NSF/CEE-82096

PB83-186817

中　美
通过建筑、城市规划和工程减轻地震灾害讨论会会议录

PROCEEDINGS OF THE P. R. C.—U. S. A.

JOINT WORKSHOP ON EARTHQUAKE DISASTER MITIGATION THROUGH ARCHITECTURE, URBAN PLANNING AND ENGINEERING

中国　北京　　Beijing　China

50272-101

REPORT DOCUMENTATION PAGE	1. REPORT NO. NSF/CEE-82096	2.	3. Recipient's Accession No. PB83 186817

4. Title and Subtitle Earthquake Disaster Mitigation Through Architecture, Urban Planning and Engineering, Final Proceedings PRC-USA Joint Workshop, Beijing, China, November 2-6, 1981	5. Report Date 1982 6.
7. Author(s) None listed	8. Performing Organization Rept. No.
9. Performing Organization Name and Address University of California at Berkeley Center for Planning and Development Research Berkeley, CA 94720	10. Project/Task/Work Unit No. 11. Contract(C) or Grant(G) No. (C) CEE8105229 (G)
12. Sponsoring Organization Name and Address Directorate for Engineering (ENG) National Science Foundation 1800 G Street, N.W. Washington, DC 20550	13. Type of Report & Period Covered Proceedings 14.

15. Supplementary Notes

Submitted by: Communications Program (OPRM)
National Science Foundation
Washington, DC 20550

16. Abstract (Limit: 200 words)

Twenty-five papers which were presented at a workshop on earthquake hazard mitigation are provided. The papers are grouped under three subtopics: (1) architectural and non-structural elements; (2) urban planning and land use; and (3) evalaution, strengthening and repairing of existing masonry structures. The relationship between building configurations and seismic performance for typical Western building types is examined and the earthquake performance of emergency service facilities is reviewed. Disaster relief works following the earthquake in Tangshan, China, are outlined. Group discussions are summarized and recommendations made during the discussions are listed. Appendices contain the workshop program, names of participants, and the itinerary of the U.S. delegation.

17. Document Analysis a. Descriptors

Earthquakes	Construction
Hazards	Dynamic structural analysis
Damage	Meetings
Buildings	

b. Identifiers/Open-Ended Terms

Ground motion	H. Lagorio, /PI

c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
NTIS	20. Security Class (This Page)	22. Price

(See ANSI-Z39.18) See Instructions on Reverse OPTIONAL FORM 272 (4-77) (Formerly NTIS-35) Department of Commerce

NOTICE

1-a

中、美

通过建筑、城市规划和工程减轻地震灾害讨论会会议录

FINAL PROCEEDINGS

P.R.C.-U.S. JOINT WORKSHOP ON EARTHQUAKE DISASTER MITIGATION THROUGH ARCHITECTURE, URBAN PLANNING AND ENGINEERING

主 办 单 位

中国国家基本建设委员会抗震办公室　　美国伯克利加州大学规划和发展研究中心

美国国家科学基金会

Workshop Sponsored by

Office of Earthquake Resistance, State Capital Construction Commission, P.R.C.
Beijing

Center for Planning and Development Research, University of California, Berkeley, California
U.S.A.

National Science Foundation
Washington, D.C.
U.S.A.

中国　北京
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中、美
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Workshop Organized by

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中国国家基本建设委员会抗震办公室筹备

中国 北京
1981. 11. 2-6

Beijing, China
November 2-6, 1981

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PREFACE

As Director of the Office of Earthquake Resistance, I am very pleased to have provide sponsorship of the workshop on Earthquake Disaster Mitigation Through Architecture, Urban Planning and Engineering.

This workshop represents a very important development in scientific exchange of earthquake disater prevention and mitigation information. This workshop has produced many beni-fits to People's Repblic of China and the United States.

Mr. Cheng Zhenwen
程震文
Director
Office of Earthquake Resistance
State Capital Construction
  Commission, P.R.C.
Beijing

INTRODUCTION

In 1979, the Chinese Earthquake Engineering Delegation headed by Vice-Minister Li Jingzhou visited the United States and attended the Second U.S. National Conference on Earthquake Engineering organized by the Earthquake Engineering Research Institute at Stanford University. During this visit, discussions were held with representatives from the National Science Foundation. Mutual interests were indicated including the topic of urban earthquake disaster mitigation.

Based on these discussions the topic of urban earthquake disaster mitigation was included in an annex to the Protocol Agreement between the National Science Foundation and the U.S. Geological Survey and the State Seismological Bureau of the People's Republic of China.

The P.R.C.-U.S. Agreement on Earthquake Studies was signed in Beijing on January 24, 1980, as an official agreement to conduct scientific exchange and develop research activities of joint concern in areas identified with earthquake hazard mitigation. The cooperative studies identified under the protocol include:

1. Exchange of scientists and information;
2. Cooperative research;
3. Covening of scientific conferences, symposia and lectures.

This workshop on earthquake disaster mitigation through architecture, urban planning and urban engineering is being developed under cooperative studies authorized by the first and third items mentioned above.

In the summer of 1980, Professor Henry Lagorio, Associate Dean for Research, College of Environmental Design, University of California at Berkeley, visited China as a member of the Earthquake Engineering Research Institute delegation headed by Dr. John Blume. At that time, he initiated detail discussion with Mr. Ye, Yaoxian, Vice Director, the Office of Earthquake Resistance, State Capital Construction Commission, for the development of a workshop on topics of Annex III, Section 6 "Urban Technology for Hazard Mitigation". The workshop proposal resulting from this discussion was received and approved by both governments.

The principal objectives of this Workshop are to:

1. Provide a forum for architects, planners, engineers and other concerned professionals to exchange experiences and scientific information on the technology of urban earthquake disaster mitigation;
2. Identify the research needs in earthquake disaster mitigation through architecture, urban planning and urban engineering of concerns to both sides;
3. Explore potential topics for cooperative research.

The Workshop was organized in three parts:

1. Architectural and Non-structural elements;
2. Urban Planning and Land Uses;
3. Evaluation, Strengthening and Repairing of Existing Masonry Structures.

Twenty five invited papers were received. Ten papers concerned the first part, eight papers related to the second part, and seven papers deal with the third part.

The Workshop was attended by thirty participants: eleven from the United States, including the representative from the National Science Foundation, and nineteen from China including representatives from the State Scientific and Technological Commission and the State Seismological Bureau.

The study of earthquake disaster mitigation through architecture and urban planning constitutes a new research activity in both countries. We believe this Workshop will significantly contribute to the advancement of research in this area in the United States and China.

It is said in China, "For all things, difficulties always occur in the beginning." In English, there is a similar saying: "A job begun is a job half done." Now, we have begun. It is hoped this modest beginning will lead to substantial growth in exchange of understanding, knowledge and experience between our two peoples.

Ye, Yaoxian
叶耀先
Vice Director
Office of Earthquake Resistance
State Capital Construction Commission
Beijing, P.R.C.

Henry Lagorio
Henry Lagorio
Associate Dean for Research
College of Environmental Design
University of California
Berkeley, California
U.S.A.

OPENING ADDRESS BY VICE MINISTER PENG MIN, SCCC

(Nov. 2, 1981)

All participants, friends,

PRC/US Joint workshop on Earthquake Disaster Mitigation Through Architecture, Urban Planning and Engineering sponsored by Earthquake Resistance Office, State Capital Construction Commission of the People's Republic of China, Center for Planning and Development Research, University of California, Berkeley, California, the United States and the National Science Foundation of the United States of America is open now based on the Annex III of the Protocol between the State Seismological Bureau of the People's Republic of China and the National Science Foundation of the United States of America and the Geological Survey of the Department of Interior of the United States of America for Scientific and Technical Cooperation in Earthquake Studies. This is a significant event. On behalf of the State Capital Construction Commission I would like to express my sincere congratulation to the opening of this joint workshop and warm welcome to all participants.

Earthquakes have brought tremendous disasters to human beings. More than 30 countries in the world are under the threat of earthquake. In future earthquakes inevitably will still cause harm to the human life. Therefore, earthquake disaster mitigation has become the subject attracting wide attention and concern in the present world. The earthquake engineering research has been developed gradually. More and more specialists from all professions and government officials have been involved in the research. This will surely bring benefit to the whole mankind.

China is one of the countries which are suffered most severely from earthquake disasters. According to the records in history, more than 830,000 people died in the 1556 earthquake of Hua county, Shaanxi Province, 200,000 people lost their lives in the 1920 earthquake of Hai Yuan County, Ning Xia, 240,000 people were killed in the 1976 Tangshan earthquake. Since the founding of the People's Republic of China, the government and the people have made great efforts in the mitigation of earthquake disasters. In the aspect of earthquake prediction we have carried on massive studies and practices. Regarding the earthquake engineering we have compiled seismic design codes and standards for ensuring the safety of structures to be constructed in seismic areas and aseismic criteria and strengthening methods for evaluation and strengthening of different structures and equipments to improve the earthquake performance of existing structures. We have organized the concerned institutions to conduct the extensive and deep-going reaearch in various fields of earthquake engineering. Moreover we have made more efforts in earthquake disaster prevention in cities and regions and in popularizing the basic knowledge of earthquake and earthquake resistance. However the mitigation of earthquake disaster through architecture, urban planning and engineering is still a new topic in this country. This workshop provides an opportunity for the scientists from both countries to exchange experiences and explore the possible projects for future cooperation. I believe we will make active and beneficial contributions in mitigation of earthquake disaster through architecture, urban planning and engineering.

Finally, I wish the workshop a great success and wish you all a happy stay in China.

POTOGRAPH OF THE PARTICIPANTS WITH MR. PENG

Front row, left to right: B. Jones, L. Selkregg, F. Krimgold, M. Wang, E. Schwartz, Ye Yaoxian, H. J. Lagorio, Peng Min, K. Steinbrugge, Cheng Zhenwen, A. Jacobs, C. Arnold, N. Hawkins, and G. McCue

Middle row, left to right: Niu Zezhen, Zou Qijia, He Guanglin, Jiang Menghou, Gao Lutai, Yu Qingkang, Zhang Shuquan, Xu Xunchu, Song Peikang, Wang Zuyi, Hu Lingyu, Shi Yunlin

Back row, left to right: Hou Minzhong, Wang Zongrong, Chen Mouxin, Yang Yucheng, Yang Wenzhong, Cheng Zuwu, Cheng Minda, Gong Yongsong, Liu Zhaofeng, Jiang Minsheng, and Ning He

ARCHITECTURAL AND
NON-STRUCTURAL
CONSIDERATIONS

ARCHITECTURAL DESIGN AND URBAN PLANNING FOR SEISMIC REGION

Ye, Yaoxian[I]

ABSTRACT

The architectural damages and urban disasters observed in recent Chinese earthquakes as well as the lessons learned from them are described in this paper.

The paper is divided into four sections. In the first section, the building systems are presented, including the earthquake performance of prevailling building systems in China, vulnerable building systems and requirements to building systems. The second section describes influence of building configuration on earthquake performance, the plan and elevation shape included. In the third section, interaction between the structural and non-structural elements, damages to non-structural elements, and the aseismic measures are specified. Finally, the importance of mitigating urban earthquake disasters, lessons learned from 1976 Tangshan Earthquake and requirements to urban planning for earthquake-prone region are described in the fourth section.

INTRODUCTION

The last few decades have witnessed that the loss of life and property mainly was caused by collapse of buildings and unrational urban planning. However, up to date, architects and urban planners mostly ignore seismic activity, and design requirements essentially deal with the structural aseismic measures of a building as it affects life safety. Less attention is given to the performance of architectural components and of the whole city during an earthquake. In fact, particularly in the architectural design and the planning stages, the decisions on earthquake disaster protection made by designers, especially by architects and planners, have critical implications for life safety and disaster mitigation.

In accordance with the lessons learned from recent Chinese earthquakes, the paper describes selection of building systems, the influence of building configuration on earthquake performance, the interaction between architectural components, and urban disasters and requirements to city planning for seismic region. The suggestions for aseismic architectural design and urban planning for earthquake-prone area also proposed in this paper.

BUILDING SYSTEMS

The safety of building during an earthquake, economic effectiveness and the function of the building etc. are all closely related to building system. Therefore, selection of the building system (or structural system) is one of the critical step in earthquake resistant design. In this aspect, it is necessary to have architect and structural engineer cooperate with each other.

Earthquake Resistant Performance

Figures 1 and 2 show the comparision of earthquake performance among

I Engineer and Vice Director, Office of Earthquake Resistance, State Capital Construction Commission, Beijing, China.

current brick structures and among reinforced concrete structures based on the experience from 1976 Tangshan Earthquake and several other earthquakes occurred in China respectively. In the Figures the earthquake resistant performance improved gradually from 1 to 8.

For the brick structures, as shown in Figure 1, the brick chimney is the weakest structure and the cracks occurred where the intensity is VI or even V. While the low silo built with brick masonry seldom collapsed even if it is in the area with intensity of X. Multi-story brick buildings with reinforced concrete constructive columns have very good performance. They may not collapse in the area with intensity of X. While most of the single-story buildings with brick columns were severely damaged or collapsed in the region with intensity of VIII.

For the reinforced concrete structures, as shown in Fig. 2, prefabricated single-story industrial building could be collapsed in the area with intensity of VIII or IX. However, the reinforced concrete frame building with infill brick walls could be intact even in the area with intensity of X.

It may be seen that different building systems make a great difference in earthquake resistant performance. Even if the same materials are used, with different building system, the earthquake performance can be different. Therefore, it is very important to select optimum structural solution in earthquake resistant design.

Vulnerable Building Systems

Some of the building systems which are optimized in non-seismic region can be vulnerable in earthquake-prone region. Hence, we should not blindly adopt the building system accepted in non-seismic area.

We should pay more attention to the following vulnerable building systems in seismic design according to the lessons learned from the recent strong earthquakes occurred in China.

Longitudinal load-bearing-wall system for multistory brick buildings
There are many advantages for longitudinal load-bearing-wall system, such as convenient operation in construction, better economic result, flexible arrangement in space, and satisfying multiple function etc. It is hardly too much to say that this system is one of the optimized building systems in non-seismic region. However, such buildings have suffered repeatedly heavy damages in the past earthquakes due to insufficient transversal load-bearing-wall. During the 1975 Haicheng Earthquake, about 40% of this type of buildings were collapsed or severely damaged in the area with intensity of VIII, and 70% with intensity of IX. For example, the longitudinal load-bearing-wall system was adopted for the both wings of the Haicheng County Guest House. There was no transversal load-bearing-wall within the length of 25.4m for this building and it was levelled to the ground with the death of some dozens of people by the earthquake. The plan of a wing of the building is shown in Figure 3.

A striking contrast with the longitudinal load—bearing—wall system is the longitudinal and transversal load-bearing-wall system. During 1975 Haieng Earthquake, the building with transversal load-bearing-wall for each

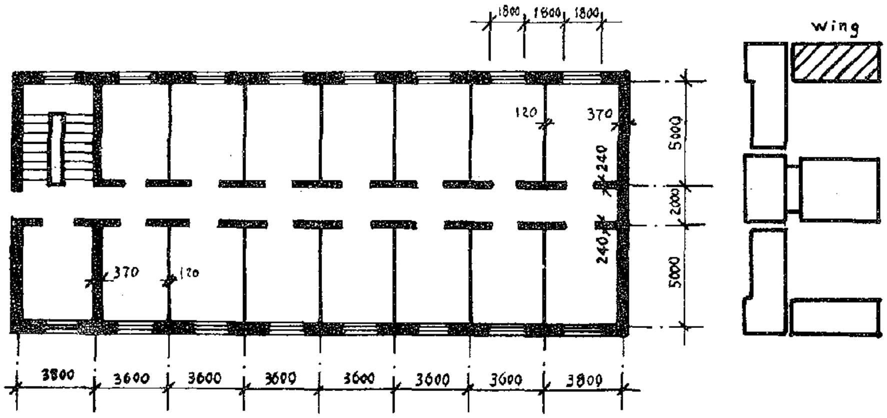


Fig. 3 Plan of a wing of the Haicheng County Guest House

one to three bays were not collapsed in the area with intensity of VIII or IX, only 35% were severely damaged. Transversal load-bearing-wall system has adequate vertical bracing elements for each bay, so that only a little damage occurred in the area with intensity of IX. The ductility and lateral load bearing capacity of the transversal load-bearing-wall system with reinforced concrete constructive columns at the intersections of the walls and with tied beams in the floor levels are greatly increased by reason of confinement of the brick walls and floor slabs. This type of buildings were not collapsed in the area with intensity of X during 1976 Tangshan Earthquake.

It proved that the transversal load-bearing-wall system should be adopted as much as possible for the multistory brick buildings in seismic region, and reinforced concrete constructive columns should be used for upgrading when necessary.

Brick column load bearing system Brick column load bearing system is a good solution for buildings with medium and small span in non-seismic areas since it is economic and convenient for construction. However, most of such type of buildings were damaged or collapsed in the area with intensity of VIII and above in the past earthquakes. Hence, it is specified in the Chinese Aseismic Design Code that the vertical reinforcements should be placed in brick columns in accordance with calculation when design intensity is VIII or IX, but not less than 4 Ø 10 and 4 Ø 12 respectively.

Flat slab system If we design for no earthquakes or mild earthquakes, flat slab system may be a good system for the buildings which need large space and with large floor load. Without girders, it's convenient in construction and the cost can be reduced by decreasing the height of the floor. However, some buildings using such kind of system were collapsed under strong ground motion by reason of the damages to columns. The collapse of the cool storage plant of Tangshan city in 1976 Tangshan Earthquake is one of the examples. The collapse of the computer centre, a four storey flat plate building in Bucharest in 1977 Romania Vranca Earthquake is another example. Although there was misarrangement of reinforcements, the collapse of the

1 chimney

2 single story building with brick columns

3 longitudinal load-bearing-wall building

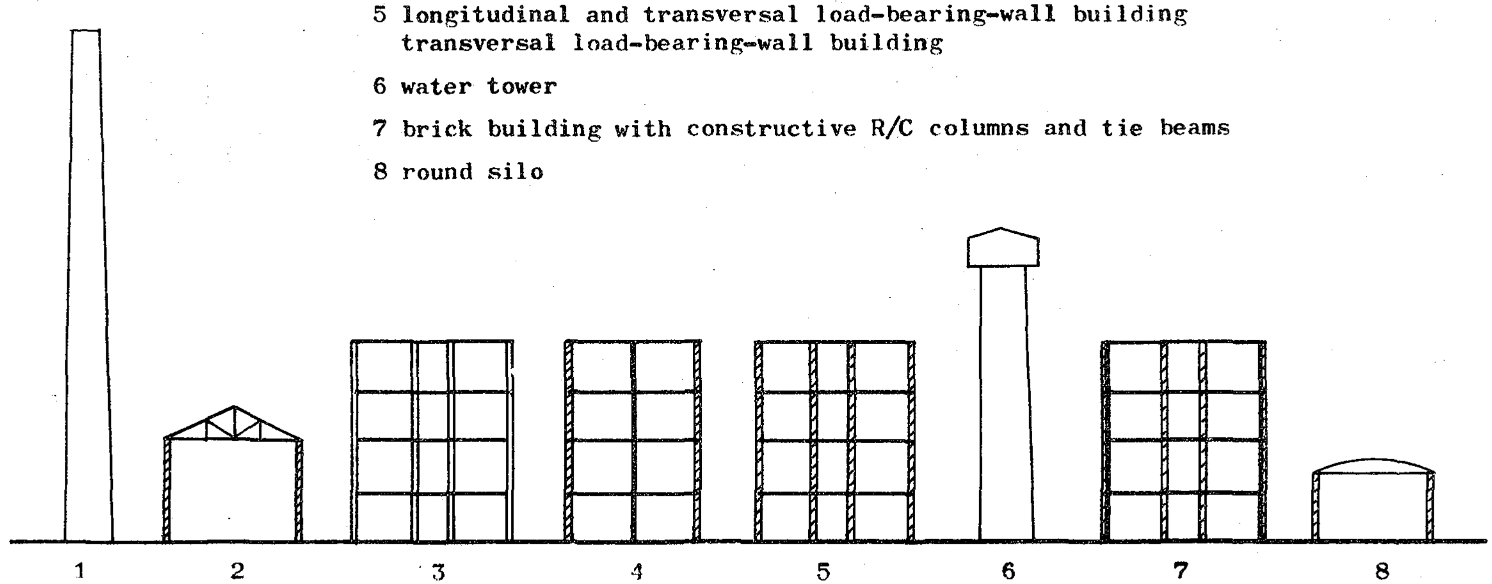
4 inner frame building

5 longitudinal and transversal load-bearing-wall building
transversal load-bearing-wall building

6 water tower

7 brick building with constructive R/C columns and tie beams

8 round silo



(The earthquake performance improved gradually from 1 to 8)

Fig. 1 The Earthquake Performance of Brick Masonry Structures

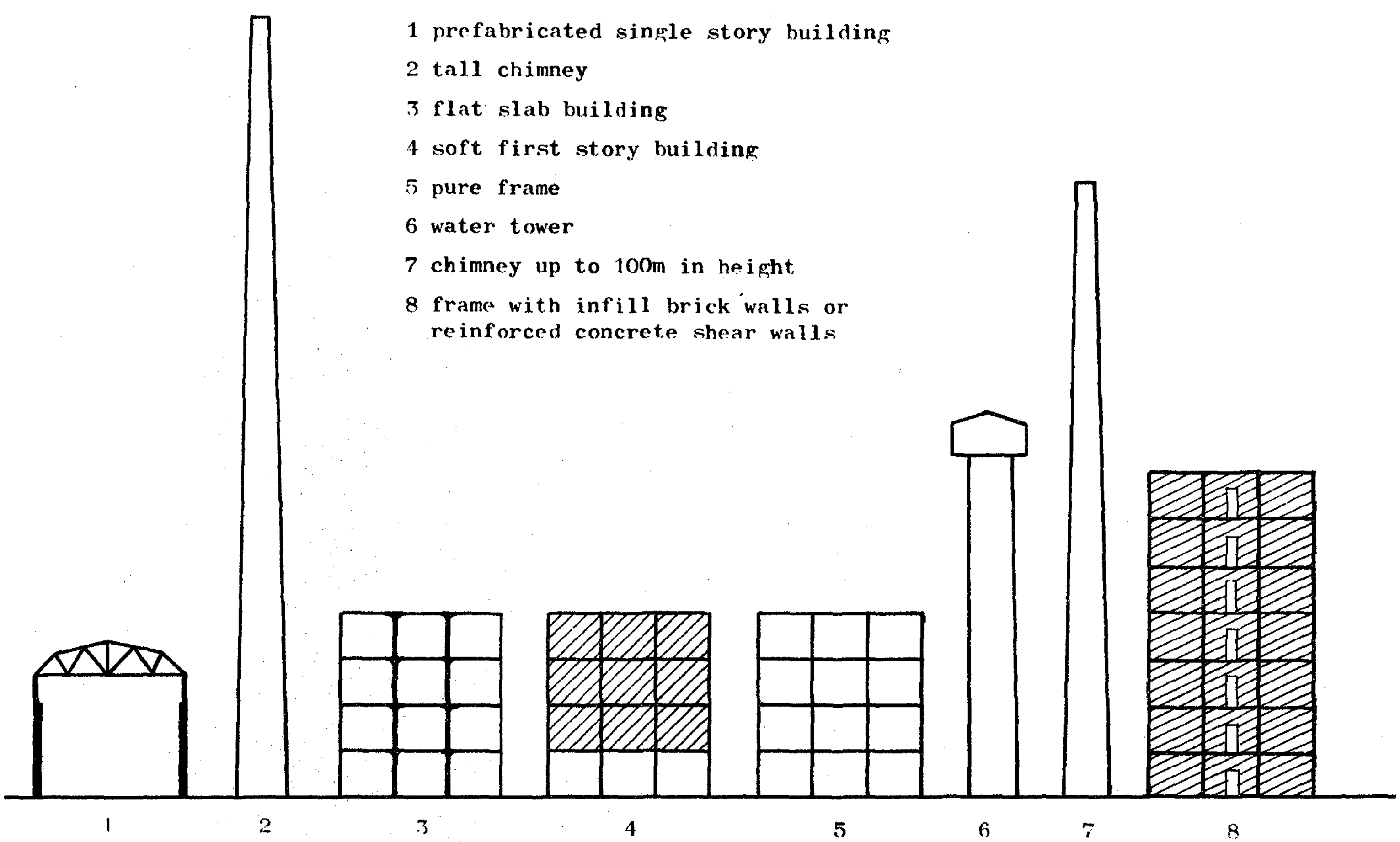


Fig. 2 The Earthquake Performance of Reinforced Concrete Structures

building was mainly affected by the building system. It follows that the shear walls should be added to the flat slab building system in the area with high intensity.

Soft first storey building system Soft first storey building system, which is called "structure supported by frame" or "building supported by chicken legs" in China, consists of soft first storey (pure columns) and rigid stories above it. This building system can provide larger space in first storey. Obviously, it is a good building system in non-seismic region to satisfy the requirements of restaurants and shops in first storey. However, in seismic area, there is an argument on adoption of this system. Many engineers don't agree to use it and the strong earthquakes show that the soft first story building system is a vulnerable, although Martel suggested to adopt the soft first story to reduce the earthquake energy inputted to the building in 1929, and it was recognized the earthquake ground motion can be isolated by the soft first storey. The damage and collapse of such a building are always caused by damage of columns in first storey. The damaged building is very difficult to repair. The collapse of soft first storey buildings were found in Ashkhabad, The Soviet Union, Earthquake(1948), Agadir Earthquake(1960), Skopje Earthquake(1963), Tangshan, China, Earthquake(1976), and Romania Vranca Earthquake(1977) etc. It follows that we have to avoid using the soft first storey building system in earthquake-prone area, especially in the area with high intensity.

Tall slender system Tall slender system mainly refers to independent tall chimneys. Brick chimneys are vulnerable system during an earthquake, and the damage usually occurs in upper part of the chimney shaft. Reinforced brick chimney with vertical steel bars and horizontal ring reinforcements also may collapse in strong earthquakes, in case there are no adequate anchors for vertical steel bars. Reinforced brick masonry chimneys often collapse in segments, which may destroy the buildings nearby. In contrast brick chimneys often collapse in brick pieces. The damage to tall slender reinforced concrete chimneys also occurred in the upper part. During the 1976 Tangshan main shock, the reinforced concrete chimney with 180m in height cracked in upper part which fell down to the ground during the largest after shock.

Requirements for Building Systems

The optimum aseismic building system is dictated by functional, economic and architectural considerations and depends on the seismicity and soil condition of the site as well as the building materials to be used. It is necessary not only to build the building on the stable site but also to have adequate lateral load bearing capacity, higher deformation ability (ductility), better integraty and a certain redundancy.

For any earthquake resistant building system, the following essential requirements should be observed in the aseismic design.

— To have adequate vertical structural elements, such as frames or shear walls, that can transmit all earthquake lateral forces in all directions to the ground and can ensure stability to the building during an earthquake by it's strength and ductility.

— To have adequate horizontal elements or diaphragms, such as roof, floor, bracings or tie rods, that can tie the structure together and can distribute all earthquake loads to the vertical structural elements.

— To have stable foundations, that can anchor the vertical structural elements and can resist all vertical and horizontal loads, including possible uplifts. In design for strong earthquakes, the individual foundations should be tied together.

— To have a certain redundant elements as the second front-line of earthquake resistance.

— To avoid irregular shapes or any abrupt change in elevation.

— To have a good matching with seismic intensity, characteristics of strong ground motions and soil conditions of the site.

BUILDING CONFIGURATION

Building Plan

It is said that building shape in plan should be as simple as possible and the rectangle is the optimum. Many Aseismic Design Codes, including Chinese code, specified that building with irregular shape in plan should be separated into several independent regular units by aseismic joints. According to this requirement some people say that an existing building with irregular plan should be separated artificially in design for strengthening.

However, the construction cost is sure to be increased by the aseismic joints. Moreover, for some buildings the irregular plan is needed in view of functional requirements, limit of the site and architectural considerations. For examples, the peasants and residents living in small cities or towns prefer to live in the house with irregular plan, either or , rather than with rectangle plan, because it has advantage of a convenient life and there is no need to build enclosing walls. Sometimes, it is necessary to build a building with -shape in plan due to limit of the site. As regards tall buildings, irregular shape in plan, such as 人 , , , , is usually needed for stability of the building under large lateral forces and to satisfy the architectural requirements. Therefore, whether the requirements for building plan specified in the codes are true remains a question to be studied.

Soon after the Tonghai Earthquake on January 5, 1970 in Yunnan Province, the author was sent to struck field to observe and report on the structural damage. At that time, the influence of building plan on earthquake performance came into the author's notice. Several interesting examples were found during the inspection of Osan County.

The single story pigpen house with the shape of in plan was stood well after the earthquake. This building had two rows of timber columns and a flat mud roof. It was located at Xiaojie Commune with intensity of IX. In contrast, all the nearby buildings with same structural system and same materials but with rectangular plan were collapsed.

The single story candy workshop with the shape of L was slightly damaged. The building system consists of brick columns and timber truss. It was located in Xiaojie Commune with intensity of IX. While the similar Commune Hospital Building with rectangular plan near in place was levelled to the ground.

The building so called "one stamp" with the shape of in plan is a traditional building in the countryside of Yunnan Province. It has a courtyard in centre. The height of two wings are lower than the main part. Its structural system is timber frames with Chinese traditional jointing method. In the area with same intensity, the "one stamp" buildings almost all stood well, the same buildings with rectangular plan, however, were mostly collapsed in the earthquake.

Shortly after the Tangshan Earthquake on July 28, 1976 in Hebei Province, the author was sent to the struck area to inspect the structural damage. According to the reports on 55 spacious brick buildings (21 buildings were in Tangshan city, 34 buildings were in Tianjin city), we found that the ratio of collapse for the buildings with irregular plan (with front and/or rear halls and/or with side halls) is lower than that with regular plan (without any front, rear, or side halls).

We also found that the round low structures have a good performance in an earthquake. For example, during 1970 Tonghai Earthquake, most of the storage buildings with brick bearing walls in Xiaojie Commune, Yunnan Province (with intensity of IX) were severely damaged or collapsed, except a brick round silo for storage of grain.

The same lesson was relearned from the 1976 Tangshan Earthquake. Few of the round silo for grain storage in mud or brick masonry in the area of grade IX-X were damaged in the earthquake, except the old ones.

The above mentioned examples indicated that:

— The structures with round plan and less height have a good behaviour for earthquake resistance.

— The capacity for prevention of collapse of the buildings with irregular plan, provided with meticulous design and construction, may not be lower than that with regular plan, since the different parts of the building may support each other for existance.

Therefore, the author recognized that it may be beneficial to separate a building into several units of simple shape in plan by aseismic joints in design for mild and moderate earthquakes, because the cost for repair can be lower. However, in design for strong earthquake, prevention of collapse became a princial problem. In this case, separating a building into several units not only needs more funds but also may be disadvantageous to hold the building in an earhtquake. This opinion, whether right or wrong, remains a question to be identified.

Building Elevation

Earthquakes repeatedly demonstrate that the structures with simple elevation shapes have the great chance of survival and the influence of the elevation shape of a building on its earthquake performance is greater than that of the plan shape.

The facade outstanding parts of buildings, either facade setbacks or penthouses, are vulnerable in an earthquake. The main reasons are as follows

— The analytical method for evaluation of the effects of facade outstanding parts is not given in the current normal aseismic design codes.

— About their overall seismic behaviour and structural details we know little.

Figure 4 shows a building with facade setback and its schematic diagram for analysis. Let W_1 and W_2 denote the lumped weights of the main building and the setback respectively. The weight W_1 is joined to the weight W_2 and to the ground by linear, massless springs, whose stiffness are K_2 and K_1 respectively, x_0 is the ground displacement. Suppose $K_2/W_2 = (K_1 + K_2)/W_1$, from dynamic analysis we have

$$P_2 = \frac{1+\sqrt{W_1/W_2}}{2} P_2'$$

where P_2 and P_2' denote the lateral load applied to the weight W_2 in Figure 4b and 4c respectively. If $W_1/W_2 = 25$, we get $P_2 = 3P_2'$. It follows that the lateral load on the weight W_2 in system of Figure 4b is much larger than that in system of Figure 4c.

Therefore, in Article 19 of the Chinese Aseismic Design Code specified that

"For checking earthquake resistant strength of penthouse, parapets and stovepipes projected over the roof, the horizontal seismic load may be taken to be 3 times the calculated values from fomula (4)".

Generally, the weight of the penthouses, parapets and stovepipes projected over the roof is much less than that of the main building, so that the magnified coefficient 3 seems to be small in some cases.

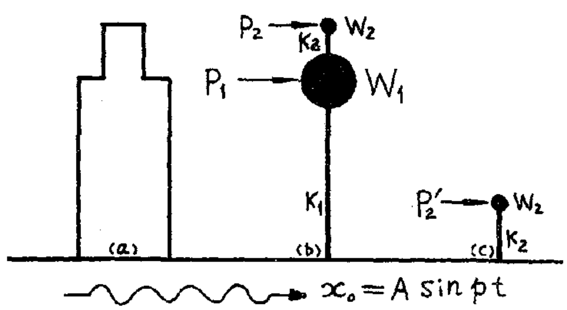


Fig. 4 Building with facade setback and its schematic diagram for analysis

INTERACTION OF STRUCTURAL AND NON-STRUCTURAL ELEMENTS

Any building always consists of two kinds of elements. The first is structural elements, which carry the dead loads, live loads, and environmental loads. The second is non-structural elements, which join or attach to the structural elements to satisfy the functional requirements.

The interaction of structural and non-structural elements involves both the influence of structural system on non-structural elements and vice versa. During an earthquake, the structural system moves by ground excitation at its base. As the structural system moves, the non-structural elements move too. This is a only reason for their damage.

There are several damage mechanisms for non-structural elements. They are:

— Damage to connections between the structural elements and the non-structural elements due to earthquake inertial forces itself.

— Change of geometrical shape of the infill elements due to drift of the structural system, especially the story drift.

— When the infill elements have a certain stiffness and its effect is ignored in design, the earthquake loads applied to them will be much larger than that of prediction in the elastic range. In this case, the damages take place not only in the infill elements but also in the structural elements joined to them. As a result, the building may be collapsed by the interaction between infill elements and structural elements.

The damage to non-structural elements and the measures for earthquake resistance are listed in Table 1 based on the lessons learned from the recent Chinese earthquakes. It is worthy of note that sometimes the cost of non-structural elements attains to almost half of the construction cost of the building. Therefore, aseismic design of non-structural elements is an important project to be studied.

URBAN EARTHQUAKE DISASTER MITIGATION

Urban Earthquake Disasters

Lessons learned and relearned from past earthquakes occurred in China and abroad showed that morden cities are more vulnerable in destructive earthquakes. This conclusion can be clearly confirmed from the following examples.

In China, during the period of recent 30 years, 11 destructive earquakes took place in main land, 9 of which occurred in countryside. The 1975 Haicheng Earthquake effected industrial cities of Yingkuo and Anshan. The 1976 Tangshan shock took place right in the down town of Tangshan city and also caused damages to certain degree in Tianjin and Beijing. The loss of life caused by Tangshan event is about 90% of the total loss of life caused by earthquakes occurred in this period of time. Concern with the economic losses, they were about 70% and 15% of the total in Tangshan and Haicheng Earthquakes respectively. It follows that the better part of the losses comes from the earthquakes occurred in city or near it.

In the United States, the total of 1,600 people were killed by earthquakes in history. About half of the total life losses were caused by the 1906 San Francisco Earthquake. The total economic losses in history were about 2 billion dollars. However, most of them were caused by San Francisco Earthquake (1906, 520 million dollars), Alaska Earthquake (1964, 500 million

TABLE 1 DAMAGE TO NON-STRUCTURAL ELEMENTS AND ASEISMIC MEASURES

type	example	damage	effect	causes of damage	aseismic measures
vertical outstanding	- brick parapet - brick stovepipe	- crack - collap-se	- death and injury of people - ruined nearby struc-tures	- amplified lateral load - weakness in brick masonry	- decreasing its height - reinforcing brick masonry
attached	- ornament - finish - face brick	- crack - fall down	- injury and death of people	- weakness in connection - drift of structure - collapse of structural element joined	- strengthening of connections - limit to drift
horizontal outstanding	- balcony - canopy	- fall down	- death and injury of people - ruined nearby struc-tures	- collapse of supporting structure	- control collapse of supporting structure
vertical	- partition - cladding	- crack - collap-se	- death and injury of people - ruined bearby struc-ture and contents of rooms	- weakness in connection with structure - lower strength and ductility	- strengthening of connections - increasing lateral load bearing capacity
infill	- brick wall - panel - window -door	- crack - collap-se - glass breakage - defor-mation	- death and injury of people - ruined nearby contents of rooms - changing response of structure - clogging door	- no account its effect in design - weakness in connection - storey drift	- considering its effect - strengthening of connections - limit to drift - separating from structure
suspended	- ceilling - hanging wall plate - hanging lamp, lan-tern	- fall down	- death and injury of people - ruined nearby structure and contents of the rooms	- weakness in connection	- strengthening of connections

dollars), and San Fernando Earthquake (1971, 523 million dollars), which were occurred in nearby cities. The experts of the United States expect that if the destructive earthquake occurs in Los Angeles in the future, the loss of life would be 15,000 people and the economic losses would be 25 billion dollars.

In Japan, during the Miyagiken-oki Earthquake of June 12, 1978, 28 people died and more than one thousand people injured. The property loss amounted 276 billion yen (1.1 billion dollars). It affected capital city and 7 counties. The disaster was supposed as one of the biggest earthquake disasters since the 1923 Kanto Earthquake. While 42 years ago, in the same area had a stronger earthquake occurred, the losses were less than this one. So that, the Japenese said with deep feeling that this earthquake fully proved that the existing cities which are guaranteed, although on the surface, by morden technology are so vulnerable in an strong earthquake.

The above mentioned earthquake disasters show the prevention of urban earthquake disaster is a main task of the earthquake disaster mitigation.

Lessons From 1976 Tangshan Earthquake

The Tangshan Earthquake of July 28, 1976 brought great disasters to Tangshan city with a urban population of about six hundred thousand and many disasters to Tianjin and Beijing. It took the lives of 242,000 people, caused 164,000 injuries. The macroepicenter was in Tangshan down town area. A lot of useful lessons for urban earthquake disaster mitigation can be drawn from this earthquake.

Density of buildings While the density of buildings and population decreased, the disaster will be mitigate. The Lunan District of Tangshan city has a highest density of buildings, which was about 70%. On the average, about 15,000 people lived in a square kilometer. The death-rate in this district was more than 45%. In other area of Tangshan city, the death-rate was about 21.3%, while in Tangshan suburbs, it was 14%. In rural area, such as Fengnan county, although the intensity over there is the same with Tangshan urban area, the death-rate was only 10%. Moreover, in the region with higher density of buildings, many people who escaped from the houses died in narrow lanes during the earthquake. The debris filled in some lanes higher than one meter. Many structures were damaged or collapsed by the debris fallen from the nearby collapsed buildings. It follows that reduction of the density of buildings and population is an effective measure for mitigation of urban earthquake disasters.

Prediction of the basic seismic intensity The basic intensity of a region denotes the maximum possible seismic intensity caused by earthquake that may be occurred in that region during a future period of one hundred years under ordinary site condition. In this country, in order to make aseismic design of the structures and to strengthen existing structures as well as to make urban planning and to select the measures for reducing disasters of a given city, the seismic hazard analysis study, i.e. basic seismic intensity estimate, is required as the initial step. The Tangshan Earthquake shows that the main cause which led to great disasters is that the basic intensity of Tangshan region had been assessed too low. In fact, before the Tangshan shock, the basic intensity in Tangshan city was of only

VI. It means that the structures were not required to be designed to resist earthquakes. Actually, the intensity in Tangshan city attained to XI during the Tangshan Earthquake. The same situations were found in Xiantai Earthquake (1966), Tonghai Earthquake (1970), and Haicheng Earthquake (1975) etc. Therefore, correct assessment of the basic intensity is a strategical and basic measure for urban disaster mitigation.

Secondary disasters Tangshan is a coal mine base in China. After the earthquake, the underground passageways were intact, but were inundated with water, which was 1.7 to 5 times that of the usual amount. The maximum gush in a well was up to $160m^3$/min. It took a year to resume due to interruption of power supply at that time. Hence, secondary power supply and emergency drainage facilities must be available for rapid recovery.

In Kaiping Chemical Plant, liquid chloride flew out due to damage of the valves. It took the lives of two people. Fortunately, it was diluted by rain water, which avoided another disaster.

Explosions and fires happened in some institutions because of damage to chemical containers. Severe disaster, however, did not take place due to prompt rescue.

The Douhe river flows through the Tangshan city. The Douhe resevoir is situated at 15km up the river. It was damaged in the earthquake. A 1700m long longitudinal cracks with width of 1-1.5m and many transversal cracks were found in the dam. As a result, the dam leaked after the earthquake. Fortunately, the dam was stood because of low water and prompt repair.

The reinforced concrete chimney of 180m high in Douhe Power Station broke at the height of 132m during the main shock, and fell down in the largest aftershock right on the day. The nearby conveying corridor was collapsed by the fallen debris. Some factory buildings were damaged or collapsed due to fallen sections or brick pieces of the brick chimneys. Therefore, the buildings should be constructed at a certain distance away from the tall chimneys.

Lifeline engineering After the earthquake, the power supply in Tangshan were completely interrupted due to collapse of buildings and damage of equipments. Power supply was temporarily restored by means of truck power on July 28. The power was supplied to Tangshan through the network on July 29. The Tangshan power plant was completely recovered by the end of 1976.

Interruption of water supply in Tangshan was mainly caused by collapse of water structures, such as water works, towers, buried pipelines and well pipes. A lot of pipelines buried under debris were difficult to repair. The potable water supply in Tangshan city was resumed on August 10, and the water supply was recovered in late September. While in Tianjin city, it was restored at the end of August.

After the earthquake, the communication in Tangshan city was entirely cut off due to the collapse of buildings and damage to equipments, lines and poles. The emergency communication to Beijing was connected through underground cables, which were intact in the morning of July 18. It was entirely restored on September 1.

The traffic was blocked up both in Tangshan and Tianjin cities after the event, since the debris filled on some roads. For example, there was no transport service on 80% of roads in Heping District, Tianjin city. After the earthquake, traffic was interrupted over 10 hours in the north-south main road in Tangshan city. Vehicles lined up 10km on the Tangfeng road because of the interruption of interurban traffic and lack of qualified traffic controlers . The collapse of Shengli bridge cut the transport service between the down town area and east mine district of Tangshan city. The failure of the Ji Canal bridge led to the periodical traffic interruption between Tangshan and Tianjin cities. All this had a severe effect on rescue work.

Both the natural gas and the contained liquefied gas were slightly damaged, and gas supply was resumed basically in late August. The contained liquefied gas not only satisfied the previous customer's needs but also provided the gas for hundred rescue medical teams.

Tangshan Airport suffered damages of only some buildings, normal operation was soon resumed.

The above mentioned facts indicated that safegurding security of the lifeline systems, especially the power supply systems, is very important to mitigate disaster, recover production and rescue.

Underground construction In the area with intensity of X-XI, underground constructions were slightly damaged, while nearly all the buildings on the ground surface were levelled to the ground. Moreover, the buildings with basements had slighter damages than those without basements. Especially, nearly thirty thousand people working underground safely returned to the ground after the main shock. It is worthy of note that the earthquake performance of the underground structures greatly depent on the behaviour of the surrounding soils. Generally, buildings on soft soil are vulnerable in an earthquake. Besides, attention should be paid to entrances and exits which might be blocked by ruins.

Open space After the earthquake, the parks and green area were very useful for taking refuge and rescue. The Fenghuang park and People's park in Tangshan down town were used as the main sanctuarys. While the green open space in Tangshan Airport was used as the medical centre. The main parks in Beijing and Tianjin city provided refuge places for 300,000 people.

Requirements of Urban Planning

The earthquake resistant requirements should be considered in making urban planning for seismic regions in order to maximum reduce earthquake disasters. The principal requirements can be listed as follows.

— Getting size and population of a city under control, and constructing more small cities and towns.

— Getting density of buildings under control, and the distance between two buildings should be larger than the sum of their height.

— Adopting the underground structures if it is possible. Critical facilities for urban communication, water supply and power supply should be installed in underground structures as much as possible. The underground structures should not be constructed in soft soil, and their entrances and exits should be situated in free fields to prevent clogging.

— The circular networks and multiple entrances and exits as well as standby facilities for power supply, communication, water supply and traffic systems shall be adopted.

— The requirements of refuge, evacuation and rescue should be considered in road planning. Each road shall have adequate width and multiple entrances and exits to prevent interruption of transport service.

— Rational land use planning should be made based on the seismic risk analysis. Riskful areas should be used as the parks or green open space.

— The water area, either rivers or lakes, should be preserved.

— The factories and storages for production or storage of materials and goods, which are vulnerable to fire and/or explosion and/or violently poisonous, should be built far away from the city. The nearby reservoir should be situated at a certain distance down the river.

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BUILDING CONFIGURATION INFLUENCE ON SEISMIC PERFORMANCE: THE WESTERN EXPERIENCE

Christopher Arnold [1]

ABSTRACT

This paper reviews the relationship between building configuration and seismic performance for typical Western building types based on experience in 20th century earthquakes.

A set of configurations are identified which, unless designed with special concern for dynamic performance, are liable to perform poorly under moderate to strong ground motion conditions.

Three of these configurations are analyzed in more detail to explain their failure mechanisms in a conceptual way. Architectural and engineering approaches to alleviate problems caused by these configurations are reviewed.

INTRODUCTION

Experience in a number of twentieth century earthquakes has made clear the fact that building configuration - the size, shape and disposition of the main structural and non-structural building elements - has a major effect on seismic performance. Moreover, there are strong indications that recent architectural forms, introduced in the United States since about 1940 and becoming extremely prevalent during the extensive building programs between 1950 - 1970, perform worse than more traditional configurations. In particular the use of wide-span frame structures, particularly those of non-ductile concrete built before the 1970's, and the much lower intensity of solid walls, contribute to forms which are intrinsically weak unless very carefully designed for considerations of dynamic behavior.

The problem is increased by the fact that our seismic building codes do not recognize these issues of building form except by general admonitions of a non-mandatory nature. Moreover, the computation of design forces based on the code formulae assumes a simple building concept with none of the irregularities itemized later in this paper. In fact, such buildings are rarely designed and so, perhaps inadvertently, the architects' concepts work against the code's attempt to provide for safety and property loss reduction.

A graphic summary of the main configurations which have been shown to present problems if poorly designed is shown in Figures 1, 2, 3, 4. Three of these are selected for more detailed study later in this paper, and analyzed in a conceptual way to explain their failure mechanisms.

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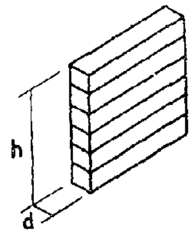
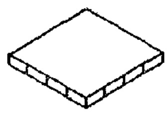
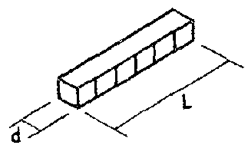
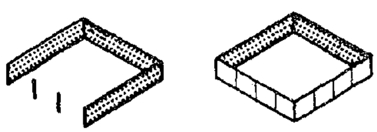
1. Architect, President, Building Systems Development, Inc., San Mateo, California.

PROBLEM CONFIGURATION	STRUCTURAL PROBLEM
A. PROBLEMS IN EXTREME DIMENSIONS	
 1. EXTREME HEIGHT-DEPTH RATIO	high overturning forces; large drift may cause extensive non-structural damage
 2. EXTREME PLAN AREA	build-up of large diaphragm forces
 3. EXTREME ELEVATION LENGTH-DEPTH RATIO (ASPECT RATIO)	build-up of large lateral forces in perimeter; big difference in resistance of two axes
B. PROBLEMS OF HORIZONTAL LAYOUT	
 1. SIMPLE PLANS a. VARIATIONS IN PERIMETER STRENGTH AND STIFFNESS	torsion caused by extreme variation in strength and stiffness between different elevations.
	Figure 1

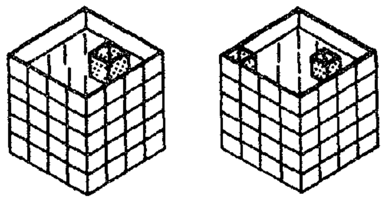
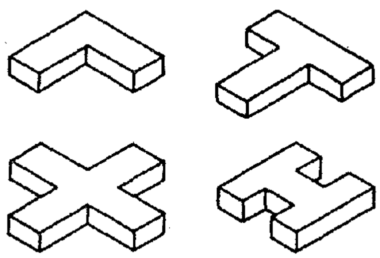
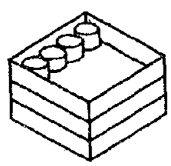
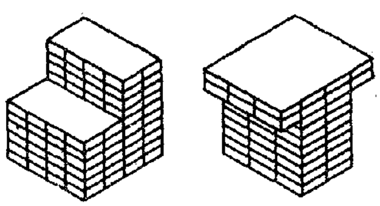
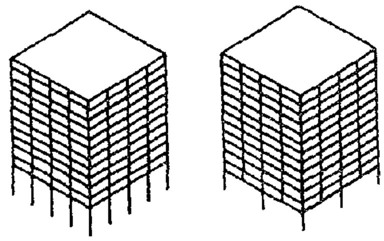
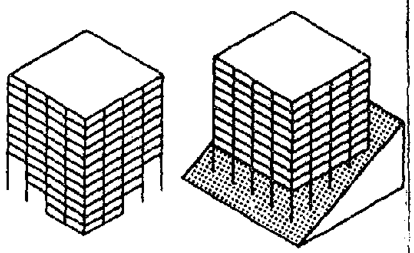
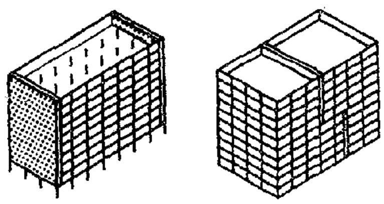
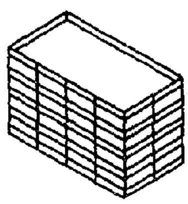
PROBLEM CONFIGURATION	STRUCTURAL PROBLEM
1. SIMPLE PLANS	
 b. FALSE SYMMETRY	torsion caused by stiff asymmetric core
 2. RE-ENTRANT CORNERS	torsion; and stress concentrations at the 'notches'
 3. MASS ECCENTRICITY	torsion; stress concentrations
C. PROBLEMS OF VERTICAL LAYOUT	
 1. VERTICAL SETBACKS & INVERTED SETBACKS	stress concentrations at notch; different periods for different parts of building; high diaphragm forces to transfer at setback

Figure 2

PROBLEM CONFIGURATION	STRUCTURAL PROBLEM
 2. SOFT STORY-FRAME	causes abrupt change of strength and stiffness at point of discontinuity
 3. VARIATIONS IN COLUMN STIFFNESS	causes changes of stiffness and much higher forces in stiffer columns
 4. DISCONTINUOUS SHEAR WALLS	results in discontinuities in load path and stress concentration in most heavily loaded elements
 5. WEAK COLUMN-STRONG BEAM	column failure occurs before beam, short column must try and accommodate story height displacement
	Figure 3

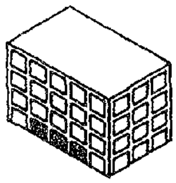
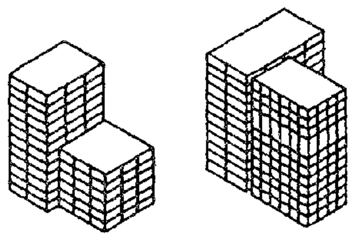
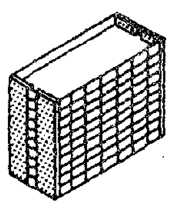
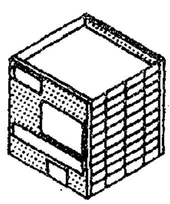
PROBLEM CONFIGURATION	STRUCTURAL PROBLEM
 6. MODIFICATIONS OF PRIMARY STRUCTURE	most serious when masonry infill modifies structural concept. creation of short, stiff columns results in stress concentration and their receiving undue proportion of loads
D. PROBLEMS OF ADJACENCY  1. BUILDING SEPARATION	possibility of pounding damage dependent on building periods, heights, drift, distance apart
E. SHEAR WALLS  1. COUPLED	incompatible deformation between strong walls and weak links
 2. RANDOM OPENINGS	may seriously degrade shear capacity at points of maximum force transfer

Figure 4

VARIATIONS IN PERIMETER STRENGTH AND STIFFNESS

A building's seismic behavior is strongly influenced by the nature of its perimeter design. If there is wide variation in strength and stiffness around the perimeter, the center of mass will not coincide with the center of resistance, and torsional forces will tend to cause the building to rotate around the center of resistance. This effect is illustrated in Figure 5.

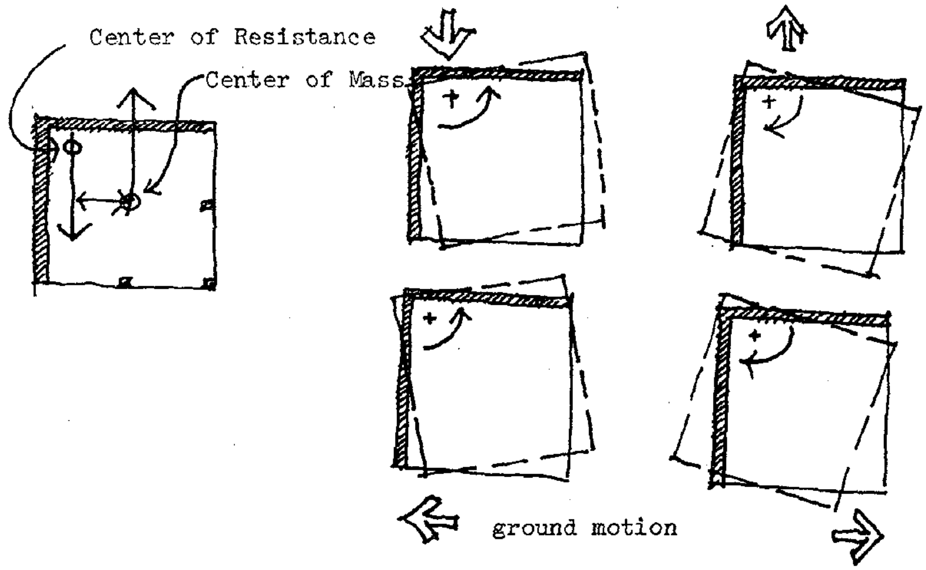


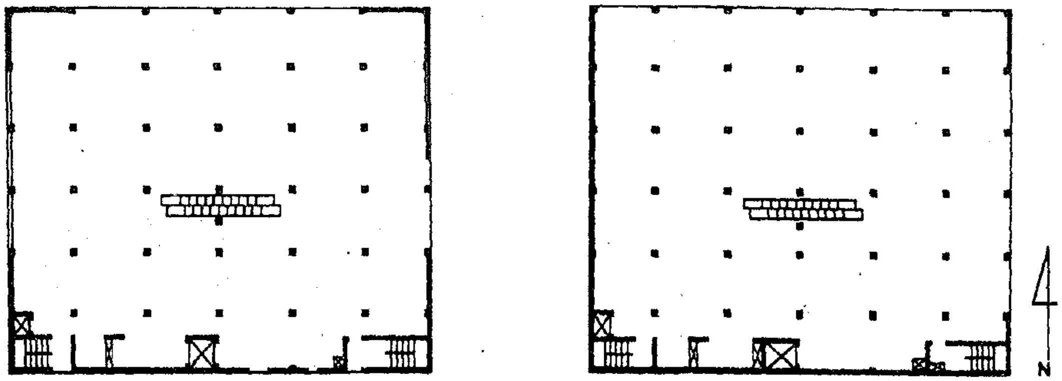
Figure 5 Torsional forces caused by variations in perimeter resistance

This condition causes much building damage and collapse. Henry Degenkolb has discussed a form of this problem in a way that clearly emphasizes its effect. (Degenkolb, 1977)

"The effect of torsion can probably be best illustrated by one of the most common building constructions in the United States if not the world. The side walls are on property lines, the rear wall is either on a property line or faces an alley. The rear wall has minimum openings, if any, but the front wall with its display windows on the street is essentially open. When shaken by an earthquake, the rear and side walls are quite rigid but the front wall is very flexible, and the roof tends to twist."

A classic instance of this kind of effect is that of the J.C. Penney building in Anchorage, in the 1964 Alaska earthquake. The building was so badly damaged by torsional forces that it had to be demolished. The store was a five story building of reinforced concrete construction. The exterior wall was a combination of poured-in-place concrete, concrete block, and precast concrete non-structural panels which were heavy, but unable to take large stresses. Steinbrugge, Manning and Degenkolb discuss the source of this torsion. (Steinbrugge, et al, 1967)

"Torsional forces were not a significant factor in the first story, since shear walls were found along all street fronts. The upper stories, however, had a structurally open north wall, and large torsional forces would arise from the U-shaped shear wall bracing system when subjected to east-west lateral forces." (Figures 6, 7)



FIRST FLOOR FOURTH FLOOR

Figure 6 Plans of J.C. Penney Store, Anchorage, Alaska

Figure 7 Damage to the J.C. Penney Store, Anchorage, Alaska
Note undamaged glass faced building in foreground.

Open front design is also common in buildings such as fire stations and motor maintenance shops, in which it is necessary to provide large doors for the movement of vehicles. In fire stations, it is particularly important to avoid even slight distortion of the front frame, for if the doors cannot be raised, the fire station is out of action at a time when its equipment may be urgently needed.

The object of any solution to this problem is to reduce the possibility of torsion. Four alternative strategies can be employed. These are illustrated in Figure 8.

The first strategy is to design a frame structure with approximately equal strength and stiffness for the entire perimeter. Opaque portions of the perimeter can be constructed of non-structural cladding, designed so that it does not affect the seismic performance of the frame (Figure 8a). This can be done either by using lightweight cladding, or by ensuring that heavy materials - such as concrete or masonry - are isolated from the frame.

A second approach is to increase the stiffness of the open facades by adding shear walls at or near the open face (Figure 8b). This solution is, of course, dependent on a design which permits this addition.

A third solution is to use a very strong moment resisting or braced frame at the open front, which approaches the solid walls in stiffness (Figure 8c). The ability to do this will be dependent on the side of the facades: a long steel frame can never approach a long concrete wall in stiffness. This is, however, a good solution for wood frame structures, such as apartment houses with a ground floor garage, because even a long steel frame can be made as stiff as plywood shear walls.

Finally, the possibility of torsion may be accepted and the structure designed to resist it (Figure 8d). This solution will only apply to relatively small structures with stiff diaphragms, which can be designed to act as a unit.

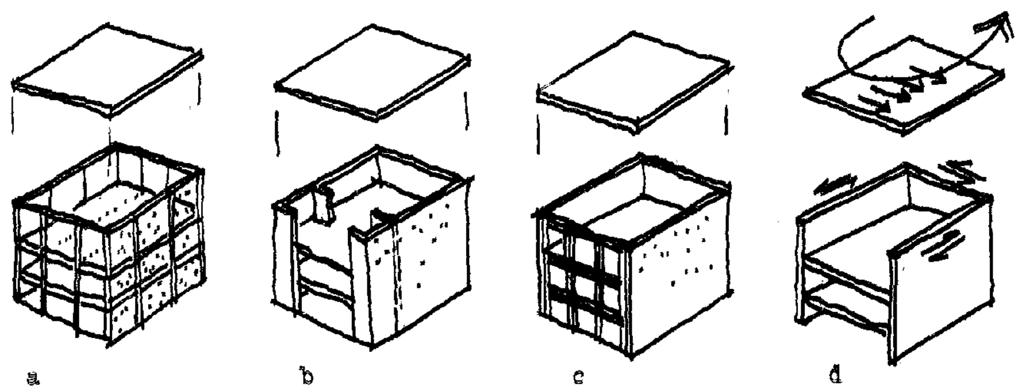


Figure 8 Solutions to perimeter variation

RE-ENTRANT CORNER CONFIGURATIONS

The re-entrant, or inside, corner is the common characteristics of overall building configurations that, in plan, assume the shape of an L, T, U, H, + or a combination of these shapes (Figure 9).

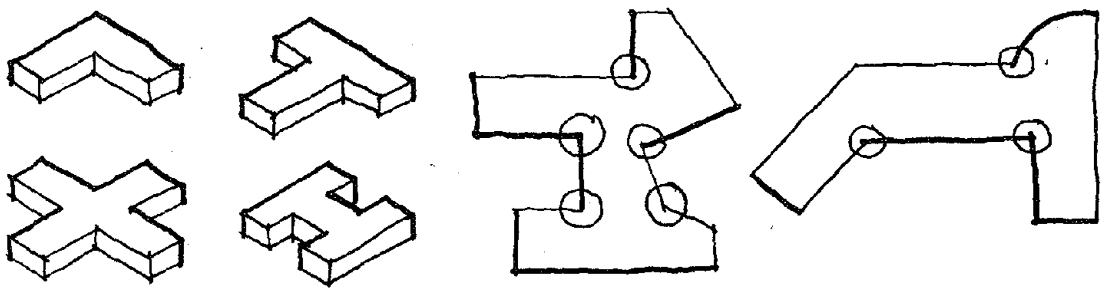


Figure 9 Re-entrant corner configurations. The re-entrant corners are circled in the two right hand sketches.

This is a most useful set of building shapes, which enable large plan areas to be accommodated in relatively compact form, while still providing a high percentage of perimeter rooms with access to air and light. The advent of air-conditioning and fluorescent lighting reduced the necessity for perimeter access for ventilation and daylight, and produced the characteristic Western deep plan forms of the mid-twentieth century. Current interest in daylighting and natural ventilation for energy conservation may result in a return to narrow buildings and the traditional set of re-entrant corner configurations.

There are two problems created by these shapes. The first is that they tend to produce variations of rigidity and hence differential motions between different portions of the building, resulting in a local stress concentration at the re-entrant corner.

This problem is illustrated in Figure 10. If ground motion with a north-south emphasis occurs, the wing oriented north-south will, purely for geometrical reasons, probably tend to be stiffer than the wing which is located east-west. The north-south wing, if it were a separate building, would tend to deflect less than the east-west wing, but the two wings are tied together and attempt to move differently at their junction, tearing and pushing each other. In addition, the forces will be dynamic; there will be to- and- fro motion causing further damage. For ground motion along the other axis, the wings reverse roles but the differential motion problem remains.

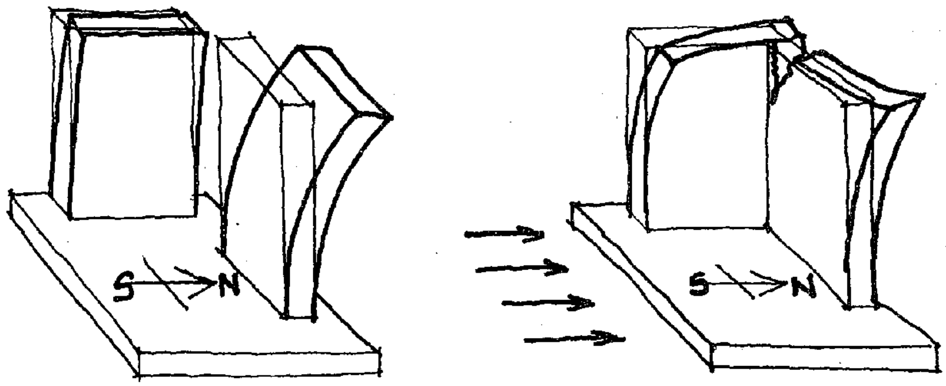


Figure 10 The comparison between two separated rectangular buildings and an L-shape

The second problem of this form is torsion, or twisting. This is caused because the center of mass and center of rigidity cannot geometrically coincide for all possible earthquake directions. The result is rotation which will tend to distort the structure in ways that will vary in nature and magnitude depending on the nature and direction of the ground motion, and result in forces that are very difficult to analyze and predict.

Examples of damage to re-entrant corner type buildings are common, and this problem was one of the first to be identified by observers.

The damage to the West Anchorage High School in the 1964 Alaska earthquake is typical, and since complete collapse did not occur, the sequence of events can reasonably be reconstructed, as was done by engineer John Meehan. (Meehan, 1967)

"One cannot be certain of the sequence or path of distress; however, it is believed that the initial damage occurred in the roof diaphragm at the vertex of the angle formed by the two portions of the classroom wing due to torsional moment developed in this diaphragm. It is also believed that, after the roof diaphragm separated at this point, each portion of the classroom wing essentially formed individual buildings, thus necessitating a redistribution of load in the shear walls. The shear walls were not capable of resisting this redistribution of load and were apparently damaged next. The exterior second-floor columns were then unable to resist the total load alone, and damage developed in these. At this point, the earthquake action stopped. No damage was observed to the more flexible center corridor columns." (Figures 11, 12)

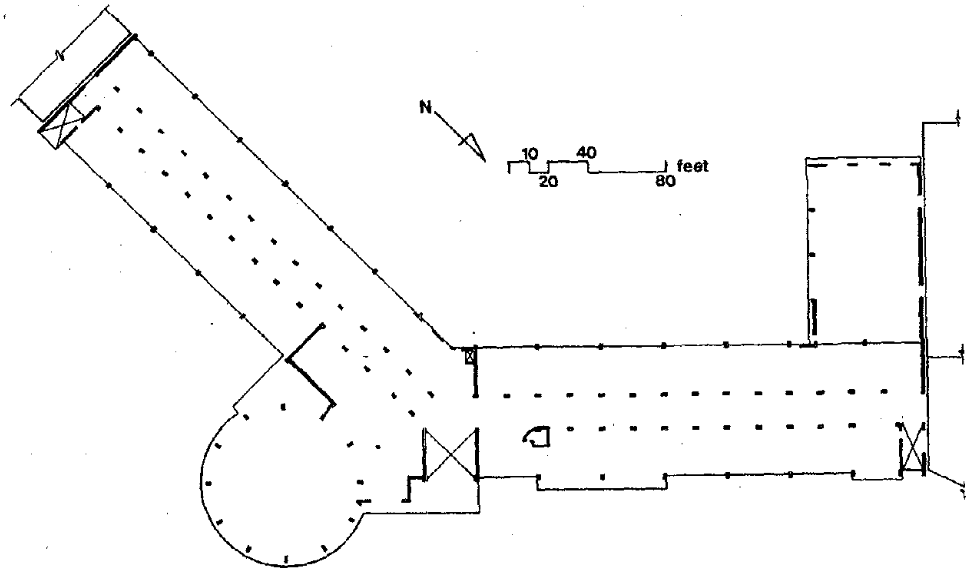


Figure 11 Plan of West Anchorage High School, Alaska, 1964

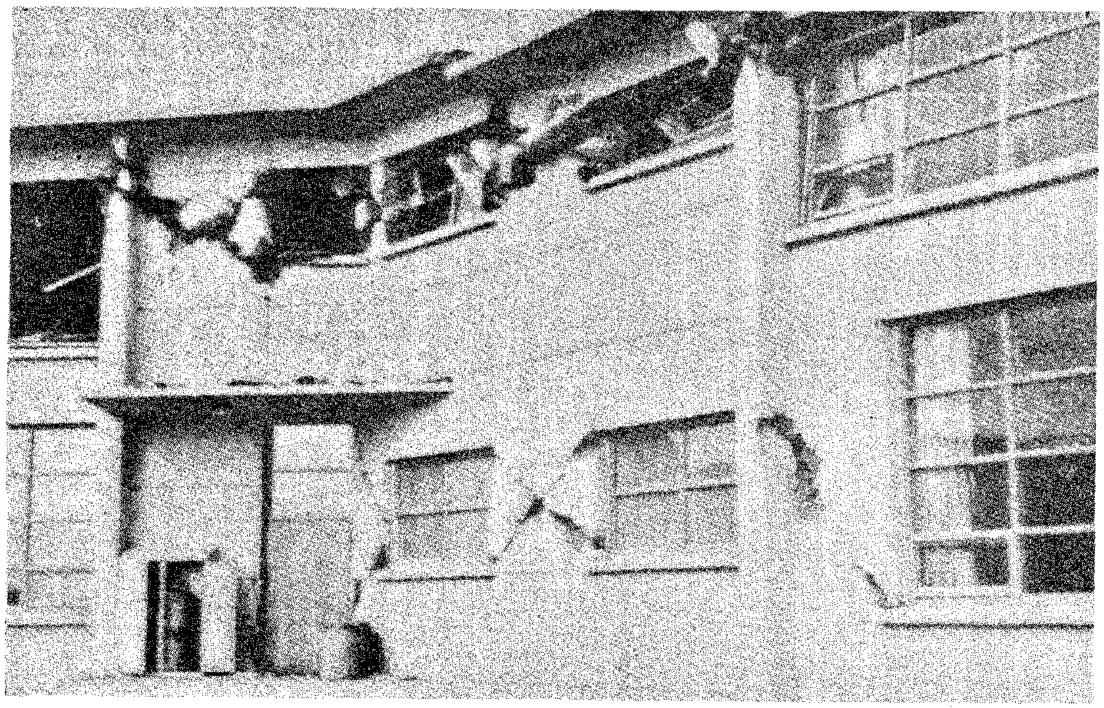


Figure 12 Damage to West Anchorage High School Alaska, 1964
Concentration of forces in the "notch"

The re-entrant corner plan well illustrates the dangers of transferring structural behavior from one scale to another. There is no comparison between a solid steel H that is only about one foot square, and a building with wings one hundred or more feet long connected by occasional floor slabs.

The latter will not behave homogeneously, and the force will be transferred through dozens of columns, beams, slabs, and connections, all varying in their strength and stiffness and transferring forces one to the other with varying eccentricity and direction.

There are two basic alternative approaches to the problem of the re-entrant corner forms: structurally to separate the building into simpler shapes, or to tie the building together more strongly.

Several considerations arise if it is decided to dispense with the separation joint and tie the building together. Collectors at the intersection can transfer forces across the intersection area, but only if the design allows for these beam-like members to extend straight across without interruption. Even better than collectors are walls in this same location (Figure 13).

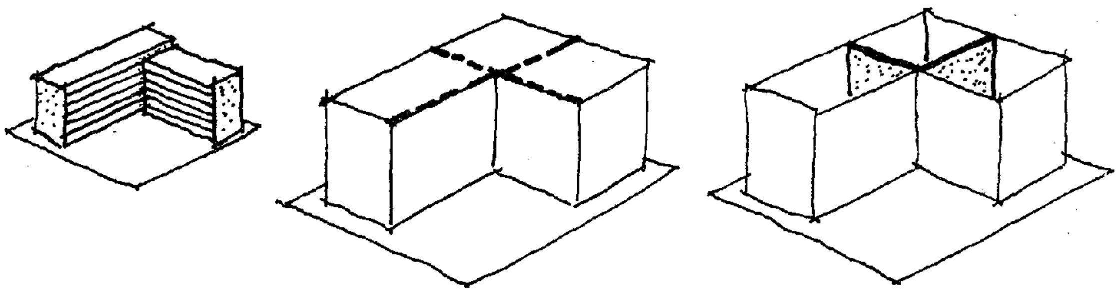


Figure 13 Building separation, or tieing the building securely together with beam-like collectors or walls.

DISCONTINUITIES OF STRENGTH AND STIFFNESS

The set of problems created by discontinuous strength and/or stiffness has been well stated by Hanson and Degenkolb. (Hanson/Degenkolb, 1975)

"if there is a single zone of weakness in the path of force transmission, or if there is a sudden change of stiffness, there is a zone of danger. Even when the structure remains elastic the response will change considerably and the distribution of forces throughout the height of the structure can vary substantially from the assumed triangular distribution. However, it is even more critical when the structure has begun to deform inelastically.

"...If it can be assumed that the code required lateral forces are based on the performance of an older style typical structure where there was no sudden change of stiffness, then the absorption of the earthquake energy is distributed throughout the structure, either uniformly or in some regular continuous pattern. If a structure has much more flexible portion under a rigid portion, most of the energy absorption is concentrated in the flexible portion and very little is absorbed in the more rigid portion above... "

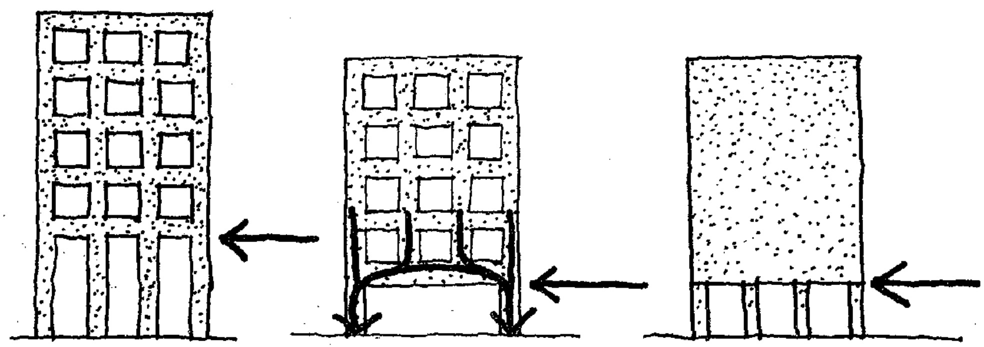
THE 'SOFT STORY'

The most prominent of the set of problems caused by discontinuous strength and stiffness is that of the 'soft story.' This term has commonly been applied to buildings whose ground level story is weaker than those above. However, a soft story at any floor creates a problem, but since the forces are generally greatest towards the base of a building, a stiffness discontinuity between the first and second floors tends to result in the most serious condition (Figure 14).

The soft story occurs when there is a significant discontinuity of strength and stiffness between the vertical structure of one floor and the remainder of the structure. This discontinuity may occur because one floor, generally the first, is significantly taller than the remainder, resulting in decreased stiffness (Figure 14a).

Discontinuity may also occur as a result of a common design concept in which all vertical framing elements are not brought down to the foundation, but some are stopped at the second floor to increase the openness at ground level (Figure 14b). This condition creates a discontinuous load path resulting in an abrupt change of strength and stiffness at the point of change.

Finally, the soft story may be created by an open floor which supports heavy structural or non-structural walls above (Figure 14c). This situation is most serious when the wall above is a shear wall, acting as a major lateral force resistant element: this condition is discussed here in more detail, since it represents an important special case of the soft story problem.



a b

Figure 14 The 'Soft Story'

When shear walls form the main lateral resistant elements of the buidling, they may be required to carry very high loads. If these walls do not line up in plan from one floor to the next, the forces created by these loads cannot flow directly down through the walls from roof to foundation, and the consequent indirect load path can result in serious overstressing at the points of discontinuity.

Olive View Hospital, which was severely damaged in the 1971 San Fernando, California earthquake, represents an extreme form of the discontinuous shear wall problem. The general vertical configuration of the main building was a 'soft' two-story layer of rigid frames on which was supported a four story (Five, counting penthouse) shear wall-plus-frame structure. The second floor extends out to form a large plaza: thus, in photographs, the main building appears to have a single soft story, rather than two (Figure 15).

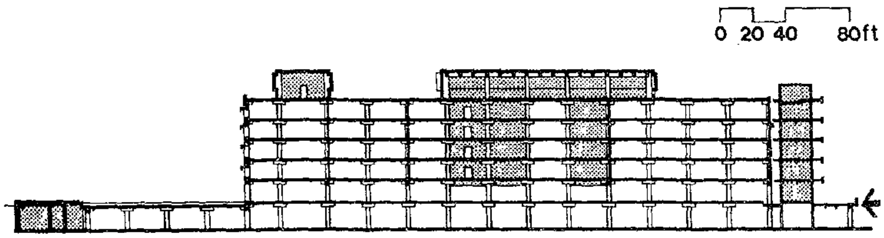


Figure 15 Olive View Hospital

The severe damage occurred in the soft story portion, which is generally to be expected. The upper stories moved as a unit, and moved so much that the columns at ground level could not accommodate such a huge displacement between their bases and tops and hence failed. The largest amount by which a column was left permanently out of plumb was 2-1/2 feet.

A discontinuity in vertical stiffness and strength leads to a concentration of stresses and damage, and the story which must hold up all the rest of the stories in a building should be the last, rather than the first, component to sacrifice. Had the columns at Olive View been more strongly reinforced, their failures would have been postponed, but it is unrealistic to think that they would have escaped damage. Thus the significant problem lies in the configuration, and not totally in the column reinforcement.

Though it is not as widely known, the stairtowers at Olive View also show a clear example of a discontinuous shear wall failure. The nature of this failure is not obvious, since the plaza formed by the extended second floor gives the towers the appearance of being only six stories in height, when actually they are seven.

These seven-story towers were independent structures, and proved incapable of standing up on their own: three overturned completely, while the fourth leaned outward 10°. The six upper stories were rigidly braced with ample solid reinforced concrete walls, but the bottom, or 'soft' story was composed of six reinforced concrete columns, which failed. The exception was the north tower, whose walls came down to the foundation directly without any discontinuity; this was the only tower which remained standing. Obviously, none of the towers was adequately built to prevent overturning, since the 10° out-of-plumb movement of the north tower might easily be called "failure," but it is clear that this flaw was compounded into total collapse only where the soft story was present (Figure 16).

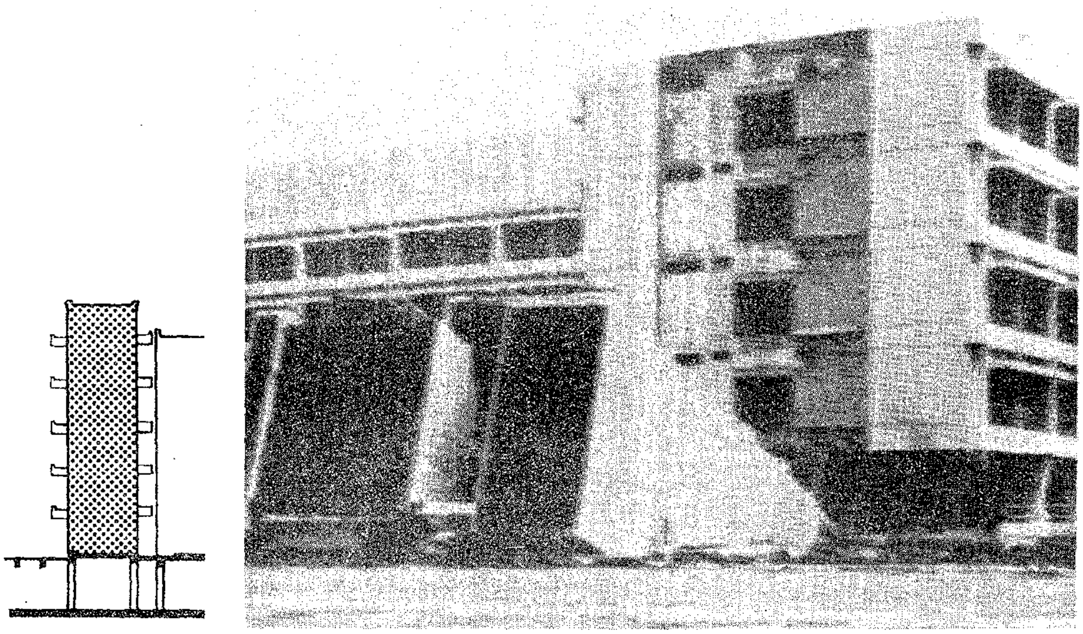


Figure 16 Stair tower section and damage.

While one may attribute the proximate cause of these stair tower failures to the detailed design of the reinforced concrete columns which failed (such as the inadequacy of their ties) and to the extreme ground motion, is clear that the configuration factor was responsible for setting up this over-stress situation. No matter how well the reinforcing is designed, a more reliable general solution would have been to eliminate the discontinuity created by the termination of the shear walls.

The solution to the problem of the discontinuous shear wall is unequivocally to eliminate the condition. To do this may create architectural problems of planning or circulation or of image. If this is so, then it indicates that the decision to use shear walls as resistant elements was wrong from the inception of the design. Conversely, if the decision is made to use shear walls, then their presence must be recognized from the beginning of schematic design, and their size and location early made the subject of careful architectural and engineering coordination.

CONCLUSION

It is clear that seismic design is a shared architectural and engineering responsibility. It is shared in the physical relationships between architectural forms and structural resistant systems, and ideally an understanding of these relationships would be present in the mind of any designer working in a seismic area. Unfortunately in the United States methods of education and practice have tended to diminish the opportunity for such understanding to become ingrained, as it should, in the designer's way of thinking, for we separate our architects and engineers in their education and, for the most part, in their practice. In fact, some architects, by intuition and thinking pattern, do have an excellent sense of structure, but they are rare, and such fortunate understanding tends to be in spite of education and practice rather than because of them.

Indeed, our conditions of practice are such that, for all but small structures, it is virtually impossible for the architect also to assume a structural design role. Similarly, a few engineers appear to have an excellent sense of the integration act that is the essence of architecture, but most engineers are content to practice their specialized trade, employed by the architect, with the role of advisor but seldom the authority to ensure that their advice is followed.

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THE SEISMIC SAFETY OF HOSPITALS IN URBAN CENTERS: THE CALIFORNIA EXPERIENCE

by

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ABSTRACT

The earthquake performance of emergency service facilities such as hospitals, ambulance, police and fire stations, and communication centers is of critical importance to immediate post-earthquake recovery efforts. These structures which must remain standing after an earthquake should also continue to function at full capacity following the event and not become a liability during search and rescue activities.

The San Fernando earthquake of February 9, 1971, in which four major hospitals were severely damaged and evacuated, clearly signaled that acute general hospitals located in urban areas of high seismic activity are particularly strategic elements for response to the human consequences of major, damaging earthquakes. In consequence the California Hospital Act of 1972 was enacted into law using the Field Act as a guide. The intent of the legislation is that all new hospitals shall be designed and constructed to be completely functional to perform all necessary services to the public after a disaster.

The basic difference between the Hospital Act of 1972 and previously established performance standards is the requirement that the building remain functional in contrast to earlier provisions requiring only the structure to survive the earthquake. The intent, therefore, is that not only the structural system, but also the architectural, mechanical, and electrical components of a hospital building must maintain their integrity. Accordingly, this paper places emphasis on the architectural, mechanical, and electrical systems of a hospital building.

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INTRODUCTION

General:

Historic records indicate that over 28 major, damaging earthquakes have occurred in California between 1812 and 1971 which resulted in either significant life loss, injury, or property damage, or combinations thereof. These do not include the many other smaller earthquakes which occurred in California over the years not associated with significant property damage.

TABLE 1

PROPERTY DAMAGE AND LIFE LOSS IN MAJOR CALIFORNIA EARTHQUAKES

Year	Location	Damage*	Deaths
1812	San Juan Capistrano	(NA)**	40
1868	Hayward	0.4	30
1872	Owens Valley	0.3	27
1906	San Francisco	24.0***	700
1915	Imperial Valley	0.9	6
1925	Santa Barbara	8.0	13
1933	Long Beach	40.0	115
1940	Imperial Valley	6.0	9
1952	Kern County	60.0	14
1954	Eureka-Arcata	2.1	1
1971	San Fernando	553.0	65

Notes: * In millions of US dollars
** Not available
*** Not including fire loss recorded at 500.0

Source: "Disaster Preparedness", Report to the Congress, Office of Emergency Preparedness, Executive Office of the President, Washington, D.C., January 1972.

Of all these major earthquakes, three greatly affected public policy relative to fire, building, health and safety codes which govern the design and construction of schools, hospitals, and other public facilities:

1. The 1906 San Francisco Earthquake Mag. = 8.3
2. The 1933 Long Beach Earthquake Mag. = 6.3
3. The 1971 San Fernando Earthquake Mag. = 6.5

The 1933 Long Beach earthquake, in which schools and public buildings performed poorly, resulted in legislation passed in 1933 providing for the seismic safety of schools through enactment of the Field Act, while a corresponding Riley Act related to the safety of public buildings. The purpose of the Field Act was to ensure that future public school buildings would be designed and constructed with sufficient earthquake resistance to protect occupants from death or injury. Since the law's passage, almost all schools built in California under Field Act standards have performed well in earthquakes. The San Fernando earthquake in which hospital buildings were damaged badly resulted in the passage of the Hospital Act of 1972.

Importance of the 1971 San Fernando Earthquake to Architects and Planners:

Study of the San Fernando earthquake is important to architects and planners because it represents the first event of significant size since the 1933 Long Beach earthquake to occur in a modern urban area located in the Los Angeles basin which had a current population of approximately 5,000,000. Though classified as a "moderate earthquake" because of its Richter Magnitude of 6.5, it none-the-less caused severe damage locally with 65 deaths and damage estimated to exceed $500,000,000, see Table 1. Important information was collected from this earthquake because of the effects of severe ground motion on contemporary reinforced concrete frame buildings, highway bridges, and communication facilities, in addition to the shaking of highrise buildings, up to 52 stories in height, in the central business district of Los Angeles City in which approximately 90% of the elevators were damaged and rendered inoperable.

After the San Fernando earthquake it became readily evident that the seismic performance of critical use facilities such as hospitals, ambulance services, police and fire stations, communication centers and other life line systems is of strategic importance to immediate post-earthquake recovery efforts. Services provided by these facilities in the urban environment are critical in meeting essential emergency needs generated by the earthquake's impact. Without them, the primary function of fire, search and rescue teams would be useless. These structures which must remain standing after an earthquake should also continue to function at full capacity after the event and not become a liability to the community during the emergency post-earthquake recovery period.

The same lesson was brought into sharp focus again immediately following the 1980 earthquake in southern Italy which occurred in the Regions of Campania and Basilicata. The destruction and loss of hospital facilities, communication services, electrical utility systems, highways, and other critical, emergency use functions made it impossible for rescue teams to operate effectively and efficiently at full capacity. The high losses which occurred, approximately 3,000 deaths, 8,000 injured, and over 200,000 homeless, were due in a major part to the damage and functional failure of these critical facilities. Injured buried in the rubble of damaged buildings were still being rescued four days after the earthquake and 33 people died in the collapse of a hospital wing, all adding to the confusion following the event.

For urban transportation planners, the importance of highway bridges and overpasses was also clearly evident after the 1971 San Fernando earthquake. The epicenter of the earthquake was located very close to four metropolitan freeways. It was the first time that modern freeway overpasses were tested as critical links to a highway transportation system feeding a large urban area. Approximately 62 bridges incurred damage varying from minor cracking to total collapse. Circulation by freeway to and from major hospitals located in the San Fernando Valley was critically curtailed when four arteries were closed to traffic. Alternate routes to metropolitan Los Angeles had to be found.

Even if none of the major hospitals in the San Fernando area had incurred damage, their effectiveness as treatment centers for the severely injured would have been diminished due to the strains placed on surface transportation caused by damaged and collapsed freeway bridges. As it was, the combination of severely damaged hospitals and freeways closed to all vehicular traffic, including ambulance service, made emergency health care most difficult, if not impossible, in the San Fernando Valley. Accordingly, after the earthquake, it was determined that accessibility to hospitals is just as important as the structural integrity of the hospital facility itself.

In the emergency post-earthquake recovery period following the 1980 earthquake in southern Italy, a similar combination of events associated with collapsed hospitals and damaged roadbeds forced rescue teams to use helicopters as the only means available to evacuate the severely injured. Even then, the effectiveness of helicopters was significantly reduced by dense fog and heavy winter snows which seriously diminished visibility in the hill town areas.

For planning purposes in post-earthquake recovery efforts, it is therefore critical to weigh physical conditions of: (a) surface routes which affect accessibility to and from critical facilities, (b) hospital facilities, and (c) weather characteristics in the area before proceeding with any evacuation strategies for the injured. In the siting and design of major hospital facilities in urban areas of high seismic activity, it becomes important to keep such considerations in mind.

Performance of Hospital and Medical Facilities in the San Fernando Earthquake:

After the 1971 San Fernando earthquake, hospital and medical facility construction generated considerable public concern in California regarding their potential of collapse in earthquakes for justifiable reason: the earthquake was particularly destructive to health care facilities located in the area. In the U.S. Department of Commerce publication "The San Fernando, California, Earthquake of February 9, 1971", Volume 1, Part A, Kesler writes as follows:

> Most of the major structures in the heavily shaken area were medical facilities. Four major hospitals (Olive View, Veterans Administration, Holy Cross, and Pacoima Memorial Lutheran) were located within a radius of 9 miles of the epicenter. At the Veterans Administration Hospital, some of the buildings that were built prior to 1933 collapsed. The other three hospitals, which were built within the last 12 years with earthquake resistant features, all suffered significant damage resulting in evacuation. There were in addition, three medical office buildings (Foothill Medical Center, Pacoima Luthern Center, and Indian Hills Medical Center), two psychiatric units (Golden State and Olive View), and one mechanical equipment building (Olive View). All, except Golden State, were damaged significantly. (Page 175).

In the aftermath of the San Fernando earthquake, it was most clear that the health care facilities are a building type especially important to the recovery of urban environments. In the same publication referenced above, Kesler continues:

> Not only are patients incapacitated in many cases and unable to take, perhaps, even simple precautions to protect themselves, let alone safely endure an interuption in care, but also medical facilities are needed urgently in the hours following widespread destruction and injury due to an earthquake. At the time of disaster, these installations must be functional, rather than being among the casualties.
>
> Hospitals, compared with other types of buildings, are special in that they, in general, will contain unusually large quantity of specialized, expensive, and delicate equipment and materials. Often such items, if damaged, cannot be replaced quickly or inexpensively. Indeed, hospitals themselves are neither financed easily nor quickly built in the first place. Repair or replacement required by earthquake damage has placed a severe financial strain on some if not all of the institutions.
>
> Serious consideration should be given to providing increased levels of safety for these important and expensive facilities. (Page 295).

CALIFORNIA HOSPITAL ACT OF 1972

The San Fernando earthquake, in which the four major hospitals indicated above were severely damaged and evacuated, clearly signaled the fact that acute general hospitals located in urban areas of high seismic activity are particularly strategic elements for response to the human consequences which follow a major, damaging seismic event. In contrast, public school buildings generally performed well, experiencing minor to moderate damage. This was attributed to California's Field Act regulations, which include, among other requirements, stricter seismic performance standards, structural plan review by the Office of the State Architect, and rigorous field inspection during construction. Public reaction to the limited seismic performance of hospitals resulted in legislation which developed and passed "The Hospital Safety Act of 1972", a direct product of California State Senate Bill 519. The Act became effective in March 1973. A copy of Senate Bill 519 as proposed in 1972 is included in Appendix A.

In 1976, specific amendments relative to the intent of the Hospital Act were passed to clarify the basic definition of a "hospital building" through California Assembly Bill 1843. (See Appendix B). The amendment stipulates that by definition, a "hospital building" shall not include any of the following:

(1) Any building in which only outpatient services are provided and which is not physically attached to a building in which inpatient services are provided.

(2) Any building used, or designed to be used, for skilled nursing facility or intermediate care facility if such building is of single-story, wood frame construction.
(3) Any buidling of single-story, wood frame construction in which only skilled nursing or intermediate care services are provided if such building is not physically attached to a building housing other patients of the health facility receiving higher levels of care.

In passing the amendment, the California Legislature recognized the relative safety of single-story, wood frame construction for use in housing patients requiring skilled nursing and therefore decided to provide for reasonable flexibility in the seismic standards for such structures. In addition, changes made by the amendment to the original Hospital Act of 1972 acknowledge diverse levels of exposure to risk between outpatients, who are ambulatory, and inpatients, who are generally incapacitated.

General Ojectives of California Hospital Act:

In passing the Hospital Act, it was the intention of the California Legislation that new hospitals, "which house patients having less than the capacity of normally healthy persons to protect themselves, and which must be completely functional to perform all necessary services to the public after a disaster, shall be designed and constructed to resist, insofar as practicable, the forces generated by earthquakes, gravity, and winds." The key words in the Senate Bill are "which must be completely functional to perform all necessary services to the public after a disaster". The basic difference, therefore, between the Hospital Act and other public health and safety acts, such as the Field Act and the Riley Act among others, is the requirement that the building remain functional in contrast to earlier provisions requiring that only the structure survive an earthquake without injury to occupants. The result, therefore, is that not only the structural system, but also the architectural, mechanical, and electrical components of a new hospital building must maintain their integrity. Since this is a new consideration to be taken into account in the design and construction of hospitals, this paper will focus on the implications relating to the architectural, mechanical, and electrical systems of health care facilities.

Title 17 - Safety of Construction of Hospitals:

The intent of the Hospital Act of 1972 is translated into actions for implementation through Title 17, Public Health and Safety Code, of the California Administrative Code which includes provisions for application of the rules and regulations prescribed. It includes requirements for damage control in which building elements, systems and equipment necessary for the complete functioning of buildings are to be designed, detailed and constructed to withstand the maximum acceleration and deflections of the basic structure without excessive displacement or damage which will disrupt essential operations and services to be performed. To meet this objective, deflection under lateral forces are to be established by a dynamic analysis or assumed to be two times the static deflection computed for the prescribed seismic or wind forces, whichever governs.

The regulations established under Title 17 are the basis for design, checking, and approval of plans and specifications for all construction or alteration of hospital buildings in the State of California. It is made clear that the application of the rules and regulations is not intended to limit the ingenuity of the designer, nor to prevent designing to a higher standard. All plans and specifications are to be prepared under the responsible charge of an architect or a structural engineer, or both. Structural, mechanical and electrical drawings and specifications must be prepared and signed by engineers registered in these respective disciplines. Monitoring of the construction work must be under the direct responsibility of the architect or professional engineer who prepared and signed the doucment for that particular work. State review of the documents for final approval prior to the start of construction is completed by the Office of Architecture and Construction, and the State Fire Marshall.

Submission of design and construction documents for prior approval by the State are made in three stages:

(1) Site Data
(2) Preliminary Plans and Outline Specifications
(3) Working Drawings and Final Specifications.

Site data information includes a Geologic and Earthquake Engineering Report which contains scientific data on an assessment of the physical characteristics of the site including the potential of earthquake damage based on geologic, foundation, and earthquake engineering investigations. The geologic investigation includes an evaluation of known active and potentially active faults, both local and regional. It also includes an assessment of slope stability and liquefaction potential of the site.

Preliminary drawings and outline specifications of the proposed buildings comprise the second submittal required for review by the Department of General Services through its Office of Architecture and Construction as well as the Office of the State Fire Marshall. The preliminary drawings are to include typical architectural and structural engineering drawings of site development plans, floor plans, exterior elevations, and location of fixed and major movable equipment. The outline specifications are to include a general description of the construction, exterior and interior finish materials, and the types of mechanical and electrical systems to be used.

The final submittal of Working Drawings, Final Specifications, and Reports for review comprises all final:

Geotechnical Reports	Architectural Drawings
Structural Drawings	Mechanical Drawings
Electrical Drawings	Complete Specifications

Again all documents are reviewed by the Department of General Services through the Office of Architecture and Construction as well as the Office of the State Fire Marshall. Since 1973 a total of $1.9 billion of new health care facility work has been submitted for approval, is currently under construction, or has been built in California.

General Design Requirements:

In developing requirements relating to the continued function of a hospital building during and after an earthquake, particular attention is given to deflection of wall assemblies, the horizontal deflection of vertical structural systems, in the plane of the wall, due to lateral forces are not to exceed 1/16 inch per foot of height of any story. Deflection from the head to sill of a glazed opening, in the plane of the wall, is not to exceed 1/32 inch per foot of height of the opening unless the glass therein is prevented from taking shear or distortion, or tempered, safety or wired glass is used.

In giving consideration to deflection, design solutions are to take into account: (a) secondary stresses induced by the deflection of the structure, or parts thereof, when such deflections create unsafe conditions, (b) deformation compatibility of structural and non-structural elements, and (c) inelastic deformation in any portion of a connection which might create unsafe conditions.

As an example of anchorage requirements, where horizontal diaphragms of wood or other materials of similar flexibility butt into masonry or concrete walls, positive direct connections using bolts, screws or other acceptable connectors, are to be provided between the walls and framing members. These connections are in addition to any other diaphragm to wall connections required for shear transfer.

Deflection capacities of building elements are developed on the capability of the element to resist the effects of earthquakes upon the structure as determined by dynamic analysis which is required for certain structures such as high-rise buildings and those of complex configuration. The analysis is to be based upon the ground motion prescribed for the site in the Geotechnic Report which is required to consider the seismic event that may be postulated with a reasonable confidence level within a 100 year period.

ARCHITECTURAL, MECHANICAL AND ELECTRICAL ENGINEERING DESIGN IMPLICATIONS

The Hospital Act of 1972 and its accompanying Title 17 Safety of Construction of Hospitals requirements have had specific impact on the design of architectural, mechanical and electrical elements when their failure has a direct bearing on the continued function of the building. Each element or system must be examined in detail to determine its role in maintaining the functionality of a health facility.

Architectural Design Considerations:

In the submittal of final working drawings and specifications, in addition to the normal floor and roof plans, elevations and sections, and schedules of finishes, doors and windows, anchorage details of all fixed items must be designed and detailed in consultation with the consulting structural engineer. Location and identifying data on all items of fixed equipment or major movable equipment such as autoclaves, sterilizers, kitchen equipment, laboratory equipment, X-ray equipment, and cubicle curtains must be shown. In the architectural set of drawings, the manner in which all non-structural partitions, window-wall assemblies, and wall opening assemblies are attached, or connected, to other components and systems of the building must be detailed. Figures 1 through 4 indicate typical connection and bracing details for interior partitions.

In the general architectural design of the building, light weight panels and wall assemblies are being used on building exteriors by designers to reduce the overall weight of the building envelope. By using lesser building masses, it is easier to control building deflection and floor to floor movement. The basic structural system of multi-story hospital buildings now tends to be shear wall, or braced frame, construction rather than moment frame. To produce an effective moment frame system in steel needed to minimize overall deflection appears to require too much steel framing to warrant economic consideration in many cases, in particular when dealing with multi-story construction of more than two or three floors. In contrast to health care facilities constructed prior to the Hospital Act of 1972, buildings tend to be less in height compared to the high slab type of construction employed for health care facilities in the past. The typical hospital building in California is being designed currently as a squat four story structure rather than the tall and slender towers which were common five years ago. There are exceptions of course, but generally speaking it is easier to control excessive deflection in a squat building type in contrast to a tall, slender one. One interesting case study illustrates this trend in architectural design relating to the building height economic constraint produced by the Hospital Act very well: a recent new hospital building was originally designed and constructed with the provision for a future addition of two additional floors on top of the original structure prior to the passage of the Hospital Act. Now a few years later under the requirements of Title 17, the additional floors could not be added to the original building as planned and still meet performance standards specified for the safe construction of hospitals.

Mechanical and Electrical Engineering Design Considerations:

In a similar manner the Hospital Act has also affected design considerations for the placement of mechanical and electrical systems and equipment in health care facilities. As the building must remain functional during and after a major earthquake, critical mechanical and electrical components must remain in place and not disrupt the continued use of the building. Under the Title 17 provisions, specific information must be indicated in the construction drawings.

Mechanical drawings must show the complete heating, ventilating, air conditioning and plumbing systems and details showing methods for fastening equipment to the structure to resist seismic forces include:

(a) Radiators and steam-heated equipment, such as sterilizers, autoclaves, warmers and steam tables
(b) Heating and steam mains, including branches with pipe sizes
(c) Sizes, types, and heating surfaces of boilers and furnaces
(d) Pumps, tanks, boiler breeching and piping and boiler room accessories
(e) Air conditioning systems with refrigerators, water and refrigerant piping, and ducts
(f) Exhaust and supply ventilating systems showing duct sizes with steam or water connections and piping
(g) Size and elevation of street sewer, house sewer, house drains, street water main and water service into the building
(h) Location and size of soil, waste and vent stacks with connections to house drains, fixtures and equipment
(i) Size and location of hot, cold and circulation water mains, branches and risers from the service entrance and tanks
(j) Riser diagram or some other acceptable method to show all plumbing stacks with vents, water risers and fixture connections for multistory buildings
(k) Gas, oxygen and special connections
(l) Fire extinguishing equipment--sprinklers; wet and dry standpipes, fire extinguishers
(m) Plumbing fixtures and fixtures which require water and drain connection

Electrical drawings which show the complete electrical system and details showing methods for fastening equipment to the structure to resist seismic forces include:

(a) Electrical service entrance with service switches, service feeds to the public service feeders, and characteristics of the light and power currents
(b) Transformers and their connections, if located in the building or on the site
(c) Plan and diagram showing main switchboard, power panels, light panels and equipment
(d) Feeder and conduit sizes with schedule of feeder breakers or switches
(e) Light outlets, receptacles, switches, power outlets and circuits, isolated electrical system
(f) Telephone layout
(g) Nurses' call system with outlets for beds, duty stations, door signal lights and annunciators
(h) Fire alarm systems with stations, sounding devices and control boards
(i) Emergency lighting system when required with outlets, transfer switch, source of supply, feeders and circuits.

Figures 5 through 6 indicate some typical details for bracing ducts, component connections to concrete members, and restraining devices for lateral and vertical loads recommended by the Sheet Metal and Air Conditioning Contractors National Association (SMACNA) for use in California. These examples were abstracted from the SMACNA publication entitled "Guidelines for Seismic Restraints of Mechanical Systems" published in 1976.

Economic Implications:

In May 1977 the California Seismic Safety Commission issued a report of a Task Committee on the Hospital Act of 1972. In Chapter VII of the report the Committee presented its findings on hospital construction costs resulting from the passage of the legislation affecting the safety of construction of health care facilities. The report indicates that

> "Based on studies by the Building Safety Board, it has been estimated that the average cost for compliance with the earthquake force requirements of the Hospital Act regulations is about a 25 percent increase in the structural components of a hospital project. The structural portions of a project account for about 12 to 15 percent of the project costs for most new construction. About 20 percent of project costs are for structural portions of alterations and additions, which make up a large portion of the hospital projects submitted to the Department of Health for approval. Therefore, the total construction cost increase attributable to structural items ranges from 3 to 5 percent. The estimated increase in costs for the mechanical and electrical portions of a hospital project, such as air conditioning ducts, piping, and electrical installations, etc., brought on by the regulations is approximately 15 percent. The mechanical and electrical items constitute about 35 percent of the entire project costs, resulting in a project cost increase of about 5 percent. Thus, the total project cost increase brought on by the new regulations is approximately 8 to 10 percent. These cost increases are in addition to the costs imposed by the 0.7 percent project review fees, inspection costs, geologic study costs and costs of project delays (discussed previously in this report)".

Areas of Additional Study:

Since passage of the Hospital Act in 1972, a severe earthquake has not occurred in a major urban area in California. Hospitals designed under the new health and safety code provisions have not been tested by an actual event as severe as the 1971 San Fernando earthquake.

Current research includes a project to study and quantify incidences of critical damage sustained by health care facilities in past earthquakes to identify and document actual failures in architectural, mechanical, and electrical equipment and systems which rendered hospitals inoperable or critically affected their optimum operation. Once this study is completed, large-scale, or full size, laboratory testing of components and systems identified as critical elements in the continued function of a hospital is recommended.

SUMMARY

The Hospital Act of 1972 has effectively changed approaches in the design and construction of health care facilities in the State of California. The intent of the legislative act is that the hospital building remain functional to perform all necessary services to the public after a disaster. The provisions in Title 17, Safety of the Construction of Hospitals, has had significant impact on architectural, mechanical and electrical engineering design standards for the seismic restraint and bracing of non-structural partitions, building contents, equipment and systems in hospitals. The overall, architectural building envelope has changed in general terms to a squat four story structure in contrast to the tall, slender multistory facilities designed in the past. The adoption of exterior light-weight cladding materials is a typical method used to reduce the overall weight of the building mass. The total increase of a project cost brought on by the new regulations is approximately 8 to 10 percent.

Since passage of the Hospital Act, California hospital buildings designed under the new provisions habe not been thoroughly tested by a major, damaging earthquake as had occurred during the 1971 San Fernando earthquake. Full scale laboratory testing of architectural, mechanical, and electrical components and assemblies critical to the continued function of a hospital is recommended.

ACKNOWLEDGEMENTS

Special acknowledgement is extended to Peter Stromberg, Seismic Safety Specialist of the California Seismic Safety Commission; Robert Barnecutt, Associate Director of Stone Marraccini and Patterson, Architects/Planners/Health Planning Consultants, San Francisco; and Milton Leong, Principal, Leong-Razzano & Associates, Inc., Consulting Engineers, Berkeley; who provided significant data and technical reports for this paper. Appreciation is given to the Sheet Metal Industry Fund of Los Angeles for use of representative bracing and connection details taken form the SMACNA publication on Guidelines for Seismic Restraints of Mechanical Systems.

Although considerable data were made available for this paper from many sources and technical documents, any material presented and conclusions drawn are based on personal interpretation and judgement by the authors and remain their responsibility.

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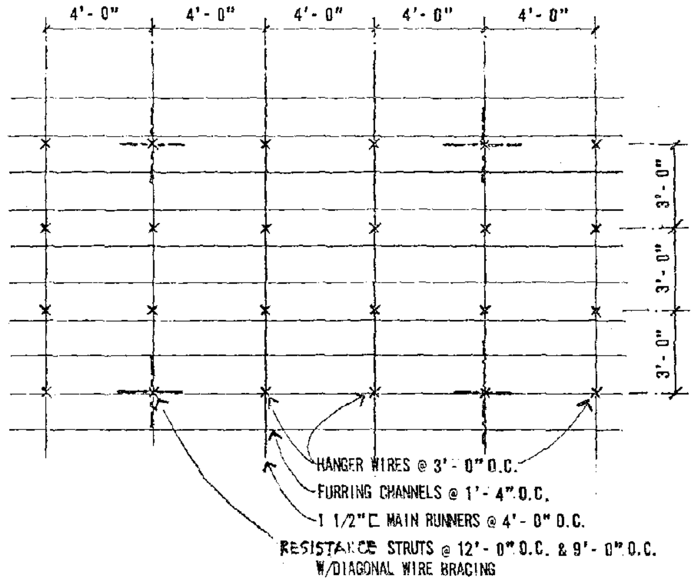
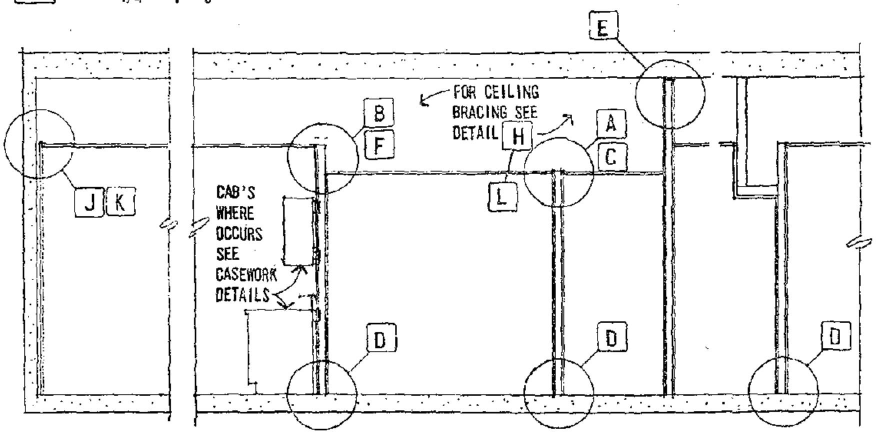
<u>Figure 1</u>

<u>Interior Partitions & Ceiling Framing</u>



DIAGRAMATIC CEILING FRAMING PLAN
1/4" = 1'- 0"

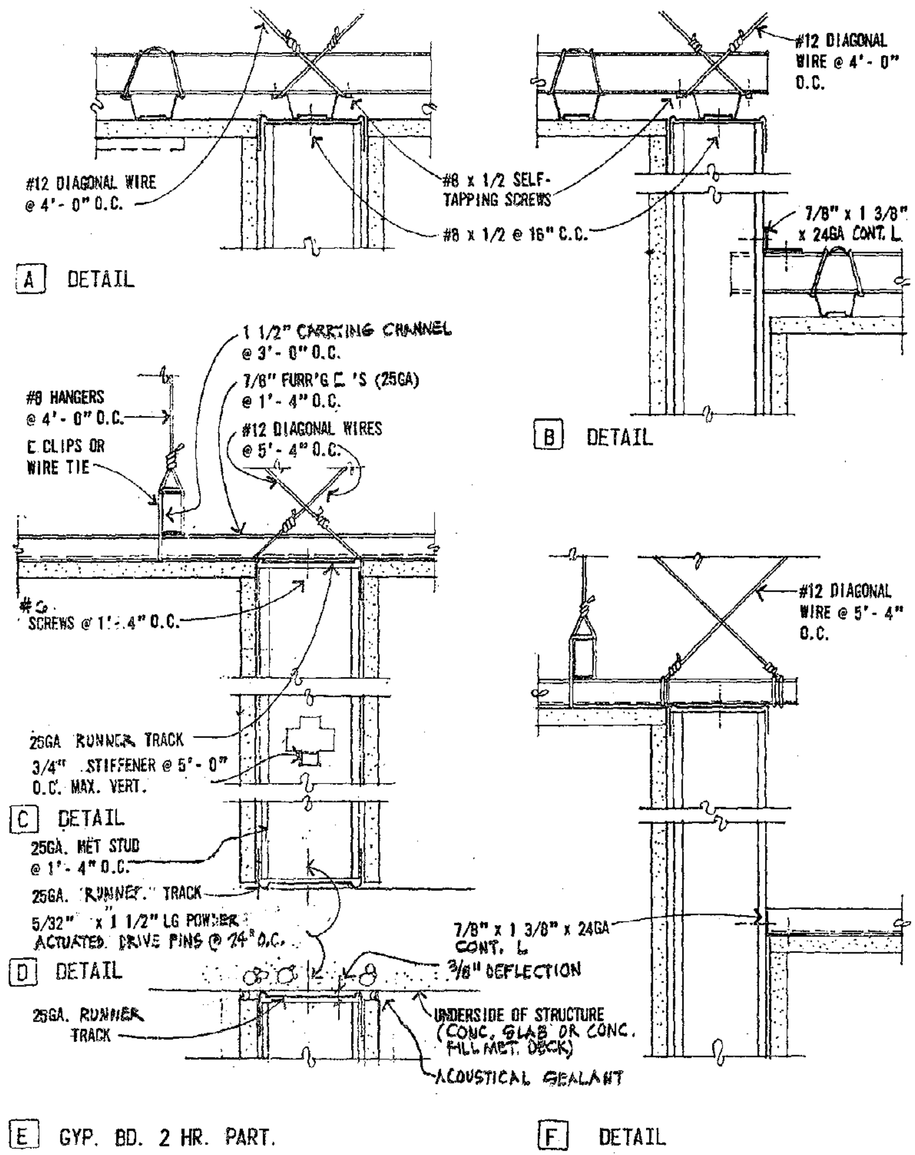


DIAGRAMATIC - SECTION OF INTERIOR PARTITIONS
1/4" = 1'- 0"

Source: Stone Marraccini and Patterson
Architects/Planners/Health Planning Consultants
San Francisco, California. 1981

Figure 2

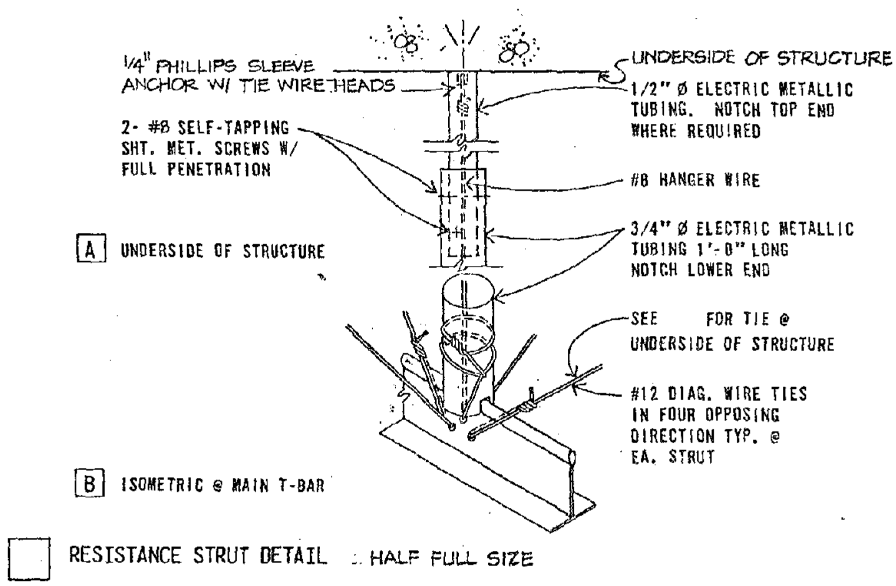
Partition Connections

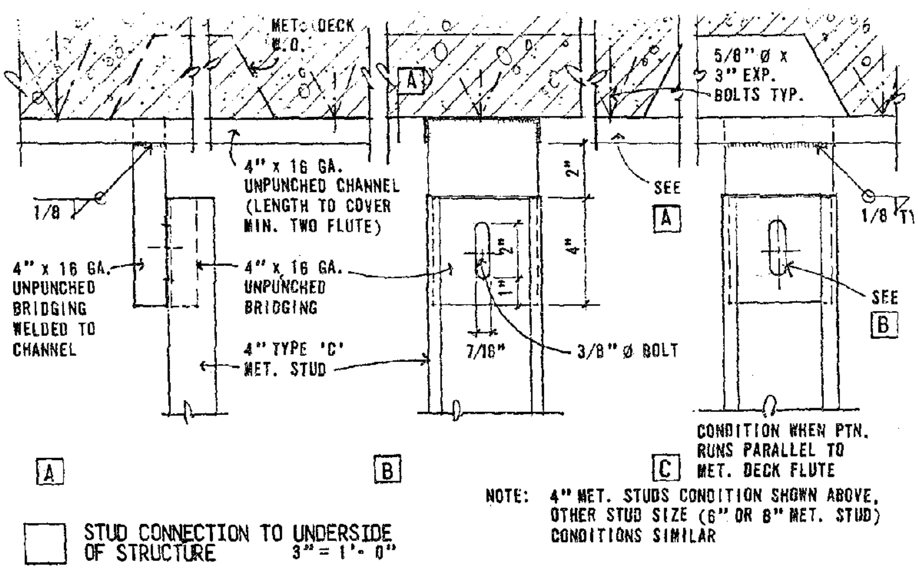


Source: Stone Marraccini and Patterson
Architects/Planners/Health Planning Consultants
San Francisco, California, 1981

Figure 3

Representative Connections

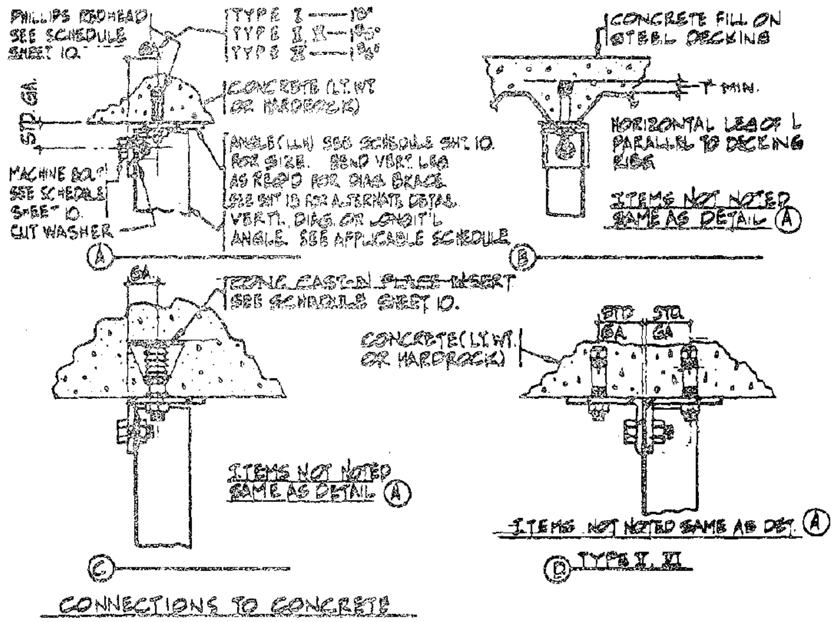


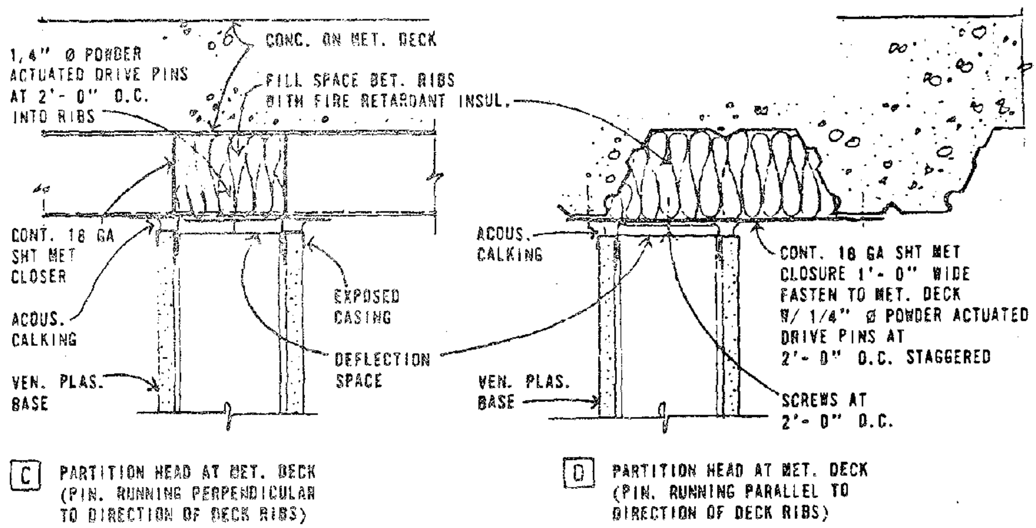


Source: Stone Marraccini and Patterson
Architects/Planners/Health Planning Consultants
San Francisco, California. 1981

Figure 4

Typical Connections

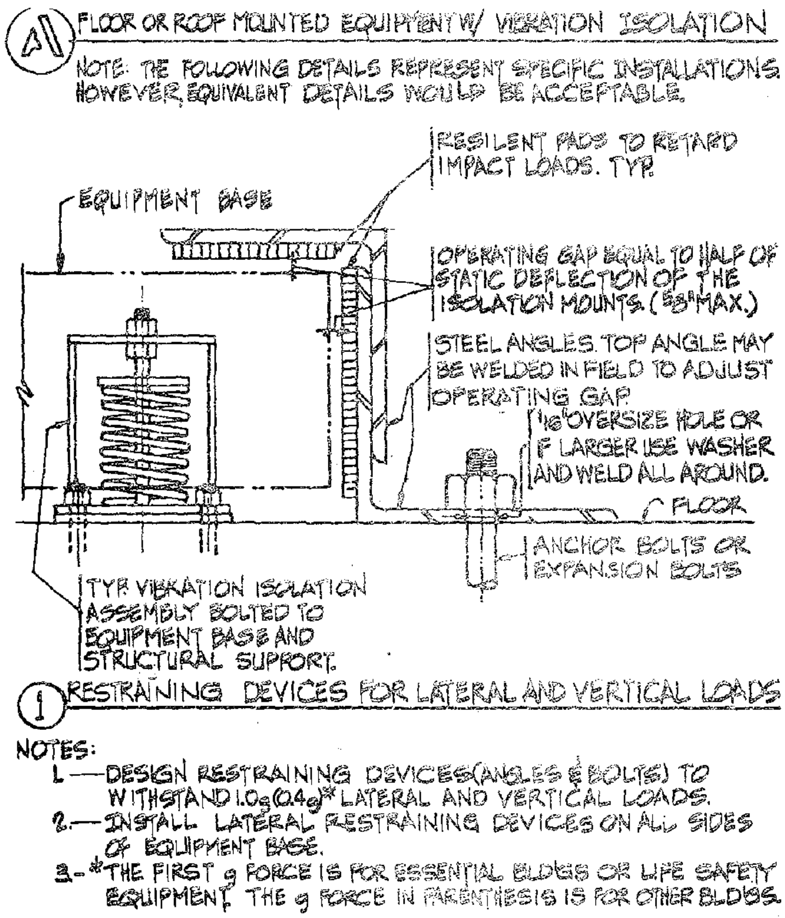




Source: Stone Marraccini and Patterson
Architects/Planners/Health Planning Consultants
San Francisco, California, 1981

Figure 5

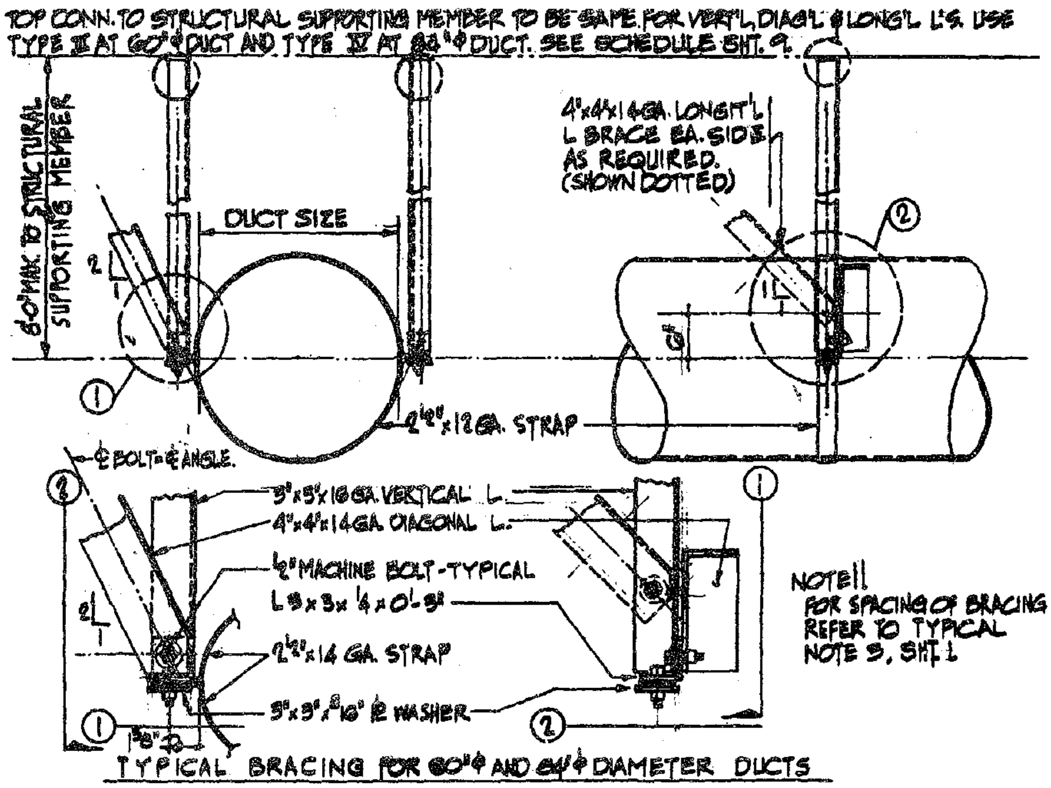
Restraining Devise for Lateral Load



Source: "Guidelines for Seismic Restraint of Mechanical Systems", 1976. Sheet Metal and Air Conditioning Contractors National Association (SMACNA). Los Angeles, California.

Figure 6

Typical Duct Bracing Detail



Source: "Guidelines for Seismic Restraint of Mechanical Systems", 1976. Sheet Metal and Air Conditioning Contractors National Association (SMACNA). Los Angeles, California.

Senate Bill No. 519

CHAPTER 1130

An act to add Division 12.5 (commencing with Section 15000) to the Health and Safety Code, relating to hospitals, and making an appropriation therefor.

[Approved by Governor November 21, 1972. Filed with Secretary of State November 21, 1972.]

LEGISLATIVE COUNSEL'S DIGEST

SB 519, Alquist. Hospital construction.

Requires the State Department of Public Health, through a contract with the Department of General Services, to (1) observe the construction of or addition to any hospital building or, if the work alters structural elements, the reconstruction or alteration of any hospital building, as it deems necessary for the protection of life and property; and (2) pass upon and approve or reject all plans for the construction or the alteration of any hospital building, independently reviewing the design and geological data to assure compliance with the requirements of the act. Requires that geological data be reviewed by an engineering geologist and structural design data be reviewed by a structural engineer.

Requires the governing board of each hospital or other hospital governing authority, before adopting any plans for such hospital building, to submit the plans to the State Department of Public Health for approval and to pay prescribed fees, specifies what must accompany the application for approval, and prescribes requirements for plans and specifications.

Creates a Hospital Building Account in the Architecture Public Building Fund, requires that all fees collected pursuant to the act be credited to such account, and continuously appropriates money in such account, without regard to fiscal years, for the use of the State Department of Public Health, subject to approval of the Department of Finance, in carrying out the provisions of the act.

Declares that no contract for the construction or alteration of any hospital building made or executed on or after the effective date of the act is valid, and prohibits payment of any money for work done under such a contract, or for any labor or materials furnished in constructing or altering any such building, unless prescribed requirements are satisfied.

Prescribes requirements re administration of the work of construction, inspection of hospital buildings and of the work of construction or alteration, and reports concerning the work of construction or alteration.

Authorizes the State Department of Public Health to call upon the Department of General Services to make a periodic review of hospital operations to assure that the hospital is adequately prepared to resist

damage caused by earthquake tremor and prescribes requirements re such review.

Authorizes the State Department of Public Health to make regulations to carry out the act.

Requires the Director of Public Health to appoint a Building Safety Board to advise and act as a board of appeals in all matters affecting seismic safety in the administration and enforcement of the act.

Declares intent of the Legislature to preempt from local jurisdictions the enforcement of building regulations adopted pursuant to this act, including plan checking, and intent of the Legislature that where local jurisdictions have more restrictive standards for enforcement of building regulations and construction supervision, such standards shall be enforced by the state.

Prescribes penalty for violations.

Defines "hospital building," "construction or alteration," "architect," "structural engineer," and "engineer geologist."

The people of the State of California do enact as follows:

SECTION 1. The Legislature finds and declares as follows:

(a) California is situated on the rim of the great Circum-Pacific seismic belt and it is inevitable that strong seismic disturbances along this belt will cause extensive property damage and endanger the lives of all people who enter or are near buildings which may collapse or be seriously damaged by such seismic disturbances.

(b) It is reasonable to expect that any building located anywhere within California will be subjected to the forces generated by a strong earthquake at least once during its life.

(c) Following the 1933 Long Beach earthquake, the Legislature enacted the so-called "Field Act" (Sections 15451 to 15466, inclusive, Education Code) as an urgency measure, which established reasonable minimum standards and procedures for the design and construction of new public school buildings. The durability during subsequent earthquakes of school buildings designed and constructed under the provisions of those statutes, when compared with the durability during the same earthquakes of other buildings not designed and constructed pursuant to the "Field Act," has repeatedly illustrated the prudence of such legislation.

(d) The San Fernando Valley earthquake of February 9, 1971, although moderate in terms of total energy release, resulted in such total collapse or damage as made many hospital buildings inoperable. Some of these damaged or destroyed hospital buildings were relatively new structures, designed and constructed to meet the standards as prescribed by most local jurisdictions throughout the State of California.

SEC. 2. It is the intent of the Legislature that hospitals, which house patients having less than the capacity of normally healthy

Appendix B

Assembly Bill No. 1843

CHAPTER 177

An act to amend Section 15001 of, and to add Section 15001.5 to, the Health and Safety Code, relating to seismic structural safety requirements for hospital buildings, and declaring the urgency thereof, to take effect immediately.

[Approved by Governor May 26, 1976. Filed with Secretary of State May 27, 1976.]

LEGISLATIVE COUNSEL'S DIGEST

AB 1843, Cullen. Hospital buildings: seismic safety requirements.

Under existing law, seismic structural safety requirements are prescribed for hospital buildings. The term "hospital building" is defined, for such purposes, as including (with certain exceptions) all licensed health facilities, which include general acute care hospitals, acute psychiatric hospitals, skilled nursing facilities, intermediate care facilities, and special hospitals.

This bill would redefine the term "hospital building," for the purposes of such seismic structural safety provisions, to include only licensed health facilities and to exclude single-story, wood frame buildings used, or designed to be used, for skilled nursing facilities or intermediate care facilities, and any single-story, wood frame building in which only skilled nursing or intermediate care services are provided if such building is not physically attached to a building housing other patients of the health facility receiving higher levels of care. However, a structural engineer would be required to submit a declaration to the State Department of Health that the design and construction of new buildings so excluded comply with prescribed requirements.

The bill would take effect immediately as an urgency statute.

The people of the State of California do enact as follows:

SECTION 1. Section 15001 of the Health and Safety Code is amended to read:

15001. (a) "Hospital building," as used in this chapter, shall include any building not specified in subdivision (b) which is used, or designed to be used, for a health facility of a type required to be licensed pursuant to Chapter 2 (commencing with Section 1250) of Division 2.

(b) "Hospital building" shall not include any of the following:

(1) Any building in which only outpatient services are provided and which is not physically attached to a building in which inpatient services are provided.

(2) Any building used, or designed to be used, for a skilled nursing

facility or intermediate care facility if such building is of single-story, wood frame construction.

(3) Any building of single-story, wood frame construction in which only skilled nursing or intermediate care services are provided if such building is not physically attached to a building housing other patients of the health facility receiving higher levels of care.

SEC. 2. Section 15001.5 is added to the Health and Safety Code, to read:

15001.5. New construction of buildings specified in paragraphs (2) and (3) of subdivision (b) of Section 15001 shall conform to the provisions of the latest edition of the Uniform Building Code of the International Conference of Building Officials. A structural engineer shall submit a declaration that, in his opinion and to the best of his knowledge, the design and construction of such buildings comply with such standards. The declaration of the structural engineer shall be transmitted to the state department and shall additionally include an assessment of the nature of the site and the potential for earthquake damage based upon engineering investigation by competent personnel.

For the purposes of Section 1265.8, the state department shall be deemed to have approved the construction of any building subject to the requirements of this section for which a declaration meeting such requirements has been received by the department.

The Legislature recognizes the relative safety of single-story, wood frame construction for use in housing patients requiring skilled nursing and intermediate care services and it is, therefore, the intent of the Legislature to provide for reasonable flexibility in seismic safety standards for such structures.

SEC. 3. This act is an urgency statute necessary for the immediate preservation of the public peace, health or safety within the meaning of Article IV of the Constitution and shall go into immediate effect. The facts constituting such necessity are:

In order to revise the term "hospital building" in the law relating to seismic safety requirements at the earliest opportunity, it is necessary that this act take immediate effect.

persons to protect themselves, and which must be completely functional to perform all necessary services to the public after a disaster, shall be designed and constructed to resist, insofar as practicable, the forces generated by earthquakes, gravity, and winds. In order to accomplish this purpose the Legislature intends to establish proper building standards for earthquake resistance based upon current knowledge, and intends that procedures for the design and construction of hospitals be subjected to independent review. It is further the intent of the Legislature that Division 12.5 (commencing with Section 15000) of the Health and Safety Code shall be administered by the State Department of Public Health, which shall contract for enforcement of such provisions with the Department of General Services which now successfully enforces the provisions of the "Field Act."

SEC. 3. Division 12.5 (commencing with Section 15000) is added to the Health and Safety Code, to read:

DIVISION 12.5. BUILDINGS USED BY THE PUBLIC

CHAPTER 1. HOSPITALS

15000. It is the intent of the Legislature that the Department of General Services shall analyze the structural systems and details, as set forth in the working drawings and specifications, and observe the construction of hospital projects and report the findings of such analysis to the state department. It is further the intent of the Legislature to preempt from local jurisdictions the enforcement of building regulations adopted pursuant to this chapter including the plan checking. It is further the intent of the Legislature that where local jurisdictions have more restrictive standards for the enforcement of building regulations and construction supervision, such standards shall be enforced by the state.

15001. "Hospital building," as used in this chapter, means and includes any building used, or designed to be used, for a hospital and shall include all of the following:

(a) All hospitals of a type required to be licensed pursuant to Chapter 2 (commencing with Section 1400) of Division 2 and facilities of a type required to be licensed pursuant to Chapter 1 (commencing with Section 7000) of Division 7 of the Welfare and Institutions Code.

(b) Institutions conducted, maintained, or operated by this state or any state department, authority, district, bureau, commission, or officer or by the Regents of the University of California, or by a board of supervisors of a county under the provisions of Chapter 2.5 (commencing with Section 1440) of Division 2, which, except for the exemption provided by Section 1415, would be encompassed by the terms of subdivision (a).

15002. "Construction or alteration," as used in this chapter, includes any construction, reconstruction, or alteration of, or addition to, any hospital building.

15003. "Architect," as used in this chapter, means a person who is certified and holds a valid license under Chapter 3 (commencing with Section 5500) of Division 3 of the Business and Professions Code.

15004. "Structural engineer," as used in this chapter, means a person who is validly certified to use the title structural engineer under Chapter 7 (commencing with Section 6700) of Division 3 of the Business and Professions Code.

15005. "Engineering geologist," as used in this chapter, means a person who is validly certified under Chapter 12.5 (commencing with Section 7800) of Division 3 of the Business and Professions Code.

15006. The state department, through its contract with the Department of General Services, shall observe the construction of, or addition to, any hospital building or, if the work alters structural elements, the reconstruction or alteration of any hospital building, as it deems necessary to comply with the provisions of this chapter for the protection of life and property.

15007. The state department, through its contract with the Department of General Services, shall pass upon and approve or reject all plans for the construction or the alteration of any hospital building, independently reviewing the design and geological data to assure compliance with requirements of this chapter. Geological data shall be reviewed by an engineering geologist and structural design data shall be reviewed by a structural engineer. The governing board of each hospital or other hospital governing authority, before adopting any plans for such hospital building, shall submit the plans to the state department for approval and shall pay the fees prescribed in this chapter.

15008. In each case, the application for approval of the plans shall be accompanied by the plans and full, complete, and accurate specifications, and structural design computations, and the specified fee, which shall comply with requirements prescribed by the state department.

15009. Plans submitted pursuant to this chapter for work which affects structural elements shall contain an assessment of the nature of the site and potential for earthquake damage, based upon geologic and engineering investigations by competent personnel of the causes of earthquake damage. One-story Type V construction of 4,000 square feet or less shall be exempt from the provisions of this section.

15010. The engineering investigation shall be correlated with the geologic evaluation made pursuant to Section 15009.

15011. The application shall be accompanied by a filing fee in an amount which the state department determines will cover the costs of administering this chapter. Such fee shall be based on a uniform percentage of the estimated construction cost, and shall not exceed 0.7 percent of the estimated construction cost.

The minimum fee in any case shall be one hundred dollars ($100). If the actual construction cost exceeds the estimated construction cost by more than 5 percent, a further fee shall be paid to the state department, based on the above schedule and computed on the amount by which the actual cost exceeds the amount of the estimated cost.

15012. All fees shall be paid into the State Treasury and credited to the Hospital Building Account, which is hereby created in the Architecture Public Building Fund, and are continuously appropriated without regard to fiscal years for the use of the state department, subject to approval of the Department of Finance, in carrying out the provisions of this chapter. Adjustments in the amounts of the fees, as determined by the state department and approved by the Department of Finance, shall be made within the limits set in Section 15011 in order to maintain a reasonable working balance in the account.

15013. All plans and specifications shall be prepared under the responsible charge of an architect or a structural engineer, or both. A structural engineer shall prepare the structural design and shall sign plans and specifications related thereto. Administration of the work of construction shall be under the responsible charge of such architect and structural engineer, except that where plans and specifications for alterations or repairs do not affect architectural or structural conditions, such plans and specifications may be prepared and work of construction may be administered by a professional engineer duly qualified to perform such services and holding a valid certificate under Chapter 7 (commencing with Section 6700) of Division 3 of the Business and Professions Code for performance of services in that branch of engineering in which said plans, specifications, and estimates and work of construction are applicable.

15014. Before letting any contract for any construction or alteration of any hospital building, the written approval of the plans as to safety of design and construction, by the state department, through its contract with the Department of General Services, shall be first had and obtained.

15015. No contract for the construction or alteration of any hospital building, made or executed on or after the effective date of this chapter by the governing board or authority of any hospital or other similar public board, body, or officer otherwise vested with authority to make or execute such a contract, is valid, and no money shall be paid for any work done under such a contract or for any labor or materials furnished in constructing or altering any such building, (1) unless the plans and specifications comply with the provisions of this chapter and the requirements prescribed by the state department, and (2) the approval thereof in writing has first been had and obtained from the state department, through its contract with the Department of General Services, and (3) the hospital building is to be accessible to,

and usable by, the physically handicapped, and (4) the plans and specifications comply with the fire and panic safety requirements of the State Fire Marshal.

