modeled was dominated by manufacturing; therefore direct shocks to this sector produced a large reduction in regional output, but forwardly and backwardly linked losses proved to be minimal. This follows from the fact that smaller sectors managed to secure sufficient supplies and markets; indirect losses were minimized as a result. In contrast, the identical shock to a small sector, such as transportation, was transmitted to the dominant sector, thereby amplifying losses.

The right hand side of Figure 3 shows the effect of internal excess capacity alone on indirect loss. The diagram indicates that, given sufficient surviving capacity, the economy will rebound to its predisaster state, albeit with a lower growth potential. With one important exception, additional external capacity (not shown) produces a similar result. In the event that imports are used to meet household unmet demands (e.g., aid sent directly to households), then the economy will remain depressed. Otherwise, the availability of imports tend to relieve supply shortages, thereby dampening indirect loss.

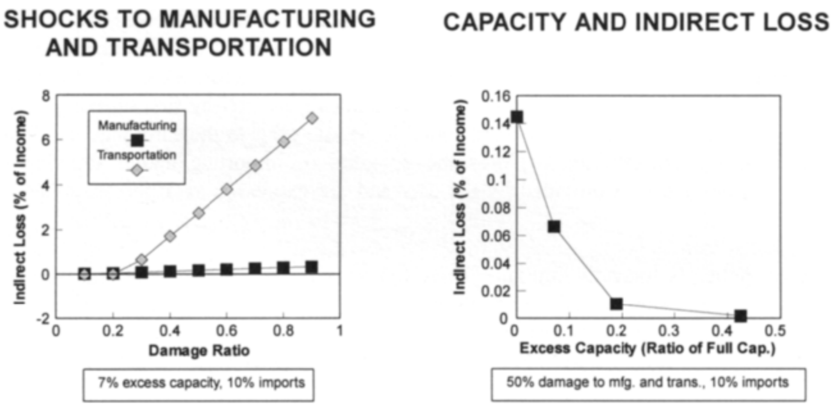

Fig. 3. Sensitivity studies: varying sector shocked and excess capacity

The Relationship Between Indirect Loss, Damage Patterns, and Rebuilding

The foregoing discussions suggest that there may not be a simple relationship between direct and indirect losses. Much depends upon the pattern of damage, which sectors sustain the greatest disruption, and their relative importance in the economy. In addition, the demand stimulus inherent in the rebuilding process would lessen indirect loss, possibly producing gains in instances where large amounts of excess capacity existed. The sensitivity of indirect loss to patterns of damage and rebuilding can only be determined through repetitive experiments with the model. Thousands of random direct shocks were simulated and the resultant indirect losses recorded. The solutions obtained for different levels of internal capacity and rebuilding were then plotted against direct loss to derive generalizable relationships.

Damage Patterns and Indirect Losses. As the above analysis pointed out, indirect losses are highly variable and are sensitive to the capacity of the region to expand production or import replacement ingredients or services. This point is underscored by the results displayed in Figure 4. These diagrams show the relative frequency of indirect and direct losses for 6000 random patterns of damage. Each of the 6000 patterns was fed to the NIBS model and the indirect loss or gain recorded. Contour maps of relative frequencies of occurrence were derived from the results. The upper right-hand insets indicate the relative gradient of each solution map. Two observations can be drawn from these maps. First, indirect losses can be substantial for small events. However, given the scenarios depicted, the probability of this occurring is relatively small. Second, indirect gains can result even when rebuilding does not occur. Although this observation has not yet been fully analyzed, a preliminary explanation of this counterintuitive result lies in the reallocation of sector activity. Two explanations seem plausible: 1) it is

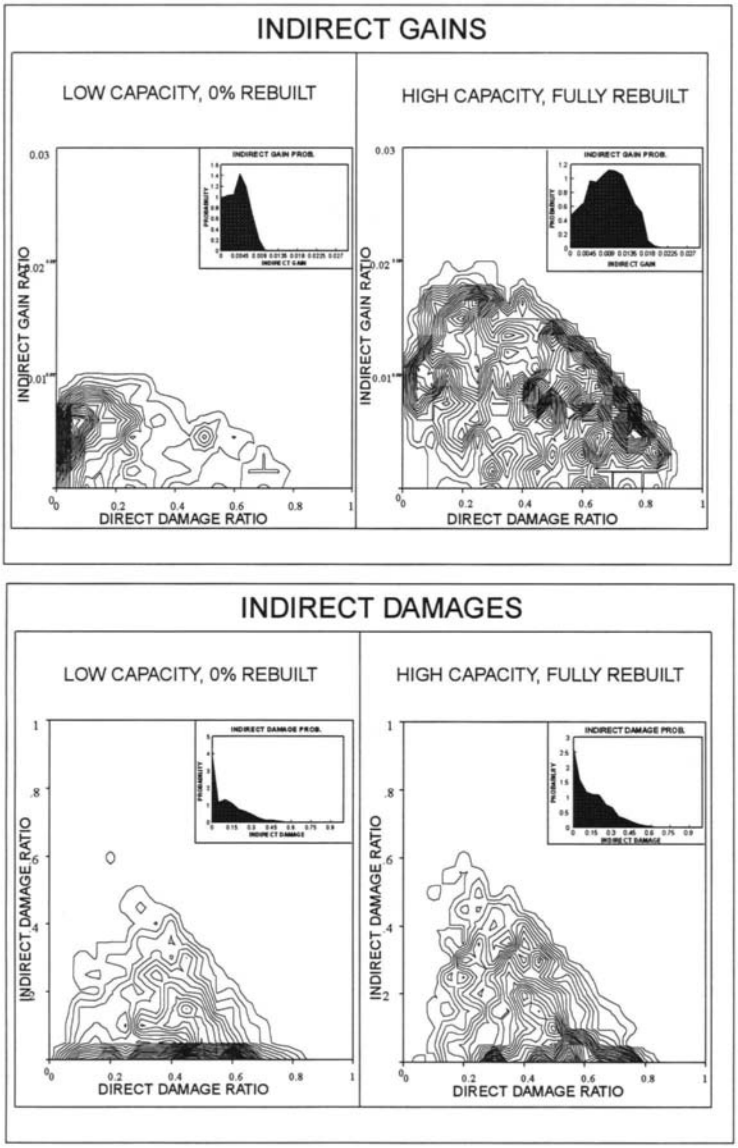

Fig. 4. Indirect loss related to patterns of damage, capacity and rebuilding

highly likely that rebalancing the economy involves reallocation of resources from less labor intensive to more labor intensive economic sectors and 2) expanded production will occur in sectors experiencing slack. This last point can be tested by observing the solution map for a fully constrained economy. Fully constrained means that no options (excess capacity, imports or exports) exist for mitigating the effects of forward and backward linkages. The top most map in Figure 5 shows the results of this experiment.

Indirect gains entirely disappear, only losses are observed. The fully constrained solution map provided in Figure 5 yields several other interesting insights into the nature of indirect losses. Indirect losses are in this case highly probable. The lower left-hand diagram in the same figure shows the probability of different loss combinations of occurring. The triangular shape of the map, lower right-hand diagram,

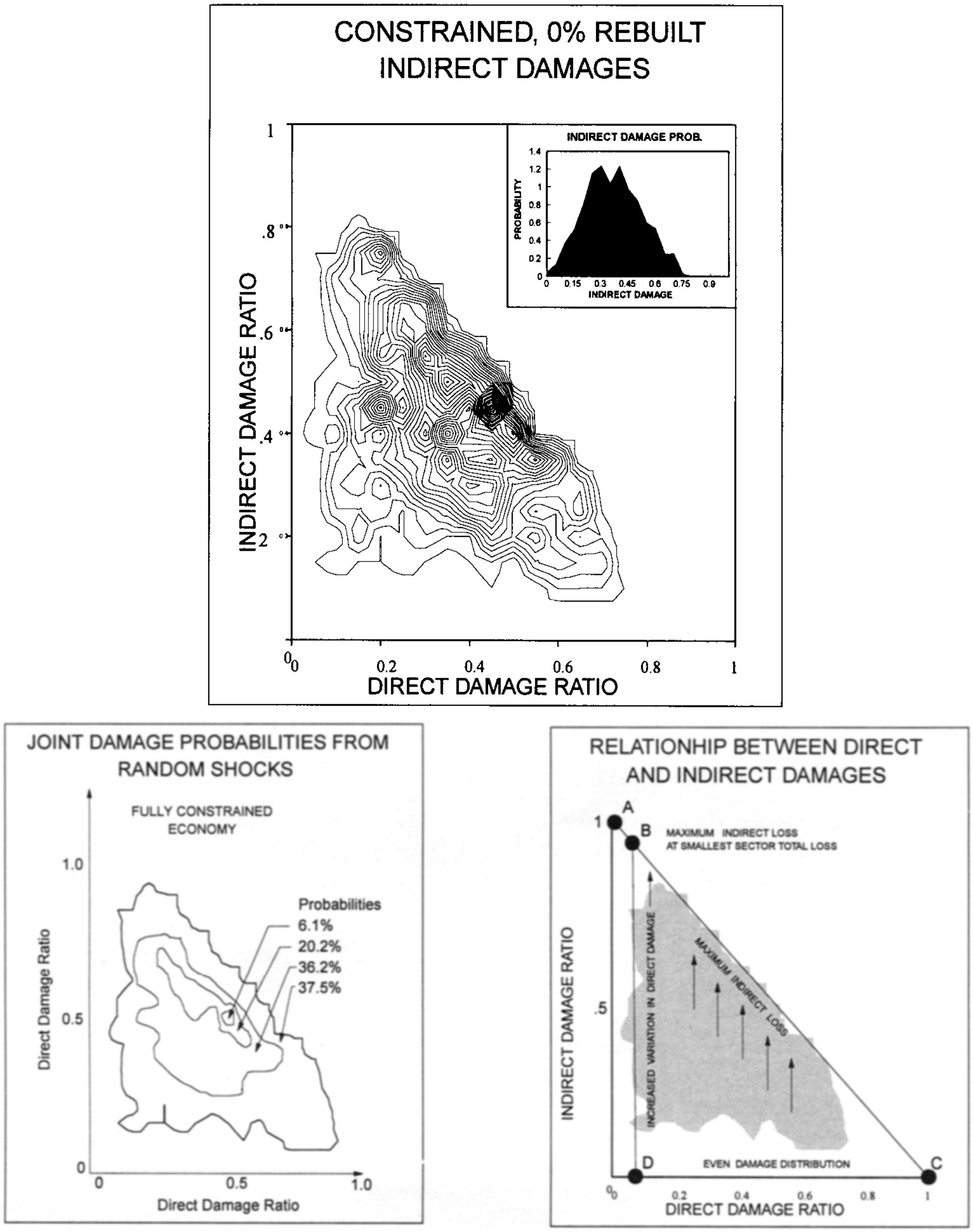

Fig. 5. Patterns of indirect loss when the economy is fully constrained

follows directly from the way in which the economy responds to damages. Point **B**, the uppermost level of indirect loss, results from a maximum shock to the smallest sector. Note, that none of the 6000 solutions provided that unique combination. Even though B proved to be improbable, other combinations of low direct loss and relatively high indirect loss were observed. The Line segment **D-C** shows the effect of uniform damage patterns. An even pattern of damages produce no indirect loss since the economy remains balanced. Only an uneven pattern of damage produces bottleneck effects and indirect losses. The line segment **A-C** can be interpreted as the indirect loss frontier. At the extreme, when direct loss is total, indirect loss must be zero. Similarly, when direct loss is total for the smallest sector, indirect loss is maximum. Hence, point A would be observed if the size of the smallest sector approached zero. Line segment **D-B** shows the influence of increased variance in the pattern of loss. The variance is zero at D and maximum at B.

The Effect of Reconstruction Spending. Whether reconstruction spending produces a positive short run gain for the economy depends upon the existence of internal and external capacity, and on the uses to which imports are put. As indicated above, if imports are directed toward meeting household final demands, then the economy's internal transactions will be short circuited. If this were to occur, then reconstruction spending would produce little in the way of internal gains, while regions outside the disaster zone would benefit most from the expenditures. Figure 6 shows this result. For reasons previously discussed, rebuilding produces no internal gain if the economy is fully constrained (upper right-hand map). Gains are observed as capacity becomes available (the lower left-hand diagram). A relatively large number of solutions are shown to lie in the negative indirect loss range (implying gains). The last map (lower right) reflects an economy which has significant amount of internal excess capacity and utilizes all imports to meet the needs of intermediate production. This, then, is the unconstrained scenario, the antithesis of the fully constrained economy. As might be expected, indirect losses disappear entirely, leaving only the residual gains from rebuilding. This unconstrained scenario produces results identical to that obtainable through conventional input-output techniques, where multipliers are used to assess the overall income effects produced by final demand changes. Although the conventional approach works in this instance, it is the only case where it is applicable. In the vast majority of cases, constraints will be encountered and the NIBS model (or some variation thereof) would be required.

A Brief Introduction to The Dynamics of Indirect Loss

The analysis just presented is static. That is, damage led to loss of function, and loss of function to economic disruption, all of which transpired instantaneously. Rebuilding, if it occurred, was accomplished over the course of a year, resulting in a loss of function over the same period of time. In addition, it was assumed that the disaster left no lingering effects on the economy. The expense of rebuilding did not alter the region's savings, debt, or future spending plans. These are, of course, gross simplifications, assumptions which hold up well for small events, but produce misleading results otherwise. What are the dynamic elements of indirect loss?

The introduction of time does not pose a formidable problem. The process of rebuilding restores lost function. Debt, if incurred, has to be repaid, thereby depressing future household spending on goods and services. These considerations have been incorporated into the NIBS model. The model was made dynamic by tracking the amount of reconstruction occurring each period and then permitting each sector to expand production accordingly. The model is resolved period by period until reconstruction is completed. If internally financed, debt payments are computed and subtracted from post reconstruction final demands. Net indirect losses are then computed over the entire time period (including the debt repayment period) and discounted accordingly.

APPLICATION TO THE GREAT HANSHIN EARTHQUAKE

The Great Hanshin Earthquake (1995) produced many of the constraints, dislocations and rebuilding stimuli that were discussed in this paper. The following is a very brief analysis of how the Kobe economy is likely to be impacted by the event. The analysis is based on news reports and first hand

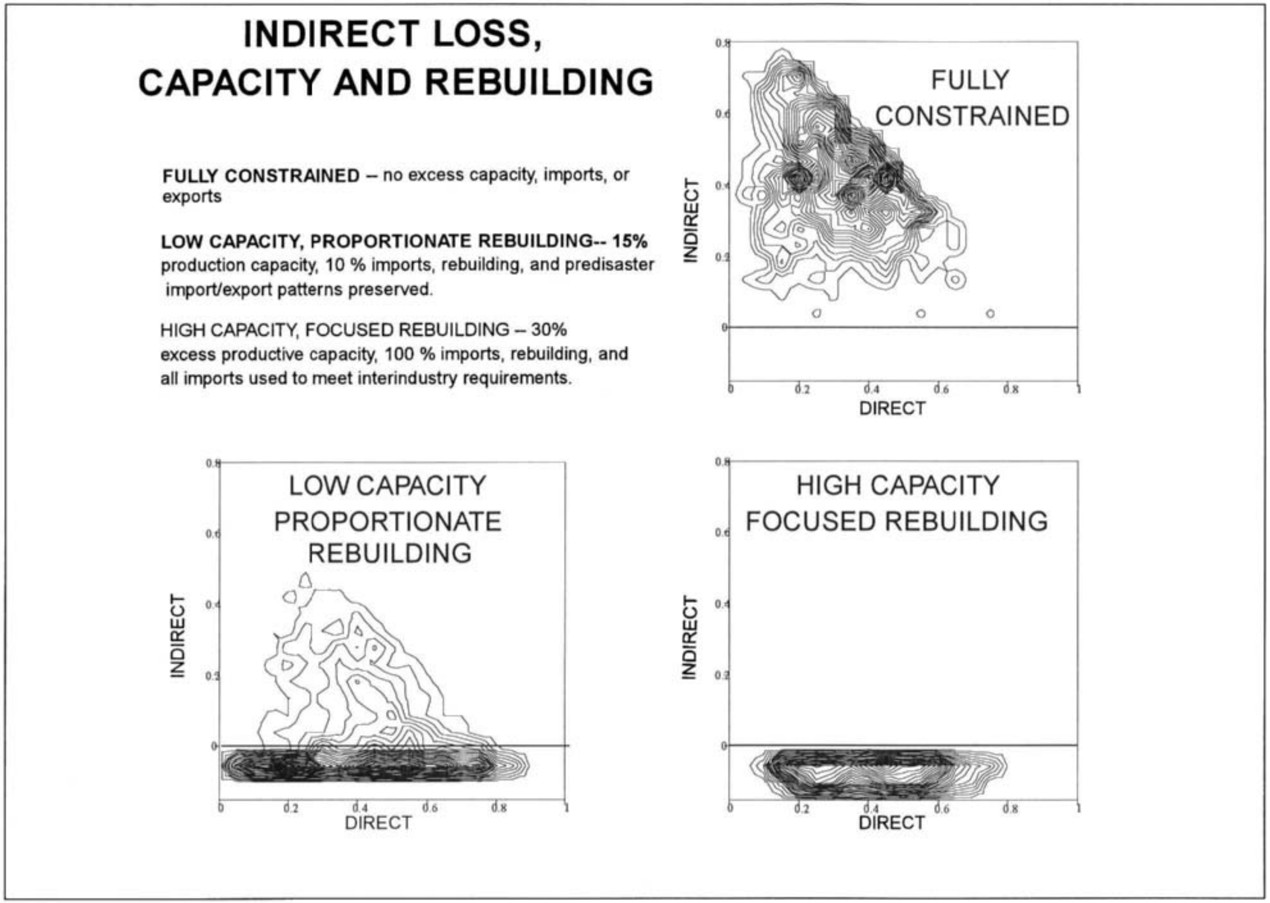

Fig. 6. The effect of rebuilding on indirect losses and gains

accounts of the Risk Management Solutions team which toured the disaster site within days of the earthquake.

The Kobe tremblor of Jan 17, 1995, killed over 5000 people, injured nearly 25 thousand, left 330 thousand homeless, and disrupted the region's factories, utilities and ports for up to a year (The Nikkei Weekly, January 23, 1995, p. 1). Estimates of both direct and indirect losses remain highly speculative. Damages to structures have been assessed at anywhere between \$30 billion and \$100 billion. Estimates of the impact on the Kobe economy are even more wide ranging, some placing the figure at 5 to 10 times the direct loss.

> "More sobering was the assessment of the assessment by the Japanese Chamber of Commerce and Industry President Kosaku Inaba that, taking into account the losses wrought by the disruption of economic activity, the quake will end up costing 40 trillion yen (\$400 billion)" (The Nikkei Weekly, January 30, 1995, p. 4)

Clearly, it is early to be projecting economic consequences for the Japanese economy, but even at this stage of the rebuilding process a few observations seem warranted.

The stimulus from rebuilding Kobe will far outweigh the economic dislocations stemming from the closure of its port, or dislocations within Kobe's manufacturing sector. This observation seems credible given that the \$100 billion price tag to rebuild Kobe is equivalent to the sum of one year's gross fixed investment in all of Japan (public, private residential and private non residential). We have learned from experiments with the NIBS Indirect Loss module that such a large stimulus would overwhelm the negative consequences stemming from disruptions of interindustry shipments (particularly given that the economy is mired in a recession, and significant amounts of excess capacity exist.) Some of the adjustments discussed earlier are emerging as important factors in dampening the earthquake's disruptive effects on interindustry shipments. Sealand, a major shipper utilizing Kobe's port, has already taken steps to find alternatives, such as diverting vessels to berths in Tokyo, Yokohama, Nagoya, Naha, and Okinawa, and chartering self sustaining cargo ships which have their own cranes. (The Nikkei Weekly, January 30, 1995, p. 4). In other cases demands are being met by drawing down inventories. Such was reported by the Kobe based Sumitomo Rubber Industries Ltd. (The Nikkei Weekly, January 30, 1995, p. 17).

The NIBS Indirect Loss model was utilized to provide a rough assessment of indirect losses which the Kobe earthquake might engender. Direct damages by economic sector along with estimated repair times, pre event excess capacity and reconstruction financing were input to the algorithm. The results, shown in Table 1, indicate that the regional rebuilding stimulus will be substantial, so much so as to cancel the effects of direct and indirect damages. The net loss to the greater regional economy (Kobe and Osaka) is projected to be \$1.05 billion. Indirect construction gains are \$129 billion. Pure indirect damages, after eliminating construction gains, are \$41 billion.

Table 1. Summary of Projected Kobe and Osaka Losses/Gains

Loss/Gain Category (- Loss, + Gain)	Discounted Amount During Reconstruction ($ Billion US)
Direct Loss	-89.44
Indirect Loss/Gain	88.39
Total Loss	-1.05
Construction Gains	129.64
Pure Indirect	-41.25

Figure 6 breaks out these income effects for more detailed examination. The total income effect

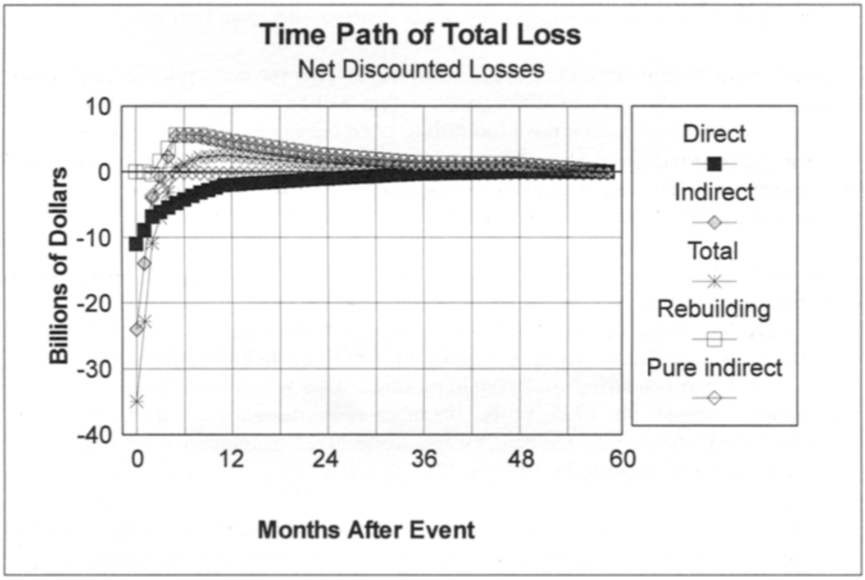

Fig. 7. The dynamic path of discounted direct and indirect losses

reflects both the positive effects of reconstruction and the costs of forward and backward linked economic dislocations. After the first 12 months the gains from reconstruction dominate; however, once reconstruction is complete at 48 months the effects of effects from dislocation effects disappear and debt repayment depresses incomes.

