Earthquake Resistance of High-Rise Systems

DIGESTS OF CASE STUDIES

OF TALL BUILDINGS DAMAGED IN EARTHQUAKES

By

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Research Supported By

NATIONAL SCIENCE FOUNDATION

Grant No. CEE-8105306

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December 1983

Fritz Engineering Laboratory Report No. 474.6

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[1] Numbers in parenthesis identify the event number consisting of the year and a sequence number

ABSTRACT

Keywords: earthquake; seismic; damage; tall buildings; structural system; nonstructural elements; failures: configuration: irregularities.

This report contains a collection of digests of detailed case-studies on tall buildings damaged in earthquakes and a preliminary assessment of the available data.

Each digest identifies site, ground motion, soil conditions: high-rise systems (structural, architectural, mechanical); design and construction related data as year of construction, governing code, design assumptions and construction practices; damage; cause for observed performance and failures, and recommendations or lessons learned.

The digest portion of the report is followed by a presentation of classification schemes for high-rise systems, earthquake, and damage, and a preliminary assessment of data. The investigated parameters material (steel versus concrete), tallness (high-rise versus medium-rise), configuration (regular versus irregular), and structural system (other versus moment-resisting frames) were found to affect earthquake damage in tall buildings in this order of importance. For all investigated parameters the first class experiences less damage than the second. Except for the last parameter, these performance differences could be confirmed as statistically significant if the data for all levels of earthquake intensity were combined. For individual intensity levels, however, this was not possible in general.

The preparation of the digests and the preliminary assessment of data form a first step in the research work on correlating earthquake performance of high-rise systems with specific characteristics of these systems.

Citation: Mikroudis, G., K., and Mueller, P. 1983, DIGESTS OF CASE STUDIES OF TALL BUILDINGS DAMAGED IN EARTHQUAKES, Technical Report No. 474.6, Fritz Engineering Laboratory, Lehigh University, Bethlehem. PA

1. INTRODUCTION

1.1 PURPOSE

This study is part of a research project being conducted at Lehigh University on the earthquake resistance of high-rise systems, under sponsorship by the National Science Foundation. The major questions addressed by this project are:

1. What are the specific characteristics of the tall building systems (structural, architectural, and mechanical) that are built throughout the world?
2. How have these systems performed in earthquakes?
3. Can the performance be correlated with particular systems?

Many data on the performance of high-rise systems are available from reports of damage evaluation teams formed to study the effects of particular earthquakes and from individual case-studies. However, a systematic correlation between specific high-rise systems and their performance is not available so far. Degenkolb [1980] claims that systems known to be deficient are being reintroduced in practice. Since California engineers are in the forefront of earthquake-resistant design, it seems clear that improved documentation of the suitability of various systems is needed.

This report contains a collection of digests of detailed case studies from the literature of post-earthquake surveys. It also includes a preliminary analysis of data from the 40 digests and another 44 well documented buildings. The digests may be valuable by themselves to a reader who would like to familiarize himself with earthquake damage patterns without having to examine the extensive source literature. They were primarily prepared, however, as the first step in the development and refinement of the classification schemes for high-rise systems, ground motion, and damage needed for the analysis of a more extensive data base.

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1.2 BACKGROUND

Correlation of the seismic performance of high-rise systems with specific characteristics of these systems involves, first, establishing ground motion - damage relationships for different classes of high-rise systems, and, second, comparing these different relationships.

Quantitative ground motion - damage relationships and methods to develop them have been reported by various investigators. Early impulses came from investigators interested in the prediction of earthquake-induced economic losses in the context of earthquake insurance and earthquake hazard mitigation policies. The field of earthquake loss prediction and damageability is relatively young and still in development. A detailed literature review is given by Scholl et al, [1982]. Basically, one can distinguish empirical and analytical/theoretical methods to derive ground motion - damage relationships.

Empirical methods rely on a statistical analysis of data from past earthquakes and usually result in average ground motion - damage relationships for large classes of buildings. The main problem lies in developing a reliable data base. Data reporting is often inconsistent in format, subjective, and incomplete, particularly regarding undamaged buildings [Scholl, 1982]. Also, in the majority of cases, ground motion is only reported in terms of Modified Mercalli Intensity (MMI), a rather crude and biased measure.

Theoretical methods are based on engineering principles and probabilistic concepts. They require analysis of a structural model of the building. Total damage is obtained by summing component damage. Local model response is related to component damage in various ways including expert judgment and, more recently, empirical component damage - component response functions derived from laboratory tests [Kustu, 1981]. Obviously, theoretical methods are directed towards individual buildings, although a typical building may represent a class of buildings. The main problem lies in a reliable structural analysis. Reliable local component response prediction requires an inelastic dynamic analysis including the effect of nonstructural elements. In either method, damage is often expressed in terms of damage ratio, the ratio of repair cost to replacement cost, or in terms of the verbally described damage states that Whitman developed and correlated with ranges of the damage ratio [Whitmanx, 1973, Council, 1981].

Two major research programs. both of which specifically address high-rise buildings, are of importance here: The program "Seismic Design Decision Analysis" conducted by Whitman et al. [1972a] at MIT and the program "Seismic Damage Assessment For High-Rise Buildings" conducted by Scholl et al. [1982] at URS/Blume & Associates. Both programs include analytical studies, [Wong, 1975, Scholl, 1982, Kustu, 1981], and empirical studies, [Whitmanx, 1973, Wong, 1975, Scholl. 1982]. Empirical results are reported in the form of the damage probability matrices introduced by Whitman, [1973], or of mean damage ratio vs. motion intensity. Both studies use the MMI scale for motion intensity. The URS/Blume study also uses the Engineering Intensity Scale (EIS) introduced by Blume, [1970]. Results compare well for MMI VIII, but differ considerably for smaller earthquakes, apparently because the MIT data base was less complete regarding undamaged structures. Results are presented for two classes of high-rise buildings: steel buildings and reinforced concrete buildings. The MIT and URS/Blume studies again agree well in that both investigators show that reinforced concrete buildings consistently exhibit higher mean damage ratios. While this result appears reasonable considering the fact that the majority of the buildings in the data base were constructed before 1930, a more refined building classification might result in a more complex picture. One might suspect, for instance, that modern reinforced concrete buildings employing stiff and ductile shear walls exhibit lower damage ratios than steel buildings relying on flexible moment-resisting frames, particularly for moderate earthquakes, for which damage is primarily confined to nonstructural elements.

Classification schemes for high-rise systems have been reported by various researchers. Regarding classification of structural systems, one can distinguish between schemes that differentiate between lateral load resisting and gravity load resisting systems, and schemes that do not. The earthquake engineering profession is accustomed to distinguishing between the two functions. However it must be realized that more often than not systems serve both functions and do not respond according to the simplified design assumptions. Falconer and Beedle, [1982]. review the different classification schemes and propose a series for consideration.

1.3 SCOPE

The program for the performance - systems correlation envisions the following steps:

1. Preparation of digests of case-studies for about 40 well-documented buildings.
2. Preparation of classification schemes for high-rise systems, damage and ground motion.
3. Preliminary assessment of data.
4. Finalization of classification schemes.
5. Collecting and classifying additional data.
6. Analysis of data.

This report presents work on the first three tasks.

Successful correlation of building performance with high-rise systems (structural, architectural, mechanical) requires that the classification schemes recognize the most important characteristics that affect performance. Classifying means generalizing and, hence, loss of data considered less important. Whether or not particular characteristics are important clearly depends on the purpose for which the classification scheme is used. Thus regularity/irregularity [ATC, 1978] of a building, which is considered important regarding seismic performance, is likely to be irrelevant in a fire hazard study.

It was decided therefore to precede the development of classification schemes with the preparation of an initial data base of about 40 buildings in the format of the digests presented in this report. The digest format ensures that only a minimum of important data is lost. The selection of 40 buildings gave a large enough number so that certain patterns would show up, allowing the identification of the most important characteristics that must be recognized by the classification schemes. For a larger number of buildings, however, the digest format is cumbersome. High-rise systems are complex, and usually each building is unique in its combination of basic systems. Classification therefore always requires some judgment from the persons performing the classification. The digests allow a research team to come to a consensus about the classification of these buildings without each one having to read the source literature.

Many of the recommendations resulting from these damage reports have already found their way into codes and resulted in more stringent requirements regarding detailing and quality control, particularly for reinforced concrete buildings [ATC, 1978, ACI, 1983]. Designers are usually reluctant to accept code changes unless there is clear evidence of damage. The digests allow designers to familiarize themselves in a reasonable amount of time with the earthquake damage patterns that have prompted these code changes. In this sense, the digests will also have some value in their own right.

Finally, the data from the digests and 44 additional buildings were used for a preliminary study. Before proceeding to the finalization of the classification schemes and a more extensive data collection and analysis, a preliminary assessment of data was made in order to gain experience with the methods that can be used in an empirical investigation of earthquake resistance of tall buildings.

1.4 ORGANIZATION

The digests are organized into chapters according to earthquakes in chronological order. Each chapter starts with a brief presentation of general information about the earthquake, such as location of epicenter, magnitude, epicentral Modified Mercalli Intensity (MMI), and area affected. Within each chapter, buildings are arranged in alphabetical order. The main criterion in selecting those buildings was the availability of reasonably detailed data.

In each digest the data are loosely organized into five sections under the headings: Ground Motion & Site, Structural System, Design $ Construction, Damage, and Cause. The section "Ground Motion & Site" contains such data as location of the site, distance from epicenter, measures for ground motion intensity (accelerograms, assigned MMI value), damage in the neighborhood, soil conditions.

"Structural System" describes general building layout and configuration, structural systems including foundation, and architectural (nonstructural) systems (which act, in spite of the name, often as structural systems). If available. building periods are also given. "Design & Construction" gives information about construction date and cost, pertinent code, design assumptions. material quality, design and construction details, quality of construction, quality control, and supervision.

The section "Damage" contains, in addition to the damage description, repair cost where available. The section "Cause" offers comments or explanations as to the seismic behavior of the building and the cause of failures (according to the judgment of the author of the case study, or, in a few instances, of the writers of the digests). Typical examples are torsional response of the building due to eccentric arrangement of cores and shear walls, soft story effect, discontinuous shear walls, interference of nonstructural elements, inadequate shear reinforcement to develop the flexural capacity of the member, discontinued column ties, etc.

Most digests are accompanied by a very simple sketch of a typical plan and elevation. Simplicity was the objective. All details and repetitious patterns are left out. They should help in understanding general building layout and configuration, and they are primarily intended to help in deciding on the regularity/irregularity of the building. Thus, locations of shear walls, cores, openings in floor diaphragms and shear walls are indicated. But the location of columns and frame lines are often omitted and indicated only by the bay dimensions.

Even though these buildings are relatively well documented, the data are often far from complete. The explanations given regarding design assumptions and causes for observed behavior are often not fully satisfactory or are incomplete. However, the writers of the digests tried to remain as close to the original report as possible.

The last two chapters, finally, present the classification schemes as they evolved so far, and a preliminary assessment of the available data. Material (steel vs. concrete), tallness (medium- vs. high-rise), configuration (regularity vs. irregularity), and structural system (pure moment-resisting frames vs. other) are studied to investigate their importance regarding seismic vulnerability of tall buildings subjected to earthquakes.

2. THE ALASKA, ANCHORAGE, EARTHQUAKE OF 1964. (64-3)[1]

The great Alaska earthquake occurred at 5:36 p.m., March 27 1964. The Richter magnitude of the earthquake was 8.4; the focus was about 12.5 miles below the surface and the generated fault progressed in a S-W direction.

The main shock epicenter was located about 80 miles east of the city of Anchorage. There are no recordings of the ground motion but the maximum ground acceleration is estimated at 0.16g. This motion was reported to have lasted about 1 1/2 to 4 minutes. The observed damage was greater to multi-story than to one- and two-story buildings. The earthquake intensity in Anchorage was estimated at VIII on the Modified Mercalli intensity scale.

The soil profile in the Anchorage area consists, at the surface, of relatively dense sandy gravel at a depth range of 20'-70'. Under this gravel is a layered light gray silty clay containing lenses and layered silt, sand and sandy gravel to depths of 200'-300'. Landslides induced by the earthquake demolished some buildings.

REFERENCES: [Ayres, 1967], [Berg, 1964], [Blume66, 1966], [Committee, 1973], [Hansen, 1965], [Whitman, 1972], [Wood, 1967].

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[1]Numbers in parenthesis identify the event number consisting of the year and a sequence number

2.1 ANCHORAGE WESTWARD HOTEL

[Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The site of the Anchorage Westward hotel is located at the W-end of the 4th Avenue landslide, in Anchorage, and was classified as being partly in a 'zone of major adjustment'. The soil at the site was sand and gravel, and design soil pressure was limited to 3.5 ksi. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The Anchorage Westward Hotel consists of a 14-story tower and two adjacent structures: a 3-story ballroom and a 6-story hotel. The tower measures about 53' x 139' in plan and about 133' in height. The 14-story tower is constructed as a steel frame with shear walls and cores, the lateral resistance being provided by the shear walls.

There are two major shear walls in the E-W direction. One is the exterior S-wall and the other is an interior wall one bay removed from the N-end of the building. The interior shear wall has 4 doorway openings in each story. In the N-S direction there are 2 principal shear walls. An exterior wall runs along the three N-bays of the W-face; it has an opening, either a window or a door, in each story. The other is an interior wall, one-bay removed from the E-face and extending over the northernmost bay of the building; it has no openings. Reinforced concrete walls forming irregularly distributed cores house the stairs and the elevators. Stirrups are used in the lintels over the door openings in the lower 8 stories only.

Shear walls and core walls contain reinforced concrete structural columns built around light steel erection columns. usually 6" wide-flange shapes. There are no shear connectors between the concrete columns and steel sections. The columns not located in shear walls are of structural steel. The beam-to-column connections in the top 6 stories are partial moment-resisting. At the top of the 8th story, just below the 6-story addition, the columns are capped with

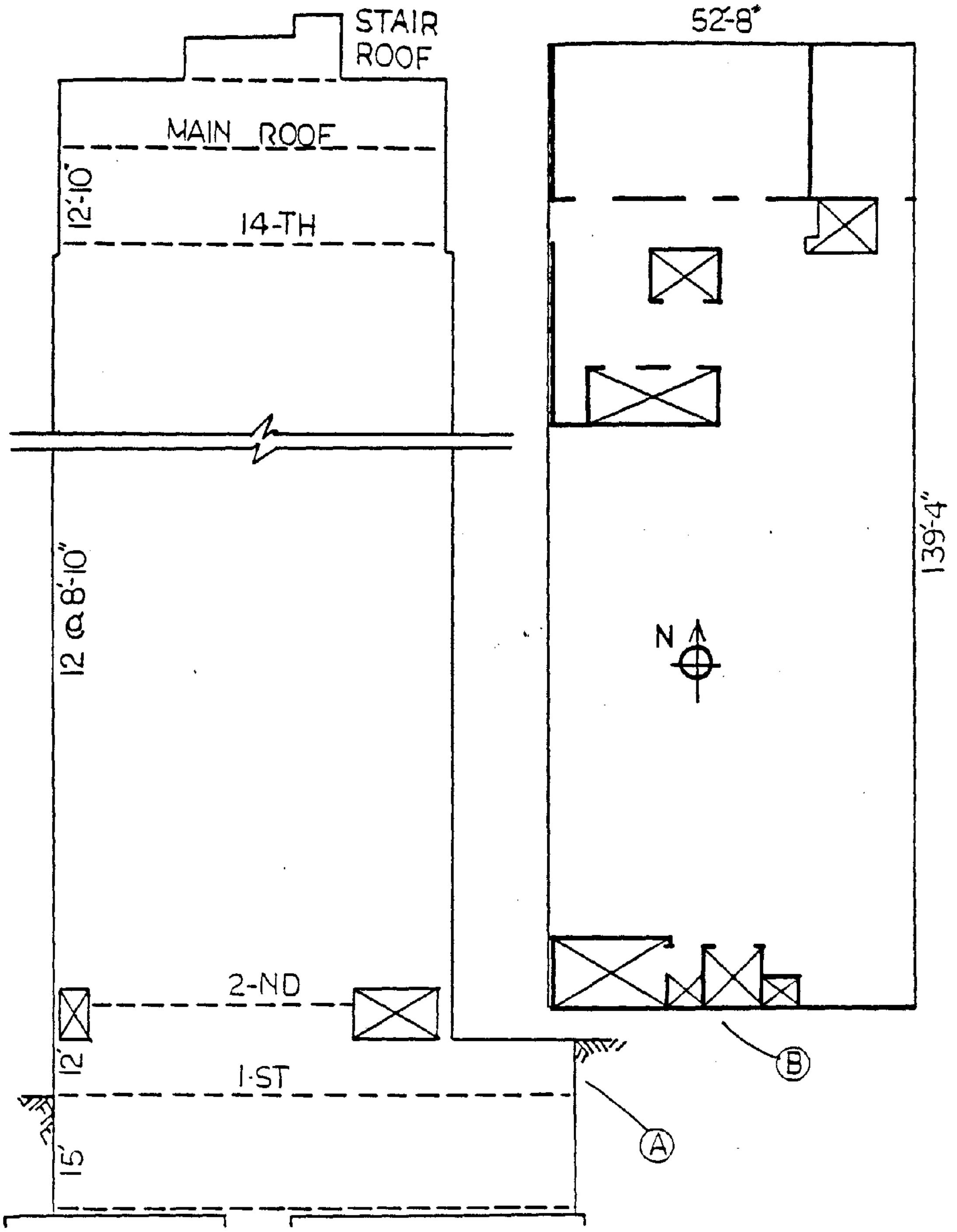

Figure 2-1: Anchorage Westward Hotel:
(A) Elevation of exterior S-wall.
(B) Typical plan.

a 1/4" steel plate welded to the column and beam above. The floors are 6 1/2"-thick one-way reinforced concrete slabs on corrugated steel forms. Footings are of reinforced concrete spread type.

Curtain walls of insulated metal mullions and glass windows form the facade. They are supported on brackets at each floor and tied to concrete inserts in the slab. The facade on the top of the tower consists of large. sloping windows with the same type of support. Interior partitions are of metal lath and plaster. Fireproofing is 3/4"-thick gypsum plaster on metal lath.

DESIGN & CONSTRUCTION

The basement and first 8 tower stories were constructed in 1959 according to the UBC zone 3 requirements; the upper 6 stories were just completed at the time of the earthquake(1964).

Vertical loads were carried by steel columns and reinforced concrete shear walls. Steel columns located in the reinforced concrete walls were erection columns, not intended to take the full vertical loads. Column-to-beam connections, in general, were not moment-resisting, since the frame was not intended to take lateral forces. The lateral force bracing system was in the form of reinforced concrete shear walls.

Concrete quality was 2.5ksi at 28 days. Reinforcing steel was of working stress 20ksi.

DAMAGE

The damage to this building has been estimated at 12% of its replacement value.

Substantial damage occurred in the exterior W-wall at the top of the 8th story. There was also significant damage in the interior E-W shear wall at the 2nd floor level. Its E-end concrete column fractured and the steel section buckled. There were no shear connectors between the steel section and the concrete, so that the contact surface was of zero shear strength in the reinforced concrete. The W-end failures were even more severe: the steel bars also buckled and the ties broke. The reinforcing bars were spliced at this level. Above this location, at the 8th story, the connection plate fractured; an examination of the walls near the S-end of the building found slight movement along several joints. In several instances laitance had not been removed from the joint. Additional damage occurred to an outside column in the 1st story, similar to the other failures and at a location where the bars were spliced. Failures occurred in the lintels above all door openings in every story in the interior E-W wall. Stirrups were used in the lintels in the lower 8 stories but not in the upper 6 stories, which experienced the worst damage.

Pounding damage occurred between the 14-story building and the ballroom building. Additional pounding damage occurred between the 6- and 14- story buildings, although they were separated by a 4" structural gap.

The facades and glazing survived the earthquake with only minor damage. A curtain wall mullion came apart from the slip joint next to the structural failure at the 8th story, but it did not fall. Typical hammering damage occurred where the tower adjoins the 3-story ballroom. The facade on the top of the tower, being subjected to heavy racking, cracked the concrete slab around the supporting inserts. The metal lath and plaster partitions were damaged, and plaster covering a damaged concrete shear wall was broken loose.

CAUSE

Since damage was more pronounced in the 9th story, which was the roof of the original building at this point, it seems reasonable to attribute this damage, in part, to construction problems at the juncture between the addition and the original building. Failures were also concentrated at points of bar splicing. The damage to the lintels over doorways was due to vertical shear caused by seismic overturning forces. Overturning stress in the shear wall, when viewed as a vertical cantilever, must be transferred from the tensional component at one end to the compressional component at the other end of the wall by means of shear through the lintels above doors. The high shearing stresses in the lintels resulted in damage vertically aligned between floors.

Some recommendations are:

- Construction joints between different stories should be designed to withstand seismic forces.

- Openings in structural members should be reinforced to become earthquake resistant.

- Avoid bar splicing at points of maximum shear.

- Provide seismic gaps between adjoining structures.

- Nonstructural elements should be isolated from seismic movements.

2.2 THE CORDOVA BUILDING

[Committee. 1973]. [Wood 1967]. [Berg. 1964].

GROUND MOTION & SITE

The Cordova building is located in the city of Anchorage. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale. No pertinent information on local site and seismic conditions was found.

STRUCTURAL SYSTEM

The Cordova building is a 6-story office building, 53'-8" x 129'-4" in plan and 66' high. oriented with its long dimension in the N-S direction and facing westward. This steel building has a full moment-resisting frame in the narrow direction, which is the strong direction of the columns. In the long direction of the building partial moment-resisting beam to column connections were used. The building has a reinforced concrete service core enclosing the stairwell and elevator shaft. The floors are 2 1/2"-thick concrete slabs on corrugated-steel forms. supported by open-web joists.

The peripheral columns are seated just below the ground floor girders on 2'-6" square piers in the line of the basement wall. Square spread footings supporting these piers are founded from 3' to 6' below the basement floor. Interior columns are seated near the basement floor level on stub piers with similar footings.

The N-face of the building is a 4"-thick concrete curtain wall. Similar curtain walls also run around the S-E corner of the building. Curtain walls are supported on the basement wall. There was evidence after the earthquake that the S-E curtain wall had not been anchored to the floor system at the 2nd and 3rd story levels. All the W-face, nearly all the E-face. and half the S-face are covered with lightweight insulated metal panels. Wood-stud partitions faced with drywall consist of panels about 3' wide, fitted into guides along the floor, ceiling, and end walls and held in position by spacerclips. All fireproofing is plaster.

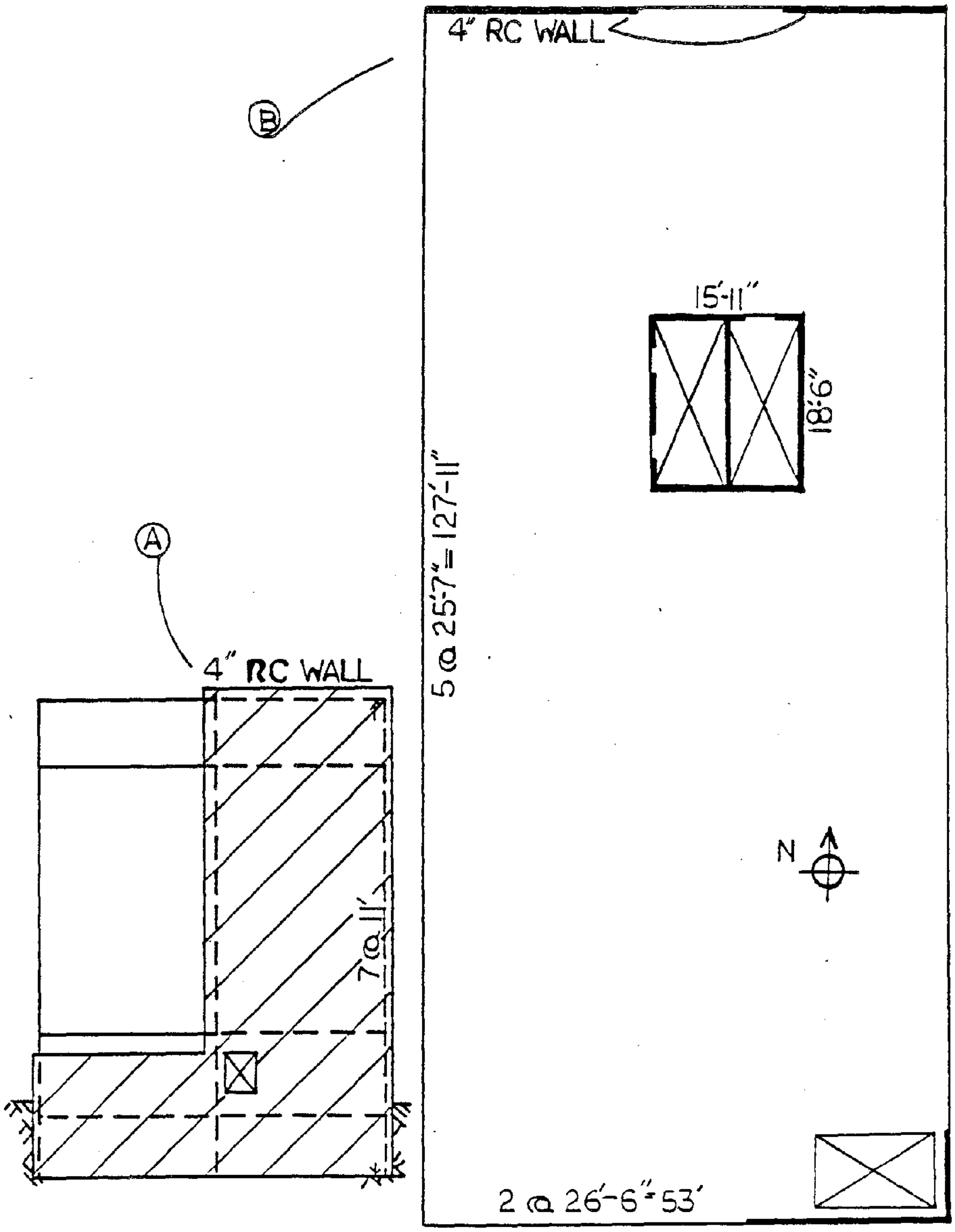

Figure 2-2: Cordova Building:
(A) S-elevation.
(B) Typical plan.

DESIGN & CONSTRUCTION

The Cordova building was designed in 1956 according to the UBC for zone 2 requirements. The original design intended the steel frames to resist the earthquake forces, while the core was supposed to contribute nothing to the lateral resistance.

All steel columns and girders are wide-flange sections. The connections are shop welded and field bolted with high-strength bolts. Core walls are 8" thick reinforced vertically with #9 bars at 12" in the N- and S-walls. #7 bars in the E- and W-walls and #4 bars in the interior wall. The horizontal steel in all walls consists of #4 bars at 12". The 28-day design strength of the floor slabs and walls was 2.5ksi while the rest of the reinforced concrete had a 28-day design strength of 3ksi.

Errors in construction are not believed to be a cause of the observed damage. However, sawdust was found in the construction joint in the S-E corner wall after the earthquake. These 4"-thick exterior reinforced concrete walls were defined as "curtain walls" on the drawings. Wire fabric was specified as wall reinforcement, but reinforcing bars were actually used.

DAMAGE

The damage to the Cordova building cost about $0.2 million to repair, nearly 1/5 of the total building cost.

Most of the damage occurred in the 1st story. The core sheared at the base of the 1st story. Failure of the core at this level was complete along the N-wall and at the N-W corner. The S-E corner column buckled severely below the 2nd floor beam and shortened about 1 1/2". The midstory stair landing was anchored to this column making it much stiffer than the rest of the columns at the 1st story. The center and west columns at the S-face buckled locally at the top and bottom of the 1st story. The penthouse collapsed.

The metal curtain wall facade was almost undamaged. Some of the aluminum mullions failed on the 1st floor next to the damaged 4" concrete curtain wall and allowed the metal curtain wall to sag. The S-E concrete curtain walls sheared at the top of the basement walls, and the corner broke open in the 1st and 2nd stories, where the walls apparently were not anchored to the framing of the floor system. Also the N-curtain wall shifted on the construction joint atop of the basement wall.

CAUSE

The interior reinforced concrete core appears to have initially resisted the major portion of the seismic forces. The steel frames were able to withstand the ground motion after failure of the core. The steel frame at the S-E corner wall sustained more damage because of the stiffening effect of the stairwell and because of its greater distance from the center of rigidity. The rigid S-E concrete curtain wall also resisted lateral forces until it failed, and then the S-E corner column buckled.

Some recommendations are:

- Stairways should be designed to avoid unfavorable interaction with the structural system.
- Seismic gaps should separate the structural frame from the nonstructural filler walls.
- Curtain walls should be securely attached to the building frame.
- Heavy rigid facades should be used only in rigid structural systems and not in relatively flexible ones.
- Glass panels and their mullions behave excellently, if they are isolated from earthquake motion.

2.3 ELMENDORF HOSPITAL

[Committee. 1973]. [Wood, 1967], [Berg. 1964].

GROUND MOTION & SITE

The Elmendorf Air Force Hospital is located about 5 miles west-northwest of downtown Anchorage. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale. No pertinent information on local site conditions was found. The maximum allowable bearing pressure for the soil of the site was specified at 9ksf.

STRUCTURAL SYSTEM

The Elmendorf Hospital is a reinforced concrete structure consisting of 3 wings. An 8" seismic joint separates the wings along their intersection. Wing A is a 7-story and basement structure. about 40' x 366' in plan. with a stairwell shaft at each end. Additionally, a central core (about 43' x 108' in plan) which houses the elevators and adjoining stairs, rises the equivalent of four more stories. Wing B is a 3-story and wing C a 2-story concrete structure of construction similar to that of wing A. The 7-story section of wing A was designed for four additional stories. A full basement is beneath all buildings. The following discussion will be devoted to wing A.

The structural system is a reinforced concrete frame with shear walls and a central core. The N-S component of lateral forces is to be resisted by shear walls around the stairs located at the W-end, E-end. and central core. The E-W component of lateral forces is to be resisted by the shear walls around the central core. There are two more E-W shear walls between the central core and wing A in the basement and lower 3 stories; these walls stop at the 4th floor and are replaced by two perpendicular (N-S), and discontinued shear walls in the central core unit that extend from 4th to 11th floor. The reinforced concrete floor system consists of one-way or two-way slabs on beams. Slab thickness is usually 6"; interior beams generally are 42" wide and 14" deep. Foundations are of the reinforced concrete spread type. The structural separations in the superstructure do not go through the footings.

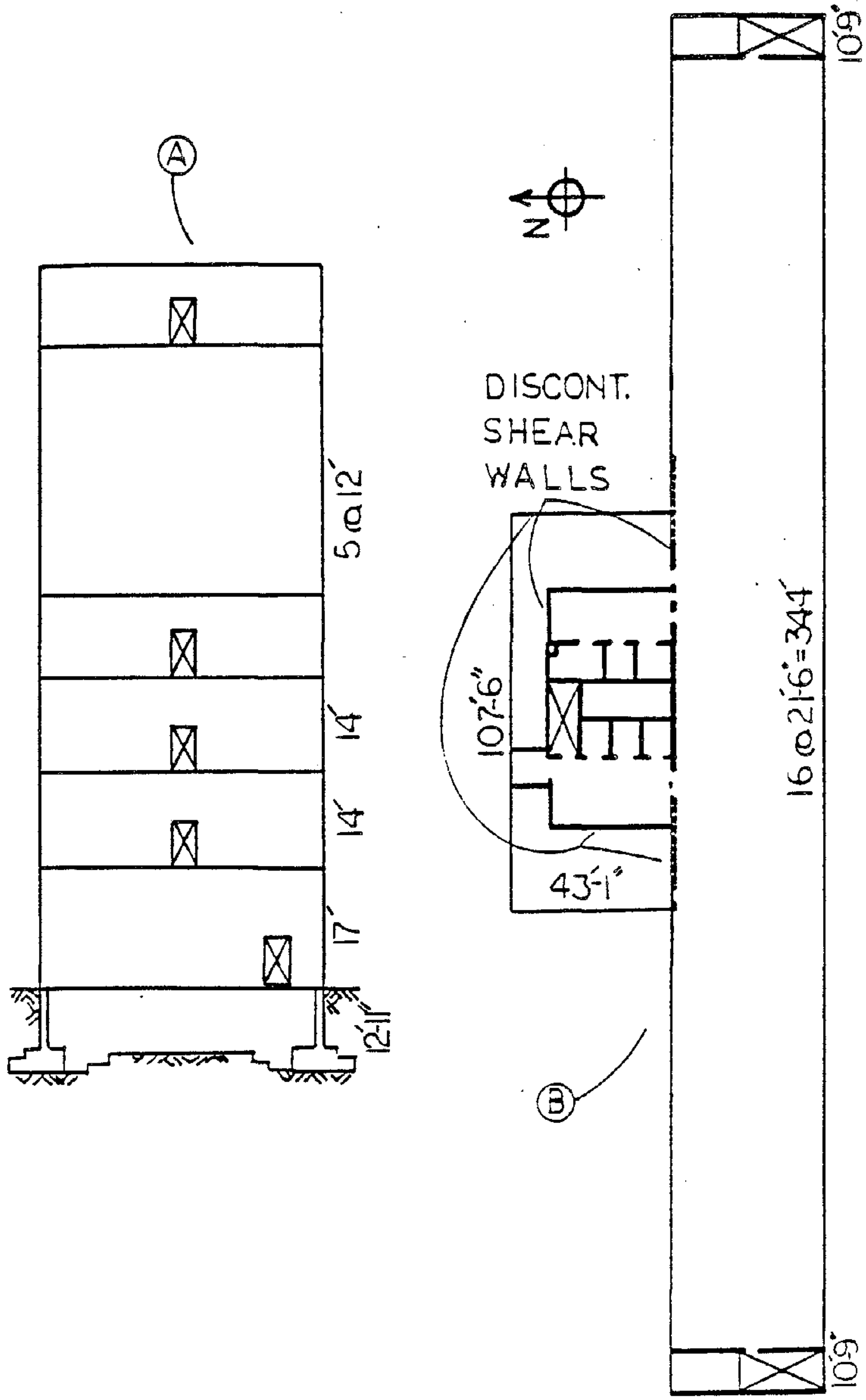

Figure 2-3: Elmendorf Hospital:
(A) Elevation of interior S-wall.
(B) Fourth floor plan of wing A and 11-story tower.

The facades consist of 8" reinforced hollow concrete block filler walls These walls are anchored to the floor with #3 bars and reinforced with trim bars around openings. Partitions are plaster walls; the floors and walls of the operating rooms are covered by tile. Marble veneer supported by wall inserts is used in the main entry.

DESIGN & CONSTRUCTION

The hospital was designed in 1952 and built in 1954-55 at a cost of $8.9 million. The seismic design followed the 1949 UBC requirements. and the concrete design and details were in accordance with the 1951 ACI code.

The 11-story core together with the shear walls of the stairwell shafts at the ends of wing A were designed to provide the main lateral force resistance. Forces were to be transmitted to these elements by the floors acting as diaphragms; some resistance was to be furnished by the columns in the exterior walls.

At each floor there are two large openings in the shear wall between wing A and central core; a deep beam above each opening was designed to make the wall act as a unit (coupled shear wall) during the earthquake. These coupling beams proved to have been poured in two parts with a longitudinal construction joint at 1/3 to 1/2 the height of the beam; they were also reduced in size because of duct openings. An 5'-4" x 6' opening not shown in the drawings was left in the elevator well of the central core between 4th and 5th floor. On a later investigation some large pockets of bad concrete were found in the same wall. Many joints proved not to have been completely cleaned and sawdust, dirt, and so forth was left during construction.

Concrete quality was specified to be 2.5ksi at 28 days, in general, and 3.75ksi at 28 days for columns.

DAMAGE

The total cost of repairs was $2.4 million of which about $2 million was for nonstructural repair; the high repair costs are due to the need for quick repair of this important building.

All shear walls showed diagonal hairline cracking. The coupling beams of the shear wall between wing A and central core failed on floors 1 to 5, large chunks of concrete having spalled. The spandrel beams connecting the outer frame to the central core were severely damaged and the damage increased with height. At the rear of the central core in the 1st story a column was shattered. There was evidence of movement between the concrete wall of the core and the floor slab. The re-entrant corners between the discontinued shear walls at the 4-th floor were severely damaged.

The interaction between the frame of the building and the filler walls produced local cracking at the beam column joints of the frame. Block filler walls in the 1st to 5th story showed large X-cracks between windows; about 70% of them had to be replaced. The plaster partitions were all badly cracked. and in some cases entire sections were loosened. The wall inserts holding the marble veneer were loosened, and some of the slabs broke. The mechanical system was severely damaged. Steam, water, and sewer lines were damaged. There was little damage to the electrical system.

CAUSE

Poor construction practice and poor workmanship was the main cause of failure of the coupling beams of the coupled shear wall. The intersection of the E-W lower shear wall with the N-S upper discontinued shear walls was damaged from the concentrated overturning forces to be transfered at that point. Column ties and possibly vertical bars were fewer than those called for in drawings.

The diagonal cracks in the filler walls indicate that the design did not prevent the transfer of the seismic motion of the structural frame to the filler walls; they resisted lateral forces and failed at their weakest points, i.e. between windows. The cracking of the frame of the building is an illustration of how a structural system can be damaged during an earthquake by not properly isolated nonstructural elements. The interstory displacement cracked the plaster partitions. Very little glass was damaged in the hospital, although the facades were cracked at the windows, because the double-hung steel-sash windows had enough space between their frames to absorb movement.

The hospital's inability to function after the earthquake emphasizes the need to give special attention to earthquake resistant design of important buildings that are urgently needed after an earthquake.

Some recommendations are:

- Seismic gaps should separate the structural frame from the nonstructural filler walls.
- Special consideration should be given to the design of coupling beams between shear walls.
- The use of noncontinuous shear walls needs careful attention to detail, both in design and construction.

2.4 THE FOUR SEASONS APARTMENT BUILDING

[Reuter, 1965], [Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The Four Seasons apartment building was located about 100' to the south of the graben that formed at the head of the L Street slide in Anchorage. Small surface cracks in a general transverse direction were observed at the building's site. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The 3 bay x 6 bay structural system of the 52' high, 6-story building was composed of 8"-thick post-tensioned concrete flat slabs supported on 10" wide-flange steel columns. Resistance to lateral loads was provided by the elevator and stairwell cores. The dimensions of a typical floor were 75'-8" x 130'-8". The central part of the lobby slab was depressed 8' below the 1st story level. A basement extended under part of the building. and a penthouse covered the roof between the two cores. The structure rested on spread footings. The 34' x 34' footing under the N-core was founded 12'-8" under the basement floor slab while the footing under the S-core was founded at an elevation 5' higher outside the basement area.

DESIGN & CONSTRUCTION

The construction of the Four Seasons apartment building had begun in the summer of 1963. At the time of the earthquake, the building was structurally complete, and work was rapidly proceeding on the interior finish. The building was designed for UBC zone 3 requirements. The design and construction appeared to be in agreement with requirements.

This was the first building in Anchorage in which lift-slab construction was used. The flat slabs were cast in two sections, post-tensioned on the ground, and then jacked vertically into position. The two cores and the strip

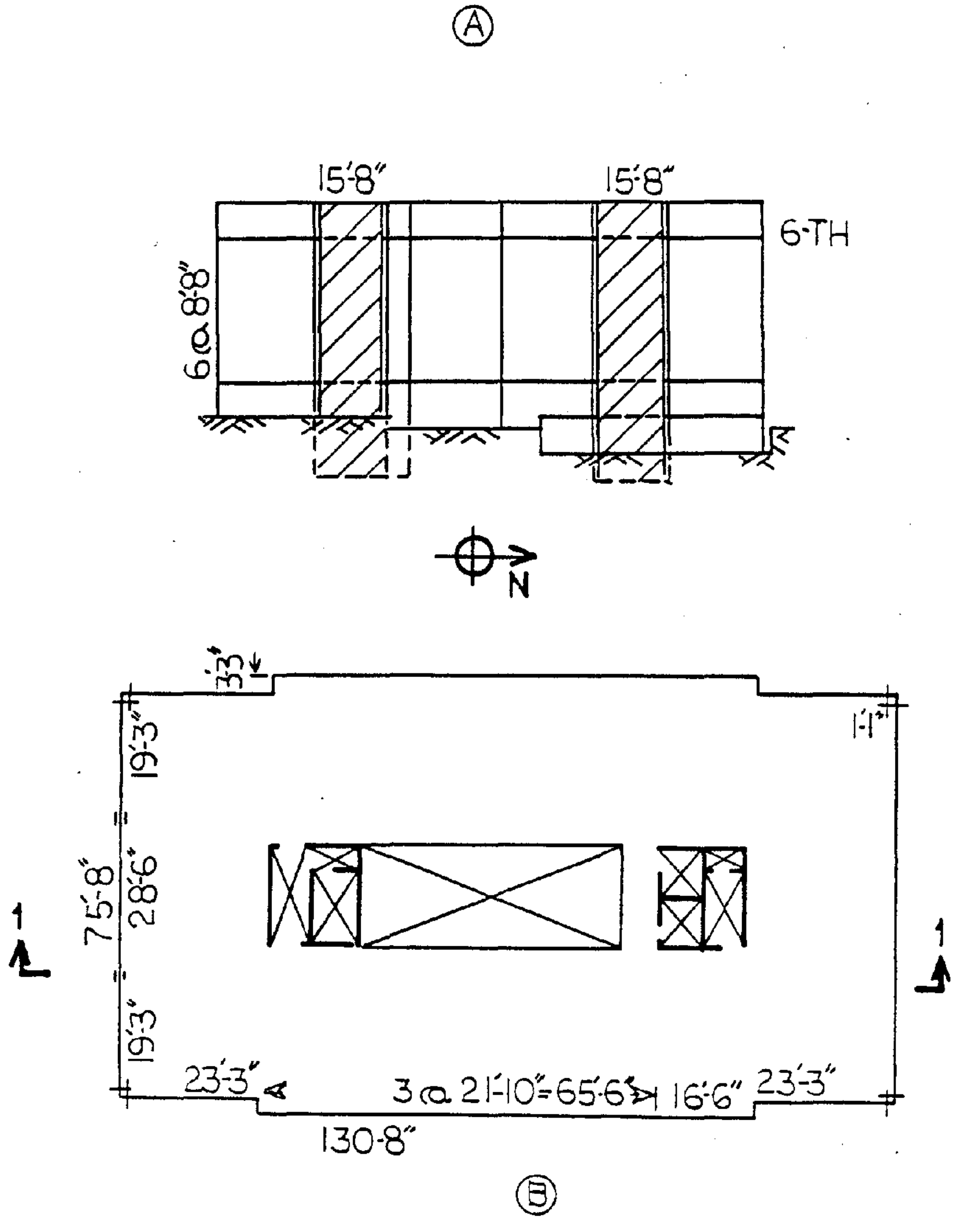

Figure 2-4: Four Seasons Apartment Building:
(A) Elevation 1-1.
(B) Typical plan.

connecting the two sections of the floor were poured after the slabs had been jacked into position. After this, steel shear heads were embedded in the slabs and welded to the steel columns. The slabs were keyed and doweled into the poured-in-place concrete core. The prestressing cables were 1/2" greased and wrapped tendons with wedge type anchorages.

DAMAGE

This structure collapsed completely, but fortunately there were no tenants in the building and no workers on the site at the time of failure.

The two cores were severed near their bases and overturned to the north, carrying the slabs with them. All vertical reinforcing bars in the cores were spliced near the core bases with a 20 - bar diameter lap. The splices of the #8 and #11 bars failed, while the #4 bars ruptured in a ductile manner.

Due to the rocking of the cores the adjacent slab areas experienced severe angular displacements and shattered completely so that the slabs could drop down. During the descent of the slabs, severe distortion at the column connection caused the shear-head connection to punch through the slab. A spectacular secondary effect was the release of many prestressing cables during the collapse.

CAUSE

The collapse of the Four Seasons apartment building has been attributed to inadequate length of lapping of the reinforcement bars at the base of the two cores. Bond failure effectively disconnected the cores from their foundations and left them free to rock and break the connections of the floor slabs to the cores. The code requirements on overlapping of bars were not adequate to prevent failure.

The analysis of the Four Seasons apartment building clearly indicates that relying solely on slender vertical concrete shafts for earthquake resistance requires more thorough engineering analysis and more conservative design than were employed in that case. The UBC requirements should be revised, so that in the future buildings of similar design will be able to withstand stronger earthquake movements without collapsing.

2.5 THE HILL BUILDING

[Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The Hill building is about 100' from the 4th Avenue and 1200' from the L Street landslide in Anchorage. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale. Detailed soil data are lacking for the site. However, design soil pressure was 6ksf.

STRUCTURAL SYSTEM

The Hill building is an 8-story office building, 100' x 180' in plan and about 114'-6" high. with a structurally separated 1-story garage and covered loading dock.

The structural system of the building combines simple steel framing with two reinforced concrete cores enclosing stairwells, elevator and utility shaft. These cores are connected by reinforced concrete beams at each floor. The framing system consists of steel beams and girders supported on the central cores, on 4 interior steel columns, and on 20 steel columns along the building perimeter. The floors are one-way reinforced concrete slabs, 5 1/2" thick. Footings are of the reinforced concrete spread type.

Plaster fireproofing was used throughout on the structural steel. Exterior faces of the building are 3"-thick insulated porcelain enamel panels, except for the penthouse, where hollow concrete blocks are used. Interior partitions consist of 8" reinforced hollow concrete blocks in the 1st story and movable metal partitions in the other stories. All light partitions and cladding are essentially isolated from the framing. Reinforced concrete columns support a canopy roof at the 1st story.

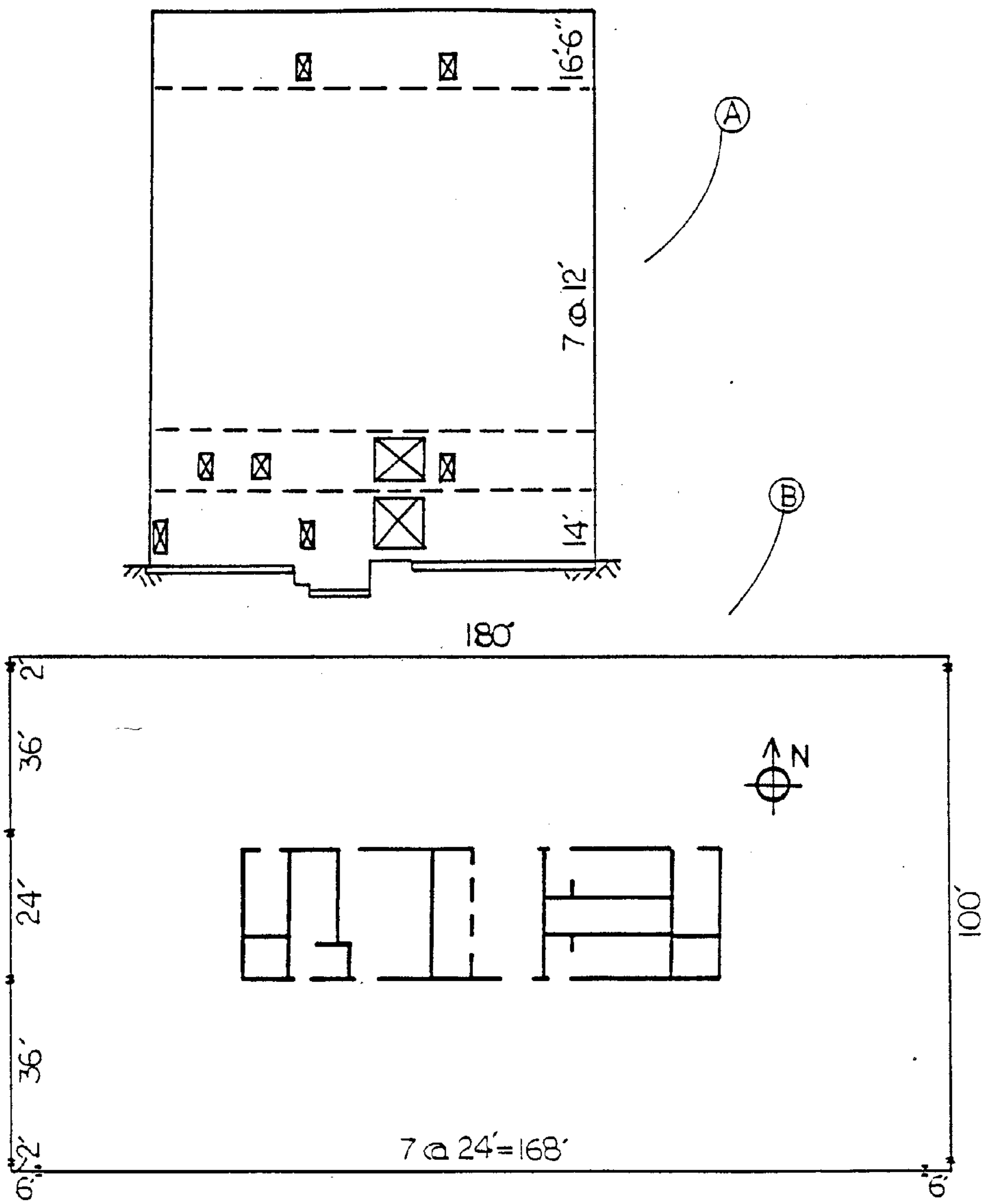

Figure 2-5: Hill Building:
(A) S-elevation of central core.
(B) Typical upper story plan.