15016. The state department, through its contract with the Department of General Services, shall make such inspection of the hospital buildings and of the work of construction or alteration as in its judgment is necessary or proper for the enforcement of this chapter and the protection of the safety of the public. The hospital governing board or authority shall provide for and require competent, adequate, and continuous inspection during construction or alteration by an inspector satisfactory to the architect or structural engineer, or both, and the state department. The inspector shall act under the direction of the architect or structural engineer, or both, and be responsible to the board or authority. Notwithstanding any other provision of this section, where alterations or repairs are to be conducted under the supervision of a professional engineer pursuant to Section 15013, the inspector need only be satisfactory to the state department and to the professional engineer, and the inspector shall act under the direction of the professional engineer. In approving any inspector, the state department shall consult with the Department of General Services.

15017. From time to time, as the work of construction or alteration progresses and whenever the Department of General Services requires, the architect or structural engineer, or both, in charge of construction or registered engineer in charge of other work, the inspector on the work, and the contractor shall each make to the Department of General Services a report, duly verified by him, upon a form prescribed by the state department, in consultation with the Department of General Services, showing, of his own personal knowledge, that the work during the period covered by the report has been performed and materials used and installed are in accordance with the approved plans and specifications, setting forth such detailed statements of fact as are required by the Department of General Services.

The term "personal knowledge," as used in this section and as applied to the architect or registered engineer, or both, means personal knowledge which is the result of such general administration of construction as is required and accepted of, and for, such persons in the construction of buildings. Such persons shall, however, use reasonable diligence to obtain the information required.

The term "personal knowledge," as applied to the inspector, means the actual personal knowledge of the inspector obtained by his personal, continuous observation of the work of construction at the construction site in all stages of progress.

15018. Upon written request to the state department by the governing board or authority of any hospital, the state department through contract with the Department of General Services shall make, or cause to be made, an examination and report on the structural condition of any hospital building subject to the payment by the governing board or authority of the actual expenses incurred by the state department.

15019. The state department may call upon the Department of General Services to make a periodic review of hospital operation to assure that the hospital is adequately prepared to resist damage caused by earthquake tremor. The review shall include, but not be limited to, evaluations of the structural safety of elevators, standby equipment and emergency procedures, and procedures and facilities for storage of dangerous gases, liquids, and solids. The governing board or authority of the hospital shall reimburse the state department for actual expenses incurred in making such review. The state department shall contract with the Department of General Services for such services.

15020. The state department, with the advice of the Department of General Services, shall from time to time make such rules and regulations as it deems necessary, proper, or suitable to effectually carry out the provisions of this chapter.

15021. There is in the state department a Building Safety Board which shall advise and act as a board of appeals with regard to seismic structural safety of hospitals. The Director of Public Health, with the advice of the Department of General Services, shall appoint the members of the Building Safety Board, which shall advise and act as a board of appeals in all matters affecting seismic structural safety in the administration and enforcement of this chapter. The board shall consist of 11 members appointed by the Director of Public Health and six ex officio members who are: the Director of Public Health, the State Architect, the State Fire Marshal, the State Geologist, the Chief of the Bureau of Health Facilities Planning and Construction in the state department and the Chief Structural Engineer of the Schoolhouse Section of the Office of Architecture and Construction in the Department of General Services. Of the appointive members, two shall be structural engineers, two shall be architects, one shall be an engineering geologist, one shall be a soils engineer, one shall be a seismologist, one shall be a mechanical engineer, one shall be an electrical engineer, and one shall be a hospital administrator. The appointive members shall serve at the pleasure of the director. He may also appoint as many other ex officio members as he may desire. Ex officio members are not entitled to vote. Board members, qualified by close connection with hospital design and construction and highly knowledgeable in their respective fields with particular reference to seismic safety, shall be appointed from nominees recommended by the governing bodies of the Structural Engineers Association of California; the California Council, American Institute of Architects; the Earthquake Engineering Research Institute; the Association of Engineering Geologists; the Consulting Engineers Association of California; the California Hospital Association. Board members shall

be residents of California.

15022. The Building Safety Board shall convene upon request of the chairman thereof. He may convene a meeting of the board whenever it may be necessary, in his judgment, for the board to meet. The board shall adopt such rules of procedure as are necessary to enable it to perform its duties. The chairman of the board shall, in his discretion, or upon instructions from the board, designate subcommittees to study and report back to the board upon any technical subject or matter for which an independent review or further study is desired. Members of the board shall be reimbursed from the Hospital Building Account in the Architecture Public Building Fund for their reasonable actual expenses in attending meetings conducted to carry out the provisions of this chapter, but shall receive no-compensation for their services.

15023. Any person who violates any provision of this chapter is guilty of a misdemeanor.

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EFFECT OF CONFIGURATION OF BUILDINGS ON EARTHQUAKE RESISTANCE AND ITS MANAGEMENT

Yang Yucheng[I], Yang Liu[II], Gao Yunxue[III], Lu Xilei[IIII]

ABSTRACT

Effect of configuration of buildings on earthquake resistance is shown in this paper, through some case history on earthquake damage and results in the study of earthquake resistance, in order to provide some experience for the architectural design so that a variety of buildings, both magnificent and earthquake resistance can be developed. At the same time, the authors attempt to show that it is more economic and effective to promote the earthquake resistance of buildings and mitigate earthquake hazard more effectively and economically by means of architectural design than structural design.

INTRODUCTION

One always desires to construct a building in a highly active seismic area not only with good earthquake resistance, but with architecturally treated beauty. A lot of earthquake damage showed, however, buildings with complex configuration and variation in elevation were easily suffered to damage; ornaments on the eaves and roof fell easily under the excitation of earthquake so that the architects' work would be limited to a certain extent.

In the "Aseismic Code for Industrial and Public Buildings"[1], requirements in the architectural design have been assigned, such as "configuration of buildings should be simple, distribution of mass and stiffness should be even and symmetry, and sudden change in elevation and plan or irregularity of configuration should be avoided as far as possible"; "Ornaments, parapets, cornices, etc., which will easily fall on the ground or separate from the roof during earthquake, should not be made or be made only in a small amount". As for the earthquake resistant measures, it states that "earthquake resistant joint should be made in order to separate the building into several units of simple configuration and evenly distributed stiffness"; "walls between windows should be of equal width and evenly arranged". Owing to the difficulty of the architect for the precise grasping the above statements, and the emphasis on the failure from experience during the

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earthquake, buildings in the seismic area have to be designed simple in form, lacking a sensation of art and beautifulness.

Reduction of the available architectural art function of buildings owing to consideration of earthquake resistance not only reflect in the construction of new buildings, but also in the strengthening of the existing buildings. Appearance of most of the strengthened buildings, using so called structural columns and collar tie beams installed outside the wall, is just like a prisoner fastened by ropes. There are no fine arts in such buildings.

It is a new topic worthy to study and explore for architects as how to combine strengthening work with architectural design so that fine arts may be involved in the reasonable aseismic design. As for this purpose, two concepts are proposed: firstly, to construct an earthquake resistant building from architectural point of view may be more economical and effective than the aseismic structural design, especially for the public buildings which are majority of buildings in a city. Therefore, whether an aseismic design is good or not is first reflected in the architectural design and not in the structural design. Secondly, architects' talent should not be bound by the regulations of the design code. Architects should develop a new type of earthquake resistant buildings in accordance with the basic principles of earthquake resistance, combining architectural art and earthquake resistance as a unity. Some earthquake damage cases and results of research are given below to provide a basis in the architectural design to develop a variety of earthquake resistant buildings.

COORDINATION OF CONFIGURATION OF BUILDING AND DEFORMATION

In the earthquake field, some buildings of complex configuration and variation in plan or elevation suffered serious damage. One may assume that it is induced by the complexity in building configuration. But, in fact, it is not perfectly true. Damage to buildings of complex configuration is not always heavier than that of simple configuration. For example, during the 1976 Tangshan earthquake in the Yuejin residential region of the Tangshan city, there is an east-corner-building (see Fig. 1a and b), only cracked (Intensity X), however, around many buildings of rectangular section collapsed. The plan of the building was of L - shape, the angle between two wings was 77° (see Fig. 2). The central part of the building was of 5 stories, two wings were of 4 stories and no deformation joint was installed between these wings. Of course, simple configuration is favourable to earthquake resistance, more exactly say, convenient for aseismic design, but if deformation of buildings of complex configuration during earthquake can be treated properly in the design, the cracking resistance or collapse resistance of such buildings will not be worse than those of simple configuration.

Coordination of deformations in all parts of a building can be summarized as following:

1. Coordination of deformation of different part on the same floor level. Horizontal drifts of all points on the particular floor during earthquake should be the same at the same time and in the same direction as far as possible, or members having small drift should be coordinated with members having large drift to resist earthquake load. However, drifts at the both ends of a building should be nearly the same.

2. Coordination of deformation of different parts on the same vertical plane in a building. Horizontal drifts of all points on the particular floor in a given vertical plane should be the same as far as possible, or members having small drift should be coordinated with members having large drift to resist earthquake load. At the same time, interstory drifts should not change abruptly..

Some examples are given below to illustrate the above two respects. Multi-story buildings of brick and concrete construction are rigid buildings, the walls of which are made by brittle material, say bricks. The strength and allowable deformation of brick walls are often low. In general, drifts of all lateral force resistant members (walls) should be nearly the same, either on the same floor level or in the same vertical plane. If the difference of drifts is much greater, walls with allowable deformation smaller will crack first, losing joint function of earthquake resistance. Some cases have proved the fact.

(1) Deformation of walls between windows or/and doors should be consistent. Generally speaking, wide walls is large in rigidity, so it is easily to crack. Drift of the wall not only depends on the width of the wall but also on the height and the load acting on it etc., the latter factors are easily neglected in the design. Therefore, it requires for the architect in the arrangement of doors and windows to make this width similarly equal and at the same time take care of the distribution of rigidity.

(2) Hallways in some multi-story brick buildings are of internal-framed structure, its horizontal drifts of deformation is greater than the main brick structure, causing much damage to the hallway itself and joining place, especially in the case of protruding halls. In Tangshan, partial collapse of the hallway of such buildings was often seen. Therefore, it requires in the design to install a deformation joint to separate the hallway from the main building or to increase its stiffness.

(3) The plane of multi-story brick buildings recently designed is seldom irregular. It is, therefore, favourable to earthquake-resistance. But, as far buildings of irregular

plan, if drifts of transverse walls on the same floor level are nearly equal and the deformation of the exterior wall in the protruding portion approaches to that of adjacent walls, no more damage would occur.

For R.C. framed structures with shear walls, small drift of shear walls should be coordinated with large drift of the frame to resist earthquake load. In order to make the frame and shear walls to work together, it is necessary to assure the floor of the structure to have a given stiffness so that earthquake load can be transferred effectively to shear walls. It is our opinion that for multi-story brick buildings, it is not suitable to add a 4-5 cm thick rigid concrete layer to the original floor slab. Although this can increase the stiffness of the floor, yet, at the same time, seismic load is also greatly increased, it would be better to add the cement and steel to the brick walls.

Theatres are another type of buildings of irregular configuration. In aseismic design, two approaches are used to deal with the irregular configuration. Firstly, the hall is separated from the front hallway and back stage of the theatre by means of deformation joint. Secondly, the hall is connected with the other parts as a whole structure. Two constructions are different. In the case of no separation, damage to the front hallway and back stage may be heavier; but in the case of separation, damage to the hall due to bending may be more obvious. These two constructions require different methods of checking in aseismic design and aseismic measures.

For buildings of non-rectangular plan, one always considers the torsional effect is prominent. In fact, it is not always true. The building located in the Yuejin region mentioned above is an example. Damage to this building due to torsional effect is not obvious. Five "L" shaped multi-story brick buildings were compared with three rectangular buildings for torsional effects after the 1975 Haicheng earthquake[2]. It is shown in Tab. 1 that, if well designed, center of stiffness and center of mass of L-shaped buildings can be consistent as well as rectangular buildings and damage due to torsion is also not obvious (computation results are consistent with the actual damage). In Anyang, there is a L-shaped building (Fig.3), corridor of the central part is located outside the exterior wall, while the corridor of the wing is located in the center of the building. Results of calculation show that the center of stiffness and center of mass of the building approach very closely. If the specified stiffness of the floor is assured, it is predicted that this building will suffer moderate damage during earthquake of intensity VII^{+}, and will collapse during earthquake of intensity X^{-}; if the building is separated by deformation joint, the center part will suffer moderate damage during earthquake of intensity VII^{-} and will collapse during earthquake of intensity of IX^{+}. From the above examples, it may be concluded that it is not necessary to separate all L-shaped buildings into two parts by joint. Other types of plan, such as [, I and Y-shapes, may be more reducible in torsional effect and more effective in earthquake resistance than L-shape.

Damage to buildings having variation in elevation often occurs in the protruding part and is generally heavier than the main building, therefore stiffness and strength of the protruding part should be properly increased in design, and rooms of large span, such as meeting rooms should not be located in the protruding parts. Some examples can be found during the Haicheng earthquake and the Tangshan earthquake.

Uneven vibration in different parts of a building should be also avoided. Measurements of dynamic characteristics of a 3-story L-shaped building in Anyang were carried out. The angle between the two wings is 104° , the ground floor is a large span store, while the two upper stories are used for apartments with interior walls. The measuring points were located as in Fig.4. At the both ends of the roof and at the junction of two wings, 4 pick-ups were installed. Vibrations were excited by exciters respectively fixing at five different locations. Although frequencies of different measuring points were similar, and the dampings measured were also approximately equal, yet in microtremor measurement, torsional effect obviously excited and the frequency at the junction were different from those at the end of the wings, i.e. local vibration effect excited. In order to reduce such effect in irregular buildings, consideration should be taken in design to increase the stiffness of the roof and floors, and to avoid too concentration of mass and stiffness of members used to resist lateral force.

Natural period of a structure should be avoided to coincide with the prominent period of ground motion in order to reduce the damage. Structures of different natural periods suffered different degrees of damage. Such examples can be easily found in the Haicheng and the Tangshan earthquakes also. It requires that in the design configuration of building dynamic characteristics should be considered.

In conclusion, earthquake resistance of a building, of course, depends on its configuration, but, more important, depends on whether deformations during earthquake are consistent or not. This should be emphasized in the architectural and the structural design.

ARCHITECTURAL LINES AND ASEISMIC MEASURES

At present, art has not got been considered in aseismic measures, but, if properly treated on the elevation of building, combine aseismic measures with architectural lines, for example the pier, pilaster, collar tie beam, gird, strip on wall between windows, sun breaker, window frame, sunk spandrel wall, etc., buildings in the earthquake region can be both artistic and earthquake resistant.

Several aseismic measures briefly described below are related to the treatment of elevation of building:

(1) Vertical lines treatment —— wall piers outside exterior wall.

In brick buildings, piers are often made outside the exterior wall or end walls for the purpose of stability, thrust bearing and decoration. On the other hand, such piers form protruding vertical lines on the wall, making the building more beautiful. These piers are effective in earthquake resistance. If piers are made at the junction of longitudinal wall and transverse wall, the section of transverse wall is increased, strengthening the connection of exterior and interior wall and, preventing overturning and collapse after shear failure of exterior wall. There were such buildings in Tangshan, piers were made every other panel. Cross cracks occurred on wall with pier between windows in the bottom floor, both sides of cracking walls fall down, but the pier and the walls on the upper stories did not collapse. No piers were made, collapse of walls on upper stories would follow often the shear failure of the wall on the bottom floor (Fig.5). If piers are made outside the longitudinal bearing wall, then, not only the area of the bearing wall is increase, but after the shear failure of the wall, owing to more core area remaining thus increasing its anti-collapse capacity.

(2) Horizontal lines treatment —— strip on wall between windows.

The horizontal lines on the exterior wall are eaves and plinth and sometimes are those between the upper and lower windows. If these horizontal portions are strengthened, it is effective for preventing cracks occurring at corners of a window. If strengthening horizontal strips are made on wall between windows, the earthquake resistance of the wall is greatly increased. Horizontal R.C. strips of 6 cm thick can be made on the wall, either protruding or retreating from the wall.

(3) Strengthened window frame protruding on wall face.

In the treatment of elevation of building, window frames protruding on wall face are sometimes used as a kind of decoration. If the frame is strengthened, the earthquake resistance of longitudinal wall will be increased. The frame is also effective for anti-cracking and anti-collapsing.

Lintel and sill can be used as the upper and the lower portion of the frame respectively. Reinforced cement mortar layer can be used as two columns of the frame. Precast R.C.

window frame can also be used instead of lintel and sill.

(4) Sunk spandrel wall.

Spandrel retreated from exterior wall are often used by architects to decorate elevation of a building. Such weakened sunk spandrel walls are easier to crack than walls between windows, which is often identified in the earthquake. In the Tangshan earthquake however, sunk spandrel walls of some buildings had "x" cracks, but these buildings did not collapse It hardly assumes that this is because of the cracks of the spandrels that makes the building not collapse, but in general case, no cracks will occur on the wall between windows after the spandrels crack (Fig.6). It is favourable for the exterior bearing wall; at the same time, repair of spandrels is easier to perform than that of walls between windows. Therefore, we assume that, in the architectural design for earthquake resistance, retreated and weakened spandrels can be used, but the cross section of which cannot be reduced too much, so that cracks may occur earlier.

SHAPE OF ROOF AND DECORATION OF EAVES

Roof and eaves are emphatically to be decorated in the architectural treatment, and are easily to be blamed in the inspection for earthquake resistance. For example, parapets, although a little bit higher than usual, are often required to remove. It is not practicable to build houses, all of which have flat roofs and without eaves and parapets in the seismic region. Variety of roofs and appropriate eaves decoration are necessary also, but their treatment should be prudent.

Effect of shape of roof, either flat roof, slope roof or arch cover, is not essential for damage. It can be justified by the record of destructive historic earthquakes. Arch cover may make more rigidly, if the roofs of the adjacent panels are made as flat roofs, the arch thrust will be equilibrated. In addition, such roof is also beautiful in appearance. Damage to dwellings with sloping wooden roof may be heavier than those with flat roof, but the weight of slope roof itself is more lighter, thus favourable to the earthquake resistance of the whole dwelling. But horizontal thrust should be avoided to produce in the structural design.

In the Tangshan earthquake, damage to non-structural elements, such as eaves decoration and attachment on the roof was neglected, because large amount of buildings collapsed. But in the Haicheng earthquake, a lot of injuries or deaths were caused by the falling of small chimneys, parapets, decoration on the roof or roof tiles etc. This was a tragedy and worthy to pay much attention. Therefore, it should be avoided to install chimney near eaves or just above the door opening. Limitation of parapets have been specified in the code. Most of parapets damaged or fallen during earthquakes were situated on slope roof or on old brick and wood buildings. Installation of parapets should depend on the vibrational characteristics of the main structure. For structures subjected to flexural deformation basically, no parapets, eaves decorations, etc. should be made; for structures subjected to shear deformation basically, such non-structural elements can be made, and height of whcih should not be limited strictly. As for protruding eaves, large amount of falling of such eaves has not yet occurred, but emphasis should be put on its balance, anchoring and the strength of the bearing wall. Poured protruding eaves at the roof level is favourable to the increase of stiffness of the whole structure.

CONCLUSION

Recently pithy style is the main feature in the architectural design. But if the configuration of buildings in seismic region is too unitary and insipid, it will lose its artistry, quality, its beauty and reduce their function. Of course, in the architectural design, it is not correct to make clumsy ornaments or beautiful configuration of building without consideration of earthquake resistance and regard to cost of construction. But architectural art should be well considered also. With the cooperation of architects and structural engineers, using experience learned from earthquakes and knowledge obtained in scientific research, not only configuration of building can be designed to have a certain degree of earthquake resistance, and a new type of buildings can be developed, practical in use, economic in construction, beautiful in appearance, resistant to earthquake. But latent potentiality in the architectural design may be fully brought out and damage to buildings may be reduced.

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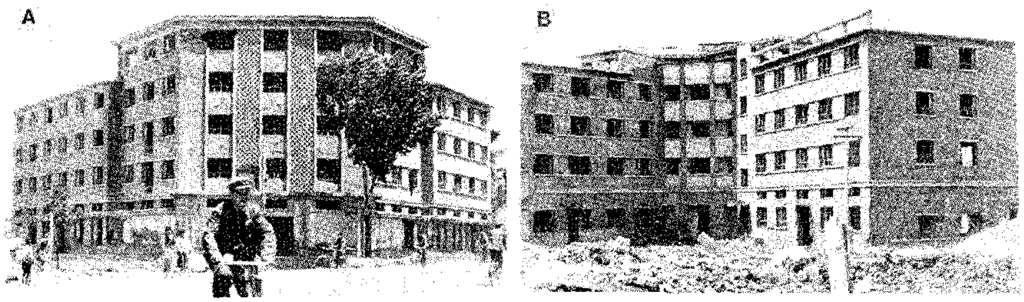


Fig. 1 The east-corner-building in the Yuejin residential region of Tangshan city cracked (Intensity X) during the 1976 Tangshan earthquake.

a. front elevation b. back elevation

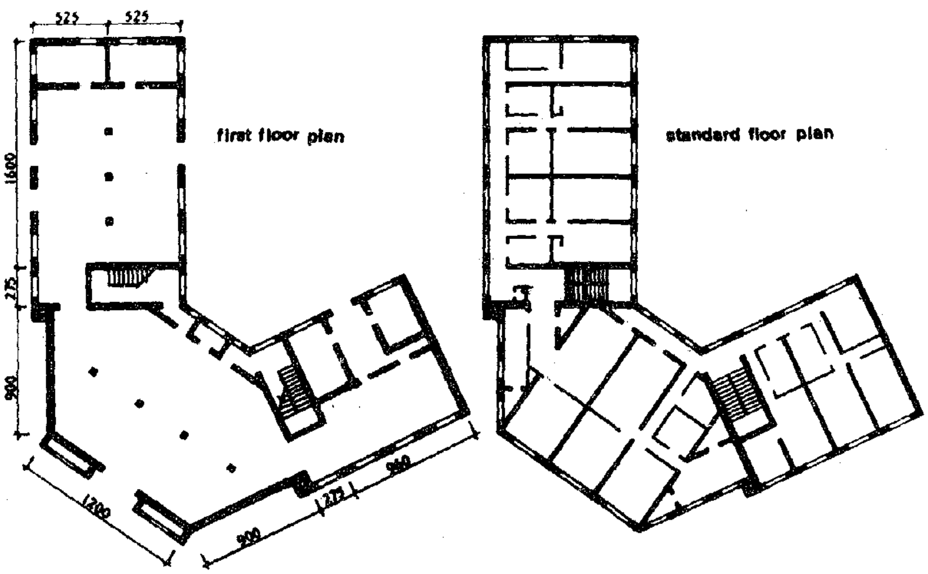


Fig. 2 Plane figure of the east-corner-building

Table 1 Torsional effect of multi-story brick buildings in Haicheng

Name of building	Plan shape	Static eccentricity e (m)	Ratio of eccentricity 2e/L (%)	Max. influencing coeff. (%)
Agricultural machine station	L	5.19	21.6	71.8
People's military bureau	L	3.70	16.4	44.2
Zhanqian hotel	L	1.89	11.7	30.2
Police bureau	L	0.96	3.9	9.1
Army guest house	L	0.49	2.8	6.0
Transformer factory	Rectangle	2.70	20.0	64.4
County guest house	Rectangle	2.48	16.9	28.1
Dormitory of the steel factory	Rectangle	0.60	2.5	4.4

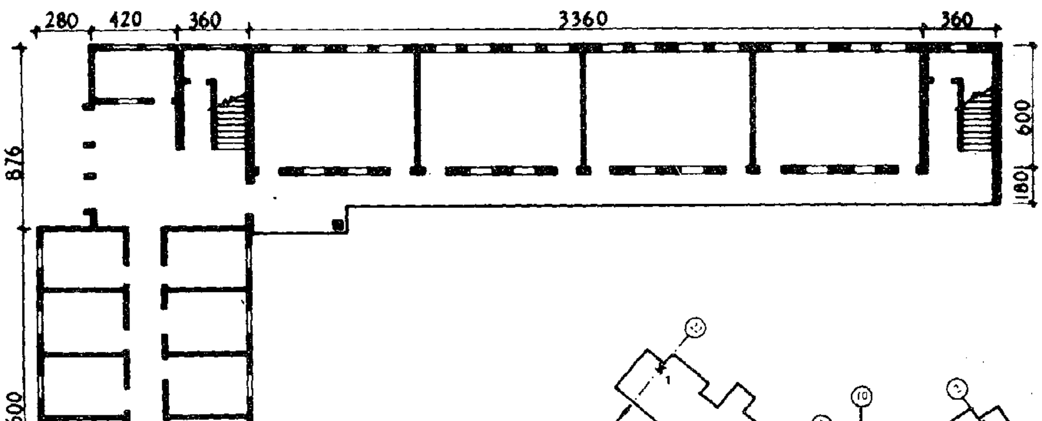


Fig. 3 Plane figure of the L-shaped building in Anyang

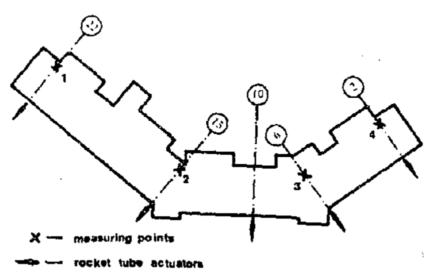
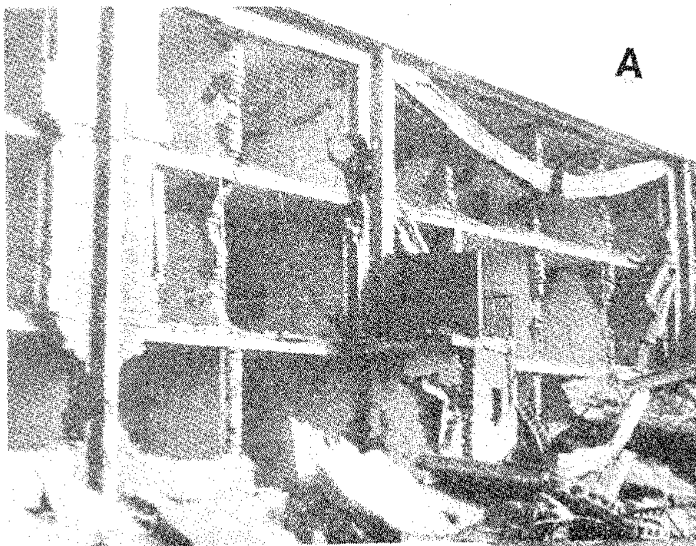


Fig. 4 Location of the measuring points on the 3-story L-shaped building in Anyang



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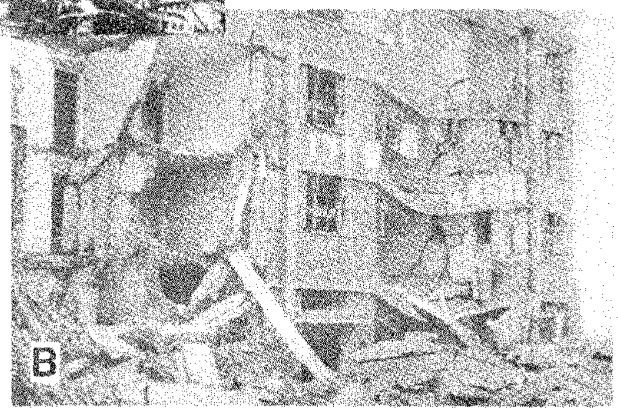

Fig. 5 Damage to building having wall piers outside during the Tangshan earthquake

a. dwelling
b. office building

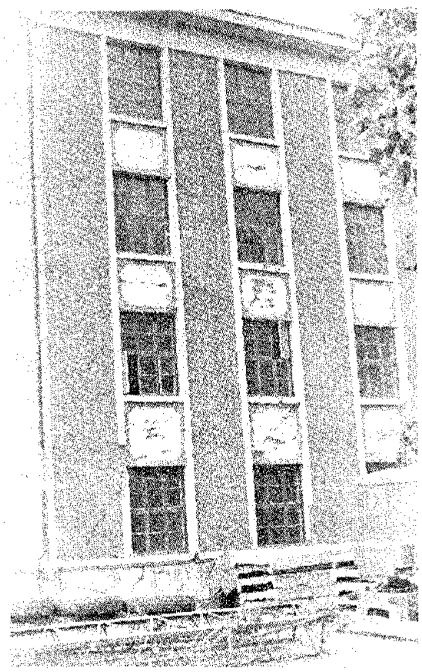

Fig. 6 Damage to building Having sunk spandrel wall during the Tangshan earthquake

DESIGN OF SECONDARY BUILDING SYSTEMS FOR SEISMIC EFFECTS: APPROACHES TO DESIGN AND LIMITS OF FEASIBILITY

By GERALD M. McCUE

ABSTRACT

Secondary systems of buildings designed according to conventional practices of architects and engineers, and meeting current building codes, have suffered damage in recent earthquakes due to vibrational and displacement effects. The evidence suggests that codes and design procedures may not include principles which are essential to satisfactory performances of secondary systems. A two-phase design process provides a means for reducing damage to secondary systems. The first phase considers both vibrational and displacement effects in the design of site and building to reduce the severity of building response that components must sustain; the second part considers vibrational and displacement effects in the design of secondary systems and their component parts. Despite systematic approaches for design, technological, practical and social factors affect their implementation and impose limits on improving performance.

Paper presented at the 1981 Joint United States/People's Republic of China Workshop on, "Earthquake Disaster Mitigation through Architecture, Urban Planning, and Engineering Under the U.S.A./P.R.C. Cooperative Research Program on Earthquake Studies," Beijing (Peking), People's Republic of China, October 28-November 14, 1981.

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INTRODUCTION

Increased attention is being given to the performance of secondary building systems during earthquakes. Previously thought to be of minor consequence, damage sustained by these systems and damage they have caused to structural systems have brought recognition of their importance. Recent earthquake damage indicates that components of secondary systems can perform significant dynamic roles. Losses from damage establish social and economic significance to performance by these systems. Studies of damage and loss as well as investigation of analytical methods and design approaches offer a basis for improved design and better performance. However, social, economic and technical factors form practical limits to levels of performance that may be expected.

This paper notes the reasons for increased attention on secondary systems and outlines development of approaches for earthquake design. A design process is presented which addresses the two major factors essential to improved performance of secondary systems. One is to reduce the severity of the dynamic conditions imposed by building earthquake response; the other is to improve the capabilities of these systems to sustain the conditions with controlled damage. Factors which limit feasibility and implementation are also introduced.

New Understanding of Functional Roles

Traditionally, design professionals have described parts of buildings by their functional roles during essentially static conditions. Parts which carried gravity loads, such as columns, beams and bearing walls, were considered "structural"; all other parts were considered "non-structural." It is now understood that during dynamic conditions, all parts interact. New terms are being proposed which convey more accurately the functional roles which parts of the buildings perform during earthquakes.

Dynamic Structure

Several terms have been proposed to describe components that provide a building's essential capacity to sustain the dynamic conditions introduced by ground motions, including "primary structure," "seismic structure," and "dynamic structure," which is used here. Dynamic structure is conceptually descriptive as it includes the structural systems that carry gravity loads as well as the other components that under dynamic conditions perform equally important structural roles.

Secondary Components and Systems

All building components not part of the dynamic structure are defined here as "secondary." A distinction is made between "component" and "system." "Component" refers to a physical entity which may be fabricated from several pieces but forms an essentially independent unit when considered dynamically. A prefabricated panel of a curtain wall, an electric transformer, or a single panel of a lay-in suspended ceiling are components. "System" refers to interconnected components which form a substructure and must be considered individually and together for their dynamic properties of response.

Except when integrated into the dynamic structure, secondary systems generally include: components of the building "enclosure systems," such as infill walls, curtain walls, windows and spandrel covers; components of the building's finish systems, such as partitions, ceilings and veneers; and components of the building service systems, such as heating, ventilating, lighting, communications and transportation.

Role and Importance of Secondary Systems

Recent earthquakes have drawn attention to the seismic performance of secondary systems. Patterns of damage indicate that vibrational and displacement effects of building response have caused unanticipated interaction between structure and secondary systems. Stiff, strong or heavy secondary components, such as stair and elevator shafts, concrete and masonry infill walls and enclosure panels, have altered building response and caused damage to the structure. Lateral displacements of flexible structures have damaged stiff secondary components, such as masonry infill walls and concrete and masonry enclosure walls. Also, such secondary systems as partitions and ceilings have affected patterns of building response. As a consequence, increased attention is being given to the dynamic interaction between systems, to the role of secondary systems in affecting building response, and to the effects of building response on the performance of secondary systems and their component parts.

Buildings suffering little or no damage to their structures have incurred significant damage to secondary components. The vulnerability of exit routes and mechanical systems critical for life safety is of particular concern. Following the 1971 San Fernando earthquake, for example, about 90 percent of the elevators in the immediate Los Angeles region were inoperable. The social and economic impact of damage to secondary systems can be extreme, since about 60 to 70 percent of the construction cost for finished buildings is for secondary systems. Of the $6.6 million damage incurred in the El Centro earthquake, $3.9 to $4.9 million has been attributed to non-structural losses. Loss of use of buildings due to damage to these systems can impose severe economic losses on owners and the community. It is now recognized that better performance of secondary systems may save lives, reduce injury and property loss, and ensure continuing community services and productive employment.

Evolution of Approach to Earthquake Design and Secondary Systems

Early studies of earthquake damage concentrated on improving safety by preventing building collapse and reducing falling debris. Design approaches used static forces to approximate earthquake stresses. Current studies are intended to limit and control damage to ensure safety and reduce social and economic losses. Design approaches simulate earthquake forces by using both static and dynamic models. For example, in the rebuilding of San Francisco following the 1906 earthquake, a 30 pounds per square foot static wind load was used to approximate seismic forces. It was not until after the 1925 Santa Barbara earthquake that the importance of mass was recognized. The Western States Uniform Code introduced .075 times weight as an equivalent horizontal static load for seismic design. Following the 1933 Long Beach earthquake, the City of Los Angeles raised the factor to .080 times weight.

After the 1940 El Centro earthquake, studies of the vibrational character of earthquakes led to recognizing dynamic factors. The 1943 Los Angeles Code was the first to relate force to story height. Studies begun in 1952 resulted in the 1957 publication, "Recommended Lateral Force Requirement for Buildings," which introduced the broad range of dynamic aspects of earthquake response. As a result of these and other studies, equivalent static forces in California codes were successively raised to a range of ~.25 to 1.00 times supported weight for various parts of buildings.

The Alaska and Santa Rosa earthquakes, both in 1969, provided new tests for contemporary theories. Changes in construction materials and techniques, as well as increases in the size of buildings, provided new precedents. In certain respects, damage to buildings designed according to current codes was greater than expected due to unanticipated interaction between structure and secondary components. This caused the Structural Engineers Association of California (SEAC) to begin a re-examination of analytical techniques and design criteria. While this was underway, the 1971 San Fernando and 1972 Managua earthquakes drew new attention to the dynamic attributes of earthquakes. The SEAC studies led to the 1978 Applied Technology Council Report, "Tentative Provisions for the Development of Seismic Regulations for Buildings," currently the most comprehensive guide to earthquake design criteria in the United States.

The development of design approach to secondary systems follows similar trends. Early requirements were for components and their anchorages to sustain horizontal static forces. Design emphasis was placed on resisting horizontal stresses in components to prevent failure and in anchorages to prevent dislocation. For relatively "stiff" traditional buildings of masonry, shear wall and braced frame construction, dislocation was the most serious design issue and this approach provided generally satisfactory solutions.

Importance of Lateral Displacements

Beginning in the 1950's, the character of many buildings in the United States began to change. Taller, longer span, more open buildings were designed to meet new functional criteria. New construction materials and changing techniques emerged for off-site fabrication and on-site installation. The need for greater adaptability over time also lead to changes in the character of contemporary buildings. Better understanding of dynamic response has lead structural engineers to design more resilient and ductile structures as a means of reducing the forces which the dynamic structure must sustain, changing the dynamic character of buildings. As a result, contemporary buildings tend to be more "flexible" than traditional ones, and therefore exhibit greater displacements during earthquakes. These trends are particularly important because of the vibrational and displacement effects which flexible buildings impose upon secondary systems.

In many respects, the approach to design of secondary systems has not kept pace with the new conditions imposed by flexible buildings. Damage due to shear and torsional stresses caused by dynamic effects still persist, caused by the displacement effects inherent in the response of more flexible buildings.

Limits of Feasibility

Currently, a range of technical, practical and social factors impede application of the most advanced knowledge and theory into design practice. Lack of empirical data as a basis for design is one problem. Technical challenges and even conceptual aspects of the way architects and engineers approach design also contribute. Lack of control of manufacturing and construction also make implementation difficult. Taken together, these factors limit the feasibility of achieving better performance in secondary systems.

Design Approach to Improve Performance

A design approach that considers both the dynamic and displacement effects of earthquake response offers promise of improved performance of secondary systems. Factors limit feasibility of high technology solutions for most buildings. Therefore, the best approach will place as much emphasis on reducing the severity of the effects imposed on these systems as upon the detailed design of the systems.

REDUCING SEVERITY OF BUILDING RESPONSE

Earthquake design should anticipate and achieve desired levels of performance for secondary systems. Control of performance begins with a first phase of design for both site and building intended to reduce the severity of the response that secondary systems must endure. The second phase is intended to achieve secondary systems that sustain the dynamic and displacement effects which are imposed. (For an outline of such an approach, see Appendix A.)

The first design phase focuses on anticipating and reducing the severity of the conditions imposed on secondary systems and may include the following steps:

- Programming,
- Selecting and analyzing the site,
- Developing building design alternatives,
- Controlling response, and
- Evolving a design concept.

Programming

Defining explicit seismic objectives is important to the design process. The programming step establishes needs and priorities, and in the case of buildings and building components in seismic zones, establishes objectives for performance during earthquakes. Building codes do not normally provide an adequate basis for design. With the partial exception of regulations for critical activities such as hospitals, codes are primarily intended to prevent total failures which might result in death or injury. As a consequence, design criteria intended to maintain a building in continued operation or to limit costs for repair or replacement are normally programmed by the design team, by architects and engineers in collaboration with the owners and operators of the facilities.

It is not usually cost-effective to design secondary systems and all their components to endure conditions imposed by the most "maximum possible" earthquake projected for the site. Often the building program establishes a tiered set of objectives identifying specific "design earthquakes" and corresponding "thresholds-of-damage" acceptable for major building systems and their components. As an example, for a design earthquake with a probable occurrence of 25 to 50 years, all secondary systems should be designed to incur only minor damage. However, for the "maximum-probable" earthquake on a 100 to 500 year basis, acceptable levels of damage should be defined depending upon the importance of the facilities to life safety and their social and economic value.

While projections of damage are imprecise, subject to margins of error, they are useful in advancing the dialogue about relative priorities among systems. Programming information also helps focus on the necessity for establishing the probable characteristics of the building response for each tier of design earthquakes as a basis for establishing the conditions that secondary systems must endure.

Program requirements for earthquake performance are usually examined in light of other functional needs for normal operations and for other emergencies, such as fire. Frequently, this creates conflicts and requires that relative priorities be set. The process also considers technical, as well as social, economic and aesthetic factors. Decisions are largely based on judgment, since imprecise projections and approximations must be used in considering conflicting requirements.

Selecting and Analyzing a Site

To achieve an optimum building response for performance of secondary systems, the design of site and building should be considered together. The importance of site selection to reduce hazard is well-documented, but the relevance of site planning has not received sufficient attention. For large and geologically complex sites, analysis may identify portions of the site more or less suitable for facilities. Identifying locations of potential ground failure, areas with the possibility of high amplification of input motions, and locations where discontinuities of geologic structure might result in different ground motions, permits site planning which may reduce the severity of seismic response. For buildings where performance of secondary systems is important, a seismic response analysis should be prepared to determine probable ground motions for a series of design earthquakes, both maximum-possible and minimum-probable for ranges of 25 to 30, 50 to 100, and 100 to 500 years. Site response is controllable only within narrow limits. The most important control is planning, intended to locate buildings, utilities, access routes, excavations and earth fills on sites of lowest hazard and lowest probable amplitude of ground motions. Occasionally it is possible to provide additional controls by removing elastic soil materials and by preparing bases under earth fills, buildings and utility stations. In certain cases, introduction of basements and particular types of foundations may also reduce severity of response by ensuring that ground motions are introduced uniformly into buildings.

Developing Building Design Alternatives

While site response is important, equally critical--and more controllable--is the building response. It is important to recall during every phase of design that the dynamic environment for secondary systems is imposed by the building response. Therefore, for buildings where performance of secondary systems is important, schematic design alternatives for the building must consider the probable characteristics of building response.

No one design or response is necessarily best. Relatively "flexible" and relatively "stiff" buildings each have advantages and disadvantages depending upon many factors: the frequency and amplification of earthquake motions, the fundamental period of vibration of the building, and the materials and methods of construction. The important point is that there are two objectives for controlling building response. The first is to reduce the severity of overall response in order to achieve an optimum and cost-effective dynamic structure, and the second is to achieve a building response that makes the design of secondary systems technically and economically feasible.

Controlling Response

Building response may be controlled only within limits. Understanding the important physical aspects which affect building response enables designers to see how the dynamic structure interacts with secondary systems and how secondary systems may influence building response.

-Stiffness: Both configuration and connectivity are encompassed in stiffness, which is one of the most important characteristics affecting building response. Structural type, such as braced or movement-resisting frame, and the materials of the dynamic structure are primary influences on building response. Occasionally, enclosure or finish components may be incorporated into the structure to produce dynamically effective systems such as web-frames, skin-frames and the tube systems. However, the more normal concern is to prevent relatively stiffer or weaker components such as enclosure walls, partitions, ceilings, and piping systems from resisting displacements of the dynamic structure.

-Configuration: Building configuration and configuration of the dynamic structure are the most fundamental factors in controlling response. Height, configuration, continuity of plan and section, and dynamic-resistive elements are all critically important.

-Mass: The mass of a building is also an important factor in overall building response. The total weight and distribution of weight are major factors in establishing the forces which must be sustained by the dynamic structure. While controllable within limits, the structure is often more a result of cost than seismic response. Concrete and masonry structures weigh several times the structures of steel and lightweight assemblies, and sustain corresponding greater forces. The mass of secondary systems, particularly enclosure and finish systems, is more subject to design control. It is also possible to reduce stresses in components and anchorages by reducing weight of secondary systems.

-Strength: Within limits, strength provides a way to control building response. Components of secondary systems frequently have relatively low strength and thus fail at low levels of stress. Increasing strength extends the severity of force which a component may accommodate before failure, although ultimate strength is not significant to response until the component is stressed to the point of yield. Making components of secondary systems stronger than necessary to sustain motion may affect the character but does not necessarily reduce the severity of building response.

-Connectivity: Control of connectivity at the interface points of components within secondary systems has a direct effect on building response. The type of interface, whether it transmits moment, axial or impact stresses, or permits free movement, is a major determinant in the interaction. Architects and engineering specialists should collaborate with structural engineers to establish the explicit role secondary systems should perform during dynamic conditions.

-Damping: Current analytical methods enable little control over damping or energy absorption in a building. The true nature of damping is only approximately known, and no reliable schemes to modify it have been developed to date.

Building response should be considered for each design earthquake with respect to two different and possibly conflicting objectives: optimum benefits for the dynamic structure, and optimum benefits for secondary systems. While a flexible and resilient building may reduce forces and costs for the dynamic structure under conditions of the maximum-probable earthquake, it may make design of secondary systems expensive and marginally feasible for lesser earthquakes. In contrast, a stiffer building may lessen the cost and increase feasibility of performance of secondary systems but result in additional costs for the dynamic structure.

Evolution of Design Concept

Development of the selected building design concept normally proceeds concurrently with generation of concepts for individual secondary systems, such as enclosure walls and partitions, since the dynamic role of these systems may be of particular relevance to the building design. Dynamic roles may differ for individual secondary systems, each with their own requirements of connectivity. Secondary components can be connected to the dynamic structure in four ways.

-Components of the enclosure, finish and service systems "integrated" into the dynamic structure: Components are fully interconnected to provide complete interaction among all parts of the dynamic structure. This solution is normally more costly, and the trend toward off-site prefabrication tends to make this level of interconnection technically difficult. It also typically makes the building less adaptible. Only carefully selected elements of the enclosure and finish systems, such as permanent elevator, stair and duct shafts and permanent enclosure walls, are normally suitable for integration into the dynamic structure.

-Secondary components "coupled" to the dynamic structure: Components of secondary systems which significantly affect the response of the building but which lack permanence or have incompatible physical properties and therefore are not suitable for integrating into the dynamic structure, are considered coupled with the dynamic structures. The interaction between these components such as masonry infill walls, concrete panel enclosure walls and heavy enclosure equipment, may provide beneficial damping of building response, but usually the design objective is to limit interaction in order to prevent damage to either the structure or the component. While components integrated into the dynamic structure are rigidly interconnected and deform with the structure, coupled elements should normally be connected to limit their potential for restraining the displacements and deforming the dynamic structure. Coupled components must be securely anchored to some portion of the structure to sustain forces of inertia and prevent dislocation. To limit interaction between components caused by horizontal displacements, connections and interface conditions must often allow for differential motions and deformations. Damping may be accomplished by the methods used to connect components to the dynamic structure, such as ceilings, partitions and curtain walls, but this approach is rarely feasible for mechanical and electrical equipment or other heavy components.

-Secondary components "uncoupled" to the dynamic structure: Components of secondary systems which are lightweight and flexible so that their effect on the structure cannot be significant, are considered uncoupled from the dynamic structure. Lightweight, panelized non-masonry partitions, panelized ceilings, doors, windows of normal size, ducts, lightweight piping and lightweight mechanical and electrical service fixtures are examples of such components. Connectivity requirements are the same as for coupled components to prevent damage to the components. Selected connections must provide positive anchorage to sustain inertial stresses and prevent dislocations caused by vibration, yet other connections may require accommodating displacements and deformation by the structure or by other components.

DESIGN OF SECONDARY SYSTEMS TO ACCOMMODATE BUILDING RESPONSE

The second design phase is intended to achieve secondary systems and components of these systems capable of sustaining the dynamic and displacement effects imposed by the building response. (For an outline of this design approach, see Appendix A.) The designer may follow several steps:

-Formulate design strategies,

-Analyze dynamic environment,

-Generate design concepts,

-Select and develop design concept,

-Analyze technical details: vibrational effects, and

-Analyze technical details: displacement effects.

Formulate Design Strategies

It is useful to begin the second phase with a programming step for secondary systems. Reference should be made to the seismic performance desired for each system for earthquakes of various probabilities described earlier. Other functional requirements such as acoustic isolation, fire separation, maintenance and adaptability should be considered, as well as visual qualities. Initial maintained costs may also be evaluated with respect to benefits provided.

Analyze Dynamic Environment

Just as ground motion activates the building, so motions of the building in a particular location activate components in that location. As a result, it is necessary to identify the qualitative and quantitative aspects of both vibrational and displacement effects for each portion of the building that may have a significantly different response. This response, referred to here as the "dynamic environment," provides design criteria for design of components in that location.

Generate Design Concepts

To design each secondary system, alternative materials and types of systems are evaluated for their ability to sustain vibrational and displacement effects imposed by the dynamic environment and for sufficiency to meet other program requirements. This is the most critical and creative step in component design. Systems and components must be analyzed for the building response projected for each design earthquake. The following considerations should be addressed jointly by architect and engineer: the dynamic role of the component, the approach to damage control, and the approach to anchorage. Systems are approached individually and collectively to ensure consistent and compatible solutions among components interconnected within the same system and among adjoining systems.

-Dynamic role of component: The role of the component must be confirmed. Will it be integrated into and designed with the dynamic structure, or will it be a secondary system? If it is to be a secondary system, a decision must be made about whether it should be coupled or uncoupled with the dynamic structure, and what type of connectivity is required.

-Approach to damage control: While the objective is to prevent damage, explicit discussion of the nature of damage that would be acceptable is useful. Designers should establish what design approach is being taken. A "tiered" damage approach is often selected, where systems of different priority are designed to sustain different design earthquakes. A "damage zone" approach may also be adopted, where weakened zones upon which the system is not dependent for its integrity may fail if displacements exceed certain magnitudes. This approach requires that the majority of the system will sustain the earthquake.

-Approach to anchorage: Anchorage types for major systems and components are reviewed. "Rigid" anchorage allows little or no relative displacements; "controlled" allows motions in one axis only; "plastic" allows for deformation, but does not fatigue to failure; and "resilient" allows controlled motions in all directions. An anchorage is selected to meet necessary conditions.

Select and Develop Design Concept

During development of the design, components are considered with respect to the physical attributes previously considered in describing factors affecting building response: stiffness, configuration, mass and strength. Of particular concern is configuration, for large irregular components do not readily accommodate changes in building geometry caused by displacements. Eccentricity of mass with respect to points of anchorage is also important because of torsional stresses which are induced. Locations for anchors should also be selected to permit displacements of the structure without damaging the component. Where possible, anchoring components to two or more parts of the structure and/or other components should be avoided. This is especially important where there is the potential for relative dimensional change between anchors or the possibility that motions of different frequency, amplitude or phase will be introduced through different points of anchorage.

Analyze Technical Details: Vibrational Effects

Components are analyzed with respect to a range of design issues which relate to both vibrational and displacement effects. Vibrations that a component must sustain are erratic, three-dimensional motions in changing trajectories, with rapid accelerations, high velocities, and abrupt changes in direction. They are described by frequency and peak accelerations. Analysis reveals the magnitude of forces involved, indicates how forces will be applied, and suggests the irregular non-axial quality of the displacements that will take place. Vibrational effects raise several specific design issues regarding relative stiffness, instability, overstress, deformation, potential for resonance and damping.

-Relative stiffness: Relative stiffness among interconnected components is critical, since each component restrains deflection and is stressed in relation to its stiffness compared with other interconnected components in the particular axis of the trajectory of each particular motion.

-Instability: Components not fully restrained or anchored symmetrically with respect to centers of gravity are subject to overturning or rotation. Analysis of where motions will be induced reveals whether anchorages can maintain stability.

-Overstress within component: Components are analyzed to determine whether shear and torsional forces caused by motions in any axis and in all directions through the specific locations of anchorage will cause overstress within the component.

-Overstress of anchorages: Anchorage devices (clips, angles and rods) and fasteners (bolts, screws and inserts) are examined for shear and withdrawal capacities under stress from forces in any axis and in all directions, not merely on orthogonal axes.

-Excessive deformation: Components and anchors are analyzed for deformation. Excessive deformation, even within elastic limits, may cause unanticipated interaction with adjacent systems or components.

-Potential for resonance: Components of secondary systems are vulnerable to damage by resonance when frequency of input motions coincides with the natural period of the component. This is similar to conditions caused by vibrations induced by mechanical equipment. Components supported on resilient mounts or anchors are especially susceptible and require damping. Large mechanical equipment is routinely analyzed for this phenomenon, but the potential exists for any system with resilient anchors.

-Damping: Altering or increasing the capacity for a component to absorb or dissipate energy of the building response is theoretically possible but untested. There are currently no practical applications. Methods of structural analysis for vibrational effects of coupled or uncoupled components are documented in the literature. Various analytical models have been introduced and vary in complexity. Methods of analysis are significantly altered by the number of different anchorages and the potential for more than one input motion through different anchors. This latter complexity should be avoided.

Analyze Technical Details: Displacement Effects

Displacements in different portions of the building are likely to be different at any point in time. For example, displacements between floors are erratic, multi-directional and non-axial, varying in dimension as illustrated in diagrams appended to this paper. The designer must attempt to visualize the fully three-dimensional character of displacements and the potential dimensional changes accommodated by components. Where vibrational effects may only require increased strength in component anchorages, displacement effects require workable strategies for sustaining geometric deformations and dimensional changes. Extreme care must be exercised so that details designed to accommodate displacement effects do not negate provisions needed for vibrational effects.

Displacement effects involve design issues regarding changes in building geometry, displacements of anchorages, dislocation, impact, and out of phase motions.

-Changes in building geometry: Relative displacements of columns, beams, floors and components of secondary systems will alter building geometry and may change boundary conditions for individual components.

-Deformation: Stresses in individual members of structure and in individual components will cause deflections and deformations and may change boundary conditions.

-Relative displacement of anchorages: Displacements in the primary structure, relative displacement between floors, and displacements due to deformation of other secondary components may cause changes in dimension between points of anchorage. Relative displacements of anchorage must be accommodated within the anchor or within the component, and may impose stresses in addition to those caused by other vibrational effects.

-Dislocation: The potential for components to become dislodged from their original positions is of particular concern. Sliding or resilient connections and isolation-mounts intended to isolate sound or vibration during normal conditions, or to accommodate differential movements during earthquakes, may permit uncontrolled motions beyond anticipated limits causing unanticipated interaction between components.

-Impact: Impact may cause severe additional stresses in components, significantly greater than those caused by vibrational effects. Any or all types of displacements may cause components to impact with the dynamic structure, with other components of the same system, or with other systems. Several impacts are often overlooked: the impact against the end of a slip-anchorage or at the limits of compressibility of spring-mounts, or between adjacent components moving in strip joints designed to accommodate differential displacements.

-Differential displacements and out of phase motions: Describing the design criteria of the dynamic environment usually means stating the difference between the extremities of lateral displacements between two points. If the maximum displacement of the third floor is anticipated to be 3/4" and of the first floor 1", then the differential displacement (in this case, "floor-to-floor drift") is defined as 1/4". However, as indicated in one of the accompanying diagrams, it is possible for corresponding locations on two adjacent floors to be displacing laterally along two different vectors. It is also possible for flexible components of different systems, such as suspended piping systems and suspended ceiling systems, or adjacent components with resilient anchors to be in different phases. This means the differential displacements may approach the sum, rather than the difference between lateral displacements.

The systematic process described provides the basis for design of secondary systems. A case study using essentially this process is included as Appendix B. Discussion of practical limitations that affect translation of theory into practice follows.

FACTORS LIMITING FEASIBILITY OF IMPROVED PERFORMANCE

Previous sections have presented the two design phases, and the objectives, approaches and issues that must be addressed to improve performance of secondary systems. There is a third phase, not within the scope of this paper, which involves supervision, fabrication, construction, and installation to create the built work. Theoretical, technical, social and economic factors affect the design phases, as well as feasibility of transferring theory into design and then into construction. Each factor is different, subject to varying degrees of control and exerting various degrees of influence. In most cases, several will be present at the same time. It is probable that the aggregate of these factors will negatively affect efforts to improve performance.

Limitations on Projecting Design Requirements

Central to the concept of a dynamic approach is the ability to project probable characteristics of energy activating a site, as well as probable site response, building response, and to some extent, component response. There are differences of opinion about the levels of accuracy possible at the end of this chain. However, even if the margin of error is significant, design approaches that make explicit and seek to accommodate specific vibrational and displacement effects are more apt to provide improved performance than approaches that omit from consideration what are now known to be important aspects of the phenomenon.

Conflicts With Other Than Seismic Criteria

There are many conflicts which pose problems of priority for designers.

-Building configuration: Building configuration derives from many factors, including the nature of activities, social amenities, views, exposure, site configuration, density of coverage and visual appearance. Imposing constraints for seismic performance frequently conflicts with one or more of these factors.

-Adaptability: The need for column-free, wall-free space suitable for rearrangement of functions may not agree with optimum structural systems for seismic performance.

-Fire separation: Requirements for draft-stops, fire separations and other fire-rated divisions within a building are frequently in conflict with seismic requirements for resilient connections. Fire divisions may also be in conflict with clearances needed to permit displacements and prevent undesirable interaction between components.

-Acoustic isolation: Isolation of components in order to eliminate transfer of sound poses many of the same problems as fire separation, and often imposes the additional problem of increased mass.

-Vibration isolation: Isolation of mechanical and electrical equipment to prevent transfer of vibrations during normal operations often requires introducing massive inertia blocks and normally requires separation by resilient anchors, requiring special attention for earthquake design.

-Locations for mechanical equipment: Optimum locations for heavy mechanical equipment for seismic performance may not coincide with what would be the most economic distribution of these systems.

Lack of Data about Dynamic Characteristics and Relative Stiffness

Secondary systems and their components pose design problems not associated with the design of dynamic structure. First, there is very little data available about pre-manufactured components, their capacity to withstand inertia, shear and torsional forces, and their capacity to accommodate stresses imposed at their points of anchorage. Also, there is rarely sufficient data about their physical properties to project dynamic characteristics.

Indeterminacy

To some reasonable extent, the dynamic structure may be isolated, its configuration defined and its material systems and relative stiffness established. It is much more difficult to assess secondary systems and their components since they are frequently composites of several materials with highly irregular configurations. Often the precise location and interconnections are established in the field during construction, and frequently these systems are altered over time, after construction is completed.

Different Stiffness Among Vectors

Secondary systems vary in their physical properties and will normally have a different stiffness in each vector of motion. A ceiling may be stiffer and attempt to restrain motions of a partition system in one direction of motion, while in another direction, piping attached to the partition may be stiffer and restrain motions of both partition and ceiling. The interconnection of secondary systems of unknown structural capacities with varying stiffness in each axial and non-axial direction of motion poses an almost unsolvable dilemma. Except for the secondary systems whose individual components can be isolated, indeterminacy establishes very real limits on accuracy of design.

Problems of Geometry

Designing components to accommodate displacements poses problems which are not always solvable with Euclidian Geometry. For example, floor-to-floor drift may be resolved in external enclosure systems in which components are essentially two-dimensional and narrow, approximately a line connecting two points. For this case, displacement of one point of anchorage with respect to the other may be achieved without deformation of the line and the panel can rotate across the face of the structure, provided the distance between the points does not change beyond the capacity of the connections to accommodate change. This principal is used in sophisticated curtain wall designs where special provisions are required at corners in order to allow rotation along adjacent wall surfaces. (See Appendix B for a case study of this type.)

It is significantly more difficult to accommodate changes in geometry in three-dimensional systems. One example is an interior partition system anchored to ceilings with elements of mechanical distribution systems anchored to the floor above and to the ceiling below. Even if separated into panels, the partitions may not rotate past the floor or ceiling without leaving clearances that may not be acceptable with respect to fire and acoustic isolation. Rotation may not be feasible at corners, and rotation of door frames may cause distortion which impedes operation of door latches. In internal three-dimensional secondary systems, there are frequently no geometrically satisfactory solutions for accommodating changes in building geometry caused by lateral displacements.

Technical Problems in Detailing

Construction details frequently recommended for seismic design do not always allow for the three-dimensional characteristics of vibratory motions. Anchorages designed to provide resistance to overturning in one axis and allow displacements along another axis attempt to resolve this problem. When considered with respect to orthogonal axes alone, this appears to be a feasible approach, but for conditions where multi-directional motions cannot be translated into motions along axes, this approach may not produce desired results.

Incompatible Approaches to Detailing

It is difficult to coordinate and achieve compatible approaches by all of the professions and technical building crafts to resolve seismic details. In addition, code provisions may require solutions that may not be consistent with one another. The best solution for accommodating inner-story drift of a partition system may be at the level above the suspended ceiling. However, some codes require rigid anchorage of mechanical services to the slab above and to the ceiling below, eliminating this approach.

Cost Premiums for Construction

All buildings must meet requirements of local building codes, but details for seismic performance in excess of codes must be evaluated for their cost relative to benefits. It is difficult to generalize about cost premiums to achieve improved seismic performance. Premiums for improving earthquake response are not as great in buildings with well-configured dynamic structures as for buildings with irregular or discontinuous structures and secondary systems. Also, cost premiums to improve expensive custom-designed components may not be great, but cost premiums to improve less expensive standard manufactured items may be extreme.

- The relative costs for the dynamic structure and enclosure, finish and service systems vary depending upon the nature of the activity, the sophistication of environmental control systems and levels of finish. In general, for finished low-rise to mid-rise buildings, 30 to 40 percent of construction cost is for the dynamic structure including foundations.

- In terms of total construction cost, it is estimated that premiums to bring three to four story buildings of regular configuration to meet 1981 California seismic codes are about 25 percent for an unfinished building where a major percentage of the total cost is for the dynamic structure, and about 10 percent for a finished building where the dynamic structure represents a smaller percentage of the total cost. The cost premium for the same buildings to meet the 1981 California Title 17 seismic code for hospitals is about 15 percent for buildings with comparable features, not including emergency services and other special features required for hospitals.

- Cost premiums to construct a three to four story movement-resisting frame and floor system of regular configuration which meets 1981 California code over non-seismic codes is in the range of 30 percent of the cost of the structure alone. The cost premium to construct a three to four story movement-resisting frame and floor system of regular configuration to meet 1981 California Title 17 seismic code for hospitals over non-seismic code is about 50 percent of the cost of the structure alone. It should be noted that for irregular or discontinuous configurations, premiums could be 100 percent or more.

-To construct a high quality custom-designed metal curtain wall system to achieve a 3/4" floor-to-floor displacement, the cost premium is about five to ten percent of the curtain wall system. To alter a standard pre-fabricated curtain wall to meet the same performance criteria, the cost premium would be 25 to 50 percent of the cost of the wall system.

-To construct suspended ceilings to meet 1981 California seismic code over non-seismic codes, the cost premium is about five percent. To change the installation details of suspended ceiling installation to special seismic details, the cost premium can be as high as 100 percent of the ceiling system.

-To install a panelized type of gypsum partition system with inherently better seismic performance, the cost premium over a standard type of gypsum brace partition is in the range of 15 percent of the partition system.

-To install mechanical and electrical supports to meet 1981 California codes over non-seismic codes, the cost premium is in the range of 100 percent of the cost for supports but the premiums may be only one or two percent of the total cost of the mechanical system.

Cost Premiums for Professional Fees

The design process presented earlier requires professional services in addition to those normally provided in non-seismic regions. Specialists in geology, seismology and soil engineering are required, as well as structural engineers and architects knowledgeable and experienced in dynamic approaches to earthquake design. Additional specialized services are necessary in several steps of the analysis and design process, and to supervise manufacturing and construction to increase the likelihood of built conditions conforming to designs. The total premiums can be 50 to 100 percent greater than professional fees for sites and buildings in non-seismic areas.

Conceptual Problems Affecting Design and Construction

Conceptual problems limit feasibility of translating current understanding of dynamic phenomena into more sophisticated design and construction solutions. Most designers, technicians, construction workers and building inspectors do not have an understanding of the violent motions of earthquakes. Their instincts may be counter-intuitive. In addition, buildings are normally conceived as an assembly of parts or systems, with separate systems for structure, space division, heating, plumbing and other functions. Separate professions are responsible for the integrity of each of the parts. There may be no one responsible for the integrity of the building as a whole. Often there is no one professional on the design team who considers the interaction between all of the parts, or the building as a dynamic whole.

Securing Constructed Work as Designed

Depending upon the degree of professional surveillance of manufacturing and installation during construction, a probability factor may be assigned for the extent to which secondary systems will be constructed as designed. These systems are particularly subject to substitution and change in material and assembly techniques. Construction craftspeople often revise special details designed to improve seismic performance, to follow more conventional construction techniques. Depending upon the extent of professional supervision, these changes can substantially limit control over performance of secondary systems.

CONCLUSION

The evolution of earthquake design shows a history of changes made possible by better understanding of seismic phenomena and new analyses of damage. Building codes and design approaches have been successively modified to account for the severity of the vibrational effects. Coefficients of acceleration have been increased and requirements for anchorages have been added to prevent dislocations. For relatively stiff buildings built with traditional methods and materials, these approaches have generally provided improved performance for secondary systems.

Recently, changes in approach to structural design as well as changes in the scale and methods and materials of construction have caused contemporary buildings to be more flexible than earlier buildings. Damage has been caused by displacement effects inherent in these buildings. This raises the question of whether codes and design approaches adequate for relatively stiff buildings can provide satisfactory performance for secondary systems in relatively flexible buildings.

To improve performance of secondary systems, a design approach considering both vibrational and displacement effects of earthquake response is required. The theoretical basis for such an approach is now established. The site-building-component interaction is sufficiently understood to permit its application in design of secondary systems. However, it is important to recognize that limits of knowledge of these systems, and aspects of the manner in which they are designed and constructed, place realistic limits on the levels of sophistication achievable.

Three different conditions place limits on feasibility. First, there is a lack of test data on the performance of secondary systems under dynamic conditions. The United States-Japan Cooperative Large Scale Testing Program for testing full-scale fabrications is extremely important. More research of this type is needed. Data is needed on the performance of individual secondary systems and assemblies of systems using conventional as well as experimental construction techniques.

Second, there appear to be practical limits on the extent that lateral displacements can be realistically accommodated by secondary systems. The sparse data available suggests that about .025 percent differential displacements between successive floors, in the range of 1/4" to 3/8" depending upon floor-to-floor height, is the damage threshold for many conventional construction systems.

Third, there are technological, geometric, economic and social factors that affect feasibility of technological solutions. Levels of performance above the damage threshold provided by conventional construction techniques will increase cost and will probably not be executed satisfactorily.

One may conclude that a coordinated design process which systematically addresses the full range of vibrational and displacement effects of earthquake response will improve the probability of improved seismic performance by secondary systems. However, because of the difficulties in achieving the design, and to maximize the probability of improved performance, equal emphasis should be placed upon: the design of the dynamic structure to achieve a building response that makes the design of secondary components feasible; and upon the design of secondary systems and their components, to sustain the dynamic conditions imposed by the building response.

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PART I. BUILDING DESIGN

TASK	CONSIDERATIONS	PRODUCT
STEP A. PROGRAMMING: Formulate objectives:		
1. Functional needs	- Desired activities and their space requirements - Mechanical, electrical, plumbing services - Circulation systems - Adaptability to future change	Building performance standards: - Functional program - Human factors program - Seismic-safety program - Fire-safety program - Budget
2. Time and cost needs	- First cost - Maintenance costs - Time schedules	
3. Human factors	- Environmental psychology - Aesthetics	
4. Emergency needs: earthquake, fire, etc.	- Life-safety - Applicable codes - Continued operational use - Costs of repair due to damage from hazard	
STEP B. SITE ANALYSIS		
1. Select location on site	- Building performance standards - Soil and geological characteristics of site - Dependability of utilities, access routes	- Suitable site for development - Locational criteria for site planning
2. Determine seismic design criteria	- Seismological history of region - Soils and geological characteristics of region and of site (proximity to faults, etc.) - Considerations listed under earthquake emergency needs (see STEP A, 4.)	- Seismic design criteria: - design peak ground acceleration - design ground response spectrum or earthquake time history
STEP C. GENERATE BUILDING DESIGN ALTERNATIVES by simultaneously performing tasks 1-3:		
1. Develop conceptual design alternatives	- Building performance standards - Seismic design criteria	- Relationships of spatial needs and circulation, mechanical, electrical, and plumbing systems - Basic systems and materials for DynS and suggested material for finish components and service systems
2. Assign probable dynamic roles to components of each design concept	- Physical compatibility characteristics of building systems and materials: mass, stiffness, configuration, connectivity	- Designation of components as DynS, CpEl, or UncEl
3. Estimate approximate building response for each building concept	- Seismic design criteria - Overall building relationships, systems, materials - Component dynamic roles (DynS, CpEl, or UncEl)	- Probable general building response to seismic loads (flexible or stiff)
4. Identify schemes that are unacceptable under any circumstances	- Building performance standards - Building relationships, systems, materials - Approximate building response	- Alternatives designated as "viable" or "unacceptable"
STEP D. EVOLUTION OF DESIGN CONCEPT For viable alternatives, cycle through the following steps:		
1. Develop more specific schemes for conceptual design alternatives	- Building performance standards - Seismic design criteria - General building response	- General building design criteria: plan dimensions and story heights; materials, weight, etc. of components
2. Project more specific pattern of building response to seismic loads	- Seismic design criteria - General building design criteria	- Specific pattern of building response: relative displacement and vibrational effects
3. Evaluate alternative design concepts	- Building performance standards - Seismic design criteria - Building design criteria - Specific pattern of building response	- Selection of final design concept OR - Determination to proceed through design development again, modifying concepts to improve their viability

PART II. COMPONENT DESIGN

TASK	CONSIDERATIONS	PRODUCT
STEP A. FORMULATE DESIGN STRATEGY		
1. Formulate objectives for component		- Component performance standards
a. Functional needs	- Functional space and circulation relationships - Location in building configuration - Interface with other building components - Adaptability to future change	
b. Time and cost needs	- First cost - Maintenance costs - Time schedules	
c. Social needs	- Aesthetics - Environmental psychology	
d. Emergency needs	- Life-safety - Applicable codes - Continued operational use - Costs of repair due to damage from hazard	
STEP B. ANALYSIS OF THE DYNAMIC ENVIRONMENT		
1. Determine Dynamic Environment with aid of structural engineer	- General building response - Potential interaction of component with other components - Location in structure	- Dynamic Environment: relative displacement and vibrational effects
2. Determine design criteria for component	- Dynamic Environment - Component performance standards - Selected design approach	- Dynamic Environment design criteria
STEP C. GENERATE COMPONENT DESIGN ALTERNATIVES		
1. Develop conceptual design alternatives	- Dynamic Environment design criteria - Component performance standards	- Component material system and configuration
2. Assign dynamic roles to components	- Physical properties: mass, stiffness, strength, configuration, interface conditions - Design approach	- Designation of each alternative component design as CpEl or UncEl
3. Determine response of each component alternative	- Dynamic Environment design criteria - Dynamic role of component	- General component response and interaction with other components
4. Identify solutions that are unacceptable under any circumstances	- Component performance standards - General component response and its effect on adjacent components	- Component design concepts designated as "viable" or "unacceptable"
STEP D. EVOLUTION OF COMPONENT DESIGN CONCEPT		
1. Develop more specific design of component	- Component performance standards - Dynamic Environment design criteria - General component response and interaction with other components	- Specific component design criteria: size, material, weight, structural support system, aesthetics, and so forth
2. Project specific pattern of component response to seismic loads	- Dynamic Environment design criteria - Specific component design criteria	- Specific pattern of component response
3. Evaluate alternative design concepts	- Component performance standards - Interaction with other components - Dynamic Environment design criteria - Specific pattern of component response	- Select component design or cycle through STEP D, 1-3 again
STEP E. FINAL DETAILING OF COMPONENT		
1. Develop details of selected component concept	- Concept chosen - Compatibility with adjacent components	- Modified design detailed in working drawings, specifications
2. Preparation and review of shop drawings	- Modified design - Contractor's capabilities - Costs	- Shop drawings of design approved by architect
STEP F. CONFIRM THAT FABRICATION AND CONSTRUCTION MEET DESIGN CRITERIA		

Building Response and Component Design: An Enclosure Wall Case Study

The effect of the Dynamic Model on the design process can be demonstrated best by an analysis of its use in the design and construction of a building. During research conducted under an earlier National Science Foundation grant, McCue Boone Tomsick's (MBT) research team had the unique opportunity of working with a design team which could test the theory's usefulness in the design of an enclosure wall system. At that time, MBT was engaged in a commission for the IBM Corporation to design a building complex of 600,000 square feet which would accommodate over two thousand people. Although specific aspects of the Dynamic Model had not yet been developed, the basic concepts had been formulated, making it possible for the design team to utilize them in the design of the IBM facility. The case study that follows summarizes important steps in the application of the Dynamic Model to the overall design of the building, and then presents the design of the enclosure wall as an Uncoupled Element for its particular Dynamic Environment.

The subsequent analysis follows the major steps of the design process as outlined in Chapter Two, beginning with programming considerations, proceeding through site analysis, overall building concepts, and concluding with the design of the enclosure wall system as one example of the use of the Dynamic Model for building component design. In order to present a clear and logical description of the design process, each step appears sequentially even though some retracing of steps occurred, as is typical in design projects. This case study demonstrates that various decisions made early in the design process can have a significant effect not only on the response of the building as a whole, but also on the design of building components for the seismic criteria of their Dynamic Environments.

PROGRAMMING

Program requirements for the IBM facility called for the provision of almost two thousand individual offices, a large computer center, a library, classrooms, and a food service facility. In addition, the office space had to be able to accommodate specific uses at initial occupancy, but remain adaptable for anticipated future changes in the client's functional requirements at the site. Good communication and direct access between key functions were the major requirements for circulation.

In terms of earthquake considerations, life-safety was the high priority, with the importance of continued operation and protection of capital investment serving as major secondary criteria. Of critical importance in terms of operation and investment was the computer center, which would be very sensitive to differential settlements and lateral movements Very early in the design process, therefore, the architects and engineers were aware that a sophisticated blend of design and seismic considerations would be required to provide the design quality, life-safety, and operational and financial protection desired by the owner.

The design would require a sophisticated blend of design and seismic criteria.

SELECTION OF SITE LOCATION

As the first step in the site analysis procedure, MBT conducted a study of the feasibility of development on the site which IBM tentatively selected. The site is located in the seismically active Northern California region, approximately ten miles northeast of the nearest trace of the San Andreas fault, and six miles southwest of the Calaveras fault (Figure 3-1).

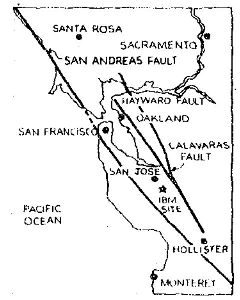


FIGURE 3-1. SITE LOCATION RELATIVE TO MAJOR FAULTS

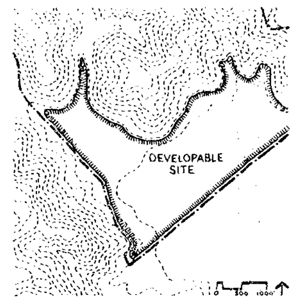
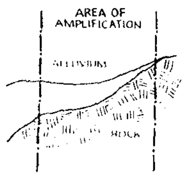
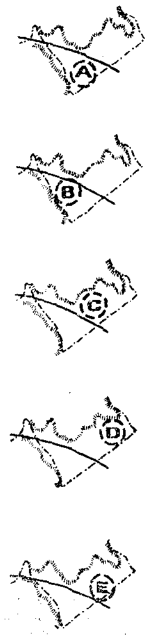


FIGURE 3-2. DEVELOPABLE SITE.

Lowney-Kaldveer Associates, the soils engineers, were informed of the intended use of the site and the likelihood of low- to medium - rise building development. They performed a preliminary soils investigation which, in combination with program requirements, determined preliminary design criteria for building siting, earthwork, and foundations. In addition to these criteria, the investigation revealed the following conditions requiring special design consideration:



- an upper, three to ten foot thick layer of moderately expansive silty clay which would require special treatment to prevent heave and resulting damage to structures.
- a relatively high water table which, under hydrostatic pressure, would cause construction problems during excavation for deep foundations;
- likely amplification of seismic waves in the area of transition between alluvial plain and hills where rock is close to the surface; thus, buildings should not be located adjacent to the base of the hills.



In the portion of the site that was otherwise desirable for development (Figure 3-2), a fault location study identified a fault passing beneath the alluvium. The location of the fault was determined by magnetometer and seismic refraction surveys, and the width of the fault trace was measured to be forty feet. The fault was considered potentially active, based on geological formations and indication of activity along other portions of the same fault. The soils engineers recommended a fifty-foot offset on either side beyond the forty-foot wide fault trace, resulting in a 140 foot wide restricted zone. The land within this restricted area did not meet the engineer's design criteria for location of structures; therefore, greenbelt, parking, recreation and roads were recommended as alternative uses. Taking into acount the configuration of the site and the location of the fault, the architects considered alternative site plans which would accommodate the desired relationships between building and parking. Since the fault trace roughly bisects the buildable site area, alternative locations fell to one side of the fault or the other. Alternative C (which placed the project to the east of the fault zone and just south of the hills) was recommended because it allowed a large buildable area, maximum advantages under zoned height requirements, efficient road access, a substantial, landscaped buffer zone, and accommodation of storm drainage requirements.

DETERMINATION OF GROUND RESPONSE DATA

Once the site was determined to be feasible for development, a seismic response analysis was performed to determine the probable characteristics of ground input motion for the chosen location on the site. The method the soils engineers utilized involved:

1) establishing an idealized soil profile at the site from test borings and laboratory test results;
2) selecting design earthquakes for the site by predicting the magnitude and dynamic characteristics of possible future earthquakes;
3) modifying available records of bedrock motions so that the ground surface accelerations could be determined;
4) analyzing the dynamic response of the soil deposit to the anticipated bedrock motions to determine ground surface accelerations.

An idealized soil profile was developed using data from the borings of preliminary and final soils investigations. Then seven earthquakes of various magnitudes and origins were selected by Lowney-Kaldveer for study for the IBM site. Their magnitudes varied from 5.25 to 8.25 on the Richter Scale, and their probability of occurrence ranged from 25-50 years to 500 years up, with the smaller magnitudes having the more frequent occurrence intervals. For each of the seven earthquakes a source accelerogram was chosen. Because accelerograms of the magnitude and location needed were not available (as is often the case), appropriate existing records were adjusted for magnitude and source. Since there was no existing record which corresponded to the largest of the seven earthquakes (8.25 on the San Andreas fault), a synthetic source accelerogram developed at the University of California at Berkeley was used to simulate an earthquake of such magnitude.

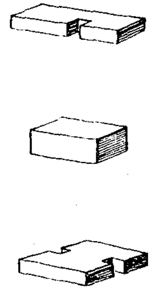
Once the source accelerograms were chosen, they were then adjusted for the distance of the site from the epicenters of the seven earthquakes. This procedure resulted in bedrock accelerograms for the site of the building. Based upon these accelerograms and the soil profile, ground motion accelerograms for the surface of the site were developed using a computer program (SHAKE) which takes into account the dynamic characteristics of

Amplification of bedrock motions would be likely due to the stiff nature of the soil materials and their limited thickness.

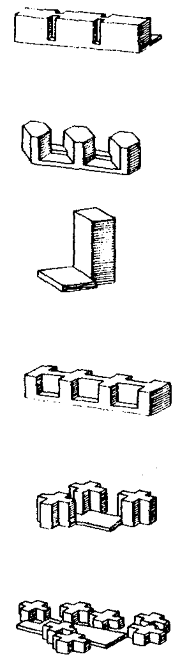
the soil materials overlying the bedrock. The natural period of the soil deposit was low, ranging from approximately 0.5 to 0.8 seconds for the seven earthquakes studied. Bedrock and ground surface acceleration were high due to the site's location close to major faults, a characteristic of most of the surrounding area which could not be avoided in site selection. Amplification of the bedrock motion was found in all seven earthquakes studied, primarily because of the very stiff nature of the soil materials and their limited thickness. The ground motion accelerograms were then used to construct response spectra for each of the seven earthquakes for damping values of two, five, and ten percent. The response spectra would then serve as the basis for design, since they record the maximum acceleration to be used for a building of known period of response.

BUILDING DESIGN ALTERNATIVES

While the ground response data were being prepared, conceptual building designs were also being developed. Then each concept was evaluated on the basis of its functional program (space requirements, services, circulation, and adaptability to future change), construction and maintenance costs, aesthetic and psychological factors, and approximate seismic response. Nine different concepts which represented the prototypical solutions to the problem were considered (See marginal diagrams). Three of these concepts, 1A, 3A, and 7C were selected for further study. Several options for the structural system were considered for each of the three schemes: steel moment frame, steel braced frame, concrete shear wall, ductile concrete moment frame, and ductile concrete tube frame. Most of the concrete systems were eliminated for the following reasons:



- a concrete ductile moment resisting frame would result in a higher cost for the structural system than steel for all schemes;
- concrete shear walls in the building cores were considered unfeasible for eight-story schemes because they would generate very large overturning moments;



- concrete tube frame solutions were considered unfeasible because they would interfere with program requirements, especially the large continuous space at the ground floor.

On the basis of these considerations, steel framing was recommended with the exception of scheme 7C, which at this point also appeared feasible with a concrete system incorporating shear walls only at the central cores.

From the three alternatives, schemes 3A and 7C were selected for more detailed analysis. Scheme 3A (Figure 3-3) was a rectangular-plan, three-story concept which achieved the desired qualities of adaptable space and a simple and economic structure. Scheme 7C (Figure 3-4) consisted of a large ground floor of continuous adaptable space with a series of eight, three-story, cruciform buildings which provided windows for over sixty percent of the individual offices on those floors, satisfying the owner's desire to maximize views of the exterior environment. Spatial configurations were further developed, tentative material and structural systems were examined, and detailed cost comparisons were made for each scheme with its possible structural systems.

At this stage in the design process, the dynamic role of other building components was considered. Each building concept was examined to determine which components might logically become part of the Dynamic Structure, which would be Coupled Elements, and which Uncoupled Elements. These dynamic role designations were used as a basis for preliminary estimates of building response and to assist in the projection of comparative construction costs.

The enclosure wall was determined to be unsuitable for incorporation into the Dynamic Structure because it did not extend to the ground at all facades, but stopped at the roof of the first floor, a discontinuity which ruled out the possibility of the wall increasing the structure's ability to sustain seismic loads. Considering the dynamic role analysis and the functional and aesthetic aspects of the two schemes, the owner and the design team agreed to proceed with scheme 7C. This scheme

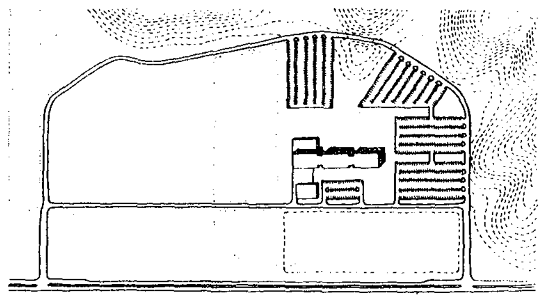


FIGURE 3-3. SCHEME 3-A/SITE PLAN

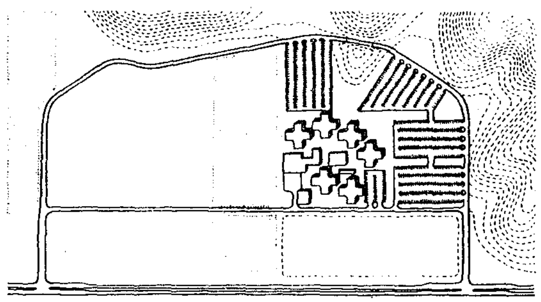


FIGURE 3-4. SCHEME 7C/SITE PLAN

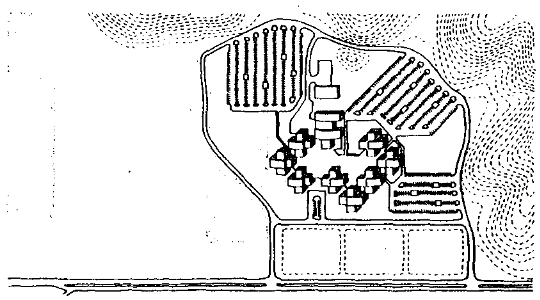


FIGURE 3-5. SCHEME 7E/SITE PLAN.

then underwent several phases of additional refinement with minor variations in plan form and story heights, and was *finally approved* under the designation 7E, as illustrated in Figure 3-5.

DESIGN DEVELOPMENT AND STRUCTURAL CONCEPTS

During design development of scheme 7E, both steel and concrete structural systems were studied in more detail to determine the most appropriate roles of various building components. The systems were then evaluated for their compatibility with the functional program, construction scheduling and cost considerations, aesthetic design quality, and response to seismic loads. Two structural alternatives were determined to be most viable:

- ductile steel moment resisting frame;
- concrete shear wall utilizing the cores of the cruciform-shaped buildings.

The concrete system had the advantages of no delay in the start of construction as would be required for the rolling schedule and delivery of steel, and no requirement for expansion joints between the plaza framing system and the retaining walls. But the steel moment frame was selected for the following reasons:

A ductile steel moment resisting frame would make it possible to avoid coincidence of ground and building periods.

- The response spectra developed generally peaked at .4 to .6 seconds, meaning that the structural system should have the relatively longer periods possible with ductile steel moment resisting frame structure but not with a concrete one. Otherwise coincidence of ground and building period would occur, causing resonance.
- Because of the weight of concrete, design seismic forces for concrete would be four times those for steel.
- Concrete has less reserve energy capacity than steel.
- The scheme 7E plan configuration made it difficult to resolve diaphragm shear forces in concrete.
- The weight of the concrete would require a more ex-

pensive and complex foundation system, whereas the steel moment frame could be supported on conventional, spread footings.

- A standard two-way waffle slab system would not be feasible for the floor plan configuration, and other concrete framing systems would be too costly.
- Interior space planning options in the computer center would be severely restricted due to the shear walls in the three-story buildings above the computer center.
- Shear walls would interfere with the mechanical distribution system since they would be located at the cores where the greatest concentration of mechanical systems would also occur.

Concrete shear walls would interfere with space planning and mechanical system layout.

PROJECTION OF BUILDING RESPONSE

Once the basic steel moment frame concept was established, Forell/Elsesser Engineers performed a preliminary structural analysis for the building, using hand calculations and in-house computer runs. These calculations established preliminary member sizes and stresses, as well as interstory drift and the fundamental period of the building. The engineers then used the building's period, calculated to be 1.0 seconds, in conjunction with the response spectra developed earlier, to determine the building's probable response. In the case of scheme 7E, the first mode of vibration was dominant because the buildings were relatively short (four stories) and thus modal superimposition did not lead to higher stress levels. The maximum acceleration computed from the response spectra substantially exceeded existing recommended code forces because of the large amplification of base rock motion due to the nature of the overlying soils. But since structures behave inelastically during major earthquakes, the response spectrum that assumes elastic behavior is incorrect, and yields maximum accelerations higher than actually occur for these quakes. As the structure goes into inelastic behavior, the damping values increase and the period lengthens, which in the case of the IBM building would decrease the seismic response. Hence, the structural engineers took this reduction into account when designing for the severest probable earthquake and the severest possible earthquake. They performed a three-dimensional computer analysis on mathematical models of the buildings. Damping, ductility, and reserve energy of the structure were taken into consideration. The response spectra for the two major earthquakes considered most probable were smoothed, and a ductility factor $\mu = 4$ and damping of 5% were used in order to determine the appropriate design level, which was found to be 15% g. For the building's period of 1.0 seconds, this level was considerably higher than the 1976 Uniform Building Code requirements of 7% g and the Applied Technology Council's tentative recommendations of about 11% g, and was nearly as great as the California Hospital's Title 17 requirement of almost 16% g (Figure 3-6). Such a high lateral force coefficient was warranted because the site had the potential for large amplification of seismic waves due to its sloping rock-alluvium soils condition (described earlier) and the fault which passed through the site. For the largest credible earthquake, a ductility factor of $\mu = 8$ and 10% damping were used. The large ductility capacity of the steel moment building was judged to be able to sustain the structure against collapse in the event of the largest credible earthquake.

For scheme 7E the first mode of vibration was dominant.

The design level was 15% g.

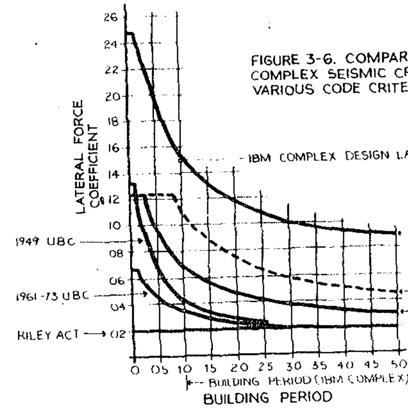


FIGURE 3-6. COMPARISON OF IBM COMPLEX SEISMIC CRITERIA WITH VARIOUS CODE CRITERIA.

The campus-like plan led to some seismic engineering complexities.

The owner's choice of a scheme incorporating a campus-like complex rather than a monolithic block-like structure led to a complex relationship of building shapes, which presented some difficulties from a seismic engineering point of view. The design consisted of a series of four-story buildings linked to a one-story computer facility in an arrangement which, becuase of its projecting wings, re-entrant corners, and asymmetrical plan, might, during an earthquake, cause parts of the building to vibrate out-of-phase, resulting in torsional forces and stress concentrations. To overcome these potential problems, the building complex was broken into smaller, less complex elements by the use of seismic expansion joints. The result was four biaxially symmetrical buildings of four stories, and three different building types with one axis of symmetry (Figure 3-7). In the larger buildings, the column stiffnesses at the lower floor were carefully adjusted to make the seismic resistance of the large first floor compatible with the smaller individual buildings above. Rotational torsion in the unsymmetrical direction was also accounted for in the design.

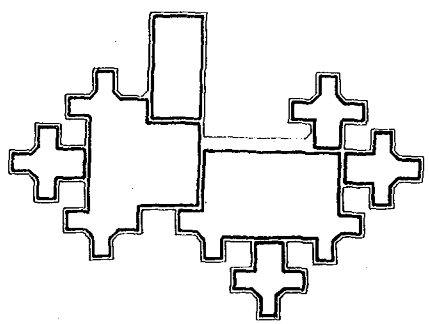
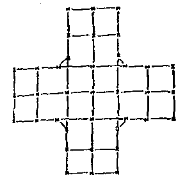


FIGURE 3-7. SECTIONS OF BUILDING COMPLEX DYNAMICALLY SEPARATED BY SEISMIC EXPANSION JOINTS.

For aesthetic reasons, the architects wanted the framing system to utilize a cantilever system at the ends of the bays of each cruciform building. Such framing would have allowed a continuous band of glass uninterrupted by the column around the outer faces of the cruciform buildings (See marginal diagram). After preliminary analysis, however, this scheme was abandoned in favor of the selected framing scheme, because of significant additional costs and the potentially larger deflections under seismic loads resulting from the less efficient lateral resisting system.



FINAL LATERAL ANALYSIS

The final lateral analysis to determine building response was made using a computer program, XTABS, in which the building is idealized by a system of independent frames interconnected by floor diaphragms which are rigid in their own plane. The program is three-dimensional in the sense that it computes translations at each floor in both axes, as well as rotation about the vertical axis. In addition, at each column, the vertical displacement and rotation is computed. Input data for the analysis consisted of the following:

- building member description: the moment of inertia, shear area, dimensions, and modulus of elasticity of each building member;
- building member location: the location of each member within the building in terms of x, y, and z coordinates from a basic reference point;
- loads: dead loads and live loads as well as the 15% g lateral load.

Building response was used to determine the Dynamic Environment for Coupled and Uncoupled Elements.

Given the above data, the XTABS computer run determined bending and axial forces for each member, deflections in terms of displacements from the vertical axis at all floors, and mode shapes. The highest deflection occurred for the dead plus live plus seismic loading condition, with a maximum interstory drift of about .75 inches. This loading situation also applied to all other frames at all other floors in the various building types. The building response, as determined by the data from the XTABS program, was used, in turn, to determine the Dynamic Environment which provided the design criteria for Coupled and Uncoupled Elements, including the enclosure wall system of this study.

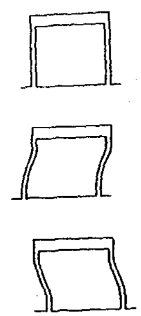
DESIGN STRATEGY FOR THE ENCLOSURE WALL

The owner's objectives for the design of the exterior wall were that it be designed for an optimum balance of aesthetics, cost, and function, and that the materials used be of the type requiring little or no maintenance. Additional requirements developed by the design team expressly for seismic considerations were the importance of continued operation of the building, the protection of the contents from damage, and the minimization of replacement costs of the wall in the case of moderate to severe earthquakes. Based upon these general requirements, more specific seismic design requirements were developed as follows:

- The enclosure wall must respond to the Dynamic Environment imposed by the Dynamic Structure during an earthquake of moderate intensity in a manner such that almost no damage would occur.
- For the severest probable earthquake, the enclosure wall must accommodate interstory drift of the Dynamic Structure without a significant amount of anchorage, framing, or panel failure, with a low probability of major glass breakage, and with only a minor amount of deformation-caused leaking. In addition, the curtain wall must be designed to avoid the possibility of its damaging the Dynamic Structure.
- For the severest possible earthquake, the enclosure wall should respond to the input motions of the Dynamic Structure such that damage to it would be minimized in order to provide a high standard of life-safety for the occupants.

In this case only the Dynamic Structure should transfer motion to the enclosure wall.

Having established these requirements, it was then necessary to determine the specific Dynamic Environment for the enclosure wall. In this case, the design team determined that only the Dynamic Structure should transfer motion to the enclosure wall. The ceiling and partition systems were to have the capability of being relocated from time to time, making it impossible to determine their influence on the response of the wall, necessitating their separation from it, and hence removing them from the Dynamic Environment for the enclosure wall. The



magnitude of the building response was determined for the various frames, one through seven, in each of the x and y directions, at each floor level. The wall was to be designed such that its connections would sustain the highest acceleration determined by the computer analysis for the given design earthquakes. In addition, the largest interstory displacement or "drift" in any one story and frame location was used as a relative displacement criterion for the design of the entire enclosure wall system. From the XTABS output, the largest drift was found to be somewhat less than 3/4". Thus, the wall was to be designed to accommodate a plus or minus 3/4" drift between floors in any horizontal direction from the at-rest position without interference with the other design objectives, which had been developed as follows:

Thermal transmission: Overall "U" value should be .4 or less and the overall shading coefficient .35 or less.

Thermal movement: Within the audible range there must be essentially noiseless contraction and expansion, both vertically and horizontally, of component materials for a temperature range of 20° F. to 180° F. without buckling, opening of joints, glass breakage, or undue stress on fasteners.

Wind pressure: The wall must be designed for both flexural and torsional stress for the following positive and negative wind pressures acting perpendicular to all planes of the curtain wall/cladding elements: less than 30' - 15 psf; 30' to 49' - 20 psf; 50' to 99' - 25 psf.

Air Infiltration: Tested in accordance with NAAMM Standards, air infiltration should not exceed 0.06 cubic feet per minute per square foot of fixed unit area.

Light transmittance: Transmittance should not be less than 20%; shading coefficient with interior blinds should not be less than 0.30.

Water infiltration: Essentially no water penetration, that is, the appearance of uncontrolled water, should occur during NAAMM Standard tests.

Self-drainage: The wall must be designed to drain to the exterior any water entering at joints or glazing reveals and any condensation occurring within the unit's construction.

EVALUATION OF ALTERNATIVE ENCLOSURE SYSTEMS

At first it was uncertain whether the enclosure wall would be designed as a Coupled or an Uncoupled Element.

During the conceptual design phase, the design team determined that the enclosure system could not be incorporated in the Dynamic Structure, but it was uncertain whether the wall would be designed as a Coupled or an Uncoupled Element. Once the conceptual design scheme was chosen and the Dynamic Environment for the wall determined, the design team then decided to examine all feasible enclosure systems, both those which would be heavy and hence Coupled Elements, and those which would be light and hence Uncoupled Elements.

Two different concrete systems were studied. The first system consisted of prefabricated concrete panels anchored directly to the Dynamic Structure and spanning the full story height (Figure 3-8). This scheme was considered to be somewhat undesirable because the heavy mullions necessary to give the panels structural integrity would conflict with preliminary facade studies which had indicated that a continuous glazing system would be the most aesthetically pleasing. The second concrete system considered was composed of precast concrete wall spandrels between columns braced by a secondary steel subframe. In this case the windows would be treated as horizontal bands interrupted only by the precast column covers (Figure 3-9).

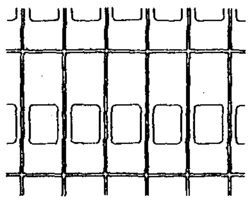


FIGURE 3-8. FULL BAY WIDTH, FULL STORY HEIGHT PANELS.

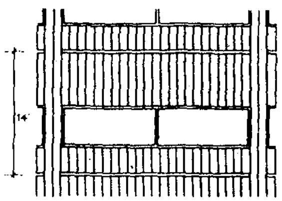


FIGURE 3-9. FULL BAY WIDTH, PARTIAL STORY HEIGHT PANELS.

The weight of the concrete panels would cause two major problems.

Both concrete systems were eventually abandoned in favor of a metal system for several reasons. The difficulty of detailing concrete systems to accommodate the interstory drift of the structural frame was a primary consideration, because conventional joints between panels would not accommodate the large potential drift of the Dynamic Structure, and because shiplap type joints would be prohibitively expensive. In addition to detailing problems posed by interstory drift, the heavy weight of concrete panels, as opposed to metal panels, would increase the force level for which the building must be designed; since the design force levels were already high because of geological site conditions, designing the structure for even higher force levels would be extremely difficult and much more costly. The second disadvantage of the concrete panels' greater weight was that heavy equipment would be required for installation, an expensive venture and one likely to cause problems on account of the large number of small courtyards in which the panels would be installed, where large and heavy equipment would be virtually impossible to use. Thus, metal panels, which would be Uncoupled Elements and therefore have little effect on the performance of the structural system, were chosen.

Metal panels were chosen and would be Uncoupled Elements.

The design team began examining . . . subframes for attaching the panels to the structure.

Having chosen metal panels, which would be relatively lightweight, the design team examined various means of anchoring the metal panels to the structural system such that the large interstory drift could be accommodated. The design team began to examine metal enclosure system alternatives utilizing a subframe system, a common practice in curtain wall construction. A subframe system allows the use of smaller panels, thus reducing the difficulty and expense of detailing. The subframe serves as the structural support for the enclosure panels, and the structural system serves as the support for the subframe system. The various connections provide the capability for movement necessary for vertical and horizontal deflections.

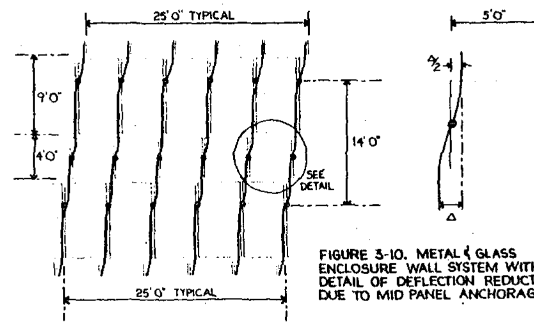


FIGURE 3-10. METAL & GLASS ENCLOSURE WALL SYSTEM WITH DETAIL OF DEFLECTION REDUCTION DUE TO MID PANEL ANCHORAGE.

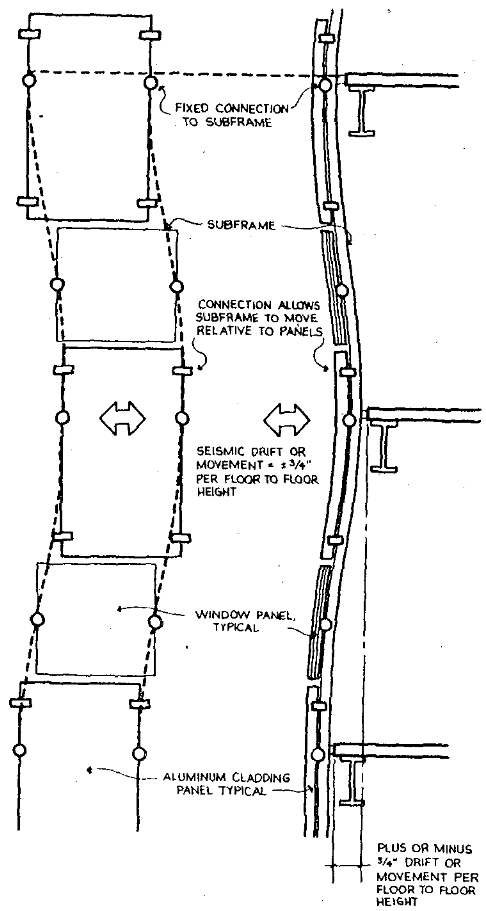


FIGURE 3-11. SCHEMATIC DETAILS SHOWING SEISMIC MOVEMENT OF METAL AND GLASS PANEL SYSTEM AS DESIGNED BY ARCHITECT AND ENCLOSURE WALL CONSULTANT.

The subframe system for anchoring panels was chosen essentially to make possible both the desired aesthetic treatment of the glass pattern and the accommodation of the relatively large interstory drift of ± 3/4 inch per story. While a subframe system is a commonly used method of supporting enclosure walls, the design of the anchorage of the panels to the subframe, and that of the subframe to the structure were, in the case of the IBM complex, unique. To the subframe system, a skin was anchored which consisted of single panels of metal and metal frame holding glass. Anchorage of each panel to the subframe was from its midpoint, thus reducing the amount of deflection that must be accommodated in any one direction by one-half (Figure 3-10). Anchors at midpoints of panels attached the panels to the subframe, while anchors at top and bottom permitted the subframe to move with respect to the panels up to ± 3/4 inch per story when necessary for the dynamic movement of the structure (Figure 3-11). In addition to the allowance for horizontal seismic drift, allowance for 1/2 inch of floor slab deflection was also designed in order to accommodate beam/building deflections. Building deflection (vertical) was accommodated in the cladding to window horizontal expansion joint. The following list presents the detailed design criteria established by the design team with the assistance of E. O. Tofflemire and Associates, the enclosure wall consultant:

Seismic drift:

The curtain wall was to be constructed to allow for a plus or minus 3/4" drift or movement in any horizontal direction between floors, as follows:

- Normal to the plane of the curtain wall:
  plus 3/4" (in),
  minus 3/4" (out);
- Parallel to the plane of the curtain wall:
  3/4" to the right,
  3/4" to the left.

Horizontal force factor for elements of structures - 'C_p' (UBC Table 23-J):

- Exterior Wall, C_p = 0.20 normal to flat surface,
- Cantilever Wall, C_p = 1.00 normal to flat surface,
- Connection, C_p = 2.00 in any direction

Wind pressures:

- 0 to 30' = 15 psf,
- 30 to 50' = 20 psf,
- 50 to 100' = 25 psf.

Deflections:

- The deflections of any metal framing member in a direction normal to the plane of the wall should not exceed 1/240 of its clear span or 3/4", whichever is less;
- The maximum deflection of any section in the plane of the glass should not exceed 1/8";
- The deflection of any horizontal member supporting glass, when carrying its full design dead load, should not exceed 1/360 of the clear span of the member or 1/8", whichever is less;
- The deflection of any member in a direction parallel to the plane of the wall, when carrying its full design load, should not exceed 75% of the design clearance dimension between the member and the top of the panel, sash, glass, etc.

Anchorage and support of curtain wall elements:

- Points of support for the curtain wall were to be braced in the three orthogonal directions to resist loads from any direction, including positive and negative wind pressures, seismic forces, etc.;
- Curtain wall elements and their applicable anchorage assemblies should be designed to accommodate thermal, seismic, and building movements without harmful effect to the curtain walls, including glass, glazing, and sealant.

Sealants:

- Sealants should be installed such that there is no adhesive or cohesive failure of joints.

Visual criteria:

As an extension of the design philosophy of the overall project, the architects set the following visual criteria:

- Because the building form is highly articulated, the enclosure wall should be relatively smooth and present a flush appearance;
- The wall should look like lightweight material, not like painted structural walls;
- The enclosure wall should honestly express the interruption of the space modules with the structural columns;
- Panels should look continuous with only a subtle indication of joints to express the means of fabrication, and without demarcations or shadows which would disjoint the wall into conspicuously separate pieces.

DETAILED DESIGN OF THE ENCLOSURE WALL

The architects and E. O. Tofflemire and Associates designed the wall as an Uncoupled Element based upon the design criteria previously outlined. The lightweight enclosure system developed in the wall's conceptual design stages was initially composed of steel mullions supporting porcelain enamel steel panels and glass panels.

The combination of flush and smooth glass and metal surfaces was consistent with the design concept . . . of a "skin" only, an Uncoupled Element.

As the design evolved, the steel panels were changed to aluminum. The weight of the enclosure system, averaging glass, aluminum panels, the mullion system, and other miscellaneous materials, was about 4.3 pounds per square foot, making the system relatively lightweight and therefore an Uncoupled Element as previsouly assumed.

The curtain wall design incorporated separate glass and aluminum panels anchored to the mullion system, which was, in turn, anchored to the Dynamic Structure. Support mullions were located 5'-0" on center, glass panels were 5' x 5', and aluminum panels 5' x 9'. Glass panels were 1/4" thick, and aluminum panels were 3/16" thick. Because of the visual criterion of a smooth-appearing wall, the glass panels were detailed to be as nearly flush with the aluminum panels as possible. This detail was accomplished by the use of 1/4" thick perimeter butt glazed glass panels with no exterior stops. This type of glass installation was feasible because the glass panels were of relatively small size. In some areas of the complex, such as the cafeteria area and interlinking corridors between adjacent cruciform buildings, the larger size of the glass panels necessitated a different glazing system which used stops. In addition to the flush detailing of the windows, the aluminum panels were designed with smooth natural finishes, and joints between panels were designed as subtle reveals to express the means of fabrication, while still maintaining the continuous look of the wall surface. Furthermore, the combination of flush and smooth glass and metal surfaces was visually consistent with the design concept of the enclosure system as a "skin" only, an Uncoupled Element, rather than a coupled or "structural" one. Stiffeners were added to maintain panel flatness within 1/8" out of plane in 5'-0". To maintain the integrity of the lightweight enclosure system at the third and fourth floor interlinking corridors, one inch movement tolerance joints were provided at either end of each corridor's enclosure wall. Finally, for the most severe seismic conditions, the design had to take into account the interaction between the two perpendicular walls at typical corner situations. To avoid the possibility of potentially high amounts of stress at the corners, special 45° corner panels were designed to disengage from the support mullion system under severe loadings, allowing room for movement of the remainder of the cladding system, thus preventing extensive damage. A typical horizontal section of the wall at the 45° corner panels is shown in Figure 3-12.

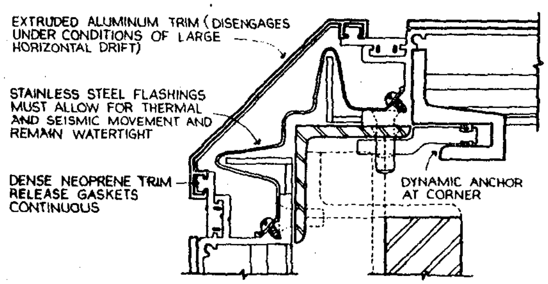


FIGURE 3-12. HORIZONTAL SECTION OF TYPICAL 45° CORNER PANELS.

By the time the design of the enclosure wall was completed and ready for bids, the materials systems chosen to meet the functional, structural, and aesthethic design criteria were the following:

Exterior cladding: Anodized aluminum sheet with clear or special color coatings.

Exposed and internal curtain wall sections: Anodized aluminum extrusions.

Flashings: Aluminum or stainless steel, as required, with mill finish.

Anchors and related structural components: Anodized aluminum extrusions, or steel with protective paint coating, as required.

Sealants:

- For glass to metal butt glazed joints: Silicone sealant compounded for an acetic acid cure.
- Secondary glass to metal and metal to metal: Silicone sealant.
- Exposed metal to metal, perimeter metal to concrete: Two part polysulfide base sealant.
- Concealed metal to metal and metal to concrete: Non-drying, non-skinning synthetic butyl rubber sealant.
- Joint fillers and back-up materials: Selected in accordance with written recommendations from the applicable sealant manufacturers for each specific application. Factors considered included shape, size, hardness, compatibility and bond breaking requirements.

SUBCONTRACTOR'S FINAL DESIGN AND DETAILING

The design of the curtain wall illustrated in the contract documents was carried out in a manner consistent with the requirements for an Uncoupled Element. The next step was the selection of a subcontractor who could manufacture and install the wall at an economical price, and yet satisfy the given objectives and requirements. The subcontractor was offered the option of modifying or adding details, subject to the architect's approval, as long as the visual and performance requirements were fulfilled.

Five companies with the capability of fabricating and installing this type of curtain wall system submitted bids. The Cupples Products Division of the H.H. Robertson Company was low bidder with a proposal based upon MBT-approved modifications and thus was awarded the subcontract.

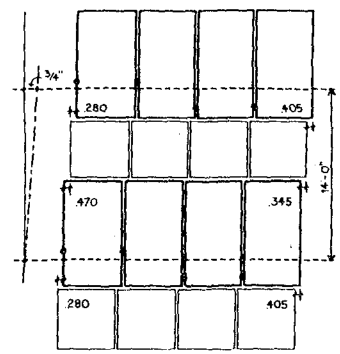


FIGURE 3-13. ELEVATION SHOWING LOCATION OF CURTAIN WALL CONNECTIONS AS DESIGNED BY SUBCONTRACTOR.

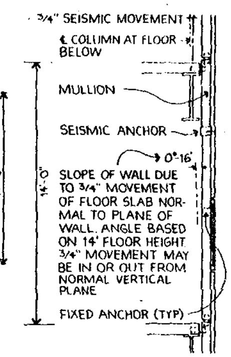


FIGURE 3-14. SECTION SHOWING SAME AS FIGURE 3-13.

Cupples Products' design group proposed changes in detailing, fabrications, and installation, but not in the concept of the curtain wall. Substantial modifications were made to the anchoring system to make the wall more efficient. The design modifications proposed in the shop drawings changed the location of the seismic anchors as shown in Figures 3-13 and 3-14. The final design still allowed 3/4" movement in and out from the plane of the wall, as well as 3/4" in either direction in the plane of the wall.

BUILDING AND TESTING OF A MOCK-UP UNIT OF THE CURTAIN WALL

Construction and installation of the test unit were performed as they would be at the actual site.

The architect's contract documents required that, upon approval of the shop drawings, the subcontractor build a full-scale mock-up of a section of the curtain wall in order to test its ability to meet the seismic and other design criteria that had been established. A full-size test unit provided a means of conducting both a visual evaluation and a performance testing of the wall. Construction and installation of the test unit were performed in the manner proposed for the completed structure and included all components, such as glass, sealants, anchor assemblies, and so forth. The structural steel frame, to which the curtain wall test unit was anchored, also simulated the actual structural frame to be constructed.

Visual review and approval was based on the quality standards outlined in the performance specification and included finish match and uniformity, joinery, tolerances, seals, flatness in smooth-faced surfaces, and so forth. Acceptable performance required approval of both the testing procedures and the resulting data submitted in a certified report.

The curtain wall mock-up unit was tested by the A. A. Sakhnovsky Construction Research Laboratory at Cupples Products' facilities in St. Louis, Missouri. The test procedures were in accordance with the testing methods and procedures described in the National Association of Architectural Metal Manufacturers (NAAMM) Standard TM-1-68T, "Methods of Test for Metal Curtain Wall." A test chamber was constructed on the interior side of the test wall with observation ports permitting examination of the interior surfaces and joints of the test assembly during the actual test periods. The interior of the air chamber was maintained at a uniform temperature of 70° F. with a relative humidity of 40%; the air pressure was varied as required for the test procedures.

The test unit consisted of a group of four aluminum frame curtain wall elevations (A,B,C, and D) constructed in the relative configurations which they would assume in the actual building complex (Figure 3-15). The mock-up was two full stories plus one spandrel plus the coping in height. Elevations A and B consisted of a vertical tubular mullion system with applied 1/8" thick aluminum spandrel panels plus separate "floating" glazing frames for single 1/4" thick perimeter butt-glazed lights; no exterior stops were utilized. Elevation C consisted of a similar mullion and panel system, but incorporated separate "floating" glazing frames for pairs of 1/4" thick exterior glass flush glazed into a tubular rail. Elevation D was a narrow vertical strip consisting only of aluminum panels. The wall also incorporated a full-height neoprene flashing at each 135° inside corner. The overall size of the test unit was approximately 45 feet wide by 35 feet high.

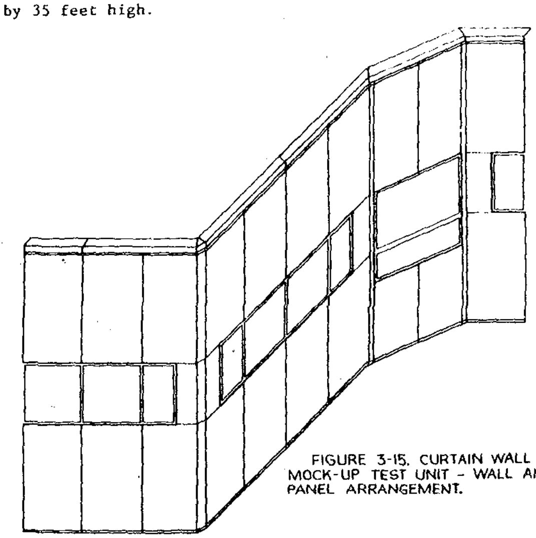


FIGURE 3-15. CURTAIN WALL MOCK-UP TEST UNIT - WALL AND PANEL ARRANGEMENT.

The mock-up unit was tested for static pressure air and water infiltration, dynamic pressure water infiltration, and static pressure-structural performance. Static racking tests to simulate seismic movement were then conducted and the preceding tests repeated. Performance requirements, the tests applied to ascertain the wall's ability to meet those requirements, and the results of these tests are presented below. The tests appear in the order in which they were conducted.

Air infiltration by static pressure:

- Test applied: Static air pressure of 1.56 psf, equivalent to a 25 mph wind.
- Design criteria: Less than 0.06 cfm per square foot or 97.4 cfm total allowed.
- Performance results: The test wall was found to be acceptable, since only 0.05 cfm per square foot or 88 cfm total, including test chamber air leakage, which could not be segregated because of weather conditions, permeated the assembly.

Water infiltration by static pressure:

- Test applied. The wall was subjected to a water spray at the rate of five gallons per hour per square foot with static pressure of five psf for 15 minutes, equivalent to 20% of the positive pressure design load; and six pressure cycles of 2.5 minute duration imposing 3.9 psf and 10 psf loads, equal to 39 mph and 63 mph winds.
- Design criteria. There was to be no water infiltration.
- Performance results. No uncontrolled water leakage occurred in any of the tests, but several drops of water entered at one weep drain tube fitting at the "C" elevation.

Structural performance tests by static pressure using full design loads:

- Test applied. The wall was subjected to ± 25 psf (positive and negative pressure design loads) to measure deflection. It should be noted that 25 psf was the design load applied to the upper 8'-6" of level 4 and 20 psf was applied to lower portion of the mock-up unit. Deflections measured at the 25 psf design loads are shown below.
- Design criteria: No glass breakage or evidence of any other damange.
- Performance results:

Elevation B:	Allowable: 1/240 or .750"	Performance Results + 25 psf	Performance Results - 25 psf
Outside corner mullion	.750"	.003"	.032"
Typical Mullion-level 3	.750	.454	.540
Typical Mullion-level 4	.750	.444	.545
Panel stiffener-level 3	.239	.122	.100
Vision sill	.239	.108	.181
Vision head	.239	.080	.156
Vision glass	-	.580	-

Elevation C:	Allowable: 1/240 or .750"	Performance Results + 25 psf	Performance Results - 25 psf
Vertical panel mullion	.323"	.012"	.030"
Vision glass	1.10	-	-

- There was no evidence of any damange or harm. Apparent excessive deflection was experienced at the vision head and at the horizontal muntin at the "C" elevation, but these included undetermined end movement which was to be measured in subsequent testing.

Seismic performance test:

- Test applied: The structural steel representing the floor at level 4 was designed so that it could be moved by means of screw jacks with respect to the floors at levels 3 and 5 (which remained fixed) to simulate seismic movement. The floor at level 4 was moved as follows:

  Inward (+) 3/4" normal to Elevation A; return to original position;
  Outward (-) 3/4" normal to Elevation A; return to original position;
  Inward (+) 3/4" normal to Elevation C (45° to Elevations A and B; return to original position;
  Outward (-) 3/4" normal to Elevation C (45° to Elevations A and B; return to original position.

- Design criteria: as stated earlier in this chapter.
- Performance results:

  Elevation A: Movement normal to the elevation produced no effect on the "A" wall. Movement at 45° caused displacement of the glazing sill closures with respect to the frame sills by 1/8". This figure was low becuase some of the 3/4" movement in the floor resulted in bending of the structure rather than movement at the test wall.

  Elevation B: Tests normal and at 45° to this elevation resulted in displacements of the glazing sill closures of 1/4" to 5/16" with respect to the glazing frame sills. When the floor was returned to its original position, some sills failed to return to their original positions by 1/16" to 1/8" due to friction or drag on neoprene weatherstripping and other wall components.

  Elevation C: No effects were noted in Elevation C.

Water infiltration by Dynamic Pressure:

- Test applied: The wall was subjected to water spray at the rate of five gallons per hour per square foot and winds from an 1800 horsepower aircraft engine wind generator at nominal 5 psf (20% of 25 psf design load) for 15 minutes.
- Design criteria: No uncontrolled water infiltration.
- Performance results: No water appeared on the interior of the wall.

Water infiltration by Static pressure (supplementary test):

- Test applied: The wall was subjected to a water spray at the rate of five gallons per hour per square foot and static pressure of 5 psf for 15 minutes, cycled pressure for 12 minutes, 10 psf for 10 minutes, and 15 psf for 10 minutes.
- Design criteria: No uncontrolled water infiltration.
- Performance results:

  There were no leaks during the 5 psf and cycled pressure tests.

At 10 psf loading after three minutes, a leak at the rate of one drop/second occurred between the top of the lower glazing frame and the head trim; total leakage was about one ounce.

At 15 psf, water leakage developed at the outside corner as a result of air and water percolation at the stacked joint at the bottom of the uppermost panels. Uncontrolled percolation occurred at the level 4 exposed corner panel weeps, with water surging periodically over the sill.

Structural performance tests by static pressure:

- Test applied: The wall was subjected to the following structural loading held for ten seconds each:

  For the lower 6' of level 4 and below:
  ± 20 psf (design load) and
  ∓ 30 psf (1.5 x design load);

  For the top 8'-6" of the mock-up:
  ± 37.5 psf (1.5 x design load)

- Design criteria and performance results:

Design criteria (L/240)	Performance Results + 20 psf	- 20 psf
.421"	.308"	.296"
.421	.207	.277
.421	.325	.310

- There was no damage or harm experienced as a result of the above tests.

Test to Failure:

Negative pressure was slowly increased from 30 psf in 10 psf increments. At -58 psf the spandrel panel frame welds at the first typical mullion in Elevation A just below the vision glass failed. This failure was accompanied by release of the adjoining glazing frames and glass failure on each side of the mullion.

Inward (+) 3/4" normal to Elevation A; return to original position;

Outward (-) 3/4" normal to Elevation A; return to original position;

Inward (+) 3/4" normal to Elevation C (45° to Elevations A and B); return to original position;

Outward (-) 3/4" normal to Elevation C (45° to Elevations A and B); return to original position.

Overall Test Results

The wall was tested in accordance with and met the architect's design criteria for static pressure air and water infiltration, static pressure structural performance and seismic racking, except that some excessive movements occurred as described in the static pressure structural test. The wall was also satisfactorily tested for dynamic pressure water infiltration.

Proper installation was necessary to ensure the wall's performance as an Uncoupled Element.

INSTALLATION OF THE CURTAIN WALL AT THE SITE

In any system designed to have a certain configuration, weight, stiffness, and connectivity, there is always the danger that the field installation may alter these characteristics to the extent that it does not perform as intended. In the case of the curtain wall, the decision that it should be an Uncoupled Element was the critical factor which would determine the wall's response during an earthquake. Should improper installation change the connectivity to the degree that all or a portion of the wall would act instead as a Coupled Element, then there would be the probability of excessive damage to the wall during a severe earthquake. Thus, good design in itself was not sufficient: the installation had to be consistent with the basic performance objectives and design criteria for the wall. For example, the design team intended that the subframe be anchored with pin connections to permit rotation, and that the 45° corner panels be installed to permit disengagement upon heavy seismic impact. Had a few extra screws or welds been added in the wrong locations to make the installation "stronger," the entire concept of the wall being lightweight and able to move in response to seismic loads would be changed. Originally, the design team thought that one would be able to pull off the 45° corner panels by hand. However, the installation resulted in panels which will disengage during an earthquake, but cannot be easily pulled off by hand. The design team's experience with the curtain wall emphasized the importance of recognizing potential changes in concepts which may occur as a result of installation; design teams must apply the necessary field inspection and testing procedures to insure that the design concept is fully carried out throughout the entire building process.

SUMMARY

The design and construction of the curtain wall system of the IBM complex is an example of the impact of the theory of the Dynamic Model on building design. The curtain wall, designed as an Uncoupled Element, was affected by even the earliest decisions in the design process. The site analysis, which noted the geological difficulties of the site, including a fault, determined the period of the site to be relatively short. This fact, in combination with an initially anticipated building period in a similar range, meant that in order to avoid a condition of resonance, the engineers had to design the building such that its period would be lengthened. Hence, the final design utilized a ductile steel moment resisting frame and was very flexible with a period of 1.0 seconds. The direct impact of this flexibility was an unusually large interstory drift of 3/4", making the design of an enclosure system especially challenging. The design team also determined that the enclosure system must be relatively lightweight, so that it would not increase the already high forces for which the building must be designed. The wall was designed, therefore, as an Uncoupled Element. The curtain wall design was unique in that its connections permitted a story to story displacement of 3/4" without significant damage, and the effectiveness of the design was confirmed by an extensive series of tests on a mock-up of the wall. Final detailing and installation successfully carried out the concept of the wall as an Uncoupled Element, thus completing the first design and construction of a building system following the concepts of the Dynamic Model.

Many people contributed to both the research effort and its design application, and special credit is due IBM for their interest in the most advanced design methods for earthquake safety.

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Gerald M. McCue, Principal-in-Charge of Design Team

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Consulting Mechanical Engineers:
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Eugene O. Tofflemire and Associates

General Contractor:
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Subcontractor for Enclosure Wall:
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OBSERVATION AND ANALYSIS OF DAMAGE TO MULTISTORY BRICK BUILDINGS IN THE 1976 TANGSHAN EARTHQUAKE

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ABSTRACT

Multistory brick buildings widely used for residential and public purposes in the city of Tangshan were severely damaged during the earthquake of July 28, 1976. We take a group of apartment houses and a classroom building in the paper for detailed analysis. The former, located in the area of intensity X, was completely collapsed. The latter, situated in the area of grade XI, was stiff enough to remain standing,even though it suffered heavily structural and nonstructural damages.

In this paper an analysis of seismic performance of multistory brick buildings from point of view of architectural and structural design practices as well as construction techniques is discussed.

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INTRODUTION

Immediately after the earthquake of Richter Magnitude of 7.8 occurred in Tangshan on July 28, 1976, a team of some teachers and a dozen of students of Qinghua University hurried to the site to observe the damage to the buildings. More than 700 multistory brick masonry buildings, including several residential quarters, office buildings, hotels and hostels, and classroom buildings were investigated.

I WORKER-PEASANT-SOLDIER DWELLING BUILDINGS

1. General Situation

W-P-S buildings consisting of nine apartment houses, Figure 1, located in the area of intensity X, were some of those suffered from the most serious damage in Tangshan. The vast majority of them were completely collapsed during the earthquake, resulting in a heavy loss of life and property. Buildings No.1 and No.2 were of three-unit two-story houses, whereas buildings from No.3 to No.9 were of five-unit three-story ones having a total length of about 100 meters each. Figure 2 shows the end unit plan of buildings No.3 to No.9, and their wall section is shown in Figure 3. All the buildings were constructed with grade 75 ($75^{kg}/cm^2$ of compressive strength) clay brick. The walls were bonded with grade 25 cement mortar, except that the outer and transverse walls were laid with grade 10 mortar at the second and third floors.

There was no reinforced concrete tie-beam in buildings No.1 and No.2 Tie-beams were installed only in buildings No.3 to No.9 at the roof level in both end bays, Figure 4, and there was no anti-seismic joint in these buildings.

2. Damage Description and Discussion

Damage to Builing No.1

Figure 5 shows the failure of the east unit of building No.1. It could be seen that the balcony at the east bay was broken up, transverse walls had partial breakage, most precast floor slabs fell down off each bay and wall cracked with a large vertical rupture along the flue of the chimney. The north wall (except the stair well) standing in line suffered a less damage than the south wall, having alternate recessions and projections. In the north front, walls weakened by window and door openings and chimneys were invariably collapsed, Figure 6. Figure 7 and Figure 8 illustrate the interior view after the earthquake. Due to the action of the vertical earthquake forces, the brick masonry shattered loose, and then roof slabs with their supporting beam beneath sank to a depth of 25cm. Both the upper and lower parts of the inner longitudinal walls, having been free from their connection with transverse wall, collapsed badly. The corridor floor slabs shattered and fell down due to tossing movements.

Damage to Building No.2

The design of building No.2 was just the same as that of building No.1, so the state of their damage was similar to each other. Four bays at the west end and two bays at the east end were completely collapsed to the ground; others partially. Figure 9 shows the elevation view of the north front after the earthquake. In construction, the eaves slabs were rested on the cantilever beams without any anchorage between them, and consequently, a large number of them fell off their supports in the earthquake. Figure 10 is a view of the damaged south front at the second floor.

The damage to the corridor at the second floor is shown in the Figure 11 and Figure 12. As soon as the inner longitudinal walls came down, beams and floor slabs followed to fall. Accordingly, the building splitted readily into two parts, Figure 13. In above figures, We can see clearly that the transverse and the longitudinal brick walls were jointed by means of straight toothing overlapping only once per five courses of brick throughout the entire height of the wall. On the surface of the toothing no indication of mortar could be seen. It is evident that the poor workmanship of bonding was one of the most serious defects which greatly reduced the seismic resistance of the wall. Figure 14 shows an instance of fractures in the corridor floors of building No.1 and No.2, caused either by heavy impact of falling members or by vertical action of violent quake shocks. Since floor slabs in the rooms were securely fixed in the walls on which they were supported, they had less bending moment at the span center than those in the corridors. Therefore, no fracture was discovered.

Damage to Buildings No.3 to No.9

Seven three-story buildings from No.3 to No.9 were almost completely flattened, except that two badly damaged bays stood at the east end of building No.8. Thanks to the existence of conduits and pipes, the damage to the water-closets at the first floor was somewhat less severe than that to other rooms.

The remains of two east end bays are illustrated in Figure 15. It was found that the outer and inner walls of one of the bays had came down. The third floor slabs in the corridor, one slab near the south wall of the room and two eaves slabs had also fallen apart. The tie-beam at the roof level ruptured at the place where chimney extended through. The flue of the chimney built at the intersection of the south and the transverse walls was exposed and visible. The X-shaped cracks occurred in the transverse walls at both the upper and lower floors, but they appeared larger and deeper in the second floor wall. Some vertical cracks were seen at the corner where balcony and chimney met. In addition, the balustrade panels of the third floor balcony, the brick piers supporting the balcony structures and eaves cantilever beams were also falling down.

The above-mentioned damage indicated that during the earthquake most buildings happened to collapse first at the outer and inner longitudinal walls, and soon afterwards the transverse walls were inflicted so seriously, that they failed to sustain the load, resulting in the failure of the upper structure, and finally crushing the structures into pieces below. A few buildings were destroyed by the earthquake at one stroke.

3. Brief Summary

There were ten other apartment houses in Yaojinlu quarter designed in the same way as W-P-S buildings. They also broke into pieces and suffered much more severe damage than their neighbors. Analysis showed that the improper treatments of architectural and structural design as well as constructional techniques were responsible for the seriousness of damage to the buildings.

(1) The Seismic Resistant Requirements were Overlooked in the Layout.

Poor Earthquake Resistance in Longitudinal Direction.

All the inner longitudinal walls of the building (nearly 100m long), except those being 3.2m long and 24cm thick at the two ends, were only 12cm in thickness, Figure 1. Undoubtedly, the earthquake resistance was much too weak in their longitudinal direction. In addition, there was poor sound and thermal insulation in the rooms adjacant to the partitions of dwellings and unheated stair halls.

Earthquake Resistance of the Wall Weakened by Window and Door Openings and Chimney Flues.

Owing to the layout with an alternately recessing and projecting exterior, the south wall stretched tortuously. Quite a few chimneys had been built not only inside the outer walls but also at the corners where stresses were concentrated near the doors and windows. Figure 16 shows a typical bay in the south wall. After deducting the chimneys and window and door openings, there were few areas of solid wall remaining for resistance to the earthquake forces. The similar problem also existed in the layout of refuse chutes in the north wall, Figure 17. Consequently, the outer walls were greatly weakened to resist the earthquake forces.

Transverse Walls Separated without Strong Connection.

So far as the wall layout was concerned, it seemed that the designer had paid much attention to lining up the transversal load-bearing walls. But the corridor ran a length of five out of seven bays along the center part of each unit, separating the transverse walls into two parts without any strong connection between them. The structure seemed to be cut into two parts once the violent shock broke the longitudinal walls, and the corridor floor slabs fell apart.

But, it was interesting to compare the W-P-S buildings with another four-story apartment house also sited in area of intensity X. It could be seen from the layout plan, Figure 18, that the architect seemed to have put special emphasis on laying the longitudinal walls, particularly the inner ones in line. In this case, the longitudinal earthquake resistance of the structure increased to a considerable extent, presenting itself as a principal cause to keep the building standing even with dangerous cracking. Therefore, special attention should be paid to the earthquake resistance of the longitudinal walls in the future design.

A long building is unfavorable to seismic resistance as the earthquake forces would not be evenly distributed to the whole structure with its poor resistance in the transverse direction. It seems acceptable that the length of a building should be less than 60m. Otherwise, anti-seismic joints should be provided.

(2) Poor Brick Masonry Inadequate in Strength.

Brick masonry is a key component for seismic resistance in a brick load-bearing wall building. The masonry strength of W-P-S buildings was extremely affected by the poor mortar used. It is suggested that cement mortar grade of at least $50^{kg}/cm^2$ should be used in the future construction.

(3) Poor Quality of Construction Techniques Aggravating the Damage.

In the W-P-S buildings, the vertical toothing method was used as the connection between longitudinal and transverse walls by overlapping once every five courses (occassionaly every ten courses). In some instances, there was no overlapping at all along the entire height of the wall. The openings provided for the wheelbarrows during construction were walled up in a slipshod manner.

The phenomena of the damage to a group of apartment buildings in the south street of New Xicun were the proof that fine quality of wall materials and good construction techniques were of great importance in resisting earthquake forces for brick wall structures.

As shown in Figure 19 and Figure 20, this building group consisted of five two-story apartment houses of the same type. The outer walls of buildings No.1, No.2 and No.3 were constructed with high grade bricks of $150^{kg}/cm^2$ and 25 grade mortar in spring, whereas both outer and inner walls of buildings No.4 and No.5 were built with low grade bricks of $50\text{–}75^{kg}/cm^2$ and with 10 grade mortar in winter.

In a word, the greatest differences between each case were the material quality of the brick wall and the season of construction. It was these differences that accounted for the remarked difference of masonry strength in earthquake resistance. As a result, after the major shock, the buildings No.4 and No.5 collapsed completely to the ground; the buildings No.1 and No.3 collapsed partially and the building No.2 survived destruction with cracks.

(4) Tie-beams were Installed in their Improper Places.

It was quite inadequate to have tie-beams only at the roof level on both sides of the end bays. In the future construction, they should be installed at every story upon all the load-bearing walls and directly tied to the precast floors and roof slabs.

II CLASSROOM BUILDING OF DAXIEZHUANG PRIMARY SCHOOL

1. General Situation

The statistics of damage to brick masonry buildings according to investigation made in New Town district after the earthquake are shown in the following table:

Kind of buildings	Amount of buildings (percentage)	Class of damage*			
		I	II	III	IV
2 or 3 story classroom building	21 (100)	10 (47.6)	5 (23.8)	1 (4.8)	5 (23.8)
Brick building for other purpose	241 (100)	24 (9.9)	46 (19.1)	37 (15.3)	134 (55.7)

* I: Cracking without collapse
II: Partial collapse
III: Most collapse
IV: Total collapse

The result of investigation showed that the classroom buildings had less damage than those similar buildings used for other purpose.

The classroom building of DAZ primary school was situated in the area of quake-intensity XI. It sustained a great many cracks all over the building, but escaped from collapse. The outer wall at first floor had thickness of 49cm; the rest, both outer and inner walls, were 37cm thick. Outer wall was built of high grade bricks ($150^{kg}/cm^2$), while inner wall was built of ordinary clay bricks. The brickwork of the first floor was laid with cement mortar in a grade of $50^{kg}/cm^2$, while that of the second and third floors was laid with $25^{kg}/cm^2$ grade. The stairs and floors above the lobby were construted with reinforced concrete poured in place. On the top of classroom, there were two double-span continuous beams connected with the tie-beams at roof and each floor structures. The tie-beams were used as window lintels as well. All the floors of the classroom, including the roof slabs, were make up of precast hollow slabs, Figure 21 and Figure 22.

2. Damage Description and Discussion

Figure 23 shows the front view of the classroom building after the earthquake. The breakage happened at two corners at the third floor. In the walls appeared various types of cracks, which got larger during the aftershocks. Probably, this building had been repeatedly subjected to the strong quake forces in all directions. The lintel at the east end on the third floor had been keeping from dropping until a few days after the major quake.

As illustrated in Figure 24, the vertical cracks appeared in the middle of the wall, and the horizontal ones occurred along the floor levels. The end wall was bulging outward due to the earthquake. This showed that the inner longitudinal wall and floor structures were so rigid that the end wall could keep itself still standing.

Figure 25 shows the wide diagonal tension cracks at a maximum width of 8cm in transverse walls. There were two closely spaced doors in the wall, and the small pier between them was bonded to the longitudinal wall by means of toothing joint with a overlapping every five courses. Hence, the pier became a weak link in the wall. In the earthquake, one of these piers completely shattered off, causing the breakage of the door lintel. If the south door had been placed to the other side of the classroom wall, there might have been less damage to the wall.

The damage to the inner longitudinal wall is shown in Figure 26 (lobby not shown). In addition to the various cracks, it should be noted that partial spalling took place near the upper corner of the end wall, where the chimneys got through (see arrow a, Figure 26). It might be well proved that the chimney built inside the wall had considerably weakened the wall to resist the shock of the earthquake.

3. Brief Summary

In general, this building was so damaged that it rendered itself irreparable. However, as compared to the nearby build ings at a fairly long distance, including those completely collapsed one-story school buildings, it still stood there. This proved that this building had structurally functioned very well during the earthquake.

Analysis indicated that the good earthquake resistant properties of this building in comparison with other similar buildings could be characterized by the following points:

(1) Making the outline in plan simple, symmetric and shorter resulted in good earthquake resistance of the structures in both transverse and longitudinal directions.

It was of a great success to layout the four classrooms symmetrically around the lobby and to partition them off into north and south parts with a thick brick wall, thus preventing classrooms from noisy confusion. By means of the solid thick wall, the functional requirements of sound insulation, structural capabilities of load-bearing and resistance to the quake forces were well integrated together. This provided some remedy for defects of earthquake resistance in the outer walls resulting from large windows in themselves and in the tortuous lobby walls. Two end walls designed without any window openings were very helpful in blocking the noise from outside, insulating cold and heat, and ensuring good day lighting. In short, the architect seemed to have made the center of mass of a building possibly close to its center of rigidity, and to have well combined archtectural treatments with seismic resistant requirements.

(2) The brick masonry had a high grade in strength.

The wall of this building was thicker than that of other three-story buildings commonly built in pre-earthquake Tangshan. The mortar was also of higher grade.

(3) It had a good structural integrity.

Stairs and floors in the lobby being constructed of monolithic concrete, tie-beams being monolithically bound to two-span continuous beams together and interlocked with the precast floor and roof slabs, all of these measures taken greatly strengthened the integrity of the structure in its entirety.

(4) It had a good quality of construction.

It was well known that this project had been praised for its best quality of construction in the city of Tangshan.

III CONCLUSION

We are of the opinion that it seems impossible to prevent multistory brick building from damage under so terrific shock as the 1976 earthquake in Tangshan. However, it is possible to build them strong enough to avoid collapse in order to ensure the people to be safely evacuated and to decrease loss of property to a minimum.

To do so requires all the architects and engineers to draw lessons carefully from the disaster of the Tangshan earthquake and take every practical measure to improve in seismic design and construction.

We can draw very useful lessons from the Tangshan earthquake to improve the seismic resistant property of multistory buildings (here mainly refer to apartment buildings). It may by summarized as follows:

1. Improving the Design of the Plan.

In designing the plan, careful consideration should be given to the layout of the structural walls so as to make the structure have a balanced resistance to earthquakes in all directions, --- for examples, placing doors, windows and chimneys properly in order to increase possible areas of solid walls and decrease the small piers to a possibly minimum amount, avoiding, if possible, tortuous walls, preventing large doors, windows and chimneys from arranging at the places where stresses concentrate, correctly laying anti-seismic joints, and so on.

2. Ensuring Adequate Strength and Increasing Ductility of Brick Masonry.

Brick and mortar for masonry must be tested before laying. Effective measure should be taken to make cast-in-place reinforced concrete constructive columns in the brick wall to improve its ductility.