It is rather easy to misconstrue these results. The Hanshin earthquake was a truly catastrophic event with real economic repercussions. Yet, at first glance the results reported in Table 1 appear to understate the loss. However, the fact remains that excess capacity in the Osaka region absorbed some of shock. Second, the results reported in Table 1 are regional. National losses must include the effects of aid and insurance. So, reconstruction gains disappear, leaving only indirect loss. Lastly, Table 1 reports only direct loss sustained by industrial and commercial sectors. Public sector and residential housing losses are not included.

CONCLUSIONS

In conclusion, several sweeping observations are worth emphasizing. This paper has demonstrated that indirect loss can be an important and significant element of regional earthquake damages. It has also demonstrated that indirect loss is elusive; it can be quite large or diminish to zero, regardless of the severity of the event. Perhaps most important, indirect loss can be masked by rebuilding expenditures, and vary depending upon the accounting stance one takes. Impacted regions may escape these losses, but the nation cannot.

In addition the paper has uncovered several principles.

1. Holding capacity and rebuilding fixed, indirect losses are inversely proportional to the size of the sector shocked.

2. Imports, can either promote or reduce indirect loss, accentuating losses if imports are used to satisfy unmet household demands, and dampening losses if used to supply industry with raw and semi-finished ingredients.

3. Shocks to a fully constrained economy produce only indirect losses. In such an economy, the probability of indirect losses exceeding direct damage is approximately 50 percent.

4. The greater the variance in the pattern of damage, the greater the indirect loss.

5. A uniform pattern of loss produces no indirect loss.

6. If the economy is fully constrained, indirect losses are maximum when the economy's smallest sector is totally destroyed.

7. If the economy if fully unconstrained, indirect losses disappear and conventional multiplier analysis can be utilized.

8. A dynamic analysis of indirect loss reflects both the forwardly and backwardly linked losses and future demand changes resulting from disaster caused indebtedness.

Lastly, the results of the thousands of simulation experiments conducted thus far suggest that conclusions drawn from case studies <u>may not</u> be generalizable. We have found that indirect loss varies widely depending upon unique preexisting economic conditions, the amount of external assistance poured into the affected region, as well as the nature and extent of direct damage.

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POST-EARTHQUAKE REHABILITATION OF DAMAGED RC FRAMES

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ABSTRACT

The purpose of this paper is to identify some of the most likely locations of damage to reinforced concrete frame systems and to describe the results of tests to reconstruct the damaged area and to improve performance in the event the building is subjected to an earthquake in the future.

INTRODUCTION

The reconstruction of damaged reinforced concrete structures is dependent on a number of factors. First the damage must not be so severe that the stability of the structure both before the reconstruction or after is questionable from an engineering or technical point of view. Second, the cost must be within the owner's means. Finally, there must be social and political acceptance of the preservation of the structure.

The need to rehabilitate structures in seismic zones in the United States has gathered momentum in recent years with the experience gained following the 1989 Loma Prieta and 1994 Northridge earthquakes. A large national effort is now underway to develop guidelines for the rehabilitation of existing structures in seismic zones. The guidelines will not address the rehabilitation of damaged structures but there will be considerable overlap in the approaches to be used in either situation. There have been a number of studies of various techniques for rehabilitating existing reinforced concrete frames conducted at the University of Texas. While the primary aim was to evaluate pre-earthquake rehabilitation, tests have been included to study the performance of damaged elements that have been repaired following damage, i.e., post-earthquake rehabilitation.

Reinforced concrete frames constructed prior to changes in US codes after the 1971 San Fernando earthquake tend to have inadequate lateral capacity, low ductility at critical hinging regions, and lack of continuity of primary reinforcement (Pincheira. 1992). The most common detailing deficiencies include column ties too widely spaced to function effectively as shear reinforcement, splices of column longitudinal reinforcement located at hinging regions and designed to transfer compression only, inadequate transverse reinforcement in joints between beams and columns, and lack of continuity of bottom reinforcing bars through the beam-column joint. Often the columns do not have sufficient shear capacity to permit the flexural capacity of the columns to be developed. As a result, the column fails before the flexural strength of the frame can be mobilized. In other cases, the column flexural strength may be less than that of the beams at critical joints. Hinges form in the columns rather than in the beams. Most current design requirements are based on a "strong column--weak beam" philosophy. Some of the approaches used to correct these deficiencies will be described below. Typical details of non-ductile frame are shown in Fig. 1.

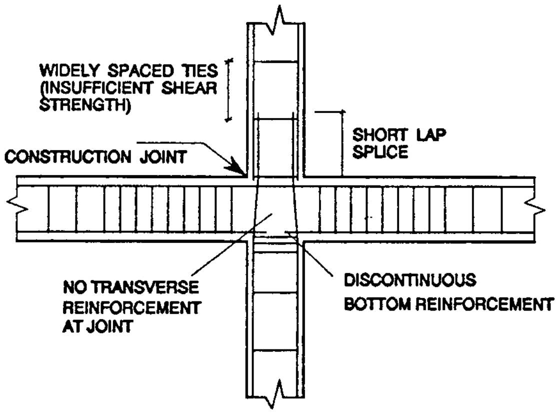


Fig. 1. Typical deficiencies in existing reinforced concrete frames

Experimental studies at the University of Texas have focussed on non-ductile reinforced concrete frames. Damage to columns, joints, and hinging regions in beams or columns has been produced under cyclic loading histories and the members damaged regions have been repaired and/or strengthened. The scope of this paper will include a discussion of the damage to such elements, the type of reconstruction undertaken, and the performance of the strengthened element or subassembly.

COLUMNS

Repair of Shear Damage

Two techniques for repair of shear damage will be described. The first involves the addition of a shotcrete jacket to repair shear damage in a short column (Bett, *et al.*, 1988) and the second addresses the problem of repairing severe short column damage in a two-story frame structure by encasing the damaged column with a large concrete jacket (Stoppenhagen, 1987). The new columns are large enough to transform the structure from a weak column--strong beam case to one in which the beams hinge before the columns fail in shear or reach flexural hinging.

Three identical column specimens were constructed at 2/3 scale. The prototype columns were 46 cm square. The test columns were 30 cm square and were reinforced with eight 19 mm longitudinal bars and 6 mm ties spaced at 20 cm. The tie spacing was much greater than permitted by current US codes. Details of the existing column are shown in Fig. 2 and the rehabilitated sections are shown in Fig. 3. The compressive strength of the concrete in the original (existing) columns was about 30 MPa and the shotcrete used to jacket the columns had a compressive strength of about 35 MPa. The steel reinforcement had a nominal yield strength of 420 MPa. The shotcrete was applied using a wet mix process. The columns were subjected to three cycles of deformation to increasing drift limits in increments of 0.5% drift. Specimen 1-1 was the existing column and was repaired by removing all the loose concrete after initial testing and adding a new shotcrete shell and transverse reinforcement (1-1R). The other two columns were strengthened by adding a 6 cm shotcrete shell with a reinforcing cage of 6 mm ties at 6 cm spacing. Columns 1-3 and 1-1R had additional 10 mm cross ties at 27 cm spacing. The new reinforcement was designed to increase only the shear capacity of the sections, the flexural capacity of the column at the ends was not increased.

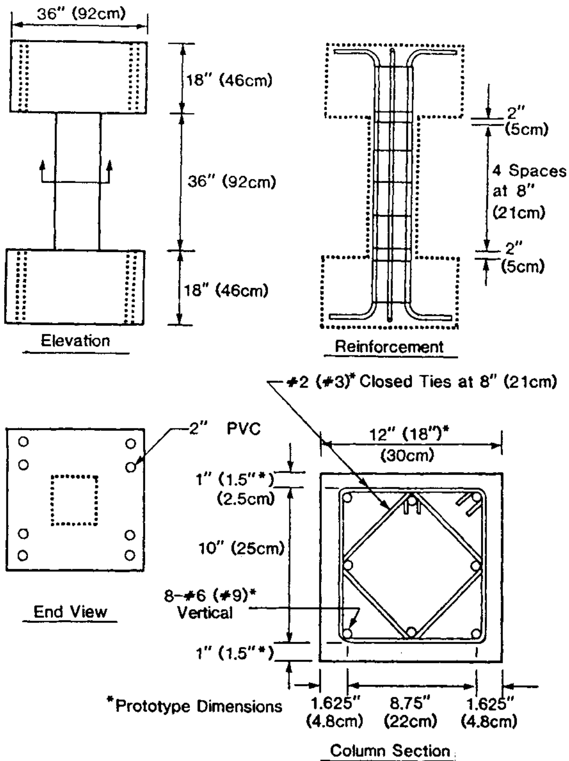
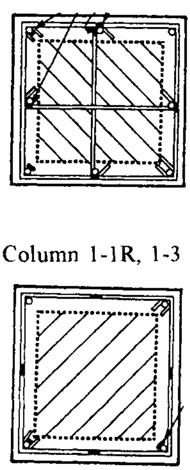


Fig. 2. Short column details, Column 1-1



Column 1-1R, 1-3

Column 1-2

Fig. 3. Jacket Details

Figure 4 shows the envelope curves for the four columns. The increase in shear capacity with the addition of the shotcrete shell and the transverse reinforcement cage is readily apparent. Of most interest is the fact that the repaired column (1-1R) performed almost identically to the two strengthened columns. Even though the concrete in the core of the column was cracked during testing (1-1), the repaired column performance was dominated by the new shotcrete shell and the added transverse reinforcement.

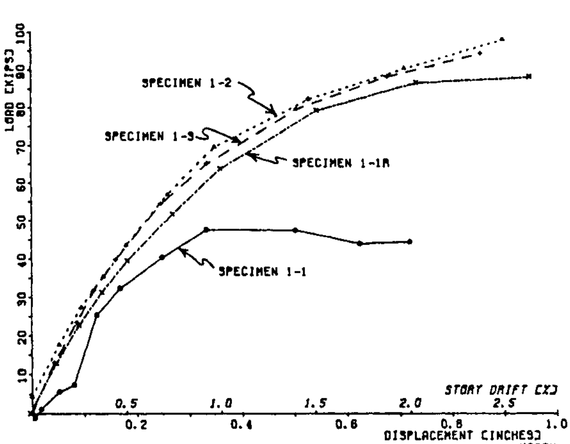


Fig. 4. Envelope curves for cyclic loading on short columns

The second study involves the repair of heavily damaged columns in a two-story frame structure shown in Fig. 5. The prototype structure was a seven story frame designed to meet the 1955 Uniform Building Code. All lateral forces were carried by the perimeter frames which had deep spandrel beams and short columns with widely spaced ties. The test frame was a 2/3 scale model of two stories of the prototype. The concrete strength was 40 MPa and the steel had a nominal yield of 420 MPa. Under lateral loading, the short columns failed in shear (Fig. 6). The repair scheme involved the construction of new columns encasing the existing damaged columns. The new column section was designed to have a flexural strength large enough to avoid shear or flexural failure in the columns and to force hinges to form in the spandrel beams.

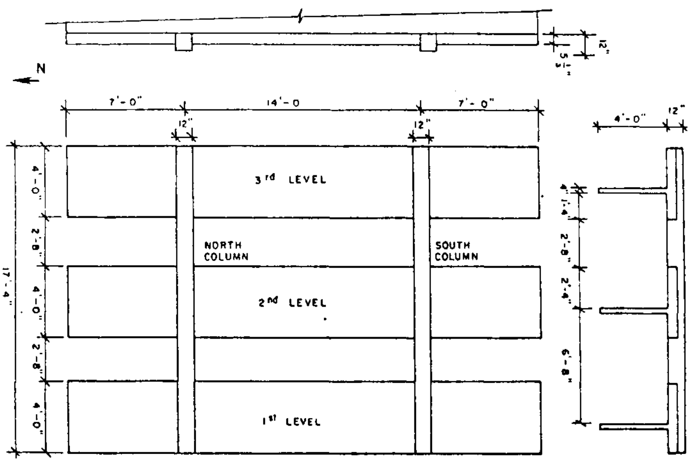


Fig. 5. Two story frame structure with short columns

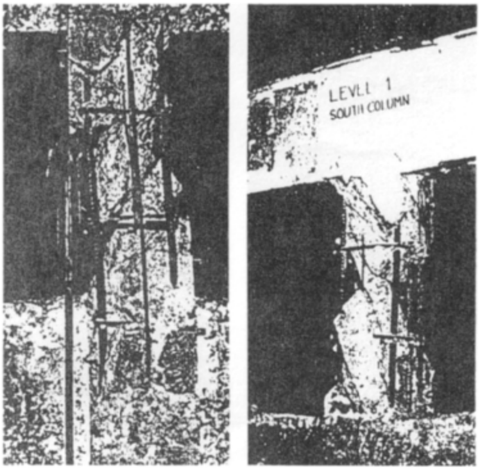


Fig. 6. Column shear damage

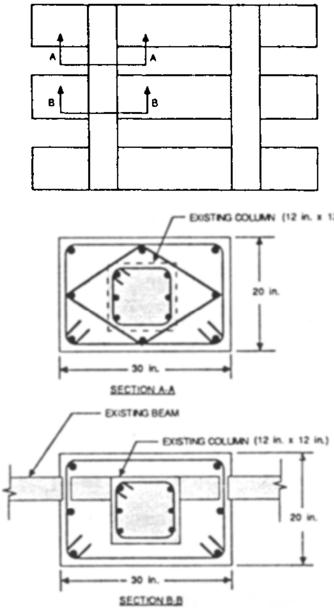


Fig. 7. Column jacket details

Details of the new columns are shown in Fig. 7. The column ties were detailed to permit the cages to be constructed around the existing columns. Loose concrete was removed from the damaged columns but fractured concrete in the column core was not removed so that the floors did not need to be shored during construction. The new columns were constructed in two lifts per floor.

Figure 8 shows the load-drift relationship for the repaired structure. The lateral capacity of the frame was controlled by hinging in the spandrel beams. The slight "spindle" shape of the hysteresis curves was due largely to the cracks in the beams from previous testing. The design objective was clearly achieved and the repaired columns functioned as intended with the section containing the heavily damaged columns performing in a nearly monolithic manner.

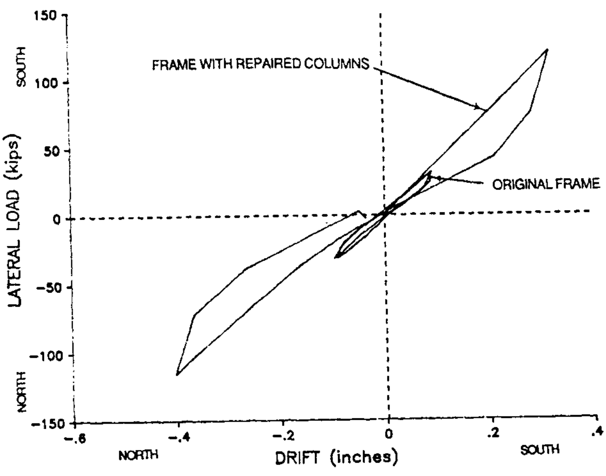


Fig. 8a. Load-drift response--all cycles

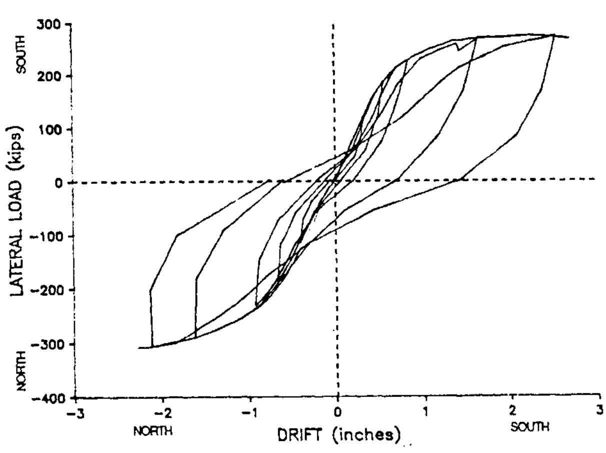


Fig. 8b. Load-drift response--all cycles

Repair of Damaged Splice Regions

Splices in column longitudinal reinforcement in buildings constructed prior to the mid-1970's were generally designed for compressive loadings only. When such frames are subjected to lateral loading, it is likely that the end regions of the columns, where the splices are located, will have large moments and the splice will be in tension. Because the splices are only confined by widely spaced ties, spalling is likely and the splice will fail before reaching tensile yield. Several techniques have been investigated to improve the confinement when such columns are damaged (Aboutaha, 1994; Valluvan, *et al.*, 1993).

Figure 9 shows the details of a column with inadequate splices. The response under cyclic loading is also shown in Fig. 9. The column was subsequently repaired by removing loose cover concrete and encasing the splice region with a steel jacket (Fig. 10). On one face, a row of anchor bolts were placed to improve the confinement in the long face of the column. As can be seen, the cyclic response was improved but the bars did not reach yield. The splice failed before the bars yielded. A third specimen was also repaired and tested but the repair involved removal of concrete from the core behind the spliced bars as shown in Fig. 11. Bolts were placed through the column in the short direction to improve confinement of the splices in the long face. The cyclic response was excellent and the bars reached yield. The improved performance was attributed primarily to the fact that sound concrete was placed around the spliced bars providing good bond on the entire perimeter of the reinforcement. Although the anchor bolts may have helped it is important to note that bolts were present in one face of the other repaired column but had little effect on the cyclic response.

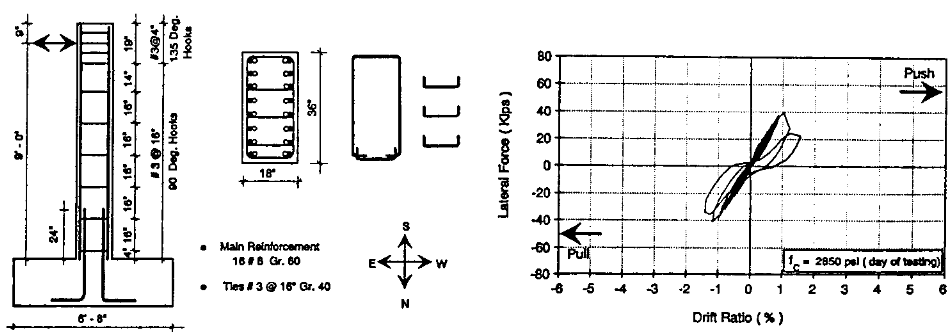


Fig. 9. Test of column with inadequate splice

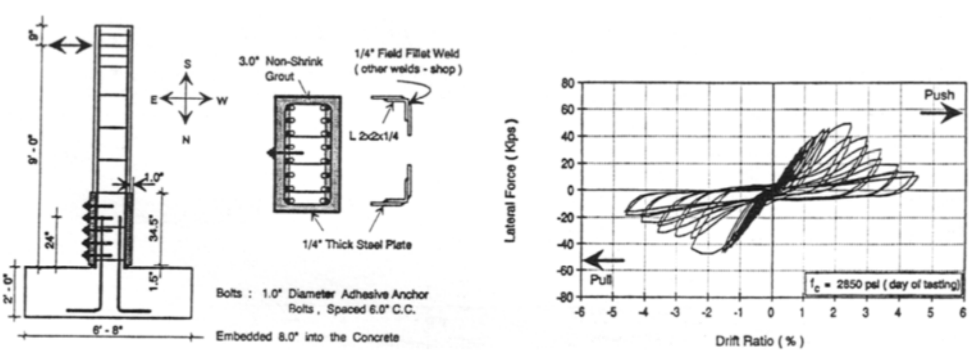


Fig. 10. Column with inadequate splice--Repaired with cover removed and steel jacket added

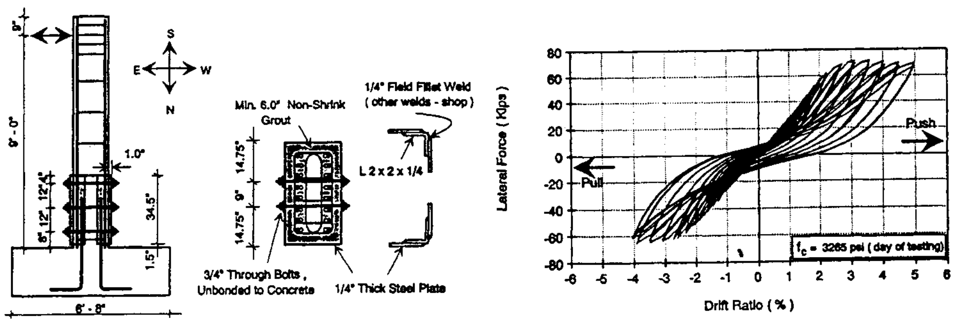


Fig. 11 Column repaired by removing concrete around splices and adding steel jacket

In another series of tests, column sections with compression splices were subjected to tensile loading. The splice regions were repaired and strengthened using a number of different techniques intended to produce only minimal changes in the cross section dimensions. Figure 12 shows some of the schemes investigated. Figure 13, 14 and 15 show envelope curves for cyclic load tests of the various schemes. The results show that schemes which did not include grouting to fill gaps between the new confining reinforcement and the concrete surface performed poorly. Likewise, schemes which required removal of existing concrete to install new ties also performed worse than expected because the concrete removed was replaced by patching material which did not bond sufficiently to the existing concrete to confine the splices in the same manner as cast-in-place concrete.