DESIGN & CONSTRUCTION

The building was designed in 1966 according to UBC zone 3 requirements. The frame was designed to resist only vertical loads. while the lateral loads were assigned to the cores. All core walls had only one curtain of steel. Beam-to-column connections in the steel frame were simple connections utilizing either ordinary or high strength bolts.

All concrete had a 28-day design strength of 2.5ksi. Reinforcing steel was of design strength of 20ksi. The frame was field checked by a testing laboratory during construction. A laboratory analysis of concrete specimens after the earthquake indicated the presence of organic material resulting in extremely low strength concrete at the 1st story points, where the shear walls failed.

DAMAGE

It is estimated that the total repair costs amounted to 20%-25% of the replacement value of the building.

The central cores dropped about 5" at one corner and 3" at another. reportedly as the concrete corewalls had pulverized just above footing level because of defective concrete. In addition the core walls were cracked, particularly in the lower stories, and the beams interconnecting them were damaged at all floors. A reinforced concrete column supporting the canopy roof failed at its connection. Slippage was observed along construction joints in the core walls. There was no damage to the steel frame. A number of bolts at the beam-to-column connections had sheared.

Exterior damage to the Hill building was slight. The lightweight curtain walls were undamaged. Some of the concrete blocks fell out of the S-wall of the penthouse, which was not reinforced. and some tiles were damaged at the front entrance. Damage elsewhere in the building was confined to the failure of hollow concrete block walls in the 1st story.

CAUSE

Lateral force resistance was to be provided by the central cores, the most rigid components, which also experienced the most severe structural damage. The failure of the core above footing level was due to a horizontal belt of poor quality concrete discovered during reconstruction. The settling of the core probably caused the failure of the unreinforced concrete block wall of the penthouse. The redistribution of stresses due to the "mushing" of concrete increased the damage in the 1st and lower stories. The shear failure of the floors adjacent to core walls was due to the high shear stresses developed in the transfer of vertical coupling forces between the two cores.

The lack of damage to the frame and exterior curtain walls is not surprising, because the frame members and their connections were relatively flexible and the curtain walls were seismically isolated. The damage to the hollow concrete blocks in the 1st floor started, once the cores fractured and all other rigid elements were subjected to shattering.

Some recommendations are:

- Improve inspection procedures.
- Rigid nonstructural elements should be isolated from seismic motion.
- Provide keys and possibly diagonal reinforcement at construction joints.

2.6 HILLSIDE APARTMENT BUILDING

[Committee, 1973]. [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The Hillside apartment building in Anchorage was constructed on a steep natural slope that dropped off toward the south at a grade of about 5' in 10'. The subsoil in the vicinity is gravely sand to a depth of approximately 39'. No significant landslides or other ground movement features were discovered after the earthquake. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

This apartment building measured 208' x 41' in plan, its long dimensions running E-W and parallel to the bluff line. Due to its location on a sloping site the front (N) elevation had 3 stories while rear (S) elevation had 5 stories.

The structural system consisted of a steel frame with stairwell cores. Steel pipe columns were used along exterior column lines and structural steel H columns on the center line. No plans for the building were available. It is not clear from the case-study whether the lateral resistance was provided by the steel frame or the stairway cores. The steel-pipe columns on the N-side of the building ranged in size from 3" diameter at the top floor to 6" diameter at street level. The interior columns along the center line of the building were 6" H columns from top floor to basement level. The spandrel beams were 10"-deep structural steel I beams. The connections of beams to columns were not moment-resisting. The building had three stairway systems. The stairs were of steel-pan construction with nonskid cement-filled threads. The wall stringers were structural steel channels, and the stairwell stringers were continuous boxes made of welded steel plates.

The floor and roof slabs were supported by 9"-deep bar joists attached to the main framing beams. The basement level had 4"-thick reinforced concrete slabs placed on grade. All other slabs were 2 1/2"-thick pumicrete. The floors were split level, the floors of the S-half being at a different level than the N-half. Floor diaphragms, therefore, were cut in half. The foundation walls of the building were of reinforced concrete with spread footings placed on firm soil.

The exterior of all spandrel beams was encased in Pumicrete. Copings and canopies were constructed from the same material. The exterior facades between the encased spandrel beams and the window sills consisted of 9"-thick cavity type pumice-concrete block walls. The block walls were nonbearing and carried by the building frame; they were neither reinforced, nor were they tied to the frame. All interior partitions were unreinforced pumicrete block units. The partitions within apartments were 4" thick. The partitions running N-S between units and around stairways were 8" thick. The partitions running E-W between the N- and S-apartments were made up of two 4" units separated by an 8" pipe space. All rooms had plaster walls and ceilings.

DESIGN & CONSTRUCTION

This building was constructed in 1951. The design and construction, however, did not comply with UBC requirements; indeed it appeared to be very weak in this respect. It appears that when this structure was built, it was outside of the city limits of Anchorage, and therefore the City's building code did not apply. In any event, the structure lacked all commonly accepted forms of earthquake resistance.

The structure did have steel pipe columns and steel beams, but may not have had a complete steel frame. Connections were, for all practical purposes, "pinned". The frame was light, and member sizes were not sufficient for lateral seismic forces.

DAMAGE

Damage to the Hillside apartment building was so extensive that it was neither safe nor fit for occupancy and in condition hazardous to the public. Rehabiliation was not economically feasible, and the building was therefore completely demolished.

The building appeared to be out of plumb in the north and south direction. Some of the connections between the north and south spandrels and the center H columns had failed at the 2nd floor. The 2nd and 3rd story northwest corner columns collapsed, letting an entire bay of the 3rd floor and roof drop onto the 2nd floor slab. Except for one large crack in the 2nd floor running north and south just inside the entrance, the roof and floors appeared to be reasonably intact. The foundations were apparently in satisfactory condition.

A large percentage of the 9"-thick block walls had collapsed, especially on the 2nd and 1st floors near the northeast and northwest corners. All interior block partitions were badly shattered, and some had completely or partially collapsed. The plumbing and heating systems had suffered some damage; the electrical system was subjected to considerable damage.

CAUSE

The Hillside apartment building was not designed for seismic forces and resisted the earthquake motion through the rigid nonstructural block walls and the structural frame. Although extensively damaged, this old building, which did not comply with the seismic code, surprisingly survived the earthquake without collapsing. This is particularly striking when compared to the modern Four Seasons apartment building, which was designed according to UBC but collapsed completely.

Some recommendations are:

- Buildings should be designed according to seismic code requirements.

- Studies should be made to clarify why some improperly designed structures can withstand earthquakes without collapsing.

2.7 J.C.PENNEY BUILDING

[Committee, 1973], [Wood. 1967]. [Berg, 1964].

GROUND MOTION & SITE

The J.C.Penney building was located about 450' to the south of the 4th Avenue landslide in Anchorage; however, no ground fractures were found beneath or around the building. Detailed information on soil conditions at the site was lacking. Maximum design soil pressure was 6 ksf. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The building was a 5-story reinforced concrete structure measuring approximately 130' x 150' in plan, 6 bays wide in each direction, and 66' high. The long axis lied in an E-W direction. Story heights were 14' for the 1st and 13' for the other stories.

The structural system consisted of flat slabs on reinforced concrete columns and peripheral shear walls. The S- and W-face of the building were formed by cast-in-place shear walls that extended the full height of the structure and had openings only in the 1st story. The E-face contained full-height shear walls at each end, the center portion being structurally open. The N-face. finally, had shear walls in the 1st story only. The shear wall bracing system for lateral forces was thus reasonably symmetrical in the 1st story, but in the upper stories the N-face was structurally open. In general, the walls were 8" reinforced concrete, although some 10" and 11" walls existed. The reinforcing steel was in the center of the 8" walls. but was placed in two curtains for the thicker walls. The 1st floor was a 4" concrete slab-on-grade. The roof and remaining floor slabs were 10"-thick reinforced concrete slabs designed with drop panels or column capitals with shear heads of reinforcing steel at all interior columns. Maximum roof and floor panel size was 22' x 26', and columns were square 20" x 20". The foundations consisted of reinforced concrete spread footings for the interior columns and a continuous spread footing around the perimeter.

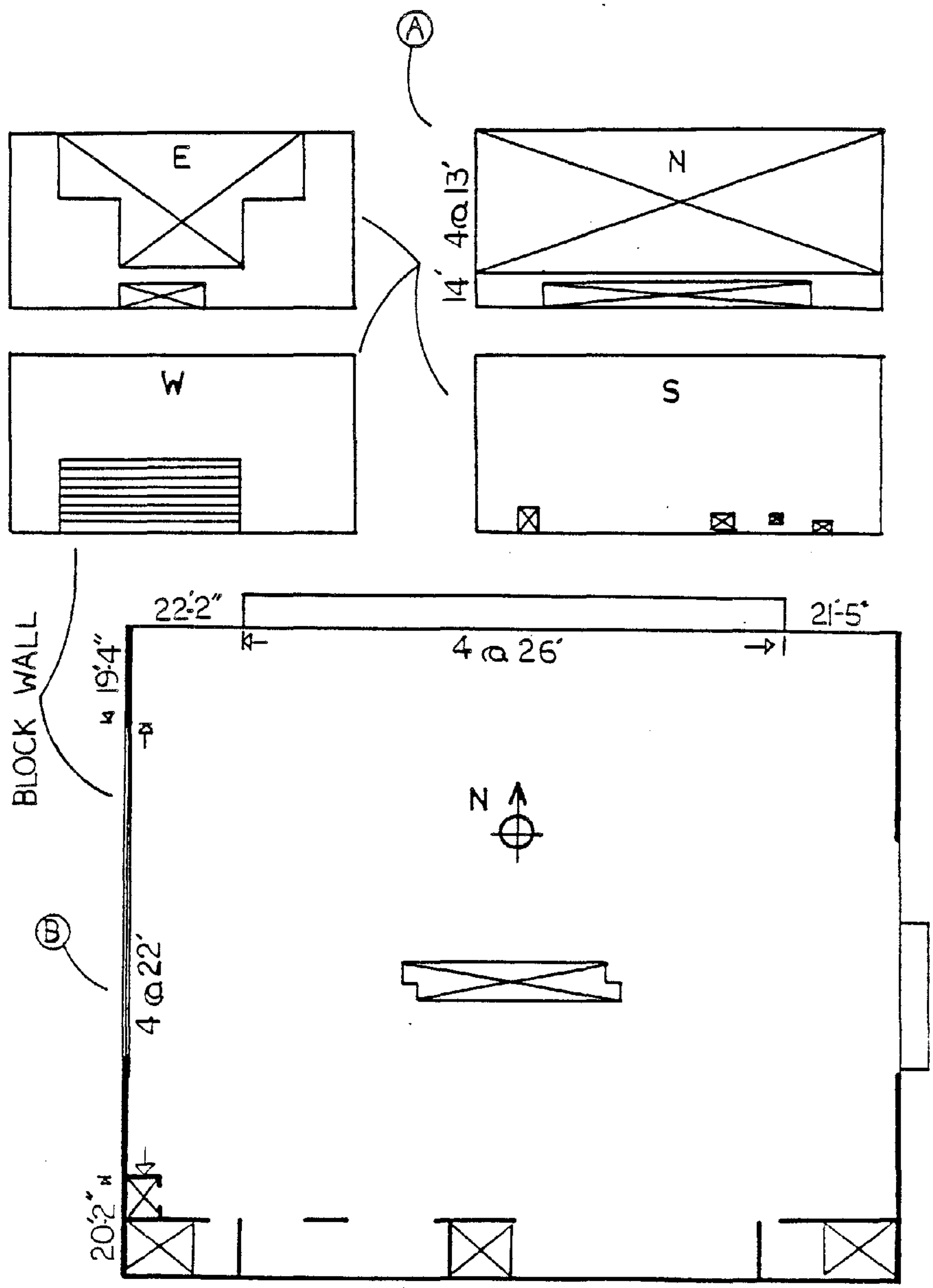

Figure 2-6: J.C. Penney Building:
(A) Elevation of peripheral shear walls.
(B) Typical upper story plan.

The N- and E- street fronts from the 2nd floor marquee level to the roof were faced with 5" precast concrete panels connected to the floor slabs. The precast panels were installed after the cast-in-place E-walls had been completed. Slots or holes were left in the shear walls to allow anchorage of the precast panels to the floor slabs. The holes were not grouted after the anchorage had been completed. It is obvious that the holes greatly reduced the shear resistance of these shear walls. On the W-elevation 8" and 10" hollow concrete block walls were found, in the first two stories at the three interior bays facing an existing building. No mention was made about partitions or other nonstructural elements.

DESIGN & CONSTRUCTION

The structure was built in 1963. At that time the city of Anchorage required that buildings be designed for zone 3 requirements of the UBC.

Apparently. lateral forces were resisted by the peripheral shear walls and columns and transfered by the floor slabs without spandrels. The precast panels were not considered to be significant lateral force resisting elements. Besides. the installation did not follow the drawings, because a note allowed the contractors to submit for approval an alternate detail.

Concrete quality was specified to be 2.5ksi for the footings, supported floors plus some columns. Certain columns had 3ksi design concrete.

DAMAGE

This building is of particular interest, because it was one of the relatively new structures that were damaged beyond repair. It was removed and replaced by a new structure after the earthquake.

The W-shear wall failed in shear at the north pier, and a portion fell several feet to the ground. The E-shear wall at the NE-corner failed. and this corner of the building collapsed. Wall movement along the 2nd floor construction joint followed a pattern commonly found in Anchorage. The laitance had not been effectively removed, there were pockets of uncompacted grout. and the concrete wall was not truly monolithic.

Most of the precast panels on the N- and E-faces of the building were also badly cracked and displaced. A woman in front of the store was killed, when one of these panels fell from the building.

CAUSE

The failure can be attributed to torsional response. In the undamaged 1st story torsional response was not significant, since shear walls were found along all street fronts. The upper stories. however. had a structurally open N-face, and large torsional response resulted from the U-shaped bracing system when subjected to an EW motion.

Many lessons on design and construction procedure may be learned from the behavior of this building.

- Highly unsymmetrical arrangements of shear walls that induce torsional oscillation, should be avoided if not accompanied by a thorough analysis and design.

- Elements connecting precast concrete panels to the building frame should be strong enough to withstand earthquake forces.

- Construction joints in concrete walls should be well made, so that they do not provide planes of weakness.

- Provide keys and possibly diagonal reinforcement at construction joints.

2.8 KNIK ARMS APARTMENT HOUSE

[Committee. 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The Knik Arms apartment building is located at the L Street landslide area in Anchorage. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale. Resurveys showed that the landmass in the L Street area slid horizontally on "dynamically sensitive saturated sands. and clayey silts", remaining largely intact, and that this movement had no significant vertical component. After the underlying soils at the site experienced liquefaction, the building was effectively isolated from the ground motion and moved horizontally about 10'. Therefore it possibly did not have as long a duration of violent shaking as did structures outside the landslide area. This period of reduced vibrational intensity may have been as long as 2 minutes or the 2nd half of the earthquake.

STRUCTURAL SYSTEM

The Knik Arms Apartment House is an approximately L-shaped structure. 74' x 123' in plan, and about 62' high. The 6-story apartment house is of cast-in-place reinforced concrete. The structural system of this building can be labeled as a box- or bearing wall- structure. Shear walls are found along the perimeter of the building and around the stair and elevator cores. These 6" to 6 3/4" thick walls are gravity load bearing and have reinforced concrete pilasters. Floors are 5 1/2" one-way slabs with clear spans of almost 17'. They are supported by exterior walls and beams spanning between interior cores and columns. The beams are essentially thickened floor sections. Foundations are reinforced concrete type with a design soil bearing pressure of 6ksf.

No mention is made about the interior partitions or any other nonstructural elements. There are glass windows in the exterior wall openings.

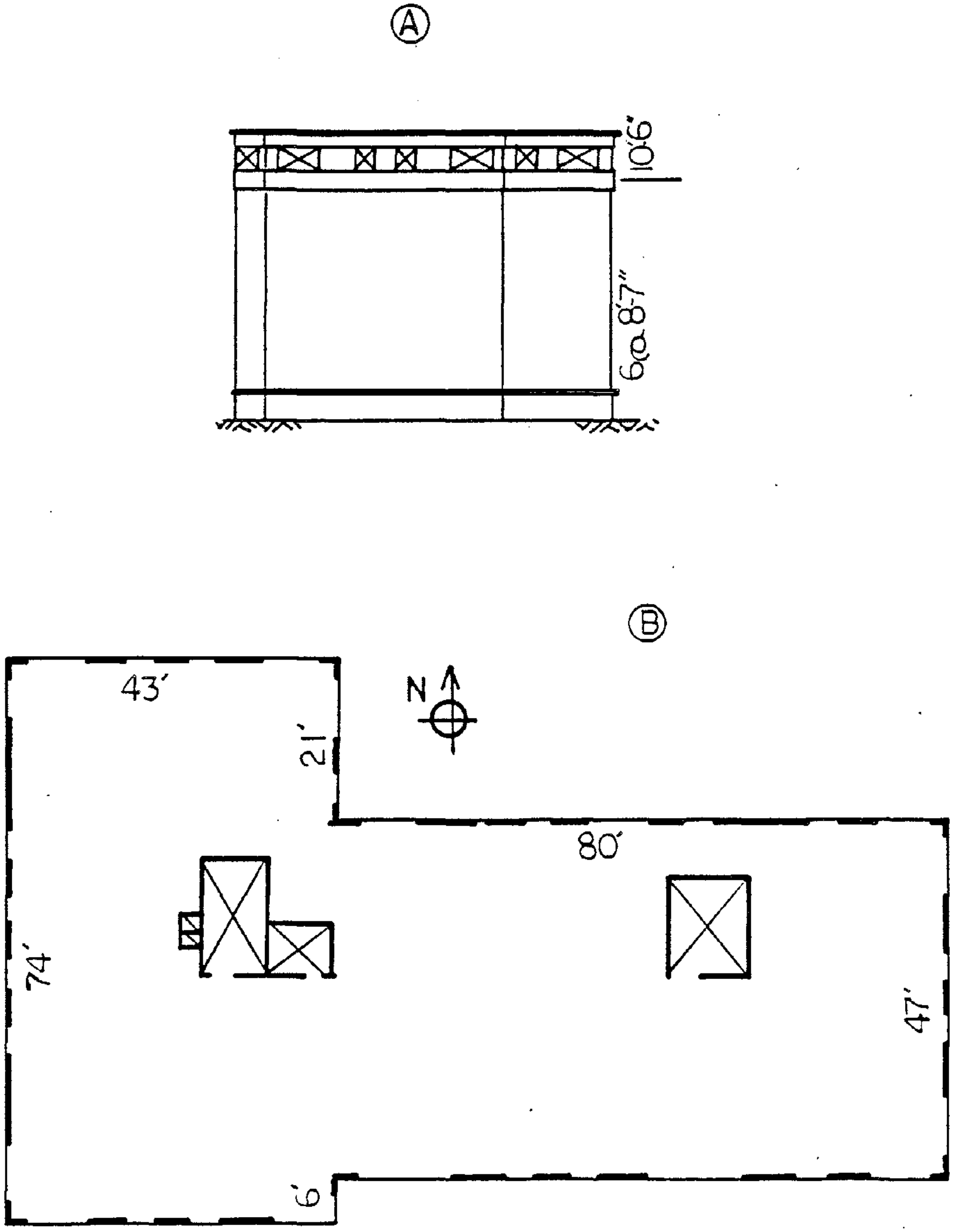

Figure 2-7: Knik Arms Apartment House:
(A) E-elevation.
(B) Typical upper story plan.

DESIGN & CONSTRUCTION

The Knik Arms Apartment House was built in 1950. Lateral forces are resisted by the reinforced concrete shear walls, the exterior walls providing the bulk of this resistance. These exterior walls have a single curtain of reinforcing steel. Conventional trim bars are found around the wall openings. Concrete quality was specified to be 2.5ksi at 28 days.

DAMAGE/CAUSE

There was no significant damage to this structure.

The explanation for this good performance may lie, in part, in the less intense vibrations experienced by this building due to the soil liquefaction at depth. The 10' building translation, occurring over an estimated 1 or 2 minutes, would not induce significant stresses in the building. Also, due to its short period, the building was not as susceptible to this earthquake with its relatively long-period motions. This short period can mainly be attributed to the shear wall structural system.

2.9 THE 1200 L STREET APARTMENT BUILDING

[Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The 1200 L Street apartment building is located about 1,300' S-E of the L Street landslide in Anchorage. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale. No pertinent information on local site conditions was found.

STRUCTURAL SYSTEM

The 1200 L Street apartment building is a 14-story reinforced concrete structure measuring 52'-4" x 129'-8" in plan and 119' in height. Its long dimension lies in the N-S direction. It is almost identical to the McKinley Building with respect to construction, orientation, and type of earthquake damage.

The building is essentially a bearing wall or box structure with exterior walls formed by piers and spandrels and a central core. The exterior wall piers between window and entrance openings are designed as columns. Exterior wall thickness is 8" from roof down to 8th floor, 10" from 8th to 2nd floor and 12" below. The 8" bearing walls and the 10" bearing walls in the 5th, 6th and 7th stories have a single curtain of steel. Spandrel lengths are 3'-4" and 6'-10". All spandrels have a gross depth of 4'. For architectural reasons the exterior spandrel faces are rusticated and set back 1 1/2" from the column face in the transverse end walls and in the two N- and S-bays of the longitudinal walls of the building. The set-back and the 1" deep rustications reduce an 8" wall to about 6" at the rustication. There are no corner columns; cantilevered spandrels are used instead. The core consists of two elements: one a composite of the elevator shaft, stack stairwell and a heating / ventilating duct, and the other a detached stairwell.

Floors are generally 5 1/2" thick reinforced concrete one-way slabs having a maximum clear span of 17'-9". These floors are supported by the exterior walls and interior reinforced concrete beams. The beams are spanning

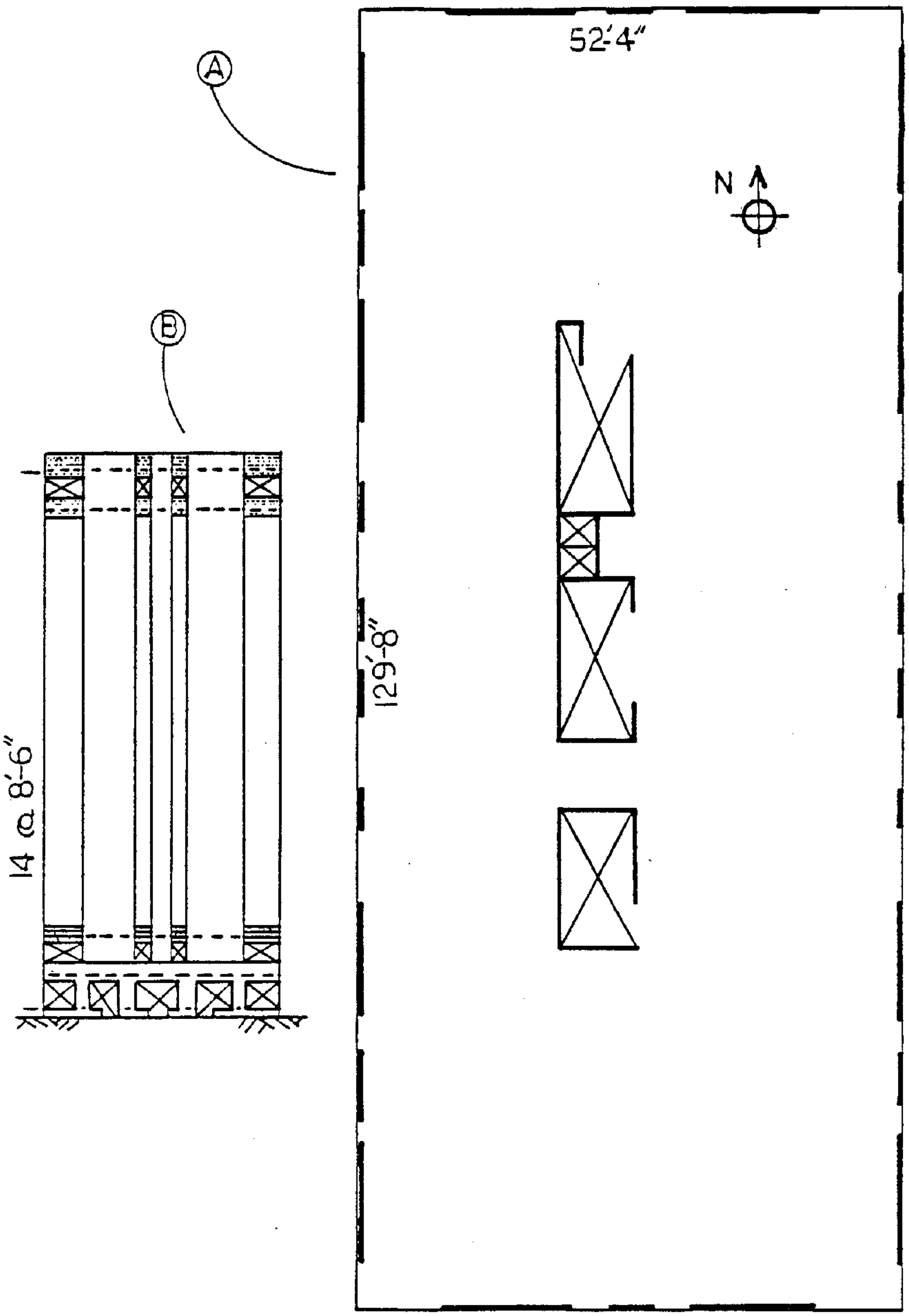

Figure 2-8: The 1200 L Street Building:
(A) Typical plan.
(B) S-E elevation.

longitudinally between cores. bearing walls and reinforced concrete columns. The floors have few openings, and these are so placed that they do not significantly reduce the strength of the floor slab as an earthquake diaphragm. Footings are of the reinforced concrete spread type.

All windows between shear walls are of glass. Plaster partitions were constructed of 2" of solid plaster with an embedded metal lath extending from floor slab to floor slab.

DESIGN & CONSTRUCTION

The $1.2 million building was originally designed for zone 2 UBC requirements and checked by the International Conference of Building Officials in 1951. Unconfirmed reports state that the Federal Housing Administration required the contractor to further strengthen the building.

Because the spandrels function as beams, although not particularly designed for this purpose, the peripheral piers and spandrels represent coupled shear walls. The exterior shear walls and the core constitute the lateral force-resisting elements of the structure. The concrete used has a cylinder strength of 3ksi.

DAMAGE

The 1200 L Street Apartment building was heavily damaged during the earthquake. The cost of repairing the building is estimated to have been in the range of $0.3 to $0.5 million. This damage came to about 30% of the replacement value of the building.

Shear failure of the spandrels in the transverse end walls was evidenced by pronounced X-cracking from the 2nd to 11th floor. In each of the two longitudinal walls. shear failures occurred in approximately 1/3 of the spandrels. Substantially all the cantilevered corner spandrels failed. In the S-end wall the westward wall pier failed completely in flexure in the 2nd story. Similar partial failures occurred in the E-ward wall pier at the 3rd floor level, and there is evidence of compression failure at the center pier. In the W-wall pronounced X-cracking occurred in four piers in the 2nd story over the main entrance of the building. All construction joints showed movement.

Nonstructural damage was major. Glass breakage at the facade was extensive. Glass was undamaged when the spandrels were only cracked. Most of the plaster partitions were damaged by cracking. The solid plaster partitions in the corridors were also damaged.

CAUSE

The piers and spandrels forming the walls of the 1200 L Street building were proportioned in such a way that very unfavorable distributions of shears and moments were developed in the spandrel beams. The cantilevered corner spandrels failed in diagonal tension as they were subjected to combined forces induced by the E-W and N-S motion. The failures in the spandrels caused excessive deformation that could not be absorbed by the sash and glass panel. The absence of pier failures in the N-end wall is attributed to the N-ward eccentricity of the central core.

Some recommendations are:

- Coupling spandrels should be designed for seismic forces.
- Corner columns should be used instead of cantilevered corner spandrels.
- Eccentricities between center of mass and center of rigidity in the lateral force-resisting system should be avoided.
- Partitions should be isolated from the slab above and from shear walls and columns.

2.10 THE McKINLEY BUILDING

[Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The McKinley building, almost identical to the 1200 L Street apartment building, is located about 800' east to the 4th Avenue landslide in Anchorage. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale. No pertinent information was found on local site conditions.

STRUCTURAL SYSTEM

This 14-story building is 119' high, measures 52'-4" x 129'-8" in plan, and has a basement, a penthouse, and a 72' free standing TV tower on the roof. An adjoining 1-story building is structurally separated from the high-rise McKinley building by 8".

The building is essentially a reinforced concrete bearing wall box-type structure with shear walls along the perimeter and a central elevator/stair core. The exterior bearing walls are 12"-thick from basement to 3rd floor, 10"-thick from 3rd to 8th floor, and 8"-thick from 8th floor to roof, with a single curtain of steel from 5th floor to roof. Window openings in these walls are between spandrels with rustications in the N- and S-sides and around the corners of the building. Spandrels with rustications are nominally 1" thinner and also rustications are about 1"-deep reducing an 8" wall to about 6" at the rustication. Cantilevering rusticated spandrels "wrap around" the corners.

Floors are generally 5 1/2" thick reinforced concrete one-way slabs having a maximum clear span of 17'-9". These floors are supported by the exterior walls and interior reinforced concrete beams. The beams are spanning longitudinally between cores, bearing walls and reinforced concrete columns. The floors have a few openings and these are so placed that they do not reduce the strength of the floor slabs as earthquake diaphragms. Footings of the building are of the reinforced concrete spread type.

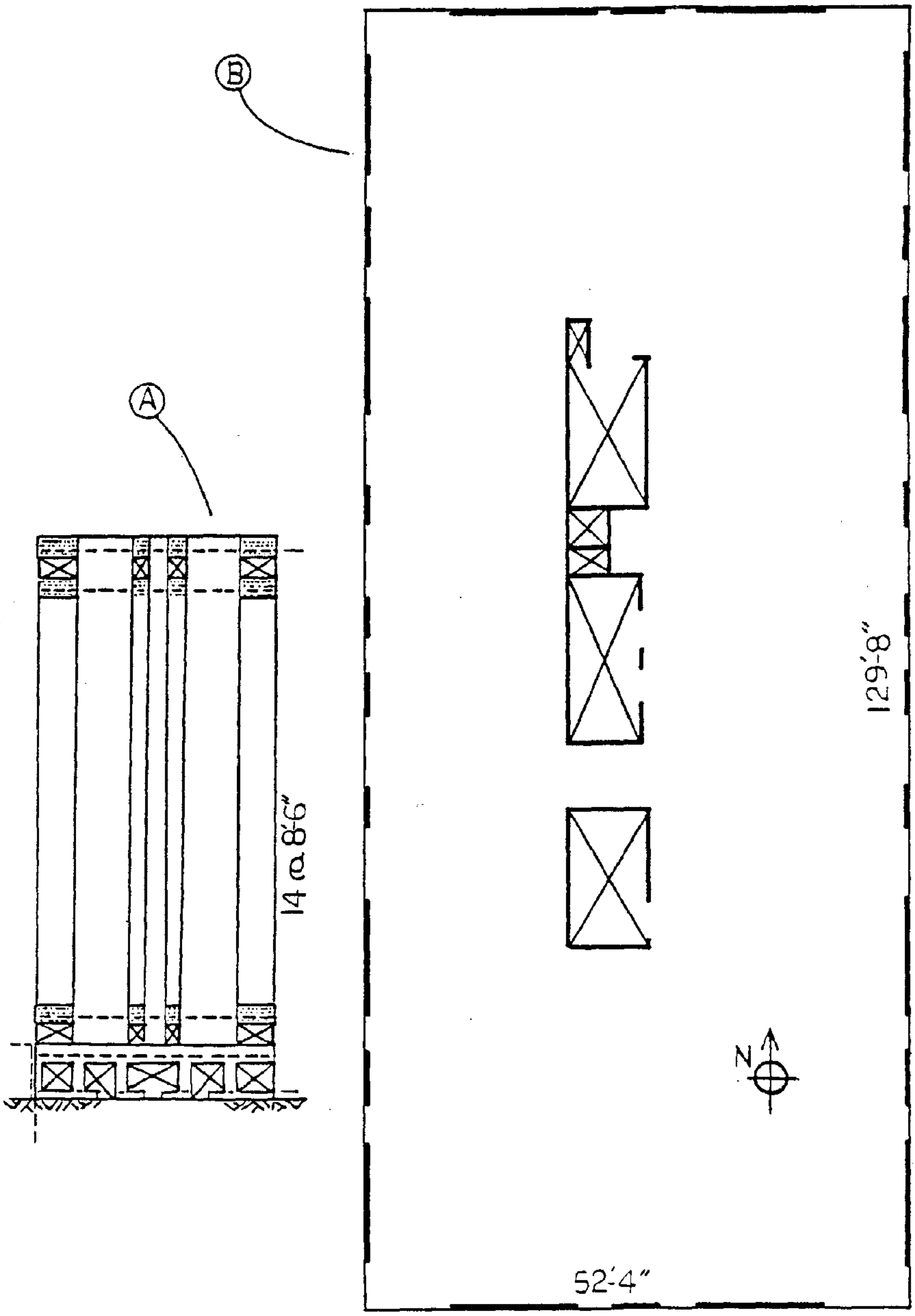

Figure 2-9: McKinley Building:
(A) S-elevation.
(B) Typical plan.

The nonbearing walls which constitute the majority of the interior partitions are 6"-thick.

DESIGN & CONSTRUCTION

The McKinley building was designed and constructed in 1951. The UBC zone 2 requirements were used in Anchorage until about 1954. Thus it is reasonable to assume the design was based on the 1949 edition of the UBC along with its seismic zone 2 factor.

The concept of the lateral force resisting system was simple. Exterior bearing walls acted as shear walls as well as bearing walls. The central core certainly contributed somehow to the buildings earthquake resistance.

A sample of the concrete used tested at 2.6ksi; data on hand would indicate that the specified 28-day strength was 2ksi.

DAMAGE

The damage to this building has reliably been placed at 40% of its replacement value.

The bearing wall in the N-face failed at the 3rd story. This failure constitutes a complete structural separation through the entire pier. The rusticated spandrel beams in the N-face were severely damaged by shear X-cracks, principally from the 3rd to 11th story. The cantilevered rusticated corner spandrels showed X-cracking from shearing forces, but also pronounced horizontal movement along the construction joint at the floor lines around the corner. The S-face did not have as severe damage to the rusticated spandrels as did the N-wall; however, one 1st story pier failed. Movements occurred along the construction joints in the core walls, more pronounced in the mid stories of the building.

Glass breakage occurred only close to failures of adjacent structural elements. Mechanical equipment in the penthouse was dislocated in many cases by the earthquake.

CAUSE

According to the sources of the digest, one can visualize the N-wall as a vertical cantilever fixed to its basement walls. The whole mechanism can be explained as a plastic hinge forming at the wall base. Total shear forces on this vertical cantilever became progressively greater in the lower stories causing the rusticated spandrels to fail. Damage to the pier along the N-wall is best explained by overturning bending, tension, and compressive forces. Another factor could be the floor slab fixed to the pier, which introduced torsion into the wall. Overturning forces can also account for the damage to cantilever rusticated spandrels at the buildings corners. Since they "wrap around" the corners, spandrel deflections are restrained and shearing stresses result in the spandrels. The same line of reasoning applies for the 1st story pier which failed by overturning forces plus shear and column bending.

A comparison of the building's design with current UBC provisions indicates that this structure would be quite deficient by today's standards. Obviously a designer in 1951 could not have anticipated seismic codes almost two decades in advance.

Some recommendations are:

- Coupling spandrels should be designed for seismic forces.
- Corner columns should be used instead of cantilevered corner spandrels.
- Mechanical equipment should be secured from earthquake movement.
- Cladding and partitions should be isolated from seismic motion.

2.11 PROVIDENCE HOSPITAL

[Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The building complex of the Providence Hospital lies about 3 miles S-E of downtown Anchorage. The earthquake intensity for the buildings in Anchorage (about 80 miles west of the Alaska earthquake epicenter) was estimated as VIII on the Modified Mercalli Intensity scale.

Detailed soil engineering information for this site has not been found. However, borings made at a nearby site suggest the typical sands and gravels underlain by clay found elsewhere in Anchorage. After the earthquake, there were no landslides or other significant similar surface ground effects in the region. Soil design pressure was 4ksf.

STRUCTURAL SYSTEM

The hospital is structurally one unit from the foundations to the 3rd floor level. Above this level it is divided into two independent units separated by 2". The E-unit is the 5-story plus penthouse main tower of the hospital building, measuring 94'-6" x 74'-4" in plan, and 62' in height. The W-unit is a 6-story core tower. A basement exists beneath the two units.

The structural system of the main tower consists of a steel frame with a reinforced concrete central core and some shear walls. The W-tower unit is a reinforced concrete core. Floors and roof are reinforced concrete on metal deck. Foundations are of the reinforced concrete type.

Exterior walls of the W-tower unit are of cast-in-place architectural concrete, while the main tower has porcelainized enamel walls. No mention is made of interior partitions or other nonstructural elements.

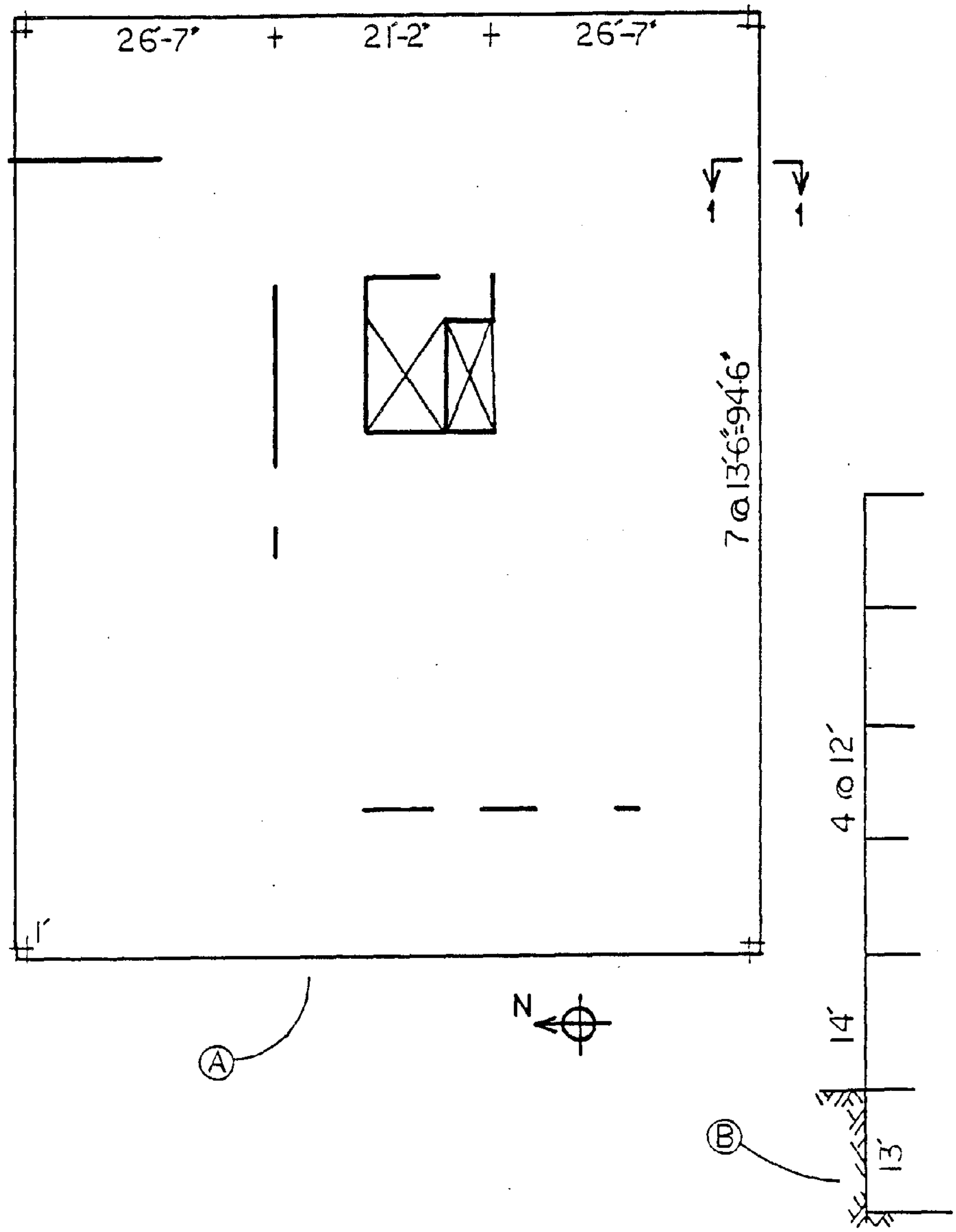

Figure 2-10: Providence Hospital:
(A) Typical main tower plan.
(B) Elevation 1-1.

DESIGN & CONSTRUCTION

The hospital was built in 1961 according to the 1958 UBC zone 3 requirements.

The lateral force resisting system is provided by the reinforced concrete walls acting as shear walls and the floors and roofs acting as diaphragms. The 1st and 2nd stories which have larger floor areas than the upper stories also have considerably more shear wall areas as bracing elements. One can consider the two structurally separated units above the 3rd floor as being two independent buildings on a common base.

All reinforced concrete had 28 day design strength of 2.5ksi. All walls were 8"-thick with all steel in a single curtain for the upper 3 stories. The duct openings in the main tower core, although on the mechanical drawings, were not shown in the architectural and structural drawings.

DAMAGE

The repair cost has been estimated at 2.5% of the buildings replacement value.

The principal earthquake damage was found in the walls of the central core of the main tower. In one 8" reinforced concrete shear wall concrete crushed above doorways in the 3rd, 4th, and 5th story; the damage was vertically aligned and progressively decreased in the upper stories. The damage was more severe at the duct openings. The adjoining walls in this central core had hairline X-cracks plus some hairline movement along several construction joints. This relatively minor cracking was more noticeable in the 3rd story.

No significantly damaged concrete was found below the 3rd story. Damage was negligible elsewhere in the building. The sheet-metal cover over the 2" structural separation worked loose between the 3rd story and the roof. The penthouse on the main tower was also damaged.

CAUSE

That there was damage to the Providence Hospital is quite significant since it was one of the better designed and constructed buildings in Anchorage.

The damage was concentrated at points of vertical discontinuities. It is apparent that from the 3rd floor down the shear walls were adequate to resist the lateral forces. Also, as previously described, the first two stories are quite different from the upper stories. The damage over doorway openings can be explained when the lintels are viewed as coupling beams between shear walls. The problem of ducts piercing shear walls is a common one and no doubt it will give trouble to many presently constructed buildings in future earthquakes.

Some recommendations are:

- Design carefully buildings with structural irregularities.
- Adequate analysis is needed of coupling beams in shear walls.
- Structural elements with openings should be properly designed for seismic forces.
- Develop design values for shear in horizontal construction joints in shear walls.

3. THE ALASKA, WHITTIER, EARTHQUAKE OF 1964. (64-3).

The great Alaska earthquake occurred at 5:36 p.m., March 27 1964. The Richter magnitude of the earthquake was 8.4; the focus was about 12.5 miles below the surface and the generated fault progressed in a S-W direction.

Whittier is a port located about 40 to 50 miles S-W of the epicenter. Submarine landslides and the resulting waves extensively damaged waterfront facilities and caused 13 deaths. Other damage was caused by consolidation of the ground. The shaking reached its maximum intensity in about 1 1/2 minutes and then gradually subsided during the next 2 1/2 minutes. The earthquake intensity at Whittier was estimated as IX on the Modified Mercalli Intensity scale. Whittier was closer to the seismic energy release than was Anchorage which may mean that the shorter period ground motions were relatively more significant at Whittier than in Anchorage.

REFERENCES: [Ayres, 1967], [Berg, 1964], [Blume66, 1966]. [Committee, 1973], [Hansen, 1965], [Whitman, 1972], [Wood, 1967].

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3.1 BUCKNER BUILDING

[Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The Buckner Building is located at Whittier. and rests mainly on bedrock; eyewitnesses at the site described the earthquake as a jarring motion. The earthquake intensity at Whittier (about 40 miles from the epicenter) was estimated as IX on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The Buckner Building is unique in that it is a complete city by itself. It was designed to provide mess. sleeping, recreational, medical, and administration facilities for 1,250 men. Because of its multipurpose function the building contained many items of specialized equipment, for example, a bowling alley, an X-ray machine. and a large oven. The major portion of the building consists of 6 stories. The long dimension of the structure lies in a NE-SW direction. The lower 2 stories are 12' high; the remaining stories are each 10' high. The structure is divided in rectangular parts by 8" joints.

The structural system of this reinforced concrete building consists of exterior shear walls enclosing the building, interior shear walls around various elevator shafts and stairwells, and interior rigid frames with infill walls. The floor system consists of reinforced concrete slabs on beams. Footings of the columns extend to the solid bedrock.

Interior partitions in the building are of two types: concrete masonry unit (both 4" and 6") and gypsum board on wood studs. The concrete masonry unit partitions were tied to the structural frame on all sides and were reinforced. However, vertical reinforcing bars had been installed only to the lower half of the partition.

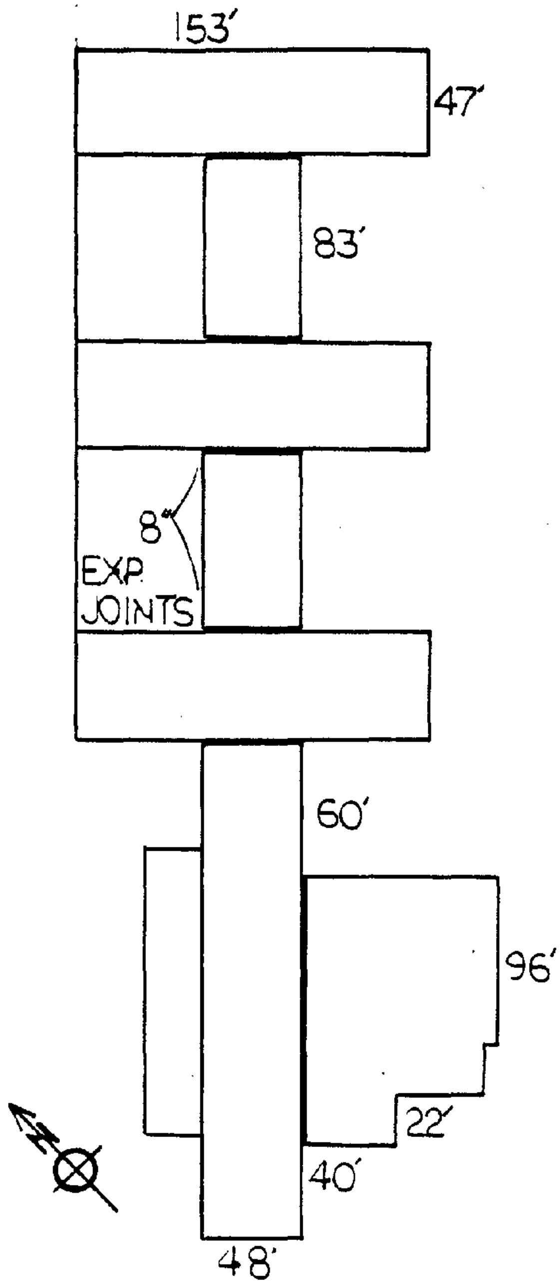

Figure 3-1: Buckner Building:
General plan configuration.

DESIGN & CONSTRUCTION

The Buckner Building complex was planned in 1951 and construction was completed in 1953.

Lateral resistance was furnished by the exterior concrete walls, the structural frames, and the stairwells and elevator cores located throughout the building. Walls for these cores are poured 8" reinforced concrete.

DAMAGE

The structural damage to the Buckner Building was negligible.

The majority of the damage was to nonstructural items. The concrete masonry partitions were severely damaged. The upper unreinforced half of the 5th story partition collapsed. At one stairwell at the E-end of the building a cold joint worked considerably. The construction joints in exterior walls showed evidence of movement; each joint was clearly visible and had fresh mortar spalls.

CAUSE

The building performed very well under the strong shaking of the Alaska earthquake. This is partly due to the stiff structural system of concrete walls enclosing the building and rigid frames and stairwells found throughout the building. This makes a short-period structure that is less affected by the long-period motion at Whittier. The underlying bedrock was also beneficial to some extent. The structure was made even stiffer by the concrete masonry unit partitions which participated in the response and failed, because they lacked reinforcement.

Some recommendations are:

- Avoid the use of unreinforced and unanchored masonry unit partitions.
- Improve the design and inspection of horizontal construction joints.

3.2 THE HODGE BUILDING

[Committee, 1973], [Wood, 1967], [Berg, 1964].

GROUND MOTION & SITE

The site of Hodge building is located at Whittier. The earthquake intensity at Whittier, (about 40 miles from the epicenter), was estimated as IX on the Modified Mercalli intensity scale. People at the site described the earthquake as a rolling and "round and round" motion. Unconsolidated deposits are at least 44' thick beneath the Hodge building. The design soil pressure was 10ksf.