3. Ensuring Integrity of Precast Floor and roof structures.

Tie-beam can play an important role in ensuring integrity of precast floor and roof, if they are well and directly interlocked with each other. The practice of taking tie-beam as lintel with one to three courses of brick apart from the floor and roof structures above, which was often adopted in Tangshan before the earthquake, must never be used, because it cannot provide integrity for the precast structure at all.

Finally, it should be emphasized that special attention should be paid to the workmanship of construction on the site, as it is one of the utmost important factors for a seismic structure.

It should be examined through practice to make sure whether the opinions mentioned above are correct or not. The mitigation of earthquake disaster through architectural design is quite a-new subject in China, about which we have practically no previous experience. There must be a lot of provlems concerning this subject, and they need to be discovered and solved by collaborated efforts on further investigation between architects and engineers with advanced experience gained at home and abroad.

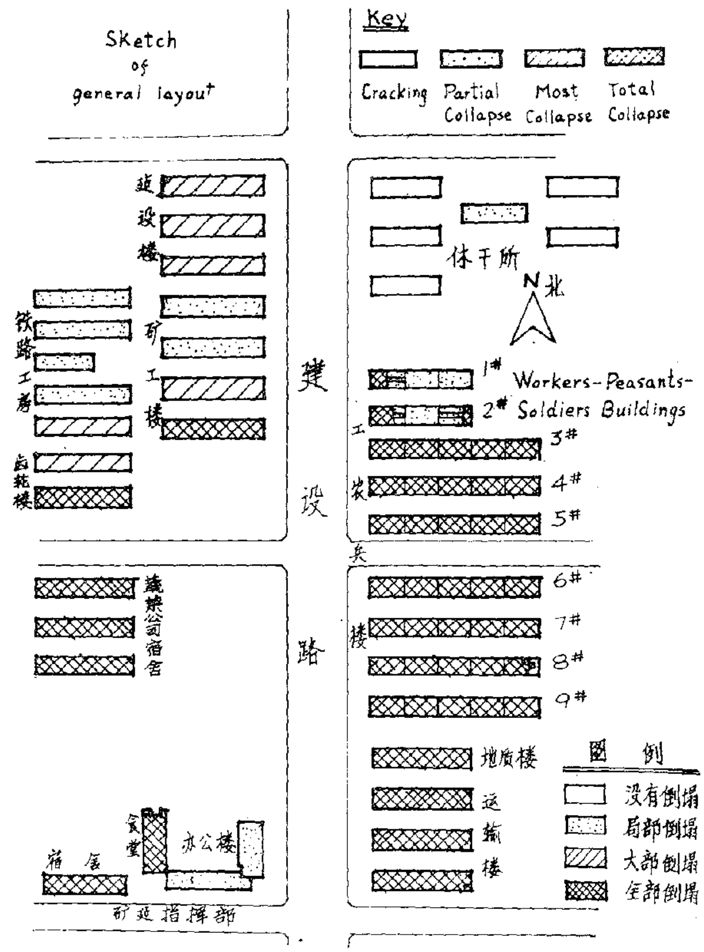


Figure 1 Sketch of general layout

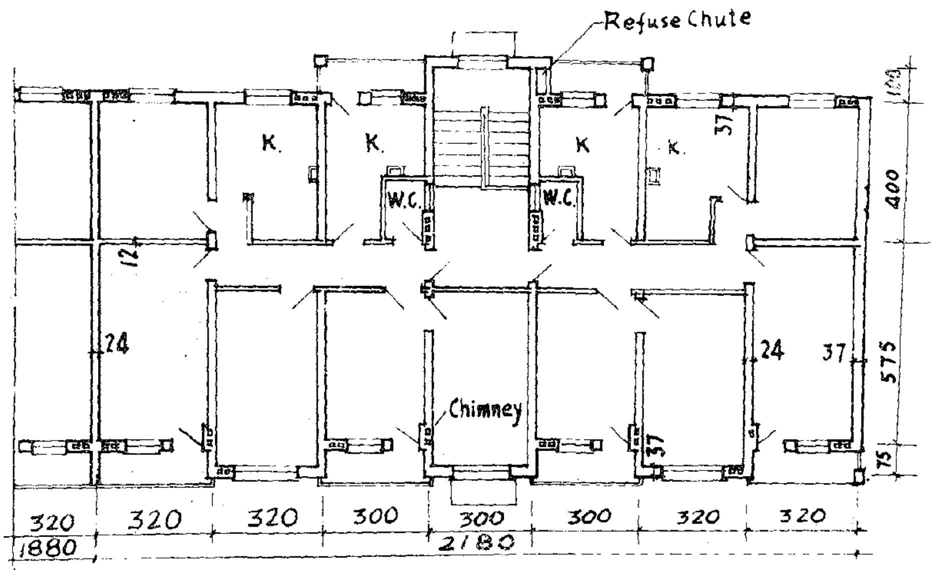


Figure 2 Unit plan at the end of bldg. No.3-No.9

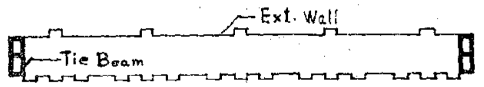
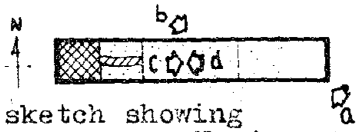


Figure 4 Position of tie-beams at roof level



A sketch showing damage to bldg.No.1 and key signs of viewpoints

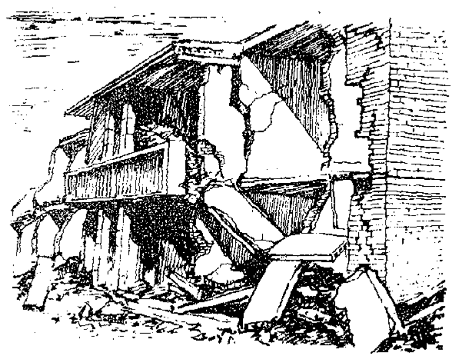


Figure 5 A view of bldg.No.1 at the east end (from vp a)

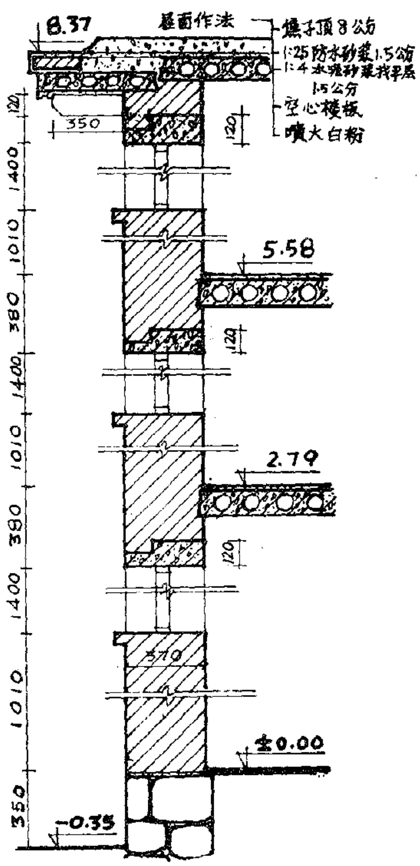


Figure 3 Wall setion

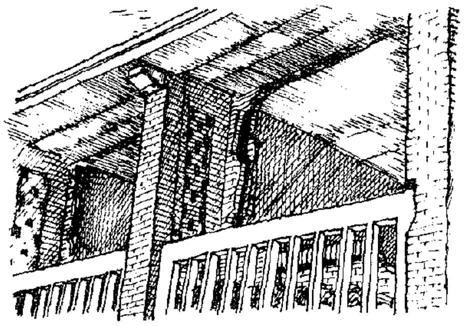


Figure 6 A view of the north balconies of bldg.No.1 (from vp b)

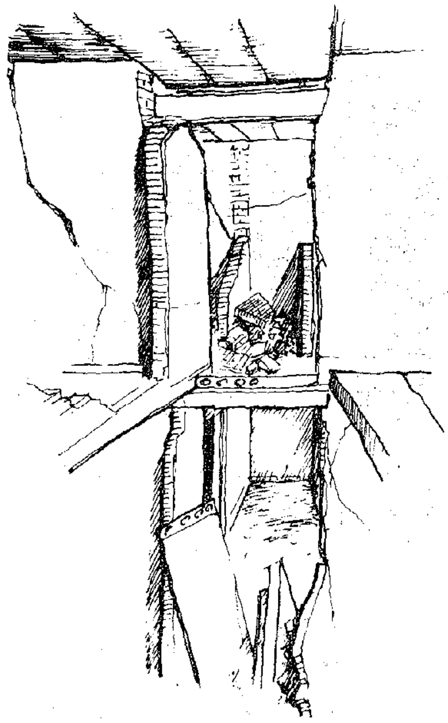
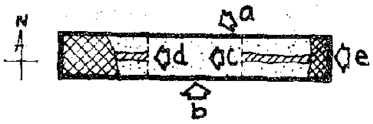


Figure 7 An view of bldg.No.1 (from vp c)



A sketch showing damage to bldg.No.2 and signs of viewpoints

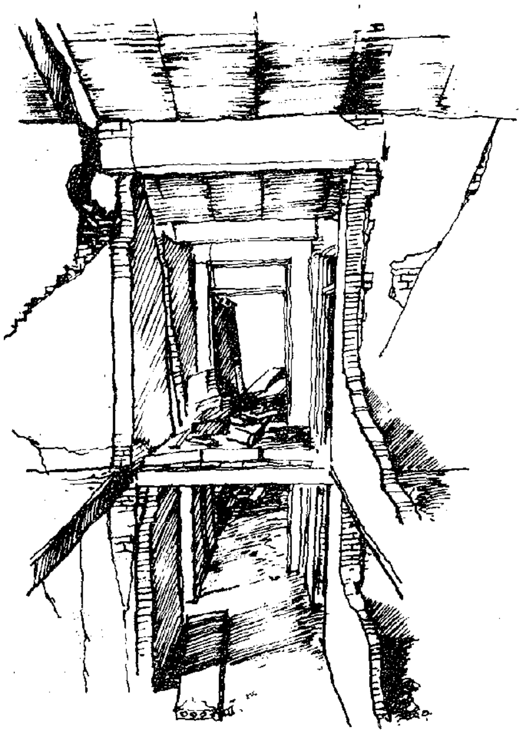


Figure 8 An interior view of bldg.No.1 (from vp d)

Figure 9 North elevation of bldg.No.2 after the earthquake

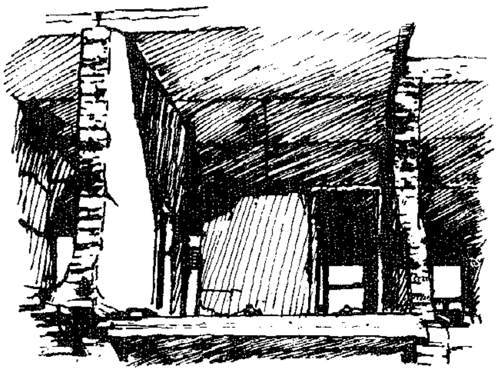


Figure 10 A close view of bldg.No.2 after the south wall thrown outward

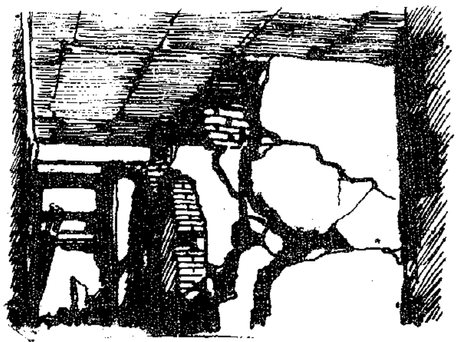


Figure 11 Corridor of bldg. No.2 (from vp c)

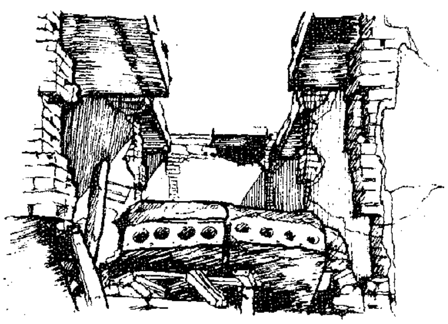


Figure 12 Corridor of bldg.No.2 at the west end after roof slabs falling down (from vp d)

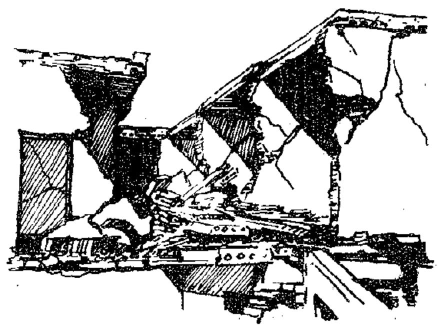


Figure 13 The east part of bldg.No.2 (from vp e)

A sketch showing damage to bldg.No.8 and key sign of viewpoint

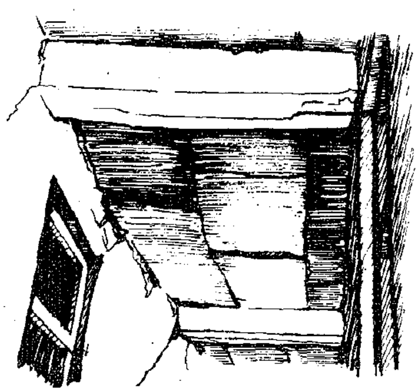


Figure 14 A view of fracture visible in the corridor slabs

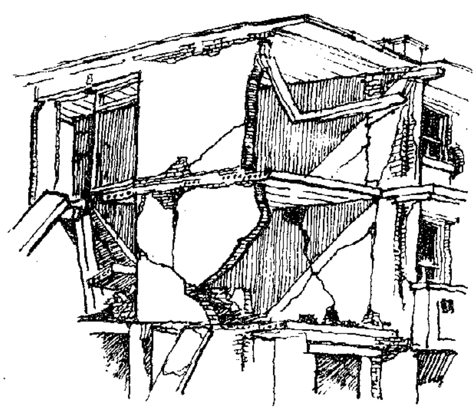


Figure 15 Remains of bldg.No.8

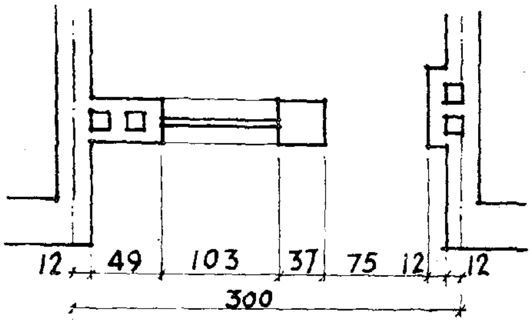


Figure 16 Part of south wall

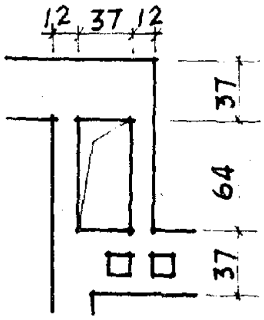


Figure 17 Refuse chute detail

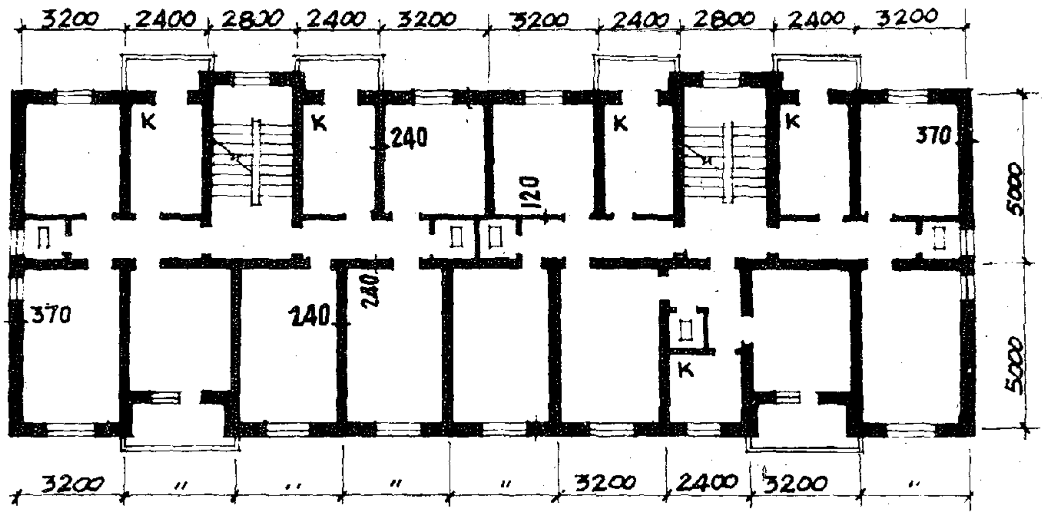


Figure 18 Floor plan

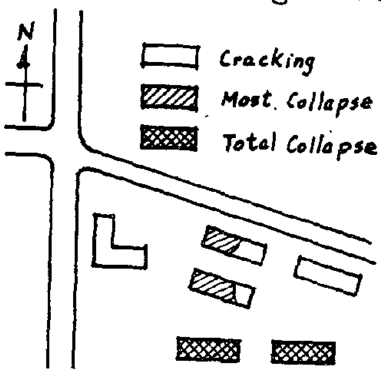


Figure 19 Sketch of general plan

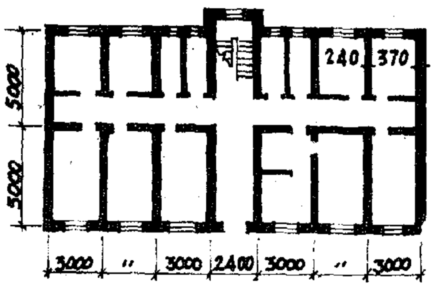


Figure 20 Unit plan

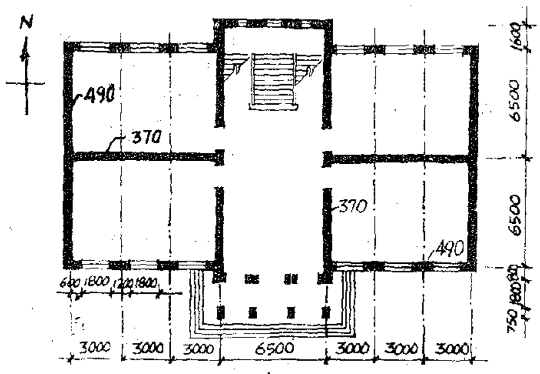


Figure 21 First floor plan

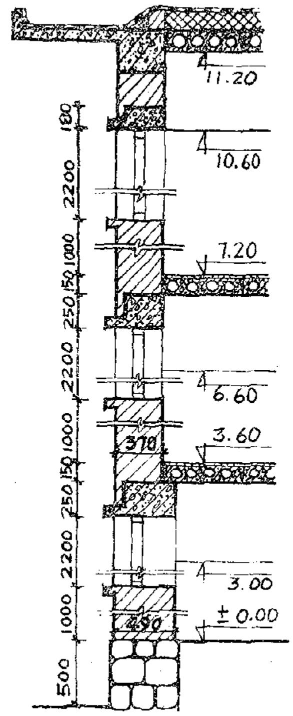


Figure 22 Wall section

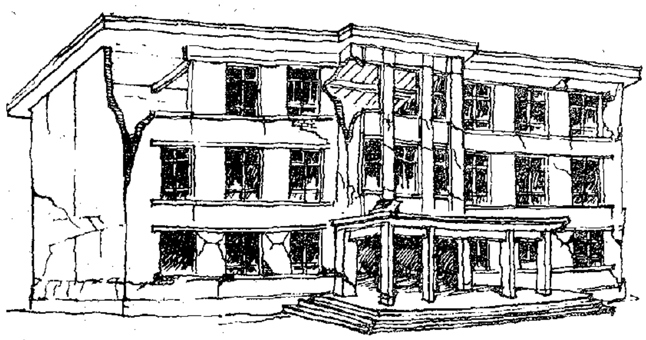


Figure 23 Damaged elevation of classroom bldg. of
DXZ primary school

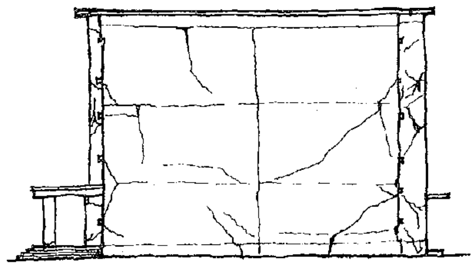


Figure 24 East elevation

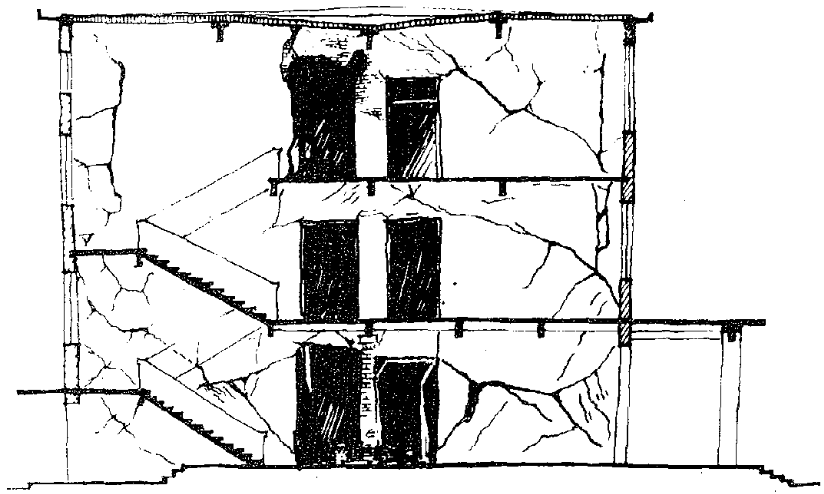


Figure 25 Transverse section

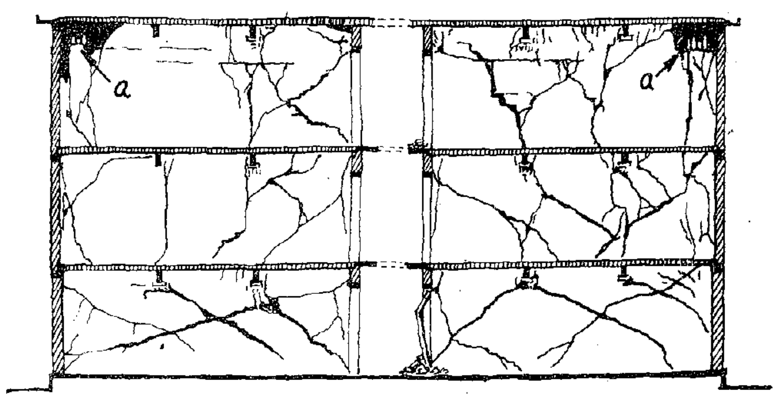


Figure 26 Longitudinal section

DAMAGE OF NON-STRUCTURAL COMPONENTS DURING THE 1976 TANGSHAN EARTHQUAKE

Yang Wen-zhong*

ABSTRACT

In the past, the architects put the function and artistry of the non-structural units of the houses above everything else, without taking the seismic resistant property into due consideration.

The paper stresses in particular the hazard suffered by the non-structural units of the houses located in Tangshan magistoseismic area. Starting from the point of reducing the earthquake hazard, when new design is in contemplation, the non-structural units should match the main structural units to the greatest extent to offer a two-fold effect of both economy and seismic resistance.

I. GENERAL DESCRIPTION

Before the earthquake the total building area in Tangshan was 14.13 million M^2, of which the dwelling houses occupied 7,800,000 M^2, 80% of the dwelling houses were single-storeyed houses and 20% were multi-storeyed brick masonry structures.

The single-storeyed brick masonry houses varied in style. Those included the thatched cottages, the little tile-roofed houses left over by early times, various types of slope-roofed houses and a great number of houses with concrete roofs and cinder concrete roofs. The maximum height of the multi-storeyed brick houses were seven stories while those of the stone houses were three stories in maximum. The room layout was set at 3-3.6 M in width and 3 M for floor in height(see Figs 1,2).

According to the custom prevailing in Tangshan, the external and internal bearing wall thicknesses of the multi-storeyed brick houses were 370MM and 240MM respectively. The wall thickness of the multi-storeyed stone houses was 400-500MM. The floor slabs were usually prefabricated hollow floor slabs spanning the short side. Only a small number of them was cast-in-situ reinforced concrete slabs. The brick masonry houses were generally built on rubble stone strip foundations.

There were two kinds of frame type houses. One was the all frame houses. These were mostly the plants located in the bases of coal, metallurgy, power generating and chemical engineering. The other was of the half-lattice frame-type houses. These were usually the plants and workshops of the light industry and public utilities, such as stores and office buildings. The frame-type houses were mostly cast-in-situ and only a small part of them

*The Tangshan Municipal Construction Bureau.

were prefabricated and assembled on site. Most of the frame-type houses had no decorations except the public buildings. All the frames were laid on separate reinforced concrete foundations.

There were two kinds of spacious single-storeyed houses. One was the heavy roof system composed of large-size sheathing, concrete or steel roof truss and various steel rod bracings, The heavy roof system generally used reinforced concrete columns as load-bearing structures. The singlespan or multispan single-storey industry plants were mostly of this structure.

The other kind was the light-weight roof system composed of asbetos tiles, corrugated sheet iron, hanging tiles, steel or wooden roof truss as well as steel or wood rod bar bracings. The light-weight roof system usually used reinforced concrete pillars, steel pillars, brick pillars and brick walls as load - bearing structures. The singlespan or multispan light industry plants, auditoriums, gyms, dining halls, warehouses, etc., were mostly of this structure. The reinforced concrete pillars and steel pillars all had their own reinforced concrete foundations. The brick pillars had their separate rubble or brick foundations and the load-bearing brick walls stood on brick masonry strip foundations.

Apart from the above-mentioned structure types, there were also houses with flat plates and with shell structures. The earthquake intensity in Tangshan area in the past was basically 6 degrees.

After the severe earthquake hit, the industrial and public buildings in the urban district of Tangshan suffered severe damage(see Figs 3,4) resulting in heavy losses of the life and property of the people.

The macroscopic epicentre lay in the southeastern part of the city proper, namely in an area to the south of the Beiging-Shanhaikuan railway. Therefore, the earthquake intensity in the district lying to the south of the railway and a part of the western district adjacent to the railway was 11 degrees. Most of the houses located in these districts were levelled to the ground.

At the roots surrounding the Phoenix Hill, Dacheng Hill and Jiachia Hill, abnormal regions with low intensity occured, resulting in a general reduction of damage of the various houses. Furthermore, many inflexible houses on the soft soil foundations along the bank of the Douhe River did not collapse thus suffering slight damage. Besides the abovementioned houses, large scale industry and mine enterprises as well as various houses in other districts were all hard hit with an intensity of 10 degrees(see Fig 5).

Through this event, we have learnt experience and also a profound lesson.

II. DAMAGE OF THE NONSTRUCTURAL COMPONENTS AND ANALYSIS

Parapet walls

The parapet walls composed of ornamental concrete blocks, the brick parapet walls of the colonnade type, the brick laid solid parapet walls and self-supporting vertical ornamental walls all bore great earthquake force. Most of them broke off at the roots and collapsed(see Figs 6,7). The cast-in situ reinforced concrete parapet walls suffered lighter damage due to their better integrity.

Roof

The flat roof could be made thermal insulating in various ways. The 8 - 12CM thermal insulating breeze laid on the roof had the disadvantage of being high in load. But the earthquake proved that it had a better integrity. When the wall corner or the prefabricated hollow floor slabs spanning the short side collapsed in local places, the breeze insulating layer could still remain intact. Cracking of the breeze insulating layer was due to change of the bearing conditions. There were two ways of placing the dry breeze or fine cinder thermal - insulating layers. One was to place a 4CM thick small-size ballast concrete rigid water-resisting layer upon the thermal insulting layer. The roof cover exhibited a very good integrity because of the laying-up of the steel wire mesh. The rigid water-resisting layer formed by small-size ballast concrete was rather sensitive to earthquake. During violent earthquake, it was liable to crack to a large extent and fail to perform its function. The other way was to firstly plaster a 2.5CM layer of cement sand mortar and then apply a flexible water-resisting layer composed of two layers of felts and three layers of tar.

During earthquake, the hollow slabs cracked along the joints or came off while the thermal insulating materials peeled off with them and the felts split and broke away.

Comparatively speaking, the roof cover made of cast-in-situ reinforced concrete was beneficial to the maintenance of the thermal-insulating layer. In a word, where the load-bearing structure of the house suffered slight damage, the roof cover was likely to be damaged to a lighter extent.

Judging from the deformation, displacement of the waterproof sheet iron at the expansion joints and from the fact that the felts broke away along the joints, we could see that the houses demonstrated different shock characteristics during the strong earthquake.

As far as the heavy roof cover system of the industry plants was concerned, dry breeze or asphalt pealite were usually laid on the roof boarding as thermal insulating material. Afterwards, two layers of felts and three layers of tar were paved. Finally coarse-grained sand was spread. Connection of the large size roof boardings with the truss and the connection between roof boardings were effected by simple welding which led to unreliable bonding. During strong shock a large part of the roof cover disintegrated resulting in the fall of the large size roof boarding. Due to concentration of stress of the structure, disintegration of the large size boardings adjacent to the bracings between the columns were even more obvious. The thermal insulating material became loose and destroyed with the dislocation, bulging, disengagement of the large size roof boardings. At the same time, the water-resisting layer cracked and broke off. If the connection of the roof boarding was secure, the thermal insulating and waterproof layers would remain basically intact during the earthquake.

Roofage of the slope roofs included hanging cement tiles, earthenware tiles, small black tiles, asbetos tiles and corrugated sheet iron, etc. As for the highly sloped roofs, the hanging tiles all slipped in outward direction. The gradient of the most slope-roof houses in the urban district of Tangshan was set at 1/5. The hanging tiles on the survival houses all remained basically intact. Corrugated sheet iron and asbetos tiles were destroyed to a less extent.

Roof facilities

Chimneys with earthenware pipes and the cast iron vent pipes were unliable to get damaged because of their good attachment to the flat roof. As far as the brick chimney, water tank, applicance chamber and staircase were concerned, those parts which projected beyond the roof were mostly supported by brick walls. During strong shock, the masonry first got damaged and then collapsed(see Fig 8). The TV antennas, ladders, steel passways and iron supports were uneasy to get damaged. All of the water tanks filled with water would undergo displacement and torsion. The conduits connecting the water tank were mostly cut off at the welding joints and flange connections.

Leaders

Leaders of the dwelling houses and public buildings in the urban district were all of the outside drop type. The leaders were generally fitted at the corner of the house where the stress was liable to concentrate during the earthquake. They usually got damaged with the collapse of the walls. Among them, the terra cotta pipe were more sensitive(see Fig 9).

Rain-gutters and inside rain leaders of the multispan industry plants were also sensitive structural elements. During strong shock, the rain-gutters made of rolled sheet iron and reinforced concrete would tend to displace or go up and down in varying degrees due to stress concentration. The leader connecting with the rain-gutter often got damaged at the top part due to shearing.

Overhanging structures

Balcony, canpy, suspended ladder, outside corridor and terrace were reinforced concrete overhanging structures commonly seen on all kinds of houses. The architects not only laid stress on their functions and sense of beauty, but also were very considerate in their structure. The static load and dynamic load were all taken into consideration with the exception of the load incurred by earthquake. Furthermore, they paid much attention to their integral connection with the house. In the course of construction, these overhanging structures were handled with much care and were mostly cast-in-situ. No trouble had occured in their daily service. In the past, it was considered that reinforced concrete overhanging structures were liable to get damaged during strong earthquake. How did things stand with them after the earthquake? During the earthquake, one situation was that the overhanging structures fell down due to loss of bearing below(see Figs 10, 11) as often seen under the circumstances where brick walls were used as load-bearing elements.

The damage is mainly due to failure of the walls. Another situation was that the overhanging structures were smashed(see Fig 12). This was often seen under the conditions where parapet walls or maintenance walls were used. The damage was mainly due to the failure of the upperstructure. Nevertheless, a large number of overhanging structures were still in good condition during the strong shock(see Figs 13,14). This showed that the overhanging structures offered good anti-seismic characteristics. Their success or failure mainly depended on the condition whether the bearing structure and the upperstructure were firmly attached. Overhanging structures in the magistoseismic area were unliable to get toppled.

Ornamental lattice

Ornamental lattice on the porches, windows and walls were made of combinations of concrete blocks and inserted pieces. In the course of construction, one method was to build directly with concrete sand mortar. The other way was to use the methods of building, welding, inserting and binding in conjunction with the placement of horizontal reinforced bars. During the earthquake, various ornamental lattices suffered slight damage and were uneasy to peel off under ordinary conditions.

Wall finish

The mortar plastering, concrete adjacent planks, concrete skirting boards, ceramic veneers and granitic plaster were ornaments of the common external and internal walls. The damage extent of the ornament depended on the degree of damage of the wall. In case cracks appeared on the wall, the ornaments would crack or broke away accordingly(see Fig 15).

Lathed walls

Damage of the lathed walls were different to some extent. Apart from the peeling-off of the wall cladding, a lot of horizontal cracks also appeared. This phenomenon was more remarkable on the lower part.

In summary, the ornaments were very sensitive.

Chimney on the attached wall

When designing civilian buildings of brick structure, the architects considered both their functions and the people's customs in utilization. On the one hand, heating in winter season and ventilation were considered. On the other hand, regularity of a room was also taken into account. Hollow smoke flues and terra cotta pipe were often installed in the 240 cm thick cross and vertical walls. The walls were remarkbly weakened because the flues were mostly placed at the juncture of the cross and vertical walls. It was the weak part viewed from the point of shockproof. During severe earthquake, the wall was seen to crack vertically at the place where the flue was installed. The terra cotta pipe was also cut off or broke away with the collapse of the walls.

Windows and doors

Windows and doors made of steel, wood or concrete were liable to deform. Meanwhile glass on the window was particularly sensitive to the shock. Therefore the glass was observed to get damaged first. Windows and doors usually deformed, curved, broke off or fell with the cracking, displacement of the wall. Generally windows and doors in houses with frame structure would undergo remarked deformation and not a single piece of glass remained. As for the low houses with great rigidity, damage of windows, doors and glass were slight. Even with houses which survived after the earthquake, the windows and doors worked unwell.

Lintels

Wood lintels, reinforced concrete lintels, brick lintels, the flat brick arch and the brick arch used in place of lintels were all indispensable structural elements. However, the supports at both the two ends were just in the region where the wall stress concentrated. Due to insufficient overlapping length at both ends, different lintels had been damaged on account of the loss of bearing. The flat brick arch would also suffer damage if no tension bars and concrete protecting layer were applied below it.

Girths and reinforced concrete structural columns*

According to the custom prevailing in Tangshan, most of the houses were fitted with reinforced concrete girths to enchance the ingegrity of the houses. In face of the strong shock, the wall of the brick masonry house first exhibited X - shaped principal tensile stress cracks and under the effect of repeated shock, the wall which had been separated into four parts could be seen to slump, displace, be smashed and collapse. The girth failed to perform its function of controlling the slumping, faulting of the already cracked wall. The reinforced concrete beam - column systems of the houses with inner frame structures

*A kind of column installed in brick wall. It runs in a continuous state, enclosing the wall and forming its rim. It is one of the measures adopted to enhance both the strength and integraty of the brick wall---similarly hereinafter.

were different from the outside load-bearing walls in rigidity and ductility. Usually, the cross walls with high rigidity and low ductility showed shear failure due to principal tensile stress while the vertical walls showed curved through cracks in the horizontal direction. The weakest point of a house with half-lattice frame structure lay in the failure of the outside load-bearing walls. The girth was unable to prevent the walls from failure. As for a house with spacious rooms (with masonry structure as load-bearing units) and industry plants with overcloak walls, the vertical walls tended, under the bending effect, to crack and break off due to the large area of the wall and due to the fact that its bending strength was much lower than its shear strength. This resulted in the collapse of the whole house. The larger the height-width ratio, the more harmful it was to the house. The broad spacing between the girths could only exert poor constraint over the walls, leading to their failure and collapse. If the girth was cut short by smoke flue and vent, and lack in reinforcement and insufficient in overlapping lengths, it would be more unfavourable to the shockproof characteristics of the house.

After the earthquake, we could see a lot of broken reinforced concrete girths in the shape of necklaces on the site (see Figs 16, 17 and 18). It proved that the girths were unable to prevent the walls from shear cracking, slumping, faulting, smashing and collapse. Thus the collapse of the houses was unavoidable. However, the girth had a good checking effect on the uneven settlement of the foundation and the development of vertical cracks.

As seen from the numerous damaged houses, the reinforced concrete structural columns demonstrated great vitality. In combination with the reinforced concrete girths, they could improve the shockproof properties of the walls. During violent earthquake, the houses, though cracked, could still stand firm on the ground. Thus, loss was reduced and people's lives saved.

The reinforced concrete structural columns on houses in mixed structure were installed with an aim to enhancing the bearing capacity of the brick walls. After the Haicheng earthquake which took place in 1975, a small number of reinforced concrete columns were fitted to strengthen the relatively weak walls(see Figs 19 and 20). These columns played important role during the severe earthquake.

The reinforced concrete structural columns, if securely attached to the girths and girders running in two directions, and rationally attached to the cross and vertical walls with careful construction, could produce good effect. They could prevent a three-storeyed house in brick-concrete structure, an eight-storeyed house with inside frame structure and a spacious house in brick load-bearing structure from collapse in presence of serious cracks. However the discontinuous reinforced concrete structural columns were ineffective for anti-seismic effect of the houses (see Figs 21, 22 and 23).

The structural columns in the vertical direction constituted, with the girths, the frame enclosing the walls, thus improving the shockproof characteristic of the walls running in two directions. The horizontal girths and girders could also constituted the rims surrounding the floor slabs, thus improving their integrity. The shear strength of the brick masonry and the effect of the frame were not brought into full play at the same period. During the elastic period, the elastic force was mainly borne by the brick masonry. It was only after the elasticity of the brick masonry failed and the integrity was weakened that the frames began to play their roles. The reinforced concrete continuous structural columns and the girths were under the effect of a combination of forces, i.e. tension, compression, shear and bend. It was noteworthy that at the juncture of structural columns and girths, articulation of plastic nature were generally formed. Oblique cracks and horizontal cracks mostly concentrated at the juncture position. The walls near the juncture position were subject to shear or got smashed. The larger the area of this part, the more serious were the cracks at the ends of the girders and columns. Therefore, the connection between the structural column and the girth were of great importance.

Rational installation of reinforced concrete continuous structural columns and girths might improve the integrity and ductility of the houses in mixed structures, the load-bearing capacity and lateral force strength of walls. It was considered as one of the best approaches of ensuring the houses in magistoseismic area not to collapse in presence of serious cracks.

Suspended ceiling

Hanging down of the laths and falling-off of the plaster from the stalk and reed mattings were common occurance. One reason was dislocation caused by the failure of iron wires and nails due to corrosion. But this often took place in local places. The other reason was the coming-off of nails. In large houses, movements of the ornamental sound-absorbing boards and the gemetrically variable wooden keels were not well coordinated. In addition, short nails were used in construction. All these resulted in the coming-off of nails. Hanging down of the suspended ceiling caused by coming-off of nails were often large in area. The suspended ceiling near the gable in industry plants often got damaged due to collapse of the gable, and the woodwool slabs peeled off with the fall of the wood batten while the glass-fibre felts came off with the fall of the fibre boards. Furthermore, the ornamental glass laid in the suspended ceiling in spacious houses were sensitive structural elements which crashed in large area during the earthquake. Yet compared with the earthquake hazards of the entire house, the damage of the suspended ceiling in undamaged houses was slight (see Fig 24).

Lamps and lighting fixture

Tungsten lamps, daylight lamps and pendent lamps were all light hanging lamps. They were firmly secured with the purlins, ceiling joist and concrete ceiling. Wall mount lamps and ceiling lamps were also securely attached to the wood keels and concrete ceilings. Except that individual lamp-shades fell down due to the use of short screws, all of the lamps and lighting fixture remained basically perfect after the earthquake.

Stair, railings

The steel railings, wood railings or reinforced concrete fences could stand the rigorous trials of the severe earthquake. However, the brick fence was heavy in weight, poor in integrity and tended to crack or topple during the earthquake.

Maintenance walls

Maintenance walls of industry plants were built with clay bricks. There were two kinds of maintenance walls for the reinforced concrete framed bents and the frames. One was filler walls laid along the inner face of the beams and columns. The other was the overcloak walls on the outer face of beams and columns. The filler wall was great in lateral force strength and rigidity and was effective to a certain degree, in controlling the deformation of framed bents and frames because the walls could absorb the shock. Brick walls showed plastic damage for being poor in tensile strength. In case the strength of the concrete was high, X - shaped cracks were usually seen. If the strength was low, the wall would crack along the mortar-chalked joint or scattered. Under the repeated shock effect, the wall would utterly collapse. It was noteworthy that in multispan plants, the longitudinal columns were often unequal in rigidity, resulting in greater damage of the concrete columns, bracings, truss and roof sheathing due to uneven distribution of force along the individual columns and roof cover system. The effect of filler walls on frames also aggravated the damage of the beams and columns. Therefore, in analyzing the strength and rigidity of the framed bents and frames, the effect of filler walls should also be taken into account.

Outcladding walls were poor in horizontal strength and their fulcrums were extremely sensitive. After the earthquake, the outcladding walls, particularly the upper parts and the longitudinal walls universally collapsed due to the reason that connection between the walls and the reinforced concrete columns was poor and the walls were low in shear strength and bending strength (see Fig 25). In case that outcladding walls were used for maintenance walls, they should be handled with great care.

In industry plants where large prestressed concrete slabs, corrugated sheet iron, asbesto tiles were used for maintenance walls, better anti-seismic effect could be obtained. They remained basically perfect after the earthquake(see Fig 26).

Colonnades and canopies

The colonnades and canopies provided at the porches differed widely in rigidity. Apart from the menace imposed by collapse of the superstructures, the colonnades and canopies also experienced, at the parts connected with the main structure, horizontal push and pull forces and vertical shearing effect. Therefore, they all suffered damage to a certain extent. These sensitive structural elements should have been separated from the main structure to avoid damage(see Fig 27).

Ground and floor

Large area of ground and floor were damaged due to collapse of the upperstructure. The fabricated floor usually tore up the concrete layer along the joints because of structural response (see Figs 28, 29 and 30). In addition, in houses built in tectonic earthquake zones, along the bank of the river or on weak foundations, the concrete grounds, terrazzo floors and tiled floors often got separated and straight cracks, arc cracks, X - shaped cracks appeared. The violent earthquake proved that the cast-in-situ reinforced concrete ground and floor could keep good integrity.

Equipment

Collapse of houses or collapse of wall in part posed great menace to various equipment. The combined high-tension switches, ventilators, porclain machinery such as scrapers, generators and forming machines were destroyed, and coal hoisting was interrupted, resulting in the failure of production operations. Pumps, electrolyzers, liquid chlorine storage tanks were destroyed, causing corrosion of metals and hazards to lives by leakage of air and liquid. Damage of pipes and the fall of overhead travelling cranes rendered the production impossible to restore(see Fig 31).

In undamaged plants, supports and hooks came off with the cracking, scattering and collapse of the walls, thus giving rise to fall of vent pipes, water supply lines, sewers, steam lines, acetylene lines and transmission lines. These piplines often fractured at the flanges or welding points due to structural response. Brittle plastic and glass pipes were sensitive to earthquake. In chemical engineering plants, damage of the pipes of this kind often incurred secondary hazards. The transmission lines installed in the reinforced concrete slabs suffered damage because the slabs subsided in local places due to change of bearing conditions. Track-bending of the elevator and misalignment of the track of overhead travelling cranes often resulted from structural response, rendering them unable to operate.

Under the repeated effect of the earthquake inertia forces, equipment would incline, shift and run out of step. Large-scale milling, boring and planing machines underwent deformation. Foundation bolts of large and middle size lathes got loosened and plucked out resulting in loss of accuracy. A heavy cement kiln, wheels and cast iron supports of a ball mill were cut off by shearing. A cupola furnace 30 m in height and 3.8 m in diameter toppled as a result of the break of welding joints at its bottom. Collapse of a silo at its lower part crushed a loco. Damage of equipment by earthquake resulted in economical loss and rendered the restoration of production activities extremely difficult.

III. EXPERIENCES AND LESSONS

Houses in Tangshan built in the past had not taken the anti-seismic facilities into account. So far as non-structural units are concerned, what the architects considered were mostly their functions and artistry. Little had been made on the research of mechanics in designing. The grave consequence resulted from earthquake has not been taken into due consideration from the angles of interaction and mutual influence. Interaction and mutual influence might fall into three categories: 1). Effect of non-structural units on the response of structural system; 2). effect of response of structural system on the non-structural units; 3). effect of non-structural units on the response of non-structural units. The earthquake force is determined by the mass, damping and structural characteristics. Therefore, for reduction of the losses of structures, the grave earthquake effect should be considered in advance and in an all-round way. The response of structural system should be reduced to a minimum. Prudent consideration of the proper choice of favourable construction sites and of structural systems, materials and the type of various structural components is more important than the computation work involved. In this way, the design goal can finally be attained in combination with elaborate construction.

Estimation of vulnerability of non-structural components:

1). Pay attention to the integral connection between the overhanging structure and the main structure. Connection of the upperstructure as well as reliability of the lower bearings are all matters needing attention;

2). As for overhanging structures, pay attention to the connection between different materials and parts;

3). As for the attached structures, their connection with the main structure should be strengthened or keep them separated;

4). Reduce the height of the vertically self-supporting structures, enhance their strength and secure a firm connection;

5). Effect of rigidity and quality of in-laid structures should be taken into consideration. Pay attention to their integral connection with the main structure;

6). Ensure a firm connection between the vertically stretching structure and the main structure, enhance their strength and lower the center of gravity;

7). As for brittle materials, care should be taken to enhance their deformation strength or keep them in isolated state.

In summary, in order to reduce the loss of the non-structural components of the houses, possible reduction of the response of structural systems should be undertaken and reasonable estimation of the vulnerability of the non-structural components should be made.

Many-sided efforts should be exerted to ensure that the non-structural components match the main structures. Only in this way can the dual purpose of effecting seismic resistant and favourable economical results be obtained.

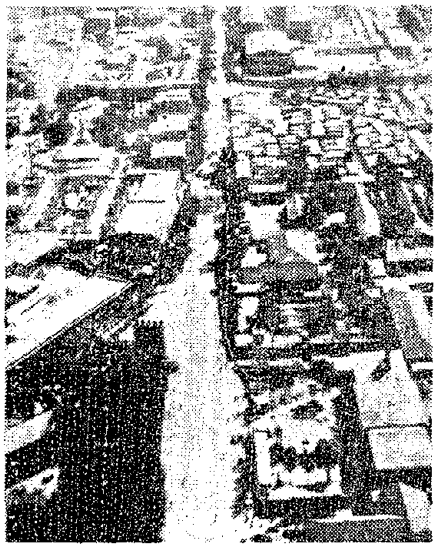


图 1　唐山市新华路街区震前原状

图 2　凤凰山南侧震前原状

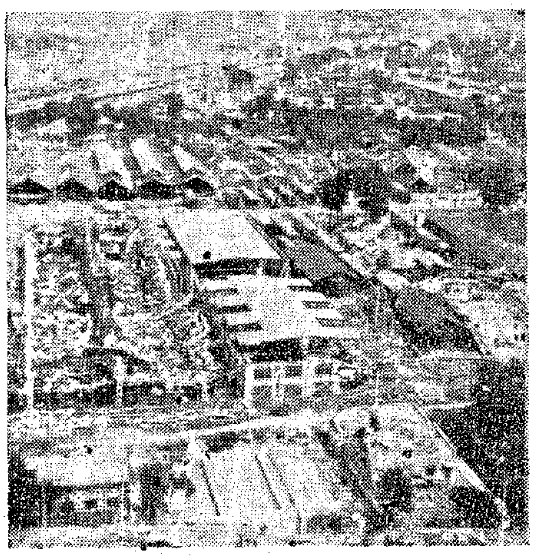


图 3　11度区唐山机车车辆厂震后状态

图 4　震后路南区复兴路南部平房区状态

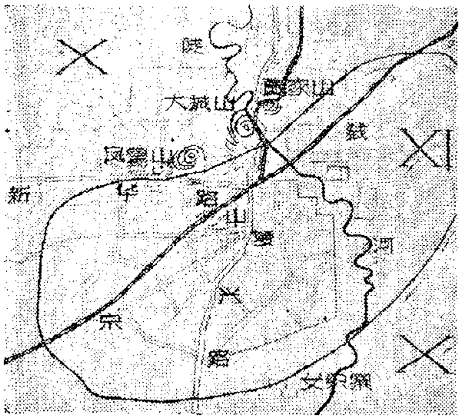


图5　唐山市地震烈度示意图

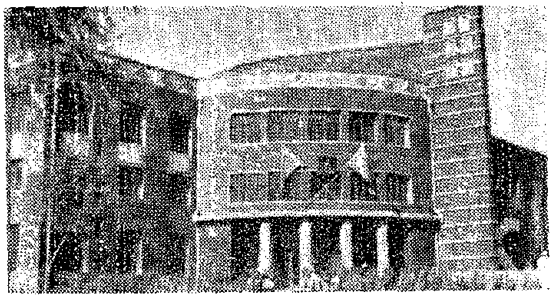


图6　唐山市华新纺织厂俱乐部震前原状

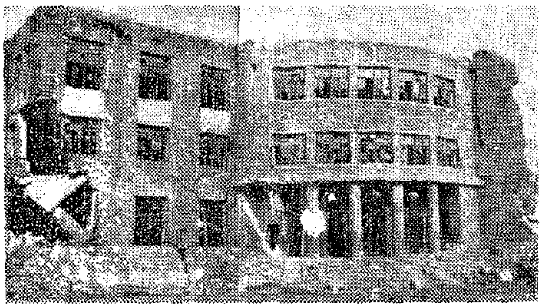


图7　11度区唐山华新纺织厂俱乐部震后仅有门厅部分残存

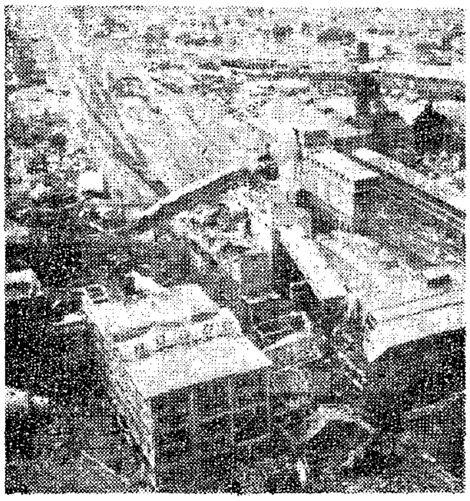


图8　开滦唐山矿全框架结构的洗煤楼震后设备间全部坍塌

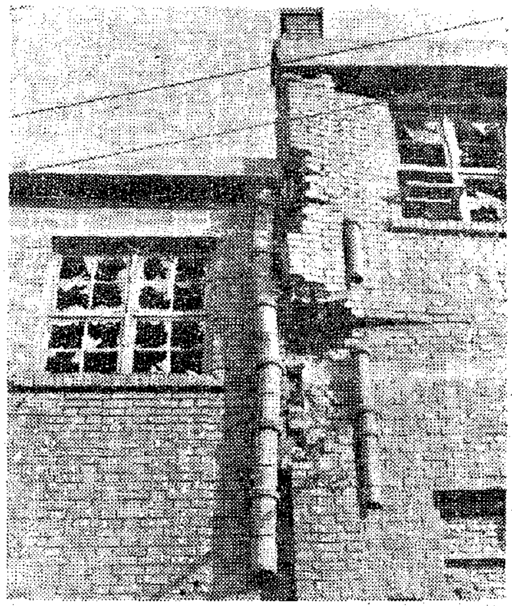


图9　11度区唐山华新纺织厂回收车间伸缩缝处，砖墙、玻璃、乚瓦落水管破坏状态

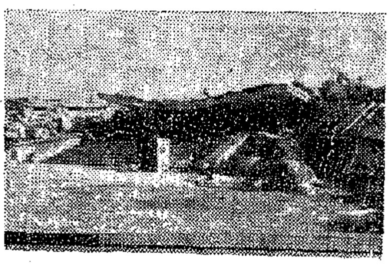


图10　11度区机车车辆厂的医院门厅、雨罩震后状态

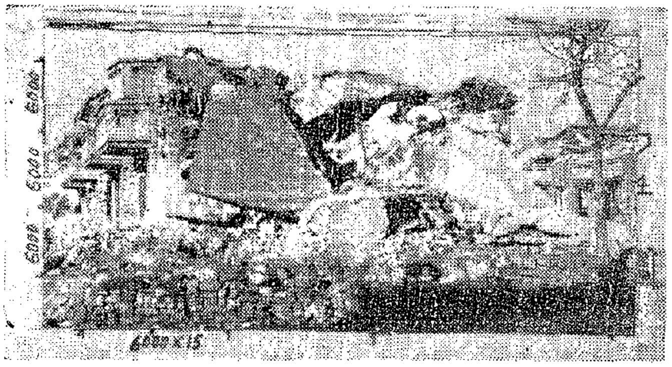


图11　10度区开滦三层砖混结构住宅楼阳台震后状态

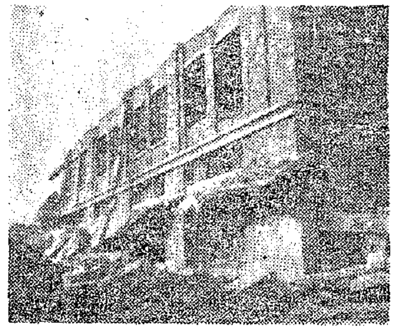


图12　10度区新市区百货商店雨罩震后状态

图13　10度区唐山第一瓷厂办公楼雨罩震后状态

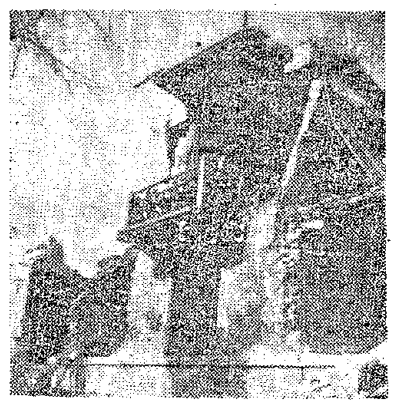
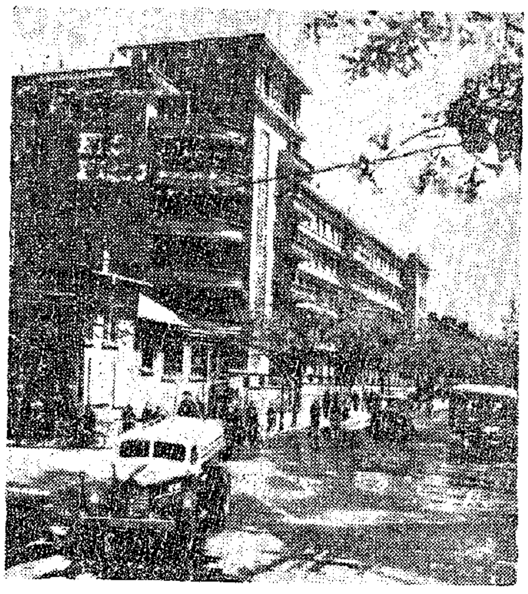


图14　10度区唐山第一招待所4号楼阳台雨罩震后状态



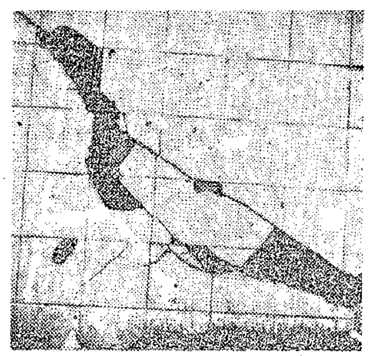


图15　10度区唐山市第四浴池砖墙上瓷砖贴面破坏状态

←图16　震前六层砖混结构的开滦医院楼

图17 震后开滦总医院转角楼部位残存

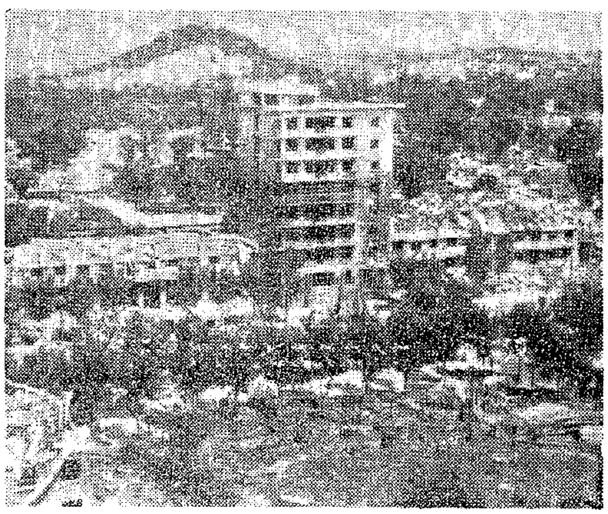


图19 10度区新华路北侧新华旅馆“。”门厅部位内框架结构“，”因为外承重墙加设了十二根钢筋砼物造柱“，”震后裂而未倒。门厅两侧五层和六层砖混结构的房屋倒塌严重“。”

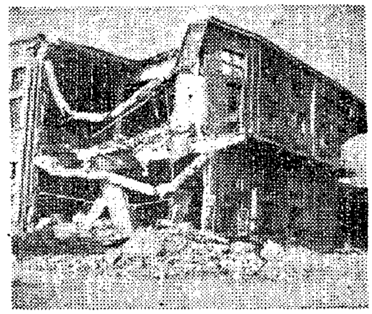


图18 10度区唐山市第九瓷厂内框架结构的成型车间震后状态

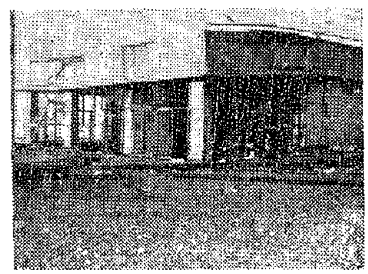


图20 新华旅馆门厅部位的屋面突出部分破坏状态

图21 11度区唐山铁道学院电机馆西侧震后状态

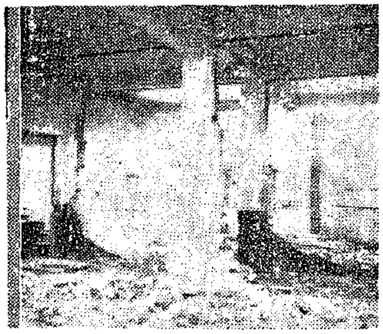


图22　电机馆的底层带有构造柱，楼板基本完好

图23　电机馆的二层无构造柱。这是圈梁的破坏状态

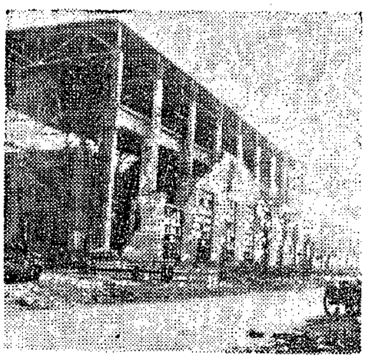


图25　10度区唐山冶金矿山机械厂组装车间外包砖墙倒塌状态

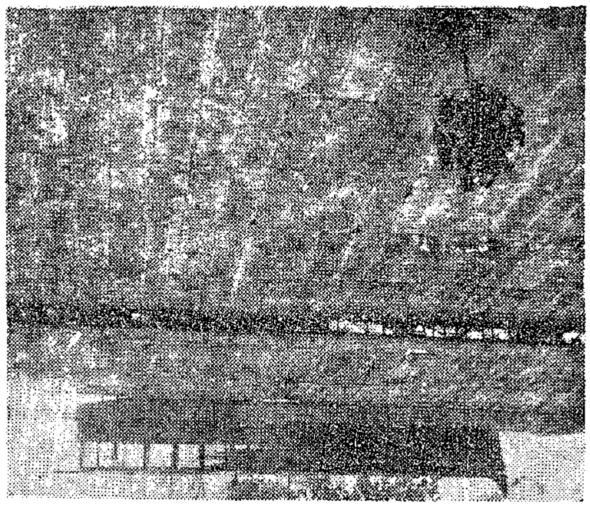


图24　11度区唐山交通局俱乐部木丝板吊顶和吊灯震后基本完好

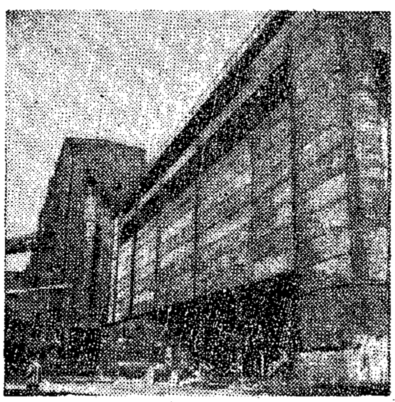


图26　10度区唐山钢铁公司二炼钢厂主厂房、予应力大墙板和瓦楞铁皮墙面震后完好

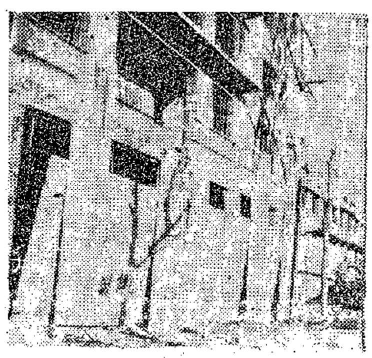


图27　新华旅馆门厅入口处雨棚倾复的状态

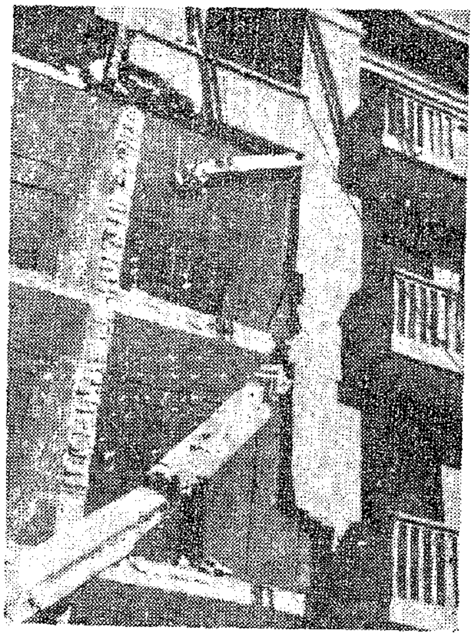


图28　10度区开滦新建四层混合结构的61楼南侧中部的破坏状态

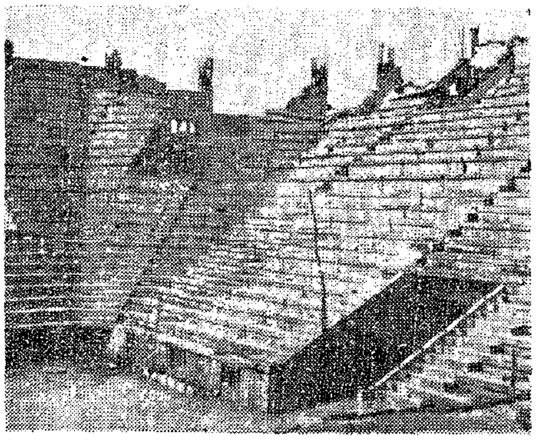


图30　10度区灯光球场震后混凝顶板严重解体

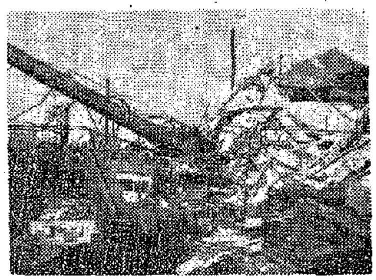


图31　11度区机车车辆厂机车厂房天车落地

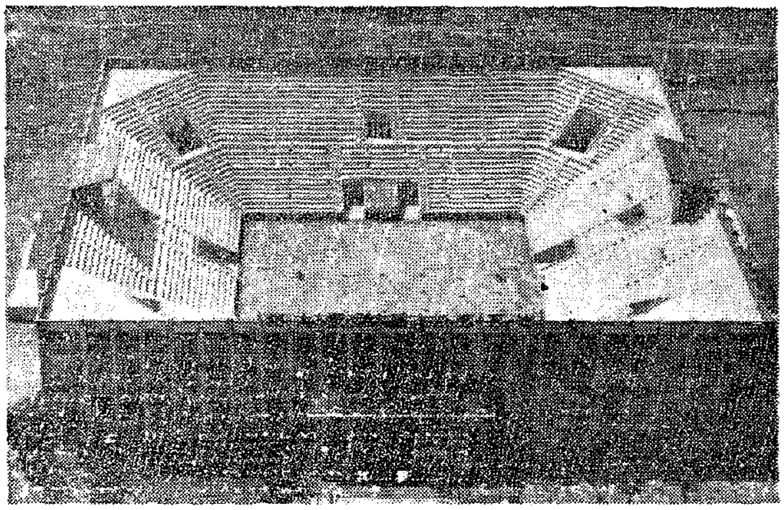


图29　唐山市灯光球场震前状态

CONSEQUENCES OF ARCHITECTURAL STYLE ON EARTHQUAKE RESISTANCE

by

MARCY L. WANG[1]

ABSTRACT

It has long been recognized that an individual building's characteristics of material, configuration, connection details, and overall concept strongly affect potential structural response in an earthquake. In contrast, architectural style, which broadly forms the framework for the choices of material, configuration, etc., has not traditionally been evaluated or seriously questioned in terms of its effect on building response in an earthquake. In regions of high seismicity, the use of design elements and concepts that are inappropriate for earthquake resistance, can frustrate even the most sophisticated of engineering strategies. Thus, overlap in the considerations of style and earthquake resistance would lead to a more location specific architectural vocabulary and most importantly, safer buildings.

The most dramatic example of how an architectural movement can be ubiquitously adopted throughout the world is that of the Modern Movement. In the 20th Century, its influence has extended far beyond its birthplace of Northern Europe and the Eastern seaboard of the U.S., both of which are places with relatively low seismicity. By now, cities in highly seismic regions, such as the West Coast of the U.S. and countries in Asia, North Africa, South America, and the Mid-East, are replete with buildings that are to varying degrees, the products of the Modern Movement.

Major earthquakes in urban areas, continue to reiterate lessons of a basic nature which are quite familiar to engineers, but which are still not obvious to an absent architectural audience. In the 1980 El Asnam Earthquake, it is paradoxical that relatively new, engineered concrete frame structures, designed after a severe earthquake in 1954, by French or Algerian architects in a Modernist fashion, suffered as much or more severe damage as their more traditionally styled neighbors. One should not infer that architectural design or style is the sole culprit in such a disaster; however, the prevailing pattern of destruction suggests a general correlation to style which cannot be ignored.

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INTRODUCTION

The effects of architectural style on the seismic behavior of buildings are often adverse and unforseeable by architectural designers. This regrettable situation exists to some degree in all well populated seismic regions for traditional, vernacular architecture as well as contemporary, modern architecture.

Historically, the difficulty in considering earthquake hazard when formulating architectural design concepts as intrinsically as other environmental problems, has been due to two major factors, first, the long recurrence period of destructive earthquakes, and second, the complex nature of seismic forces.

Compared to other natural environmental and climatic phenomena such as temperature extremes, seasonal hurricanes, monsoons, tornados, or snowstorms, the recurrence periods of major destructive earthquakes are irregular and extremely long. Both human memory and life span tend to be shorter than the recurrence periods of major earthquakes; it is not surprising that architectural styles indigenous to a particular region would reflect adaptations to certain commonly expected natural conditions or events, yet such an accommodation in style for seismic reasons is far more rare. Even in aftermaths of earthquake disasters, when rebuilding of towns or cities takes place with a high consciousness of seismic hazard integrated into building design and planning, subsequent generations soon forget the lessons.

The nature of earthquake forces is both potentially enormous in magnitude, and difficult to analyze. Even the simple correlation between building weight and seismic inertial force on a structure might not be obvious. Furthermore, consciousness of such elementary principles would be of little help if, as in many regions in the world, the only readily available building materials are heavy and unreinforced masonry of stone, adobe, or brick. Even today, scientists find phenomena such as soil structure interactions and seismic wave attenuation, more complex the more they are studied.

In recent times, assuming that there are no major economic obstacles or limitations of materials, small to moderately sized buildings can usually be easily designed to a degree of acceptable earthquake risk. In the state of California, subsequent to several major earthquakes starting with the 1906 San Francisco Earthquake, building codes and legislation have aided engineers and architects in designing buildings to be seismically safer. Consequently, there have been relatively few fatalities in California buildings that have been built according to these codes. One of California's advantages is, of course, its healthy supply of wood, steel and reinforced concrete, which has mostly replaced previously dominant masonry for important structures.

Although the U.S. has suffered few deaths in recent earthquakes, there have been alarming though infrequent instances of multi-story modern, "seismically engineered" buildings that have collapsed during moderate to large earthquakes. In California, the low death toll resulting from collapsed buildings is partly due to fortuitous conditions such as the time of day of earthquake, and to the fact that no shock approaching the 1906 Quake's magnitude has recurred in a densely populated area. Earthquake struck cities around the world in the past decade, however, reveal statistics that are much more grim in terms of deaths and injuries of people in collapsing modern buildings. For example, only one year ago, in El Asnam, Algeria, it is estimated that over 20,000 people died, many of them in modern (post 1954) buildings. Since the materials of modern construction, i.e. steel and reinforced concrete, can be made to be even more strong and ductile than materials of previous eras, and construction and engineering are more advanced in modern times than ever, the large proportion of modern compared to older building failures is at first glance bewildering.

THE ROLE OF MODERNISM AS A STYLISTIC AND SEISMIC PROBLEM

In countless cases of failed modern buildings, engineers dispatched to investigate the causes for failure discover that a major defect of the building lies not in the engineering, but rather the overall architectural design concept, comprising those elements and configurations, which are largely related to stylistic or functional decisions. Architects are usually and conspicuously absent from earthquake disaster investigations that commonly include geologists and engineers. Likewise, the architectural profession on the whole is relatively oblivious to the impact and relevance that architectural style has in regard to building seismic resistance.

Architectural tendencies of the twentieth century are important to examine in terms of effects of style on seismic behavior. The unprecedented universality of a single style that "Modernism" can boast has led to the almost interchangeable use of the words, "modern" defined as contemporary, or of the recent past, and "Modern" capitalized or "Modernist", alluding to the "Modern Movement" in architecture that originated in Western Europe early in this century. For most architects who now automatically design buildings with a Modernist vocabulary, the omnipresence of Modernist expression has obscured the original intents and issues that the founders of the Movement debated. Thus, not only is the engineer often confounded by stylistic and seemingly arbitrary architectural decisions, the architect himself often has unwittingly slipped into the easy trap of recycling Modernist expressions without really considering their consequences in seismic and other terms. Due to the typical segregation and working styles of architecture and engineering disciplines in the U.S. and in many other countries, the engineer is often unable to influence the early architectural concept of a building with respect to earthquake hazard, but is relegated to making the best of a difficult situation. Considering 1) the ubiquity of modern, and Modernist architecture, 2) its poor track record in earthquakes, and 3) the contradiction of poor performance despite the availability of high level technology and excellent building materials, architects should be compelled to analyze the roots of the Modernist tradition, and the aspects of style which so drastically contribute to building failures.

The definitions of Modernism in an architectural context are as numerous as the number of architects who contributed to its evolution. Whether Modern Architecture consists of a set of formalist rules as did preceding architectural fashions, or consists in contrast, of a way of thinking and a philosophy that manifests itself automatically in the physical entity of a building without conscious imposition of elements of style, depends upon which Modern Architect one consults. Upon arriving from Germany, in the U.S. to teach at Harvard University in the 1930's, Walter Gropius stated,

> "My intention is not to introduce, a so to speak, cut and dried 'Modern Style', from Europe, but rather to introduce a method of approach which allows one to tackle a problem according to its peculiar conditions. I want (a young architect) independently to create true, genuine forms out of the technical, economic, and social conditions in which he finds himself instead of imposing a learned formula onto surroundings which may call for an entirely different solution."[2]

Ludwig Mies van der Rohe, who held a similar view of the role of "form" in architecture, said, "Form is not the aim of our work, but only the result; we should develop the new forms from the very nature of the new problems."[3] Although neither of these pioneers included seismic resistance as one of the "new problems" to be solved, the spirit of including all major factors of a building's surroundings into the style, was foremost in the philosophy.

The lofty concept of modern architecture as the means to customize solutions to individual architectural problems, unfettered by stylistic presuppositions, was not universally shared. Charles Eduard Jeanneret, better known as Le Corbusier, presented his definition of "style" and arguments for why a modern style is necessary, in his famous document Ver Une Architecture in 1923.

> "Style is a unit of principle animating all the work of an epoch, the result of a state of mind which has it own special character . . ."[4]

He further explains this need for a new style:

> "The history of Architecture unfolds itself slowly across the centuries as a modification of structure and ornament, but in the last fifty years, steel and concrete have brought about new conquests, which are the index of a greater capacity for construction, and of an architecture in which the old codes have been overturned. We challenge the past, we shall learn . . ." that a style belonging to our own period has come about; and there has been a Revolution."[5]

As one of the principal prophets of modern architecture, Le Corbusier dogmatically spelled out the requisite physical elements of modern architecture which he felt was necessary to insure its proper execution.

Almost every architect educated in the Western World is familiar with the "five points of a new architecture" that Le Corbusier published in his

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2. Architectural Record (May 1937).

3. Blake, Peter (1970).

4. Le Corbusier (1927).

5. Ibid.

Ouevres Completes. Le Corbusier wrote that the new architecture MUST have:

1) pilotis (or first story columns to free the ground).

2) a free plan (free of the systematic restrictions of bearing walls).

3) a free facade (an external skin such as a curtain wall which is independent of the structure holding the building up).

4) roof terraces (which are encouraged to be constructed with sand covered by thick cement slabs laid w/staggered joints, the joints seeded with grass.)

5) strip windows (horizontal lengths or strips of fenestration which could occur as uninterrupted bands).[6]

It is equally as likely that almost no structural engineer has heard of "the five points", but upon hearing of their description, would recognize several of these elements as very familiar from schemes that architects regularly hand them to structurally design. Structural engineers would recognize pilotis as "soft stories" which have been the failing point of dozens of modern buildings in earthquakes all over the world; the free plan as the generator of highly assymmetric plans which result in excessive torsion during the earthquakes; the free facade which may pose complicated motion and interaction between the structure and non-structural elements; the roof terraces, which involve an absurdly heavy load of thick concrete, sand and soil; and strip windows, that effectively destroy the resistance of shear walls, or create short columns that are highly susceptible to shear failures.

In the course of his career, Le Corbusier designed several never-built projects of administrative skyscrapers intended for diverse places such as Algiers, Nemours, (both in North Africa), Antwerp, Paris and Buenos Aires, in which his points of architectural programme (pilotis, roof garden, etc.) were rigorously applied. The inability of Le Corbusier to have these projects built belies the eventual widespread acceptance of his points that became known as the general elements of the international repertoire of modern architecture. The appearance of such architectural points in major buildings throughout the world, attest to the ultimate influence that Le Corbusier had on townscapes and cityscapes, independent of climate, culture, and seismicity. By the nineteen fifties, many architects were already becoming aware that Corbusier had become a towering architectural influence, largely due to the ease of copying those elements he advocated, and that the curtain wall or free facade, and pilotis, "had become conventional formulae like the columns and pediments in neoclassical times, that is, they had been accepted in obedience to a preordained poetics, not to resolve the real demand of the country."[7] It should be

6. Le Corbusier (1967).

7. Benevolo, Leonardo (1971).

noted, that in his writings, Corbusier backed his theories with plenty of functional, social, economic, and climatic, as well as philosophical-political rationale. He and his architectural peers initially were tacitly speaking of architecture in the context of Western Europe, although the self-fulfilling label of the "International Style" (given to a major strain of modern architecture) clouds this very important point. Whatever the conscientious and sensitive thought that was behind Le Corbusier's and other Modernists' theories, they were subsequently interpreted as rules-of-thumb for architectural modernist design.

It is this type of undiscriminating formalism in Modern Architecture which has resulted in the inappropriateness of many buildings in cities around the world, culturally, climatically, and even seismically. As the 1981 recipient of the American Institute of Architect gold medal, Jose Luis Sert observed in a recent interview:

> "I must say that I am most horrified when I see the application of modern architecture in some countries, even remote countries where they didn't know what they were working with...where some of the elements of modern architecture are copies, and repeated, misinterpreted, badly built. To me, that is the best argument against modern architecture."[8]

Undoubtedly, third world countries suffer the most from this type of unquestioning acceptance of Western technology and cultural impositions. The lack of native engineering and other technical training in major building projects, result in foreign supervision which usually is from countries which share neither environmental nor cultural characteristics.

In retrospect, of all the giants of modern architecture, including Mies, Wright, Gropius, etc., Le Corbusier has had the most blatant effect on the look of modern buildings around the world. The cause of his strong influence in Europe and colonial French North Africa is obvious; his indelible mark on the rest of the world may be due to the directness with which he defined his elements of modern architecture in his prolific and passionate writings, and also due to the coincidence that he worked primarily in reinforced concrete, which is still the most accessible modern material in much of the world. It is by chance that many of these elements of Le Corbusier's invention, are antithetical to principles of good seismic resistance when explicitly applied onto buildings in earthquake zones.

Assessment of current architecture in the Modern style, and preceding fashions of building design in regard to their effect on seismic resistance of buildings, may help designers consider the repercussions that future architectural styles imported from non-seismic areas may have on resistance to earthquakes. Disaffection with the Modern style has recently lead to a movement, noncommitally called "Post Modernism." The term, "Post Modernism" is as difficult to define as "Modernism", and is still such a young trend that the various strains of this "after Modernism" style have not produced many large buildings yet. It is also not clear whether the movement will survive long enough to have as universal an impact on world architecture as the Modern Movement did. Superficially, it appears that

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8. Architectural Record (May 1981).

the "going back to zero" and elimination of traditional historical reference that Modernists advocated, is being challenged. The seismic implications of Post Modernism, at this point, seem limited since there is no one dominant consensus among its proponents as to what it constitutes. The elimination of certain Modernist tendencies such as pilotis, is an improvement seismically, however, the vogue of stone curtain walls, pediments, and other ornamentation on skyscrapers which have emerged in some recent American works, are less desirable seismic aspects of the trend. Like Art Nouveau, for example, the notion of Post Modernism is particularly arty and without regard for factors beyond visual for the genesis of its forms and definition. Post Modernism is so totally detached in spirit from the functional, technical aspects of building, that the juxtaposition of the words "Post Modernism and seismic concerns" have quite an odd ring. Architects in earthquake prone areas must take particular caution when designing in such an image oriented style, to not overlook the less ethereal but very real dangers of seismic hazard.

The balance of this paper will consist of examining the effects of contemporary and traditional architecture on the seismic behavior of buildings in Algeria, California, and Japan. These seismic areas represent architecturally, historically, and economically diverse situations.

CASE STUDY I - ALGERIA, A DEVELOPING COUNTRY

Like much of North Africa, Algeria regularly experiences strong seismic activity due to its location on the collision belt of the continental African plate, and the overriding Eurasian plate. Despite a high degree of seismicity, there has not emerged a style of architecture which specifically reflects this environmental hazard through the choice of configuration, materials, or architectural elements, in this country. Rather, the architectural influences on Algerian buildings, like other aspects of Algerian culture, are more of an eclectic result of the region's turbulent history, which includes occupation or invasion by such diverse groups as the Romans, Turks, Berbers, Spanish, Arabs, and French. Buildings which were constructed through the years of French Colonization (1848-1962) are still quite prevalent in Algeria. In the capital city of Algiers, the ancient Casbah is surrounded by 19th and 20th Century buildings which are architecturally reminiscent of buildings in French Mediterranean cities.

Through much of the French Colonial era, the problem of seismic resistance was not foremost on the list of problems that the French found most worrisome about building in Algeria and the other North African territories. A March, 1936 issue of L'Architecture d'Aujourd'Hui, which was devoted to the subject of French architecture in North Africa, summarizes the concerns that Europeans had about building in that part of the world. In the articles which were about Algeria, the major problems cited were the differences in cultural, aesthetic, and sanitation standards that Europeans and Algerians were accustomed to in their respective housing. The French were especially disturbed by the lack of adequate sanitation facilities in Algerian households, to which the Europeans attributed a high mortality rate among the native Algerians. For their own housing and more prestigious public buildings, the French would import architecturally current styles from their homeland, usually incorporating modification for climatic, and insect pest conditions. In the case of both European and indigenous buildings, the threat of earthquake was hardly mentioned and never stressed as a major problem of construction.

The same issue of L'Architecture d'Aujourd'Hui contained proposed town plans prepared by Le Corbusier for Algiers, and Nemours. Although, the proposals must have appeared quite radical and futuristic at the time, and although those specific city plans were never implemented,[9] the sketches of the buildings that Le Corbusier envisioned in his plan, are startlingly similar to buildings that the French were to erect in Algiers two decades later. Le Corbusier's Ouevres Completes, contains a 1933 proposal for the prototypical Algiers apartment building (fig. 1) which Le Corbusier describes in this way:

> "The building is located on a site characteristic of this hillside city. A primary proposal: there should be a municipal regulation obliging all buildings along the boulevard paralleling the bay to be constructed on columns, thus leaving the ground floor entirely free so as to allow the inhabitants of Algiers, an unobstructed view to the sea."[10]

These proposals which were published by L'Architecture d'Aujourd'Hui, and by Le Corbusier in his books, La Cite Radieuse (or the The Radiant City) and Ouevres Completes, 1910-1965, are interesting for their prophetic accuracy. The highrise apartment building shown in figure 2,[11] is part of a five building complex which was constructed by the French in Algiers, in 1952. Such reinforced concrete buildings with the ground story pilotis (columns) and intermediate floor pilotis (columns) which are referred to by structural engineers as "structural discontinuities", and "soft stories" are extremely common in Algiers; they constitute one of the more frightening seismic hazards that the city could realize in the case of a major earthquake.

The inevitable occurrence of a major destructive earthquake happened during the French occupation of Algeria on September 9, 1954, in the town of Oreleansville (now known by its Arab name, "El Asnam"), which is located 230 km. or 150 miles west of Algiers. In this 6.7 Richter Magnitude shock, most of the town was destroyed, and 1300 people were killed. This event abruptly awakened the French to the enormous earthquake hazard that existed, and they immediately began the reconstruction of Orleansville (on the same site) with intentions of making it much more earthquake resistant.

9. Le Corbusier believed that his plan was rejected due to "the weakness of the authorities." In 1934, he write, "Algiers drops out of sight like a magnificent body, but covered by the sickening scabs of a skin disease. A body which could be revealed in all its magnificence through the judicious influence of form. . .But I have been expelled, the doors have been shut in my face. I am leaving and deeply I feel: I am right, I am right, I am right. . ." from The Radiant City (1964).

10. Le Corbusier (1964)

11. L'Architecture d'Aujourd'Hui (June 1955).

The June, 1955 issue of L'Architecture d'Aujourd'Hui published extensive documentation of the destruction, ambitious planning schemes for the new city, and several reinforced concrete building projects which were already under construction, The cause of the great devastation in this earthquake was attributed to the bad quality of construction in old, traditional, masonry architecture which was often neither reinforced nor tied.[12] Steel frame construction was encouraged for this area, however, the expense of steel in this region, precluded its widespread use, even after the devastating earthquake, and the traditional masonry buildings were largely replaced by concrete frame structures.

Typical construction from the French period of building after the 1954 earthquake consisted of two way reinforced concrete frames with three meter (about ten feet) modules. The floors were hollow precast concrete elements with a 4 to 5 cm. thick topping of unreinforced concrete. Interior as well as exterior walls were usually built of hollow precast concrete infill. Such construction was generally used for two, three, and four story buildings, in El Asnam. Usually, the building was elevated from the ground story on pilotis (columns) for major, official buildings, and sometimes housing, whereas apartment buildings were commonly built upon a short crawl space supported by stubby columns. This crawl space, called a "vide sanitaire" (sanitary void) was not a stylistic but rather a construction convention inherited from France. In addition to widely used reinforced concrete, in which they had great confidence, compared to the old masonry construction, the French devised and applied a rudimentary earthquake engineering code for building in Algeria.

Following the expulsion of the French in the 1962 Revolution that gave Algeria independence, the Algerians assumed the task of keeping up with the enormous housing and other construction needs in their country. While the general mood of the country at that emotional time, was to do away with vestiges of their French colonial past, many aspects of French culture were so entwined with the Arab, Berber, and other Algerian influences, that it was difficult to segregate and evict them. Thus, it was easy enough to symbolically change street names from the French, to Arabic, and to disregard the French seismic code for construction. Much more difficult to eradicate, was the trend of Corbusian construction, the form of which was the only modern type that the Algerians, and much of the world for that matter, was familiar with. Thus, the concrete frame with masonry infill, often completed with ground story pilotis or vide sanitaires was more of a French tradition which the Algerians automatically continued, very probably without any architectural convictions about what they were building. Understandably, the housing shortage and the need for immediate post war reconstruction, were the crises which led the government to concentrate on volume of building rather than analysis of what to build. Superficially, the Algerian buildings eventually began to assume more elaborate ornamentation, which was quite "un-Modern", but which is also not identifiable with any particular style, and probably is derived from an individual designer's notion of Arabic, historic allusion. (Their version of "Post Modernism"?) Usually this addition of ornament, either applied or in the complication of architectural forms, only aggravated the seismic problems.

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12. Ibid.

On October 10, 1980, a series of major shocks, the largest of which was 7.2 Richter Magnitude, originated on the same fault on which the 1954 earthquake occurred. In small, very rural villages close to the earthquake epicenter, innumerable people died in the predictable collapse of traditional, unreinforced, stone and adobe buildings. The city of El Asnam (previously called Orleansville), which had been almost completely reconstructed only some twenty years prior to this event, again suffered complete and immediate collapse of dozens of buildings; this time they were modern reinforced concrete buildings. It is estimated, that approximately eighty percent of El Asnam was irreparably destroyed in the 1980 earthquake.

The buildings of El Asnam prior to the last earthquake, could be roughly divided into three categories: first, those few buildings which withstood the 1954 event, and were strengthened; second, those buildings which the French built, and are described in a previous paragraph; third, the buildings built by the Algerians after Independence. Not surprisingly, many of the buildings in the first category did not survive this second large earthquake. The modern buildings of categories two and three, on the whole, behaved worse than the more traditional buildings of category one, which were generally constructed of massive unreinforced masonry walls with heavy tile roofs.

In several cases, modern buildings suffered complete pancake failures; structural failure was immediate and resulted in the heavy floor and roof slabs of concrete stacked up one on top of the other. The Hotel Cheliffe, the FLN (National Liberation Front), the Town Hall, the Palace of Justice, the Police Station, are just a few examples of pancake failures in which hundreds of people perished. In most of these failures, collapse was so complete, that there was no way to determine exactly what these buildings looked like, or how they exactly failed. The most awesome structural failure in terms of size of building, extent of failure, and lives lost, is that of the Ain Nasr Market, a huge square block (100m x 100m) of mixed use commercial and housing functions. The development, which had three and four stories, contained shops and restaurants on the ground floor, and housing on the upper levels. This complex, which was built by the French after the 1954 earthquake, completely collapsed, except for one corner that remained precariously standing. It was from this small fraction of the structure, that investigating engineers determined the main causes of the devastating failure. General characteristics of the building were enormously heavy slabs of concrete, exceptionally thick heavy concrete nonbearing partitions, were supported by relatively spindly ground story pilotis. The specific mechanism of failure, can be seen in the way the slabs are beginning to fail in the portion of the building that was left standing. The inner columns were apparently designed weaker than the columns on the perimeter of the building, and failed first, followed by the inward pancaking of slabs, and inevitable failure of the rest of the columns. Deaths in this building complex alone exceeded the one thousand mark.

There were several examples of buildings nearly completing construction by the Algerians, which suffered severe collapse. In each case, the buildings were of reinforced concrete, and suffered major damage due to soft story failure, and other configuration problems. A new medical clinic, almost completed but not yet occupied, is an example of a Modern building with Algerian ornamentation that contributes to its distinctive appearance. The building is elevated on pilotis, more for the functional purpose of accomodating a garage on the gound story than as a conscious carry over from the Modernist trademark of lifting the building visually off the ground. An almost identical twin building adjacent to this clinic, differs only in the lack of ground story columns that provide garage space. The situation is almost laboratory like in providing the comparison in behavior of two nearly identical structures in an earthquake, with the sole difference of ground story stiffness. It is also believed, that the location of the stiff interior elevator core, was placed in such a way that large torsional moments resulted. The clinic collapsed, its twin (without pilotis) did not.

Another Algerian construction project which partially collapsed, was that of the new Cultural Center, which was built of concrete frame and masonry infill. In figure 3a the two structures are pictured, one completely collapsed, and one still standing. They appear to have been identical, and the reasons that one collapsed and other didn't is not clear. The standing structure, however, does provide some clues as to contributing factors for its twin's failure. The cultural center buildings are of that more ornamental and heavy "Algerian" style of architecture, superimposed upon a regular Modernist concrete frame. This adds seismic risk. The heavy cantilevered portion of facade, that seems to be more of a stylistic expression than any conceivable functional requirement, is also unwise in any normal situation, let alone a highly seismic one.

Finally, not far from the downtown of El Asnam, a new housing project (fig. 4), which bore a remarkable resemblance to "peasant farmer" housing sketches that Le Corbusier published in the twenties,[13] consisting of dozens of two story housing units elevated on piloti, were completely destroyed as a result of the failure of the piloti. Again, it was extremely fortunate that no one was living in these units yet. During investigation of the disaster, it was not uncommon to discover in the rubble, that in addition to unusually thick and heavy concrete or masonry wall loads, the heavy roof slabs were even further loaded with large quantities of sand. The decision to use heavy walls and roof slabs insulated with sand, stems from the need for protection from the summer heat, and in an example of how accomodating one environmental condition could aggravate another situation.

Less visually spectacular failures, but resulting in as great an economical loss, were the common failures of vide sanitaires, the short columns of which absorbed much of the lateral force. Multistory concrete buildings were often undamaged above the vide sanitaire, however, were dropped or tilted by several feet as a result of the short column failures.

It was noted by the U.S. Earthquake Reconnaisance Investigative team in their report, that,

> "No new lessons were learned from the performance of buildings during this earthquake...the collapse of

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13. "The peasant farmer is to be separated from Mother Earth - his house is to be built up in the air, in a clear, rarified atmosphere; raised above the vegetative, crawling and fertile earth; he is to live, descending from the cloistered dwelling to care for the few mechanical tasks that a rationally planned and run farm now calls for." p. 10 C.B. Troedsson Two Standpoints Toward Modern Architecture (1951).

these buildings did not occur because they were not engineered structures, or because attempts were made to economize the use of structural materials. The collapse occurred due to the fact that the buildings were not architecturally designed and engineered for the effects of strong earthquake ground motions."[14]

In summary, many of those architectural characteristics which greatly affected the seismic resistance of modern buildings can be traced to the European International Style, particularly Le Corbusier, whose well intended advocacy of the universal use of pilotis, did not consider the dire consequences of their performance in earthquakes. Another product of the Modern Movement, the frame, can be an extremely good type of structure in earthquake areas, however, in third world countries in particular, such frames can be difficult to properly design, in reinforced concrete. The very serious problem of quality control and construction supervision in Algeria, further accentuates this type of building's weakness.

The Algerian government, which had already been concerned with the problem of earthquake hazard in urban areas prior to the 1980 El Asnam earthquake, had commissioned a group of Stanford University engineers to create an earthquake building code for Algeria, based upon historic seismicity. Subsequent to the 1980 El Asnam Earthquake, a code of Algerian regulations, which contain many similar items to the U.S.'s SEAOC Recommendations, has been produced. Emphasis of the code was placed on restricting the types of conrete construction allowed. For examples, in the El Asnam regions, no 'ductile' frames will be allowed, since design practice, material quality, and construction methods just cannot provide the required performance. All concrete systems must be braced by continuous 100 percent percent seismic load resisting shear walls,[15] in place of the Corbusian frames that have been in such common use. (Fig. 5a & 5b)

This radical limitation on building type is not extreme considering that El Asnam was destroyed twice in a little over two decades. What will be interesting to watch for, is the spontaneous effect that this edict has upon the architectural style of buildings in the second reconstructed city of El Asnam.

CASE STUDY II - CALIFORNIA

The settlement of California after the 1849 Gold Rush, occurred in a remarkably short time, during which the state was transformed from wilderness to a highly developed built environment in many areas. It was merely a century ago that the railroad and other new industries produced an economy which stimulated the building of cities and universities, that required the construction of large public, educational, and commercial buildings. The Californians, who were anxious to emulate the culture of Europe and Eastern North America, imported architectural styles often without much adaptation, for their own buildings. The use of certain

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14. V. Bertero, H. Shah et al. (1981).

15. T. Zsutty (July 1981).

traditions of architecture had symbolic and social significance in the late 19th Century, and Revival styles (Greek, Gothic, Italianate, Second Empire) were extremely popular.[16]

In the 1800's, a popular awareness of earthquakes existed in California simultaneously with the fashion of heavy and highly ornamented architecture even after strong shocks occurred in 1857, 1865 and 1868. The earthquake of 1868, centered in Hayward, damaged every building in the town, and cornices were especially victim to the shaking. San Francisco had its share of damage in that event, nevertheless, the desire for monumental architecture was stronger than a fear of earthquakes.

The price of insistence on styles inappropriate to the seismic location, came in the form of San Francisco's devastation during the Great 1906 Earthquake. Parapets, cornices, and ornamentation of Revival Styles could not withstand the tremendous seismic forces, and masonry construction which was inherent to these fashions, suffered badly due to its heavy weight, faulty workmanship, poor materials, and lack of reinforcement. San Francisco City Hall was representative of disregard for appropriate construction belying the grandeur of massive buildings. This brick structure of irregular plan, featured a central tower, prominently topped by a steel framed dome. The shock denuded the tower skeleton of bricks and many of the walls toppled.

It is possible that the small urban scale of San Francisco, which is one of the more charming distinctions of the city, and the dearth (until recently) of very high rise buildings, was in part due a remembrance of the 1906 catastrophe. An engineer of the time, Charles Derleth Jr. stated his opinion that many failures in the 1906 Earthquake were due to excessive building height; he recommended two stories as a height limit for school buildings and places of assembly.[17] As economic pressures mount, and confidence in technology rises, real estate developers increasingly invest in high rise buildings despite the possible seismic consequences, and despite the loss of San Francisco's traditional scale and urban character.

South of San Francisco, on the bucolic campus of Stanford University, severe failures befell major buildings, such as the Memorial Church. The University buildings were of sandstone masonry and influenced by the midwestern architect, Henry Hobson Richardson's style of heavy rusticated stone bearing wall construction. This Richardson Romanesque style

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16. One analysis of this fascination with architectural revivalism in San Francisco during this era, is given by Judd Kahn in his book, Imperial San Francisco - Politics and Planning in an American City 1897-1906 (1979): "First, city merchants saw San Francisco as the capital of a developing American commercial empire in the Pacific. Second, both the planners and their sponsors took as their models the great imperial cities of Europe; the Athens of Pericles, the Rome of the Caesars, the Paris of Napolean III, and Baron Haussmann. The architecture itself and the layout of streets, parks and other open spaces were intended to inspire awe in an unruly public, to encourage identification with the glories of a state that stood enshrined in enormous public buildings."

17. Jordan, David S. (1907).

was blamed for many of the campus's building failures. Although the founders, Jane and Leland Stanford were certainly aware of the existence of earthquakes in California, they could not be deterred from choosing this style of building for the University which was built as a memorial to their only son. The emotional implication which that style of building had for Jane Stanford is expressed in her writings: "The 'stone age' which has been so frequently alluded to, no doubt was irritating and tedious to some connected with the University, but to me the erection of these stone buildings has had a deep and important significance."[18]

In the decades following the 1906 earthquake, California became a leader in earthquake legislation, codes and research. Nonetheless, from the architectural standpoint, the state was no less immune to the strong influence of Modern Movement leaders. The late California architect, George Simonds recalled in 1968:

> Going back only as far as I can remember, the twenties and early thirties, styles in small buildings drew inspiration from the Cotswold cottage, the Brittany half timber, the Tudor cottage...the Mediterranean of Spanish, Southern French and Italian influences...followed by the rediscovery of our own heritage - the Cape Cod cottage, the Pennsylvania salt box, and Monterey Colonial. Whatever the style, such buildings had a certain common denominator, symmetrical distribution of cell like spaces with partitions from floor to ceiling...These buildings, whatever their antecedents, whatever their construction had an inherent stiffness.
>
> During this same period, the prophets of the new appeared: Frank Lloyd Wright, and Le Corbusier. They freed us from the rigid and symmetrical styles of the past with "organic" architecture and the "new internationalism," characterized by symmetry, by open plans and great expanses of glass... and buildings on stilts called "pilotes" (sic) in the vernacular.
>
> Gone is the inherent strength and stiffness of the old... The use of pilotes (sic) on the ground floor, with open space for pedestrians where the summer breezes waft and the winter winds howl, provides little resistance to horizontal forces.[19]

Mr. Simond's evaluation of the effect that elements of modern style, such as piloti, could have on the earthquake resistance of buildings was astute and prophetic. In fact, in the same year that Simonds wrote this article, the design for the Imperial County Services Building (which is by now a famous example of a modernist seismic failure) was underway, and it was only three years later that the Olive View Hospital was fated to fail. Both the failures of the Imperial County Services Building (El Centro, 1979), and the Olivé View Hospital (San Fernando, 1971) are unequivocal

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18. Allen, Peter C. (1980).

19. Simonds, G. (March 1968).

confirmation of the view that architectural concept may be more detrimental to the seismic survival of a building than any other design decision.

The most recent major failure of a modern building in a California earthquake, that of the Imperial County Service Building, is a classic example of how, despite excellent technology, materials, engineering, and public awareness of earthquakes, an important building can fail in a moderately large earthquake due to architectural concept. This Modernist, six story, reinforced concrete office building, which was elevated off the ground by piloti, failed to the point of near collapse, at its piloti. An investigation of the failure did not reveal abnormally substandard concrete or workmanship, the foundation was excellent, and the structural design had been executed according to the latest seismic requirements of the Uniform Building Code. It is quite obvious that the Modernist style of the building was the real weakness in its seismic design.[20]

The high level of earthquake consciousness that exists in the town of El Centro, makes this particular failure especially interesting. The townspeople, who frequently experience small to moderate seismic activity, had been assured that the new Imperial County Services Building would be much more earthquake resistant than the older facility across the street, thanks to modern technology. (The structurally undamaged neo-classic older building is visible in figure 6, beyond the failed pilotis of the newer structure.) It is possible that the Imperial County Services Building's designers either felt that the building configuration was not so detrimental to seismic safety, or that the structural design could compensate for weaknesses introduced by architectural concepts.

Ironically, the configuration and open ground story of the Imperial County Services Building, encouraged by the Modern Style, did not enhance either the function or the aesthetics of the building. The structure seemed to have assumed its form to follow the fashion of modern architecture. A conscientious design sensitivity to the environmental requirements of El Centro, including that of seismic resistance may well have resulted in a richer architectural product in style and function.

Although much data about structural behavior is being gathered from laboratory testing by universities and industry, these experiments can represent only a minute fraction of the existing architectural conditions in California. Unfortunately, the empirical results of non-structural component and structural tests, rarely correlate architectural style to engineering interpretations in such a way that architects can directly apply the new knowledge.

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20. The mechanism of failure in this three by five bay structure, was the result of an east west moment resistant frame system, in conjunction with a more rigid north-south shear wall system, which were designed to accomodate the architectural conception. On the east side of the building, a massive concrete shear wall was eccentrically situated over four piloti. In response to random earthquake motion, the frame behaved flexibly in the east west direction, while the east shear wall placed a simultaneous compressive load on the east piloti which lead to hinge failures in the frame direction of these piloti. Other details, such as insufficient confining steel in the column, may have aggravated the problem.

The effect of heavy architectural elements on light structural frameworks may turn out to be very important in another great urban earthquake. Concrete panel wall systems, or brise soleil, and granite or travertine curtain walls (which are becoming a popular Post Modern application) are certain to have a more powerful effect on a flexible steel moment resisting frame, than the light aluminum panels of the recent Hi Tech fashion.

CASE STUDY III - JAPAN

The evolution of Japanese architecture is perhaps the most interesting among countries located in high seismic zones. Prior to modern times, Japan's architectural heritage had been chiefly influenced by that of China, in addition to her own climatic and seismic conditions. The country's narrow width and location between the Pacific Ocean and Asian continent, gives Japan countless microclimates, many of which include extremely hot, humid summers, and torrential monsoon rains. Japan's situation over a plate boundary where one tenth of the world's seismic energy is released, accounts for the approximately dozen earthquakes over Richter Magnitude 7.0 that have occurred there in the last hundred years alone.

The extensive use of wood, especially cypress, and nearly absent use of masonry (until European influences in the 1800's) are major aspects which distinguish traditional Japanese architecture from Chinese architecture. The reasons for this deviation are numerous: Japanese forests have been an abundant and replenishible source of building materials; wood is light and ductile, thus appropriate for seismic areas; post and beam construction in wood is a climatically desirable form of construction in the humid, warm, Pacific zones.[21]

Although traditional Japanese architecture was exceedingly sophisticated in achievement of aesthetic and environmental excellence, the style and construction still subjected buildings to some common natural hazards. Wood construction has always been vulnerable to the wind fires of the region, and the heavy roof tiles which are a beautiful but hazardous architectural feature adopted from China, have killed countless Japanese in earthquakes. Both of these aspects of Japanese building were major problems in the Great Kanto (or Tokyo) Earthquake of 1923.

In the late nineteenth century, Japan's centuries old architectural traditions underwent a great upheaval, concurrent with and in fact due to the political downfall of the Tokugawa Shogunate in 1867 followed by the pressure of foreign countries on Japan to discard its seclusion policy. An unprecedented current of Western influence was force fed to the Japanese in all aspects of the culture, including architecture. In 1868, an

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21. Since the thermally moderating effect of the Pacific Ocean gives Japan a generally mild climate, the seasonal monsoon rains are the most demanding climatic conditions that traditional Japanese architecture responds to. Steep roof slopes were adopted to drain the great rainfalls, and this in turn, necessitated long floor plans, and horizontally extended elevations. Unbearably hot and humid summers were alleviated by the openness and lightness intrinsic to the post and beam construction of Japanese design.

Englishman, T.J. Waters, was the first of several Europeans to introduce Western masonry construction to Japan. Not only was this construction the complete conceptual antithesis of Japanese post and beam building, the pretentious European eclectic styles were totally foreign to the Japanese. As a result, the Japanese had to resort to sheer imitation of these stone structures, such as in the Akasaka Detached Palace, in Tokyo, executed in French court style in 1899.

The absurdity of transplanting Western Eclectic styles into Japan was eventually realized not only in terms of the inappropriateness of the style to the culture, but also with regard to the poor earthquake resistance of such construction. The Great Earthquake of Nobi in 1891, was a motivation for Riki Sano[22] and others, to pursue studies on earthquake resistant construction. By 1895, steel frame and reinforced concrete buildings were introduced to Japan and by 1909, were in wide use. By 1920 this type of construction largely replaced masonry buildings.

Just when the Japanese began to actively question the conflict of traditional Japanese Western Eclectic styles in their country,[23] the modern movement in architecture was beginning to take on momentum in Germany, Austria and other parts of Europe and was soon to reach Japan. The spirit and theory of the modern movement was much closer to that of traditional Japanese architecture than the Western Eclectic styles which could not be easily assimilated into Asian culture. The Modern movement itself, was influenced by Asian concepts of aesthetics, especially Japanese. The elegance of Katsura was "discovered" and admired by European Modernists such as Bruno Taut and Walter Gropius. Even Frank Lloyd Wright, who usually denied the existence of inspirations beyond his own genius, admitted that "Japanese prints had attracted me and had taught me a great deal. The elimination of the insignificant, the process of simplication of which I had already embarked, found confirmation in these prints...The art of the Japanese was a more autonomous produce of more authochthonous conditions of life and work, therefore in my view much closer to the modern spirit than the art of any European civilization living or dead."[24]

In the 1920's, the Japanese reciprocated Wright's appreciation of Japanese art, when they invited him to design the Imperial Hotel in Tokyo. This structure, which is by now of legendary status among examples of earthquake resistance success stories,[25] was aesthetically a marvelous

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22. Riki Sano continued to be a leader of Japanese earthquake resistance construction in the 20th Century. In 1941 he chaired the compilation of papers for a book entitled "The Principles of Earthquake Resistant Construction," a comprehensive and pioneering document on the subject.

23. At the 1909 Convention of the Architectural Institute of Japan, the problem of the coexistence of Western and Japanese architecture was hotly debated although no definite conclusion was reached.

24. Benevolu, Leonardo (1971).

25. Robert Reitherman's excellent article on the myth of this hotel, (AIA Journal, June, 1980) considers several questions about the validity of several of the claims concerning the structure's remarkable resistance to the 1923 Earthquake.

building of masonry on a steel frame, in which both Japanese influences and Wright's own organic philosophy were elgantly blended. Although the effectiveness of Wright's amateur engineering theories that he incorpbrated into the design (such as floating the building on the soft soil) are problematic, at least he considered seismic hazard to be a major criteria of the building design, and the hotel escaped serious damage in the 1923 Kanto Earthquake. Despite the hotel's demolition in 1968, the building continues to be a model of sublime architectural design which is executed with a high consciousness of seismic hazard.

Cross poll nization of Japanese and European ideas in architecture flourished in the nineteen tens and twenties. The Japanese were more inclined to embrace the Modern Movement on their own terms, as a new architecture which was created partly in response to the technologies emerging in the new century. Numerous young architects of Japan had direct contact with the rising European Modernists in these decades; Kunio Maekawa and Junzo Sakakura were disciples of Le Corbusier, and Iwao Yamowaki was affiliated with the Bauhaus.

The Kanto Earthquake of 1923, which demolished Tokyo and Yokohama by both ground shaking and fire occurred during the rising trend of modernism in Japan. The tragic event provided architects further opportunity to rationalize the new technologies of the Modern Movement, in order to create an architecture that was stronger and more earthquake resistant than the buildings destroyed in the earthquake.

During the Imperialist and World War years of the thirties and forties, Modern architecture was considered politically out of step with the nationalistic fervor. Fascistic, semi Western architecture, such as the National Diet building of 1936, and later, ultra nationalistic architecture, as expressed in the Imperial Museum of Tokyo in 1938, turned out to be merely an interlude in the development of the Modern style in Japan. In the post war years, when Japan had made enough of an economic recovery, and began to catch up on housing and other building needs, Modern Architecture made a stronger appearance than ever, this time without the rationalization for its seismic resistant potential that the 1923 Earthquake provided. In fact, the movement had become almost purely an aesthetic, dialectical one, since in the immediate years following the war, architects could only design "paper architecture," rather than building with functional requirements. It is no wonder that buildings such as the Reader's Digest Tokyo Branch Office executed by a foreigner, Antoni Raymond, with its bold cantilever beam construction and questionable seismic resistance, created controversies among construction engineers on whether it would be strong enough in a severe earthquake.

The look of modern Japanese architecture was similar to that of the Western World although most of the architects were in fact Japanese. Both steel frame, and reinforced concrete structures were erected on a large scale. The forms and elements and expressions assumed by the most renowned Japanese architects such as Tange, Maekawa, and Yanagi, took on distinctly Corbusian looks. Bold reinforced concrete buildings, were often lifted from the ground on pilotis, and provided with brise soleil, or strips of fenestration on the facades. The Hiroshima Peace Center by Tange '56 is a monumental group of reinforced concrete buildings with an abundant use of pilotis (fig. 7).

Beyond the obvious Western influence of Corbusier, Mies, Nervi, and other Modernists, there is a distinctly Japanese character to many of the bold reinforced concrete structures that alluded to the wooden tradition of Japan's native architecture. Unfortunately, the hovering giant members in concrete, carry much greater potential seismic disaster along with their great weight than the ancient timber elements that they recall. Unlike the predecessor style of Japanese traditional architecture, the forms are derived, not from the environmental stringencies of seismic and climatic conditions, but rather from a total image orientation allied with an over-confident reliance on modern technology to carry the contradiction through.

The skill and facility of Japanese Modern architectural forms are undeniable. Their visual impact is seductive both to their creators, and to architects from the rest of the world. By the sixties and seventies, modern Japanese architects began to enjoy a recognition which ranked them with the greatest modernists of the twentieth century. But the emphasis on form as sculptural and artistic rather than functional and environmentally responsive, is clearly evident in the absence of mention of seismic problems, and rate mention of climatic conditions, in recent literature on Japanese architecture. Publications such as Japan Architect and the slick Global Architecture strictly promote the visual aspect of architecture works.

Japan's current dichotomy between its state of the art seismic research, and its leading edge in world architecture, is even more extreme then that of the United States, especially considering the leadership it has achieved in both fields, and the percentage of the country that lies in severe seismic zones.

Rather than tsunami, or ground failure (such as the spectacular liquifaction at Niigata in 1964), ground shaking is the effect of earthquakes which causes modern building failure that is linked to poor architectural concept. As in the United States, Algeria, and other countries, reinforced concrete structures tend to fail in earthquakes more than other types of modern construction. Failures of such buildings are more common to Japan, compared to the United States, in proportion to the greater seismicity of the archipelago. The following examples of recent examples in modern Japanese architecture, are merely two instances of a phenomenon that is not rare.

A 7.9 earthquake severely shook northern Honshu and southern Hokkaido on May 16, 1968, and was officially named the "Tokachi-Oki Earthquake of 1968" due to the existence of several previous major earthquakes in that region. Nearlu 700 buildings totally collapsed, and over fifty people died in the event. In this disastrous earthquake, several reinforced concrete and concrete block buildings sustained major damage, although no steel building suffered severe damage. There were not many reinforced concrete buildings over four stories high, in the area, thus most of the severely damaged concrete buildings were low in height. Nearly all of the damaged buildings had been designed in accordance with the Japanese Building Standard Law and Structural Standards by the Architectural Institute of Japan, which requires design for at least 18% gravity. It was generally found, that buildings with well distributed shear walls (that is, not too eccentrically placed, and oriented to resist lateral force in two orthogonal directions), behaved very well. On the other hand, most failures occurred mainly due to ground shaking, and to "Framed structures with very weak (shear) walls or no walls" in which "some buildings failed at the top and bottom portion of the columns due to bending moment and axial load, and (barely) escaped a total collapse. Those severe damages were liable to be found in such buildings where very weak walls were present and where in addition the walls were arranged extremely eccentrically so that twisting vibration was liable to superpose the normal vibration."[26]

A situation similar to that of the Imperial County Services Building, where a modern building collapsed across the street from an older structure it replaced, occurred in the 1968 Tokachi-Oki Earthquake, in the case of the Hachinohe City Hall which had been designed to the latest Japanese seismic standards. The older City Hall of wood, was essentially undamaged, although it was so old that "the ridge line was waving and the wood lath base of the wall was partially decayed."[27] (fig. 8a) In contrast, the new city hall (fig. 32b), which was a reinforced concrete building with a basement and five storied penthouse, suffered remarkable damage in the collapse of the top of the penthouse which fell onto the main portion of the building, in addition to failure in shear and bending of columns on the first and second story. Results of tests for concrete strength, taken from samples of the building's columns showed that the material had more than required strength. The problem was apparently due to an overall architectural concept problem, where the configuration resulted in an "unfavorable distribution of the stiffness,"[28] in the conclusion of the Architectural Institute of Japan.

Another building, of the many which failed in this quake, Hakodate College, severely failed due to architecturally related design features. The 1966 reinforced concrete building had an extremely long and narrow plan, that was angled 135 degrees at a point one third from one end. What appeared to be a three story building after the earthquake was the consequence of a failure to a four story building, where all the ground story columns and walls collapsed. There were almost no shear walls in the structure. It has been concluded by investigators of the damage that shear force which acted in the long direction of the building played the leading role in the failure, and the extremely long configuration of the plan led to a complication of the structure's dynamic behavior, such that "damage must have been essentially concerned with the basic concept of design"[29] rather than specific engineering design.

CONCLUSION

The examples of modern building failures in Japan and the U.S.A., in which material quality was above reproach, and seismic design was up to the latest engineering requirements, emphasizes the problem that even technologically advanced countries face; that is, how can one control and codify regulations for seismic design and building safety, considering the

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26. Suzuki, Z. (1971).

27. Ibid.

28. Ibid.

29. Ibid.

infinite possibilities of architectural design, without repressing those conceptual possibilities. Like the United States' Uniform Building Code, the Japanese National Building Code attempts to express the spirit of the code without unreasonably limiting architectural creativity. The 1980 version of the Japanese code states: "The purpose of this aseismic design method is that the buildings shall withstand with almost no damage, the moderate earthquake motions which would occur several times during the use of the buildings, and shall not collapse, nor harm human lives by the severe earthquake motions which would occur less than once during the use of the building."[30] The Japanese code is more strict than the American code in the requirement that "Buildings exceeding sixty meters in height shall have the special permission of the Minister of Construction through the detailed review of the dynamic behavior of the structure by the Board of technical members."[31] Despite even this limitation, it is clear that the Code is not preventing seismically delinquent buildings from being constructed.

Artistic self indulgence in architectural design, is perhaps the smaller part of the problem. More importantly, there must be realization of the consequences that architectural style, and concept have on seismic behavior of buildings. This is the first step toward more responsible design. In the process, the architectural profession may find that rather than being a limit on creativity, seismic considerations, like other environmental issues, can provide profound design expression. Cities in seismic areas around the world continue to become higher and denser with new buildings which are often designed with style as one of the predominant criteria. Considering the complexity of earthquake forces and the high level of engineering capability in many parts of the world, the tragic outcome of a major earthquake in any modern city can only be guessed. What is certain, is that the architect is in an excellent position to sweeten the odds without necessarily reducing the cultural and aesthetic expression that has always been central to the art of building.

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30. Japanese National Building Code 1980 (1980).

31. Ibid.

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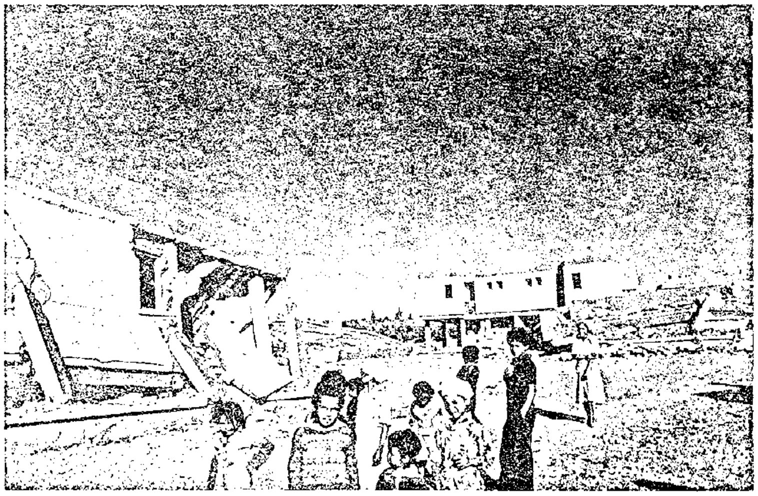


Figure 4a. Several nearly completed, concrete dwellings in this government built complex on the edge of El Asnam, collapsed on their piloti. The inspiration for their design may have had its source in Corbusier's early sketches for peasant farmer housing, an example of which is shown in Figure 13b.

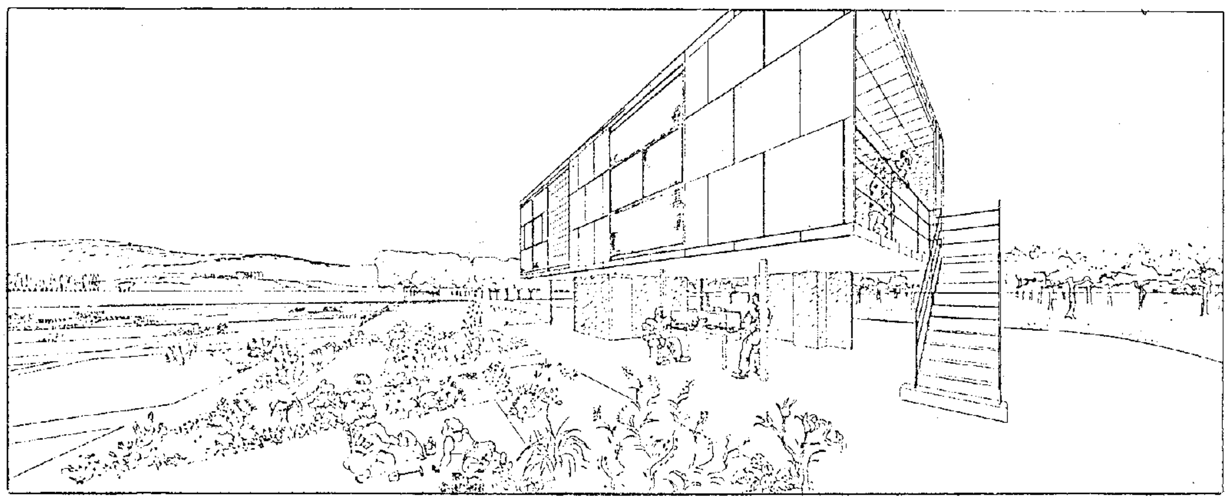


Figure 13b. Le Corbusier's romantic notion of separating the peasant farmer from Mother Earth by raising his house up in the air in a clear rarified atmosphere, was illustrated in sketches, and published in his Ouevres, 1929-34.

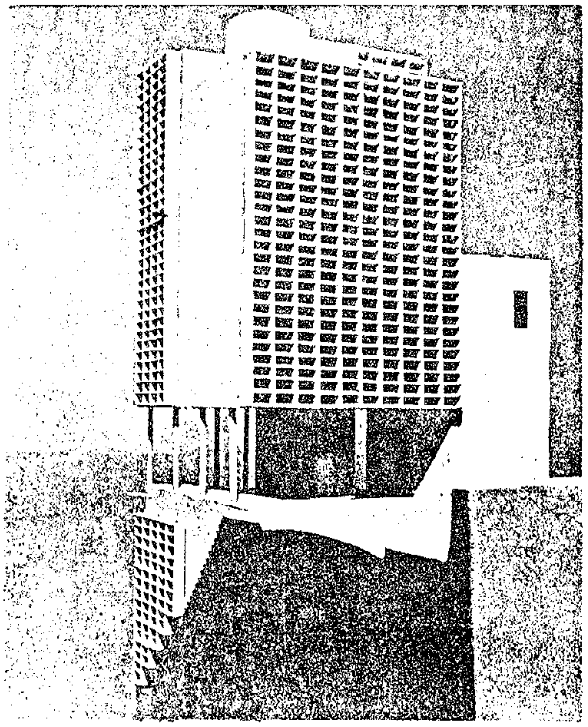


Figure 5a. Model of Le Corbusier's proposal for a prototypical apartment building in Algiers.

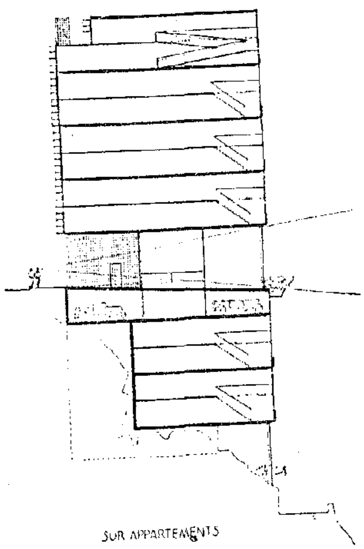


Figure 5b. A sketch of the proposal that demonstrates the architect's intention of giving everyone a good view of the city by lifting the building off the ground on pilotis.

From Le Corbusier 1964

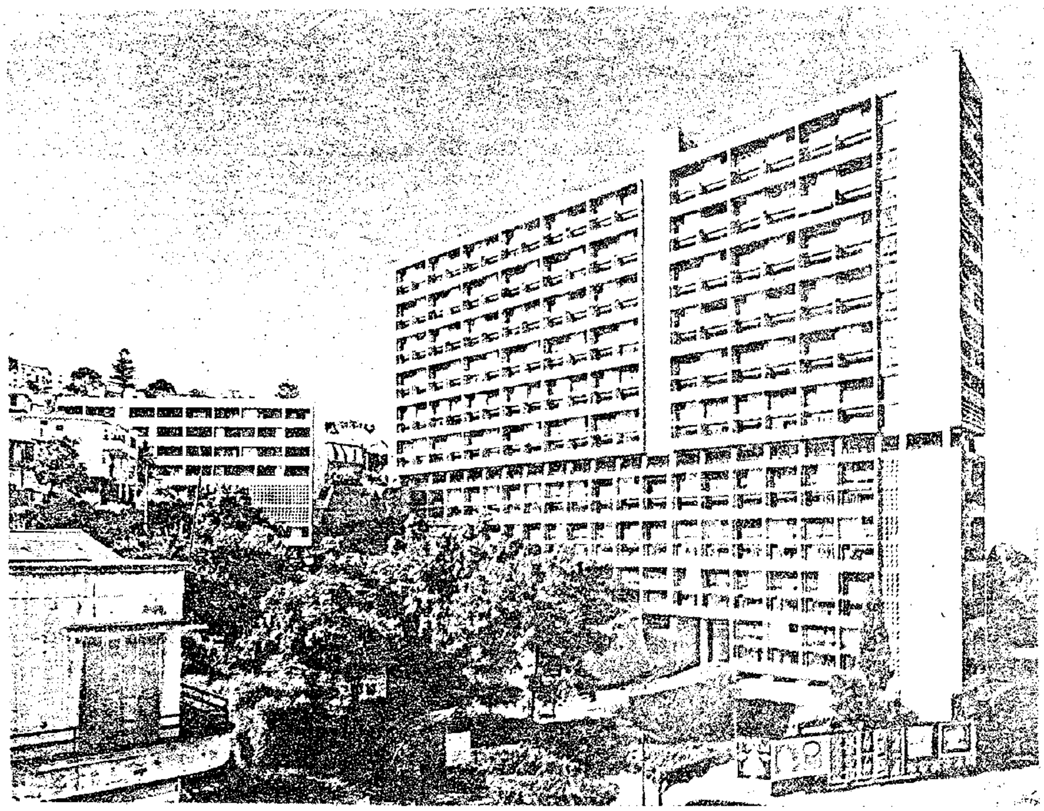


Figure 6. This apartment building with an intermediate story on pilotis, is part of a five building complex built by the French in the early 1950's.

From L'Architecture d'Aujourd'Hui June 1955

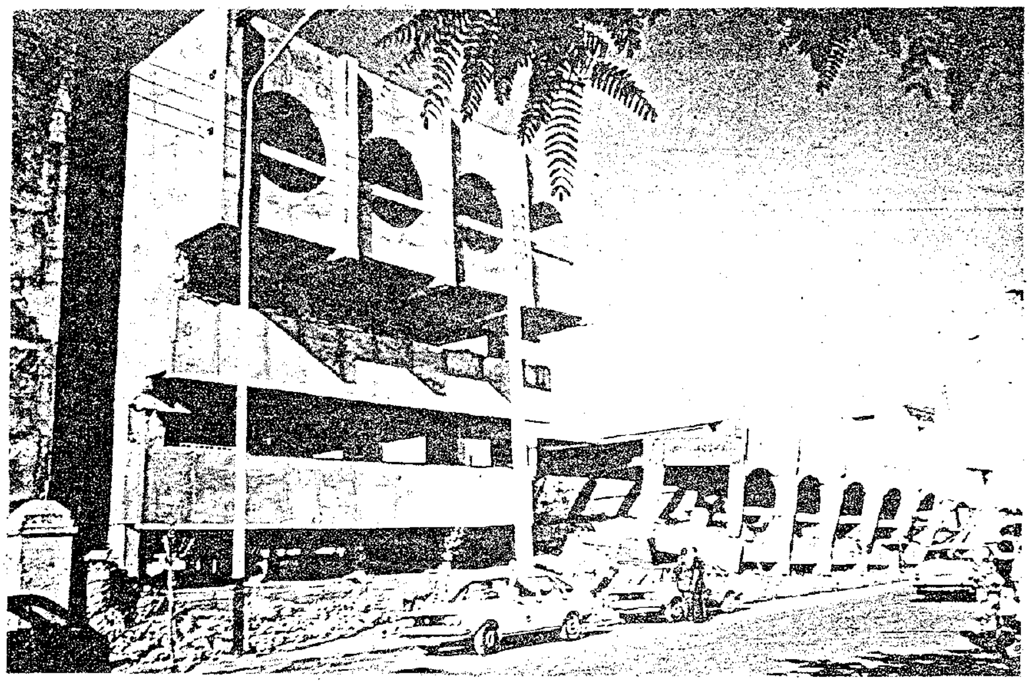


Figure 15a. These buildings were to be the city's Cultural Center. The right structure is completely collapsed.

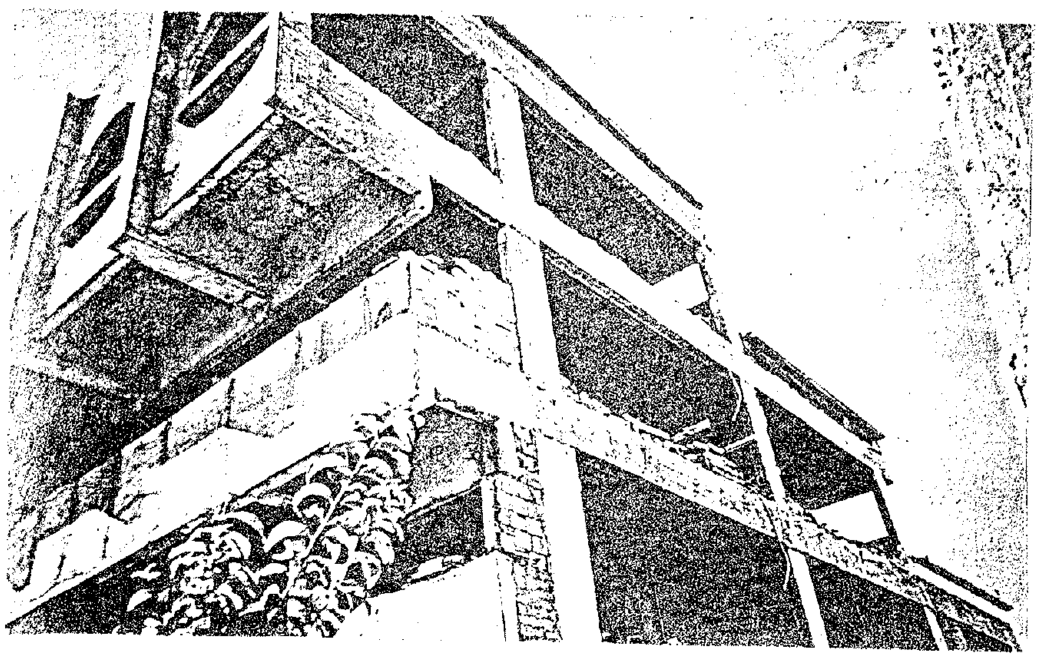


Figure 15b. Construction of the Cultural Center was of concrete frame and masonry infill.

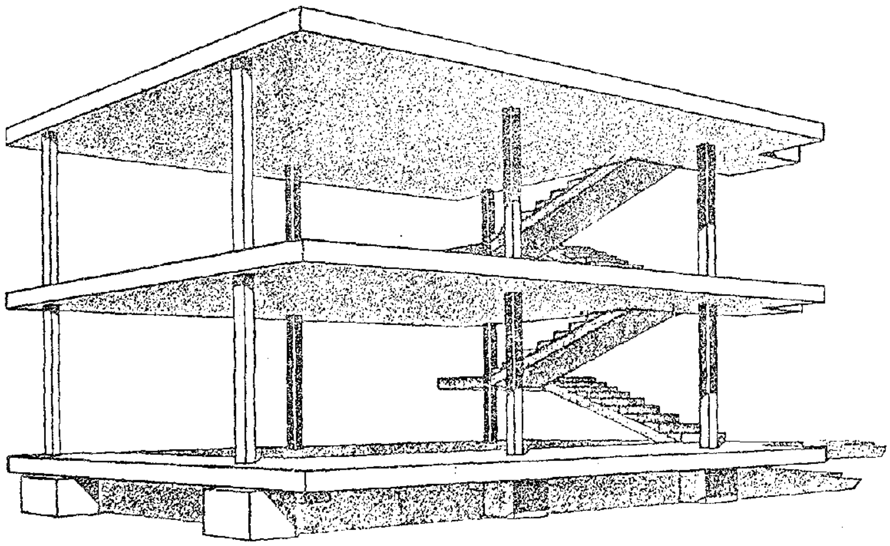


Figure 17a. Le Corbusier believed that the frame skeleton would forever "free the plan".

Le Corbusier 1967

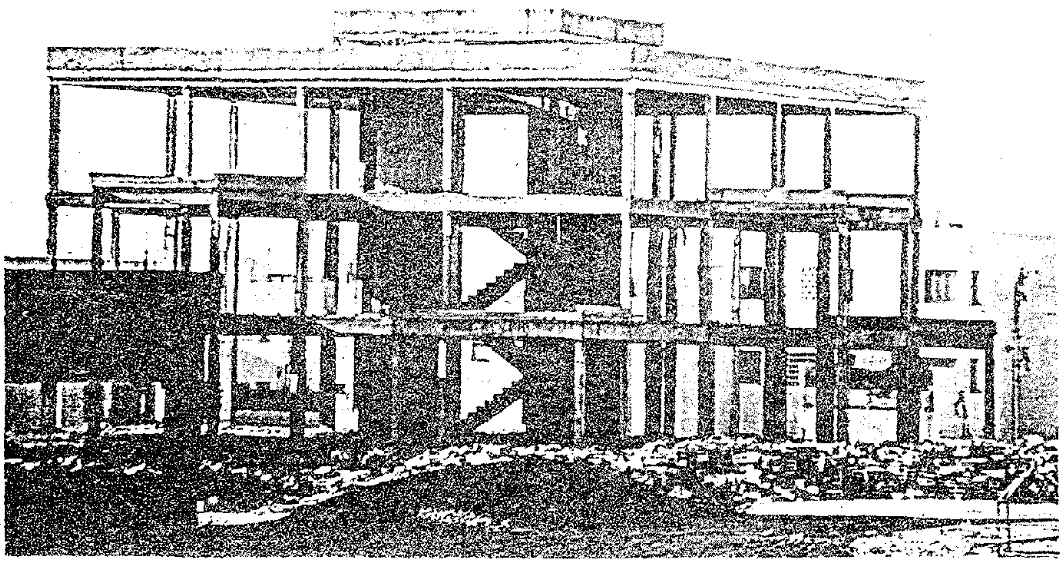
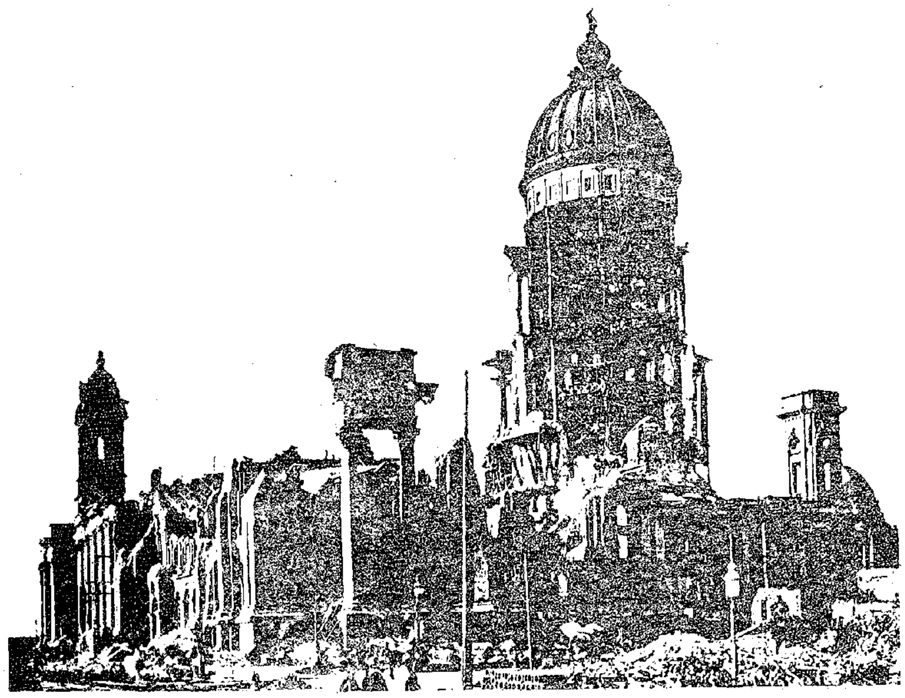
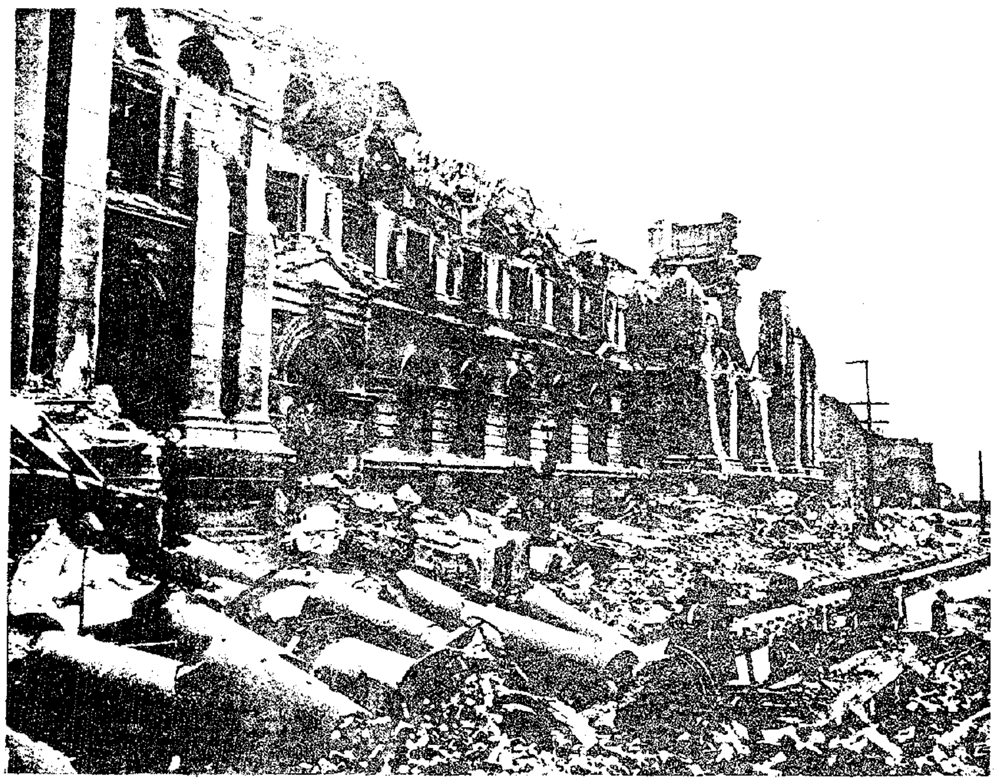


Figure 17b. This very common type of concrete frame construction proved to be inadequate for the rigorous seismic conditions.





Figures 19a and 19b. San Francisco City Hall, after the 1906 Earthquake.