Details of splice strengthening	Maximum load (K)	Fraction of actual tensile capacity (f_y = 121 K)	Failure mode
unstrengthened	80	0.66	splice failure
steel angles & straps (ungrouted)	120	0.99	end connection failure
steel angles & straps (grouted)	150	1.24	large deformations outside the splice region
steel angles & straps (grouted)	140	1.16	large deformations outside the splice region
external ties (grouted)	145	1.20	splice failure
external ties (ungrouted)	90	0.74	splice failure
external ties (partially grouted)	135	1.12	splice failure
additional internal ties	115	0.95	splice failure
welded splice	131	1.08	extensively damaged specimen
welded splice with an additional tie	145	1.20	yielding of additional tie

Fig. 12. Splice strengthening schemes

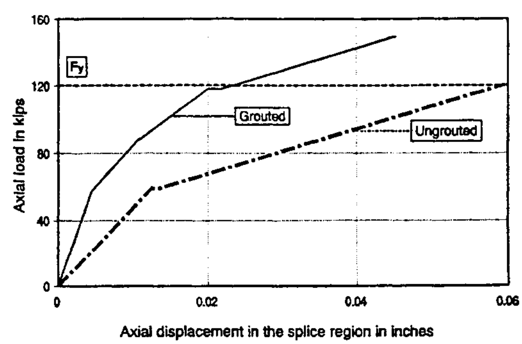


Fig. 13. Tests with straps and angles

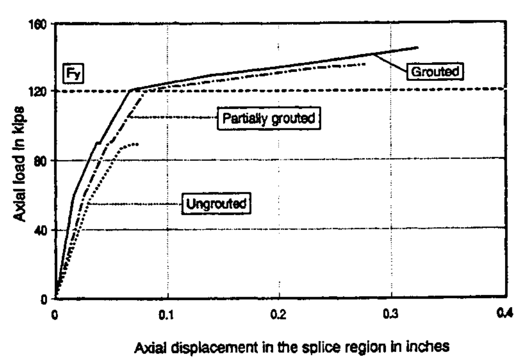


Fig. 14. Results of tests with external ties

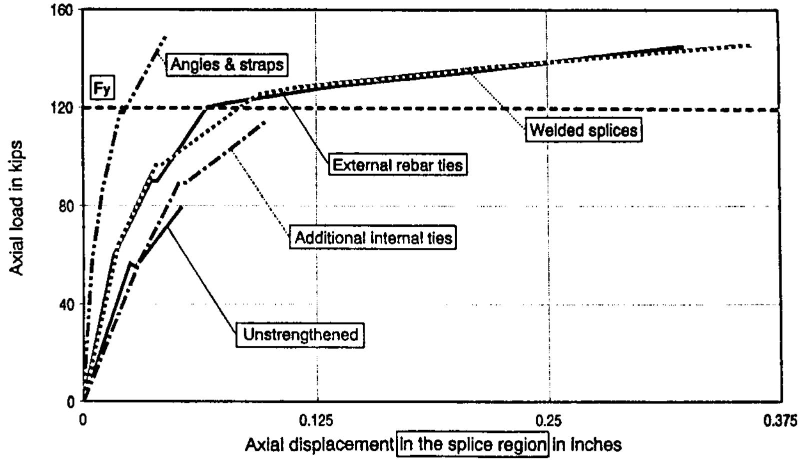


Fig. 15. Comparison of results with different splice repair schemes

BEAM-COLUMN JOINTS

As mentioned previously, the beam-column joint region in reinforced concrete frames is often highly stressed and, in older buildings, may lack the confinement and continuity of reinforcement needed to perform well under cyclic lateral forces. Figure 16 shows a schematic view of a beam column joint which was tested under bidirectional lateral loadings (Alcocer, *et al.*, 1993). The member dimensions and reinforcement resulted in a joint with strong beams and weak columns. The response was quite poor with the column hinging at relatively low lateral loads. The column was repaired by encasing the column with a new cast-in-place jacket containing additional longitudinal and transverse reinforcement as shown in Fig. 17. Because it was extremely difficult to place transverse joint reinforcement through existing floor elements, a special structural steel cage was fabricated to confine the joint region (Fig. 18).

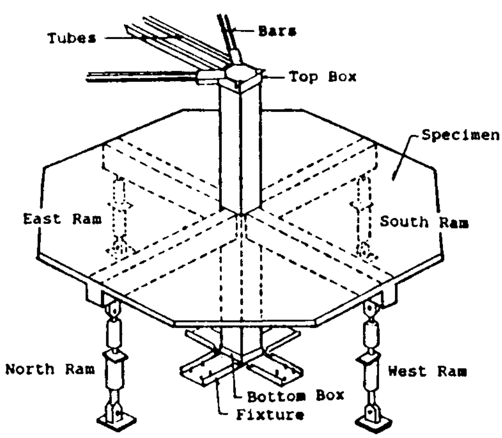


Fig. 16. Schematic of beam column joint subassembly

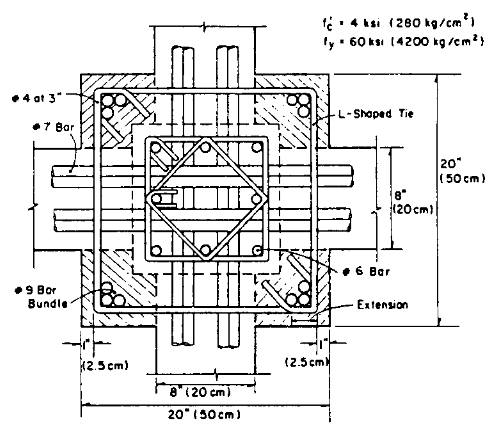


Fig. 17. Section through repaired column

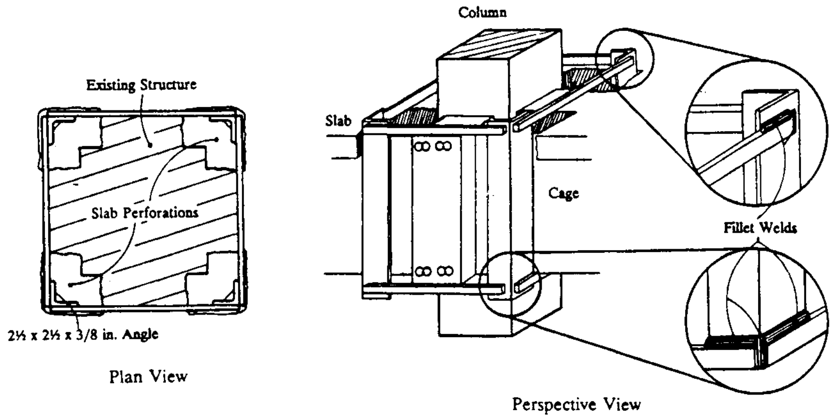


Fig. 18. Special structural steel cage for joint region

The results of lateral load tests on the repaired joint subassembly are shown in Fig. 19. With the strengthened column, the specimen reached levels which resulted in yielding of the beams. In effect, the subassembly was converted from a strong beam--weak column system to one in which the column no longer controlled the response.

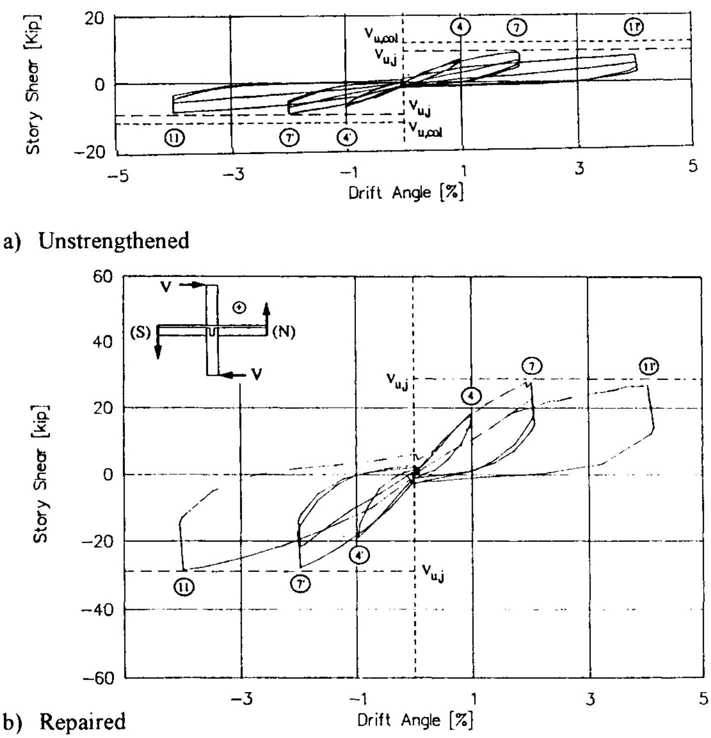


Fig. 19a. Story shear vs drift for beam-column joint subassembly

CONCLUSIONS

The tests of various reinforced concrete structural elements and subassemblies indicate that damaged elements can be repaired to perform in a manner comparable to that of similar elements strengthened before an earthquake occurs. In cases where shear failures have occurred, jacketing with concrete or steel shells gave reliable results. The sections behaved as would be expected if the sections had been cast monolithically. By jacketing columns, it was possible to change a weak column--strong beam system into one controlled by hinging in the beams.

In cases where splice deficiencies were corrected, reliable performance was obtained by removing damaged concrete around the entire bar perimeter and replacing it with new concrete. Jackets or confining elements such as straps or external ties were effective only if they were grouted to eliminate gaps between the concrete surface and the new elements.

ACKNOWLEDGEMENTS

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EARTHQUAKE DAMAGE OBSERVATIONS SHAPE FUTURE BUILDING CODES

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ABSTRACT

Recent California earthquakes demonstrated that while California's current building codes are generally adequate to protect life-safety, they fall short of what is needed for protection from economic disaster and the loss of essential services after an earthquake. Efforts are underway to translate these observations into tangible improvements to the way buildings are designed. New design approaches for producing buildings, which not only protect life-safety, but are also capable of reliably satisfying a number of enhanced performance goals are being developed.

One effort by the Structural Engineers Association of California entitled Vision 2000 is chartered to produce a framework for design and construction standards which will yield buildings of predictable earthquake performance. Employing a performance- based engineering methodology, Vision 2000 marks a milestone in development of future seismic codes. ATC-33 is a compatible effort which focuses on the seismic rehabilitation of existing buildings also using a performance based design approach.

INTRODUCTION

Revisions and refinements of seismic codes have traditionally been based on observations of building damage - each earthquake has made its contribution to improving building safety. In the U.S., it was the damaging Long Beach, California earthquake in 1933 that inspired codification of minimum seismic design standards. These standards were developed with the goal of protecting life-safety by preventing structural collapse.

Modern U.S. building codes have grown and developed from this platform. California's building code is the Uniform Building Code (UBC), one of three model building codes used by local and state jurisdictions in the United States. The UBC is maintained and published by the International Conference of Building Officials (ICBO). Historically, ICBO has adopted the provisions of the Structural Engineers Association of California (SEAOC) Blue Book for the UBC seismic provisions.

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Recommended lateral force requirements contained within the SEAOC Blue Book (Reference No. 1) are intended *to maintain public safety in the extreme earthquakes likely to occur at the building's site.* Secondary goals include control of property damage and maintenance of function in more moderate events that are expected to occur more often.

Following each major earthquake, engineers have observed the damage sustained by buildings, with particular interest in the performance of structures conforming to the current code at the time. Where unacceptable damage has been observed, the building code provisions have been modified to prevent the recurrence of such damage in future events. As a result, current building codes enforced in seismically active regions of the U.S. are believed to provide good levels of life-safety protection in buildings which are properly constructed. However, these codes appear less reliable with regard to the secondary goals of minimizing property damage in moderate and small events.

RECENT EARTHQUAKE DAMAGE OBSERVATIONS

In recent years, California, due to its active seismicity and relatively modern inventory of buildings, has provided a good laboratory in which to judge the effectiveness of building codes in meeting their intended performance goals. The limited life loss and injury which has occurred in recent large magnitude California events, notably in the 1989 Loma Prieta and the 1994 Northridge earthquakes, indicates that current building codes based on the Blue Book are largely successful in fulfilling the primary objective of maintaining public safety. However, the massive economic losses and interruption of operations at critical facilities points to other risks that remain largely unmitigated. The Loma Prieta earthquake, a M7.1 event, caused only moderate ground shaking in the San Francisco Bay Area. Despite the relatively limited levels of shaking produced, the earthquake caused more than $7 billion in damage with some modern structures, designed to recent editions of the building code, incurring damage so severe as to cause extended loss of use.

The M6.7 Northridge earthquake produced an estimated $20 billion of economic loss. The loss included buildings designed in accordance with modern buildings codes suffering damage which rendered them unusable following the earthquake. Some were damaged so severely that reconstruction was chosen over repair. In other cases, the utility of the building was forfeited after the earthquake even though the structural system performed without damage. The Olive View Hospital, constructed on the site of an earlier building which collapsed in a 1971 earthquake, was designed in accordance with the most rigorous current standards with the intent of remaining functional following any earthquake. Yet, while this building experienced no structural damage in the Northridge event, it was evacuated due to water damage resulting from failed building utilities.

Engineers and public policy makers agree that it is unacceptable to experience this magnitude of loss in relatively frequent and moderate events (Reference No. 2). There is general consensus that the collective focus of the earthquake engineering community needs to improve and broaden from safety, to include damage control and functionality.

VISION 2000

Faced with a need to repair and reconstruct many hundreds of buildings after the Northridge earthquake, the California Office of Emergency Services contracted with the Structural Engineers Association of California to develop recommendations for performance-based design and construction procedures. The Vision 2000 Committee of SEAOC had been previously established with a mission *to develop the framework for procedures that yield structures of predictable seismic performance.* Consistent with the committee's mission and SEAOC's contracted obligations with the State Office of Emergency Services, the Vision 2000 committee has recently developed recommendations for performance-based design and construction procedures (Reference No. 3).

The Vision 2000 report presents the conceptual framework for performance-based engineering of earthquake resistant structures. Performance-based seismic engineering is shown to encompass the full scope of engineering tasks necessary to create structures with predictable and controllable seismic performance, within established levels of risk. This scope of seismic engineering is significantly expanded from current U.S. design practices. It begins with the selection of performance objectives and identification of seismic hazards, continues with conceptual, preliminary and final designs, design acceptability checks and design review and concludes with quality assurance during construction and building maintenance after construction. The methodology is illustrated in a flow chart in Figure 1.

Some of the major steps in the Performance-Based Engineering methodology are described herein.

Performance Levels

Under the concept of performance-based engineering, buildings are explicitly designed and constructed to resist earthquakes of different severities within specified limiting levels of damage. Figure 2 illustrates the entire spectrum of damage states which a building may experience when subjected to ground motions of increasing severity. The Vision 2000 committee has defined 4 individual performance levels in terms meaningful to the building user community:

Fully Operational - A performance level in which essentially no damage has occurred. The building is occupiable and all equipment and services important to the building's basic occupancy and function are in service.

Operational - A performance level in which moderate damage to nonstructural elements and contents, and light damage to structural elements has occurred. The damage is limited and does not compromise the life-safety of the building for occupancy. It would be available for occupancy for its normal intended function immediately following the earthquake, however, damage to some contents, utilities and nonstructural components may partially disrupt some normal functions.

Life-Safe - A performance level in which moderate damage to structural and nonstructural elements and contents has occurred. Egress from the building is not substantially impaired. In some cases, the building would not be viable for immediate post-earthquake beneficial occupancy. The building is likely to be repairable, although it may not be economically practical to do so.

Near Collapse - An extreme damage state in which the lateral and vertical resistance of the building have been substantially compromised. Additional loading could result in partial or total collapse of the structure. The building would be unsafe for occupancy and repair may not be technically feasible.

Table 1 describes permissible levels of damage to the various systems and subsystems which form buildings, for each of the defined performance levels. Table 2 describes permissible damage to the various components which comprise the vertical elements of the typical building lateral force resisting system.

Earthquake Hazards

For any site, there is a continuous spectrum of earthquakes which may affect it, ranging from distant small magnitude events that produce negligible hazards, to local large magnitude events which produce hazards that are potentially highly damaging. Performance-based engineering seeks to control the levels of damage experienced by a building over the full spectrum of events which may occur. In order to permit practical application of this approach, a series of discrete earthquake events has been selected from among the entire spectrum. These discrete earthquakes have been defined as illustrated in Table 3.

Design Performance Objective

A design performance objective is an expression of the desired performance level for the building for each earthquake design level. Design performance objectives are selected based on building occupancy, importance of functions occurring within a building, economic considerations including costs associated with building damage repair and business interruption, and considerations of the potential importance of the building as a historic cultural resource. Minimum performance objectives for buildings of different occupancies and uses have been provided by the Vision 2000 committee. Figure 3 summarizes these recommendations.

Engineering Procedures for Building Structures

Several modified existing design approaches and several new approaches still in development can be applied to performance-based design. The comprehensive design, displacement-based design and energy-based design approaches are powerful tools in development that can directly address the inelastic response of structures and can be used with any performance objectives.

The general force/strength, simplified force/strength and prescriptive approaches are modifications and enhancements of current code practice using elastic or simplified inelastic design procedures. These approaches each have restricted application.

Acceptability analysis is performed with the design to verify that performance objectives are met. Acceptability criteria are set in terms of limiting values of structural response parameters (drift, stress, ductility demand, etc.) associated with the defining damage levels for each performance objective. The acceptability analysis employs various elastic or inelastic analysis techniques to measure the structural seismic response in comparison to the limiting acceptability criteria.

Future Development

Publication of the Vision 2000 report contributes to the development of a conceptual framework for performance-based seismic engineering of structures. Future goals include the mid-term goals of developing the conceptual framework into a guideline document for performance-based seismic engineering and to begin incrementally incorporating the concepts in to the existing seismic design codes. The long-term goal is to draft the guidelines into code form and to complete the overhaul of seismic design practice by the year 2000 to 2005.

ATC-33 GUIDELINES AND COMMENTARY FOR THE SEISMIC REHABILITATION OF EXISTING STRUCTURES

The ATC-33 project is a multi-year effort to provide consensus guidelines for the seismic rehabilitation of existing buildings. The project is a joint effort of the Building Seismic Safety Council (BSSC), the American Society of Civil Engineers (ASCE) and the Applied Technology Council (ATC). The effort is being funded by the Federal Emergency Management Agency (FEMA).

Performance Basis

The ATC-33 team has used performance-based design concepts as the model for the Guidelines document (Reference No. 4). Inherent in the procedure is the selection of a Performance Level to be achieved through rehabilitation of an existing building, termed a "Rehabilitation Objective". ATC-33 permits the user to select any earthquake hazard level, for example, earthquake with return periods of 100, 500 or 1,000 years, and to design to achieve a number of different performance objectives ranging from prevention of collapse to preservation of building suitability for occupancy. A standard safety objective is defined in the Guidelines and recommended, but not required, as a minimum level for design. The standard safety objective includes two levels of design. Under this objective, design is performed to achieve both the life-safety performance level for earthquakes with a 500-year return period and the collapse prevention level for earthquakes with a 2500-year return period. Design to this objective is intended to result in a low, but not negligible, risk of life loss in any earthquake which could affect the building.

For buildings with important post earthquake functions such as hospitals and fire stations, the Commentary recommends an enhanced performance objective. The enhanced objective also includes a two level performance check consisting of the immediate occupancy performance level for earthquakes with a 500-year return period and the life-safety level for earthquakes with a 2500-year return period. The intent of this objective is to result in buildings with a low risk of occupancy interruption and a very low risk of life-safety endangerment in any earthquakes which could affect them.

Analysis

In order to allow determination of the performance level produced by a given earthquake hazard in a specific building design, ATC-33 relies on a "pushover" conceptual model of building behavior. In this model, varying earthquake hazards are envisaged as inducing increasing lateral displacements in the structure. If the elastic and inelastic strength and deformation properties of the individual components which comprise the building are known, then at a given level of induced lateral displacement, it is possible to determine the force demand and also deformation demands on each component. For each performance level defined in the Guidelines, specific acceptance criteria will be provided for both component force and deformation demands for the various beams, columns, walls and other elements which comprise common structures.

Evaluation of achieving a specified performance level for a defined earthquake demand is based on predicting the amount of lateral displacement induced by the demand in the building, identifying the individual component force and deformation demands at this displacement level, and checking against the acceptance criteria provided for the desired performance level.

CONCLUSION

New design approaches for producing buildings which not only protect life-safety, but are also capable of reliably satisfying a number of enhanced performance goals, are becoming available for use. Two U.S. efforts, the Vision 2000 and ATC-33 projects, represent important milestones in the development of the next generation of seismic codes. These efforts employ performance-based engineering concepts with the intent of yielding structures with predictable seismic performance within defined levels of risk and reliability. Through these efforts, building owners, architects, engineers and the financial community will have the opportunity to make informed decisions regarding performance of structures in the event of an earthquake.

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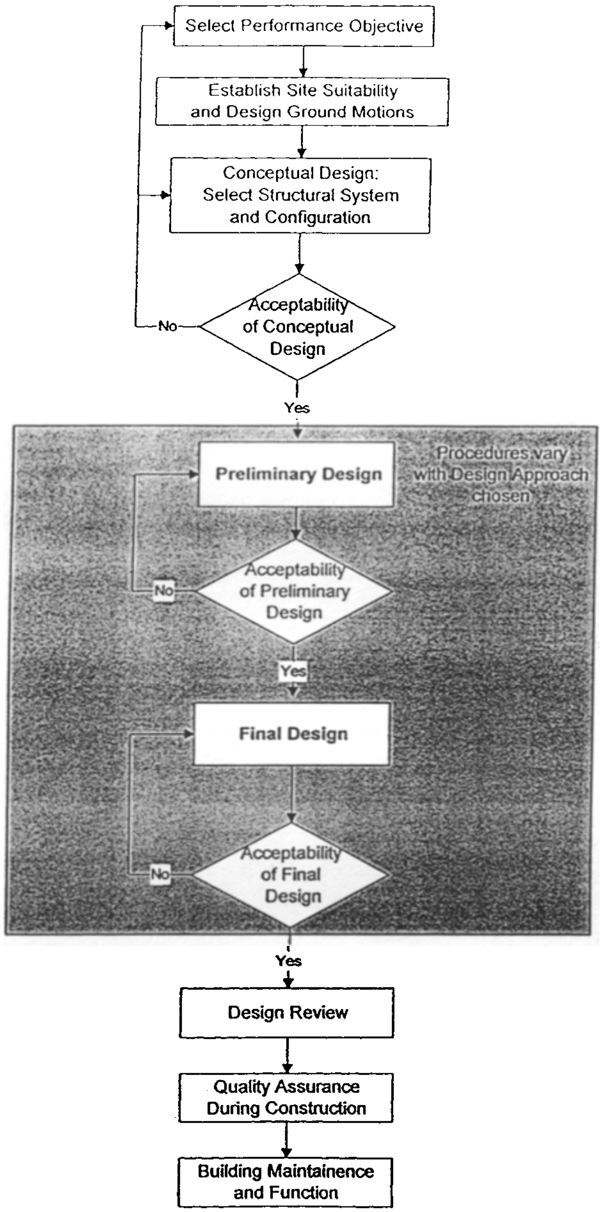