STRUCTURAL SYSTEM

This apartment house is a 134' high, 14-story structure with a penthouse and a basement. It is divided into 3 monolithic units separated by 8" crumble joints. The central unit is rectangular, 47' x 83' in plan, and the two end units are L-shaped with an overall dimension of 96' x 91'. Each of the L-shaped units has a stair tower and two elevator shafts. The story height is 10'-8" for the basement and ground story and 8'-8" for the remainder of the building.

The Hodge building is of cast-in-place reinforced concrete. The structural system consists of exterior bearing/shear walls and interior frames and cores. The majority of the beams are 14" x 20". The columns were designed for a maximum thickness of 14" and the width was varied with load. Floor slabs span almost 20'. The basement is a 4" nonstructural slab on grade. The remainder of the floors are one-way slabs on beams, most of which are 6 1/2" thick. Footings are of the reinforced concrete type.

The exterior bearing walls have exposed concrete finish. The numerous interior partitions are of hollow concrete block construction. They are attached to the structural frame by dovetail anchors. As far as could be determined, there was no reinforcing steel in the partition walls [Committee, 1973]. In bathrooms the partitions have ceramic tile finish.

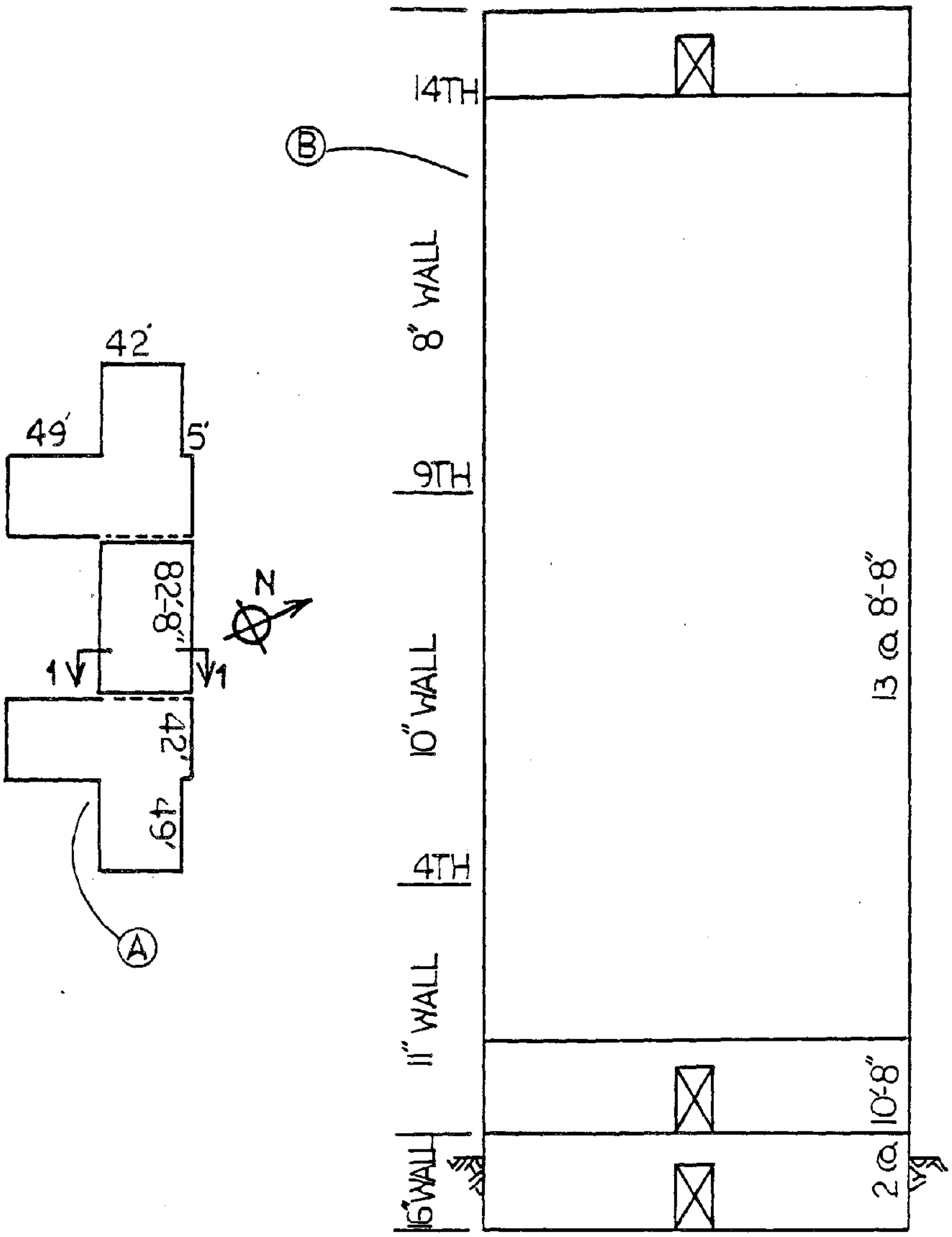

Figure 3-2: Hodge Building:
(A) Plan configuration.
(B) Elevation 1-1 of shear wall.

DESIGN & CONSTRUCTION

The structural design of the building was planned in accordance with the ACI Bulletin (ACI 318-51) and to the Pacific Coast UBC for zone 3 requirements. Lateral resistance was provided by the exterior reinforced concrete shear walls. All structural concrete used in the building had compressive strength of 2.5ksi. Concrete quality appeared to be excellent from a visual inspection standpoint. All reinforcing steel was of intermediate grade conforming to ASTM Spec.A-305.

The shear and overturning effect produced by the floor shears were distributed to the shear walls in proportion to their relative rigidity. The floors were assumed to be rigid diaphragms. The wall piers and spandrels were designed to develop the wall as a cantilever beam above the foundations. The outside walls were poured up to the bottom of the story above. The floor slab was then poured adding a cold joint.

The unreinforced concrete block walls were not anchored at their boundaries, although the design drawings stated that K-web reinforcing was to be placed in the hollow concrete block at specified horizontal joints [Wood, 1967][2].

DAMAGE

The Hodge building suffered moderate damage during the earthquake.

The principal structural damage in the building occurred in the central unit, at the lintel beams over corridor door openings. The 14th floor lintel failed in shear. This lintel was weakened by pipe openings not shown in the structural drawings. Lintels in other stories experienced severe diagonal cracking and concrete spalling, which exposed reinforcement. The damage decreased at lower stories. A deep lintel at the 1st floor level showed

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[2]Apparently, this is a contradiction between two different sources of digests. The authors believe that some of the partitions were not properly anchored in this building.

X- cracking. The cold joints at floor levels in the exterior walls moved during the earthquake. Movement along construction joints was not noted anywhere else in the building, indicating a better quality cold joint than that found in Anchorage buildings. Other structural damage throughout the building was minor.

The steel plate cover over the 8" earthquake joint at the E-end of the center portion of the Hodge building was damaged by N-S movement of the two building units. Many of the concrete masonry unit partitions on the top floors were severely damaged; on the 13th and 14th floor most had been knocked down. On the 12th floor there was a noticeable reduction in the amount of damage. The partitions lacked reinforcement, and the failures occurred at the mortar joints. The ceramic tile-finished partition on the 13th floor collapsed, whereas the identical partition on the 8th floor cracked but did not collapsed. The ceramic tile finish probably strengthened these partitions, but not enough to prevent the failures in the 13th floor where the lateral forces were greater.

CAUSE

The bad detailing of the lintel beams over doorway openings was the main cause of damage which followed a pattern similar to that found in other buildings in Anchorage. The increased damaged at the upper floors is in accordance with the greater interstory displacements which occur at higher levels of the shear walls. In view of the experience in Whittier and Anchorage, the use of unreinforced and unanchored unit-masonry partitions. which are prohibited by code, should never be allowed even under extenuating circumstances.

Some recommendations are:

- Provide either adequate strength or ductility to coupling beam/girders between shear walls.
- Develop design values for shear in horizontal construction joints in shear walls.
- Avoid the use of unreinforced unanchored masonry-unit partitions.

4. THE CARACAS, VENEZUELA, EARTHQUAKE OF 1967. (67-7)

At 8:00 p.m. on Saturday night, July 29 1967, a 6.5 Richter magnitude earthquake occurred in the Caribbean sea. The focal depth was estimated to be about 10 miles. The epicenter was approximately 30 miles NW of the city of Caracas. It has been estimated that the total duration of the earthquake was approximately 60 seconds. The strong motion portion lasted 1/4 to 1/3 of this time.

Of an estimated 10,000 multistory buildings in the Caracas area less than 300 suffered structural damage. About a dozen buildings suffered damage in the beach area. The strongest ground motion at Caracas did not appear to be as severe as that experienced in Caraballeda. Caraballeda lies 10 miles N of Caracas on the Caribbean coast, approximately 40 miles E of the epicenter, but closer to the main fault running parallel to the coast. Although the earthquake was moderate, it killed about 266 people in north central Venezuela; 156 of these deaths are related to four buildings that collapsed in Caracas, and 43 to a partial collapse of a multistory building in Caraballeda.

REFERENCES: [Fintel, 1967], [Hanson, 1969], [Skinner, 1969], [Skinner, 1968], [Sozen, 1968], [Steinbrugge, 1968], [Whitman, 1968].

4.1 CAROMAY BUILDING

[Seed, 1970], [Fintel, 1967], [Hanson, 1969], [Skinner, 1968], [Sozen, 1968], [Steinbrugge, 1968].

GROUND MOTION & SITE

The Caromay Building is situated in the city of Caracas on 10'-deep fill, which was placed over an alluvial soil with sand, clay and large boulders. The earthquake intensity at Caracas (about 30 miles SE of the epicenter) was estimated as VIII on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The Caromay Apartment Building, a relatively new structure, is curved in plan and has 19 stories plus basement and penthouse. The building is 159' high, and story heights are 8'-10". The plan consists of eight full radial bents plus one partial bent in the center. The building is regular in plan except at the basement parking area. This level is very irregular, concrete retaining walls being connected with the structure on the N- and W-sides.

The structural system is a reinforced concrete frame. Wide, flat beams acting as girders and deeper girders at the central and end column lines span in radial direction. Along the three circumferential column lines there are 24" wide by 10" deep beams/joists with tile fillers. Although the soil capacity was 5ksf, the building was founded on piles for economic reasons.

The interior partitions are tile walls following the frame lines arranged symmetrically with respect to the central elevator shaft. Apparently. cladding was also of tile wall construction.

DESIGN & CONSTRUCTION

The building was designed according to the 1955 edition of the Venezuela code(MOP). The overturning moments calculated by this code are equivalent to the zone 2 moments of the 1967 UBC without the "J" factor reduction. In

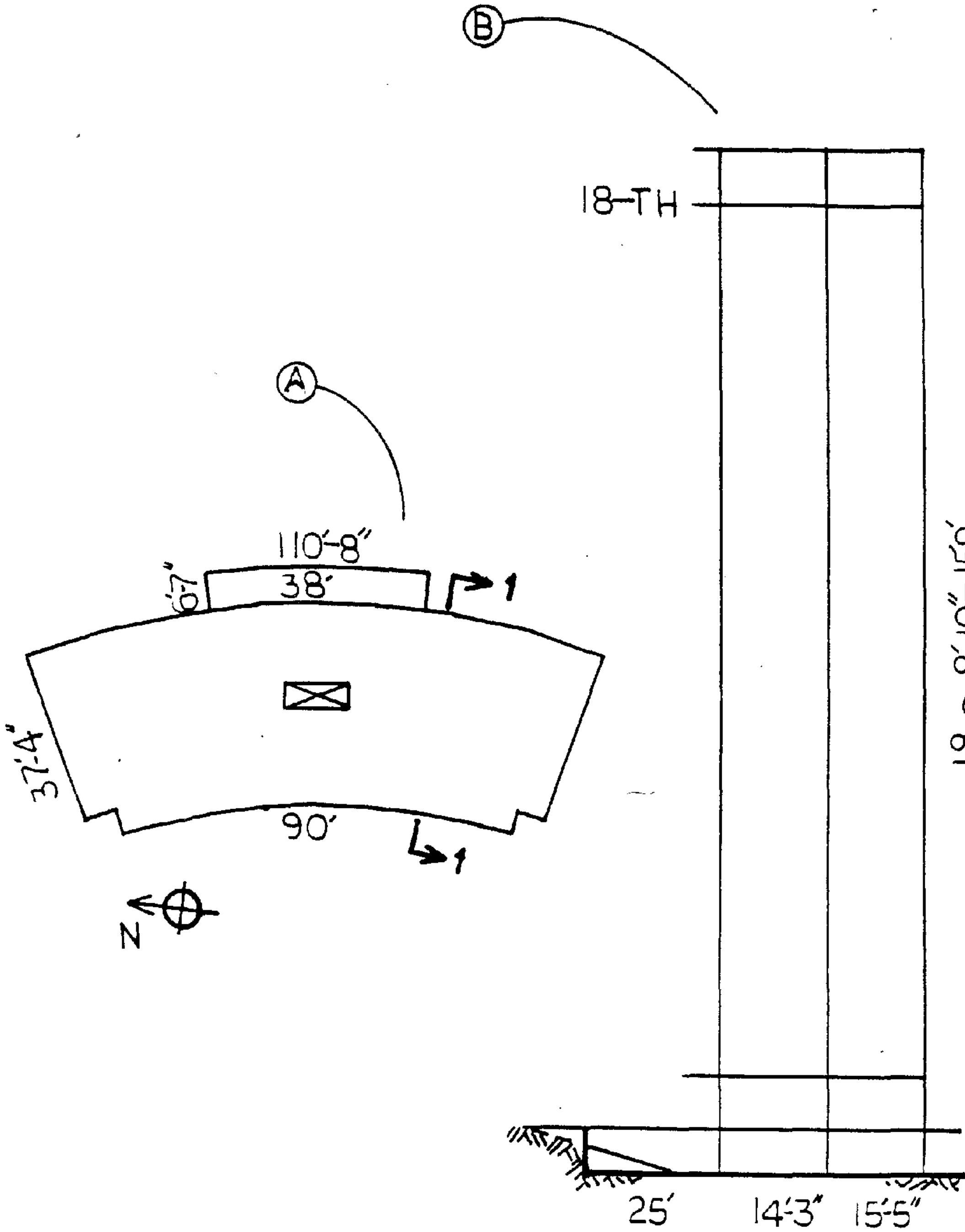


Figure 4-1: Caromay Building:
(A) Typical plan.
(B) Elevation 1-1.

actuality, the minimum base design shear of 2.5% used in design was equivalent to UBC zone 3 requirements.

Reinforcing steel is comparable to intermediate grade with a working stress of 17ksi, while concrete, apparently of good quality, was specified to be 3ksi. Actual cylinder strengths were over 4ksi.

DAMAGE

This building suffered major structural and nonstructural damage.

The most spectacular damage occurred at the basement level, where several columns failed at about mid-height. These were classic compression failures with cone shaped concrete spalling and outward buckling of the 1" bars. The column failures and their obvious importance overshadowed the possible significance of the cracks in the beams found on a later examination. All girders on the radial lines were cracked at the ground level; cracking decreased in quantity and severity up to about the sixth floor.

Damage to tile partitions was maximum in the lower stories and decreased towards the top. There was little damage above the 6th floor. The lower portions of the tile end walls were severely broken. The stairway, which was being used to evacuate the personal possessions of the tenants, was badly shattered up to about the 10th floor.

CAUSE

The location of the failed columns and the incipient failures suggest that there were sufficient interior tile walls to make the building act as a cantilever unit forming a plastic hinge at the lower stories. The quality of the concrete and the location and type of failures indicate tremendous overturning moments and base shears corresponding to an equivalent lateral acceleration of about 0.20g. The damaging motion was in the EW (radial) direction. However, there must have been substantial motion in the NS direction as indicated by the stair failures and the cracks in the concrete girders. The solid tile walls on end lines were severely damaged between the 2nd and 3rd floor, but were in remarkably good condition above that point.

Some lessons learned from the behavior of this building are:

- Overturning forces can occur that are greatly in excess of those anticipated by codes or previous studies.

- Nonstructural elements can be damaged if they are not isolated from seismic motion.

4.2 CHARAIMA BUILDING

[Seed, 1970], [Mahin, 1974], [Fintel, 1967], [Hanson, 1969], [Skinner, 1968], [Sozen, 1968], [Steinbrugge, 1968].

GROUND MOTION & SITE

The Charaima apartment building is located approximately 10 miles north of Caracas on the Caribbean coast in Caraballeda. All beaches in this area are the result of alluvial deposits from the streams cutting valleys into the mountain sides.

No ground motion accelerograms were recorded during the 1967 earthquake. The earthquake intensity at Caraballeda (about 40 miles east of the epicenter), was estimated as VIII on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The 11-story building is about 120' high, and measures 62'-5" x 177' in plan. The floor plan is rectangular and consists of 10 evenly spaced three-bay reinforced concrete frames supported by individual footings on pile groups. A penthouse for mechanical equipment is located above the stairwell.

The structural system is a reinforced concrete frame. Most columns are rectangular except for a few spirally reinforced columns in the first and second story. In the longitudinal direction of the framing system, haunched T-beams are used. Shallow rectangular beams are used in the transverse direction except for girders adjacent to the stairwell and elevator shaft, which are considerably deeper. The floor system is a thin slab with tile blocks supported by transverse joists.

Typical partitions are very light and, thus, of no structural value. Brick masonry walls enclose the elevator shaft and hollow clay tile walls are located in the end frames above the first floor.

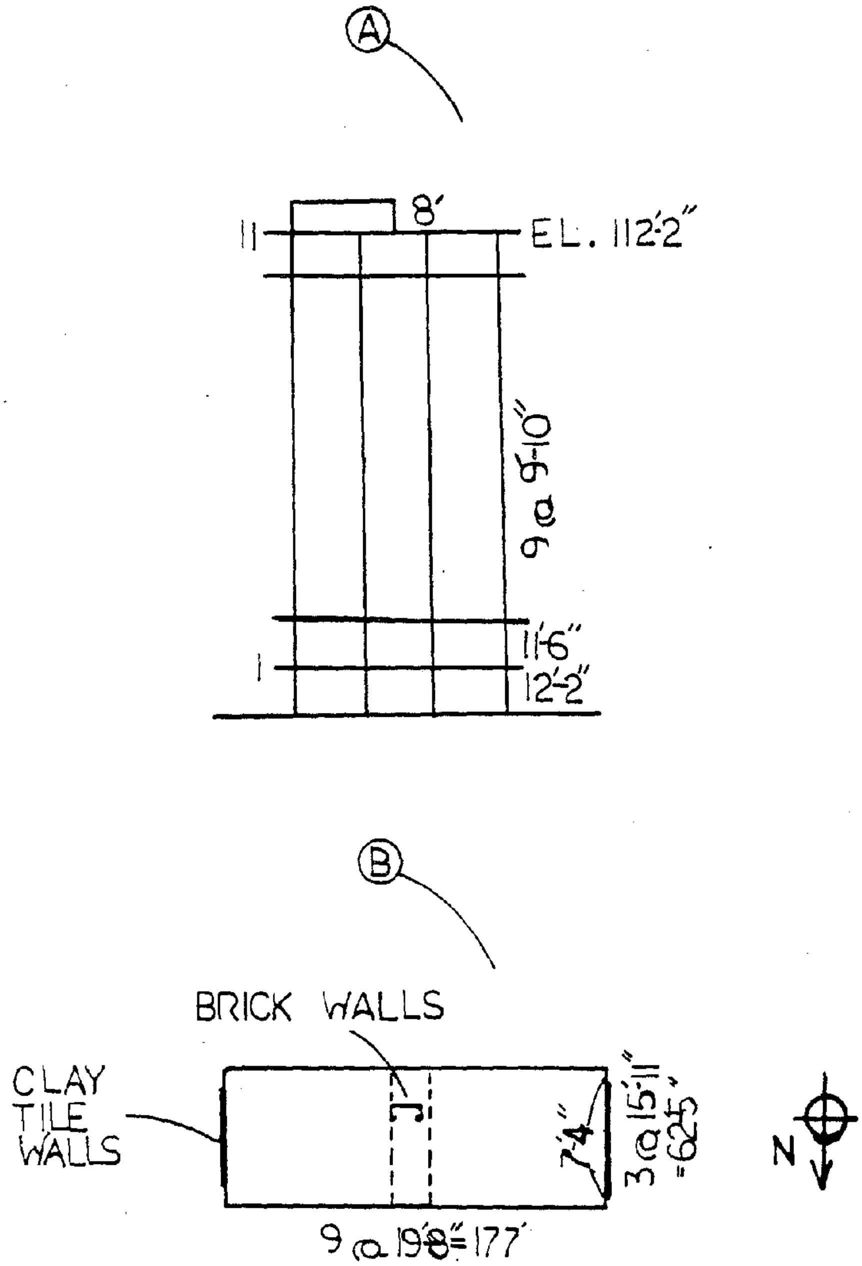


Figure 4-2: Charaima Building:
(A) Typical transverse frame.
(B) Typical plan.

DESIGN & CONSTRUCTION

The Charaima building was designed in 1954 to be 10 stories tall, using working stress methods in accordance with the 1947 building code of Venezuela. Generally, following the ACI code of 1951, it does not satisfy current recommendations for ductile, moment-resisting frames. A major shortcoming was the low amount of transverse reinforcement in most of the columns, typically No. 2 or No. 3 ties at 7.9" (200 mm) spacing. Construction was halted at the 7th story level, while an additional 11th story was designed.

The structural system was designed to resist critical combinations of dead, live, wind, and earthquake loading. Seismic loads were represented by static lateral forces at each floor level equal to 5% of the total dead load and 50% of the live loads on that floor. For seismic loads, a 33% increase in allowable stresses was permitted. Only approximate analyses were used to determine design loads, and any dynamic effects were disregarded. No walls or partitions were treated as structural members.

Intermediate grade (40 ksi) reinforcement and normal stone aggregate concrete with a strength of 2.13ksi were specified. An investigation of the structure after the earthquake revealed that design specifications were met by the builders, except that understrength concrete (about 1.6ksi) was apparently used in the collapsed stories. Deficiencies in construction methods or materials were not believed to be the principal cause of failure.

DAMAGE

As a result of the earthquake the top four stories collapsed, their slabs finally resting on top of each other. The collapse caused 42 deaths. In addition, some column failures occurred near the elevator shaft down to the 4th story. The failure appeared to be oriented toward the south in the transverse direction.

The remainder of the building suffered considerable nonstructural damage.

CAUSE

Elastic and nonlinear analyses show that the failure was due to brittle columns located adjacent to the elevator shaft. Here the stronger and stiffer girders caused the columns to yield with the result that their axial loads were redistributed to adjacent columns. All columns in the top three stories were identical. Thus the columns of story nine were most critically stressed, especially those adjacent to the elevator shaft, which had to resist additional loads from mechanical equipment and penthouse.

In lower floors, ductile beams yielded before columns, which were more heavily reinforced. This indicates an inconsistency in the structural system due to strong girders around the elevator shaft forcing yielding in the adjoining brittle columns.

Some recommendations are:

- Yielding should be initialized in and confined to girders rather than columns.
- Irregularities and inconsistencies in the structural system should be avoided.

4.3 MACUTO SHERATON HOTEL

[Seed, 1970], [Fintel, 1967], [Hanson, 1969], [Skinner, 1968], [Sozen, 1968], [Steinbrugge, 1968].

GROUND MOTION & SITE

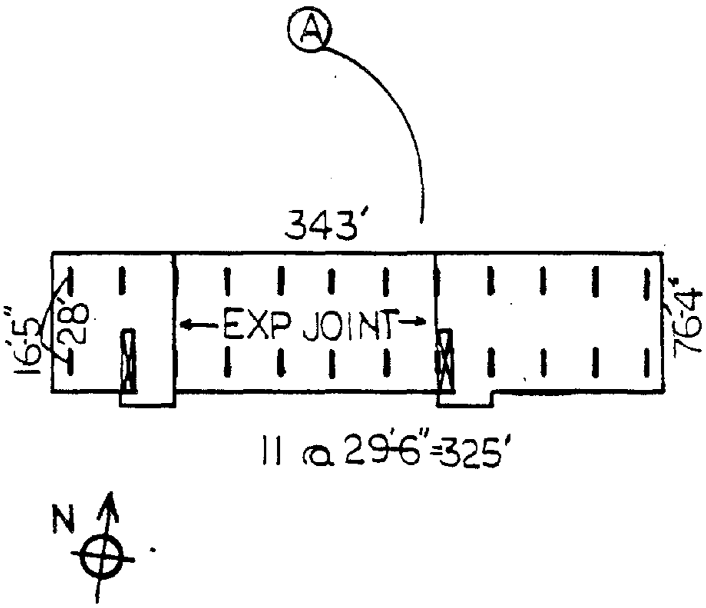
The Macuto Sheraton Hotel comprises a complex of structures built on reclaimed land at the Caraballeda Beach. This area is a large alluvium deposit produced by three rivers which feed into the sea. The lagoon was dredged with the resulting fill material used to extend the area of usable land. It appears that all fills are uncompacted. No soil borings are available to estimate the alluvial depth near the damaged buildings. After the earthquake, at the beach adjacent to the hotel, a series of fissures appeared, 18" to 24" deep and 6" to 8" wide.

The earthquake caused damage throughout the area, the more severe damage apparently being concentrated toward the ocean. The collapse of quite a few low level buildings and the partial collapse of a multistory building indicates an intense ground motion. The earthquake intensity at Caraballeda, (about 40 miles east of the epicenter), was estimated as VIII on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The main structure of the Macuto Sheraton Hotel is a 120' high, 10-story reinforced concrete building, about 343' x 77' in plan, which is separated into 3 structural units by expansion joints. The two end sections contain the stairs, elevators, and services. The central section is very regular and should be relatively easy to analyze, since the cross section is similar for each bent.

The structural system is a shear wall frame on bottom "soft" stories with columns; it consists of 28" deep beams spanning in both directions between transverse wall piers. The transverse wall piers are 16'-5" long and vary in thickness from 18" at the lowest typical story to 10" at the 5th story. At the mezzanine ceiling level the wall piers are discontinued and supported by two 43" diameter columns at each wall end. The space needed for the transfer girder is 6 1/2' high and is used as utility space. The transfer beam is a very heavily reinforced tapered girder, 43" thick



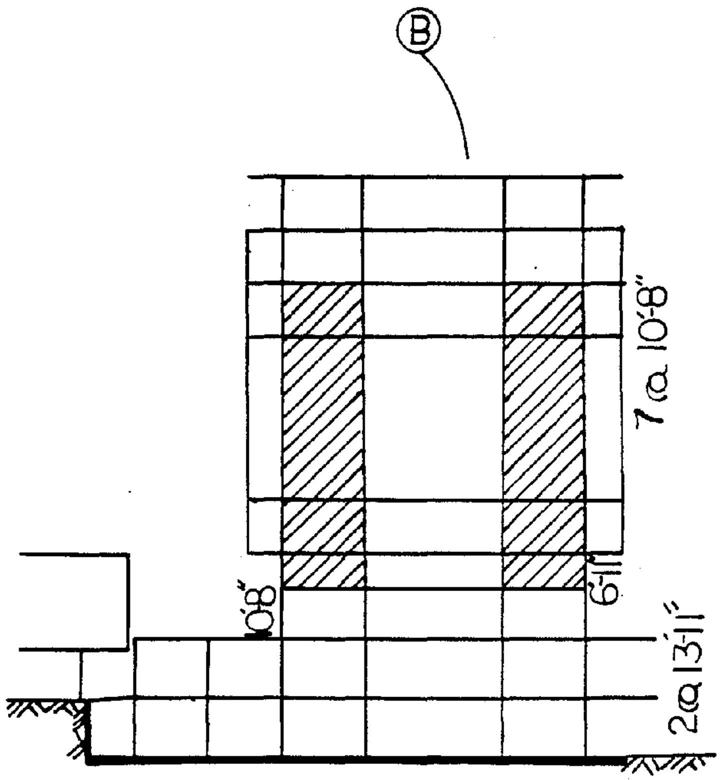


Figure 4-3: Macuto Sheraton Hotel:
(A) Typical plan.
(B) Typical transverse elevation.

at the mezzanine and 18" thick at the 1st floor. In the top two stories the wall piers are discontinued, too, and replaced by 10"-thick columns located above each wall end.

Typical floor construction consists of one-way joist slabs with tile fillers between joists, supported on the 28" deep beams connecting the wall piers. The slab cantilevers 7'-9" beyond the wall piers. In the mezzanine and lower floors two-way joists with tile fillers are used, supported on the 28" deep beams. Foundations are combined footings resting on concrete piles. The pile caps are interconnected with heavy tie beams in each direction.

Tile walls extend the length of the building on both sides of the central corridor. All interior partitions are tile except for the 16'-5" long concrete walls. Apparently, exterior walls are also tile walls.

DESIGN & CONSTRUCTION

The building was constructed in 1958 according to the then current Venezuela building code regulations, which approximate the UBC zone 2 requirements. Concrete used was of good quality, with a compressive strength probably higher than 3ksi.

The large round mezzanine columns were shown in the drawings with as many as 40 longitudinal 1" bars and 1/2" ties at 5 3/4". After the earthquake, inspection showed 34 1 1/4" smooth bars. The 1/2" tie bars were evidently meant to be lapped at 24" - measurements of 25" and 27" were made in the field.

DAMAGE

This building suffered severe structural and moderate nonstructural damage.

The immediately observable damage of greatest importance was the failure of the large 43" diameter columns below the mezzanine ceiling level. All columns on intermediate column lines were badly shattered throughout the length of the building, whereas on the outer column lines they were damaged to a smaller extent, the least damage being found in the end columns. All of the beams of the mezzanine floor were found to have double diagonal cracks, when spanning longitudinally, whereas vertical hinging cracks were found in the transverse beams. Pounding by a heavy concrete walkway cover caused severe damage to the entrance columns. At the utility space between the ceiling of the mezzanine and the floor of the 1st typical floor. several of the longitudinal 28" deep beams were cracked. At the ceiling of the mezzanine three steel trusses rested on a ledge of the entrance structure with about 4" of bearing, but with no anchor bolts. The movement between the two structures was enough to allow these trusses to slip off their bearings and crush to the floor.

The exterior of the hotel showed some damage to the tile walls. The lobby floor at the W-end broke away from the tile exterior wall. The exterior canopy in the front -a concrete slab supported on pipe columns - moved to the south causing some damage to the tile walls where pounding occurred. Minor concrete construction and tile walls at the elevator and stair portions were damaged. In the typical stories, partitions were broken, some ceilings fell, furniture was overturned. and many plumbing pipes broke, especially on the lower levels. Progressing up through the building. this type of damage diminished until about the 4th floor, where there were fewer cracks and even floor lamps did not overturn. Stair damage was considerable in the lower portion of the building.

CAUSE

The combination of columns for the bottom stories and shear walls for the upper stories resulted in a "soft-story" response of the building, most of the damage being concentrated at the lower levels. Since all columns were of the same size, it is perhaps significant that those with the largest vertical load suffered the heaviest damage. The upper part of the building behaved essentially as a shear wall structure, most of the nonstructural damage being concentrated at lower typical stories, which dissipated most of the remaining seismic energy.

Some recommendations are:

- A thorough study of the "flexible first story" concept of design should be made before it is attempted in a major earthquake resistant structure.

- Sufficient separation is needed in order to avoid pounding of structures.

- Provide support to longitudinal column reinforcing bars by adequate ties.

4.4 MENE GRANDE BUILDING

[Seed, 1970], [Fintel, 1967], [Hanson, 1969], [Skinner, 1968], [Sozen, 1968], [Steinbrugge, 1968].

GROUND MOTION & SITE

The Mene Grande Building is located in the metropolitan Caracas area. The earthquake intensity at Caracas, (about 30 miles southeast of the epicenter). was estimated as VIII on the Modified Mercalli Intensity scale. No pertinent information on local site conditions was found. Allowable soil pressure was specified at 8.2psf.

STRUCTURAL SYSTEM

The Mene Grande building is a modern office building. The typical floor plan is an "I" shape, 128' x 132' in overall dimensions, with two 39' x 132' apartment units forming the flanges and a 28' x 50' elevator and stair core forming the web of the "I". It has two basements for parking, a ground floor. and 15 elevated floors plus a penthouse. Ground story height is 13'-3 1/2" and typical stories are 11'-9 3/4" high.

The structural system is a moment resisting reinforced concrete frame. Floor girders and columns are spaced to form 5-bay frames in the flanges perpendicular to the web and 2-bay frames in the flanges parallel to the web. The floor slabs are supported by 6" x 12" joists with tile fillers spaced about 22" on center. Foundations consist of spread footings.

Tile walls are used for all four exterior ends of the building, the walls of the core, and around elevators, stairs and toilets in the connecting core. Partitions in the lower 10 stories are movable lightweight partitions. In the upper 5 floors, all office partitions are of tile.

DESIGN & CONSTRUCTION

The building was completed in early 1966. The lateral force formulae used in the design were the same as the Zone 2 requirements of the '64 UBC, which for this building approximated the MOP Normas. Base shear corresponded to an

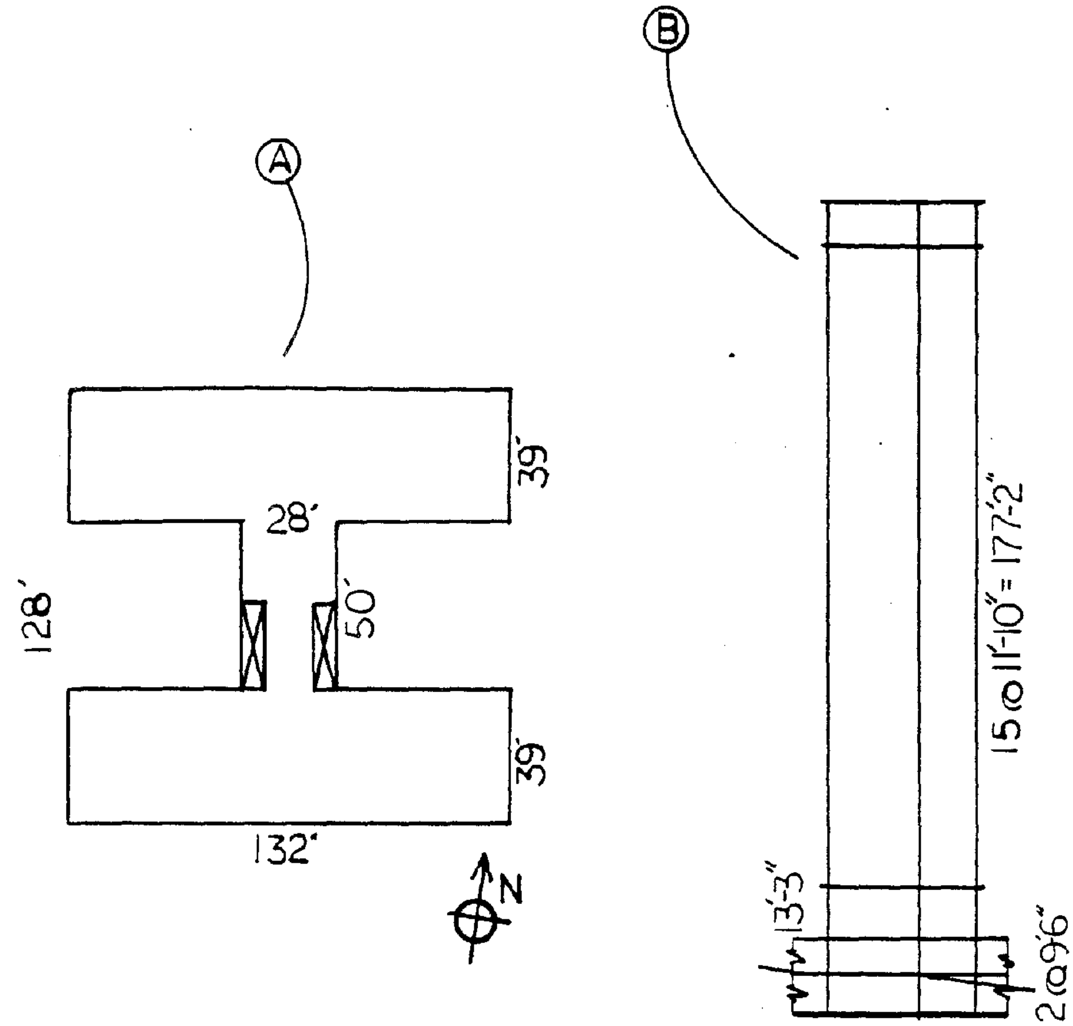


Figure 4-4: Mene Grande Building:
(A) Typical plan.
(B) Typical end-face elevation.

acceleration of 1.43%g. The design assumptions were the same as used in many areas of the world. All lateral forces were resisted by frame action. The tile walls were not assumed to carry lateral loads nor to affect the stiffness of the bents in any way.

Specified concrete strength was 4.2 ksi in the columns and 3.5 ksi in the floor system. The actual strengths exceeded these requirements. The original design contemplated intermediate grade steel with working stress 20ksi, but actual construction used Heliacero steel of 28ksi. The concrete frame was designed by the ultimate strength methods using the provisions of ACI 318-63. A review of the design after the earthquake indicated that the column sizes were selected conservatively, only 80% of the capacity being used.

During the construction, the engineers became aware of the latest recommendations on column reinforcing for lateral loads. Extra vertical steel was added at column to beam joints from the ground floor up and extra column ties were added from 2nd and 3rd floor up.

DAMAGE

This building experienced heavy structural and considerable nonstructural damage. The most obvious and spectacular damage was the failure of 7 of the 8 corner columns of the wings in the lobby story. Columns at the S-wing failed between the ground and second floor. The N-E corner column failed one floor higher; the N-W column was cracked at two floor levels, but did not fail; however, the beam at this corner was cracked. Most beams had hinging cracks in the 2nd to about the 6th floor. Above that level cracks decreased. Many of these cracks were quite small. Some floor cracks were found, especially where the wings connected with the core.

There was considerable nonstructural damage to the tile walls in the lower floors. The amount of damage decreased with height. Stairs were somewhat damaged at the lower floors, but were usable.

CAUSE

The solid infill panels of tile provided a shear-wall-like stiffening effect. The stiff tile walls, not considered in design, attracted the overturning forces to the bents which enclose them. These forces much greater than anticipated, caused the 8 corner columns to fail in compression, even though one would expect beams to yield first. The concurrent action of seismic forces in all 3 directions possibly also contributed to the overstressing of these corner columns. The stiffening effect of the tile cores made this tall building act as a cantilever with plastic hinging at the base and therefore causing most of the damage to lower levels.

Some recommendations are:

- Include the effect of solid nonstructural walls in the lateral design considerations.
- Give special attention to the design of corner columns.
- Yielding and failure should be initiated in and confined to beams rather than columns.

5. SAN FERNANDO, CALIFORNIA EARTHQUAKE OF FEBRUARY 9 1971. (71-2).

The San Fernando, California earthquake occurred at 6:01 a.m. (local time) on February 9. 1971, killed 58 persons and caused over 2,500 hospital treated injuries in the San Fernando Valley. Direct damage to buildings and other structures exceeded half a billion dollars.

The earthquake's epicenter was in the San Gabriel Mountains, its strong motion lasted for 12 seconds, and its magnitude has been assigned as 6.4 on the Richter scale. Most of the severe damage and major losses were along a narrow band of surface faulting that runs E-W on the valley floor.

REFERENCES: [Algermissen, 1973], [Campbell, 1976], [Chang, 1976], [Duke, 1971], [Duke, 1972], [Matthiesen, 1972], [Moran, 1973], [Murphy, 1973], [Steinbrugge, 1971], [Whitmanx, 1973].

5.1 AVENUE OF THE STARS BUILDING

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Avenue of the Stars building is located in the city of Los Angeles about 24 miles south of the epicenter.

Strong motion accelerographs located at basement, 9th floor and roof level recorded the maximum building accelerations: 0.170g, 0.110g, 0.070g, respectively, in the major axis direction (S.44.W) and 0.120g, 0.180g, 0.140g in the minor axis direction (N.46.W). The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

Subsurface conditions are generally fine sand throughout the depth of the foundations.

STRUCTURAL SYSTEM

This office building has 20 stories above and 4 parking levels below ground level. Plan dimensions are 110' x 240' for the building and 318' x 303' for the basement. A 2-story mechanical penthouse occupied about 20% of the roof area.

The structural system consists of moment-resisting steel frames in longitudinal direction (S.44.W) and braced steel frames in transverse direction (N.46.W). The building foundation consists of driven-steel I-beam piles under the main structural tower and spread footings elsewhere. The steel piles are 72' long and capped in groups of 3 to 10 piles. All pile caps are connected with 2' x 2' reinforced concrete tie beams.

Nothing is known about internal partitions, but the curtain wall, apparently, consists of glass between precast concrete facade panels.

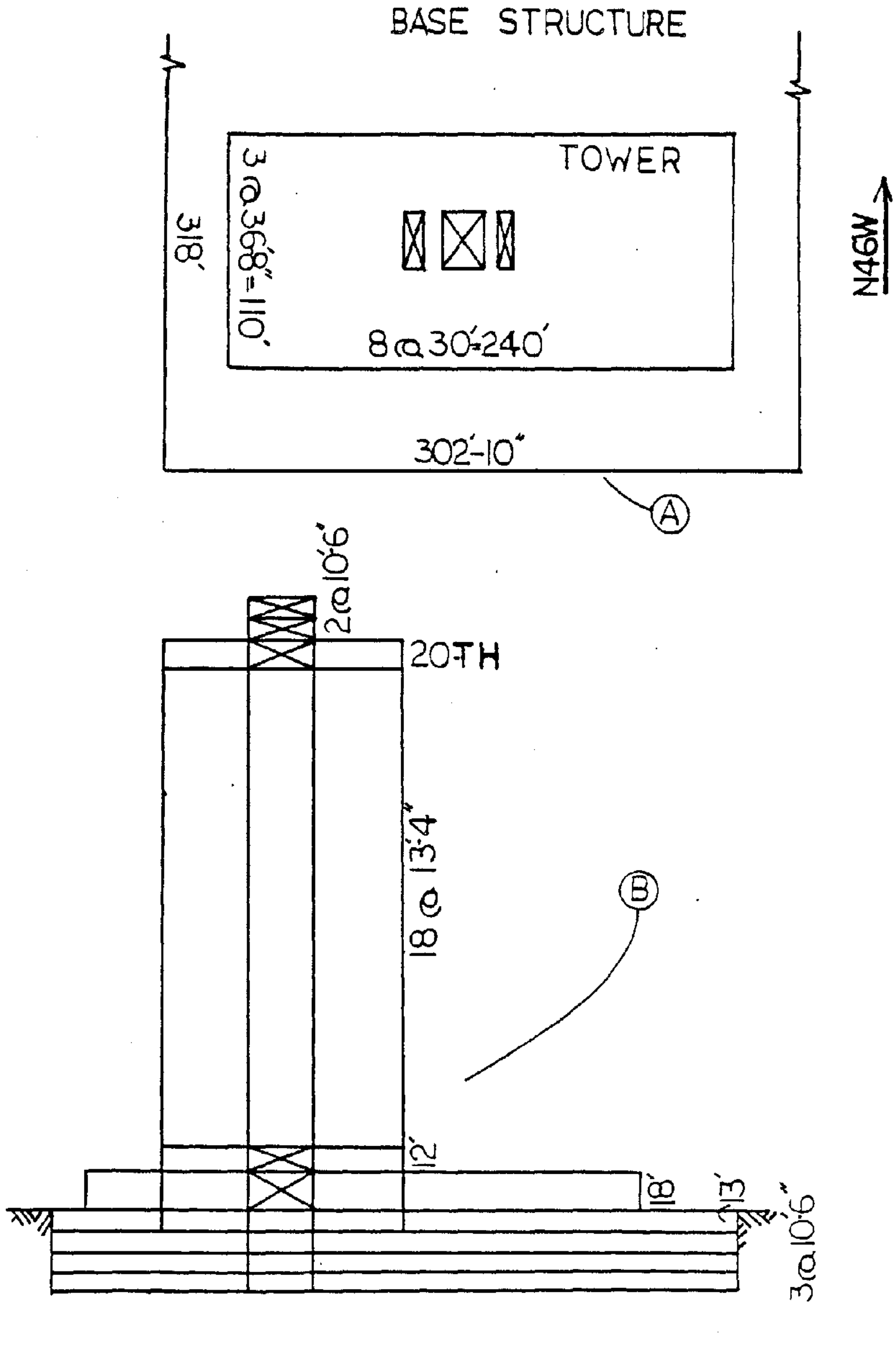


Figure 5-1: Avenue of the Stars Building:
(A) Typical plan.
(B) Typical elevation of N.46.W frame.

DESIGN & CONSTRUCTION

The building was constructed according to the Los Angeles building code. Lateral earthquake forces are resisted in the major direction by four A-36 ductile steel moment-resisting frames, and in the minor direction by five A-36 X-braced steel frames. Construction methods and quality appeared to be in accordance with specifications.

DAMAGE

No major structural and only minor nonstructural damage was experienced during the earthquake. Repair costs were estimated to be approximately $20,000 for painting over minor interior wall cracks in stairwells and offices, replacement of ceiling tiles, repairing the damage at the interfaces between the main tower and the low level connecting structures, and replacement of a small broken window.

CAUSE

The design and construction of the Avenue of the Stars building was such that no structural damage resulted from the earthquake. However, better estimation of earthquake movements effects on partitions and separation between structures would have resulted in less nonstructural damage. The ratios of the roof to basement Fourier modulus at the building's natural frequencies show that the X-braced framing system amplified the basement motion about 80% more than the moment-resisting frame. This increase was attributed to the existence of soil - structure interaction where the soil has nonlinear stiffness characteristics.

Some recommendations are:

- Design nonstructural elements for seismic motions.
- A better estimate of seismic separation requirements is needed.

5.2 BANK OF CALIFORNIA

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Bank of California Building is located in the Sherman Oaks district of Los Angeles, some 17 miles from the epicenter.

Earthquake motions were recorded by accelerographs located at the roof, 7th, and ground floor; the peak accelerations were: 0.277g, 0.262g, 0.230g, respectively, for the longitudinal (N.11 E), 0.188g, 0.255g, 0.155g for the transverse (N.79 W), 0.150g, 0.172g, 0.108g for the vertical component. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

Soil conditions are silt and silty sand with lesser deposits of clay and sand; water level is at 53'.

STRUCTURAL SYSTEM

This office building is a 174'-4" high, 12-story reinforced concrete structure with plan dimensions of 60' x 161' except at the 1st story, where they are 90' x 161'. Story heights are 13' except for the 1st story which is 16' high. A mechanical penthouse occupies some 30% of the roof area. A 1-story low-rise structure is attached at the E-side by means of a parapet.

The structural system consists of reinforced concrete frames. Two 8"-thick, 11'-6" long shear walls, each 2 stories high, rise along the W-face. A 10"-thick, 1-story high shear wall rises along the property line; this wall is not part of the tower, and supports the low-rise structure. In the transverse direction moment-resisting frames extend the full height of the structure along the exterior column lines, whereas the interior frames extend only up to the 3rd floor. Above this floor interior columns continue to the roof, while the beams framing into them are merely wide joists reinforced to carry only vertical loads. Similarly, in the longitudinal direction, interior frames are designed only for vertical loads. Exterior spandrel beams are offset

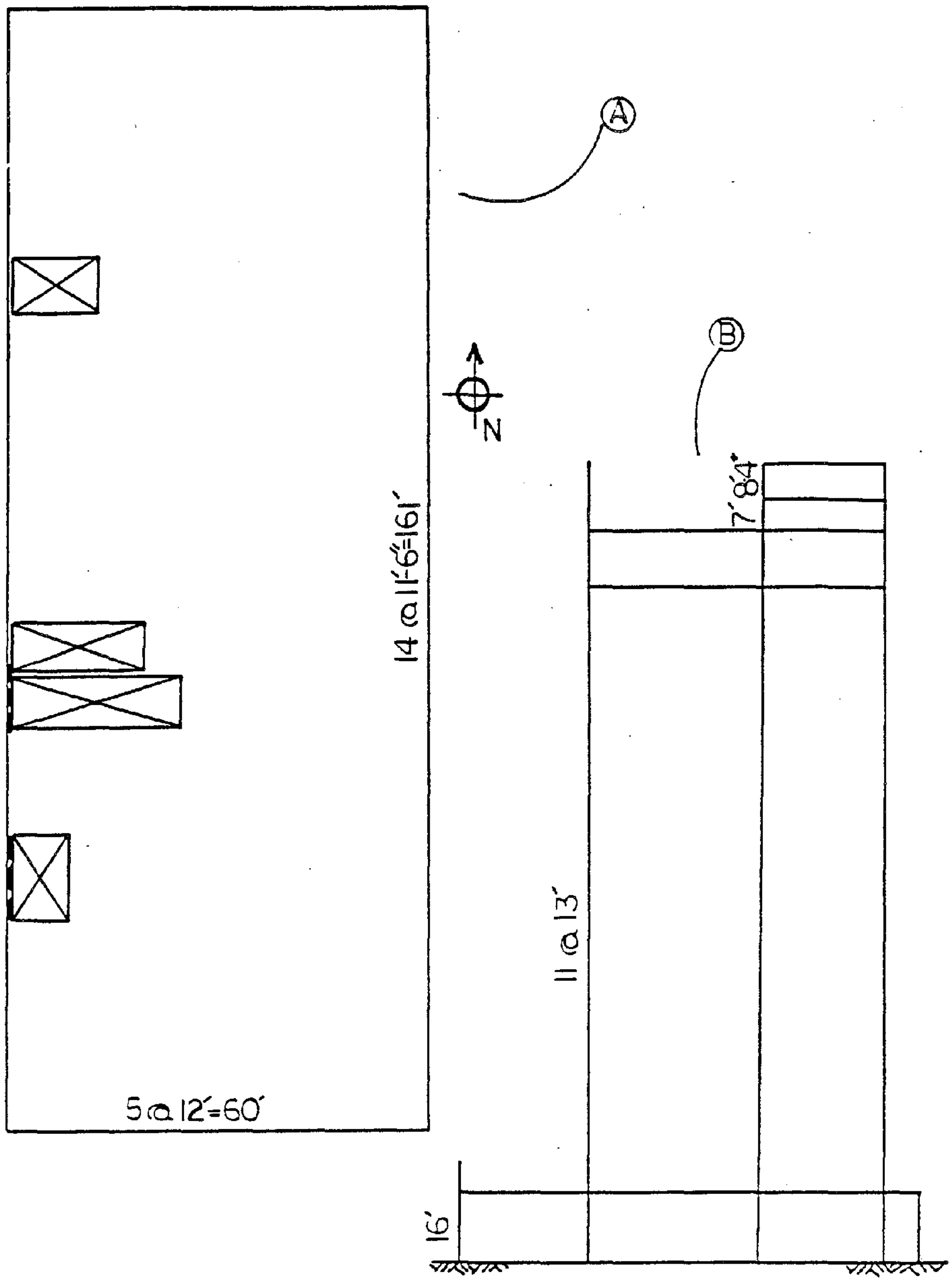


Figure 5-2: Bank of California:
(A) Typical tower plan.
(B) Typical transverse elevation.

by 3" from the exterior column face. These beams have a half-width nonstructural extension at the 3rd floor and at the 2nd floor they are set back 3' from the column line and frame into girders rather than columns. Typical floor construction consists of 4 1/2" slab on a 17"-deep pan-joist system, which spans from girder to girder. Because the upper soils are only moderately firm and tend to become weaker and more compressible when wet, pile foundations were provided. Piles are drilled and cast-in-place concrete 35'to 50' long.