From Bancroft Library, U.C. Berkeley

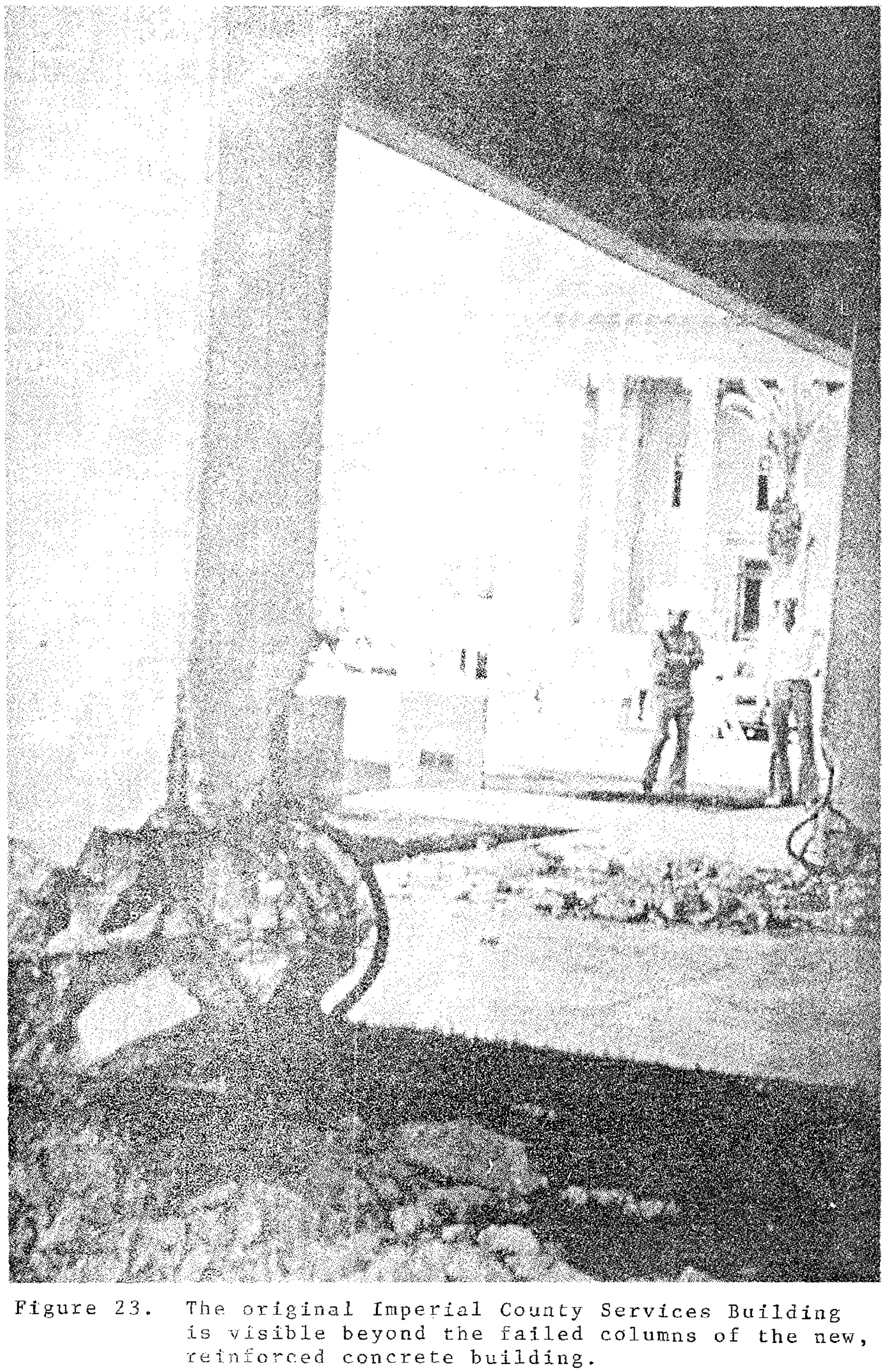


Figure 23. The original Imperial County Services Building is visible beyond the failed columns of the new, reinforced concrete building.

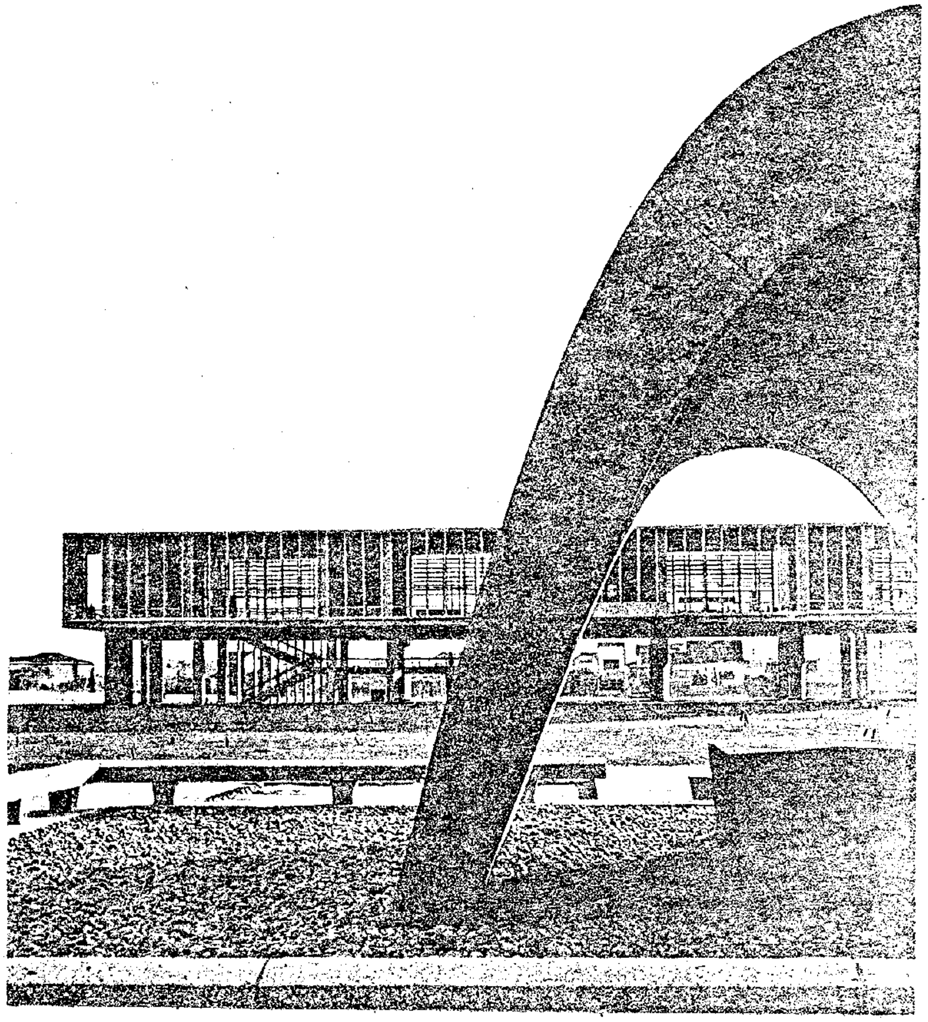


Figure 29. Kenzo Tange's Hiroshima Peace Centre, Memorial and Museum.

From U. Kultermann 1967

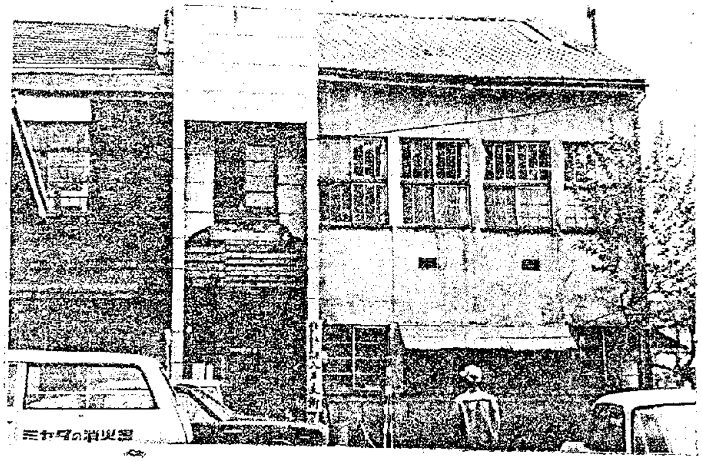


Fig. 32a. Old wood city hall, Hachinohe.

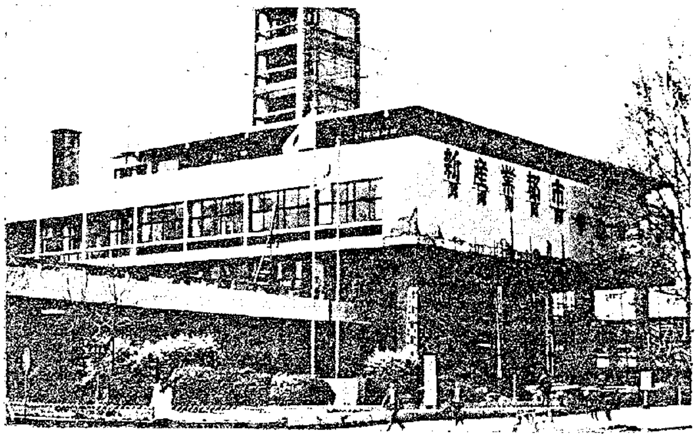
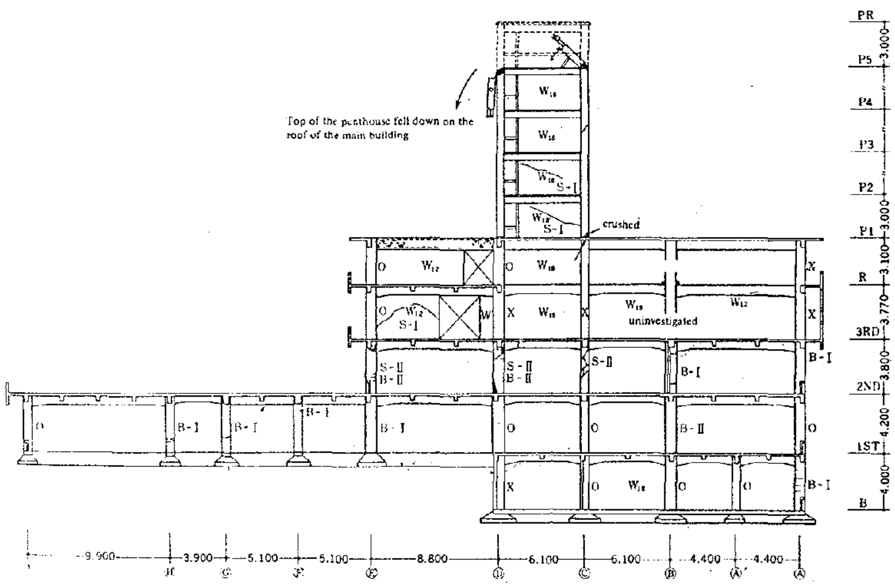


Fig. 32b. New reinforced concrete city hall, Hachinohe.

Fig. 32c. Section of new reinforced concrete city hall.(below)

From Suzuki (1971)



EARTHQUAKE DESTRUCTION ANALYSIS OF BUILDING CONSTRUCTION AND IMPROVEMENT PROPOSALS

Chen Mou-xin*, Xu Jia-feng*, Ruan Zhi-da*

SYNOPSIS

Seismic damage survey of building construction in the Tangshan earthquake indicates that the main structure of a building usually remains intact or is only slightly damaged during an earthquake, but damages may be caused on a large scale at positions such as deformation joints, parapets, non-load-bearing partitions, etc. due to irrational construction and so forth, and bring about a considerable amount of restoration work and expenditure. This paper enumerates the various kinds of damage phenomena of these positions, analyses the causes of their occurrence, and attempts to propose corresponding construction designs for their improvement with the hope that through endeavours made in the direction of construction designing, various minor elements of the construction of a building may have the aseismic capacity congruent with that of the main structure, thereby, reducing earthquake damage of buildings to the minimum.

INTRODUCTION

The Tangshan earthquake brought about the collapse and destruction of a large amount of buildings. In the areas of 11° intensity, practically all buildings collapsed; in 10° regions, almost 90% of the buildings of every description collapsed or were seriously damaged; in 9 and 8° areas, destruction was 35 and 10% respectively; in the 6 and 7° regions in which Beijing municipality happened to be located, damage was 3 - 4%. It is evident that with the decline in earthquake intensity with respect to distance from the seismic centre, the rate of structural damage of houses also decreases. Nevertheless, in areas of

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relatively low earthquake intensity above mentioned, the amount of non-structural damage was large and fairly widespread. Thus, a very acute problem is raised. In the Beijing and Tianjin area, deformation joints, parapets and non-load-bearing partitions were damaged quite a lot. Besides, facing tiles on the exterior faces of the walls and various kinds of decorative fixtures on same were shaken loose and fell, door and window panes were shattered, suspended ceilings cracked and dropped down. These non-structural damages not only posed the problem of safety but also brought about a huge amount of restoration work and expenditure besides affecting the normal use of buildings. These problems are especially conspicuous with massive public buildings.

Through the earthquake damage investigation, it can be seen that the cause of damage of the above non-structural components lies in the irrational building construction of these fixtures or lack of appropriate aseismic measures. In the past, people paid more attention to the aseismic performance of the main structure as it has a direct effect on the security of life and property. As to the aseismic requirements of building construction, proper understanding was wanting, experience and methods were also lacking, and so the problem found no appropriate solution in designing. These accounted for the non-conformity of aseismic capacity of non-structural components and the main structure. Therefore, to make a serious study on the aseismic construction of non-structural components with corresponding construction design principles and measures so that during earthquakes reaching the designed intensity, these components, as well as the main structure, remain undamaged or are only slightly damaged, thus, reducing the amount of restoration work after the earthquake and preventing them from collapsing and injuring people during a high intensity earthquake is an important topic that should not be overlooked in building construction designing.

Let us deal with the earthquake damage analysis of deformation joints, parapets and non-load-bearing partitions of buildings severally and the proposals for their improvement. This is the result of the investigation and survey carried out on 66 engineering projects in Beijing, Tianjin, Tanshan, Guye and Qianan after the Tangshan earthquake.

I. Deformation Joints

1. Damage Conditions

If deformation joints were not provided, in buildings with complex planar configuration, or marked changes in building height and rigidity, or in buildings where the high and low portions were joined by corbels, damages following an earthquake were found to occur in varying degress of magnitude (Fig. 1-4). In buildings where deformation joints were provided, in the past consideration during design mainly dealt with expansion or settlement joints, but not with aseismic consideration, thus, cracking of building decorations and wall bodies in an earthquake is a common phenomenon, especially in a highrise. In some skeletons with exterior bearing wall or bearing wall construction, double walls were not built at the junction of deformation joints, and damage was found on the side having the opening (Fig. 5). Even in buildings provided with aseismic joints, damage of varying degrees also occurred.

The damage conditions at different places of the deformation joint:

(1) Exterior Walls. Construction practices that easily caused damages included: (a) Covering joints -- Brick buttresses or sham pillars were used to cover up the joints. They fell as the earthquake occurred; (b) Small joints -- larger gaps of about 7 - 10 cm were left in joints of structures, but building decoration and finishing made them much smaller, leaving about 2 - 3cm only. Following the earthquake, these decorations and finishes were detached and fell down; (c) Clogging up of joints -- Hard substances like nogs, hollow bricks, etc. were set into the joints so that they got stuck in them. Vee-shape tinplates were nailed over them or they were plastered. During an earthquake, grounds got loose, tinplate covers detached and pieces of bricks fell down (Fig. 6).

Aseismic joints of a few highrises had wider joints left in their structure. Their exterior walls took on a contractile folded-type of flexible construction which remained intact basically after the earthquake (Fig. 7). However, if it were not properly designed or if the amount of deformation provided for were too small, such a construction may also show damages in an earthquake (Fig. 8).

(2) Interior Walls (including the portion of the inner walls and the ceilings): Construction practices to be dealt with below have often caused damages: (a) No deformation joints provided in the decoration and finishing -- Deformation joints were provided in the structure, but they were stuffed dead with long square timber during decorating and finishing. Mortar was directly applied over them. Cracks appeared with the earthquake. (b) Fixed plank cover -- Plank covers for joints were nailed at both ends when some projects were under construction. Both plank and grounds were detached and fallen during the earthquake; (c) Clogging up of joints -- In some projects under construction, grounds used for fixing cover planks were clogged into joints. These grounds became loose when the earthquake came, and the cover planks fell down; (d) Joints hidden by door frames -- In some projects, deformation joints on the passages were covered up by door frames which were deformed during an earthquake. Doors could no longer be opened and closed then -- a hindrance to evacuation (Fig. 9).

In most projects, the width of deformation joints were not large, and moveable plank covers that permitted one-way displacement were used. Most of these basically remained intact in the earthquake. However, when such a method was practiced in highrises, local damages appeared at the higher storeys (Fig. 10).

Some aseismic joints of highrises made use of flexible bonding construction, i.e., moveable plank covers, both ends of which were held fast by clamps (Fig. 11), or folded-type metal sheets fixed at both ends, etc. (Fig. 12). Damage was generally slight with such a practice. However, some cover sheets were found to be twisted, pulled out and fallen.

(3) Floors: Some deformation joints were provided for floor structures, but in the flooring surface and backing, none was provided and cracks appeared after the earthquake. Most projects have employed highly elastic rubber strips to fill the joints (Fig. 13). These have generally remained intact after the earthquake. In some others, aseismic joints were wider, floor joint locations were covered with moveable planks both sides of which were stuffed with rubber strips, but between the ends of plank covers and the faces of the walls, there were no rubber strips to serve as partitions, and local damages due to collision occurred (Fig. 14).

(4) Roofs: In ordinary projects with flat roofs, the edges alongside the joints were flanged and coping was done with a moveable precast reinforced concrete sheet or tinplate over it (Fig. 15). This remained undamaged after the earthquake. In some projects where deformation joints were provided for the roofs between high/low buildings, the upright gaps between the concrete flashing on the face of the wall and the flanged edge of the roof was very narrow (Fig. 16). During the earthquake, collision cracked the flashing which detached and fell.

Besides, along the deformation joints there were the parapets, eaves, decoration lines of wall faces, etc. that intersected them. Very often no break was seen along the joints, or the gaps left were too narrow. Thus, damages due to collision were liable to occur.

2. Causes of Damages. Causes that accounted for various kinds of damages of the deformation joint construction can be summarised as follows:

(1) In buildings where abrupt differences occur in plan or height, or where the rigidity of the structure changes suddenly, no deformation joints were provided, non-conformity of amplitude and frequency of adjacent buildings in an earthquake, or the concentration of stress will cause damages.

(2) In skeleton with exterior bearing wall or bearing wall construction where double walls are not used on both sides of the deformation joint and where the wall on the open side is not pro-

perly attached, a weak link occurs which will be damaged first in an earthquake.

(3) The damage of most deformation joints is due to insufficient allowance which often contributes toward damages in collisions. The width adopted for expansion and settlement joints is commonly 2 - 3 cm. This can hardly cope with the amount of structural deformation in an earthquake. Some structures left relatively wider joints, but decoration and finishing coupled to narrow them a lot. Some buildings did have the width of joints between wall bodies guaranteed in size, but at parapets, eaves and architraves on wall sides, no gaps were seen or the clearance left was too narrow to offer ample room for deformation in earthquakes.

(4) When 3-dimensional deformation occurs at deformation joints during an earthquake, the horizontal one is the most conspicuous. As construction and material used for the majority of deformation joints only allow of relative horizontal displacement, especially those places using superfluous building decoration such as pilasters for covering joints, sham wall pillars, etc. that restrain the forward-backward sway of both sides, have become positions that are seriously damaged. In some projects, insufficient allowance was seen provided for vertical motion at deformation joints between high/low portions on the roof, thus, restraining vertical deformation, and as a result, seismic damage is inevitable.

(5) In order to make construction work simpler, some projects leave narrower joints which have even been blocked up completely, thereby, giving rise to damage.

3. Suggestions for Improvement

(1) The provision of aseismic joints with width conforming to stipulations defined in current aseismic design code should be stressed.

(2) The decoration construction of aseismic joints should, in principle, consider allowances for 3-dimensional spatial deformation the amount of which should correspond with structural joint widths specified in the code. No rigid material should be

used to block up the joints, to get things stuck into them or to make them smaller, etc. For vertical motion like that between the high/low joints on the roof, an allowance of not less than 3 cm should be provided. (If it serves as the settlement joint as well, sufficient settlement should be considered).

(3) Flexible connections are suitable for aseismic joint constructions, but in order to cut down cost and simplify construction, projects of different standards should be dealt with separately. Generally speaking, in buildings of lower standard where the width of joints are smaller, and at joint positions where double walls are to be built, the joint for the exterior wall need not be decorated. The interior wall is easy to redress and simple practices like using rigid material fixed on one side to cover up the joints can be employed. In highrises, the width of joints is usually broader, and flexible connections should be set up for both the outer and the inner walls.

(4) Interior partitions, suspended ceilings, doors and windows, fixed counters, pipe lines, etc. should not be built across deformation joints as far as possible. When it is difficult to do so, proper measures have to be taken, for instance, at the place of crossing try to set up flexible connections or treat it as a deformation joint itself. At in-door deformation joints, no door and window frames should be used to cover them up.

(5) The realisation of the rational design scheme of a project should be ensured in carrying out the construction. Attention should be paid not to let any bits of broken brick, mortar, wood block and other oddities get stuck or drop into the joints.

II. Parapets

1. Damage Conditions

Current practices resorted to in building parapets include in situ concrete or using precast reinforced concrete, metal railing, composite parapets (masonry with reinforced concrete balusters) and brick parapets. The strength of reinforced concrete and metal railing parapets are both high in themselves with good integrity and can be firmly attached to the structure beneath.

The next comes composite parapets. No damage phenomena were found on these few types of parapets except collision damages at expansion joints of individual projects. Brick parapets were damaged more often (including solid ones of brick, the same with brick balusters, and those laid with opennings). In the Tianjin area, brick parapets and high facades of old buildings were seriously damaged and many collapsed. In newly-built buildings, the construction was more rational. Brick parapets that were not high had rarely been damaged, though there were individual cases of collapse seen. In the Beijing area, parapets were relatively less damaged, in a few cases some collapsed. In areas where intensity exceeded 10 degrees, low brick parapets behaved consistently with the main structure on some of the new buildings that withstood the attack of the earthquake, and cases of their collapsing before the main structure were rarely noted.

Besides the total collapse of whole parapets, other forms of damage were seen. Most of these were due to collision at the expansion joints. Others include cracking, swaying outwards or partially collapsing. At the base of the parapets along the roof, local or complete horizontal cracking of the whole parapet was often noted. In serious cases, whole flanks were swayed outward. In some instances, slant cracks occurred at the corners and at local protrusions (including fixtures that were embedded, like lamp posts, flagstaffs, etc.) of the parapet. Partial collapses often occured at the high portions of the parapet. Brick parapets laid with openings in them were often broken at the balustrades.

2. Causes of Damages: As parapets are built on the highest part of the buildings, this very feature and the whip lash effect of seismic force combine to subject them to a stronger seismic force, easily damaging the parapets. Taking the parapets themselves into consideration, the causes of their damage may be analysed as follows (Take brick parapets for example):

(1) The strength and integrity of such parapets themselves are poor. They cannot withstand seismic load and so are damaged easily. In a certain project, the grade of mortar used for

laying the parapet was very low, and the masonry work very poorly executed. During the earthquake, a part of them collapsed; the other part crushed. In other projects, the outer finishing of the parapets cracked, and rain water seeped in. Thus, the masonry was subjected to years of freeze-thaw attack which lowered their strength considerably. During the earthquake, the place of bonding between it and the roof cracked throughout. Damage on the side facing north was more marked. The integrity of brick-laid parapets with openings is poor. During earthquakes, brick-laid balustrades (capped with unbroken reinforced concrete) form the weakest link of the parapet are broken down first.

(2) Over high parapets augment seismic force and diminish their own seismic load-bearing capacity relatively. This is the reason why so many "high facades" collapsed. For instance, there is a dormitory in a school in Beijing. A portion of its parapet laid with hollow tile stood 2.7m high -- 1.6 m higher than the rest. During the earthquake, the portion that was over high the others collapsed completely.

(3) Poor bonding of the parapet and the structure underneath it is also one of the important reasons for its damage. This bonding is often weakened by roof slabs extending into the wall, or the set-in of asphalt felt on the roof into it, thereby, breaking the bond between the parapet and the main structure. During earthquakes, this juncture cracks first or loses its stability, and so comes the collapse. For instance, there was a brick parapet on one of the residential buildings in Beijing. As the roof slabs intruded 6 cm into the wall together with the set-in of the waterproofing sheets at the same level, claiming another 6, what was left for the base of the parapet was 12 cm. During the earthquake, the entire parapet fell over (Fig. 17). In another project, the actual effective section at the base of the parapet weakened by the waterproof layer of the roof was also 12 cm only. A horizontal crack along the base of the parapet appeared (Fig. 18).

(4) Individual projects have concrete lamp posts embedded in the brick parapets for installation lighting of the roof

terrace. These posts are tall and so displacements are large in the earthquake and cracks emerged as the posts tore at the parapets.

(5) Collisions also occur at the deformation joints of parapets. Insufficient clearance for the joints or without breaks provided on the concrete copings of the parapets or the railings brought about their damage.

3. Suggestions for Improvement

(1) When buildings need parapets, every attempt should be made to employ metal railings, in situ concrete or precast reinforced concrete for their construction if conditions permit. Attention should be given to the strength, integrity, and their firm bonding with the main structures and the rational construction of their deformation joints.

(2) The aseismic performance of the composite type of parapets is inferior to that of the above mentioned, but it is better than masonry ones. If these were to be adopted, attention should be shown to make the bonding of their concrete balusters with the main structure as strong as possible. The balusters and the filler brick wall should be fastened by steel bars. Steel bars inside the concrete coping and those within the balusters should be tied firmly. Spacing between the concrete balusters should not be too large.

(3) When brick parapets are to be constructed, they should not only be built according to stipulations defined in aseismic code but attention should be shown to the few points below: (a) Try to avoid the reduction of strength of the bonding of parapet foot and the main structure as caused by the extension of roof slabs into the wall and do not lay the parapet on the roof slabs directly. (b) Do not let the set-in of asphalt felt reduce the bond at the base of the parapet. To cope with this, pea gravel concrete may be used to press down on the position where asphalt felt are upturned. (c) The capping of the parapet should be concreted full length with steel bars in situ. (d) No lamp posts, flagstaffs, enormous advertisement boards, etc. should be embedded in the parapet. These fixtures should in no way be connected to it. Besides drip holes, no other opening like hidden lamp caskets,

air exhausts, etc. should be left in it. (e) Ample allowance should be given to deformation joints of the parapets whose free ends on both sides of the joints should be strengthened. (f) Non-anchored brick parapets laid with openings should not be adopted.

III. Non-load-bearing Partitions

1. Damage Conditions: main structures were intact, but non-load-bearing partitions damaged. This condition mostly occured in reinforced concrete structures. At present, those adopted generally fall under three structural systems -- the skeleton construction, the skeleton combined with shearwall and the shearwall. Infill walls in pure skeleton buildings were very commonly damaged. The conditions of damage generally found were alternating cracks in the walls or horizontal-vertical cracks at the interjection with beam and column, and cases of partial of the infill of some collapsed. The infill wall of the skeleton-shear wall structure fared better relatively, and with non-load-bearing walls of the shearwall type, the damage was even slighter. In our survey of this type of projects in Beijing, no heavy damage was found.

Viewed from materials used in non-load-bearing partitions, it can be seen that damage was relatively widespread after the earthquake when clay bricks, breeze bricks, or hollow tile were used. Damage of the walls laid with hollow bricks was rather serious. It was slighter when partitions were of light material. Lightweight boards fared better than blocks. If aerocrete slabs, carburetted lime slabs, wood fibre boards, etc. were used, only individual match-bond splices showed traces of tiny cracks. But when aerocrete blocks were used for partitioning, relatively more cracks appeared, and some of them cracked slantwise. Before the Tangshan earthquake, gypsum boards were little used in building projects. In the rooms on the 16th storey of the eastern high-rise of Beijing Hotel, experimental use was made of wood studs with gypsum boards and aerocrete blocks for partitioning. After the earthquake, the former was not damaged at all, while the latter cracked. In our survey, the partitions of this kind were not found to be destroyed.

At present, rigid bonding is generally employed in joining non-load-bearing partitions to the main structure. In regions where earthquake intensity was low, partitions with steel bars pre-set in the columns so that it could be better and more tensely fastened to the main structure were seldom cracked. In partitions reinforced with R.C. bands or reinforced with steel bars well fastened to the columns, there was slighter damage seen. But with partitions that were too long, too high and poorly bonded with the main structure or not laid to the underside of the beam/slab, thereby, leaving a gap on the upper edge, the damage was serious.

Some walls were made of two kinds of material such as partitions built of wood lath plastered combined with brick, or hollow and solid bricks alternately bonded horizontally and vertically, or with fire hydrants and vanity cabinets installed in them. Following the earthquake, most places where two materials were matched cracked.

In some projects brick partitions did not bond well with the underside of the beam or slab during construction, mortar was not fully stuffed into the gap to make a good bonding. Following the earthquake, horizontal cracks were seen to occur along the entire interface between the partition and the beam/slab.

2. Causes of Damages: Seismic damage of the non-load-bearing partition is generally related to the deformation of the main structure, the material selected for construction, the integrity of the partition itself, and its bondage to the main structure.

With the same kind of partition, if the rigidity of the main structure is high, its deformation will be small, and the damage of the partition slight; otherwise, the damage will be relatively serious.

If light material with a certain degree of flexibility is used for the partition, the material itself has a certain amount of deformation capacity, so that it can cope with the inter-storey deformation of the main structure with quite good aseismic results. On the other hand, a partition built with heavy rigid material has greater mass which will produce a stronger seismic force. Coupled with a smaller degree of allowable deformation of this kind of partition itself, it is easy to see the cause of its relatively widespread damage.

Partitions built up of two kinds of material usually lack proper joining measures either horizontally or vertically and so affect their integrity. The greater the difference of mass between the two materials used, the easier it is damaged. Other components embedded in the partition will also weaken it if there are no measures to strengthen their matching. The concentration of stress in it will make it prone to damage.

Poor joining of the partition and the main structure is another cause of damage. Infill walls and skeleton columns without tension joint bars, too large spacing between such bars, improper bonding of the partition and the slab/beam, etc. are some examples. The damage caused by poor joining of partition and the main structure often results in cracks at the junction of connection. If it is serious, out of plane displacement of the partition is the result or it may collapse if it loses its stability.

3. Suggestions for Improvement

(1) The aseismic capacity of non-load-bearing partitions should correspond with that of the main structure. If the latter is flexible, the partition should be able to cope with a larger amount of deformation. In this case, flexible connections are more feasible. When the main structure has higher rigidity, then its deformation is smaller, and rigid connections should be used. Whatever the connection may be, the partition itself should have better integrity and be reliably joined to the main structure.

(2) When conditions permit, light board or block material should be used for the partition as far as possible. Materials like gypsum boards, aerocrete slabs, lightweight blocks, etc. are some of them.

(3) In order to ensure good integrity of the partition, every attempt should be made to avoid using two kinds of material on the same partition, otherwise, some steps have to be taken to strengthen the junction of matching. Measures such as placing tie bars horizontally at this place or staggered bonding of the materials, etc. are practical. The partition should not be laid

only to level the height of the suspended ceiling and leave a gap on the upper edge. If this be so, measures should be taken so that firm connection with the main structure is provided. When the block-built partition is too wide and too high, reinforced brick layers and reinforced concrete belts should be provided and spaced at certain intervals for better connection to the main structure.

(4) There should be firm connections joining the non-load-bearing partition and the main structure. Generally, pre-set tie bars spaced at certain intervals in the columns have to be built-in in the block-built partitions or join to the reinforcementary belts of the wall. If boards are nailed over the framework to act as partition, iron parts have to be pre-set in the column to be welded firmly to the framework or bolts may be used to join them. If strip boards are used as partition, and adhesives are used to bond it to the beam/slab, the upper end of the board strips should be held fast by clamps that are fixed firmly to the beam/slab above so that it may not be shaken loose by an earthquake and fall down.

In order to cope with relatively larger deformation of the main structure, and minimise the amount of restoration work after the earthquake, it is feasible for partitions to use flexible connections, but joint construction should satisfy sound insulation and fire-fighting requirements.

This paper is written on the basis of "Earthquake Destruction Survey of Construction Decoration and Finishing of Buildings". The original report was drafted by Chen Mou-xin, Xu Jia-feng, Ruan Zhi-da, and Huang Ping-liang. Others participating in the study are Han Fu-sheng and Sung Yuan-lin.

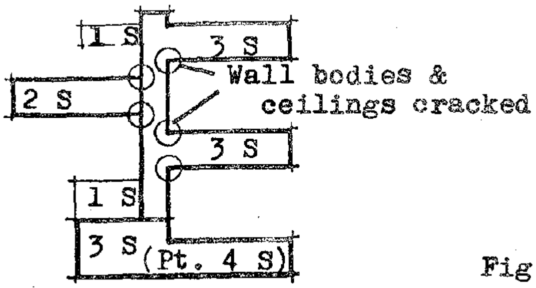

Fig. 1 Planar configuration of a hospital in Tianjin: complex and without provision for joints. All damages at the positions of re-entrant corners

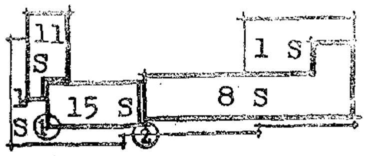

Fig. 2 Damage condition of civil aviation building, Beijing: (1) Connection of ground floor of the 15-storey & brick wall of the 1-st. blocks cracked; (2) The aseismic joint of the 15- and the 8-storey blocks was not extended to the exterior wall of the 1-storey block cracked

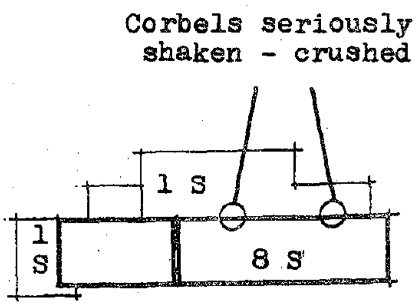

Fig. 3 Tianjin Friendship Guest-house: 8- and 1-storey blocks connected by corbels crushed seriously

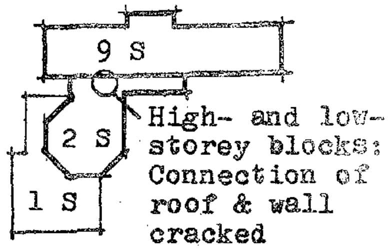

Fig. 4 Beijing Peace Guest-house: Damage condition of connection of high- and low-storey blocks

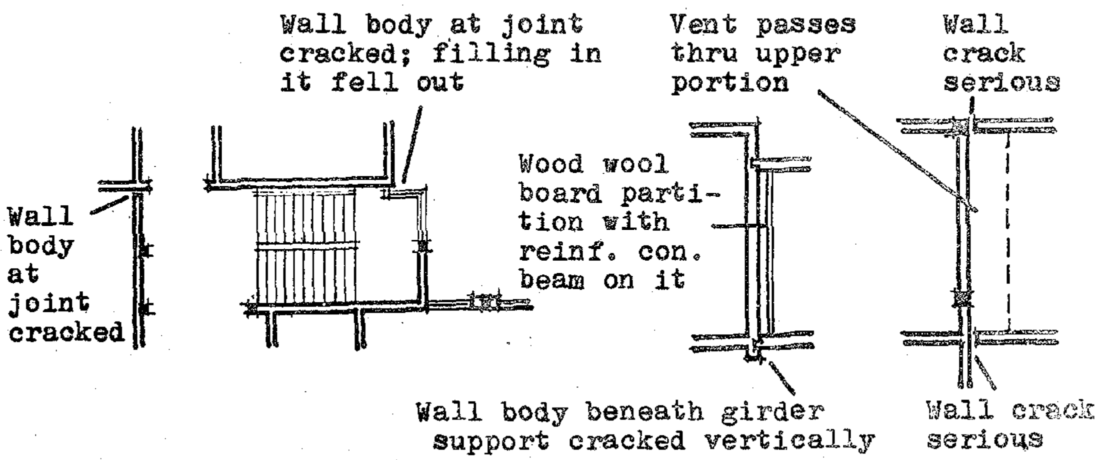

Fig. 5 Damage conditions of junction at deformation joints built as single wall

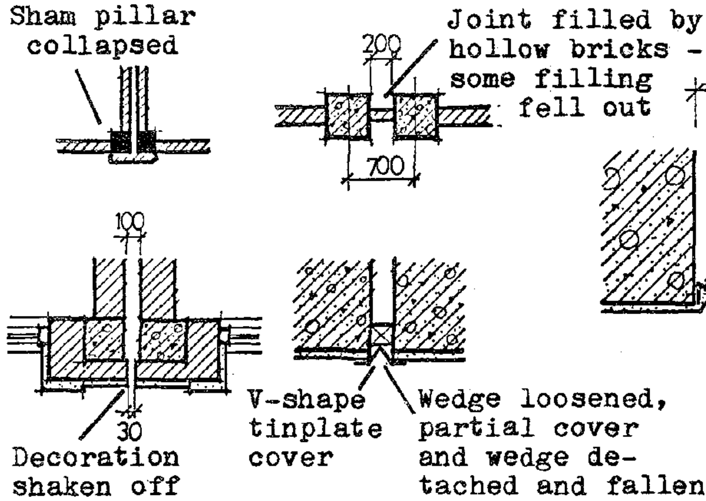

Fig. 6 Damage condition at exterior wall deformation joint (Upper left: covering of joint; Lower left: small joint; upper & lower right: stuffing of joints)

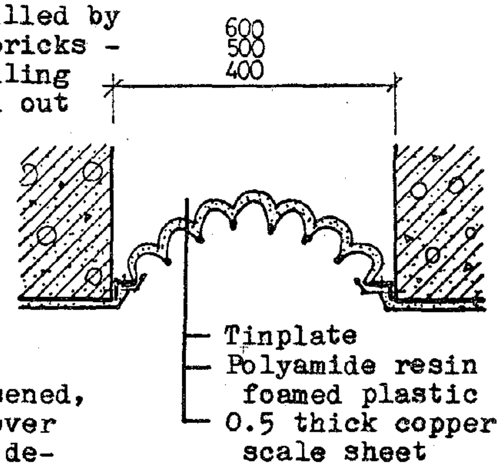

Fig. 7 Flexible connection of exterior wall aseismic joint, Beijing Hotel

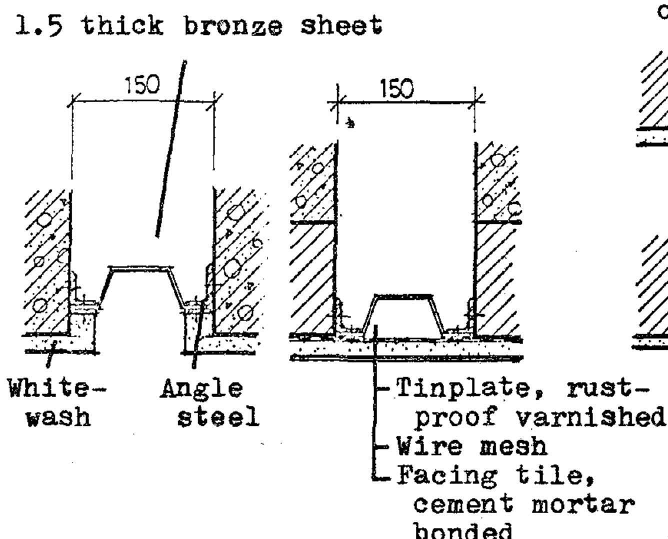

Fig. 8 Aseismic joint construction of Tianjin Friendship Guesthouse: (Left: Window partitioning column - bronze detached & fallen, angle steel displaced after quaking; Right: spandrel - angle steel & facing tile detached & fallen after quaking)

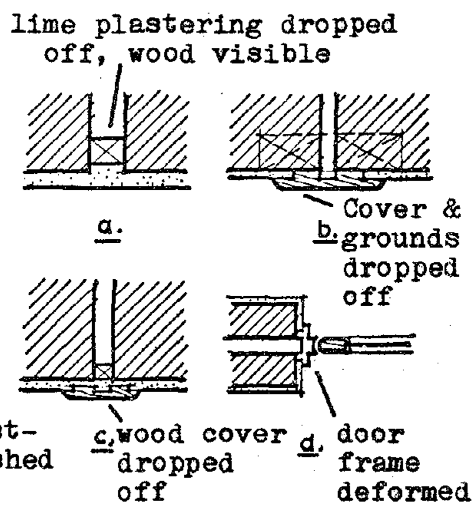

Fig. 9 Damage condition at interior wall deformation joint
(a. No gap after finishing
b. Fixed cover board
c. Joint stuffed with grounds
d. Door frame hide-out joint)

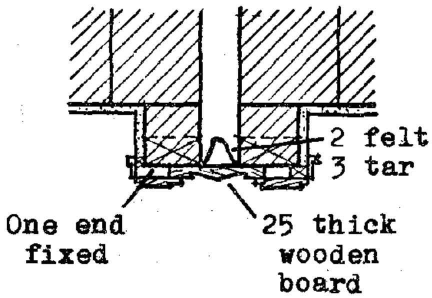

Fig. 10 Moveable cover permissive of 1-way displacement.

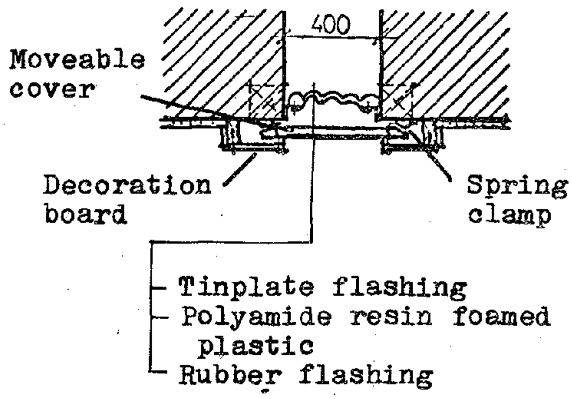

Fig. 11 Flexible connection at aseismic joint, Beijing Hotel - Moveable cover displaced, else intact.

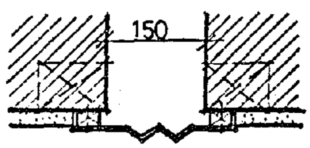

Fig. 12 Flexible connection at aseismic joint, Tianjin Fr.Gh.Al. cover pulled out & dropped.

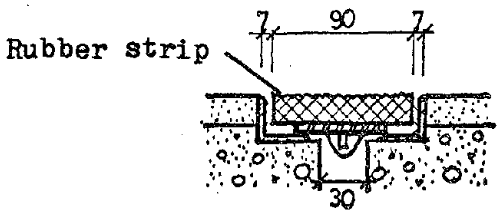

Fig. 13 Common practices for ground floor deformation joint.

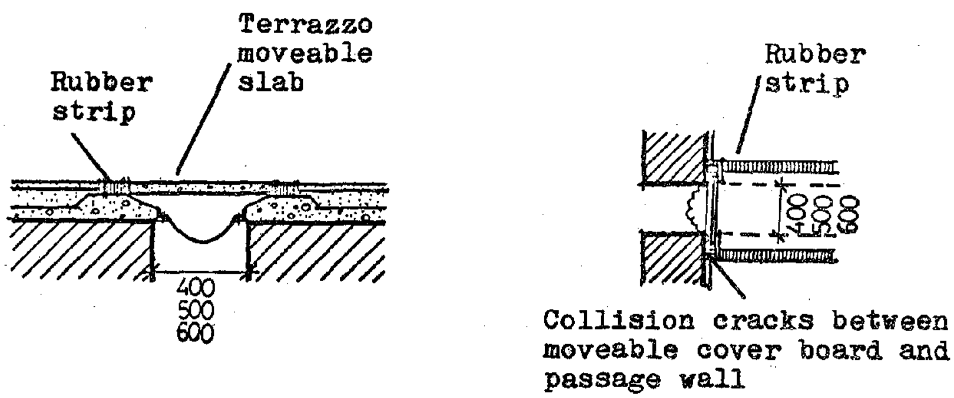

Fig. 14 Flexible connection of ground floor deformation joint, Beijing Hotel
(Left: section Right: plan)

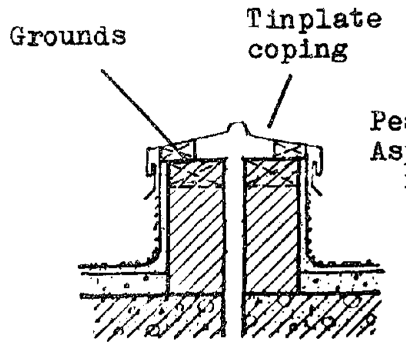

Fig. 15 Common practices for deformation joint of roof

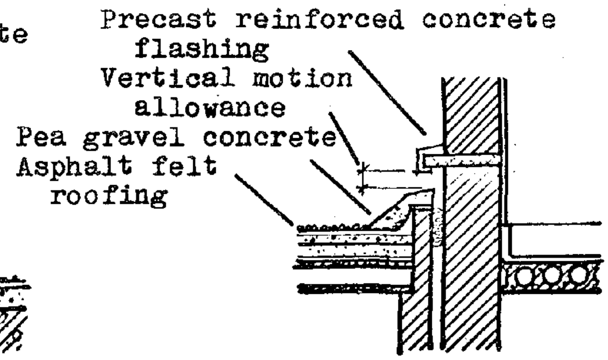

Fig. 16 Practices for high- and low-flashings on roof

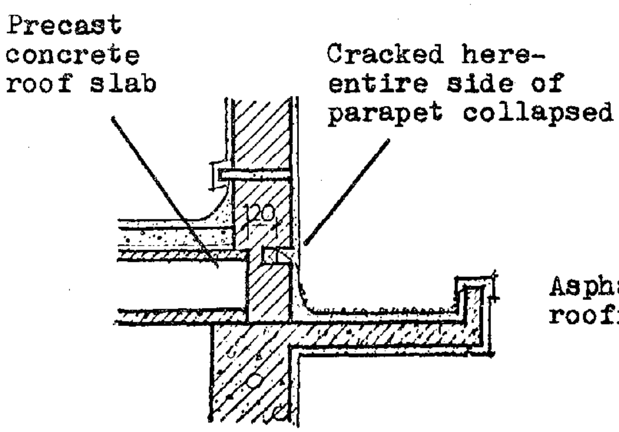

Fig. 17 Profile (sectional) of parapet of a residence in Beijing

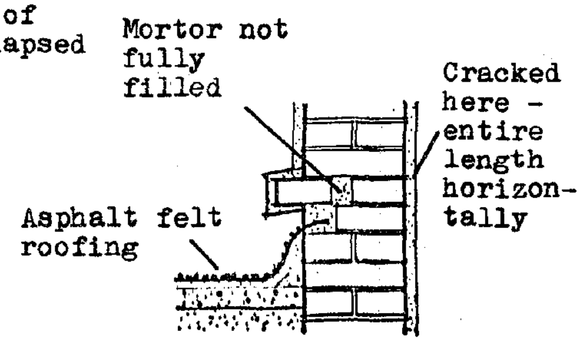

Fig. 18 Profile (sectional) of parapet of engine room bldg. of Beijing Telephone Office

SOME PROBLEMS ON ARCHITECTURAL DESIGN OF SINGLE STOREY FACTORY BUILDINGS IN SEISMIC AREA

by

He Guang-lin *

ABSTRACT

Based on a series of seismic damage investigation to many single storey machine-shops in Tianjin. the author illustrates the necessities and possibilities of taking anti-seismic problems into consideration in architecture design. A brief statement and analysis of seismic damage features of machine-shops are given. Finally, some proposals about how to consider anti-seismic problems, such as "planning", "Spatial combination" and "Wall treatment", are offered.

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INTRODUCTION

The city of Tianjin was seriously affected by the violent Tangshan-Fengnan earthquake of July 28, 1976, and large numbers of factory buildings were damaged in varying degrees.

According to the statistical figures made by ten industry bureaus of Tianjin, in Tianjin and Tang-gu districts with earthquake intensity of 8 degrees, the total floor area of factory buildings amounts to 11.7×10^6 M^2, in which 42.5% of them (namely 4.7×10^6 M^2) were collapsed and destroyed. In the meantime, the statistics given by the First and Second Bureau of Machine Industry and the Bureau of Metallurgy suggest that the collapsed building area of 2.08×10^6 M^2 is 39.6% of the total single-storey factory building area (5.26×10^6 M^2). Meanwhile, in 40 multi-storeyed framed factory buildings, 23 were destroyed to different extents, namely 57.5%; and in 84 inner-frame buildings, 66 were damaged in varying degrees, namely 78.5%. But in Han-gu district with earthquake intensity of 9 degrees, the factory building floor area is of 393×10^3 M^2, even 94% of which (namely 369×10^3 M^2) were collapsed. All the facts as above mentioned indicate that in earthquake regions with earthquake intensity of 8 or 9 degrees, the damage condition of factory buildings looks quite serious.

From August 6 to September 6, 1976, a group sent by Tianjin University made an investigation and an appraisal on seismic damage features of typical factory buildings subordinated to four corporations under the First Bureau of Machine Industry. Meanwhile, it still made a visit to several big factories, such as the Tianjin Heavy Machine-shop, Tianjin Tractor Factory, Xinhe and Xingang Shipyards, etc. In accordance with the level of seismic damage, the 67 workshops of 23 factories are listed as follows.

The list of classification* and statistics with respect to seismic damage level

Corporation which the Workshops are subordinate to	Kind					Summation
	I	II	III	IV	V	
Machine Tool corporation	–	1	11	17	15	44
Universal Machine corporation	–	4	4	4	1	13
Machine Fittings corporation	–	–	2	5	1	8

Corporation which the Workshops are subordinate to	Kind					Summation
	I	II	III	IV	V	
Electrical Machine and Equipment corporation	–	–	1	–	1	2
Summation	0	5	18	26	18	67
percentage	–	7.5%	26.8%	38.9%	26.8%	100%

Of the 67 single-storey factory buildings being investigated and appraised, their types were of reinforced concrete trestle system on the whole. 65.7% of them were under the fourth and fifth kind of seismic damage and this percentage may even come to 92.5%, if those under the third kind of seismic damage are included. It is obvious that so far as middle and small factory buildings are concerned (the span $\leqslant$24 M, the height of bottom chord of roof truss$\leqslant$15 M, the lifting capacity of crane $\leq$ 20∼50 T), the type of structure as above mentioned can reduce the seismic influence, and resist the seismic intensity of 8 degrees, provided that the "planning", "Spatial Combination" and "Joint Construction" are reasonably taken into consideration in architecture design.

CONDITION AND ANALYSIS OF THE SEISMIC DAMAGE

The seismic damage features of the single-storey machine-shop are as follows: some parts of the roof members wrecked; roofs collapsed; some parts of the reinforced concrete columns cracked and broken to pieces; struts destroyed and unstabilized; brick walls cracked, inclined outward and collapsed. Now, a brief illustration of seismic damage features in respect of architecture design is given below.

1. Roof System. It with roof-slab in large scale and reinforced concrete roof truss being adopted as a standardized design and construction.

(1) Roof—Light. Roof-light of type "Π" protruding from

*The classification of seismic damage:

I -- thoroughly collapsed;
II -- seriously collapsed;
III -- general collapsed;
IV -- slight collapsed;
V -- fundamentally undamaged.

the roof is very heavy because reinforced concrete roof-light skeleton and roof-slab in large scale are generally adopted. As a result, the seismic forces increase by several times due to the whipping effect and a severe seismic damage will be induced.

All the seismic damages of roof-light happened at the positions of its struts. Under the action of the longitudinal seismic force, the struts became unstable and the roof-light skeleton was destroyed, such as the roof-light projecting from the roof of blacksmith shop of the Xinhe Shipyard in Tang-gu. The roof-light skeleton inclined towards one direction and partially tumbled down due to the failure of the struts (see Fig.1).

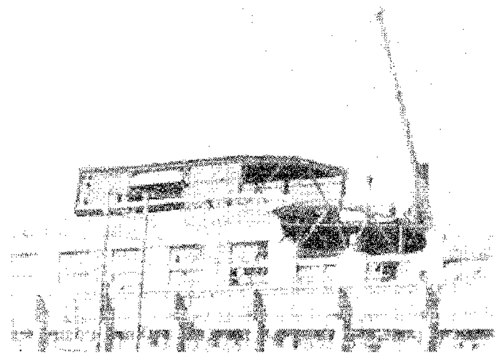

Fig. 1

Recently, the roof-light of the "sinking" type has been widely adopted. Because it is located below the roofing hence the entirety of the roof is strengthened and the seismic damage is not discovered. Fig.2 shows the roof-light of a turbine shop of the Xingang Shipyard in Tang-gu. Its roof-light of the "sinking" type is not damaged.

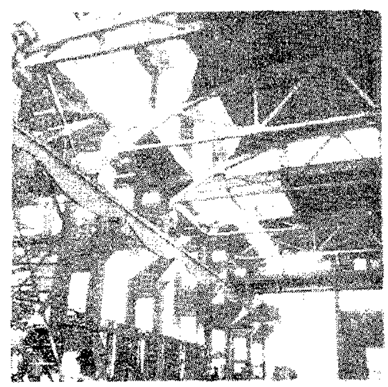

Fig. 2

(2) Roofing Members. Roof-slab in large scale is not only the commonly adopted roofing members but also the principal symbol of heavy roof.

Roof-slab in large scale has the following seismic damages: upper part of the wall tumbled, fell onto the roofing and smashed the roof-slab, as shown in Fig.3; the welding seam in the joint of roof-slab and roof truss was out of order, and the roof truss inclined out of the plane, so that the roof-slab fell down and cannot worked as longitudinal strut, as shown in Fig.4.

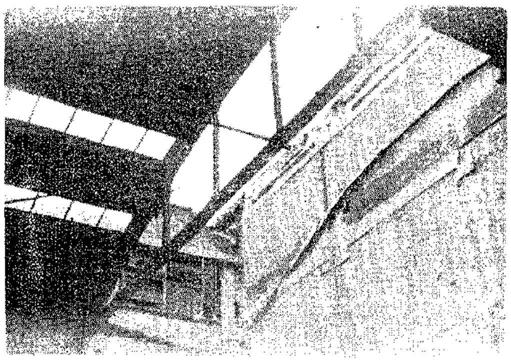

Fig. 3

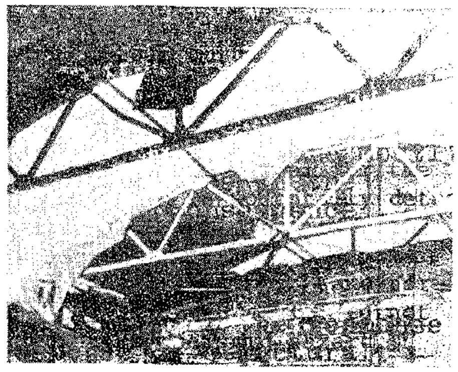

Fig. 4

(3) Roof-Truss. There are three kinds of failure:

a. The roof truss member cracked for lack of strength in itself;

b. A few roof trusses collapsed (see Fig.5) by reason of: less strength of reinforced concrete column head and the joint of column and roof truss was not secure;

c. The common condition of seismic damage of roof truss is as follows: it was necessary to install vertical reinforced concrete strut truss because the two end struts of trapezoid roof truss were of comparatively large length. As the longitudinal seismic force acted on the structure, the end upper chord and the end strut of the trapezoid roof truss were distorted by the thrust of the strut truss due to its comparatively large rigidity. Then the trapezoid roof truss inclined out of plane and both the end upper chord and the end strut cracked, so that the roofing supporting point to the roof-slab slid out, as shown in Fig.6.

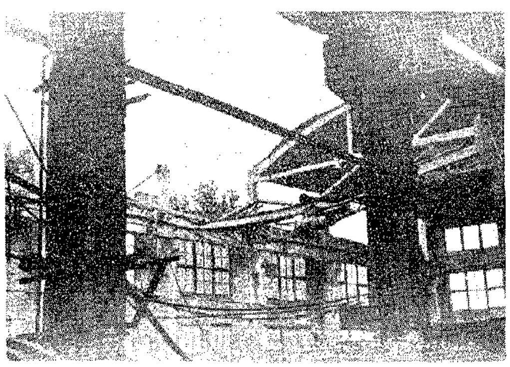

Fig. 5

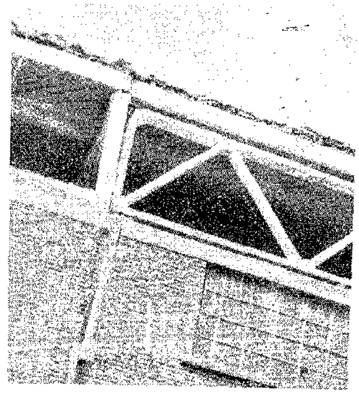

Fig. 6

2. [illegible] Column. At present, the reinforced concrete column is commonly adopted in most of the single storey factory buildings. Under the action of the seismic force, the typical damage of columns always occurs at both the level of the top of crane beam and the level of connection between the higher span part and lower span part. The former is due to the comparatively smaller section size of upper column (400 x 300 MM). In the case of heavy roof system, the seismic force is relatively larger and the column is so slender as to be destroyed by the compression and the bending at the level of the top of crane beam, as shown by the hull shop of the Xingang Shipyard in Fig.7. The latter is due to the difference between rigidities of the higher span part and the lower span part. Hence, under the action of lateral seismic force, the vibration frequency and the vibration type of the two parts of higher span and lower span are different, so that they impact one another. Meanwhile, influenced by the higher vibration type, the column destroys at the place of the sudden change of rigidity, as shown by the turbine shop of the Xinhe Shipyard in Fig.8.

Fig. 7

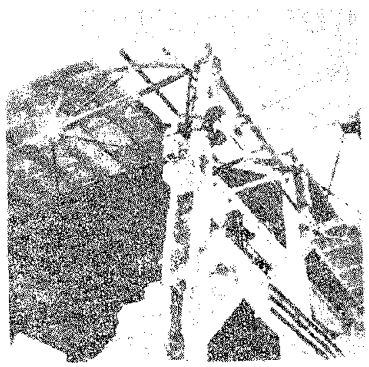

Fig. 8

3. Cladding Wall. Because the brickwork is mainly adopted, the cladding walls of all factory buildings are liable to be destroyed by seismic force. The slight case is the occurrence of wall cracks. The serious case is the collapse of wall.

(1) The parts of gable and longitudinal walls above the highest gird collapsed, as shown in Fig.9 and Fig.10. Conventionally, the highest gird is installed at the level of the column top, especially if the trapezoid roof truss and the thin webbed girder are adopted, and the height of wall above gird may approximately reach to 2 M, and even to 3 M at the middle part of the gable. Being located at the highest position, these above mentioned walls with heavy weight

are easy to collapse because their unpleasant bind with the main structure make it in a state of cantilever structure.

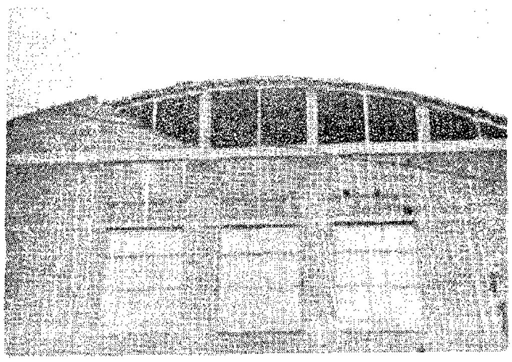

Fig. 9

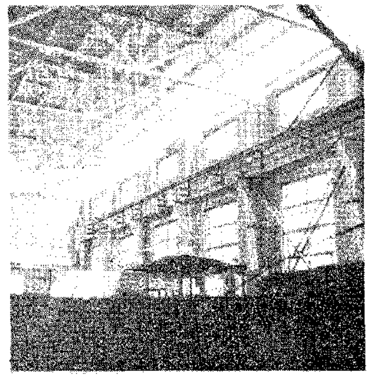

Fig. 10

(2) Gable cracked diagonally and the parts of wall collapsed down, as shown by the machine shop of the Sixth Machine Factory in Fig.11. This case chiefly appears in the factory buildings with the roof of large rigidity, which can transmit the seismic force to the gable, so that it is easy to be destroyed by the shear force.

Fig. 11

(3) Gable inclines outwardly, so the roofing asphalt felt cracks. Fig.12 shows that the foundry gable of the grinding machine factory inclined outwardly about 10 CM. Fig.13 shows that the foundry gable of the Xinhe Shipyard seriously inclined outwardly and collapsed to the extent that the columns of resisting wind force ruptured just at the place where the section area changed. As everyone knows, when the seismic forces act on the factory building along its lengthwise direction, the stability of the whole factory building is guaranteed by column bracing. But the ability to resist seismic force by the gable itself is very weak, so the seismic force acting on the gable is mainly resisted by the column of resisting wind force. The above mentioned seismic damage will take place, if there is a bad bind between the gable and the column of resisting wind force.

Fig. 12

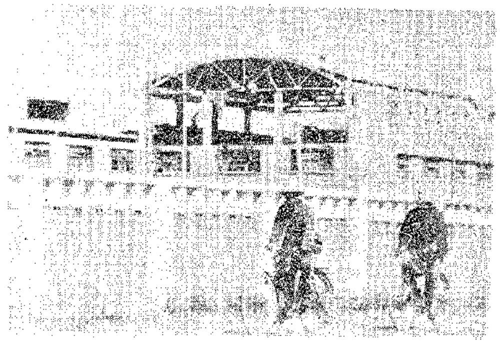

Fig. 13

(4) Longitudinal wall inclined outwardly, so that the roofing asphalt felt was torn with crack width of 25 CM, as shown by the foundry of the grinding machine factory in Fig. 14. In serious case, some parts of longitudinal wall collaspsed and the wall separated from column, as shown in Fig. 15. By reason of unpleasant bind between wall and column, under the action of seismic force, the column and wall separate each other due to their different periods of self-vibration.

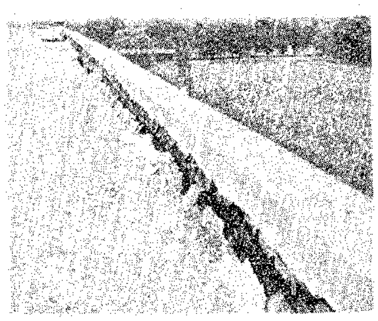

Fig. 14

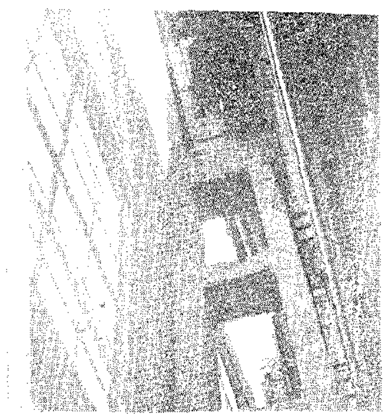

Fig. 15

(5) Secondary Calamity Due To Collapse Of Wall.

a. In general, the collapsed positions of wall were at the topmost part, hence the roof of the lower span part was smashed and collapsed by the large energy of wall released as it fell down, as shown by the third foundry in Fig.16.

b. Door openings used as entrance of factory building are often put on the gable and the longitudinal wall. As earthquake happens, these door openings will serve as the main passages, hence the collapse of wall will induce the accident of personal injury.

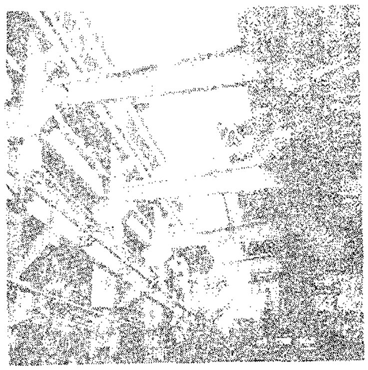

Fig. 16

INQUISITIONS OF SOME ARCHITECTURE DESIGN PROBLEMS

From the above description of investigations and analysis about seismic damages in Tianjin district, we consider it still feasible to construct the single storey factory buildings with reinforced concrete trestle structure system. Besides the antiseismic calculation and structure design, now we inquire into the architecture design in order to lighten the seismic damages as follows:

1. Planning.

(1) Selection of Plans— The irregular plans with the spans being perpendicular to each other such as "L", "T" and "п" etc. are unfavourable for transmission of seismic forces. Because the forces are twisting and turning, the strains will not be homogenous and will be easy to make concentration of stresses. If the strengths of elements and wall are not big enough, it is easy to make seismic damages as shown by the foundry shop of the Tianjin Tractor Factory and the machine shop of the Tianjin Forging Press Factory in Fig.17 and 18. For this reason the plans should

Fig. 17

be designed in regular forms such as rectangular or near square forms of single span shops or parallel multi-span shops.

(2) It is suitable not to have the length of shops too long, otherwise the function of space would be reduced. When the horizontal seismic forces act, the shop gets longitudinal flexures. As the rigidity of the two gable walls of the shop are larger, horizontal displacements are smaller, while as the rigidity of the middle part of walls are smaller, the horizontal displacements are larger, and the walls and columns of the middle part are easy to be broken. Besides, considering the action of rigidity of roofing and the influence of ground movements, the length of the shop should also be as short as possible.

Fig. 18

(3) The attached rooms or cross partition walls between the columns of the shop will cause the elements and walls to break due to the strengthening of the rigidity of local elements, which will induce the concentration of stress under seismic force, as shown in Fig.19. Therefore in architecture design the following points are noteworthy.

(a) The evenness of rigidity of different parts of factory building should be noticed;

(b) The nonbearing partition walls should be laid beside the external edges of the columns and combined with them reliably, as shown in Fig.20;

(c) The fabricated light partition walls should be located as much as possible.

Fig. 19

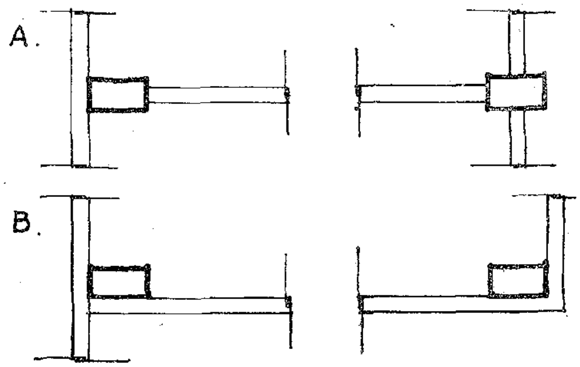

Fig.20 — A. Former practice.
B. Suggestible method.

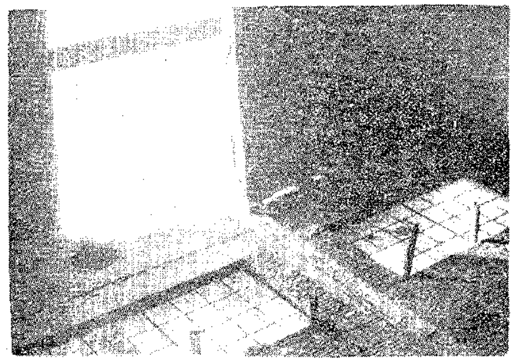

Fig. 21

(4) Formerly, in some designs, the trestle would not be located at the place near the gable wall. Although some columns were located the trusses were not. And the roof-slab are directly located on the gable walls. In this condition while the longitudinal seismic force act on the roof-slab, it is bumping against the gable walls repeatedly. The gable walls are inclined outward and collapsed. At last the roof-slabs fall down, as shown in Fig.21. Therefore in architecture design we should keep the trestle structure system integrated and avoid using enclosure structures as bearing walls.

2. Spatial Combination.

(1) Consideration of High and Low Shop Blocks — The change of heights of shop blocks would directly influence the transmission of seismic forces. Owing to the differences between periods of self-vibration and vibrating range, under the action of the seismic force, the part of connection between the high and low shop blocks is easy to get destroyed due to the concentration of stresses, as shown in Fig.8 and 22—the chassis and assembling shop of the Tianjin Tractor Factory. Therefore, in designing, we should adopt equal heights of shop blocks to the full, under the prerequisite of satisfying the presupposition of technology. If there is a necessity of being designed with different heights of shop blocks, it is suitable to erect

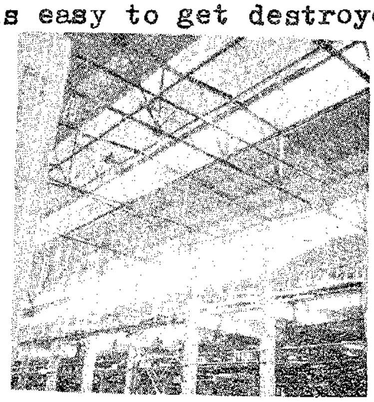

Fig. 22

additional columns, in order to separate the two parts of high and low spans as two independent units, or lay light panels at the connecting place of the two parts.

(2) Selection of Types of Roof-Light —— As above mentioned, the "Π" type of roof-light projecting from roofing is put on the position of change of rigidity at which it is easy to be destroyed with seismic forces. In designing, therefore, we should avoid adopting the section with projecting "Π" type of roof-light and it is suitable to adopt the regular section. So the flat roof-light is used for the daylighting of the shop and the "sinking" roof-light is used for the daylighting and ventilating of the shop. For the special ventilating of the shop, when the projecting roof-light using to shelter from the wind must be adopted, it is better to design the roof-light with steel skeleton, and light roofing so as to lighten greatly the weight of structures. They are shown in Fig. 2, 3, 23 and 24.

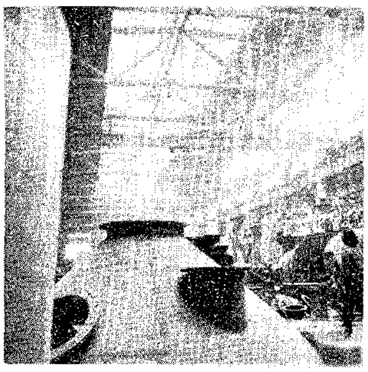

Fig. 23

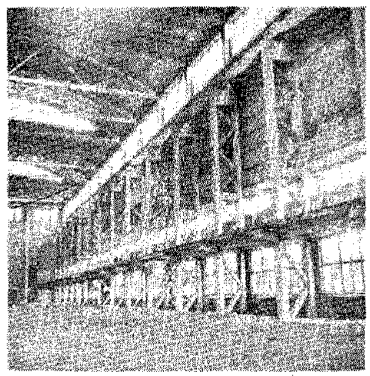

Fig. 24

(3) Consideration of Attached Buildings ——

a. The main shop is often to be built with some lower auxiliary rooms which are close to the side of the main shop. Under the action of seismic forces, due to the difference of height and rigidity of the two parts, the factory building has a character of bearing capacity of parallel high and low blocks and is easy to be destroyed, and auxiliary rooms are collapsed. For this reason, it is better not to build the attached rooms; if necessary the attached rooms should be built separately as independent structure system.

b. Some auxiliary rooms are built, being attached to the corner of the shop. The auxiliary rooms are constructed with bearing brick columns, reinforced concrete thin webbed girders, and roof-slabs in large scale and their structures are separated from the main shop. But they are not sealed systems of structures, while under the action of seismic forces, owing to the rigidity being very small, they cannot bear the seismic forces, so that they collapse seriously as shown in Fig.25 a, b. Therefore, adopting the sealed system of wall is reasonable,

a. b.

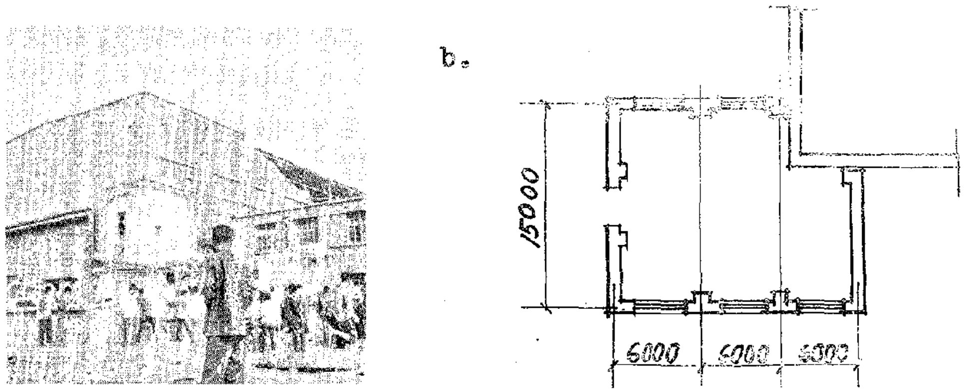

Fig. 25 --- a. collapsed condition of dust remover room of foundry shop in Tianjin Tractor Factory
b. the plan of dust remover room.

3. Treatment of Wall,

Although cladding wall isn't the main structure of factory building it plays a very important role. Cladding wall treated unreasonably will induce secondary calamity more serious than disaster of itself. So that we must pay attention to the treatment of wall in architecture design and construction. According to the condition and analysis of seismic damage, these measures should be adopted as follows:

(1) Brick is a brittle material, so the ability of receiving the shearing and pulling forces is not satisfactory. When it's laid along external edge of a column, seismic damage of brick work will be general and serious due to the bad bind between the column and brick work. In investigation we discovered that there wasn't seismic damage in case of factory building adopting panel in large scale, as shown in Fig. 26. It indicates that the panel has better earthquake-resistant ability due to its sufficient rigidity, reliability and flexiability of joint with columns. So we should widely spread the using of panels.

When brick wall is adopted, its joint with the column should be strengthened. Formerly in the standard of earth-

...ake resistance the vertical distance between the connecting rods hadn't an explicite rule. According to analysis of the seismic damage, the connecting rods should be arranged sparsely on the lower part and densely on the upper part of column. In architecture design, we suggest that two connecting rods (φ6) are to be adopted on every layer, the vertical distance between levels are 500 MM and 600 MM for the upper part and the lower part respectively. Type of arranging rods is shown in Fig.27.

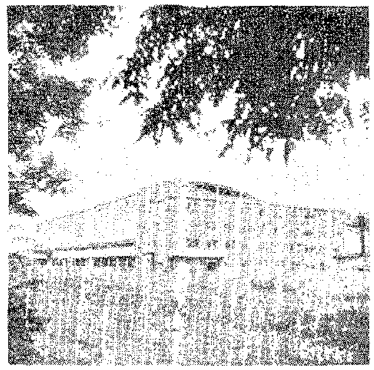

Fig. 26

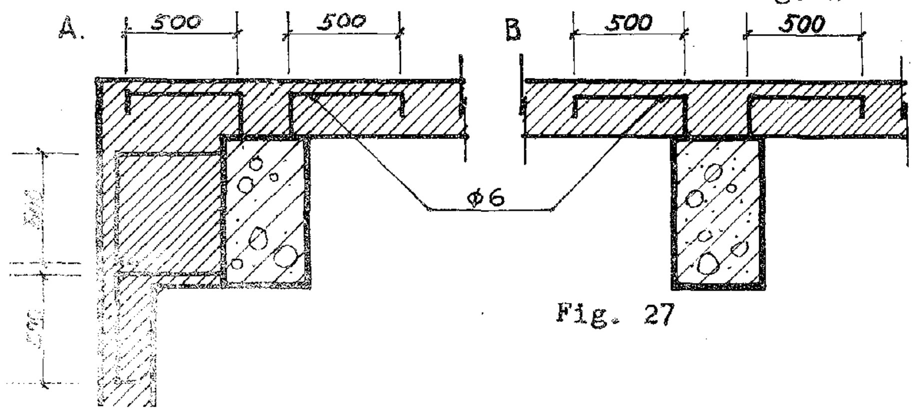

Fig. 27

(2) Cladding wall laid between columns has the action of column bracing and strengthening entire rigidity of the factory building, so no seismic damage was found in the longitudinal and gable wall, as shown in Fig.28. All the facts indicate that the above mentioned method is a better seismic-resistant measure. In architecture design the cladding wall should be laid between the columns for single span factory building.

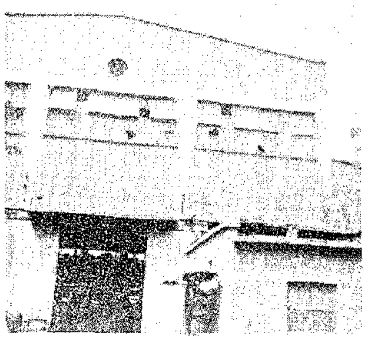

Fig. 28

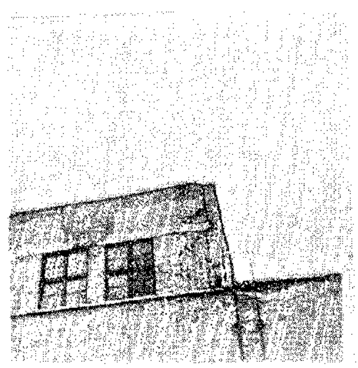

Fig. 29

(3) Interfenestral wall and corner of gable produce [illegible] crack and are collapsed as shown in Fig.11, 29.

By reason of small rigidity of interfenestral wall, under the action of seismic force, it cracks at the place of the section change. In architecture design, I am of the opinion that the construction rods should be disposed at the corner of cladding wall and the type of arrangement is shown in Fig.30. At the same time, in the design of elevation, measures should be adopted as follows (as shown in Fig.31):

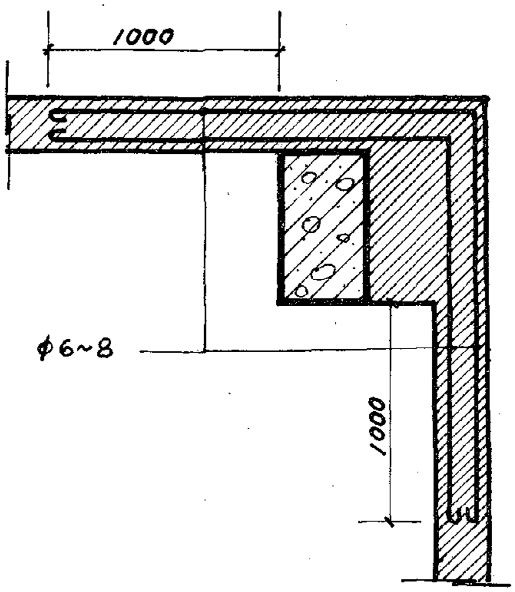

Fig. 30

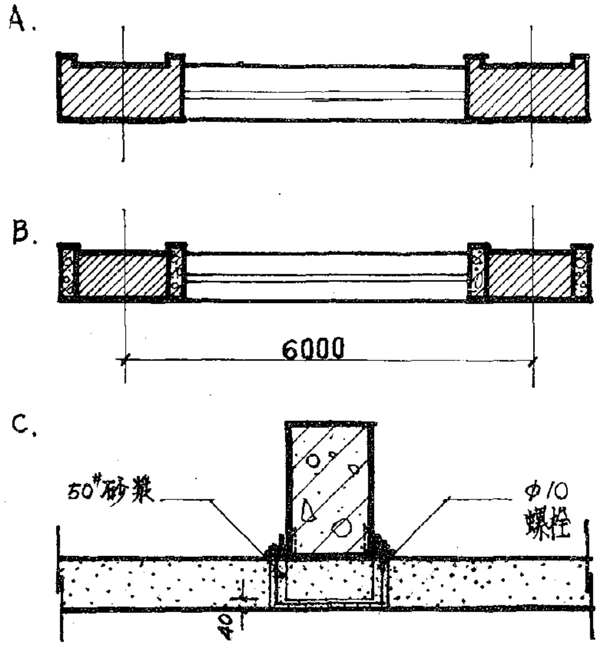

Fig. 31

a. Within 6 M distance from corner of cladding wall, windows shouldn't be set up;

b. Interfenestral wall laid with full cement mortar will be divided equally in width as far as possible. When the interfenestration $\leqslant 1.2$ M, the panel or brick wall, which has either set-off or reinforced concrete case in cooperation with mouldings of cladding wall, should be adopted.

c. A few windows are set up on gable, but on the upper part of gable above the gird, which is located at the level of column top, windows shouldn't be set.

(4) Parapet works as a cantilever structure, the seismic damage and secondary calamity are serious. In the former "Earthquake-Resistant Standard (TJ11-74)" and "Standard of

Earthquake-Resistant Appraisal for Factory and Civil Buildings in the Region of Beijing and Tianjin (1975)", the height of parapet was limited to 80 CM. It is too high and should be reduced as far as possible. For the tall building we should keep off parapet and adopt eaves gutter. If we adopt parapet, its height shouldn't be larger than 50 CM and the bind with roofing should be strengthened, as shown in Fig.32.

(5) Seismic damage of brick wall is small due to rational arrangement of gird and dependable bind between gird and column. The vertical distance between girds, arranged to be close to equal distance, should be reduced. In Tianjin region, the vertical distance between girds at upper part of brick wall shouldn't be larger than 4 M. It may be just a little larger than 4 M for lower part. The general brick wall and the suspended wall located at the connecting place of high span part and low span part should be installed with the closed cast insitu reinforced concrete girds. On the level of top of column or end strut of roof truss, these beams are connected with columns or roof truss by four rods being not smaller than 12 MM in diameter. On the top of gable, sleeping beam having reliable bind with roofing system should be set up, as shown by the gear shop of the Tianjin Tractor Factory in Fig.33. So that under the action of seismic force, the sleeping beam and the parapet above were not collapsed. But the brick wall under the sleeping beam was collapsed due to the outward inclining.

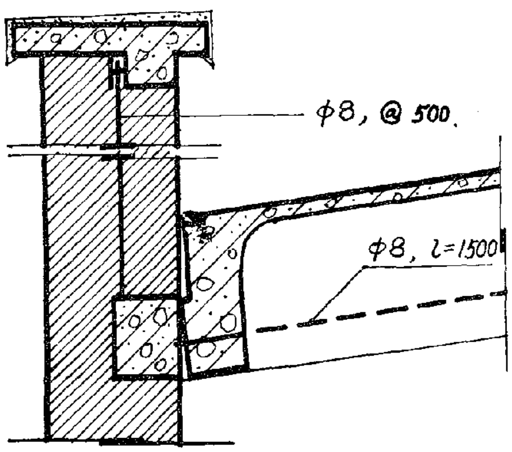

Fig. 32

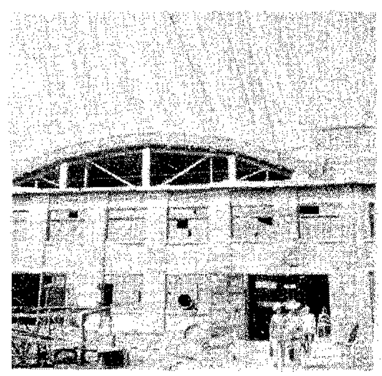

Fig. 33

(6) Investigation expressed that in many factory buildings the brick walls of either side of expansion joint were injured seriously due to bumping against each other. As expansion joint adopted the tongue-and-groove or overlap joint

in plane, the brick wall is destroyed seriously more than ever. In architecture design, earthquake-resistant joint combined with expansion and settlement joints should be adopted as a flush joint in plane. According to design intensity, soil constituents of foundation, rigidity and height of factory building, the width of earthquake-resistant joint generally isn't selected less than 50-90 MM, as joint parallel to trestle. The width of joint may be larger than the former and is commonly selected 100-150 MM, as joint is perpendicular to trestle or situated at place, where spans intersect perpendicularly to each other. Because the width of joint is larger, hence the construction of joint should be treated additionally in architecture design, as shown in Fig. 34.

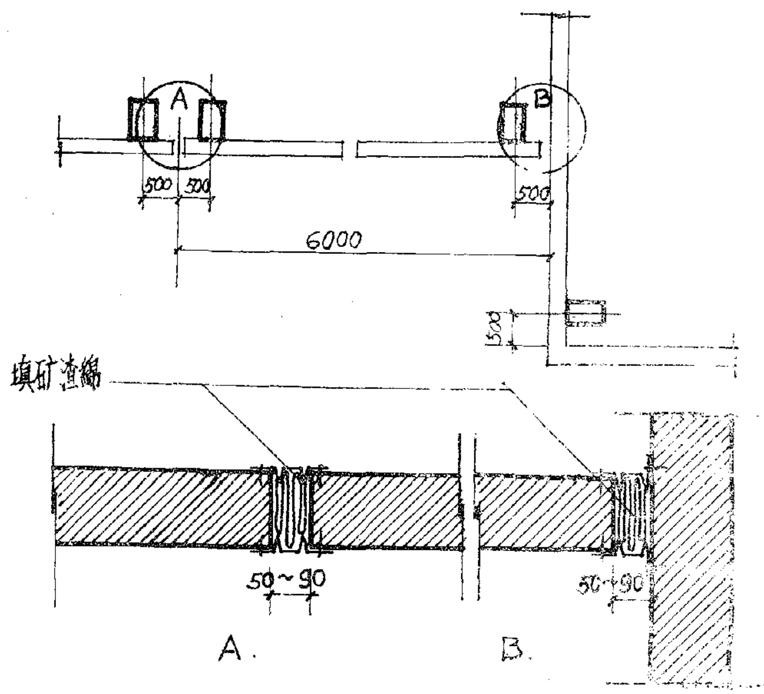

Fig. 34

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In view of the investigation and analysis of seismic damage are rather limited, in this paper the author only inquires into some problems on architecture design of single storey factory building in Tianjin seismic area in order to reduce seismic damage. Inappropriate analysis is unavoidable and please kindly give me your advice.

URBAN PLANNING

AND

LAND USE

THE SEISMIC SAFETY PLAN FOR SAN FRANCISCO: ITS PREPARATION AND ADOPTION

by

Allan B. Jacobs*

ABSTRACT

In 1971 the legislature of the State of California passed legislation that required each local governmental jurisdiction to prepare a plan for seismic safety to be adopted as a part of its master plan. It was required that such plans identify and appraise seismic hazards in the community as well as plan for protection from geologic hazards and fires. San Francisco is well known for its devastating earthquake and fire of 1906. Nevertheless, as late as 1973 it did not have a plan to deal directly with the possibility of another event of similar magnitude.

Although the San Francisco Department of City Planning could boast of a highly competent staff in 1973, it did not possess some of the highly technical expertise needed to address many of the subject matters associated with potential seismic hazards and protection. Highly qualified experts were hired to undertake the required studies dealing with such subjects as potential landslide areas, liquifaction hazard areas, subsidence hazard areas, tsunami inundation areas and estimated building damage levels. This work was prepared in close collaboration with the staff. Staff, too, had its primary responsibilities in such areas as strong-motion instrumentation, building parapets, emergency operations, and post-disaster financial assistance. Coordination with other agencies was required. A preliminary plan was produced in 1974 and, after public hearings, a final plan was adopted by the Planning Commission.

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Introduction - Why a Plan

In 1971 the legislature of the State of California passed legislation that required each local governmental jurisdiction to prepare a plan for seismic safety to be adopted as a part of its master plan.[1] San Francisco, though famous for the devastating earthquake and fire of 1906 and with every good reason to expect another quake within the life expectancy of most of its residents, had no such plan as late as 1973. This paper will review and comment upon how a plan was prepared (as well as why none had been prepared until then), the nature and content of the studies carried out as part of the plan, the final plan document, and the process of its adoption in 1974. Toward the end, progress made since then will also be reviewed.

Although it is true that San Francisco in the early 1970s was without a plan that identified seismic hazards or addressed such questions as protection from geologic hazards and fires, it would be highly erroneous to conclude that the city government had done nothing since 1906 to prevent the kinds of loss to life and property that occurred earlier. San Francisco, like most American cities, has a building code whose basic intent is to insure the structural safety of man made construction and thereby to protect the public. That code is prepared and maintained by the Bureau of Building Inspection of the Department of Public Works. Particularly since 1948, the code had been continually updated to account for newly identified risks, in part based upon analyses of the performance of buildings in earthquakes elsewhere (such as those in Caracas, Mexico City, Iran, Morocco, Turkey, and Colombia).[2] The Bureau's staff had worked closely with local engineers in updating the code and it is fair to say that it was (and is) a technologically sound basis for construction in San Francisco. So it can be said that the city government was taking steps, continually, to insure that new construction would minimize loss of life and property. Moreover, San Francisco, also like most other major American cities, had an Office of Emergency Services, under the Mayor, responsible for the preparation, coordination and implementation of emergency plans in case of disaster. It was the general understanding, however, that the office was understaffed and underfunded, but it did at least exist and people in government were knowledgeable about it.

These efforts not withstanding, it is true that San Francisco was without the sort of plan called for in the State legislature. In a city vulnerable to another earthquake as shown in Figure 1 it is interesting to speculate as to why that would be so (understanding that it is possible only to <u>speculate</u> on this particular question). First, it may be that this is an issue whose consequences are potentially so great--fire, collapsed buildings, death, panic, total disruption of normal daily life--that people are not willing to deal with it. Those prospects may be too hard to face. Then, too, it may well be that people feel this is an issue about which little or nothing can be done in any case. That is, the forces at work <u>are so great and likely to happen without warning</u>, that people feel helpless, and calling that sort of matter to someone's attention, that is, the potential of something devastating that they can do little about, is not something they want to hear. In any case, they may feel that it--an earthquake and its

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[1] California Government Code, sec. 65302(f), 1971.

[2] Code changes were made in 1956, 1962, 1965, and 1968.

damage--cannot really happen to them, individually. Or, perhaps they are willing to risk it; to tempt fate. Another explanation would relate to technology. After all, they might reason, we have come a long way since 1906; we know more about building and what are all those building codes for, after all, if not to insure against significant damage in case of another quake. Finally, there is the question of economics. There may well be a perception that preparedness and preventative planning is very expensive in regards to earthquakes and when faced with a choice people would rather spend limited resources on today's needs than to spend them on something that might not happen, or is not urgent. It is the kind of spending that people may feel can be postponed. There could be other explanations as well. But then one might ask, might not elected and appointed officials take steps to plan for an earthquake in any case, regardless of how people in general might feel on the subject? A reasonable answer would be that local officials really are representative in this case and have the same responses as their constituents.

Explanations (or speculations) such as these may seem irrelevant in light of the state mandate to prepare a plan to deal with seismic safety.[3] In the absence of local initiative, a higher level of government enjoined local governments that they must deal with the subject.

Responsibility for the Plan - Who Did It

The task of preparing a plan was the responsibility of the San Francisco Department of City Planning. The charter of the city not only establishes the department[4], it also charges it (among other duties) with the preparation and maintenance of a Master Plan. Subsequent state legislation had mandated the preparation of nine master elements, of which seismic safety was but one[5]. The department, of course, had any number of additional legal responsibilities as well as non-mandated activities that engaged its staff. These included zoning responsibilities, historic preservation, short- and long-range housing programs, neighborhood planning, design and planning review of proposed private development projects, enforcement of planning legislation, demographic and economic research, and public information. So the Department was not without activities to keep its staff busy.

Perhaps for this reason more than for any other, the department leadership did not welcome the new requirement of having to prepare a plan for seismic safety; it was fully engaged with other matters at the time. Then, too, there was the question of budget and resources. The state legislation, though it had required local governments to prepare plan elements had not provided the funding for the local people to do the work.

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[3]They do, however, help to explain why no such plan had been prepared and they may give some indication of how forcefully a community will implement such a plan once it is prepared.

[4]Under a seven member commission, five of whose citizen members are appointed by the Mayor.

[5]The other eight: Land Use, Circulation, Housing, Conservation, Open Space, Noise, Scenic Highway, and Safety.

Plan preparation would cost money and staff that was not readily available and which might require division of energies from other, locally-determined priorities. Staff of the department did not like being told what they had to do by people of another layer of government. Such was the nature of feelings about local self-determination in San Francisco at that time. Finally, the planners were concerned about what was likely to happen to such a plan once it was prepared. If the assessments about citizen lack of concern were correct then the planners wondered what might become of a plan once it was prepared. They were a bit reluctant to go to all that trouble if no one locally was interested.

These misgivings notwithstanding, once it was determined that a plan was required, the leadership of the department determined that it should represent the best possible, highest quality professional quality work. For such a technically demanding subject in such a sensitive city they did not want to produce a second-rate product.

A first step was to get some idea of the nature of the work that would be required. This was done by the senior staff of the department in consultation with staff of the most obvious other city agency--the Bureau of Building Inspection of the Department of Public Works--and with one or two local seismic experts. Fortunately, these people do exist locally, in part because local architects and engineers have to deal with the subject regularly in regards to major new construction and also because of long-standing earthquake research at the University of California, across the Bay in Berkeley.

It was determined early that although the planning department could boast of a highly competent staff, it did not possess the highly technical expertise needed to address many of the subject matters associated with seismic hazards and protection. But for their part, the technical experts were not knowledgeable or sophisticated in city planning matters associated with seismic hazards and protection, or in local governmental organization, governmental processes, or legislation that are critical to successful urban planning efforts. Based upon these considerations it was decided to employ highly qualified consultants to undertake required studies dealing with such subjects as potential landslide areas, liquifaction hazard areas, subsidence hazard areas, tsunami inundation areas and estimated building damage levels. This work would have to be prepared in close collaboration with the staff who would be expected to know the planning implications of the findings and translate the technical work into a plan document. Staff, too, would have primary responsibilities, in such areas as strong-motion instrumentation, building parapets, emergency operations, and post-disaster financial assistance. One planner working full time and two less experienced staff professionals, all working under the direction of a senior planner, were assigned to these tasks. They would also be responsible for coordination with other relevant agencies. A committee of local private professionals (engineers), as well as representatives from public agencies, was used to help advise on all of the work.

The Research and Its Funding

Considerable research and several specific investigations were required to provide an understanding of the nature and scope of seismic problems, and to enable development of meaningful planning policies for reduction of risks from seismic hazards.[6] Over a period of approximately one and one-half years the planning staff produced five short background reports on discreet subjects, while the outside consultants, John A. Blume and Associates, produced a single, major work that included all of their investigations and conclusions on geologic and structural hazards in the city.

The background reports of the staff are reviewed here, in brief:

1. Strong-Motion Instrumentation.[7] This report documented existing federal, state and local programs to locate accelerographs in San Francisco buildings in order to help determine over an extended period how man-made structures respond to earthquakes. It also proposed a process to locate new instruments at the best locations on a selective basis.

2. San Francisco Parapet Ordinance.[8] Ironically, many of the buildings most valued by San Franciscans for their aesthetic qualities also represent potential life hazards in the event of an earthquake. The older often brick structures ironically have parapets which, if not securely maintained and tied, are likely to fall on pedestrians (and possibly taking building walls with them). Maintenance by private owners is very costly and enforcement of the parapet ordinance (passed in 1969) could easily result in removal of building features that contribute greatly to the city's unique visual appearance. This report called attention to the situation, documented the buildings in question, and made preliminary recommendations regarding the retention of these architectural features.

3. Post-Disaster Financial Assistance.[9] This report points out that once capital for construction becomes available after a major earthquake (Anchorage, Alaska, for example) the pressures for rapid redevelopment can be so strong as to make the same building mistakes that helped cause damage in the first place. In addition to observations as to sources of funds available for reconstruction, the report makes recommendations for modifications of funding processes and for reconstruction strategies that can avoid repetition of previous problems.

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[6] San Francisco Department of City Planning, "Community Safety" (A Proposal for Citizen Review, July, 1974, p.2).

[7] San Francisco Department of City Planning, "Background Paper #1, Strong-Motion Instrumentation", January, 1974.

[8] San Francisco Department of City Planning, "Background Report #2, San Francisco Parapet Ordinance", January, 1974.

[9] San Francisco Department of City Planning, "Background Report #3, Post Disaster Financial Assistance", January, 1974.

4. Emergency Operating Center.[10] This report documented the inadequacy of the existing emergency center (not large enough, poor structural design, not well equipped, not well located) described and evaluated alternative sites, and reviewed other, existing emergency operations of police, fire, and medical services. The importance of accelerating and reinforcing existing, practiced governmental functions rather than to introduce a new, unfamiliar apparatus to exercise coordination and leadership during an emergency was stressed.

5. Emergency Operations Play.[11] An evaluation of the many facets of city emergency operations was undertaken to determine the adequacy of preparedness. Although it was concluded that the city was well prepared "on paper", any number of weaknesses in operational capabilities were pointed out. These had to do with the untested nature of the emergency operations plan, a lack of good communications and the like. Public apathy and low funding levels were pointed out and a series of preliminary recommendations to correct the problems--mostly communications, public education, pre-planning, and exercises--were discussed.

Although there is no indication of active faults within the boundaries of San Francisco, the city is in close proximity to the San Andreas fault which caused the 1906 disaster and to the Hayward and Calaveras faults east of the bay, see Figure 1. And, of course, San Francisco is for all intents and purposes a totally built-up city. There is little underdeveloped land (except public parks and a military base). It followed, then, that San Francisco begin to approach the problem of seismic hazards by considering the interaction between already existing structures and their underlying geology. The two major areas of research--geological investigations and structural investigations--were carried out by the consultants.

The Geologic Evaluations were based on information selected from published and unpublished investigations performed by government agencies, private firms and individuals. Most of the information was presented in the form of maps, each dealing with a different geologic hazard. Accordingly, the following phenomena were mapped and evaluated for the city (see Figures 2 - 8).

- Soil and rock types and their areal distribution
- Fault locations (there are no active faults but it is reasonable to assume that a future quake will occur on the San Andreas fault)
- Distribution of Seismic Intensity in the various parts of the city
- Potential Landslide Areas

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[10]San Francisco Department of City Planning, "Background Report #4, Emergency Operations Center", February, 1974.

[11]San Francisco Department of City Planning, "Background Report #5, Emergency Operations Plan", March, 1974.

- Liquifaction: where conditions for this phenomenon are present
- Tsunami and Seiches areas
- Areas of Inundation in the case of major Reservoir Failure
- Areas of likely Subsidence.

In short, a number of conclusions followed from the geologic evaluations:

- It is not necessary to restrict land use in the vicinity of inactive faults.
- The lower elevations of San Francisco, where construction is on man-made fill and soft Bay muds, will undergo the highest degree of ground shaking in an earthquake similar to that of 1906.
- Similarly, the land fill and soft Bay mud areas along the northern and eastern coastlines and the western coast where there are sand dunes and high water tables are the areas where conditions for liquifaction are most present.
- The areas of possible subsidence are also the lower, filled areas.
- For the future, there should be on-site appraisal for buildings proposed in areas where liquifaction and subsidence are possible.
- Tsunamis and seiches do not pose a major threat.
- Evacuation plans should be developed and circulated in areas where inundation due to reservoir failure is a possibility.
- Land use in areas subject to landslide should be restricted and detailed individual on-site evaluations undertaken before any building permits are issued.
- The potential primary hazard to human life is not directly due to earthquake effects of ground shaking or ground failure, but to collapse or damage of man-made structures. Hazard from ground shaking in a future major quake would be the most serious because of destruction to existing weak, poorly constructed, or poorly designed buildings.
- To protect new construction against ground shaking, restrictive land use zoning is not necessary because engineering design and construction procedures can minimize structural damage due to ground shaking and can be incorporated into the building code.

The main thrust of the Structural Evaluation studies was directed at estimating damage to existing buildings that may result from a hypothetical repetition of the 8.3 (magnitude) 1906 earthquake. To gain some perspective as to what that might mean, it is important to recall that although damage was severe, over 25,000 San Francisco buildings still in service in 1973 survived that earthquake and the fire that followed. The number would have been greater had there been no fire. Of 44 completed major pre-1906 buildings, 38 were put back into use after repairs. Of the total 400,000 people living in the city at that time the estimated deaths from all causes was only 700-800.[12]

To determine the extent of structural hazards on extensive evaluation of existing structures was undertaken. This evaluation was based on data provided by city records, not on individual inspections or structural computations for specific buildings. Effort went into considering the types of buildings in the city and how those types have performed in other earthquakes, including the 1906 earthquake, the 1971 San Fernando earthquake, and other recent events outside of the U.S. Attention was given to building code requirements over the years, particularly to buildings built before the 1948 seismic code. One finding was that while many older buildings do not meet the seismic code requirements they may nonetheless be quite strong and resilient because of heavier non-structural materials and elements than those which are found in many contemporary buildings.

This was the first study for San Francisco to consider damage to all buildings and the first to provide damage level estimates on a block by block basis. Ultimately, using occupancy use, age, construction type, and personal judgement of course, formulas were developed to determine estimated levels of damage throughout the city. Potential damage estimates were aggregated on a whole block basis.

Four estimated potential damage levels were projected, ranging from severe (extensive to complete damage of non-structural elements, major structural damage, some collapse, many buildings for which replacement would be more economic or desireable than repair) to slight damage (some cracked and damage to walls, partitions and stairwells, broken chimneys, dislodged parapets and ornaments, minor structural damage).

These projections where shown graphically on a map, see Figure 9, and, as might be expected, the most severe estimated damage would be in the older areas of the city on the fringes of the intense downtown area and along the old waterfront areas where pre-code (1948) "type C" construction (masonry or concrete walled buildings with wood floors and roofs) is most prevalent. Two other maps were also presented, one showing the living unit density, by block, of these pre-code buildings and the other showing the pre-code non-residential concentrations.

Estimates of damage to structures other than buildings were also made. Some damage to freeways is to be expected. The two major bridges--San Francisco-Oakland Bay Bridge and Golden Gate Bridge--are expected to perform well, although performance of their approaches was questionable.

The study made estimates of possible deaths and injuries in a 1906-type earthquake. They were expressed as a percentage of deaths or injuries to the total number of people in each of the severe to slight damage areas at the time of an earthquake.

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[12]John A. Blume & Associates, Engineers, "San Francisco Seismic Safety Investigating", June, 1974.

Beyond these studies and estimates, the state of various lifelines and emergency services was also examined. It was generally concluded that many of the public utility companies, the transportation agencies, and emergency services were deficient in the adequacy of their seismic planning. Improvement in some of the lifeline support facilities (gas and electric) was underway. The high pressure water system (fire fighting) presented some concern and it was concluded that approaches to the major bridges constituted a problem as indicated earlier.

In the end, the engineers who prepared the structural evaluation concluded that a reduction of potential hazards in future building was achievable through: (a) planning that could avoid uncontrollable geologic hazards in siting, (b) constant monitoring and updating of the building code to insure that new construction has adequate resistance, and (c) by new building that, over time, would replace high risk structures. For existing poor risk buildings a step by step process was postulated to eliminate hazards, starting with the pre-code "type C" buildings. Priority would be given to the buildings that housed public uses, then to the high living density "type C" buildings, and then to other areas where the probability of severe damage was strong (see Figures 10 and 11). The assessment also pointed out the relatively minor costs associated with improvement to older, typical San Francisco two- to three storey wood frame, narrow buildings by adding resistance in the transverse directions with more bracing in the form of walls, partitions and knee braces.

In all, then, the geologic and structural evaluations, together with the work on parapets, strong motion instruments, financial reserves, and the state of emergency preparedness gave to officials and residents of San Francisco a reasonably complete picture of the likelihood of future damage in a major earthquake, the state of current planning and preparedness, and several ideas of what could be done to minimize future risk.

The Seismic Safety Plan

The investigations and research by the consultants and the planning staff did not, of course, constitute a plan. Rather, they were the basis for a plan that the staff was to prepare in a manner similar to other master plan elements (e.g., elements for transportation, residence, urban design, parks and recreation). The plan dealing with seismic safety (under the title "Community Safety") took the form of a series of objectives and policies, followed by recommended programs to carry out the plan. In this case there were four objectives, dealing with life safety, preservation, emergency operations, and reconstruction.

The first objective was to reduce hazards to life safety and to minimize property damage and economic dislocations resulting from future earthquakes. This objective required a confrontation with the idea of acceptable risk, understanding that it would be impossible and perhaps not even economically feasible to eliminate all risk. The determination of a level of acceptable risk, the staff concluded, should be based upon the importance of a structure to the general welfare of the community (e.g., a hospital), the hazard a particular use might present to the larger community, the intensity of use or the density of occupancy, whether or not exposure to risk is voluntary or not (e.g., shopping in a store versus being in a convalescent hospital), and economic factors related to how much the community might be willing to pay to reduce the risk. The first policy, to achieve the objective, proposed three classifications to be dealt with by building standards and risk abatement programs:

- a basic level of risk where no structural collapse endangering lives would occur, which could be handled by the building code (for single and two-family homes, apartments, stores, industrial buildings)

- a lowest risk category for such functions as emergency operations, fire fighting facilities, ambulance dispatch centers, and public records storage structures

- an intermediate level of risk for such buildings as large stores and apartment complexes, hotels, schools, theaters and convalescent hospitals.

Other policies under this objective were directed to orderly abatement of hazards from existing structures, abatement of hazards in identified "Special Geological Study Areas" (Figure 12) where potential ground failure and inundation hazards exist, the requirement of special geologic or soil engineering studies in such areas prior to any construction, modification of land use plans to be consistent with the risk categories, and periodic review and upgrading of relevant codes and building standards.

The second objective was to preserve the architectural character of buildings and structures important to the city's unique visual image, but consistent with life safety considerations. The policy under this objective called for a joint effort between the Bureau of Building Inspection, whose responsibility it is to enforce the parapet ordinance, and the Planning Department, to eliminate the hazards from architecturally significant buildings but to do so in such a way as to maintain their design integrity.

The third objective was to the subject of adequate emergency operations preparation. Policies to achieve the objective deal with up-to-date emergency operations plans and equipment, fire prevention and fire-fighting capability, the need for emergency access routes for both emergency operations and evacuation, and continued public education.

The last objective was concerned with reconstruction and the policies directed to: (a) insuring that development after an earthquake take place in a timely fashion (via a reconstruction planning committee), (b) reducing pressures for unnecessarily rapid construction, and (c) rebuilding in accordance with the city's overall comprehensive plan.

It is noteworthy that the plan contained no major proposals for drastic change to the existing character and physical design of the city to follow a major earthquake. In contrast, the planners might have looked at such a catastrophic event as an opportunity, one to reconstruct the city as they would like if they only had a chance, or at the very least to correct major problems through new construction in the event that an earthquake gave the city that chance. There was no such "ideal" in the proposed plan document. Why? On the one hand it is fair to say that this would have been an inappropriate document in which to make massive reconstruction proposals in that it would have seemed that the planners were proposing to feed off of disaster. Then too, the planners could say that they had already dealt with that issue, at least in part, in the other master plan elements, such as for housing, transportation, urban design, and open space. But perhaps the more significant reason is that those responsible for the plan were by and large satisfied with the physical makeup of the city as it existed. They, too, were hoping that a major earthquake would not happen. They did not want to design a new San Francisco.

The Plan Adoption Process

The proposed plan was published as a proposal for citizen review in July, 1974.[13] At the same time, a presentation was made to the Planning Commission (that oversees the work of the staff) at a public meeting. Copies of the plan were made available to citizen and business organizations, elected officials, other governmental departments, and any citizen who was interested. This was to be followed by a period of review for all who were concerned and public hearings where those concerns could be aired and discussed publicly, before the Commission. This was the process that had been followed with other plan proposals.

It is worth noting here that the period of presentation, review and comment for other plans had been lively and full of interest and citizen participation. (The plans for urban design or transportation are but two of many examples). Meetings were widely attended and there was much debate. Often, many meetings were held and the preliminary plans changed more than once in response to objections, concerns, and new ideas that emerged during the process. The period between initial plan presentation and final adoption might be longer than six months and the plan might be significantly different than the initial proposal.

That was not to be the case this time. Few people (perhaps 30) attended the initial presentation and there were relatively few questions or requests for additional information or staff presentations by citizen groups in the period prior to the public hearing. Nor was the public hearing typical by San Francisco standards. Proposed minor changes were generated more by staffs of other departments, by the consultants and by the planners themselves than by citizens. The plan was adopted by the Planning Commission in September. It seems safe to conclude that the planning staff's original assessment of the interest of elected officials and of citizens was correct: for most people in San Francisco, even if they recognized the potential chaos and misery that could be generated by another major earthquake, this was not an issue of immediacy and therefore not of particular interest.

The plan, once passed, was sent to the Board of Supervisors (San Francisco's City Council), the Mayor, and to departments of government that might be affected. It is important to realize that under San Francisco's governmental structure and process the master plan (sometimes referred to as general plan) is an advisory document. It does not necessarily mandate actions. Implementation is always a long process that requires any number of separate actions that may involve legislation, direct actions by government, financial support, commitment of staffs, and involvement with state and national programs. The plan, like all plans, represented a policy document.

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[13]San Francisco Department of City Planning, "Community Safety Plan: A Proposal for Citizen Review, July 1974.

Developments Since Adoption of the Plan in 1974:

To the extent that San Francisco looks different in 1981 than it did in 1974, the differences cannot necessarily be attributed to the plan, at least not directly. That would be expected. The new buildings are larger and taller (especially downtown) than those they have replaced but that is because of economic rationale and current style rather than for reasons of seismic safety. In some cases the new buildings have replaced older pre-code, "type C" buildings that might not have withstood a severe earthquake, but it is once again the economics of the time that has dictated the change, not the plan. It is reasonable to observe, however, that the newer buildings respond to updated structural standards reflected in the building code and therefore make for a safer environment.

Visible change, of course, is by no means the major indicator of a plan's effectiveness or of a community that is more prepared for an earthquake or is safer than it would have been. Legal, administrative and procedural changes as well as detailed construction improvements are as likely to be conclusive in planning for community safety as are those changes that came from direct public and private actions. Whereas a detailed accounting of what has happened in San Francisco in regard to seismic safety since 1974 is not available, interviews with planners who worked on the plan (most have since left the city) and who are presently members of the planning staff can give some idea of progress.

In terms of the life safety issues as they relate to new buildings, the environmental impact reports that are required for all new major structures make it common for both developers and city staff to consult the geologic maps that were prepared as part of the plan in order to determine if any situations exist that require special consideration. Staff of the Bureau of Building Inspection might well have made such determinations in any case. Nonetheless, it seems reasonable to conclude that the plan has at least helped to provide a greater general awareness of the seismic issue among developers and city staff.

Again in relation to new development, there have been major changes in the city's zoning legislation since 1974. The question, then, is whether or not the seismic plan had any impact on the changes. Apparently it has not had a direct impact (according to the staff interviewed). However, it is expected that detailed planning for the area immediately south of downtown and on the eastern waterfront will start in the near future. It will be recalled that these areas are precisely those where many geologic factors combine so that a significant impact from a major earthquake can be expected. It is fair to anticipate that seismic considerations will be a factor in the detailed planning.

From time to time since 1974 there have been statements (usually from City Hall) about the need to renovate the older masonry buildings, especially in those areas adjacent to the downtown area. Many people live in those buildings and many of them have limited incomes. Replacement housing for them at affordable prices do not exist. They tend to see the seismic safety issue as one that will displace them from their dwellings and so they see the issue in political terms and tend to resist code enforcement. Nonetheless, some buildings have been renovated and they have been brought up to seismic standards when that occurred. Apparently the impetus to renovation is not seismic safety in these cases, but rather, seismic safety is a spinoff.

In regards to preservation of architecturally significant buildings it is clear that the parapet ordinance acts against preservation and results in a quandary. To date the planning staff has tried to use persuasion as a means of encouraging owners to maintain important architectural features while correcting for safety reasons. Consideration of an incentive that would permit building owners to transfer their development rights to other sites while renovating the older structures is taking place.

A more detailed plan for emergency operations than existed at the time of the Plan (and which was so strongly criticized) was apparently completed and the Office of Emergency Preparedness if apparently better staffed and more effective than was previously the case. A public information document on earthquakes has been produced and distributed.

The last objective of the plan had to do with reconstruction after an earthquake. It is not apparent that any of the more specific recommendations, such as that for a reconstruction planning committee, has as yet been implemented.

In all, then, it does not appear that the plan itself has thus far had a major impact on either the nature of construction in the city or how it goes about its business.

A Brief Conclusion

This accounting of San Francisco's plan to deal with a future earthquake of the magnitude that caused such damage in 1906 permits some tentative, cautious conclusions. I say tentative and cautious because I am not sure that I am right even though I was the Director of the San Francisco Department of City Planning when the plan was prepared. Obviously, I had many misgivings about the plan and how likely or unlikely were the chances of implementation. So it is possible that any conclusions would be self-serving. Also, it is very difficult to conclude with any certainty just how effective the plan objective and policy focus has been, especially in only seven years since its adoption. Therefore the caution. With those caveats, several reasonable conclusions that seem to follow from this case study are:

- The plan might never have been prepared had it not been mandated by the State.

- Regardless of the plan, a considerable amount had already been done by the Bureau of Building Inspection to avoid major problems that could come with a major earthquake. These include major and continuous revisions to the building code and the parapet safety ordinance.

- The plan has not had a major, direct effect on land use plans, preservation or post-earthquake preparation.

- It is difficult to get people concerned, before the event, about a major earthquake and how to avoid its consequences, and a plan is not likely to be the key to focusing public concern.

- Studies undertaken as part of the plan provided a sound base for determining where specialized, sight-by-site geologic studies should be required prior to construction. These studies are an effective part of the plan.

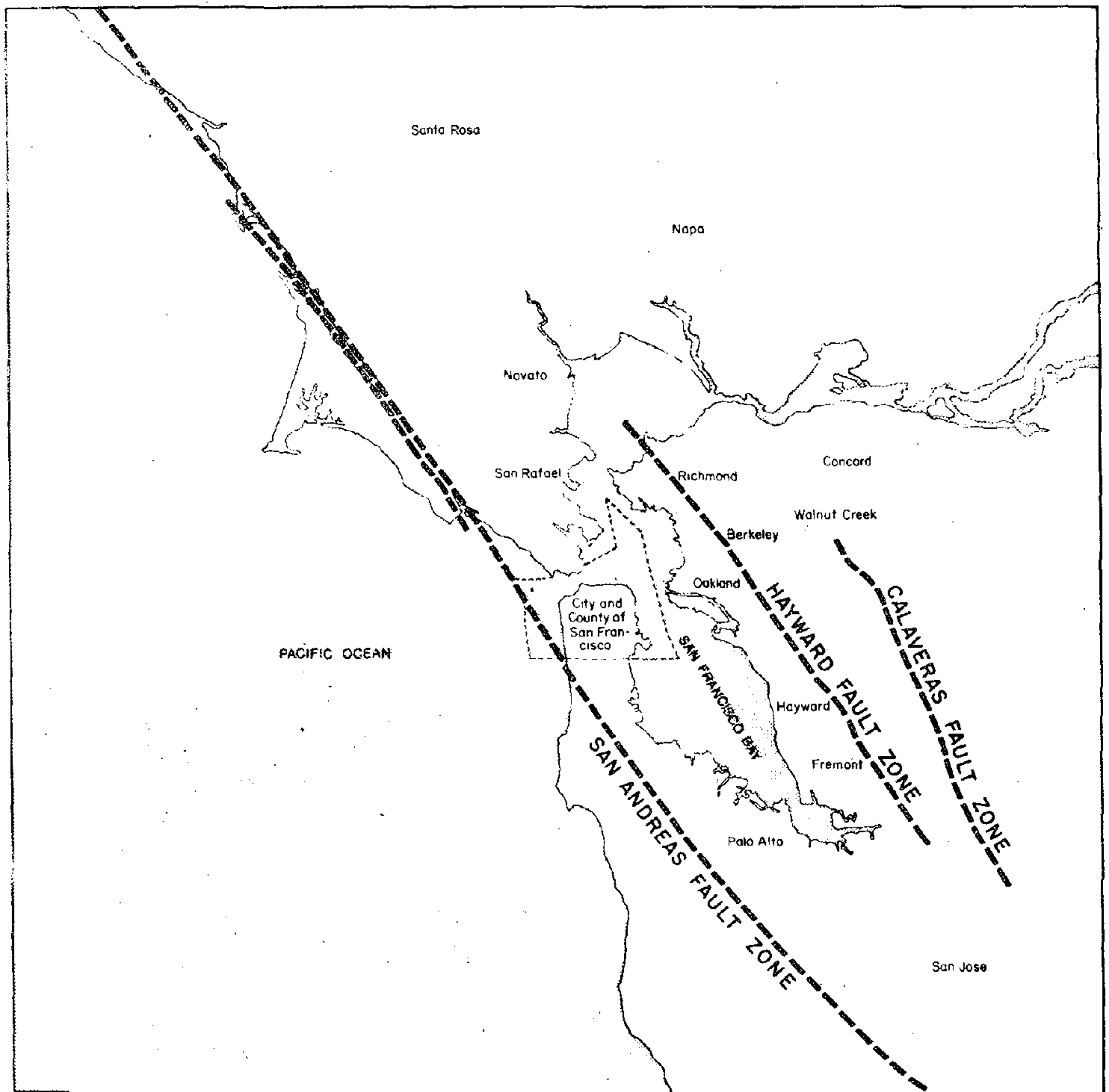
- Structural studies provide a good basis for determination of acceptable risk, for arriving at priorities of where to start risk abatement, and for showing how many existing homes can be made safer with little cost.

- Offices of Emergency Preparedness, like San Francisco's, may find it difficult to be effective simply because they are special: because they (and their staffs) are not a normal, significant part of day-to-day government and are therefore alien and even suspect to those who are.

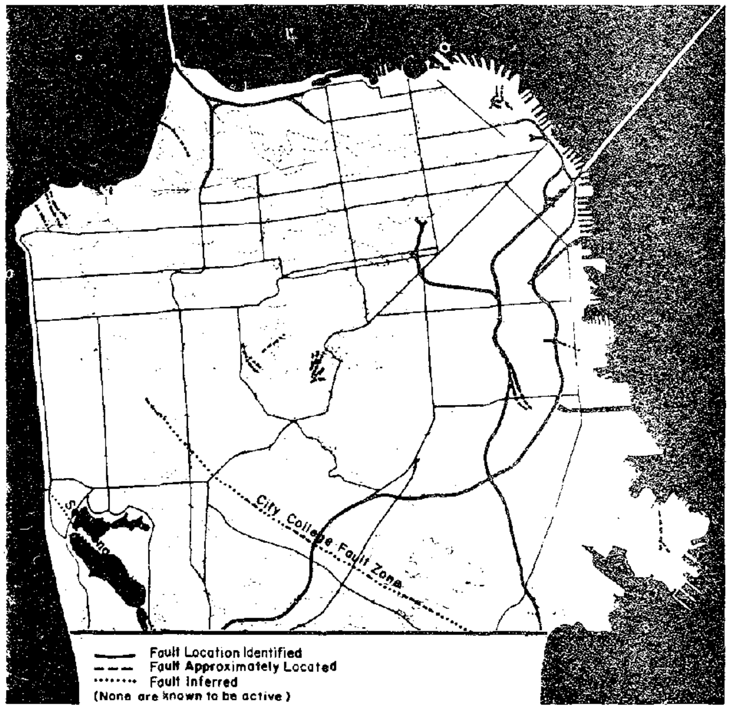
- It may be more appropriate to designate responsibilities for emergencies to those agencies that must deal with emergencies "normally", most notably the police, fire, and emergency health departments.

- In the long run, public education, including drills, in schools and places of work on a regular basis may be one of the most effective tools to dealing with the immediate consequences of earthquakes.

This working paper has been prepared, in part, on the basis of personal experiences while Director of the City Planning Department of San Francisco from 1967 to 1975. Conclusions drawn relate directly to observations made during the development of the Seismic Safety Plan. It should be noted that positions taken and methods used in the development of the plan could vary significantly in other regions, communities, and countries.

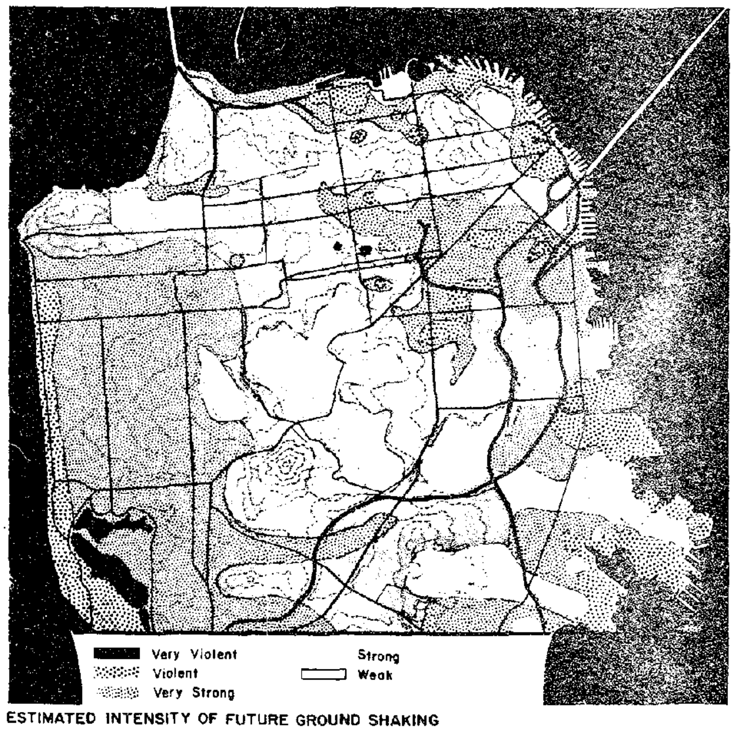


ACTIVE FAULTS IN SAN FRANCISCO BAY AREA
Source: U.S. Geological Survey / J. Schlocker, 1970



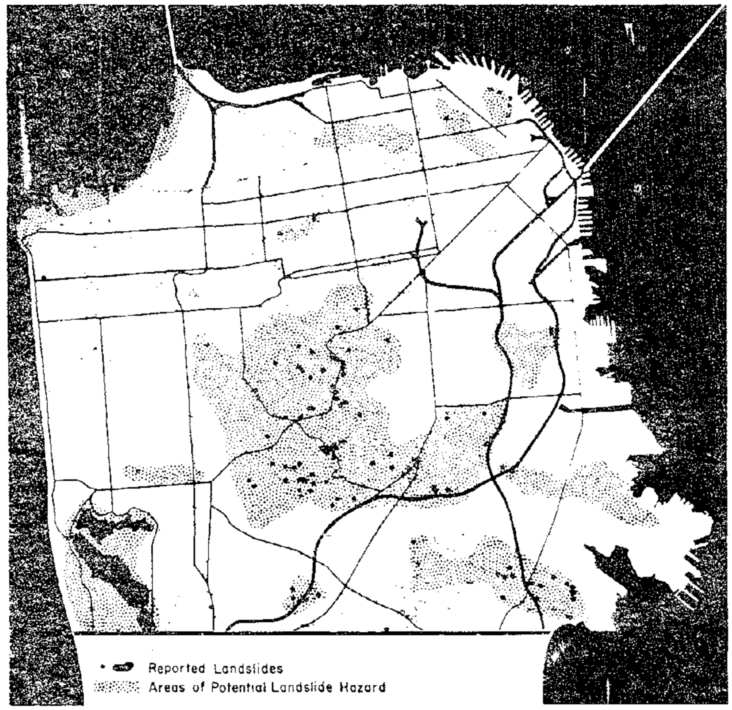
FAULT LOCATION IDENTIFIED

Figure 2



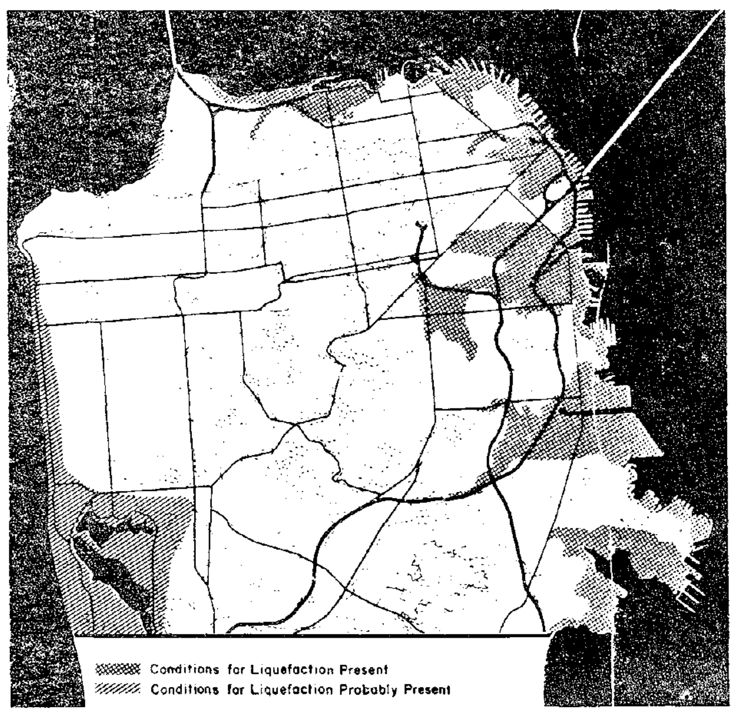
ESTIMATED INTENSITY OF FUTURE GROUND SHAKING

Figure 3



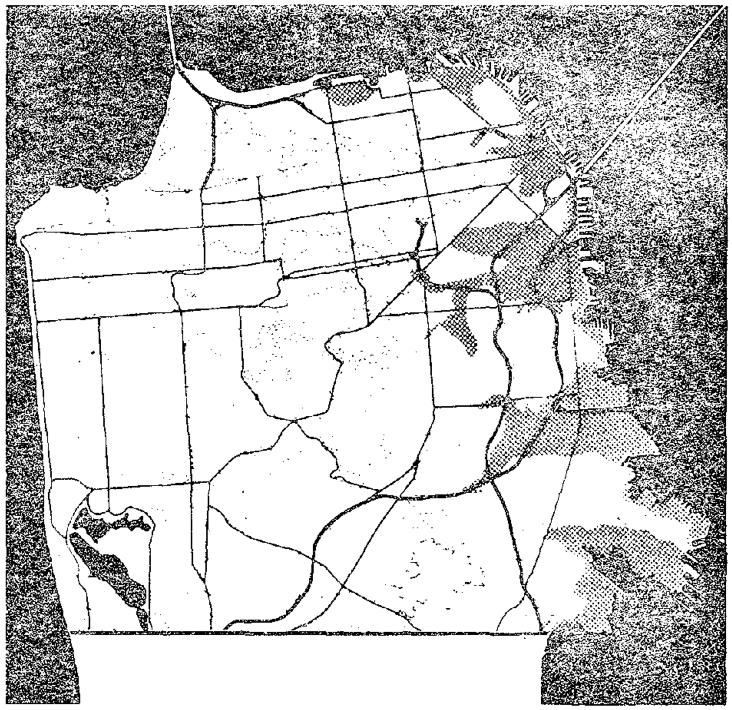
POTENTIAL LANDSLIDE AREAS

Figure 4



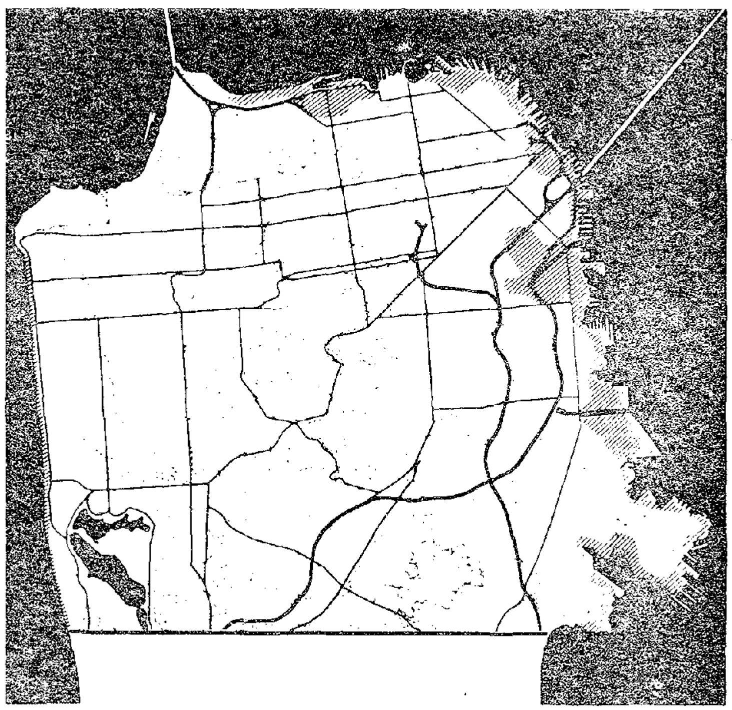
POTENTIAL LIQUEFACTION HAZARD AREAS

Figure 5



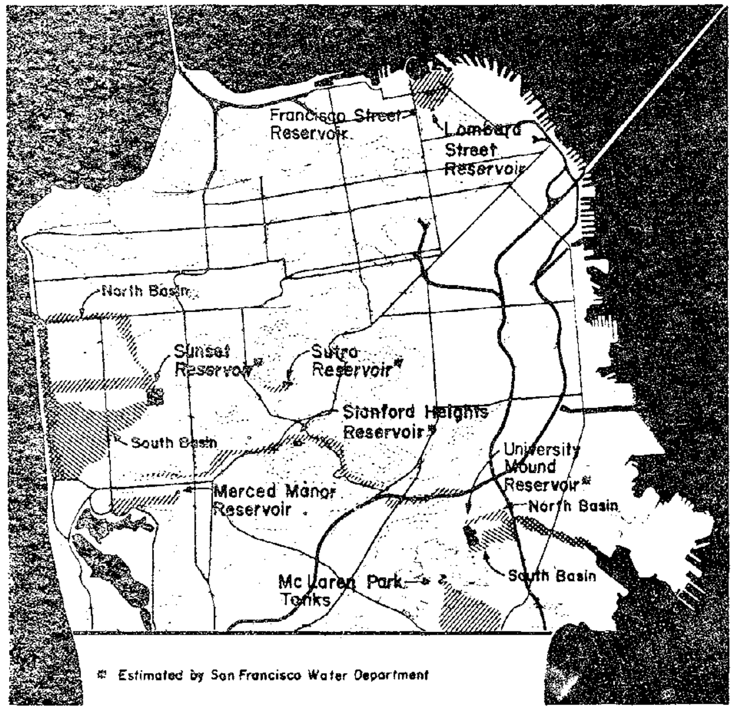
POTENTIAL SUBSIDENCE HAZARD AREAS

Figure 6



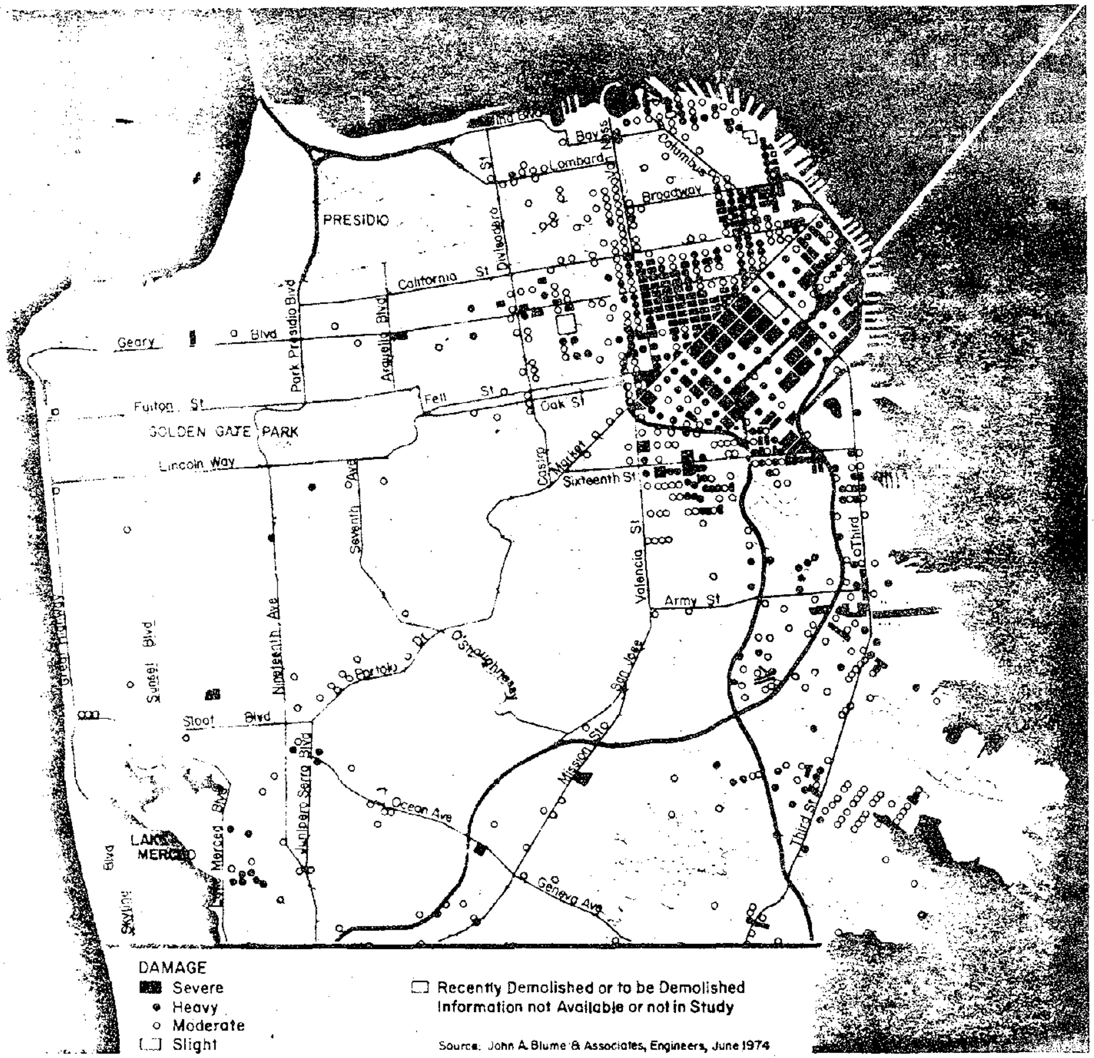
POTENTIAL TSUNAMI INUNDATION AREAS: Due to 20 Foot Tsunami at the Golden Gate

Figure 7



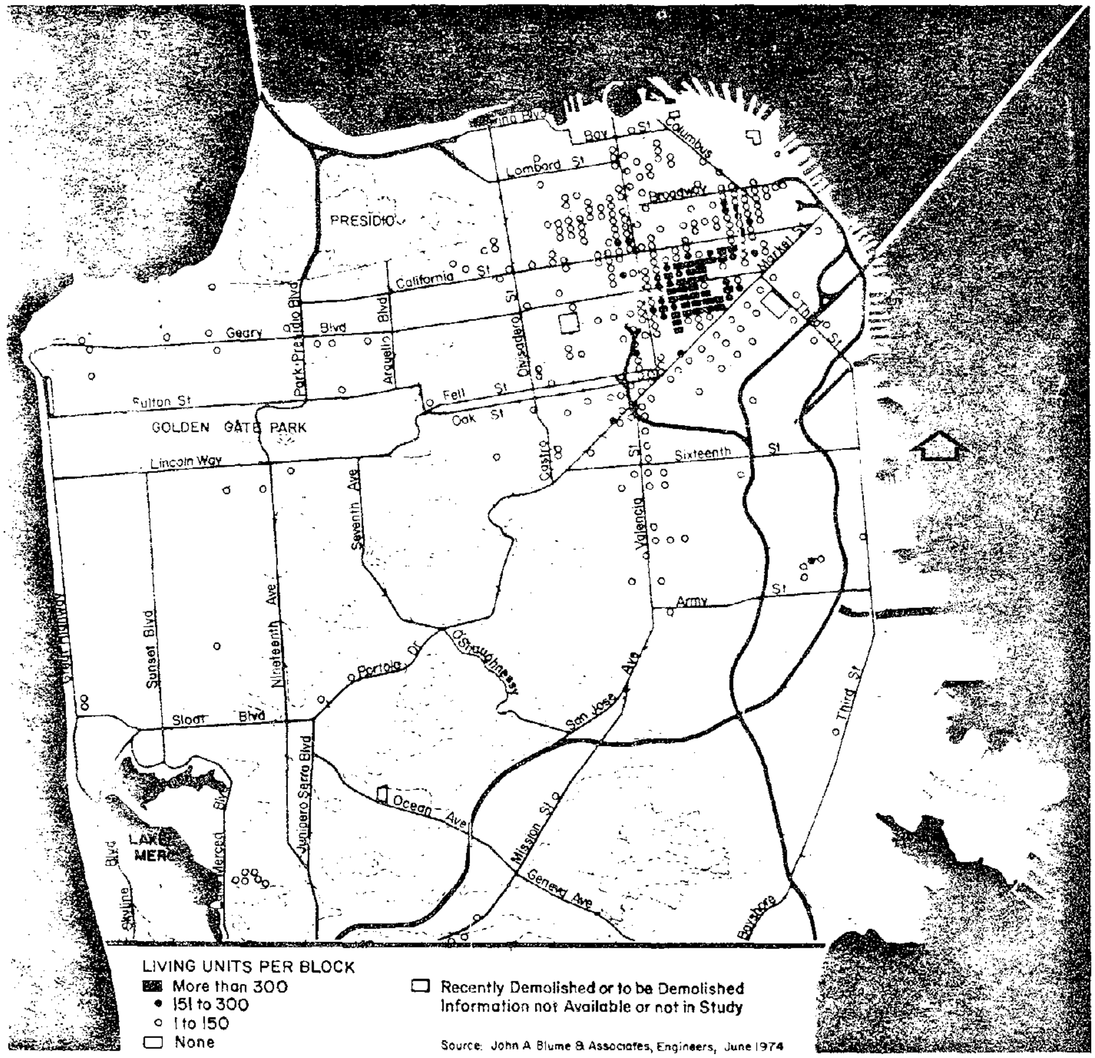
POTENTIAL INUNDATION AREAS DUE TO RESERVOIR FAILURE

Figure 8



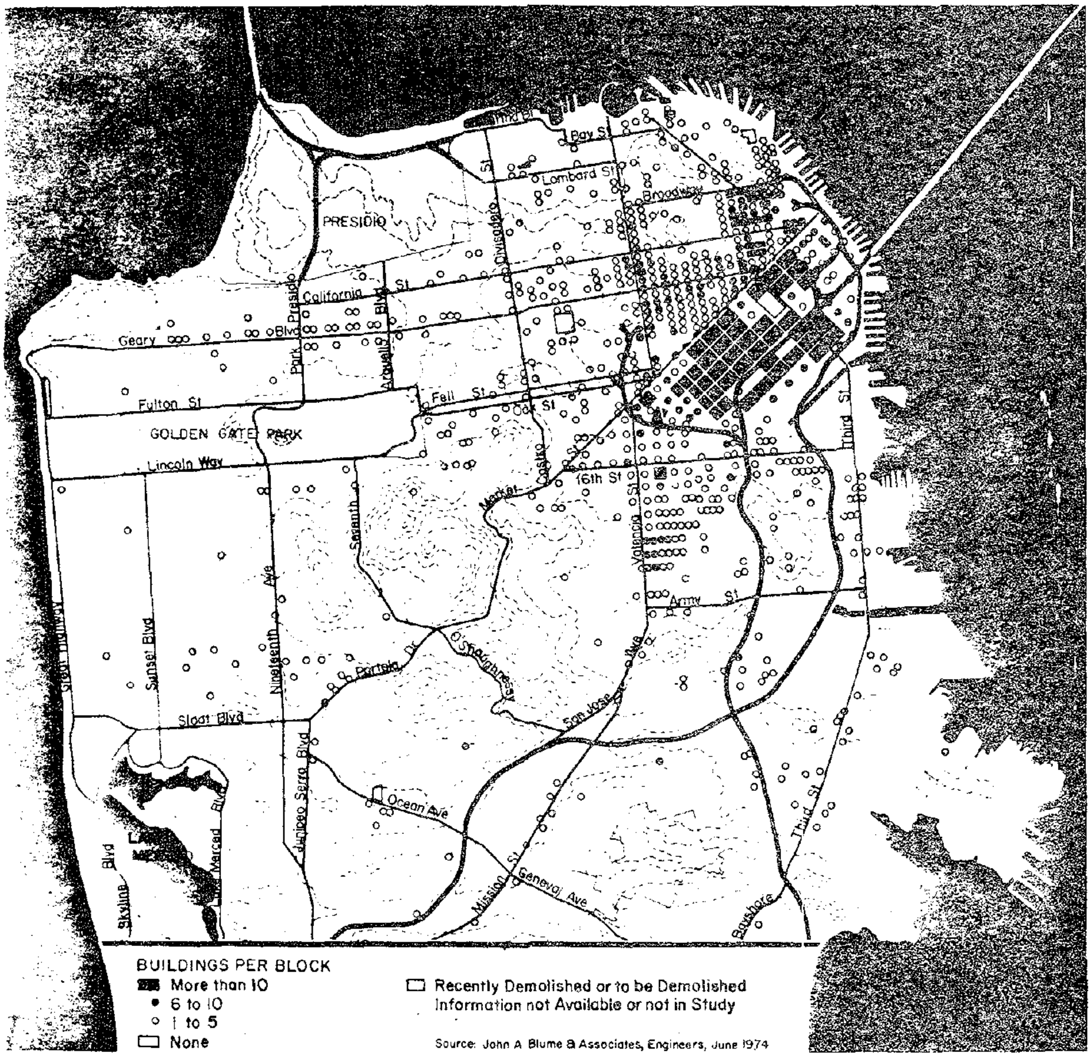
ESTIMATED BUILDING DAMAGE LEVELS FOR A "1906 TYPE" EARTHQUAKE

Figure 9



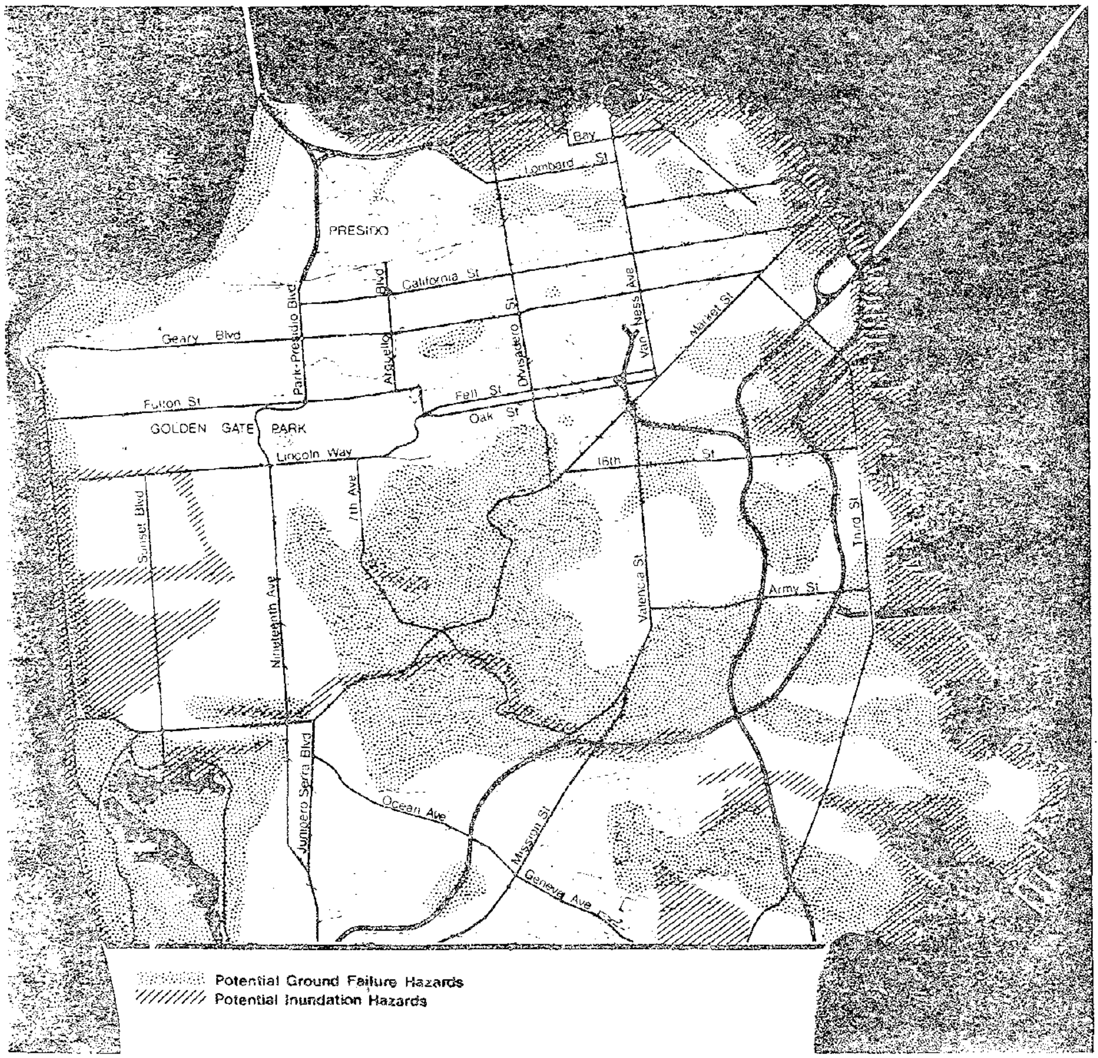
DENSITY OF LIVING UNITS IN PRE-CODE TYPE C BUILDINGS

Figure 10



DENSITY OF PRE-CODE TYPE C NON-RESIDENTIAL BUILDINGS

Figure 11



SPECIAL GEOLOGIC STUDY AREAS

Figure 12

PLANNING FOR THE RECONSTRUCTION OF
EARTHQUAKE STRICKEN COMMUNITIES

Barclay G. Jones[1]

ABSTRACT

A disastrous seismic event is a crisis in the historical evolution of the economic system in the impacted region. Since seismic activity is highly localized in portions of the earth's surface, the particular system has probably experienced a long series of such crises at sporadic intervals throughout its history. Reconstruction planning should be approached in the context of the continuum of the system's evolution. Planning should guide rebuilding so as not to result either in stasis in the system nor excessive discontinuity but rather the facilitation of development trends. At the same time, it should improve the rapidity and efficiency of the process and insure that the reconstructed system is less vulnerable to future earthquakes. To accomplish these purposes, reconstruction plans must take into account the trends that have characterized the changing composition and spatial distribution of the population and economic activity within the region and make use of the rebuilding process to facilitate adjustment to new requirements. Analytical techniques must be applied rapidly to develop a sufficiently profound comprehension of the nature of the spatial system in the impacted area and the way in which it has been transforming over time to be of use in making decisions about the location of activities as reconstruction occurs. The specific character of sites and locations for activities must be matched with the particular vulnerability of those activities, and the structures which house them should be designed to minimize risk.