Figure 1: Methodology for Performance Based Engineering

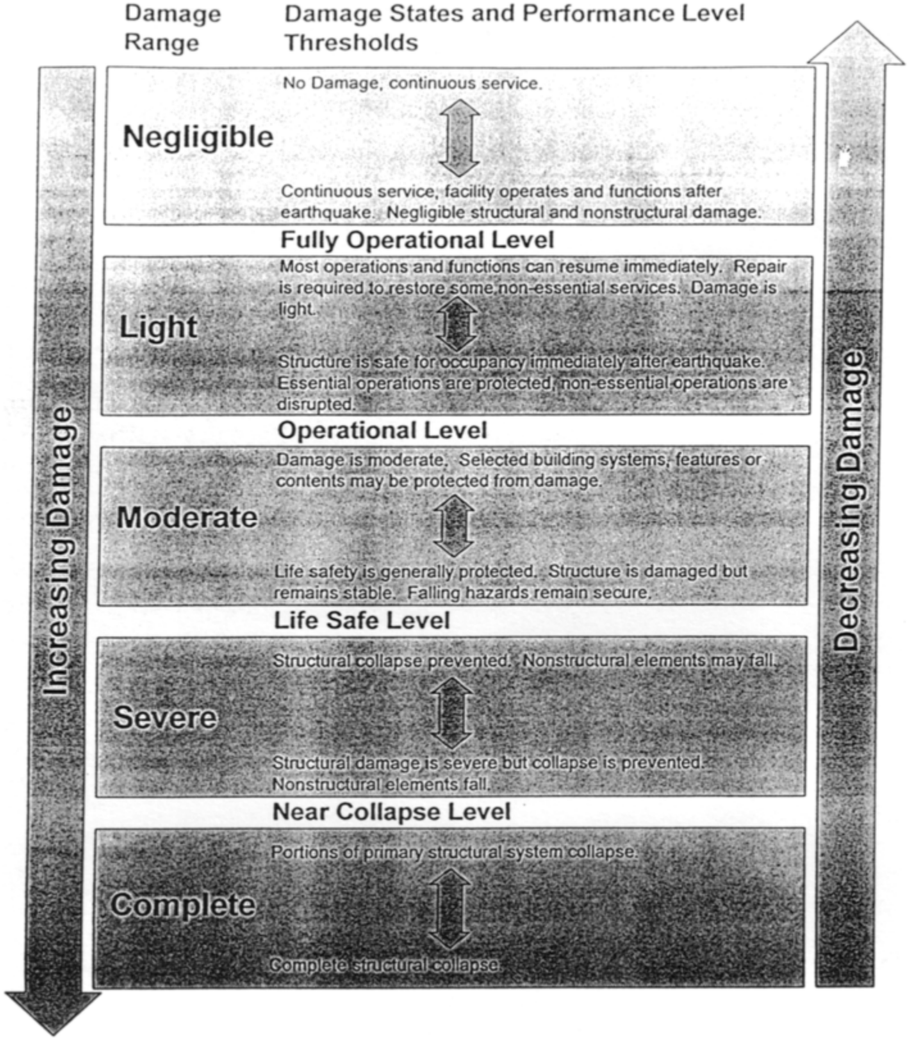

Figure 2: Spectrum of Seismic Damage States

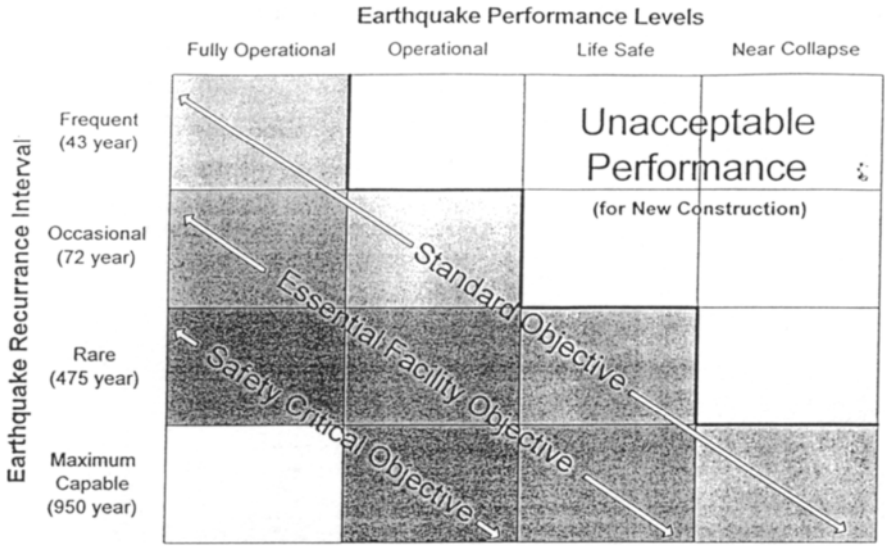

Figure 3: Recommended Seismic Performance Objectives for Buildings

System Description	Performance Level: Fully Operational	Operational	Life Safe	Near Collapse
Overall Building Damage	Negligible	Light	Moderate	Severe
Permissible Transient Drift	<0.2%+/-	<0.5%+/-	<1.5%+/-	<2.5%+/-
Permissible Permanent Drift	Negligible.	Negligible.	<0.5%+/-	<2.5%+/-
Vertical Load Carrying Element Damage	Negligible.	Negligible.	Light to moderate, but substantial capacity remains to carry gravity demands.	Moderate to Heavy, but elements continue to support gravity loads.
Lateral Load Carrying Element Damage	Negligible - generally elastic response; no significant loss of strength or stiffness.	Light - nearly elastic response; original strength and stiffness substantially retained. Minor cracking/yielding of structural elements; repair implemented at convenience.	Moderate - reduced residual strength and stiffness but lateral system remains functional.	Negligible residual strength and stiffness. No story collapse mechanisms but large permanent drifts.
Damage to Architectural Systems	Negligible damage to cladding, glazing, partitions, ceilings, finishes, etc. Isolated elements may require repair at users convenience.	Light to moderate damage to architectural systems. Essential and select protected items undamaged. Hazardous materials contained.	Moderate to severe damage to architectural systems, but large falling hazards not created. Hazardous materials contained.	Severe damage to architectural systems. Some elements may dislodge and fall.
Egress Systems	Not impaired.	No major obstructions in exit corridors. Elevators can be restarted following minor servicing.	No major obstructions in exit corridors. Elevators may be out of service for an extended period.	Egress may be obstructed.
Mechanical/Electrical / Plumbing Systems	Functional.	Equipment essential to function and fire/life safety systems operate. Other systems may require repair. Temporary utility service provided as required.	Some equipment dislodged or overturned. Many systems not functional. Piping, conduit ruptured.	Sever damage and permanent disruption of systems.
Damage to Contents	Some light damage to contents may occur. Hazardous materials secured and undamaged.	Light to moderate damage. Critical contents and hazardous materials secured.	Moderate to severe damage to contents. Hazardous material contained.	Severe damage to contents. Hazardous material may not be contained.
Repair	Not required.	At owners/tenants convenience.	Possible - building may be closed.	Probably not practical.
Effect on Occupancy	Continuous occupancy possible.	Immediate re-occupancy possible.	Indefinite loss of use.	Potential permanent loss of use.

Table 1: General Damage Descriptions by Performance Levels and Systems

		Performance Level			
Elements	**Type**	**Fully Operational**	**Operational**	**Life Safe**	**Near Collapse**
Concrete Frames	**Primary**	Negligible.	Minor hair-line cracking; limited yielding possible at a few locations; no crushing (strains below 0.003).	Extensive damage to beams; spalling of cover and shear cracking (<1/8 ") for ductile columns. Minor spalling in non-ductile columns. Joints cracked <1/8 inch thickness.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.
	Secondary	Negligible.	Same as primary.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.	Extensive spalling in columns (possible shortening) and beams. Severe joint damage. Some reinforcing buckled.
Steel Moment Frames	**Primary**	Negligible.	Minor local yielding at a few places. No observable fractures. Minor buckling or observable permanent distortion of members.	Hinges form; local buckling of some beam elements; severe joint distortion. Isolated connection failures. A few elements may experience fracture.	Extensive distortion of beams and column panels. Many fractures at connections.
	Secondary	Negligible.	Minor yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.	Extensive distortion of beams and column panels. Many fractures at connections.	Extensive distortion of beams and column panels. Many fractures at connections.
Braced Steel Frames	**Primary**	Negligible.	Minor yielding or buckling of braces. No out-of-plane distortions.	Many braces yield or buckle but do not totally fail; many connections may fail.	Extensive yielding and buckling of braces. Many braces and their connections may fail.
	Secondary	Negligible.	Same as primary.	Same as primary.	Same as primary.

Table 2: Performance Levels and Structural Damage - Vertical Elements

Design Earthquake	Recurrence Interval	Probability of Exceedance
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very Rare	970 years (1,2)	10% in 100 years (1,2)

(1) For sites located within 25 miles of an active fault or seismic zone, the hazard parameters for the "Very Rare Earthquake" need not exceed those calculated at a mean +1 confidence level for the maximum deterministic event the fault or source zone is deemed capable of generating based on substantiated, geologic data.

(2) For sites located within zones of low seismicity, the hazard parameters for the "Very Rare Earthquake" should be based on a calculated Maximum Capable Earthquake for the region.

Table 3: Earthquake Design Levels

SOME ASPECTS OF EARTHQUAKE RESISTANT DESIGN IN CANADA AND CHINA

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ABSTRACT

This paper compares the earthquake provisions of the National Building Code of Canada (NBCC 1990) with those of the Aseismic Design Building Code of China (ADBCC 1989). Items dealt with in this examination include: (1) the probability of exceedance and return periods controlling earthquake zoning, and the selection of ground motion parameters in relation to earthquake magnitude and epicentral distance; (2) the assignment of ground motion intensity levels associated with the minor, moderate and major earthquakes contemplated in the design philosophy governing earthquake codes; (3) the evaluation of earthquake loads, including suggested methods of analysis and incorporating the influence of importance of structure, soil-structure interaction, vertical ground motion, and design spectrum; (4) the strength and service load requirements, including control of drift.

A critical analysis of the comparative study is offered. The presentation should be of interest to designers and code formulation bodies. NBCC notation is used whenever possible.

KEYWORDS

Aseismic design code; seismic zoning; ground motion; Canadian code; Chinese code; seismic response; section design; strength; deformation.

INTRODUCTION

The present paper considers recent improvements in the seismic design codes of Canada and the People's

Republic of China, and offers a comparative analysis of the earthquake codes of these two countries.

Seismic zoning maps of Canada and China are based on a statistical analysis of earthquakes experienced by different regions within and adjacent to these countries. Both countries use a probability of exceedance of 10% in 50 years to define zonal ground motion parameters, and assign this probability level to identify a moderate earthquake. Their earthquake design philosophy is also the same, namely that structures in general should be able to:

resist minor earthquakes without damage.
resist moderate earthquakes without structural damage (or with slight, reparable damage).
resist major earthquakes without collapse but with some structural as well as non-structural damage.

To ensure these conditions are satisfied, certain procedures of earthquake analysis are set down in the codes, and structural response limitations as well as structural design details are specified. The latter material occupies more than 60% of ADBCC.

SEISMIC ZONING AND GROUND MOTION PARAMETERS

Canada and China use their own, but analogous, methods to assess seismic risk and develop seismic zoning maps. These methods incorporate historical records and geological and tectonic information.

Characteristics of the ground motion spectrum at a given site depend on earthquake magnitude and epicentral distance, and the response of different structures varies in relation to the earthquake and structural characteristics. Short-period structures experience more damage when subjected to a moderate, nearby earthquake than to a large, distant earthquake. The damage in this case is proportional to the peak horizontal ground acceleration (PHA). On the other hand, long-period structures experience more damage when subjected to a large magnitude, distant earthquake than to a moderate, nearby earthquake. In this case, the damage is proportional to peak horizontal ground velocity (PHV).

The Canadian Code uses both PHA and PHV as zoning parameters for design. When the PHA/PHV ratio is low, the velocity dominates; that is, the sites are influenced by distant, major (large) earthquakes (e.g., Prince Rupert). When the ratio is high, the acceleration dominates, and the sites are influenced by moderate, nearby earthquakes (e.g., Montreal). By using both parameters, NBCC takes into account information on the frequency content of potential earthquakes.

China's historical record of earthquakes goes back more than 2,000 years. Generally these records describe earthquakes by an intensity scale; China has used this historical intensity information, together with modern instrument measurements and geological/tectonic conditions, as an indirect parameter for zoning. China's new intensity scale is similar to the modified Mercalli (MM) intensity scale. By analyzing macro-devastation of earthquakes in China, various approximate relationships between PHA and intensity have been established. For reference purposes, Professor Liu Huixian has provided the following expression:

$$A = 10^{(I\lg 2 - 0.01)} \tag{1}$$

where A = peak horizontal ground acceleration and I = earthquake intensity.

As summarized in Table 1, China's seismic design code, ADBCC, relates PHA to its intensity scale for design purposes. This parameter represents only a single ground motion measurement. In order to account for the influence of earthquake magnitude and epicentral distance on the frequency content of earthquake ground motion, the Engineering Mechanics Institute of the Chinese Academy has statistically analyzed the spectra of the main shock and numerous after shocks of the 1976 Tangshan earthquake. The results, which are presented in Fig.1, show that:

short-period oscillations are predominant for nearby, moderate earthquakes and the period corresponding to the peak point of the response spectra is around 0.1 seconds.

long-period oscillations are predominant for large, distant earthquakes and the period corresponding to the peak point of the response spectra is around 1.0 seconds.

Table 1 Design PHA and α_{max} in ADBCC

level of earthquake	probability of exceedance in 50 years	K and α_{max} for various intensity: ratio of PHA to gravity acceleration, K			earthquake influence coefficient, α_{max}		
		VII	VIII	XI	VII	VIII	XI
minor, frequent event	63.2%	0.04	0.08	0.16	0.08	0.16	0.32
moderate, zonal intensity	10.0%	0.125	0.25	0.50	0.23	0.45	0.90
major, rare event	2-3%	0.25	0.40	0.65	0.50	0.90	1.40

* Earthquake influence coefficient is the product of the ratio of PHA to gravity acceleration K by the maximum magnification factor of the response spectrum (see Section "General approach of NBCC and ADBCC").

** Chinese earthquake intensity scale is similar to MM scale. Here VII, VIII, IX are Chinese intensity scales.

ADBCC describes a set of characteristic periods on the design spectrum; these are similar in concept to "predominant periods". The code specifies how these periods are influenced by earthquake magnitude and epicentral distance. Chinese design spectrum will be discussed in Section 4 of this paper.

The Chinese zoning map contains information concerning the earthquake hypocentre.

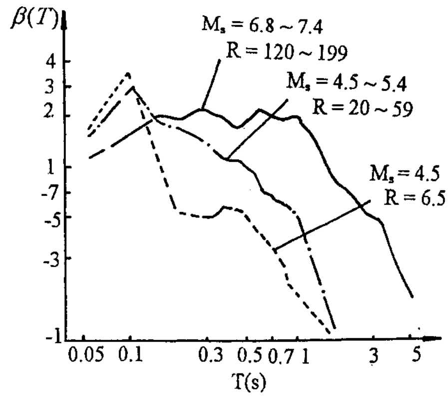

Fig. 1 Configuration of Response Spectra for Various Earthquake Magnitudes and Epicentral Distances

ADBCC also recommends the use of microzoning results, if available and selected ground motion records for the design of important structures.

GENERAL APPROACH OF NBCC AND ADBCC

Various methods for evaluating seismic response of buildings are suggested in NBCC and ADBCC. For example, ADBCC notes that:

1. Low-rise buidings (structures up to 40m in height)having uniformly distributed mass and rigidity and mainly shear-type deformation, or structures whose response is similar to that of a single degree of freedom system, can be analyzed by the base shear method.

2. High-rise and less regular buildings can be analyzed by the elastic modal superposition method.

3. For important structures (Catalogue A buildings in ADBCC), or unusually irregular buildings, a nonlinear time-history dynamic analysis should be performed with selected earthquake records.

The first two methods are based on the design spectrum approach, which is discussed in section 4 of this paper. When the third method is employed, ADBCC specifies that appropriate earthquake records or artificial input be selected with due consideration to earthquake magnitude (or intensity), epicentral distance and geological conditions of the construction site. Furthermore, Chinese code also requires that base shear determined by the third method must not be less than 80% of that evaluated by the first two methods.

The 1990 NBCC specifies that the minimum lateral seismic force, V, be calulated in accordance with the following formula:

$$V = V_e / RU \tag{2}$$

where V_e is an equivalent lateral seismic force representing elastic response. Evaluation of V_e will be discussed later. R is a modification factor reflecting the capability of a structural system to dissipate energy through inelastic behaviour. U is a calibration factor (equal to 0.6). It is applied to maintain the design base shear at the same level stipulated in the 1985 edition of the NBCC for buildings with good to excellent capability of resisting seismic loads consistent with the R factors used.

This format is quite similar to the old 1978 Chinese code, but the 1989 ADBCC provides an improved version. The 1978 Chinese code suggested that $V = CV_e$, where C, defined as the "structure influence factor" or "structure behaviour factor", (see Table 2) reflects the capacity of a structure to dissipate energy during inelastic behaviour, and has the same physical meaning as R.

The 1978 Chinese code used ground motion parameters associated with the basic intensity of a moderate earthquake to evaluate earthquake action. Because the code philosophy anticipated elasto-plastic behaviour under a moderate earthquake, resulting stresses were lower than those obtained by the corresponding imaginary elastic response, and the C factor was introduced to account for this reduction. The factored effects of earthquake action were superimposed with those of other loads. However, this led to a certain confusion in the theory.

Table 2 Structure influence factor C and intensity decrement ΔI

type of lateral load	resisting system	C	ΔI
ductile moment-resisting frame	steel	0.25	2.0
	R.C.	0.30	1.74
R.C. moment-resisting frame	with R.C. wall	0.30 - 0.35	1.74 - 1.51
R.C.	wall	0.35 - 0.40	1.51 -1 .32
unreinforced	masonry	0.45	1.150
hinged bent	steel column	0.30	1.74
	R.C. column	0.35	1.51
	masonry column	0.45	1.32
chimney, water-tank tower, tall but flexible structures	steel	0.35	1.51
	R.C.	0.40	1.32
	masonry	0.50	1.00
timber	structures	0.25	2.00

If we assume that C is used to reduce the ground motion level of a moderate earthquake, rather than the "fictitious" elastic response, we can consider the structure still responds elastically under the reduced level of ground motion. The 1989 ADBCC stipulates that the base shear is given by

$$V = V_{em} \quad (3)$$

where V_{em} is the equivalent lateral force resulting from elastic response to reduced intensity, which is defined as a minor earthquake by ADBCC. A minor earthquake and its ground motion parameters are determined in the following manner.

Assume A is the PHA corresponding to basic intensity I of a moderate earthquake, and A_c is the PHA corresponding to the reduced intensity I_c of a minor earthquake. From our supposition that $A_c/A = C$, the decrement of earthquake intensity can be derived from Eq. (1) as

$$\Delta I = I - I_c = \lg [1/C] / \lg 2 \quad (4)$$

Decrements of intensity for various structures (with various C-values)are listed in Table 2. From the table we note that the reduced intensity is 1-2 scales lower than the corresponding basic intensity.

Basic intensity (a moderate earthquake) is defined as an earthquake having a probability of exceedance of 10% in 50 years. Figure 2 shows the probability density function derived from statistics of Chinese earthquakes. Mean intensity value, I_c, corresponding to frequent events, is about 1.55 scale units lower than basic intensity I. The probability of exceedance in 50 years of mean value intensity I_c is approximately 63.2%. ADBCC defines this as the intensity of a "minor earthquake" and takes the PHA of the minor earthquake as about 1/3 of the PHA of a moderate (basic) earthquake, that is A_c=A/3. This mean (reduced)intensity value corresponds to using C = 0.34 to reduce the basic intensity. The ductility of various structures, or their elements can be quite different. Thus C can differ from 0.34. To account for this variation, and to consider ductility more carefully, ADBCC introduces a regulation factor γ_{RE} in the design process.

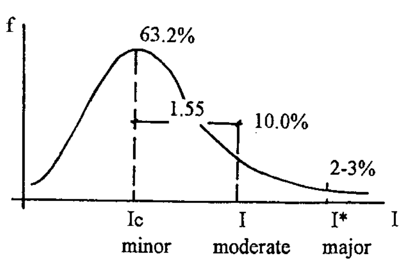

Fig. 2 Probability Density of Earthquake Intensity

Earthquakes which are more severe than the moderate event, but whose probability of exceedance is correspondingly smaller, must also be accounted for in design. ADBCC defines an earthquake intensity with a probability of exceedance of 2-3% in 50 years as a "major or violent earthquake". Using the corresponding parameters of this major earthquake, ADBCC specifies that an elasto-plastic analysis be performed in order to control deformations of structures (especially ductile structures or buildings with weak portions) and to ensure that the collapse of buildings does not occur. Based on the probability of exceedance of various intensities, and the economic conditions in China, ADBCC specifies the PHA of major earthquakes in VII, VIII, and IX intensity zones as 6, 5, and 4 times the PHA of corresponding minor earthquakes for these zones(see Table 1).

Ground motion parameters corresponding to a minor earthquake are used to establish the cross-section properties of a structure. Ground motion parameters related to a major earthquake are used to check the drift and collapse potential of building, while those related to the (zonal) moderate earthquake are used in the selection of construction sites and structural details.

Thus, ADBCC divides the design earthquake intensities for ground motion parameters into three levels -- minor, moderate and major earhtquakes -- and the analysis for seismic response into two stages -- a check for building strength and a check for the control of deformations.

ANALYSIS OF EARTHQUAKE ACTION

Equivalent Lateral Seismic Force and Response Spectra

The main earthquake analysis approaches of both NBCC and ADBCC are based upon response spectrum theory. In Eq. (2), which denotes NBCC provisions for establishing minimum lateral seismic force V, the equivalent lateral seismic force representing elastic response, V_e, is calculated in accordance with the following formula:

$$V_e = \nu \, SIFW \tag{5}$$

ν is the zonal velocity ratio, i.e. the specified zonal PHV expressed as a ratio to 1 m/s, and S is the seismic response factor for a unit value of zonal PHA ratio as defined in NBCC and Fig. 3. Importance factor of the structure, I, foundation factor, F, and gravitational load, W, are discussed later. Note that only the PHV is made explicitly by the NBCC; the PHA is implicitly introduced by means of seismic response factor S, which depends on the fundamental period of vibration of the building. Velocity related seismic zone Z_v and the acceleration related seismic zone Z_a pertain to a particular location.

In ADBCC, horizontal earthquake force corresponding to a single degree of freedom structure is calculated in accordance with the existing formula:

$$V_e = KSW \tag{6}$$

or

$$V_e = \alpha W \tag{7}$$

In these formulae, W is defined as "representative gravitational load"; this load is discussed below. K is the ratio of zonal PHA to gravity acceleration. Because these formulae are used to check the strength of a structure in the first stages of design. K is the value corresponding to a "minor earthquake". S is the seismic response factor for a unit value of acceleration ratio. For design purposes, the Chinese code specifies five normalized S-spectra to account for the site position, earthquake magnitude and epicentral distance.

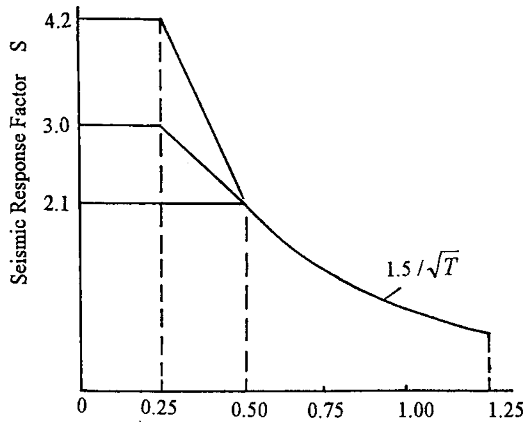

Fig. 3 Seismic Response Factor S of NBCC

The periods corresponding to peak values of the five curves are 0.2, 0.3, 0.4, 0.65 and 0.85 seconds (see Table 3) and peak value for every curve is approximately equal to 2.25. Coefficient α is used directly in ADBCC instead of α and S. α is defined as an earthquake influence coefficient.

Table 3 Characteristic period of site

distance from epicenter	category of site soil			
	I	II	III	IV
nearby	0.20	0.30	0.40	0.65
distant	0.25	0.40	0.55	0.85

$$\alpha = K\,S \tag{8}$$

Although the acceleration ratio, K, varies with zonal intensity, it remains constant for every given intensity.