Nonstructural elements consist of gypsum wallboard and metal stud partitions. Enclosure of the building consists of 2.5'-high metal stud supported cladding, except for a concrete wall at the 3rd floor; a continuous curtain wall between columns stands on top of them. The mechanical penthouse consists of concrete masonry block walls with a metal deck and a steel roof system.

DESIGN & CONSTRUCTION

The structure was designed in 1969 under the requirements of the '68 Los Angeles Building Code and completed in 1970 at a cost of $4 million. Except at the W-side, where two shear walls extend to the 3rd floor, lateral forces are resisted in each direction by moment-resisting concrete frames.

All floor slab and girder construction consists of lightweight concrete (3ksi) reinforced with Grade-40 steel. For the rectangular tied columns regular weight concrete (4ksi) and Grade-60 steel are used.

During construction the structural engineer followed standard inspection procedures; he assumed full responsibility for interpretation of drawings and for periodical inspections. A full-time deputy building inspector was also provided, as well as a part-time city inspector. However, the low-rise connection to the office tower columns at the 2nd floor level was poured monolithicly during construction; original design plans specified a seismic separation at this point. Another weak detail that facilitated construction was at the column - joist - girder connection; lightweight concrete in the girder and joist was partly poured into the column. In addition, main bottom reinforcement was found without confining stirrups within the column zone.

DAMAGE

The Bank of California experienced moderate structural and extensive nonstructural damage. Repairs totaled $44,000; $12,000 was spent on epoxy repair of damaged concrete elements.

Generally, visible structural damage was moderate and consisted of minor cracking and spalling of concrete. However, extensive cracking occurred at the exposed girder stubs at the 2nd floor due to the torsion induced. In the 3rd to 11th story columns spalled at the floor level, particularly on the inside face. A series of cracks was observed in the floor slabs around columns. Horizontal hairline cracks were observed at 3rd floor spandrels along construction joints. A cold joint that bonded the low roof structure to an E-face column, not designed to be integral, sheared free during the earthquake.

Nonstructural damage was distributed extensively between the 6th and 11th floor. Partitions running in the E-W direction pulled away from exterior columns; partition cracking also occurred in the stairwells at these levels. Ceiling tiles fell out. As the building displaced laterally, racking of the partitions and shortening of the hung ceiling took place. Damage also occurred to mechanical equipment and building contents. Potted plants and water bottles toppled. In the mechanical penthouse a compressor came off its mounting and a cooling tower support buckled.

CAUSE

The building resisted the earthquake by inelastic action. with local yielding of reinforcement, and with cracking and localized spalling of concrete. Calculated elastic story shears were 2.5 times greater than code values and overturning moments about 2.0 times. Architectural damage was related to interstory displacements. and some mechanical equipment and building contents were damaged by shaking alone. Improved detailing and construction procedures could have resulted in less structural and nonstructural damage.

Some recommendations are:

- Increase minimum code seismic force requirements if the equivalent static force method is retained.

- Members of lateral force resisting systems should be provided with ductile characteristics. Regular and lightweight concrete should not be mixed at column girder connections. Concrete at girder - column joints must be confined by hoop reinforcement.

- Girders and spandrels should frame into columns without offsets or other avoidable eccentricities.

- Seismic resistance can be improved if the configuration of the structure is regular.

- Expansion joints, flashings, partitions, and stairwells should be designed for seismic movements.

5.3 BUNKER HILL TOWER

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971],

GROUND MOTION & SITE

The 32-story Bunker Hill Tower is situated approximately 26 miles from the epicenter of the San Fernando earthquake.

Strong-motion accelerographs were located at the roof, 16th floor. and ground floor. These instruments showed a maximum ground acceleration of 0.143g in the longitudinal direction and maximum rooftop displacement of 20.34" in the transverse direction. Dynamic analysis of the building was correlated with the accelerograph readings through adjustment of damping percentages. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

Soil consists primarily of firm shale and sandstone. Material is weathered and fractured near the surface but becomes more consistent at deeper depths.

STRUCTURAL SYSTEM

The 336'-8" high 32-story tower measures 90' x 125' in plan. An adjoining plaza and parking garage on the N-side is separated at all levels by seismic separation joints. p There are offices in the lower three stories and apartments in the remaining stories. A mechanical penthouse covers about 30% of the roof area.

The structural system of this steel tower consists of moment-resisting perimeter frames and an internal "gravity load only" space frame. Exterior column spacing is 5'-9" along the perimeter, but interior beams frame into alternate columns. All exterior girder - column connections are welded and capable to develop the full moment capacity of the member. At the girder - column connections the column web is reinforced by stiff plates. Beams extending to the interior framing at 11'-6" spacing support a 5" reinforced concrete floor slab.

Foundations consist of either spread footings,

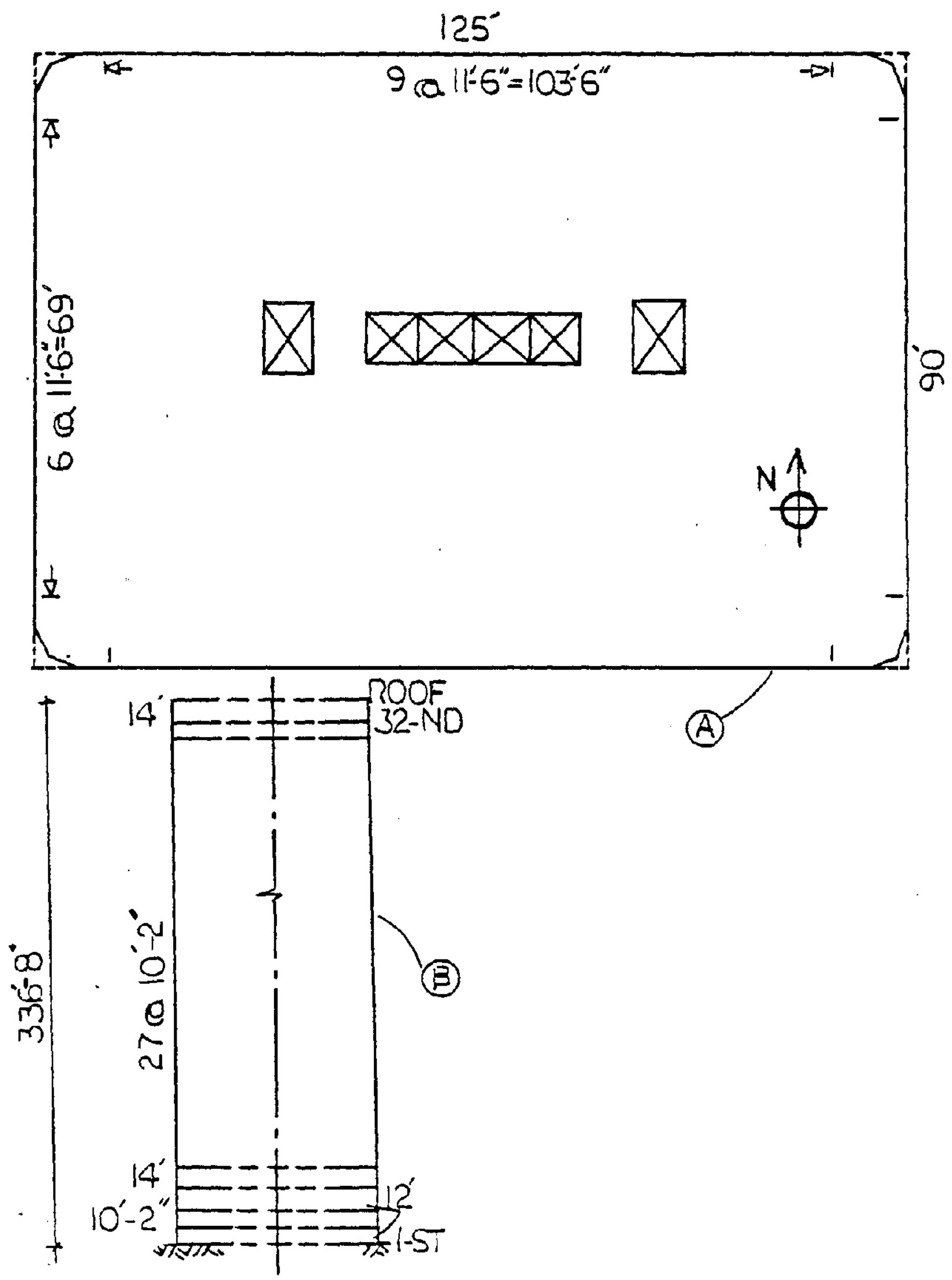


Figure 5-3: Bunker Hill Tower:
(A) Typical plan.
(B) Typical transverse elevation.

continuous strip footings. or grade beams on belled caissons. Individual spread footings support the interior columns. which are not part of the lateral force-resisting system. The perimeter columns are supported by a continuous foundation with caissons on the southern half of the building to avoid undue stress on a tunnel located nearby.

All beams and girders have spray-on fireproofing, and columns are encased in fireproof gypsum wallboard. Interior partitions of gypsum wallboard enclose the elevator shafts, stairwells, duct shafts, apartments, and rest rooms. Block walls are used in the lower levels, detailed such that the frame will move independently of the block walls. A seismically isolated curtain wall consisting primarily of glass between the columns encloses the building.

DESIGN & CONSTRUCTION

The building was designed in 1967 under the Los Angeles City Building Code and met the 1970 UBC requirements. Lateral forces in each direction are resisted by tube action of the moment-resisting perimeter frames. The interior space frame was designed to carry vertical loads only. All shapes were rolled except the corner columns, which were box sections of fabricated plate.

Reinforcing steel was Grade40 (40ksi). All concrete was of 3ksi compressive strength, lightweight above the first floor and regular below. All structural steel was A-36 (36 ksi) except for the box columns below the fifth floor which were A-441 (46 ksi). A full-time inspector was present and the building was constructed as designed.

DAMAGE

No damage to structural elements and only minimal damage to nonstructural components was observed. Nonstructural damage consisted of some cracking in partitions and ceilings. which required patching and painting. Also three windows were broken by objects falling against them during the earthquake. Four of the elevators were put out of service, two for several hours, one for a day, and one for two days. A cable had to be replaced on one elevator after it jumped its sheave and formed a kink.

CAUSE

The structural system performed very well with no structural failures and limited nonstructural damage. Dynamic analysis indicated some minor local yielding in a few girders and at the corner columns near the ground floor; but because most of the members were covered, this could not be verified. This behavior is very good considering that the computed elastic dynamic shears are 2.8 to 3.0 times and computed dynamic overturning moments 2.5 to 2.8 times larger than 1970 UBC minimum values with J=1.0.

Some recommendations are:

- Increase base shear and overturning moment for design if equivalent static method is retained.
- Use dynamic analysis with one or more hypothetical design earthquakes.
- Frames should be designed so that inelastic behavior is initialized and confined to girders.
- Architectural elements should be designed for seismic movement based on interstory drifts.
- Equipment and building contents should be secured against seismic movement.

5.4 CERTIFIED LIFE BUILDING

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Certified Life Building is located approximately 17 miles south of the epicenter of the San Fernando Valley earthquake.

Ambient vibration surveys of the building that were performed before the earthquake and shortly afterward, indicate that the fundamental building periods before, during, and after the earthquake were respectively: 0.81, 1.08, 0.90 seconds in the N-S, and 0.88, 1.13, 0.96 seconds in the E-W direction. Three strong-motion accelerographs were installed in the 14-story building. These were mounted on the ground floor, sixth floor, and rooftop. Maximum accelerations at the ground floor were 0.26g, 0.20g, and 0.10g in the transverse, longitudinal, and vertical directions respectively. Total maximum displacements of 0.23' transversely and 0.16' longitudinally were recorded at the rooftop level. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

Soil at the site consists of moderately soft silty sand and clay. At about 30 feet the soils become firmer and ground water was encountered between 26 and 30 feet below grade.

STRUCTURAL SYSTEM

This 164' tall reinforced concrete structure consists of a 14-story tower with a three story setback. Adjacent to the setback was a parking garage separated from the main structure by a 3" seismic gap. The tower measures 76' x 156' in plan and the setback 121' x 179'-6". A mechanical penthouse occupies 40% of the roof area.

The structural system of the tower consists of shear walls at both transverse faces, spandrel frames along the longitudinal faces, and a core in the center. Interior columns, spandrel frames, core, and shear walls support 8" concrete flat slabs. The 12" shear walls are continuous to the ground floor.

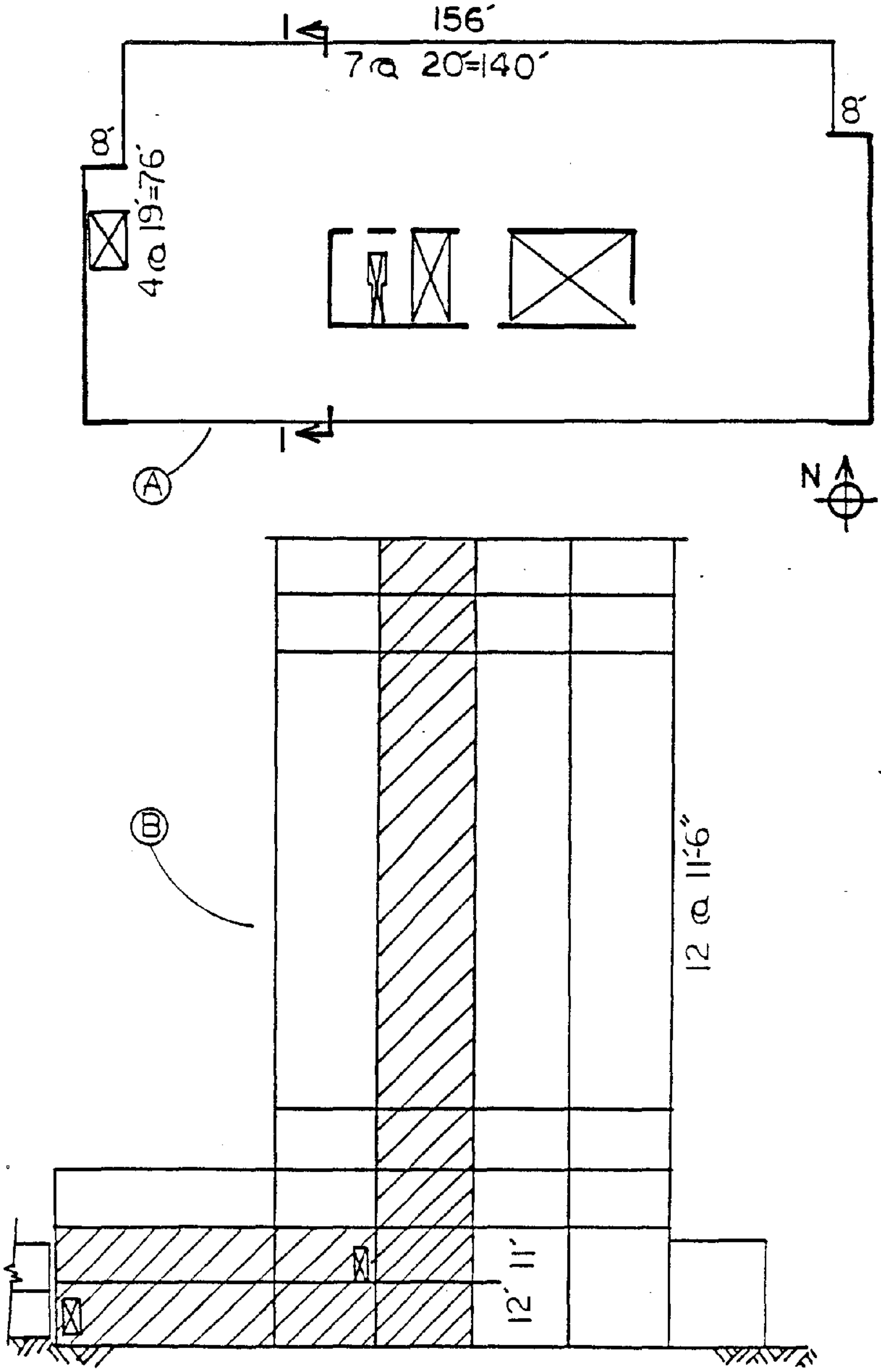


Figure 5-4: Certified Life Building:
(A) Typical plan.
(B) Elevation 1-1.

This building rests on 245 cast-in-place concrete piles. The 45 piles located under the central tower are battered. The footings, supported by piles, are connected by tie beams or grade beams, where wall support is needed.

On the bottom two floors, 12" block walls enclose the setback and portions of the tower base. Glass curtain walls enclose the exterior spandrel frames. Apparently, partitions are drywalls and plaster walls enclose the stairwells.

DESIGN & CONSTRUCTION

The building was designed under the lateral force provisions of the 1964 Los Angeles City Building Code. From the foundation level up, the structure was designed to resist all lateral forces by the shear walls and the cores only (K=1.33). Any resistance provided by the spandrel frames and the flat slab - column system was neglected.

For analysis, the three story base was considered as a separate structure with tower shears and overturning moments added to the design forces in the structure. Efforts were made to minimize torsional response under lateral loading. This was accomplished by proportioning wall sizes and spacing to minimize eccentricities between center of floor mass and center of lateral building stiffness. Overturning moments were reduced using J-factors. a practice that has since been abolished.

Reinforcing steel is 40 ksi in spandrels and slabs, and 60 ksi in columns. Walls and columns are of regular stone concrete (3ksi and 4ksi), and spandrels and slabs are lightweight concrete of the same strength. All construction procedures and workmanship complied to applicable code requirements.

DAMAGE

This building suffered only minor damage. About $30,000 was spent on mechanical repairs and $2.000 on repairing cracked drywall and plaster. All other damage was considered as normal building maintenance.

Hair line cracks were observed in the exterior shear walls and over door lintels of core walls at lower levels. No other structural damage was observed.

Nonstructural damage was limited to cracks in drywall partitions and plaster walls around stairwells caused by interstory displacements. The 12th level suffered cracked wallpaper, overturned water coolers, and fallen drapes. In the 4th and 9th story, mounted bookcases broke free, ceiling tiles cracked at the 5th and 8th story and some ceiling tile fell in the bank area.

The motor on the HVAC chiller on the rooftop burned out, and fuses in the elevators blew out, all attributed to the earthquake.

CAUSE

Maximum shears determined from dynamic elastic analysis exceeded design and code values by 30% to 70%, depending on direction. The overturning moments exceeded code requirements by 30% to 115% using the J-factor reductions. Without these reductions the moments exceeded code values by 30% transversely and were 10% below the code longitudinally. A significant portion of the maximum building response was in the 2nd mode. Though pierced by mechanical ducts, stair and elevator doors, the cores performed well due to the large amount of reinforcing around these openings. The general performance of the structure was linear elastic with some yielding in the shear walls due to peak overturning moments.

With the offset, this building would be vertically irregular according to ATC 3-06 [ATC, 1978]. However, the irregularity was considered in the analysis.

Some recommendations are:

- Investigate whether the overturning moment reduction factor, J, should be eliminated from seismic codes.

- Recorded building periods, when compared to calculated values, indicate that foundation characteristics should probably be included in the mathematical modeling of the building.

- Review code practice to see if modifications should be made to account for the higher mode effects in medium-rise structures.

5.5 HOLIDAY INN, MARENGO STREET

[Murphy, 1973], [Blume71, 1971], [Blume73, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Holiday Inn at Marengo street, Los Angeles, is located at about 26 miles south of the epicenter of the San Fernando earthquake.

Strong motion accelerographs located at the roof, 4th floor and 1st floor (ground level) recorded the following peak accelerations: 0.426g, 0.261g, 0.147g, respectively, in the transverse direction (S.52.W), 0.247g, 0.199g, 0.139g in the longitudinal direction (N.38.W), and 0.140g, 0.109g, 0.086g in the vertical direction. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

Geological source data indicate that the site lies on older alluvium. The underlying soil is primarily clayey silt, sandy silt and silty fine sand.

STRUCTURAL SYSTEM

The Holiday Inn is a 7-story reinforced concrete structure. about 66' high. Typical plan dimensions are approximately 63' x 150'. A mechanical penthouse covers about 20% of the roof area.

The structural system consists of flat slabs on interior columns with column - spandrel beam perimeter frames. Except for two small canopies at the 1st story the plan configuration is the same for each story. The typical column spacing is 20' in the transverse and 19' in the longitudinal direction. Spandrel beams run along the perimeter of the flat slabs.

The foundation system consists of 35' long friction piles centered under the main building columns. A grid of tie beams and foundation beams connects all pile caps. The 1st floor is a slab on grade over about 2' of compacted fill.

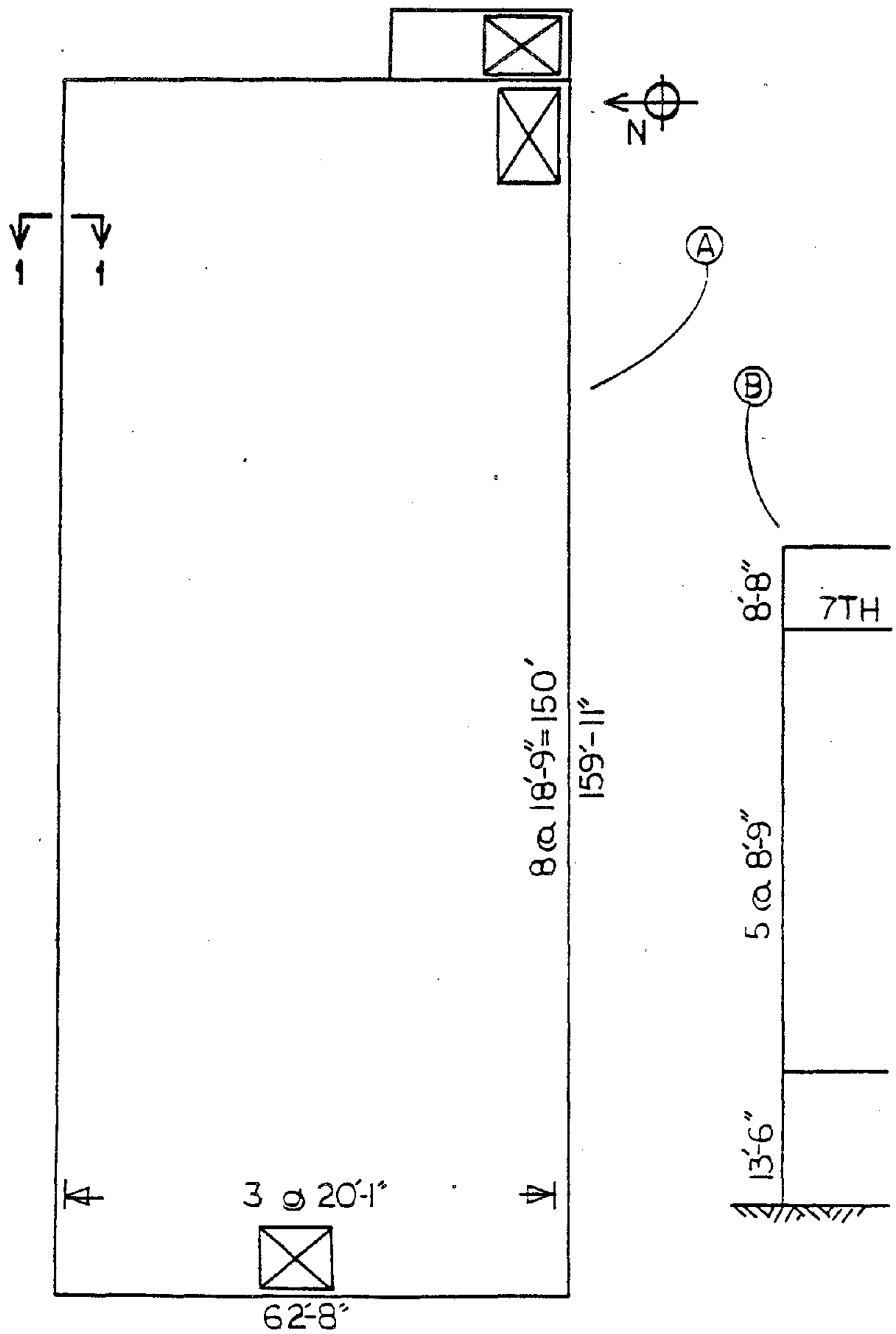


Figure 5-5: Holiday Inn, Marengo Street:
(A) Typical plan.
(B) Typical transverse elevation (1-1).

Interior partitions are gypsum wallboard on metal studs. Cement plaster, 1" thick, is used for exterior facing at each end of the building. Double 16" gauge metal studs support the cement plaster. The N-side of the building along the back column line has 4 bays of brick masonry walls located between ground and 2nd floor and separated by expansion joints from the columns and spandrels.

DESIGN & CONSTRUCTION

Designed in 1965 and constructed at a cost of about $1.3 million this building is essentially identical to the Holiday Inn at Orion Avenue. It meets the requirements of the Los Angeles building code at that time.

Lateral forces are resisted by both the column - slab interior frames and the column - spandrel beam exterior frames. The additional stiffness provided by the spandrel beams creates exterior frames that are twice as stiff as the interior frames. All interior partitions and exterior brick walls were considered nonstructural. The structure is constructed of regular weight reinforced concrete. Cylinder strength is 5ksi for the 1st story columns, 4ksi for the 2nd story columns, beams and slabs, and 3ksi for all upper stories. Reinforcing steel is Grade 40 for beams and slabs, but Grade 60 for columns.

The building apparently was built according to the specifications.

DAMAGE

This building suffered only minor structural, but considerable nonstructural damage. Repair of the damage cost approximately $95,000. Structural repair amounted to roughly $2,500 of that figure; the remainder was nonstructural damage.

The structural repair was required at the intermediate stair landing between the 1st and 2nd floors at the S-E corner column, where the elevator shaft is. Cracking and spalling occurred at the slab and beam column joints.

Nonstructural damage occurred in almost every guest room. Whereas drywall panels had to be replaced in the Orion Avenue structure, the cracks in the Holiday Inn at Marengo street were smaller and could be repaired. Only 9 bathtubs and no water closets had to be replaced. Windows and doors in every guest room required alignment and adjustment. Some sliding windows tilted in their frames, but no glass was broken. Cracks were observed in the exterior plaster.

CAUSE

During the earthquake the structure responded at amplitudes that exceeded the elastic limits of a substantial number of girders. Earthquake forces, calculated by elastic dynamic analysis, exceeded prescribed code minimums by a factor of 4 to 5. Interstory displacements exceeded 1/2". Although none of the wall elements was designed as a part of the lateral force resisting system, they did contribute in varying degrees to the stiffness of the structure. This accounts for the moderate amount of nonstructural damage.

Some recommendations are:

- The effect of nonstructural elements should be included in the lateral force design criteria.

- Nonstructural elements should be designed for seismic motions.

5.6 HOLIDAY INN - ORION AVENUE

[Blume , 1971], [Blume , 1973], [Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Holiday Inn at Orion Avenue, Los Angeles, is located about 13 miles south of the epicenter of the San Fernando earthquake.

Strong-motion accelerographs located at the roof, 4th floor, and ground floor showed 40 seconds of motion. Maximum accelerations at ground level were 0.251g transversely, 0.134g longitudinally and 0.180g vertically. Maximum displacement at the roof was 7.7" transversely, and 5.5" longitudinally. The earthquake intensity at the site was specified as VIII on the Modified Mercalli Intensity scale.

The soil of this site, in the center of the San Fernando Valley, consists of fine sandy silts and silty fine sands from alluvium deposits.

STRUCTURAL SYSTEM

The Holiday Inn is a 66' high, 7-story reinforced concrete building measuring approximately 150' x 63' in plan. A penthouse with mechanical equipment covers approximately 10% of the roof area.

The structural system consists of flat slabs on interior columns with column-girder perimeter frames. Except for two small one-story canopies on the ground floor, the plan configuration is similar in each story. Rectangular tied columns are spaced at 20' transversely and 19' longitudinally and support 8 1/2" thick flat slabs. Spandrel beams run along the perimeter of the flat slabs. The foundations consist of cast-in-place concrete piles supporting individual pile caps connected with grade beams. A slab on grade over about 2' of fill forms the ground floor. There are no basements.

Interior partitions are generally gypsum wallboard on metal studs. Cement plaster, 1" thick is used at each end

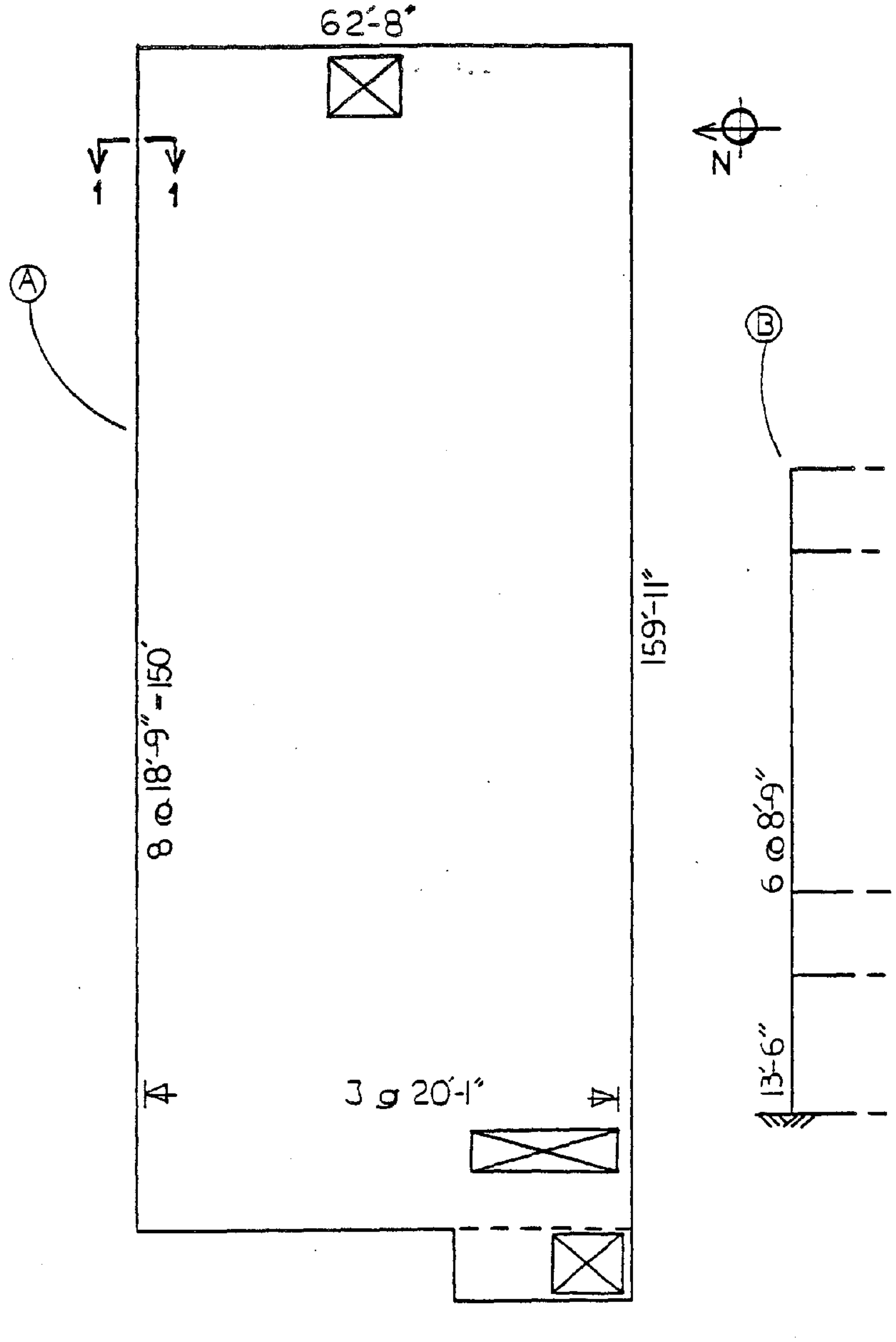


Figure 5-6: Holiday Inn, Orion Avenue:
(A) Typical plan.
(B) Typical transverse elevation (1-1).

of the building and in the stair and elevator shafts. The plaster is supported by a metal stud frame. In the 1st story, there are four bays of brick wall with 1" joints between the walls and columns and 1/2" joints between walls and underside of second floor spandrels. The remainder of the building enclosure is glass.

DESIGN & CONSTRUCTION

The structure was designed in 1965 to specifications equivalent to the 1967 UBC. Lateral forces are resisted by both interior column-slab frames and exterior column-spandrel frames. The stiffness of the exterior frames is approximately twice that of the interior frames. Any structural contribution from the partitions and the eccentrically placed brick exterior walls has been neglected.

This was a typical design for Holiday Inns; another Holiday Inn, about 16 miles southeast on Marengo St., has the same details and floor plans. No mention is made of design analysis techniques, but they are assumed to be the static equivalent load method.

Reinforcing steel was 60 ksi in the columns and 40 ksi elsewhere. Regular weight concrete was used. Concrete strength was 5ksi for 1st story columns- 4ksi for 2nd story columns and slabs, and 3ksi for all other stories. Construction methods and quality appeared to be within specifications.

DAMAGE

This building suffered only minor structural, but extensive nonstructural damage. Repair of structural damage cost less than $2,000. Repair costs for nonstructural damage were not available.

Structural damage was limited to cracking in a 2nd floor beam-column joint, above a brick wall, and some spalling at column pour joints underneath beam and spandrel connections.

Nonstructural damage was extensive. Most of the damage occurred in the 2nd and 3rd stories, while the damage was less severe in the higher stories. Gypsum wallboard buckled and cracked, 45 bathtubs and 12 toilets had to be replaced. Tile had to be patched, grouted, or replaced in over half the bathrooms. No windows were broken, but many needed caulking and alignment, and many doors needed adjustment. Architectural concrete spalled, where it was attached to structural concrete columns on the 1st floor. Exterior cement plaster cracked and spalled.

CAUSE

Elastic dynamic analysis showed that the structure resisted substantially higher seismic forces than required by code. Maximum base shears were calculated to be four to five times code requirements and overturning moments were six to nine times greater than code values. Most of the girders went beyond their elastic limits along with some of the exterior columns in the transverse frames. These columns experienced moments and shear forces high enough to cause yielding, which may have redistributed the forces. Shears were within 200 psi of the ultimate capacity of reinforced concrete columns.

Modeling and analysis of the building gave a close correlation between actual damage and predicted damage under similar loading. Participation of nonstructural elements was evidenced by the period increase after 6 seconds of motion. At this moment all resistance due to interior walls and curtain walls was overcome and the frame was beginning to yield.

Some recommendations are:

- Use more sophisticated analysis as part of the design procedure.
- Include the effect of nonstructural elements into the lateral load resistance calculations.

5.7 HOLY CROSS HOSPITAL

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Holy Cross Hospital, is located in Los Angeles, approximately 9 miles SW of the epicenter of the San Fernando earthquake. Across the street is the Indian Hills Medical Center, which suffered damage amounting to 10% of the original building cost. There were no accelerographs in the building, but maximum ground accelerations were estimated to be 0.4g to 0.5g. The earthquake intensity at the site was specified as IX on the Modified Mercalli Intensity scale.

Soil reports indicate nonuniform alluvial deposits of mixed sand, silt, and clay. The upper soils are low in density and shear resistance, but become stronger with increasing depth. Though fairly close to the epicenter, no ground fissures or upheaving was observed in the area.

STRUCTURAL SYSTEM

This reinforced concrete building consists of a 7-story tower. 89' x 184' in plan, a 3-story wing to the north, and 1-story wings at each end. A single story basement extends under the tower.

The structural system is a reinforced concrete space frame with irregularly distributed 8" shear walls running in both directions. Most of the shear walls are discontinuous from top to bottom. Joists 14" deep supporting a 3"-slab frame into beams of the same depth, spandrels on the exterior column lines, and shear walls along the interior column lines. Spandrels at the transverse ends of the building were located eccentrically flush against the inside face of the columns. Tower, 1-, and 3-story wings are of similar construction. Cast-in-place friction piles with tie beams between footings were used for the foundation. The basement was a 4" thick slab on grade.

Interior partitions consisted of steel studs and plaster, ceilings of suspended lathing and plaster. No mention is made of the cladding system of this building.

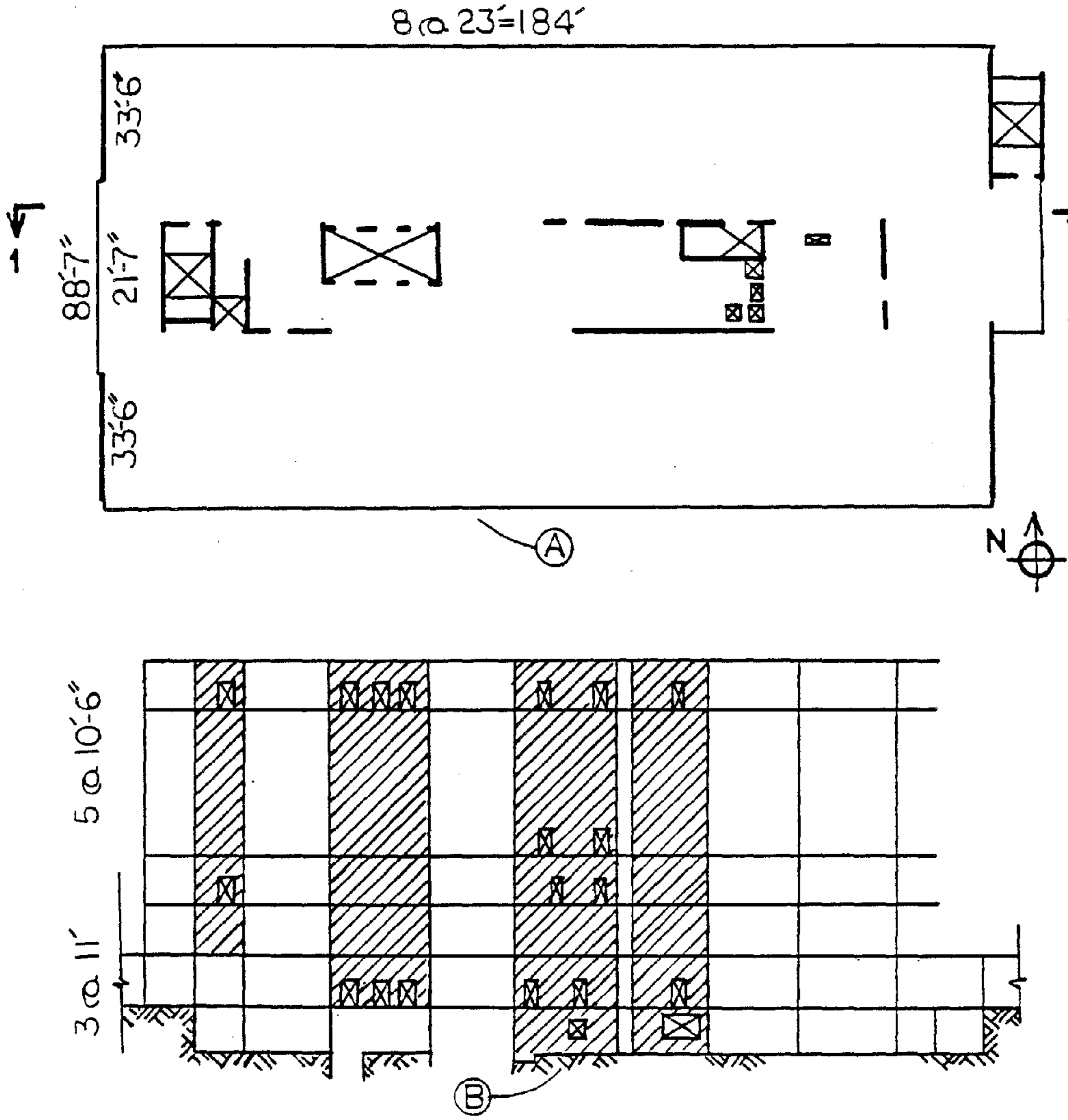


Figure 5-7: Holy Cross Hospital:
(A) Typical plan.
(B) Elevation 1-1.

DESIGN & CONSTRUCTION

This building was designed in accordance with the 1959 Los Angeles City Building Code as a shear wall building. The frame was designed for gravity loads only. At the points of the discontinuity in the shear walls, reliance was placed on the joist-slab system to act as a diaphragm to transfer shear. The layout and design is not unusual, but the discontinuities create a complex lateral force resisting system. This building was designed to accommodate three additional stories at a later date.

Reinforcing steel used was 20 ksi and 33 ksi for the structural steel in the canopies and penthouse. Lightweight concrete (3ksi) was used in all floor systems and regular rock concrete (5ksi) was used in columns and shear walls. Lightweight concrete intruded into columns and walls at slab levels, where also vertical reinforcement splices were located. This layer of weaker concrete was considered in design by using an allowable stress in the shear walls based on the lower concrete strength. The construction practices apparently met code requirements at the time of construction.

DAMAGE

This building suffered major damage. The rehabiliation of the facility required the removal of the top two stories of the main tower. Repair costs amounted to 48% of the replacement cost. This includes a reduction of 20% in floor area due to the removal of the top two floors. Severe structural damage occurred in shear walls, floor systems and columns, primarily in the lower four stories. Excessive diaphragm loading cracked floors on the 2nd, 3rd, and 4th levels along the west face.

At the 3rd floor, west shear walls cracked at the joints between lightweight and regular concrete, and a west wall end-column shattered. The east shear wall failed at the 4th floor at the location of the lightweight concrete layer and the splice of the column reinforcement acting as vertical flexural reinforcement at the wall ends. The splices failed and the lightweight concrete crushed. A wall around the east stairwell failed at the pour line of the first floor, and some light vertical reinforcing ruptured. Many longitudinal shear walls showed X-cracking over door openings permitting displacements, which left some doors inoperative.

The large inelastic deflections of the shear walls caused columns to carry seismic moments and shears as well as the design axial load. Column cracking was most severe in the 2nd, 3rd, and 4th story. In the longitudinal direction many spandrels crushed in flexural compression and columns cracked, primarily in the 4th story. Framing members between the tower and the 3-story wing were cracked indicating independent motion.

No record of nonstructural damage was found but it must have been major.

CAUSE

Most of the damage was the result of poor detailing and design practices. Lightweight concrete in floor pours resulted in some shear wall failures, even though allowances were made for strength differences. A staggered splice or more confinement (wall column ties) would have helped in preventing the splice failure in the east wall.

Lack of enough shear reinforcement in lintels over wall openings was compounded by holes for mechanical ducts. The shallow lintels acted as coupling beams between shear walls and should have been designed as such. The failure of the floor system in many locations can be attributed to the combination of inadequate diaphragm capacity with discontinuous shear walls. The columns designed to carry vertical loads only, had to resist high shears and moments resulting from inelastic shear wall deformations.

The 3-story north wing was of sufficient size to cause dynamic irregularities. Separation of this wing with an independent structural system and seismic gaps would have improved the buildings performance.

By current standards, this building would be deficient in a number of areas. There are discontinuities in the shear walls and diaphragm system. According to ATC 3-06 [ATC, 1978], the plan and vertical geometry of the building must be classified as irregular. The building did satisfy the basic requirement of not collapsing under seismic loads, but inspection indicates that damage would have been much greater, if the ground motion had lasted over a longer interval.

Some recommendations are:

- Vertical load resisting columns should be designed to resist shears produced by the ultimate moment capacity of the sections and column ties should be continued to the ends.

- Avoid lightweight concrete intrusions from floor systems in shear walls.

- Individual wings should be separated by seismic gaps.

- Reinforcement splices should be staggered.

- Increase shear reinforcing in wall elements over openings as well as in diaphragms, and particularly at the bottom of discontinued shear walls.

5.8 INDIAN HILLS MEDICAL CENTER

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Indian Hills Medical Center, in the city of Los Angeles, is located approximately 9 miles southwest of the epicenter of the San Fernando earthquake. There were no accelerographs at or near the building, but estimates put the maximum ground acceleration at 0.40g to 0.50g. Ground motion was severe enough to demolish some one- and two-story buildings in the neighborhood and to cause major damage to the 7-story Holy Cross Hospital across the street. The earthquake intensity at the site was specified as IX on the Modified Mercalli Intensity scale.

Underlying subsoils vary with sands, clays, silts, and combinations of each. Test borings to a depth of 50' showed no ground water.

STRUCTURAL SYSTEM

This 7-story reinforced concrete building is about 101' high and measures approximately 80' x 171' in plan. A penthouse is located on the roof.

The structural system consists of reinforced concrete transverse frames and shear walls that are regularly distributed along the perimeter of the building. In general, the configuration is regular and symmetrical in plan except for an offset at the S-face. Shear walls are located in the end bays in longitudinal direction, and in the center bays and at the offset in the transverse direction.

Beams running transversely at 19' spacing support 6 1/2" floor slabs and frame into columns or exterior shear walls. The typical shear wall is 8" thick and reinforced with #5 bars at 18" each way. The ends of shear walls are designed as columns. There are no spandrel beams along the perimeter of the building, rather the slabs are additionally reinforced. Floor system and story height are similar from the 2nd to 6th story. However the 1st story is higher and contains a suspended mezzanine floor.