INTRODUCTION

Slippage along the edge of one of the great tectonic plates that form the crust of the earth releases seismic energy, and an earthquake occurs. The seismic event is space and time specific. It has a focus and the projection of this point on the surface of the earth is the epicenter. There will be a region of varying size within which strong ground motion occurs. The size of the region will vary with, among other things, the depth of the focus and the amount of seismic energy released. The impacted region has topographical features and soil characteristics. The physical effects of the seismic event on the earth's surface may be various. There can be fault displacement, either vertical or horizontal, strong tremors, liquifaction of the earth, rock slides and earth slides, sea surges or tsunamis, and others (Bolt, 1976).

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The impacted region has human occupants in all probability. It will have a population of a given size and given demographic characteristics. The population will be organized into a social, economic, political system. In developing this system, the population will have created a whole series of physical structures. These include modification of the landscape for the exploitation of natural resources through horticulture, animal husbandry, silviculture, the extraction of minerals, and the management of water. This will have resulted in the clearing of forests, the cultivation of fields, the creation of pasture, planting of timber lots, digging of mines and quarries, erecting elaborate agricultural terraces, creating ponds and channels, building fences and walls. In addition the population will have invested vast amounts of time, energy and resources in the building of structures which can be classified as of two types: those intended for human occupancy and those intended to be unoccupied. Occupied structures are primarily for protection from the elements and animal and human predators. Fairly standard relationships exist between the number of buildings and the size of population in urban centers and regions. While these relationships vary with technology and income as well as cultural and other factors, there is reasonable stability in them.[2] Buildings provide shelter for economic activities of various kinds, residentiary activities, social and cultural activities. Many of these structures have been elaborated with tremendous care and have symbolic significance beyond their immediate use as historic, artistic or cultural monuments. Unoccupied structures are extremely numerous in their varieties. They include wells, dams, docks, roads, causeways, bridges, tunnels, aqueducts, canals, and many other elements. The unoccupied structures usually represent by far the greatest percentage of all manmade structures in the region, and their initial cost and replacement value usually vastly exceeds that of occupied structures. The human population, in addition to structures, will also have accumulated over long periods of production a large number of artifacts. These include tools, implements, clothing and various sorts of textiles, furnishings, machinery and equipment, vehicles and ships. These artifacts are necessary, not merely in the conduct of economic activity but in the pursuit of everyday life (Bates, 1981). Many of these artifacts will also be objects of beauty and have symbolic value historically, culturally or socially.

The destruction that results is a consequence of the interaction in time and space of the seismic event and the elements of the social and economic system. The severity of the impact of the earthquake is measured in these terms. There is an impact on the population within the region that results in mortality, in injury, and in adverse effects on physical and psychological health. Most of these effects do not result directly from the seismic event. They result indirectly through the effect of the seismic event on structures and the interaction of the population with the structures.[3] The extent of the effect is, therefore,

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[2]For a brief discussion of these relationships and their use in earthquake relief and recovery planning, see (Jones, 1981). For a more complete discussion of the model of the relationships and presentation of data and analysis, see (Jones, Manson, Mulford, and Chain, 1976).

[3]In Automotive Crash Injury Research, it is customary to make a distinction between the primary and the secondary accident. The primary

extremely time sensitive since there are diurnal and seasonal and other temporal patterns of occupancy and association with structures. An earthquake in the middle of the night may find most people asleep in their residences. One in the morning of a workday will find them at their places of work. One on a holiday in warm weather may find most of them outdoors.[4] The number and configuration of victims will modify the human resources in the region. Not only will the population be a different size than before the event, but since mortality is not randomly distributed but varies with age, sex, and income, the demographic characteristics will have changed (Glass, et al, 1977).

The seismic event will also have a substantial impact on the regional, economic system depending upon a number of factors. The interrelationships between people and between people and the environment they inhabit in the impacted region will have changed. Past relationships may no longer exist at all, and new relationships may have been created. For one example, in the Friuli earthquake of 1976, it seems that a certain amount of the archaic alpine agriculture which might have persisted for another generation was eliminated quite rapidly. To some extent, that was a result of the evacuation of the population out of the impacted region to available shelters elsewhere for the duration of the approaching winter. With no one left to tend the livestock, they had to be slaughtered. Consequently, both the physical structures and the stock of animals associated with an animal husbandry activity were eliminated (Cattarinussi, 1981).

Many of the landscape features may have changed drastically. Earth slides and rock slides may have changed the character of large areas and eliminated many physical elements. Subsidence, fault displacement, the devastation of sea surges and tsunamis may have substantially altered the landscape and destroyed many of the modifications that had been made by the population to make it productive and useful for their purposes. Among the features that may have been radically changed are waterways, water impoundments, estuaries and natural harbors.

Of course great damage may have occurred to the physical structures, and this is usually considered the most important impact of an earthquake. Certainly most of the attention that is given to reducing vulnerability focuses on structures. Both occupied and unoccupied structures will have suffered. The failure of both will have caused indirect effects on the human population and the artifacts that are so necessary to the operation of the system. Buildings in which economic

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accident refers to the interaction of the vehicle with the environment which results in damage to the vehicle. The secondary accident refers to interactions of the occupants with the vehicle and other environments as a consequence of the primary accident. Most of this research has focused principally on the secondary accident. In earthquake research conversely, most research has focused on the damage to the structure.

[4]Diurnal and seasonal distributions of population have been studied through daytime-nighttime population research and more recently through time budget research. For an empirical study of this kind and an extensive review of the literature, see (Arbeit, 1981).

activities are carried out will have suffered damage. The implements, equipment and machinery housed within them, as well as the inventories of fuel, materials and products, may have been destroyed. Nonstructural damage to the contents of these buildings may be even more severe than damage to the structure itself. Residential buildings may have been lost depriving parts of the population of shelter. But even more importantly, they may have lost their entire accumulation of household implements, clothing, textiles, furnishings, including artifacts of tremendous symbolic value to them. Structures in which cultural and other public services are housed may have suffered similar kinds of damage. Of particular importance is damage to structures representing historic and cultural monuments which may have provided the community with tremendously important symbolic value and contributed to the stature of the community in the larger regional system. Damage to unoccupied structures such as those associated with transportation, the supply of water, fuel and energy, may be extremely important. Of course, there can be direct and indirect effects on the population from this also. The collapse of dams may result in secondary disasters of floods and long term impacts from the deprivation of the source of water for irrigation, water supply, energy, or whatever the original purpose of the dam was.

The loss of the inventory of artifacts in the impacted area has already been alluded to several times. However, because of its extreme importance, it should be emphasized again. The loss of artifacts can in terms of economic value far exceed the loss in physical structures, and furthermore the ability of the system to resume the normal level of production and capacity for producing income may be far more dependent on the artifacts than on the structures. In the crosscultural level of living indices mentioned earlier such as those by Bates, residential structures are included as only one of the elements of a much longer list of artifacts.

The destructiveness of a seismic event is measured in terms of the capability of the regional economic system to operate after the event as it did before. The estimates of the economic loss sustained is measured in this way. The estimates of the cost of reconstruction is basically what will be necessary to make the system capable of operating at the previous level.

RELIEF AND RECOVERY PROCESSES

Immediately following a disastrous seismic event there is a period which is usually referred to as the relief phase. The normal response of the population in the impacted area after brief periods of panic and numbing shock is that of a great burst of energy. There is an intensive mobilization of local human resources. People work at feverish pitches for vast numbers of hours to the point of exhaustion. A sense of cooperation pervades, and individuals travel around the area finding ways in which they can provide assistance to others. Survivors have to be rescued; the injured have to be treated; the dead have to be buried; water supply and other environmental health systems must be restored; food supplies assembled; shelter acquired for oneself and others. Debris must be cleared, artifacts recovered, structures and artifacts repaired, and attempts made as quickly as possible to restore to some degree the operating capability of the system.

The reconstruction process follows. In a very natural way, the population will set rapidly about reconstruction. More extensive attempts to recover and salvage, repair and restore artifacts will be a major preoccupation. The structures both occupied and unoccupied which contribute to the productive capacity of the region will receive first order attention. Demolition of ruined structures and clearing of rubble, repair and rebuilding are the normal phases of activity. Experience in many parts of the world suggests priority will be given usually first to those elements which relate to economic productivity, secondly to investments in community services infrastructure such as transport and communications facilities, then residential structures, and finally cultural elements.[5]

The next phase is that of the recovery of the region. Human socioeconomic systems are extraordinarily resilient. The system will recover. The extent to which it does and the speed and completeness of recovery will depend upon a variety of factors. The degree and the nature of destruction, the extent of damage, and the specific nature of the loss of human resources will all play important roles. A vitally important factor will also be the secular trend that has characterized the level of activity in the region over the previous period. If the region has been developing, growing and becoming increasingly productive, the recovery period will be relatively brief and the region may rapidly transcend previous levels of accomplishment and even seem to benefit from the disaster. However, if the region has been undergoing a steady decline from previously high levels of development and activity, the disaster may accellerate the process, because surviving elements which were destroyed may be far beyond the capability of the system to restore. Furthermore, they may no longer have the usefulness they did at previous levels of development.

An extremely important intervening variable in this process is that of extra-regional assistance. Human resources are likely to appear rapidly on the scene from surrounding regions. These may include highly specialized technical personnel as well as just ordinary manpower resources. Seismologists, earthquake engineers and others may appear to help assess damage, identify repairable structures, and estimate reconstruction costs. Technicians trained in locating and rescuing survivors, and medical personnel to treat the injured and prevent further deterioration of environmental health are likely to be needed, too. Material and supplies can be expected to be offered. Food and water and medical supplies are often quite necessary. Replacements of textile artifacts that have been lost such as clothing, blankets, and other household implements, are very useful when there has been much residential destruction. Temporary shelter, some of which can be used for rather long duration, is also customary.

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[5] There are a number of notable occasions on which the reconstruction of cultural monuments has taken precedence over almost everything else after a very destructive disastrous event. This seems to have been particularly the case when the monuments had great symbolic significance and were important to re-establish the identity of the stricken population. One of the classic examples is the rebuilding of the Old Market Square in Warsaw after World War II (Ciborowski, 1964).

This summary description of the nature of a seismic disaster and the phases through which the regional economic system that occupies the impacted region proceeds is intended as background against which to consider the area of this paper. A process has been described. The question to be addressed here is how can planning assist in such a way as to make the process work better. The assumption is that we are dealing with a system in an impacted area that has suffered a disaster. Primary attention will be given to the reconstruction process rather than to preventive measures. However, these are not inseparable and some considerations will span between them.

PLANNING FOR RECONSTRUCTION

Objectives

Planning can assist the reconstruction and recovery process basically in three ways. First, it can increase the efficiency and rapidity with which reconstruction and recovery occurs. Second, it can help to guide the reconstruction process so that the rebuilt regional economic system is less vulnerable to the disruptions of seismic events than it was before the disaster. Third, and quite importantly, it can help to see that the reconstruction efforts and the immense investments in rebuilding that are made are carried out in such a way as to promote the development of the region in an optimal fashion.

It is immediately apparent that there are obvious conflicts between these three objectives. The most efficient and rapid way of restoring the stricken community may leave it equally vulnerable as before and impede its development. Reducing the vulnerability of the reconstructed community to seismic disasters may make the recovery period substantially longer and the process less efficient. It can also be carried out in such a way as to impede growth and development. Carrying out reconstruction with the primary purpose of promoting development may delay the recovery process inordinantly and reduce its efficiency extensively. It also may result in a new system which is even more vulnerable certainly in terms of higher levels of economic loss than the previous community.[6] Likewise pursuing any two of the three objectives could seriously jeopardize the achivement of the third. With such inherent conflicts between the objectives, obviously trade offs will have to be made, and these must be given most thoughtful consideration.

Continuity in Change

After a social and economic system has been devastateu by a natural disaster, there are tendencies towards one of two polar positions. The first of these is to restore the status quo. Put back everything exactly

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[6]The disaster literature is filled with embarrassing examples of instances in which older buildings survived new ones. At Al Asnam, Algeria, many of the losses of buildings in the earthquake of 1980 were modern style buildings which had been built in the reconstruction after the earthquake of 1954 by foreign designers. While the historic town of Budva received extensive damage in the Montenegran earthquake of 1979, the

where it was the way it was. The second propensity is to view the situation as a tabula rasa. This assumes there are no givens and that anything can be put anywhere without reference to what was there before or where it was located. The first approach has the tendency to freeze the community in a completely archaic pattern, but now no longer in plant and equipment which is largely or completely amortized but in brand new plant and equipment the burden of which must be born for many years. Change and development can be stifled. The second approach frequently imposes a pattern of activities as well as a pattern of locations and spatial interrelationships which are totally foreign to the community and which may be far beyond the cultural capacity of the population to absorb. In the extreme, it may consist of putting a peasant who has lost a traditional dwelling in a modern house equipt with plumbing, wiring, heating, cooking and hardware that are appropriate to a completely different life style than his own. The system may collapse of its own weight because the change that was imposed was too drastic, and maintenance and operation are beyond the resources of the community to sustain. Institutions, instruments, and personnel intended to assist in reconstruction processes often have built-in constraints or prejudices toward one approach or the other. Various forms of insurance in the United States and other countries are available only for the restoration and rebuilding of what was lost. Prefabricated housing frequently is of a type indigenous to the donor rather than the recipient region. Foreign experts and the financial assistance associated with them may have one prejudice or the other. Foreign aid from nearby alpine regions was sympathetic to the desire of Friuli natives after 1976 to restore the archaic pattern of their alpine lifestyle. The foreign assistance supplied by the United Nations to Skopje after 1963 imposed a modern, international style, land-extensive urban plant which was completely alien to the Macedonian culture.

It is extremely important to recognize that systems are constantly in the process of social evolution. Their progress from one time period to the next is part of a social continuum. The continuum must be respected by those who are doing the planning for these systems. It is conceptually impossible ever to maintain completely the status quo. It is equally impossible ever really to be confronted with a tabula rasa. The approach that planning must take is to determine where in its evolution the system occupying the impacted region was and try to promote its progress as expeditiously as possible to the next stage of the established trend.

It is necessary to avoid the dangers, inherent in both polar approaches, to the healthy development that can result from optimal reconstruction. This is difficult because the tendency towards both approaches have roots deep in human nature. The danger of putting things back as they were is that it can lead to retrogression rather than progressive development of the system. The motivations to do so are quite understandable. First there is a very real grief for a lost environment[7] which may also be compounded by grief over lost interpersonal associations

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most spectacular destruction occurred in the new hotels along the beach outside the city walls.

[7]There are numerous historical cases of populations that have culturally yearned for their lost environments from which they have been displaced for generations. In some instances, this is perpetuated in the

connected with victims of the disaster. Grieving for a lost neighborhood and home has been found to be very important in the case of the disloc tion resulting from urban renewal. A fine example of this literature can be found in (Fried, 1963). For the late middle aged and elderly, this can be a serious matter. Artifacts lost in the disaster represent the accumulation of many years. The number of years that were necessary and the earning capacity required to accumulate the artifacts no longer remain for the individual, and the possibility of restitution is quite hopeless. That older persons are more subject to despair and other psychological trauma after disasters, as has been shown in a number of studies, is not surprising. From an institutional point of view, in many instances it is vastly simpler to replace what was lost rather than attempt to make recompence for the amount of the loss. This is the reason that in the United States and other countries many forms of insurance operate in this fashion. Fire dam to a home which does not completely destroy the structure is normally reimbursed through paying the contractor's fees for repairing the damage. Many forms of automobile insurance pay the repair bill from accidents directly rather than making the money representing the extent of the damage available to the owner to expend as he wishes. Obviously, financial institutions which have possible equity in various forms of property find their operations greatly simplified by procedures of this kind.

Levels of satisfaction are related through aspirations and expectations to previous levels of achievement. Prior to the disaster, the population had a level of contentment which it would like to recover. That it is closely associated with objects, structures, communities and environments in highly particular ways should not be surprising. However, each individual had some idea of how things could be at least somewhat better. Given the option, many individuals would appear to prefer to reassemble something approximating the previous physical situation with a number of improvements some of which had been desired but unattainable for long periods of time. It would seem advisable, therefore, to separate restitution from restoration as much as possible in the instruments that provide financial resources for relief and recovery. It would also follow that as many decisions as possible should be left in the hands of local inhabitants who were intimately familiar with the environment and the system and had probably given a great deal of thought as to how it could be improved.

For the stranger or the outside expert, exactly the reverse seems to be the case. The pre-existing spatial order and the system that occupied it appear to be replete with irrationalities. Destruction represents an ideal opportunity to eliminate these senseless irregularities and impose a higher order of rationality. This perception is compounded by an insensitivity to the pecularities of the environment and the system that could only come with long familiarity. Furthermore, external values are likely to be imposed relating to systems with which

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folklore of the people and, in other instances, results in a depressed kind of alientation. Cases have been recorded of this effect among Sephardic communities that were uprooted from Spain during the Inquisition and resettled in the Eastern Mediterranean.

the stranger is more familiar and, therefore, the inherent order and rationality of which is more apparent. After the Friuli earthquake in 1976, Swiss, German and Austrian experts who were from alpine areas were far more sympathetic to the values and desires for reconstruction held by the alpine Italians than were the Italians from the plains below the Alps within the Alto Adige region. The desire is to regularize the land platting which appears to be the disorderly accretion of generations of more or less arbitrary decisions. Next align the streets and make them orderly. Place the buildings in a uniform manner on their plots. Clean up the irregularities in style, period, design, materials used, and other elements that detract from the visual order and add a degree of chaos to the appearance. The result can be vastly more orderly, sanitary, extremely nea , and terribuly dull. In the worst case, a completely alien environment can be created, the maintenance of which is dependent upon methods, materials and even fittings which are too costly to maintain because they require the import of artifacts and even skills in an economy that cannot sustain that level of trade. This is not to say that important insights about the nature of the system and the environment cannot be introduced by external experts who are looking at the situation with fresh eyes. Impedimenta which locals had lived with for most of their lives and considered inevitable are seen easy to correct and remove. Opportunities for development and change that are entirely sympathetic to the character of the place and the people may have been overlooked. Expert outside assistance can be extremely helpful and in many instances necessary.

What is most desirable is to mount a reconstruction and recovery effort which will continue and facilitate the evolution of the region. The process of readjustment may be eased in cases where the development trend is downward from previously higher levels. In some instances, this may even require urban euthanasia in which obsolete settlements which no longer have any purpose and cannot be sustained by the system are eliminated completely. Impediments to development trends must be removed and facilities for implementing and accellerating the process installed. The fundamental key is the human resources that the population of the region represents. The quality of these human resources must be improved and their development facilitated. However, this must be done within the context of their cultural and demographic characteristics. The second key is that of increasing the productivity of these human resources. This may be accomplished by modernizing the economic productive plant of the activities existing within the region. It almost certainly will include improving access to the community and means of facilitating movement and interaction within the region. For example, after the Friuli earthquake in 1976, a severely damaged village high at the end of a valley in the Slovenian Alps had a new needle factory installed in it to improve the economic base. Perhaps more attention should have been given to improving the alignment of the road connecting the village to nearby larger urban centers to facilitate commutation of resident workers to employment opportunities elsewhere.

Some of the most important kinds of developments that can occur are those things which remove differences in the quality of life and the standard of living between communities within the region. These do a great deal to stabilize population and patterns of distribution of economic activities and maintain the viability and orderly development of the regional economic system. To accomplish these objectives, those charged with planning the reconstruction effort must fully comprehend the system and understand all its elements very deeply. Everything that is done in the reconstruction process must conform in a very profound way with the nature of the existing system in the region and the trends of its evolution. Otherwise the activities are likely to be counterproductive and lead to less than optimal results. They will be alien to the character of the region and, therefore, destructive of it.

A methodology for planning for the reconstruction of stricken regions needs to be delineated. The task that has just been described may seem entirely too complex and require too much study and research over too long a period of time to be at all useful in the exigencies of a reconstruction process. That is not the case. There is enough understanding of regional economic systems that their essential features can be comprehended fairly rapidly. Sufficient information is available for many areas to make the necessary study rather simple and, for most regions, to bring it well within the range of accomplishment.

Methodology

Regional Level[8]. The starting point is to study the population of the region. The way in which it has been changing in size and demographic characteristics over time is an important element of the regional evolution. Secondly, it is necessary to look at the changing distribution of the population within the region not only in aggregate numbers but in detailed characteristics as well. Some of the population will be dispersed and some will be concentrated in a hierarchy of urban centers. There is now enough familiarity with urban systems to anticipate what the nature of this hierarchy and size distribution of urban centers is likely to be. The hierarchy needs to be determined and the ways in which the distribution has been changing over time established. Population will be clustered in hamlets, villages, towns and cities. It will be dispersed in rural nonfarm and part-time farming residences and in farms. The changing percent of the population that is clustered and dispersed is a basic piece of information. The nature of the dispersion of the population with respect to land area needs to be investigated also. Some index of concentration should be applied for this purpose. A very satisfactory one is the Lorenz Curve and the Gini Coefficient. Time series analysis of Gini Coefficients can show graphically the changing patterns of spatial dispersion and provide a basis for projection (Avioli, 1976). The centers of concentration from hamlets to cities will be characterized by a skewed distribution. The rank size rule and the log normal distribution are the most frequently employed indices (Simon, 1957a). The parameters of these distributions change over time to accommodate the changing size of the concentrated population and various characteristics of the system which influence the hierarchical relationships.[9]

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[8]Much of the methodology that is described in the following section was developed and empirically demonstrated in (Jones and Mars, 1974).

[9]For a large scale study of the changing hierarchical patterns of cities in a developing country as related to economic, social, and political factors, see (Farid, 1978).

Trends in the evolving size and characteristics of population can be projected. The past changes in the way in which population is distributed over the surface of the region can be studied and expectations of future changes developed. Changes in the hierarchy of urban centers will also become apparent and projections for the evolving pattern made. After projections of clustered and dispersed populations are made, indices of aereal concentration such as Gini Coefficients projected, and changing parameters of hierarchical distributions estimated, it will be necessary to fit these projections to actural spatial and physical characteristics of the region, that is, the subregions and population centers. Time series analyses of population trends for subareas within the region should be made independently and reconciled with the regional projections.

The population is engaged in a set of economic activities. Changing patterns of participation in these activities need to be documented. The changing structure of economic activities as reflected in the mix of industrial sectors has to be established, projected and related to the population. The sectoral mix of a regional economy changes over time in response to a variety of factors such as favorableness of regional location, national and world markets, and so forth. A major factor that has been identified is technological change. The income elasticity of demand is different for different sectors. Demand is less elastic for the products of extractive and fabricative industries and more elastic for the activities of distributive and service sectors. As technological advance occurs through development, there is a tendency for the labor force to shift out of agriculture and manufacturing and into trades and services (Simon, 1957b) (Jones, 1979).

An important characteristic of economic activities is their scale. Changing distributions of establishment by size as measured by employment, value added, output or other measure need to be determined. This, of course, must be done for each major sector of the economy. Hierarchies of activities by sector by size will result, and patterns of evolution will appear in which some sectors will be moving toward increasing scale while others will be moving in the opposite direction. The size distribution of establishments usulaly conforms to one of a series of skewed distribtuions. Perhaps the most frequently applied distribution is the log normal (Silberman, 1967). This is the distribution that was used in the study that forms the basis of this methodology as mentioned above. The parameters of the distribution change over time to accommodate changing numbers of establishments and employees and changing scales of establishments as reflections of institutional and technological factors.[10]

The economic activities are distributed in space. There will be some relationship between the hierarchies of enterprises by sector and the hierarchies of urban centers. As these two hierarchies coincide, the differing economic and social functions of cummunities derives (Skinner, 1964-65) (Johnson, 1976). The nature of the changes in functions and the

[10]For a demonstration of these changes in one industrial sector and an exploration of the implications for regional economies, see (Mars, 1979)

implications for population distribution derive from this. Depending upon the kinds of changes that have been occurring in the size distributions and numbers of establishments in the various sectors, the relative position of communities within the urban hierarchy of the region will be determined.[11] Some communities will have been becoming increasingly favored. Long-term secular trends in the economies of various communities will be apparent and these will be of importance. Attitudes of managers and operators of enterprises in declining communities frequently are perverse leading to evolutionary patterns which are less than optimal.[12]

Community Level. With the knowledge of future sizes and spatial patterns of populations and sets of economic activities, we have the basis for reconstruction plans. Emerging general patterns of distribution throughout the region should become fairly clear, and any single community can be seen in the context of the larger regional social and economic system. Planning for reconstruction at the community level can then proceed. There are two essential elements that must be taken into consideration at the scale of the community. These are site conditions and structural characteristics. Any site that is to be used as the location for residence, economic activities or other purposes has two important characteristics for consideration of vulnerability. The first of these is topography which comprises its elevation, its relationship to other nearby sites, its slope and similar characteristics. Vulnerability to flooding, earth and rock slides and other hazards are related to topography. The second characteristic is concerned with the nature of the soil. Whether it is rock, hard-packed soil, alluvium or silt will determine its bearing capacity and the nature of its behavior under seismic conditions. Micro-zonation studies should be carried out in communities subject to seismic hazards as the basis for determining the characteristics of sites and locations. Structures house functions which require different degrees and kinds of protection. Structures have characteristics such as shape, height, bulk which cause them to behave differently in seismic conditions. They are also built of different materials and have different configurations of non-structural elements. The design of their foundations and their structural components can control the way in which different types of structures behave on different kinds of sites. This, of course, has been the focus of the immense accumulation of experience and the purpose of theoretical and laboratory studies in the field of earthquake engineering.

It is suggested that the sites existing within a community be inspected and rated in terms of their vulnerability to various kinds of natural hazards such as earthquakes. Activities housed within structures should also be analyzed in terms of their vulnerability to natural hazards including seismic events. Matching activities and sites should lead

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[11]The seminal work on the relationship between the hierarchical distribution of activities and urban centers and their locations in space is, of course (Christaller, 1966) and (Lösch, 1954).

[12]Systematic entrepreneurial failure in declining regions is described by the concept of Regional Opportunity Loss. A summary review of a number of studies of this concept is reported in (Jones and Clark, 1976).

to requirements for different kinds of structural conditions so that vulnerability reduction objectives are achieved. Using this approach a hazard rating would be developed for each site not merely for earthquakes but other kinds of natural disasters as well. A vulnerability rating would be developed for each activity. The combination of the ratings for the sites ~~with~~ those for the activities would determine what actions would need to be taken. Appropriate actions to reduce vulnerability would be required by various codes such as building, use, and occupancy codes.[13] A controlled system such as this would permit maximum flexibility in locating activities on sites, if the costs of achieving vulnerability standards were justified.

For example, suppose a specific site is in an area where the soil would be subject to liquefaction in a seismic event. It is in a zone with a high vulnerability rating, let us call it S-4. It is proposed to use this site for a certain activity involving the assembly of sensitive population, say children. Perhaps this is an elementary school, call it Activity Number 7983. Activity 7983 with a large number of other activities has been assigned a vulnerability classification, call it V-6. When it is proposed to place a V-6 activity on an S-4 site special conditions of various codes apply. These codes may include Building, Plumbing and Heating, Occupancy and other codes. The conditions may require more elaborate foundations, special structural provisions, limit height, require greater ease of eggress, forbid the use of certain materials and so forth. As a consequence, the use of that site for that activity might be extremely costly. A nearby site may be equally suitable and have an S-2 rating which would have less stringent requirements for V-6 activities. On the other hand, no more suitable site may exist because the zone is very extensive in relation to the children to be served.

Conversely, another activity such as a roofed market structure, call it Activity 6148 may be assigned a vulnerability rating of V-2. The code conditions that apply when using an S-4 site for a V-2 activity may be much less stringent. The relative costliness of the project may be slight compared with other locations. However, one would expect there would be different code provisions for V-2 activities on S-4 sites as compared with S-2 sites.

What has been described is an approach to reconstruction planning which proceeds from the macro-spatial to the micro-spatial level and provides a comprehensive methodology within which to explore interrelationships. The classical model of land use planning has not been discussed. The purposes for which this very ingenious methodology was developed, that of providing a comprehensive perspective of a community to produce a master plan to guide its growth by converting all activities into a standard medium of exchange--acres of land use, is substantially different from the purpose at hand. Adapting the classical model to the present circumstance would result in what would appear to be a very cumbersome instrument. Somewhat similar conclusions have been reached by Mader (Spangle, 1980). The alternative approach that has been suggested is felt to be superior. The customary approach to the problem of reducing community vulnerability

[13]This approach is developed and described at greater length in (Jones, 1980).

to seismic hazards is that of matching micro-zonation of topography and soil characteristics to land use controls such as zoning. It is felt that the approach suggested here which is the utilization of building and occupancy codes in relation to micro-zonation studies provides a technique that is both more precise and more flexible. The role of the reconstruction planner is identified as that of assisting in the evolution of the system that occupies the stricken region. The seismic event that precipitated the disaster is viewed merely as one of a series of similar recurrent episodes in the region's history. The reconstruction process is viewed as part of the long continuum of evolution in the course of which the regional system has been undergoing many different kinds of modification throughout time. The role that is assigned is far less commanding than most planners might like it to be. On the other hand, it is probably far more demanding than most planners have the capability of performing.

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URBAN LANDUSE PLANNING IN SEISMIC REGIONS

by

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ABSTRACT

The paper is based on summerization of lessons learned from the great hazards befallen on the city of Tangshan and surrounding area after the great 1976 july 28th earthquake and enumerates the direct earthquake hazards as the principal causes of city's destruction. Surveys of urban landuse is conducted to analyse the impacts of hazards on landuse and the differentials of seismic effects on different urban locale. The paper attempts to point out the significance of integrating the types of buildings with appropriate landuse in urban planning in seismic region and the need to adopt policies for option of landuse designed for protection from direct effects of earthquake hazards.

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I. Introduction

China is situated in the middle of two of the most active seismic belts in the world: the Circum-Pacific belt and the Trans-Asiatic belt. She is an earthquake prone country. From 180 AD to 1976 AD, 664 earthquakes of magnitude over 6 were recorded and occurences of earthquakes of high intensity have been markedly frequent in this century accounting for more than 70% of total occurences. During the 70's, earthquake over magnitude 7 occured practically every year numbering more than 70, and mostly in densely populated urban areas with burgeoning industrial development with disastrous losses.

The July 28th 1976 Tangshan earthquake of magnitude 7.8 is the worst disaster yet encountered, a painful lesson dearly paid. The epicentre is in urban area to the south of railway where an intensity of 11 was recorded. From it, seismic waves extended to a very vast area. The total area effected by seismic hazards, from intensity 11 to 7, is estimated at 42,765 sq. kms. (fig.1) Fengnan Prefecture, in Tnagshan region and metropolises of Tianjin and Beijing all suffered varying degrees of destruction from impacts. According to statistics of losses to properties in the region of Beijing, Tianjin and Tangshan, 80% of industrial establishments and 94% of dwellings collapsed to the ground and irrevocably damaged; 61% of properties in Tianjin sustained various degrees of damage of which 36% collapsed and beyond repair; in Beijing 10.7% oc city's properties were damaged of which 3% being seriously damaged. Casualties were heavy; in Tangshan there were 148,022 dead and 81,630 wounded, the most tragic loss the country yet suffered. After the shake, the entire urban infrastructure was completely disrupted. With the exception of air pasage, all inter-regional transport and communication were severed. With city's activities brought to an abrupt end, life of inhabitants after the initial shake was exceedingly dificult and to add insult to injury, relief and salvage works were greatly hindered by the aftermath of the quake. From this, it is evident that prevention of earthquake hazards especially in urban areas is of paramount importance if we are to safeguard life and properties of their inhabitants for which purpose, scientifically planned and economically feasible urban development programs for seismic regions are the most important prerequisite to safeguard the wellfare of our posterity.

Two aspects accounting for the destruction wrought upon the city by the earthquake, they are, firstly, the direct hazards brought about by transmission of seismic waves through sub-strata to act upon buildings and structures on ground surface to cause destruction; and secondly, as results of destruction of the above surface structures and the underground public utility networks, indirect hazards such as fire, flood and outbreak of epidemic follow in the wake of direct hazards. Often it happens that due to certain factors, indirect hazards can play more serious havoc than the direct ones. In the recent several earthquakes of high intensity, however, with centralized state power and the will of the people to combat natural disaster, we have been successful to carry out immediate and effective relief works and to confine losses from indirect hazards to smallest possible extent. Nevertheless, indirect hazards though deserving our vigilance, the main causes of destruction still steer from the direct hazards.

II. Correlation of earthquake hazards and geotectonic considerations

From macrocosmic point of view, direct hazards, per se, are related to seismic magnitude, focal depth and distance from the eqicenter but on the other hand as facts stand, the crux of seismic hazards lies in the geotectonics of the region which is in fact the function of hazards. The distance from epicentre actually bears little relation to the degree of destruction. For example, the Feb. 4th 1975 Haicheng earthquake of magnitude 7.3 which is in the southern part of

Liaoning province noted for the complicated geotectonic, and geophysiognomy, the isoseismal of that quake (fig. 1) clearly demonstrates correlation of intensity distribution effects and geotectonics of the land designated for specific use. The presence of three large faults, of north north-easterly, north-westerly and east-westerly direction running here-under which manifest in the topographical features of the terrain here which is clearly divided into two parts by a line drawn from Anshan, Yingkou and County Gan, one in the east and the other in the west. The eastern part is a hilly region formed of paleo-metamorphic and igneous rocks and the wester part is an extensive alluvium toward the sea which account for considerable seismic intensity differentials decrease in the opposite directions as from the dividing line; in the east, owing to the transition from fan alluvium to hills, intensity decrease is markedly rapid while in the west the decrease is slow as approaching the coast but extends to a great distance, thus, a configuration of '+' appears in the isoseismal diagram. Furthermore, the foundation soil in the area is such that in the east it consists of essentially category I & II soils while in the west, category II & III. But, in the east there exists anomalous patches of low seismic intensity in an area of high intensity whereas in the west, reverse is the case, patches of high intensity in otherwise low intensity area. For instance, in the town of 'Big Stone Bridge', an intensity 9 is recorded but on its fringe because of its proximity to the hills and of its bedrock foundation, hazards are noticeably less serious, whereas the town of Haicheng being situated on the river terrace of Haicheng river, on the fringe of fan alluvium with high ground water table and poor soil conditions suffer an anomaly of intensity 9 in an area of otherwise intensity 8; city of Yingkou is situated on the coastal plain with category III soil effected by an intensity 8, the factory buildings there, however, are subjected to hazards of intensity 9. Isoseismal configuration of Tangshan earthquake and the pattern of anomalous intensity distributions are also typical, for examples, in and around Yutian, areas of anomalous intensity 6 within and area of intensity 7; in Ninghe, anomalous intensity 9 within an area of intensity 8; In Tianjing, anomalous intensity 8 within intensity 7 and Tanggu fare the same. So, it is evident that this anomaly of high intensity within the low and vice versa is a phenomenon associated with the correlation of geotectonics and geophsiognomy of landuse. Such being the case within a seismic area so it is with the city where owing to the existance of the same factors so exists anomaly of sismic intensity in urban area. Therefore, identification of subterranean geotectonics of urban lands is first step towards effective prevention of direct seismic hazards, descriptions required as follows:

A. Tectonics of faults.

Knowledge of faults tectonics — distribution of faults and their directions — is an important aspect of description to assess possible hazards. Past seismic hazards have testified to the fact that Seismic fracture of faults and seismic induced fracture are two essentially different aspects.

Seismic fracture of fault is that small part of an active fault with latent seismicity and that is the basic determinant of seismic intensity. Accordingly, suthorities concerned have made the recommandation that seismic fault refers to the possible displacement or fracture in the next hundred years, in this fault.

Prior to Tangshan quake, a seismic active fault had already been identified in the urban area to the south of the city's Beijing and Shanhaiguan railway line where the epicenter of the 1976 earthquake is. The fault runs in north easterly direction essentially corresponding with the ground rupture occurred which cuts across Jixiang Lu (road) causing road surface to be displaced 1.25 m horizontally and 0.6 m vertically (fig.3). Either side of the ground rupture experienced most intensive shake, the recorded intensity is 11, and within an area of 8 square kilomters, scores of buildings of two and three storeys high and practically all bungalows collapsed to the ground; the Internal Combustion Engine Factory, Gear Manufacturer, Light Industry Machine Works, Rolling Stock Factory (fig. 4) anc etc. sustain irreplacable destruction, more than half of rows of single span r.c. workshops collapsed. The cause of destruction is attributed to the focal depth (12 to 16 km.) apart, fault displacement, acceleration of ground motion, arching and rupture of ground surface leading to deformation and fracture of structures to cause devastating destruction. Therefore, it may be said that areas in the neighbourhood of an active seismic fault is a danger zone for building use.

Geotectonic faults not related with seismicity are often encountered in engineering projects, their hazards manifestations are such as to show no obvious increase of destruction as testified by the experiences of the 1970 Tonghai earthquake in Yunnan province where tens of villages are situated over geotectonic faults. The same also happened in the 1975 Haicheng earthquake where it is known that a large and deep fault existed but as the rupture traversed the whole area, no discernible increase of intensity is recorded from evidences of damages to all the villages in the effected area; informations gathered from the 1976 Tangshan earthquake also substantiate the aforementioned phenomena. However, presence of faults is after all a geotectonic weakness, especially dangerous where there faults are juxtaposed either horizontally or vertically with the seismic faults spatially. Such areas deserve the closest attention. Therefore it is desirable to delineate such areas as unsuitable for development as stipulated by our codes.

B. Shallow bedrock.

As mentioned above, the county of Yutian is in the area of anomalous low intensity. This is due the shalow bedrock there under and thin surface cover, so the town is blessed with a firm foundation. According to the County Chronicle, the 1679 Pinggu earthquake — the town being situated near a ford of three rivers — records an intensity 11. The County of Wuqin being equi-distant from the epicenter as Yutian suffer great losses as stated in the chronicle 'government buildings and temples collapsed 8 out of 10. Dwellings tilted and collapsed. Level grounds suddenly arched up with black water oozing out profusedly. casualties were big with great many dead.' Counties of Liao, Changli, Leding, being much farther from the epicenter also suffer from varying losses. Yet, the Chronicle of Yutian records that 'temples and dwellings in and around the Eastern Capital collapsed in great number, only Yutian survived the ordeal without loss'. During 1976 Tangshan earthquake of magnitude 7.8, intensity at the epicenter is also 11, the County City of Yutian and its fringes suffer little losses only few ingividual old and dilapidated properties have peelings off their external walls and eaves falling down. No ground rupture and no liquefaction, the intensity only reaches 6, an anomaly in intensity 7 area. In Tangshan urban area engulfed by an intensity 10, the Metal Sawing Tools Manufacturer, the Machine Accessaries Factory, the Building Ceramics Factory being situated on slopes of shallow bedrock of Dacheng Mountain, hazards experienced only corresponding to intensity 7 to 8. In order to probe further into these phenomena, government departments concerned conducted a detailed survey of hazards there such as the four workshops of the Metal Sawing Tools Manufacturer which were of bearing piers with light-weight roofs, the state of damages as found as follows: 15% medium damage, 18% light damage, basically intact 37%. No building collapsed nor seriously damaged, generally damages confined to slight wall crackings and only few instances where bricks were crushed by compression and dislocation of bricks observed. The Mining Machinery Works in the area of same intensity but located on poor subsoil, suffered heavier losses than the other buildings in the locale. According to statistics from a survey of damages to 12 workshops in the plant, the state of destruction as follows: 36% complete collapse, serious damage 48%, light damage 16%. The two factories only a few hundreds meters apart, yet suffer great differences in damages the former undergoing an intensity equivalent of 7 to 8 and the latter, intensity 10. Cases like these are quite frequent in our chronicles suffient to show that intensity on shallow bedrock foundation (i.e. category I soil) is usually 1 degree less that on category II soil.

C. Normal Stable Foundation Soil.

That is the category II soil of consistant formation and medium strength. During Tangshan earthquake whether be in areas of intensity 7, 8 in Tianjin or areas of intensity 11 in Tangshan, no evidence of increase in hazards to the foundations presents itself. In siesimic regions, this is fairly satisfactory land for building use.

D. Sandy foundation soil liable to liquefaction.

During earthquake, the loose sandy soils, especially the saturated, on being compressed by the mass of soils above, oozed out together with fine sand through ground rupture in jets of sand and bubble of water to cause cracks in and displacement of foundation, greatly jeopardizing the bearing strength of building foundation and consequently the surface structure which is manifest both in the 1975 Haicheng and 1976 Tangshan earthquakes. Haicheng west is situated on an alluvial flat plain with thin ground cover over a strata of fine loamy sand of considerable depth and high ground water table (1 - 3m.) extensive liquefaction occurred in areas where intensity only reached 7 and suffered a great deal of damage. To the south of Tangshan seismic area lies coastal plain formed by tidal ebb and river deposit of sand to give thin covers to the ground here, in parts not more than 3 meters with ground water table 2-3 meters on average and less than one meter close to the sea. In this area is situated a synthesis workshop of the Chemical Fertilizer Plant belong to the Baigu Zhuang Farm. After the quake, wall foundation of the shop subsided as much as 70 cm, shop floor cracked as result of ground arching, some of cracks as wide as 30 cm, the synthsis tower tilted to an inclination of 51 in 1000. Within the urban area of Tianjin there are seven subterranean ancient riverbeds and 45 ancient buried bogs, during earthquake extensive liquefaction occurred in areas near these ancient sites and existing rivers, causing deformation and cracking, subsidence, fracture or tilting of goundation to the buildings which suffered serious shearing and tilting. According to statistics from survey of damages collected by streets and blocks as areal units for investigation, there are 62 blocks and 9 streets that suffered complete collapse and serious destruction; the area effected is 15% of the total urban area. For examples, the Tianjin Engineering Works, the First Machine Works, the Blanket Manufacturer all situated near the ancient reverbed of Haihe with soil of poor bearing strength and aggregation of loamy sand and sandy loamy clay; and with very high ground water table (0.8 m). During the earthquake, the sandy strata is liquefied causing bubbling of sand and water to appear over a very large area. In Tianjin Engineering Works, more than 200 places of sand and water ejection and in the casting shop alone, ejection sand is esitmated to be at 350 cu m. Ground ruptures appear in several factories, some reaching a width of 80 to 150 mm causing further destruction to the buildings. All factory foundations deformed to varying degrees to cause supporting columns to undue settlement and tilting; the relative maximum column settlement reaching 300 mm (fig. 6) resulting in the serious cracking and displacement in wall enclosue, shop floor fracture, displacement, arching or subsidence.

A noteworthy phenomenon is that under the action of strong seismicity, liquefaction, albeit causes deformation to the ground surface, consequently the extensive subsidence of foundations and serious damages to the buildings, but fewer buildings in this area collapse as compared with those on grounds less liable to liquefaction as testified by many examples in Haicheng and Tangshan earthquakes. For instance, Xuanzhuang in the county of Fengnan there is very extensive liquefaction during the quake, despite of large number of houses damaged but none of the houses collapse to the ground and quite a number of houses suffer only slight damage. On the other hand, the neighbouring 'Peddy Field' Commune fare very badly; practically all houses there collapse to the ground.

E. Soft foundation soil.

During earthquake, the ground structures cause the foundation soil to further subsisdence and undue settlements thereby the structures above subside or tilt to serious damage. Hazards thereof are also obvious. For instance, the 'Sea View Mansion' estate in Tianjin New Harbour built on coastal alluvium with poor bearing strength of 4 tons/sq.m. only. Prior to the earthquake, 3 and 4 storey housing blocks there had already settled to 20 to 40 cm and shown slight inclination; after the quake, wall foundations settled further as much as 10 to 20 cm in case of 3-storey blocks and more than 20 cm in case of 4-storey blocks, and inclination increased to 17 in 1000.

F. Foundation soils of varying bearing strength

Ancient rever beds, buried ponds and bogs, gullies, urban refills on one hand and shallow bedrocks, fan alluvium and so on, on the other hand, all belong to this category of foundation soils. Hazards manifestations during earthquake consist chiefly of generation of ground rupture and foundation sliding to cause wall cracking and undue settlements to damage buildings. For instances, in Tianjin, most of ground ruptures occur along banks of ancient river beds, such as the one kilometer long and 25 cm wide rupture in the Unity Village, Dingzigu, causes a row of buildings to slide 15 to 20 cm. The Blanket Manufacturer to the east of the ancient riverbed of Haihe, during the quake, many ground ruptures appear in the east part of the factory compound within a belt of 100 m in width and structures thereon are traversed by two principal ruptures into three sections causing wall cracks, the largest of which having a width of 500 mm, and toppling of walls and roofs falling in many places (fig. 7). Again, owing to the passage of an ancient river bed under the main building of Beijing Optical Instruments Factory, an intensity 7 brings serious damages to the factory.

G. Various unusual topographical features.

Banks of rivers and lakes, buried ponds and bogs, gullies cliffs of non-rock formation and isolated hillocks are locations susceptible to heavier hazards as evidenced by records of past earthquakes. During Tangshan quake sliding of banks occur along banks of Dou He (intensity 10) and the same also happens along banks of Hai He and Ziya He in Tianjin (intensity 8), all there ruptures running parallel with the river. Land on banks of Dou He being low lying and flat & densely built on, river Banks slide towards the midstream of the river and sliding occur to belts 50-70 m of land along the river bank due loss of stability of soil structure. 20 to 30 mm wide rupture parallel with the embankment of Hai He occur in the grounds of Tianjin Cotton Textil Mill, No. II and No. IV, wall crackes, the widest of which being 20 mm, appear on walls which the rupture traverse. Built on the fringe of a water pond and alluvium of sand, Tangshan Liuding transformer station subsides and tilts in the quake, window sills lowered to ground level and serious damages sustained. The r.c. pavilion on the isolalted hillock of the Phoenix Hill, though built upon bedrock suffer damages more serious than from hazards of hill slopes of same bedrock.

III. Pre-emption of land use for prevention of hazards

Macro phenomena indicate that under the same seismic action, hazards in and around a region may develop various manifestations, some causing comparatively no damages, some, slight damages and some serious nearly verging on complete destruction. Such seismic impact differentials in micro areas are evidently related with the topographical features on the terrain. There fore, for urban planning in seismic region, it is imperative to carry out scientific survey and analysis of all data related to seismic geology, engineering geology and past records of earthquakes so as to arrive at rational planning of land use, to synthesize design of buildings with the topography of land use. This may be of significance to the prevention of direct hazards. To this end, the chief measures for adoption may be as follows:

A. Pre-emption of stable, shallow bedrock and generally stablized foundation soil for industrial development and housing estates. This category of land of high bearing capacity on account of its compactness and consistent mass, shorter outstanding cycle, less displacement in ground motion. Seismic force transmits through the subsoil and building foundation to act on the building so, an aseismically well designed building can better stand up to seismic hazards on ground like this. But, performance of this category of land is more reliable than an aseismic building design; it may reduce the macro intensity by one to two degrees and is favourable for urban land use, especially so in case of flexible structures which with low vibration frequency can absorb certain seismic energy and with their longer vibration cycle can fall out steps with the outstanding cycle of the subsoil to avoid the generation of resonance and hence of high earthquake resistant potential. Therefore, this is an ideal land use for industrial development and housing projects in urban area. Industry is usually the economy base of a city, the correct selection of land use for industry should be the main concern of any urban planner, not only to meet the needs of the industry but also to plan for the safety from seismic hazards. Design should satisfy these two requirments with an integrated solution. For instance, to locate on firm soil with good bearing capacity over shallow bedrock of short outstanding cycle, factories with heavy loading requiring foundation of high bearing capacity and the adoption of multitruss, frame, truss supports and such like flexible structures is most favourable for seismic reistance and also economizes on building costs, Such factories may be for metallurgical, mechanical engineering, electrical, chemical industries as well as for warehouses, silos to store combustible, explosive, poisonous raw materials or finished products. As for textile industry, light industry, precision instrument industry, food processing industry, owing to their lighter loading, the type of structures adopted is usually of flexible structure with longer vibration period or of medium strength, hence pre-emption of sites with consistent soil mass and fairly good bearing capacity and shorter outstanding period such as with the normal stabilized soil, will enhance seismic resistance of the building and also facilitate construction.

Similarly, selection of land use for housing is a very significant factor in prevention of seismic hazards. Half of urban land use is for housing which constitutes a major component in urban structure directly related with protection of life and properties and costs of urban development. Hence pre-emption of shallow bedrock for development of highrise housing projects; generally stablized foundation soils for erection of multi-storey residential blocks, will be desirable.

B. Soft clayey soil, silt and urban fills and etc. liable to liquefaction are to be avoided for building use. During earthquake, the saturated fine sandy strata under the ground surface cover will yield to liquid state thereby losing all its capacity to resist shearing, thereupon, if structures are not designed to meet such an exigency, will subside in case of ground rupture, displacement, subsidence and etc. Similarly, the saturated soft clayey soil silt and loose urban fill which on being aoted upon by instant loading of the building mass above during an earthquake will produce deformation by shearing to cause the building further subsiding and tilting. Precisely due to loss of soil stability, buildings unduly settle to cause sudden increase in strain in parts of super-structure and consequently fracture of wall and connections under tension. Therefore, it can be said that the destruction of ground structures is mostly due to loss of effectiveness of the foundation soil. However, there are other problems deserving our attention: (1) though the effectiveness of foundation soil is not yet complete, but due to the excessive looseness of soil, excessively large amplitude and longer oscillation time, buildings can be damaged as result of excessive displacement. (2) The long seismic outstanding period and low frequency of the loose foundation soil can easily concur with the vibration period of the flexible structure to generate resonance to produce heavier hazards. On the other hand, it's relative performance is favourable to the low-rise rigid structures. (3) Hazards on ground with soil of good resistance yet liable to liquefaction are much less than those not liable to liquefaction. That is due to the fact that liquefaction reduces the transmission of seismic shear wave which attenuates seismic effects thus to reduce the seismic forces acting on the ground structures. On the whole, this category of land, nevertheless, is not so favourable to seismic resistance as the firm, uniform and stable soil in macro intensity, is usually higher by one degree. It is therefore not a desirable land use for development. If possible, large and medium factories and important public buildings should not be located, ordinary small factories and low-rise dwellings of small building volumes and rigid monolithic structures with shorter vibration period may be considered, if must. And if required, for certain large and medium engineering projects designated in such area, structural devices such as consolidation of the foundation may be adopted to the constricts of economy, of course.

C. The dictates of certain unusual topographical features of the land.

(1) Sudden changes in topographical features; such as long and narrow ridge, isolated hillock and etc. produce whiplash effects during earthquake and increasing oscillation to bring greater hazards to buildings than the open and flat land would. This category of land is not favourable for building use such as factories and dwellings; they may be reserved for use of parks, nurseries and so on.

(2) Grounds liable to deformation; factors of land deformation are complex, they may be geotectonic factors, such as belt of fractured fault, belt near the junction of geotectonic fault and seismic fault. They may be factors of non-uniform soil structure, soft and loose soil formation, high ground water table and so on. They may be of such manifestations as fan alluvium, urban fills, ancient river beds, buried ponds and bogs as well as banks of the existing river and swamps and so on. During earthquake, these aforementioned lands are liable to local subsidence, sliding, displacement and latent ground rupture and in seismic intensity usually one to two degrees higher that the macro intensity. These lands are not to be used for building purposes, and developed if must as for recreation, parking and such like only.

D. To keep out of land where active seismic faults exist. Ground motion in such areas can be intensive during earthquake because of presence of well developed geotectonic faults to trigger off new ground sliding to effect very extensive area thereby causing great hazards to buildings, such areas are to be delineated as dangerous zones for development.

Hence, areas in and around seismic faults with basic intensity 10 and above are on no account to be designated for development of new towns; for such areas happen to be in the existing city, effective measures must be adopted with first priority for strengthening to resist earthquake to be followed by gradual transformation of urban land use and buildings for other uses; such as open air stockpiling grounds, prefabrication sites, green belts and warehouses requiring only few employees and so on; urban areas already experience visitation of heavy seismic hazards should be evicted and rebuilt elsewhere.

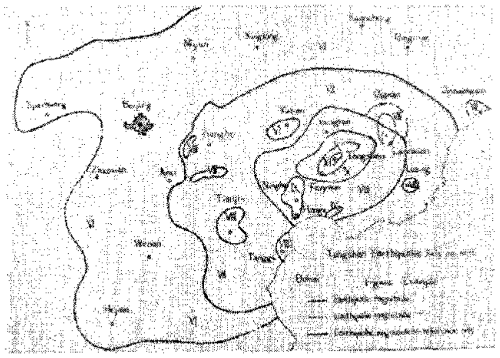
E. Beware of areas with repeated visitations of earthquake hazards in the past.

Chronicles of seismic hazards have shown patterns of cycles of seismic activities and the appatent repetition of hazards. For instances, Beijing is situated on the active seismic belt of north China, a region much frequented by seismic activities. From local chronicles, there have been, till this day, 11 destructive earthquakes in Beijing and its environs. From 1665 to 1730 Beijing experienced 3 earthquakes of intensity 8. Hazards befallen on the west and north outskirts of the city and urban areas in and around Desheng Men practically corresponded those of the 1976 Tangshan earthquake effects (intensity 7). This is due to presence of a fault running along a line joining the Lotus Pond, Exhibition Road, 6-shop Pit and the Peace Estate. And, during July 28th 1976 Tangshan earthquake, the Peace, River West and No. 2 Bridge districts in Tianjin suffer much more serious damages than elsewhere and in the same year, November 15th Ninghe earthquake of 6.9 magnitude adds insult to injury for Tianjin where hazards are once again confined to the same districts as struck by the previous quake. According to statistics, in the latter earthquake, number of houses collapsed in districts of the Peace and River West amount to 91.6% of the total houses collapsed in Tianjin.

This goes to prove that the repetitive occurences of seismic hazards can cause very serious cumulative losses, therefore, identification of such hazards-prone areas is essential for delineation of urban land use and adoption of recommandations for strengthering of structures to resist seismic hazards.

To sum up, by scientific analysis of seismic impacts on micro areas in relation to topographical conditions of land use and the dynamic characteristics of structures thereon and by prescribing rational land use by judicious planning, it may be possible to prepare an environment to better cope with earthquake hazards and to confine or reduce losses from the direct hazards, to economize cost of prevention of seismic hazards as well as to give sound guidelines for rehabilitation of old city core.

Therefore, it can be concluded that urban land use policy designed to prevent destruction from direct hazards for the safety of the city is the important raison d'etre for urban planning in seismic regions.



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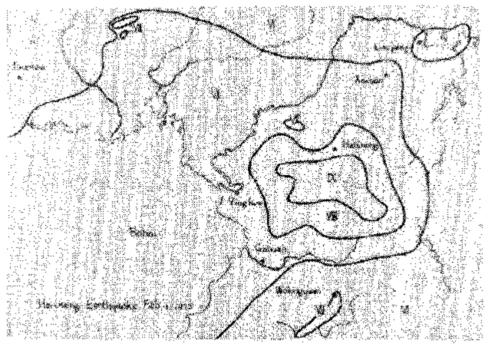

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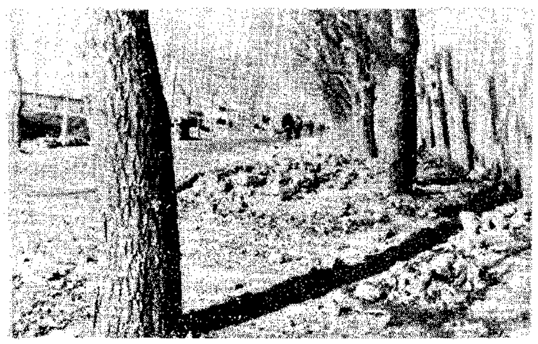

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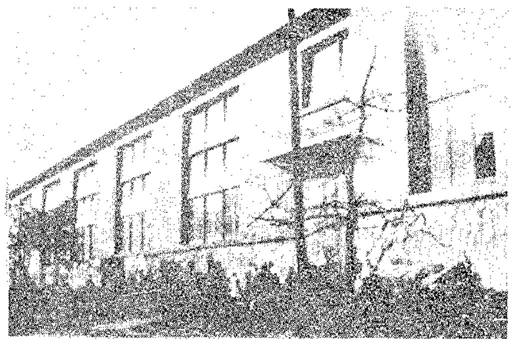

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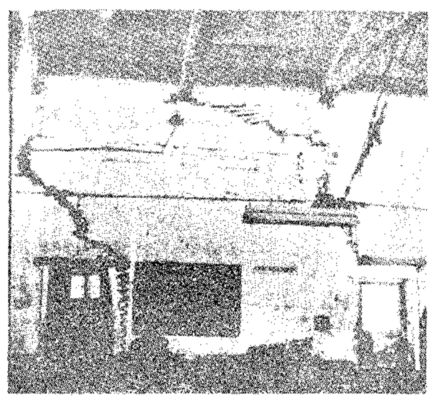

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RECONSTRUCTION PLANNING FOR TANGSHAN CITY AFTER 1976 TANGSHAN EARTHQUAKE

Hou Min-zhong*

SYNOPSIS

After Tangshan was hard hit by the violent earthquake which took place in 1976, post-earthquake reconstruction work was set about. Since then, all the experts engaged in seismic engineering in various countries throughout the world have been showing their concern over the situation in this area. In this paper, the author gives a general description of the engineering geological characteristics of the old urban district and the change of shock field. Furthermore, particular reference is made to the anti-seismic consideration through planning process. Finally, several proposals aimed at reducing possible earthquake disaster through municipal planning process are put forward.

I. ENGINEERING GEOLOGICAL CHARACTERISTICS OF THE ORIGINAL URBAN DISTRICT AND GROUND CHANGES AFTER THE EARTHQUAKE

Located in the northeast part of North China Plain, the city of Tangshan happens to be the crossing point of east-west Yanshan fold fault zone and north-east Cangdong faulted zone. The city land, bordering Yanshan Mountain in the north and Behai Sea in the south, gets lower southeastwards, with an altitude varying from 13 M to 50 M. Dacheng Hill, Jiajia Hill and Fenghuang Hill are main hills in the urban district, through which the Douhe River runs from north to south. The northern part of the city is a land of tectonic erosional topography while the southern known as Douhe alluvial plain with an alluvial, diluvial and accumulational land form. The city lands can be dided into three geomorphic units, i.e. denuded relic mountain, first grade terrace and second grade terrace. According to the ground conditions, they can also be classified as ground soils in Categories I, II, III (refer to Fig 1--Types of Ground Soils and Morphological Divisions).

Given below is a brief account of all the three types:

Category I Ground Soil(bed rock): It is of denuded relic mountain composed mainly of Ordovician limestone and Permo-carboniferous sandstone, clay rock and coal formation. It has a bearing capacity(R) of 50 t/m^2 after weathered, with a shear modulus(Go) of more than 24.3×10^3 t/m^2, being favourable area with seismic resistant performance.

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Categoty II Ground Soil: It is generally the second grade terrace of the Douhe River, consisting of alluvial, diluvial and accumulational clayey soil, distributed in the west and north part of the urban district with an average thickness of 3 - 6 M. The natural volume weight is 1.8 - 2.0 g/cm^3, with a compression coefficient of 0.006 - 0.016 cm^2/kg and a strength of 18 - 25 t/m^2, it provides good foundations for buildings.

Underlain the clayey soil is a sandy layer of silt and fine sand, which consists mainly of quartz, then of feldspar and a small quantity of dark-coloured minerals, homogeneously grained, comparatively pure and well sorted. The standard penetration resistance is normally over 30 - 50 strokes. The natural volume weight is 1.86 - 2.07 g/cm^3, with a relative density above 90%. The foundation bearing capacity(R) is 20 t/m^2 above water, 15 t/m^2 under water, the ground water level being 4 - 8 m below the surface.

Measurement shows that the velocity of transverse wave(Vs) in category I ground soil is normally between 250 - 600 m/sec, and that of longitudinal wave(Vp) is 1300 - 1500 m/sec. The shear modulus(Go) varies from 15.3 X 10^3 to 24.3 X 10^3t/m^2.

Category III Ground Soil: It generally belongs to the first grade terrace, distributed long and narrow along both banks of the Douhe River. It is composed of light mild clay, saturated silt and also lose dump soil. It is relatively low in bearing capacity which is generally on the order of 7 - 11 t/m^2. The standard penetration resistance is 9 - 26 strokes, and reckoned as a liquefacient area with 8 degrees, unsuitable for building heavy structures.

By and large, the ground soils in category II and III above the bed rock are all Quaternary alluvial or diluvial or accumulational deposits. The thickness gradually increases from the urban center hills outwards, varying from 0 m to 300 m. Abrupt changes in thickness may be seen near fault surfaces which shows that it is obviously controlled by palacotectonics (refer to Fig 2— The Geological Section A - A' and Fig 3— Change of Quaternary Isopach Before and After the Earthquake).

Due to the reason that Tangshan has expanded with the development of mining activities, most of the buildings concentrated in the vicinity of the mining area. Being unfortunately situated in a fractured zone, it is a city possessing potential hazards(Fig 4). At last, a devastating earthquake(M = 7.8) took place in 1976, leading to a catastrophe rarely seen in history.

The macroscopic epicentre lay in the Lunan District (the district lying to the south of the Beijing - Shanhaiguan Railway) with an intensity level of 11 degrees in the magistoseismic area. It developed eastnorthernly taking the form of an ellipse with the long axis of 9.5 km and the short axis 5 km, covering an area of 33.25 km^2. After the event, the whole city was reduced to rubble. There appeared a ground fissure with a length of over 8 km in the neighbourhood of the Fuxing Street, with a general strike of NNE 30° and a mono-strike of NE 40 - 50°(refer to Fig 5— The Diagram of Intensity Isorithm and Positions of Ground Fissures). The fissure assumed a right-lateral torsion in the clockwise direction, with a horizontal separation of 1.5 m and a vertical throw of 0.6 m. The Phoenix Hill lying to the northeast of the fissured zone was pushed 1.36 m away in the eastnorth direction. Here we can see the astonishing energy of the violent earthquake.

Field measurements showed that the ground elevation in the urban district also underwent great change. Under the adverse effect of the earthquake force, landslip along the bank of the Douhe River occurred in several places and the phenomenon of liquefaction of sandy soil was seen, causing the damage of bridges as well as tilt and settlement of structures.

II. GENERAL FEATURES OF THE CONSTRUCTION PLAN OF NEW TANGSHAN

1. Overall layout

In pre-liberation days, Tangshan was divided into two disticts, 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 the post-earthquake specific conditions, the development of small cities claims precedence over all others. Therefore, the construction plan of the old urban district was subject to considerable revision. According to the revised plan, the old urban district is to be restored on the basis of the original Lubei District. The original steel plants, power generating plants are to be reserved. The local and municipal party and government organizations will also be set up here because it is the political, economical and cultural center. The population is controlled at a figure of some 250,000.

The old Lunan District gives priority to the development of scenic spots and a small number of warehouses and small industries which arise no environmental problems will be set up. The subsidence pits due to mining will be transformed into lakes. Memorial hall in memory of the victims will be erected. Some representative earthquake relics are to be preserved. Reference rooms concerning with the earthquake are to be built for the scientific workers both within and abroad to carry out investigation, go sightseeing or pay visits. Main factories, enterprises and the inhabitants will move out and settle in the new district at the eastern part of Fengren County to form a satellite city with a population of about 150,000.

The eastern mining district is to be restored on the basis of the individual Kailan collieries with a population of 300,000.

In this way, the whole city is divided into three large parts which are separated from each other at a distance of some 25 km. The three parts are linked up by the Beijing-Shanhaiguan, Tungxian-Tuozitou and Tangshan-Zunhua railways as well as three highways, i.e. Tangshan-Fengnan, Tangshan-Guye and Fengnan-Guye.

Furthermore, the plants and mines (such as the Majiagou Colliery, Jinggezhuang Colliery and the Douhe Power Generating Plant) which were originally scattered around the periphery of the old urban district will eastablish their own small industrial towns. The chemical engineering plants will be moved out (For details, please refer to Fig 6-- The Schematic Diagram of The Construction Plan).

2. Functional division of the new urban district

The new construction plan has remoulded the originally unreasonable layout of functional divisions as well as the irrational situations which manifested themselves in a mix-up and criss-cross of the city with the counties, and factories with residential quarters. In the light of the practical situations of the three large parts and adhering to the principle of facilitating both production and daily life, a rational overall layout has been arranged, allowing the industrial area, residential area and stockpiling area clearly demarcated. Each one is characterized by its own distingushing features and style.

Dacheng Hill is the only scenic spot in the urban district. The Douhe River meanders from the north towards the east and then runs southwards through 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 is made of such natural conditions. The steel plants and porcelain factories are to be set up to the east of the river, while to the north of the river, automobile factory and engineering industry established. Allocated to the west of the river will be residential quarters with the Dacheng Hill serving as a natural segregation zone. At the western part of the urban district, a small industrial area is to be allocated on the border of the residential area, where unharmful industries such as electronics, food and light industries are set up. The arrangement of industries according to their nature 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 to the trunk lines, it is very easy of access. The administrative, cultural, business and sports centers are also set up here, thus forming an integral architectural complex, relatively centralized yet with organic relation. Provided in the administrative centre are Party and government organizations, mass organizations. Being arranged centrally, it is convenient for them to get in touch with each other. Installed in the cultural and business centres are general bazzars, specialized stores, cinemas, banks, post offices, hotels, bookstores and scientific center, etc. While provided in the sports centre are various sports facilities such as stadiums, gyms and swinning pools.

The living area, being close to the centre part of the urban district and to the industrial district, makes things convenient for the people and is also helpful to the reduction of the burden on traffic.

The warehouses are arranged in the vicinity of railway freight yard or on the border of the living quarters in accordance with their applications.

The new urban district will develop eastwards from the Fengren County. Behind it is the Huanxiang River and lying in front is the Tungtuo Railway. The western part of the new urban district is allocated as residential quarters while the eastern part designated as industrial and stockpiling district. A great number of large size plants such as the rolling stock plant, gear plant, light industry machinery plants and Huaxin Textile Mill are all set up here. In addition, a large park is developed by taking advantage of the bend of the Huanxiang River.

The eastern mining district will be little mining towns composed of five dispersedly distributed Kailan Collieries. The Chaokezhuang region borders on the Baiyun Hill and 3 km southwards lies the Tangjiazhang region which stands opposite to Linxi region. To the south of Tangjiazhuang region flows the Shahe River.Across the river, the Lujiatuo region and Fangezhang region face each other. The five regions, each having its independent layout, are linked up by highways forming a comprehensive setup.

3. Planning of The Residential District

There will be 30,000 - 50,000 inhabitants in each residential area which is composed of five small dwelling quarters. Each area covers an area of 70 - 100 hectres. Within which business, cultural, educational and sanitary facilities are provided. In each small dwelling quarter, there will be a residents committe, a middle school, a primary school, a club for the younger generations, a kindergarden, a nursery, a food market, a grain shop, snack bars, barbers, a savings bank and bicycle parks with an overall construction area occupying 10 -12 % of the residence area and averaging 1.2 - 1.4 m^2 per capita. In each residential area, there is a street office, a local police station, hardware stores, department stores, theatres, a children's centre, a restaurant, drugstores, a neighbourhood service centre, a post office, a clinic, a bookstore, a gas conditioning station and a heating station with a construction area of 0.5 m^2 per capita.

The dwelling houses are usually four/five-storyed houses with an area of 40 - 50 m^2 for each household. In order to make the arrangement rich and colourful, some six-storyed houses are also arranged locally. Each household in the newly-built house shares 1 - 3 living rooms and enjoys the use of private kitchen, toilet cabinet, gas fitting and heating radiator. Each household is provided with a watt-hour meter, a water meter and a gas meter, For enviromental improvement and beautifying the city, afforestation is arranged according to three levels, i.e. the municipal level, the administrative district level and the living area level with an area averaging 6 m^2 per capita. There are 26 parks, eight are of the municipal level, and eighteen are of the district level. Along the sea shores and roadsides, embellishment is well underway. The square between houses are also decorated with flowers and lawn. Thus a comprehensive embellishment system has been formed.

4. Traffic planning in the urban district

In pre-earthquake days, the streets in Tangshan were narrow and crooked, with many T - shaped road junctions. The traffic was impeded for lack of outgoing roads. To tackle this problems, measures have been taken in the new plan to widen the roads and open up new trunk lines. Apart from these, all the T - shaped toad junctions have been broken through to form a good transport and communication network. The trunk lines have been widened from original 30 m to the present 40 - 45 m, while the sub-trunk lines, from 20 m to 30 - 35 m; the 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 intersects with safety islands. 8 trunk lines, each having a length over 40 m, are built with three-slabbed pavements. Lane lines are provided to ensure safe driving. Moreover a special lane leading from the living district directly to the industrial region has been opened up for bicycle riders.

On account of the reason that the Beijing-Shanhaiguan Railway with much coal sterilized below it, runs through the city, it is therefore decided to change its route by moving it to the western part of the urban district to avoid traffic troubles. The new railway station will be set up at the westernmost part. The original Beijing-Shanhaiguan Railway will be changed over to a special industry line. At the same time, the suberb trains will be open to traffic. With the change-over of the Beijing - Shanhaiguan Railway, readjustment of the special industry lines will then be made accordingly to solve the traffic problems.

III. EARTHQUAKE RESISTANCE

CONSIDERATIONS IN PLANNING PROCESS

At the nation-wide anti-seismic meeting, Tangshan was reckoned as one of the cities possessing earthquake hazards. During reconstruction, earthquake resistant performance has been taken into full account from various aspects with regard to the selection of construction grounds, architectural design and construction.

Selection of proposed construction sites

Extensive work had been undertaken in the planning stage and in the course of its execution, with an aim to reducing the hazards in future earthquake. The work involved geological exploration, general survey of sources of water, division of areas of high seismic activity, analysis of vulnerability and topographic survey, etc. Based on the information thus obtained, taking the effects of various factories into consideration and through rational selection of construction areas, the irrational overall layout of the old Tangshan city can be changed and a new city constructed by using modern technology.

The Lunan district was located in the active fractured zone. During the July 28 earthquake, almost all the industrial and civil structures collapsed, resulting in severe damage. Furthermore, coal seams are sterilized underground. Although in some districts, the coal seams are left unmined for the time being, yet in the long run, voids will have to be formed. Hence, priority has been given to this district in the proposed new plan. This involves the moving -out of the key plants, enterprises, inhabitants as well as the setting-up of new district in the eastern part of Fengren County where favourable geological and hydrological conditions are provided.

The old urban district in new Tangshan will be developed on the basis of the old Lubei district. The construction area selected is situated to the north of the subsidence trough due to mining. This area possesses favourable geological conditions, with construction grounds mostly in categories I and II and also free of the adverse effect of mining subsidence.

The eastern mining district will be developed according to the above-mentioned layout. Compared with large cities, it has the advantages that it is favourable for earthquake resistance and relief work. It offers further merits of having a good transport services, convenience for the people to evacuate, for fire fighting and rescue activities. Such a small city is not only in keeping with the actual demands but also capable of reducing future earthquake disaster.

It has to be emphasised here 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 Tangshàn earthquake of 1976, all the structures, particularly those lifelines such as hospitals, waterworks, communication systems, power supply and fire stations ceased to be functional to perform all necessary services because of the irrational overall layout of the city and the deficiency in earthquake resistance measures. Therefore, after the event, the 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 being unable to be put under timely treatment. In a sense, we may say that the severe damage of the lifeline structures aggravated the disaster. It issues a serious warning to the countries of high seismic activity. Attention has been paid to the problems in the new 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 jam problems encountered after the earthquake, more outgoing roads are to be provided so as to keep in close contact with the neighbouring cities, i.e. Beijing, Tientsin and Chinhuangtao.

Water supply: Measures will be taken to establish a ringlike water supply system with more than one decentralized water sources and water works. Mines and enterprises with large water consumption must provide water sources for themselves. Water wells in rural area should be well preserved for urgent needs. At the urban squares or in open fields, water hydrants or underground fire hydrants are to be provided for supplying relief drinking water and for fire fighting.

Power supply: Ringlike power supply system is to be adopted. The four power stations located at Beijing, Tientsin and Tang - shan respectively are connected with a 220,000 V high - tension transmission line so as to avoid power failure in case of the destruction of any one of the four stations.

Urban communication: A wireless communication system in combination with wired system is used and the operating centers are set in different places. The cables are mostly buried underground and linked up in roundabout way so as to be able to maintain functional following the event.

Residential district should be built far from the ground exhibiting liquefaction and from the areas with fractured zones, landslides and limestone caves. Otherwise, technical measures should be taken. For example, in the course of the implementation of the new plan, a small number of residential quarters were constructed at the bend of the Douhe River with unfavourable geological condition (showing liquefaction) on account of the reasons that the task was pressing and time was running short. The difficulties in moving the earthquake proof sheds and in taking over the land also contributed to the adoption of such an expedient measure. In order to prevent the buildings from presenting cracks due to unevensettlement of the ground, measures were taken to enlarge the cross section of the foundation, reduce the load on unit area and strengthen the structural components. For the sake of keeping away from the adverse effect incurred by landslip, an area with a width of 60 - 80 m along the river was kept as shelter belt. The 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 eastnorth direction. According to the new plan, an area with a width of 100 meters was kept as afforestation zone so as to beautify the city and reduce the hazards caused by future earthquake. The Tangshan Mining Institute was located in an area with bed rock ridge and a abrupt change of the thickness of the Quaternary System(refer to Geological Section A - A'). Under the reflect effect of the ridge and the magnification effect of the stratum of the Quaternary System, the institute suffered severe damage. This area being located in the vicinity of the urban centre, it is decided to reconstruct the institute on the former site. For reducing the damage in another event, technical measures have been taken to reduce the height of the buildings and strengthen their structures. In the No. 25 residential area, a subsidece cave appeared after the earthquake, Exploration showed that the cause was attributed to the existance of underground limestone caves. Under the effect of surface water, soil erosion occured above the bed rock thus resulting in the ground subsidence. To tackle this problem, ground projecting method was used to locate the peripherally stable zone by means of the angle of internal friction of fine sand in water(26°). On the contrary, in areas with shallow bed rock and high bearing capacity, high buildings and important service facilities are built. These include the Workers Hospital, the No. 2 Hospital, the fourteen-storyed Local Guest House, the twelve-storyed No. 2 City Guest House and the sixteen-storyed Xinhua Hotel, etc. In order to facilitate evacuation and avoid injuries due to collapse of structures, ample space is left between the buildings. Based on the experience gained through the Tangshan earthquake, a three to six-story building generally collapsed to an extent of 2h/3 (h = height of the building).

During devastating earthquake, the buildings on both sides may collapse. Therefore, it is appropriate to have a thoroughfare with a width of 5 - 6 meters between two buildings for easy evacuation. The thoroughfare should be flat and straight. Open ground should also be kept and water supply facilities additionally fitted to make the open space suitable to serve as a refuge during the earthquake.

According to the national stipulations, Tangshan is specified as a city against an intensity level of 8°. Hence, in the construction design, except for the lifeline structures which have to be properly fortified (fortified against an intensity level of 8°, yet checked and accepted according to 9°), all the ordinary industrial and civil buildings are fortified against 8°. Proceeding from actual conditions, three types of construction configurations are adopted, namely, "concrete pouring internally, brick laying externally or pouring internally, cladded externally", "unreinforced masonry construction with structural columns", " frame and light-weight panel construction".

In order to check the occurance and development of secondary hazards, the chemical engineering enterprises and warehouses stockpiling inflammables, explosives and poison have been moved out of the urban district and set up in separated places. The reservoir dams at the upper reaches of the Douhe River have been strengthened in accordance with the requirements in confronting the biggest flood that may occur in a century and the demand on its seismic resistance performance. The river course has been widened and straightened for an increase of its flood discharge capacity to ensure safety of the city.

IV. SEVERAL SPECIFIC PROPOSALS

Based on the experiences gained through the reconstruction work of post-earthquake Tangshan and the lessons drawn from all previous earthquakes throughout the world, the author would like to put forward several unripe proposals concerning the problems which merit consideration in city planning process:

1. Development of satellite cities and open-up of small cities and towns are important approaches for reducing earthquake hazards through planning process.

2. In selection of suitable places for the construction of a new city, stringent rules should be set with regard to water

source conditions as well as flood discharge conditions. The city should, by no means, be constructed in an unfavourable area.

3. Construction work in subsidence area due to mining should strictly implement the relevant stipulations issued by the state. Construction of important structures in the affected area is in no case permitted.

4. In state-specified areas where precautions must be made against earthquake, the earthquake resistance performance of lifeline service facilities such as communication centre, police, ambulance and fire stations, waterworks and hospitals must be raised up to the extent that they are capable of remaining functional during the earthquake.

V. ACKNOWLEDGEMENTS

The author wishes to thank Baoming Ouyang, vice-commander and chief engineer of the Architectural Planning and Designing Headquarters, for his direction and assistance in the preparation of this paper.

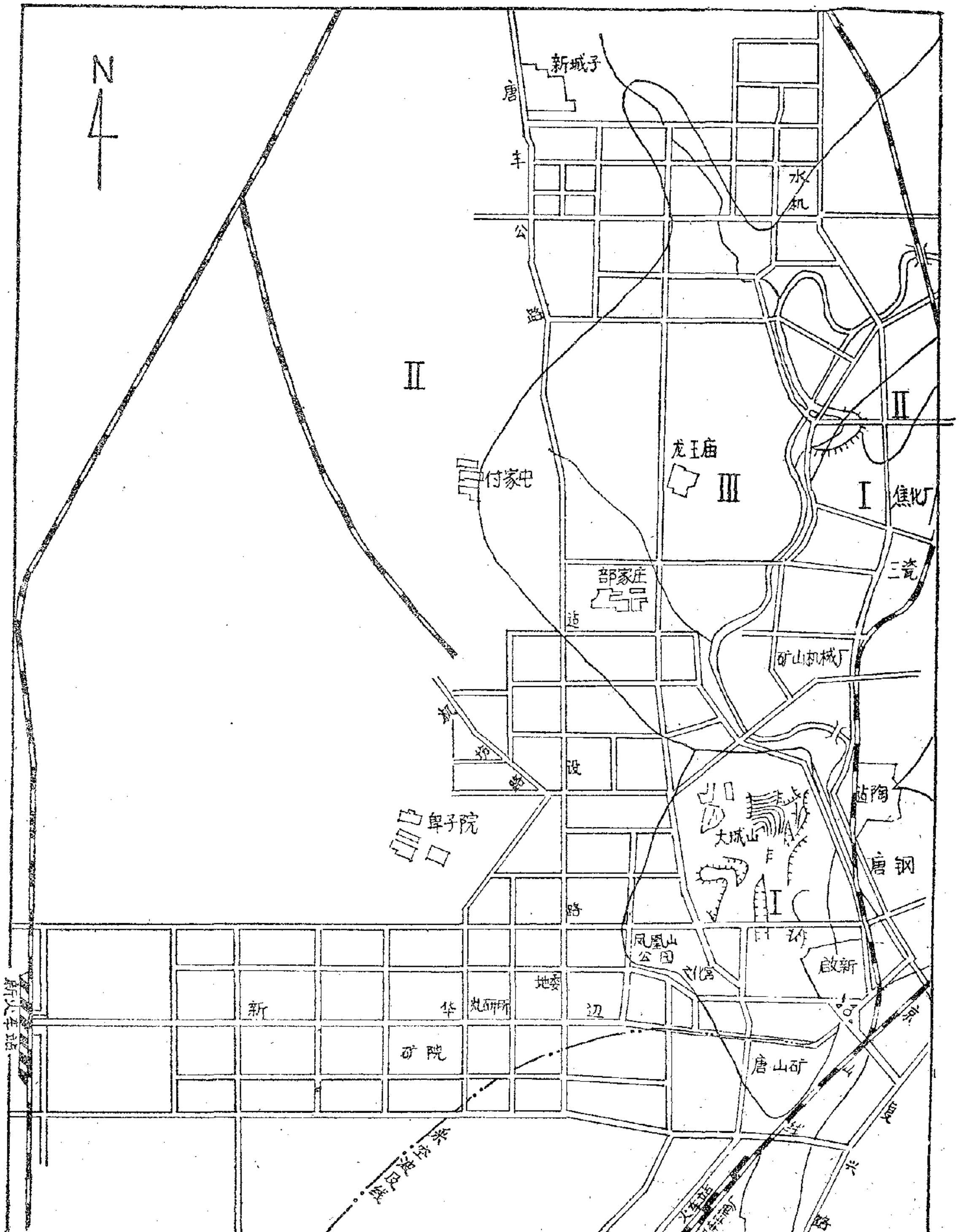

图 1　　唐山市场地岩土类型及地貌分区　　0　0.5　1(公里)

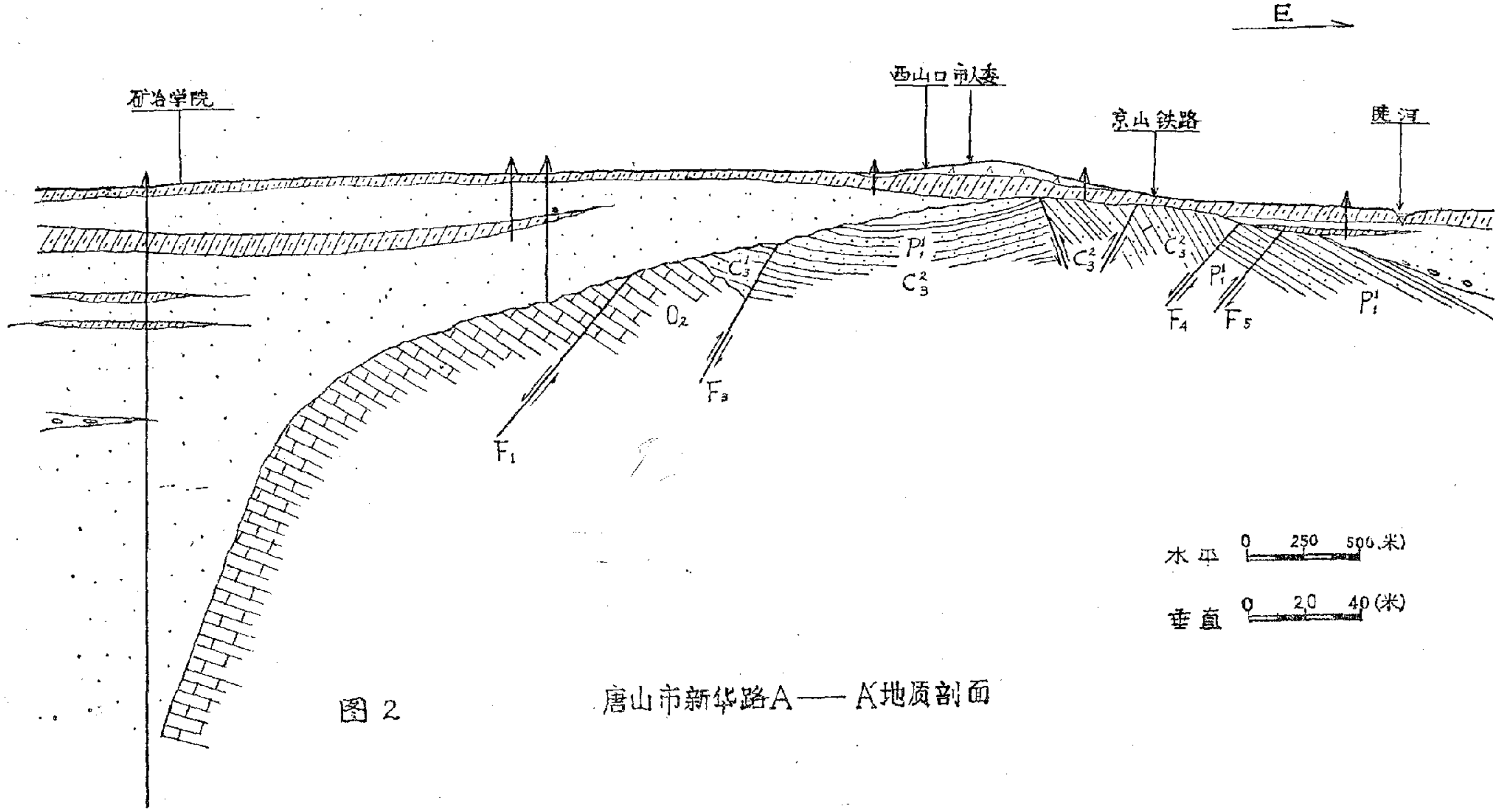

图 2　唐山市新华路A——A′地质剖面

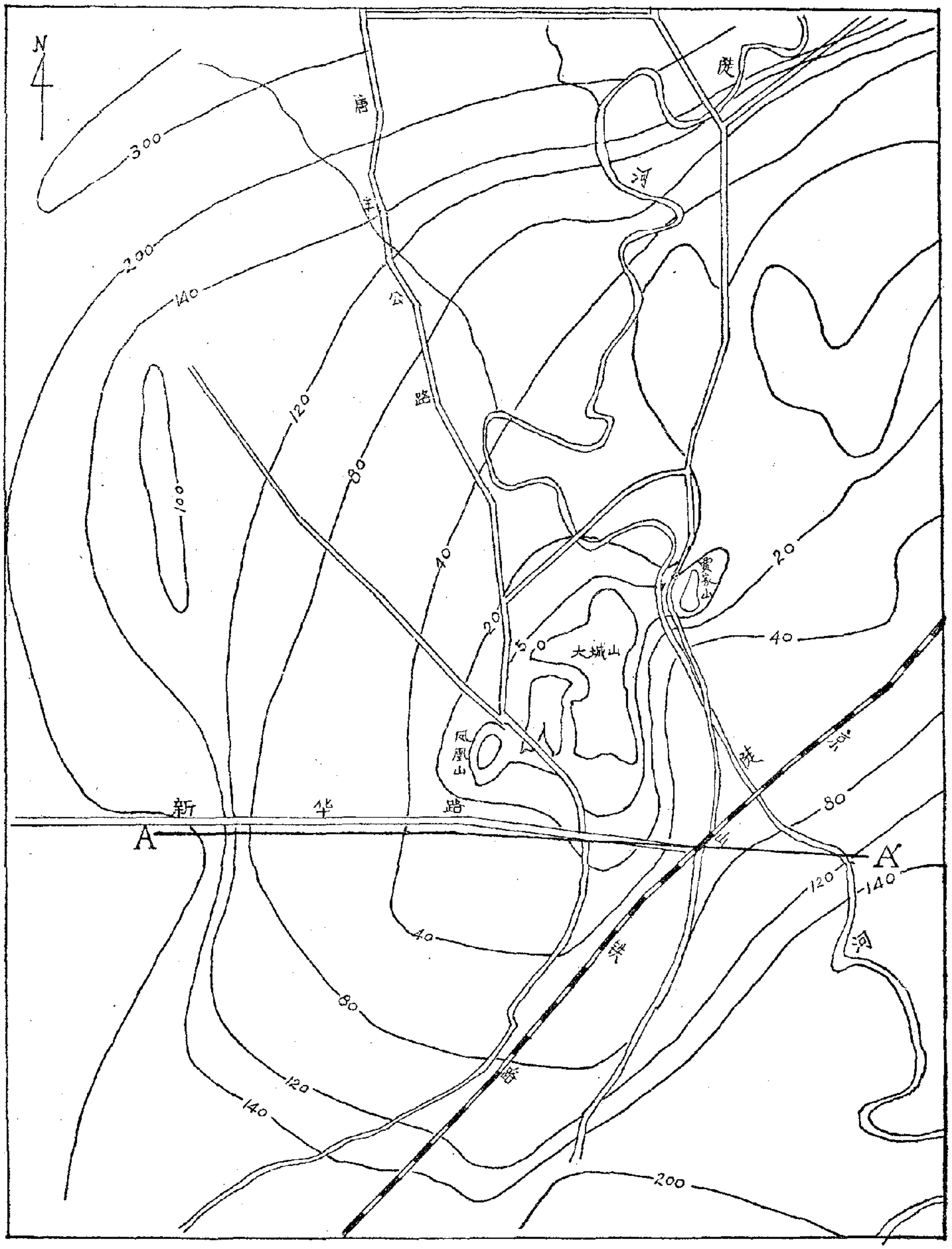

图3　唐山市第四系等厚线

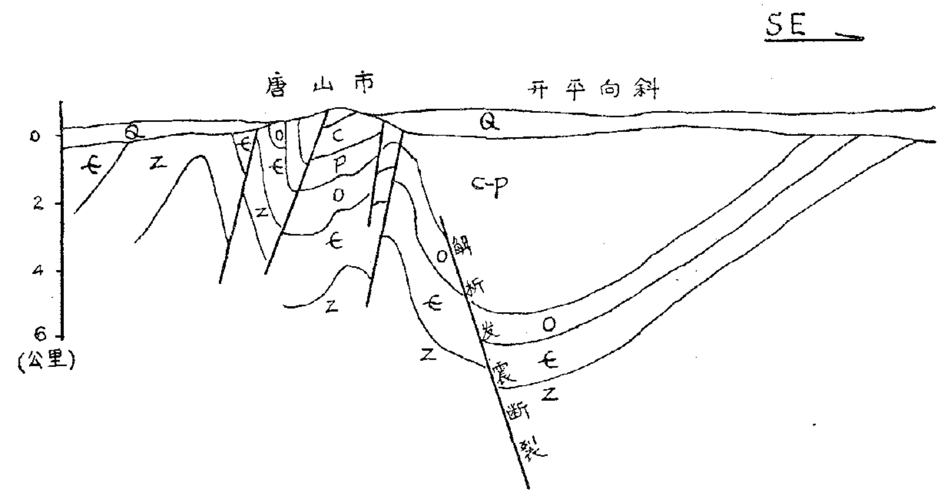

图4　　唐山市地质构造剖面

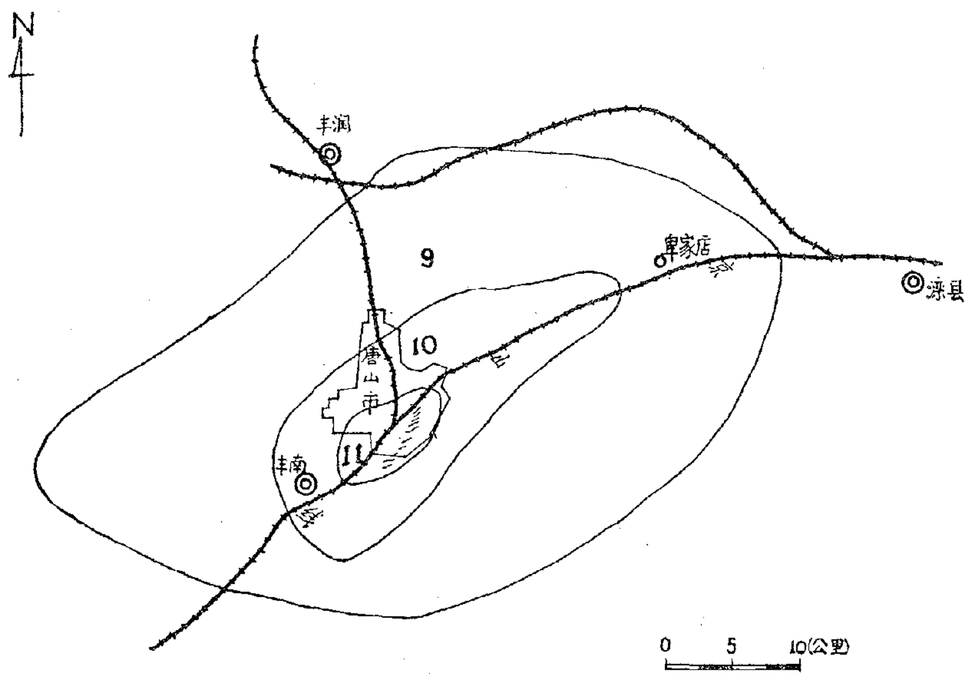

图5　　唐山市烈度等值线和地面裂缝位置

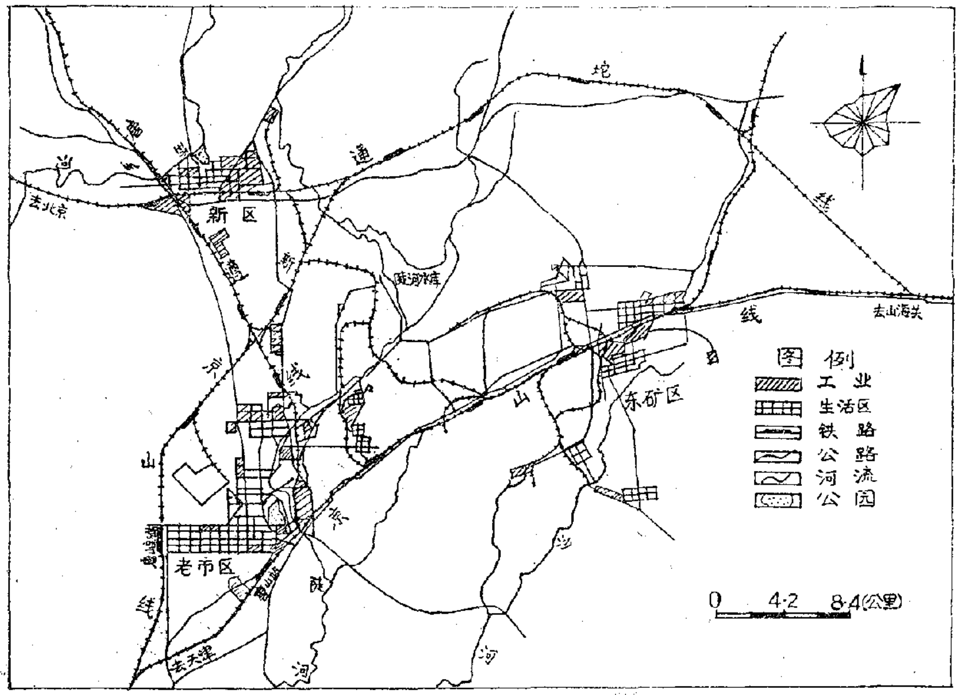

图6 唐山市总体规划图

CITY TRAFFIC PLANNING AND EMERGENCY MEASURES AFTER AN EARTHQUAKE IN SEISMIC AREAS

Xu Xunchu[I]

ABSTRACT

Through the practice of disaster relief works after Tangshan earthquake, the article illustrates the importance of the traffic for disaster relief in a city suffering from strong earthquake.

The article points out the condition of destruction of city's external traffic facilities, the questions existed of urban traffic during disaster relieving, thereby puts forward the emergency measures to be taken for the urban traffic planning and for the disaster relieving communication and transportation after earthquake. These are to ensure the maximum efficiency of disaster relieving works during the critical moment of "time is life".

China is a country in which earthquakes often occur. Through the disaster relieves in the past, it is recognized that in order to mitigate disaster from earthquake, we must have zoning of various regions according to magnitudes of earthquake, plan the city rationally, and by means of engineering measures strengthen the anti-seismic properties of structures. But in old cities, many restrictive conditions may be encountered, which make the rational planning hard to realize. At present, earthquake forecasts are not accurate enough, when earthquakes occur, heavy losses are not avoidable. For this reason, earthquake relief work must still be taken seriously, and at the critical moment of "time is life", the traffic for disaster relieving is of the first importance.

CONDITION OF DESTRUCTION OF EXTERNAL CITY TRAFFIC FACILITIES

Highway (in areas of intensity over grade VIII) may be:

1. In areas of high ground water table under roadbed and saturated silt or silty sandy loam, the soil is liquefied, and serious sand boiling and water oozing appear, uneven

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settlement, cracks of road pavement, soil slides of slopes or even slides of sections of riverside road into the river often take place;

2. Where roads were widened by stages, unequal settlement and longitudinal cracks between the new and old filled roadbeds, occur due to poor integration;

3. Where retaining walls of roadway at bridge heads extend to river beach, and the outer surface of bridge abutment has considerable free height, or the foundation is weak, the river banks are liable to slide or settle unevenly.

Bridge

Bridges are most easily destroyed, thus giving impact upon disaster relief traffic.

1. Liquefication of sandy soil of weak foundation or slide of river banks make shallowly embeded abutments displace, incline, shrink the bridge length and cause the bridges arched upwardly. In case of pipe piling foundation built in place, high piling foundation, when the dead weight of the upper structure is great, considerable displacement may occur which make the piles break and consequently the girders drop or arches break;

2. Weakness of lateral bracing of upper structure often results in considerable lateral displacement and drop of lateral beams;

3. When the width of pier cap or pier capping beam is too narrow, the pillars of the bearing incline and drop, the longitudinal beams collapse;

4. In case of long multispan bridges, because of difference in displacements of various piers, or the accumulative displacement in upper structure is too big owing to incessant aftershocks, the girders may drop.

To sum up, in areas with intensity grade X - XI, almost all bridges were severely destroyed. But at railroad grade separation, the intersecting road is in the form of tunnel, the foundations of railroad and bridge pier are comparatively stable, bridges are not damaged. If the structure of bridges is box shaped, they are still stronger. In areas with intensity grade IX, bridges are damaged to more than a moderate degree. In areas with intensity grade VIII, only a small number of bridges are damaged to more than a moderate degree, the others are slightly damaged or remain undamaged. The disaster relief traffic only requires that the beams of a bridge do not drop, so that through rush repairs it can maintain traffic.

Railroad

1. Subgrade settles and craks, roadway is bent in a sinuous form or there is an airspace under the track;

2. The running passenger trains derail, the running freight trains overturn, thus aggravate the damage of the roadway;

3. Bridgepiers and abutments shift, break, topple and fall, and as a result the beams drop; nevertheless, under the same intensity, the damages of railroad bridges are lighter than those of highway bridges;

4. The facilities of stations and depots, communication and

signal installations are severely damaged.

The repairs of railroads damaged by earthquake demand higher standard than of highway, and the time needed for repairs of railroad is also longer than that of highway.

The earthquake in Tangshan occured before dawn on July 28, 1976. After energetic emergency repairs, the telegram and telephone communications of Tangshan with Beijing, Liaoning, Hopei were restored on the same day;

On July 29, Beijing transmitted electricity to Tangshan, the airport was put into intensive disaster relief works;

On Aug. 5, all external highways and bridges had temporarily been repaired, and were opened to traffic completely;

On Aug. 10, Beijing Shanhaiguan double track railroad was opened to traffic.

Obviously, the first to play great role in disaster relief works is road traffic.

CONDITIONS OF URBAN ROAD TRAFFIC AFTER THE EARTHQUAKE

The damages suffersd by the roads in the city are generally lighter than those of highways. Heavier damages are:

Collapse of the buildings along the roads and streets caused the blockade of traffic.

In the old city districts where buildings are clustered, and population is dense, with narrow roads not sufficient during daily traffic peak hours, the conditions are serious when earthquake occurs.

The roads in Lunan District of Tangshan are only 6 - 8 m. wide except three main roads. The roads were all covered by rubbles after the earthquake, and the ruins had a thickness of about one meter, the original streets and lanes could not be recognized. Those who escaped from houses were all crushed underneath, suffered more serious than in the case when people remained in their houses and were crushed within the houses. The rate of death in Lunan District amounted to 45%.

Along either side of the wider streets in the old district, the houses collapsed and the ruins had a width of 5-6 m., moreover, fragments of lime-earth roofing and precast structural elements after the earthquake were hard to be cleared away and became hindrance to the traffic.

In new city districts, where the streets were considerably wide with wide sidewalks, green belts or with low-speed traffic lanes, the central traffic lanes would not be hindered.

Influence of artificial factors on traffic.

After intensive initial shocks, in areas with intensity grade X - XI, there appeared ruins, only a small number of houses on hill suffered lighter damage. At this moment, a large number of people died and were wounded by direct hit and by heavy structural members fall on them, only a small number of people escaped by sheer luck. The self recovering power of the entire city was very weak. The urgent need was the rescue of life, water and food sufficient to sustain life and medical treatment. The wounded and the dead rescued and dug out from the ruins were placed on arterial roads, waiting for treatment and retreat.

In the afternoon, the peasants from suburbs came into the city in a continuous stream, some of them looked for their own relatives and friends. Those who could drive helped conveying the wounded out of the city. Only a small number of arterial roads could be passed freely. The volume of traffic was very small.

The city began to be in a mess in the evening of the day when it rained in torrents. Then came the second intensive aftershock which destroyed the passable bridges and cracked houses, as a result, the people lying under the ruins were once again hit or suffocated to death.

Following the continuous aftershocks, various exaggerations of what people had seen, heard or guessed were widely spreading which aggravated people's sense of terror, and urged them to flee from the city. Owing to the timely guide of the cadres of different levels, such kind of blind action had greatly been reduced.

The stability of the city depended greatly on: the distribution by helicopters of newspaper providing reliable reports about the earthquake and the disaster relieving actions taken by the government; all remaining policemen who were on duty to maintain public order; self-consciousness of workers and staff members to stand fast at their post for dealing with emergency, thus a large number of secondary disasters had been eliminated or mitigated. Strict implement of law and discipline was also very important, it strengthened constantly the public order.

Traffic for the disaster relief

On the second and third days after the earthquake, the traffic of the city was extremely intensive. Having delayed by the destruction of bridges, the incoming vehicles for disaster relief from all directions were arriving in the city through twists, turns and round-about routes, and the traffic volume had registered an sudden increase of 200% to 300%, the traffic duration was also long, they were much higher than the normal duty of the highway and designed traffic capacity, and thus lowered the vehicle speed and impeded the traffic.

When the big streams of traffic arrived at the verge of the city, more serious traffic jams were produced, because:

1. The city had only one north entry, there had been already many cars conveying the wounded and corpses in a stream going out of the city, now this stream met with an opposite one of vehicles which was four times as big, the four lane Tangshan - Fengrun highway therefore could not bear the burden;

2. From the suburbs to the city districts, the traffic

lane width become narrower and narrower, besides, under the influence of collapsed buildings, of the wounded and dead placed along the roads, the state of affairs was still more serious at the road intersections and the traffic capacity was much smaller than the traffic volume;

3. When the traffic was impeded, the vehicles conveying disaster relief goods and materials could not go further and distributed their loads where they stopped, thus obstructed the passage of the vehicles behind them;

4. Lack of traffic policemen who could dredge the traffic in time.

Whenever traffic jams appeared, the vehicles in the rear went ahead regardless of the consequences, arranging side by side in the same direction in 4 - 5 lines, the length of autocade might be more than ten kilometers, as a result, it was difficult to advance and dredging was extremely difficult. For instance, the traffic jam of Wenhua Road lasted more than ten hours.

In view of such serious state of affairs, on the third day after the earthquake. "The Central Traffic Command Headquaters" was established, which consisted of subordinate units of the Ministry of Public Security, the Ministry of Communications and the provinces and cities concerned. Under the Headquaters were established Beijing, Tianjin and Hopei Branches and hundreds of Traffic Control Points. A large number of control personnel, dispatched by other provinces and cities, carried out a unified control over the road traffic of Tangshan and the road traffic concerning the disaster relief for Tangshan. They undertook:

1. The traffic command and control;

2. The maintenance of traffic order and disposition of traffic accidents;

3. The coordination of works concerning the collection, distribution and transshipment of the wounded, the dead and the relief goods and materials.

This timely measure, together with the day by day increased number of repaired highway bridges and lines made the efficiency of communications and transportations heighten incessantly; the public order was fine. In order to meet the increasing requirement of traffic volume, the temporary bridges for multiple lanes were erected for organizing oneway traffic in either direction to raise greatly the traffic capacity of the roads and bridges.

<u>The influence of road-side temporary buildings on the rescue traffic</u>

With the continuously arrival of relief goods and materials to the disaster area, batches of temporary buildings, located mainly on the sidewalks, in the nearby ruins or on the smooth space sites were erected in the heavily damaged city. People gathered on the arerial roads and in places where convenient contacts with society and convenient traffic were available. As a result, the effective width of roads was further narrowed. The situations in those areas which previously had narrow roads become more serious.

In cities which were affected by strong earthquakes, even in areas with intensity grade VII - VIII, building were lightly damaged. In incessant aftershocks, people would all dwell in temporary sheds, it is extremely difficult to establish another " urban residential district " in a short time on the spare sites of the city.

The density of population in Heping District of Tianjin was 47,700 persons per square kilometer and the density of construction in Hongqiao District of Tianjin was as high as 60 - 70%. The average width of the roads in the city was only 9.1 m., therefore, once earthquake occurs, only 200,000 persons can be moved to the public open space or sport grounds in schools and the others have to squeeze themself into the narrow sidewalks. Staying near the damaged buildings they were unsafe and also blocked the traffic. Furthermore, some narrow roads and lanes had been blocken up by collapsed buildings and so that the urban traffic was at a standstill for a time. This not only seriously affected the supply of daily necessities, the removal of refuse and excrements, but also the rush-repair and rescue works in urban areas.

The disaster in Beijing was lighter than those in Tianjin, the area of public open spaces and roads was roomier than in Tianjin. The erection of sheds was all organized and the ruins and rubbishes (amounted to more than 1,000,000 T) were removed in time, so there were no traffic jams. In Dongchen District of Beijing, temporary water supply facilities and public lavotories had been constructed under the sidewalks of all main streets to meet the needs of gathering masses taking part in parades served the purpose as well in earthquake. They provided convenient conditions for erecting temporary sheds and improved greatly sanitary condition of the city.

THE ROAD PLANNING OF A CITY MUST MEET THE TRAFFIC NEEDS AFTER THE EARTHQUAKE

1. A city must have several exits ensuring traffic thoroughfare. The earthquake intensities are different for different parts of the city and the rescue vehicles always come from the parts of low-intensity to those of high-intensity, as the number of passable roads and bridges is getting smaller, the traffic volume is also concentrated and often exceeds the traffic capacity of the roads. Therefore, between the exurban towns and between urban radial roads of a city, outer ringroads have to be set up to link up the exits of the city, so as to disperse traffic, avoid traffic jams, rescue simultaneously all disaster areas as quick as possible. The radial arteries connected with city's exits must be as wide as 30 - 40 m. and preferably be islands separated roads to ensure the thoroughfare of emergency vehicles.

2. Arterialroad system must be kept unimpeded both before and after the shock and its density ought to be over 2 km/km^2. The cross section of arteries should be in the form of " three separated pavements ". Roads and traffic lanes should be sufficiently wide, particularly green belts of roads should be wide enough and be integrated with green spaces in residential areas and parks in the city. The area of public green spaces should be preserved and used as asylum after an earthquake.

3. Arteries connected with such facilities like passenger stations, freight terminals, wharfs, airports etc. should be kept clear, with preferably more than two thoroughfares. Squares in front of these facilities should be spacious to facilitate concentration and dispersion of passenger and freight flows after earthquake.

4. Width of secondary roads and lanes should be greater than the sum of heights of buildings along both sides.

5. Bridges and grade separations in urban areas should be designed for one grade of intensity higher than the basic intensity. Grade separation had better been tunnel typed. Had bridges of grade separation fallen in, the loops would ensure for vehicles to pass by detour· near grade separation for crossing railway, grade crossing should be set up for emergency use. Similarly, ferry positions should be considered for river courses in city.

PLANNING OF EMERGENCY MEASURES FOR DISASTER RELIEF TRAFFIC AFTER EARTHQUAKE

1. A mobile route scouting unit and powerful traffic controlling brigade should be organized. According to the conditions and transport capacity of our country, the optimum traffic pattern with which largest number of people can be moved in shortest period of time for rescue work is to make use of roads. In order to save time for rescue personnels in scouting their path, it is most effective to utilize helicopters to reconnoitre various roads leading to seismic centre and their traffic capacity, then routes for rescue forces can be rapidly determined. It is also very important that scouting units equipped with two-way radio are dispatched, so that the following can be done as soon as possible:

to organize traffic departments concerned along routes to erect eye-catching signboards and traffic control posts;

to work out rush-repair plan and measures for damaged bridges and others.

After occurence of earthquake a vigorous brigade for traffic control should be organized by the central traffic control department concerned (including local and external personnels) to control and disperse traffic in earthquake stricken areas so as to bring efficiency of rescue units into full play.

2. Highway and municipal administration should draw out in advance plans for reinforcement and rush-repairs of important bridges and crossings. To prevent falling of bridge beams is of great important. Practice has proved that, on the basis of earthquake forecasts and analysis of disaster condition, it will be very useful

to classify the character of roads, condition of bridges, and degree of difficulty for repairing them into primary and secondary ones,

to determine the most important routes which must be guaranteed for thoroughfare and the secondary ones,

to analyse one by one bridges, tunnels and mountain passes and draw out rush-repair plan for them.

The advantages are as follow:

a) rescue units at different levels have a centralized leadship and the repair range of every road section, the rescue orientation, the responsibility of various professionals are clear and definite and they maintain close ties with local masses;

b) several project rescue plans have been prepared to deal with different conditions of earthquake and countermeasures may be taken immediately;

c) sites for reserving materials for rush-repairs are clear and definite, they may be transported without delay.

3. A location plan of points for materials transportation and distribution must be made before earthquake in order to avoid case in which less dispatched goods and materials are received where impedance of road traffic is greater. Therefore, a number of sites with sufficient area rationally scattered must be reserved in urban areas, they can be used as sports grounds for schools, bus terminal parking lots or open spaces of parks in normal times, and convenient road connection between them should be set up. Enterprises for food storing and processing and pharmaceutical storehouses ought to be located by arteries to ensure prompt transportation.

After the earthquake in Tangshan, it was advantageous to the prompt rescue and transfer of wounded that the airport was chosen as the command center. But the road leading to the airport was too narrow, it had only two-lanes of dead end type and had therefore been very seriously blocked up. Henceforth, more than two roads or exits should be put up at the very least in order to arrange up-down traffics and speed up vehicles manoeuvring in area like this.

The great quantity of rescue goods and materials arriving at the city after railroad had reopened was up to more than ten times that of the freight transported by vehicles, it was very important to transfer the goods and materials and vacate the freight yards in time promptly. Fortunately the railroad was reopened ten days after the shock so the city had enough time and was capable to prepare sites for freight yards, to organize the labour power for loading and unloading, to dispatch and distribute vehicles and materials. Eight railroad yards for loading and unloading had been set up, later increased to more than ten, ensuring the supply of large amount of building materials and articles for daily use.

4. The transport facilities for passengers and freights should meet the needs after earthquake. Parking and maintenance areas for vehicles should be located in the open-air, so that vehicles are in good condition after earthquake, and can set out quickly to rescue the wounded and transfer materials. Extensive sites may be used for rescue command center and buses as medical stations or wards.

Large-, middle- and small-sized vehicles should be used jointly, especially small-sized motorized or manpowered tricycles having a capacity below 500 kg can pass through narrow lanes or roads of bad traffic condition in early days after earthquake and play a remarkable role in transportation. For instant, the transport capacity of small-sized vehicles was about one-third of the total after the earthquake in Tianjin.

The characteristics of transportation after earthquake (as compared with normal) are:

a) Roads are in bad condition, speed of vehicles is generally retarded.

b) One way traffic is the chief form of transportation, so utilization ratio of route is low. (it is over 70% before earthquake, after earthquake it decreases to about 50%).

c) Though dispatch ratio of vehicles is high, both transport volume and amount of freight transport cycles greatly decrease, but because there is a decrease of freight volume in whole city (for factories produce less or stop their production) transport task can still be completed.

PLANNING FOR EARTHQUAKE-PRONE REGIONS

Lidia L. Selkregg[1]

Abstract

As proved by the results of the planning process that took place in Alaska after the March 27, 1964 earthquake, decisions were made too soon and with insufficient data, and many recommendations still have not been implemented. The Alaska experience points out that planning for earthquake-prone regions must be an ongoing process rather than a sporadic response to a disaster or to temporary concerns. A "risk component" must become an integral part of the comprehensive planning process rather than being treated as a separate program. To be effective the planning process must include examination of constraints to and opportunities for the implementation of the recommendations made and an estimate of costs and benefits of alternatives. This phase of hazard planning needs more attention and direction.

Methodologies for evaluating the seismic risk of urban areas are available and are being refined. The data obtained from these studies should be coupled with economic and social data to prepare long-range development plans. The plan should include an evaluation of present conditions, including land use and condition of structures, and graphically reflect the effects that the proposed plan would have on the present urban setting. Greater emphasis must be placed on community participation and education of policy makers to foster understanding of the magnitude of the topics and the responsibility of providing for public safety.

The process points to the need for developing an interagency, interdisciplinary approach to data retrieval and planning, with a lead agency selected to coordinate the planning process. Overspecializations and development of administrative systems responding to separate specialized fields will continue to interfere with the success of any planning process and of hazard planning in particular. The institutional changes necessary to improve planning implementation need more analysis.

Introduction

Southcentral Alaska is part of a vast, continuous, seismically active belt that circumscribes the Pacific Ocean basin (Figure 1). This region is one of North America's most seismically active, experiencing thousands of earth shocks each year. On March 27, 1964, one of the greatest earthquakes in history struck southcentral Alaska. With a magnitude recorded at between 8.4 and 8.6 on the Richter Scale and a duration of approximately five minutes, measurable vertical and horizontal dislocations of land surface were greater than for any previous earthquake (Figure 2).

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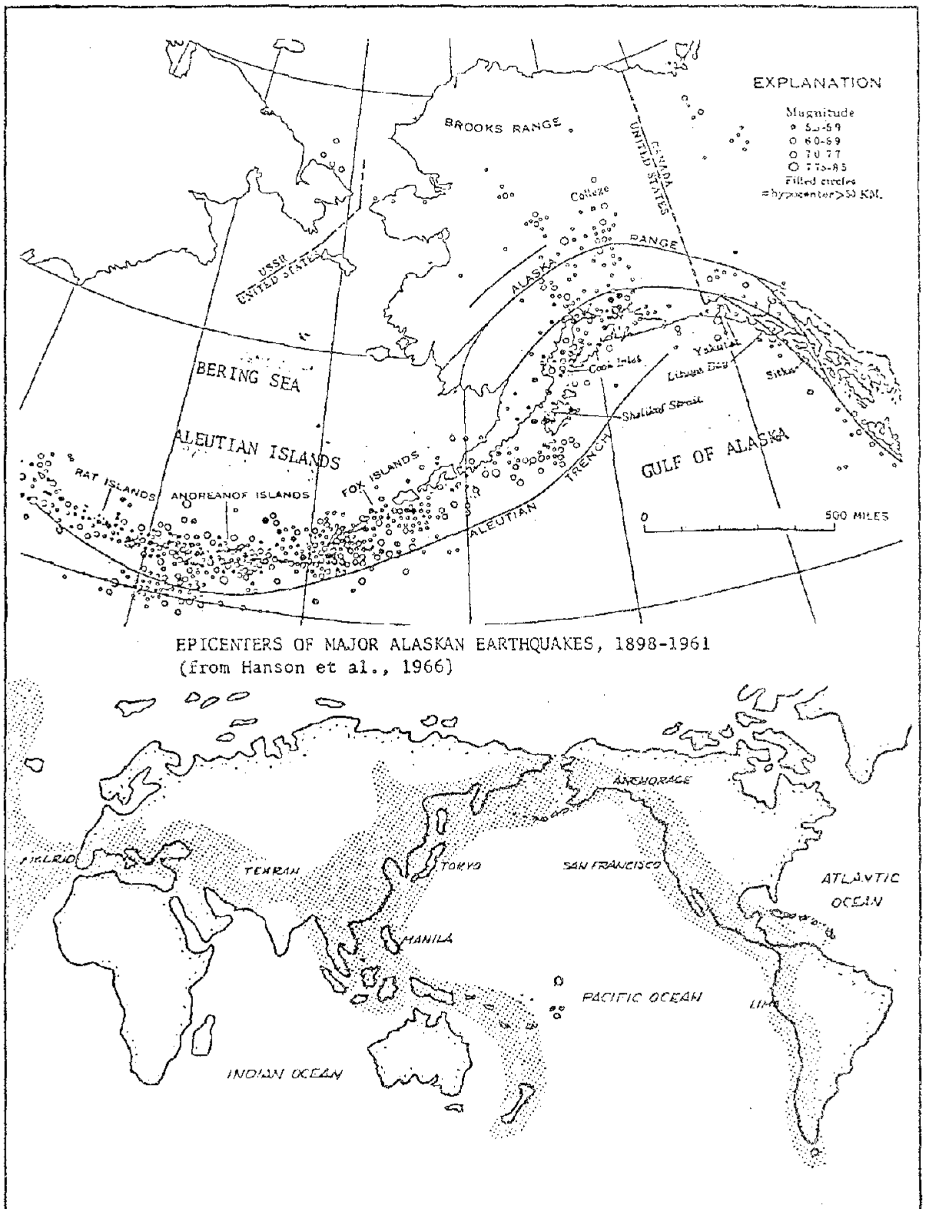


Figure 1. Earthquake Belts of the World.

Figure 1. Earthquake Belts of the World.
These belts coincide with the Earth's orogenic zones and contain most of the Earth's active volcanoes.

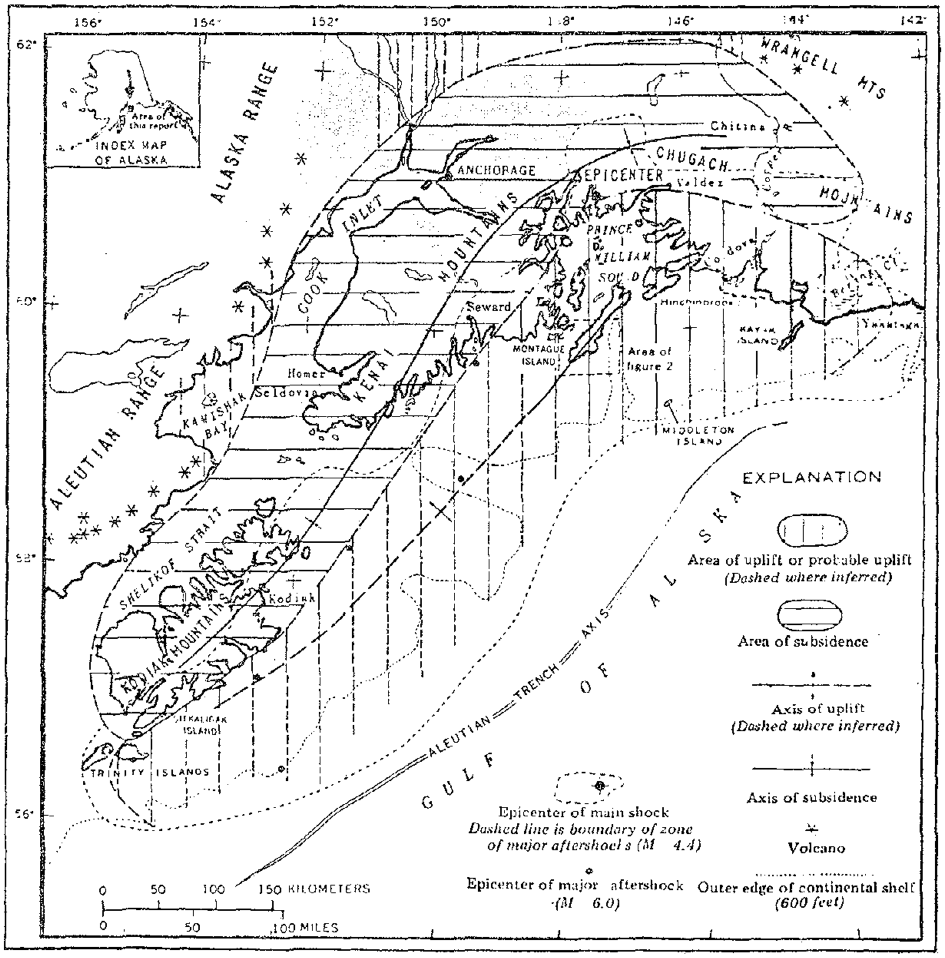


Figure 2.

Setting of Montague Island with Respect to Regional Tectonic Deformation and Seismicity that Accompanied the March 27, 1964 Earthquake.

U.S. Geological Survey Professional Paper 543-G, "Surface Faults on Montague Island Associated with the 1964 Alaska Earthquake".

Few of its effects on earth processes and on the man-made environment were new to geologists and engineers; however, never before had so many effects been available for study. Strong ground motion had induced many snowslides, rockfalls, and subaerial and submarine landslides. The submarine landslides had created local sea waves, that combined with a major tsunami generated by crustal deformation, destroyed ports and facilities in several coastal communites, covered sessile organisms and salmon-spawning beds with silt, and leveled forests, affecting the economy of extensive areas. Tectonic elevation and depression caused extensive damage to biota of coastal areas. Seismic vibration, vertical displacement and water waves affected all coastal communities in southcentral Alaska (Figure 3).

The earthquake had crippled Alaska's economic base. Public and private property loss was more than $300 million (1964 value). Hundreds of homes were uninhabitable. In Anchorage, the most sophisticated community in the state, three large landslides were triggered in the business district and two of the most valuable residential areas. In Seward and Valdez large submarine landslides and land subsidence destroyed the industrial waterfront. In Valdez the instability of the soil throughout the community made the whole community uninhabitable and in need of relocation. In Kodiak land subsidence coupled with the effects of inundation resulting from a tsunami, destroyed industrial and commercial sectors of the community. In Seldovia, land subsidence left industrial waterfront, commercial facilities, and many residences under water at high tide. In Cordova land uplift left the small-boat harbor, the city dock, and other waterfront facilities high and dry. Ports, highways, and the railroad facilities were damaged or destroyed.

The death rate was relatively low in relation to the magnitude of the event, due to the sparse population of southcentral Alaska at that time and to the fact that the earthquake occurred on a holiday--Good Friday--at a time when many public buildings were vacant. Only 115 lives were lost; however, if schools and public buildings had been occupied, the tragedy would have assumed greater dimensions.

> Had we planned this earthquake, we could not have chosen a better time. In the late afternoon of Good Friday, many office buildings were closed and many persons were driving home in their automobiles, a relatively safe place to be. Everyone was awake and most persons were clothed. Even more important, they had their shoes on, usually an important point in Alaska survival. (Wilson 1964)

Planning Considerations

The mechanisms of the Alaska earthquake, its effects on the environment and people, and the participation of governments in the recovery have been recorded in depth in The Great Alaska Earthquake of 1964, published by the National Academy of Science in eight volumes--Geology, Seismology and Geodesy, Hydrology, Biology, Oceanography and Coastal Engineering, Engineering, Human Ecology, Summary and Recommendations.

The recommendations made in these studies have set guidelines for nationwide earthquake hazard research and establishment of policies directed to hazard mitigation. But, what about the awareness of these

			Principal Causes of Damage								Types of Structures Damaged							
Place	Population	No. of Deaths	Subsidence	Uplift	Landslides	Submarine landslides	Ground cracks	Vibrations	Waves	Fire	Homes	Businesses and Public	Military	Harbor	Water Supply	Other Utilities	Highways	Airports
Southcentral Alaska	107,916	115																
Afognak	190	0	•						•		•	•		•	•			
Anchorage and military bases	80,726	9			•		•	•			•	•		•	•	•	•	•
Cape St. Elias	4	1			•				•									
Chenega	80	23		•					•		•							
Chugiak	51	0						•							•			
Cordova	1,128	0		•			•		•		•	•		•	•	•		
Cordova FAA airport	40	0					•	•			•	•			•	•	•	•
Eagle River	130	0						•							•			
Ellemar	1	0		•								•						
Girdwood	63	0	•				•	•			•	•						
Homer	1,247	0	•			•		•				•		•			•	
Hope	44	0	•								•				•			
Kaguyak	36	3							•		•	•						
Kodiak and military bases	4,788	15	•						•		•	•	•	•	•	•	•	•
Kodiak Fisheries	2	0						•	•					•	•	•		
McCord	8	0						•			•	•						
Old Harbor	193	0							•		•	•		•	•			
Ouzinkie	2'4	0	•						•		•	•		•				
Point Nowell	1	1							•		•							
Point Whitshed	–	1		•					•		•							
Portage	71	0	•				•				•	•					•	
Port Ashton	–	1							•									
Port Nellie Juan	3	3							•				•	•				
Seldovia	460	0									•	•		•				•
Seward	1,891	13	•			•	•	•	•	•	•	•	•	•	•	•	•	•
Tatitlek	–	–		•							•			•				
Valdez	1,000	31	•			•	•	•	•	•	•	•		•	•	•	•	•
Whittier	70	13	•			•		•	•	•	•	•	•	•	•	•	•	•

Figure 3. Coastal Communities in Southcentral Alaska Affected by the March 1964 Earthquake.

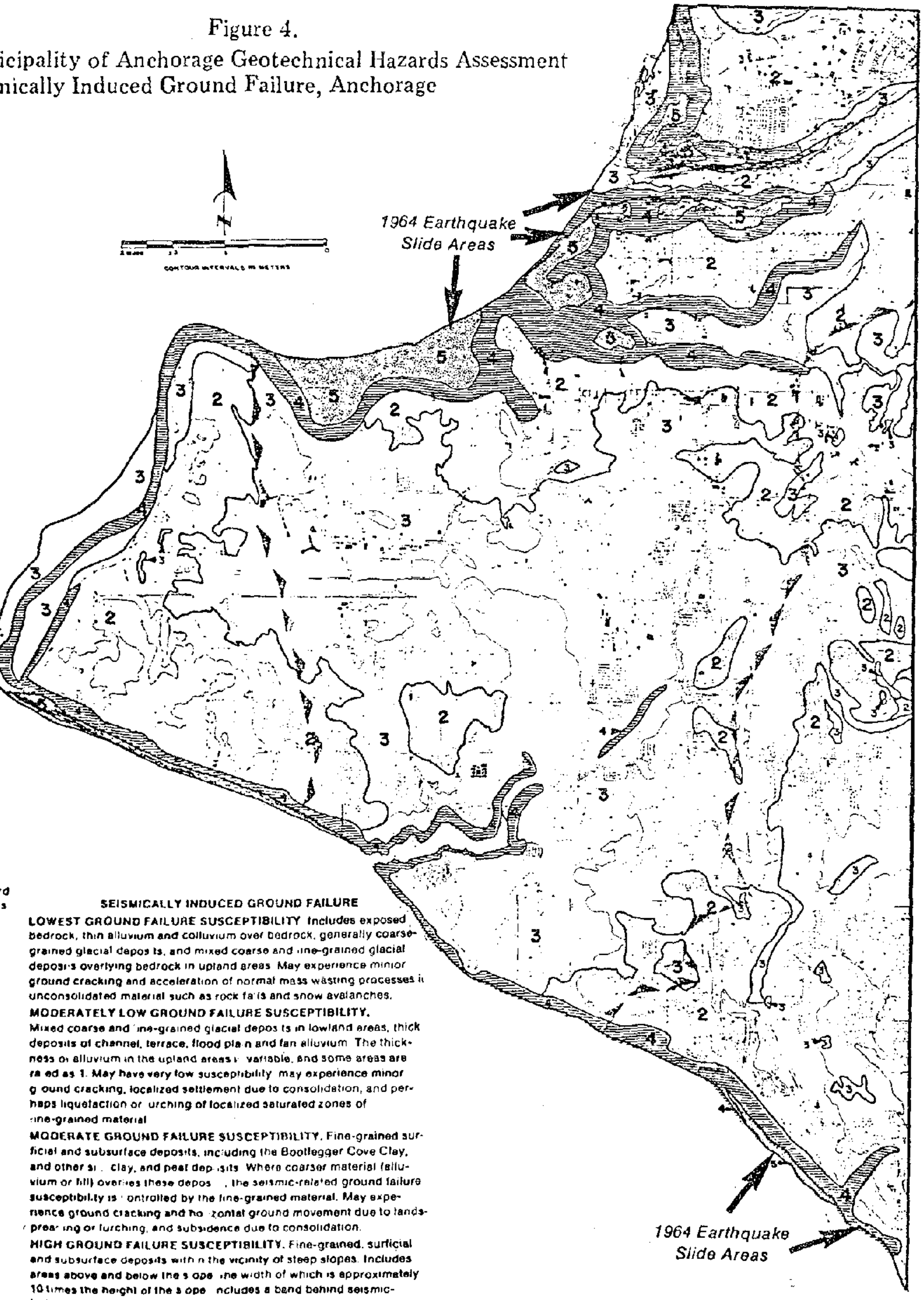
studies and recommendations in Alaska where the earthquake occurred? It is the opinion of many scientists and planners that if a major earthquake were to occur today, the state and its major communities would be at a level of readiness similar to that of the March 27, 1964 earthquake. In fact, as a result of increased population and development that has occurred in upper Cook Inlet in the last 10 years, some believe that another earthquake would have an even greater impact on commerce and people.