Therefore, the curves of influence coefficient α are similar to the curves of response factor S.

ADBCC stipulates that the earthquake influence coefficient α be determined from Fig. 4, with consideration given to geology of site, epicentral distance and fundamental period of structure. Maximum values of α listed in Table 1 are obtained from

$$\alpha_{max} = S_{max}\,K = 2.25K \tag{9}$$

The code requires that the minimum value of α shall not be less than 0.2 α_{max}.

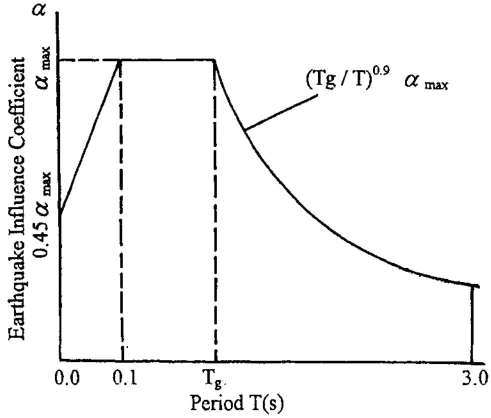

Fig. 4 Seismic Response Influence Coefficient (ADBCC)
T_g - Characteristic Period

Importance Category of Buildings

Both NBCC and ADBCC prescribe special measures to ensure that structures designed for essential public services remain operative after an earthquake. NBCC introduces an importance factor I (see Eq. 4) to specify higher earthquake forces, thereby achieving a greater level of relative safety. The code stipulates that the importance factor I shall equal 1.5 for all post-disaster buildings, 1.3 for schools and 1.0 for all other buildings. ADBCC deals with the problem of important structures by first classifying structures into four categories and then specifying various design methods for the different categories.

From the above, it may be seen that the basic approach followed by NBCC for dealing with important structures is to increase the required seismic design forces. The treatment used by ADBCC differs from the foregoing. ADBCC deals with different classification (importance) levels of structures by specifying selected ground motion records or parameters and by methods of analysis and requirements for details.

Influence of Sub-Soil

Soil conditions at a site have been shown to exert a major influence on the type and amount of structural damage resulting from an earthquake. Aseismic codes account for situations in which influence of soil conditions may play an important role in structural response.

NBCC accounts for soil amplification potential by classflying soil conditions into four types and assigning a foundation factor F to each type. Classification takes into account both the material and depth of the superficial layers. Although ADBCC also classifies the subsoil into four types (and also the site), it does not introduce the concept of a foundation factor. Instead ADBCC changes the characteristic period of the spectrum to account for the predominant period of the site soil.

Soil-structure interaction is not considered explicitly in NBCC and this phenomenon is only dealt with in a perfunctory manner. By comparison, in the Chinese code the analysis of earthquake action is based on the assumption that structures are founded on a rigid base. For the design of high-rise reinforced concrete buildings constructed on type III and IV sites and having concrete box foundations or raft foundations of good rigidity, soil-structure interaction can be accounted for by reducing the resulting earthquake forces by 80-90%, depending on the type of structure and site. Drift calculation of a rigid-based building can also be reduced in the same manner.

Apart from offering some appropriate references outlining the assessment of liquefaction potential of foundation soils, NBCC does not provide any provisions to handle this important material behaviour. On the other hand, Chinese practice deals with this matter by specifying two steps in the assessment of liquefaction potential:

A formula is included in the code to calculate a liquefaction index which is used to assess the potential severity of liquefaction, i.e. to assess depth and scope of liquefaction as well as the thickness of the liquified soil.

Countermeasures are suggested depending on the category of importance of buildings and the liquefaction of their subsoils.

Representative Value of Gravitational Loads W

Earthquake load is the shock-induced inertia force of gravitational load. This gravitational load is normally taken as the total dead load plus that portion of the possible live load can be reasonably expected to be present when an earthquake occurs. ADBCC defines this total combination of loads as "representative gravitational load".

In NBCC the load combination, W, is taken as the dead load plus 25% of the design snow load, 60% of the storage load for areas so used, and the full (load) contents of any tanks.

In ADBCC the value W is a combination of the specific weight of the structure plus live loads which are timed by the combination factors, listed in Table 4.

Vertical Seismic Force

Ground motion during an earthquake is multidirectional and contains a vertical component. In some cases this component may be substantial. In the immediate vicinity of the epicentre, the vertical component can be higher than the horizontal component; at greater epicentral distances its amplitude can be on the order of 30-60% of the horizontal amplitude. Chinese researchers have suggested that the character of the vertical acceleration response spectrum is almost the same as that of the horizontal acceleration response spectrum.

Table 4. Combination factors of gravitational live load in ADBCC

live loads			ψL
snow			0.5
dust of insdustry			0.5
maintenance load on roof			0
floor live load	counted realistically		1.0
	equivalent uniformly distributing	depository of books or archives	0.8
		other civil use and occupancy	0.5
weight hanging from crane		hard hook	0.3
		suspending hook	0

Both NBCC and ADBCC provide some guidance in relation to vertical seismic force. NBCC notes that vertical acceleration may have led to instability or unusual reductions in the safety factors of certain structures. As a result, NBCC points out that dynamic analysis should be employed when vertical accelerations become an important design consideration. In addition to the use of dynamic analysis, ADBCC offers some simplified methods to assess vertical seismic forces.

(l) Vertical seismic forces in tower-type structures, such as chimneys and tall buildings, shall be calculated and then distributed in accordance with the following formulas:

$$V_v = \alpha_{max} W \tag{10}$$

$$F_{xv} = W_X h_X W / \sum_{i=1}^{n} W_i h_i \tag{11}$$

where V_v = specific value of total vertical seismic force; F_{XV} = vertical force applied to level x; α_{max} - vertical earthquake influence coefficient, which is taken as 65% of the corresponding horizontal earthquake influence coefficient in Table 1; w = representative value used to calculate horizontal seismic forces.

(2) Vertical seismic load applied to plate-type girder roofs and trusses with spans greater than 24m shall be equal to the product of $K_V W_V$, where K_V is the vertical seismic force listed in ADBCC.

(3) The specific value of vertical seismic force on cantilevers and other large-span structures shall be taken

as 10% and 20% of its representative value of gravitational load for earthquake intensities of VIII and IX respectively.

STRENGTH AND DEFORMATION OF THE STRUCTURE

Cross-Section Design

Once lateral earthquake loads are evaluated, the resulting bending moments, shears and axial stresses acting on the structural cross-sections can be calculated and combined with the effects of other loads and actions. Stresses on the cross-sections can then be checked to verify whether they satisfy the relevant structural codes.

In NBCC the seismic forces correspond to a "moderate earthquake" and these forces are reduced by considering the ductility appropriate to the structural type. In ADBCC the seismic forces correspond to a"minor earthquake" and these forces are not reduced since the structure must essentially remain elastic during a minor event. The ability of the structure to resist minor earthquakes without damage is verified by checking the stress induced in the cross-section design.

Both NBCC and ADBCC invoke reliability theory when considering load combinations involving earthquakes and other loads. In NBCC this combination is expressed as:

$$S = \alpha_D S_D + \gamma\ \psi\,(\alpha_L S_L + \alpha_Q S_Q + \alpha_T S_T) \tag{12}$$

and in ADBCC as:

$$S = \alpha_D S_D + \alpha_{Qn} S_{Qn} + \alpha_{QV} S_{QV} + \psi_L \alpha_L S_L + \psi_W \alpha_W S_W \tag{13}$$

In Eq.(12),S_D, S_L, S_Q and S_T represent, respectively, the effect caused by specified values of dead load due to intended use, live load due to wind or earthquake and loads due to contraction or expansion. From Eq. (13), note that ADBCC subdivides S_Q into S_{Qn} and S_{QV} representing the effect of horizontal earthquake action and vertical earthquake action repectively. S_W is the effect of a specified wind load, which would only be included for tower-type structures and high-rise buildings; for these cases, the load combination factor, ψ_W, would be taken as 0.2. For other types of buildings or structures this effect could not be included; that is, $\psi_W = 0$.

Load factors α_D, α_L, α_Q and α_T are listed in Table 5-1. The value of the load combination factor ψ (or ψ_L, ψ_W) depends on the number of live load items included in Eq. (12) or (13).

Earthquake load effect S_Q in NBCC is calculated from the parameters of a moderate earthquake ;the effects S_{Qn}, and S_{QV} are derived from the parameters of a minor earthquake.

NBCC also defines an adjustment (importance) factor γ that shall not be less than 1.0, except those buildings for which it can be shown that collapse is not likely to cause injury or other serious consequences; in such cases the factor γ shall not be less than 0.8. As previously noted in Section 3, this represents a special case of the importance factor I, which is included in the base shear equation for calculating earthquake loads.

Unlike NBCC, ADBCC does not combine earthquake load in buildings with loads due to contraction or expansion. Chinese practice only combines earthquake loads with wind loads in the design of tall buildings or

Table 5-1 Load factors for seismic design

type	case	NBCC	ADBCC
		1.25	1.20
α_D	favourable*	0.85	0.80 - 1.00**
α_L		1.50	1.00 - 1.40
	horizontal E/Q load only	1.00 - 1.50***	α_{Eh} = 1.30
α_Q	vertical E/Q load only		α_{EV} = 1.30
	both		α_{Eh} = 1.30 α_{EV} = 0.50
α_T		1.25	

* Dead loads are favourable to stability of the structure, i.e. they resist overturning, uplift or reversal of load effect.

** α_L for gravitational live load shall conform to Table 3.

*** α_Q = 1.00 for earthquake or 1.00 for wind.

structures. NBCC represents S_Q as the effect from wind or earthquake, whichever produces the more unfavourable consequences. It does not consider the combination of earthquake load with wind load.

Earthquake provisions of both NBCC and ADBCC are based on limit state design principles. A building and its structural components are designed to have sufficient strength and stability so that the factored resistance is greater than or equal to the load requirement specified by Eqs. (12) or (13) . This is expressed in ADBCC by:

$$S \leq f / \gamma_{RE} \tag{14}$$

where f is load strength capacity calculated by the appropriate structural code and γ_{RE} is a regulation factor, defined earlier in this paper.

In ADBCC, structures which satisfy Eq. (14) are able to resist "minor earthquakes" without damage, i.e. they remain in the elastic range. Because such structures also possess some ductility, they also are able to resist "moderate earthquakes" (basic intensity earthquake) without irreparable structural damage.

Drift and Separation

Deformation limits, which are imposed to minimize non-structural damage and to avoid collapse, often control the design of multi-storey buildings. Many codes require certain deformation criteria to be satisfied which may be more stringent than the traditional strength approach and may therefore govern the earthquake design, as NBCC's stated principles for checking deformation.

ADBCC requires that the drift be checked not only in terms of the acceptable damage to non-structural components, but also to ensure that the collapse of structures is prevented. To specifically check on drift involves two separate calculations.

(1) Evaluation of elastic drift under "minor earthquake": Elastic incremental drift or inter-storey drift of frame-shear wall structures resulting from a minor earthquake shall satisfy

$$\Delta U_e \leq \theta_e h \qquad (15)$$

where ΔU_e equals elastic incremental drift calculated by taking α_{max} for minor earthquakes (see Table 1); in this calculation all load factors are taken as 1.00. θ_e is elastic incremental drift limitation (see Table 5-2) and h is storey height.

(2) Evaluation of elasto-plastic drift under major earthquakes: Elasto-plastic deformations of soft stories or weak portions of the following types of structures shall be checked with respect to a "major earthquake".

(a) Tall single-storey reinforced concrete columns in bent workshops which are situated on category III or IV sites in earthquake intensity zone VIII, or on any site in earthquake intensity zone IX.

Table 5-2 Elastic incremental drift limitation

Type of Structure	Condition	$[\theta_e]$
frame	incorporation of infilled wall	1/550
	exclusion of any interaction	1/450
frame-shear wall	higher-standard decorated public	1/800
	building ordinary building	1/650

(b) Reinforced category A structures.

(c) Frame structures and masonry buildings having a ground storey of frame structure whose yield strength coefficient is less than 0.50. The yield strength coefficient of a storey is the ratio between storey shear force carrying capacity V_y and storey elastic shear force V_e imposed by a major earthquake:

$$\zeta_y = V_y / V_e \tag{16}$$

where V_y is evaluated from the specific shear strength and actual reinforcement of structural members involved and V_e is determined by elastic analysis under the action of a major earthquake.

Elasto-plastic incremental drift of a soft storey or weak portion of a structure should satisfy

$$\Delta U_p \leq \theta_p h \tag{17}$$

where ΔU_p is elasto-plastic drift calculated by taking α_{max} for major earthquakes (see Table1). ADBCC has suggested a practical method for calculating ΔU_p. θ_p is the elasto-plastic incremental drift limitation (see Table 5-3). ADBCC also allows some conditions under which this limitation may be increased.

Table 5-3 Elasto-plastic incremental

Type of Structures	$[\theta_p]$
bent of single storey workshop with reinforced concrete columns	1/30
frame and infilled frame	1/50
frame of ground storey under upper masonry stories	1/70

Both NBCC and ADBCC intend that all portions of a structure be designed to act as an integral unit in resisting earthquake action. When separation of portions of a structure is desirable or unavoidable, the amount of separation is specified for various structures in the two codes.

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AN ADAPTIVE CONTROL OF SPATIAL-TEMPORAL DISCRETIZATION ERROR IN FINITE ELEMENT ANALYSIS OF DYNAMIC PROBLEMS

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ABSTRACT

The application of adaptive finite element method to dynamic problems is investigated. Both the kinetic and strain energy errors induced by space and time discretization were estimated in a consistent manner and controlled by the simultaneous use of the adaptive mesh generation and the automatic time stepping. Also an optimal ratio of spatial discretization error to temporal discretization error was discussed. In this study it was found that the best performance can be obtained when the specified spatial and temporal discretization errors have the same value. Numerical examples are carried out to verify the performance of the procedure.

KEYWORDS

Adaptive analysis; dynamic analysis; Newmark method, error estimation, discretization error, automatic time stepping; wave propagation; FEM.

INTRODUCTION

Since the early development and application of the finite element method, attempts have been made to obtain the information about finite element discretization errors for better solutions. Accompanying the efforts to evaluate the discretization errors, the adaptive finite element method for more effective solution became one of the popular branches of the finite element method during the last decade. Especially for the finite element analysis of dynamic problems, it is not reasonable to proceed with a fixed mesh and fixed time step as the locations of steep stress regions or damping out of the energy is changing from time to time.

There are two basic issues for the adaptive finite element analysis; i.e. the error estimation and the adaptive control of the error. For elliptic problems the discretization error occurs from spatial discretization and the control of the error is effectively achieved by an adaptive mesh generation. To date a considerable success has been achieved on the problems of elliptic type, such as linear elastostatic problems (Zienkiewicz and Zhu, 1987; Babuska and Reinboldt, 1979). Unlike the adaptive methods in elliptic problems in which only the spatial discretization error of displacement is concerned, more error sources such as the truncation in a time integration have to be considered for hyperbolic problems such as dynamic problems. Therefore for hyperbolic type problems a combined posteriori error estimate including both space and time discretization is needed (Zeng and Wiberg, 1992). The adaptive control of the error performed by the adaptive mesh generation and automatic time stepping has been suggested by Choi and Chung (1994).

There is already a rich literature on adaptive methods for transient problems, but almost all works deal with the estimation and the adaptive control of either the spatial or the time discretization error only. Among the works on adaptive control of the spatial discretization error Probert et al. (1991, 1992) proposed an adaptive finite element method which controls the spatial discretization error to the solution of transient heat conduction problems and compressible flow problems. Zeng and Wiberg (1992) extended an a posteriori error estimator developed by Zienkiewicz and Zhu (1987) for elliptic problems to the dynamic analysis to estimate the spatial discretization error at a certain time and attempted to make the adaptive analysis by an automatic remeshing scheme. Bajer et al. (1991) studied an adaptive technique in the dynamic elastic-viscoplastic problem by space-time elements with moving spatial nodes, where the modification of spatial meshes is made according to an interpolation-based error indicator. Joo and Wilson (1988) solved the structural dynamic problems by an adaptive mesh refinement based on Ritz vectors and the a posteriori energy norm of residual errors.

For the conditionally stable time integration schemes, such as those used in explicit methods, the proper time step is primarily related to the stability criterion, as represented by a critical time step. Methods for automatic selection of time steps for the central difference method have been proposed by Park and Underwood (1980). The situation is more complicated when an unconditionally stable integration scheme, such as the Newmark method, is used. Bergan and Mollestad (1985) suggested an objective criteria for the performance and guidelines for making an adaptive time stepping algorithm for practical applications. Zienkiewicz and Xie (1991) proposed an error estimator by comparing the Newmark solution with the exact solutions obtained from the expanded Taylor series. They also proposed an adaptive time stepping procedure which uses the time discretization error estimator for dynamic analysis. Wiberg and Li (1993) developed a more precise error estimator which can evaluate the errors of displacement and velocities by a post-processing technique.

There is virtually no works reported in the published literature on the simultaneous consideration of the effects of space and time discretization which is desirable for the analysis of transient problems. In this study both the kinetic and strain energy errors induced by the space and time discretization were estimated in a consistent manner. These temporal and spatial discretization errors are controlled by the simultaneous use of adaptive mesh generation and automatic time stepping at every time stage. The optimal ratio of spatial and temporal discretization error to the total error is also discussed.

DISCRETIZATION ERRORS

To solve a dynamic problem by finite element method, the domain of interest is subdivided first into a number of elements. Then the semidiscrete Galerkin approximation can be used to obtain an integral formulation which is usually referred to as the weak form.

After evaluation of the integrals, a set of algebraic equations with initial conditions in a matrix form is obtained as follows.

$$M\ddot{U}_t + C\dot{U}_t + KU_t = F_t, \quad t \in (0,T) \tag{1a}$$

$$U_t(x,0) = U_0 \ , \qquad \dot{U}_t(x,0) = \dot{U}_0 \tag{1b}$$

where M is the mass matrix, C is the viscous damping matrix, K is the stiffness matrix, F_t is the vector of applied forces, and U_t, $\dot{U}_t$, $\ddot{U}_t$ are the displacement, velocity and acceleration vectors, respectively. The aforementioned discretization of continuous spatial domain during the formulation of finite elements induces the spatial discretization error, $e_S(x,t_n)$ which can be written as follows:

$$e_S(x,t_n) = u(x,t_n) - u^h(x,t_n) \tag{2}$$

where $u(x,t_n)$ is the exact solution of dynamic problem and $u^h(x,t_n)$ the solution to semidiscrete Galerkin approximation.

To obtain the transient responses, Eq. (1) is solved with certain time integration scheme. In the direct time integration, the approach is to write Eq. (1) at a specific instant of time $t = n\Delta t$,

$$M\ddot{U}_n + C\dot{U}_n + KU_n = F_n \tag{3}$$

where subscript n denotes the number of time steps, $n\Delta t$ and Δt are the current time and the size of time step, respectively. Generally, when a single-step scheme like the Newmark method is used for the direct time integration, the variation of the acceleration in each time step is assumed to be either constant or linear. This approximation yields a discontinuous distribution for the acceleration in the time domain and induces the temporal discretization error, $e_T(x,t_n)$ as follows:

$$e_T(x,t_n) = u^h(x,t_n) - U_n(x) \tag{4}$$

where $U_n(x^h)$ is the solution to Eq. (3) at time $t = t_n$.

The total discretization error which contains both the spatial and temporal discretization errors in the finite element solution can be expressed as

$$e(x,t_n) = u(x,t_n) - U_n(x) \tag{5}$$

Then, for any choice of norm (Oden and Reddy 1976)

$$\begin{aligned}\|e(x,t_n)\| &= \|u(x,t_n) - U_n(x) + u^h(x,t_n) - u^h(x,t_n)\| \\ &= \|e_S(x,t_n) + e_T(x,t_n)\| \;\le\; \|e_S(x,t_n)\| + \|e_T(x,t_n)\|\end{aligned} \tag{6}$$

Therefore the spatial discretization error and time discretization error can be estimated separately and the upper bound of the total discretization error can be evaluated by adding up both errors.

ERROR ESTIMATES

In this study, for the consistent measure of the temporal and spatial discretization errors, an energy norm is taken. Let the total energy of the body be denoted by

$$E(u,\dot{u}) = \tfrac{1}{2}\left[(\dot{u}, \rho\dot{u}) + a(u,u)\right] \tag{7}$$

And replacing the displacement u by the error of displacement e and the velocity $\dot{u}$ by the error of velocity $\dot{e}$, the total energy of the error is obtained as follows:

$$E(e,\dot{e}) = \tfrac{1}{2}\left[(\dot{e}, \rho\dot{e}) + a(e,e)\right] \tag{8}$$

The square root of $E(.,.)$ defines a energy norm and from Eq. (6) the following energy norm of the error is obtained.

$$E(e,\dot{e})^{1/2} \le E(e_S,\dot{e}_S)^{1/2} + E(e_T,\dot{e}_T)^{1/2} \tag{9}$$

The upper bound of total energy norm of the error can be obtained by adding the energy norms of spatial discretization error and temporal discretization error.

Estimate of Temporal Discretization Error

In dynamic analysis the methods of direct time integration are popular and the choice of method is strongly problem-dependent. Most of the useful implicit methods including the Newmark method are unconditionally stable and have no restriction on the time step size other than as required for accuracy. The Newmark method uses following two basic assumptions:

$$U_{n+1} = U_n + \dot{U}_n \Delta t + [(1-2\beta)\ddot{U}_n + 2\beta \ddot{U}_{n+1}]\frac{\Delta t^2}{2} \quad (10a)$$

$$\dot{U}_{n+1} = \dot{U}_n + [(1-\gamma)\ddot{U}_n + \gamma \ddot{U}_{n+1}]\Delta t \quad (10b)$$

where U_{n+1}, $\dot{U}_{n+1}$ and $\ddot{U}_{n+1}$ are respectively the displacement, velocity and acceleration vectors at time $t = t_n + \Delta t$, Δt is the time step size, β and γ are parameters. The Newmark method contains, as special cases, many widely used practical methods. When the average acceleration method ($\beta = \frac{1}{4}$, $\gamma = \frac{1}{2}$) is used, the variation of acceleration in each time step is assumed to be constant and equal to the average of the accelerations at the two ends of a time step. And when the linear acceleration method ($\beta = \frac{1}{6}$, $\gamma = \frac{1}{2}$) is used, the acceleration is assumed to vary linearly. In fact, since the acceleration varies continuously at the entire time domain, these assumptions yield a discontinuous distribution for the acceleration and the temporal discretization error which can be reduced by the choice of smaller time step size may occur.