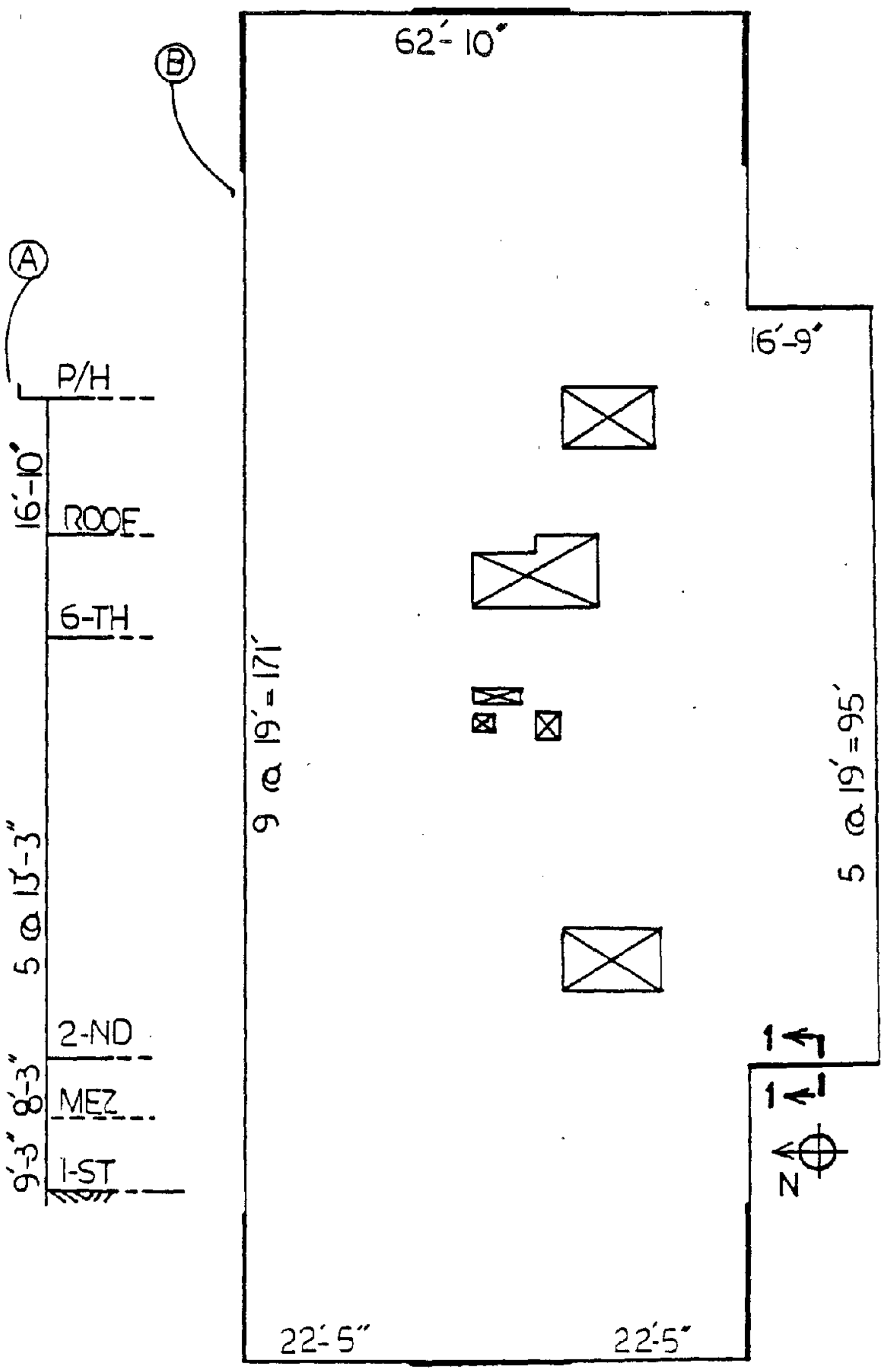


Figure 5-8: Indian Hills Medical Center:
(A) Elevation 1-1.
(B) Typical plan.

Foundations consist of cast-in-place concrete piles; all pile caps are connected by grade beams. There are no basements, and the ground floor is a 4" reinforced concrete slab on a gravel base.

The building enclosure is a light curtain wall construction, except where shear walls are located. Interior partitions consist of gypsum wallboard on metal studs.

DESIGN & CONSTRUCTION

The structure was designed under the 1966 edition of the Los Angeles City Building Code using K=1.0. Based on analysis using the 1971 Los Angeles Building Code, the base shear is 0.045 W in each direction (W=dead load on all floors). If the base shear were to follow the distribution set up by code criteria, it would require a base shear of 2 to 2 1/2 times the code shear to reach the ultimate capacities of the shear walls.

All slab and beam concrete is lightweight (3ksi) and all other concrete is regular rock concrete. The concrete strength of columns and walls below the 2nd floor and mezzanine slab is 5ksi and above that level is 3.75ksi. A-431 and A-432 (60ksi) grade reinforcing was used in all columns and for main reinforcing in beam and slabs. All other reinforcing was 40 ksi. There were no unusual features about construction or quality control. A weak detail were the points, where the lightweight concrete slab extended through the regular concrete of shear walls and columns.

DAMAGE

This building suffered moderate damage which amounted to approximately $150,000 or 9% of the original cost.

All shear walls in the lower levels developed X-cracks indicating high shear stresses. Some of the shear walls cracked at construction joints at floor lines reflecting the intrusion of lightweight concrete. The ends of shear walls, although designed as columns, suffered crumbling at splices, due to shear and axial loadings. In places, where the shear walls tied into concrete girders, damage was found at the connections. At least one interior column-girder connection suffered damage. Several shear walls offset transversely at the floor line, indicating that "the reinforcing stepped out also". From the 2nd floor to the penthouse, the building suffered more than 80 separate incidents of damage ranging from hairline cracks to major spalling. No mention was made of nonstructural damage to curtain walls or mechanical systems.

CAUSE

The building behaved within the design parameters which met or exceeded the governing building code. but the distribution of damage indicates some weaknesses that can be improved upon in future code revisions. Most immediate is the complications presented by the intrusion of lower strength lightweight concrete into higher strength stone concrete.

Crushing and spalling of lap-splice areas in columns and shear walls shows a problem involving confinement and splicing methods. Another area in question is the action of shear walls framed by transverse girders versus shear walls framed into the slab only. Special consideration must be given in the design of this important detail.

By ATC 3-06 guidelines [ATC, 1978], this building is geometrically irregular in plan due to the dimensions of the offset relative to the overall building, and vertically irregular at the mezzanine level.

Some recommendations are:

- Avoid intrusions of light weight concrete in vertical load carrying elements.

- Develop improved design recommendations for lap splices.

5.9 KAJIMA INTERNATIONAL BUILDING

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Kajima International Building is located in the Los Angeles basin about 26 miles southeast of the epicenter of the San Fernando earthquake.

Strong motion accelerographs mounted on the floor slab at the basement, 8th floor and roof level recorded peak accelerations of 0.110g, 0.207g, 0.180g, respectively, in the N-S (longitudinal) direction, 0.137g, 0.184g, 0.170g in the E-W (transverse) direction, and 0.056g, 0.078g, 0.193g in the vertical direction. From ambient vibration surveys the fundamental periods of the building before, during, and after the earthquake were found to be 1.80, 2.92, and 2.10 seconds, respectively, in the N-S direction, and 1.80, 2.80, and 2.15 in the E-W direction. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

Soils at the site consist of fill deposits and sandy overburden, which in turn are underlain by silt stone. The overburden soils are firm to a depth of about 15'; below this depth overburden soils and siltstone are firm to very firm. Ground water was encountered at a depth ranging from 23' to 28'.

STRUCTURAL SYSTEM

The Kajima International Building consists of a 202' high, 15-story office tower measuring approximately 66' x 96' in plan, and an adjacent 3-story parking structure. The two structures have a seismic separation that starts at the common basement level floor slab. A mechanical penthouse occupies 20% of the roof area.

The structural system of the tower is a 3-D moment resisting steel space frame. Full moment-resisting connections are provided between beams and columns. All columns are anchored to the top of spread footings. The floor system consists of lightweight concrete slabs.

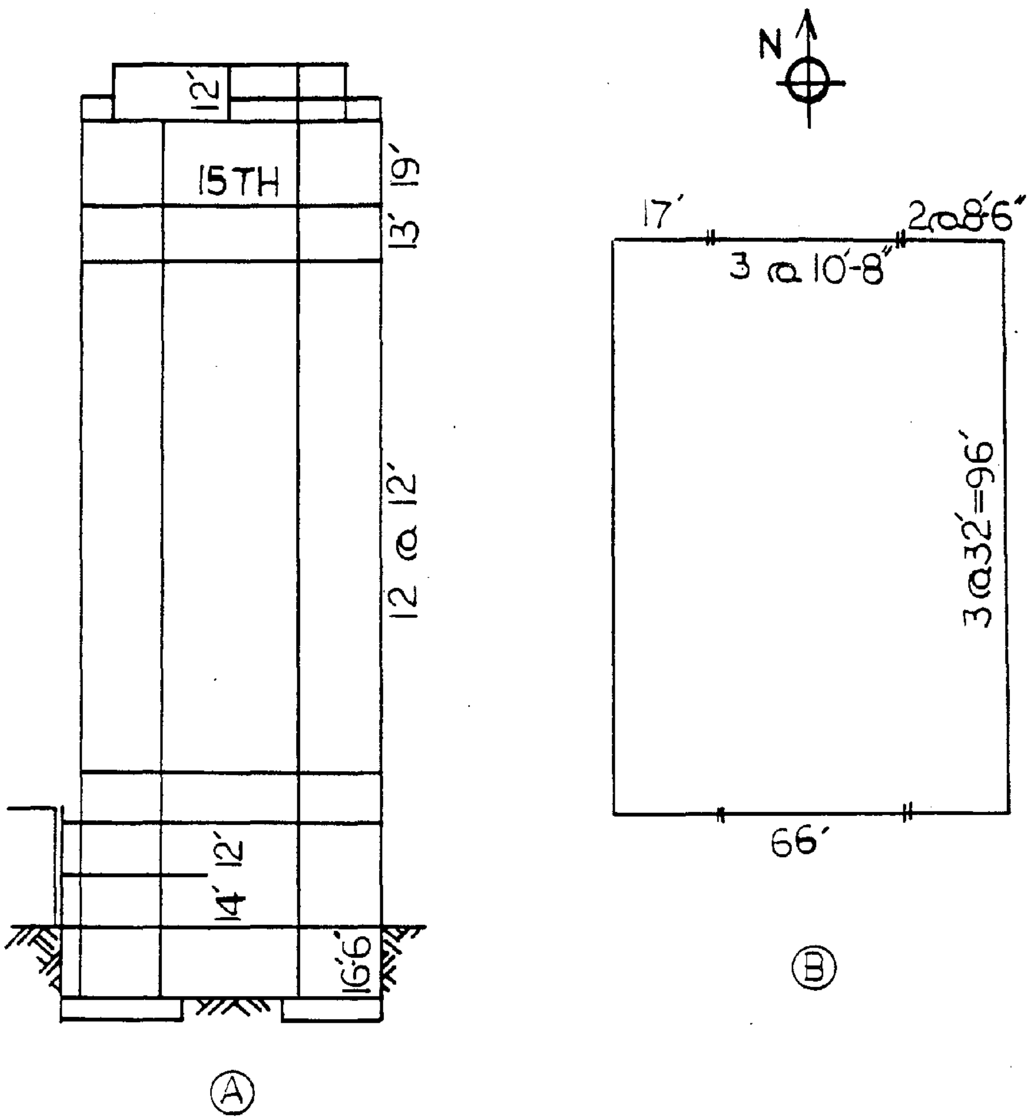


Figure 5-9: Kajima International Building:
(A) E-W transverse elevation.
(B) Typical plan.

Spread footings were used to distribute the main column loads from the office tower to the firm soils at a depth of 15'. The footings were combined in pairs due to property line limitations. Along the W-side of the tower there is a 2-story concrete block firewall supported on 24"-diameter caissons averaging 28' in length. The 2-story firewall is designed with a seismic slip joint.

Concrete encasement was used as fire protection for all exterior columns up to the 6th floor level. Drywall was used in multilayers as fire protection for all other columns. Large concrete spandrels, 6' in depth, were used as part of the exterior facade of the building. The remaining curtain wall consists mostly of glass. Plaster partitions are used throughout the building.

DESIGN & CONSTRUCTION

The Kajima International Building was designed in 1966 under the building code requirements of the city of Los Angeles. From ground level up the structure was designed to resist lateral forces as a 100% ductile moment-resisting frame. Every effort has been made by the designer to keep the center of floor mass and center of lateral building stiffness as close as possible, to minimize torsional response. Lateral forces were designed to be transfered from the structure to the ground through passive soil resistance and friction.

Concrete from foundation to the 1st floor is stone aggregate (3ksi). Above the 1st floor lightweight concrete (3ksi) is used in all structural floor slabs. All temperature reinforcing steel is Grade 40 and all primary reinforcement Grade 60. All structural steel is rolled sections with fy=36ksi.

DAMAGE

There was no structural damage to the $3 million building as a result of the San Fernando earthquake.

However, nonstructural damage to plaster partitions around the elevator shaft and stairwell was estimated at $1,000 by the owner. Damage consisted primarily of cracking and chipping. Glass panels shifted in most of the frames and some cracked. Repair costs amounted to $100. The office tower and parking structure actually impacted during the earthquake at the seismic separation joint. In addition, slip marks were observed at the top of the 2-story firewall· which indicated that this joint functioned as designed.

CAUSE

The general performance of the structure was linear-elastic, with some lengthening of building periods during the earthquake. Maximum stress levels in 15% of the frame members (especially columns) were calculated to have exceeded initial yield values at least once during the earthquake. The dynamic story shears and overturning moments exceeded design and code values by 115% and 140% respectively in the N-S direction. Vertical roof accelerations exceeded the horizontal, thus indicating that vertical amplification of ground motion can be significant.

Some recommendations are:

- A more realistic evaluation of earthquake loading on frame members and corner columns in particular is needed.

- Nonstructural elements should be designed for earthquake motions.

- A more realistic evaluation of seismic separations is needed.

- Further investigation of vertical acceleration effects is desirable.

5.10 KB VALLEY CENTER

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The KB Valley Center is located in the city of Los Angeles at the southern part of the San Fernando Valley, approximately 17 miles south of the epicenter of the San Fernando earthquake.

Three strong motion accelerographs located at the basement, 9th floor, and roof, recorded peak accelerations of: 0.132g, 0.180g, 0.220g, respectively, in the N-S (transverse) direction, 0.153g. 0.136g. 0.231g in the E-W (longitudinal) direction, and 0.134g, 0.215g, 0.211g in the vertical direction. The fundamental building periods recorded during the earthquake were 3.2 seconds in the N-S direction, and 3.0 seconds in the E-W direction. The earthquake intensity at the site was specified as VII of the Modified Mercalli Intensity scale.

The soils consist generally of clayey sands and silty sands. Below a depth of 8' all soils at the site are dense. The ground water level is at a depth of 30' to 34'.

STRUCTURAL SYSTEM

The KB Valley Center consists of a 16-story office tower and adjacent 4-story parking structure. The structure above the 5th floor forms the office tower, which is set back in plan from the lower floors. The approximately 211' high tower measures about 87' x 169' in plan and the lower stories about 97' x 220'. Above the roof level there is a 1-story mechanical penthouse, covering 40% of the roof area.

The structural system consists of a vertical load-carrying 3-D steel space frame and moment resisting perimeter frames. Deep (42") girders are used to stiffen the perimeter frames, thus minimizing lateral story drift under wind or earthquake loading. For the corner columns moment connections are provided only in the N-S direction. In the E-W direction. pinned connections are used to minimize bending moments about the weak axis of the column. The corner columns are anchored to the pile caps of the

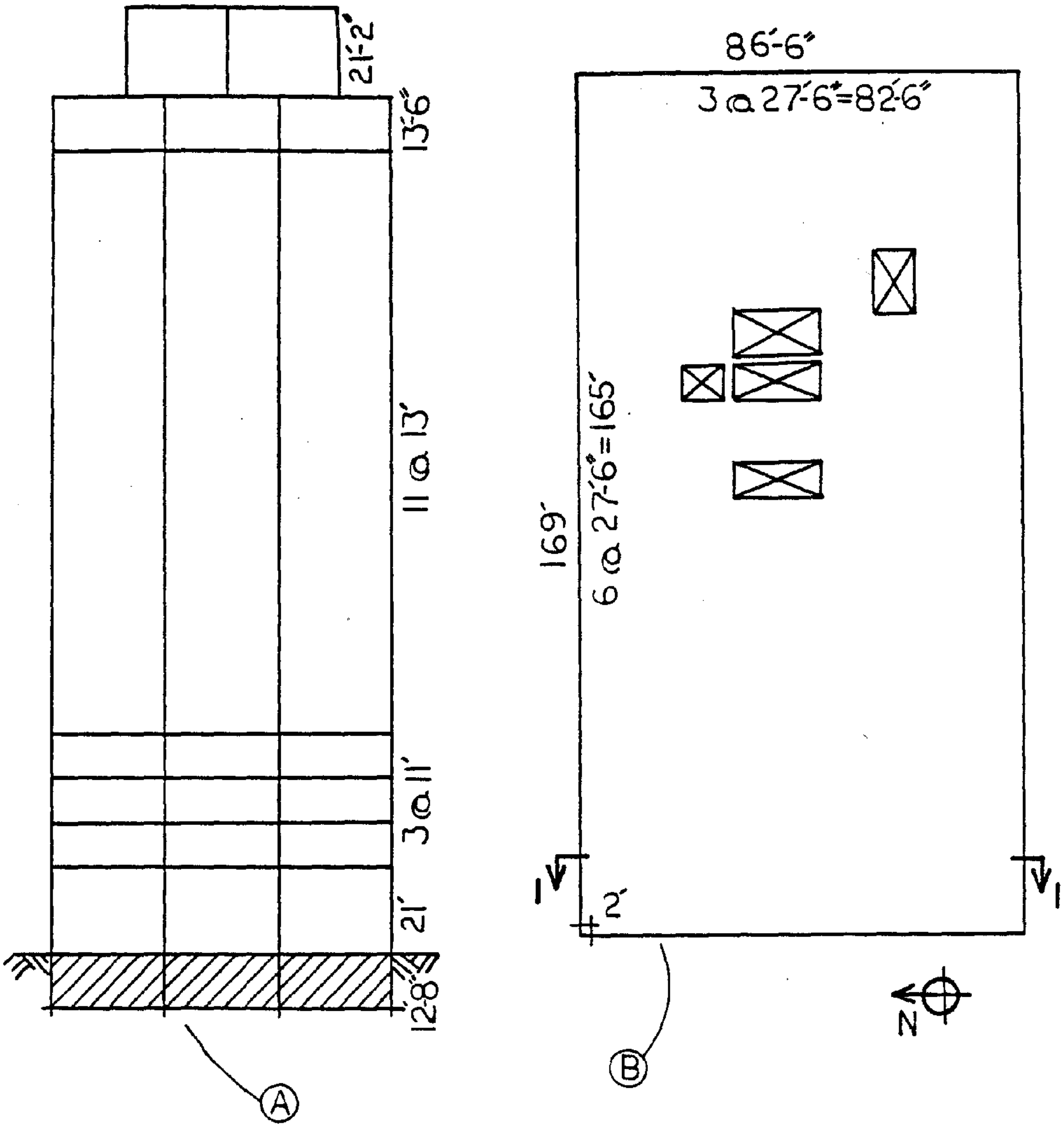


Figure 5-10: KB Valley Center:
(A) Elevation 1-1.
(B) Tower floor plan.

foundation to prevent possible uplift under lateral loading. The floor system consists of lightweight concrete slabs in composite construction with the beams of the frame. This system forms a relatively rigid diaphragm for lateral loads.

The foundation system used to support the structure on the firm, dense soil layers consists of driven step-tapered piles. averaging 54' in length. Reinforced concrete tie beams are used between pile caps in several locations, where the lateral resistance of piles and pile caps is not sufficient to meet the imposed code loads.

A 3-story high concrete fire wall along the western edge of the building is designed with a seismic slip joint in the N-S direction. A 2" seismic gap separates the concrete parking structure from the tower to minimize building eccentricities that would be present, were the two structures tied together. No mention is made about nonstructural elements except that columns are encased in 4"-concrete fireproofing and that nonstructural block walls enclose the building, designed with seismic slip joints at each floor level. Nonstructural walls also enclose the elevator and stair shafts.

DESIGN & CONSTRUCTION

The KB Valley Center was designed in 1969 and constructed in 1970 under the building code requirements of the city of Los Angeles. From ground level up the perimeter frames were designed to resist all lateral forces as 100% moment-resisting frames. All other frames were designed for vertical loads only. The designer made every effort to keep the center of floor mass and center of lateral stiffness as close as possible to minimize torsional effects under lateral loading.

Concrete from the foundation up through the 1st floor is stone aggregate (145pcf). Above the 1st floor light weight concrete (115pcf) is used in all structural floor slabs. All primary reinforcing steel is Grade 60. All temperature steel is Grade 40. The structural steel for all rolled sections and steel plates is A-36. Columns not part of seismic frame are of Grade 50 structural steel.

The building apparently was built according to the specifications.

DAMAGE

No damage to the structural elements of the building was observed. Minor nonstructural damage occurred in partitions, at seismic joints, and in mechanical equipment mounts. The repair costs amounted to an estimated $3,000. Construction costs in 1970 were $4 million.

The steel plate that covers the 2" expansion joint between office tower and parking garage, buckled under the relative movement between the two structures. There were indications at the joint that the buildings had actually impacted at the upper floors of the parking structure. The seismic slip joint of the 3-story block wall, however, showed no sign of movement. At the roof level, a 40-ton condenser bounced laterally on its spring supports, bending the 1"-diameter tie-down bolts.

CAUSE

Acceleration records available indicate that the general performance of the structure was almost linear-elastic with only minor lengthening of building periods during the earthquake. This indicates a loss of stiffness partly due to cracking of nonstructural elements such as partitions, and concrete fireproofing. Elastic dynamic analysis after the earthquake indicates that the envelope of dynamic maximum force response exceeded current code design forces by 110% in the N-S direction, and by 170% in the E-W direction. Maximum stress levels exceeded initial yield stress at least once during the earthquake in 30% of the members in the N-S frames and in 80% of the lateral force-resisting frames in the E-W direction. Columns reached initial yield stress before girders. A significant portion of the maximum building response was in the 2nd and 3rd modes.

Some recommendations are:

- Frames should be designed so that inelastic behavior is initiated in and confined to girders rather than columns.

- Current code practice should be reviewed to account for higher mode effects in medium-rise structures.

- A realistic approach should be followed in evaluating the actually required earthquake separations.

- In the case of KB Valley Center, the peak vertical acceleration was as large as the horizontal component at the roof.This fact alone warrants further investigation of vertical acceleration effects.

- Code provisions must be developed for earthquake design of equipment supports.

5.11 MUIR MEDICAL CENTER

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Muir Medical Center is located in the city of Hollywood, about 21 miles south of the epicenter of the San Fernando earthquake.

Strong motion accelerographs installed in the building at the basement, 6th floor, and roof level recorded peak accelerations of 0.088g, 0.122g, 0.122g, respectively, in the longitudinal (N-S) direction, 0.102g, 0.195g, 0.214g in the transverse direction, and 0.065g, 0.150g, 0.220g in the vertical direction. The fundamental building periods as measured before, during, and after the earthquake were: 0.90, 1.4, and 1.02 seconds in the N-S direction, and 1.03, 1.6, and 1.14 seconds in the E-W direction, respectively. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

The upper layers of soil at the site are moderately firm silty sands composed of natural soils and fills. At depths of about 21' to 26' the soil is a silty clay and is uniformly firm.

STRUCTURAL SYSTEM

The Muir Medical Center consists of an 11-story office tower, approximately 89' x 144' in plan and about 149' high, surrounded by a 1-story bank, pharmacy, and restaurant facility. Beneath the entire structure is a 1-story garage. The 2nd floor is set back in plan from the ground floor. The tower, in turn, is set back in plan from the 2nd floor. A 2-story penthouse is provided to support elevator equipment.

The structural system of this reinforced concrete building consists of an interior space frame formed by 9" flat slabs on columns with tapered column capitals and of perimeter frames with deep spandrel beams. The flat slabs and deep girders are designed to work together with the columns as a moment-resisting frame. Perimeter framing of the tower is extended above the roof level for architectural

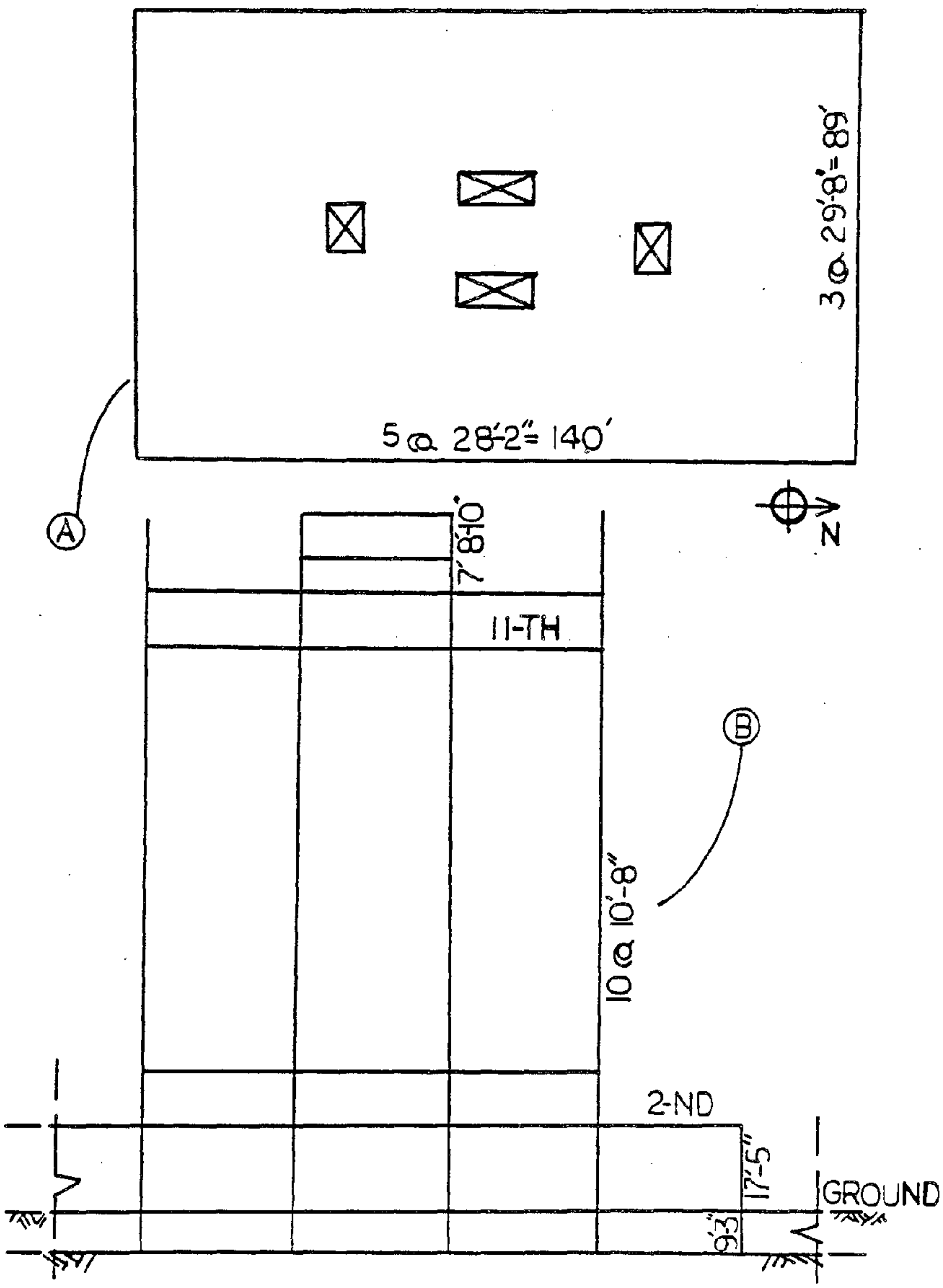


Figure 5-11: Muir Medical Center:
(A) Typical plan.
(B) Typical transverse elevation.

reasons. Perimeter basement walls serve as shear walls in resisting seismic forces.

To minimize differential settlement, the foundations consist of drilled belled caissons and cast-in-place concrete piles, extending into the underlying firm layer. The caissons are tied together by reinforced concrete beams located just below the grade slab.

Curtain walls. consisting mostly of glass, enclose the building. No mention is made of interior partitions or other nonstructural elements except that stairwells are enclosed by drywalls and that there are some concrete canopy arches at the 2nd floor.

DESIGN & CONSTRUCTION

The $4.5 million reinforced concrete structure was designed in 1966 under the building code requirements of the city of Los Angeles. From ground level up the structure was designed to resist lateral forces as a 100% moment-resisting space frame with K=0.67. Both the interior space frame formed by flat slabs and columns and the perimeter frames are designed as moment-resisting frames. The subterranean parking with its shear walls was designed as a 1-story building with K=1.33. Member forces were found by computer analysis of a 2-D model of the structural frames and shear walls subjected to the static code seismic forces. No reduction was applied to overturning moments although allowed by the code. The perimeter frames with the deep spandrels provided 70% of the lateral force resistance, while the flat slab - interior column system accounted for the rest. To improve earthquake resistance, a tapered drop panel was adopted so that a plastic hinge would form in the slab at the perimeter of the panel. Confinement ties and spiral reinforcement were extended up through the tapered panels.

Concrete from foundation to 1st floor is stone aggregate concrete with a strength of 4.5ksi for piles and external caissons and 3ksi for the rest. Above the 1st floor lightweight concrete with a strength of 3ksi is used. Reinforcing steel in slabs, spandrels and walls is Grade 40 and in columns and caissons Grade 60.

DAMAGE

There was no observed structural damage as a result of the San Fernando earthquake. However, nonstructural damage to partitions and exterior glass is estimated at $2,000.

The major repairs to drywalls occurred in the stairwell walls between the 3rd and 6th floors. Damage consisted primarily of separation at tapered joints and paint cracking. Glass breakage occurred between ground floor window mullions and the 2nd floor canopy arches. All glass panels at the canopy developed horizontal cracks and had to be replaced; they were constructed without provisions for horizontal slippage.

CAUSE

The general performance of the structure was linear elastic with only minor lengthening of building periods during the earthquake. Based on elastic dynamic analysis of the building, the level of the lateral forces developed exceeded the design static forces by 20% in the N-S direction and by 100% in the E-W direction. Accordingly, brittle glass and partitions cracked in the E-W direction under interstory displacements of 1/2" or more.

Some recommendations are:

- Nonstructural elements should be designed to accommodate earthquake motion.
- A guide of practice should be prepared for instrument location. The high vertical accelerations at the roof and 6th floor probably reflect local slab amplification.
- The peak vertical acceleration was larger than the horizontal component at the roof. This fact alone warrants further investigation of vertical acceleration effects.

5.12 OLIVE VIEW MEDICAL CENTER

[Mahin, 1975], [Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

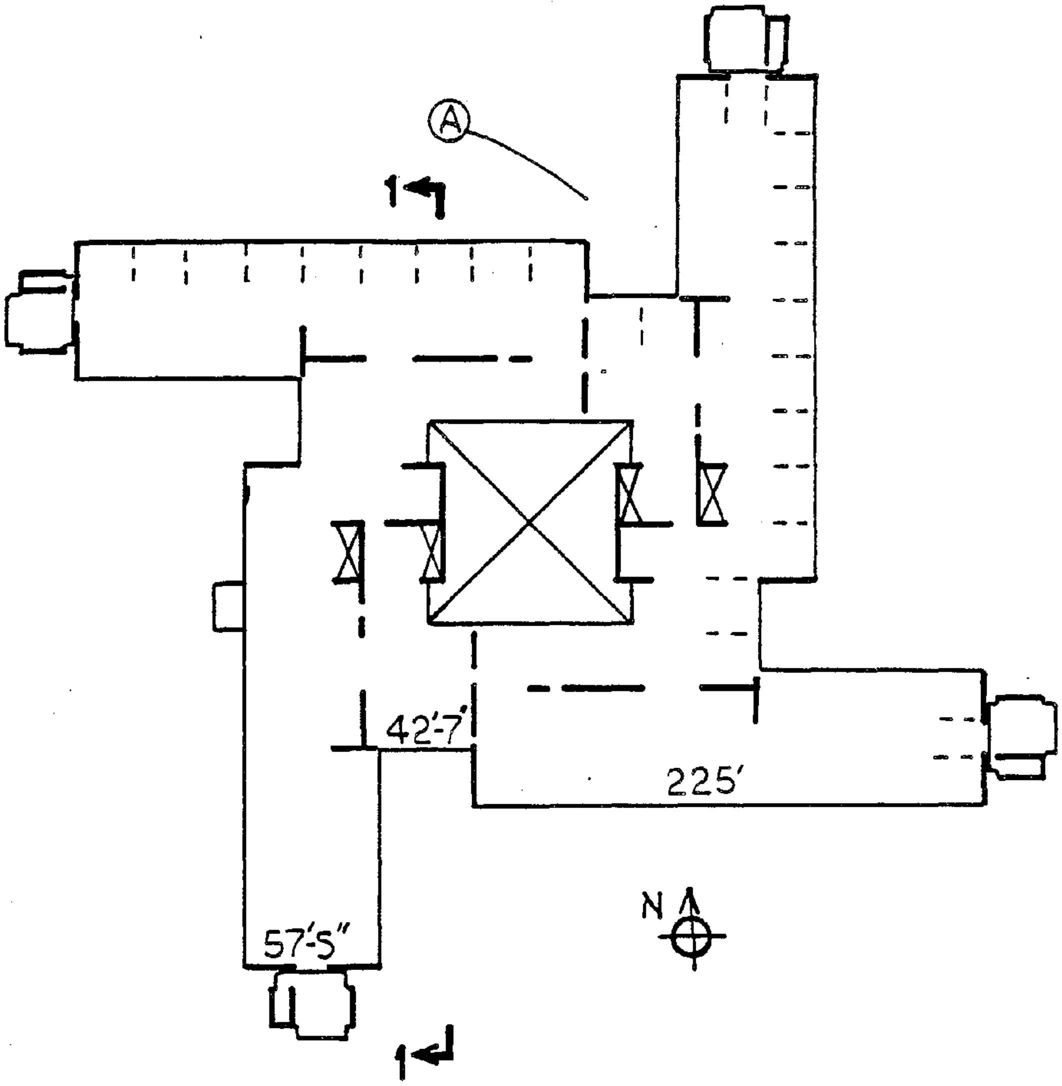
The Olive View Medical Center lies at the base of the San Gabriel mountains, 6 miles southwest of the epicenter of the San Fernando earthquake. From accelerograph records from nearby sites, the ground motion was estimated to exceed 0.50g. At the hospital's site the intensity of the motion was estimated as XI on the Modified Mercalli Intensity scale.

Much of the hospital complex consisted of older buildings. Constructed of wood frames, unreinforced brick, or hollow tile masonry, many suffered considerable damage or collapsed. Small wood frame residential-type structures remained relatively undamaged along with two concrete portal framed buildings designed to resist lateral loads.

The hospital structures were located on an alluvial fan consisting of unconsolidated sands and gravels interspersed with rocks and large boulders. Underlying granite bedrock was at a depth of 200' to 300'. Although two faults lie within 1 mile of the hospital, there was no evidence of recent activity, all damage resulting from horizontal and vertical ground motion. A survey showed an average uplift for the site of about 1.6'.

STRUCTURAL SYSTEM

The main hospital building (medical treatment and care unit) consists of four rectangular 5-story wings supported on a single, large 1-story base. The wings intersect each other at right angles forming an open courtyard in the center. The roof of the 1-story base is heavily landscaped, supporting 21 inches of concrete waterproofing and earth fill. This mass comprises 27% of the total dead load mass of the building. Because of the slopping terrain, the grade at the N- and W- face is level with the first floor, while the grade at the S- and E- face is at the ground floor level. Basement walls of the N- and W- faces were designed as cantilever retaining walls, separated from the floor system by 4".



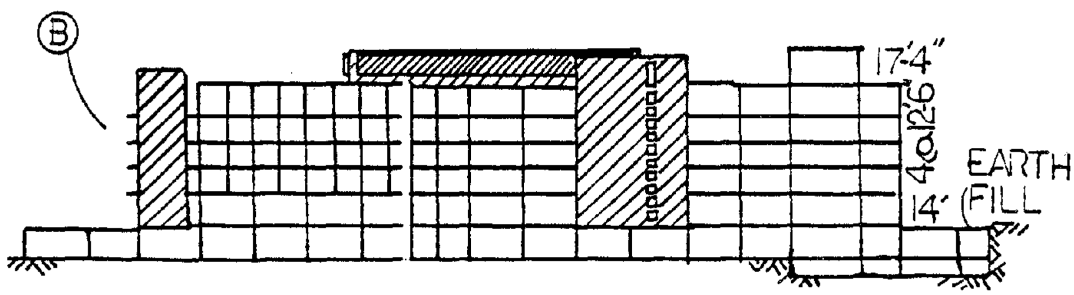


Figure 5-12: Olive View Medical Center:
(A) Typical plan.
(B) Elevation 1-1.

The basic framing scheme is a two-way flat slab system with drop panels at the columns. The top four stories contain shear walls, which are discontinued above the 2nd story and supported by a 2-story moment-resisting frame. Some of the walls in the N-S direction continue to the 1st story. In the extended portion of the ground story, tied rectangular columns are used. Elsewhere in the first two stories and all interior bays of higher stories, spiral columns with rectangular sections are used. All columns are supported by spread footings on undisturbed soil.

Each wing has a free-standing stair structure separated from the main building by about 4". Stair tower walls are terminated at the 1st-floor level and supported by a beam and column system except for the N-tower, where the walls extend to the foundation. Nonstructural elements were not explicitly mentioned in the original case-study, but, apparently, partitions of masonry wall construction and cladding of precast concrete panels were used.

DESIGN & CONSTRUCTION

The Olive View Medical Center was completed in accordance with the 1964 edition of the UBC in 1970, only 4 months prior to the earthquake. In effect, the building may be described as a 4-story rigid box structure supported on 1- and 2-story soft frames. In the top four stories, the shear walls were designed to resist the entire design lateral forces, while the remaining flat slab system was proportioned to carry 25% of the design lateral forces (dual system, K=0.8). The bottom two stories were designed as moment-resisting frames resisting 100% of the lateral load (K=0.67).

A greater design base shear than required by code was used, because the upper stories were considered to form a setback and their lateral loads were applied as a concentrated load at the top of the 2-story base structure. The stair towers were designed to resist lateral forces with reinforced concrete shear walls (K=1.00). A 33% increase in allowable stresses was used when considering seismic forces.

Regular weight stone concrete with a specified strength of 3ksi was used except for the columns in the bottom two stories, for which the specified strength was 5ksi. Reinforcing steel was 40ksi deformed bars except for the longitudinal column bars, which were 60ksi. Field and mill records along with specimens tested after the earthquake generally showed material strengths much higher than specified. Construction methods and quality appeared to be in accordance with design specifications. The only weakness found was that spiral column reinforcement was terminated early.

DAMAGE

This building suffered severe structural and nonstructural damage and it had to be demolished after the earthquake.

The most critical structural damage was concentrated in the bottom two stories which acted as soft stories, while the stories above were only moderately damaged. The bottom two stories suffered severe permanent deformations consisting of a translation towards the northeast combined with a clockwise rotation of the structure above the ground story. The displacements reached 10" in the ground story and 30" in the 1st story. Pounding of the building against the retaining walls at the N- and W- face caused these walls to move by 6".

All the tied columns, located primarily in the extended part of the base structure, failed. Most of the spirally reinforced columns suffered considerable spalling and cracking in the bottom two stories. Some columns failed completely due to spirals being terminated before the joint. Shear walls in the upper stories suffered spalling and diagonal cracking. Serious damage to the slabs and drop panels was primarily limited to the first two floors.

Three of the four stair towers overturned. The fourth tower on the N- side, the only tower whose shear walls extended to the foundation, was out of level by approximately 2'.

Nonstructural masonry walls were torn loose, many precast concrete panels were dislodged, interior partitions, ceilings and other architectural features were severely damaged, and mechanical and electrical equipment failed to function.

CAUSE

The extremely poor behavior of this building can be attributed to the irregularities in the structural system and in the mass distribution. None of the shear walls extended to the ground. With shear walls in the top four stories and moment-resisting frames in the bottom two stories the building effectively responded as a rigid concrete box on a soft story in which all deformations were concentrated. The problems due to the structural discontinuities were compounded by the irregularity in the mass distribution resulting from the large earth fill mass on the roof of the extended ground story. This mass resulted in both high vertical and high lateral inertia forces, which led to the brittle failure of the tied columns. The drop panel slab of the ground story roof provided little fixity at the top of the tied columns, resulting in a relatively ineffective lateral load resisting system. More effective were the lower level frames along the perimeters of the wings. Their columns had spiral reinforcement and were well restrained by the beams and shear walls of the wings.

The free-standing stair towers also responded essentially as a rigid concrete box on a soft story. They overturned due to brittle shear failure of the tied columns supporting the discontinued shear walls. While the spiral columns used elsewhere were capable of developing their ultimate flexural capacity, the tied columns of similar section prematurely failed in shear.

That the building exceeded most code requirements, indicates the weakness of the "equivalent static load" method of design. An unusually shaped, irregular structure built over a fault system requires a much more sophisticated level of analysis and design. This building was irregular in plan, elevation, mass ratio, and lateral load resisting system, according to ATC 3-06 [ATC, 1978].

5.13 SHERATON UNIVERSAL HOTEL

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971],

GROUND MOTION & SITE

Located in the Universal City area of Los Angeles, the Sheraton Universal Hotel lies about 19 miles south of the epicenter of the San Fernando earthquake.

Two strong motion accelerographs, located at the 20th floor and at basement level, recorded approximately the first 28 seconds of earthquake motions. Recorded accelerations at the roof and basement levels were 0.120g, 0,175g, respectively, in the transverse direction(N-S), 0.195g, 0.165g in the longitudinal direction(E-W), and 0.260g, 0.087g in the vertical direction. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

The underlying soil deposits consist primarily of bedded sandstones with deposits of shale and clay. A geologic study of the bedding revealed no adverse bedding planes; but a wide fault zone crosses the site and occupies the area immediately beneath the foundations of the central tower.

STRUCTURAL SYSTEM

The building is an approximately 210' high, 20-story reinforced concrete structure that serves as a hotel and convention center. Plan dimensions of the central tower extending from the 4th floor to the roof, are typically 183'-6" x 57'-10". The plan dimensions of lobby story, ground story, and basement are 198'-7" x 96'-4". The central tower portion is separated from the rest of the structure by a seismic joint.

The structural system of this reinforced concrete building consists of moment-resisting frames in each direction, except for the basement which contains 12" thick reinforced concrete shear walls. Typical framing consists of columns spaced at about 19' on center in the transverse and 13' in the longitudinal direction, with interconnecting floor girders in each direction. Exterior columns on the

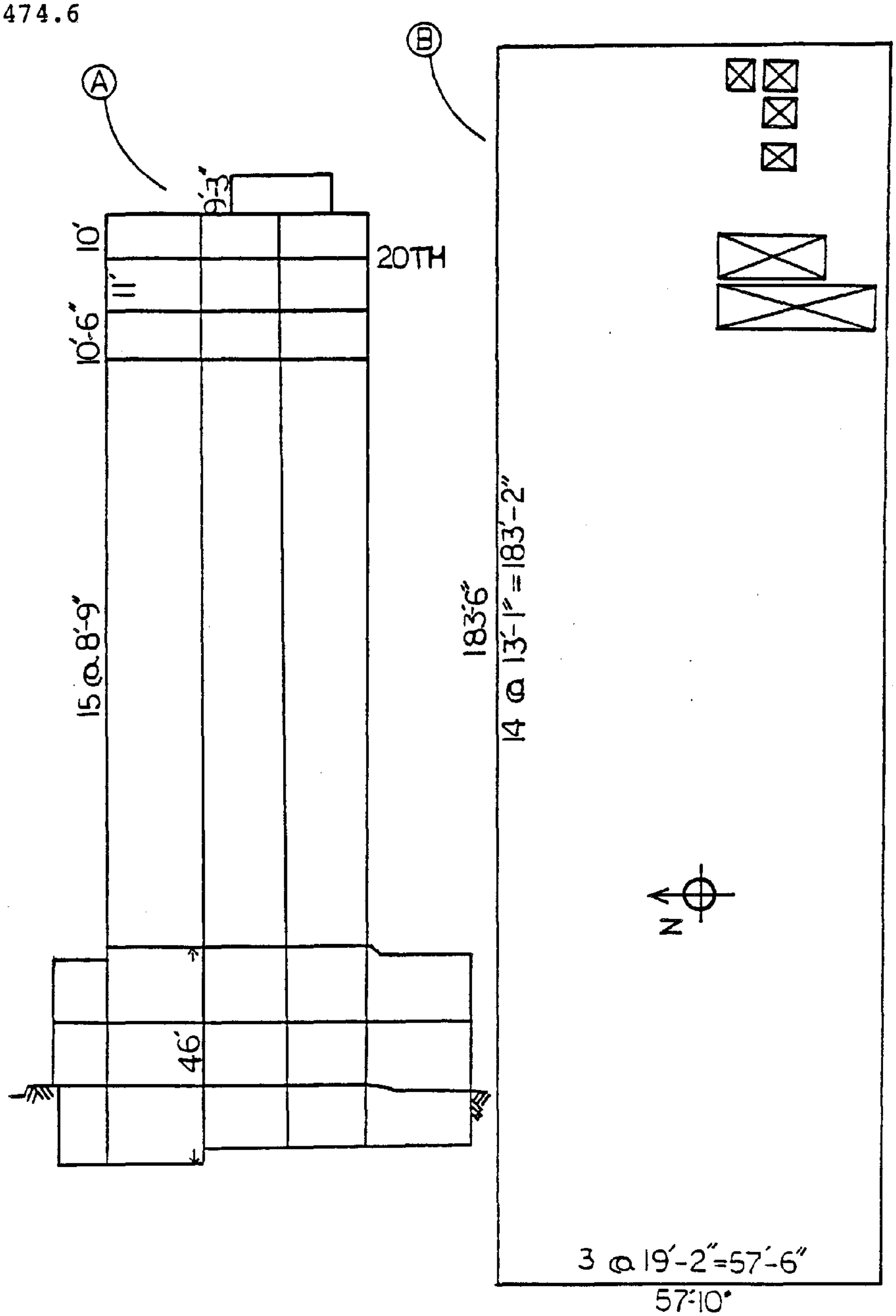


Figure 5-13: Sheraton Universal Hotel:
(A) Typical transverse elevation.
(B) Typical tower plan.

N- and S- faces taper from a 20" x 18" section at top and bottom to 20" x 15" at midstory height. Floor slabs are typically 4.5" thick in guest rooms and 6" in corridors. At the lobby and ground floor the slab thickness is 5". Reinforced concrete spread footings comprise the foundations for the structure. Design soil bearing capacity was 6ksf for dead plus live load.

Typical interior partitions consist of gypsum wallboard on metal studs, or gypsum coreboard. Some plaster partitions are located in the ground story. Plaster walls in the longitudinal direction are secured to the structural frame on all edges. In the transverse direction, plaster walls are separated from the building frame with a 3/8" seismic gap by means of neoprene filler strips. The E- and W-facade consist of 4" thick precast concrete panels connected to the spandrel beams by strap anchors and separated by a 3/8" seismic gap. Slotted bolt holes at the top connections allow relative movement between panel and frame. In the longitudinal direction, the entire facade consist of glass curtain walls located between columns.

DESIGN & CONSTRUCTION

This building, completed in 1968 at costs of $7.5 million, has been designed as a ductile moment-resisting frame, meeting the requirements of the 1966 Los Angeles City Building Code.

Lightweight concrete (110psf) with a compressive strength of 3ksi was used above the ground floor. Columns 10th floor to roof were of 4ksi. All concrete, from basement to ground floor was regular weight (150psf) with a compressive strength of 3ksi. All reinforcement was Grade 40 deformed billet bars, except in columns from foundation to 10th floor, where Grade 60 was used.

The designer followed standard inspection procedures during construction. The structural engineer provided a full-time licensed inspector to inspect construction and to interpret drawings.

DAMAGE

The $7.5 million hotel suffered only slight damage, nonstructural damage totaling $2,100. The only known structural damage occurred at a 3rd floor corner column that suffered minor spalling. No reinforcing steel was exposed.

The seismic joint cover at the low-rise roof of the 3rd floor suffered a 3/4" permanent displacement in the E-W direction. At the lobby floor, an aluminum seismic joint buckled. A water seal was broken at the roof coping at both sides of the seismic joint. At the ground story stairwell, evidence of seismic joint movement was apparent. Horizontal cracking appeared at the ground floorline in this stairwell and at the underside of the beam in the lobby floor above the stairwell. A mosaic tile mural mounted on columns adjacent to the seismic joint at the ground floor was damaged, apparently by impact of the adjacent wing. Plaster walls in the ground story and the gypsum wallboard partitions in the E-W direction throughout the building suffered cracking. A band of cracking in partitions started in the 14th story at the E-end and extended to the 10th story at the W-end. It consisted primarily of diagonal cracks, extending from the upper corners of the doors to the ceilings, in the E-W filler walls on the intermediate column lines. These walls apparently were secured on all four edges.

CAUSE

The building responded to the earthquake in an essentially linear-elastic manner, but with an equivalent viscous damping of 10% of critical. Calculated building response indicates that no major structural damage would be expected, although earthquake forces generally were greater than prescribed code minimums. After 6 seconds of motion the apparent period of the structure elongated, which indicates that enough force had been generated to overcome any bond between structural and nonstructural elements. Only the bare structural frames resisted the earthquake motion.

Some recommendations are:

- Expansion joints, flashings, partitions, and stairwells should be designed for seismic movements. The amount of movement to be designed into these elements should be based upon maximum possible interstory drifts, rather than upon deflections computed for code seismic forces (unless code seismic forces are increased considerably).

- Strong-motion recording devices should be placed in high-rise buildings in such a way that they provide a record of true seismic motion without any undue contributions from real or accidental torsional effects. Vertical records should be taken at or near a column to avoid local effects from flexible elements, such as thin slabs.

5.14 UNION BANK BUILDING

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Union Bank Building, in Sherman Oaks, is located approximately 17 miles south of the epicenter of the San Fernando earthquake. Across the street is the Bank of California, which suffered only moderate structural and nonstructural damage and was instrumented with strong-motion accelerographs. The ground motion experienced by the Union Bank was similar to that recorded at the Bank of California. The peak accelerations recorded at the ground floor of the Bank of California were 0.155g, 0.230g, and 0.108g in the longitudinal, transverse, and vertical directions, respectively, of the Union Bank Building. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale.

The underlying soil consists of silt, clay, sand, and combinations of the same. The soils are recent alluvial deposits, generally only moderately firm with soft layers at varying depths.