Why? There is no doubt that the overall technical competence needed to assure safe development in earthquake-prone regions has been refined since 1964. The state legislature passed the Alaska Disaster Act, which requires that the Division of Emergency Services study areas subject to shifting, subsidence, flood, or other catastrophic occurrences and recommends appropriate changes in zoning regulations, other land use regulations, or building requirements for areas susceptible to a disaster. Also, the legislature passed the Alaska Coastal Management Act, which contains a section on geophysical hazards (GAAC 80.050) that directs districts (local governments) and state agencies to identify known geophysical hazard areas and areas of high development potential in which there is a substantial possibility that geophysical hazards may occur. Development in these areas cannot be approved by the appropriate state or local authority until siting, design, and construction measures for minimizing property damage and protecting against loss of life have been provided.

In 1979, to comply with the state requirements, the Municipality of Anchorage contracted with the firm of Harding-Lawson & Associates to conduct a geotechnical hazards assessment study of the Anchorage area. In addition to mapping seismic risk areas (Figure 4), the study identified and delineated other hazards--wind, coastal erosion, snow and rock avalanche areas, permafrost zones, and areas subject to glaciation. The purpose of the study was to meet the requirements of the Coastal Management Act by identifying and delineating potentially hazardous areas and to identify areas that needed additional study and evaluation for the development of specific land use regulation and risk mitigation measures. To date, the Anchorage Municipal Assembly (the local governing body) has failed to adopt the study and maps; these were accepted only as professional reports. Thus, they carry no official weight, and no specific mitigation or implementation measures have resulted from these efforts.

Studies requested by local governments immediately after the earthquake and conducted by the U.S. Geological Survey for various communities (Dobrovolney 1971; Schmoll and Dobrovolney 1971, 1972a, 1972b, 1974a, 1974b; Lemke 1967; Selkregg 1972; Miller 1972) have been ignored in the preparation of local comprehensive plans. Why? My experience as a geologist, planner, and an elected member of the Anchorge Municipal Assembly points to the fact that to ensure implementation of geophysical hazard studies, the information obtained from such studies must be made an integral part of the total planning process. Because environmental hazards affect all components of a comprehensive plan, knowledge of seismic-induced risks must be evaluated in developing guidelines and standards for allocation of land use, development of transportation and utilities, and location of public facilities, along with the evaluation of social and economic projections for community growth.

Figure 4.
Municipality of Anchorage Geotechnical Hazards Assessment
Seismically Induced Ground Failure, Anchorage



Hazard Zones

INCREASE IN SUSCEPTIBILITY TO GROUND FAILURE

SEISMICALLY INDUCED GROUND FAILURE

1 — **LOWEST GROUND FAILURE SUSCEPTIBILITY** Includes exposed bedrock, thin alluvium and colluvium over bedrock, generally coarse-grained glacial depos ts, and mixed coarse and ine-grained glacial deposi s overlying bedrock in upland areas May experience minor ground cracking and acceleration of normal mass wasting processes i unconsolidated material such as rock fa ls and snow avalanches.

2 — **MODERATELY LOW GROUND FAILURE SUSCEPTIBILITY.** Mixed coarse and ine-grained glacial depos ts in lowland areas, thick deposits of channel, terrace, flood pla n and fan alluvium The thickness o alluvium in the upland areas variable, and some areas are ra ed as 1. May have very low susceptibility may experience minor g ound cracking, localized settlement due to consolidation, and perhaps liquefaction or urching of localized saturated zones of ine-grained material

3 — **MODERATE GROUND FAILURE SUSCEPTIBILITY.** Fine-grained surficial and subsurface deposits, including the Bootlegger Cove Clay, and other si clay, and peat dep sits Where coarser material (alluvium or fill) overlies these depos , the seismic-related ground failure susceptibility is ontrolled by the fine-grained material. May experience ground cracking and ho zontal ground movement due to landsprea ing or lurching, and subsidence due to consolidation.

4 — **HIGH GROUND FAILURE SUSCEPTIBILITY.** Fine-grained, surficial and subsurface deposits with n the vicinity of steep slopes. Includes areas above and below the s ope the width of which is approximately 10 times the height of the s ope ncludes a band behind seismic-induced landslides, the width of which is approximately 10 times the slop height in the slide a ea Highly susceptible to all types of seismically-induced ground fa lure, including liquefaction, translational sliding, lurch ng landspread ng, cracking and subsidence

5 — **VERY HIGH GROUND FAILURE SUSCEPTIBILTIY.** Areas of previous seismically induced landslides Includes the zone of tension cracks above the headward scarp and the toe bulge or pressure ridge areas. Although portions of hese previous slides may remain relatively undisturbed from future strong shaking, these slides will be the more likely site of uture seismically nduced sliding.

APPROXIMATE EXTENT OF BOOTLEGGER COVE CLAY. Arrows toward area underlain by Bootlegger Cove Clay.

NOTES

a) Base map origin: Municipality of Anchorage Planning Department.

b) Limitations: Interpretive map for general planning based upon Municipality-wide geological and other data which is of varying accuracy and completeness as explained in the accompanying report titled "Geotechnical Hazards Assessment Study," Municipality of Anchorage prepared by Harding-Lawson Associates Anc orage, Alaska Geotechnical hazards assessment and subsurface conditions relative to construction at specific s tes should be determined by qualified engineers and geologists through appropriate investigations of those sites.

A major earthquake could devastate all or any of the planning components--land use, utilities, transportation, and socioeconomic services--with major economic consequences. The earthquake's effect on the economic base would in turn impede the effective recovery of all or any one of the planning components (Figure 5). The public and the policy makers should be made aware of this fact. Through maps, overlays, and charts, simple presentations should be prepared to explain the possible effects of a future disaster. Need for studies should be related to the total welfare of the community. Data on geology, hydrology, soil, and slope presently prepared as "special studies" must become an integral part of the information used by the technical staff and presented to the public and the policy makers when decisions are made on land use allocation, construction of roads and utilities, location of public facilities, and evaluation of stability of various neighborhoods.

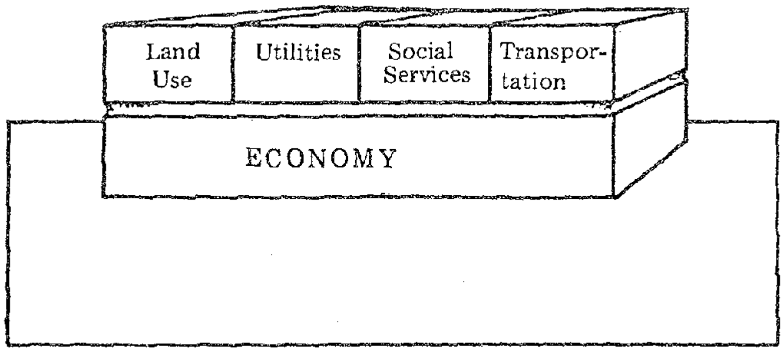
Unless this is accomplished, the same factors that affected reconstruction after the 1964 earthquake will occur again. Geologists and engineers then knew that southcentral Alaska lay in an area of high seismic activity. Reports had been written to guide planning in populated areas (Miller and Dobrovolby 1960). However, this knowledge had not been recognized and used by local, state, and federal planners in preparing planning documents for regional and municipal governments.

The reconstruction of the communities affected by the March 27, 1964 earthquake, therefore, was based on studies conducted immediately after the disaster by teams of scientists gathered under the auspices of emergency planning directives (Ink 1970). These task forces were composed mainly of physical scientists and planners. They remained functional until their recommendations were submitted to various agencies, departments, or city administrators. They did not review or discuss findings and recommendations with the public and local elected officials. Press releases (Figure 6) were used to inform the public of major issues, such as establishment of "high risk" in residential and commercial districts, without explanation of how and why these decisions had been reached. The Scientific and Engineering Task Force, which was responsible for recommending where reconstruction could or could not occur, was dissolved six months after the earthquake. This action left a series of firm recommendations dangling. There were no clear plans for enforcement, no procedures for adjusting or relaxing restrictions after ground stabilization had occurred and for when more sophisticated mitigation measures could be developed in zoning and building codes. The task force had no time to evaluate the long-range economic impact of their recommendations on the cities or to relate their recommendations to a regional growth development plan.

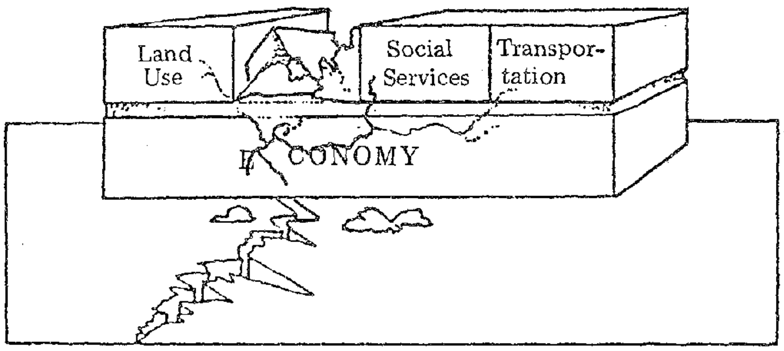
To obtain federal assistance, local governments agreed--without conviction--to change designations and to limit development areas. Later, many of the commitments were either forgotten or ignored. This is why, today, extensive new construction has occurred on or adjacent to slide areas in Anchorage, and the recommendations of the task force have been challenged in Kodiak, Seward, and Valdez.

Before disaster strikes, that is the time to stress community participation and education of policy makers so that both groups understand the multitude of topics and responsibility that each has in promoting public safety. The identification of seismic-hazard areas and research

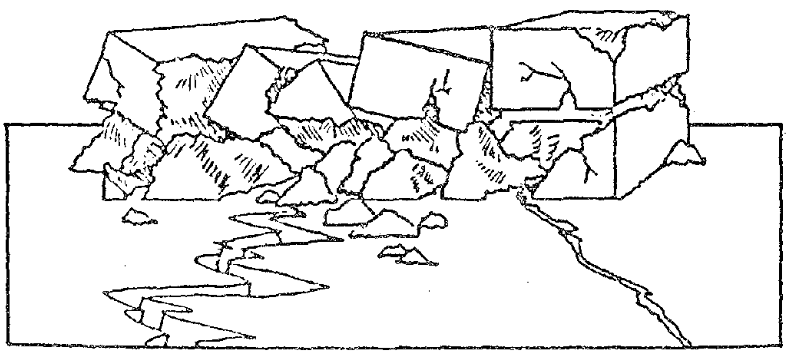
Figure 5. Results of Disaster on Comprehensive Plan Components



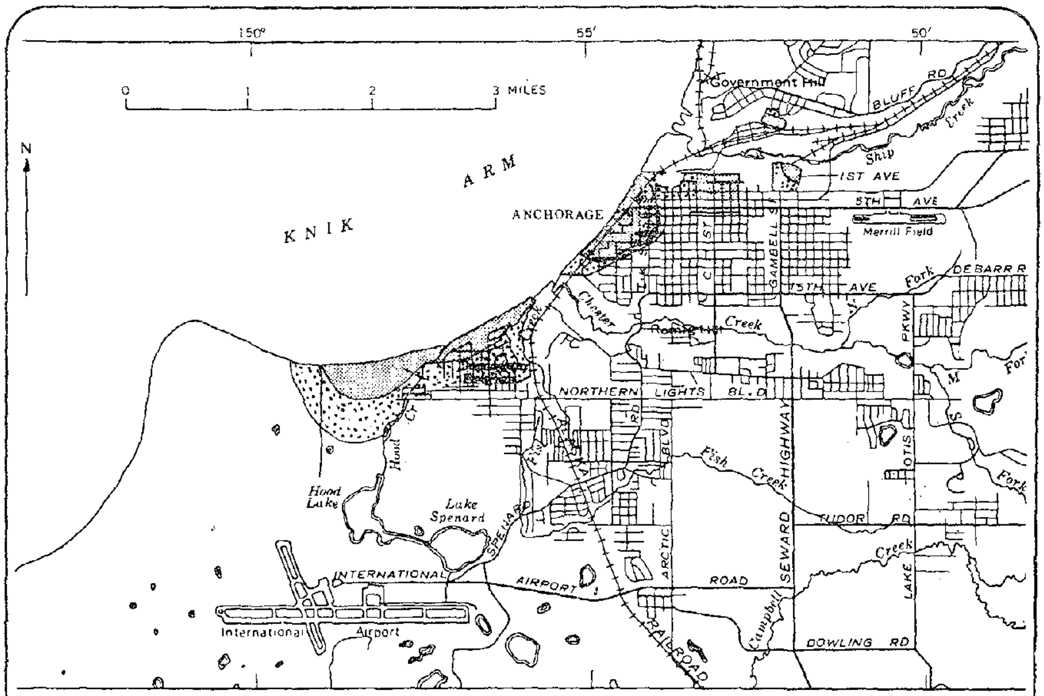
A city exists as an economic entity. The economic base supports the stability of comprehensive plan components.



If a disaster disrupts one or more components, it would result in instability of the economic base.



Economic breakdown would erode the stability of the entire urban structure and affect the regional economy.



EXPLANATION

Nominal-risk area

Little likelihood of landslides except for small slumps, largely in artificial fill. In all other respects risks are no greater than is normally expected in the construction industry where structures are built on a thick sequence of unconsolidated sediments. Current Uniform Building Code for Seismic Zone 3 applies both to new buildings and to plans for rehabilitation of earthquake-damaged structures. Special engineering consideration should be given to construction near the top, at the base and on steep slopes, especially wherever the Bootlegger Cove Clay is present. No filling, cutting, or construction should be permitted that will steepen or increase the load on or above these slopes.

Unstable area

Land considered unstable in the event of future earthquakes unless stabilization is attained. No new construction and only limited rehabilitation is recommended unless stabilization is attained. It is recommended that after stabilization new buildings on Fourth Avenue, L-K Streets, and Government Hill slides be limited to light structures not over two stories high. No buildings are recommended on the Turnagain Heights slide between the bluff and tidewater, nor on the First Avenue slides, even after stabilization. If stabilization is not effected, land will be "high-risk" classification.

Provisional-nominal-risk area

Reclassification to "nominal-risk" in these areas is contingent on stabilization of adjacent slide areas or stabilization within the areas themselves. If stabilization is not effected, land will be "high-risk" classification.

The Fourth Avenue slide was stabilized by construction of a buttress and special drainage treatment.

Source:
Committee on the Alaska Earthquake, National Academy of Sciences, 1971. **The Great Alaska Earthquake of 1964: Geology.**

Figure 6. Map and Press Notice Released September 8, 1964, Representing Final Recommendations in Risk Classification of Anchorage by the Scientific and Engineering Task Force.

aimed at understanding and preventing seismic induced disasters will remain an esoteric exercise unless adequate steps are taken to achieve proper communication between researchers, planners, and policy makers. This communication is the base for achieving direct input into the decision-making process at the local government level.

A city exists as an economic entity. To be effective the planning process must evaluate the economic implications of proposed recommendations. This evaluation must consider short-term and long-term costs and benefits of urban areas as well as their area of economic influence. To date, cost/benefit analysis has not been part of comprehensive risk planning.

The Alaska experience clearly points out that the evaluation of the physical environment alone is not enough to ensure safe redevelopment. Although there were excellent interdisciplinary teams evaluating and studying the physical effects of the earthquake, they lacked the assistance of sociologists, economists, and planners to evaluate the effects on people and the urban economy. Even more important, these teams were not coordinated by a local lead agency able to relate the findings to state and local political infrastructures. Consequently, the effectiveness of the task forces was lost when they were dismantled six months after the earthquake. Without continued guidance by scientists and planners, local governments fall prey to greed, temporary solutions, and speculative answers.

Summary and Recommendations

The Alaska experience exemplifies the need for improving the methodology applied in the preparation of regional and municipal plans in seismic risk areas. As posteven time passes, memories of the March 27, 1964 earthquake fade regardless of the physical studies conducted. The general public believes that another earthquake will never happen. Therefore, developing sound land use laws to protect the citizen has met with more and more resistance. Physical data, including geology, seismology, topography, hydrology, and soil, must be evaluated along with information on land use, conditions of structures, and land value, and ownership and related to a "risk component" developed as an integral part of the comprehensive planning process.

Study of seismic hazards must become a major component of the baseline data developed as part of the planning process rather than considered as a special study programmed sporadically in response to temporary and limited funding sources or interests. The baseline data must be updated on a continuing basis to assure that new information, methodologies, and concepts are used in preparation of comprehensive plans that include regulations directed to mitigation of seismic risk and set guidelines for post-earthquake recovery. This would require the coordination and cooperation of many disciplines and agencies at the local, state, and federal level. All agencies should relate and tap the same baseline data when implementing programs or projects as construction of utilities, roads, ports, airports, development of residential, commercial, and industrial districts or in building schools, hospitals, or other social projects.

Scientists and planners must build communication skills to inform the public and the policy makers of their findings. Consideration of seismic risk in urban areas will gain greater prominence only if scientists promote understanding of their work, meet with planners and other responsible public officials on a personal working-level basis, learn how to communi-cate with them, evaluate their needs, and design their own products to be of optimum use. This effort would add to society's value of scientific research. This broadening of scientific responsibility would enhance public understanding and support of research needed to assist policy makers in setting guidelines for public safety.

Overspecialization and administrative division of specialized fields further impeded long-range recovery programs following the March 27, 1964 earthquake. Extensive studies were conducted on the physical environment, but little emphasis was placed on socioeconomic research. Also evident were conflicts on agency guidelines, time tables for implementation of programs, and funding of specific projects, which interfered with the continuity of the implementation process and diluted recommendations made after the disaster. Institutional changes will be necessary if interdisciplinary coordination is to be effective. Unfortunately, federal, state, and local agencies' specializations and limitations are now even narrower than in March 1964.

The Alaska experience clearly demonstrates that the following components of hazard planning can help minimize damage and aid in post-disaster recovery.

1. Existence of a plan which includes both physical and human considerations and is defensible from the engineering and socioeconomic standpoints.

2. All agencies involved in pre- and post-disaster planning organized under one coordinating entity.

3. Urban development planning meshed with long-range planning for the region and the state.

4. Regular, established communication with the communities, news media, and appropriate governmental agencies and policy makers to educate them and to develop the support needed for a long-lasting commitment to research, planning and implementation of recommendations.

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A FEW PROBLEMS OF LAND USE IN RELATION TO EARTHQUAKE-PROOF IN TIANJIN CITY

by

Wang Zun Kuo Song Be Kan Su Shih Kuang

Abstract

Zone of Tangshan-Fengnan caused 7.8 scales strong earthquake, in which effect on Tianjim City as result for earthquake a lot of building were collapse and dangered to safe of peoples life. After the earthquake, we was investigating for the seismic destructiveness and the properties of the earthquake in the built-up district. The text in the foundation of survey, only is a few problems of land use in city planning layout in relation to earthquake proof.

In the planning of land use, its considerated planning of earthquake-proof. This is initial view only, the text run on land use at the abnormal area of strong earthquake, Land use of the pit-pool and green, Layout of dangers factory and warehouse, Underground live-line. Planning of road system and so on.

The Tianjin Department of City Planning

(1) Chief resident engineer,
(2) Engineer,
(3) Photographer.

1. Seismic backgroung for Tianjin City.

1) General description of the geologic structure.

The geologic structure of Tianjin City lies on the northeast of the settle-belt in northern area of China(Huabeis Place) which belong to latitude geologic structure and neocathaysion system. In the deeper strataa of the plain,northeast, in which is form a relative arrangement in consist of the foundation base of plain, Rift of the foundation base according to its property and trend may be divided into groups: reversed rift of compression-torsion style, that is mainly with distributed to northward, northeast(Tianjin Norths rift, Baidangkou-Wests rift and cangdongs rift), and positive rift of non-torsion style, that is mainly which distributed to northward, west west. (Chengling-zhuanges rift and Hai-river rift). There are showing in the Fig. 1.

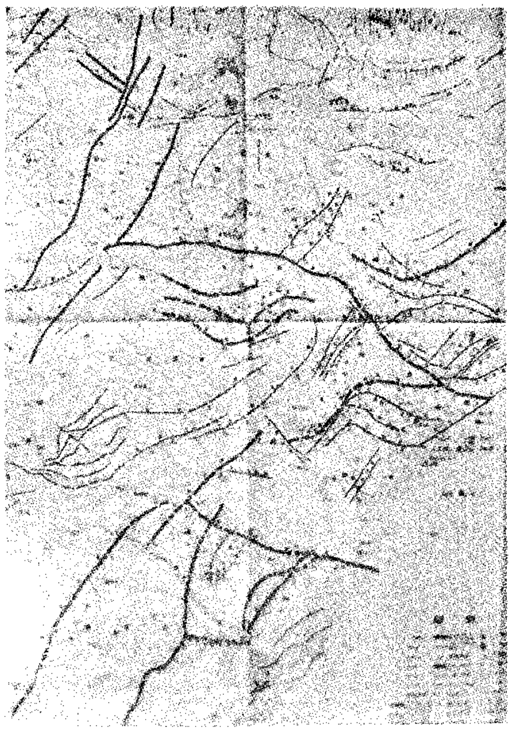


Fig. . Geologic structure system, Tianjin City.

2) Geological conditions for the epi-surface.

Influence of industrial and civil architectures on the foundation soil that thickness is about 30 meters, they (upper) and holocene series, its may be divided into three stratum:

a) Lowest-stratum is belong to land sedimentary facies (from late-period, upper pleistocene epoch to early-period, holocene epoch).

b) The stratum of center is marine deposit (middle holocene series).

c) The upper of stratum is land sedimentary facies (late-period holocene series). see Fig. 2.

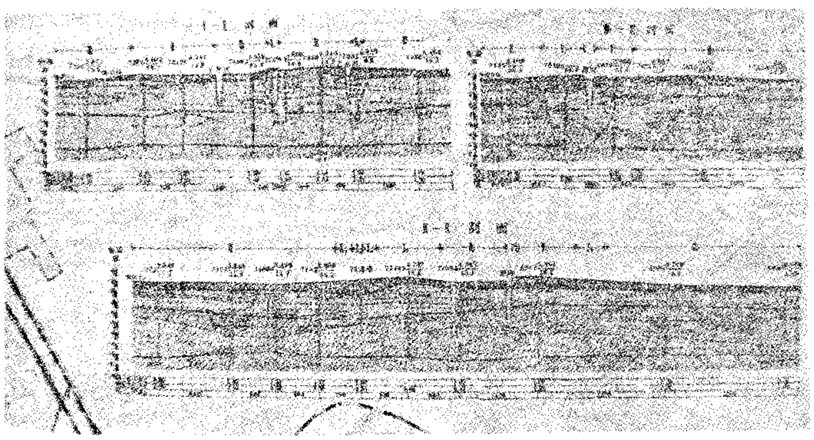


Fig. 2: The section of geological horizon for shallows stratums.

On the land sedimentary facies of late-period holocene epoch, its accomulated to neogenic sediment or artificial fill soils. The thickness of this fill soils stratum about 2-5 meters, may be divided into three types:

a) Alluvial soil:

Formation from clayey sand of Hai-river through alluvial effort mainly in distributed to Hai-river-South and Hai-river-North, the are light clayey loam mainly.

b) Mixed fill soils:

Include to three kinds for ruins soil, furnace lime soil and garbage soil that in distributed to old city zone.

c) Plain fill soil:

Mainly in distributed to edge zone of built-up district of Tianjin, there are about 30% to zone of total fill soils for the built-up district, there are clayey soils mainly.

Inthe built-up district, Neogenic sediment is distributed in the accumulation area of recent river, old channel and abandoned old channel, that is discard be cause of river straighten through human. there are light mainly, see Fig. 3.

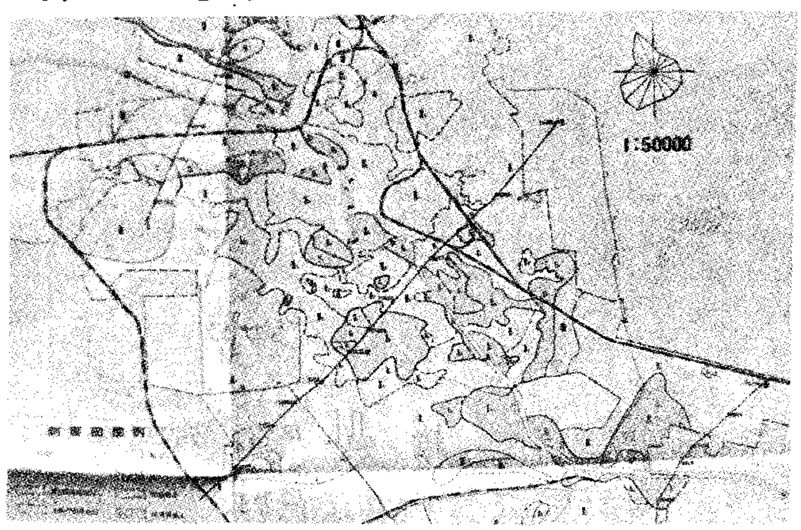
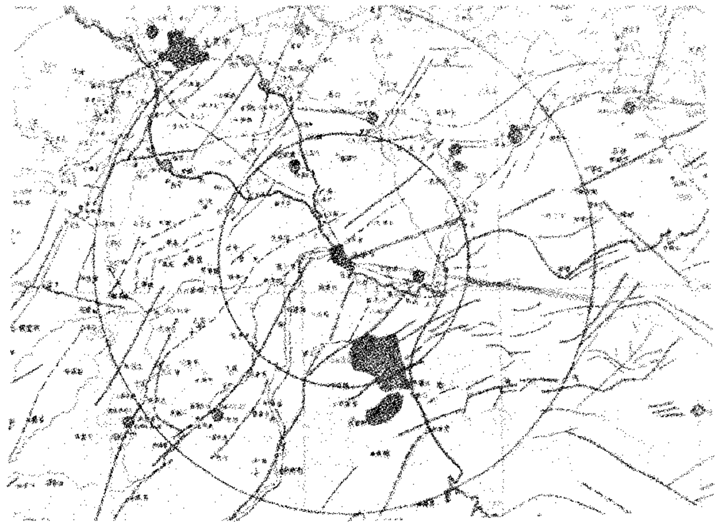
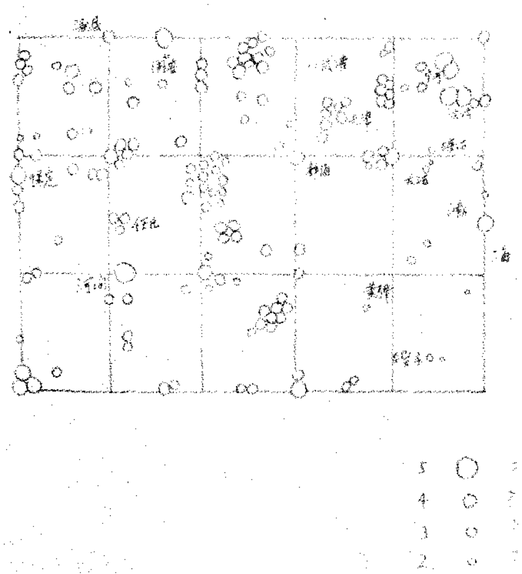


Fig. 3 . Distributed drawing for the neogenic sediment and articial fill soil.

3) Characteristic of Tianjins large earthquake historied in the lately for 300 years.

For Tianjins earthquake, begin records 70 B.C. (Hahn Dynasty, Hahn Xuans Yellow Emperer), Up to 1976s, in regards to recorded earthquake already has 94 times, In the lately for 300 years, larger earthquake have 33 times take place. Fig. 4. showed to the earthquake of 5 scales and above 5 scales in Tianjin, in Tianjin and in distant journey Tianjim and Fig. 5 showed to the distributed earthquake from 3 scales to 6 scales in Tianjin City.




5
4
3
2

Inaccordance with showing figures through analysis was without destructive earthquake take place in Tianjin City 9 generally speaking, the destructive earthquake is above 6 scales). If assume in center of city, there are 3 times above 5 scales in the range of 65km, and have 12 times in the range of 65km. and have 12 times in the range of 130 km, 9 times take place beyond range of 130 km, these earthquake had effect on Tianjin City. In particular, Zone of Tangshan-Fengnan caused to 7.8 scales strong earthquake, in which effect on Tianjin City, as result for earthquake a lot of building were collapsed and dangerd to safe of peoples life. As a result of earthquake, serious zone had Ninghe and Haugu, middle serious zone had the district (built-up district and Baodi), Jinghai and Wuqing were lighter destructions. In the district (with sone area), emerge to seismic-geological phenomenon, e.g. ground surface fissure, curved arch, slump and subsidence, and sand jet and belch water and so on. After the earthquake, we was investigating for the seismic destructiveness and the properties of the earthquake in the built-up di district. (on layout to 37 shallow hole of the explore, soil samles had 171, and two penetration test). Through initial analysis, drawing a seismic intensity be distributed for the built-up district, its indicate from the analysis of the figure, seismic intensity had most serious in which Heping and Hexi, Nankai and Honggiao is middle scale, Hebei and Hedong is most light. In addition, we take a lot of photogragh for earthquake analysis from this photograghs may be result fillowing:

a) Ground surface fissure:

Mainly be distributed in the areas of Heping and Hexi, The fissure width being 1-2 generally, Max. had 30, fissure direction had two:mainly be NE (20,40, 60,) and NW (20, 40,) they are perpendicular each other in crisscross distance had 3-60, In addition trace phenomenon for horizontal movement is existance about the horizonutal displacement had 3-20, the fissure length had severral-ten to the one hundred meters, Some fissure had 400-1000 meters length. see Fig. 6.

Fig. 6. Ground surface fissure, Chen-Dui-Zi.

b) Sand jet and belch water.

Mainly be distributed in the two areas of Heping and Hexi. Sand jet and belch water had several-ten places, and sand jet point had ten thousand places. Then sand jet and belch water major to producted with ground surface fissure. The sand content for each hole of the sand jet general had 0.1-1 m a few holes for sand content had 5 m, colour for sand jet major to be glay-yellow, yellow-glay and yellow-brown, They are loamy soil and silt sand soil. see Fig 7.

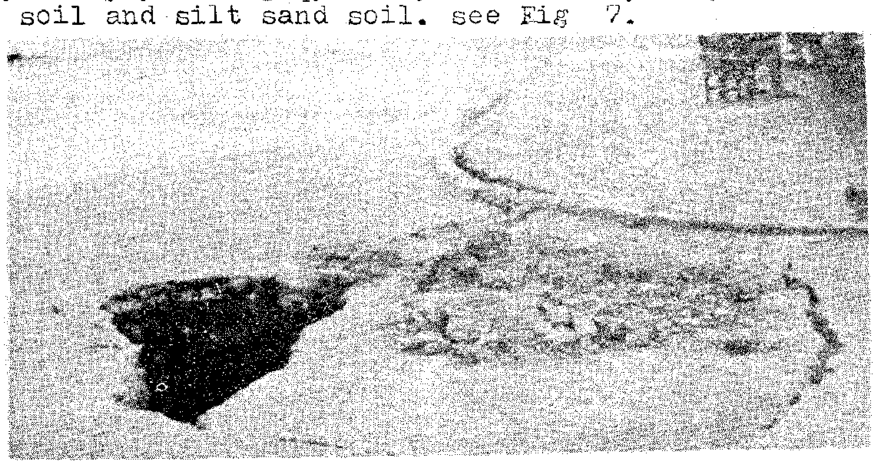


Fig. 7. Sand jet and belch water, Four-New-Bridge.

c) Settlement of the ground surface.

Settlement is 30cm for along two banks of Hai-river. Settlement of Zhanzhuagzi area to about 70 cm. but the ground between Tianjin-Tanggu highway and Hai-river rising 20-390cm. see Fig. 8.

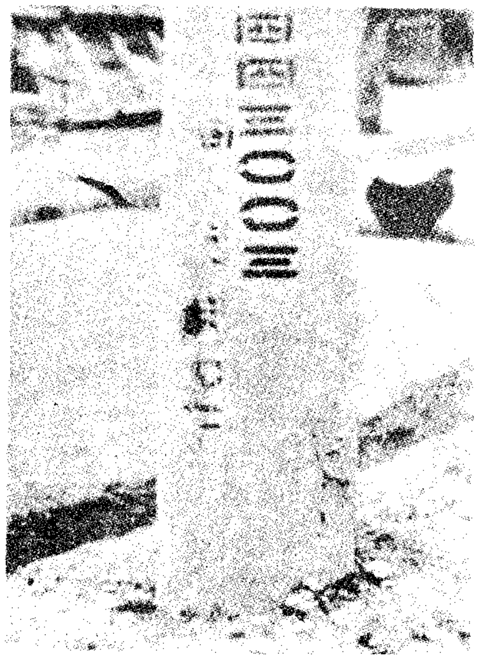


Fig. 8. Settlement of the ground surface, Ding-Shi-Gu.

d) Buildings destruction.

In general, the building had settlement, fissure and even to collapse take place, owing to site appeared to the slide, displacement and ground surface fissure in the ground deformed and old channel on account of ground swell, settle and relief etc, in which across on areas of the ground fissure as well as areas of serious sand jet and belch water. general to three groups cocluded to follow:

a) Collapsed building mainly to contributed at the zone of Heping and Hexi, see Fig. 9.

b) Destructive building seriously, mainly to contributed at the two banks of Hai-river. see Fig. 10.

c) Destructive building generally. Mainly to distributed at the edge of the district and areas of east-north direction. see Fig. 11.

Fig. 9. collapsed building
Lan-Zhou street.

Fig. 10. Destructive building seriously,
Tianjin-Hotel.

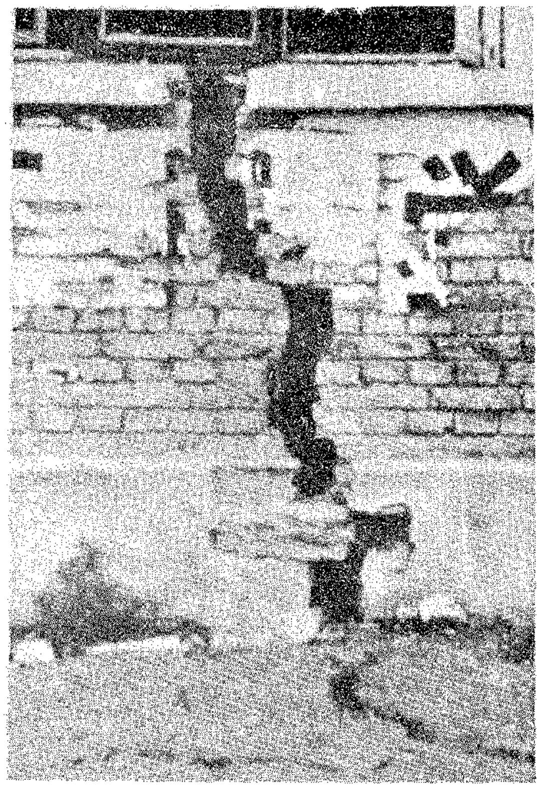


Fig. 11. Destructive building generally.

2. Cosideration on the earthquake-proof in the city planning layout.

Tianjin is a super-large city, Population in the district had 3 millions, Areas of the district had been 16089 hectares, Average area for each person had 53.2m, Up to date, Ten industrial district and many dwelling district had been forming, that archi-tec tectural density and density of population is very high. we think, for the district over lands is very hight, Important subject for the consideration and research is equilibrum of construtive lands for the layout, full utilize for space of the cities and collaborate coordinating correlation between the land use and earthquake under this situation. As a consequence, Several problems was presenting follow:

1) Land use at the abnormal area of strong earthquake.

In the abnormal area of strong earthquake had unusual situation for ground surface fissure, settle, sand jet and berch water and so on. in the event of earthquake take place and lead to building to collpse.

Therefore, we suggested, On the old channel of Daqing-river along to north-side of Timba-highway should not to building and take up a green-belt had a suitable width. that is take apart the industrial district on the road-south with the dwelling district on road-north, bring about effect for protected to sanitation and take refuge apace. At the dwelling district on the road-north, its carry out from the lower transitived to high story gradually, In the interest of avoided to the slide and movement of new areas for the old channel. After the building had been collapse owing to earthquake no longer restoration at old channel cutoff for the lower reaches of Hai-river on the earth east-southfor the time being deserted areas in principle retained make green by planting tree. and comstructived to aport

count, parking place, building mainly to constructived for 2- 3 storys Liulings area at south bank of Hai-river, convalescent hospital building by all means restoration demand to avoid crisscross beld of ground rift in the deconstruction for the design and no longer will being added new building, However, retained to make green by planting trees on the area of serious jet sand and belch water.

In part nuilding weed to consolidation and other a part as collapse house demend to restoration on the Heping and Hexi areas, that being had densed populations and on the row buildings zone. Zone of the dwelling is serious collapse and the jet sand and belch water on the Loiuzhou-street may be building to some lower story house, and contrustived to some up to 14 level story for tall-buildings along main streets. But, the pile-foundation must through wind-fill soil into lower yelloww soil foundations, In order to decreased densities of building that make up some small green zone in the interests of take refuge when earthquake take place in the zone of new building.

Park of river bank was stand up for retainted width green zone in the Hai-river banks, Connot to were building in the range of 30 meters in order to make use of river bank roads. In order to empty a lot of area for green ground and may be building to tall building beyond the range of 100 meters, There are more than green and some lower level-story house on the old channel of Hai-river at Xoangzhuangzi and so on .

Inthe hope of connot or few building to dwelling in the intensity of earthquake in Ⅷ - Ⅵ scales will be distributing parks, aport count, parking place, lower house and a few tall building. Utilize fully for unusual zone of safe isoland of the Ⅷ scales will be builded to large public building and tall dwellimg constructed to 4-level story building in 1:1.5 distance generally in the normal zone of the seismic intensity Ⅵ scales.

see Fig. 12.

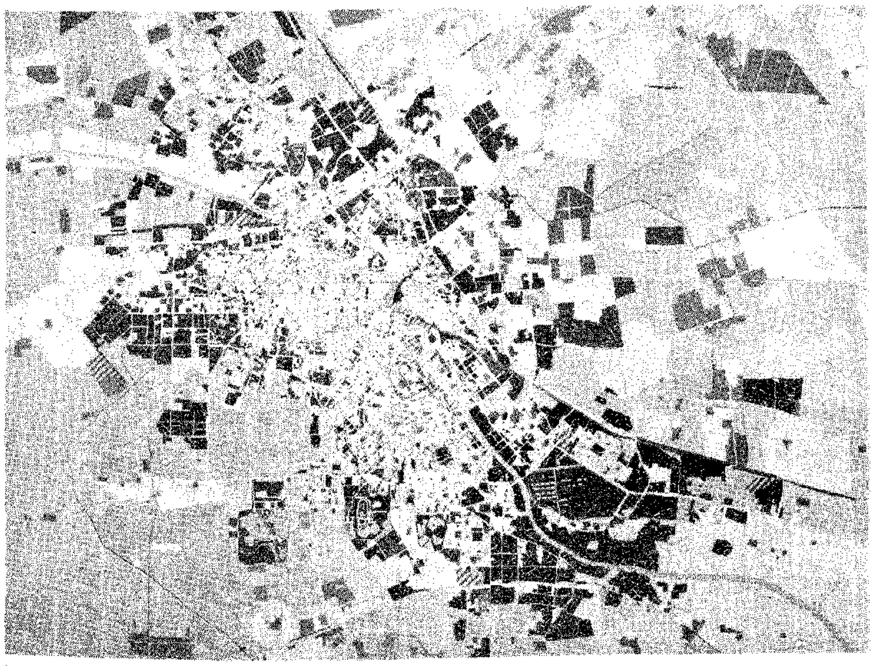


Fig. 12. Land use, Tianjin City.

2) Utilization in the natural water-bodies of the pit-pool for the earthquake-proof.

The district had many pit-pool at present. This pit-pool being had large lose in past when the earthquake take place result were filling in order to builded. Adjust retain effect in order to cities rain-water for removed had werken at the same time, Therefore, this pit-pool by means of retained and link to with river-road formed to water-systems. Bankes of pit-pool shall builded parks. This water of the pit-pool in order to urgent saving after the earthquake take place.

3) Earthquake accident-proof place and zoning of building groupe.

Dwelling zone being had 99 is divided for planning the district. Population for each dwelling zone shall had about 300-400 hundred and divided into small areas and the blocks. Population for small areas shall had about 100 hundred and each block shall had about 20-30 hundred.Each person will had 2-4m for green index of dwelling district. we will used to primciple of centralize and disperse build to center park will was construction in the each dwelling district and connected to several small parks. There green and space-system will form storage grain, water and sawing service on these large and smallparks for take refuge zone. Distance from the dwelling district to take refuge zone was being the more inmate the more nice. In order to safety for dispersed cityite and building along to dwelling district. System will builded for different levels fire station and initial fire was perish that need to certain lands, it will be planning for dwelling district in order to prevent diffusions of fire after the earthquake take place. Zone of take refuge by all means integrate planning in the total district as long as system of space-green was being very good. see Fig.13.

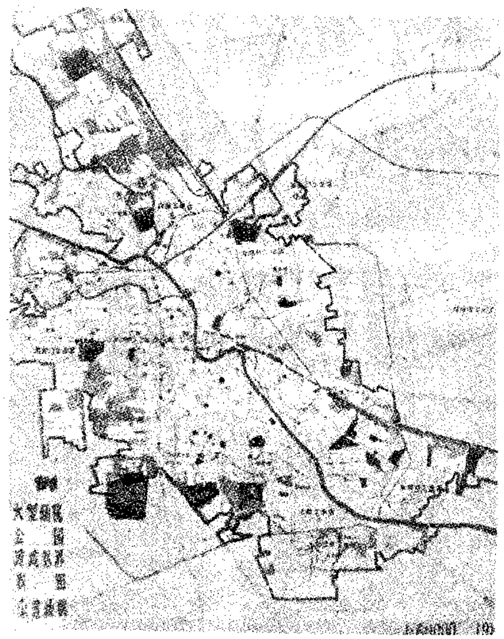


Fig. 13. System drawing of space and green, Tianjin City.

4) Problem of central management for the product and warehouse of dangers.

Warehouse land of Tianjin City about 7% in total built-up district. These warehouse storage to grain, coal, building material and other ware as well as included to the dangers. Producting of using chemical danger had works 1162 at present. Many works warehouse mixed with dwelling shall been danger safety directly in distruting each corner of the district. Inaddition, special warehouse for the danger was being small menber and mixed storage warehouse is majouity, Therefore, we suggesting to the works and warehouse about producted danger no longer builded in the district, Oriqina works for the producted burntable and explosivable danger in the district should more from urban to planning zone of chemical industry gradually or change to non-dangerous producted, Works and warehouse for storage the danger should retainedthe district of safety beyond 50 meters. In order to decresed to presure for safety of the district and should build to several special warehouse for danger in the suburban when planning. In addition, divided into certain land for works and warehouse of dangerin the industrialz zone. In order to safety added to green apart belt should have strenthen management about works and warehouse in producting danger temporary no move at the district. Fire service of each works and warehouse should be forming net-system with city-public fire service.

5) Safety working for underground live-line able was being assured.

If the earthquake take place, should using water for the live, prevent fire and product as well as lighting and owing of cities, in which to system of the water supply and power supply is very important. For water-works lave three river-water-works in Tieyuan, Lingzhuang and Mazhuang at present. In addition to well-water-works 14 in order to abnormal times, planning two river-water-works in Xinkaihe and Guijiamatan shall be build in future, shall be forming poly-water-source for the water supply and linkage forming circular pipe-net was being assure to nimble water supply. Other four water-works supply to total urban centinually in the case of one work was being destruction owing to earthquake take place. System of supply power from suburban high-voltage system will power to urban at a time. But, four high-voltage (220kv) in to urban should have distance certainly apart with building. In order to safety for supply power in which lower-voltage transformer station. should be distributing balance and forming a circular-line from the system of the distribution. In order to safety when the earthquake take place and urban-art demended, replace a high-air cable with on underground cable in the planning supply power line. Therefore, situation of the water-power service and pipe-net line in demand land for the ground and underground should be assuring, in planning utilization of land for the urban.

Cities road-net should arrage appropriately in order saving traffic able assure to unblocked when the earthquake take plce.

We the earthquake take place considerating fifteen outlet road in planning. This road shall be linking to suburban-highway, and planning suburban-highway emerging circularly cionncted to this outlet road in order to saving vehicle able to come in and go out. Assored to traffic able to unblocked when the earthquake take place shall have two circular-road pass through every zone of the urban.

Several measure in consideration of decresed to seismic destructiveness in view of the utilization of the land from the above mentioned. This is initial view only.

Thanks.

Architectural and Planning Research
for Earthquake Hazard Mitigation in the
United States
by Dr. Frederick Krimgold *

I. Background of the NSF Earthquake Hazard Mitigation Program

Destruction due to earthquake has plagued civilization for thousands of years. It is only in relatively recent times that the problem of constructing facilities to resist the effects of earthquakes has been approached in a scientific way.

In the United States, some interest was generated after the 1906 San Francisco Earthquake, but it was not until the 1933 Long Beach Earthquake that a building code was developed to address problems of earthquake resistant design and attempts were initiated to make measurements of the actual ground motion which occurs during an earthquake. During these early years little was known about the motion realized in an earthquake. The ability to process any strong motion records obtained or to calculate the dynamic response of structures as a result of such strong ground motion was severely limited due to a lack of fundamental methods of analysis and because no high speed computational equipment was available for making such studies. Sporadic progress was made using rudimentary shaking tables and simplified structural models and, in the late 1940's and the early 1950's, using analog computers.

The state-of-the-art for earthquake design which evolved during this period depended on the use of a statics approach to a dynamic problem. Responsibility for most work in the field of Earthquake Engineering lay with the Structural Engineering profession (particularly SEAOC, the Structural Engineers Association of California) and primary emphasis in research and regulation was on prevention of structural collapse.

The National Science Foundation was established as an agency of the Federal Government in 1950 to support basic research in the natural, physical and social sciences and engineering. NSF is one of may Federal agencies which support research, however, NSF support is focused on basic research programs at universities. The policy of the National Science Foundation is determined by the National Science Board which is made up of distiguished scientists.

Specific research proposals are evaluated and selected for support on the basis of scientific peer review. The activities of the Science Foundation reflect the interests and judgement of the academic research community.

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During the early years of the existence of The National Science Foundation a few grants were made in the Engineering Program on earthquake effects. In 1962 the Engineering Program was changed to an Engineering Division and several specific programs were established. One of these programs was the Engineering Mechanics Program which was charged with the responsibility for supporting research in the areas of Civil, Mechanical, Aeronautical, Architectural and other branches of Engineering.

In 1962 the Engineering Mechanics Program identified a general area of opportunity in consideration of impacts of natural hazards on natural and man-made systems. Important hazards included floods, wind, expansive soils, earthquakes, storm surges, tsunamis, land slides, and large-scale land subsidence.

Of these natural hazard studies, earthquake engineering emerged as a major focus of research activity. During the period of 1963 to 1971 the annual research budget for earthquake engineering grew from $1.1 million to $2.5 million.

Some important accomplishments during these early years of the program were:

- O Support of the publication of the Earthquake Engineering Research Digest
- O Establishment of the University Council for Earthquake Engineering Research
- O Establishment at the National Research Council of the Natural Disasters committee which could dispatch teams to collect perishable information after natural disasters
- O Development of the report "Earthquake Engineering Research" by the National Academy of Engineering (Reference # 1)
- O Development of the 20 foot by 20 foot shaking table at the University of California, Berkeley
- O Establishment of the National Information Centers for Earthquake Engineering with two major components - (1) Publication and Strong-Motion Data and (2) Computer software dissemination
- O Establishment of cooperative research programs with Yugoslavia and Japan.

On a general level, substantial progress was made in structural dynamics theory, understanding of the dynamic behavior of construction materials, soil dynamics, and strong motion instrumentation.

In 1972 the Earthquake Engineering component of the Natural Hazards research activity was transfered to the newly established Research Applied to National Needs Program. As an applied research program the earthquake research effort expanded to a multidisciplinary effort. New emphasis was placed on the application of research findings in professional practice and in public policy.

In 1972 a multidisciplinary study including geographers, sociologists, and engineers was assembled at the University of Colorado to assess research on 15 national hazards, including earthquakes, and to make recommendations on future research opportunities Which might have relevence for public policy in hazard mitigation. In 1975 the Societal Response to Natural Hazards Program was created and the 1975 University of Colorado Report Assessment of Research on

Natural Hazards, (see references # 2,3,4,), became part of the basis for strengthening research on the social, economic, political, and legal consequences of disasters and natural hazards.

In 1976 in response to a request of the Science Advisor to the President, an Advisory Group on Earthquake Prediction and Hazard Mitigation was established. The Advisory Group, under the leadership of Professor Nathan Newmark, prepared a detailed plan for earthquake research to be carried out by two Federal Agencies; The United States Geological Survey and The National Science Foundation (see reference #5). According to the plan the U.S.G.S. was to have primary responsibility for research in prediction, induced seismicity and hazards assessment. The National Science Foundation was to have primary responsibility for research in fundamental earthquake studies, (Division of Earth Sciences), engineering and research for utilization (RANN).

The Earthquake Hazards Reduction Act of 1977 (see reference #6) established a "National Earthquake Hazards Reduction Program" which assigned responsibilites for research, operational activities and overall program coordination. The Federal Emergency Management Agency is responsible for coordination and management of the federal program (see reference #7) and the U.S.G.S. and The National Science Foundation have primary responsiblity for earthquake research. The Earthquake Hazards Reduction Act of 1977 assigned the chief responsibilty for supporting research on earthquake engineering, architecture, planning, and related policy aspects of the earthquake hazard to The National Science Foundation.

In 1978 the Earthquake Hazard Mitigation Program was established as a part of the Problem Focused Research Applications Division of NSF. In response to the Earthquake Hazards Reduction Act of 1977 the Program was structured under three elements: Siting, Design, and Policy. [In this configuration of the program, planning research was included in both the Siting and Policy, and architectural research was included in the Design element.]

Current Status of the Program

In March of 1981, with the establishement of the Engineering Directorate at NSF, the Earthquake Hazard Mitigation Program was incorporated into the Division of Civil and Environmental Engineering. The program is structured under 19 objectives including basic research, applied research and research applications in engineering, architecture, planning and the social sciences (see reference # 8).

The current objectives of the program are:

1. To gain understanding of the nature and distribution of destructive earthquake ground motion;

2. To develop and install instrumentation to measure strong earthquake ground motions and their effect on constructed facilities;

3. To develop through experimental and analytical research an understanding of the behavior of geotechnical materials subjected to destructive earthquake loadings;

4. To develop analytical, numerical and computer methods to study and predict dynamic response of structural systems;

5. To experimentally determine the structural properties of materials, elements and systems subjected to intense cyclic dynamic loads;

6. To develop methods to evaluate the hazard potential of existing buildings and structural systems;

7. To develop methods of analysis and design to reduce damage to non-structural and architectural systems subjected to damaging dynamic loads;

8. To develop methods to reduce earthquake vulnerability of buildings and urban regions through improved architectural and urban planning analysis;

9. To develop improved methods to predict destructive seismic effects on coastal and inland waterways;

10. To develop methods to predict seismic effects on distributed lifeline facilities;

11. To continue and improve procedures for rapid response to optimize learning from post-disaster earthquake studies;

12. To develop international cooperative research programs which take advantage of unique research opportunities.

13. To improve existing and develop new information transfer programs to speed the flow of information developed through research to operational government agencies and to design professionals;

14. To develop improved methods to assess and predict the safety of dam-reservoir systems;

15. To study the interaction between design considerations for other hazards and seismic resistance of constructed facilities;

16. To develop knowledge on the socioeconomic aspects of hazards mitigation;

17. To increase the base of knowledge on preparedness for earthquakes and other hazards;

18. To improve the understanding of disaster impacts and response;

19. To provide a basis for improving the dissemination of information on earthquake hazards and its utilization by decision makers and the public.

Between 1971 and 1981 the budget for the program expanded from $2.5 million to $19.6 million.

II. Background of Architectural and Planning Research in the Earthquake Hazard Mitigation Program

In 1976, as part of the expansion of applied research in the earthquake research program, projects were initiated in the areas of architectural and planning research. Early work in these disciplines began with grants to research units associated with relevant professional societies, The AIA (American Institute of Architects)/Research Corporation and AIP (American Institute of Planners, now incorporated into the American Planning Association). Research adgendas were developed (see references # 9, 10) and materials were prepared for dissemination to practicing professionals and training institutes (see reference # 11) were held for faculty to improve professional instruction and to encourage development of needed research capability.

Initial grants were also made to research oriented architectural and planning firms to study basic problems including seismic performance of non-structural building elements, the influence of architectural configuration on structural performance, and the relationalship of land use management to seismic hazard reduction.

With the establishment of an Earthquake Hazard Mitigation Program in 1978, planning research was incorporated into the Siting Element and architectural research was included in the Design Element.

A listing of Architecture and Planning research projects supported by the National Science Foundatiton between 1975 and 1980 is included in Appendix #1. (also see reference # 12)

III. The US-PRC Protocol of Scientific and Technical Cooperation in Earthquake Studies

In January 1980 an agreement was signed by the Directors of the National Science Foundation and the U. S. Geological Survey on the American side and the State Seismological Bureau on the Chinese side for bilateral cooperation in earthquake studies (see reference # 13).

Article 3 of the Protocol establishes the forms of cooperation as:

1. Exchange of scientists, specialists, delegations, and of scientific and technical information;

2. Cooperative research on subjects of mutual interest, including devising and installing of instruments and equipment and the analysis of data therefrom;

3. Joint organization of scientific conferences, symposia and lectures;

4. Such other forms of cooperation as are mutually agreed.

Annex III to the Protocol for Scientific and Technical Cooperation in Earthquake Studies covers cooperative research on earthquake engineering and hazard mitigation and Section 6 of that Annex covers architectural and planning research under the study of technology for urban earthquake hazards reduction.

The following considerations are included to study measures for urban planning and architectural design:

(a) The study of effects of building and infrastructure system failures on life safety and recovery;

(b) The study of measures to decrease vulnerability of infrastructure systems in urban areas;

(c) The study of mitigation measures for critical facilities such as hospitals, police and fire stations and communications systems;

(d) The study of techniques for seismic reliability identification, reinforcement, and other alternative procedures for reducing vulnerability of existing structures;

(e) The development of techniques for assessing vulnerability of habitational structures;

(f) The study of planning and management of land use as measures for reducing vulnerability of buildings and infrastructure systems in urbanized areas including both pre- and post-earthquake contexts.

The implementation plan for the Year 1981 provides for a related workshop on microzonation techniques for earthquake hazards reduction in the fall of 1981 in China. That workshop contains elements of planning research including interpretation of geotechnical information for application in land use management and is scheduled in advance of the workshop on technology for urban earthquake hazards reduction.

The US-PRC Workshop on Earthquake Disaster Mitigation through Architecture, Urban Planning and Engineering has been organized under the Protocol for Cooperative Earthquake Studies. It is intended that it should conform with the objectives of Article III of the protocol. The Workshop is expected to provide for the establishment of direct contact between researchers, the exchange of technical information and the development of cooperative research proposals on subjects of mutual interest.

Development of US-PRC Cooperative Research Proposals for Architectural and Planning Research

One of the primary objectives of the bilateral workshop is the establishment of productive long term relationships between researchers and research institutions. An important mechanism for establishing and maintaining those relationships is the pursuit of collaborative research projects. In the course of the workshop it is anticipated that research topics of mutual interest and areas of productive collaboration will be identified. The pursuit of collaborative research efforts can be supported under the terms of the Protocol (see reference # 14). Proposal evaluation on the U. S. side will involve the standard peer review procedures of the Foundation with reference to the following criteria:

0 prospective contribution to scientific knkowledge;

0 competence and complementarity of the researchers, U. S. and Chinese, in the proposed area of cooperative activity;

- O appropriateness of the design of the project research plans
- O prospective benefit from internationl cooperation;
- O appropriateness of permanent equipment request.

Evaluation procedures and criteria on the Chinese side may vary from those of NSF.

In preparation of a collaborative research proposal the U.S. and Chinese principal investigators must reach agreement on:

- O the substance and format of the proposed project;
- O the starting date and duration of the project;
- O participating staff
- O division of work; where it will be performed, when and by whom. (Visits are generally limited to one visit in each direction within one year. The duration of visits does not normally exceed six months.)
- O planned use of research facilities in the United States and China.

<u>Conclusion</u>

Over the past twenty years a vigorous and productive program of Earthquake Studies has been developed in the United States. More recently, in the past five years, architectural and planning researchers have come to play an active role in the Earthquake Hazard Mitigation Program. It is now hoped under the terms of the US-PRC Protocol for Scientific and Technical Cooperation in Earthquake Studies that researchers in the United States and China may work together to advance scientific understanding and assure public safety in both countries.

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NON-STRUCTURAL COMPONENTS

1980	Investigation of the Seismic Resistance of Interior Building Partitions	R.W. Anderson	Agbabian & Associates El Segundo, CA 90245
	Methodology for Evaluating and Improving Operational and Non-Structural Aspects of Office Buildings	M.E. Durkin	Space for People Los Angeles, CA 90057
	Development of a Unified Approach to the Design of Window Glass Subjected to Dynamic Forces	J.E. Minor	Texas Tech Univ. Lubbock, TX 79409
	Influence of Non-Structural Cladding on Dynamic Properties and Response of High-Rise Buildings	B.J. Goodno	Georgia Inst. of Tech. Atlanta, GA 30332
1979	Influence of Non-Structural Cladding on Dynamic Properties and Response of High-Rise Buildings	B.J. Goodno	Georgia Inst. of Tech. Atlanta, GA 30332
	Development of a Unified Approach to the Design of Window Glass Subjected to Dynamic Forces	J.E. Minor	Texas Tech Univ Lubbock, TX 79409
	Study of Behavior of Architectural (Non-Structural) Building Components During Earthquakes	S.S. Rihal	Cal State Univ San Luis Obispo, CA
	Seismic Behavior of Precast Curtain Walls in High Rise Buildings	D.C. Perry	Univ of Idaho Moscow, ID 83843
	Seismic Investigation and Design Criteria for Industrial Storage Racks	J.A. Blume	John A. Blume & Assoc. San Francisco, CA 94105
1978	Seismic Investigation and Design Criteria for Industrial Storage Racks	J.A. Blume	John A. Blume & Assoc. San Francisco, CA 94105
	Influence of Nonstructural Cladding on Dynamic Properties and Response of Highrise Buildings	B.J. Goodno	Georgia Inst. of Tech Atlanta, GA 30332
	Unified Approach to the Design of Window Glass Subjected to Dynamic Loads	J.E. Minor	Texas Tech Univ Lubbock, TX 79409
	Seismic Behavior of Precast Curtain Walls in High Rise Buildings	D.C. Perry	Washington State U Pullman, WA 99163
	Seismic Behavior of Precast Curtain Walls in High Rise Buildings	R.L. Sack	University of Idaho Moscow, ID 83843
1976	Building Enclosure and Finish Systems: Design Procedures Considering Interaction of Building Components During Earthquakes	G.M. McCue	McCue, Boone, Tomsick San Francisco, CA
	Seismic Investigation and Design Criteria for Industrial Storage Racks	J.A. Blume	John A. Blume & Assoc. El Segundo, CA 90245

EQUIPMENT

1980	Analysis of Mechanical Equipment Under Earthquake Excitation	S.F. Masri	U of Southern CA Los Angeles, CA
1979	Improving Earthquake Resistance of Power Transmission Substations	A.J. Schiff	Purdue University Lafayette, IN 47907
1978	Improving Earthquake Resistance of Power Transmission Substations	A.J. Schiff	Purdue University Lafayette, IN 47907
1977	Structural Connections in Industrial Installations Subject to Earthquake	G.C. Driscoll	Lehigh University Bethlehem, PA 18015
1976	Assessment of Seismic Design of High-Rise Elevator Systems	K.L. Merz	Ayres and Hayakawa Los Angeles, CA
	Seismic Resistance of Fossil-Fuel Power Plants	J.L. Bogdanoff	Purdue University Lafayette, IN 47907
1975	Seismic Resistant Design of Mechanical and Electical Systems	T.R. Simonson	Simonson Consultants San Francisco, CA 94105

DESIGN DECISIONS/CODES

1980	Introduction of Earthquake Hazard Mitigation Through Multi-Hazard Techniques in Areas of Low Concern for Seismic Risk	J. Loss	Univ of Maryland College Park, MD 20742
	Development of a Model Recertification of Building Process	R. Warburton	U of Miami Coral Gables, FL 33124
	Seismic Design Decisions in the Building Process	M. Brill	Buffalo Org./Soc. Innov. Buffalo, NY 14216
1979	Seismic Behavior and Design of Urban Area Tunnel Linings	S.C. Anand	Clemson University Clemson, S.C. 29631
	Development of Revisions to American National Standard A58-Building Code Requirements for Minimum Design Loads in Buildings and Other Structures	B. Ellingwood	Nat Bureau of Standards Gaithersburg, MD 20234
	Multi Design Approach to Seismic Safety	E.W. Kennett	AIA Washington, D.C. 20006
	The Integration of Seismic Design Principles into Preliminary Architectural Design	K.I. Britz	Caregie Mellon Univ Pittsburgh, PA 15213
	Introduction of Earthquake Hazard Mitigation Through Multi-Hazard Techniques in Areas of Low Concern for Seismic Risk	J. Loss	University of Maryland College Park, MD 20742
	Conference on Urban Design and Seismic Safety at a Cooperative International Level	H.J. Lagorio	U of Hawaii-Manoa Honolulu, HI 96822
1977	Summer Institute for Architectural Design for Earthquake Disaster Mitigation	J.P. Eberhard	AIA Research Corporation Washington, D.C. 20006
	Implementation Planning for Seismic Design Provisions for Buildings	C. Culver	Nat. Bureau of Standards Gaithersburg, MD 20234
	Collaborative Research: Formulation and Expression of Seismic Design	R.N. Wright	Nat. Bureau of Standards Gaithersburg, MD 20234
1976	Optimal Earthquake Design of Energy Production, Storage, and Distribution Systems	A. Freudenthal	George Washington Univ Washington, D.C. 20006
	Formulation and Expression of Seismic Design Provisions	R.N. Wright	Nat. Bureau of Standards Gaithersburg, MD 20234

NON-ENGINEERED/LOW RISE STRUCTURES

1980	Workshop on Seismic Performance of Low Rise Buildings	A.K. Gupta	N.C. State/Raleigh Raleigh, NC 27607
	Earthquake Resistance Cost-Benefits of Factory Produced Houses and Mobile Homes	S. Winter	Steven Winter Assoc. New York, NY 10001
	International Workshop on Earthen Building Materials and Methods in Seismic Areas	G.W. May	U of New Mexico Albuquerque, NM 87106
	Morphology of Tensile and Pneumatic Structural Systems for Seismic Design	S.P. Gill	Abri Inc. Cambridge, MA 02139
1979	Workshop on Seismic Resistance of Non-Engineered Structures	A.J. Gupta	IL Inst. of Tech Chicago, IL 60616
	Dissemination of Earthquake Damage Mitigation Techniques to Homeowners and Renters	I.D. Terner	U of Cal/Berkeley Berkeley, CA 94720
1976	Earthquake Stability of Reinforced Earth Structures	K.L. Lee	U of Cal/Los Angeles Los Angeles, CA 90024

OCCUPANT BEHAVIOR/BUILDING FUNCTION

1980	Methodology for Evaluating and Improving Operational and Non-Structural Aspects of Office Buildings	M.E. Durkin	Space for People Los Angeles, CA 90057
	Methods and Costs of Maintaining Hospital Functions in Earthquakes	C. Arnold	Bldg. Systems Dev. San Francisco, CA 94111
1979	Methods and Costs of Maintaining Hospital Functions in Earthquakes	C. Arnold	Bldg. Systems Dev San Francisco, CA 94111
1977	Seismic Safety Design for Police and Fire Stations	E.W. Kennett	AIA Research Corp. Washington, D.C. 20006

GENERAL ARCHITECTURE

1980	Earthquake Forces on Buildings: A Workbook-Primer for Architects	J.L. Briscoe	Univ of Oregon Eugene, OR 97403
	Improving Productivity in Building Construction	E.S. Townsley	Nat. Academy of Sciences Washington, D.C. 20418
	Fire Loss In Earthquakes	I.J. Oppenheim	Carnegie-Mellon Univ Pittsburg, PA 15213
	Multiprotection Design Summer Institute	D. Sullivan	FEMA Washington, D.C. 20472
	Seismic Institute for Building Officials	C. Healer	Education Dev. Center Newton, MA 02101
	Public Response to Geologic Hazards	T.F. Saarinen	University of Arizona Tucson, AZ 85721
1979	1979 Multiprotection Design Institute; August 6 through August 17, 1979; DCPA Staff College	G.T. Goforth	Department of Defense Washington, D.C. 20301
	Patterns of Housing Density and Type: A Basis for Analyzing Earthquake Resistance	U.P. Gauchat	Harvard University Cambridge, MA 02138
	An Analytical Technique for Establishing Emergency Services Planning Policy: Effects of Building Characteristics on Forecasts of Seismically-Induced Route Blockage	D. Swartz	San Jose State San Jose, CA 95192
1978	A Summer Institute on Multiprotection Design	G.T. Goforth	Department of Defense Washington, D.C. 20301
	Fire Resistance of Epoxy Repaired Concrete Structures	J.M. Plecnik	Cal State/ Long Beach Long Beach, CA 90840
	Summer Seismic Institutes for Architectural Faculty	E.W. Kennett	AIA Research Corporation Washington, D.C. 20006
1977	Building Configuration and Seismic Design	C. Arnold	Bldg. Systems Dev. San Francisco, CA 94111
	Summer Institute on Multiprotection Design	B. Wobbeking	ASEE Washington, D.C. 20036
1976	Summer Institute on Protective Design	B. Wobbeking	ASEE Washington, D.C. 20036
1975	Summer Institute on Protective Design	B. Wobbeking	ASEE Washington, D.C. 20036

Architect's Role In Reducing Earthquake Damage to Buildings	J.P. Eberhard	AIA Research Corp. Washington, D.C. 20006
Workshop on Earthquake Disaster Mitigation by the Architectural Profession	D.M. Wilson	AIA Research Corp. Washington, D.C. 20006

RISK ANALYSIS

1980	Earthquake Safety of Water Distribution Networks Based on the Concept of Balanced Risk	R.T. Eguchi	J.H. Wiggins Co. Redondo Beach, CA
	Tsunami Risk Analysis	C.C. Tung	N.C. State/Raleigh Raleigh, N.C. 27607
	Underground Lifelines in a Seismic Environment - Phase II	M.L. Baron	Weidlinger Assoc. New York, NY 10022
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EXISTING, HAZARDOUS
BUILDING PROBLEM

ASEISMIC EVALUATION OF BRICK STRUCTURES

Gong Yongsong[I]

ABSTRACT

The evaluation of earthquake resistant performance of the brick structures is a critical problem in design of strengthening for existing buildings.

The principles and methods for evaluation of existing brick structures in this country are described in this paper. Some problems which need to be further studied in the near future, such as the safety of evaluation for the whole structure under strong motion, are also indicated in this paper.

INTRODUCTION

In many areas of the world, people started to build brick houses thousands of years ago. Up to now many buildings are still being built with brick masonry, although steel, reinforced concrete materials etc. have widely been used. Because the brick masonry has some advantages, for instance, it can be available in local places, convenient for manual operation, and low construction cost. For the advantages mentioned above, brick masonry will be still a main building material in some countries and regions for quite a long time. But brick masonry also has many disadvantages : such as lower shear strength, especially during the earthquake, such material has poor ductility etc. For reasons given above, the damage ratio of this structure were rather high in the past earthquakes.

During Tangshan earthquake of July 28, 1976, more than 90% of brick houses in Tangshan city were collapsed or severely damaged. However, many engineering damage observation reports in China show that, under the attack of strong earthquake, some buildings were not severely damaged or collapsed in the area with intensity of 7, 8, and 9. Particularly some buildings, which had been designed or strengthened for earthquake resistance could basically remain intact or only slightly damaged. It shows that, brick structures can be adopted in seismic regions provided aseismic measures are taken in design. There are a lot of buildings in the world, especially in the third world countries, such buildings are being used and will be continually used for quite a long time. In order to safegard the human life and property, it is quite necessary to make evaluation of earthquake performance and earthquake damage estimation and strengthening of existing buildings. Some other old memorial brick buildings also need evaluation and strengthening to prevent damage during the strong earthquakes to be occurred in the future.

Aseismic evaluation for brick structure means to evaluate the safety reliability of the building in the area with specified intensity for

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evaluation and to work out the strengthening measures for critical components of the building. It shows that aseismic evaluation is a principal step in earthquake disaster estimation and building strengthening.

BASIC PRINCIPAL OF ASEISMIC EVALUATION

1. Estimate of earthquake risk is based on the earthquake intensity zone map of China. While evaluating, the intensity adopted is in conformity with "Aseismic Design Code for Industrial and Civil Buildings" (TJ 11-78) (here in after referred to as "Aseismic Code") (1).

Evaluation will be done in the areas with intensity of 7 and above. The intensity for evaluation is determined according to the importance of the buildings. For ordinary buildings, it is equal to basic intensity. For the exceedingly important buildings, the intensity for evaluation should be raised one grade higher. For less important buildings, the intensity for evaluation can be lowered by one grade. However, it shall not be lowered when its basic intensity is 7.

2. The evaluation criterion of aseismic performance of buildings is based on the "Aseismic Criterion for Evaluation of Industrial and Civil Buildings" (TJ 23-77) (here in after referred to as "Evaluation Criterion"(2), which allows some damage to the buildings which need repair after an earthquake, but the damages will not endanger human life and the valuable production equipment. "Aseismic Code" not only safeguards human life and valuable production equipment but also insures that the buildings can be still serviceable without repairing or with moderate repairing after earthquakes. With the comparision of these two criteria, we could see that the safety criterion of buildings of "Evaluation Criterion" is lower than that of "Aseismic Code". The reason to allow this difference is that it's uneconomic or difficult to strengthen the existing building up to the requirements in the "Aseismic Code".

3. The concrete requirements of technical measures of "Aseismic Code" is higher than that of "Evaluation Criterion"or equal to it, because the safety of "Aseismic Code" is higher than the safety of "Evaluation Criterion". On the other hand, new buildings designed according to "Aseismic Code" would naturally satisfy the requirement of "Evaluation Criterion", and there is no need to be evaluated.

4. For some buildings which can be used at present, but their aseismic capacity is low, and strengthening of them is uneconomic, the following measures should be taken:

A. Taking possible measure of strengthening to improve aseismic capacity of the building.

B. Such building should be used as the less important building accomadating very few people.

C. Taking safety measures for people and equipments or evacuate them before earthquake.

PROCEDURE OF ASEISMIC EVALUATION AND STRENGTHENING

The procedure of aseismic evaluation will be done according to flow chart shown in Fig.1.

THE MAIN CONTENT OF TYPES OF BRICK BUILDINGS AND ASEISMIC EVALUATION / STRENGTHENING

The area with intensity of 7 and above is about 3 million square kilometers in China, in which there are various types of brick structures, following are 5 main types.

1. Multistory brick building, including framed first story buildings, inner frame with columns in multirow or single row.
2. Spacious brick building.
3. Single story factory buildings with brick columns.
4. Brick chimneys.
5. Brick-stacks or columns-supporting water towers.

The various types of brick structures mentioned above show that the common characteristic of these structures is that brick walls or columns are vertical load bearing elements of building as well as lateral force resistant element, the building would collapse, once these elements are damaged. Brick is a fragile material, the shear, bending and tensile strength of which are far less than compressive strength, so the safety of such structure may be adequate if there is no earthquake, but it would be very low, once an earthquake occurs. So that the main task of aseismic evaluation for such building is to evaluate the earthquake performance of brick structures by means of checking computation of aseismic strengthening and evaluating the structural system and elements. The former emphasizes the analysis of aseismic calculation theory, while the latter stresses the summary of seismic experience of such structure, specific contents are shown as follows.

1. The mark of brick wall/column
2. The ratio of the horizontal cross sectional area of aseismic walls to the floor area of a building or brick column strength
3. The thickness and space of the aseismic walls
4. Height of multistory brick building
5. The integrity of building (integrity of building and roof, position and number of girth joint between longitudinal and transverse walls, joint between various elements)
6. The sectional elements of brick structure, such as gable wall, parapet wall, small on-roof chimney, etc.
7. Other elements, such as wood elements, reinforced concrete elements etc.

CHECKING COMPUTATION OF ASEISMIC STRENGTH

Basic theory of aseismic strength checking computation for brick structure is the same with "Aseismic Code", which adopts theory of elastic response acceleration spectrum and shear beam model for multistory brick

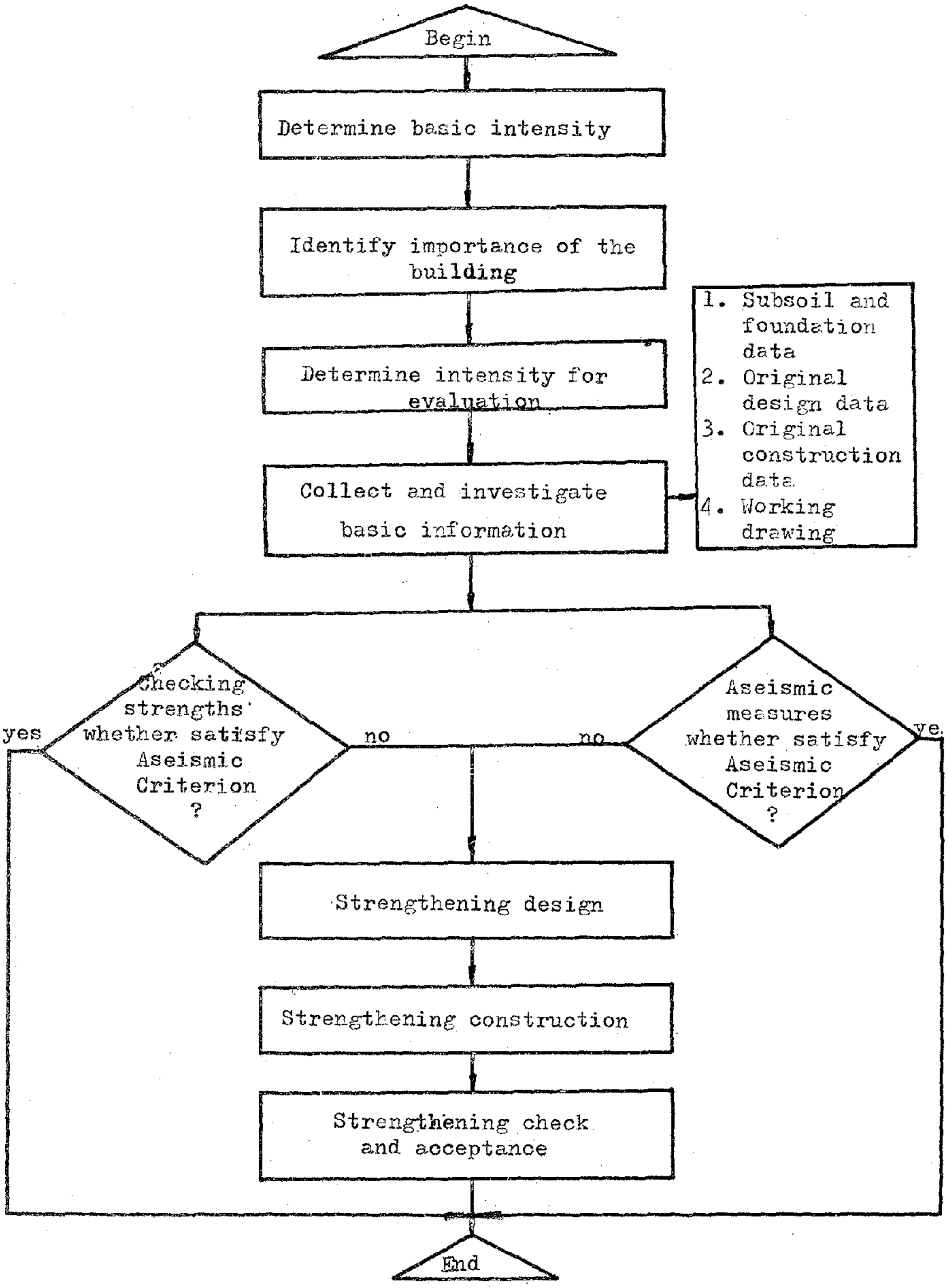

Figure 1. Flow chart for Aseismic Evaluation

building. Owing to reasons mentioned in previous paragraphs, safety coefficient of aseismic checking computation used in "Evaluation Criterion" is generally lower than that in "Aseismic Code". When basic intensity is 7, the safety coefficient would not be lowered. When the intensity for evaluation is 8 or 9, the safety coefficient of multistory brick building would be lowered by 30%, the safety coefficient of all frame on first story and multistory inner frame would be lowered by 15%. The determination of this degraded coefficient was also gained from the aseismic experience of brick building in recent earthquakes occurred in China.

Safety coefficient of brick building for evaluation or strengthening is given in Table 1.

Table 1 Safety coefficient of brick building for evaluation and strengthening

Element	Type of stress	Intensity		
		7	8	9
Wall of multistory brick building	Shear stress	2.0	1.4	1.4
Wall of inner frame multistory building	Shear stress	2.0	1.7	1.7
Wall of brick-wood building	Shear stress	2.0	1.6	-
Brick column of single story building	Eccentric compression stress	1.7		
Brick-stacks-supporting water tower or chimney	Eccentric compression stress	1.84		

There are a lot of multistory brick buildings in China. In order to facilitate aseismic strength checking computation of brick wall, "Evaluation Criterion" provides values and formula of minimum area ratio $(A/F)_{min}$ of the brick building up to six stories, including all frame on first story, and multistory inner frame. The formulas are as follows:

$$\left[\frac{A}{F}\right]_{min}=\frac{K\xi C\alpha_{max}\left(\frac{2}{n+1}\right)w}{\sqrt{R_j(R_j+\sigma_0)}}\sum_{i=j}^{n} i \qquad (1)$$

Where

k=Factor of safety

ξ=Coefficient of non-uniform distribution for shear stress at a cross-section

A=Total horizontal cross-section area of all walls

F=Building area corresponding to the jth floor

C=Structure related coefficient

α_{max}=Maximum value
n=Number of stories of the building
w=Weight of building per unit floor area
R_j=Principal tensile strength of brick masonry
σ_0=Average compresive stress on brick wall to the jth floor

$$\sigma_0=\frac{h\gamma(n-j+1)}{10^4(1-\zeta)} \qquad \text{for non-bearing wall}$$

$$\sigma_0=\frac{(bh\gamma+q)(n-j+1)}{10^4 b(1-\zeta)} \qquad \text{for bearing wall}$$

h=Average height of building story
b=Net thickness of aseismic wall
q=Total load on floor per unit length
γ=Weight of brick masonry per unit volume
ζ=Ratio of the door or window width to the wall length

The strength checking of aseismic evaluation would be greatly reduced due to the calculation method and some diagrams given above. With the same purpose, the "Evaluation Criterion" provides evaluation and strengthening requirements of brick chimney which are widely adopted in many cities of our country. The requirements are shown in Table 2.

Table 2 Requirements of Brick Chimney for Evaluation or Strengthening

Height (m)	Intensity for Evaluation	Site soil	Steel strips: Vertical	Steel strips: Circular	Reinforced concrete: Vertical Reinf.	Reinforced concrete: Circular Reinf.
30	7	I, II	8-8×60	-6×60 @2000	φ8@300	φ6@250
		III				
	8	I, II				
		III	8-8×80		φ14@300	
	9	I, II				
40	7	I, II	8-8×60	-6×60 @2000	φ10@300	
		III				
	8	I, II				
		III	8-8×80		φ14@300	
	9	I, II				
50	7	I, II	8-8×80	-6×60 @1500	φ12@300	
		III				
	8	I, II				
		III	12-10×80		φ16@300	
	9	I, II				

It is very important to provide a simplified evaluation method, and it is also a principal aspect to judge whether the practical value of a evaluation criterion could be widely applied.

SOME PROBLEMS WHICH NEED FURTHER STUDY

The basic theory of aseismic strength technique of brick structure will depend on understanding of dynamic behaviour of brick material, elements and structures. It is very difficult to accurately determine a complete aseismic evaluation technique for brick structure, because physical and dynamic behaviour of brick structure are very unstable due to material and construction condition, further more it is difficult to make dynamic experiment for it belongs to fragile material. So aseismic evaluation of brick structure mainly depends on lessons learned from destructive earthquakes. This problem will need to be further studied. Besides these, the author would raise following problems for discussion and research.

1. The aseismic behaviour evaluation of brick building must start from building integrity. The methods provided by "Evaluation Criterion" are mostly for each element of the building, so it can not reflect the safety concept of building integrity during an earthquake.

2. Basic intensity in China is supposed the largest intensity in certain region within future 100 years, but it does not mean that every earthquake will influence this region with the same intensity. Therefore, the evaluation of existing building should be based on the various aseismic safety, and some measures of strengthening should be taken.

3. Aseismic Evaluation is the answer to the questions of whether the building could meet the aseismic requirements? whether strengthening is needed? how to meet the requirement of strengthening after aseismic behaviour judgement of building, it is not only a technical matter, but also a matter of safety determination and economic analysis. Therefore, the determination of some principles of the aseismic evaluation should be established on the basis of comprehensive analysis of safety and economy, so that the evaluation may be more reasonable in technology and economy.

4. Because many existing buildings often have been built without aseismic consideration in original design in some cities or regions, large amount of buildings would need aseismic evaluation, which requires either accurate theoretical basis or simplified methods of calculation. In addition, the adoption of classification evaluation may also be studied according to importance of buildings and structures.

ACKNOWLEDGEMENT

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BUILDING REHABILITATION STRATEGIES IN SEATTLE, WASHINGTON

by

Neil M. Hawkins [I] and J. F. Stanton [II]

ABSTRACT

Knowledge on seismicity in the Puget Sound Region and its effects on buildings and building rehabilitation strategies is summarized. Details of the philosophical considerations dictating the rehabilitation strategies for two buildings, and details of those strategies are given. For one building, Grand Central On the Park, the strategies are typical of those adopted for creative seismic retrofitting of commercial turn-of-the-century masonry structures. For the second building, The Olympic Hotel, the strategies are typical of those adopted recently for government-owned historic structures.

INTRODUCTION

Seattle is the most populous city in the Pacific Northwest of the United States. It is a young city and therefore the designers of many of the older buildings still standing had few earthquake records to influence their choice of lateral load resisting systems. The first European settlers arrived in Seattle in 1852. By 1861 when the University of Washington was established, the population was 302, and by 1890 when statehood was granted the population was about 50,000. Growth in the 1880's was spurred dramatically by the arrival of the railroad and the provisioning of prospectors headed for the goldfields of Alaska. Today, the population of the greater Seattle-Tacoma area is more than two million. Seattle is a major port for trade with the Far East, a center for the timber industry and the home of the Boeing Aircraft Company.

Many of the older masonry buildings in downtown Seattle date from the 1890's and the citizens of the city have repeatedly demonstrated their desire to retain such older structures as symbols of their cultural and social heritage. Accordingly, since the 1970's the city government has actively pursued a program of encouraging preservation and rehabilitation of older buildings. Two types of city ordinances, as well as a general philosophical

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attitude in the City's Building Department, affect directly rehabilitation strategies. One type of ordinance establishes Historic Preservation Districts. There are six such Districts each covered by a separate ordinance. The first such ordinance in 1970 covered Pioneer Square, the original center of Seattle. Pioneer Square is an area of about 25 city blocks that contains many of the four- to five-story masonry buildings that were built after the major fire of 1889 destroyed the previously predominate wooden structures. For such Districts a Historic Preservation Board is established (1) and structures within the District cannot be altered, demolished, restored, etc., without a permit issued by the City Superintendent of Buildings and approved by the Historic Preservation Board. Generally boards are concerned with retaining the exterior architectural features of existing buildings and the historic consistency of the District. The second type of ordinance established a Landmarks Preservation Board that recommends to the City Council structures to be designated as "Landmarks" because of their architectural and/or historic significance. Landmarks, so designated, become monuments scattered throughout the city and subject to similar alteration criteria as buildings in Historic Districts.

Since 1973 Washington State law has mandated use of the Uniform Building Code (3). Specific provisions of that Code can only be superseded by City Ordinances more stringent than the corresponding provisions of UBC. Section 104 of UBC deals with application of the Code to Existing Buildings. A revised City ordinance permits the Building Official to work with designers on an individual basis on the rehabilitation of Landmark and Historic District buildings. Thus, the structural engineer need not bring these buildings up to the same life safety standards as new buildings. Since the development of this revised ordinance, the Building Department has adopted the philosophy that building rehabilitation is more of an art than a science and that generally it is inappropriate to utilize code provisions intended for new buildings for the rehabilitation of older buildings (4).

This paper describes the rehabilitation procedures utilized for two structures, Grand Central On the Park in Pioneer Square and the Olympic Hotel. Grand Central was one of the first buildings rehabilitated under the City's Revised Section 104 ordinance. The rehabilitation philosophy and procedures adopted for that structure are typical of those subsequently utilized for creative solutions to the seismic retrofitting of historic privately owned commercial structures. The Olympic Hotel is a Landmark building owned by the University of Washington. The philosophy and procedures adopted for that building are typical of those utilized more recently for government owned historic structures.

EARTHQUAKE ACTIVITY AND SOIL CONDITIONS

For any rehabilitation project, an understanding of prior seismic history and soil conditions are important. Several writers (5,6) have reported the Seattle region's characteristics. Earthquakes have been recorded in the region since the first European settlement was established in 1833. There have been major quakes in 1872, 1949 and 1965. While researchers have variously placed the epicenter of the 1872 quake over a wide area of the State, the epicenters of the 1949 and 1965 quakes were undoubtedly in southern Puget Sound. Between 1820 and 1857, three of the five volcanoes in Washington State were active. However, until the 1980 eruption of Mt. St. Helens, that fact was conveniently neglected (7).

Soil conditions in the Puget Sound Region have been determined by at least three periods of glaciation and deposition during the Quaternary Period. Resulting glacial and glacial-fluvial deposits cover most of the Region and have depths of over 1,000 meters in the Seattle area. The glacial deposits have effectively prevented any detailed study of the geological structure of the area and that structure has had to be inferred from seismograph records. Since 1970, a network of short period seismographic stations has been maintained in Western Washington. From those records, it has been shown that the geophysical picture of Fig. 1 is appropriate. Between the offset San Andreas and Queen Charlotte faults, there are a series of ridges and fracture zones. Approximately 500 km west of Puget Sound there is a zone of spreading termed the Juan de Fuca ridge. That ridge is pushing a small portion of the earth's surface, the Juan de Fuca plate, beneath the North American plate at the rate of about 3 cms per year. The resulting subduction zone extends downwards in an easterly direction beneath Washington State and creates a north-south chain of volcanoes that extends from northern California to southern Canada. The large magnitude 1949 and 1965 quakes were undoubtedly caused by that subduction. They had deep focal depths of about 40 and 60 kilometers, respectively. Approximate epicenters are shown on Fig. 2. The exact locations are, however, in a sense unimportant due to the deep focal depths. The whole Puget Sound Region is badly shaken by such earthquakes. Geophysical studies indicate that this mechanism can cause earthquakes of Richter magnitude 8 about once in 800 years and earthquakes of magnitude 6.5 about once in 60 years. The geophysical mechanism involved differs from that for California quakes where shearing actions occur along major fault lines and where the energy release is likely to be at the earth's surface and cause near field effects. The deeper focal depths for Washington quakes result in their effects being distributed over a wider area of the earth's surface than California quakes and in there being greater energy dissipation before the shock waves reach the earth's surface.

Small earthquakes identified by the Western Washington seismograph network during a six-month time period are also shown in Fig. 2. Those quakes have a diffuse distribution in the central basin consistent with the geophysical picture of Fig. 1. However, there is also a definite alignment of the small earthquake data on NNW axis extending from Mt. St. Helens to the south end of Puget Sound and indicated by the broken line in Fig. 2. That alignment is also confirmed by radar imagery and appears to be a fault in the making. Precise measurements taken since St. Helens eruption show locations to the west of that line are moving northward with respect to locations about 3 miles to the east. Geophysical studies indicate that surface earthquakes of magnitude 6 are likely on that line. The 1971 San Fernando earthquake was of magnitude 6.4.

Even as late as 1946 there was little recognition by building authorities of the earthquake risk in Puget Sound. UBC 1946 placed Western Washington in the lowest intensity seismic zone. Two mild shocks in 1945 and 1946 caused officials to upgrade the area to a zone of moderate risk. Since 1949, the area has been recognized as a zone of high risk and now it is clear that at least two geophysical mechanisms can cause earthquakes and that the form of the resulting risk differs from that in California.

THE 1949 AND 1965 EARTHQUAKES

At Olympia, the city closest to the 1949 epicenter, the destructive phase of the quake lasted about one minute. The maximum ground acceleration was about 0.3g and the maximum ground displacement about 1cm. In Seattle, corresponding figures were 0.1g and 0.3cm. Eight people died due primarily to falling parapets and gables. Most damage was to structures located on low-lying soft-land areas or fill along the Sound and adjacent buildings. In the Pioneer Square area of Seattle, most of the damage was to the masonry buildings on timber piles driven into fill material. Buildings on more solid ground suffered only facing cracks or failures of parapet walls. In general, damage was greatest for older masonry buildings three to five stories in height.

The 1965 earthquake was characterized by a large zone of intensity VII and two small pockets of intensity VIII centered in the Seattle area. Maximum ground accelerations were about 0.2g and displacements about 2cm. Some of the more spectacular damage was difficult to evaluate since it occurred to structures in which it was obvious that damage caused by the 1949 earthquake had either not been discovered or not been repaired. Again, major damage in downtown Seattle was to older three to five story masonry buildings located on poor soil conditions.

In both the 1949 and 1965 earthquakes, there was little fore or after shock activity, no evidence of surface faulting except due to soil settlement and damage was associated primarily with vibration effects. It is now accepted that in Seattle there are no major geologic factors to be considered and that there should be microzonation for anticipated earthquake effects based primarily on soil conditions.

In applying its revised Section 104 of UBC, the Seattle Building Department relies heavily on the performance exhibited by an older building in the 1949 and 1965 earthquakes. If that performance was satisfactory, no specific standard is applied for rehabilitation. Both Grand Central and the Olympic performed well during the 1949 and 1965 earthquakes. Therefore, the attitude of the Building Official has been that those full scale tests are more meaningful than the application of mathematical formulas developed for new structures with greater predictability of performance. However, this reliance in prior performance means that any changes in the building's lateral resistive characteristics are discouraged by the Building Official.

GRAND CENTRAL ON THE PARK

The Grand Central is located on the site of Seattle's first major theatre. The Squire Opera House was a wooden structure built in 1879 and operated as an entertainment center for shows, recitations, magic, opera and drama until destroyed by the fire of 1889. The current masonry building was erected in 1893. It is believed to be the work of architect Elmer Fisher who is said to have built over 100 similar structures in Seattle following the 1889 fire. Grand Central was initially office space for the Domestic Steam Heat and Lighting Company until converted to a hotel in 1895. Renamed the Grand Central in 1899, it continued to be listed in directories as a hotel until 1968. Intended originally as an establishment for wealthy gold miners, it became a cheap hotel as strikes petered out and Pioneer Square became disreputable in the early 1900's. By 1928, the hotel operation was confined to the upper floors and the ground floor housed a tavern, pawnbrokers and men's inexpensive clothing store. The building was acquired by architect Ralph Anderson and developer Alan Black in 1971 for $220,000. They completed its rehabilitation at a cost of $1,500,000 in 1973. Little difficulty was encountered with Building Officials in application of Section 104 because the occupancy was downgraded to specialty shops and restaurants on the basement and ground floors and offices on the second through fourth floors. As the first structure rehabilitated in Pioneer Square, Grant Central was instrumental in drawing the attention of Seattle's citizens to the possibilities for making old buildings as amenable as new.

An isometric view of the structure is shown in Fig. 3. Its architecture is best described as Romanesque-Victorian. Originally the building was five stories. With the regrading of downtown Seattle to eliminate some steep slopes, the sidewalks were raised to the level of the second floor and the building became four stories and a basement. On the north side, there is a party wall shared with the adjacent building. The east side opens on to a cobbled square and the south and west sides onto sidewalks along adjacent streets.

Although the Grand Central rode out the 1949 and 1965 earthquakes with little damage, the building, and especially the lime mortar between the bricks, was in poor shape when rehabilitation was commenced in 1971. The building is on spread footings on an old N-S spit of glacial till at the edge of Puget Sound. Fill material lies to the east and west of the building. Similar adjacent buildings supported on timber piles driven through that fill material suffered significant damage in the 1949 and 1965 quakes. The mortar in the buildings is soft enough that the effective period of the structure is lengthened over that for a similar height cement mortar building. Apparently lime mortar buildings of the height shown had roughly the same period as their pile foundations and the resulting resonance contributed markedly to their damage.

In addition to the exterior walls, there were two interior E-W walls extending over the full height of the buildings and three other E-W walls discontinued at the second floor level. Those walls were 21 in. thick in the basement, 17½ in. thick for the first through third floors and 13 in. thick in the top floors. There was one interior N-W wall discontinued at the second floor level. The main vertical members of the envelope had thicknesses similar to the interior walls and were supported on brick pillars. The intermediate vertical members were cast iron columns from the basement to the second floor and 13 in. thick masonry thereafter. The building's floors were generally 1x8 in. sheathing supported on 2x12 in. beams at 2 ft. centers. The beams were supported by recessing their soffits approximately 3 in. into the masonry walls. There were two stairways in the building; a large central one exiting through the main entrance on the west side and a smaller one exiting through the opening in the center of the south side. With the many perforations in the exterior walls, the strength of the building was obviously inadequate in the N-S direction. Further, there were no elements present in the building that would be recognized as earthquake resistant by modern building codes.

There were three elements to the rehabilitation: architectural, fire and structural. In the architectural rehabilitation, the exterior masonry was completely refurbished and much of the interior of the building gutted and rebuilt. An arcade was cut through the center of the building from the main entrance in an E-W direction, a large arched fireplace built at the ground floor level, an elevator added and open wells used to bring light into the center of the building. The architectural rehabilitation costs were $1,275,000.

The main elements of the fire rehabilitation were the introduction of a third stairwell extending over the height of the building on the northeast side and the encasement in concrete of some of the interior cast iron columns at the basement level. That concrete was then faced with bricks. The fire rehabilitation costs were $75,000.

With virtually no basis on which to estimate the seismic resistance of the building, the structural engineer Dean Ratti opted for an overall rehabilitation philosophy rather than a procedure that would provide lateral load resisting elements with known strength and ductility. The philosophy was to tie the elements of the building together by:

(1) providing a new plywood overlay to the existing floors and roof,

(2) connecting the floors and roof to the exterior and interior masonry walls by metal straps, and

(3) adding a continuous reinforced concrete bond beam to the top of all walls extending through the roof.

Four considerations were instrumental to the development of that philosophy: stiffness, creep, legal and cost. When new stiff elements such as reinforced concrete shear walls are added to older buildings, those elements attract and concentrate inertia forces. Therefore to avoid undesirable effects, those elements have to be added to almost every existing masonry wall of the structure. Old lime mortar brick creeps more than stiff new brick or a shotcrete reinforced concrete shear wall. Therefore, if either of those elements are used to face an old lime mortar wall, buckling can occur in the new thinner element. Grand Central shared a north facing wall with a similar height adjacent building. The metal strips, described previously, could not be attached to that wall for legal reasons. Similarly, if the Grand Central was strengthened or stiffened in such a manner that its period was changed markedly relative to that of the adjoining structure, then there could be severe legal implications if the adjoining structure suffered significant damage in the next earthquake. Finally, the cost of rehabilitating the structure by the methods adopted would be considerably less than the cost of rehabilitating it through the addition of shotcrete walls. The seismic rehabilitation costs were $150,000, approximately one-tenth of the total rehabilitation costs. Net income from the structure after depreciation and taxes is $125,000 and it is anticipated that the pay-back period to recoup the initial investment in the structure will be 12 years. If the more expensive shotcrete approach had been used, the greater seismic rehabilitation costs would have probably increased the pay-back period to in excess of 20 years. Financing of the rehabilitation would have been much more difficult to obtain.

Details of the metal straps used to connect the floors to the walls are shown in Fig. 4a and details of the bond beams and straps at the roof levels in Fig. 4b. The 4 ft. long ¼ in. thick metal straps were recessed into the sheathing and then 20d nails driven through the 3/8 in. plywood into either the existing joist or into new solid wood blocking inserted between the joists. The metal straps which were 4 ft. on-centers were welded in place to an 8x4x ½ in. angle attached 4 ft. on-centers to the masonry wall by 3/4 in. diameter mild steel bolts that extended through the depth of the wall and were anchored by a plate and bolt on the opposite face of the wall. At the roof level, the 8x16 in. bond beams were reinforced top and bottom by 2 No. 4 bars, faced with brick and anchored vertically into the wall by 3/4 in. diameter 2 ft. long, dowels inserted at 4 ft. centers in 2½ in. diameter grout holes. The dowels passed centrally through the bond beam and the end of the metal strap. The straps cost approximately $60 per 4 ft. module in 1971. That cost rose to about $120/module in 1975, but has now dropped back again to $60/module in 1981 as contractors have become more familiar with their installation.

In addition to tying the building together, the engineer provided two new 2x4 stud "shear" walls in the N-S direction. Those walls were faced on one side with 3/8 in. plywood connected with 8d nails spaced 2½ in. on-centers around their edges to the studs. Those walls were in turn bolted and nailed to the floor and bolted, in a manner similar to the metal straps, to the masonry walls. These plywood shear walls were intended only as a second line of defense in the structure and were carefully specified in order that future users may think twice before removing them.

Although this method of tying the structure together is widely accepted for older buildings, there is almost total ignorance on how such buildings will behave and what earthquake resistance can be associated with them. Obviously reliance is still being placed on the shear strength of the brick walls and in such cases in situ tests to determine the strength and lateral load stiffness, characteristics of such walls seem highly desirable. Then, a specific level of seismic resistance could be associated with the structure and management techniques such as special protection of exits and entrances or provision of shelters used where that level is inappropriately low compared to the forces expected for the 60-year return period earthquake.

THE OLYMPIC HOTEL

Background: The Olympic has been for many years the most prestigious hotel in Seattle. It is located on a 10-acre Tract that was the original site of the University of Washington. Since 1904 the University has leased parcels of the Tract for long periods in a policy aimed at generating the maximum long-term revenue consistent with sound long-range development of the Tract. Buildings constructed on the Tract by leaseholders revert to the University at the end of the lease.

The first stage of the Olympic was built in 1923 under a lease expiring in 1954. In 1952 that lease was renegotiated and Western International Hotels signed a 42-year agreement that also severely restricted their rights to develop other hotel holdings in Seattle. Although they expended considerable monies on modernizing the Olympic, by 1978 Western International was finding operation of the Hotel a losing proposition and they indicated to the University their desire to terminate the lease as early as 1979.

The University examined various possible options for the Olympic site in the 1980's. Those options included extensive remodeling, major additions, and demolition and replacement. There was overwhelming public opposition to demolition. The Hotel construction had been financed primarily through bond sales to the public and the grandeur of the public rooms, remembered by many, remained essentially untouched. Foes of demolition succeeded in having the Olympic placed on the National Register of Historic Buildings and designated as a Landmark by the Seattle City Council. The latter action resulted in litigation between the City and the University as to which of those public bodies had final authority over developments on the site.

In 1978 the University solicited proposals for renovation and subsequent operation of the Olympic. There was only one respondent and their financial terms were not satisfactory. Inquiries showed that several other organizations were interested in responding but had held back because of uncertainties in seismic rehabilitation requirements for the structure. Those requirements affected significantly the economics of any proposal. The issue was further complicated by the Landmark litigation. The Building Official for the City of Seattle maintained that Section 104 of their Code gave him the authority to determine what seismic rehabilitation was necessary. By contrast the University, as a State authority, felt obligated to require proof of some known level of seismic resistance for the rehabilitated structure.

In May 1979 the University decided to issue a second call for proposals to rehabilitate the Olympic and included with that call a preliminary assessment of the seismic safety of the Hotel by the authors. That assessment (8) detailed the nature of the lateral load resisting elements in the Hotel, recommended a level of seismic resistance to which the Hotel should be upgraded, and identified possible strategies for that upgrading. The University was particularly sensitive to the issue of seismic risk because insurance premiums for buildings on the Tract are tied to that risk (8,9). The authors assisted the University in evaluating the three proposals it received and later worked with the architect and structural engineers for the developer on conceptual aspects of the seismic rehabilitation procedure.

Original Structure: Typical plan and sectional views of the original structure are shown in Fig. 5. The Hotel was built in three phases. The bottom three floors of the U-shaped structure, the upper floors over the center of the U and the northwest wing were built in 1923. Floors 4 through 11 of the northeast wing were added in 1929 when additional financing became available. A theater located in the center of the U was demolished in 1955 and replaced by a three-level entrance facility. All three sections of the Hotel were designed by different architect/engineers.

The lower four stories were constructed around moment-resistant steel frames. The steel was generally I, channel or built-up sections connected by large gusset plates, knee braces and rivets. The steel was of good quality with a minimum yield strength of 30 ksi. The steel columns were connected to reinforced concrete spread footings, some of which were reinforced with grillages. The footings were founded on deep glacial till. Fireproofing was provided by encasing the steel columns in concrete. The panels between steel columns were concrete shear walls below the lobby level and unreinforced masonry walls above that level. The lobby level and the adjoining northwest wing on the same floor were open and spanned by trusses or plate girders. There were few lateral load resisting elements at the lobby level and the eccentricity of the center of mass of the building from its center of resistance at the level was large.

The upper floors were constructed of reinforced concrete with moment-resisting frames in the longitudinal direction and concrete shear walls at the north ends of the east and west wings. Those walls stopped at the lobby level steel frame. The interior partitions were unreinforced clay tile and the exterior walls were unreinforced masonry not tied to the concrete frame. The concrete shear walls and floor slabs had less reinforcement than that needed to develop the tensile strength of the concrete.

The Hotel was a very heavy structure with average dead loads of 160 psf per floor. Design live loads were small being 40 psf for the upper floors, 100 psf for assembly areas and 20 psf for lateral loads. Based on those design values, the lateral strength of the load resisting elements according to modern design concepts was only 1.3% of gravity. That value was well below the 9% required for the Seattle area by 1979 UBC.

The ornamental work such as cornices and balconies on the outside of the structure was not well secured to the building and, in several instances, part of the restraining steel work had rusted away. Those elements represented a severe potential hazard to pedestrians on the sidewalks surrounding the building.

The Hotel's seismic resistance was severely deficient by today's concepts. There was an incomplete lateral load resisting system, widespread use of brittle materials including inadequate reinforcement in the upper walls and floors, a weak and flexible main story at the lobby level, discontinuous shear walls, ornamental cornices, etc., not properly tied to the main structure and exterior masonry walls not tied to the main frames. The University recognized those deficiencies as potentially contributing to a dangerous seismic condition despite the Hotel's satisfactory performance in the 1949 and 1965 earthquakes. They concluded that the past good performance could not be interpreted as meaning the same performance would be repeated in future earthquakes. Another earthquake of the same severity as the 1949 quake could cause a vastly different response should any of the characteristics of that earthquake (direction of approach, frequency content, location of epicenter, etc.) be different from those for the 1949 quake, or any of the interior partitions of the structure be removed during the renovation. Second, the masonry which must have provided most of the resistance in previous quakes may have been weakened by those actions, by moisture, temperature and other weathering effects.

Recommended Remedial Actions: Altering the Hotel to satisfy fully the 1979 UBC requirements would have been difficult, would have destroyed some of the more noteworthy architectural aspects of the building, and would have been financially unattractive to a developer. The authors recommended to the University that the calculable seismic resistance of the Hotel be upgraded to a level intermediate to that required by UBC 1979. They recommended provision of a lateral resistance equal to at least 5% of gravity. That value was the design strength that would have been required by UBC 1973. Further, it represented the limit beyond which diaphragm action would cause cracking in critical locations in the inadequately reinforced concrete floors of the upper portion of the structure. The concept of upgrading to an intermediate aseismic design level was accepted by the Seattle Building Official. The authors suggested that the upgrading be accomplished by a combination of mass reduction and the addition of strengthening elements such as shear walls. However they further noted that with careful examination, experimental testing preferably of the insitu walls, and the addition of certain strengthening elements in regions of weak construction it might be possible to improve the seismic characteristics of the Hotel to an acceptable level without major structural changes. When the results of the authors' study were presented at an NBS-NCSBCS Conference (8), it was generally agreed that it was not feasible to bring the Hotel up to UBC 1979 standards and that strengthening to an intermediate level was the best technical solution. However, substantial concern was also expressed that by adopting such an approach, any structural engineer could be assuming considerable liability. Several attorneys suggested that the City's Landmark ordinance would provide only limited immunity to the structural engineer.

In 1979 the University issued a second call for proposals for renovating and operating the Olympic. Three submissions were received. For two of the proposals the seismic renovations suggested were similar. The Hotel was to be upgraded by adding reinforced concrete shear walls. The third proposal suggested that reliance be placed on the prior satisfactory performance of the Olympic in the 1949 and 1965 earthquakes and that only minor structural changes be made. It was concluded that considerable analysis and possibly limited experimentation would be needed to justify that approach.

Remodel: The Four Seasons' proposal, with the addition of shear walls, was accepted. Four Seasons was to spend $32 million in architectural renovations and the costs of seismic rehabilitation was to be shared with the University. The number of guest rooms was to be reduced by 40%, the main entrance relocated and a garden court and health club added. The building was to be brought back to a state reminiscent of the elegant twenties and luxuries provided similar to those people expect in the best European hotels. Renovations are now nearing completion and reopening of the Hotel is scheduled for May 1982.

Four Seasons' initial seismic rehabilitation concept called for the addition of reinforced concrete shear walls through the depth of the wings and along the inside of the exterior masonry walls facing the old theater location. The positions proposed for those walls are shown in Fig. 6 by thick lines. From the basement through to the bottom of the reinforced concrete frame (third floor), the walls were to be 10 in. thick and reinforced with No. 5 bars at 6 in. centers in both directions and for increasing story levels, shear wall details changed as indicated on Fig. 6. Between the ground and third-floor levels where they were large 14 ft. high arched windows in the exterior masonry walls, an interior framework of relatively heavy steel sections was to be fabricated and welded to the existing steel frame. The masonry was to be tied to the steel framing by ½ in. diameter 16 in. long anchors spaced 24 in. on-centers. For the third floor and above where there was a concrete frame, the shear walls were to be tied by 1¼ in. diameter 12 in. long expansion anchors to the existing concrete beams and columns. Anchors were to be spaced 10 in. on-centers in 10 in. walls, 18 in. in 8 in. walls and 24 in. in 6 in. walls. Anchors were to be placed around the complete perimeter of each shear panel and the masonry tied back to the concrete wall by ¼ in. diameter rods spaced 4 ft. on-centers in each direction. Where there was no shear wall added, the masonry was to be tied at its top to the concrete frame through a connecting steel angle bolted to the frame and the wall with 5/8 in. expansion anchors. While this rehabilitation scheme would have undoubtedly performed satisfactorily in any subsequent earthquake, it did not take maximum advantage of the existing elements of the building. Further, this rehabilitation scheme was costed out at $8 million with the anchor bolts alone costing more than $2 million. Seismic rehabilitation costs equal to 25% of the total rehabilitation costs were unexpected making financing more difficult and placing the future of the whole project in jeopardy.

The alternative of relying on the existing masonry elements and strengthening the structure in limited areas only was therefore explored. First, a careful examination was made of the total structure, then, in-situ testing was conducted of the masonry, and finally, a strengthening scheme designed to take maximum advantage of the attributes of the existing elements of the structure. The inspection involved photographing and close-up visual examination of the masonry and the terra cotta. Where any cracking was observed, portions of that terra cotta were removed to reveal stiffening channels and attachment connections. The masonry and mortar in the Olympic were found to be of much higher quality than that in other older masonry buildings such as the Grand Central. However, many of the stiffening channels and attachments for the terra cotta were found to be badly rusted. A program of complete removal and reattachment or replacement of that terra cotta was obviously necessary.

In-place testing of the masonry was conducted at the bottom level of the concrete frame in six locations. The masonry walls were three wythes thick. Load-deformation relationships were obtained for the shear resistance of the two interior wythes. That approach neglected the contribution of the stronger exterior brick and the interior plaster. The test arrangement is illustrated in Fig. 7a. At each test location, the adjoining window was removed and a masonry prism 8 in. wide, 8 in. high and 16 in. in the direction of loading created by making a vertical cut in the plane of the wall that separated the exterior wythe from the inner wythes and a vertical cut transverse to the plane of the wall to separate the prism from the rest of the wall. The cuts coincided with mortar planes. Loading was by a jack inserted in the window opening and bearing against an 8 in. square by 1½ in. thick plate bedded on high strength plaster on one side of the test prism. Shear deformations across the upper and low boundaries of the prism were measured with dial gages. Four prisms from the 1923 portion of the builidng were tested and two from the 1929 portion. In only one case was the loading apparatus adequate to test the masonry prism to failure. A shear strength of 172 psi was obtained for a shearing area of 295 sq. in. The resulting shear stress-displacement curve is shown in Fig. 7b, together with the result for test 6 where the prism was unloaded and reloaded and the ultimate strength was still not exceeded at a stress of 230 psi. Based on these results, the City accepted use of a 50 psi permissible shear stress on the masonry and the classification of 9 in. of the 13 in. thick masonry walls as in-fill walls. The masonry walls were bounded on four sides by either steel or concrete beams and columns and with the masonry tied back to that frame for forces normal to the wall, the in-plane shear capacity of the walls would exceed that measured in the tests. By utilizing all masonry and concrete shear wall elements in the structure and distributing earthquake forces to those elements in accordance with their rigidities, it was found the structure could take forces of 6 to 6.5% of gravity at shear stresses not exceeding 50 psi in the masonry. While the foregoing testing and analysis documented the reasons for the good overall performance of the building in the 1949 and 1965 earthquakes, calculations and engineering judgment showed also that strengthening was needed in certain critical areas where the framing was incomplete or relatively light. As indicated in Fig. 8a, structural steel shear walls and framing were added in the lower three stories at the north ends of the east and west wings in order to offset termination of the concrete shear walls at those locations at the third floor level. The structural steel shear wall at the ground floor was a 3/8 in. stiffened plate. A similar framing scheme was used for the north face of the central part of the building opening onto the central courtyard. As indicated in Fig. 8b, the first story levels of the west, south and east walls surrounding the large windows were reinforced with additional steel framing connected to the existing framing and also used to support the masonry walls. As indicated in Fig. 8c, cold formed steel stud frameworks were fabricated for each shear panel of the exterior walls above the second floor, that framing connected to the existing concrete framing by 3/4 in. wedge angles at 24 in. centers and the exterior masonry anchored to the stud framework with ½ in. grouted bolts at 36 in. centers. These steel stud frameworks were also convenient for insertion of insulation and for attachment of interior finishes. Finally, as illustrated in Fig. 8d, steel ties were added for all floor diaphragms from the bottom of the concrete frame to the roof in order to tie the inadequately reinforced concrete floors into the main core of the structure. The cost of this second alternative for seismic retrofitting was estimated at $4 million and the job let for bid using that scheme. The cost of the seismic rehabilitation has been tracked separately during construction and found to be $3.8 million. The cost of the architectural renovations has been $34 million.

CONCLUDING REMARKS

It is generally inappropriate to use building code provisions intended for new buildings for the seismic rehabilitation of older buildings. However, by systematic investigation, much can also be done to change the development of seismic rehabilitation schemes from an art to a science. In the Seattle area, the form of the construction is primarily a function of the age of the building and the appropriate seismic retrofitting scheme will differ for differing forms. If a building has performed satisfactorily in previous earthquakes, the basis for that satisfactory performance should be documented by analysis and in-place testing before the seismic retrofitting scheme is developed. That scheme should take maximum advantage of the possible contributions of existing elements of the structure to the earthquake resistance and should be developed so that it is as compatible as possible with simultaneous energy retrofit and architectural renovation schemes. The retrofitting schemes developed for the Olympic Hotel suggested that laboratory investigations should be made of the use of steel plate shear walls as shear reinforcement for masonry panels, the use of steel stud walls as shear reinforcement for masonry panels as well as anchoring frameworks for out of plane loading of those panels, and requirements for anchoring masonry panels to surrounding frames and back-up shear panels of either shotcrete or steel.

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ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance of City of Seattle officials, their colleagues, Dr. C. W. Roeder and Padriac Burke, and the consulting engineering firms of Ratti and Fossatti, and Andersen, Bjornstad, Kane and Jacobs for their assistance in the preparation of this paper. Studies for this paper were made possible by grants from the U.S. National Science Foundation, and particularly grants CEE-8100531 and PFR-8021118, and by support from the Board of Regents, University of Washington.

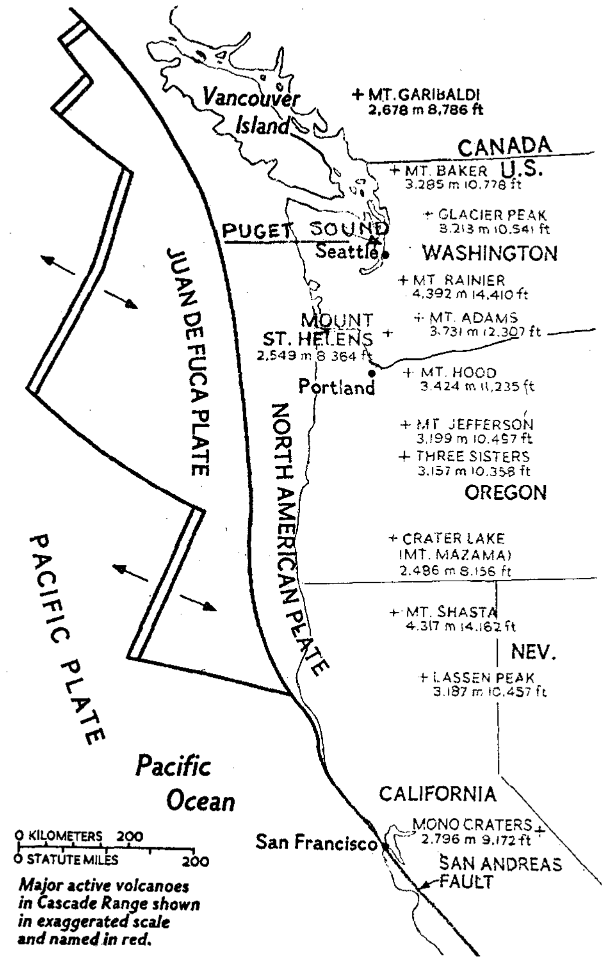
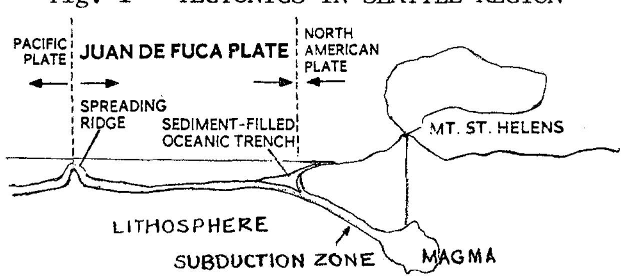


Fig. 1 TECTONICS IN SEATTLE REGION



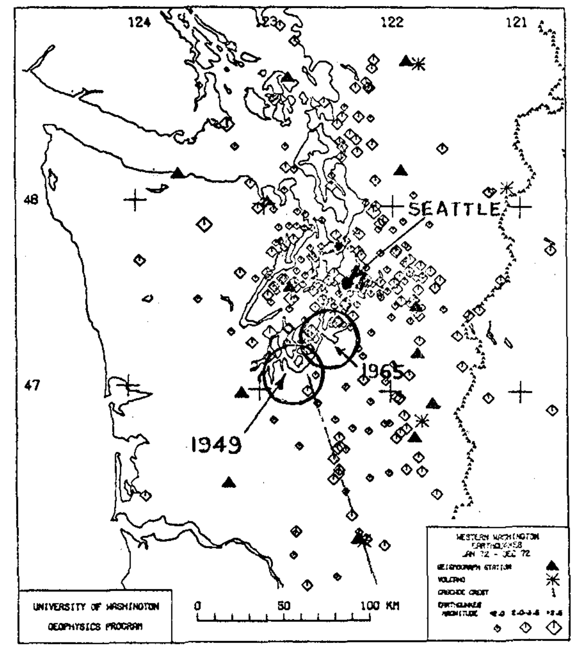


FIG. 2 EARTHQUAKE EPICENTERS SEATTLE REGION

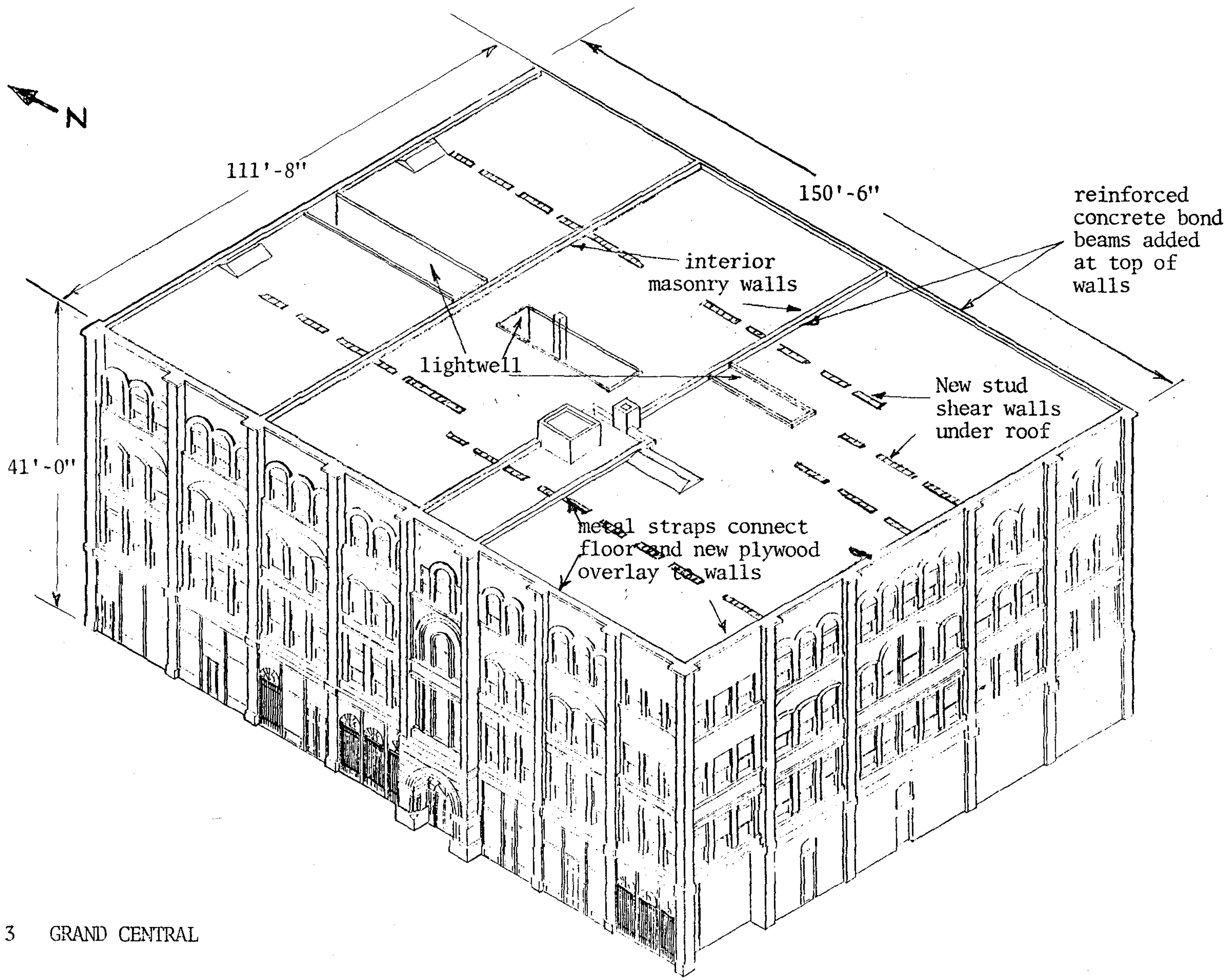


FIG. 3 GRAND CENTRAL

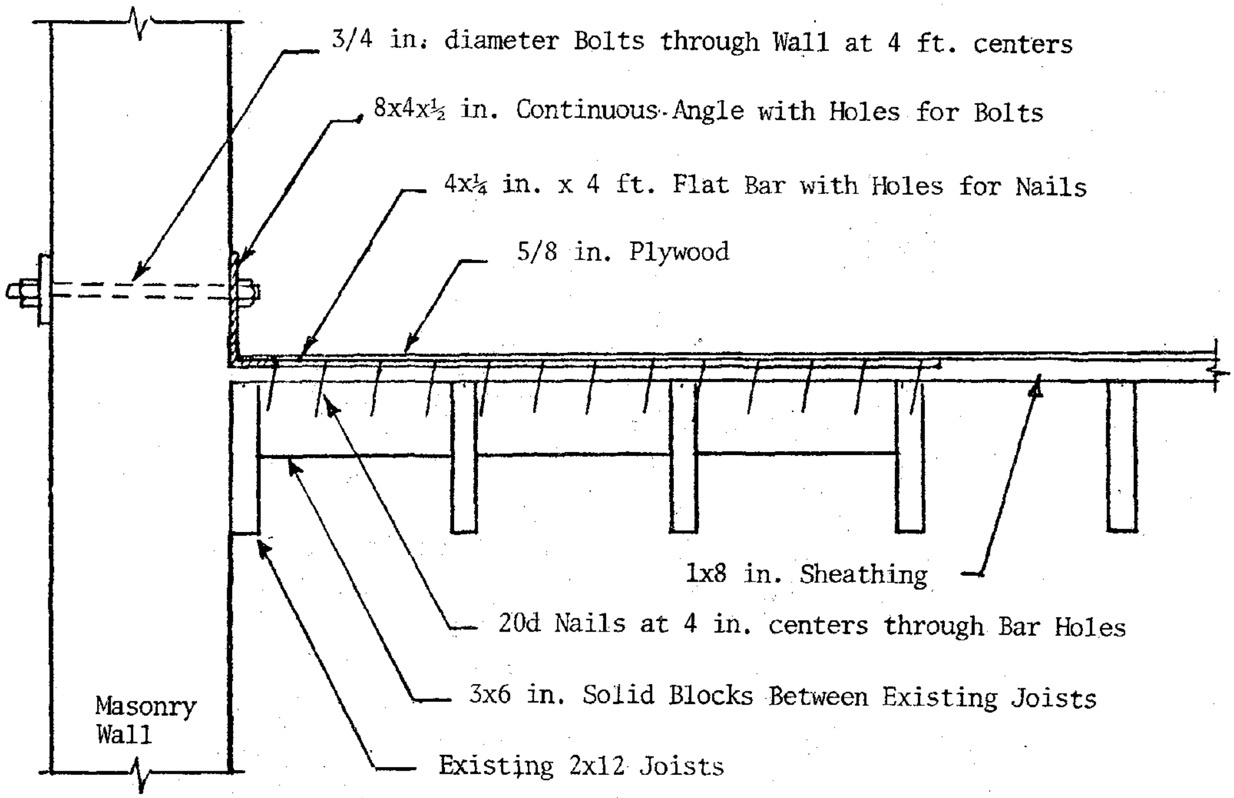


FIG. 4a FLOOR TO WALL CONNECTION

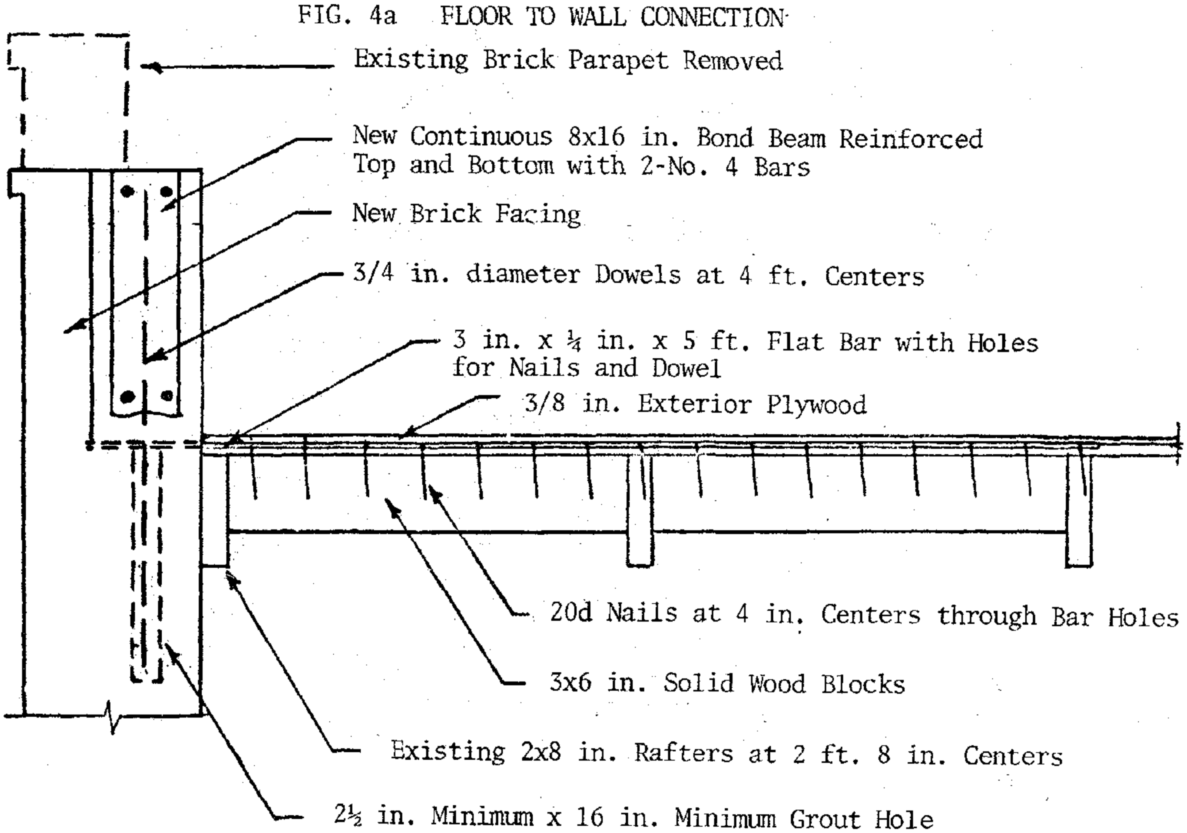


FIG. 4b ROOF TO WALL CONNECTION AND PARAPET BOND BEAM

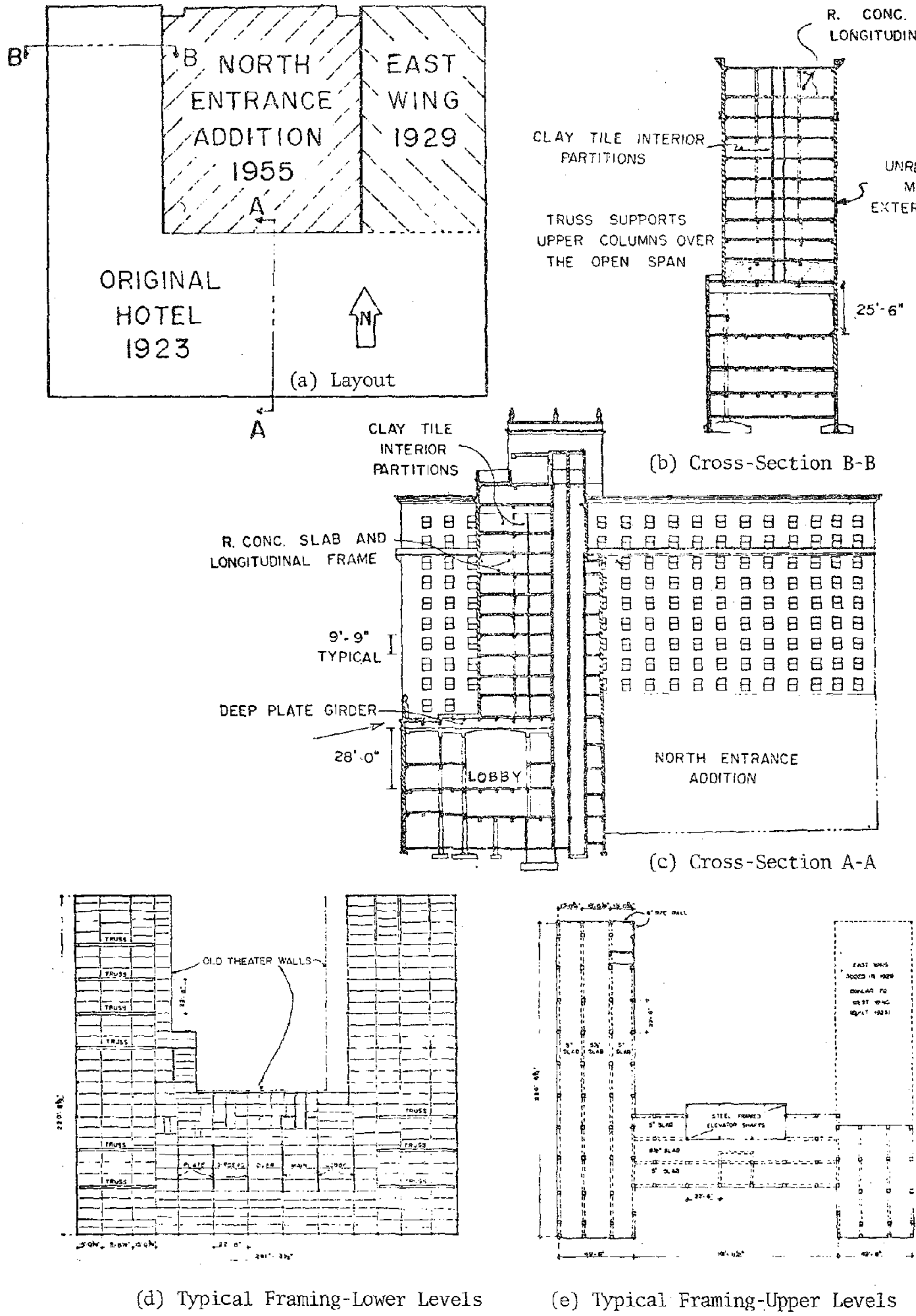
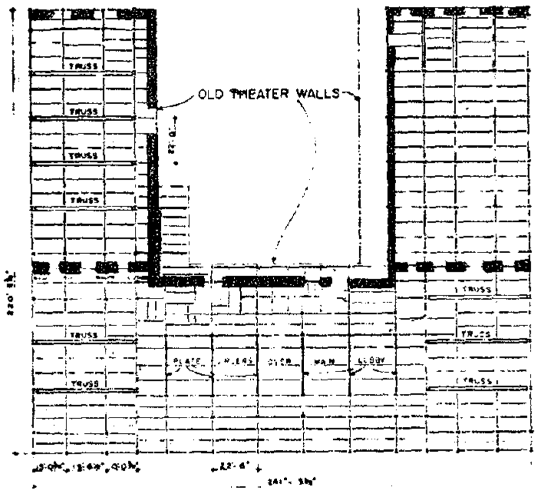
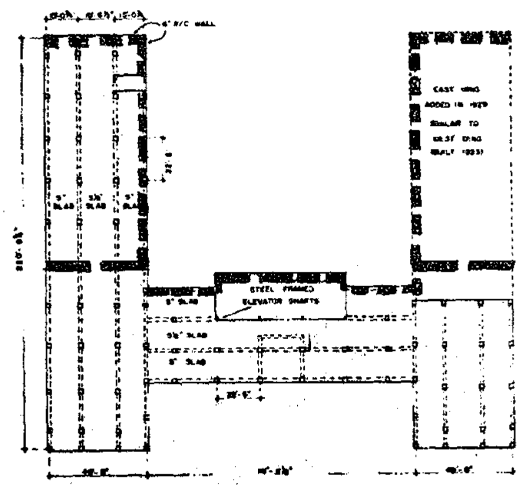


FIG. 5 OLYMPIC HOTEL PRIOR TO REMODEL



Ground - Second Floor
(Existing Steel Frame)

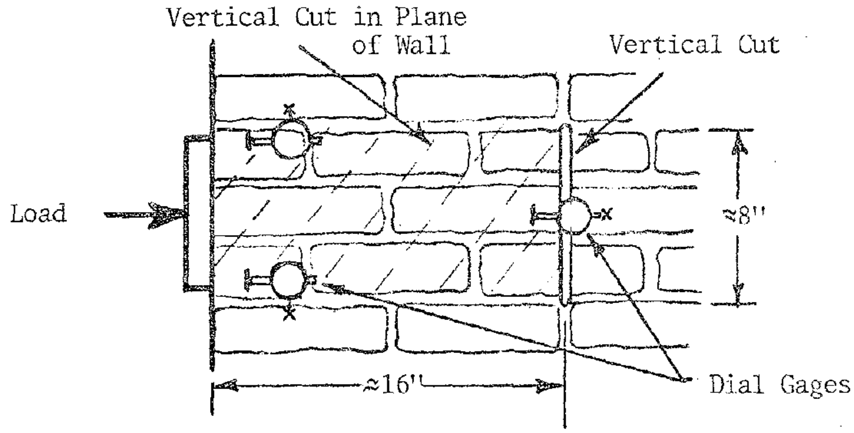


Third Floor - Eleventh Floor
(Existing Concrete Frame)

New Shear Wall

Floors	Wall Thickness inches	Reinforcement	Layout
ground - 2	10	No. 5 at 6 in.	all bays
3	8	No. 4 at 9 in.	all bays
4 - 7	6	No. 4 at 12 in.	all bays
8 and above	6	No. 4 at 12 in.	alternate bays

FIG. 6 PROPOSAL FOR REHABILITATION OF OLYMPIC HOTEL WITH SHEAR WALLS



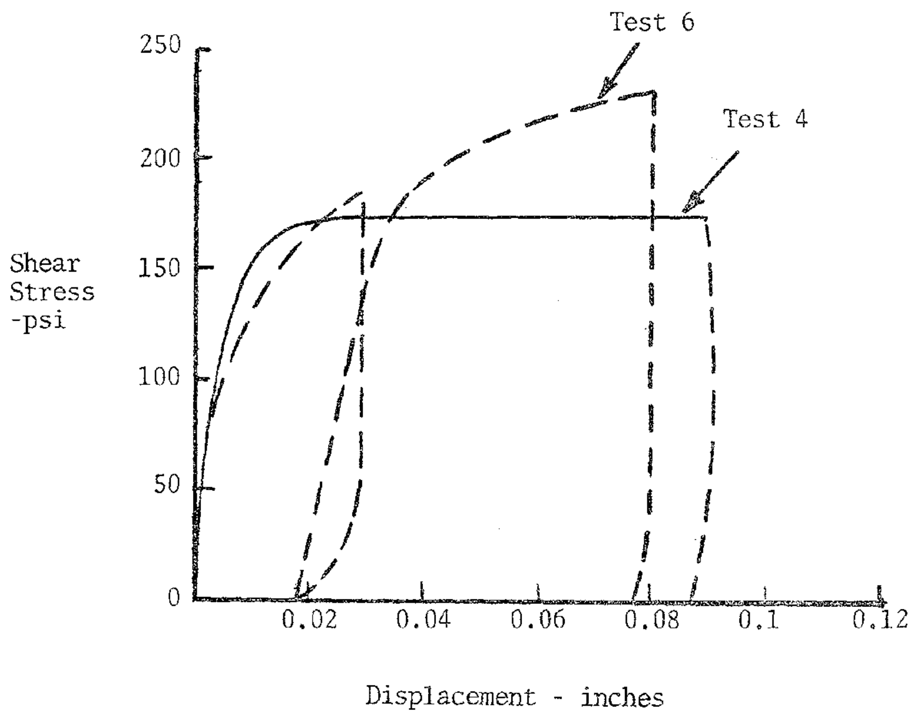


FIG. 7(b) SHEAR STRESS-DISPLACEMENT CURVES FOR IN-SITU TESTS

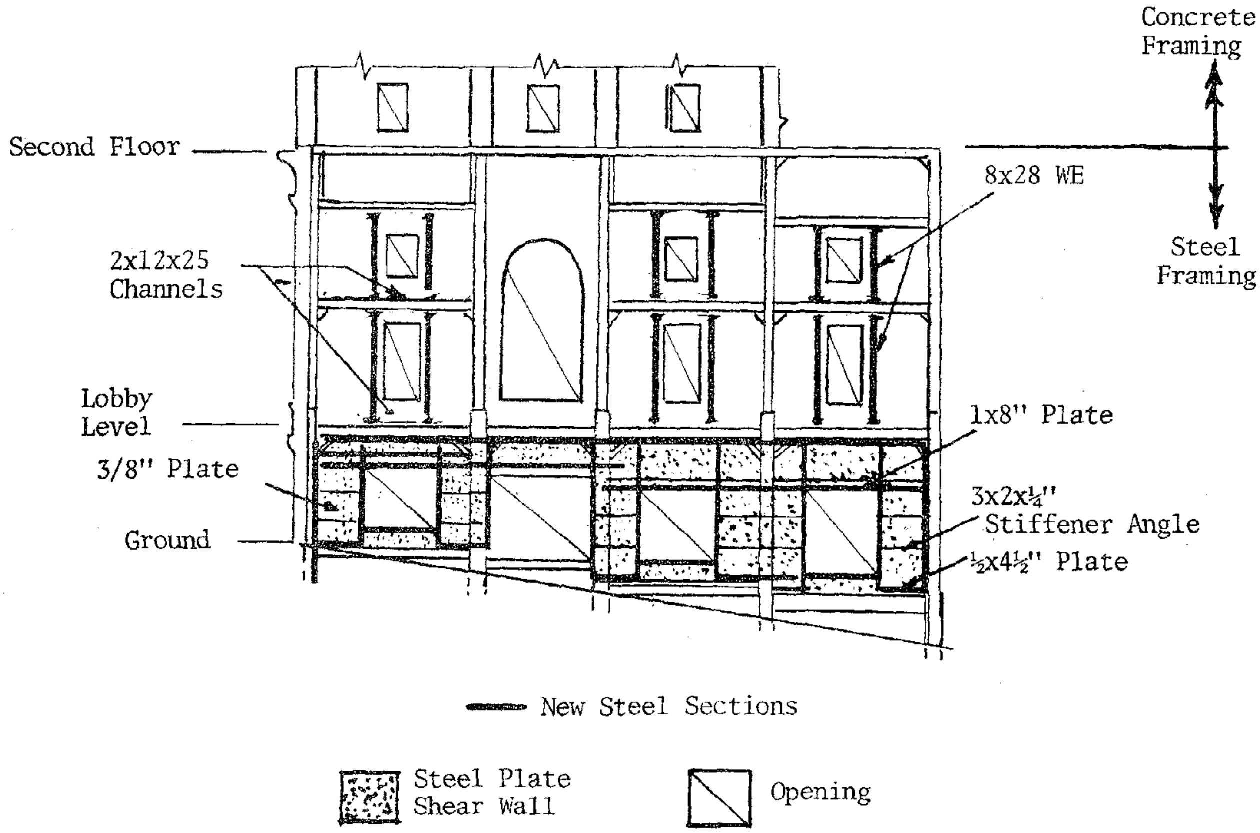


FIG. 8(a) STRUCTURAL STEEL SHEAR WALL AND FRAMING REINFORCEMENT AT BOTTOM OF NORTH END OF EAST WING

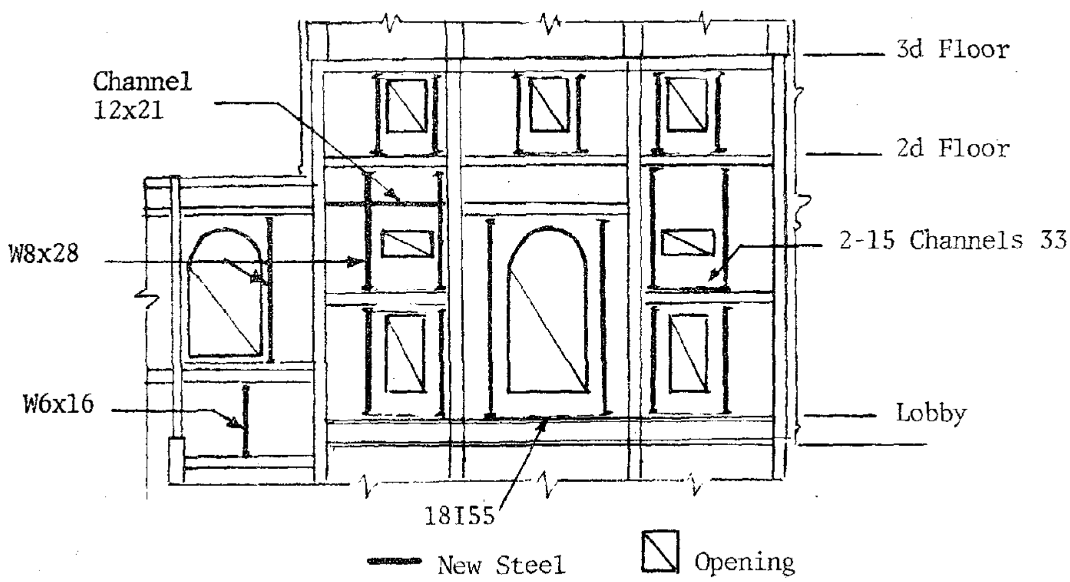


FIG. 8(b) STEEL FRAMING REINFORCEMENT SOUTH FACE

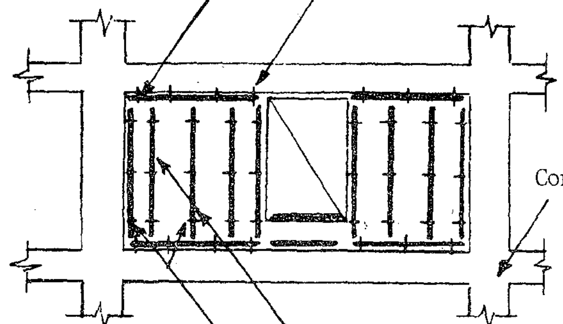


FIG. 8(c) STEEL STUD FRAMEWORK FOR EXTERIOR MASONRY WALLS

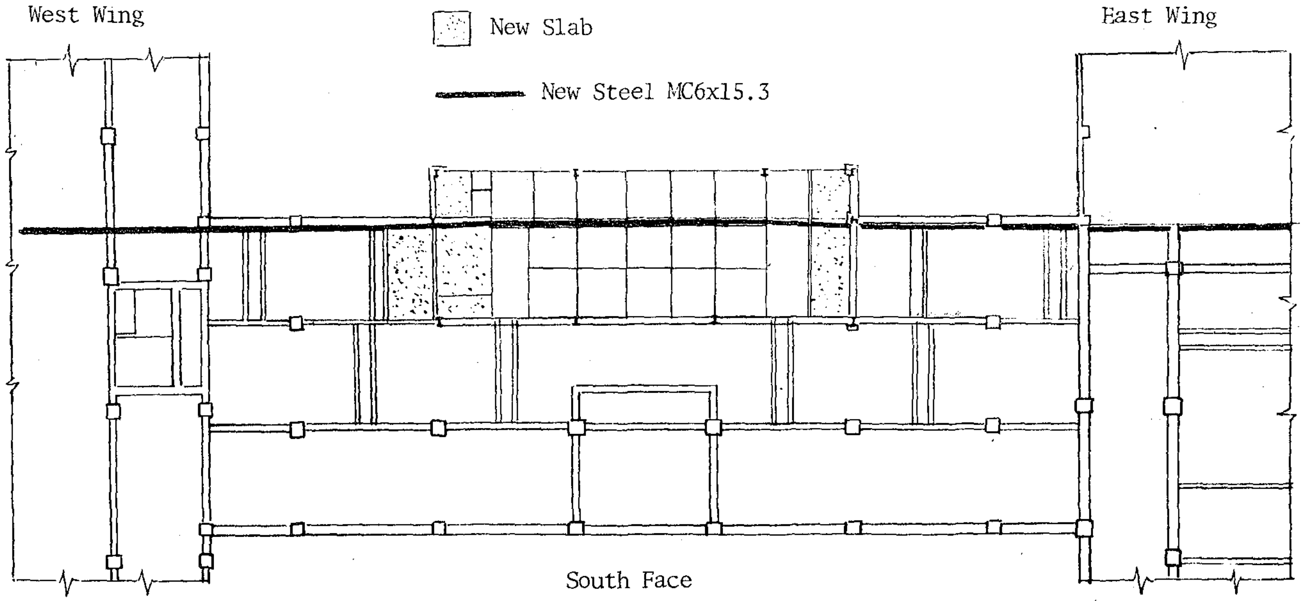


FIG. 8(d) NEW TIE FOR FLOOR DIAPHRAGMS FLOOR 3 THROUGH 11

CALCULATING METHODS OF STRENGTHENED AND REPAIRED BRICK MASONRY STRUCTURES FOR EARTHQUAKE RESISTANCE

Niu, Zezhen[I]

ABSTRACT

Calculating methods of strengthening and repairing measures of brick masonry structures are described in this paper. These methods are based on the available testing data of the institutes concerned. The strengthening and repairing measures used are follows.