Let us consider a time interval $[t, t+\Delta t]$ and assume that $\tau \in [t, t+\Delta t]$. The temporal discretization error of acceleration at time τ is

$$\ddot{e}(\tau) = \ddot{u}^n - \ddot{u}^{ex}(\tau) \quad (11)$$

where $\ddot{u}^n$ is the assumed acceleration and $\ddot{u}^{ex}$ is the real acceleration which varies continuously. Suppose that the solutions at time station t are exact. Then, the time discretization error of velocity solution at time τ and $t+\Delta t$ can be estimated by

$$\dot{e}(\tau) = \int_t^\tau \ddot{e}(\tau')d\tau' \quad (12a)$$

$$\dot{e}(t+\Delta t) = \ddot{u}^{nv}\Delta t - \int_t^{t+\Delta t} \ddot{u}^{ex}(\tau')d\tau' \quad (12b)$$

where

$$\ddot{u}^{nv} = (1-\gamma)\ddot{u}_t + \gamma \ddot{u}_{t+\Delta t} \quad (13)$$

And the error of displacement solution can be estimated by

$$e(\tau) = \int_t^\tau \dot{e}(\tau')d\tau' \quad (14a)$$

$$e(t+\Delta t) = \frac{\ddot{u}^{nd}}{2}\Delta t^2 - \int_t^{t+\Delta t}\int_t^{t+\tau} \ddot{u}^{ex}(\tau')d\tau' d\tau \quad (14b)$$

where

$$\ddot{u}^{nd} = (1-2\beta)\ddot{u}_t + 2\beta \ddot{u}_{t+\Delta t} \quad (15)$$

Since, the exact value of the acceleration $\ddot{u}^{ex}(\tau)$ cannot be obtained in most real problems, it is desirable to approximate the acceleration by a higher order function than the order of the assumed acceleration function in the Newmark method.

Zienkiewicz and Xie (1991) proposed a local error estimator by comparing the Newmark solution with the exact solutions expanded in the Taylor series. However, this error estimator uses the linearly approximated solution as an exact solution. It can only estimate the errors for displacements and cannot measure the errors for velocities since the positive values and negative values of acceleration error cancel out during the integration by Eq. 12.(See Fig. 1a) Accordingly, the error estimate in the total energy norm is not obtainable by this method. Moreover when $\beta = \frac{1}{6}$ and $\gamma = \frac{1}{2}$, the Newmark scheme itself is equivalent to the linear acceleration method in which the acceleration is assumed to be linear. This error estimator can measure neither the strain energy of error nor the kinetic energy of the error. Also if this estimator is used for an adaptive time stepping, the time step size may be changed too frequently. A step size change usually requires an inversion of a new effective stiffness matrix, which is expensive for implicit schemes. Wiberg and Li (1993) derived a formulation for linearly varying third-order derivatives and, based on this, they obtained a posteriori error estimates for displacements and velocities.

In this study, a quadratic function is used for the approximation for $\ddot{u}^{ex}(\tau)$ and the corresponding parameter for the function is obtained from accelerations at three time stations; $t-\Delta t$, t and $t+\Delta t$. This approximation for the exact acceleration gives a more accurate error estimation than a linear approximation of acceleration since it consists of connecting the successive three points as a group on the curve expressed by a second-degree parabola.(See Fig. 1)

A pointwise definition of error, as given in Eq. (11), is generally difficult to use in the measure for adaptive control, and the energy norm is more conveniently adopted. Therefore, as a posteriori error estimate, the energy norm of temporal discretization error can be obtained as follows :

$$E(e_T,\dot{e}_T)^{1/2} = \left(\tfrac{1}{2}\dot{e}_T{}^T M\dot{e}_T + \tfrac{1}{2}e_T{}^T K e_T\right)^{1/2} \tag{16}$$

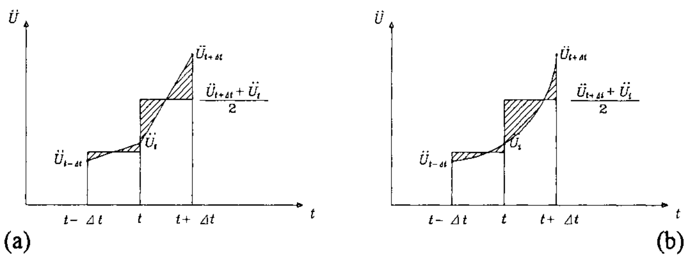


Fig. 1 Accelerations assumed in the Newmark time integration (constant average acceleration) and a post-processed continuous accelerations : (a) Linear and (b) Quadratic approximation

Estimate of Spatial Discretization Error

Among the types of error estimators on spatial discretization, the simpler one presented by Zienkiewicz and Zhu (1987) in plane elasticity problems was shown to be effective. Its application to plate bending analysis using transition element has also been reported (Choi and Park 1992). Ewing (1990) pointed out that the error estimate in strain energy norm for elliptic problems could be extended to the dynamic problems. In addition, Zeng and Wiberg (1992) extended this error estimate to dynamic analysis. According to them, if a sufficiently small time step size is chosen, the total energy of the error is mainly due to the strain energy of the error at most stages. Hence, the total energy norm of the spatial discretization error is given approximately as follows

$$E\left(e_S,\dot{e}_S\right)^{1/2} \cong E\left(e_S\right)^{1/2} \tag{17}$$

Eq. (17) implies that the spatial discretization error in dynamic problems can be approximately estimated by ignoring the kinetic energy of the error. The spatial discretization error estimator $\left\|e_S^i\right\|$ in the displacement for an element i is defined as follows :

$$\begin{aligned}\|e_S^i\| &= \left(\int_{\Omega_i} e_S{}^T K e_S d\Omega\right)^{1/2} \\ &= \left(\int_{\Omega_i} (\overline{\sigma}-\sigma_h)^T D^{-1} (\overline{\sigma}-\sigma_h) d\Omega\right)^{1/2}\end{aligned} \tag{18}$$

where $\overline{\sigma}$ is the smoothed stress which is an approximation to the exact stress, σ_h is the finite element approximation for stress and D is the elasticity matrix. To obtain the continuous stress $\overline{\sigma}$ based on σ_h, the global stress smoothing procedure by the least square method was used in this study. The energy norm of the spatial discretization error for the whole solution domain can be calculated approximately by summing up the squares of the local error estimators in Eq. (18) over individual elements, that is,

$$E(e_S)^{1/2} = \left\{\sum_{i=1}^{n} \left(\|e_S^i\|\right)^2\right\}^{1/2} \tag{19}$$

where n is the total number of elements.

COMBINED ADAPTIVE PROCEDURE

An adaptive analysis should find a discretization in the least cost, such that the local error is uniformly distributed and within a given tolerance over the entire spatial/time domain. To control the relative error which is defined as an error norm of energy divided by the total energy norm, the time step size and the mesh distribution should be modified at the same time based on the local error estimate and the prescribed error tolerance.

From Eqs. (9) and (17) the total energy norm of the discretization error can be estimated by simply combining the energy norms of the spatial discretization error and time discretization error, that is,

$$E(e,\dot{e})^{1/2} \cong E(e_T,\dot{e}_T)^{1/2} + E(e_S)^{1/2} \tag{20}$$

On the other hand, nevertheless the user specifies the relative error tolerance η, and the portion of the spatial discretization error out of the total error and that of the temporal discretization error should also be specified. Accordingly, parameters can be defined as follows

$$\varepsilon = \frac{\eta_S}{\eta} \qquad \text{and} \qquad \delta = \frac{\eta_T}{\eta} \tag{21}$$

where η_S and η_T are the prescribed relative energy norm of spatial and temporal discretization errors, respectively, and the sum of two parameters is 1.0 ($\varepsilon + \delta = 1.0$). Optimal values for this parameters will be discussed in the numerical studies.

The combined spatial and temporal adaptive procedure proposed in this paper can be summarized as follows :

[STEP 1] Carry out the finite element analysis with the initial or previous mesh and the time step until the prescribed termination time is reached.

[STEP 2] Estimate the temporal discretization error and check whether the value of temporal discretization error is within the range of specified error tolerance. If the error is not within that range, change the current time step size and go to [STEP 1]. Otherwise go to the next step.

[STEP 3] Estimate the spatial discretization error and check whether the value of the spatial discretization error is within the range of specified error tolerance. If the error is not within the range, change the mesh distribution and go to [STEP 1]. Otherwise forward one time step and go to [STEP 1]. For practical reasons, when there is no change in the mesh distribution after mesh generation, forward to the next time stage.

Overall adaptive procedure which is proposed in this study is symbolically depicted in Fig. 2.

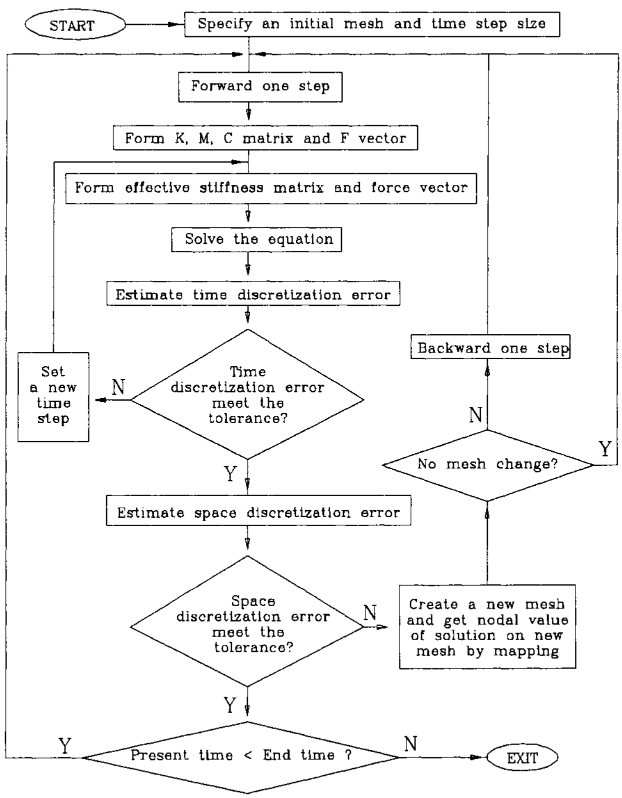


Fig. 2 Adaptive procedure

Control of Temporal Discretization Error

If error estimate does not satisfy the given error tolerance, the time step size is updated according to the local refinement index until the required accuracy is achieved. As many other studies (Zienkiewicz and Xie 1991) in which the error per step is used for error control, a lower error limit and upper error limit are introduced by

$$\gamma_1 \delta\eta \le \frac{E(e_T, \dot{e}_T)^{1/2}}{E(u, \dot{u})^{1/2}} \le \gamma_2 \delta\eta \tag{22}$$

where $0 \le \gamma_1 \le 1$ and $\gamma_2 \ge 1$ are two parameters. Whenever Eq. (22) is satisfied, the solution is accepted and the time integration is proceeded to the next step without changing the time step size. However, if the upper limit is violated, reject the solution, update the time step size and perform a re-calculation for the current time.

For the Newmark integration, the rate of convergence of the local error should achieve $O(\Delta t^3)$. Therefore when Eq. (22) is not satisfied a new time step size $\Delta t'$ may be predicted as

$$\Delta t' = \left(\frac{\delta\eta E(u,\dot{u})^{1/2}}{E(e_T,\dot{e}_T)^{1/2}} \right)^{1/3} \Delta t \tag{23}$$

where Δt is the current time step size.

Control of Spatial Discretization Error

For each time station, the estimation of spatial discretization error as described in the preceding section is made first. If the given error tolerance is not satisfied, the mesh is updated according to the local refinement parameter until the required accuracy is achieved. In order to perform the computation more efficiently and economically for a given tolerance, a lower error limit and an upper error limit are introduced as

$$\gamma_1 \varepsilon\eta \le \frac{E(e_S)^{1/2}}{E(u,\dot{u})^{1/2}} \le \gamma_2 \varepsilon\eta \tag{24}$$

If this condition is not satisfied, the spatial mesh needs to be updated and a re-calculation for the current time should be performed. Based on the optimal mesh hypothesis, in order that the energy norm of the error for an element should be within a prescribed error bound, the critical error is defined as

$$\left\| e_S^c \right\| = \frac{\varepsilon\eta E(u,\dot{u})}{\sqrt{n}} \tag{25}$$

This critical error measure provides a refinement criterion, that is, any element for which the energy norm of the error calculated from Eq. (18) is greater than the critical error norm should be refined. A new element size required over the domain of each present element can be predicted by the use of the well-known fact that the convergence rate of the error is $O(h^p)$ for optimal mesh. Thus in each subdomain the predicted mesh size h_k^a is required as

$$h_k^a = \left(\frac{\left\| e_S^c \right\|}{\left\| e_S^i \right\|} \right)^{1/p} h_k \tag{26}$$

where h_k is existing mesh size.

Because of the mesh modification by adaptive methods, the value of all history dependent variables must be remapped to the location of new nodal points. The variables defined at nodal points, such as accelerations and displacements, can be easily remapped using the shape functions of an element such that

$$x(a) = N_i(a)\, x_i \tag{27}$$

where $x(a)$ is the value at the location of new nodal points, x_i is the value at the i th node (in local numbering) of the element to which the new point belongs and a is the position of the new point in the parametric coordinate system. If the problems without damping subjected to a shock-like external load and the corresponding meshes are modified too frequently or abruptly, the remapping error becomes considerably large.

There are generally two approaches for the mesh modification in an adaptive analysis. The one is the element refinement/coarsening approach and the other is remeshing approach. With the element refinement/coarsening approach, the element that violates the allowable error range is subdivided into smaller elements or merged into a bigger element. The remeshing approach involves completely regenerating a new mesh, either in regions of high error only, or over the entire domain. Unlike the element refinement/coarsening approach, the remeshing approach changes the location of the whole nodes. Therefore, from the view point of minimizing the remapping error, the element refinement/coarsening approach is more efficient than the remeshing approach. For this reason, the element refinement/coarsening approach is used in this study.

NUMERICAL EXAMPLES

A single-degree-of-freedom problem was solved to evaluate the accuracy of the proposed time discretization error estimate. And to demonstrate the performance of the proposed adaptive procedure, uni-dimensional tests were carried out since the results can be verified intuitively with ease. Several test problems with different error tolerances η, and parameters ε and δ were analyzed to obtain the reasonable value of ε or δ.

Estimate of Time Discretization Error

To investigate the performance of the proposed time discretization error estimate, a SDOF model (Fig. 4) for which an exact solution can be obtained analytically is considered. This dynamic system subjected to a sinusoidal loading can be presented by the following second order differential equation:

$$\ddot{d} + 0.19596\dot{d} + 6d = \sin t \tag{28}$$

When the Newmark integration step forwards from a time station t_n to t_{n+1}, the exact local error of displacement and velocity for this time step is obtained respectively by

$$e_{ex} = d_{n+1}^N - d(t_{n+1}) \tag{29a}$$

$$\dot{e}_{ex} = \dot{d}_{n+1}^N - \dot{d}(t_{n+1}) \tag{29b}$$

where d_{n+1}^N and $\dot{d}_{n+1}^N$ are respectively the displacement and velocity given by Newmark's scheme at time t_{n+1}, and $d(t_{n+1})$ and $\dot{d}(t_{n+1})$ are obtained by integrating Eq. (28) analytically from t_n to t_{n+1} taking the values d_n^N and $\dot{d}_n^N$ as initial conditions (Zeng et al. 1992). The relative error distributions for the energy norm with different time step sizes and integration schemes are shown in Figs. 4 and 5. Noting that the proposed estimate gives an accurate measure of the relative energy norm of exact error induced by time discretization, this scheme can be used in the following example.

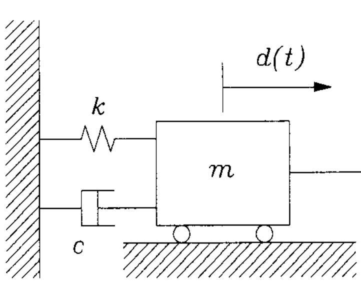
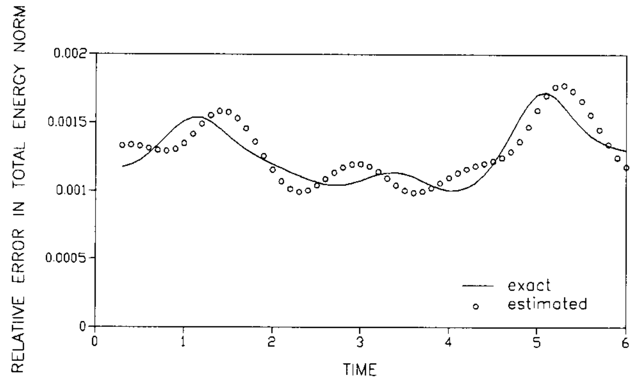
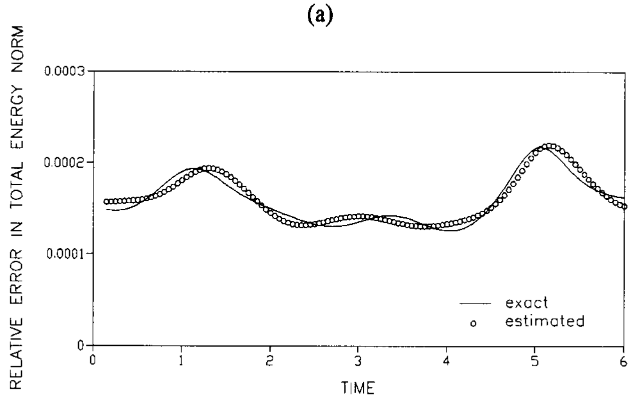


Fig. 3 A single-degree-of-freedom model

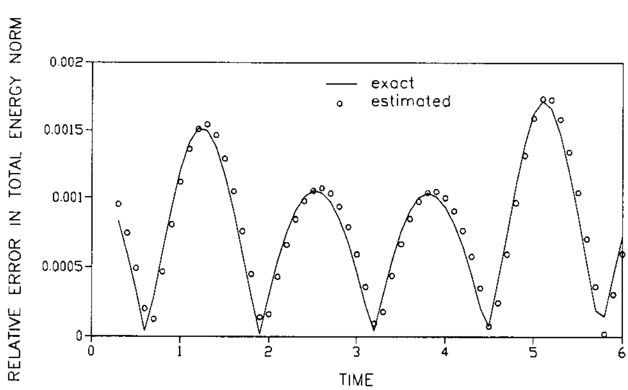


(a)

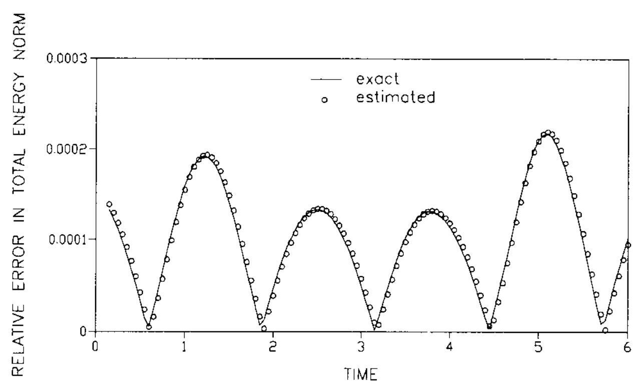


(b)

Fig. 4 Relative error of energy norm for constant average acceleration method : (a)$\Delta t = 0.1$; (b)$\Delta t = 0.05$



(a)



(b)

Fig. 5 Relative error of energy norm for linear acceleration method : (a)Δt =0.1 ; (b)Δt =0.05

Adaptive Analysis of One-dimensional Problem

An one-dimensional elastic bar (Fig. 6) subjected to a sinusoidal pulse is analyzed to demonstrate the effectiveness of the proposed adaptive analysis procedure. The sinusoidal pulse is given by

$$f(t)=\begin{cases}\sin^2 8\pi t & \text{if } \ 0\le t\le 0.125\\ 0 & \text{if } \ \ t>0.125\end{cases} \tag{30}$$

The Rayleigh damping is assumed and Rayleigh damping coefficients are set to 0.02.

Two uniform meshes and constant time step sizes are used for the conventional analysis. The first one which consists of 300 degrees of freedom is analyzed with a constant time step size of $\Delta t = 0.01$. In the second, a mesh consisting of 30 degrees of freedom is used and the time step size is set to 0.03(Table 1). And as an adaptive analysis, the dynamic responses in a time interval of]0.0, 10.0[are to be computed with parameters $\gamma_1 = 0.5$, $\gamma_2 = 1.5$ and $\varepsilon = 0.5$ $(\delta = 0.5)$ and two different relative error tolerance $\eta = 0.01$ and 0.02. Cases E and F are partial adaptive analyses with the control of time discretization error only and space discretization error only, respectively. Detailed conditions and computing times of each cases are shown in Tables 1 and 2. It should be noted that the optimal values of and . are not known yet. In Fig. 7, it can be observed that the shape of stress wave varies from a sharp peak to a rather smoothed shape as the energy is being damped out. Also observed are the reflections of stress wave at the free and fixed ends. In Figs. 8 and 9, the variation of time step sizes and the meshe adaptation for two different cases are plotted. The element refinement/coarsening algorithm used in this study is not affected by the initial meshes which the user defined (Cases C and D) and both cases show a virtually the same variation of meshes after a first few iterations. The relative errors adaptively controlled within the allowable range are shown in Figs. 10 and 11.

The comparison between the solutions of conventional and adaptive analysis is shown in Figs. 12 and 13. It is noticed that the adaptive analysis gives more accurate results than conventional analysis with same computing time. The Newmark's average acceleration method is non-dissipative and unconditionally stable, but its accuracy in the transient analysis of wave propagation problems depends not only on the spatial discretization but also on the temporal discretization (Wang, Murti and Valliapan 1992). Figs. 12 and 13, show that the solutions of Case C (simultaneous adaptive control of spatial and temporal discretization errors) are more accurate than the solutions with either of Case E (adaptive control of temporal discretization error only) and Case F (adaptive control of spatial discretization error only). Therefore the simultaneous control of spatial and temporal discretization errors is more practical than the adaptive control of only one of those errors.