STRUCTURAL SYSTEM

The building is a 13-story reinforced concrete structure with two basements and a mechanical penthouse covering approximately 20% of the main roof. Plan dimensions of the tower are 75' x 193'. Total building height is approximately 204'; story heights are 11'-9" except for the 1st story, which measures 23'-6".

The structural system is a reinforced concrete space frame. Shear walls are located in the basements only. Rectangular tied columns are spaced at 27' longitudinally and at 37'-6" transversely. The floor system consists of 4 1/2" thick one-way slabs on intermediate concrete beams spanning between the frame girders. The foundation consists of cast-in-place concrete piles supporting individual footings.

Nonstructural partitions consisting of wallboard on metal studs enclose elevator shafts, stairwells, duct

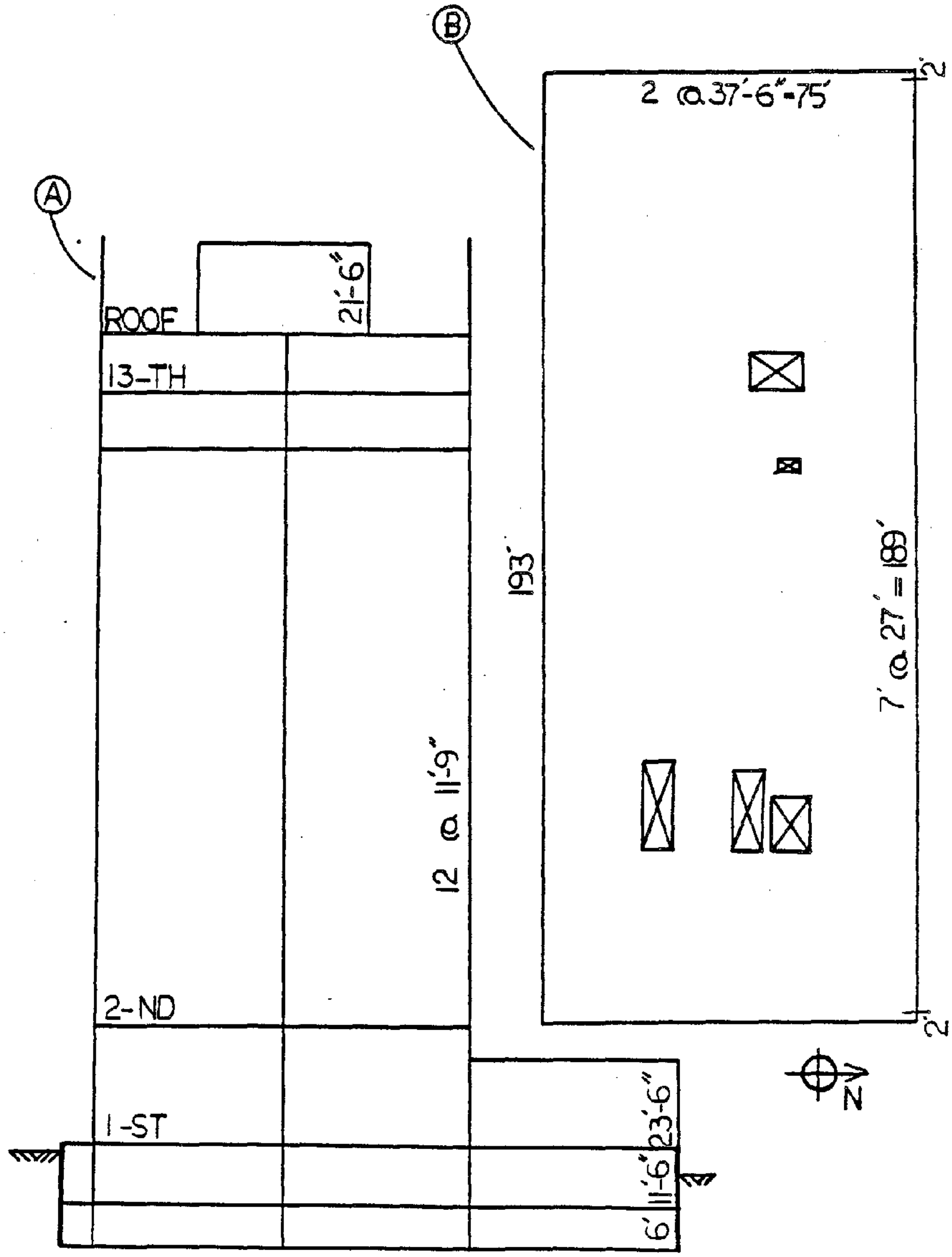


Figure 5-14: Union Bank Building:
(A) Typical elevation.
(B) Typical tower plan.

shafts, and restrooms. The building is enclosed by glass windows extending from spandrel to ceiling.

DESIGN & CONSTRUCTION

The building was designed in 1964 under the Los Angeles City Building Code then in effect. The building was designed to resist both gravity loads and lateral loads in each direction by moment-resisting frame action. In the basement, shear walls transfer most of the shear from the tower to the ground.

Lightweight concrete (3.75ksi) was used above the 2nd floor. All concrete in the 1st story and below was regular aggregate concrete. Regular concrete strength ranged from 2.5ksi in the piles to 5ksi in columns up to 2nd floor, some floor systems, and foundations. Construction methods and quality appeared to be in accordance with specifications.

DAMAGE

This building suffered moderate damage. Damage costs are estimated at $80,000 in structural and $17,500 in nonstructural damage. Structural damage occurred in the four corner columns, which cracked in the vicinity of the 2nd-floor spandrel beams. Some hairline cracks also appeared in the 2nd floor spandrel beams.

Nonstructural damage occurred mainly in the bottom four and the top two stories. Plaster walls cracked and portions of plaster fell in the elevator shafts and stairwells. Ceramic tiles in public restrooms cracked at corners of partitions from the 2nd to 5th story, while tile damage at all other stories was minor. Some partitions buckled in the 2nd story, and marble veneer panels around the elevators in the ground story pulled away from the wall. Large areas of acoustic ceiling tile (12" x 48") fell in the 2nd and 3rd stories. Other damage included minor window breakage, four inoperative elevators due to fallen plaster, and damage to steel stairs, which pulled away from their support landings (broken the welds) in the three bottom stories.

CAUSE

The building behaved well under the earthquake loading. Lateral forces stressed the corner columns and interior column-spandrel connections beyond the elastic limit, but no failures occurred. The cracks at the corner columns are believed to be the result of longitudinal spandrel reinforcing not extending far enough into the column. Rigid stair stringers had no allowance for lateral movement relative to the frame.

Though the building performed well, it would have to be classified according to ATC 3-06 [ATC, 1978] as vertically irregular because of a high 1st story.

Some recommendations are:

- Attention should be given to corner columns and the combined effects of two-way frame action.
- Stair stringer connections should provide for lateral frame movement.
- Pay close attention to reinforcing placement in the beam-to-column connections.

5.15 UNION BANK SQUARE

[Murphy, 1973], [Moran, 1973], [Steinbrugge, 1971].

GROUND MOTION & SITE

The Union Bank Square (UBS) building is located in the city of Los Angeles, 26 miles from the epicenter of the San Fernando earthquake.

Three strong-motion instruments were placed at the second basement, 19th, and 39th floors. The recorded peak ground accelerations were 0.06g, 0.14g, and 0.13g in the longitudinal, transverse, and vertical directions, respectively. Displacements at the 19th floor level peaked at approximately 9" transversely and 7" longitudinally. The ground shaking exhibited about 10 seconds of strong motion and 50 seconds of low-amplitude, long-period, rolling motion. The earthquake intensity at the site was specified as VII on the Modified Mercalli Intensity scale. Soil consists of brown silty weathered shale becoming massive gray shale at greater depths.

STRUCTURAL SYSTEM

The UBS tower cantilevers a total height of 42 stories (536') from the 2nd basement level to the roof. Of this, 39 stories (495') project above the adjacent plaza level. The tower is 98' x 196' in plan; the bottom 3 stories of the tower are surrounded by 3 levels of parking along with a plaza level (302' x 514'). There is a 2" seismic gap separating the tower from the adjoining parking garage and a 3" gap between the garage and a retaining wall.

The structural system consists of moment-resisting steel frames combined with reinforced concrete shear walls that extend from the foundation into the 1st tower story (plaza). The frames are 100% moment resisting except for the interior longitudinal frames which are not moment-resisting. Framing is regular throughout the tower; perimeter column spacing is 14'. Transverse frames frame into every second perimeter column (28'). The exterior footings are continuous as well as the transverse footings of the shear core. Individual spread footings support all other columns.

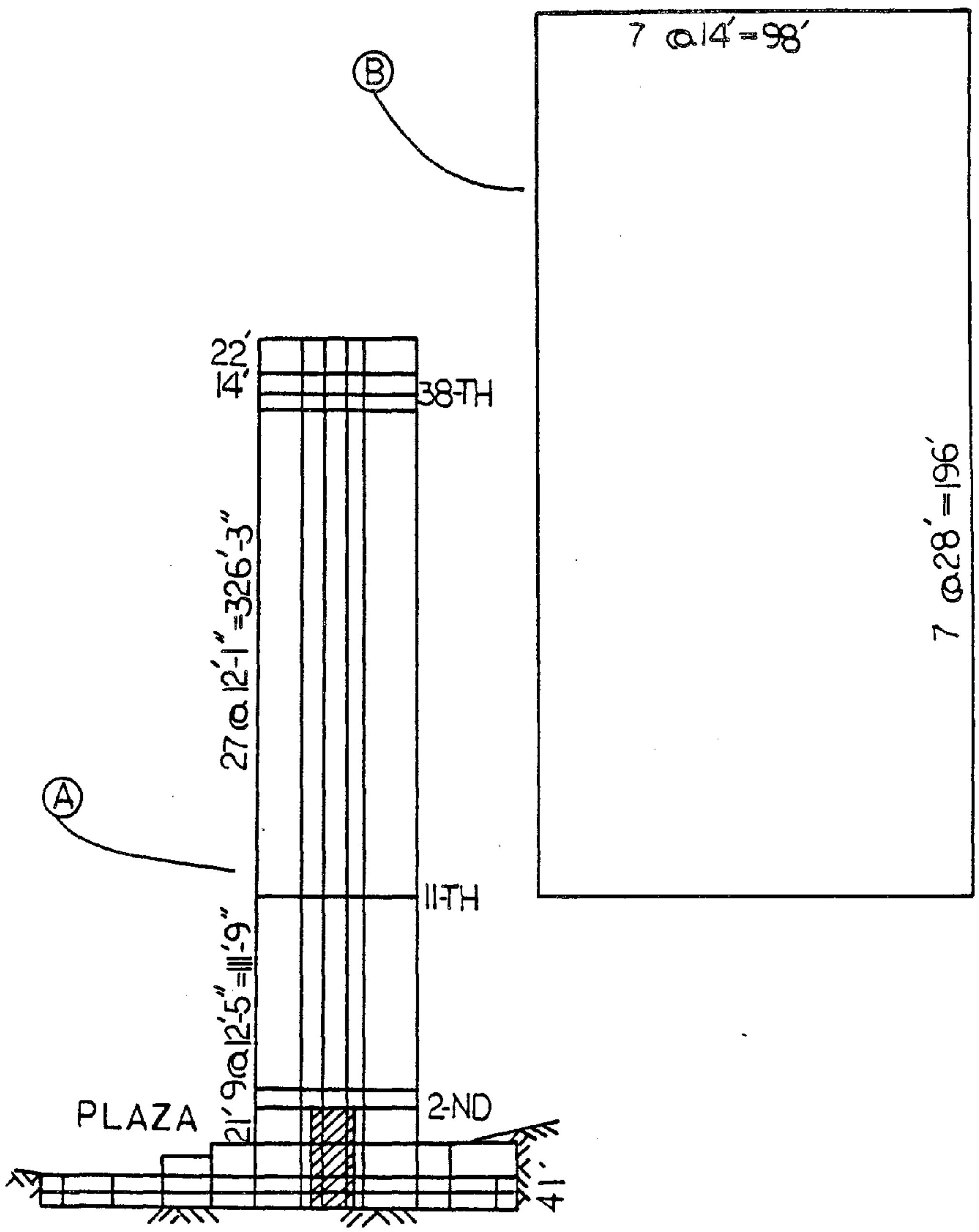


Figure 5-15: Union Bank Square:
(A) Transverse elevation.
(B) Typical plan.

Curtain walls consisting mostly of glass enclose the building. No mention was made of interior partitions or other nonstructural elements, but, apparently, there were nonstructural core walls and stair shafts of plaster in the building.

DESIGN & CONSTRUCTION

Designed in 1964, the UBS was one of the first very tall buildings constructed in Los Angeles. Many dynamic analysis computer techniques were pioneered in its design.

Gravity and live loads were based on standard code provisions, but the wind loads were increased with elevation. They exceeded local code requirements (30 psf vs. 20 psf), but were in conformity with the UBC. This wind loading produced shears in the lower stories, which exceeded the design seismic shears in the lower stories by only a small margin. The wind overturning moments greatly exceeded the design seismic overturning moments, but were less than the seismic moments indicated by a dynamic analysis. Design static seismic loadings exceeded local requirements by a factor of 2; they were selected to obtain a frame which would exhibit only nominal plastic behavior for response to the 1940 El Centro earthquake.

Dynamic analysis was used for member proportioning in order to obtain a structure of uniform strength. This resulted in small story drifts due to wind and seismic loading. Frame members were designed to develop plastic hinges without buckling. All connections in the moment resisting frames are welded connections capable to develop the plastic capacity of the beams. Column splices were designed to develop the plastic capacities of the girders at the connections above and below the splice. Most of the steel was A36 (36 ksi), stronger steel (42 ksi) being used in some exterior columns, reportedly, to maintain uniform column shortening due to vertical stresses.

The building apparently was built according to the specifications. Regular in plan, the soft story effect of the plaza level is compensated for by the shear walls extending out of the basements. Both the shear walls and the steel framing have the capacity to independently resist 100% of the lateral loading.

DAMAGE

Initial damage costs for repair of this $30 million structure were estimated at considerably less than $100,000; (the high premiums and deductibles of earthquake insurance are not accounted for). All earthquake damage was limited to nonstructural elements.

Nonstructural damage included plaster cracking in the longitudinal core walls and stair shafts, minor tile damage in the restrooms, and some caulking around plumbing fixtures. Seismic gap joints between the tower and garage were ruptured at the beam seat connections. Elevators were out of service for 6 to 8 hours, and some free standing bookcases overturned.

CAUSE

The UBS seismic behavior was entirely elastic resulting in no structural damage. The novelty of the computer dynamic analysis and the building's tallness probably instigated more thought behind the design of the UBS than would have been the case for a less challenging design of a smaller, more typical structure. It was shown that tall structures in downtown Los Angeles with long fundamental periods, greater than 2.5 seconds, sustained seismic loads well in excess of the local building codes.

6. THE MANAGUA, NICARAGUA EARTHQUAKE OF 1972. (72-12)

An earthquake having a Richter magnitude of 6 1/4 occured near Managua, Nicaragua, on December 23, 1972. Damage patterns suggested that the field epicenter was nearly directly under the city. At least one minor foreshock preceded and a series of aftershocks followed the main shock.

Only one accelerograph record, about 3 miles west from the epicenter, was obtained in this earthquake; the peak ground acceleration was 0.39g in the E-W, 0.34g in the N-S, and 0.33g in the vertical direction. The high amplitude portion of the record lasts for 5 seconds and has a "nominal" acceleration of 0.20g.

The city of Managua is underlain by a thick sequence of bedded volcanic deposits. The materials are poorly to moderately consolidated and of relatively low density. At many locations in a broad area centered around the city surface cracks developed. Fault-caused damage to structures in Managua is especially difficult to differentiate from damage caused by the severe ground motion produced by the earthquake. However, noticeable corridors of damage were observed along the two major fault traces in the eastern portion of the city.

REFERENCES: [Dewey, 1973], [EERI, 1973], [Facioli, 1973], [Hansen, 1973], [Knudson, 1973], [Leeds, 1973], [McLean, 1973], [Meehan, 1973], [Pereira, 1973], [Shah, 1973], [Valera, 1973], [Wright, 1973].

6.1 BANCO CENTRAL DE NICARAGUA

[Lin, 1973], [Wyllie, 1973], [EERI, 1973], [Meehan, 1973], [Wright, 1973].

GROUND MOTION & SITE

The Banco Central de Nicaragua building is located in the city of Managua. The earthquake intensity at the site was specified as IX on the Modified Mercalli Intensity scale. No pertinent information about the soil conditions was found.

STRUCTURAL SYSTEM

The Banco Central de Nicaragua is a 15-story tower with an enlarged floor area below the 4th floor, which accommodates a delegate assembly room and public banking facilities. Security vaults and mechanical equipment are contained in a deep basement, which covers the entire site. The tower measures approximately 145' x 47' in plan.

The structural system of this reinforced concrete building consists of moment-resisting frames in the tower stories, a flat slab system from the 4th floor down, and highly eccentrically placed shear walls enclosing the elevators at the W-end. Numerous security walls exist in the basement and 1st story, while the 1st through 4th stories contain numerous hollow-tile infill walls.

In the tower stories, closely spaced (4.6') columns (8" x 27 1/2") and beams along the perimeter support 18" deep beams spanning the full transverse width of the building. There are no interior columns. Slabs, approximately 2" thick, span between beams. At the 4th floor the closely spaced columns terminate at transfer girders (47" x 63"), which transfer the building loads to a total of ten columns, each 39" x 61" in size. The 2nd, 3rd, and 4th floors are cored flat slabs, and the 1st floor is a solid flat slab 18" thick. With the exception of the numerous security walls in the basement and 1st story, permanent partitions are of hollow-tile construction. The delegate assembly room on the S-side of the tower is framed with structural steel roof trusses, which are supported on reinforced concrete framing with hollow-tile infill walls.

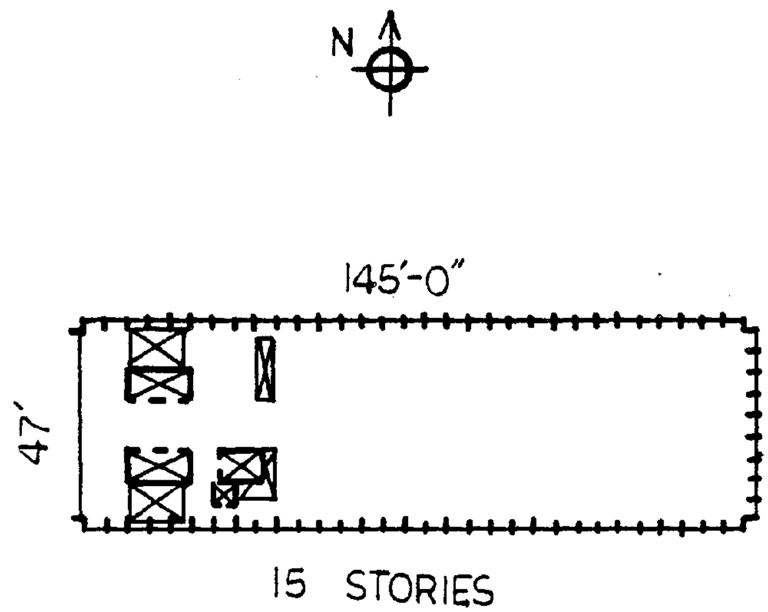


Figure 6-1: Banco Central de Nicaragua: Typical plan.

DESIGN & CONSTRUCTION

The structure's drawings are dated 1961, and the building was dedicated in 1964. The tower reportedly was designed for seismic forces derived from a lateral force coefficient of 0.10, which was uniform over the height. Furthermore a wind force equivalent to 10psf was applied to the structure acting simultaneously with the design seismic forces. This lateral loading criterion used in the design of the Banco Central de Nicaragua was considerably in excess of any building code requirements in the U.S. at that time.

DAMAGE

This building suffered considerable structural and nonstructural damage.

Structural damage was most heavy in the tower stories. Virtually all of the closely spaced reinforced concrete columns developed cracks at top and bottom in each story, and many exhibited considerable spalling. Cracks were found in the tower floor slabs immediately east of the elevator cores. The elevator walls in the lower stories experienced some diagonal shear cracking, indicating that they resisted sizeable loads. At the corner of some of the walls in the 4th and 5th story the concrete crushed, indicating high overturning moments. Small beams between the elevator walls were heavily damaged, as the walls interacted with each other and with the floor system. In the lower stories structural damage is considerably lighter, with the exception of the collapse of the delegate assembly room. The columns and beams of this room were only nominally sized. The hollow-tile walls apparently were intended to brace that portion of the structure, but they were obviously inadequate. Damage in the basement area was very slight, which was due to the high strength and rigidity provided by the heavy retaining and security walls.

Many of the hollow-tile infill walls in the 1st through 4th story eventually either failed or caused considerable damage to the columns between them. However, they added considerable initial stiffness and strength to these stories. There was extensive damage to architectural finishes such as ceilings, marble veneer, and windows in the lower floors.

Most striking in this tower was the extreme damage to the building contents. Mechanical equipment was also damaged during the earthquake. The elevators ceased functioning, when their motor generators slid on the floor and the counterweights left their guide rails. A water tank in the penthouse, rocking in its saddle, was damaged. Otner tanks and equipment slid on the floors. Broken water pipes caused extensive water damage in lower stories.

CAUSE

The Banco Central de Nicaragua relied on flexible frame action for its primary lateral resistance. However, the stiffer concrete elevator walls acted as shear walls and attracted high seismic forces, which damaged both elevator walls and floor slabs. Tension failures and cracks in the floor slabs adjacent to the elevators indicate that the floors were not adequately designed and reinforced for diaphragm action. The flexibility of the structure caused movements, which increased the damage of suspended ceilings, partitions and marble veneer. According to ATC 3-06 guidelines [ATC, 1978], this building is irregular both in plan and elevation.

Some recommendations are:

- The effects of torsion must be considered when the locations of the centers of mass and rigidity do not coincide.

- Horizontal diaphragms must be adequately reinforced to transfer all lateral loads to shear walls.

- Effects of lateral inertia forces on nonstructural elements, equipment and building contents must be considered in design.

6.2 BANCO DE AMERICA

[Mahin, 1975], [Rojahn, 1973], [Salna, 1973], [Sozen, 1973], [EERI, 1973], [Meehan, 1973], [Wright, 1973].

GROUND MOTION & SITE

The Banco de America building was adjacent to one of the main surface ruptures that traversed the city of Managua during the December 23, 1972 earthquake. The building lies on layered volcanic deposits (primarily cantera, a rock-like volcanic tuff agglomerate). The intensity of the ground motion at the site was specified as IX on the Modified Mercalli Intensity scale.

STRUCTURAL SYSTEM

The Banco de America building is a square, 75' x 75', 18-story, about 215' high tower with two basements extending in the east-west direction.

The structural system of this reinforced concrete building is very clear and regular. Square, two-way flat slabs are supported by closely spaced, T-shaped columns along the perimeter and by four large, L-shaped coupled cores symmetrically located in the interior. The columns have the same cross-section and reinforcement over the full height of the building except at the ground level. The reinforcement content is 1.43% and #4 ties are at 9.8" (250mm) spacing. The four cores are essentially of similar shape with details changing at the 4th, 11th, and 17th floor. Only the exterior core walls continue to the roof at the 18th story. The cross-sectional area of the shear walls is 4.1% of the total floor area, a higher value than typically used.

The L-shaped cores are coupled by pairs of coupling girders. Interior girders have the same thickness as the shear walls they frame into [9.8" (250mm)], while the exterior girders and remaining shear walls have different thicknesses. All the coupling girders between the first and penthouse level have 20" x 10" (50 x 25 cm) openings for ventilation ducts. On even-numbered levels, a 11.32" (3.45 m) square opening in the center of the floor was framed with beams running into the shear walls. The building rests on a single, deep mat foundation.

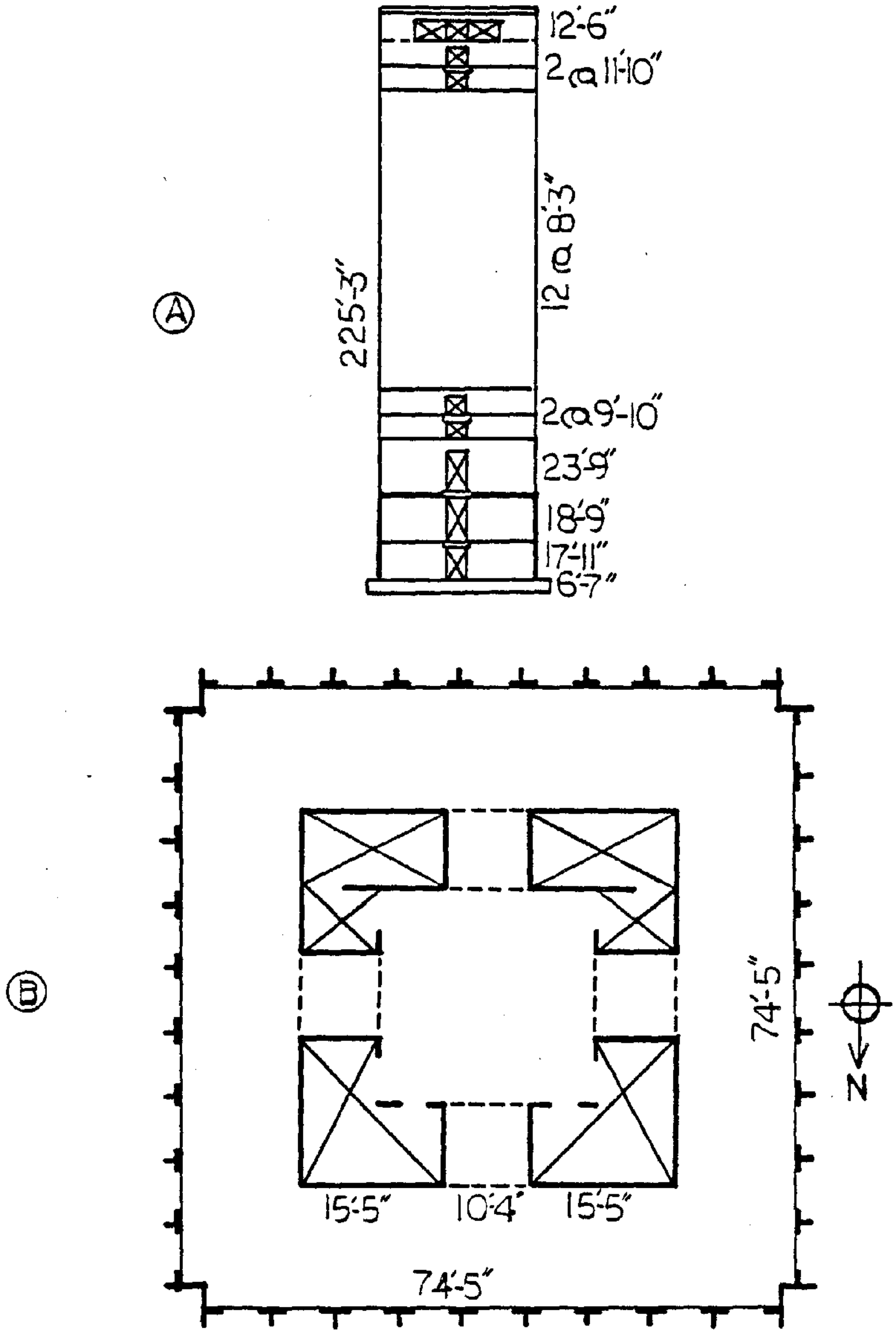


Figure 6-2: Banco de America:
(A) Typical elevation.
(B) Typical plan.

The few partitions existing in the tower portion of the building are constructed from lightweight hardboard.

DESIGN & CONSTRUCTION

Designed and built between 1963 and 1967, the building generally complied with the working stress provisions of the UBC then in force. Design assumed that lateral loads are resisted by the coupled shear walls, while perimeter columns serve only a gravity load bearing and architectural function. Design shear at street level was 5.55% of the total dead load (K=1.33). A 33% increase in allowable stress for seismic loads was used. Due to the distribution of the base shear over the height of the building, shears in the upper levels and overturning moments in the lower levels did not meet UBC code requirements.

In the design of the cores only the walls facing the exterior of the building were considered effective. This conservative approach yielded cores significantly stronger than required. Shear reinforcement in the coupling girders did not meet code requirements. Intermediate grade reinforcement and stone concrete (4ksi) was used. Quality of workmanship apparently was good.

DAMAGE

Damage to the Banco de America building was minor, even though one of the fault ruptures passed along the sidewalk adjacent to the building.

The main structural damage occurred in the coupling girders in the E-W axis of the building, which failed in shear at the duct openings. Extensive diagonal cracking occurred in the deeper unpierced coupling girders at the penthouse level. Floor slabs cracked above the failed coupling girders, at the slab - perimeter column connections, and along some of the slab - shear wall connections. In the upper stories, the shear walls suffered some diagonal and horizontal cracking and, over doorways, vertical cracking. No damage was visible in the perimeter columns.

Damage to nonstructural elements was minimal due to their flexibility and connection details. The elevators were inoperable, but the stairways remained clear of debris.

CAUSE

Despite the systematic failure of the coupling beams, the overall evaluation of the performance of the building must be a positive one. Except for the beam damage, there was very little structural and nonstructural damage in the building, a fact attributable to the stiffness as well as strength of the shear walls. A structural system combining ductile coupled shear walls with a ductile framed perimeter tube, appears to be excellent for resisting strong earthquake motions. The symmetry of the structural system and the large ratio of shear wall to floor area contributed to the excellent behavior of this building.

The primary reason for the coupling girder failures was insufficient shear capacity. The girders were strong in flexure and spanned a short distance creating high shears which were not considered in design. To maintain the ductility of the system, flexural reinforcement should be reduced and shear strength increased, so that the flexural capacity can be developed without premature shear failure. This will allow inelastic flexural deformations without substantial loss of strength under large numbers of load reversals. Loss of the coupling action due to shear failure has increased the natural period, displacements, and drifts. In this specific case, this softening appears to have actually helped the building.

Damage to the slab connections at the shear wall and the perimeter columns would have been reduced, had the coupling girders functioned properly. The eccentric slab - perimeter column connection is a poor detail and should be avoided in design.

This building is regular in plan, elevation, mass distribution, and lateral resistance according to ATC 3-06 guidelines [ATC, 1978].

Some recommendations are:

- Girders should be reinforced for the shear forces associated with the development of the actual flexural capacity rather than for code design shears.

- Buildings with regular and simple structural systems have more chances to survive a severe earthquake.

6.3 ENALUF ADMINISTRATION BUILDING

[Hanson, 1973], [Lin, 1973], [Nicoletti, 1973], [EERI, 1973], [Meehan, 1973], [Wright, 1973].

GROUND MOTION & SITE

The ENALUF administration building is located in downtown Managua. The earthquake intensity at the site was specified as VIII on the Modified Mercalli Intensity scale. No pertinent information was available on the soil conditions.

STRUCTURAL SYSTEM

The ENALUF Administration Building is the main office building for the electric power company. The main part of this reinforced concrete building measures 60' x 140' in plan and comprises 8 stories, each about 12' high. Five full stories plus a top "mechanical" story are completely above highest ground level. The ground level varies from the floor of the 1st story on the N-side of the building to the ceiling of the 1st story on the S-side. A full basement is below the 1st story.

The structural system consists of four longitudinal reinforced concrete frames and shear walls forming central cores. The shear walls of the cores were typically 8" thick and had openings. The largest columns vary in size from 20" square in the basement to 16" square in the 2nd and higher stories. The smallest columns in the basement measure 18" square. All columns are equally spaced at 20' in both directions and arranged on 4 frame lines in the longitudinal (E-W) direction, and on 8 lines in the transverse (N-S) direction. Girders run in the E-W direction. The floor system consists of precast joists, 13" (33 cm) thick, and cast-in-place concrete on arch type filler forms between joists.

Outside views of the building show that a heavy curtain wall is discontinued above the 1st story. Between the 1st story columns there are arches of architectural concrete in both directions. Information about interior partitions and other nonstructural elements was not available.

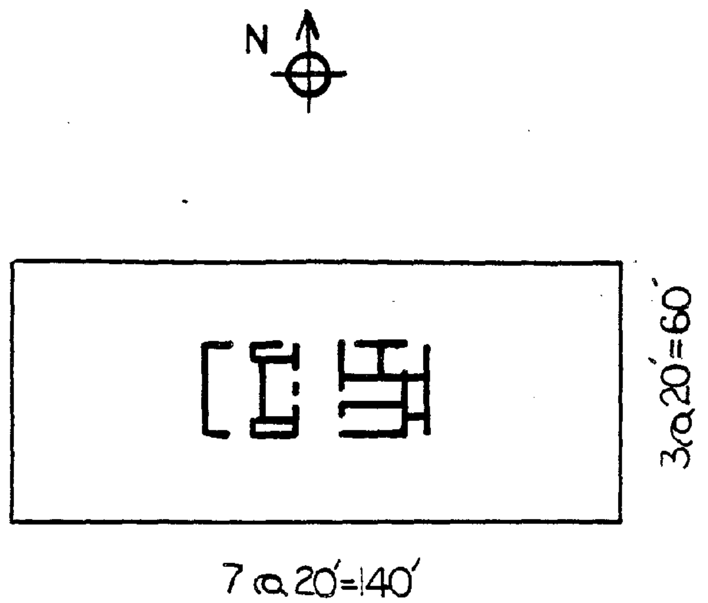


Figure 6-3: EMALUF Adminstration Building: Typical plan.

DESIGN & CONSTRUCTION

The design and construction of the ENALUF building appeared to be in accordance with the specifications. However, the observed locations of the openings in the core shear walls did not fit the structural plans, although they did fit the revised architectural plans.

DAMAGE

This building suffered only minor damage. It remained unoccupied for a few days after the earthquake.

Overall, structural damage was slight. Exterior columns suffered minor cracking. The core shear walls in the 1st and 2nd story showed cracks, some of which were large enough to suggest possible yielding of the reinforcing steel. The columns near the cores were lightly cracked all the way up to the 4th floor.

Nonstructural damage reported consisted of spalling of plaster and cracking of architectural concrete at exterior columns.

CAUSE

On the whole, the performance of the building was good. There was very little architectural damage. The minor cracking of the shear cores points, in addition to possible errors in construction, to the problems associated with the design of shear walls with openings.

The concentration of damage in the 1st story can partly be attributed to the termination of the heavy curtain wall, which created a slightly more flexible story at this level. The stiffness provided by the shear walls resulted in considerably less nonstructural damage than usually was observed during this earthquake in buildings relying on frame action only.

6.4 HOTEL INTERCONTINENTAL

[Aktan, 1973], [EERI, 1973], [Meehan, 1973], [Wright, 1973].

GROUND MOTION & SITE

The Intercontinental Hotel is located at the southern edge of downtown Managua. The earthquake intensity in this area was VIII on the Modified Mercalli Intensity scale. No pertinent information about the soil conditions at the site was found.

STRUCTURAL SYSTEM

The Hotel Intercontinental is a 9-story building with mechanical penthouse and partial basement. The shape is somewhat unusual in that the E- and W-face have several setbacks from bottom to top creating a pyramidal appearance. The 1st story has plan dimensions of 312' x 93' and a height of 16.4'. The 3rd story is set back measuring 265' x 48.5'. In each succeeding story the long dimension is reduced by 24'. The 9th story houses a lounge. Above is a mechanical penthouse. Typical story heights are 9.7'. The partial basement covers about half of the 1st story area.

The structural system of this building consists of reinforced concrete frames, concrete block exterior infill walls, and reinforced concrete shear walls around elevator shafts and in several other locations. The latter extend only up to the 2nd floor level. In the south or rear, the concrete exterior wall for the basement extends up to the 2nd floor except for 72' at the ends where there is no basement and the exterior wall is nonstructural. Columns are tied columns spaced at 24'. The floor system consists of concrete slabs with precast concrete joists and tile fillers. The slabs span between the joists, which are supported on reinforced concrete beams.

All interior partitions are of hollow-tile construction in all stories, except in the mechanical penthouse.

DESIGN & CONSTRUCTION

Not much is known about the design assumptions and construction methods used in this building.

DAMAGE

The Hotel Intercontinental suffered significant structural and severe nonstructural damage. The hotel was occupied at the time of the earthquake, but was vacated and not open to public after the earthquake.

The major structural damage occurred at the mechanical penthouse and in the 2nd story. The mechanical penthouse experienced a complete column failure and collapsed. At the 2nd floor level a perimeter beam failed in tension causing the failure of four tied columns at the W-end due to the subsequent excessive movement. The perimeter beam at this point was inadequate to transfer the high forces to the rigid concrete shear wall.

The concrete frame suffered only some minor cracking. However, the exterior infill walls from 2nd to 5th story experienced extensive damage. Interior partition damage was severe in the penthouse story and slight in the top story. It increased to moderate moving down to about the 2nd story and was slight again in the 1st story. Furniture and fixtures were moved and thrown over in rooms.

CAUSE

The building experienced extensive damage to the concrete block walls, which provided the primary stiffness of the structure. Although not designed as structural elements, these walls resisted most of the seismic forces together with the shear walls found at several locations in the building. If the concrete block walls had not been present, the total damage would have been much more extensive. The penthouse, lacking infill walls, experienced a column failure. Its exterior columns were shorter because of an intermediate beam, which supported the bottom of precast panels. The damage pattern can be explained by the tapered shape of the building resulting in increased mass and stiffness at lower stories. Seismic forces and damage were concentrated into the concrete block walls at lower levels except for the 1st story where shear walls were present to resist these forces.

Some recommendations are:

- Consider the effect of structural and nonstructural walls when combined with moment resisting frames.
- Setbacks and other irregularities deserve increased research.
- Improve the art of earthquake design for nonstructural items.

6.5 SOCIAL SERVICES BUILDING

[EERI, 1973], [Meehan, 1973], [Wright, 1973].

GROUND MOTION & SITE

The Social Services Building is located in downtown Managua. The earthquake intensity at the site was specified as VIII on the Modified Mercalli scale. No pertinent information about the soil conditions was found.

STRUCTURAL SYSTEM

The Social Services Building is a reinforced concrete structure with a 9-story tower, and two 2-story wings facing east. It is a "T" shaped building, the tower forming the web and the wings the flanges of the "T". A basement extends under the wings. The tower portion is rectangular and measures about 43' X 116' in plan.

The structural system is a reinforced concrete frame with shear walls enclosing an elevator core. The elevator is located near the center of the building and has 8" thick reinforced concrete walls. Two rows of 5 spirally reinforced columns, spaced at 25.6' on center, support the two-way joist floor system. The floors cantilever on all four sides over the exterior columns. The floor system is 16"deep including the slab (2" thick). Precast concrete forms are used to form the joists. The spiral columns are 37" in diameter in the basement and decrease gradually to 16" in the 9th story. The 2-story wings are structurally connected to the tower and framed similarly with two-way joist floor slabs and spirally reinforced columns. The foundations are spread footings.

The rear or W-wall of the tower consists of hollow unit masonry supported on the cantilevered slabs. The other exterior walls are all of curtain-wall construction.

DESIGN & CONSTRUCTION

The building was constructed in the period 1960 to 1962. Information on design assumptions and construction methods and practices was not available.

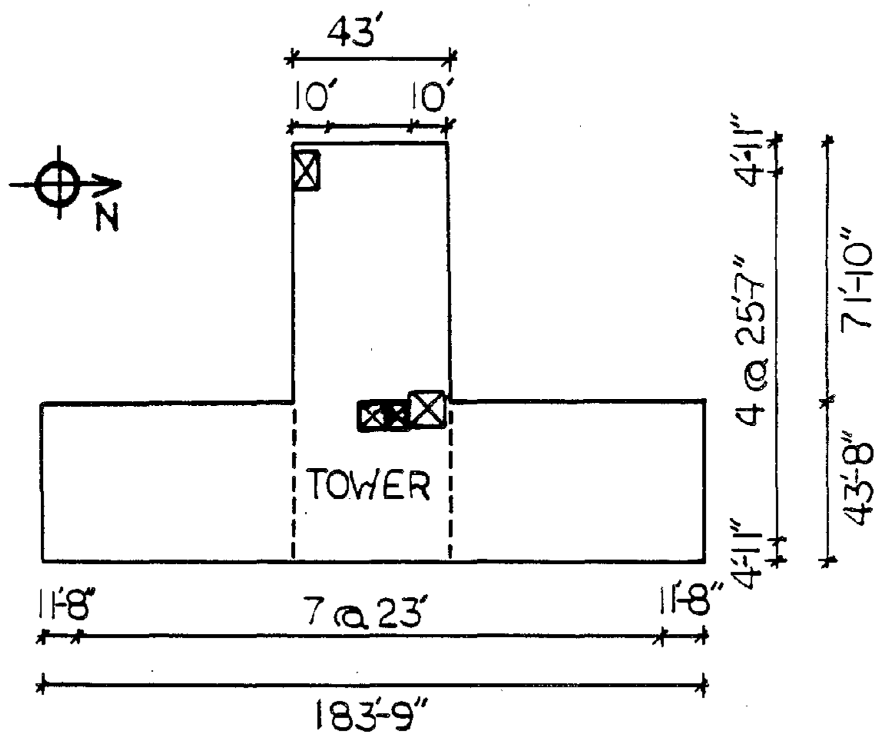


Figure 6-4: Social Services Building: Typical plan.

DAMAGE

The Social Security Building suffered significant structural and moderate nonstructural damage.

The large spiral columns showed spalling at their ends at some locations. There were numerous flexural cracks in the floor slabs. The cracks on the top of the slab were usually near the ends of the top reinforcement. There were also extensive cracks in the slab soffits. Cracks across the entire building were observed at the W- and E-face of the elevator core due to diaphragm tension and slab bending at that location. Most of the slab cracks run N-S indicating primary building response in the E-W direction (in the "T" stem). However, there were also some E-W cracks, especially at the re-entrant corners of the "T". A mechanical penthouse collapsed; apparently, anchorage of the vertical column reinforcement in the slab failed. This illustrates how spiral columns cannot perform in a ductile manner when their anchorage is inadequate to develop their flexural strength.

The interior stairways were heavily damaged due to strut action between the deflecting stories. Partitions and exterior curtain walls, although slightly damaged, performed remarkably well.

CAUSE

The importance of adequate continuous top and bottom reinforcement in slabs or beams is clearly demonstrated in the Social Security Building. In actual earthquake response with forces exceeding those used in conventional design, tension will develop in the top and bottom of continuous slabs or beams in regions not indicated by the usual calculations. Continuous top and bottom reinforcement not only provides resistance to these tension or moment reversals, but also ties the structure together, thus minimizing diaphragm distress.

Some additional recommendations are:

- Improved confinement and reinforcing details are necessary, in order to achieve adequate ductility of structural components.

- Attention should be given to the design of buildings with structural irregularities.

6.6 SUPREME COURT BUILDING

[EERI, 1973], [Meehan, 1973], [Wright, 1973].

GROUND MOTION & SITE

The Supreme Court Building is located to the west of the downtown section of Managua. The earthquake intensity at the site was estimated as VIII on the Modified Mercalli Intensity scale. No information was available on the local soil conditions.

STRUCTURAL SYSTEM

The Supreme Court Building is a reinforced concrete structure consisting of a 6-story tower and a 2-story portion surrounding the tower on the W-, N-, and S-side. Tower and 2-story portion are structurally separated. A basement extends below the building. The tower measures 92' x 82' in plan. The 2-story low-rise structure is 242' x 95' in plan. The lower structure has numerous open bays dividing it into separate substructures. These substructures are inter-connected only by corridors and nominal beams on column lines. In one case an expansion joint was provided across the corridor.

The structural system of the tower consists of reinforced concrete frames and L-shaped shear walls at the four corners. The L-shaped shear walls are 12" thick and each leg measures 13' in length. The frames consist of tied columns and cast-in-place post-tensioned beams. The floor systems are slabs on precast concrete joists spanning between the post-tensioned beams. The framing of the low-rise structure is similar to the tower. There were a few concrete walls in the low-rise structure, none being effective as shear walls.

The entire complex contained numerous hollow-tile walls, many of them with marble veneers.

DESIGN & CONSTRUCTION

The Supreme Court building, of modern concrete construction, was built in 1967. The framing was quite complicated and relied on frame action for the low rise portion and evidently on shear walls for the structurally separated tower portion.

DAMAGE

The tower with its shear walls had very little structural damage. The low-rise structure had considerably more damage, both structural and nonstructural.

Structural damage was most spectacular in the low-rise structure in the areas adjacent to open bays. Inadequate capacity of the floor diaphragm due to discontinuities resulted in heavy damage to perimeter beams. Beams bounding open bays were more severely damaged. Beam column joints exhibited some spalling in certain locations. In the tower there was some column damage in the 6th story. Some cracking was noted in the concrete walls of the tower, primarily from overturning moments, but also some shear and pounding-induced cracking. There was pounding between the tower and the low-rise structure.

The building suffered extensive damage to ceilings and hollow-tile walls. Numerous hollow-tile walls cracked and shattered; several tile walls at the top floor and roof of the tower collapsed. Some heavy ceilings collapsed also. Marble veneer was extensively damaged.

CAUSE

The low-rise structure, relying only on flexible frames for seismic resistance, experienced greater nonstructural damage than the tower structure with the L-shaped shear walls. The rigid nonstructural filler walls were damaged by interaction with the frames. Discontinuities in the floor diaphragms of the low-rise portion were not adequately compensated by reinforcement and this resulted in increased damage at these locations. The penthouse, as a separately attached structure on the roof, was apparently subjected to amplified motion and experienced significant damage.

Some recommendations are:

- The stiffness and strength of nonstuctural and structural walls must be considered when combined with moment resisting frame.

- Adequate design of horizontal diaphragms to transfer lateral loads is needed.

- A better evaluation of seismic gaps should be made.

6.7 TELCOR BUILDING

[EERI, 1973], [Meehan, 1973], [Wright, 1973].

GROUND MOTION & SITE

The Telcor Building is located in the city of Managua at the N-W corner of 6a Calle and 2a Avenida. In that area, the earthquake intensity was VIII on the Modified Mercalli Intensity scale. No pertinent information on the soil conditions was available.

STRUCTURAL SYSTEM

The Telcor Building is a 7-story reinforced concrete building with a partial mezzanine. The building, used as office space for telephone communications personnel, is approximately 85' x 47' in plan and 89' high.

The structural system of this building consists of reinforced concrete moment-resisting frames and eccentrically located shear walls enclosing the elevator shaft and stairwell at the W-end. Longitudinal frames are located on the perimeter only. Transverse frames span the full width of the building using post-tensioned reinforced concrete beams. The post-tensioning tendons are parabolically placed, and the beams contain both top and bottom mild steel. The beams are typically 31 1/2" deep and 12" wide and increase in width to 18" near the columns. The columns are typically 18" x 27 1/2". The corner columns have a major dimension of 53", tapering larger below the 2nd floor, apparently for architectural appearance. The floors are framed by precast concrete joists, spaced at 24.6", which span up to 13.6' between the beams. Hollow ceramic blocks fit between the joists, and a 2" poured-in-place reinforced slab covers the blocks and joists. The foundations are spread footings.

The walls forming the E- and W- facades as well as the infill panels to windowsill height on the N- and S- facades are of hollow-tile construction.

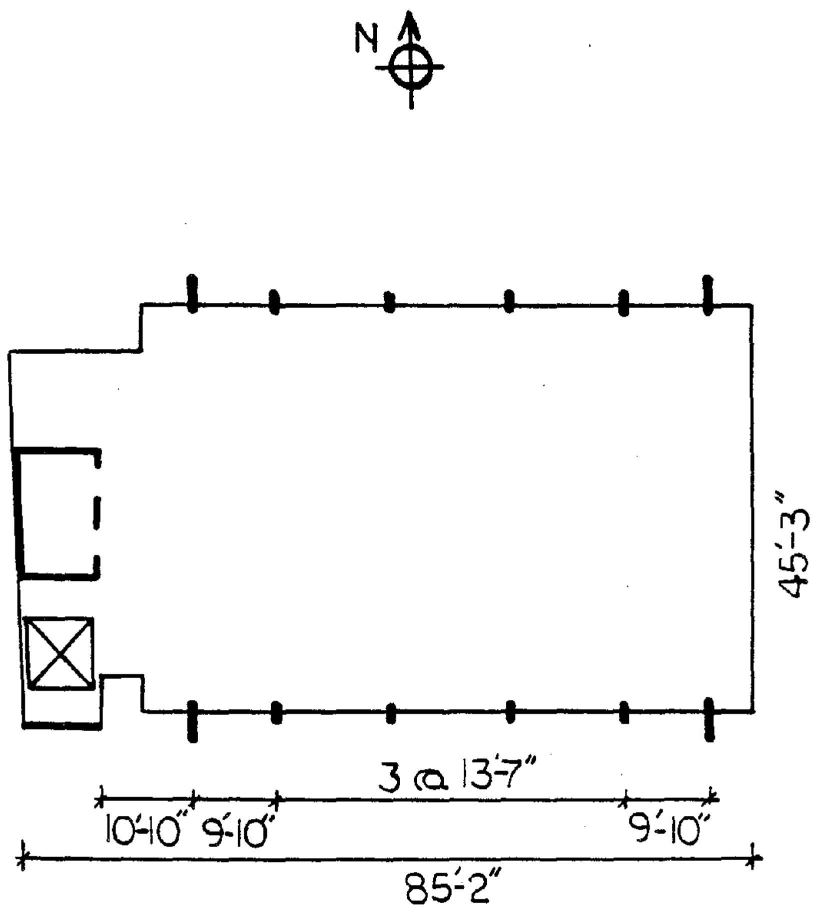


Figure 6-5: Telcor Building:
Typical plan.