1. Cement mortar coating, reinforced cement mortar coating attached to the wall surfaces or reinforced concrete columns with tie beams were used for strengthening walls.

2. Reinforced cement mortar attached to the surfaces or steel angles at the corners were used for strengthening columns.

3. Vertical and circular steel strips attached to the surface were used for strengthening chimneys.

This paper shows that the calculating results with these methods are more close to the testing data than those calculated by the available current methods.

INTRODUCTION

Brick masonry structures are widely used in China for residential houses, public buildings, and some industrial buildings. A lot of brick masonry buildings which were not designed to resist the vertical and lateral loads produced by strong ground motion severely damaged or collapsed in the past Chinese earthquakes.

After 1975 Haicheng earthquake, some buildings in Beijing and Tianjin cities, where the strong earthquake might occur, were strengthened according to the medium-term earthauake prediction. During the Tangshan earthquake, the strengthened buildings stood well or were slightly damaged only. This event shows that strengthening and upgrading of existing buildings located in the area where destructive earthquakes might occur in the near future are an effective measure for earthquake disaster mitigation.

After Tangshan earthquake, strengthening and repairing works were spread in 37 major cities of this country. During the recent four years, about 92 million square meters of buildings were strengthened. In the same time, an extensive program, supported by the Office of Earthquake Resistance of State Capital Construction Commission, has been under taken at various institutions to develop calculating methods and aseismic measures for structures to be strengthened and repaired. The "Aseismic Criterion for Evaluation

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of Industrial and Civil Buildings" (TJ 23-77) was issued in force on December 1, 1977 (1). The reports on aseismic strengthening techniques and measures for various types of buildings were published.

The purposes of strengthening for brick structures are mainly to increase lateral earthquake resistant capacity of the vertical structural systems (such as walls, brick columns) and to increase ductility and integraty of the brick masonry buildings.

Calculating methods of strengthened and repaired brick masonry structures for earthquake resistance are described in this paper. These methods are developed in the light of the available testing data of the institutions concerned. The strengthening and repairing measures in common use are as follows.

— Cement mortar coating, reinforced cement mortar coating attached to the wall surfaces or reinforced concrete columns with tie beams were used to strengthen walls.

— Reinforced cement mortar coating attached to the column surfaces or steel angles at the column corners were used to strengthen columns.

— Vertical and circular steel strips attached to the shaft surfaces were used for strengthening chimneys.

This paper shows that the calculating results given by the author's recommened methods are closer to testing data than those calculated by available methods.

STRENGTHENING AND REPAIRING OF BRICK WALLS

Using Cement Mortar or Reinforced Cement Mortar Coating

An example of a wall strengthened or repaired by cement mortar coating attached to the wall surfaces is shown schematically in Fig. 1. Fig. 2 shows an example of a wall strengthened or repaired by reinforced cement mortar coating attached to the wall surfaces. The lateral load bearing capacity P and stiffness B of brick walls strengthened or repaired with cement mortar coating or reinforced cement mortar coating attached to the wall surfaces should be equal to the sum of corresponding values of the brick walls and the additional coatings. Let P_c, P_s and P_g denote lateral load bearing capacity of the wall, the additional cement mortar coating and the net of reinforcement respectively, and their corresponding effective coefficients are α_c, α_s and α_g respectively. Thus, we obtain

$$P = \frac{m}{\xi}\left(\alpha_c P_c + \alpha_s P_s + \alpha_g P_g\right) \tag{1}$$

where

m = coefficient of construction condition, usually, m=0.8

ξ = coefficient considering nonuniform distribution of shear stresses, and ξ = 1.2 for a rectangular cross section

Denoting the elastic modulus and the cross sectional area of the brick

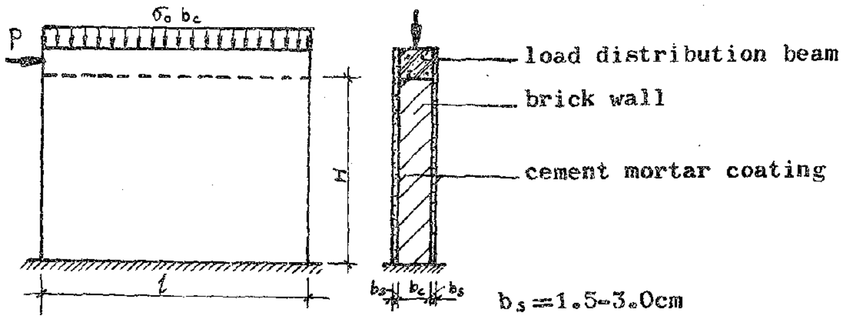


Fig.1 Brick wall strengthened by cement mortar coatings attached to the wall surfaces

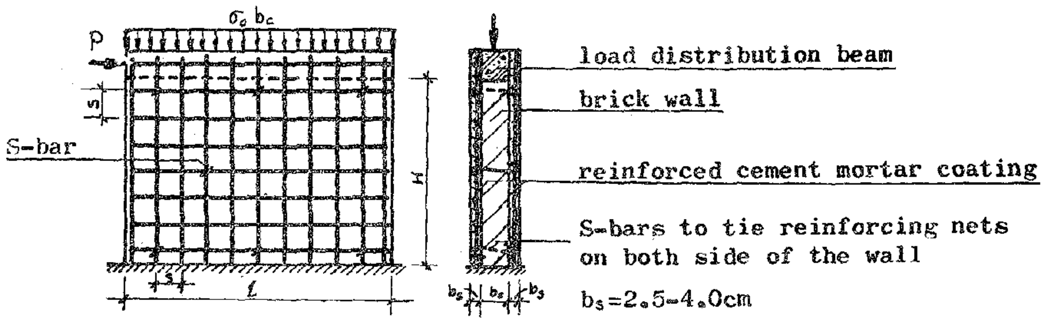


Fig.2 Brick wall strengthened by reinforced cement mortar coatings attached to the wall surfaces

wall by E_c and A_c respectively, and those of the additional cement mortar coating by E_s and A_s respectively. Then, the relative stiffness of the strengthened wall can be written as:

$$B = \beta E_c A_c + E_s A_s \tag{2}$$

where

β = coefficient of quality of the original wall considering decrease of its stiffness due to cracks

P_c, P_s, P_g, α_c, α_s, α_g = coefficients given in Table 1.

The above mentioned coefficients α_c, α_s, and α_g in Table 1 are obtained by statistical analysis of available testing data as follows.

Uncracked Wall

The ultimate tension deformation of mortar approximately equal to half of that of brick masonry. Therefore, lateral load bearing capacity of the wall strengthened by cement mortar coating depends on the ultimate loading capacity of the mortar. Taking α_s= 1.0 when mortar attained the ultimate

TABLE 1 COEFFICIENTS FOR CALCULATING LATERAL LOAD BEARING CAPACITY AND STIFFNESS OF STRENGTHENED WALLS

measures for strengthening	condition of wall	α_c	P_c	α_s	P_s	α_g	P_g	β
cement mortar coating	cracked	0.84	$f\sigma_0 A_c$	1.0	$R_{sj}A_s$	—	—	0.84
	uncracked	$0.2+0.13\frac{\sigma_0}{R_j}$	$R_\tau A_c$	1.0	$R_{sj}A_s$	—	—	1.0
reinforced cement mortar coating	cracked	1.0	$f\sigma_0 A_c$	0	$R_{sj}A_s$	1.0	$\frac{2n_s R_g a_g l}{s}$	0.84
	uncracked	$0.1+0.06\frac{\sigma_0}{R_j}$	$R_\tau A_c$	0.84	$R_{sj}A_s$	1.0	$\frac{2n_s R_g a_g l}{s}$	1.0

In Table 1:

l = length of the brick wall

σ_0 = mean compression stress acting on the brick wall

R_τ = shear strength of brick masonry,

$$R_\tau = R_j\sqrt{1+\frac{\sigma_0}{R_j}}$$

R_j = tension strength of brick masonry, and it can be taken as the shear strength along the stepped cross section of brick masonry specified in the "Design Code for Brick and Stone Structures" (2)

R_{sj} = shear strength of the mortar, in case of lack of experimental data, it can be taken as

$$R_{sj}=2\sqrt{R_s}$$

R_s = compression strength of mortar

a_g = cross sectional area of a steel bar

R_g = design strength of the reinforcement

n_s = number of additional coating layers

s = spacing of reinforcement

f = frictional coefficient at the cracks of the wall, generally, f=0.7

loading capacity and $\alpha_g = 0$ by reason of without reinforcement in coating, from equation (1) we have

$$\alpha_c=\frac{1}{P_c}\left(\frac{\xi}{m}P-P_s\right)$$

According to experimental results listed in Table 2, the relationship of α_c versus P is shown in Fig. 3. It indicates that the coefficient α_c is not a constant, but increases with the value of P. The coefficient α_c is a linear function of σ_0/R_j as indicated in Fig. 4. From which the α_c is given by

$$\alpha_c=0.2+0.13\,\sigma_0/R_j$$

The testing results show that for most of the strengthened walls coated with reinforced cement mortar, the loss of bearing capacity is caused by the yield of steel bars in case of normal content of reinforcement. When the yield point of steel bars is attained, we take $\alpha_g=1.0$. Then the lateral load bearing capacity of brick wall and cement mortar coating will be:

$$\alpha_c P_c+\alpha_s P_s=\frac{\xi}{m}P-P_g$$

Using the least square method and the experimental data listed in Table 2,

TABLE 2 TESTING AND CALCULATING VALUES OF LATERAL LOAD BEARING CAPACITY OF STRENGTHENED BRICK WALLS

wall condition	measures for strengthening	specimen		brick wall				coating				compression stress on wall (kg/cm^2)	lateral load bearing capacity (ton)	
		group	number	grade of brick	grade of mortar	cross section (cmxcm)	ratio of height to width	grade of mortar	thickness (cm)	reinforcement diam. (mm)	reinforcement spacing (mm)		test.	calcu.
uncracked	cement mortar coating	I	2	75	10.8	24×180	1:1.2	114	2×2.8	—	—	9.7 11.1	28.0 32.0	28.2 30.6
		II	3	100	10.0	24×200	1:2	166	2×3.0	—	—	3.5	20.5 19.3 21.0	20.0
	reinforced cement mortar coating	III	3	75	15.9	37×175	1:1	142	2×2.6	6.7	350	4.17 3.63 3.94	26.6 22.3 25.5	26.8 26.3 26.5
		IV	4	75	10.8	24×180	1:1.2	114	2×2.8	6.5	225	11.58	36.0 38.0 36.0 40.0	36.6
		V	3	100	10.0	24×200	1:2	166	2×3.0	6.0	250	3.5	21.3 25.5 29.8	26.3
		VI	3	100	15.6	24×200	1:3.15	122	2×3.25	6.0	200	3.5	39.0 36.5 29.5	35.0
		VII	3	100	11.9	12×200	1:3.15	112	2×3.25	6.0	200	3.5	32.5 37.7 35.2	34.2

CONTINUED FROM TABLE 2

cracked	cement mortar coating	VIII	2	—	—	37×175	1:1	45	2×3.0	—	—	2.48 2.40	16.0 15.9	15.95 15.75
		IX	2	—	—	37×175	1:1	100	2×2.3 2×1.6	—	—	2.74 2.16	18.0 13.0	18.05 13.18
		X	1	—	—	24×200	1:3.15	180	2×3.25	—	—	2.75	28.2	28.40
	reinforced cement mortar coating	XI	3	—	—	37×175	1:1	—	0	6.7	600	2.44 2.36 2.32	13.9 13.9 14.2	14.00 13.70 13.60
		XII	3	—	—	37×175	1:1	98	2×2.5	6.7	350	4.01 3.80 3.82	22.3 22.2 22.2	23.40 22.80 22.80
		XIII	3	—	—	24×200	1:3.15	110	2×3.25	6.0	250	2.75	20.5 21.0 21.8	20.70

Remarks:

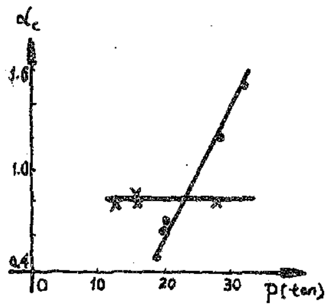
1. Double coatings were used to strengthen a wall for all specimens
2. Shear strength of cement mortar is taken as
   $R_{j,j}=2\sqrt{R_s}$
3. Testing data quote from reports of following institutions:
   — Design Institute of Department of Building Construction, Tianjin University (group I, IV)
   — Building Research Institute of Shanxi Province (group II, V)
   — Aseismic Strengthening Research Group of Liaoning Province (group III, VIII, IX, XI, XII)
   — Beijing Architectural Design Institute (group VI, VII, X, VIII)

the coefficients α_c and α_s may be expressed as

$$\alpha_c=0.1+0.06\,\sigma_0/R_j$$

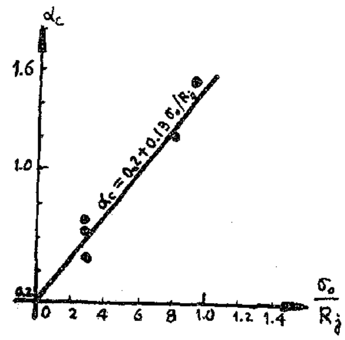
$$\alpha_s=0.84$$

respectively.



testing values
● for uncracked wall
x for cracked wall

Fig. 3 Relationship of α_c vs P of the wall strengthened by cement mortar coating



● testing values

Fig. 4 Relationship of σ_0/R_j vs α_c of the uncracked wall strengthened by cement mortar coatings

<u>Cracked Wall</u>

For cracked walls, the lateral load bearing capacity of brick masonry contributed by friction is

$$P_c=\alpha_c f \sigma_0 A_0$$

which is the main part of total lateral load bearing capacity. For the strengthened wall with cement mortar coatings, we have $\alpha_c = 0.84$, when $\alpha_s = 1.0$ (see Fig. 3). While for the strengthened wall with reinforced cement mortar coatings, in accordance with the least square method, we obtain $\alpha_c = 1.0$, $\alpha_s = 0$, when $\alpha_g = 1.0$.

The values of lateral load bearing capacity calculated by equation (1) and Table 1 are given in Table 2, and the corresponding testing data are also listed in this Table. It shows that the calculating results have a good coincidence with testing data. Coefficient β in Table 1 is obtained by the ratio of testing values of shear strain of the strengthened or repaired cracked and uncracked walls. The testing result shows that reinforcement has no significant influence on the shear strain. Therefore, we conclude that the stiffness of the reinforced cement mortar coating has no difference from that of the cement mortar coating.

Using Additional Reinforced Concrete Columns

Since the brick wall is surrounded and confined by the additional reinforced concrete columns and the tie beams or steel rod, both the ductility or deformation ability and the lateral load bearing capacity of the wall can be increased.

Analysing available testing data of the brick walls strengthened by additional reinforced concrete columns and the 1/4 model of a strengthened brick building, the author developed a improved method. In which lateral load bearing capacity of strengthened wall can be computed in two stages: the initial cracking stage and the thoroughly cracked stage.

When the wall begins to crack, i.e. in its initial cracking stage, the lateral load bearing capacity is given by (Fig. 5a)

$$P = \left(1 + \frac{2D_{kh}}{D_{kc}}\right)\frac{R_\tau A_c}{\xi} \tag{3}$$

where

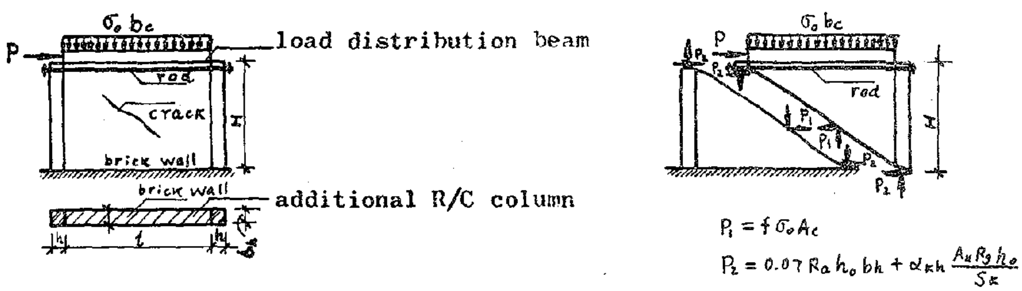
$$D_{kh} = \frac{b_h E_h}{\frac{H}{h}\left[4 + \left(\frac{H}{h}\right)^2\right]}$$ — lateral stiffness of a column

$$D_{kc} = \frac{b_c E_c}{\frac{H}{l}\left[4 + \left(\frac{H}{l}\right)^2\right]}$$ — lateral stiffness of the brick wall

E_h = elastic modulus of concrete

Let $\frac{b_h}{b_c} = \xi_b$, $\frac{E_h}{E_c} = \xi_E$, $\frac{h}{l} = \xi_l$, $\frac{H}{l} = \xi_h$, we can rewrite equation (3) in the following form

$$P = \left(1 + 2\xi_b\xi_E\xi_l^3\frac{\xi_h^2+4}{\xi_h^2+4\xi_l^2}\right)\frac{R_\tau A_c}{\xi}$$



(a) initial cracking stage (b) thoroughly cracked stage

Fig. 5 Brick wall strengthened by additional reinforced concrete columns with rod

When the wall is thoroughly cracked, i.e. in its thoroughly cracked stage, as shown schematically in Fig. 5b, lateral load bearing capacity of the wall is the sum of the resistant forces due to friction in brick masonry and the shear bearing capacity of additional reinforced concrete columns.

such that

$$P = f\sigma_0 A_c + 2\times\left(0.07R_a b_k h_0 + \alpha_{kh}\frac{A_k R_g h_0}{s_k}\right) \qquad (4)$$

where

R_a = axial standard compression strength of concrete

A_k = total cross sectional area of all hoops placed in a cross section of a column

s_k = spacing of the hoops

α_{kh} = shear strength-related coefficient, it can be obtained by reference (3)

Thus, as lateral load bearing capacity of strengthened wall by additional reinforced concrete columns we can take larger value of that calculated by equations (3) and (4). It should be noted that in this case tension resistant force of the tie beam or rod should be larger than the shear resistant force of one column. Otherwise, the second term in the right hand of the equation (4) shouldn't multiply by two. Table 3 shows that lateral load bearing capacity calculated by the method developed in this paper has a good agreement with the corresponding testing values.

STRENGTHENING OF BRICK COLUMNS

Most of the buildings with brick columns situated in the area with intensity of VIII and above were severely damaged or collapsed due to lack of the bend-bearing capacity of the brick columns in the light of Chinese earthquake experience. Therefore, according to the requirements of the Aseismic Evaluation Criterion (1), the vertical steel bars should be placed in the brick columns of buildings located in the region with intensities of VIII and IX. When the brick columns do not coincide with these requirements, they should be strengthened.

Using Reinforced Concrete Mortar Coatings

The forces acting on cross section of strengthened brick column and wall pier are shown in Fig. 6a and 6b respectively. The reinforcement content in coatings should be determined by calculation and should not less than 4 ϕ 10 and 4 ϕ 12 for design intensities of VIII and IX respectively (1). Stiffness B and ultimate moment M of the strengthened brick column can be calculated by following formulae:

$$B = 0.8E_0 I_c + E_s\left[I_s + \left(\frac{E_g}{E_s} - 1\right)\Sigma A_{gi} z_i^2\right] \qquad (5)$$

for brick column:

$$M = R_g \Sigma A_{gi}\frac{h_{0i}^2}{h_{01}} + (W_1 + W_2)\left(\frac{h}{2} - a'\right) \qquad (6)$$

for brick wall pier:

$$M = R_g \Sigma A_{gi}\frac{h_{0i}^2}{h_{01}} + W_1(y_1 - a) + W_2'(y_2 - a') \qquad (7)$$

where

W_1 = weight of roof

W_2 = weight of strengthened brick column

TABLE 3 TESTING AND CALCULATING VALUES OF LATERAL LOAD BEARING CAPACITY OF WALLS STRENGTHENED BY REINFORCED CONCRETE COLUMNS

type: specimen	type: column	group	number	brick wall: grade of brick	brick wall: grade of mortar	brick wall: thickness x length x width (cm)	reinforced concrete column: strength of concrete (kg/cm^2)	reinforced concrete column: cross section (cm^2)	reinforced concrete column: height (cm)	reinforcement: main	reinforcement: stirrups	compression stress on wall (kg/cm^2)	lateral load bearing capacity (ton): test.	lateral load bearing capacity (ton): calcu.
single wall	addition.	I	3	100	16.9	12×200 ×63	225	2– 12×12	63	4φ8	φ6@100	3.5	16.2 13.6 12.9	13.84
single wall	original	II	3	100	50	12×176 ×74	150	2– 12×12	74	4φ8	φ6@100	3.5	12.5	11.63
single wall	original	III	3	100	50	12×176 ×74	150	2– 12×12	74	4φ8	φ6@100	0	6.6	6.46
4-story model (1/4 scale)	addition.	IV	1	100	30	3– 5.3×245 ×75	161	4– 5.3×6	75	4φ6	φ3@100	2– 1.17 1– 1.76	8.93	9.16
4-story model (1/4 scale)	original	V	1	100	30	3– 5.3×235 ×75	161	4– 5.3×6	75	4φ6	φ3@100	2– 1.17 1– 1.76	9.01	8.79

Remark: Testing values quote from the Beijing Architectural Design Institute

W_2' = weight of strengthened brick wall pier

E_0 = original elastic modulus of brick masonry, and $E_0 = 2E_c$ when there is no adequate data, where E_c denotes elastic modulus of brick masonry specified in reference (2)

I_c = moment of inertia of brick column about the axis through centroid of converting cross section of strengthened brick column or brick wall pier

I_s = moment of inertia of cement mortar coating about the axis through centroid of converting cross section of strengthened brick column or brick wall pier

z_i = distance from centroid of ith row reinforcement to centroid of converting cross section of strengthened brick column

a' = distance from center of gravity of compression reinforcement to the nearest side to the cross section

It should be noted that equations (6) and (7) are suitable for the buildings with light roof and brick columns using lower reinforcement ratio for strengthening.

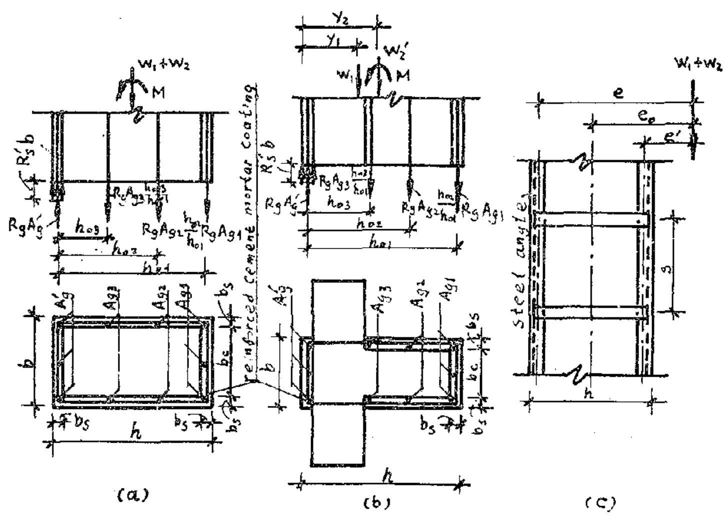


Fig. 6 Schematic diagram for calculating strength of cross section of strengthened brick column

Using Steel Angles at the Corners of Brick Column

Fig. 6c shows the forces acting on a cross section of a brick column strengthened by steel angles. The stiffness B of the strengthened brick column may be taken as

$$B = 0.8E_0I_c + 0.9E_gI_g \tag{8}$$

The formulae for checking strength of strengthened brick columns are as follows

$$
\left.
\begin{aligned}
1.05Rb_e x\left(e_0+\frac{x}{2}-\frac{h}{2}\right)+R_g A_g(e'-e)=0 \\
K(W_1+W_2)\leqslant 1.05\varphi Rb_e x
\end{aligned}
\right\} \tag{9}
$$

where

- A_g = cross sectional area of steel angles in each side of strengthened brick column
- E_g = elastic modulus of steel angle
- I_g = moment of inertia of all steel angles about the axis through centroid of cross section of brick column
- R = compression strength of brick masonry
- x = height of compression zone of cross section
- φ = longitudinal bending coefficient of strengthened brick column, which can be calculated by formula for composite (reinforced concrete and brick masonry) element specified in reference (2).

The comparision between testing data and calculating results of a brick column strengthened by above mentioned two measures is listed in Table 4. It follows that the testing data are consistent with the calculating ones.

STRENGTHENING OF BRICK CHIMNEY

The brick chimney is a vulnerable structure during an earthquake. It may be damaged even in the area with intensity of VI. The failure modes of brick chimney mainly are follows:

- —— top part fell down
- —— ruptures and cracks took place in brick masonry shaft
- —— brick pieces fell down to the surrounding ground

In the area with intensity of VII and VIII, the damages were found in the upper part of the chimneys. While in the area with intensity of IX and X, sometimes the damages took place in the lower part, sometimes even at the bottom. Therefore, the Aseismic Evaluation Criterion (1) specified that all brick chimneys built in earthquake-prone region with intensity of VII and above should be reinforced or strengthened. The experience of strong earthquakes shows that using vertical and circular steel strips attached and surrounded to the exterior surface of the chimney is one of the effective measures for strengthening.

Let r_1 and r_2 denote interior and exterior radius of the chimney shaft respectively. For simplicity, in checking strengths, we use an equivalent steel cylinder shell with radius r_3 and cross sectional area A_g instead of vertical strips, as shown in Fig. 7. Supposing that: in the tension zone, the stresses in steel reach the design tension strength and ignore the effect of brick masonry; and in the compression zone, both stresses in steel and in brick masonry reach the design compression strength and the compression stresses distribute uniformly; then equations of equilibrim for axial forces and for moments in considering cross section of a shaft are

$$KN+1.2A_gR_g(1-\alpha)-1.2A_gR_g\alpha-1.05RA\alpha=0 \tag{10}$$

$$KN\frac{e_0}{\eta}\leqslant\left(0.7RA\frac{r_1^2+r_1r_2+r_2^2}{r_1+r_2}+2.4R_gA_gr_3\right)\frac{\sin\pi\alpha}{\pi} \tag{11}$$

TABLE 4 TESTING AND CALCULATING VALUES FOR STRENGTHENED BRICK COLUMNS

measures for strengthening	number of specimen	brick column				reinforced cement mortar coating				steel angles	fundamental period (sec)		earthquake resistant capacity (kg)	
		brick masonry		cross section (cm^2)	height (cm)	mortar		reinforcement						
		compres. strength (kg/cm^2)	modulus of elasticity			compres. strength (kg/cm^2)	modulus of elasticity	main	stirrup		test.	calcu.	test.	calcu.
reinforced cement mortar coating attached to column corners	3	45	3.84×10^4	37×37	294	145	1.09×10^5	12ϕ8	ϕ6@200	—	0.068	0.069	$\frac{M}{H}=900$	915
steel angles attached to corners of column	3	45	3.84×10^4	37×37	294	—	—	—	—	4-∠50×5	0.082	0.072	N=3620	3330

respectively.
where

- A = considered cross sectional area of brick shaft
- η = modified coefficient of longitudinal bending coefficient given by the reference (2)

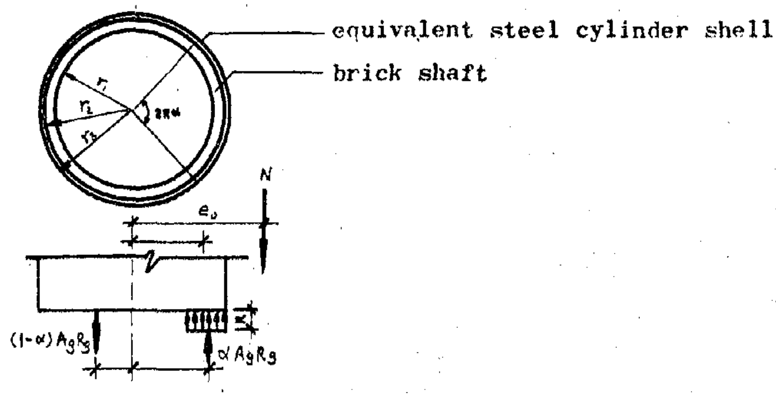


Fig. 7 Schematic diagram for checking the strength of a cross section of strengthened chimney

In equation (10), coefficient 1.05 is the ratio between ultimate load and corresponding calculating value. The ultimate load comes from axial and eccentric compression test of 29 composite masonry specimens conducted by Hunan University. The coefficient of 1.2 in equation (10) is the ratio between ultimate and design strength of reinforcement. When checking the strength of a cross section, first, from equation (10) we find

$$a = \frac{KN + 1.2A_gR_g}{1.05RA + 2.4A_gR_g}$$

then calculate safety factor from equation (11) and compare it with those specified in the Aseismic Evaluation Criterion (1).

The earthquake damages to three brick chimneys in Tianjin urban area due to 1976 Tangshan earthquake and their checking results by equations (10) and (11) are listed in Table 5. When checking the strengths, the lateral earthquake loads are calculated by Aseismic Design Code (4). The Table shows that real damage levels of chimneys due to earthquake consistent with calculating levels. It follows that computation methods for checking developed in this paper is applicable for practical use.

CONCLUSION

Finally, we have come to the following conclusions:

1. Strengthening measures described in this paper can actually increase earthquake resistant capacity of brick structures, and their lateral load bearing capacity can be obtained by methods given in this paper.

2. Most of calculating formulae proposed in this paper are empirical.

TABLE 5 DAMAGES TO BRICK CHIMNEYS IN TIANJIN URBAN AREA WITH INTENSITY OF VIII IN 1976 TANGSHAN EARTHQUAKE AND THEIR COMPUTATION RESULTS THROUGH CHECKING

location of chimney	condition of chimneys before earthquake				computation results through checking obtained by equations (10) and (11)	damages
	height (m)	brick masonry		condition of seismic design		
		mark of brick	mark of mortar			
Tianjin Glass Fiber Factory	40	100	50	18 steel bars in diameter of 8mm are placed in shaft masonry up the height of 20m to the top according to requirements for design intensity VII specified in Aseismic Design Code(TJ 11-74)	K=0.97 at the level of 27m in height	collapse of upper shaft above 20m in height
Tianjin Glass Fiber Factory	50	100	50 (0–25m) 25 (25–50m)	using 16 vertical steel bars in diameter of 22mm and circular steel angles to strengthen the shaft masonry above 25m in height	K=1.72 at the level of 37m in height	slightly damaged and still can be used after earthquake twist and buckling of few reinforcements in diameter 22mm at the level of 35m in height and cracks occurred in shaft
Tianjin Power Equipment Plant	45	—	—	no account taken of earthquake resistant requirements	$e_0 > 0.8y$ at the level of 35m in height	many cracks were found at the level at 37m high

The coefficients in these formulae are obtained by statistic analysis based on available testing data. In the future, they should be modified with the increase of testing data.

3. Seismic analyses for strengthening brick structures are more complicated. Some problems, such as behavior of structural components and complete structures of brick buildings under earthquake loading, reliability of testing results obtained by models with medium or small scales, and procedures for evaluating efficiency of strengthening measures etc, need further researches in detail in the near future.

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REPAIR OF DAMAGED STUCTURES IN TANGSHAN CITY

Zhang Shu-quan*

ABSTRACT

Studies in techniques employed for renovating the houses suffering damage due to earthquake are of great importance to the restoration of normal life and production activities of a city hit by earthquake. When dealing with a damaged house, opinions and design projects as to the possibility and measures adopted for its repairing are many and varied. Selection of optimal renovation project results in a cutdown of cost and work hours. In this paper relevant discussions are carried out and reference is made to the damage features as well as the techniques applied for repairing the multi-storied brick houses after the violent earthquake which took place in Tangshan. By way of concrete example, renovation work made on the multi-stoeied brick houses, spacious houses, single-storied workshops and chimneys located in Tangshan are also described.

I. INVESTIGATION AND APPRAISAL OF THE HOUSES AFTER THE EARTHQUAKE

Whenever a violent earthquake($M \geq 6$) takes place, various houses will suffer damage to various degrees. Hence restoration work should be timely undertaken to meet the people's demands in daily life and to maintain the normal production activities. Repair of the houses which have been severely damaged during the earthquake is superior to the building of new houses in that the former has the merits of being low in cost and less in time required. Particularly the restoration of water supply, power supply and hospital facilities is, in all cases, in urgent need. Ancient buildings as well as memorial buildings also need to be restored to their original states. Therefore, following a violent earthquake, organization work should be undertaken to immediately carry out the investigation and appraisal of the damage conditions and seriousness of various houses. Meanwhile, an important work as to filling in the form and checking in according to classification should be underway.

Damage of multi-storeyed brick houses falls into four categories:

1. The main structure is basically in good condition. Only the upper stuctures such as the parapet walls and the chimneys standing on the houses are damaged.

2. Negligeable cracks appear on the main structure, and most parts of the non-structural elements have been destroyed. For example, part of the separation partition collapse and a

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greater part of them are seen to have cracked. Parapet walls and chimneys mostly collapse.

3. The main structure has been severely destoyed, yet still merits restoration.

4. The main structure has collapsed in local places. Walls are smashed, inclined. As a result, the house doesn't deserve restoration.

As for the houses in categories 2 and 3, thorough investigation and appraisal work should be carried out. At the same time, note down the positions where damages have taken place and try to find out about the original designing data, construction and maintenance informations. Finally work out a rational restoration plan.

A rational restoration plan should conform to the following conditions:

1. Restore the house to its original functions of utilization. The strengthening columns and girths provided should not influence the operation of service pipes, the free movement of windows and doors as well as the usable floor area.

2. Improve the functions of utilization of a house. Modify the original design through restoration to make it more convenient and reasonable. As for an old house, efforts should be made to raise its standard of utilization so as to meet the requirements of modern life. For example, additional sanitary equipment may be installed, the separation partition increased or decreased. Furthermore the walls can be applied with plastic paper. Restoration of production workshops should be carried out in combination with advanced scientific technology and the requirements of technological improvement.

3. Render the anti-seismic charateristic of a house to be in line with the anti-seismic requirement in relation to intensity level. In so doing, the house is not to be restored merely to its former state through covering up the cracks with ornaments. Special attention should be paid to the strengthening of the damaged parts and the improvement of anti-seismic charateristic of the house. Arrangement should be made to ensure a firm connection between the damaged and undamaged parts because the already damaged parts will be the most sensitive positions in future earthquake.

4. Save on materials and facilitale the restoration work. Several design plans should be in hand for the restoration of a house and the optimal plan should be selected on the basis of techno-economical comparison. The strengthening columns and girths should not be installed at random without taking into consideration the position whether it has been damaged or not.

5. The additional structures installed during restoration

such as columns and girths must not affect the elevation beauty. A house with exposed angle steel and bolts may suggest a sense of danger. Therefore, all the strengthening structures which may affect the sense of beauty should be concealed up or converted into decorations.

II. DAMAGE CHARACTERISTICS OF MULTI-STOREYED BRICK HOUSES DURING VIOLENT EARTHQUAKE

On account of the reason that before the earthquake the basic level of intensity was 6 degrees, design of all the multi-storeyed brick-concrete houses had not taken the anti-seismic facilities into due consideration. The thicknesses of the inner walls and outer walls were generally 360MM and 240MM respectively. The separation partition walls were 120MM in thickness. Grade of the bricks was $50^{\#}$ - $70^{\#}$ and that of the cement-sand mortar was $25^{\#}$, $10^{\#}$ or less. Therefore, after the earthquake with an intensity level of 11 degrees, 90% of the municipal houses collapsed because most of them were located in regions with intensity levels of 11 and 10 degrees. Some of these houses did not collapse in presence of cracks because of the good quality of the walls(Grade of bricks was over 75 and that of the mortar over 25), the provision of girths and foundation girths as well as their better ground conditions. A part of them can be restored for reuse. Their damage characteristics were as follows:

1. The wall between the windows, the wall projecting below the window, the inner cross wall and longitudinal wall were seen to have oblique cracks, crosswise cracks, horizontal cracks and vertical cracks. The 120MM thick separation partition collapsed(see Fig 1).

2. The side walls inclined outward or collapsed(see Fig2).

3. The end bays collapsed in local places(see Fig 3).

4. The gable showed oblique, horizontal and vertical cracks, Part of it collapsed(see Figs 4,5 and 6).

5. The corner of the house partly collapsed(see Fig 7).

6. The projecting part of the wall collapsed in local places (see Fig 8).

7. The wall below the beam smashed or collapsed(see Fig 9).

8. Vertical cracks appeared on the wall below the balcony (see Fig 10).

9. The smoke flues and refuse chutes were destroyed (see Fig 11).

10. The parapet wall and the chimney standing on the roof all collapsed.

III. RESTORATION TECHNIQUES

1. Repair of the cracks on the wall.

   a). New bricks are inserted into the two sides of the crack with cement-sand mortar(see Fig 12).

   b). Vertical cracks are filled up with key blocks of reinforced concrete or concrete(see Fig 13).

   c). 50MM X 50MM grooves are cut on both the two edges of the crack and then place reinforcing cramp iron to make it more secure. Finally plaster a layer of cement-sand mortar(1 : 3),(see Fig 14).

   d). Fill up the cracks with pressurizing cement grouting. Add 2% (W/W) polyvinyl alcohol to the cement paste as suspending agent. The process is cheaper than chemical grouting with resin grout.

2. Repair of the side wall.

   a). In case tensional cracks are seen at the juncture of side wall and the inside cross wall, apply reinforced concrete supporting pillars(see Fig 15).

   b). The side wall collapsed or its inclination had exceeded the accepted limit: rebuild the wall, install reinforced concrete pillars and fit two ∅ 6 MM reinforcing bars at an interval of five layers of bricks(see Fig 16).

   c). In case the cross and vertical walls cracked slightly: use reinforcing bars and angle steel to bind them. (see Fig 17).

3. Repair of the wall between two windows.

   a). Slight cracks: after filling up the cracks, plaster cement-sand mortar or other fit-up materials.

   b). Seriously cracked or dislocated: remove it and rebuild a new one. Clad it with reinforced concrete plus either angle steel or iron hoop(see Figs 18 and 19).

4. Cracks appeared at the corner of the side wall: apply reinforced concrete in the form of cramp iron(see Fig 20).

5. The cross wall cracked seriously yet it was inconvenient to tear it down and rebuild a new one:

   a). Remove the plaster on one or both sides of the wall and then inject a layer of 50MM thick concrete with steel mesh reinforcement(see Fig 21).

   b). At the points equal to h/3 and 2h/3(h=storey height), apply two reinforced concrete ribbons(see Fig 22).

6. Repair of the brick lintels over the doors and windows:

a). Apply reinforcing bars below the lintel and then plaster cement-sand mortar(1 : 3) (see Fig 23).

b). Install angle steel lintels(see Fig 24).

c). The lintel cracked seriously and dislocation occured: support the floor slabs and fit reinforced concrete lintels. Rebuild the wall above the lintel.

7. Repair of the 120MM thick separation partition:

a). It cracked seriously and appeared to be out of perpendicular:remove it and rebuild a new one. While rebuilding, pay attention to its attachment to the load-bearing wall(see Fig 25), the floor slab(see Figs 26 and 27) and the beam.

b). If it cracked slightly, fill up the cracks; If vertical cracks were seen at the place where the separation partition met with the load-bearing wall, install reinforced concrete key blocks(see Fig 28); In addition, angle steel and reinforcing bars are placed under the floor slab(see Fig 29).

IV. CONCRETE RESTORATION EXAMPLES OF DAMAGED CONSTRUCTION

1. Repair of the Kitchen of the big dining hall of the 2nd Municipal Guest House.

The 40-meter long dining hall with a span of 18 meters could accomodate some 800 people. It was brick-walled, strengthened with pillars, steel trussed with cemented tile coverings. Above the window level was a reinforced girth. The quake caused the walls between windows seriously cracked, some even smashed. The upper part of the gable at western end collapsed and roofs on the two end rooms fell down. The kitchen was of beam and slab structure, with brick wall as load-bearing wall. The walls of the kitchen inclined showing cracks on their most parts, and part of them was smashed. According to the original restoration plan, it was to be repaired as a post-earthquake "earthquakeproof house" in the ordinary way. The general approach was to build a light-weighted wall (plaster on lath) from the window level which was about one meter in height.

This was to meet the demand of the people who were panic-stricken, filled with fear that another severe earthquake might come because of the frequent occurance of aftershocks. But in doing so, a large amount of timber would have been needed and it was also impossible for the construction work to be finished within 20 days in winter season. Therefore the conventional method was done away with, and a new method of strengthening the walls was finally adopted. As for the badly smashed and mislocated walls between the windows(three places in all), the method employed was to support the girth and build new walls. In addition, the walls under the truss were applied with angle steel as the combined pillars, and angle steel was also fitted at the window openings. Between the window opening and the angle steel pillar inside the walls, additionally installed were ø16MM reinforcing bars and iron ribbons which were welded to the pillars and iron steel between the window openings. In case of strong aftershock, the walls could be prevented from falling into the hall and hurting the people(see Figs 30 and 31).

Kitchen: The precast slabs were removed, but the cast-in-situ reinforced concrete girder reserved. Above the girder, we made a new truss. We also removed the smashed walls and re-build new ones. At the inside of the wall under the truss, reinforced concrete pillars were mounted. The upper beam of the kitchen range could not be supported by pillars for further strengthening because the kitchen range was reserved. It was therefore strengthened by suspended stiffening girder extending from the girth(see Fig 32). The adoption of the rational restoration plan resulted in a saving of timber over $100M^3$ and the work was completed on schedule(see Fig 33). The pumping house of the Municipal Water Works was repaired in the same way and water supply was initiated ahead of time(see Fig34).

2. Repair of the No. 3 building of the 2nd Municipal Guest House.

The three-storeyed building with a storey height of 3M was built with 75# bricks, 50# mortar and prestressed hollow slabs. At the time when the earthquake took place, the principal part of the project had just been completed. On account of the reason that the foundation soil was uneven in composition, a settlement joint was provided at the middle part, and foundation girth as well as upper girth were additionally fitted.

After the quake, the walls were seen to crack universally. The walls of the two end rooms collapsed in local places. The slabs on the third floor were smashed with the fall of the roof slabs.

Repair methods used were as follows:

a). Reinforced concrete columns were installed as foundation (see Figs 35 - 40).

b). Splint beams(L-1, L-2) were provided below the floor slabs. The reinforcing bars protruding into the column to a depth equal to 30 times of the diameter of the bar(see Fig 41).

c). Two ø 12MM reinforcing bars were applied on the inner cross wall at the places equal to h/3 and 2h/3(h=storey height) (see Fig 42).

d). 50MM X 50MM grooves were cut along the edges of the cracks and put into cement-sand mortar with a ratio of 1 : 2.5 . As for the vertical crack, two ø 8MM cramp irons were fitted at an interval of 500 MM.

The two storeyed No. 2 building(see Fig 43),the four storeyed office building of the Hebei Geological Prospecting Party, the Guest house of the Dou River Power Generating Plant, (see Fig 44), the Exhibition Center(see Fig 45 taken after the earthquake; Fig 46 taken after restoration), and the office building of the 11th Middle School(see Fig 47 and Fig 48) were all repaired in the same way.

3. Repair of the assembly shop of the Electrical Machinery Plant.

The assembly shop was built in 1975. It was a single-storey workshop with precast I-shaped reinforced concrete columns and a steel truss with a span of 18M. It had large size roof boardings. The columns were spaced at 6M. The workshop was 90M in length with twin-column expansion joint. The level of the top of the column was 10.5M and that of the OX leg, 6.9M. A 15-ton crane was provided there. After the earthquake, the workshop inclined towards the south. The top of the column displaced with a magnitute of 340MM. Cracks were seen at both the roof and the upper part of the column and the concrete smashed in local places.

Repair method: At first, a 50-ton hoist and a 10-ton hoist were used to bring the columns at one side of the expansion joint, namely at one half of the workshop, to their vertical positions. However, the efforts met with a failure with the break-off of the wire rope. After thorough study, a 100MM deep notch was cut at the northern side of the column at a place about 800MM above the ground to lay bare the main reinforcing bars. Then six hoists(three for each part of the workshop) were used to bring the columns to their vertical positions. This met with great success. Finally the columns were cladded with reinforced concrete. The capital investiment was several hundred thousand Yuan less than that of building a new workshop(see Figs 49-54).

4. Repair of the metalwork shop.

The shop was located at a distance of 25M from the assembly shop. The precast concrete columns were rectangular in shape. It was provided with steel truss and cement roofing tiles. A 5 - ton crane was in use.

After the earthquake, the shop was seen to incline slightly towards the north. Some of the columns showed cracks at the roots. Due to the slight damage, the shop appeared to incline slightly. Therefore, it was left there as it was and only at the roots of the columns with cracks, reinforced concrete was applied(see Fig 55). But two years later, the columns initially without cracks(see Fig 56) were found to show cracks and so were the reinforced concrete claddings later applied. The shop showed marked inclination towards the north. Observation showed that the top of the column had inclined 140MM towards the north and the deformation was still underway. It showed that it was infeasible to repair the shop by mere strengthening without resorting to measures to correct it. Therefore, the shop needed to be repaired a second time.

5. Repair of the 36M high brick chimney of the No. 7 Porcelain Plant.

The bottom of the chimney was 3M in diameter(OD) and the upper part was 1.69M(OD). Wall thicknesses: Below 2.5M - 473 MM; 2.5 - 9.5M - 355MM; above 9.5M - 230MM. At the places of 9.5M and 16.5M, a 250MM high girth with six main reinforcing bars 12MM in diameter was installed each. The height of the reinforced concrete foundation was 700MM with a diameter of 5.3M. The steel ribbon hoops originally applied were spaced at 2 meters. After the earthquake, the chimney showed cracks and the upper five meters were seen to displace. The repair method used was to remove the top part and have it rebuilt. In addition, 18 (45X5 MM) angle steels were welded around the chimney body at different places(see Figs 57 and 58).

6. Repair of the 30M high chimney of the No. 3 Porcelain Plant.

The chimney with an O.D. diameter of 2.8M at the bottom showed cracks and dislocation of bricks after the earthquake. The repair method employed: The lower part above the foundation was cladded with a 1.5M high reinforced concrete ribbon.

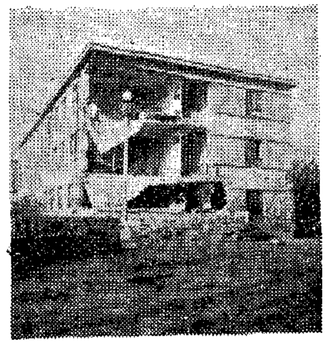
Above 15M, another 1.2M high reinforced concrete ribbon was applied. The uppermost 1.5M was removed and rebuilt. The reinforcing bars run through the whole chimney body from the top to the bottom. As for the part some 12M above the ground with exposed reinforcing bars, cement grouting was applied. The hoop reinforcement was welded to the vertical binders(see Figs 59 and 60).

7. Repair of the 60M high brick chimney of the Coking Plant.

During the earthquake, the upper 5 meters collapsed and vertical cracks appeared on the chimney body resulting in the dislocation of bricks. The repair method employed: 100MM thick reinforced concrete was applied at the height of 30-50M and the top 5 meters were rebuilt(see Fig 61).

Fig 1

Fig 2



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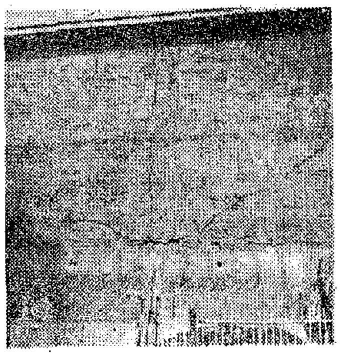

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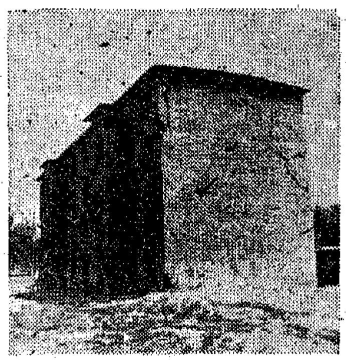


Fig 5

Fig 6

Fig 7

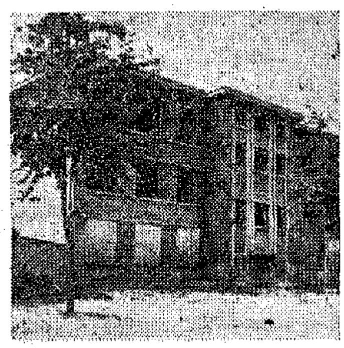


Fig 8

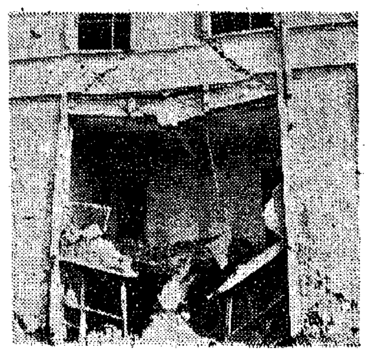


Fig 9

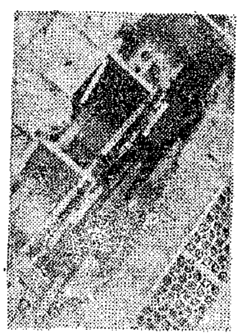
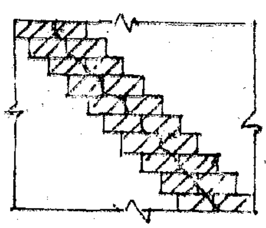


Fig10

Fig11



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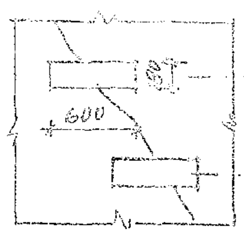

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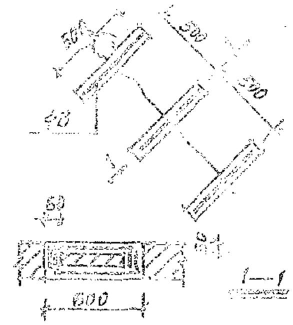

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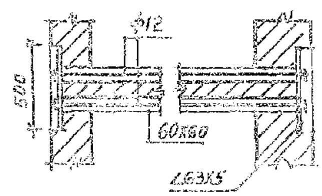

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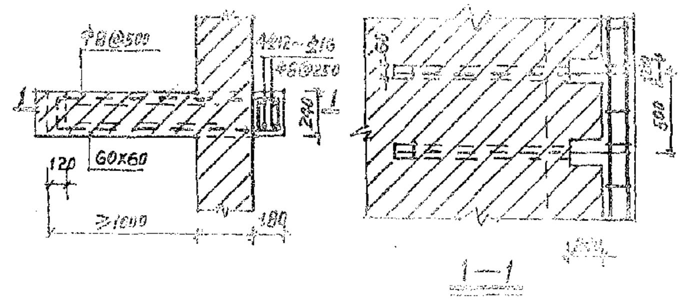

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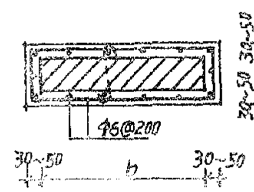

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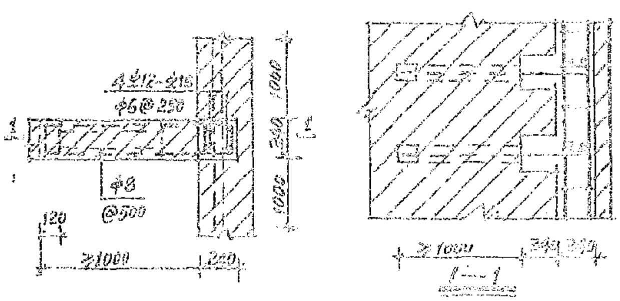

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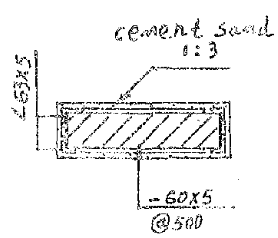

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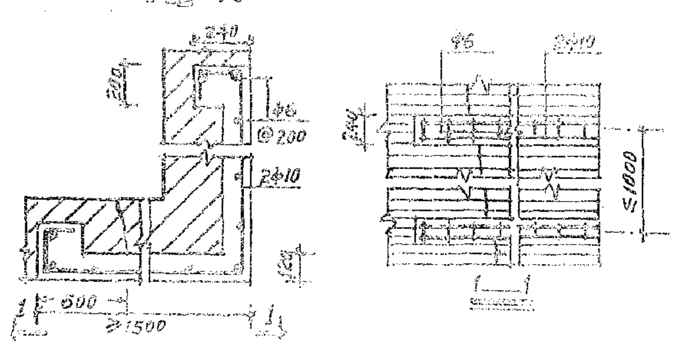

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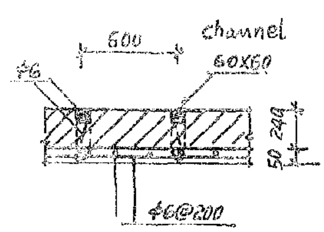

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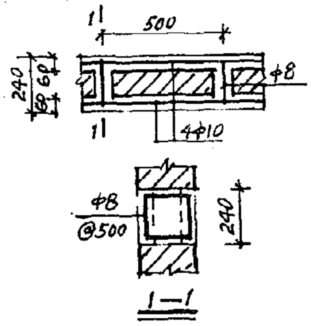
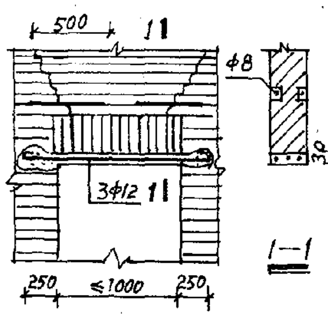


Fig 22



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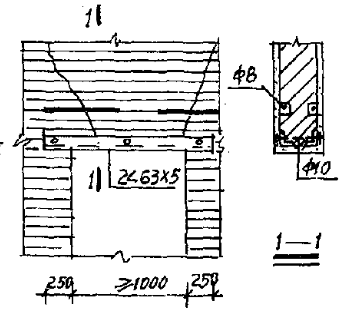


Fig 24

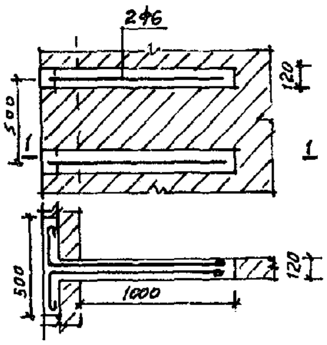


Fig 25

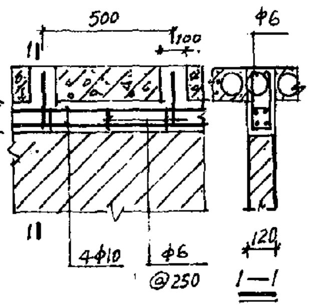
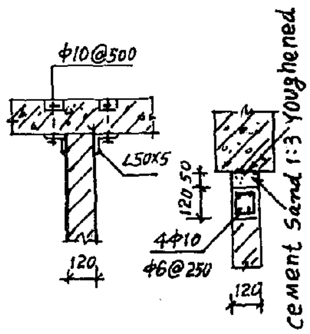


Fig 26



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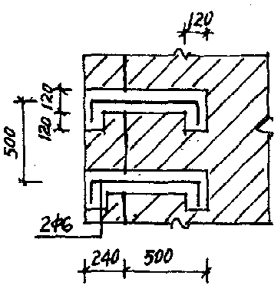


Fig 28

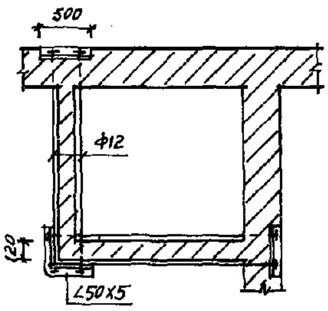


Fig 29

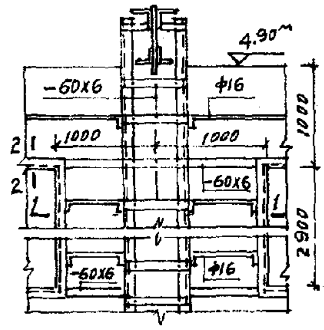
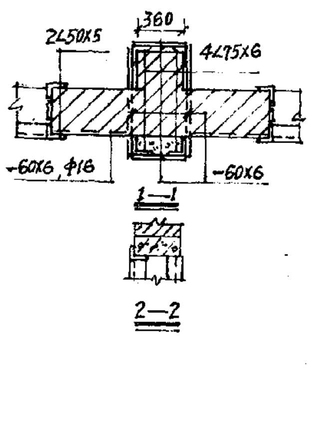


Fig 30



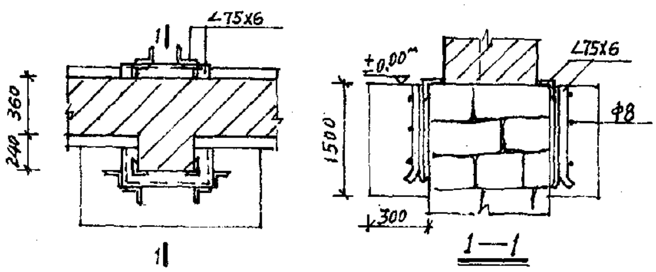
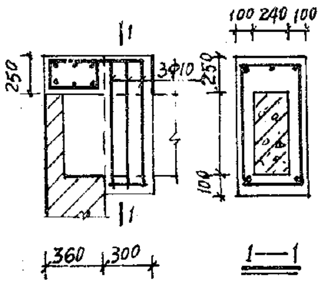


Fig 31



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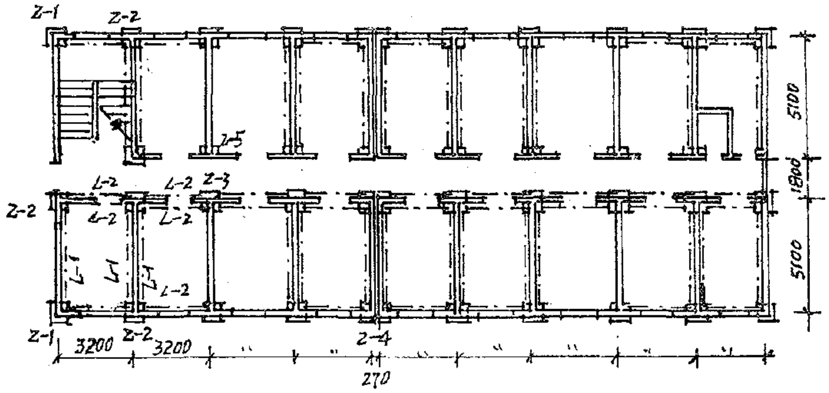
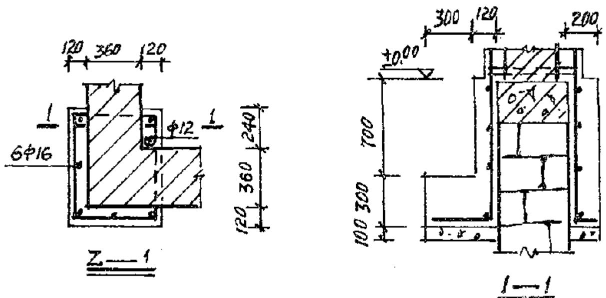


Fig 35



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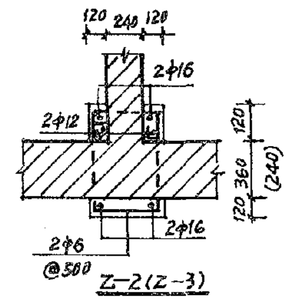

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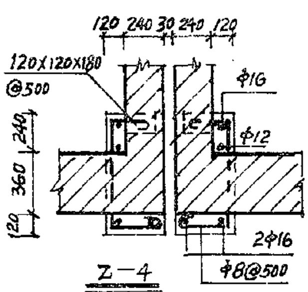

Fig 38

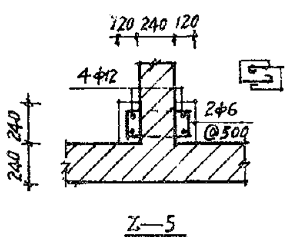

Fig 39

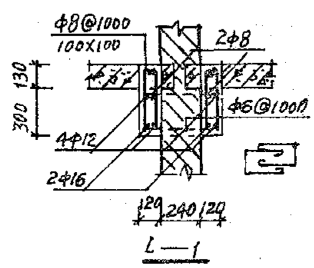

Fig 41

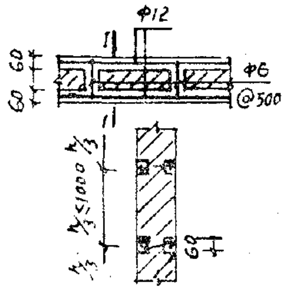

Fig42

Fig40

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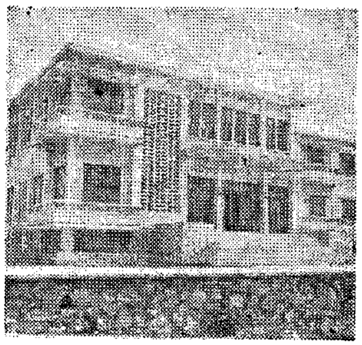

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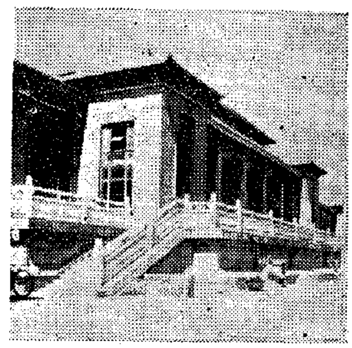

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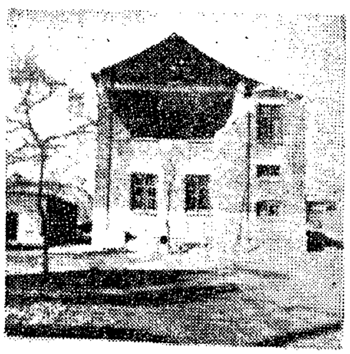


Fig47

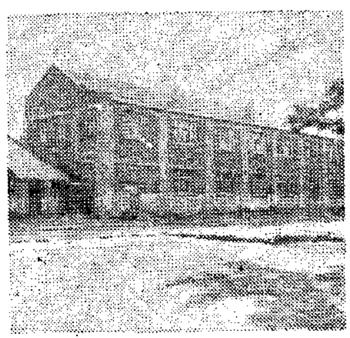
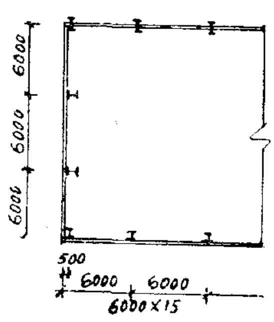


Fig48

Fig49

Fig50



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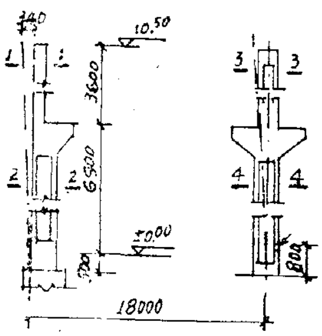

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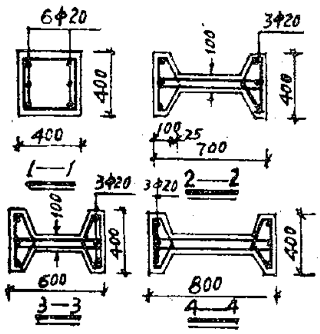

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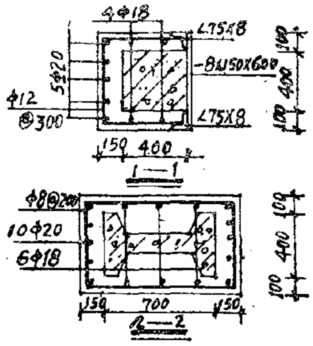

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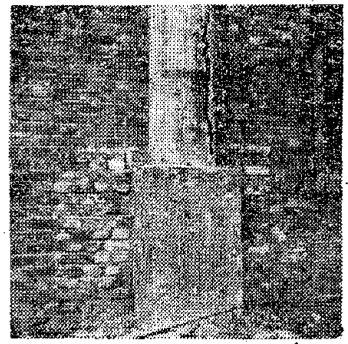

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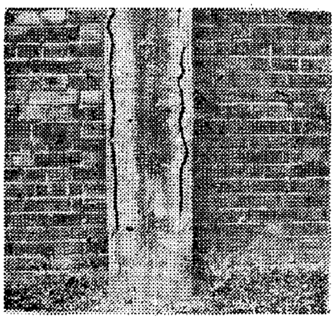

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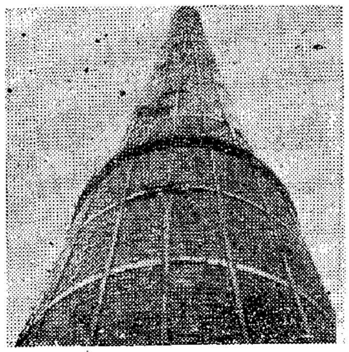

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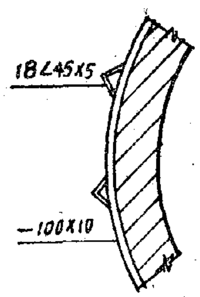

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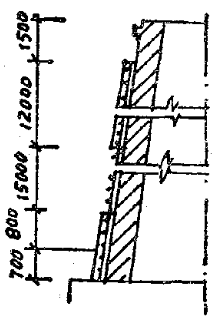

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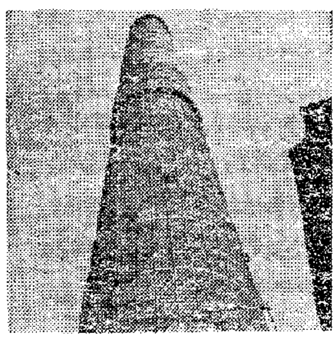

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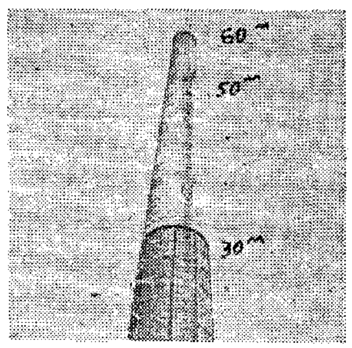

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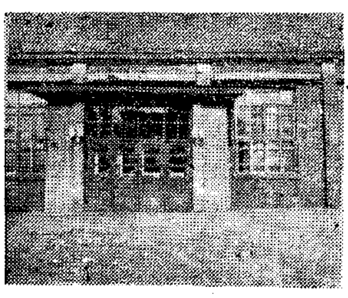

Fig33

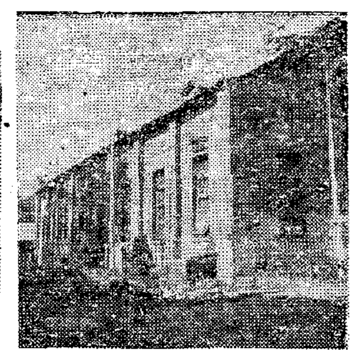

Fig34

EXPERIMENTAL RESEARCH ON THE SEISMIC BEHAVIOR OF MASONRY PIERS

by

Hugh D. McNiven

ABSTRACT

In an earthquake the most vulnerable buildings are those constructed of masonry and the most vulnerable part of the building is the pier between wall openings. If such piers could be made more resistant to damage, the chances for the loss of the building and even loss of life could be significantly reduced.

The experimental and analytical program conducted at the Earthquake Engineering Research Center at the University of California, Berkeley, is devoted to improving the seismic resistance of masonry piers. The paper describes the experimental program and its findings and how these results have led us to ways of improving not only the strength of a pier, but its ductility, the factors which must be improved if catastrophic damage is to be avoided.

INTRODUCTION

We are concerned in this paper with structures built of masonry, either brick or concrete block, and how such structures are apt to behave when subjected to seismic forces. If we consider masonry as a material and judge its properties in the simplest terms, we have to recognize that it is brittle and weak in tension, so that of all building materials it is potentially the most vulnerable to dynamic loads. I say potentially because we will learn in the course of the paper there is much that we can do to blunt these weaknesses.

What we have learned about masonry, and its response to seismic type loads, has come as the result of an experimental program extending back to 1972. The program is still active and our present plans are to continue it for several years. Accordingly, part of the paper represents accomplished fact, and part speculation derived from work we are now doing or have planned for the near future.

The object of the program is to try to improve the behavior of masonry when it is subjected to seismic loads. By "improve" we mean enhance the ability of masonry to undergo large deformation without severe damage and improve its ability to absorb energy. I will discuss later how we measure these abilities.

Given a particular masonry element (in this study a pier), there are a limited number of parameters that can influence its seismic response.

A. Type of Masonry - We study three kinds of masonry: Hollow Concrete

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Block (HCBL), Hollow Clay Brick (HCBR) and double wythe Solid Clay Brick (CBRC).

B. Geometry of Element - We study three rectangular piers whose height-to-width ratios are 2:1, 1:1 and 1:2.

C. Extent of Grouting - We study both partial and fully grouted piers.

D. Steel Reinforcing - Amount, location and distribution of vertical and horizontal reinforcing bars.

Of the four influences, the most effective for improvement is the reinforcing. Accordingly, the bulk of our attention in this paper will be directed to the effective use of reinforcing bars.

To understand what it is that we impose on the masonry, it is necessary to understand the test setups used in the experiments. There are three and all three will be discussed.

TEST SET UPS

After an extensive review of the literature dealing with earthquake resistance of masonry, we concluded that exterior wall panels penetrated by numerous window openings (Fig. 1) are the components of multi-story masonry buildings most frequently damaged in earthquakes, and accordingly, we decided to make an experimental study of the seismic behavior of such components. A testing fixture was designed to subject typical full-scale window piers to combined static vertical (gravity) and cyclic lateral (seismic) loads (Fig. 2). The test equipment permits lateral loads to be applied in the plane of the piers, using displacement controlled actuators with a maximum capacity of 450 kips. A vertical load may be applied to the piers through the springs and rollers shown above the spandrel beam in Fig. 2. The double pier tests were carried out with initial bearing stresses varying from zero to 500 psi.

With this load applied we were able next to impose a horizontal displacement at the top of the wall, while the bottom remained fixed. This displacement was applied by means of an actuator equipped with a load cell to record the load necessary to realize the displacement. The displacement was applied cyclically with a frequency of 0.02 Hz so that inertial effects are minimal. The amplitudes of displacement were imposed in groups of three and were increased montonically until the wall could no longer resist the horizontal load. We also followed the ability of the wall to sustain horizontal load after damage began and increased; that is after the horizontal resistance began to diminish.

The data resulting from a series of tests on a particular pier were plotted as horizontal resisting forces vs horizontal displacement, so that a hysteresis loop was formed for each cycle of loading. The complete data for a test would take the form of a series of hysteresis loops. A typical array of loops is shown in Fig. 3. For reasons that will be shown later, a line representing the envelope of the complete array of hysteresis loops, proves to be extremely useful. A number of such envelopes is shown in Figure 4.

These double pier tests were successful because the specimen used reproduced faithfully the conditions in a highrise building. Each specimen,

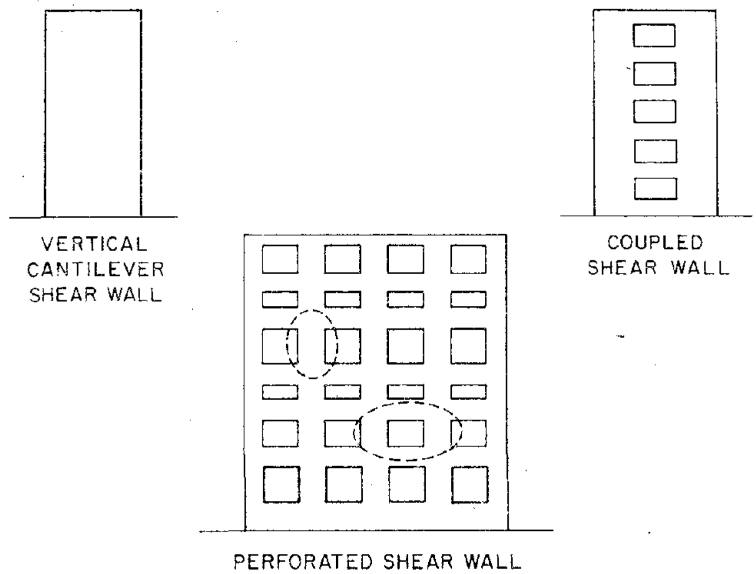


FIGURE 1: TYPICAL MASONRY BUILDING

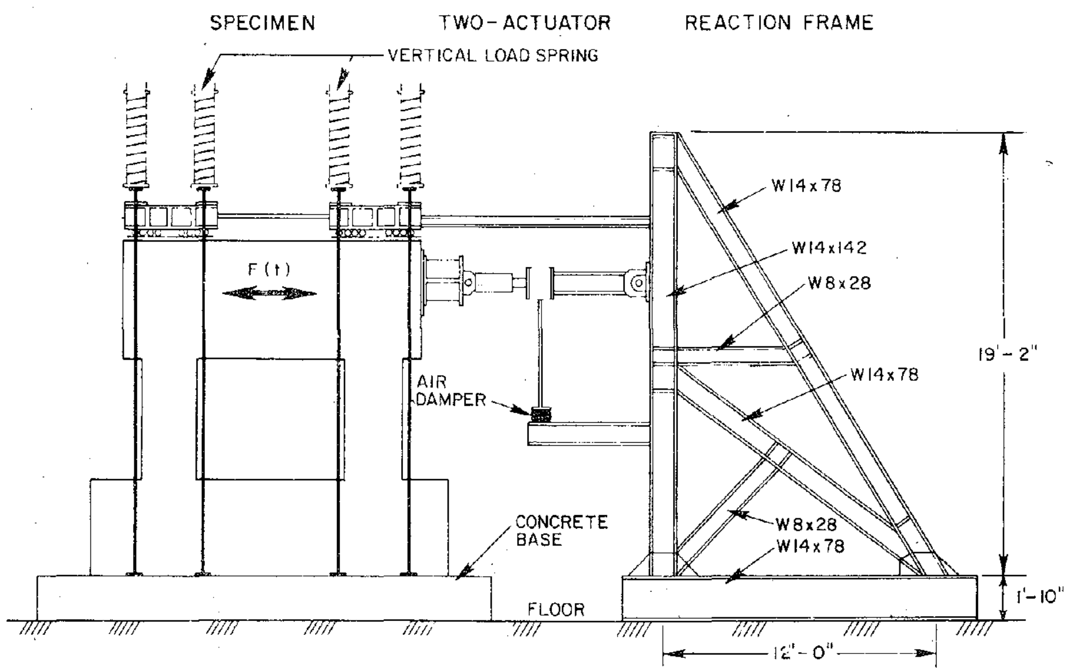
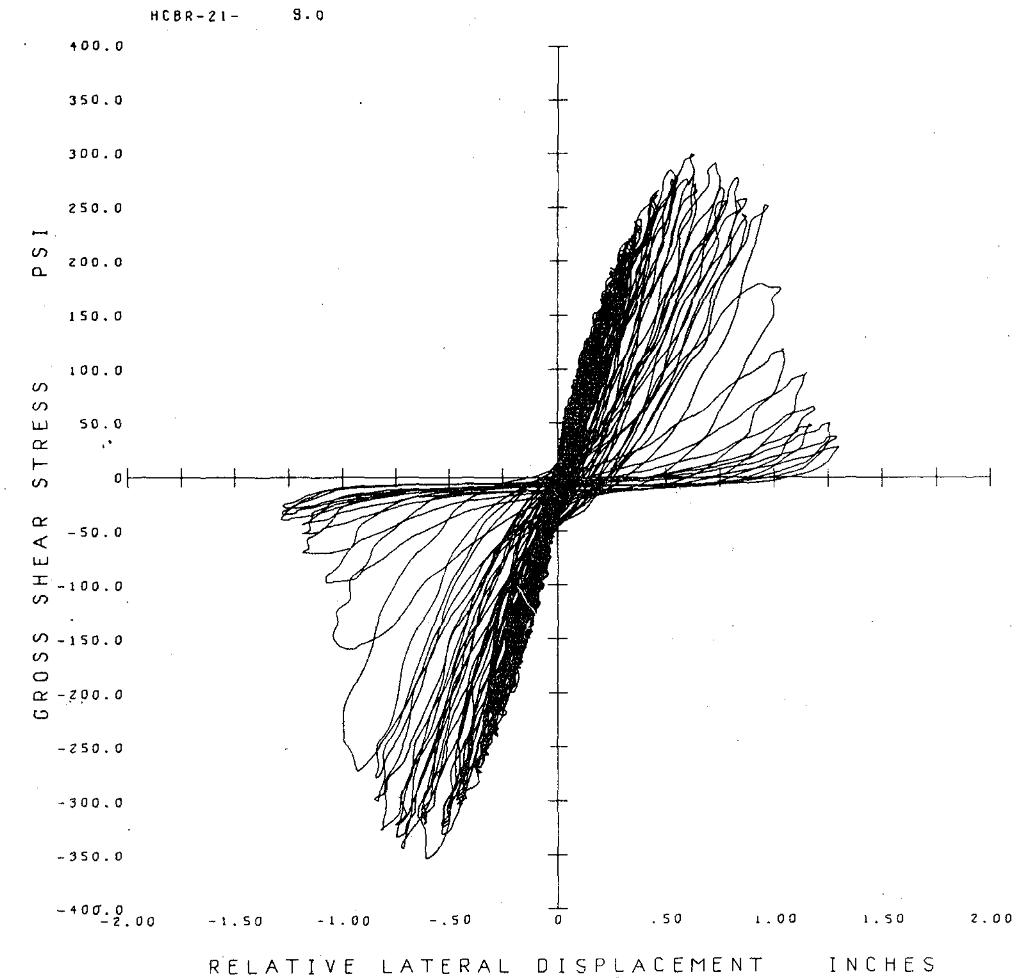


FIGURE 2: TEST SET UP FOR DOUBLE PIERS



SHEAR-STRESS-DEFLECTION DIAGRAM

FIGURE 3: ARRAY OF HYSTERESIS LOOPS

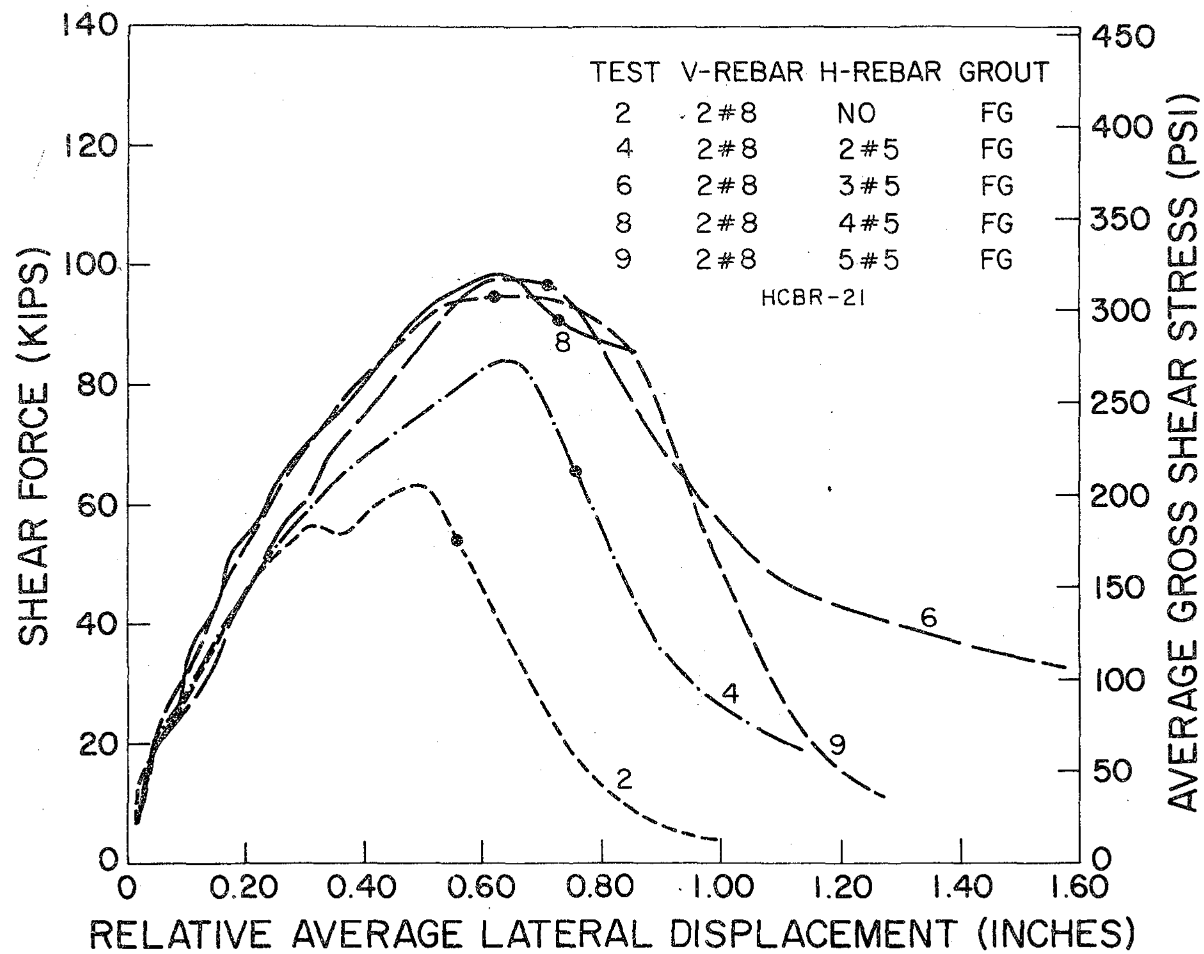


FIGURE 4: TYPICAL HYSTERESIS ENVELOPES

however, proved to be prohibitively expensive both in terms of time to build and test and in cost. Because of this, we decided to continue the program using single pier specimens. These could be tested for a fraction of the cost of the double pier specimens.

The first test set up for the single pier is shown in Fig. 5. The figure shows that rotation of the pier at the top is prevented by vertical steel members connecting boundary beams at the top and bottom.

Just recently we have changed the test set up in a significant way.(Fig.6). The vertical rotation constrainers have been replaced by actuators. These actuators can be used to impose a specified vertical load on the pier, each exerting the same downward force. Superimposed on these forces are additional equal and opposite actuator forces which impose a moment at the top resulting in a rotation. These vertical actuators are coupled by means of a servo-mechanism to the horizontal actuators which impose the horizontal displacements at the top of the pier. As the horizontal displacement changes, the force in the horizontal actuators changes, and in turn, the forces in the vertical actuators change and change by whatever amount we choose. In all our tests with this modified test set up to date we have chosen to keep the rotation at the top of the piers equal to zero simulating fixed boundary conditions.

The total experimental test program to the date of this paper consists of tests on 17 double pier specimens, 69 tests on single piers under the old test set up, and 20 on single piers with the new test set up.

TEST RESULTS

I have already noted that by recording a horizontal displacement, imposed by the horizontal actuator, and the resisting force of the test specimen corresponding to that displacement, we could subsequently plot a point on the force-displacement grid. A number of such points derived from a complete cycle of displacement form the basis of a hysteresis loop. In addition to these data, the vertical load was recorded corresponding to each increment of displacement. Variation of this vertical load during a test will prove to be significant. As well as recorded data, visual data was also recorded by means of a series of photographs. One side of the specimen was whitewashed to enhance an observor's ability to identify and trace the progress of cracking during a test. The cracks were marked with heavy black lines and numbered according to the increment of displacement producing the origin or progression of a crack. A photograph was taken immediately following the marking and identification of the crack progress. Typical progression failures are exhibited in Figures (7) and (8). The resulting set of photographs were important in identifying the mode of failure of a pier. This identification is extremely important in that different modes have to be treated differently when improvement of behavior is attempted.

MODES OF FAILURE

There are two principal modes of failure: flexural and shear (or diagonal tension) failures.

The flexural mode is identified when horizontal cracks develop at the top and bottom of a pier and the ultimate strength of the pier is controlled by the tensile yielding strength of the vertical reinforcing bars. The

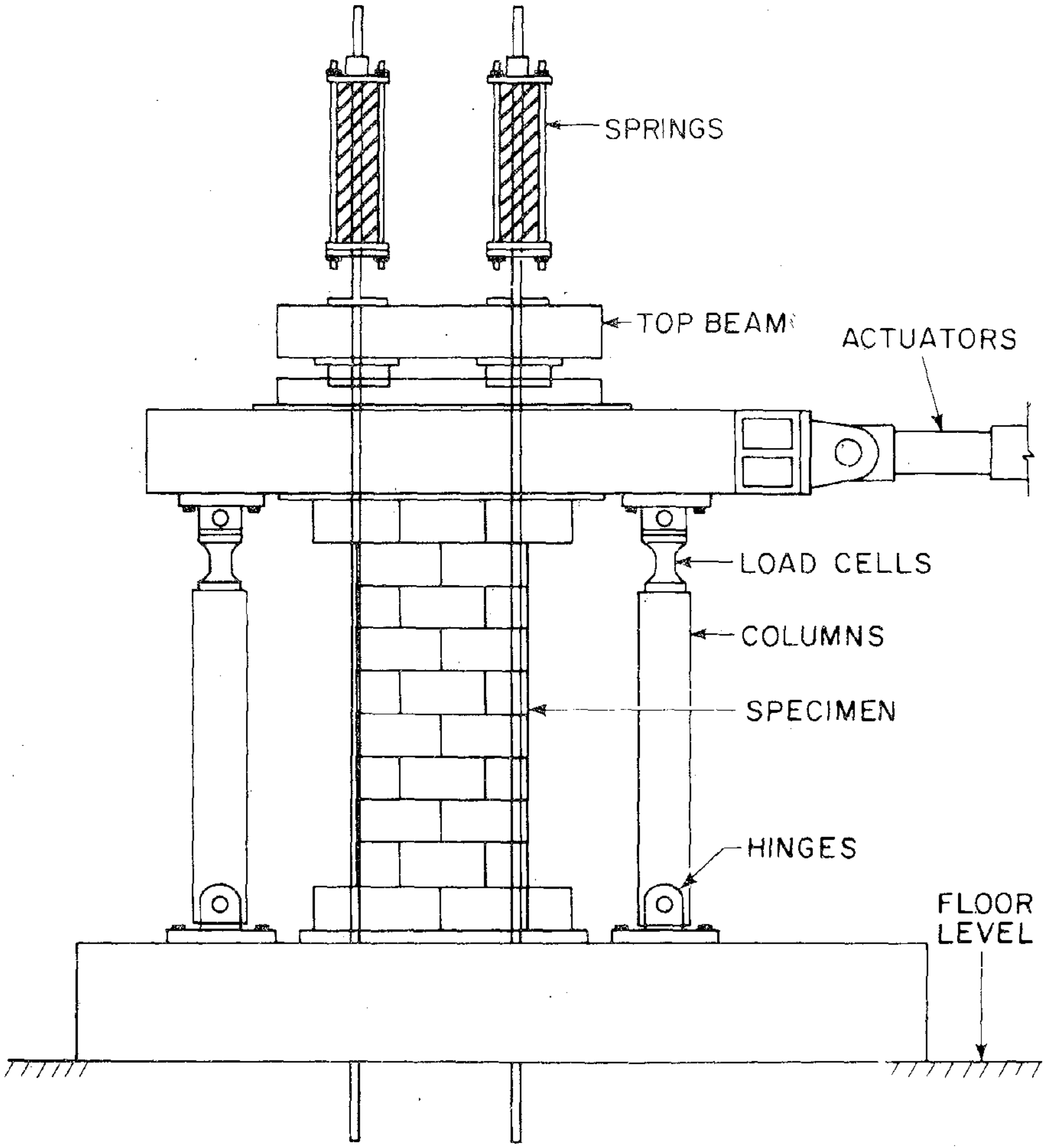


FIGURE 5. FIRST SINGLE PIER TEST SET UP

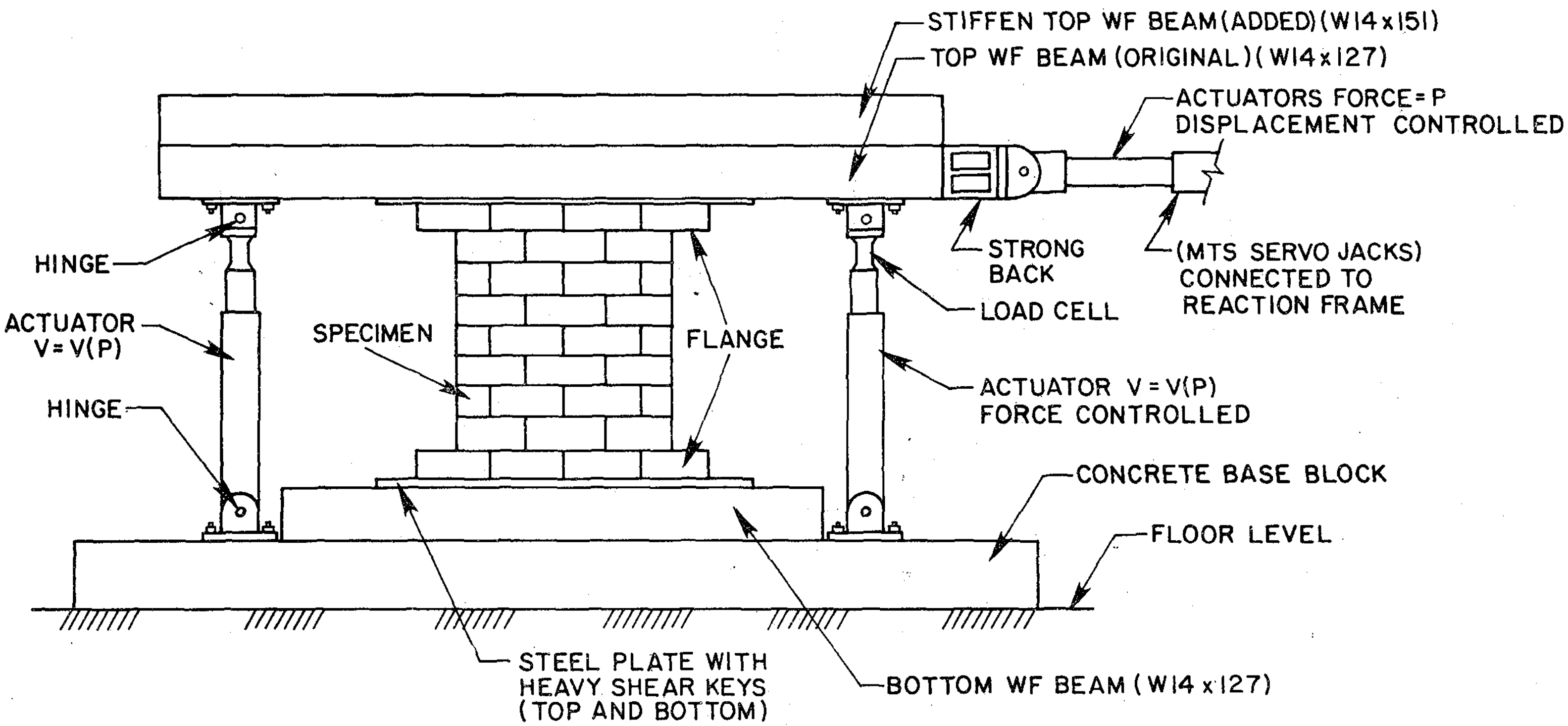
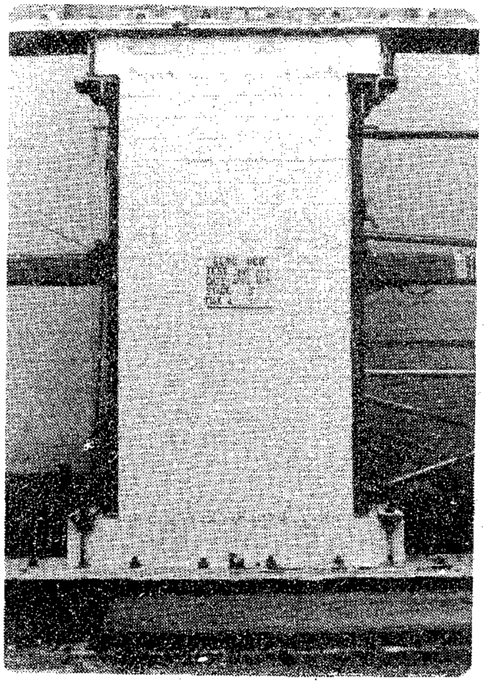
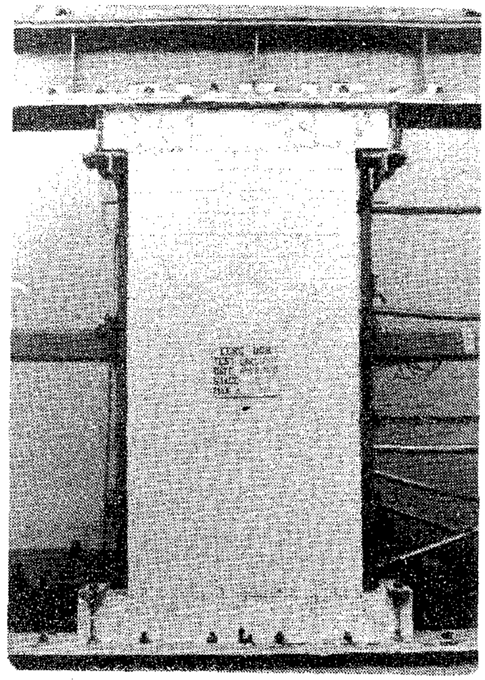


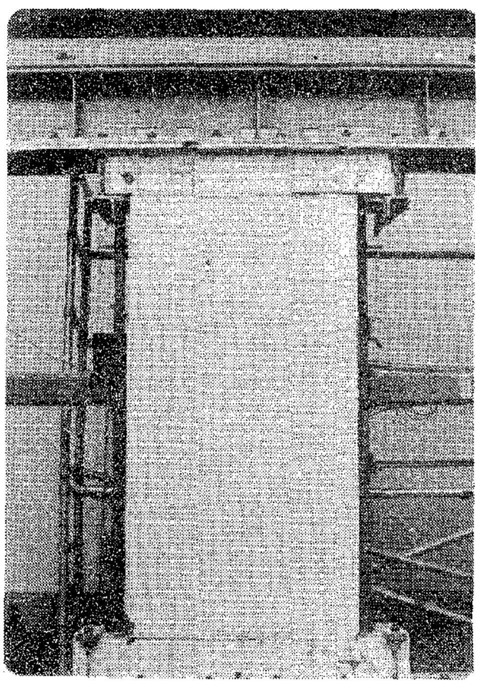
FIGURE 6. SECOND SINGLE PIER TEST SET UP



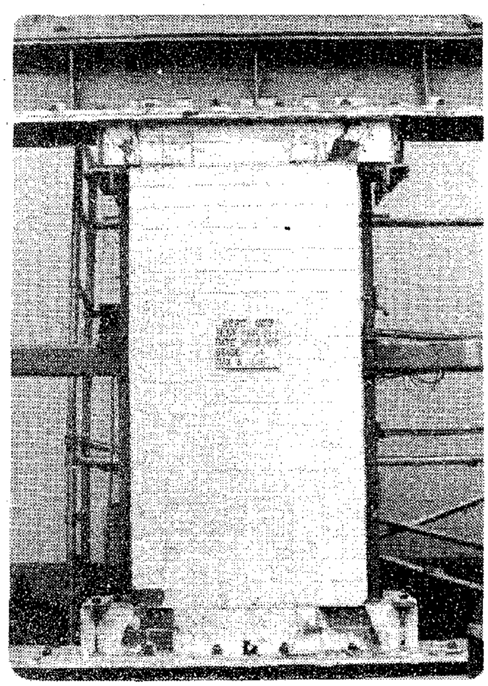
(A)



(B)



(C)



(D)

FIGURE 7. PROGRESS OF A FLEXURAL FAILURE

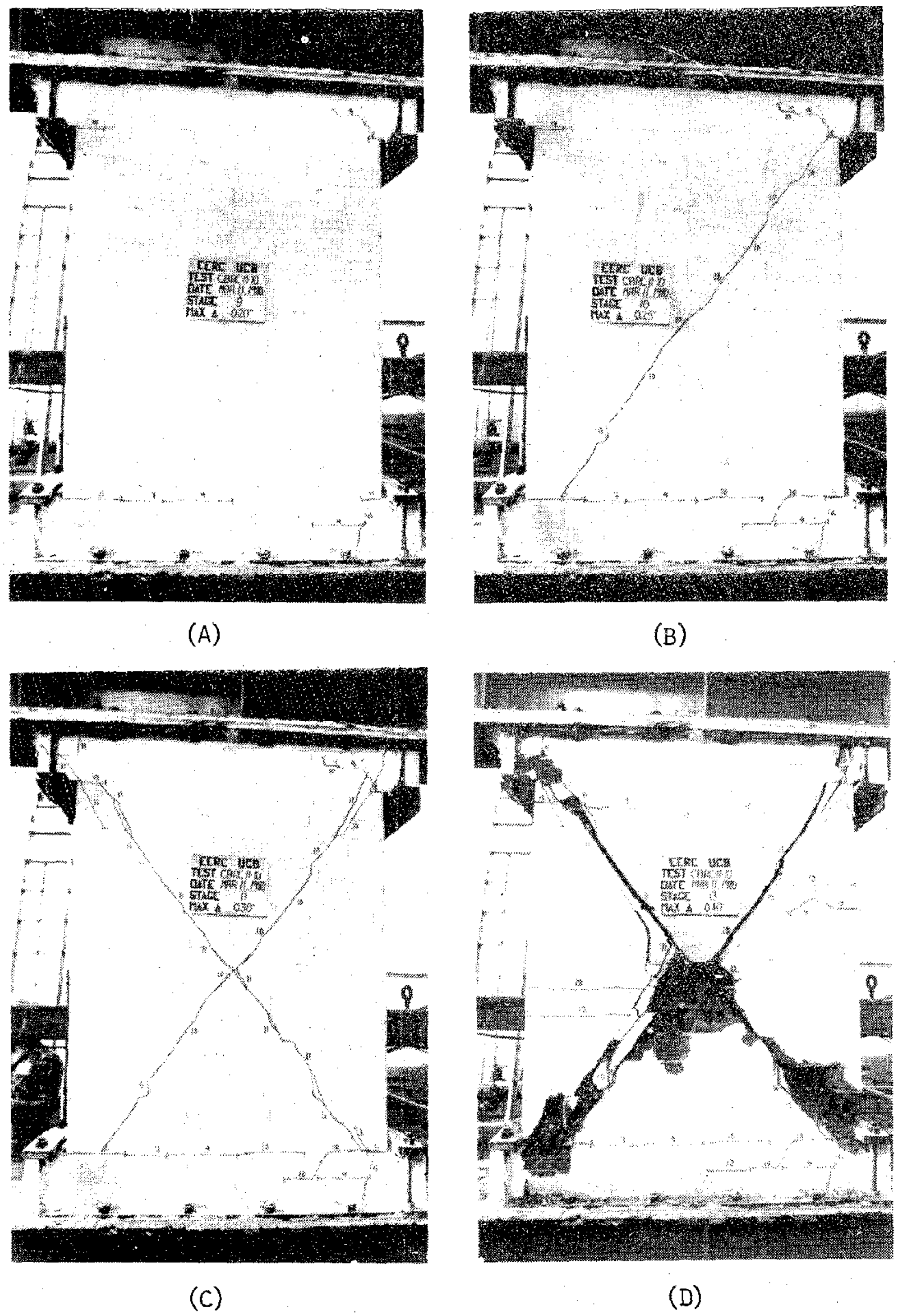


FIGURE 8. PROGRESS OF A SHEAR FAILURE

final mechanism of failure is usually due to crushing at the compression toe of the pier. Figure (7) shows the progress of a flexural failure.

The shear mode of failure is characterized by early flexural cracks, but which are soon augmented by diagonal cracks that extend through a partial zone of a pier. As the horizontal displacement increases, large diagonal cracks (X-cracks) form when the diagonal tensile stresses in the pier reach tensile strength capacity of the masonry. Figure (8) shows the progress of a shear failure.

Because most failures in past earthquakes have been characterized by diagonal cracks, many research programs have concentrated on this type of failure mechanism. Test techniques used by Greenley and Cattaneo [4], and others, induce diagonal tension or shear mode of failure. Scrivener [5], Meli [6], Williams [7], and Priestley and Bridgeman [8] recognize that there are two possible modes of failure for cantilever piers. In addition to the shear or diagonal tension mode of failure, they recognize that, for certain piers, a flexural failure can occur.

In our own experimental program we have been able to induce flexural failure in only two specimens in the double pier tests, and six specimens in the new test set up for single piers. With the old test set up all of the single piers exhibited shear failures. It was this characteristic of the old test set up that led us to examine all of the test data to ascertain if the type of failure is an inherent characteristic of the pier or is partly due to the test set up. By studying the actual vertical load on the pier we found that the initial load, which realistically should remain constant, actually increased as the horizontal displacement increased, meaning that the additional vertical load was due to the vertical steel constrainers. The large vertical load would not allow for a flexural failure, thus the failure was always in shear. The test results were valid, but for the somewhat special case when the vertical load is large, that is for piers at the lower stories of a high rise building. Furthermore, they are useful only up to the maximum resisting load. The inelastic behavior is inadmissible.

INELASTIC BEHAVIOR OF PIERS

It is of course important that a pier be designed so that the maximum horizontal resisting force be as large as possible. In terms of the safety of the structure, however, the behavior after failure begins is even more important. In unreinforced masonry, failure at the maximum load is sudden and severe. Our major effort at the present and in the immediate future is directed to improving this behavior.

We must return to the definition of "improvement" that I made at the beginning of the paper. "By improve we mean enhance the ability of masonry to undergo large deformation without severe damage and improve its ability to absorb energy". Since our program is primarily experimental we must have a measure of these behaviors. If we examine Figure 4 we see that the hysteresis envelope is a very good measure of both properties. We will be attempting to design piers whose experimental behavior results in an envelope that drops off gradually following the maximum load (that is the horizontal resisting load drops off gradually) and extends with this gradual slope as far as possible in the direction of imposed displacement. The area under this envelope is not exactly the energy absorbed by the pier but is a very good indicator of relative energies absorbed by different piers.

Earlier in the paper I described the factors that influence the response of masonry to seismic forces. Many of these: the type of masonry, the geometry of a pier, etc. may be beyond the control of the engineer. Without question the most single significant contribution to the behavior of a pier is the use of steel reinforcing. To get an insight into the best way to use reinforcing, we recognize the simularities and the differences between masonry and concrete. For the similarities we can benefit from design practice in reinforced concrete, for the differences we are on our own.

Masonry and concrete are similar in their weaknesses: both are brittle and are weak in tension. Masonry is different from concrete in that reinforcing can be placed only in two perpendicular directions and in itself does not provide anchorage. We will also find that the two-material nature of masonry introduces a problem not encountered in concrete.

The treatment that we envision is very much dependent on whether the mode of failure for the pier is flexural or shear. With the new test set up we can, by prescribing the vertical load and holding it constant, dictate whether the mode of failure for a given pier will be flexural or shear.

First, we study flexural failure. At this writing we have conducted six tests with the new test set up that have been designed to produce flexural failure. The hysteresis envelopes derived from all six tests show promise, but also room for improvement. In these tests the envelopes extend upward to the maximum resisting force, which coincides with the beginning of yielding of the vertical reinforcing bars in tension. The envelope then begins to extend horizontally, which is desirable, but drops off quickly at a displacement coincident with the beginning of crushing of the compression toe (see Fig. 7). Following crushing, a vertical crack propagates upward, above the crushing, followed usually by the spalling of a large section of the toe, exposing the vertical reinforcing, leaving it vulnerable to buckling. This is a problem that has not arisen in reinforced concrete elements in flexure.

We plan two quite different ways to improve the behavior. First, we plan to reduce the size of the vertical reinforcing bars in an attempt to increase the ductility of the pier (or in other words increase the extent in the horizontal direction of the hysteresis envelope) before crushing begins.

The more important improvement is to delay crushing by improving conditions at the compression toe. The idea for improvement comes from tests on prisms carried out in the program some time ago. At that time a theory was developed that we think is worth exploring. The theory is based on the recognition that the mortar and masonry have different Poisson's ratios, the ratio in the mortar being larger. When the compressive stress (which is the same for both) becomes severe, the horizontal strain in the mortar is considerably larger than that in the masonry, setting up large horizontal tensile stresses in the masonry, causing it to crack. This theory is supported by experiments. When thin steel plates were inserted in the mortar to reduce the horizontal strain, the crushing and spalling of the prisms was considerably delayed, allowing the prism to sustain much higher compressive loads before failure. Our future experimental programs will include a number of piers without plates and a number with plates in perhaps three courses in the vulnerable locations.

The need for improvement when the mode of failure is shear is even more urgent. In the new test set up a shear failure is easily induced simply by having the vertical load relatively large. Our tests to date have shown that, when the maximum load is reached, failure begins and progresses very rapidly so that, following maximum load, the envelope drops off very steeply. This is the most detrimental characteristic of masonry as a building material in seismic zones.

The horizontal resisting force begins to drop off when a major diagonal crack forms (see Fig. 8(B)). In reinforced concrete the same behavior is evident but is improved through the use of steel stirrups. The stirrups, by holding the concrete intact, ensure the integrity of the mass allowing for further increase in load.

We have at our disposal both vertical and horizontal reinforcing. For the 2:1 piers (Fig. 8), the orientation of the crack leads us to believe that horizontal reinforcing would be more effective than vertical. During the last year we have been focusing on the shear mode of failure, both with the purpose of qualifying results derived from tests with the old test set up and investigating new parameters. The parameters we have focused on so far are:

1. Variable amount of horizontal reinforcement.
2. Variable distribution of vertical reinforcement.
3. Different types of anchorage.

In all these tests the axial force has been held constant at 108 kips, corresponding to 400 p.s.i. axial stress, a heavy load.

The major findings and conclusions are the following:

1. Effect of horizontal reinforcement - Results from tests with the old test set up indicated that an increase in the horizontal reinforcement over a certain steel percentage (0.29%) was ineffective in increasing the ultimate shear capacity and ductility of the masonry piers. However, some doubt remained about the inelastic range due to the lack of full control over the axial forces imposed by the test set up.

   Testing with the new test set up verifies these results. There is no apparent improvement in either the ultimate shear capacity of the masonry piers or the inelastic behavior. This is concluded from the tests with a steel percentage of 0.20% and 0.50%.

2. Effect of distribution of vertical reinforcing - As Priestley [9] has maintained that vertical reinforcing bars and their distribution can be effective in delaying shear failure, we conducted several tests to explore this possibility.

   There are six tests to be examined when evaluating this parameter. These tests show no definite trend towards improving the pier behavior but some indications are apparent. These are:

   a. Ultimate strength increases with a better distribution of the vertical steel. This is somewhat contradicted by a lower strength for 6#4 bars than for 4#5 bars. However, this may be due to insufficient anchorage of the center four bars to the testing frame both at top and bottom in the former case. Tentative plans have been made to study this parameter further.

b. Similarly the ductility increases from HCBR-11-19 on one hand and HCBR-11-21 and HCBR-11-23 on the other, whereas no trend of increased ductility is observed for HCBR-11-20, HCBR-11-22 and HCBR-11-24.

3. Effect of the different types of anchorage - Three types of anchorage were tested - 90° hook, 180° hook and a closed loop consisting of two #4 bars welded to a steel plate at each end. The vertical bars were contained inside this closed loop. The closed loop with its 2#4 bars provided 28% more steel area than the other anchorage types with their #5 bar, but judging from item 1, above this does not noticeably affect the pier behavior.

It was found that the ultimate shear strength and the inelastic behavior (ductility) are vastly improved by changing the anchorage from the 90° hook to one of the other types of anchorages. On the other hand no definite improvement in the pier behavior is apparent when changing from the 180° hook to the plate type or vice versa, see Figure 9.

PLANS FOR FUTURE RESEARCH

1. Horizontal Reinforcing: As the improvement in the anchorage of the horizontal reinforcing has been the major contribution in improving the behavior of the pier, we will pursue this further. We plan to use, not reinforcing rods, but a commercial product called Dur-o-wall to extend the ductility perhaps even further. This is a truss made out of wire to be laid flat in the layer of mortar.

2. Flexural Failure: Flexural failure exhibits desirable characteristics but we feel that even this can be improved. We will endeavor to realize this improvement by reducing the size of the vertical bars and by using toe plates in the critical compression zones described earlier.

3. Type of Loading: To date our imposed horizontal displacement amplitude has increased monotonically. This is not very realistic, so we plan a series of tests in which the displacement amplitude resembles that imposed by an earthquake. We plan to contract a mathematical model which will predict the wall response and the response to an earthquake type input is necessary to construct the model.

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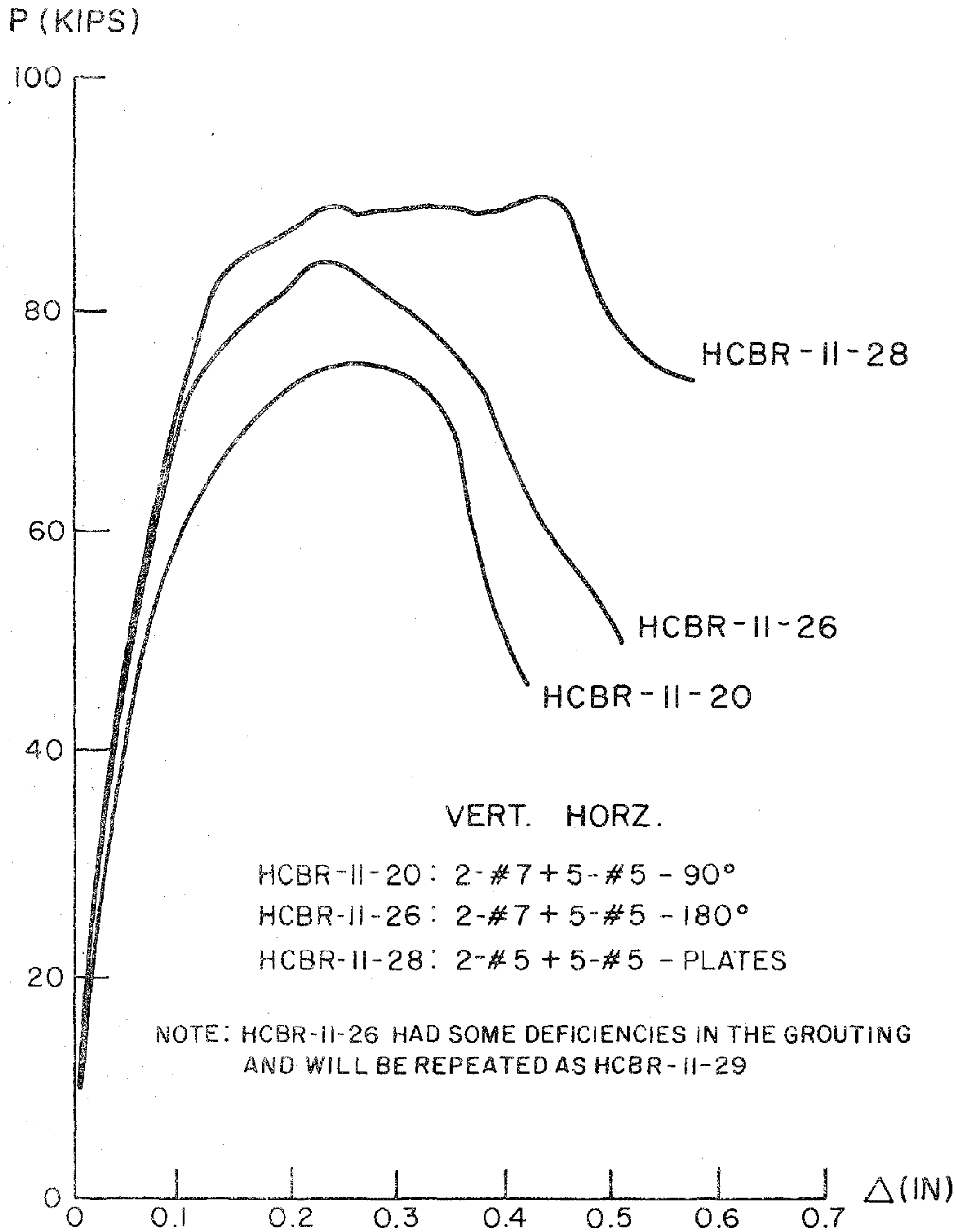

FIGURE 9. EFFECT OF ANCHORAGE ON THE DUCTILITY OF A PIER

3. Hidalgo, P.A., Mayes, R.L., McNiven, H. D. and Clough, R. W., "Cyclic Loading Tests of Masonry Single Piers, Volume 3 - Height to Width Ratio of 0.5," EERC Report No. 79/12, University of California, Berkeley, 1979.

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7. Williams, D. W., "Seismic Behaviour of Reinforced Masonry Shear Walls," Ph. D. Thesis, University of Canterbury, Christchurch, New Zealand, 1971.

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EARTHQUAKE HAZARD REDUCTION FOR EXISTING BUILDINGS IN THE CITY OF LOS ANGELES

by

EARL SCHWARTZ [1]

ABSTRACT

The evolution of the earthquake hazard reduction process for existing pre-1934 buildings in the City of Los Angeles is outlined from 1933 to the present. Emphasis on successful code enforcement is the underlining theme of this paper.

The adoption and implementation by the City of Los Angeles of the following program are detailed:

- Parapet correction program
- Limitation of use in unreinforced masonry buildings.
- Study program to reduce earthquake hazards in unreinforced masonry buildings.
- Study program to estimate reconstruction costs using proposed standards.
- Earthquake Hazard Reduction Program adopted in 1981 for pre-1934 unreinforced masonry buildings.

The efforts and progress by the City Council and the Deparment of Building and Safety in bringing about the adoption and implementation of the Earthquake Hazard Reduction ordinance are discussed. The code development process, testing programs and case histories of buildings strengthened using new earthquake standards are described. Social and economic factors are identified and explored.

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Department of Building and Safety
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THE NEED

The United States Government released a report[2] in 1973 indicating 20,000 people could be killed if a major earthquake struck populated areas of the Los Angeles Basin. The report attributed a major source of these fatalities to the area's large inventory of earthquake hazardous buildings. Damage studies[3-6] and reports from past earthquakes in Long Beach (1933) and San Fernando (1971), as well as many others throughout the seismic world have generally reached the same conclusion: Nonengineered unreinforced masonry buildings constitute the greatest threat to public safety because of the probability of collapse during an earthquake.

As these factors were realized, the City of Los Angeles, began to take positive steps to minimize the casualties that might occur in buildings as a result of earthquake damage. This was done at the municipal level because there is no mandatory national building code. Although there are some building codes that we used over large regions of the United States, it is the responsibility of local government to adopt and enforce their own building regulations. These regulations apply only to privately owned buildings as state and federally owned buildings are exempt from local laws.

PAST PROGRAMS

Initially, the City of Los Angeles adopted mandatory earthquake code requirements for new construction in October 1933. No requirements were established at that time for existing buildings. Consequently many pre-1934 earthquake hazardous buildings still exist. Because of this, certain earthquake regulations have been developed over the years.

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[2] National Oceanic and Atmospheric Administration, A Study of Earthquake Losses in the Los Angeles, California Area, U.S. Department of Commerce, 1973

[3] Joint Committee on Seismic Safety, Meeting the Earthquake Challenge, California Division of Mines and Geology as Spec. Pub. 45, 1974

[4] Task Force-Rachel Gulliver Dunne, Chairman, Consensus Report of the Task Force on Earthquake Prediction City of Los Angeles Mayor's Office, City of Los Angeles, October 1978

[5] Hazardous Building Committee, Hazardous Buildings, State of California - Seismic Safety Commission, March 8, 1979

[6] National Oceanic and Atmospheric Administrations, San Fernando, California, Earthquake of February 9, 1971, Vol. I Part B, 1973, pp 639-653

In 1949, a retroactive building code ordinance was adopted. This signaled the start of a successful 20-year parapet correction program. The unstable parapets adjacent to public ways of approximately 20,000 pre-1934 buildings were strengthened or removed. Many parapet walls were corrected as shown on Figure 1. Earlier damage reports from previous earthquakes throughout the west coast region of the United Stated demonstrated the danger and need for such an ordinance.

Many persons were being killed and injured by falling brick parapets as they tried to escape from earthquake shaken buildings. The value of the program was well demonstrated during the San Fernando Earthquake in 1971. Instances were cited where anchored walls remained in place, while unanchored walls on the same buildings failed and fell to the ground. In addition, installation of roof anchors tended to hold the building together so that complete collapse was prevented.

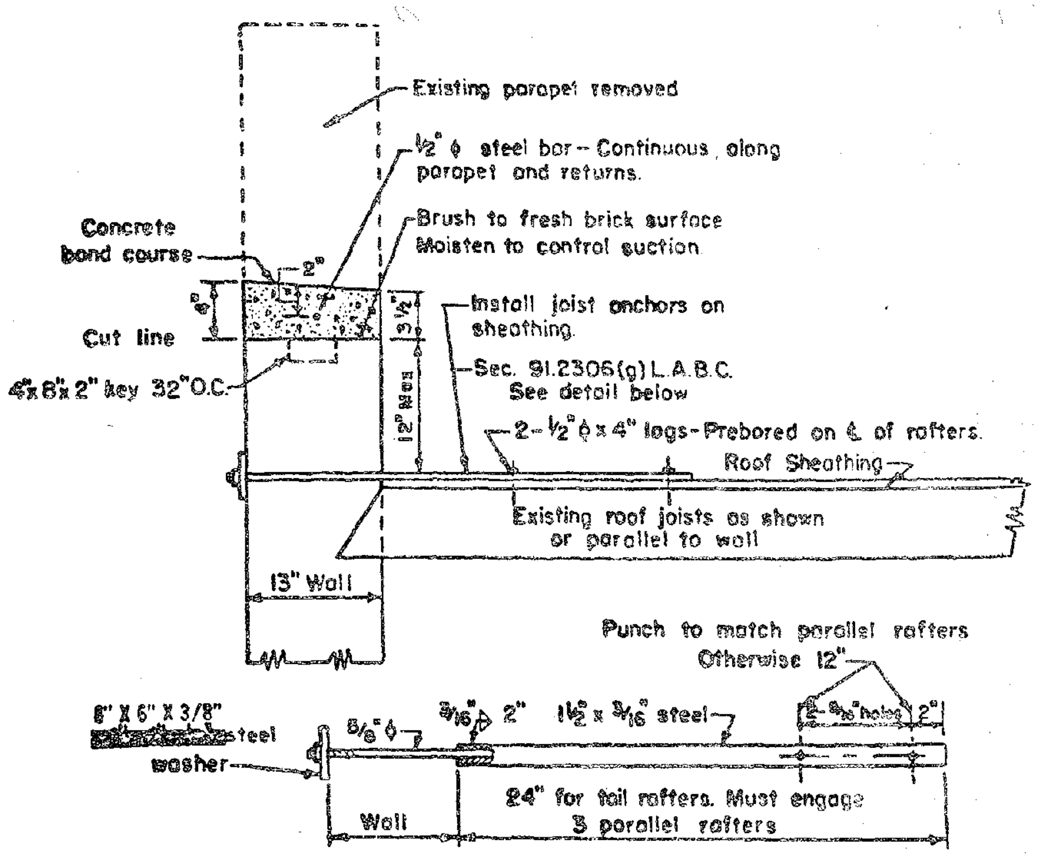
In 1965, the Department of Building and Safety of the City of Los Angeles adopted building regulations which set specific limitations and guidelined for buildings where a change to a more hazardous occupancy or an increase in occupant load was contemplated. These building are required to comply with current structural requirements. Because of high strengthening costs associated with updating to current Code standards, reuse of pre-1934 unreinforced masonry buildings were severely limited.

BACKGROUND LEADING TO EARTHQUAKE HAZARD REDUCTION

A City Council resolution in 1973 requested the Department of Building and Safety to perform a study in an effort to develop a program dealing with pre-1934 unreinforced masonry buildings. After many public hearings, an earthquake safety program was developed that included all pre-1934 unreinforced masonry buildings, with the exception of one and two family dwellings.

While this program was being developed, further interest and concern were generated with development and approval of the Seismic Safety Plan by the City Council in 1975. The Seismic Safety Plan is mandated by the State of California. It is now part of the General Plan for the City of Los Angeles and it specifies that a systematic, time-phased program be undertaken to strengthen or abate structures that do not meet Seismic Safety Standards, giving priority to pre-1934 unreinforced masonry structures.

City of Los Angeles
Department of Building & Safety



ANCHOR DETAIL

TYPICAL SKETCH "A"

Showing parapet correction along public ways or exit ways.

NOTES:

Check for need of permanent guardrail-
Sec. 91.4404 L.A.B.C.
Return bond course on all intersecting masonry walls as approved.
Maximum permissible spacing of through anchors 6 ft. o.c.

Arrange with the Department of Water and Power for any necessary relocation of power wires now located adjacent subject parapets.

SUPERINTENDENT OF BUILDING

B&S Form B-80 R8-60

FIGURE 1 - PARAPET CORRECTION DETAIL

After holding additional public hearings, the City Council in 1977, delayed action on the proposed program but did approve adoption of an earthquake study plan which included:

A city-wide survey to be conducted by the Department of Building and Safety over a period of two years for the purpose of identifying and cataloguing pre-1934 unreinforced masonry bearing wall buildings; and

A Special Earthquake Study Committee was to be appointed to develop a comprehensive earthquake safety ordinance for pre-1934 unreinforced masonry bearing wall buildings.

To implement the city-wide survey, the Department of Building and Safety established the Earthquake Safety Division under the direction of the author of this paper. Six building inspectors were specially trained to identify unreinforced masonry buildings and to gather specific information for each building identified. This information was recorded on computer-oriented forms to facilitate data cataloging. In addition, each building was photographed for identification purposes.

Los Angeles has 15 council districts, each divided into census tracts. Each inspector was assigned a council district which was inspected block by block and then by census tract until the entire council district was completed. Once a council district was surveyed, information from the inspection data forms was fed into the computer system. Records from the Parapet Correction Program, old building permits, and Sanborn Maps helped the inspection staff. The entire survey, covering 490 square miles, and the transfer of statistical data to the computer were accomplished in less than two years.

DEVELOPMENT OF CODE

While the Department of Building and Safety conducted its survey, the Special Earthquake Study Committee, comprised of numerous interested citizens, was selected. The Committee consisted of architects, engineers, geologists, building owners, property managers, attorneys, and members of the insurance and building industries. Two subcommittees were formed: The Technical Development Subcommittee, to formulate technical design standards for the new code; and the Impact Evaluation Subcommittee, to investigate social and financial ramifications of the proposed code. The initial and most important policy statement established by the Committee was that <u>life safety</u> be the basic philosophy behind the new proposal.

DEVELOPMENT OF CODE - Continued

Eighteen months later, the Committee produced "Division 68 - Earthquake Hazard Reduction in Existing Buildings", a proposed amendment to the Los Angeles Building Code (See Appendix). The proposed ordinance provided standards and procedures for identification and classification of unreinforced masonry bearing wall buildings based on their present use.

Priorities and limitations were also established under which building owners are officially notified and time limits for compliance were set. Priorities were based on the importance of the building and its occupant load capacity.

Reasonable performance standards for earthquake resistance were also included in Division 68. Each affected building is required to be structurally analyzed and the building's weak elements are then strengthened. These standards (Figure 2) are similar to the design criteria specified in the building code between 1934 and 1956. Allowable values are assigned to building elements as they exist and increased values are permitted when these elements are strengthened. Wood shear walls, which are not permitted to resist earthquake forces in new masonry construction, may now be used, under this proposal, to brace existing unreinforced masonry walls. Existing unreinforced masonry walls may not need any additional reinforcing if they meet these conditions:

The brick and mortar are tested to ensure they meet minimum strength requirements.

The wall height to thickness ratio and the earthquake stresses are within allowable values.

Division 68 allows flexibility and handling unusual circumstances through an appeal process. This permits consideration of alternative materials, methods of testing and various types of construction systems for earthquake strengthening.

TYPE OF BUILDING	FORCE FACTOR (% OF G)
Essential Buildings	0.186
High Risk Buildings	0.133
Medium Risk Buildings	0.100
Low Risk Buildings	0.100

FIGURE 2 - HORIZONTAL FORCE FACTORS

TESTING PROGRAM

To substantiate and document certain aspects of the proposed code, Ben Schmid[6], John Kariotis[7], George Battey[8], and the City of Los Angeles joined to develop and conduct a unique testing program on several existing masonry buildings. A condemned three-story apartment building was the major testing source. Several walls of the apartment building were prepared for full-scale in-place testing of wall sections, existing wall anchorage, and shear testing of newly installed bolts. In-plane bending and shear tests, diagonal compression tests and out-of-plane bending tests were performed on in-place wall sections.

In addition, a series of shear tests were conducted on individual bricks located in the exterior walls throughout the building. The purpose was to formulate a simple rapid field test as an alternative to core testing. Core testing proved unsatisfactory as a viable method of evaluating and determining mortar quality in most unreinforced masonry walls. Over 20 in-place shear tests, as shown on Figure 3, were made at various locations and on each floor level to correlate the shear value with variable dead-load superimposed on the individual brick. The bed joints of the outer wythe of the masonry were tested in shear by laterally displacing a single brick relative to the adjacent bricks in the wythe. The opposite head joint of the brick that was to be tested was removed and cleaned prior to testing. Minimum quality mortar occurs, as specified in the proposal, when 80 percent of the shear tests are not to be less than the total of 30 psi plus the axial stress in the wall at the point of the test. The shear stress is based on the gross area of both bed joints and occurs when movement of the brick is first observed.

The objective of the in-plane pier shear test was to determine the relationship between the shear tests on an individual brick versus the lateral strength of an entire pier. Two piers between three adjacent windows were prepared for simultaneous testing as shown on Figure 4. Suitable cribbing and two 30-ton calibrated jacks were installed. The piers were then pushed away from each other until cracks were formed.