Table 1 Conventional analysis

	NDOF	Time step size	Computing Time(sec)
Case A	300	0.01	592.09
Case B	30	0.03	24.16

Table 2 Adaptive analysis

	η	ε	δ	Initial NDOF	Initial time step size	Computing time(sec)
Case C	0.02	0.5	0.5	100	0.01	19.11
Case D	0.01	0.5	0.5	20	0.01	42.51
Case E	0.01	-	1.0	70	0.01	21.91
Case F	0.01	1.0	-	20	0.03	21.10

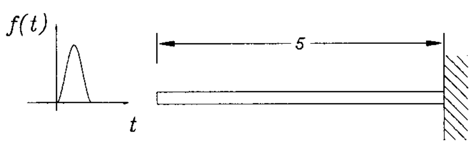


Fig. 6 One-dimensional model (Material properties are : $E = 10.0$, $A = 10.0$, $\rho = 5.0$)

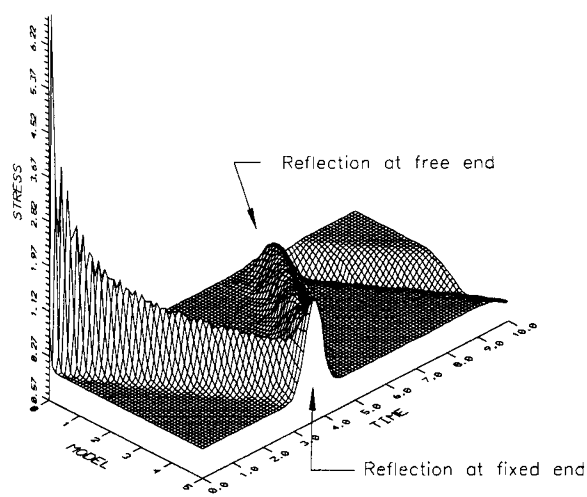


Fig. 7 Stress wave propagation

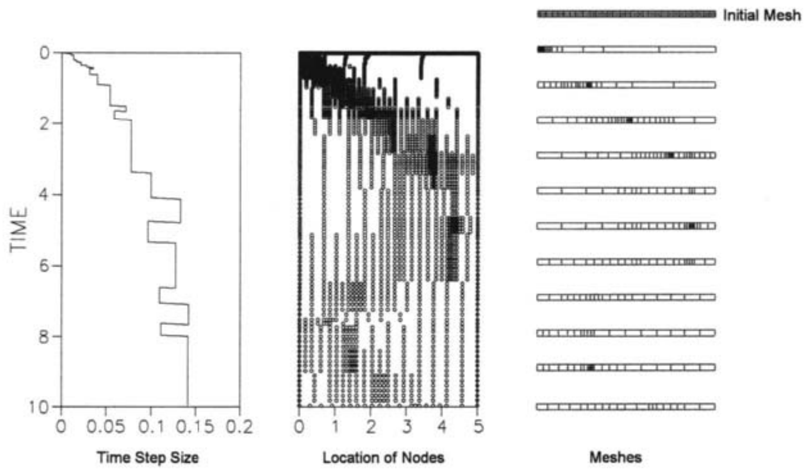


Fig. 8 Variation of time step sizes and meshes in the bar (Case C)

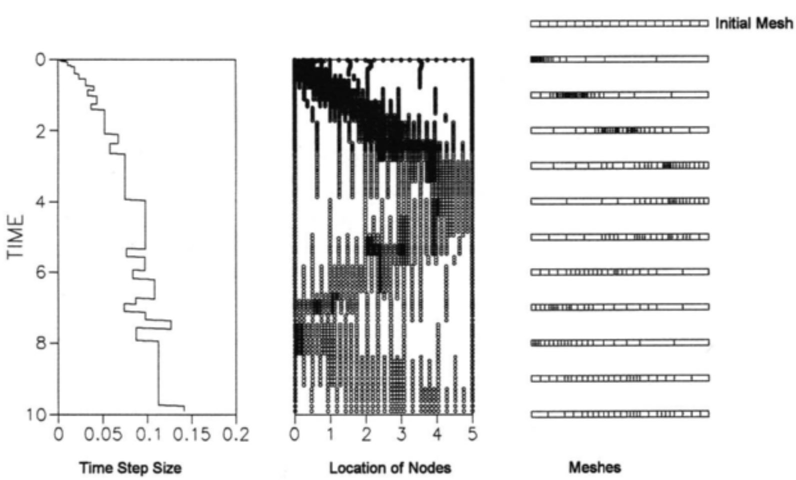


Fig. 9 Variation of time step sizes and meshes in the bar (Case D)

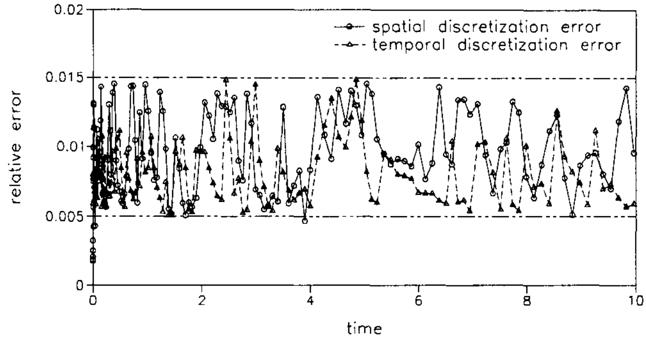


Fig. 10 Relative error achieved in the adaptive computation (Case C)

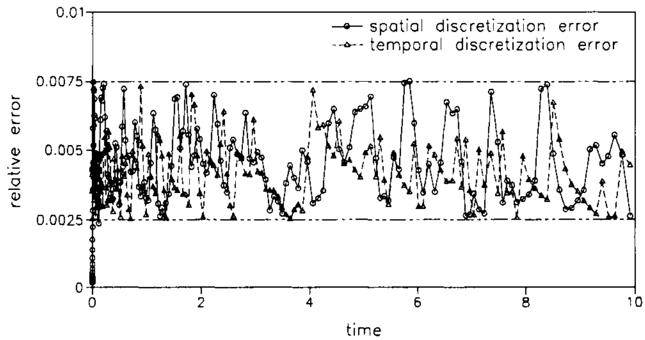


Fig. 11 Relative error achieved in the adaptive computation (Case D)

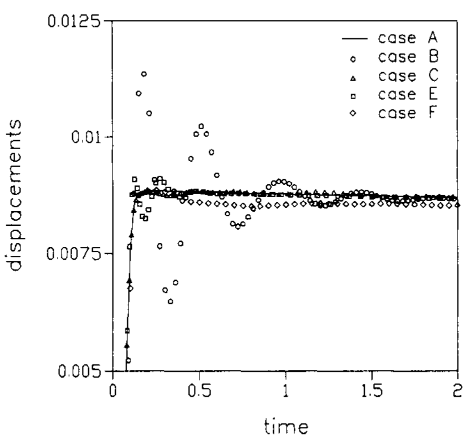


Fig. 12 Comparison of displacements at the free end, $x = 0.0$ in time

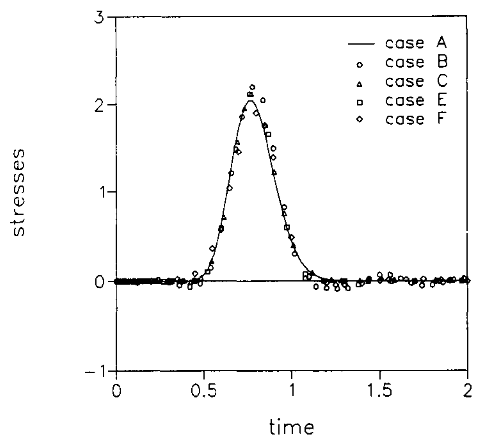


Fig. 13 Comparison of stresses at $x = 1.0$ in time

The Optimal Value of the Parameter, ε *or* δ

In order to find the optimal value of ε or δ, a parametric study on the problem used in previous section is carried out with various ratio of spatial and time discretization error tolerance. In Fig. 14 the relationship between the total computing time and ε or δ is shown. It is seen that when a proper value of ε or δ is selected, the total computing time can be reduced greatly at the same error tolerance. In this study it can be observed that the reasonable value of ε or δ is about 0.5 in the aspect of computing time which is also the value used in the preceding example. For two and three dimensional problems, however, it would not be the case because, unlike the control of the temporal discretization error, the control of the spatial discretization error is more expensive and time consuming than the one dimensional case.

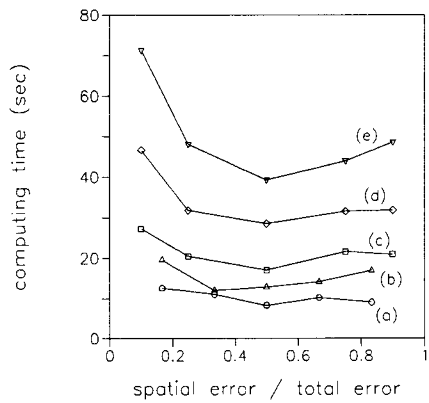


Fig. 14 Comparison of computing time : (a) $\eta = 0.12$; (b) $\eta = 0.06$; (c) $\eta = 0.04$; (d) $\eta = 0.02$; (e) $\eta = 0.01$

CONCLUSIONS

In this study the spatial discretization error and the temporal discretization error were estimated consistently and an effective algorithm which controlls the errors automatically and simultaneously by adaptive modification of the mesh distribution and time step size is proposed. In such a way, the best performance attainable by the finite element analysis of dynamic problems can be obtained.

The temporal discretization error can be estimated simply by comparing the solution obtained by the Newmark method with solutions obtained by the locally exact quadratic function. This error estimate converges to the exact error as the size of time step is decreased. Since the error estimate by quadratic function is not affected by the specific time integration method used, the temporal discretization error estimate proposed in this study can be applied to the various single step method. The remapping error occurred in control procedure of spatial discretization error becomes considerably large for the problems without damping. Therefore more extended studies are needed on the remapping technique. Based on the parametric studies a reasonable ratio of spatial and time discretization errors, which should be specified by the user in an adaptive analysis of dynamic problem is also proposed.

The adaptive procedure in this study can be extended to two dimensional and three dimensional problems with minimum modification. For this, however, the use of an effective automatic mesh generator is recommended.

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IMPROVEMENT OF EARTHQUAKE-TSUNAMI WARNING SYSTEMS AND HUMANWARE MANAGEMENT

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ABSTRACT

In 1993 and 1994, we had two earthquakes in Hokkaido island, Japan. Both earthquakes accompanied with tsunamis. From our field study, radio communication of tsunami warning systems and issue of evacuation are inadequate in almost every local area due to lack of the information facilities transmitted to individual residence and misunderstanding on tsunami behavior in local government. We have sequential events in the process of the communication in which every event may introduce and enlarge the disaster damage. In order to mitigate the human damage due to immediate coming tsunamis or well-prepared tsunamis after earthquake, it is necessary to arrange tsunami evacuation manuals in every local areas. Numerical simulation technique is very useful to give practical information about the tsunami behavior such as the highest tsunami and the shortest arrival time. Disaster management is also very important to mitigate tsunami damage. This was composed with hardware, software, humanware and commandware management. A disaster coordination system was also proposed.

KEYWORDS

Disaster management; software; humanware; tsunami warning; evacuation manual; Great Hanshin-Awaji Earthquake Disaster; Disaster lesson; Hokkaido Toho-Oki Earthquake; Hokkaido-Nansei-Oki Earthquake

INTRODUCTION

The 1993 Hokkaido Nansei-Oki Earthquake (M=7.8) occurred on 22:17 of 12 July and caused huge tsunami damages along its southwest coast facing the Japan Sea, in particular in Okushiri island as shown in Fig. 1. Tsunami attacked this area on around 22:22, only five minutes after the earthquake. For the issue of tsunami warning by JMA (Japan Meteorological Agency), it took about ten minutes after earthquakes. It includes tsunami height, announcement of tsunami warning to local governments and mass media and communication to residents. Therefore, in the tsunami-prone area with the threat of immediate coming tsunamis, it is necessary for residents to evacuate by themselves without waiting for announcement of tsunami warning. However, at the present time, any evacuation manual is not prepared in the case of immediate coming tsunamis. Moreover, it is rather difficult to expect for residents to evacuate quickly because of lack of knowledge of tsunami characteristics.

The Hokkaido Toho-Oki Earthquake(M=8.1) hit on 22:23 of 4 October and moderate size tsunami came at cities and towns located along the Shiretoko peninsula and Kushiro district after around 20 to 30 minutes of the occurrence of the earthquake as also shown in Fig. 1. We didn't have much damage due to the tsunamis, but many problems were appeared in spite of well-prepared tsunamis. At that time, JMA had improved tsunami warning systems and issued the warning with three minutes after the earthquake. However, local governments could not well manage this tsunami warning with several reasons. Though the electric current was cut off due to the earthquake, many residents about 30,000 took a quick refuge to public shelters such as school buildings and community centers. Most of them were locked to prevent robbery and some locate at the coastal lowland areas which were not enough safe against tsunamis. We introduce the time-dependent event.

Finally, the 1995 Great Hanshin-Awaji earthquake disaster(M=7.2) came about on 5:46 of 17 January. The disaster was the worst after world war II in our country. The death and missing toll was 5,504, the number of the injured was more than 39,000 and estimated property damage was more than 135 billion US dollar. During this 30 years, we have never had the natural disaster with the death toll of more than 300. And also this is truly urban disasters which has never been experienced in our highly informed urban areas. Therefore, our disaster management in relation to humanware and commandware were very poor in local government level as well as central one. After five month of the disaster, about 20,000 residents were still obliged to stay at shelters. We would like to introduce disaster management and lessons given by the catastrophic disasters.

COMMUNICATION OF TSUNAMI WARNING AND DAMAGE-ENLARGEMENT FACTORS

Tsunami Warning Systems and Their Problems

In order to make clear the actual conditions of disaster prevention radio communication systems we did field survey in the damaged areas including Okushiri island. It was found that only two of 16 local governments prepared this system completely. This is general in our country. Many political leaders of local governments can not understand to the importance of software and to facilitate disaster prevention radio communication systems. This is due to not only financial problems but also lack of long-term perspective on disasters with low frequency.

In the flow chart of issue of tsunami warning and its communication, we have several uncertain factors as shown in Fig. 2. They may lead to enlargement of damages due to tsunamis and are summarized as follows:

1) In the case of combination of small earthquakes and big tsunamis (this is defined as tsunami earthquake. Historically, on the Sanriku coast, we have had this type of tsunami disaster), tsunami warning is not always issued due to difficulty of numerical prediction. For this problem, the development of seismology is expected to solve the mechanism of earthquake energy and tsunami generation.

2) Warning information of tsunami characteristics are very rough such as tsunami at the height of more than 3m or less than 3m and big area division (our land is divided only 18 areas). Moreover, heads of local governments such as mayor, town head and village chief have insufficient knowledge about tsunamis. This leads to delayed announcement of evacuation and late preparedness for recovery. One of the solutions of those problems is that JMA issues more accurate information of tsunami characteristics in every local area.

3) Inadequate communication of tsunami evacuation. Communication facilities and evacuation manuals have to be prepared in every area.

4) Even if recommendation or order of evacuation is timely issued, it is impossible for all residents to respond to it quickly because of lack of information about tsunami behavior.

5) At present, every local government specifies public refuge sites and buildings. Some of them locate at low-lying areas and evacuation routes are not completed. The most routes are very narrow without lighting at night. Moreover, residents want to use cars to take refuge, but local governments prohibit it. At night, every public buildings are locked up so that people can not enter into them at once.

Human damage occurs at every sequential event of 1) to 5). In order to mitigate the damage, scenarios for improvement have to be made clear and the efforts should be continued with financial support of our government. In the case of immediate tsunamis which come at certain areas within several minutes after earthquake, it is necessary to prepare evacuation manuals for quick action of every resident. The following studies are based on accurate knowledge about tsunamis.

1) Declaration of dangerous areas due to tsunamis
2) Spread of knowledge about tsunamis and their disasters
3) Estimation of plausible tsunami variables such as the highest tsunami and the shortest arrival time

The 1993 Hokkaido Nansei-Oki Earthquake and Tsunami Disaster

Due to the tsunamis, 231 residents were killed. Most victims lived in Okushiri island as shown in Fig. 1. In the island, the first attacking tsunami at the height of 10m came within five minutes after the earthquake. In 1983, we had Nihonkai-cyubu earthquake tsunami and one fisherman was killed in this island. As the countermeasure of the aftermath, the crown height of the coastal dike was at the height of T.P.+5.5m

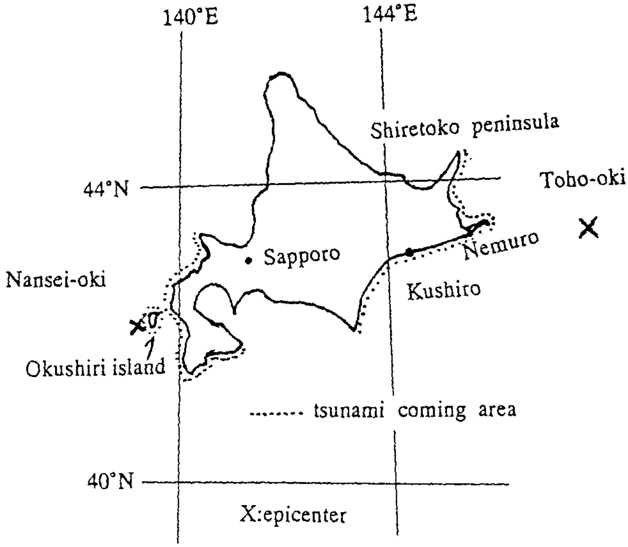


Fig. 1. Location of the epicenters of the 1993 Hokkaido Nansei-Oki and the Hokkaido Toho-Oki earthquakes and tsunamis attacked areas

Occurrence of earthquake

① In the case of tsunami earthquake, tsunami warning is not always issued due to difficulty of prediction.

Issue of tsunami warning

② Tsunami information is not clear in the warning

Issue of recommendation of evacuation

③ Communication system is insufficient.

④ Some residents do not obey with the recommendation.

Beginning of evacuation

⑤ Evacuation site and route are inadequate against tsunami.

End of evacuation

Fig. 2. Flow chart of communication of tsunami warning

(roughly, Tokyo Pile means around M.S.L. and tidal range is about 30cm in this area.) which was 1m increase in comparison with the old one. Before getting great tsunami warning (attached "great" means that the tsunami height is more than 3m.) issued by JMA, tsunami came. Many residents quickly responded to this earthquake due to the lessons from the 1983 tsunamis, but in Aonae area which locates at the south end of the island with dense population, roads were narrow and distance from edge of the residential area to the behind hill were about 500m. Without streetlight and with traffic jam of evacuation cars, many people were killed on their way to the hill.

Disaster Countermeasure Act was enforced in 1961 in Japan. This was established through the catastrophic victims of 5101 due to storm surges generated the storm surges due to Ise Bay typhoon. At that time, our society was poor to improve the function of infrastructures such as river levee and tsunami breakwater. Moreover, we have never had catastrophic disasters for more than three decades after the Ise Bay disaster. Automatically, public investment of our governments for disaster prevention has decreased year by year. This Act does not promote to use financial budget for preceding disaster preparedness and is effective only for recovery from the actual disaster damage. According to the Act, every local government prepared disaster prevention planning which have never been revised in most local governments because of no experience of catastrophic disasters. The disaster prevention planning became an armchair theory.

The 1994 Hokkaido Toho-Oki Earthquake and Tsunami Disaster

After the earthquake, we were waiting for tsunamis, because we have much time to prepare tsunami coming due to long distance between the epicenter and east area of Hokkaido island. Tsunami warning was quickly issued by JMA but many residents about 30,000 took a refuge before getting the warning. This is due to the lesson from the 1993 Hokkaido Nansei-Oki earthquake. TV program presented miserable scenes and dreadfulness of gigantic tsunamis at the moment of the disaster.

At Kushiro city which is the largest in this district with the population of more than 150,000, the city mayor issued the evacuation recommendation about five hours after the tsunami coming. Most local government officers believed that huge tsunamis do not attack Kusiro city from historical view point. Fortunately, tsunami height was around 1m so that the downtown area was only inundated at the depth of 10 to 20cm. The tsunami coincided with high water. If we had big tsunamis, the damages might reach to unbelievable rate. The city headquarters could not understand the tsunami behavior, but after that they said at the city assembly that they would like to avoid the panic. In our long history of natural disaster, we have never had a panic. At some towns, the tsunami watergate could not be closed. In ghost towns due to all residents' evacuation to the safer place, so many tracks and cars were driven on the national coastal road. This is a typical sectionalism. In this case, the national load are controlled and maintained by the Kushiro road office. This belongs to our central government, therefore local government can not stop the traffic even if under tsunami warning. However, the office director can not get any information about the local conditions of tsunamis even if disasters in daylight working time . This is a typical example of the lack of emergency management in our country.

TSUNAMI EVACUATION MANUALS

When we feel earthquakes and are anxious about tsunami coming, quick evacuation is promoted under accurate knowledge about the tsunamis which are characterized under local conditions. Fortunately we have long history of huge tsunami disaster in which the location of epicenter and magnitude of earthquakes were analyzed. Figure 3 shows the supposed epicenters of earthquakes with magnitude of more than eight along Nankai Trough in which the Philippine Sea plate has subducted under the Eurasia plate at the annual rate of around 5cm since five million years ago. The disasters occur about 100 to 150 years interval as shown in Table 1. Numerical simulation can make tsunami arrival maps and changes of the tsunami height as typically shown in Fig. 4(a) and (b).