DESIGN & CONSTRUCTION

The Telcor Building was constructed in 1967. The design intended that the lateral loads are to be resisted by moment resisting frame action.

DAMAGE

The building suffered extensive structural and nonstructural damage. Repair costs are unknown.

Practically all structural members were damaged. Reinforced concrete walls were heavily cracked and all floors had separated from the elevator core, where reinforcing was inadequate to transfer forces between the floor diaphragms and the concrete walls. Some floors had cracks about 1" wide. Some columns experienced spectacular failures. However, many other columns suffered only very slight shear cracking. The hollow-tile windowsill-height infill panels definitely interacted with the columns, damaged them, and, in turn, were heavily shattered themselves. The post-tensioned beams supporting the mezzanine near the E-end had diagonal cracks. A few upper floor beams also located near the E- end of the building had similar diagonal cracks. It appears that these cracks were a result of seismic frame action, vertical accelerations, and reverse curvature of the tendons near the columns.

A large amount of damage was found in partitions and in the exterior walls of the main elevator and stair exit. Damage in the penthouse was severe - those elevator counterweights that could be seen, had left their guides. A roof tank shifted about 2' breaking pipes and flooding the building below. Elevator machines were placed on large concrete pads with rubber pads under them. The whole units shifted on the rubber pads.

CAUSE

The building appeared to respond to the earthquake primarily in the E-W direction. Although it was designed to resist lateral forces by frame action, the elevator walls at the W-end were stiffer than the frames and initially resisted a high percentage of the lateral forces. Strong N-S excitation would have accentuated torsional response because of the eccentricity of the concrete core and, undoubtedly, would have increased the damage. The diaphragm failures at the shear walls illustrate the need to provide for potential tension forces in floor diaphragms. The authors of the source literature believe that, "the cracks in the post-tensioned beams indicate that caution is required, when frames using post-tensioning are used to resist lateral forces", although the eccentricity of the core might be as likely an explanation for the cracking at the E-end.

As in other frame buildings the flexibility of the frames resulted in heavy damage to nonstructural elements.

Some recommendations are:

- Adequately design horizontal diaphragms.
- Avoid eccentricities in the lateral force resisting system.
- Secure equipment and building contents against seismic movements.
- The stiffness and strength of nonstructural and structural walls must be considered when they are combined with moment resisting frames.

7. THE IMPERIAL VALLEY EARTHQUAKE OF OCTOBER 1979. (79-10).

A moderate magnitude earthquake (M=6.6) occurred on October 15, 1979 in the southern Imperial Valley of California. The earthquake had a shallow focal depth and was generated by lateral slip on the N-W trending Imperial fault. Faulting produced approximately 19 miles (30 km) of surface rupture. The earthquake was very similar to the Imperial Valley earthquake of May 18, 1940 (M=6.7). The maximum recorded acceleration of the October 15, 1979 earthquake at El Centro was 0.38g vertical and 0.40g horizontal. The duration of strong shaking (>0.1g) at El Centro was about 7 seconds.

Damage from the earthquake, estimated to be $30 million, was most evident in residential areas of Southern Imperial County and northwestern Baja California. Structures damaged included the multi-million dollar Imperial County Services Building in El Centro, mobile homes, a concrete block wall, bridge abutments, and metal grain elevators. The agriculture industry also suffered high dollar losses from the earthquake.

REFERENCES: [Gonzalez, 1980], [Brandow, 1980], [Garr, 1979], [Real, 1979], [Porcella, 1979].

7.1 IMPERIAL COUNTY SERVICES BUILDING

[Gonzalez, 1980], [Brandow, 1980], [Garr, 1979].

GROUND MOTION & SITE

The Imperial County Services Building is located in El Centro, approximately 4.7 miles S-W of the epicenter of the Imperial Valley earthquake. Accelerometer readings from the building indicate 10 seconds of strong shaking with 0.30g peak ground acceleration. The earthquake intensity at the site was estimated as VIII on the Modified Mercalli Intensity scale.

The building lies on alluvium material consisting primarily of sand with interbeds of clay.

STRUCTURAL SYSTEM

The Imperial County Services Building is a 6-story reinforced concrete office building, 136'-10" x 85'-4" in plan and 81'-8" high. The building is 5 bays long (E-W direction) and 3 bays wide (N-S direction), all bays being 25' long.

The structural system consists of four longitudinal frames and transverse shear walls. There are no longitudinal shear walls in the building. At the E- and W- ends of the building, the exterior facade above the 2nd floor is a shear wall located 5'-11'' outside of the first column line. These exterior walls are the only transverse walls above the 2nd floor; they extend the full width of the building, but are discontinued at the 2nd floor, leaving open ends in the 1st story. The two ends are slightly different; the W- end wall has a "smoke tower" opening in the middle of the wall, and, in addition, there is a center bay shear wall in the 1st story, which is "set back" in line with the interior columns. The E-end wall is completely discontinued and supported on the "set back" columns. Three more shear walls are found inside the 1st story center bay.

The four 1st-story walls are 12" thick. The exterior walls are 7 1/2" thick in the 2nd story and 7" thick above that story. Longitudinal framing is comprised of 10" wide by 4'-6" deep spandrels (except below the 2nd floor level)

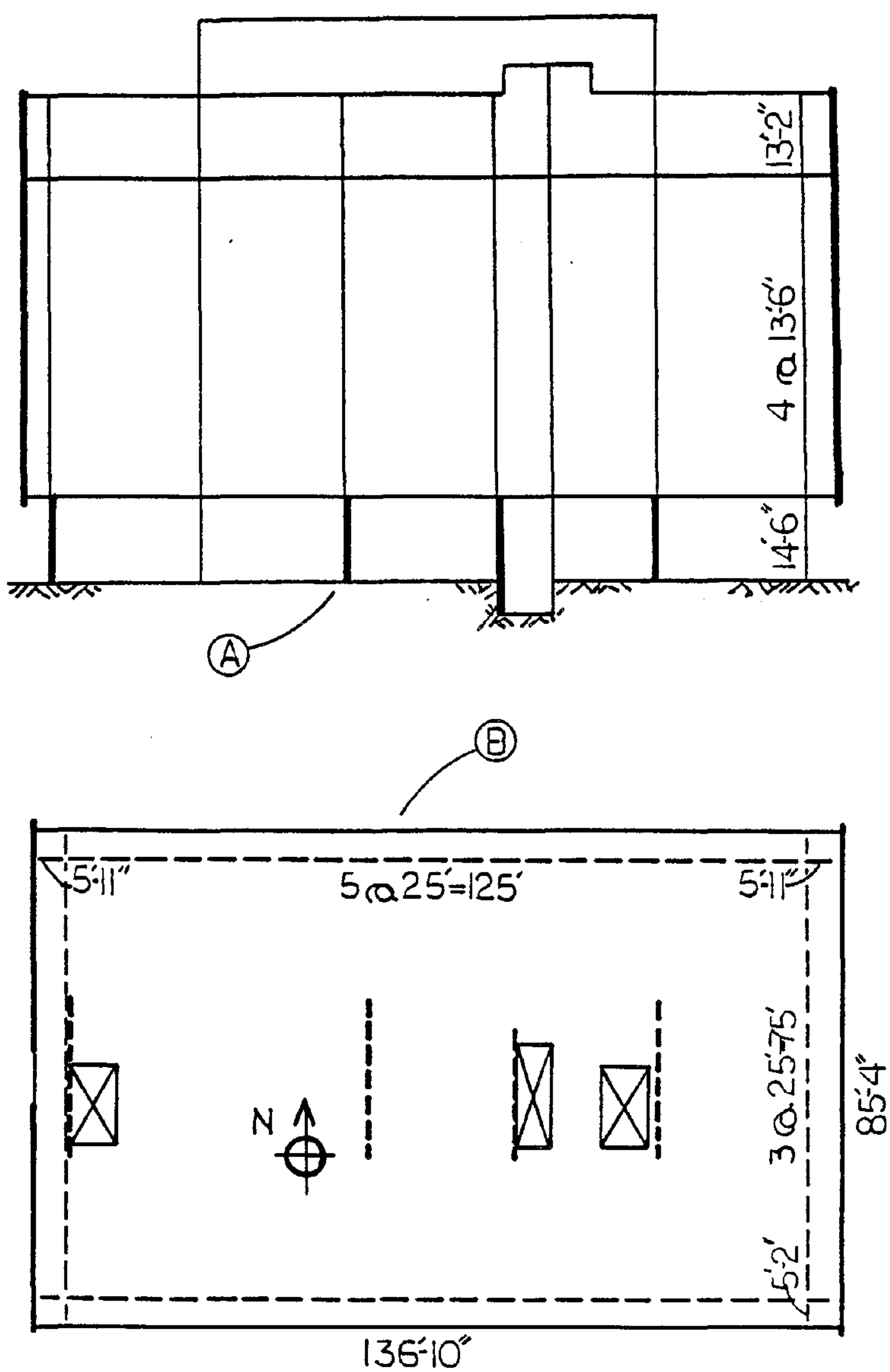


Figure 7-1: Imperial County Services Building:
(A) Typical longitudinal elevation
(B) Typical plan

on the exterior column lines, and 2' wide by 2'-6" deep girders on the two interior column lines. The interior columns are all 24" square. The exterior columns are 24" square up to the 2nd floor. Above that level they are tapered in width from 18" to 10" over a length of 5'-10". The 2nd floor slab extends roughly the same distance to form a platform for the building above, again reflecting a building on stilts. The floor slabs are 5" thick at the 2nd floor and 3" thick at the other floors. They are supported on pan joists running in the transverse direction.

The building is founded on a Raymond step-taper concrete pile foundation. The piles are interconnected with reinforced concrete link beams.

DESIGN & CONSTRUCTION

The Imperial County Services Building was designed in 1968 according to the 1967 edition of UBC. It was completed in 1971 at a cost of $1.87 million.

The earthquake forces are resisted by four moment resisting frames in the longitudinal direction and by the discontinuous exterior shear walls in the transverse direction. At the 1st story these walls are replaced by four 25' wide shear walls. The 1st story walls include the set-back W-end wall and three interior walls, all located in the center bay.

Some ductile concrete type details required by the 1967 UBC were used in the design, but not throughout. For example, column ties were extended through the girder depths, column bars were spliced at midheight and continuous top and bottom steel was used in the girders. These provisions did not include the current code requirements for special transverse column reinforcement full height under discontinuous shear walls.

DAMAGE

In an area, where most of the older, unreinforced masonry buildings suffered little damage, the failure of the Imperial County Services Building was dramatic. Although it did not collapse, it had to be demolished and replaced by a new building.

Structural damage was mostly limited to the 1st story; all four columns at the E-end failed. Because of the failure of the columns, the 1st story shortened by one foot or more on that line causing the end bay framing to hinge at the 1st interior column line. A major crack across the building at that location was visible on all floors. In other parts of the 1st story, some columns suffered spalling of concrete and typical X-cracking just above the ground floor slab. No significant structural damage was observed in the upper stories. However shear cracks were found in the floor diaphragms. The exterior shear walls exhibited diagonal tension cracks and showed effects of minor movement along construction joints, but there was no major distress in the walls.

The interior of the building was a mess from the standpoint of its contents. However ceiling and partitions were in good condition.

CAUSE

The condition at the E-end represents a classic instance of shear wall discontinuity: an abrupt change of strength and stiffness occurred at the 2nd floor, where the shear wall was terminated. The failure of the 1st story columns resulted from combined high axial force and overturning moments in both principal directions of the building. The crack patterns observed in the columns indicate that the columns were subjected, near their bases, to a high shear in the E-W direction, which is parallel to the unbraced frames, not to the shear walls. An unfortunate detail was that the closely spaced ties at the lower end or the column did not extend significantly above the slab on grade. It appears also that the fixity of the column bases provided by the pile caps resulted in higher bending moments at that location than at the top of the 1st story columns. For the corner columns the overturning moments may have exceeded design moments by as much as 4 times. At the W-end the stiff ground level shear wall protected the columns from large axial and shear forces.

Some recommendations are:

- Discontinuities in the structural system should be carefully designed; otherwise they should be avoided.

- Change code load factors for those earthquake induced forces on columns, that are not controlled by girder hinging.

- Where ductility of a system is not achievable, design members for higher factor of safety.

8. PRELIMINARY ASSESSMENT OF DATA

The scope of this chapter is to make a preliminary assessment of the data that has been collected in this study this far, and to gain experience with the methods that can be used in an empirical investigation of the earthquake resistance of high-rise systems.

As stated in the introduction, the ultimate objective of the project is to investigate the correlation of tall building systems and earthquake damage. Conceptually, the problem can be formulated as a relationship between earthquake, E, vulnerability of tall buildings, V, and damage, D:

$$D=f(V,E) \tag{1}$$

The vulnerability of high-rise buildings, in turn, is expected to depend on their characteristics, e.g. material, M, configuration, C, tallness, T, structural system, S, architectural system, A, etc.

$$V=g(M,C,T,S,A,...) \tag{2}$$

The goal is to investigate which of those characteristics or parameters significantly influence vulnerability and, hence, damage and how they do so. The next three sections present the classification of the characteristics of tall building systems, earthquakes, and damage. The fourth section presents the data organization, and the fifth, finally, analysis and results.

8.1 CLASSIFICATION OF TALL BUILDING SYSTEMS

The definition of a tall building, as described in the Monograph of the Council of Tall Buildings and Urban Habitat, specifies no minimum height: "The important criterion is whether or not the design is affected by some aspect of tallness" [Council, 1978]. In this study, information on buildings with 6 to 40 stories was collected. While tallness is usually expressed by height or number of stories, in the context of an earthquake study, the fundamental period (T) of the building is a useful measure, because seismic response directly depends on it. In the following analysis of the data the fundamental period is therefore used to distinguish tall and less tall buildings.

The building systems that need to be classified are the structural, architectural, and mechanical systems. In the literature, we find two general approaches of classifying tall buildings: analytical and synthetical classification schemes. In the analytical approach [Lu 1974, ATC 1978, UBC], structural systems are dissected into subsystems according to function (gravity load resisting system, lateral load resisting system, energy dissipation system), and classified based on the type of these subsystems. A drawback of this approach is that structural systems often serve multiple functions and the functional distinctions are rather artificial. In the synthetical approach [Schueller 1977] structural systems are classified in their entirety based on the type of framing concept used (frame, core, wall, tube systems). A drawback of this approach is that each new framing concept requires a new class.

The classification scheme proposed by Falconer and Beedle [1981], combines both approaches (Appendix A, Tables A-1 to A-4). At a first level the scheme follows the synthetical approach. The structural system is classified into four "prime" classes (bearing wall, core, frame, tube), and their combinations. While the first level requires minimum information (sketch of plan and elevation), the classification of structural bracing at the second level asks for more detail. Because structural bracing inherently relates to the lateral load resisting function, an element of the analytical approach is introduced at this level. Also classified at this level are configuration (regularity, irregularity of plan and elevation), floor framing, and material. The structural system classification is accompanied by the classification of architectural and mechanical systems.

Depending on the nature of the problem studied, different degrees of detail in classification are needed. Certain aspects of a tall building important for one study may not be relevant for an other. Using the Falconer-Beedle classification in this work, some refinements were deemed necessary and are proposed in Appendix A (Tables A-1 to A-4). In these Tables all additions are marked with an asterisk (*). Some more structural systems that can be classified by a plan and elevation sketch alone are added at the first level. Some other additions of systems needed for an earthquake study are suggested at the second level. One is the classification of foundation systems. Finally, in the classification of architectural systems, the distinctions of weight and isolation of the nonstructural elements from the structural framing are introduced.

8.2 CLASSIFICATION OF EARTHQUAKES

Earthquakes are generally characterized by their magnitude and their intensity. The magnitude (M) is a measure of the earthquake at the source and is related to the energy released by the fault rupture. Because the ground motion decreases with distance from the source, a measure for the local destructiveness is also needed. The severity of a ground motion at a given site is measured by the intensity. Subjective scales or instrumental measures are used to characterize the intensity.

Subjective scales are based on observations of the effects of a ground motion on natural and man-made objects. They represent the most commonly used and available measure of intensity. In the U.S., earthquakes are typically rated using the Modified Mercalli Intensity (MMI) scale, whereas in other parts of the world the Medvedev - Sponheuer-Karnik (MSK), Japanese Meteorological Agency (JMA), Rossi-Forel (RF), and other scales are used.

An instrumental measure containing complete information on ground motion is the accelerogram. Yet, destructiveness is difficult to assess with accelerograms alone, without dynamic analysis of structures. The peak ground acceleration is often chosen when a one-parameter description of earthquakes is needed. Equally important, however, are the peak ground velocity, the peak ground displacement, the frequency content, and the duration of the earthquake. For engineering purposes, the most useful instrumental measure is the response spectrum. Except for effects of duration and long acceleration pulses, elastic response spectra contain most of the information needed to assess structural response. To characterize ground motion by a single quantity, the response spectrum intensity introduced by Housner (the integral of the spectral pseudo-velocity over the range of structural periods) can be used rather than the entire response spectrum. Another measure derived from response spectra is the Engineering Intensity Scale (EIS) [Blum, 1970]. Essentially, it is a classification scheme for reporting ranges of spectral pseudo-velocity for ranges of building periods.

The primary problem of subjective scales in the present context is a methodological one. One objective of this study is to derive the vulnerability, V, of a particular class of buildings from the earthquake intensity, E, and the damage, D. Clearly the earthquake does not depend on the vulnerability of the buildings and, hence, neither should the measure for earthquake intensity, E. However, if building damage is used to assign MMI, then MMI also depends on the general level of vulnerability of the buildings in the area. In other words, MMI measures both the earthquake and the general vulnerability level. Therefore, comparisons between the performance of buildings in areas with significantly differing levels of design and construction quality are problematical. Scholl [1982] mentions that MMI is not particularly suited for tall building studies because it is based on low-rise building damage and the destructiveness of an earthquake is quite different for low-rise and high-rise buildings. However, this problem is common to all one-parameter characterizations of earthquakes, which are independent of building period. The same problem arises with instrumental one-parameter characterizations of earthquakes. Information is lost, and this is reflected in a significant scatter of results. For many buildings in the case-studies, instrumental data are lacking. In such cases the empirical correlation between EIS and MMI for different building periods and soil types reported by Scholl [1982] is useful. Converting MMI to EIS using these relationships makes the intensity measure dependent on the building period in an average sense, although the previously mentioned methodological problem, of course, remains. Scholl used both measures for an empirical study, but the theoretical advantages of EIS have not yet been confirmed.

In spite of the drawbacks of the subjective scales discussed, earthquakes are classified in the present work on the basis of MMI (Appendix A, Table A-12). The main reason for using this classification criterion is the availability of data. For future work it is envisioned to classify earthquakes also using EIS as an instrumental measure. Where instrumental data are not available, the MMI-EIS correlation of Scholl can be used.

8.3 CLASSIFICATION OF DAMAGE

Similarly as earthquake intensity, damage can be measured using subjective and "instrumental" or objective scales. These scales may be used to classify either overall damage, or damage of building systems and components.

A subjective scale for overall damage is usually given in the form of Damage States (DSs). Each DS is defined by a verbal description of the degree of structural and nonstructural damage. A set of DSs from 0 to 8 ("0"=no damage, "8"=collapse), which was first introduced by Whitman [1973], is shown in the Monograph [Council, 1978]. The same scale has also been used by other researchers with some modifications [Scholl, 1982]. This last version is shown in Table 8-1.

An objective measure of overall damage used by previous investigators is the damage ratio (DR). It is defined as the ratio of the repair cost to the replacement cost of the building. In many cases, however, this information is difficult to find from the case-studies of damaged buildings. Even when repair costs are available, the replacement cost must usually be estimated based on the original construction cost.

Most frequently, a description of structural and nonstructural damage is reported but the DR is not given. For these cases a relationship between DR and DS would be helpful. Such a relationship is reported by Whitman [1973] (Appendix, Table A-5). Scholl [1982] introduced some modifications (Table 8-1). A comparison of the two versions shows that, in general, the same damage descriptions are used by both, but Scholl generally assigns lower DRs for the same DSs. This points to the subjectivity inherent in the DS classification: the assignment of DS depends on the judgment of the researcher. Different views on the severity of damage may result in different DS assignments and, hence, DR ranges. In the present work, the DRs are used to classify the overall damage (and, when not available, the Scholl DS - DR relationship is used [Table 8-1]).

Likewise, DRs can also be used for the classification of damage in building subsystems and components. Whenever the exact DR is not available, a DS description can be assigned. A DS-DR relationship for components, similar to that for overall damage, can be developed. Such a relationship for component damage is not available.

Table 8-1: Overall Damage Classification and associated ranges of Damage Ratios [Scholl et al. 1982]

DS	Level of Damage	DR(%) CDR	DR(%) Range
0	No damage	0.0	0-0.05
1	Negligible or minor nonstructural damage --a few walls and partitions cracked, incidental mechanical and electrical damage	0.1	0.05-0.30
2	Localized nonstructural damage -- more extensive cracking (but still not widespread); possibly damage to elevators and other mechanical/ electrical components	0.5	0.30-1.25
3	Widespread nonstructural damage -- possibly a few beams and columns cracked, although not noticeable	2	1.25-3.5
4	Minor structural damage --obvious cracking or yielding in a few structural members; substantial nonstructural damage with widespread cracking	5	3.5-7.5
5	Substantial structural damage requiring repair or replacement of some structural members; associated extensive nonstructural damage	10	7.5-20
6	Major structural damage requiring repair or replacement of many structural members; associated nonstructural damage requiring repairs to major portion of interior; building vacated during repairs	30	20-65
7	Building condemned	80	65-100
8	Collapse	100	100+

(Note: DS=Damage State, DR=Damage Ratio =Ratio of repair cost to replacement cost, CDR=Central Damage Ratio)

However, DS categories such as those proposed by Beedle [1980] may be used. These categories (Appendix, Table A-6) are used in the present work to classify the damage of systems and components.

The case studies of buildings damaged in earthquakes include valuable information in the form of the opinions of the damage evaluation teams regarding the mechanisms of failure and the lessons learned from each particular case. Such information can be classified according to the technical reason for the observed damage and according to the ultimate cause of damage. The technical reason is usually reported as inadequate design, bad construction practices, or as insufficient consideration of effects of irregularities, etc. These are considered "critical" characteristics for the performance of the building. Moreover, particular structural or nonstructural elements may be identified as the origin and cause of damage. These are considered "critical" elements for the performance of the building. The technical reason is thus classified according to critical elements and critical characteristics. The ultimate cause is classified as accepted risk, error (in design, in construction, insufficient code provisions, other), or undetected. The classification scheme for damage evaluation used in the present work is shown in the Appendix (Table A-6).

8.4 DATA ORGANIZATION

The data collected in this project passed through several stages of condensation. For the 40 buildings in the Digests, the flow of data was as follows:
Case studies => Digests => Data Forms => Classification
For another 44 buildings (Appendix B, Table B-1) which are also used in the following analysis, the "Digest" step was omitted. The reasons for the "Digest" step have been given in the introduction. The Data Forms still contain most of the data available from the case studies, but in a concentrated form. In the classification step, finally, the data is reduced to a sequence of numbers.

Form 1 (Table A-7) includes general building information such as name, city, country, address, use, material, year of construction, cost, height, number of stories, plan dimensions, plan area, gross area, calculated building periods, and references for the sources of information.

In Form 2 (Table A-8), a sketch of plan and elevation of the building is given followed by a number assigned according to the proposed classification of tall building systems. Furthermore, detailed information is given on the structural, nonstructural and other building systems, as well as on the design methods and construction practices.

Form 3 (Table A-9) includes data on many earthquake characteristics in addition to MMI and EIS that can be used in the classification. Data are divided into two categories: general (earthquake specific), and local (site specific). The general characteristics include the ground motion measures at the epicenter such as M, maximum epicentral MMI, duration, maximum epicentral acceleration, etc. To avoid repetition of data for buildings subjected to the same earthquake, simply the code number for the particular earthquake is given. The local characteristics are the available ground motion measures at a given site such as MMI, EIS, duration, peak acceleration, peak velocity, peak displacement in the two horizontal and vertical directions. Information on the ground motion characteristics is supplemented with a classification of soil conditions [ATC, 1978], and the measured periods before, during, and after the earthquake. This information allows conversion of MMI to EIS using the correlation established by Scholl.

Form 4 (Table A-10) contains under the heading "Damage Description" all the necessary information for the classification of overall damage, component damage, and damage evaluation. Damage ratio and damage state are given for the overall damage, whereas for the component damage a listing of building systems and components is given to which appropriate damage states can be assigned. Under the heading "Damage Evaluation" technical reason and ultimate causes of damage are reported and classified. Information on methods of analysis and recommendations from case studies are given as well.

The information collected was intended to be used for the extensive analysis of high-rise buildings, earthquake, and damage. A data base was designed to include a large number of buildings (at least 200), however at the present stage it contains the 40 buildings found in the Digests and another 44 well documented buildings. The only criterion for the selection of these buildings was the availability and quality of information. This implies that the data base consists mainly of damaged buildings, because damage evaluation teams have usually concentrated their efforts almost solely on damaged buildings. Data on undamaged buildings subjected to the same earthquakes is extremely scarce. Other investigators [Scholl, 1982] searched for such data in alternate sources of information, like fire insurance maps. These sources do not contain enough data on the structural and other important tall building systems [Scholl, 1982]. Therefore, it is intended to start a world survey for this purpose.

A computer program was developed for the study of the available data. It can be used as the data base will increase. It can generate lists of selected systems from the data, tables with combinations of parameters, and can calculate statistics as means, medians, and standard deviations of DRs for selected parameters.

8.5 DATA ANALYSIS AND RESULTS

GENERAL

In order to make a preliminary assessment of the available data, the data is first compared to other data available from the literature. Then certain expectations generally accepted by experts in earthquake engineering are investigated. To the degree that the data will exhibit those expected trends, confidence will be gained in both data and methods.

Earthquake engineering specialists generally agree that steel structures are more forgiving to bad design and construction practices than concrete structures. During the last decade an impressive amount of research has been conducted on the earthquake resistant design of concrete structures. However, the new knowledge and understanding of concrete buildings was not yet available at the time that the majority of the buildings in this data base were designed. Therefore it can be expected that steel buildings experience less damage than concrete buildings.

In general, high-rise buildings are better engineered structures than low-rise buildings, and more care and sophistication is put into their design. Also codes usually are more conservative and have more stringent requirements for the design of tall buildings. Thus it is anticipated that high-rise structures experience less damage than low-rise structures.

Another expectation relates to the behavior of buildings with irregular configuration. Usually irregular structures are more difficult to analyze and design than regular structures, and thus are more susceptible to problems. Modern seismic codes [ATC, 1978] therefore distinguish between the two types of structures and call for special attention to the design of irregular structures. In many cases, especially with buildings of older construction, such a consideration to irregularity is lacking. Therefore, it can be anticipated that irregular buildings in this data base experience more damage than regular buildings.

Safety against collapse has been the major preoccupation of earthquake engineering. However, in addition to safety, damage control is very important for the successful performance of a high-rise system. In a tall building a large proportion of the repair costs can result from damage to nonstructural elements, which may constitute up to 80% of the construction cost. Flexible structures tend to exhibit more nonstructural damage than stiff structures. Buildings with a pure moment-resisting frame as a structural system are more flexible than buildings employing other structural systems with additional stiffening elements (such as shear walls). Therefore, pure moment-resisting frame structures may exhibit more damage than other structural systems.

Although many more characteristics are included in the data base, the present preliminary study will thus investigate the effect and importance of the following:

- Material of the high-rise system (steel versus concrete)
- Tallness of the high-rise system (high-rise versus medium-rise)
- Configuration of tall buildings (regular versus irregular)
- Structural system of tall buildings (pure moment-resisting frames versus other)
- MMI for the earthquake

Results will be presented in the form of damage probability matrices, mean damage ratios, and median damage ratios. The damage probability matrix, introduced by Whitman [1973], gives the probability distribution of damage states for a particular class of structures at various earthquake intensities. An example is shown in Table A-11, in Appendix A. The mean damage ratio is defined as [Whitman, 1973]:

$$MDR_I = (1/n_I)\Sigma DR_{iI} \tag{3}$$

where, n_I= total number of buildings subjected to earthquake intensity I.
DR_{iI}=Damage Ratio for i-th building subjected to intensity I.

If the damage ratio is not known but the damage state can be assigned, a mean damage ratio can be calculated using the central damage ratios given in Table 8-1 for each damage state:

$$MDR_I = (1/n_I)\Sigma n_{DS\,I}CDR_{DS} =$$

$$= \Sigma p_{DS\,I}CDR_{DS} \qquad (4)$$

where $n_{DS\,I}$=number of buildings experiencing damage state DS when subjected to intensity I.
$p_{DS\,I}$=probability that a building experiences damage state DS when subjected to intensity I

If the central damage ratios were the true mean damage ratios of all buildings in a damage state, Eq.(4) should converge to Eq.(3) as n_I increases. In this study mean damage ratios were calculated using Eq.(3) when damage ratios were available. When damage ratios were not available for some buildings, Eq.(4) was used and combined with Eq.(3) to give the mean damage ratio of all buildings in a certain category.

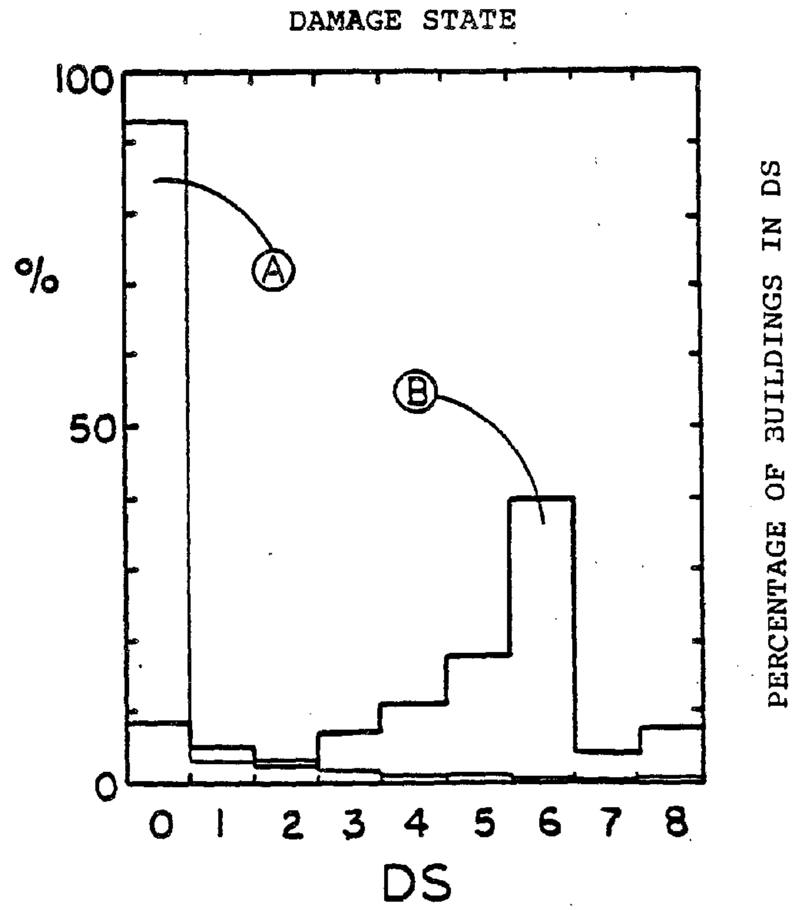


Figure 8-1: Comparison of data bases: (A) Scholl[1982], (B) this study.

COMPARISON OF DATA

The first and most important question is whether the data base constitutes a representative sample of the total population of buildings subjected to earthquakes. In Figures 8-1 and 8-2 the present data base is therefore first compared to the data bases used in two other studies [Whitman, 1973], [Scholl, 1982]. Figure 8-1 shows a comparison between the damage state distribution of the present sample and the much more extensive sample used in the study by Scholl et al. Clearly, the present data base consists mainly of damaged buildings, while in the more representative sample of the Scholl study the majority of the buildings is undamaged. Thus, any conclusions from the present sample relate to damaged buildings rather than to all buildings subjected to an earthquake. It must be realized though, that the majority of information about undamaged buildings in the Scholl study stems from fire insurance maps. Usually little more than the material is known about the structural system of these undamaged buildings [Scholl, 1982], which precludes their use for the questions addressed in this report without further data acquisition. For this reason, the authors of the Scholl study did not make any comparison of different structural systems other than steel versus concrete.

Figure 8-2 compares plots of mean damage ratios (MDRs) for steel and concrete buildings from 3 different sources: the present study, the study by Whitman et al., and the study by Scholl et al. The Scholl curves lie below the Whitman curves, which, in turn, mostly lie below the curves of this study. This again demonstrates that the data base of Scholl et al. includes more undamaged buildings than the data bases of Whitman and of this study. It also shows what effect the ignoring of undamaged buildings can have. A notable characteristic of the curves from the Whitman study and from this study is that they do not monotonically increase. This unrealistic result can be explained by the bias towards damaged buildings of these two data bases. Because there are more undamaged buildings at lower earthquake intensities, neglecting undamaged buildings increases the mean damage ratios at lower intensities relative to those at higher intensities [Scholl, 1982]. As evidenced in Figure 8-2 and all following figures, this distortion can be so significant that the mean damage ratios at lower MMI levels become larger than those at higher MMI levels. It is interesting to note, however, that all 3 groups of curves agree in that steel buildings experience less damage than concrete buildings. This would indicate that differences in mean damage ratios of various systems may be less sensitive to the bias of the data base than the absolute values of mean damage ratio. Although this observation may encourage further study of building systems based on the present data, it may not be true for parameters other than material.

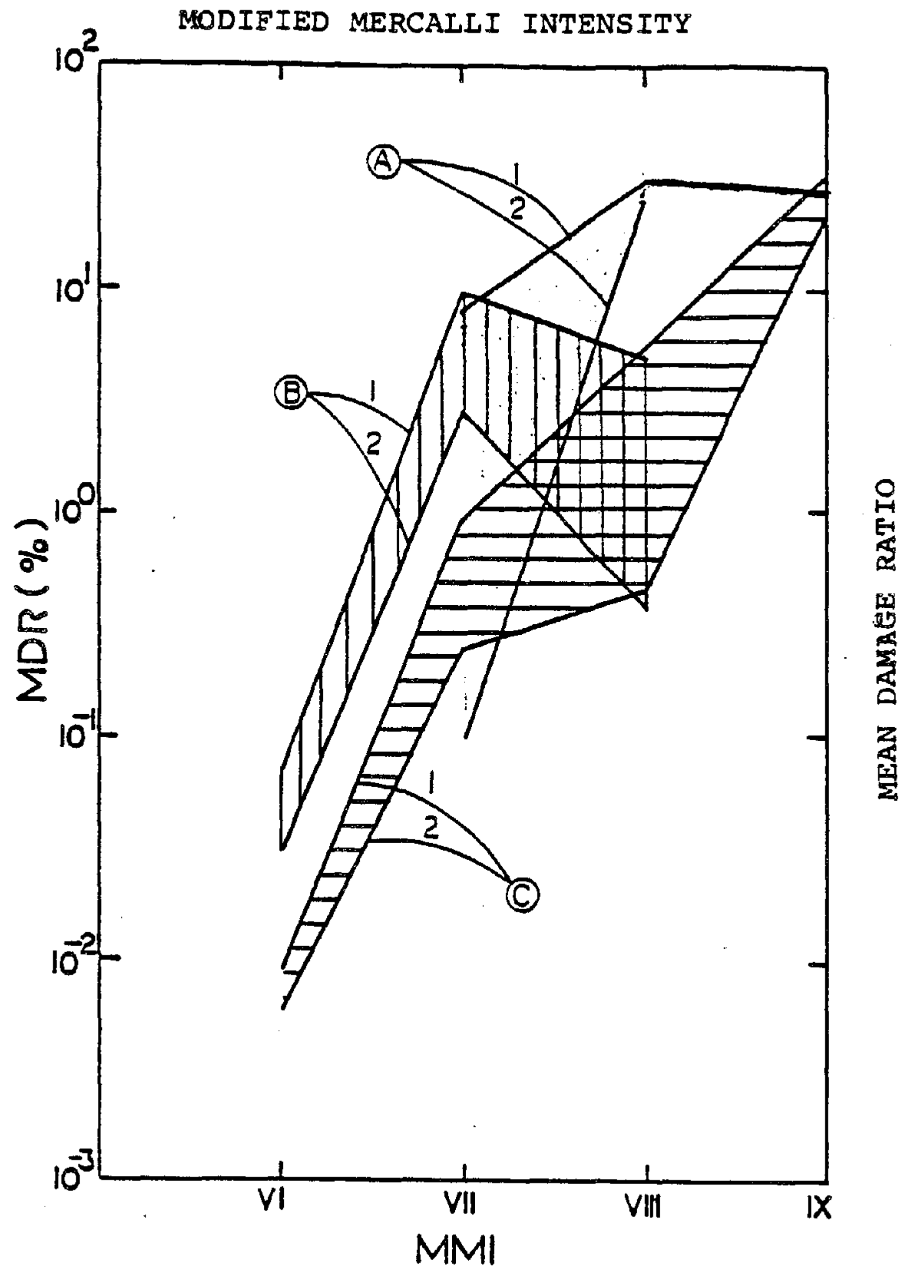


Figure 8-2: Comparison of (1) concrete, (2) steel buildings from 3 different sources:
(A) this study, (B) Whitman[1973], (C) Scholl[1982].

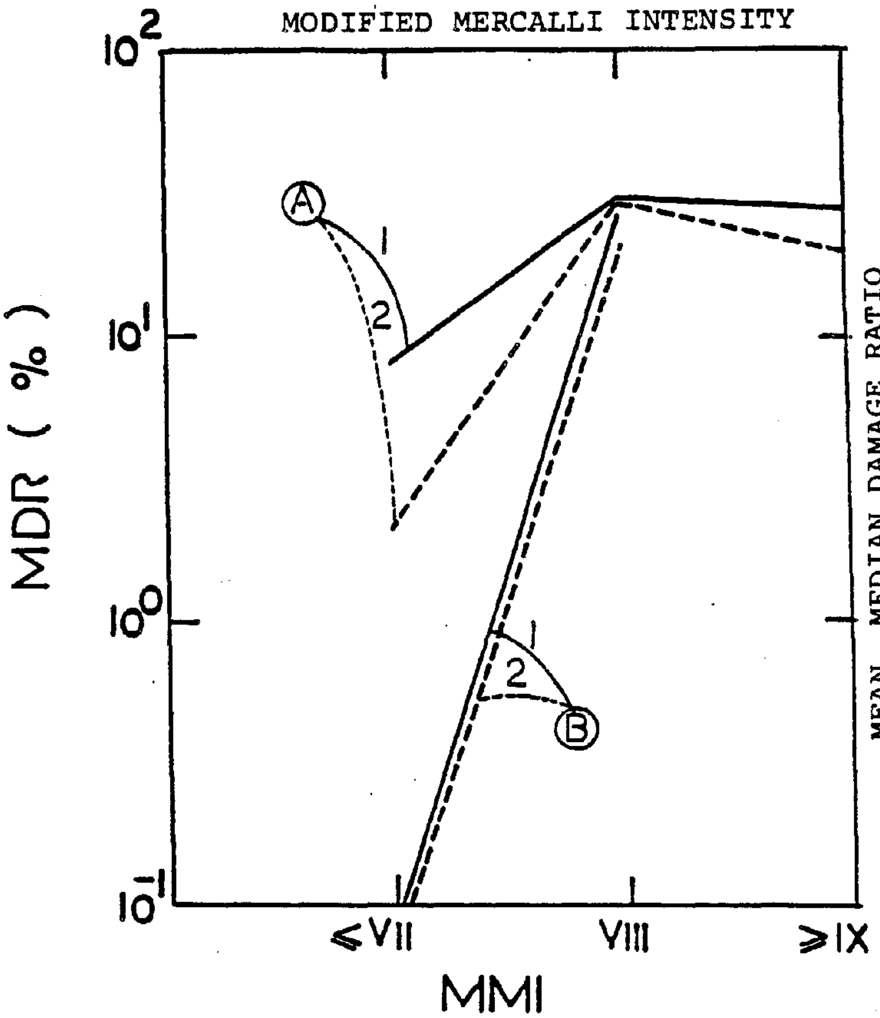


Figure 8-3: Comparison of (1) mean and (2) median damage ratios between (A) concrete and (B) steel buildings.

COMPARISON OF STEEL AND CONCRETE BUILDINGS

In the following is investigated whether the present data base confirms the expected trends discussed at the beginning of this section. Although Figures 8-2 and 8-3 show that the sample means and medians of the damage ratio are always lower for steel than for concrete, this is not sufficient to conclude that the same is true for the "true" means and medians, which are not known. The question is whether the observed differences in mean and median damage ratios are significant in view of the scatter. Figure 8-4 compares damage probability matrices for steel and concrete buildings. In addition to the probabilities, the histograms of the distribution are also shown to ease comparison. It should be noted that the lowest earthquake intensity class contains mainly buildings subjected to MMI=VII. However, a few buildings subjected to MMI=VI, whose number is insufficient to be treated separately, is also included in this class, which is therefore referenced as MMI<VII in the text. Clearly there is significant dispersion in damage state and damage ratio. This is also evident from the fact that the standard deviations and means are of the same order of magnitude.

In order to determine whether the observed deviation between the two samples is statistically significant, a statistical test is applied. Because the distribution of the original population is unknown and the samples often are small, a nonparametric test is used. Nonparametric tests do not test for the difference of means but for the difference of other types of "averages". The Mann-Whitney (M-W) test [Noether, 1976, Book, 1977] used in the following tests for the difference of medians. The null hypothesis

H_0: "The median damage ratio of steel buildings is equal to the median damage ratio of concrete buildings"

is tested against the alternative

H_1: "The median damage ratio of steel buildings is less than the median damage ratio of concrete buildings"

by ranking the combined two samples according to damage ratios. Then, assuming that the unknown "true" medians are equal (hypothesis), the probability is calculated that the rank sum of a sample is equal to or larger (or smaller depending on the case) than the actually observed value. These probabilities are shown in Figure 8-4 and in similar following figures. If this probability is high, the difference observed in sample medians is likely to be a random deviation and the hypothesis of equal medians cannot be rejected. If this probability is low, on the other hand, the observed difference is unlikely to be a random deviation. Rather a significant deviation is indicated. The hypothesis of equal medians is unlikely and can be rejected. The probability dividing acceptance and rejection region for the hypothesis, the significance level of the test, is customarily chosen at 5% or 1%.

The probabilities derived from the M-W test are 0.1% for MMI levels $\leq$VII and VIII combined, 4.0% for MMI<VII, and

DAMAGE PROBABILITY MATRIX / OTHER STATISTICS

MMI		(VI)&VII		VIII		≥IX		ALL	
DS	DR(%)	S	C	S	C	S	C	S	C
		*	*	*	*	*	*	*	*
8	100+			14	9		6	7	7
7	65-100				4		6		4
6	20-65			43	50		38	22	41
5	7.5-20		22		22		25		23
4	3.5-7.5			14	11		19	7	11
3	1.25-3.5		11	29	2		6	13	4
2	0.30-1.25	14	22					7	3
1	0.05-0.30	43	11		2			22	3
0	0-0.05	43	33					22	4
No. of Buildings		7	9	7	45		16	14	70
Mean DR(%)		0.091	8.052	26.929	30.913		28.375	13.510	27.394
St. Dev.(%)		0.114	12.644	33.819	29.555		30.964	26.866	29.001
Median DR(%)		0.000	2.000	20.000	30.000		20.000	1.165	30.000
M-W Prob.(%)		4.01		20.3				0.1	

(Note:*numbers are probabilities in percent, while the histograms show number of buildings and not probabilities. MMI=Modified Mercalli Intensity, DS=Damage State, DR=Damage Ratio, M-W=Mann-Whitney)

Figure 8-4: Damage Probability Matrices, histograms, and other statistics for Steel (S) and Concrete (C) buildings.

20.3% for MMI=VIII. Thus, based on a significance level of 5%, the evidence from MMI levels $\leq$VII and VIII combined indicates a significant difference in performance between steel and concrete buildings. For this case steel buildings experience a 51% smaller mean damage ratio and a 96% smaller median damage ratio. However the evidence from individual MMI levels is inconclusive. For MMI=VIII with the largest samples the difference is not significant.

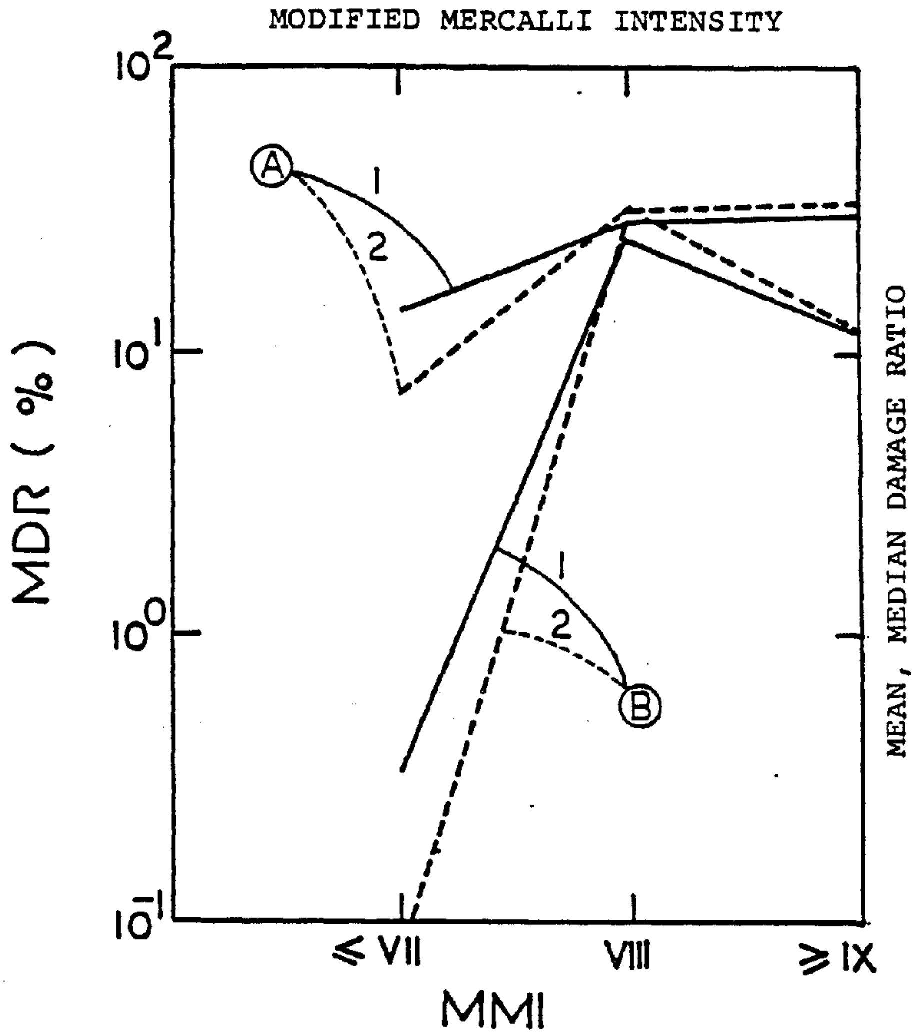


Figure 8-5: Comparison of (1) mean, (2) median damage ratios between (A) medium- and (B) high- rise buildings.

COMPARISON OF HIGH-RISE AND MEDIUM-RISE BUILDINGS

The parameter "tallness" is investigated next. Tall buildings in the data base are divided into two categories using the fundamental period as a criterion. Buildings with a fundamental period less than 1 second are classified as "medium-rise", those with a fundamental period greater than 1 second as "high-rise". When the periods were not available from measurements or dynamic analyses, they were calculated using the formulas [ATC, 1978]:

For moment-resisting frames: $T=CH^{3/4}$

where, $C=0.035$ for steel
$C=0.025$ for concrete

For all other buildings: $T=0.05\ H/(L^{1/2})$

where, T=fundamental period (sec)
H=height (ft)
L=plan dimension (ft)

In the "medium-rise" category concrete buildings have approximately 6 to 14 stories, steel buildings 6 to 9 stories. This difference in maximum height reflects the higher stiffness of concrete buildings.

The results regarding tallness are presented in Figures 8-5 and 8-6. Figure 8-5 shows that the mean and median damage ratios of high-rise buildings are, as expected, lower than those of medium-rise buildings. For all MMI levels combined, high-rises experience a 63% smaller mean damage ratio and a 80% smaller median damage ratio. However, for individual MMI levels those percentages may be as low as 26% and 0% (MMI=VIII). As shown by the histograms in Figure 8-6, there is a considerable dispersion. The probabilities derived from the M-W test are 0.2% for all MMI levels combined, 2.4% for MMI$\leqslant$VII, 61% for MMI=VIII, and 31.6% for MMI$\geqslant$IX. Thus, based on a significance level of 5%, the combined evidence from all MMI levels indicates a significant difference in performance between high-rise and medium-rise buildings. However, for individual MMI levels this is only true for MMI$\leqslant$VII. For both MMI=VIII and MMI$\geqslant$IX the difference cannot be confirmed as significant.