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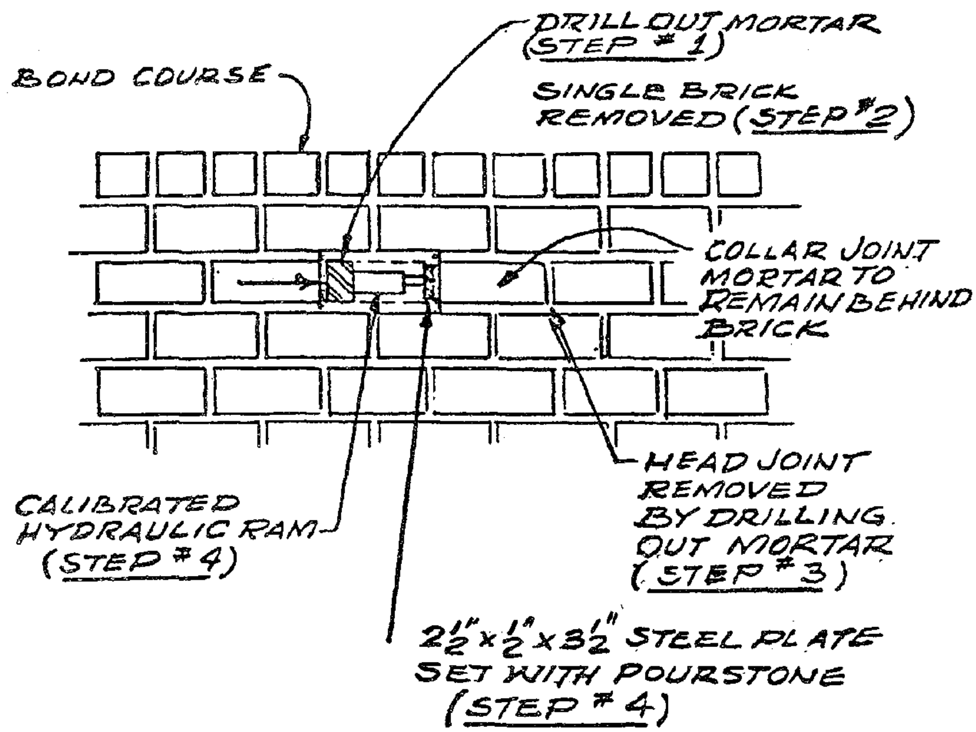
[6] Ben Schmid, Consulting Structural Engineer, Pasadena, California

[7] John Kariotis, President of Kariotis Associates, South Pasadena, California

[8] George Battey, President of Smith-Emery Co., Los Angeles California

IN-PLACE MORTAR PUSH TESTS



PROCEDURE:

STEP 1. EXISTING MORTAR DRILLED OUT WITH 5/16" DIAMETER MASONRY DRILL X 4" LONG.

STEP 2. REMOVE BRICK.

STEP 3. DRILL OUT HEAD JOINT MORTAR X 4" DEEP.

STEP 4. INSTALL JACK AND TEST.

STEP 5. "V", mortar (LBS./SQ. IN.) = $\dfrac{\text{P (LOAD IN LBS.)}}{\text{2 X FLAT AREA OF BRICK (SQ. INS.)}}$ (TYPICALLY 3-7/8" X 8")

FIGURE 3 - IN-PLACE BRICK AND MORTAR TEST

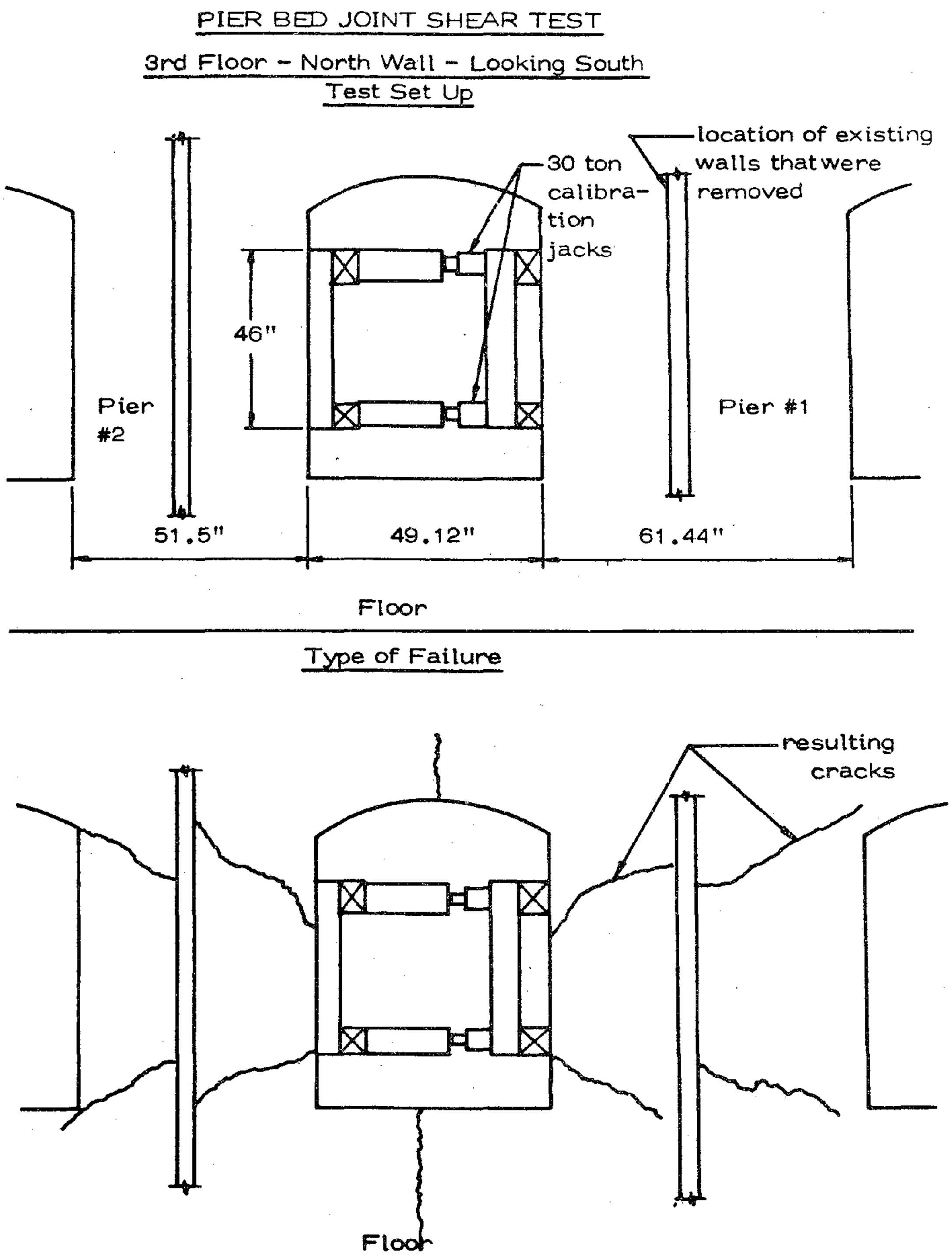


FIGURE 4 - DOUBLE PIER SHEAR TEST

Shear stresses of 23 and 27 psi were developed based on the gross area of the two piers. This compared with the individual brick shear test of 55 to 60 psi in the same location. The reason for this typical difference is due to large voids and poor quality of construction of the inner wythes. The shear value for the gross pier section was found to be in the order of 40 to 45% of the strength of the individual brick shear.

A diagonal compression test on a 48" X 48"(1.2 m. X 1.2 m.) section wall by 13"(33 cm.) thick as shown in Figure 5 was performed. A circular saw was used to cut a specimen in place in the wall. A 30-ton jack was located in the upper corner with a bearing angle at the opposite corner. The wallette was maintained completely free around the edges. Jack pressure was applied and the pier failed at a value of 24 psi. Another 30" (.76 m.) square diagonal compression test was also performed on the same wall resulting in a shear failure at 39 psi. These values are in close correlation with the wall piers shears that had been tested previously.

Field tests were also performed to determine the maximum compression value of the brick and mortar at the compression face of the wall. The test results indicated that these walls could develop an ultimate compressive strength of 400 to 500-psi, well above any forces that would be exerted in any of the planned large scale tests and also well above the maximum design allowable compressive strength of 100 psi permitted in the proposed code.

To evaluate the performance walls for perpendicular forces, a push-pull test was done on a 4'(1.2 m.) wide by 10'(3 m.) by 13"(33 cm.) wall specimen. Figure 6 shows the specimen with the jacking beam and the push-pull hydraulic jack at the mid-height of the wall. The force was applied, first by pushing the wall out and then reversing to pull in on the wall. The test load was increased by 800 pound increments until a maximum of 4800 pounds had been reached. At this point there was over an 80% G force being applied to the wall and the deflection of the wall had increased to 1/2"(1.3 cm.). Cracks that developed were readily visible on the plastered inside face of the wall and were measured and recorded. The wall specimen returned to the original wall position each time the wall was brought back to the zero force level. It was only when the pier had been subjected to more than 4000 pounds that it developed a permanent minor set.

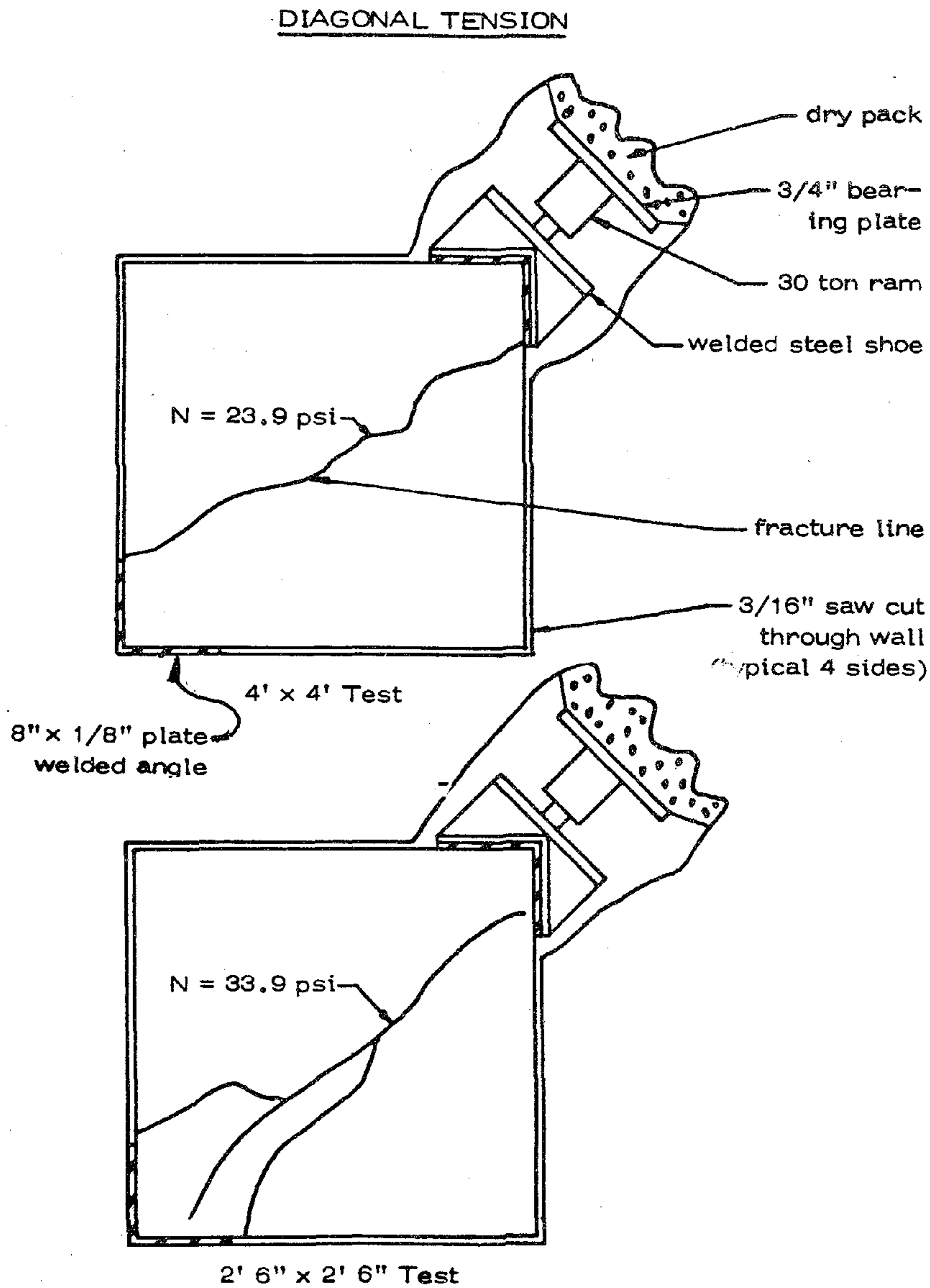


FIGURE 5 - DIAGONAL COMPRESSION TEST

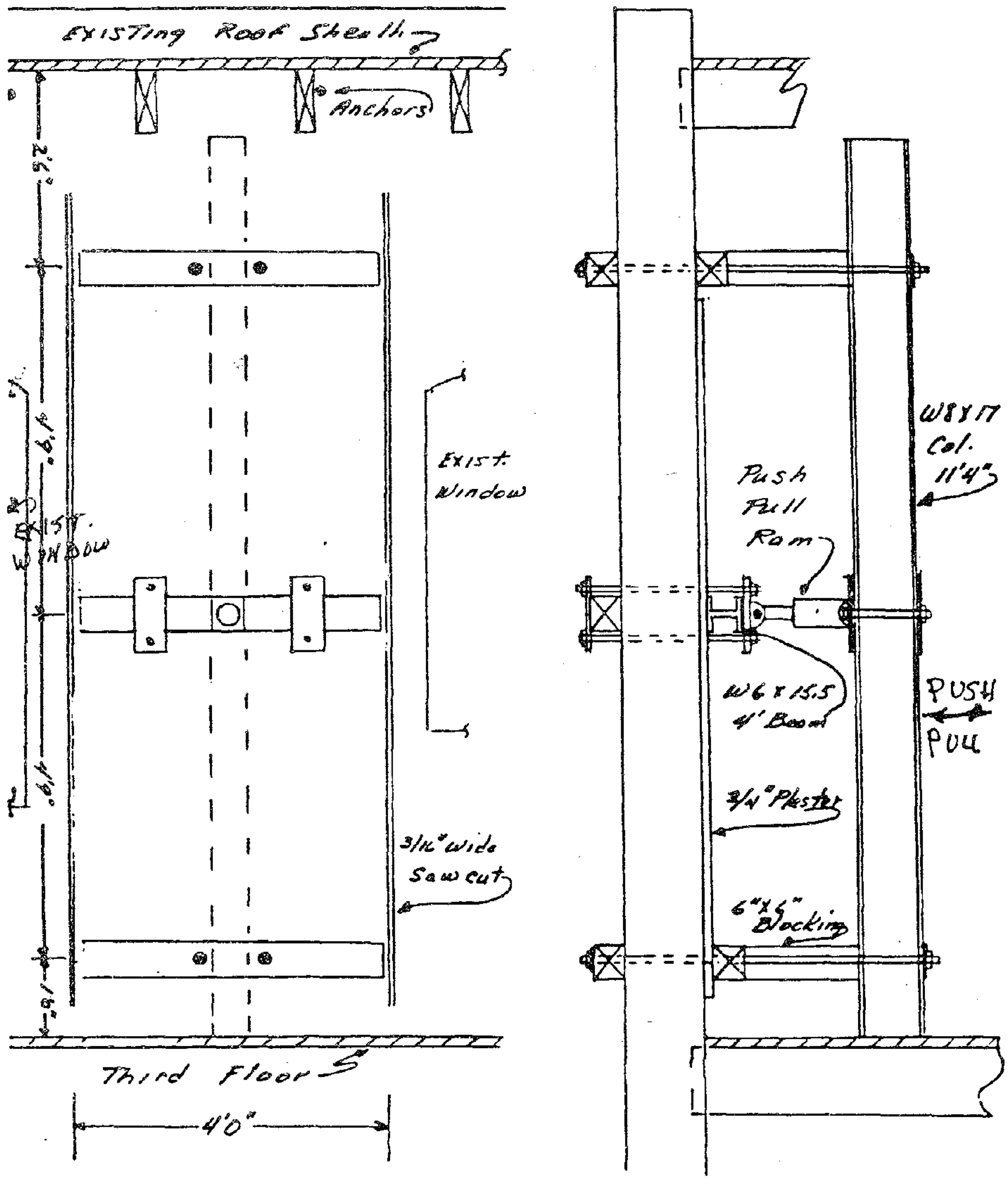


FIGURE 6 - OUT OF PLANE BENDING TEST OF WALL

TESTING PROGRAM - Continued

Since these older unreinforced masonry walls are fairly soft, connections had devised to properly attach these walls to the internal framing systems. It was found that the 2-1/2" (6.4 cm.) diameter holes 8"(20.3 cm.) deep could be readily drilled into those walls by unskilled workmen using a commercial core drill powered by an electric non-impact drill. By drypacking the bolt in the center of the hole an effective concrete plug was nominal bearing values. Various size bolts wood and steel ledgers were tested in both the vertical and horizontal directions. These tests confirmed that bolts installed as tested could be designed using the allowable shear values specified in current building codes standards for bolts in plain masonry.

Three originally installed wall anchors were tested in place for their masonry pull out capacity. These anchors which are attached to a pin embedded in the unreinforced masonry wall were 3/4"(1.9 cm.) in diameter. Properly installed wall anchors can be very beneficial in preventing walls from pulling away from a building during an earthquake. The anchors tested survived a 3000 pound pull test. More detailed test data is fully described in the Structural Engineers Association Convention Proceedings[9] of 1978.

CITY-WIDE SURVEY RESULTS

The results of the Department of Buildings and Safety's city-wide survey have been entered into a computer file. Programs have been completed to assess pertinent information relating to the 8,000 unreinforced masonry buildings that were identified as a result of the survey. These include:

A complete survey address list of all buildings;

A compilation of the total number of buildings by use as shown in Figure 7; and

A summation of floor area, dwelling units, employees and occupant load in each council district.

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[9] B. Schmid, J. Kariotis, and E. Schwartz, "Tentative Los Angeles Ordinance and Testing Program for Unreinforced Masonry Buildings", Structural Engineers Association Proceedings of 1978 pp 96-153

CITY-WIDE SURVEY RESULTS - Continued

Compiled survey data revealed that most of the unreinforced masonry buildings were concentrated in the older section of the city. Of the buildings identified, roughly 50% were commercial, 30% were industrial and approximately 20% were residential. In addition, the total floo area affected by the program is approximately 7.8 million square meters, and the total occupant load affected, as specif ed in the building code, exceeds 990,000 people. This information was extremely helpful in aiding the City Council in its deliberations regarding the proposed code.

USE	NUMBER OF BUILDINGS IDENTIFIED
COMMERCIAL	2,769
INDUSTRIAL	1,944
RESIDENTIAL	790
MIXED	1,583
GARAGE	502
OTHERS	243
TOTAL	7,831

FIGURE 7 - USES OF PRE-1934 UNREINFORCED MASONRY BUILDINGS SURVEYED

COST FACTORS

Two significant factors lead to the final approval of the code. First, in early 1980, Jack M. Fratt, General Manager of the Department of Building and Safety, awarded a contract to the firm of Wheeler and Gray[10] to analyze five sample buildings to determine the cost to strengthen these structures using the proposed standards. Since these code standards were new and unique, no past cost data was available for decision-makers. The summary of the cost study results as shown in Figure 8 was very helpful to the City Council in their deliberations. The study indicates that the average building could be strengthened to Division 68 standards for less then $10 per square foot (160 yuans/m.^{2}) of floor area.

[10] Wheeler and Gray, Consulting Structural Engineers, Los Angeles, California

BUILDING DESCRIPTION	NO. OF STORIES	PROJECT COST (1) TOTAL	PROJECT COST (1) PER SQ.FT. / SQ. M.	REPLACEMENT COST (2) TOTAL	REPLACEMENT COST (2) PER SQ.FT. / SQ. M.
Apartment 33,400 sq. ft. 3,600 sq. m.	4	\$ 208,000 (Y)359,000	\$ 6.22 (Y) 995	\$ 1,430,000 (Y)2,465,000	\$ 42.80 (Y) 6848
Apartment and Industrial 17,200 sq. ft. 1,850 sq. m.	3	\$ 207,000 (Y)357,000	\$ 12.08 (Y) 1933	\$ 756,000 (Y)1,303,000	\$ 40.00 (Y) 6400
Warehouse 6,400 sq. ft. 690 sq. m.	1	\$ 55,600 (Y) 95,800	\$ 8.70 (Y) 1392	\$ 166,000 (Y) 286,000	\$ 25.00 (Y) 4144
Industrial 10,800 sq. ft. 1,160 sq. m.	1+ mez.	\$ 86,500 (Y)149,000	\$ 7.90 (Y) 1264	\$ 281,000 (Y) 484,000	\$ 26.00 (Y) 4160
Commercial 14,000 sq. ft. 1,500 sq. m.	2	\$ 148,000 (Y)273,000	\$ 10.60 (Y) 1696	\$ 679,000 (Y)1,171,000	\$ 48.50 (Y) 7760
Average 16,400 sq. ft. 1,760 sq. m.		\$ 141,000 (Y)243,000	\$ 8.60 (Y) 1376	\$ 662,000 (Y)1,141,000	\$ 40.39 (Y) 6462

(1) Includes Contractor's Profit, Overhead, Contingencies, Engineering, Testing and Building Permit Fees.

(2) Based on Marshall and Swift Valuation Service.

(3) Based on April 1980 Costs from Wheeler & Gray report.

FIGURE 8 - SUMMARY OF COST STUDY BASED ON DIVISION 68 EARTHQUAKE STANDARDS (3)

ENCOURAGEMENT BY THE STATE OF CALIFORNIA

Although the State of California has not established seismic standards for existing buildings, guidance and encouragement are given to local jurisdiction to develop their own earthquake safety program. A state law was passed in 1980 which authorizes local jurisdictions to adopt reconstruction standards for earthquake hazardous buildings. In addition, as of this writing, the State of California is now considering legislation that would authorize a local seismic safety building rehabilitation loan program. This program would encourage building owners to strengthen their buildings by making favorable term loans available.

DEVELOPMENT OF A TWO-PHASE CONSTRUCTION SCHEDULE

An alternative compliance schedule was developed and added to the code by the Department of Building and Safety during the middle of 1980. This was the second significant factor leading to final code approval. The alternative which consists of a two-phase construction program affords building owners additional flexibility to complete the required structural strengthening. A complete wall anchoring system is required to be installed as the first phase in an accelerated period of time - within a year from the date the order is issued.

If the anchors are installed under this alternative, the time limits to complete the remaining structural strengthening (3 years) would automatically be extended from one to seven years depending on the rating classification of the subject building, see Figure 9 and 10. This additional time would allow building owners to better coordinate their construction time should they have tenant or resource problems. More importantly, the installation of wall anchors will dramatically increase the earthquake-resistant behavior of most buildings.

TIME LIMITS FOR BUILDING OWNER

REQUIRED ACTION BY OWNER	OBTAIN BUILDING PERMIT WITHIN	COMMENCE CONSTRUCTION WITHIN	COMPLETE CONSTRUCTION WITHIN
Complete Structural Alterations	1 year	180 days(1)	3 years
Wall Anchor Installation	180 days	270 days	1 year

(1) Measured from date of building permit issuance.

FIGURE 9 - DIVISION 68 TIME LIMITS

SERVICE PRIORITIES AND EXTENDED TIME PROVISIONS

RATING CLASSIFICATION	OCCUPANT LOAD	MINIMUM TIME PRIOR TO SERVICE OF ORDER	TIME EXTENSION FOR FULL COMPLIANCE
Essential	Any	0	1 year
High Risk	100 or more	90 days	3 years
Medium Risk	100 or more	1 year	5 years
	More than 50, but less than 100	2 years	6 years
	More than 19, but less than 51	3 years	6 years
Low Risk	Less than 20	4 years	7 years

FIGURE 10 - DIVISION 68 TIME LIMITS

CITY COUNCIL ADOPTS DIVISION 68

In January 1981, after eight years of study and deliberation, the City of Los Angeles adopted ordinance No. 154,807 which added Division 68 to the Building Code. The goal of the program as adopted is to strengthen the already identified 8,000 buildings affected by this program within the next 15 to 20 years and hopefully before the next earthquake. A successfully completed Earthquake Safety Program is expected to save thousands of lives when the next large earthquake shakes the Los Angeles Area. Highlights of the ordinance are shown below. The entire text of Division 68 may be found in the Appendix

HIGHLIGHTS OF DIVISION 68 AS ADOPTED

PURPOSE
1. To establish minimum earthquake standards for existing buildings.
2. To reduce risk of death and injury in the event of an earthquake.

SCOPE - Applies to all pre-1934 unreinforced masonry bearing wall buildings except for detached residential buildings having less than five dwelling units.

PRIORITIES - Four rating classifications are established to determine priorities for enforcement and magnitude of earthquake forces. See Figure 10.

GENERAL REQUIREMENTS - Building owners are required to hire a licensed engineer or architect to determine the building's earthquake deficiencies and to structurally alter these deficiencies, if any, to the established standards.

TIME LIMITS - Building owners are ordered to comply based on priority classification. Essential and high occupant capacity buildings will be required to comply first; low occupant load buildings will be last. See Figure 9 and 10 for actual limits.

APPEALS - An appeal procedure is established.

EARTHQUAKE STANDARDS - Earthquake performance standards are outlined. These standards reflect code levels in effect between 1940 and 1960 which are approximately 50 to 70 percent of the 1980 L.A. City Building Code level for new construction.

USE OF EXISTING MATERIALS - Allowable design values are listed for existing materials and the strengthening of existing materials.

EVALUATION OF UNREINFORCED MASONRY CONSTRUCTION - Test procedures are outlined to evaluate the strength of existing unreinforced masonry wall. See Figure 3.

FUTURE DEVELOPMENTS

Division 68 as adopted is not perfect. Experience, additional testing, and technology will bring improvements. An extensive testing program has recently been conducted in Los Angeles by A.B.K. - A joint venture of Agbabian, Barnes and Kariotis[11]. The testing, funded by the National Science Foundation, involved full-scale wall sections and various types of diaphragms which were subjected to simulated earthquake forces. In addition, the Structural Engineers Association of California is currently evaluating certain sections of Division 68. Results of the testing program and the deliberations of the Structural Engineers Association will improve and refine the standards of Division 68. These standards have and continue to be used as an earthquake safety guide for other communities throughout the United States.

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[11] A.B.K., a joint venture of Agbabian Associates, S.B. Barnes and Associates, Kariotis and Associates, Los Angeles Area.

CONCLUSION

The Earthquake Safety Division of the Department of Building and Safety is currently enforcing the regulations set forth in Division 68. Based on priorities established by the regulations, owners of essential buildings have already been served with compliance orders. Owners of high risk buildings are now being served with compliance orders based on the buildings occupant load. Some building owners are now strengthening their buildings to meet Division 68 standards on a voluntary basis and they have welcomed these standards as a viable and reasonable alternative to the present, more restrictive standards for new construction. It is the goal for the City of Los Angeles that all 8,000 orders be served by 1986 and that all the affected building be strengthened before the next earthquake.

APPENDIX

FULL TEXT OF DIVISION 68

Ordinance No. 154,807

An ordinance adding Division 68 of Article 1 of Chapter IX of the Los Angeles Municipal Code relative to earthquake hazard reduction in existing buildings.

Section 1. Article 1 of Chapter IX of the Los Angeles Municipal Code is hereby amended to add a Division 68 to read:

DIVISION 68 — EARTHQUAKE HAZARD REDUCTION IN EXISTING BUILDINGS

SEC. 91.6801. PURPOSE:

The purpose of this Division is to promote public safety and welfare by reducing the risk of death or injury that may result from the effects of earthquakes on unreinforced masonry bearing wall buildings constructed before 1934. Such buildings have been widely recognized for their sustaining of life hazardous damage as a result of partial or complete collapse during past moderate to strong earthquakes.

The provisions of this Division are minimum standards for structural seismic resistance established primarily to reduce the risk of life loss or injury and will not necessarily prevent loss of life or injury or prevent earthquake damage to an existing building which complies with these standards. This Division shall not require existing electrical, plumbing, mechanical or fire safety systems to be altered unless they constitute a hazard to life or property.

This Division provides systematic procedures and standards for identification and classification of unreinforced masonry bearing wall buildings based on their present use. Priorities, time periods and standards are also established under which these buildings are required to be structurally analyzed and anchored. Where the analysis determines deficiencies, this Division requires the building to be strengthened or demolished.

Portions of the State Historical Building Code (SHBC) established under Part 8, Title 24 of the California Administrative Code are included in this Division.

SEC. 91.6802. SCOPE:

The provisions of this Division shall apply to all builings constructed or under construction prior to October 6, 1933, or for which a building permit was issued prior to October 6, 1933, which on the effective date of this ordinance have unreinforced masonry bearing walls as defined herein.

EXCEPTION: This Division shall not apply to detached one or two story family dwellings and detached apartment houses containing less than five dwelling units and used solely for residential purposes.

SEC. 91.6803. DEFINITIONS:

For purposes of this Division, the applicable definitions in Sections 91.2301 and 91.2305 of this Code and the following shall apply:

Essential Building: Any building housing a hospital or other medical facility having surgery or emergency treatment areas; fire or police stations; municipal government disaster operation and communication centers.

High Risk Building: Any building, not classified an essential building, having an occupant load as determined by Section 91.3301(d) of this Code of 100 occupants or more.

EXCEPTION: A high risk building shall not include the following:

1. Any building having exterior walls braced with masonry crosswalls or wood frame crosswalls spaced less than 40 feet apart in each story.
2. Any building used for its intended purpose, as determined by the Department, for less than 20 hours per week.

Historical Building: Any building designated as an historical building by an appropriate Federal, State or City jurisdiction.

Low Risk Building: Any building, not classified an essential building, having an occupant load as determined by Section 91.3301(d) of less than 20 occupants.

Medium Risk Building: Any building, not classified as a high risk building or an essential building, having an occupant load as determined by Section 91.3301(d) of 20 occupants or more.

Unreinforced Masonry Bearing Wall: A masonry wall having all of the following characteristics:

1. Provides the vertical support for a floor or roof.
2. The total superimposed load is over 100 pounds per linear foot.
3. The area of reinforcing steel is less than 50 percent of that required by Section 91.2418(e) of this Code.

SEC. 91.6804. RATING CLASSIFICATIONS:

The rating classifications as exhibited in Table No. 68-A are hereby established and each building within the scope of this Division shall be placed in one such rating classification by the Department. The total occupant load of the entire building as determined by Section 91.3301(d) shall be used to determine the rating classification.

EXCEPTION: For the purpose of this Division, portions of buildings constructed to act independently when resisting seismic forces may be placed in separate rating classifications.

TABLE NO. 68-A
RATING CLASSIFICATIONS

Type of Building	Classification
Essential Building	I
High Risk Building	II
Medium Risk Building	III
Low Risk Building	IV

SEC. 91.6805. GENERAL REQUIREMENTS:

The owner of each building within the scope of this Division shall cause a structural analysis to be made of the building by a civil or structural engineer or arthitect licensed by the State of California; and, if the building does not meet the minimum earthquake standards specified in this Division, the owner shall cause it to be structurally altered to conform to such standards; or cause the building to be demolished.

The owner of a building within the scope of this Division shall comply with the requirements set forth above by submitting to the Department for review within the stated time limits:

a. Within 270 days after the service of the order, a structural analysis. Such analysis which is subject to approval by the Department, shall demonstrate that the building meets the minimum requirements of this Division; or

b. Within 270 days after the service of the order, the structural analysis and plans for the proposed structural alterations of the building necessary to comply to the minimum requirements of this Division; or

c. Within 120 days after service of the order, plans for the installation of wall anchors in accordance with the requirements specified in Section 91.6808(c); or

d. Within 270 days after the service of the order, plans for the demolition of the building.

After plans are submitted and approved by the Department, the owner shall obtain a building permit, commence and complete the required construction or demolition within the time limits set forth in No. Table 68-B. These time limits shall begin to run from the date the order is served in accordance with Section 91.6806(a) and (b).

TABLE NO. 68-B
TIME LIMITS FOR COMPLIANCE

Required Action By Owner	Obtain Building Permit Within	Commence Construction Within	Complete Construction Within
Complete Structural Alterations or Building Demolition	1 year	180 days*	3 years
Wall Anchor Installation	180 days	270 days	1 year

*Measured from date of building permit issuance.

Owners electing to comply with Item c of this Section are also required to comply with Items b or d of this Section provided, however, that the 270-day period provided for in such Items b and d and the time limits for obtaining a building permit, commencing construction and completing construction for complete structural alterations or building demolition set forth in Table No. 68-B shall be extended in accordance with Table No. 68-C. Each such extended time limit, except the time limit for commencing construction shall begin to run from the date the order is served in accordance with Section 91.6806 (b). The time limit for commencing construction shall commence to run from the date the building permit is issued.

TABLE NO. 68-C
EXTENSIONS OF TIME AND SERVICE PRIORITIES

Rating Classification	Occupant Load	Extension of Time if Wall Anchors are Installed	Minimum Time Periods for Service of Order
I (Highest Priority)	Any	1 year	0
II	100 or more	3 years	90 days
III	100 or more	5 years	1 year
	More than 50, but less than 100	6 years	2 years
	More than 19, but less than 51	6 years	3 years
IV (Lowest Priority)	Less than 20	7 years	4 years

SEC. 91.6806. ADMINISTRATION:

(a) Service of Order. The Department shall issue an order, as provided in Section 91.6806(b), to the owner of each building within the scope of this Division in accordance with the minimum time periods for service of such orders set forth in Table No. 68-C. The minimum time period for the service of such orders shall be measured from the effective date of this Division. The Department shall upon receipt of a written request from the owner, order a building to comply with this Division prior to the normal service date for such building set forth in this Section.

(b) Contents of Order. The order shall be written and shall be served either personally or by certified or registered mail upon the owner as shown on the last equalized assessment, and upon the person, if any, in apparent charge or control of the building. The order shall specify that the building has been determined by the Department to be within the scope of this Division and, therefore, is required to meet the minimum seismic standards of this Division. The order shall specify the rating classification of the building and shall be accompanied by a copy of Section 91.6805 which sets forth the owner's alternatives and time limits for compliance.

(c) Appeal From Order. The owner or person in charge or control of the building may appeal the Department's initial determination that the building is within the scope of this Division to the Board of Building and Safety Commissioners. Such appeal shall be filed with the Board within 60 days from the service date of the order described in Section 91.6806(b). Any such appeal shall be decided by the Board no later than 60 days after the date that the appeal is filed. Such appeal shall be made in writing upon appropriate forms provided therefor, by the Department and the grounds thereof shall be stated clearly and concisely. Each appeal shall be accompanied by a filing fee as set forth in Table 4-A of Section 98.0403 of the Los Angeles Municipal Code.

Appeals or requests for slight modifications from any other determinations, orders or actions by the Department pursuant to this Division, shall be made in accordance with the procedures established in Section 98.0403.

(d) Recordation. At the time that the Department serves the aforementioned order, the Superintendent of Building shall file with the Office of the County Recorder a certificate stating that the subject building is within the scope of Division 68 — Earthquake Hazard Reduction in Existing Buildings — of the Los Angeles Municipal Code. The certificate shall also state that the owner thereof has been ordered to structurally analyze the building and to structurally alter or demolish it where compliance with Division 68 is not exhibited.

If the building is either demolished, found not to be within the scope of this Division, or is structurally capable of resisting minimum seismic forces required by this Division as a result of structural alterations or an analysis, the Superintendent of Building shall file with the Office of the County Recorder a certificate terminating the status of the subject building as being classified within the scope of Division 68 — Earthquake Hazard Reduction in Existing Buildings — of the Los Angeles Municipal Code.

(e) Enforcement. If the owner or other person in charge or control of the subject building fails to comply with any order issued by the Department pursuant to this Division within any of the time limits set forth in Section 91.6805, the Superintendent of Building shall order that the entire building be vacated and that the building remain vacated until such order has been complied with. If compliance with such order has not been accomplished within 90 days after the date the building has been ordered vacated or such additional time as may have been granted by the Board and the Superintendent may order its demolition in accordance with the provisions of Section 91.0103(o) of this Code.

SEC. 91.6807. HISTORICAL BUILDINGS:

(a) General. The standards and procedures established by this Division shall apply in all respects to an historical building except that as a means to preserve original architectural elements and facilitate restoration, an historical building may, in addition, comply with the special provisions set forth in this Section.

(b) Unburned Clay Masonry or Abode. Existing or re-erected walls of abode construction shall conform to the following:

1. Unreinforced abode masonry wall shall not exceed a height or length to thickness ratio of 5, for exterior bearing walls and must be provided with a reinforced bond beam at the top, interconnecting all walls. Minimum beam depth shall be 6 inches and a minimum width of 8 inches less than the wall width. Minimum wall thickness shall be 18 inches for exterior bearing walls and 10 inches for abode partitions. No abode structure shall exceed one story in height unless the historic evidence indicates a two story height. In such cases the height to thickness ratio shall be the same as above for the first floor based on the total two-story height and the second floor wall thickness shall not exceed the ratio 5 by more than 20 percent. Bond beams shall be provided at the roof and second floor levels.

2. Foundation footings shall be reinforced concrete under newly reconstructed walls and shall be 50 percent wider than the wall above, soil conditions permitting, except that the foundation wall may be 4 inches less in width than the wall above if a rock, burned brick, or stabilized adobe facing is necessary to provide authenticity.

3. New or existing unstabilized brick and abode brick masonry shall test to 75 percent of the compressive strength as set forth in Section 91.2405(f) of this Code. Unstabilized brick may be used where existing bricks are unstabilized and where the building is not susceptible to flooding conditions or direct exposure. Abode may be allowed a maximum value of 3 pounds per square inch for shear with no increase for lateral forces.

4. Mortar may be of the same soil composition and stabilization as the brick in lieu of cement mortar.

5. Nominal tension stresses due to seismic forces normal to the wall may be neglected if the wall meets thickness requirements and shear values allowed by this subsection.

(c) Archaic Materials. Allowable stresses for archaic materials not specified in this Code shall be based on substantiating research data or engineering judgment subject to the Department's satisfaction.

(d) Alternative materials and SHBC Advisory Review. Alternative materials, design or methods of construction will be considered as set forth in Section 91.6809(d). In addition, when a request for an alternative proposed design, material or method of construction is being considered, the Department may file written request for opinion to the State Historical Building Code Advisory Board for its consideration, advice or findings in accordance with the SHBC.

SEC. 91.6808. ANALYSIS AND DESIGN

(a) General. Every structure within the scope of this Division shall be analyzed and constructed to resist minimum total lateral seismic forces assumed to act noncurrently in the direction of each of the main axes of the structure in accordance with the following equation:

$$V = IKCSW \qquad (68\text{-}1)$$

The value of IKCS need not exceed the values set forth in Table No. 68-D based on the applicable rating classification of the building.

TABLE NO. 68-D
HORIZONTAL FORCE FACTORS BASED ON RATING CLASSIFICATION

Rating Classification	IKCS
I	0.186
II	0.133
III and IV	0.100

(b) Lateral Forces on Elements of Structures. Parts or parts of structures shall be analyzed and designed for lateral loads in accordance with Section 91.2305(d) of this Code but not less than the value from the following equation:

$$F_p = IC_pSW_p \qquad (68\text{-}2)$$

For the provisions of this subsection, the produce of IS need not exceed the values as set forth in Table No. 68-E.

EXCEPTION: Unreinforced masonry walls in buildings not having a rating classification of I may be analyzed in accordance with Section 91.6809.

TABLE NO. 68-E
HORIZONTAL FORCE FACTORS "IS"
FOR PARTS OR PORTIONS OF STRUCTURES

Rating Classification	IS
I	1.50
II	1.00
III and IV	0.75

(c) Anchorage and Interconnection. Anchorage and interconnection of all parts, portions and elements of the structure shall be analyzed and designed for lateral forces in accordance with Table No. 23-B of this Code and the equation $F_p = IC_pSW_p$ as modified by Table No. 68-E. Minimum anchorage of masonry walls to each floor or roof shall resist a minimum force of 200 pounds per linear foot acting normal to the wall at the level of the floor or roof.

(d) Level of Required Repair. Alterations and repairs required to meet the provisions of this Division shall comply with all other applicable requirements of this Code unless specifically provided for in this Division.

(e) Required Analysis.

1. General. Except as modified herein, the analysis and design relating to the structural alteration of existing structures within the scope of this Division shall be in accordance with the analysis specified in Division 23 of this Code.

2. Continuous Stress Path. A complete, continuous stress path from every part or portion of the structure to the ground shall be provided for the required horizontal forces.

3. Positive Connections. All parts, portions or elements of the structure shall be interconnected by positive means.

(f) Analysis Procedure.

1. General. Stresses in materials and existing construction utilized to transfer seismic forces from the ground to parts or portions of the structure shall conform to those permitted by the Code and those materials and types of construction specified in Section 91.6809.

2. Connections. Materials and connectors used for interconnection of parts and portions of the structure shall conform to the Code.

3. Unreinforced Masonry Walls. Unreinforced masonry walls shall be analyzed as specified in Section 91.2417 to withstand all vertical loads as specified in Division 23 of this Code in addition to the seismic forces required by this Division. Such walls shall meet the minimum requirements set forth in Sections 91.2418 and 91.2419 of this Code. The 50 percent increase in the seismic force factor for shear walls as specified in Table No. 24-H of this Code may be omitted in the computation of seismic loads to existing shear walls.

No allowable tension stress will be permitted in unreinforced masonry walls. Walls not capable of resisting the required design forces specified in this Division shall be strengthened or shall be removed and replaced.

EXCEPTIONS:

1. Unreinforced masonry walls in buildings not classified as Rating I pursuant to Table No. 68-A may be analyzed in accordance with Section 91.6809.

2. Unreinforced masonry walls which carry no design loads other than its own weight may be considered as veneer if they are adequately anchored to new supporting elements.

(g) Combination of Vertical and Seismic Forces.

1. New Materials. All new materials introduced into the structure to meet the requirements of this Section which are subjected to combined vertical and horizontal forces shall comply with Section 91.2305(k) of this Code.

2. Existing Materials. When the stress in existing lateral force resisting elements are due to a combination of dead loads plus live loads plus seismic loads, the allowable working stress specified in the Code may be increased 100 percent. However, no increase will be permitted in the stresses allowed in Section 91.6809 and the stresses in members due only to seismic and dead loads shall not exceed the

values permitted by Section 91.2301(g) of this Code.

3. Allowable Reduction of Bending Stress by Vertical Load. In calculating tensile fiber stress due to seismic forces required by this Division, the maximum tensile fiber stress may be reduced by the full direct stress due to vertical dead loads.

SEC. 91.6809 MATERIALS OF CONSTRUCTION.

(a) General. All materials permitted by this Code including their appropriate allowable stresses and those existing configurations of materials specified herein may be utilized to meet the requirements of this Division.

(b) Existing Materials.

1. Unreinforced Masonry Walls. Unreinforced masonry walls analyzed in accordance with this Section may provide vertical support for roof and floor construction and resistance to lateral loads. The bonding of such walls shall be as specified in Section 91.2412(b) of this Code.

Tension stresses due to seismic forces normal to the wall may be neglected if the wall does not exceed the height or length to thickness ratio and the in-plane shear stresses due to seismic loads as set forth in Table No. 68-F.

TABLE NO. 68-F
ALLOWABLE VALUE OF UNREINFORCED
MASONRY WALLS WITH MINIMUM QUALITY MORTAR(1)

Rating Classification	Maximum Ratio Unsupported Height or Length to Thickness	Seismic In-Plane Shear Stress Based on Gross Area
I	Not applicable (2)	Not applicable (2)
II	9	3 psi (3)
III	10	3 psi (3)
IV	12	3 psi (3)

NOTES:

(1) Minimum quality mortar shall be determined by laboratory testing in accordance with Section 91.6809(e).

(2) Walls of buildings within rating classification I shall be analyzed in accordance with Section 91.6808(f).

(3) Allowable shear stress may be increased in accordance with Section 91.6809(g).

The wall height or length may be measured horizontally to supporting elements provided the stiffness of the supporting member is at least twice as stiff as the tributary wall. Stiffness shall be based on the gross section.

2. Existing Roof, Floors, Walls, Footings, and Wood Framing. Existing materials including wood shear walls utilized in the described configuration may be used as part of the lateral load resisting system, provided that the stresses in these materials do not exceed the values shown in Table No. 68-G.

TABLE NO. 68-G
VALUES FOR EXISTING MATERIALS

Materials or Configuration of materials (1)	Allowable Values
1. Horizontal Diaphragms	
a. Roofs with straight sheathing and roofing applied directly to the sheathing.	150 lbs. per foot for seismic shear.
b. Roofs with diagonal sheathing and roofing applied directly to the sheathing.	400 lbs. per foot for seismic shear.
c. Floors with straight tongue and groove sheathing.	150 lbs. per foot for seismic shear.
d. Floors with straight sheathing and finished wood flooring.	300 lbs. per foot for seismic shear.
e. Floors with diagonal sheathing and finished wood flooring.	400 lbs. per foot for seismic shear.
f. Floors or roofs with straight sheathing and plaster applied to the joist or rafters. (2)	Add 50 lbs. per foot to the allowable values for Items 1a and 1c.
2. Shear Walls	
a. Wood stud walls with wood lath and plaster.	50 lbs. per foot each side for seismic shear.
b. Wood stud walls with plaster and lath other than wood lath.	100 lbs. per foot each side for seismic shear.
3. Plain Concrete Footings	f_c = 1,500 psi unless otherwise shown by tests.
4. Douglas Fir Wood	Allowable stress same as No. 1 D.F.
5. Reinforcing Steel	f_t = 20,000 lbs. per square inch maximum
6. Structural Steel	f_t = 20,000 lbs per square inch maximum.

NOTES:

(1) Material must be sound and in good condition.

(2) The wood lath and plaster must be reattached to existing joists or rafters in a manner approved by the Department.

(c) Strengthening of Existing Materials. New materials including wood shear walls may be utilized to strengthen portions of the existing seismic resisting system in the described configurations provided that the stresses do not exceed the values shown in Table No. 68-H.

TABLE N[illegible]
ALLOWABLE VALUES OF NEW MATERIALS USED
IN CONJUNCTION WITH EXISTING CONSTRUCTION

New Materials or Configuration of Materials	Allowable Values
1. Horizontal Diaphragms	
Plywood sheathing applied directly over existing straight sheathing with ends of plywood sheets bearing on joists or rafters and edges of plywood located on center of individual sheathing boards.	Same as specified in Table No. 25-J of this Code for blocked diaphragms.
2 Shear Walls	
a. Plywood sheathing applied directly over existing wood studs. No value shall be given to plywood applied over existing plaster or wood sheathing.	Same as values specified in Table No. 25-J for shear walls.
b. Dry wall or plaster applied directly over existing wood studs.	75 percent of the values specified in Table No. 25-N.
c. Dry wall or plaster applied to plywood sheathing over existing wood studs	33-1/3 percent of the values specified in Table No. 25-N.
3. Shear Bolts	
Shear bolts and shear dowels embedded a minimum of 8 inches into unreinforced masonry walls. Bolt centered in a 2-1/2 inch diameter hole with dry-pack or non-shrink grout around circumference of bolt or dowel. (1)	100 percent of the values for plain masonry specified in Table No. 24-F. No values larger than those given for 3/4 inch bolts shall be used.
4. Tension Bolts	
Tension bolts and tension dowels extending entirely through unreinforced masonry walls secured with bearing plates on far side of wall with at least 30 sq. inches of area. (2)	1,200 lbs. per bolt or dowel.
5. Infilled Walls	
Reinforced masonry infilled openings in existing unreinforced masonry walls with dowels to match reinforcing.	Same as values specified for unreinforced masonry walls.
6. Reinforced Masonry	
Masonry piers and walls reinforced per Section 91.2418 and designed for tributary loads.	Same as values specified in Table No. 24-G
7. Reinforced Concrete	
Concrete footings, walls and piers reinforced as specified in Division 26 and designed for tributary loads.	Same as values specified in Division 26 of this Code
8. Existing Foundation Pressure	
Foundation pressures for structures exhibiting no evidence of settlement.	Calculated existing foundation pressures due to maximum dead load plus live load may be increased 25 percent for dead load, and may be increased 50 percent for dead load plus seismic load required by this Division.

NOTES:

(1) Bolts and dowels shall be tested as specified in Section 91.6809(f).

(2) Bolts and dowels shall be 1/2 inch minimum in diameter.

NOTES:

(1) Bolts and dowels shall be tested as specified in Section 91.6809(f).

(2) Bol's and dowels shall be 1/2 inch minimum in diameter.

(d) Alternate Materials. Alternate materials, designs and methods of construction may be approved by the Department in accordance with the provisions of Division 5 of Article 8 of Chapter IX of the Los Angeles Municipal Code

(e) Minimum Acceptable Quality of Existing Unreinforced Masonry Walls.

1. General Provisions. All unreinforced masonry walls utilized to carry vertical loads and seismic forces parallel and perpendicular to the wall plane shall be tested as specified in this Subsection. All masonry quality shall equal or exceed the minimum standards established herein or shall be removed and replaced by new materials. The quality of mortar in all masonry walls shall be determined by performing in-place shear tests or by testing eight inch diameter cores. Alternative methods of testing may be approved by the Department. Nothing shall prevent pointing with cement mortar of all masonry wall joints before the tests are first made. If the exterior joints are pointed then the inside face must also be pointed. Prior to any pointing, the wall surface must be sand or water blasted to remove loose and deteriorated mortar. All preparation and cement mortar pointing shall be done under the continuous inspection of a Registered Deputy Building Inspector. At the conclusion of the inspection, the inspector shall submit a written report to the licensed engineer or architect responsible for the seismic analysis of the building setting forth the result of the work inspected. Such report shall be submitted to the Department for approval as part of the structural analysis. All testing shall be performed in accordance with the requirements specified in this Subsection by a testing agency approved by the Department. An accurate record of all such tests and their location in the building shall be recorded and these results shall be submitted to the Department for approval as part of the structural analysis.

2. Number and Location of Tests. The minimum number of tests shall be two per wall or line of wall elements resisting a common force, or 1 per 1,500 square feet of wall surface, with a minimum of eight tests in any case. The exact test or core location shall be determined at the building site by the licensed engineer or architect responsible for the seismic analysis of the subject building.

3. In-Place Shear Tests. The bed joints of the outer wythe of the masonry shall be tested in shear by laterally displacing a single brick relative to the adjacent bricks in the wythe. The opposite head joint of the brick to be tested shall be removed and cleaned prior to testing. The minimum quality mortar in 80 percent of the shear tests shall not be less than the total of 30 psi plus the axial stress in the wall at the point of the test. The shear stress shall be based on the gross area of both bed joints and shall be that at which movement of the brick is first observed.

4. Core Tests. A minimum number of mortar test specimens equal to the number of required cores shall be prepared from the cores and tested as specified herein. The mortar joint of the outer wythe of the masonry core shall be tested in shear by placing the circular core section in a compression testing machine with the mortar bed joint rotated 15 degrees from the axis of the applied load. The mortar joint tested in shear shall have an average ultimate stress of 20 psi based on the gross area. The average shall be obtained from the total number of cores made. If test specimens cannot be made from cores taken then the shear value shall be reported as zero.

(f) Testing of Shear Bolts. One-fourth of all new shear bolts and dowels embedded in unreinforced masonry walls shall be tested by a Registered Deputy Building Inspector using a torque calibrated wrench to the following minimum torques:

1/2" diameter bolts or dowels	40 foot lbs
5/8" diameter bolts or dowels	50 foot lbs
3/4" diameter bolts or dowels	60 foot lbs

No bolts exceeding 3/4" shall be used. All nuts shall be installed over malleable iron or plate washers when bearing on wood and heavy cut washers when bearing on steel.

g. Determination of Allowable Stresses or Design Methods Based on Test Results.

1. Design Shear Values. Design seismic in-plane shear stresses greater than permitted in Table No. 68-F shall be substantiated by tests performed as specified in Section 91.6809(e) 3 and 4.

Design stresses shall be related to test results obtained in accordance with Table No. 68-I. Intermediate values between 3 and 5 psi may be interpolated.

TABLE NO. 68-I
ALLOWABLE SHEAR STRESS FOR
TESTED UNREINFORCED MASONRY WALLS

Eighty Percent of Test Results in psi Not Less Than	Average Test Results of Cores in psi	Seismic In-Plane Shear Based on Gross Area
30 plus axial stress	20	3 psi*
40 plus axial stress	27	4 psi*
50 plus axial stress or more	33 or more	5 psi*

*Allowable shear stress may be increased by addition of 10 percent of the axial stress due to the weight of the wall directly above.

2. Design Compression and Tension Values. Compression stresses for unreinforced masonry having a minimum design shear value of 3 psi shall not exceed 100 psi. Design tension values for unreinforced masonry shall not be permitted.

SEC. 91.6810. INFORMATION RQUIRED ON PLANS:

(a) General. In addition to the seismic analysis required elsewhere in this Division, the licensed engineer or architect responsible for the seismic analysis of the building shall determine and record the information required by this Section on the approved plans.

(b) Construction Details. The following construction details shall be made part of the approved plans:

1. All unreinforced masonry walls shall be anchored to all floors and roofs with tension bolts through the wall or by exsting rod anchors at the maximum anchor spacing of six feet. All existing rod anchors shall be secured to joists or rafters by bolting to develop the required forces. The Department may require testing by an approved testing agency to verify adequacy embedded ends of existing rod anchors.

2. Diaphragm chord stresses of horizontal diaphragms shall be developed in existing materials or by addition of new materials.

3 Where wood roof or floor members other than rafters or joists are supported in masonry pockets, ledgers or columns shall be installed to support vertical loads of the roof or floor members.

4. Parapets and exterior wall appendages not capable of resisting the forces specified in this Division shall be removed, stabilized or braced to insure that the parapets and appendages remain in their original position.

5. All deteriorated mortar joints in unreinforced masonry walls shall be pointed with cement motar. Prior to any pointing, the wall surface must be sand or water blasted to remove loose and deteriorated mortar. All preparation and pointing shall be done under the continuous inspection of a Registered Deputy Building Inspector certified to inspect masonry or concrete. At the conclusion of the project, the inspector shall submit a written report to the Department setting forth the portion of work inspected.

6. Repair details of any cracked or damaged unreinforced masonry wall required to resist forces specified in this Division.

(c) Existing Construction. The following existing construction information shall be made part of the approved plans:

1. The appropriate age of building.

2. The typical footing width, depth and maximum soil bearing for dead plus live loads.

3. The type and dimensions of existing walls and the size and spacing of floor and roof members.

4. The extent and type of existing wall anchorage to floors and roof.

5. The extent and type of parapet corrections which were performed in accordance with Section 91.0103(b) of this Code.

6. Accurately dimensioned floor plans and masonry wall elevations showing dimensioned openings, piers, wall thickness and heights.

7. The location of cracks or damaged portions of unreinforced masonry walls requiring repairs.

8. The type of interior wall surfaces and if reinstalling or anchoring of ceiling plaster is necessary.

9. The general condition of the mortar joints and if the joints need pointing.

Sec. 2. The City Clerk shall certify to the passage of this ordinance and cause the same to be published in some daily newspaper printed and published in the City of Los Angeles.

I hereby certify that the foregoing ordinance introduced at the meeting of the Council of the City of Los Angeles of December 16, 1980 and was passed at its meeting of January 7, 1981.

REX E. LAYTON,
City Clerk,
By Charles J. Port, Deputy.

Approved January 7, 1981.

JOEL WACHS,
Acting Mayor

File No. 73-721, 74-4395, and 79-4244
(JD26841) Jan 13

EXISTING HAZARDOUS BUILDINGS:
ASSESSING DIRECT POST-EARTHQUAKE EFFECTS

Karl V. Steinbrugge*

ABSTRACT

Planning problems arising from existing hazardous buildings may be measured through an assessment of the potential for:

1. Life loss and injuries,
2. Direct property damage,
3. Functional impairments (such as damage to an easily repaired aqueduct, but having consequential effects on fire-fighting, drinking water, etc.).

A summary of portions of the major United States studies on the first two subjects may be found on the following pages.

Life loss and injuries are functions of:

A. Construction type and number of occupants. Experience provides a relationship between construction type and life loss; these numerical parameters for United States practice are given in this paper. Methodology for determining life losses is, to a large degree, transferable from nation to nation.

B. Off-site effects having consequential effects, such as dam failure or tsunami generation. Methodology transfer to other regions is normally in general terms, and area-specific parameters are usually established for each study area. Therefore, this subject will receive no further attention in this paper.

The evaluation of property damage may be influenced by reconstruction standards which, in the possible urgent needs for post-earthquake housing and also for demands on available resources, may be lowered from pre-earthquake standards. Conversely, large life loss may result in improved standards for new if not for existing buildings. Therefore, pre-earthquake studies are usually predicated on the cost of repairs necessary to rebuild buildings to their original condition. Transfer functions may be developed to estimate the impact of improved (or relaxed standards). Included in this paper is information which relates construction types to earthquake intensity (i.e., to damage).

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Introduction

Three different magnitude earthquakes may be considered for pre-earthquake hazard assessments and for mitigation purposes. First, the maximum probable earthquake ($M = 8.3$ for California) which overwhelms local and state resources and which requires massive nationwide assistance. Second, smaller but more frequent earthquakes (such as $M = 6$ and $M = 7$) which may be satisfactorily responded to by state and local governments. Only the computational methodology is considered in this paper.

An assessment of direct post-earthquake effects and needed mitigation measures usually begins with estimates of the potential life loss and injuries. A second kind of assessment is often also requested, namely, what is the extent of the damage and the resources necessary to repair or replace the damaged buildings. Alternately for mitigation programs, cost-effective ways for improving safety are desired.

Certain methodologies for assessing potential life loss and monetary loss have been developed in the United States, particularily for California, and it seems reasonable that portions of these methodologies are tranferable to other nations. Conversely, the experience of other nations certainly can improve the procedures used in United States.

Procedures for making loss estimates in the United States began about 15 years ago in connection with the Federal Government's efforts to plan earthquake disaster response activities. One phase of these studies was later continued at the University of California in Berkeley as a part of their analysis of life hazards should the earthquake active Hayward fault, which bisects the Berkeley Campus, generate a major earthquake. The latest developments in life hazard estimation techniques was completed in a study by the Seismic Safety Commission of the State of California; this will be discussed in more detail in a following section.

Building Classification Systems

Potential loss assessments are directly related to building types and their expected earthquake performance characteristics. Past experience has clearly shown that certain construction types such as light mass wood frame (if they have no heavy roof or floor loads) will survive excellently. Conversely, unreinforced masonry with sand-lime mortar is usually associated with heavy life loss and property damage.

Due to differences in construction types within individual nations as well as among countries, no single building classification system can be universally used without some modification. However, there are many parallels in modern building types and data on these types are transferable. Additionally, earthquake design criteria are very similar throughout the world; earthquake engineering design technologies have been freely exchanged at the periodic international conferences on earthquake engineering. Further, the design techniques using structural steel frame and reinforced concrete frame, including precast concrete, have features which allow considerable transferability. Perhaps the greatest difficulties arise in the evaluation of mixed construction; for example, how much reinforcing is required in unit masonry such as brick to qualify it for "good" construction? Exactly what are "good" roof-to-wall ties?

Despite the foregoing caveats, it has been United States experience that data on the earthquake performance of various structural types throughout the world have utility in determining American loss parameters. Obviously, competent structural engineering judgment is needed when evaluating non-local data.

It is not always understood that most earthquake regulations and earthquake building codes are intended to protect life and only secondarily to protect property. For example in United States, the underlying philosophy for the earthquake provisions in building codes is:

> "Resist major earthquakes, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage"..... "In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. This, however, depends upon a number of factors, including the type of construction selected for the structure".... --from "Commentary on the Fourth Edition of the Recommended Lateral Force Requirements", page 1, Seismology Committee, Structural Engineers Association of California.

Under these circumstances, it follows that the building classification system used for life loss and injury assessments can (and does) vary somewhat from that used for property loss assessments.

Appendix A is one commonly used building classification system. To avoid unduly complicating this paper, differences between a building's life hazard characteristics and its monetary loss characteristics have been eliminated in Appendix A, except to point out the need to consider long period ground motions on tall buildings.

Death and Injury Assessments

A disaster response planner is interested in the numerical values for the estimated deaths and injuries, and their geographic distribution. The city planner, architect, and engineer wishes additional information, namely, what can be done before the earthquake to reduce the life hazard. Also, since unlimited resources to strengthen existing buildings and to build stronger new buildings are rarely (if ever) sufficient, then the city planner, architect, and engineer must know how best to allocate these resources to achieve maximum benefit-cost. In other words, an equitable priority must be established among buildings, and the remainder of this secion discusses one such rational method.

The following is based on and partially extracted from the California Seismic Safety Commission report entitled "Evaluating the Seismic Hazard of State Owned Buildings", SSC 79-01, 1979. The writer wishes to acknowledge the very substantial contributions made by F.E. McClure, H.J. Degenkolb, and R.A. Olson who, along with the author, constituted the committee which prepared the State of California report.

From a public policy standpoint, the long range goal is to eventually reduce the life loss to two per ten thousand (2:10,000) persons for all types of buildings in the event of the maximum credible earthquake. This recognizes that the current state of the art of earthquake engineering does not allow for earthquake "proof" buildings, and that the eventual limiting value is currently estimated at 2:10,000. The 2:10,000 figure is based on studies of past damaging earthquakes and represents the expected loss of life in small wood-frame structures which, because of their flexibility and light weight, are considered to be the safest type of structure in general use in the United States. Meanwhile, however, realistic current attainable life safety goals (LSRG) for various building classes are given in the last column of Table 1.

The heart of the matter in establishing the benefit-cost ratio is to determine the number of postulated lives saved per reconstruction dollar (or other monetary unit), hereafter termed the BCR. Thereafter, reconstruction priority is readily determined from any list of buildings by their BCR. "Reconstruction" is used in the context of post-disaster repairs or pre-disaster strengthening. Mathematically, the BCR may be expressed:

$$\mathrm{BCR} = \frac{(\mathrm{LSR})\times(\mathrm{ECOa})\times(\mathrm{SCF}) - (\mathrm{LSRG})\times(\mathrm{ECOb})\times(\mathrm{SCF})}{(10{,}000)\times(\mathrm{RC})}$$

Where:

BCR — <u>Benefit-Cost Ratio</u>, being the number of postulated lives saved per reconstruction dollar (or any other monetary unit).

LSR — <u>Life-Safety Ratio</u>, being the postulated number of fatalities per 10,000 building occupants prior to reconstruction for a particular type of structure for the level of shaking appropriate to the seismic zone in which the structure is located. Table 1 contains LSR data for various classes of buildings.

ECOa — <u>Equivalent Continuous Occupancy prior to reconstruction</u>, being the theoretical estimated number of persons continuously occupying the structure on a 24 hour basis, 365 days per year.

ECOb Same as ECOa, except after reconstruction. (Occupancy changes could result in a different ECO before and after reconstruction.)

SCF <u>Seismicity Correction Factor</u>, being a coefficient applied to subzones of the study area to account for differences in seismicity. This factor is 1 where the seismicity is uniform throughout the study area.

LSRG <u>Life Safety Ratio Goal</u>, being the attainable life safety goal that could be achieved by changing the use of or strengthening the building. Based on experience, Table 1 contains estimates for attainable life safety goals for various classes of buildings listed in more detail in Appendix A.

RC <u>Reconstruction Cost</u>, being the cost to strengthen a given type of building so as to reduce the life hazard to the Life Safety Goal (LSRG) specified for the particular class of building in question.

The detailed procedural methodology developed in the cited reference has not been summarized here since much of it is not transferable but a computational example is included.

In overview, it should be understood that the methodology is based on the premise that all persons are provided with an equal degree of safety, whether academic or non-academic, and whether administrator or non-administrator.

Further, the procedure allows for considerable planning latitude. For example, a high-hazard building due to poor quality construction and large occupancy load can be changed to a seldom entered warehouse containing non-damageable goods, thereby changing the reconstruction priorities.

Computational Example:

Assume: 100 bed hospital with 90% average occupancy.
Building class to be 4A non-ductile.
Reconstruction cost (RC) is $500,000.
Occupancy not to change (i.e., ECOa = ECOb).
Seismicity of the study area is a constant (i.e., SCF = 1).

To determine the ECO (Equivalent Continuous Occupancy):

From disaster response studies in California, it is known that:
At 2:00pm, there are 1.6 non-patients/bed in the hospital,
At 4:30pm, there are 1.4 non-patients/bed in the hospital,
At 2:30am, there are 0.34 non-patients/bed in the hospital.

Bed patients:	100 x 0.90	= 90
8 hours of 2:00pm equivalent:	100 x (1/3) x 1.6	= 53.3
8 hours of 4:30pm equivalent:	100 x (1/3) x 1.4	= 46.7
8 hours of 2:30am equivalent:	100 x (1/3) x 0.34	= 11.3
	ECO	= 201.3

(Alternatively, if gross area were known, then the ECO could be calculated on a unit area basis. However, all computations for all occupancies must be on a consistent basis, regardless which system is used.)

Since the building class is "4A non-ductile", then from Table 1:
LSR = 50
LSRG = 15.

Finally:

$$BCR = \frac{(50)x(201.3)x(1) - (15)x(201.3)}{(10,000)x(500,000)}$$

$$= 1.409 \times 10^{-6}$$

This process is to be repeated for all buildings under consideration in the study area, and reconstruction priorities determined from the ranking of the numerical results. Alternately, occupancies can be changed or occupant loads changed, thereby also changing priorities.

Assessment of Potential Property Damage

The evaluation of property damage may be influenced by reconstruction standards which, in the possible urgent needs for post-earthquake housing and also for demands on available resources, may be lowered from pre-earthquake standards. Conversely, large life loss may result in improved standards for new if not for existing buildings. Therefore, pre-earthquake studies are usually predicated on the cost of repairs necessary to rebuild buildings to their original condition. Transfer functions may be developed to estimate the impact of improved (or relaxed standards).

The first step in the property damage assessment process is to establish an isoseismal map, or other equivalent map of the study area, using the best inputs from seismologists and geologists working collaboratively with engineers and architects. The next step is to prepare tables or graphs which display intensity (or effective accelleration, or other seismic parameter) as a function of damage in terms of percentage of replacement cost. Then, aggregate losses are readily determined by multiplying the inventory of buildings of each class of construction by the percent loss. The methodology is simple. However, the cost of completing a thorough inventory may exceed 80% of the analysis costs; a poorly prepared or incomplete inventory will weaken or destroy the credibility of the study.

Figure 1 is a generalized graph showing the damage relationships for certain classes of structures. Consider Curves #1 and #2 in Figure 1 which show the characteristic damage patterns for certain kinds of flexible-frame multistory buildings. Both buildings are considered to be equally earthquake resistive from a design standpoint, with certain nonstructural elements being the only construction variable. The lower loss vs. intensity values of both curves are represented by a flattened portion to represent the minor real or minor imagined losses often noted at large distances from the earthquake. At the high intensity end of these curves and if the largest losses are less than 100 percent (such as no collapse), then the curves flatten. This occurs because increasing nonstructural damage no longer requires proportionate repair costs; for example, patching and painting may cost little more for the repair of a badly cracked wall than for a less severely cracked wall.

A possible effect of occupancy can be seen by comparing Curves #1 and #2 in Figure 1. A warehouse or manufacturing structure might have a minimal number of partitions (Curve #2) whereas a hotel would have numerous partitions (Curve #1). The vertical spread between the curves represents the difference in loss due to occupancy-related construction. Significant exceptions may exist to this vertical spread between curves. For example, if the partitions are of a high value type which are subject to little damage prior to building collapse (shear wall building), then the two curves may essentially coincide.

Curves #3 and #4 in Figure 1 have the characteristic shapes for losses to rigid unit masonry buildings. The beginning of the curves at low loss levels represents hairline cracks at partition-masonry wall intersections and similar kinds of minor damage. The steepness of the straight line represents brittle failure of the walls and/or roof-to-wall connections. Actually, for a specific building, the straight line should be replaced by a jagged line, since loss would really be a series of step functions, with each step representing another brittle failure.

Curve #5 is representative of an older steel frame multistory building which is not expected to collapse despite heavy damage. For the lower intensities, it is quite possible that the steel frame building may have significantly more damage than a "collapse hazard" unreinforced brick bearing wall structure with lime mortar, but the pattern would reverse at higher intensities. This is particularily true when long period effects are considered.

Figure 2 is a rather simplified, but practical, intensity vs. percent loss relationship adapted from "Estimation of Earthquake Losses to Buildings (Except Single Family Dwellings)" by Algermissen, Steinbrugge, and Lagorio, U.S.G.S. Open-File Report 78-441, 1978. Building classes in connection with Figure 2 are listed in Appendix A. Long period effects to tall buildings as a result of major distant earthquakes can be developed from Figure 2; special algorithems are often preferable since, in the United States, the Modified Marcalli Intensity scale is not truly applicable in these instances.

The computational process is:

1. Inventory buildings in the study area by class and value.

2. Assign the expected intensity (or equivalent damage estimation parameter).

3. Multiply each building's monetary value by its expected loss (such as Figure 2), and add all of these together into suitable groupings.

As previously stated, the computations are elementary. The significant problems arise when developing adequate inventories, a suitable building classification system, and loss-intensity relationships which are practical. None of these computational systems are a substitute for a detailed analysis which can be made for an individual building.

Recommendation

It is desirable to re-examine and extend damage, death, and injury information known from past earthquakes in order to better understand and quantify the relationships among them. Data extension can also be accomplished by transferring relevant information between nations. It is therefore recommended that joint studies be undertaken by China and the United States which are intended to define, evaluate, and make available damage and life loss/injury data on a common basis insofar as practical.

TABLE 1

LIFE SAFETY RATIOS (LSR)

	Simplified Description of Building Class	LIFE SAFETY RATIOS (LSR) Earthquake Resistive Buildings	Non-EQ Resistive Buildings	LSRG
1A	Small wood frame	2	4	2
1B	Large wood frame	5	10	5
2A	Small all-metal	2	4	2
2B	Large all-metal	8	15	8
3A	Steel frame, superior	5	10	5
3B	Steel frame, ordinary	15	40	10
3C	Steel frame, intermediate	10	25	5
3D	Steel frame, wood floors	25	50	15
3E	Steel frame, 3A large a.	25	50	15
3F	Steel frame, 3B, 3C large a.	--	1500	15

		Non-ductile Concrete	Ductile Concrete		
4A	Reinf. conc., superior	50	25	100	15
4B	Reinf. conc., ordinary	300	75	1000	25
4C	Reinf. conc., intermediate	200	50	500	25
4D	Reinf. conc., precast	500	75	1500	25
4E	Reinf. conc., wood floors	800	100	2000	25
4F	Reinf. conc., 4A large a.	75	50	200	35
4G	Reinf. conc., 4B, 4C large a.	1000	200	2500	35

5A	Small mixed constr., dwellings	10	200	10
5B	Mixed constr., superior EQ resistive	15	800	15
5C	Mixed constr., ordinary EQ resistive	20	1000	15
5D	Mixed constr., intermediate EQ resist.	40	2000	15
5E	Mixed constr., unreinforced masonry	--	4000	15
5F	Mixed constr., adobe, hollow tile	--	5000	15

ABBREVIATIONS: EQ - earthquake
resist. - resistive
constr. - construction
a. - large areas such as auditoriums

LSR and LSRG - see text

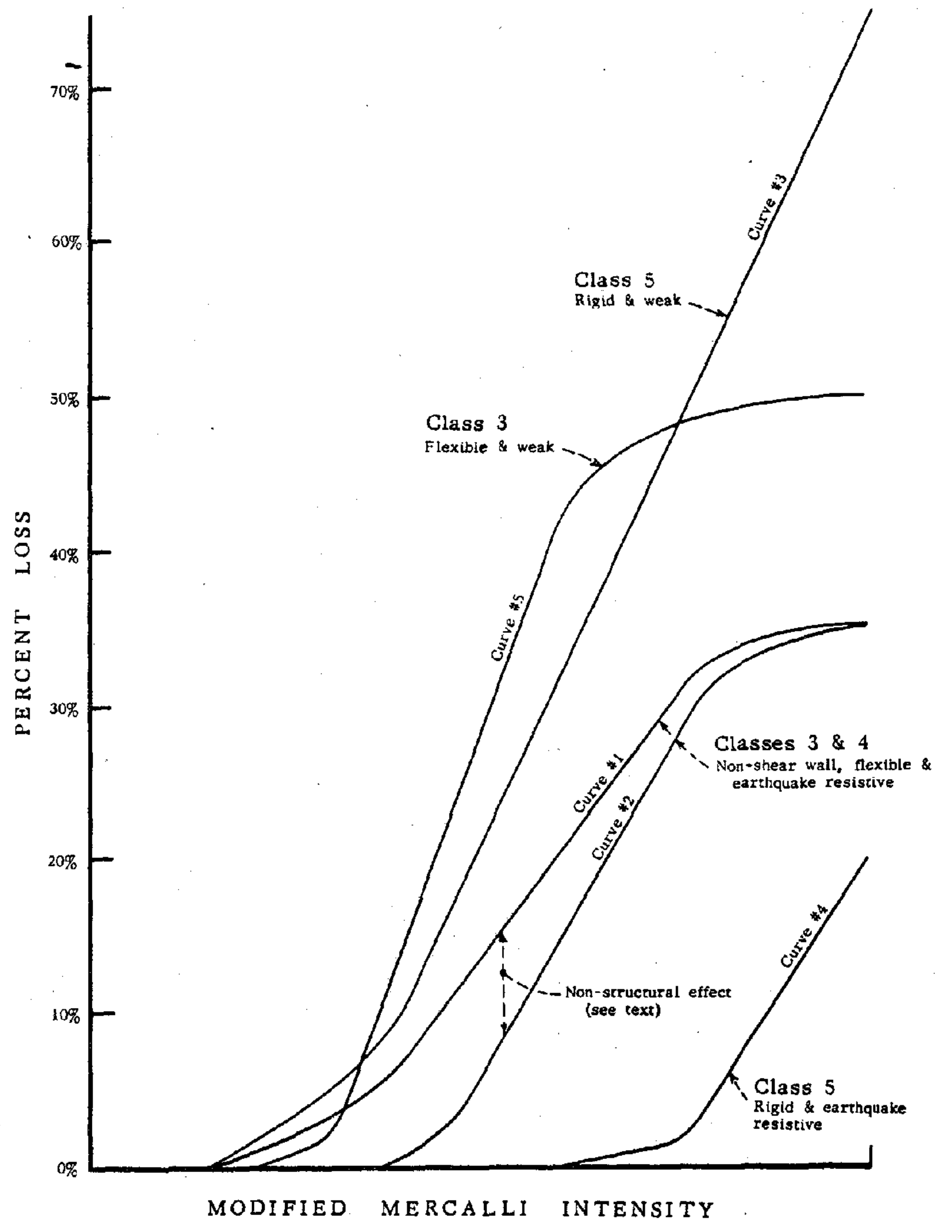

FIGURE 1. Generalized loss vs. Modififed Mercalli Intensity relationships.

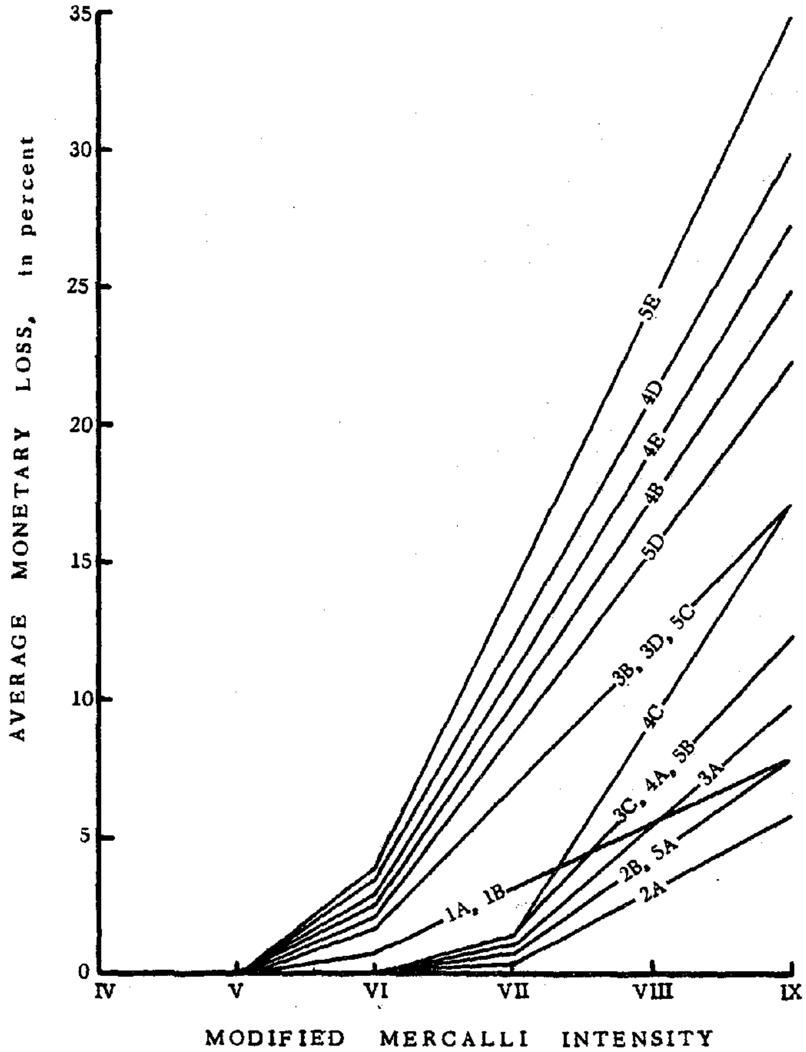

FIGURE 2. Simplified loss vs. Modified Mercalli Intensity relationships applicable in California, USA.

APPENDIX A

Building Class Definitions

Slightly modified from "Evaluating the Seismic Hazard of State Owned Buildings", California Seismic Safety Commission, 1979. A few differences exist in their application to life safety assessments as opposed to monetary loss assessments, principally in the number of classes and subclasses.

Wood Frame

Class 1A: Wood frame buildings up to 3 stories in height and not over 4000 square feet per floor. Includes wood frame with stucco, wood, plywood, and light metal sheathing.

Class 1B: Wood frame buildings which do not qualify as Class 1A.

All-metal Buildings

Class 2A: One story all-metal buildings which have a floor area not exceeding 20,000 square feet.

Class 2B: All-metal buildings which do not qualify as Class 2A.

Steel Frame Buildings

Subclasses as a function of story height should be added for monetary loss assessments in order to include damage from long period effects to buildings located at large distances from major earthquakes.

Class 3A: Buildings which have a complete steel frame with all loads carried by the steel frame. Floors and roofs shall be of poured-in-place reinforced concrete, or of concrete fill on metal decking welded to the steel frame (open web steel joists excluded). Exterior walls shall be of poured-in-place reinforced concrete or of reinforced unit masonry placed within the frame. Buildings shall have at least a width-to-height ratio not exceeding 1 to 4. Not qualifying are buildings having column-free areas greater than 2500 square feet such as auditoriums.

Class 3B: Same type of frame, floors, and roof as Class 3A, except that roofs on buildings over three stories may be of any material. Exterior and interior walls may be of any non-load carrying material. Excludes column-free areas greater than 2500 square feet.

Class 3C: Buildings having some of the favorable characteristics of Class 3A, but otherwise falling into 3B.

Class 3D: Buildings having a complete steel frame with floors and roofs of any material, and with walls of any non-load bearing materials.

Class 3E: Auditoriums, theaters, and large spaces otherwise meeting the requirements of Class 3A.

Class 3F: Auditoriums, theaters, and large spaces otherwise meeting the requirements of Class 3B or Class 3C.

Reinforced Concrete, Combined Reinforced Concrete and Steel Frame

Class 4A, 4B, and 4C buildings shall have all vertical loads carried by a structural system consisting of one or a combination of the following: (a) poured-in-place reinforced concrete frames, (b) poured-in-place reinforced concrete bearing walls, (c) partial structural steel with (a) and/or (b). Floors and roofs shall be of poured-in-place reinforced concrete, except that materials other than reinforced concrete may be used for the roof on buildings over three stories.

Subclasses as a function of story height should be added for monetary loss assessments in order to include damage from long period effects on buildings located at large distances from major earthquakes.

Class 4A: Buildings having a structural system as defined above with poured-in-place reinforced concrete exterior walls or reinforced unit masonry exterior walls placed within the frame. Buildings shall have a least width-to-height ratio of not exceeding 1 to 3. Not qualifying are buildings having column-free areas greater than 2500 square feet such as auditoriums.

Class 4B: Buildings having a structural system as defined above with exterior and interior non-bearing walls of any material.

Class 4C: Buildings having some of the favorable characteristics of Class 4A but otherwise falling into Class 4B.

Class 4D: Buildings having (a) a partial or complete load carrying system of precast concrete, and/or (b) reinforced concrete lift slab floors and/or roofs, and (c) otherwise qualifying for Classes 4A, 4B, or 4C.

Class 4E: Buildings having a complete reinforced concrete frame, or a complete frame of combined reinforced concrete and structural steel. Floors and roof may be of any material while walls may be of any non-load bearing material.

Class 4F: Auditoriums, theaters, gymnasiums, and large spaces which otherwise meet the requirement of Class 4A.

Class 4G: Auditoriums, theaters, gymnasiums, and large spaces which otherwise meet the requirements of Classes 4B, 4C, or 4E.

Mixed Construction

Class 5A: Buildings of mixed construction up to three stories in height and not over 4000 square feet per floor, constructed of poured-in-place reinforced concrete or reinforced masonry walls, and with wood or metal deck floors and roofs.

Class 5B: One story buildings having superior earthquake damage control features including exterior walls of (a) poured-in-place reinforced concrete, and/or (b) precast reinforced concrete, and/or (c) reinforced brick masonry or reinforced hollow concrete block masonry. Roofs and supported floors shall be of wood or metal diaphragm assemblies. Interior bearing walls shall be of wood frame or any one of a combination of the aforementioned wall materials.

Class 5C: One story buildings having construction materials listed for Class 5B, but with ordinary earthquake damage control features.

Class 5D: Buildings having reinforced concrete load bearing walls with floors and roofs of wood and not qualifying for Class 4E. Also included are buildings of any height having Class 5B materials of construction, including wall reinforcement; also buildings with roof and supported floors of reinforced concrete (precast or otherwise) not qualifying for Class 4.

Class 5E: Buildings having unreinforced solid unit masonry of unreinforced brick, unreinforced concrete brick, unreinforced stone, unreinforced concrete, when the loads are carried in whole or in part by the wall and partitions. Interior partitions may be wood frame or of the aforementioned materials. Roof and floors may be of any material. Not qualifying are buildings with non-reinforced load carrying walls of hollow clay tile or other hollow unit masonry, adobe, or cavity construction.

Class 5F: Buildings having load carrying walls of hollow clay tile or other hollow unit masonry construction, adobe, and cavity wall construction.

SUB-GROUP
DISCUSSIONS

SUB-GROUP DISCUSSION SESSIONS

INTRODUCTION:

In addition to the exchange of scientific information between colleagues which was achieved through the presentation of professional papers, another primary objective of the joint US/PRC cooperative Workshop was to identify and develop specific recommendations, or a comprehensive agenda, for a broad program in earthquake hazards mitigation research relating to the needs of the planning and design professions including architecture, planning, urban design, and engineering. To meet this goal participants focused their observations on the transfer of relevant technical information and exchange of academic personnel in addition to identifying potential research activities of mutual concern and common interests.

The Workshop participants were divided into three sub-group discussion sessions according to their interests on a voluntary basis. These three sessions dealt with specific topics relating to the fundamental objectives of the Workshop as follows: Architectural and Non-Structural Elements, Urban Planning and Land Use Considerations, and Existing Hazardous Buildings Problem. The task of each sub-group discussion session was to complete a comprehensive review of the topic area, define typical problems, and develop related research issues and potential joint projects for future study. The three discussion groups represented basic areas of mutual concern in which research could be directed toward the mitigation of life loss and property damage in earthquakes. The three topic areas were not necessarily isolated entities, and the overlapping of issues and concerns were expected and welcomed.

The participants of the workshop were encouraged to identify as many researchable issues as possible during the discussion sessions. All of the topics discussed were intended to add to the body of knowledge or "state-of-the-art" techniques being assembled to deal with the earthquake hazards mitigation through architecture, planning and engineering in the United States of America and the People's Republic of China. Specific instructions were assigned to each of the three sub-group discussion sessions to include in their review recommendations on the:

1) Exchange of technical information, materials, and personnel.
2) Identification of future cooperative research projects of joint concern and mutual interest.

SUB-GROUP DISCUSSION SESSION 1: ARCHITECTURAL & NON-STRUCTURAL ELEMENTS:

Co-Chairmen:

Gerald McCue	He, Guang-Lim
Graduate School of Design	Department of Architecture
Harvard University, Cambridge	Tianjin University, Tianjin

Participants:

Christopher Arnold
Building System Development, Inc.
San Mateo, California

Chen, Mou-Xin
Institute of Architectural Design
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Jiang, Menghon
North-Western College of
Architecture and Civil
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Henry J. Lagorio
Center for Planning and
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Liu, Zhaofeng
Department of Architecture
Qinghua University, Beijing

Marcy Wang
Department of Architecture
University of California, Berkeley

Yang, Wenzhong
Tangshan Municipal Construction
Bureau, Tangshan

Yang, Yucheng
Institute of Architectural Design
Academia Sinica, Harbin

Interpreter:

Nie, Fenglan
Foreign Affairs Division
State Capitol Construction Commission
Beijing

General:

Discussion ranged over a number of issues which may be categorized under three headings: (1) Architectural design for seismic effects, (2) Urban design for seismic effects, and (3) Design education. The particular problems China faces as a developing country are aggravated by the exceptional demands posed by seismic hazards. China, currently, does not have the capacity to rapidly adopt high technology solutions, but must seek methods that draw upon innovative adaptations of traditional materials and systems of construction. Under such circumstances, the conceptual development of a building through architectural design is even of greater significance in China than in the United States.

Participants clearly indicated that the problems posed by seismic hazards must be addressed at the scale of urban design and studied at the infrastructure level as well as in individual buildings. Questions of land coverage, density, arrangement of building groups, and overall building hazards must be considered as an urban design problem at neighborhood and regional scales.

Central to China's progress in addressing seismic hazards is the need to improve educational programs in architecture and urban design so that the design professionals may take a more active role of leadership. This would also enable the architectural and urban design professional reach a level of sufficient proficiency in theory and analysis to permit innovative collaborations with the engineering professions.

The session stressed the need to understand the basic differences in design and construction practices between China and the United States. In the People's Republic of China indications are that the architectural layout of the buildings is more important than in the United States. American designers have the opportunity to develop a basic structural system, or frame, within a regular, or irregular, volume or overall form, whereas for the Chinese professional the structure and form are the same. In the United States, the designer has the flexibility to utilize an independent structural frame in contrast to China where the basic material of construction is masonry in which the typical building is constructed as a composite structural frame with compressive walls, or all load bearing walls. As a result, in the United States a clearer distinction exists between the basic building structure, engineered for seismic response, and everything else in the building which is specifically defined as being non-structural and analyzed separately but coordinated with the performance of the structural system.

It is evident that a great deal of interest existed among the participants in the differences between architectural style, expression, and character. Relationships between the basic architectural style of a building and its configuration were discussed extensively as to their influence on the performance of a building during a damaging earthquake. As a starting point, it was clearly indicated that the differences in building construction practice and design must be thoroughly understood as being part of societal and cultural needs of each country. Immediate areas of significant differences were noted between both countries and all participants became aware of the concerns and dangers found in a simple transfer of Western approaches to Chinese problems or the utilization of simplistic solutions foreign to both cultures.

Recommendations made during the discussions also acknowledged the importance of the educational process. The improvement of educational programs for all design professionals was emphasized in order to develop an understanding and equal exchange of technical information on an interdisciplinary level. Further development of exchanges of faculty members with American universities, preparation of teaching materials and tools for use in China, teaching workshops, and graduate interchange at research levels were all identified as priority areas in the educational component of earthquake hazards mitigation goals.

Recommendations of Sub-Group Discussion Session 1:

A. Exchange of Technical Information, Materials, & Personnel:

1) An immediate reciprocal exchange of slides on representative examples of earthquake damage, building performance, design recommendations, repair and strengthening of hazardous buildings, test procedures in the laboratory and field, land use mapping, seismic safety urban planning, architectural and non-structural components, building lay-out and configuration, and other considerations was urged as an initial product of the Workshop. On a long-term basis it was suggested that such a mutual exchange of slides could be the start of developing a "use volume" on earthquake hazards mitigation programs to be shared by both countries.

2) United States participants requested post-earthquake recovery and technical material be collected from the Chinese on pre-planning experiences related to complete reconstruction of major urban centers severely damaged, or destroyed, by earthquakes or other major natural disasters. A specific request was made for a copy of the master plan, or urban design plan, for the reconstruction of the City of Tangshan which was destroyed by the 1976 earthquake in the Hopeh Province.

3) Interchange of techniques on the post-earthquake mapping of urban damage patterns, building damage, and damage estimates was urged. This included methods used in the mapping of geological hazards for use in urban design approaches.

4) Chinese delegates requested specific information from the U.S. participants in the discussion session on:

- (a) Seismic design approaches for small and medium size factories, including light industrial buildings.
- (b) Information on the design and construction of prefabricated buildings, including precise data on case study problems which have recently emerged relative to the seismic limitations in these buildings.
- (c) Fabrication and construction details of flexible connections for rigid wall panels to frame structures.
- (d) Data on multi-story industrial buildings in masonry and reinforced concrete systems.
- (e) Publications and books on teaching structural design principles and design approaches to architectural students. Examples mentioned were Shodeck's book as a representative type.
- (f) Particular desires were expressed for copies of the "MBT Report" by McCue, the "Building Configuration Study" by Arnold, and the "Architects and Earthquakes" booklet by Lagorio, et al, under the auspices of the AIA/Research Corporation, among others.
- (g) Detailed information on "clean-rooms" and other "high-tech" facilities, not necessarily limited to seismic considerations.
- (h) Copies of California Hospital Act of 1972 and Title 17, Safety Construction of Hospitals.

5) Additional information on Chinese products were sought by the U.S. delegates on the following prototypes:

- (a) Neighborhoods
- (b) Residential design
- (c) Other relevant building types

6) As part of an educational component urged by all discussion participants as a means of improving academic programs in architecture and urban design, it was recommended that the following activities and interchange of materials should be undertaken:

- (a) Exchanges of faculty and advanced graduate students with U.S. and P.R.C. universities.
- (b) Teachers' Workshops. Suggested as the necessary first step in "teaching the teachers".

(c) Preparation and development of teaching materials for use in China.

(d) Workshops and materials prepared for exchange are to address, among other considerations, economics, dynamic theory (particularly need for students in architecture and urban design), and concerns for visual expressions and elements of architectural style.

B. Identification of Future Cooperative Research Projects:

1) A high priority item is the necessity to explore specific requirements in architectural design imposed by construction systems peculiar to China. Study and assessment of these systems to include composite structures, braced frame and shear walls, as well as the limitations and potentialities of prefabricated structures.

2) Studies which address issues of building continuity, relative flexibility, displacements and deformations imposed by mass and configuration on seismic resisting elements, the function of rigidity, and other general architectural design constraints.

3) Development and assessment of approaches to the design of prototypes and of specific cases in building design as a means of exploring adaptation of architectural design techniques in housing, light industry, heavy industrial facilities, critical facility buildings, and buildings of mixed uses that pose conflicting geometric demands.

4) Investigations of appropriate performance standards and proper building code criteria, suitable design approaches, and prototypes for architectural detailing of non-structural components and their resistance to both vibrational and displacement characteristics.

5) Urban design problems addressing questions of land coverage, density, arrangement of building groups and complexes, infrastructure building hazards, and other physical facilities at the neighborhood scale.

6) Urban scale studies on appropriate residential form, density and building heights. Effective comparisons between high-rise/high-density versus low-rise/high density solutions relative to neighborhood and regional considerations.

7) Design studies relative to urban scale integration of housing with high intensity utilization. Considerations of location and the role of critical facilities and their relationships to the composite infrastructures of a city and metropolitan area.

8) Appropriate field investigation studies, analytical work, theoretical surveys, and experiments on models which focus on building configuration research. Particularly important in view of the current re-evaluation and re-assessment in China of rectangular versus re-entrant corner building types which require additional study on design

variables such as size, height and aspect ratio of wings, type of structural systems, and other design considerations.

9) Statistical analysis research and model tests to determine appropriate design procedures for masonry buildings. As brick is the predominant material of construction in China and an important building material in the United States, additional studies to be conducted to increase the resistance capacity of brick and masonry construction.

10) Research on the earthquake performance of elevators, stairways, and systems of egress, in various building types. Study of their relationships in placement and location in a building system including their impact on the integrity of structural systems and building configuration issues.

11) Analysis of steel uses for large spans in industrial, low-rise building systems with particular attention given to the problems of connections and tying the building together.

12) Development of data which focuses on the importance of keeping critical facilities buildings functional after earthquakes. Verification of code standards with field experience on realistic operational issues.

13) Study of earthquake performance of precast and prestressed reinforced concrete building systems and verification of the initial design assumptions with actual field experiences.

14) Engage in comparative studies between the U.S. and P.R.C. programs in general building design, urban reconstruction, repair and strengthening of existing buildings, post-earthquake mapping techniques, and earthquake performance of prototype buildings.

SUB-GROUP DISCUSSION SESSION 2: URBAN PLANNING & LAND USE CONSIDERATIONS:

Co-Chairmen:

Lidia Selkregg
School of Business and
Public Administration
University of Alaska, Anchorage

Xu, Xunchu
Department of Architecture
Tongji University, Shanghai

Participants:

Grao, Lutai
Department of Architectural
Engineering, Beijing Institute
of Architectural Engineering

Song, Pei Kang
Department of City Planning,
City of Tianjin, Tianjin

Hou, Minzhong
The Design Group,
Municipal Construction Command
Tangshan

Allan Jacobs
Department of City & Regional Planning
University of California, Berkeley

Barclay Jones
Department of Urban Planning
Cornell University, Ithaca
Ithaca, New York

Wang, Zuyi
Urban Planning & Research Institute
State General Administration of
Urban Development, Beijing

Yu, Qingkang
Urban Planning & Research Institute
State General Administration of
Urban Development, Beijing

Zheng, Zuwu
Department of Science & Technology,
Bureau of City Planning, Municipality
of Beijing, Beijing

Interpreter:

Cheng Minda
Office of Earthquake Resistance
Beijing

General:

After opening general remarks by both Co-Chairmen, each stressed the importance of planning as a guide to post-earthquake recovery and to long-range pre-disaster application of disaster mitigation programs. The need for improved methods of communication between planners, engineers, architects, sociologists and policy makers to insure that reconstruction plans as well as long-range development plans for urban areas be guided by the most advanced knowledge in the various fields was also stressed. Maximum coordination among all agencies involved in recovery programs was considered a priority in planning and in implementation of programs.

Specifically, the group discussed planning "issues of joint concern" and explored "areas of common interest" in urban planning and land use programs in earthquake hazards mitigation. These topics were addressed and discussed in terms of future interaction and study through:

1) Exchange of technical information and scientific data.
2) Availability and appropriateness of potential funding of future research activities.
3) Development and preparation of joint reports and cooperative studies.
4) Exchange of personnel in the pursuit of joint research initiatives.
5) Exchange of materials and personnel through the generation of collective research projects.

At the conclusion of the discussion session, participants indicated that the exchange of ideas and concerns was very valuable to the continued development of earthquake hazards mitigation efforts in both countries. The group expressed their desire to continue dialogue on the subject in future meetings with similar open discussion. The value in establishing an exchange of ideas and publications among the delegates was acknowledged as the initial step in meeting the objectives of the discussion session. The importance of continuing open exchange now that strong contacts and personal relationships were made during the workshop period was also emphasized.

Representatives of the University of Beijing and the University of Shanghai described the exchange of students and faculty presently carried out by both universities. They pointed out that these activities were presently restricted to the faculties of the Engineering and Architecture departments and hoped to be able to expand the programs to include the planning field. Individuals from public agencies, city planning departments and State General Administrations for Urban Development observed that they have contacts in the U.S. and that they are at present engaged in exchange visits. The desire to continue this type of exchange was strongly expressed. It was also suggested that in designing schedules or travel, maximum benefit could be obtained by inter-universities, inter-cities and inter-agencies scheduling of activities. In traveling, maximum use should be made of stops between major cities to take advantage of opportunities offered by various agencies or schools en route.

Recommendations of Sub-Group Discussion Session 2:

A. Exchange of Technical Information, Materials, & Personnel:

1) Exchange of information between various universities, cities and staff of administrative agencies on topics related to development of physical data base needed for planning, mapping, computerization of physical and social data, evaluation of regional resource economics, and design criteria for urban center. (Specifically mentioned as examples were: City of Anchorage and Tangshan City, University of Alaska, University of California, Cornell University, Qinghua University, Beijing and Tongji University, Shanghai, among other major universities).

2) Information about the reconstruction of Tangshan City, how the process was carried out, how the decisions were made about land allocation, population densities and economic activities to be accommodated by the reconstruction; a) related to types of land use and geological setting, and b) methods and theory by which residential density was established for different housing developments and allocation of other land uses in respect to geological conditions.

3) Lack of information about Chinese and American planning regulations, administrative agencies, priorities and standards, particularly in regard to seismic conditions were pointed out. Desires to learn more about these topics was expressed by several participants.

4) Need for additional knowledge in methodologies related to allocation of land uses and city design. This factor was discussed not only for construction in seismic regions, but for city planning in general. Stressed was the need to develop better relationships between planners, engineers and architects in the development of cities in seismic and non-seismic regions.

5) Exchange of academic personnel and planning professionals as an educational model to learn about respective approaches to geological seismic mapping methods and techniques is assigned a high priority to serve as a base for planning programs in seismic regions.

6) Exchange of educational materials on disaster prevention methods for short and long-range planning codes, engineering initiatives, urban planning, land use, architectural activities, and other interventions) is needed immediately as an initial conclusion of the workshop. This exchange should include: text books, films, slides, curriculum development, and other educational tools and data as well as laboratory results. This material is identified as useful input to improved communications between scientists, researchers, the public and policy makers in addition to advancing teaching methodologies.

B. <u>Identification of Future Cooperative Research Projects:</u>

1) Development of methodologies for immediate relief and long-range planning efforts following an earthquake disaster. Case studies of actual experiences in planning activities during post-earthquake recovery efforts utilizing representatives and persons who have worked on urban reconstruction projects, such as for example, the cities impacted by the Tangshan earthquake of 1976 and the Good Friday earthquake in Alaska in 1964, are appropriate in this category. Comparative studies between U.S. and P.R.C. cities are cited as potential components of a research project.

2) Development of reconnaissance survey techniques for planners to be applied immediately after disasters to determine how the urban system has failed. Techniques for surveying structure failure have been highly developed and standarized; however, techniques for determining in what ways the regional social and economic systems have been impacted have not been explored. (Exchange of knowledge among Universities was considered a priority. Cornell University was especially mentioned.)

3) Further development and refinement of techniques for making rapid surveys in post disaster situations to serve as a guide to immediate urban reconstruction efforts. Design and development standards for collection and updating of data on city infrastructure, in seismic prone regions (this model could be later used for all disasters - floods, fire, wind, etc.). Listed as examples of data to be acquired were: utilities, housing, type and condition of public facilities, etc.

4) Analysis and design of urban and inter-urban transportation systems in seismic regions to allow for maximum safety of roads, airports, and ports.

5) Identify and assess large scale regional planning options and potential implementation of land use patterns for areas scheduled for new growth and development. Generation of new planning theory for urban areas located in high risk regions.

6) Evaluation and development of planning efforts for location and function of emergency service facilities and critical infrastructure relationships in the preparation of earthquake hazards mitigation programs for urban centers.

SUB-GROUP DISCUSSION SESSION 3: EXISTING BUILDING PROBLEMS

Co-Chairmen:

Neil Hawkins
Department of Engineering
Seattle, Washington

Niu, Zezhen
Institute of Building Earthquake Engineering
Chinese Academy of Building Research
Beijing

Participants:

Gong, Yongsong
Office of Earthquake Resistance
State Capitol Construction Commission, Beijing

Karl V. Steinbrugge
Structural Engineer, and Professor Emeritus
University of California, Berkeley

Earl Schwartz
Department of Building & Safety, City of Los Angeles
Los Angeles, California

Zhang, Shuquan
Tangshan Institute of Architectural Design
Tangshan

Interpreter:

Ning, He
Foreign Affairs Division
State Capitol Construction Commission
Beijing

General:

As the first item of business, each member described their normal work activities. That discussion helped identification of individual interests and concerns. All members of the sub-group were basically structural engineers. It was recognized that the Workshop had clearly identified the need for a multi-disciplinary approach to earthquake disaster mitigation and that the sub-group would have to provide the engineer input to that approach. In accordance with the scope of the Workshop, the sub-group presumed that it should confine its interests to masonry buildings.

As the second item of business, the sub-group drew up a list of items on which specific information or materials should be exchanged. Those items are listed in sections which follow.

The problem of existing hazardous buildings in the urban setting was recognized as a critical component of earthquake hazards mitigation programs common to each country. Methods of evaluating the level of hazard posed by existing buildings in urban districts of high density are needed to approach the problem on an equitable basis where limited resources are a factor. Programs in the repair and strengthening of older existing buidlings located in areas of high seismic risk were described by participants with experience in the field.

Final discussion among participants concerned the development of recommendations for future cooperative activities or research. Following a period of general discussion, the recommendations were arranged in a prioritized listing. Each recommendation is accompanied by a statement of justification which summarizes and clarifies the reasons for that recommendation.

Recommendations of Sub-Group Discussion Session 3:

A. Exchange of Technical Information, Materials, and Personnel:

1) Information on NSF earthquake engineering program staff and its relation to overall NSF staff organization, and counterpart, staff, and organizational chart for Capitol Construction Commission,

2) Description of the interrelationship between State Capital Construction Commission, Chinese Academy of Building Research, State Seismological Bureau, State Scientific and Technological Commission, Tangshan Municipal Construction Bureau and the Universities.

3) Test data for strengthening tests conducted in Los Angeles under Earl Schwartz and in P.R.C. under Niu Zezhen.

U.S. to provide to P.R.C:

(a) Masonry Design Handbook.
(b) Precast Concrete Institute Handbooks for:
  1. Structural Design
  2. Architectural Design
(c) American Concrete Institute Design Handbook.
(d) American Concrete Institute Manual of Concrete Practice.
(e) Proceedings of May 1981, Los Angeles, California, National Science Foundation sponsored workshop on Precast Concrete Construction for Seismic Zones.
(f) Proceedings of NSF workshop on Masonry Construction for Seismic Zones.
(g) List of contacts for obtaining permission for visits to Los Angeles, San Francisco, and Seattle building departments and for visits to construction materials plants and universities in those regions.

P.R.C. to provide U.S.:

(a) Information on PRC earthquake intensities, how that scale was derived, and how it is best described.

(b) Aseismic Criterion for Evaluation of Industrial and Civil Buildings (TJ23-77), 1977.
(c) Precast Concrete and Reinforced Concrete Design and Aseismic Evaluation Criteria.

B. <u>Identification of Future Cooperative Research Projects:</u>

1) Joint design project with PRC-type masonry building to be designed for construction in Los Angeles area and US-type masonry building designed for construction in Tangshan area. The project would include laboratory testing of components, if considered necessary to vindicate design assumptions.

This project would involve design of facilities only and would not include construction. This project would offer a vehicle for continuing exchange of scholars, information, and design methods between the two countries. Development of the projects would require participation by practicing architects and structural engineers as well as academics and building officials. The projects would have to take account of local material constraints, construction procedure constraints, and local design and regulation considerations.

2) Evaluation of strengthening and repairing techniques for masonry buildings, including:

(a) cost-benefit ratios for repairing versus strengthening and reconstruction,
(b) examination of the performance of repaired structures in aftershocks, and
(c) the correlation of that performance with results obtained in laboratory tests on specimens representative of portions of those repaired structures. Studies should concentrate first on buildings that were strengthened prior to the Tangshan earthquake. The evaluation of strengthening methods for existing masonry buildings should include:

(1) Steel straps and plywood diaphragms (U.S.).
(2) Reinforced concrete ring beams with transverse ties to cast-in-place or precast floors (P.R.C. and U.S.).
(3) Exterior reinforced concrete frames (P.R.C. and U.S.).
(4) Interior steel frames (U.S.).
(5) Interior shear walls of either:

i. pneumatically applied concrete (P.R.C. and U.S.).
ii. plywood on wood studs (U.S.).
iii. metal or plywood on light gage steel studs (U.S.).

That evaluation should include consideration of attachment to existing walls and the effects of openings,

The Workshop showed that there is considerable laboratory and practical experience in the U.S. and China concerning strengthening and repairing techniques. However, for essentially similar masonry structures, the strengthening and repairing techniques of the two countries are radically different. Each country has little information on the techniques used by the other, and little experience with the real effectiveness of their respective strengthening and repairing techniques for actual earthquakes. The performance of buildings in the Tangshan earthquake is the exception. There are also wide gaps in the range of test data for the different techniques used by each country. More data on the applicability of the different techniques should be developed and information on existing data exchanged between the two countries.

3) Development of a plan for collection and exchange of information on the performance of masonry buildings following the next damaging earthquake in either the P.R.C. or U.S.A.

Standard forms for collection of data on masonry buildings damaged by earthquakes have been developed both in the P.R.C. and U.S. Information of those forms should be exchanged and suggestions made for development of a common form and a common plan of action for collection of data following the next damaging earthquake.

4) Development of procedures for evaluation of earthquake resistance of existing masonry buildings.

In both the P.R.C. and U.S. existing masonry buildings often do not contain the aseismic design measures required in new building codes (such as the P.R.C. Code). Techniques for evaluating the strength of building components have been developed in the U.S. The applicability of those techniques to P.R.C. buildings and the implications of such testing for evaluation of the overall seismic resistance of existing masonry buildings need examination.

5) Development of statistics in terms of death and injury ratios for areas of differing intensity in the Tangshan and Haicheng earthquakes, and for both unreinforced and reinforced buildings.

Although such a task is not within the scope of the protocol agreement between the U.S. and P.R.C., this task, if accomplished, could have considerable implications for assessment of earthquake risk, likely deaths, and damage in future earthquakes in either the U.S. or other countries. In particular, such data would be of considerable significance for earthquake risk assessments in third world countries.

6) Performance, connection, tie and layout requirements for precast concrete in masonry buildings.

Precast concrete units are frequently used, particularly for floors and roofs in new masonry structures in both the U.S. and P.R.C. The size of such units, rules governing their layout, and the requirements for ties between walls and the precast units differ markedly in the two countries. The reasons for those differences should be identified, the performance of precast in the two countries evaluated and recommendations developed for additional testing or design requirements in both countries.

7) Construction procedures for masonry buildings.

The quality of masonry structures and their performance in earthquakes depends markedly on construction procedures used. Neither P.R.C. nor U.S. engineers are familiar with construction procedures used in the other country. However, they recognize that those procedures critically affect design requirements. Information on construction procedures should be exchanged and the impact of those procedures on likely seismic performance assessed. This recommendation can effectively be accomplished as part of the work envisaged in recommendation 1.

8) Layout considerations for new construction (aseismic measures of Chinese code).

P.R.C. data appear to show that the aseismic measures specified in the recent Chinese code have increased construction costs by approximately 50 percent. Many of those aseismic measures concern layout considerations. U.S. personnel are interested in reviewing those considerations, determining their basis, and considering their applicability in the U.S. This recommendation can effectively be accomplished as part of the work envisaged in recommendation 1.

SUMMARY AND CONCLUSIONS OF WORKSHOP

The workshop clearly identified the goals of earthquake hazards mitigation efforts with public safety and damage control objectives. The responsibility for the design and planning of our urban centers rests with the planning and design professions which include architecture, planning, urban design, and engineering. To be successful an interdisciplinary approach is needed in which each profession makes a specific contribution toward the solution of problem areas in the reduction of earthquake hazards. During the discussion session which followed the formal presentation of papers, many problem areas were recognized as needing immediate attention.

During the course of the Workshop, it was also acknowledged that this represented the first time that architects and planners from the U.S. and P.R.C. met as a formal group with representatives from the engineering disciplines to address problems of joint interest and common concern. This type of exchange at the international level must continue now that the Workshop successfully established a starting point for the pursuit of future joint research projects on a cooperative basis and the need for the continued exchange of technical information between both countries. The need to have a "follow-up" reciprocal Workshop within a two year period in the U.S. was clearly endorsed. Dialogue on the subject of seismic safety in our cities must continue.

A common issue which emerged from the Workshop was the realization that many gaps of knowledge exist in earthquake hazard mitigation efforts. In several instances during the discussion sessions methods used to currently address safety issues were clearly characterized as "state-of-the-art" approaches to a most complex problem. Many questions and concerns remain unanswered and, as a result, much research must be done. Although many projects in damage reduction have been categorically studied, it is clear that elements of our urban environments remain vulnerable to severe earthquakes. Inhabitants, buildings, systems, contents, services and functions are subjected to a multitude of interrelated and complex situations of stress. The Workshop and the specific contributions made by all participants assisted in shaping a common understanding of the problem. From this, it is hoped that a more integrated approach to accurate and applicable results is possible in the future through major interdisciplinary research activity. Considering the technical complexity of efforts in earthquake hazards reduction, it is understandable that a balanced approach must be taken. Basic principles of scientific investigation and analysis must be followed.

Participants in the Workshop clearly and successfully identified critical topic areas needed in the exchange of technical information, materials, and personnel between the two countries. During the discussion sessions, a full range of future joint, cooperative research projects were established and targeted for development as realistic efforts keyed to specific recommendations. On a more general and representative basis, Figure 1 and Figure 2 (below) display recommendations on (a) the exchange of technical information, materials, and personnel, and (b) potential joint, cooperative research activities of mutual interest and joint concern to both countries. The two Figures display representative categories only, and, accordingly, reference is made to the preceeding pages which describe specific initiatives and activities in detail as products of the three sub-group discussion sessions.

In closing, it is hoped that the recommendations on, and potential results of exchange activities and future joint, cooperative research efforts to follow will make our cities safer places so that the disasters of the past will not be repeated. Through joint, cooperative efforts this goal of improving the seismic safety of urban environments, which as established as one of the fundamental objectives of this Workshop, will be successfully addressed.

FIGURE 1

GENERAL RECOMMENDATIONS
FOR EXCHANGE OF TECHNICAL INFORMATION, MATERIALS, AND PERSONNEL
REPRESENTATIVE CATEGORIES

GROUP NO.	SUB-DISCUSSION GROUP	BUILDING DESIGN	REPAIR & STRENGTHENING EXIST. BUILDINGS	TECHNICAL MATERIAL & INFORMATION	TEST DATA	URBAN DESIGN	PROCEEDING OF SEISMIC WORKSHOPS	ORGANIZATION CHART OF INSTITUTIONS	LIST OF CONTACTS FOR HOSTING VISITS	EXCHANGE OF PERSONNEL	CRITICAL FACILITIES AND SERVICES DATA	MAPPING TECHNIQUES	WORKSHOPS AND TEACHING SEMINARS
1	ARCHITECTURAL & NON STRUCTURAL CONSTRUCTION	●	●	●	●	●	●		●	●	●	●	●
2	URBAN PLANNING & LAND USE		●	●		●			●	●	●	●	●
3	EXISTING BUILDINGS	●	●	●	●	●	●	●	●	●	●	●	●

FIGURE 2

GENERAL RECOMMENDATIONS FOR JOINT COOPERATIVE ACTIVITIES
REPRESENTATIVE CATEGORIES

GROUP NO.	SUB-DISCUSSION GROUP	BUILDING DESIGN	REPAIR & STRENGTHEN EXIST. BUILDINGS	DEVELOPMENT OF TECHNICAL MATERIAL / INFORMATION	LABORATORY TEST EXPERIMENTS	PRINCIPLES & THEORY OF URBAN DESIGN	PLANNING AND DESIGN EDUCATION	DAMAGE ASSESSMENTS	COLLECTION OF EARTHQUAKE FIELD DATA	AFTER DISASTER SURVEY AND MAPPING TECHNIQUES	URBAN PLANNING & DESIGN OF CRITICAL FACILITES	WORKSHOPS & SEMINARS	URBAN TRANS-PORTATION STUDIES
1	ARCHITECTURAL & NON STRUCTURAL CONSTRUCTION	●	●	●	●	●	●	●	●	●	●	●	
2	URBAN PLANNING & LAND USE		●	●		●	●	●	●	●	●	●	●
3	EXISTING BUILDINGS	●	●	●	●	●	●	●	●	●	●	●	

THE CLOSING REMARKS BY VICE MINISTER PENG MIN, SCCC

(afternoon Nov. 6th, 1981)

Participants, Friends:

After 5 days papers presentation and discussions at the joint workshop "Earthquake Disaster Mitigation Through Architecture, Urban Planning and Engineering", today it comes to an end. On behalf of the State Capital Construction Commission of the People's Republic of China, I wish to extend the warm congratuations on the complete success of this workshop.

As everybody pointed out in the papers and discussions that the people's casualties and economic loss plagued by earthquakes are mainly resulted from the collapse and destruction of the buildings and resulted from the unreasonable urban planning. From this workshop we may view that there are indeed great potentiality and broad prospect on earthquake hazards mitigation by developing the measures through architecture, urban planning and engineering.

The damage of the building components or non-structural components of the building could not only kill and injure people, destroy the surrounding articles, but also it holds the considerably dominant position in the lose caused by the damage of the architectures. For an example, according to the record of the concerned information, the total loss suffered from the destruction of the architectures in the earthquake of San Fernando in the United States in 1971 is 200 million pounds, in which half of the loss was induced by the damage of non-structural components. In our country, the earthquake-prone areas are 1/3 of the total national land, experiencing more or less dozens times of seismic shocks at between 5 to 6 of magnitutes each year. Although these did not give the severe impact on people's life, but the major part of the property loss were effected from the failure of the non-structural components. Therefore to study the measures taken in the earthquake resistant design on the building components of housing is no doubt very important.

The unreasonableness of the urban planning in the seismic risk areas would not only make the cities paralysed during earthquake, but also would impede the effective work of the relieving, recovery and rebuilding post-earthquakes, thus it would make the seismic disaster even worse. The experience in Tangshan of our country has fully shown this problem.

The evaluating and strengthening of the existing buildings pre-earthquakes is an efficient measures for minimizing the seismic damages. This has already as proved by the results of several earthquakes occurred in our country in recent years. Some strengthened buildings in Tienjin after 1975's Haicheng earthquake were basically in good condition in the event of Tangshan earthquake in 1976. Besides, before the disaster of Daofu country of Sichuan province in 1981 struck, the post office of the countyyhad been strengthened , in spite of all the nearby buildings collapsed this post office building still remained undamaged. So after 15 minutes post-earthquake the office got contact with outside. All above facts are very convincing examples. Today in our country the housing with 92 million square meters have already been strengthened, in addition to many bridges, facilities and some important dams and reservoirs. Our government has determined to go on the work of reinforcing and earthquake-proofing in a planned way in order to achieve the greater benefits.

Through the simple reviews we may see that our architects, urban planners and engineers will have the bright prospects for earthquake hazards mitigation by architecture, urban planning and engineering.

At this workshop we have exchanged the achievements in our research work as well as the experiences in this field between two countries. And also we have explored the possibilities of the future cooperation. This is a good begining. I hope we will develop the enduring exchanges and cooperations in the area of the seismic engineering between China and the United States. As both of our two countries are seismically active countries, so I believe that these mutual exchanges and cooperations would be beneficial for both sides.

In conclusion, I would like to wish participants and friends the good health, and I also wish the American friends a pleasant stay in China.

APPENDIXES

附录 I

中　　美

通过建筑、城市规划和工程减轻地震灾害讨论会

日程表及代表名单

Appendix I

P.R.C.—U.S.A.

JOINT WORKSHOP ON EARTHQUAKE DISASTER MITIGATION THROUGH ARCHITECTURE, URBAN PLANNING AND ENGINEERING

PROGRAM AND LIST OF PARTICIPANTS

中国　北京

1981年11月2—6日

Beijing China

November 2—6, 1981

中　　美
通过建筑、城市规划和工程减轻地震灾害
讨　论　会

本讨论会系根据"中美地震研究科学技术合作议定书"附件三
"地震工程与减轻地震灾害合作研究"计划举行

1981年11月2日至6日

中国　北京

中方主席：叶耀先

美方主席：拉格里奥

主　办　单　位：

中国国家建委抗震办公室，美国伯克利加州大学规划和发展研究中心
美国国家科学基金会

筹　备　单　位：

中国国家建委抗震办公室

筹备委员会名单：

主任：叶耀先

秘书：赵云栋
龚永松

成员：高　越
胡陵玉
刘志刚
宁　和
师云林
綦泰益

会 议 日 程

一、1981.11.1.　　　美国代表团抵京

二、1981.11.2.　　　星期一

1.代表报到　　　8:00～8:30

2.会议开幕　　　8:30～10:00

会议主席：叶耀先（中方）

拉格里奥（美方）

(1)国家建委付主任彭敏同志致欢迎词

(2)报告会议筹备工作：叶耀先，拉格里奥

3.第一次会议　　10:00～12:00

（一）建筑和非结构构件抗震

会议主席：何广麟　（中方）

拉格里奥（美方）

论文宣读：

(1)叶耀先：地震区的建筑设计和城市规划

(2)阿诺德：建筑体形对抗震性能的影响

4.午餐　　12:00～13:30

5.继续第一次会议　　13:30～17:30

论文宣读：

(3)拉格里奥：城市中心医院的地震安全：加州经验

(4)杨玉成：关于建筑体形和立面处理的若干抗震问题

(5)麦克尔：次级建筑系统的抗震设计：设计方法与可行性限制

(6)刘兆丰：1976唐山地震多层砖房震害实例和调查分析

三、1981.11.3.　　　星期二

1.继续第一次会议　　8:00～12:00

论文宣读：

(7)杨文忠：1976年唐山地震唐山市房屋非结构构件的震害

(8)王玛莎：建筑风格对抗震上的影响

(9)陈谋莘：建筑构造的震害分析和改进意见

(10)何广麟：地震区单层厂房建筑设计中的几个问题

2.午餐　　　12:00～13:30

3.第二次会议　　13:30～17:00

（二）地震区的城市规划和土地利用

会议主席：余庆康　（中方）

雅克布斯（美方）

论文宣读：

(11)雅克布斯：旧金山市地震安全规划：规划的制定与通过

(12)侯民忠：1976年唐山地震后唐山市的重建规划

(13)琼　斯：地震灾区的重建规划

(14)王祖毅：地震区城市用地问题

四、1981.11.4　　　　星期三

1.继续第二次会议　8:00～12:00

论文宣读：

(15)徐循初：地震区城市交通运输规划和震后应急措施

(16)塞尔克雷格：地震区规划

(17)宋培抗：天津市区土地利用中与抗震有关的几个问题

(18)克利姆格尔德：美国关于减轻地震灾害的建筑与规划研究

2.午餐　　12:00～13:30

3.第三次会议　13:00～17:30

（三）砖结构的抗震鉴定、加固和修复

会议主席：叶耀先（中方）

斯坦布鲁奇（美方）

论文宣读：

(19)龚永松：砖结构的抗震鉴定

(20)郝肯斯：华盛顿西雅图房屋修建方针

(21)钮泽蓁：砖结构抗震加固计算

(22)麦克尼弗恩：圬工窗间墙抗震性能试验研究

五、1981.11.5　　　　星期四

1.继续第三次会议　8:00～12:00

论文宣读：

(23)张树全：唐山市震损房屋的修复

(24)施沃兹：减轻洛杉矶市现有建筑的地震灾害

(25)斯坦布鲁尔：现有危险性房屋；震后地震影响评定

2.午餐　　12:00～13:30

3.分组讨论说明　13:30～14:00

叶耀先

拉格里奥

4.各组集中　　14:00～14:15

5.分组讨论　　14:15～17:30

(1)建筑和非结构构件组

组长：　麦克尔（美方）

何广麟（中方）

(2)城市规划和土地利用组

组长：　塞尔克雷格（美方）

徐循初（中方）

(3)现有房屋的鉴定、加固和修复组

组长： 郝肯斯 （美方）

钮泽蓁（中方）

六、1981.11.6 星期五

1.继续分组讨论 8:30～9:30

2.全体会议 9:30～12:00

会议主席：叶耀先（中方）

拉格里奥（美方）

1)小组讨论情况介绍：各分组组长

2)一般讨论

3.午餐 12:00～13:30

4.会议闭幕 13:30～14:30

专题讨论会小结

拉格里奥（美方）

叶耀先（中方）

闭幕讲话 彭敏付主任

5.会议结束 14:30

6.参观清华大学 14:30～17:30

代 表 名 单

美 国 代 表

1.拉格里奥：

代表团团长，伯克利，加利福尼亚大学环境设计学院规划与发展研究中心主任

2.阿诺德

加利福尼亚旧金山建筑系统开发公司总裁

3.麦克尔

剑桥哈佛大学设计研究生院院长

4.雅克布斯

伯克利：加利福尼亚大学、地区规划部教授

5.琼斯

纽约科内尔大学城市、地区规划系教授

6.塞尔克雷格

安克雷奇阿拉斯加大学商业和公共管理学院教授

7.斯坦布鲁奇

加利福尼亚结构工程师荣誉退休教授

8.郝肯斯

华盛顿大学土木工程系教授

9.施沃兹

加利福尼亚洛杉矶建筑安全部地震安全处处长

10.克利姆格尔德

国家科学基金会土木和环境工程处减轻地震灾害计划经理

11.麦克尼弗恩

伯克利，加利福尼亚大学地震工程研究中心主任

12.王玛莎

伯克利，加利福尼亚大学建筑系教授

中 国 代 表

1.叶耀先

国家建委抗震办公室副主任、工程师

2.龚永松

国家建委抗震办公室技术处负责人，工程师

3.汪宗荣

国家科委二局副处长，工程师

4.邹其嘉

国家地震局外事处副处长

5.戴念慈

中国建筑科学研究院总建筑师

6.钮泽蓁

中国建筑科学研究院工程抗震研究所研究室副主任、工程师

7.余庆康

国家城建总局城市规划设计研究所副总建筑师

8.王祖毅

国家城建总局城市规划设计研究所工程师

9.吴良镛

北京清华大学建筑系主任、教授

10.刘兆丰

北京清华大学建筑系讲师

11.徐循初

上海同济大学建筑系副教授

12.何广麟

天津大学建筑系副教授

13.蒋孟厚

西安西北建工学院建筑系副主任，副教授

14.高履泰

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15.郑祖武

北京市城市规划局副总工程师

16.杨玉成

哈尔滨中国科学院工程力学研究所助理研究员

17.陈谋莘

北京市建筑设计院工程师

18.宋培抗

天津市城市规划局工程师

19.候民忠

唐山市建设指挥部工程师

20.杨文忠

唐山市城市建设局工程师

21.张树全

唐山市设计院工程师

下列代表曾应邀参加会议，但因事未能出席：

吴良镛　北京清华大学

戴念慈　北京中国建筑科学研究院

麦克尼弗恩　伯克利，加利福尼亚大学地震工程研究中心

P. R. C. —U. S. A.

JOINT WORKSHOP ON EARTHQUAKE DISASTER MITIGATION THROUGH ARCHITECTURE, URBAN PLANNING AND ENGINEERING

THIS WORKSHOP IS HELD UNDER THE
P.R.C.—U.S.A. PROTOCOL FOR SCIENTIFIC AND TECHNICAL
COOPERATION IN EARTHQUAKE STUDIES
(ANNEX Ⅲ "COOPERATION RESEARCH ON EARTHQUAKE
ENGINEERING AND HAZARD MITIGATION")

November 2—7, 1981.
Beijing, China

Co-Chairman
Ye, Yaoxian (P.R.C. side)
Henry J. Lagorio (U.S.A. side)

Sponsored by the
Office of Earthquake Resistance, State Capital Construction Commission of P.R.C.
Center for Planning and Development Research, University of California,
Berkeley, California, U, S, A, National Science Foundation of U.S.A.

Organized by
Office of Earthquake Resistance, State Capital Construction Commission of P.R.C.

Members of The Organizing Committee

Chairman	Ye, Yaoxian
Secretary	Zhao, Yundong
	Gong, Yongsong
Members	Gao, Yue
	Hu, Lingyu
	Liu, Zhigang
	Ning, He
	Shi, Yunlin
	Li, Taiyi

WORKSHOP PROGRAM

Ⅰ. November 1, 1981 Sunday

Arrival of U.S.A. participants

Ⅱ. November 2, 1981 Monday

1. Registration and opening of workshop, 8:00—8:30 AM

2. Formal Convening of workshop, 8:30—10:00 AM

Chairmen: Ye Yaoxian (P.R.C. side)

H.J. Lagorio (U.S.A. side)

Welcoming Address by: Vice Minister of SCCC Peng Min

Opening Remarks on Organization of Workshop:

Ye Yaoxian (P.R.C. side)

H.J. Lagorio (U.S.A. side)

3. Session 1: 10:00～12:00 AM

(Ⅰ). Architectural and Non-Structural considerations

Session Chairman:

He Guanglin (P.R.C. side)

H.J. Lagorio (U.S.A. side)

Presentation of Papers:

1) Ye Yaoxian

Architectural Design and Urban Planning for Seismic Region

2) Christopher Arnold

Building Configuration Influence on Seismic Performance: The western Experience

4. Lunch: 12:00—1:30 PM

5. Continuation of Session 1: 1:30—5:30 PM

Presentation of Papers:

3) Henry J. Lagorio

The Seismic Safety of Hospitals in Urban Centers: The California Experience

4) Yang Yucheng

Effect of Configuration of Building on Earthquake Resistance and Its Management

5) Gerald McCue

Design of secondary Building Systems for Seismic Effects: Approoches to Design on Limits of Feasibitity

6) Liu Zhaofeng

Observation and Analysis of Damage to Multistory Brick Building in the 1976 Tangshan Eanthquake

Ⅲ. November 3, 1981 Tuesday

1. Continuation of Session 1: 8:00~12:00 AM

Presentation of Papers:

7) Yang Wenzhong

Damage of Non-Structural Components During the 1976 Tangshan Earthquak

8) Marcy Wang

Consequenes of Architectural Style on Earthquake Resistance

9) Chen Mouxin

Earthquake Destruction Analysis of Building Construction and Improvemente Proposals

10) He Guanglin

Some Problems on Architectural Design of Single Storey Factory Building in Seismic Area

2. Lunch: 12:00~1:30 PM

3. Session 2: 1:30~5:30 PM

(Ⅱ) Urban Planning and Land Use

Session chairmen:

Yu Qingkang (P.R.C. side)

Allan B. Jacobs (U.S.A. side)

Presentation of Papers:

11) Allan Jacobs

The Seismic Safety Plan for San Francisco: Its Preparation and Adoption

12) Hou Minzhong

Reconstruction Planning for Tangshan City After 1976 Tangshan Earthquake

13) Barclay Jones

Planning for the Reconstruction of Earthquake Stricken Communities

14) Wang Zuyi

Urban Land use Planing in Seismic Regions

Ⅳ. November 4, 1981 Wednesday

1. Continuation of Session 2: 8:00~12:00 AM

Presentation of Papers:

15) Xu Xunchu

City Traffic Planning and Emergency Measures After an Earthquake in Seismic Areas

16) Lidia Selkregg

Planning for Earthquake-Prone Regions

17) Song Bekan

A Few Problems of Land Use in Relation to Earthquake-Proof in Tianjin City

18)Frederick Krimgold

Architectural and Planning Research for Earthquake Hazard Mitigation in the United States

2.Lunch: 12:00—1:30 PM

3.Session 3: 1:30—5:30 PM

(Ⅲ)Existing Buildings

Session Chairmen:

Ye Yaoxian (P.R.C. side)

Karl Steinbrugge (U.S.A. side)

Presentation of Papers:

19)Gong Yongsong

Aseismic Evaluation for Brick Structures

20)Neil Hawkins

Building Rehabilitation Strategies in Seattle, Washington

21)Niu Zezhen

Calculating Methods of Strengthened and Repaired Brick Masonry Structures for Earthquake Resistance

V. November 5, 1981 Thursday

1.Continuation of Session 3: 8:00~12:00 AM

Presentation of Papers:

22)Zhang Shuquan

Repair of Damaged Structures in Tangshan City

23)Earl Schwartz

Earthquake Hazard Reduction for Existing Building in the City of Los Angeles

24)Karl Steinbrugge

Existing Hazardous Buildings: Assessing Direct Post-Earthquake Effects

2.Lunch: 12:00—1:30 PM

3.Sub-Group Discussions:

Opening Remarks on Organization of Discussion sessions: 1:30~2:00 PM

Ye Yeoxian (P.R.C. side)

H.J.Lagorio (U.S.A. side)

4.Division into Sub-Groups: 2:00~2:15 PM

5.Sub-Group Discussion Sessions: 2:15—5:30 PM

(1)Architectural and non-Structural Considerations

Sub-Group Discussion Chairmen:

Gerald McCue (U.S.A. side)

He Guanglin (P.R.C.side)

(2)Urban planning and Land Use

Sub-Group Discussion Chairmen

Lidia Selkregg (U.S.A. side)

Xu Xunchu (P.R.C. side)

(3)Existing Building

Sub—Group Discussion Chairmen

Niel Hankins (U.S.A. side).

Niu Zezhen (P.R.C. side).

VI. Novembet 6, 1981 Friday

1. Continuation of Sub—Groups Discussion: 8:30—9:30 AM

2. Plenary General Session: 9:30—12:00 AM.

Chairmen:

H.J.Lagorio (U.S.A. side).

Ye Yaoxian (P.R.C. side).

1)Reporting of Sub—Goup

Chairmen of Sub—Groups

2)General Discussion

3. Lunch 12:00—1:30 PM

4. Concluding Workshop 1:30~2:30 PM

Chairman:

Ye Yaoxian (P.R.C. side)

H.J.Lagorio (U.S.A. side)

Closing Remarks: Vice Minister Peng Min

5. Adjournment of Morkshop 2:30 PM.

6. Visiting Qinghua University 2:30—5:30 PM.

LIST OF PARTICIPANTS

Participants From United States

1.Henry J. Lagorio (Delegation Head)
Director, Center for Planning and Development Research
College of Environmental Design, UC Berkeley

2.Christopher Arnold
President, Building Systems Development, Inc.San Francisco, California

3.Gerald McCue
Dean, Graduate School of Design, Harvard University, Cambridge

4.Allan Jacobs
Professor, Department of City and Regional Planning, UC Berkeley

5.Barclay Jones
Professor, Department of City and Regional Planning, Cornell University, Ithaca, New york

6.Lidia Selkregg
Professor, School of Business and Public Administration, University of Alaska, Anchorage

7.Karl Steinbrugge
Structural Engineer and Professor Emertus El Cerrito, California

8.Neil Hawkins
Professor, Department of Civil Engineering University of Washington, Seattle, Washington

9.Earl Schwartz
Chief, Earthquake Safety Division, Department of Building and Safety, Los Angeles, California

10.Frederick Krimgold
Program Manager, Earthquake Hazards Mhtigation, Division of Civil and Environmental Engineering National Science Foundation

11.Hugh McNiven
Director, Earthquake Engineering Research Center, Richmond Field Station, US Berkeley

12.Marcy Wang
Professor, Department of Architecture, UC Berkeley

Participants From People's Republic of China

1.Ye Yaoxian
Engineer, Vice Directer, Office of Earthquake Resistance,
State Capital Construction Commission, Beijing, China

2.Gong Yongsong
Engineer, Leading Member of Technologial Division, Office of Earthquake Resistance, State Capital Construction Commission, Beijing, China

3.Wang Zongrong
Engineer and Deputy Chief, Second Bureau, State Scientific and Technological Commission, Beijing, China

4.Zou Qijia
Deputy Chief Division of Foreign Affairs
State Seismological Bureau, Beijing, China

5.Dai Nianci
Chief Architect, Chinese Academy of Building Research, Beijing, China

6.Niu Zezhen
Engineer and Deputy Head of Research Division,
Institute of Building Earthquake Engineering,
Chinese Academy of Building Research, Beijing, China

7.Yu Qingkang
Deputy Chief Architect, Urban Planning and Research Institute,State General Administration of Urban Development,Beijing, China

8.Wang Zuyi
Engineer, Urban Planning and Research Institute, State General Administratoin of Urban Development, Beijing, China

9.Wu Liangyong
Professor, Dean of Department of Architecture, Qinghua University, Beijing, China

10.Liu Zhaofeng
Lecturer, Department of Architecture, Qinghua University, Beijing, China

11.Xu Xunchu
Associate Professor,Department of Architecture,Tongji University, Shanghai, China

12.He Guanglin
Associate Professor and Deputy Director, Teaching Group of Architectural Design, Department of Architecture, Tianjin University,

13.Jiang Menghou

Associate Professor, North—Western College of Architecture and Civil Engineering

14. Gao Lutai

Associate Professor, Vice Dean of Department of Architectural Engineering Beijing Institute of Architectural Engineering

15. Cheng Zuwu

Deputy Chief Engineer and Directer of Department of Science and Technology, Bureau of City—planning, Municipality of Beijing,

16. Yang Yucheng

Research Associate, Institute of Engineering Mechanics (IEM), Academia Sinica, Harbin

17. Chen Mouxin

Architect, Beijing Institute of Architectural Design

18. Song Peikang

Engineer, Tianjin Department of City Planning

19. Hou Minzhong

Engineer, Vice head of The Designing Group of the Tangshan Municipal Constuction Command

20. Yang Wenzhong

Engineer, Tangshan Municipal Construction Bureau

21. Zhang Shuqur

Engineer, Tangshan Institute of Architectural Design

The following were invited to participate in the Workshop, but because of scheduling difficulties were unable to attend:

Wu, Liangyong, Qinghua University, Beijing

Dai, Nianci, Chinese Academy of Building Research, Beijing

McNiven, Hugh, Earthguake Engineering Research Center, Richmond Field Station, US Berkeley

APPENDIX II ITINERAY OF THE U.S.A. DELEGATION

Sunday, November 1, 1981
Evening — Arrived in Beijing

Monday, November 2
Morning and Afternoon — Attended Workshop
Evening — Attended Welcoming banquet hosted by the SCCC

Tuesday, November 3 — Attended Workshop

Wednesday, November 4 — Attended Workshop

Thursday, November 5 — Attended Workshop

Friday, November 6
Morning — Attended Workshop
Afternoon — Attended workshop
Visited Qinghua(Tsinghua) University. Met with:
Wang Tan Professor and Vice Chairman of Architectural Department
Cai Junfu Associate Professor and Vice Chairwoman of Architectural Department
Gao Yilan Associate Professor and Head of the First Research Division of Architectural Design, Department of Architecture
Paul Chow(Zhou Puyi) Professor, Department of Architecture
Wu Huanjia Associate Professor and Head of Teaching and Research Section of Architectural History and Theory
Wang Guozhou Professor and Chairman of Department of Civil and Environmental Engineering
Shen Juming Associate Professor and Head of Research Laboratory of Earthquake Engineering and Blast Resistant Engineering
Wang Chuanzhi Professor, Department of Civil and Environmental Engineering
Liu Zhaofeng Lecturer, Department of Architecture
Fang Erhua Lecturer, Department of Civil and Environmental Engineering

Evening — Banquet hosted by U.S.A. Delegation

Saturday, November 7 — Visited the Great Wall and Summer Palace

Sunday, November 8
Morning — Departed Beijing by train for trip to Tianjin. Arrived in Tianjin and visited First Machine Factory of Tianjin and damaged building in 1976 Tangshan Earthquake. Met with:

	Liu Wenfen — Vice Director of Tianjin Capital Construction Commission
	Liu Yukun — Vice Head of Office of Earthquake Resistance in Tianjin
	Wei Sushen — Engineer, Office of Earthquake Resistance in Tianjin
	Jin Kuoliang — Deputy Head of Tianjin Earthquake Engineering Institute
	Wang Chungtsun — Deputy Head of Tianjin Earthquake Engineering Institute
	Shun Chiunyu — Deputy Head of Office of Construction, First Machine Factory in Tianjin
Noon	Attended banquet hosted by TCCC
Afternoon	Visited Shuishang Park.
	Departed Tianjin for train trip to Tangshan
Evening	Arrived in Tangshan. Met with representatives from the Hebei Province Capital Construction Commission for welcome and discussions.
Monday, November 9	
Morning	Toured Tangshan to see offset of the fault, earthquake ruins and reconstruction work. Met with:
	Wang Zixing — Deputy Director, Capital Construction Commission of Hebei Province
	Zhao Fengming — Chairman of the Foreign Affairs Office of Tangshan Municipality
	Li Baoxian — Deputy Director, Capital Construction Commission of Tangshan Municipality
	Cao Yuhua — Engineer
	Jiang Henien — Interpreter, Foreign Affairs Office of Tangshan Municipality
	Li Chingyu — Chairman of Plant Office, Tangshan Plant of Locomotive and Carriage of Railway Ministry
	Li Zhantian — Chairman of Institute Office, Mining and Metallurgy Institute of Hebei Province
	Zhang Yinfang — President, Twelfth Middle School of Tangshan City
Noon	Attended banquet hosted by HCCC
Afternoon	Some delegates visited construction site of First Guest House of Tangshan Prefecture. Other delegates visited Sixth Factory of Tangshan Ceramics Company. Met with:
	Pan Bingchen — Director, Sixth Factory of Tangshan Ceramics Company
	Departed Tangshan for train trip to Beijing