In Fig. 4(a), the most inner semicircular shaded area shows the minimum arrival time of tsunami is less than 10min. This area can be decided with results of numerical calculation of tsunami propagation in which the epicenter is shift in the domain of more than 200km in the east-west direction along Nankai trough. Therefore, residents living in this predicted area have to take refuge just after earthquake. Due to low frequent disasters, historical experiences and regents of residents are not always adequate to judge the tsunami behavior. Residents have to take a quick refuge after the big disaster. Residents who live in the most outer semicircular area in Fig. 4(a) have more than 20 min. to have tsunamis after the earthquake. They can judge with the tsunami information which will be issued by JMA after the earthquake. In Fig. 4(b), the changes of the highest water level at Tanabe city is depicted. It was found that the height changes from 1 to 3.4m with

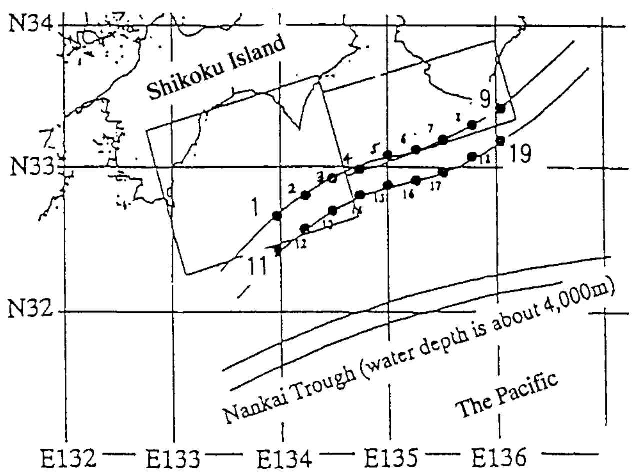
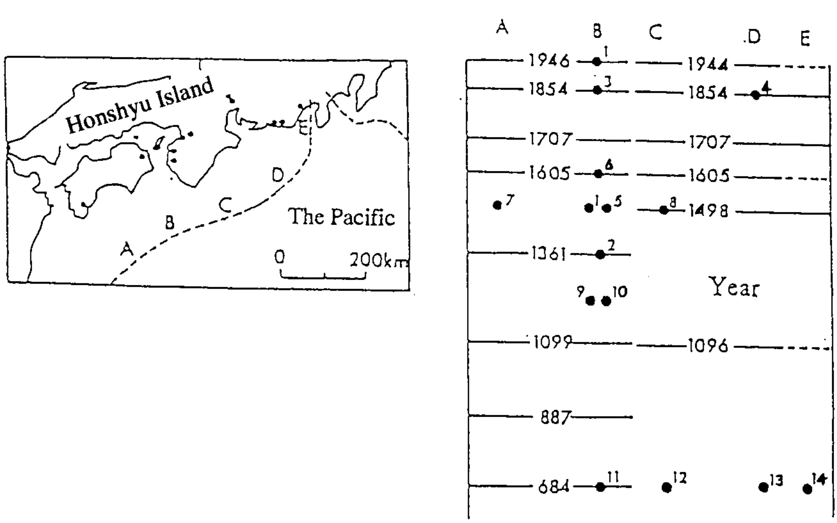
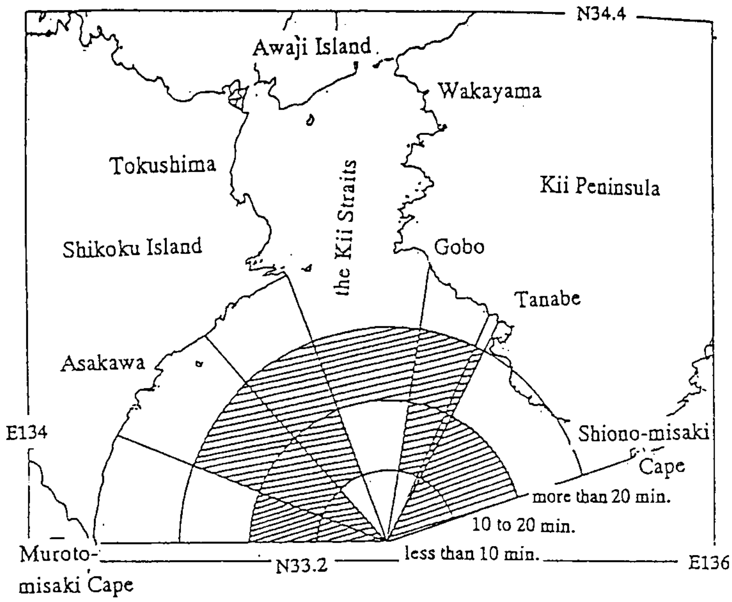


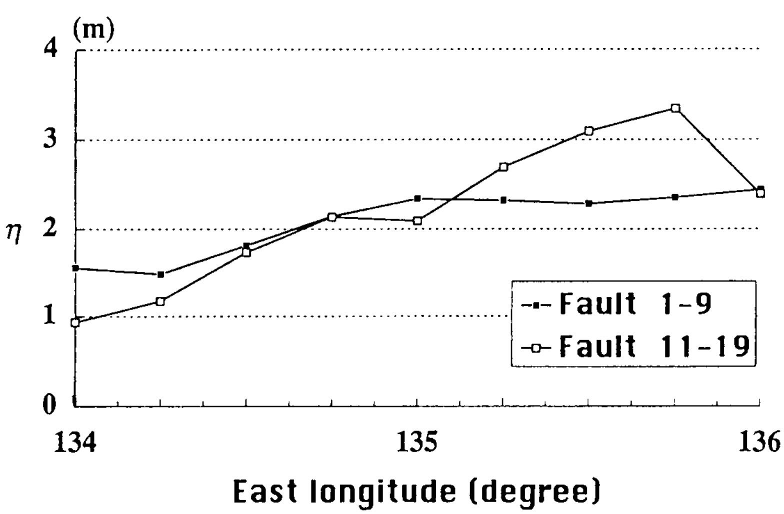
Fig. 3. Supposed epicenters of huge earthquakes along the Nankai trough

Table 1. Historical occurrence of huge earthquakes (M≧8) along the Nankai trough (black points with number shows the location of ruins with liquefaction excavated by Sangawa)





(a) Minimum arrival time of peak of first tsunami



(b) Changes of the highest water level at Tanabe, Wakayama Prefecture

Fig. 4. Characteristics of tsunamis in accompany with earthquakes at Nankai trough

the changes of epicenter. Therefore, the location of epicenter is very important factor to decide the tsunami height. Residents can be given these variables through numerical simulation. Tsunami potential can be estimated with combination of occurrence probability of earthquake and tsunami characteristics. In the case of Tanabe city, risk to life of tsunami disaster at present is 0.0032. This is about half of death rate of diseases(0.0076). Residents can be easily understood about the dangers of tsunami disaster in comparison with other dangers due to diseases and traffic accidents.

HUMANWARE MANAGEMENT AND DISASTER LESSON

Humanware Management

We traditionally divide disaster management into two categories such as hardware management and software one. The former is hard-countermeasures such as construction of tsunami breakwaters and building code of anti-seismic function. The latter is soft-countermeasures (information) such as disaster culture and disaster lesson. Software covers many items in relation to disaster information, knowledge and wisdom. In a moment of disaster occurrence, search and rescue are the most important response of our action. If the number of the dead and the injured are small, software management can include all events related human behavior. In the case of catastrophe, however, it is better to divide software into two parts such as "non-human information" software and humanware. As shown in Table 2, humanware includes so many as follows:

1)In the category of risk management(before disaster), "preparedness" includes establishment of search and rescue, registration and training of volunteers and exercise of psychological counselors.

2)In the category of crisis management(after disaster), "response" includes softening of PTSD, change of hospitals for the badly injured, arrange of funeral and introduction of missing person.

The disaster prevention systems are composed with combination of hardware, software and humanware. They have to be well controlled with disaster management. Urban disasters have many characteristics so that many damage-enlargement scenarios have to be found out because large and dense populated urban area has many aspects of human activities changed with time. Especially, in mega cities so many people are working, shopping, moving and gathering. In urban disasters, the disaster weak are not always old persons and children but also people who stay in a moment of disaster. Therefore, in urban disasters the disaster weak are defined as persons who can not survive by themselves at the disaster. Consequently, humanware management is very important to reduce the number of the victims at the disaster.

Disaster Lesson

We had a first experience of Hanshin-Awaji earthquake disaster as urban disasters on 17 January 1995. The human and property damages were quite huge. Table 3 shows the list of the damages. Due to fortune three decades, we all forget the existence of catastrophic disasters in Kansai metropolitan area whose population is about ten million including three big cities such as Osaka, Kobe and Kyoto. In the area, we have

Table 2. Disaster management and humanware

	emergency management	
	risk management	crisis management
hardware	**reduction or mitigation** return period, encounter probability, fail-safe, redundancy	**recovery** reinforced lifeline, temporal housing
software	**abundance** disaster information, training, planning, education, evacuation manual	**communication** recovery information of lifeline, provision of emergency necessities
humanware	**preparedness** search & rescue, volunteer, psychological counsellor	**response** care for PTSD, emergency medical care
commandware	**tactics** headquater of disaster measure, command system, management system	**strategy** logistics, long-term reconstruction

Table 3. Human and property damage in the Great Hanshin-Awaji Earthquake Disaster

Human damage		Facility and construction damage	
dead	5,502	education institution	766
missing	2	road	9,403
severely wounded	1,819	bridge	321
slightly wounded	25,029	river levee	427
under checking	14,679	landslide	367
House damage		block−made wall	1,385
collapse	100,282	Lifeline damage	(housefold)
half ruined	108,402	water supply	1,277,851
minor damage	185,756	city gas	856,835
Building damage		electric power supply	1,111,640
Public use	549	telephone	286,231
Others	3,126	Fire	294

had some typhoon disasters and heavy rainfall ones but not earthquake disasters since 1868 (Meiji era started as modern ages). Every cities and towns has disaster planning but it is an armchair theory because no one can consider existence of earthquake whose epicenter is close to the metropolitan area. It is, of course, hard to get disaster lessons from the disaster in the midst of study but we try to show some of them as follows:

1)Safety: More than 70% of the victims was generated due to collapse of old wooden made Japanese style houses. Many old concrete made buildings were also destroyed. We have to reinforce old houses, buildings and social structures such as highway and bridges urgently. Our building code was revised in 1978 due to Miyagiken-Oki earthquake disaster(M=7.8). In the 1995 Hanshin-Awaji earthquake disaster, the collapsed ratio of buildings which were construct under the new code is more than 40% smaller than that of old buildings under or out of the old code.

2)Land: In our country, land is very expensive. The "land" standard instead of gold one has supported our rapidly economical development. Our capitalists had borrowed much money from banks on the security of land. This has been guaranteed by the increase of land price. However, this situation was broken a few years ago and now our economy is at the stage of recession. For public works, it was very difficult to rearrange city function under residents' agreement. We have severe inner city problems such as dense populated and poor housing areas with narrow roads and many smaller enterprises in the areas. Therefore, in any urban area such as in Tokyo and in Osaka, quite different vulnerability (it is good in very modern business district and poor in inner city) against disasters coexists in urban area.

3)Network: In every disaster planning in prefectures(states), cities and towns, catastrophic and so wide-area affected urban disasters are not considered. Only fire fighting and the police have agreed cooperative support at prefecture level. Quick response of neighboring local governments is very important on urgent demands for every task force. Information network is also necessary to have quick response on disasters. The system has to be equipped as fail-safe function and applied to small scale disasters as training for operators.

4)Redundancy: The reliability of lifeline systems depends on redundancy as well as good code for anti-seismic function. In our country, financial departments have much power to decide the budget for redundancy of infrastructures. They believe that redundancy is an additional safety factor of only 10 to 20 percent. Moreover, our structural engineers tend to make over accurate manuals for construction. In order to improve the manuals, it is necessary to have experiences of disasters but unfortunately most of them are free from disasters. They have only visited foreign country, especially United States and "watched" the disasters such as the 1989 Loma Prieta and the 1994 Northridge earthquake disasters. After the disasters they say that this is unbelievable phenomena which has never occurred before in Japan. In Hanshin-Awaji earthquake disaster they said that the Sanyo Shinkansen and new designed highway were partly broken due to "unbelievable" vertical acceleration (at Kobe University, vertical acceleration was 446.5Gal and horizontal ones were 269.8Gal in the north-south direction and 306.3Gal in the east-west direction respectively). Even if it was true, the broken pattern of structures such as piers and beams showed no existence of redundancy or persistency.

5)Vulnerability: The death tall at Kobe (six wards), Ashiya and Nishinomiya cities was about 4,200. The number of population in this area is 1,580,000 and density of population 7,000/km^2. Therefore, risk to lives is 0.0027. From our simulation, the estimated maximum death toll is 8,450. This means that we have other hidden scenarios which may lead to larger human damages. For example, passengers will be killed due to turned over trains, business persons due collapsed buildings, shoppers and passersby due to collapsed department stores and shopping streets. On 17 and 18 January, fortunately we had no wind and the area of the burnt district remained 100ha. If strong monsoon did come, many residents under the broken houses could not survive.

CONCLUSIONS

As our society has become well, every resident has to be equipped with adequate knowledge about natural disasters. In the tsunami-prone areas, the highest height and the shortest arrival time of tsunamis are very useful knowledge to take a quick refuge just after earthquake. The evacuation manuals are prepared with these practical information. This process is public informed consent. Human ware as defined here is also very important to manage catastrophic disaster and to reduce human damages and get quick rehabilitation. The lesson from the 1995 Hanshin-Awaji earthquake disaster are summarized as safety, land, network, redundancy, vulnerability. They are not enough because the disaster does not come to an end. Additional lessons may be proposed at the process of rehabilitation.

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SIGNS OF DESTRUCTION OF A GREAT EARTHQUAKE AND HUMAN SENSITIVITY TO AN EARTHQUAKE: ON INITIAL DISPLACEMENT OF A TECTONIC EARTHQUAKE

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ABSTRACT

When an earthquake takes place, signs of its force and human sensitivity beforehand as well as actual tectonic destruction can be analyzed. From this, a theory of initial displacement is deduced and offered for discussion.

KEYWORDS

Tectonic earthquake; destruction; initial displacement; earth crust; deformation; response spectrum.

INTRODUCTION

While the Tangshan earthquake took place (28 July 1976) many Chinese seismographers and others personally viewed the earthquake spot. They saw the occurrence of the catastrophe and process of its development (Wang Jingming, 1993) in such a way that the main properties of seismic destruction were observed (see Fig. 1).

There was shear destruction of the east and west plates of the fault. The first elastic rebound in the north-south direction occurred and in the twinkling of an eye disappeared with surprising range.

The second back motion of the counter rebound in the north-south direction was a little weak. The third rebound occurred in the north-south direction slowly fluttered and shook along the east-west direction.

The direction in which buildings collapse and people and objects fall has a definite rule. It conforms to the opposite direction of the first elastic rebound of the east and west plates (in most cases).

In line with analysis of earthquake damage and human sensitivity, the first elastic rebound was found out to be initial displacement. This causes great damage to engineering construction and cannot be ignored. Initial displacement poses great significance if combined with analysis of force received by the earth's crust.

EARTHQUAKE PROCESS

Fluid substance in the earth's interior is called the mantle (while it may be a fact, it has no influence on the analysis of earthquake destruction). Movement of the earth includes both revolution and rotation. Centrifugal force of the earth's rotation is 365 times that of its revolution. (The former takes a year and the latter takes a day.) Tides of the ocean reflect this. The earth's outer crust and fluid substances are at an M line that is not a circular sphere (analysis omitted). This sphere presses against the mantle (thickness of the earth's crust differs everywhere due to this spherical body). Also the earth's crust has various levels, vertical and slanting from which displacement and deformation of the east and west plates originate. As shown in Fig. 1, prior to an earthquake the east and west plates are respectively deformed along the opposite direction. This is due to resisting shear body preventing displacement and deformation in the fault between the east and west plates at the centre of an earthquake. So the earthquake does not occur and deformation is slow. In light of static analysis using the finite element method and continuous bodies defecated and substituted by rods (analysis omitted), we find the distribution of forces for the east and west plates on both sides of the fault.

When the force of resisting shear is not enough at point 0, (see Fig. 1) destruction occurs. In accordance with dynamic analysis (analysis omitted), we know that the energy of elastic rebound is strong along direction 1 of the east and west plates on both sides of the fault in the earth's crust. Energy of elastic rebound along direction 2 is less strong (analysis omitted), and along direction 3 it becomes more weak. As known, when the earth's crust moves along directions 1,2,3, the east and west plates are deformed and displaced. The strain of pulling, pressing and shearing causes displacement along the vertical direction (including toward the centre of the earth--plumb line). This is called the vertical direction of the main direction of elastic rebound or interlocked direction, the transfer of which is slow to the outside (analysis omitted). From the above, it can be seen that the theoretical analysis is consistent with actual damage from the earthquake.

This is taken as resources of turbulent motion. The reason is the initial displacement is an elastic rebound and the source of turbulent motion of the outward transfer of the seismic wave. Using inference of elastic dynamics (omitted), the energy from the source of turbulent motion is strong, as is its response. Actual damage of the earthquake coincided with the direction (as a result of inertial force in the opposite direction) and then the energy slowly weakened.

As known, the intrinsic swinging motion causes the seismograph to deform at the first and second inertial vibration, and the high frequency of the seismographic record is distorted. So it is difficult to catch inertial displacement. While our inference and analysis are simple, the train of thought is true. Initial displacement should not be ignored in structural design for engineering construction.

EARTHQUAKE DAMAGE

As documented, the earthquake occurred along the direction of north and south by east, shaking first to the south and then to the north. During the third round of shaking, buildings collapsed. Is it resonant destruction or not? It is known that resonances should coincide, and the external force to the system should do positive work. If changed cycles are not in harmony, there is no resonance. If there is only a direction with a vibration, there is no harmony at the second vibration nor is there resonance. If there is harmony, the first and second vibration should have the same cycles. So the structure should deform under conditions of harmony and ultimately be destroyed. If deformation is intensified, the range of vibration should be lengthened as the cycles are the same. Shock distance is also lengthened, and acceleration quickened. As a matter of fact, in line with actual earthquake damage and theoretical analysis (see Fig. 1), the energy along direction 1 is strongest the first time and then becomes weaker. (Farther from the centre of earthquake, there is no such condition, as the seismic waves transferred from point 0 run a course from that point increasing and then decreasing. This weakening results from the strain of pulling, pressing and shearing which occurs along the direction, and the sphere of vibration is small. [omitted]). Therefore cycles are not the same, not intensified deformation, longer duration or quicker acceleration. Energy intensifies if acceleration is quicker. Where the energy comes from, with no resonance at the centre of an earthquake remains a question.

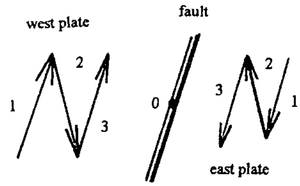

Fig. 1 Model of Elastic Rebound in Tangshan Earthquake

Despite location, the top and bottom of columns of one-story factories and frameworks as well as columns of coal bins were greatly deformed after the earthquake. Steel and concrete were crushed and broken. But there are no crevices from the bottom of the column up to the top, or from the top down to the bottom. We understand it as a continuous function. When the foundation deforms greatly by displacement, the structure has no time to follow up motion (inertial force) so deformation cannot be transferred soon enough. Thus the top and bottom of the column were crushed under the function of inertial support. If there is resonance, crevices will occur at intervals between the top and bottom of the column. (Through a test of dynamic destruction, we found that the column had a horizontal crevice with 2-4 mm from the base upward). Some examples of earthquake damage are given in the next section. From these, we can understand that it is unreasonable to have resonance if crevices appeared and 60% of the cycle is increased (see Fig. 2).

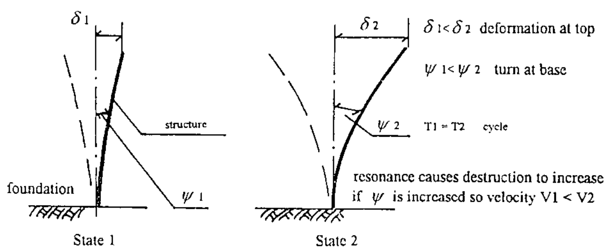

Fig. 2 Test of Dynamic Destruction

ANALYSIS OF EARTHQUAKE DAMAGE

As stated above, initial displacement has strong energy and, along with it, strong acceleration. So it may be on a par with the development of resonance. Without denying the significance of resonance in an earthquake, some damage can also be explained by the theory of initial displacement.

Five aspects of this phenomenon are discussed below.

Results of Calculation for a Chimney with a Height of 180m

Using the theory of initial displacement showed that actual damage to a chimney stalk with a height of 130.5m coincides with those results. It is assumed that the chimney is parallel with X axis and the foundation on the ground is X=0. Upwards is positive. When initial displacement is transferred to X of the chimney, an equation can be used. It is based on the bending moment of the fixed beams at both ends by X span X, axis is upwards, H-X segment of the total height of the chimney stalk is called inertial support. From the bending moment a number is derived. This number is assigned 0; the value of X is solved. This value is the maximum bending moment and is found to coincide with actual earthquake damage. At the same time it is unreasonable to use the theory of resonance for destruction of a chimney at Douhe electric power plant during two phases of the Tangshan earthquake. An earthquake occurring at 3:40 a.m. caused crevices in the chimney at a height of 130.5m and made it bend several centimeters. As a matter of fact, the crevices appeared in the chimney and the cycle increased 60%.

Another earthquake occurred that afternoon and also caused damage at the height of 130.5m. At that time, the upper part of the chimney fell. This can be explained by the theory of initial displacement and has nothing to do with the cycle. So it is reasonable for damage to occur a height of 130.5m on the chimney during two phases of the earthquake.

The Earthquake has a Directional Property

It was reported that the direction of the collapse of a building is not random fluctuation (as previously thought) but has definite laws. For example, at first, the east plate moved to the south but people and vertical objects tumbled to the north. The west plate turned toward the north but people tumbled toward the south. It was also reported that just before Tonghai earthquake, someone fell down as though suddenly pushed from behind. That resulted from initial force and strong acceleration. It was purely the result of initial displacement; resonance could not have done it.

Destruction of an Ancient Octagonal Pavilion

The ancient octagonal pavilion on the top of Fenghuang Mountain is an one-story steel and concrete structure. During Tangshan earthquake, most of the fixed bending moment at both ends of the columns was crushed. The top of the pavilion has inertial support. When initial displacement and its response occurred, both ends of the fixed column structure were bent. It was not possible for the whole structure to turn, because mass centre and rigid centre are located at same position. Every column receives bending force if the whole structure turns. There is no indication that any of the columns turned (the ground was not displaced). Even if the destruction was caused by turning, both ends of the column were completely crushed. Values of bending moments at the centre and at both ends of the columns were the same. At least the centre of column should have been broken. As a matter of fact, there was no damage to it. We understand it as both ends fixed. This is a typical sign of damage from initial displacement. As known, except around the centre of an earthquake, the surface of the earth turns very little. Out-of-step transitional motion also produces a 180° turn. For example, when a scaffold breaks and one end starts falling, it is vertical at first and then, depending upon kinetic energy from gravity with distance from the support point, it turns 180° as it falls to the ground. This has happened at construction sites. From this, it may be seen that the turning of the chimney and other objects during the earthquake resulted from parallel motion. Destruction of the octagonal pavilion is not convincingly explained by high resonance; the pavilion is a low building of one story. It is an example of nonresonant destruction.

Deformation of underground Column Well

A round column well became an ellipse, which indicates there was not permanent elastic deformation near the source of the earthquake; the dominant direction of the earthquake force was recorded. Someone saw the roof of a building thrown into a courtyard (Tonghai earthquake) and people on the second floor of the building thrown onto the roof.

These facts indicate that the destruction was caused by strong inertial force as held by the theory of initial displacement. So it is illogical here to use the theory of resonance as an explanation.

X-Type Serious Crevices appeared on a Gable Wall

They arose from initial displacement and its response which includes first displacement as well as second and third displacement as well as their responses. Appearance of X crevices on the gable wall should be noted. A direction can be determined. When displacement moves toward the east, only the first crevice A appeared. Another crevice B of X crevices remains unformed. Under this condition, crevice B cannot be produced; it occurs only when the displacement reverses toward the west. After crevices appeared in the structure, the length of the cycle increased around 60% that of original structure. The difference between the cycles is about 60% of that between crevice A and crevice B. As the two cycles do not coincide, it cannot be said that resonance exists. As known, the direction of the external force (displacement) and the direction of motion in the whole cycle are consistent. External force (displacement) plays a positive role. The energy provided from outside at this time is strongest,the phenomenon called resonance.

Resonance should be consistent or occur in steps. The crevices which X formed were caused by two different cycles, as they show different frequencies. So it is appropriate to explain the phenomenon by using initial displacement (omitted).

THEORY OF INITIAL DISPLACEMENT

Seismic theory has a history of decades. Moreover, seismic load involves complicated kinetics. When a quake occurs, the surface motion of earth is also complicated. Thus a theory takes time to develop. At present, we offer tentative ideas for discussion. Our point of view is as follows:

1) Initial displacement and its response should be obtained from current seismographic records.
2) Response spectrum of initial displacement should be added to anti-seismic design considerations.
3) A concept of inertial support should be introduced.
4) Time curve of earthquake acceleration should be used for design in accord with comprehensive seismographic record.
5) Time curve for design should be determined on the basis of item 4.

CONCLUSIONS

Research discussed here still has far to go. The earth changes its acceleration every day by one cycle. Given its annual cycle, the value of rotation of the earth increases 365 times more than that of its revolution which can reach 0.2g. This is only one factor in future work.

At present, we have meager knowledge and the level is limited. So we hope this article can lead to something of greater value.

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KEYWORD INDEX

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