DAMAGE PROBABILITY MATRIX | OTHER STATISTICS

MMI		(VI)&VII		VIII		≥IX		ALL	
DS	DR(%)	M	H	M	H	M	H	M	H
		*	*	*	*	*	*	*	*
8	100+			12		7		10	
7	65-100			5		7		5	
6	20-65	40		42	73	44	50	42	37
5	7.5-20			20	27	21		18	13
4	3.5-7.5	20		12		14	50	13	4
3	1.25-3.5	20	9	7		7		8	4
2	0.30-1.25		18						8
1	0.05-0.30		27	2				2	3
0	0-0.05	20	46					2	21
No. of Buildings		5	11	41	11	14	2	60	24
Mean DR(%)		13.860	0.346	32.210	23.818	31.357	17.500	30.482	11.284
St. Dev.(%)		14.973	0.635	33.082	9.185	32.073	17.678	31.773	12.785
Median DR(%)		7.300	0.080	30.000	30.000	30.000	17.500	30.000	6.150
M-W Prob.(%)		2.4		61.0		31.6		0.2	

(Note: *numbers are probabilities in percent, while the histograms show number of buildings and not percentages. MMI=Modified Mercalli Intensity, DS=Damage State, DR=Damage Ratio, M-W=Mann-Whitney.)

Figure 8-6: Damage Probability Matrices, histograms, and other statistics for Medium- (M) and High- (H) Rise buildings.

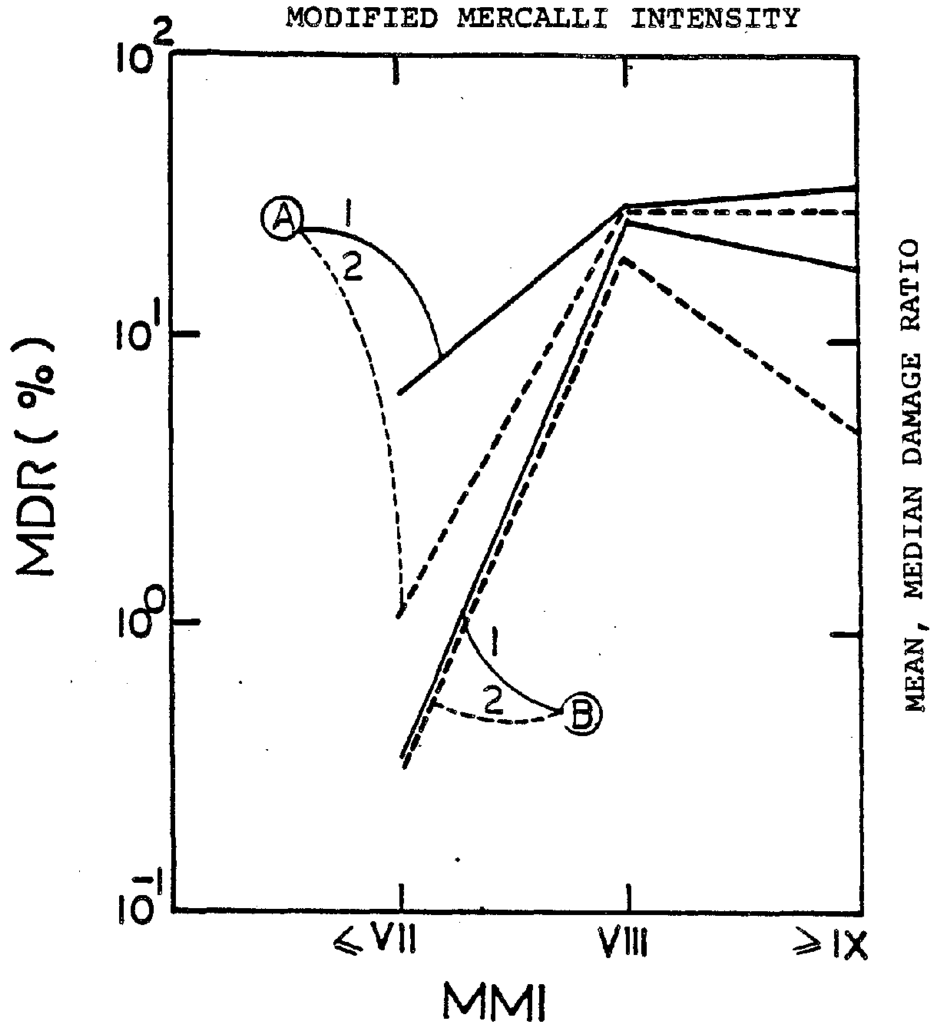


Figure 8-7: Comparison of (1) mean, (2) median damage ratios between (A) irregular and (B) regular buildings.

COMPARISON OF REGULAR AND IRREGULAR BUILDINGS

To investigate the parameter "configuration", buildings are classified as regular or irregular according to the ATC guidelines [ATC, 1978]. Figure 8-7 presents mean and median damage ratios and Figure 8-8 damage probability matrices, histograms, and other statistical information.

As expected, the mean and median damage ratios for regular buildings are lower than those of irregular buildings. For all MMI levels combined, regular buildings

MMI		(VI)&VII		VIII		≥IX		ALL	
DS	DR(%)	I	R	I	R	I	R	I	R
		*	*	*	*	*	*	*	*
8	100+			8	17		11	5	12
7	65-100			5		13		5	
6	20-65	20		61	24	62	11	53	14
5	7.5-20			13	32	25	22	12	23
4	3.5-7.5	10		8	17		45	7	23
3	1.25-3.5	20		5			11	9	4
2	0.30-1.25		60						12
1	0.05-0.30	30						5	
0	0-0.05	20	40		8			4	12
No. of Buildings		10	5	39	12	8	9	57	26
Mean DR(%)		6.635	1.753	32.192	28.508	36.000	19.000	28.243	20.072
St. Dev.(%)		12.346	3.133	28.247	36.267	28.626	31.496	27.770	31.112
Median DR(%)		1.050	0.330	30.000	10.000	30.000	5.000	30.000	9.000
M-W Prob.(%)		25.1		11.9		1.0		1.2	

(Note: *numbers are probabilities in percent, while the histograms show number of buildings and not probabilities. MMI=Modified Mercalli Intensity, DS=Damage State, DR=Damage Ratio, M-W=Mann-Whitney)

Figure 8-8: Damage Probability Matrices, histograms, and other statistics for Regular (R) and Irregular (I) buildings.

experience a 29% smaller mean damage ratio and a 70% smaller median damage ratio. However for MMI=VIII these percentages are only 11% and 67%. The probabilities derived from the M-W test are 1.2% for all MMI levels combined, 25.1% for MMI$\leq$VII, 11.9% for MMI=VIII, and 1.0% for MMI$\geq$IX. Therefore, using a significance level of 5%, the evidence from all MMI levels combined indicates a significant difference in performance between regular and irregular structures. The same is true at MMI$\geq$IX, but for MMI levels VIII and $\leq$VII the difference cannot be confirmed as significant.

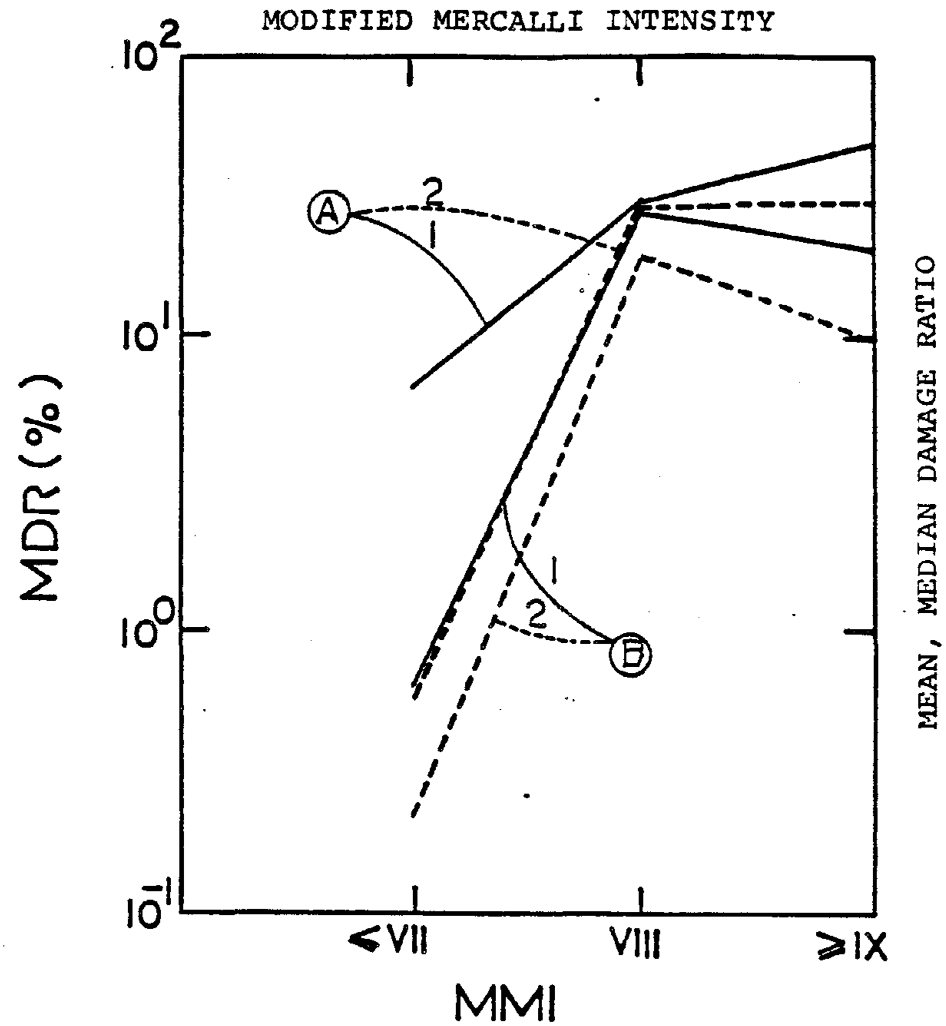


Figure 8-9: Comparison of (1) mean, (2) median damage ratios between (A) pure moment-resisting frames and (B) other structures.

COMPARISON OF PURE MOMENT-RESISTING FRAMES AND OTHER STRUCTURAL SYSTEMS

The investigation of "structural system" as a parameter influencing damage is confined to a comparison between pure moment-resisting frames and all other systems.

Figures 8-9 and 8-10 show that "other" structural systems experience 14% lower mean damage ratio and 67% lower median damage ratio than pure moment-resisting frames for all MMI levels combined. However for MMI=VIII, these percentages are only 7% and 27%. The probabilities derived from the M-W test are 9.7% for all MMI levels combined, 30.9% for MMI$\leq$VII, 16.1% for MMI=VIII, and 2.4% for MMI$\geq$IX. Thus, using a significance level of 5%, the observed difference cannot be confirmed as significant neither for all MMI levels combined nor for MMI=VIII and MMI$\leq$VII. Only for MMI$\geq$IX the test indicates a significant difference in performance between pure moment-resisting frames and other structural systems.

DAMAGE PROBABILITY MATRIX — OTHER STATISTICS

MMI		(VI)&VII		VIII		≥IX		ALL	
DS	DR(%)	F	O	F	O	F	O	F	O
		*	*	*	*	*	*	*	*
8	100+			13	5	20		11	3
7	65-100				10		9		8
6	20-65			53	42	80	18	45	30
5	7.5-20	30		25	14		37	23	19
4	3.5-7.5			6	14		27	4	17
3	1.25-3.5	10	25	3	10		9	4	11
2	0.30-1.25	10	25					2	3
1	0.05-0.30	20	25		5			4	6
0	0-0.05	30	25					6	3
No. of Buildings		10	4	32	21	5	11	47	36
Mean DR(%)		7.070	0.608	31.310	29.080	44.000	21.270	27.503	23.530
St. Dev.(%)		12.290	0.939	28.270	31.930	31.300	29.470	27.998	30.181
Median DR(%)		0.600	0.215	30.000	22.000	30.000	10.000	30.000	10.000
M-W Prob.(%)		30.9		16.1		2.4		9.7	

(Note: *numbers are probabilities in percent, while the histograms show number of buildings and not probabilities. MMI=Mofified Mercalli Intensity, DS=Damage State, DR=Damage Ratio, M-W=Mann-Whitney.)

Figure 8-10: Damage Probability Matrices, histograms, and other statistics for Pure Moment-Resisting Frames (F) and Other (O) buildings.

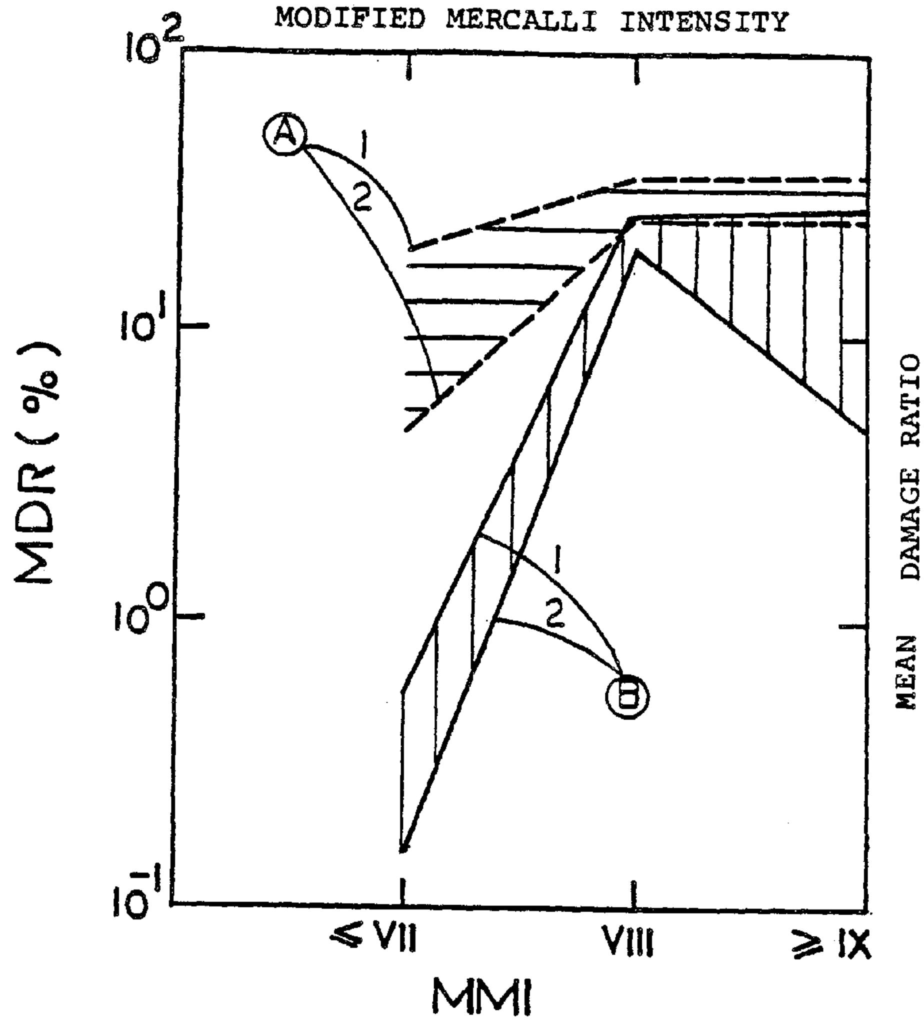


Figure 8-11: Comparison of (1) irregular, and (2) regular structures within (A) medium- and (B) high- rise buildings.

COMPARISONS INVOLVING TWO PARAMETERS

In the following two examples for investigating combinations of parameters are given, namely configuration-tallness and configuration-structural system. Due to the constraints imposed on this preliminary study, only mean damage ratios are calculated; the significance of the observed difference is not asserted.

Figure 8-11 shows that the mean damage ratios of regular structures are lower than those of irregular structures for both high- and medium-rise buildings. For all MMI levels combined, regular medium-rises experience a 36% lower mean damage ratio than irregular medium-rises, whereas regular high-rises experience a 67% lower mean damage ratio than irregular high-rises. This would indicate that irregularity affects taller buildings more than lower buildings. This result could be explained by the observation that taller buildings usually have similar layouts over many stories and are using cleaner and more regular structural systems than lower buildings. Thus, the difference between a regular and irregular structural system might be larger in high-rises than in medium- and low-rises.

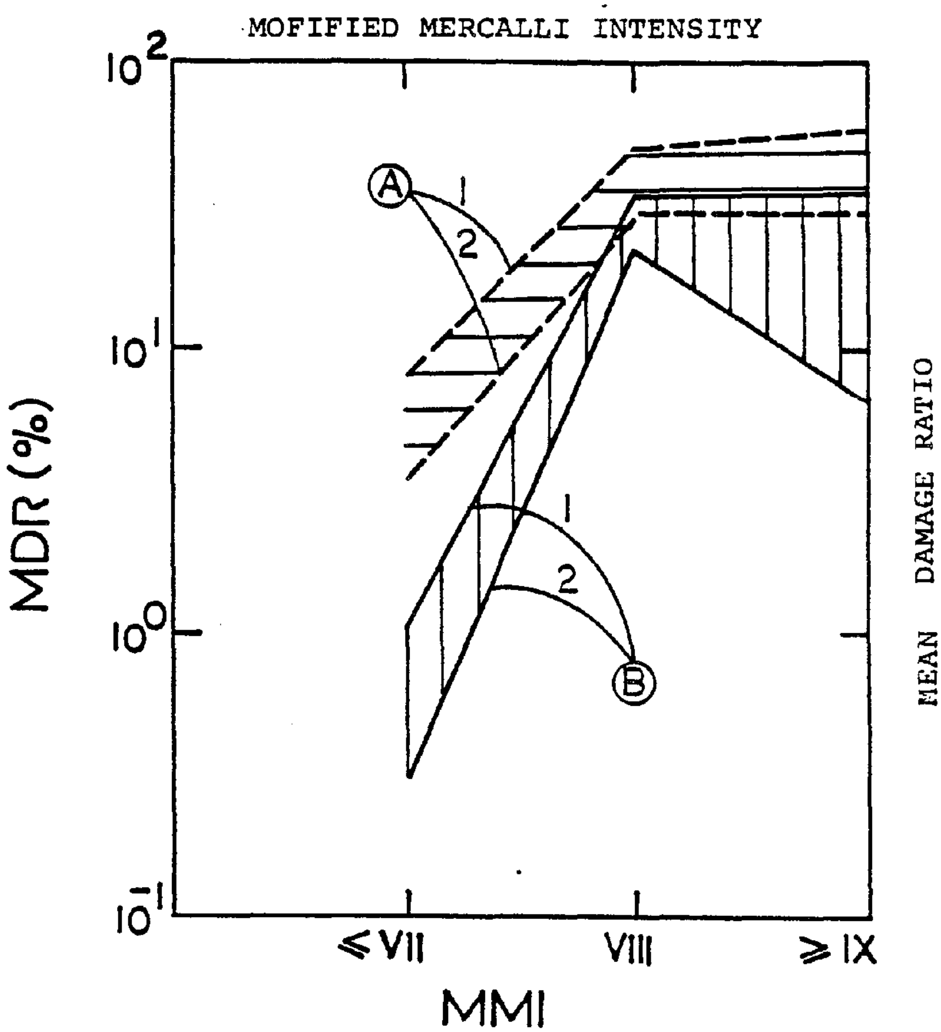


Figure 8-12: Comparison of (1) irregular, and (2) regular structures within (A) pure moment-resisting frames and (B) other structures.

Figure 8-12 shows that the mean damage ratios of regular buildings are lower than those of irregular buildings for both pure moment-resisting frames and "other" structural systems. For all MMI levels combined, regular pure moment-resisting frames experience a 33% lower mean damage ratio than irregular pure moment-resisting frames, whereas regular "other" structures experience a 50% lower mean damage ratio than irregular "other" structures. While this may indicate that irregularity affects pure moment-resisting frames less than other structures, this result may as well simply reflect the classification scheme. A building with a moment-resisting frame was classified as "other" structure, whenever shear walls or other bracing systems were present. Therefore, practically all severe cases of irregularity, such as soft stories and discontinued shear walls (e.g. Olive View Hospital, Imperial County Building) or eccentric walls (e.g. Banco Central de Nicaragua) are excluded from the class of pure moment-resisting frames. This points to the extreme care with which these results should be interpreted and qualified.

DISCUSSION OF COMMON TRENDS

Some general observations regarding the results presented so far are appropriate at this point. The plots of mean and median damage ratios (Figures 8-3, 8-5, 8-7, 8-9, 8-11, 8-12) show remarkably similar patterns. They increase with intensity from MMI⩽VII to MMI=VIII, but are independent of intensity or even decrease with intensity from MMI=VIII to MMI⩾IX. The reason for this general trend, as explained earlier, is the lack of data on undamaged buildings. All curves also show smaller differences at MMI=VIII than at MMI⩽VII and MMI⩾IX. Finally, also the probabilities obtained from the M-W test, which are summarized in Table 8-2, show similar trends for all parameters. At a significance level of 5%, the observed differences are statistically significant, in general, for all MMI levels combined, but not for individual MMI levels.

The M-W probabilities depend on sample size, magnitude of difference in medians, and scatter. Table 8-2 compares these probabilities, the sample size, and the differences (%) in medians observed. For all MMI levels combined the sample size is the largest (83 & 84) and the differences in medians are relatively large (67% to 96%). Thus the M-W probabilities are, in general, smaller than the probabilities calculated at individual MMI levels, where either the sample size or the difference in medians is smaller. The effect of sample size can be observed by comparing the first column for MMI⩽VII with the last for all MMI levels. The difference in medians are very similar for the two columns, and the dispersion in damage ratios for MMI⩽VII is certainly not larger than for all MMI levels combined. The significantly larger M-W probabilities for MMI⩽VII must therefore be attributed to the small sample size. The large M-W probabilities for MMI=VIII, on the other hand, where the sample is quite large, are due to relatively small differences in medians relative to the dispersion.

It is somewhat surprising that the differences in performance are smallest and least significant for MMI=VIII, which contains the largest sample. It may be that the earthquake intensity measure, MMI does not sufficiently differentiate between different ground motions. Similarly it is somewhat unexpected that combining the data for all earthquake intensities results in the lowest M-W probabilities. For one might expect that the increase in dispersion due to disregarding an important parameter (earthquake intensity) would offset the effect of increased sample size. The results appear to indicate, therefore, that the classification of earthquake intensity lacks precision. Again this might be attributed to the drawbacks of the MMI scale, but the fact that the damage-intensity relationship of this study is distorted due to the lack of data on undamaged buildings, probably also plays a role.

Table 8-2: Comparison of parameters affecting damage

PARAMETERS	MMI levels ≤VII	VIII	≥IX	ALL
	M-W Probabilities (%)			
Steel/Concrete	4.0	20.3	-	0.1
High-/Medium-rise	2.4	61.0	31.6	0.2
Regular/Irregular	25.1	11.9	1.0	1.2
Other/Moment-Resisting Frames	30.9	16.1	2.4	9.7
	Combined Sample Size			
Steel/Concrete	16	52	16	84
High-/Medium-rise	16	52	16	84
Regular/Irregular	15	51	17	83
Other/Moment-Resisting Frames	14	53	16	83
	Median differences (%) *			
Steel/Concrete	96	33	-	96
High-/Medium-Rise	99	0	42	80
Regular/Irregular	69	67	83	70
Other/Moment-Resisting Frames	64	27	67	67

(NOTE: * Percentage by which median Damage Ratio of first class is smaller than that of second class.)

COMPARISON OF PARAMETERS AFFECTING DAMAGE

Finally it is investigated which of the studied parameters, material, tallness, configuration, and structural system are the most important regarding seismic vulnerability. This question is addressed using only the combined evidence from all MMI levels, which is summarized in the last column of Table 8-2. If a significance level of 1% is chosen, only material and tallness are significant parameters affecting damage (whereas for configuration and structural system the hypothesis of no difference in medians cannot be rejected). Using a significance level of 5% configuration may also be considered a significant parameter. Structural system becomes significant only at a significance level of 10%. It must be remembered, though, that none of these parameters could be confirmed as significant for the majority of individual MMI levels at any of the three significance levels.

The order of importance of the studied parameters is thus material (steel/concrete), tallness (high-/medium-rise), configuration (regular/irregular), structural system (pure moment-resisting frames/others). The importance of material and configuration mainly reflects insufficient knowledge. The rapid progress in the last decade regarding detailing of reinforced concrete for ductility and analysis of complex irregular structures will likely decrease the importance of material and configuration, at least for new buildings. Reinforced concrete buildings designed on the basis of the latest research results are excellent earthquake resistant structures. Assisted by powerful computer methods, a competent designer can also make work an irregular building. Similarly, the last rank of structural system might be rationalized arguing that the ingenuity and competence of the designer together with detailing and construction quality are probably more important than the theoretical advantages of one structural system over another. A competently designed "bad" system may perform as well as a poorly designed "good" system. However, such a conclusion is not warranted before other trends regarding structural system are investigated. Structural systems using central cores only or flat-plate construction have often experienced problems. Nevertheless, the order of importance found appears to justify the emphasis that the new ATC model code [ATC, 1978] places on good detailing, particularly for reinforced concrete structures, and on more sophisticated analysis techniques for irregular structures.

EVALUATION OF RESULTS

Concluding this chapter, an assessment of the results is given. Have the present data confirmed the expected trends? Has confidence been gained in the data base and methods applied?

The results are somewhat inconclusive. For all four parameters investigated, the situation is the same. All mean and median damage ratios, whether calculated for all or for individual MMI levels, exhibit differences in the direction of the expected trends. Both means and medians show qualitatively the same behavior. If the data for all MMI levels is combined, the differences in performance are statistically significant for all parameters with the exception of structural system (pure moment-resisting frame versus "others").

Thus, for all MMI levels together the expected trends are confirmed for three out of the four trends investigated. However, based on the evidence of the data for the individual MMI levels, the differences in performance cannot be confirmed as statistically significant in the majority of the cases.

These somewhat unexpected and inconclusive results require further investigation and may have several reasons. It may be that the sample sizes are insufficient. It may also be that the drawbacks of the earthquake intensity measure used, MMI, show up. This could be investigated using EIS as a measure. Such recommendations together with the results are summarized in the next chapter.

9. SUMMARY AND CONCLUSIONS

This report presents, in Chapter 2 through 7, digests of case-studies of tall buildings damaged in earthquakes, and, in Chapter 8, a preliminary analysis of the data.

Reviewing the digests, it is evident without rigorous analysis that:

- Buildings with regular configuration, clear structural system satisfying code requirements, and nonstructural elements that are isolated from the structural system, generally perform well, if attention has been paid to good detailing and rigorous quality control procedures have been followed during construction.

- Buildings with irregular configuration and complex structural system, buildings with nonstructural elements that are not isolated from seismic movement of the structural system, buildings that have been designed with little attention to good detailing practice, and buildings that have been constructed with no or poor quality control procedures, often experience problems.

It is striking that a relatively small number of problems occur again and again. The recommendations and lessons learned that are listed at the end of each digest, attest this trend in a qualitative though not rigorous quantitative sense. They are summarized in Section 9.1.

In the preliminary study presented in Chapter 8, the data on the 40 buildings contained in the digests and on 44 additional buildings are quantitatively analyzed. The parameters that are investigated are material, tallness, configuration, and structural system. Although this data-base does not contain a sufficient number of undamaged buildings in comparison to a previous study, both studies reveal similar trends regarding the material parameter. This agreement encouraged the investigation of additional parameters. A synopsis of the results and conclusions from this preliminary analysis is presented in Sections 9.2 to 9.4.

9.1 DIGESTS OF CASE-STUDIES

While final conclusions regarding the relative importance of the factors affecting the performance of tall buildings in earthquakes require a comprehensive quantitative analysis, the digests and, in particular, the recommendations and lessons learned from each building already provide valuable advice to designers. The recommendations and lessons most frequently mentioned in the case-studies are therefore summarized below.

NONSTRUCTURAL ELEMENTS

- Improve the art of earthquake design of nonstructural elements. Expansion joints, flushings, cladding, partitions, and stairwells should be designed for seismic movements based on realistic estimates of inter-story drifts. Equipment and building contents should be secured against earthquake motion.

- Nonstructural elements may change the anticipated performance of the designed structure. The effect of rigid and heavy nonstructural walls should be considered in the design of flexible structures such as moment-resisting frames.

- Sufficient separation between structures is needed in order to avoid pounding of buildings.

STRUCTURAL ELEMENTS REQUIRING SPECIAL ATTENTION

- Yielding should be initialized in and confined to girders rather than columns.

- Special consideration should be given to the design of coupling beams between shear walls.

- Improved confinement and reinforcement details are necessary in order to achieve adequate ductility of members.

- Improve the design and inspection procedures for horizontal construction joints of concrete walls.

- Give special consideration to the design of corner columns and to the effects of 2-way frame action.

- Horizontal diaphragms must be adequately reinforced to transfer all lateral loads to shear walls.

CONFIGURATION

- Buildings with regular and simple structural systems have more chances to survive a severe earthquake. Setbacks, discontinued shear walls, eccentricities between the center of mass and the center of rigidity of the lateral force resisting system as well as other irregularities require much more sophisticated analysis than what is covered by simple code procedures.

- A thorough study of the "flexible first-story" concept of design should be made before it is attempted in a major earthquake resistant structure.

CODE REQUIREMENTS

- Improve inspection procedures.

- Seismic forces can occur that are greatly in excess of those anticipated by codes; increase minimum code requirements if the equivalent static force method is retained.

ANALYSIS TECHNIQUES

- Use more sophisticated analysis as part of the design procedure for irregular buildings.

- Further investigation of vertical acceleration effects is needed.

- Account for higher mode effects in medium-rise buildings.

9.2 CLASSIFICATION SCHEMES AND METHODS USED

It is difficult to develop a classification scheme for structural systems that is logical in structure and simple and yet covers the myriads of variations and combinations that are possible in and between each "prime" structural system. This is particularly true if buildings have to be classified from all over the world from regions with quite different construction practices. Usually, not well engineered, lower buildings were more difficult to classify than taller buildings with clean and clear structural systems. In not well engineered or older buildings it is often not clear how the designer intended to resist lateral loads. In such cases the distinction between gravity and lateral load resisting systems is difficult. On the other hand, in very well engineered buildings (like tubes), gravity and lateral load resisting system are often deliberately combined, and the distinction between those systems is artificial. Thus for studies like this, classification on the basis of "general appearance" (synthetical classification) appears to be more suited. Classification on the basis of the distinction of gravity and lateral load resisting system (analytical classification), on the other hand, is suited for design, because it forces designers to have a clear concept of how particular loads are resisted, even though both functions may be combined into one physical system.

In damage classification, problems arise when information is lacking on damage ratios (DRs). Subjectivity when assigning damage states (DSs) is inevitable. Based on the experience gained with damage classification, it is believed that the modifications introduced by Scholl in the DS-DR relationship of Whitman are appropriate. Classification on the basis of DS description on the one hand, and DR on the other, appeared to agree better with the modifications. It was even more difficult to assign damage states to subsystems and components. Such detail was feasible only in a few very well documented buildings.

The classification of earthquakes is subjected to the shortcomings of having to use a subjective scale such as Modified Mercalli Intensity (MMI) due to unavailability of better data. The fact that in the majority of cases the observed differences could not be confirmed as significant at individual MMI levels, may reflect these shortcomings. Using Engineering Intensity Scale (EIS) may result in improvement of the classification.

Regarding the methods used in the analysis, one should mention the limitations of the empirical approach. The quality of the results depends on the quality of the sample used. The sample must be representative of the buildings subjected to earthquakes, but this is difficult to achieve due to the lack of information mainly on undamaged buildings. In addition, the subjectivity of MMI and DS, as already discussed, must be kept in mind when interpreting results. Finally, the sample size may become a factor. Certain details needed for an in-depth analysis of building systems may not be available for a large number of buildings. Then, analysis using small samples, possibly, may not confirm any significant differences. Therefore samples of sufficient size, representative of buildings subjected to earthquakes are needed. Some recommendations towards this objective are:

- Damage evaluation teams should also report on undamaged buildings.

- Standardized data collection forms (similar to those used in the present study or in the Scholl study), would facilitate reporting by damage evaluation teams on undamaged buildings and buildings for which detailed case studies are not warranted.

- A generally accepted terminology and a more systematic and consistent method of reporting by damage evaluation teams would facilitate studies such as this.

Despite the limitations discussed, the empirical approach resulted in an improved documentation, and gave additional insight and understanding of the behavior of tall buildings in earthquakes.

9.3 PERFORMANCE OF SYSTEMS

Based on the combined evidence for all MMI levels and using a significance level of 5%, the following trends could be confirmed as statistically significant:

- Steel buildings experience a 51% smaller mean DR and a 96% smaller median DR than concrete buildings.

- High-rise buildings experience a 63% smaller mean DR and a 80% smaller median DR than medium-rise buildings.

- Regular buildings experience a 29% smaller mean DR and a 70% smaller median DR than irregular buildings.

While "other" structures experience a 14% smaller mean DR and 67% smaller median DR than pure moment-resisting frames, this trend could not be confirmed as statistically significant. Moreover, inclusion of more undamaged buildings in the study may change these observed trends.

Based on the evidence for individual MMI levels, the above trends could not be confirmed as statistically significant in the majority of the cases. This is true whether the selected significance level is 1%, 5% or 10%.

Based on the evidence from all MMI levels combined the order of importance of the investigated parameters can be specified. If a significance level of 1% is chosen, material (steel/concrete) and tallness (high-/medium-rise) --or fundamental period-- are the significant parameters affecting damage. At a significance level of 5% configuration (regularity/irregularity) of the building becomes significant as well. The structural system (pure moment-resisting frames/others) would be significant if a level of 10% was specified. The last rank of structural system indicates only that the difference between frames and "other" structures may not be significant. Comparisons between other types of structural systems may lead to different results. Nevertheless, the order of importance found appears to justify the emphasis that ATC [1978] places on good detailing of reinforced concrete structures and on more sophisticated analysis for irregular structures.

From the digests it is evident that quality of design, detailing, and construction is one of the most important parameters. It must be realized, though, that quality is assessed after the fact, through a deliberate search for defects in severely damaged buildings, while in lightly or undamaged buildings quality remains unknown. This parameter was therefore not investigated in this preliminary study. However, it is intended to investigate in the more extensive study, what the most frequent technical reasons and ultimate causes for damage are.

The conclusions presented here must be used with care and attention to the limitations stated in the previous sections. The study at individual MMI levels did not give enough evidence to confirm the trends that were significant for all MMI levels combined. For more reliable results the quality and size of the samples need to be improved. In particular, much more detailed data on undamaged buildings is needed. But as these data are not usually available in the literature and the quality of information on damaged buildings is quite variable, other investigators concluded that the analytical approach might be more promising [Scholl, 1982]. Whereas the results of this study do not conclusively show that the empirical approach will be successful, it seems reasonable to expect that improvements in size and quality of the sample and use of EIS for earthquake classification might reveal which systems or combinations of systems in tall buildings are more effective and which are less so in resisting earthquakes.

9.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Due to the limited scope of this preliminary study and the small sample size, only a few parameters have been investigated. However, more parameters that were envisioned to be examined in a more extensive study are recommended for further research.

- Use Engineering Intensity Scale (EIS) instead of Modified Mercalli Intensity (MMI) for classification of earthquakes in order to investigate whether this improves results.

- Investigate in more detail the performance of various structural systems, in particular, the performance of buildings employing central cores only as lateral load resisting system. In ATC [1978] it is mentioned that use of a central core alone to resist lateral forces in a building may represent some type of irregularity. Empirical studies might substantiate this statement. In addition, different types of bracing systems such as shear walls, cores, or rigid frames should be compared.

- Investigate the effect of building age, applicable code, and quality control procedures on the performance of tall buildings.

- Investigate the effect of isolation of nonstructural elements from the structural framing as well as the importance of their weight on the degree of damage in tall buildings.

- Investigate more parameter combinations, e.g. investigate whether the performance difference between steel and concrete buildings depends on building age, applicable code, etc.

- Investigate what are the most frequent technical reasons and ultimate causes for damage.

For a more reliable data assessment, future research should concentrate on the following activities:

- Include more undamaged buildings in the study. Work, if possible, with representative samples of buildings, where each building has been randomly and independently chosen from the population under study.

- Increase the number of buildings studied as well as the detail of information for each building as required by the classification schemes.

- Establish damage state-damage ratio relationships for building subsystems and components.

- To improve the quality of the data, reporting on buildings that have been subjected to earthquakes should become more consistent and systematic. A common terminology (for structural systems, members, types of failures, damage etc.) and structure of reporting should be developed. Some recommendations in this regard are given in section 9.2.

ACKNOWLEDGMENTS

The study reported herein was conducted as part of a larger project "Earthquake Resistance of High-Rise Systems" (Lynn S. Beedle, Principal Investigator), with financial support provided by the National Science Foundation under Grant No. CEE-8105306.

The authors would like to express their gratitude to Lynn S. Beedle, Director of the Fritz Engineering Laboratory, for his support and guidance, and to David A. VanHorn, Chairman of the Department of Civil Engineering, for his assistance. The valuable comments and criticisms of Lynn S. Beedle, Le-Wu Lu, and George C. Driscoll are appreciated and their contribution is gratefully acknowledged. Thanks are also due to Tom D. Cole, who helped in the data collection and literature survey.

APPENDIX A

CLASSIFICATION OF TALL BUILDING SYSTEMS

AND EARTHQUAKE DAMAGE THERETO

Table A-1

FALCONER - BEEDLE CLASSIFICATION SCHEME (AMENDED)

Level A: PRIME FRAMING SYSTEMS AND COMBINATIONS

BEARING WALL
10 Bearing Wall(BW)
11 BW & frame
12 BW & core

FRAME
50*Frame(F)
51 Simple Frame(SF)
52 Semi-Rigid Frame (SRF)
53 Rigid Frame (RF)
54 F & shear walls
55 SF & shear walls
56 SRF & shear walls
57 RF & shear walls
58 F & core
59 SF & core
60 SRF & core
61 RF & core
62 Exterior truss frame
63*F & braced frame
64*SF & braced frame
65*SRF & braced frame
66*RF & braced frame

CORE
20*Core(C)
21 Perimeter Core (PC)
22 C w/suspended floors(CS)
23 C w/cantilevered floors(CL)
24*Central Core (CC)
25*Offset Core (OC)
26 C & frame
27 PC & frame
28 CS & frame
29 CL & frame
30*CC & frame
31*OC & frame
32 C & shear walls
33 PC & shear walls
34 CS & shear walls
35*CL & shear walls
36*CC & shear walls
37*OC & shear walls
38 PC & CC

TUBE
80*Tube(T)
81 Framed Tube (FT)
82 Trussed Tube (TT)
83 Deep Spandrel Tube (DST)
84 Perforated Shell T (PST)
85 T-in-Tube
86 FT-in-Tube
87 TT-in-Tube
88 DST-in-Tube
89 PST-in-Tube
90*T w/interior columns
91 FT w/interior columns
92 TT w/interior columns
93 DST w/interior columns
94 PST w/interior columns
95 Bundled Tube

* denotes suggested additions to the original classification

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Table A-2

FALCONER - BEEDLE CLASSIFICATION SCHEME (AMENDED)

LEVEL B: BRACING SUBSYSTEM

FRAME BRACING	STEEL CORE BRACING
10*Concentrically Brac. Frame	30*Concentrically Brac. Core
11 Single Diagonal Bracing	31 Sing. Diag. Bracing
12 Double Diag. Bracing	32 Double Diag. Bracing
13 Horizontal K Bracing	33 Hor. K Bracing
14 Vertical K Bracing	34 Vert. K Bracing
15 Knee Bracing	35 Knee Bracing
16 Lattice Bracing	36 Lattice Bracing
20*Eccentrically Braced Frame	40*Eccentrically Braced Core
21*Eccentric Diag. Bracing	41*Eccentric Diag. Bracing
22*Eccentric K Bracing	42*Eccentric K Bracing

*MOMENT RESISTING FRAMES

50 Moment-Resisting Frame (MRF)
51 Ordinary MRF
52 Ductile MRF
53 Ductile MRF (Dual system)

SHEAR WALL BRACING	*CONCRETE CORE BRACING*
60 Shear Wall (SW)	80 Core (C)
61 Simple Shear Wall (SSW)	81 Simple Core (SC)
62 Coupled Shear Wall (CSW)	82 Coupled Core (CC)
63 Ductile SW	83 Ductile C
64 Ductile SSW	84 Ductile SC
65 Ductile CSW	85 Ductile CC

Table A-3

FALCONER - BEEDLE CLASSIFICATION SCHEME (AMENDED)

LEVEL C: FLOOR FRAMING

STEEL	CONCRETE	COMPOSITE
10*Steel	20*Concrete	30*Composite
11 Pre-fabricated	21 Flat Slab	31 Steel beam & slab (SBS)
12 Steel beam & Deck	22 Flat Plate	32 Steel joist & slab (SJS)
13 Steel joist & Deck	23 Waffle Slab	33 SBS on Metal Deck
	24 Beam & Slab	34 SJS on Metal Deck
	25 Joist & Slab(JS)	35 Concrete Encased Beam
	26 JS one-way	
	27 JS two-way	

LEVEL D: CONFIGURATION

PLAN	*ELEVATION*
0 Regular	0 Regular
1 Irregular	1 Irregular
2 Offsets, asymmetric plan	2 Offsets in elevation
3 Eccentricities in lateral resisting system	3 Changes in lateral load resistance or mass
4 Eccentric Core	4 Discontinued shear walls / cores, soft-stories
5 Eccentric shear walls or braced cores	5 Changes in story height
6 Large or irregular diaphragm openings	6 Changes in gravity load resisting system

FOUNDATION

10 Footings	20 Piles
11 Spread Footings	21 Piles & Caissons
12 Strap Footings	22 Piles & Footings
13 Wall Footings	23 Caissons
14 Combined Footings	24 Caissons & Footings
15 Mat Foundation	

Table A-4

FALCONER - BEEDLE CLASSIFICATION SCHEME (AMENDED)

MATERIAL

10*Steel (St)	20*Concrete (C)
11 Structural St	21 Reinforced C-I-P C
12 High Strength Low-Alloy St	22 Prestressed C-I-P C
13 Mixed Steels	23 Reinforced Precast C
	24 Prestressed Precast C
30 Vertically Mixed	25 Mixed Concretes
31 Composite action St & C	
	40*Masonry (M)
	41 Unreinforced M
50 Wood	42 Reinforced M

CLADDING

CLADDING TYPE	*MATERIAL	INSTALLATION
1 Custom Walls	1 Concrete	1 Stick
2 Standard Walls	2 Precast Concrete panel	2 Unit
		3 Unit & Mullion
	3 Concrete block	4 Panel
	4 Masonry unit	5 Column/Cover/ Spandrel
	5 Drywall	
	6 Lath & Plaster	6*Isolated
	7 Metal	
	8 Glass	

PARTITION

PERMANENT	DEMOUNTABLE
11 Masonry Brick (MB)	21 Post & Infill Panels (PIP)
12 Concrete Block (CB)	22 Post & Overlay Panels (POP)
13*MB isolated	23 Postless (PL)
14*CB isolated	24*PIP isolated
	25*POP isolated
	26*PL isolated

Table A-4 (continued)
FALCONER - BEEDLE CLASSIFICATION SCHEME (AMENDED)

```
Typical designator:          FRAME:66.5012.12.02.20.13.166.14

Rigid Frame & Braced Frame---------*
Moment-Resisting Frame (Long. dir.)---*
Double Diagonal Bracing(Tran. dir.)-----*
Steel Beam & Deck Floor--------------------*
Regular in Plan------------------------------*
Offsets in Elevation-------------------------*
Pile Foundation----------------------------------*
Mixed Steels----------------------------------------*
Custom Walls------------------------------------------*
Lath & Plaster------------------------------------------*
Isolated cladding-----------------------------------------*
Concrete Block Isolated-----------------------------------*
```

Table A-5: Earthquake Damage States
[Whitman et al., 1973, Council, 1978]

DS	Level of Damage	DR(%) CDR	DR(%) Range
0	No damage	0.03	0-0.05
1	Negligible damage	0.08	0.05-0.14
2	Minor nonstructural damage --a few walls and partitions cracked, incidental mechanical and electrical damage	0.24	0.14-0.40
3	Substantial nonstructural damage -- more extensive cracking (but still not widespread); possibly damage to elevators and other mechanical/ electrical components	0.67	0.40-1.1
4	Widespread nonstructural damage -- possibly a few beams and columns cracked, although not noticeable	2	1.1-3.2
5	Minor structural damage --obvious cracking or yielding in a few structural members; substantion nonstructural damage with widespread cracking	5	3.2-9
6	Substantial structural damage requiring repair or replacement of some structural members; associated extensive nonstructural damage	15	9-25
7	Major structural damage requiring repair or replacement of many structural members; associated nonstructural damage requiring repairs to major portion of interior; building vacated during repairs	45	25-70
8	Collapse or condemnation	100	70-

(Note: DS=Damage State, DR=Damage Ratio =Ratio of cost to replacement value, CDR=Central Damage Ratio)

Table A-6: Damage classification

DAMAGE CLASSIFICATION FOR BUILDING SYSTEMS AND COMPONENTS

DS	Damage Description
0	No damage
1	Some damage, repairable, not widespread
2	Repair, stiffening, or patching required
3	Extensive damage, repair, partial replacement possible
4	Total failure

DAMAGE EVALUATION CLASSIFICATION

Technical Reason		Ultimate Cause
Critical Elements	Crit. Characteristics	
Struct. Vertical	Irregularity	Accepted risk
Str. Horizontal	Design Quality	Error (Design
Str. Connections	Construction Qual.	Construction
Foundation		Code
Nonstructural Elem.		Other)
Seismic separations		Undetected

Table A-7

General Building Data (Form 1)

1. BUILDING ID

NAME
CITY
COUNTRY
ADDRESS

COMPLETED
COST
USE

MATERIAL

NO OF STORIES
HEIGHT
PLAN DIMENSIONS
PLAN AREA
GROSS AREA

BUILDING PERIODS

SOURCES

Table A-8: Building Systems Data (Form 2)

2.BUILDING DESCRIPTION

PLAN ELEVATION

Structural

FRAMING
BRACING
FLOOR
CONFIGURATION
MATERIAL

FOUNDATION

Architectural

PARTITION____________CLADDING____________OTHER_____________

Mechanical

PLUMBING__________HVAC____________VERT TRANP____________

ELECTRICAL

Design-Construction

CODE__________EARTHQUAKE PROVISIONS__________LOADS________
LATERAL RESISTANCE_____________SEISMIC SEPARATIONS________
DESIGN QUALITY____CONSTRUCTION QUAL___MATERIAL QUAL___
REMARKS

Table A-9: Earthquake Data (Form 3)

3. EARTHQUAKE

General

NAME
PLACE/COORDINATES
DATE/TIME
RICHTER MAGNITUDE
AFTERSHOCKS
FOCAL DEPTH
FAULT, FAULT LENGTH
MAX INTENSITY
MAX GROUND ACCELERATION

DAMAGE COST
CASUALTIES
AFFECTED AREA
DAMAGE DESCRIPTION

Local

EPICENTRAL DISTANCE
MMI INTENSITY
EIS INTENSITY
EFFECTS IN THE NEIGHBORHOOD
DURATION
PEAK GROUND ACCELERATION

RECORDS AVAILABLE

Ax=	Ay=	Az=
Vx=	Vy=	Vz=
Dx=	Dy=	Dz=

BUILDING PERIODS: Tx= Ty=

SOIL(ATC)
SOIL CONDITIONS(DESCRIBE)
LANDSLIDES
BEARING PRESSURE

Table A-10: Data Collection (Form 4)

4. DAMAGE

Damage Description

DAMAGE RATIO (DR)
DAMAGE STATE (DS)
CASUALTIES
REPAIR COST______Structural_______Nonstructural______

SYSTEMS INVOLVED
Structural________Mechanical________Architectural_____

Framing____Bracing____Floor____Partition____Cladding____
Plumbing____HVAC____Vertical Transportation____Contents____

ELEMENTS INVOLVED
Structural Vertical____[Columns____Walls____]
Structural Horizontal____[Floors____Beams____Girders____]
Structural Connections_____[Bm-Cln____Bm-Slb____Slb-Cln____]
Foundations______[Spread______Pile______])

Evaluation

TECHNICAL REASON
Critical Elements: [Str.Vert.____Str.Hor.____Str.Foun.____
Str.Conn.______Nonstr.Elem.______Seism. Sep.______]
Critical Characteristics: [Str.Irreg.____Constr.Pract.____
Inadeq.Design______]

ULTIMATE CAUSE
Accepted risk____error[design____ construction____code____
none____] Undetected____

PREDICTION/PERFORMANCE
Linear Analysis: [agree______no______not av.______]
Nonlinear Analysis: [agree______no______not av.______]

RECOMMENDATIONS

Table A-11: General Form of Damage Probability Matrix

DS	Structural Damage	Nonstructural Damage	DR (%)	Earthquake Intensity <VII	VIII	>IX	ALL
0	None	None	0-0.05	33	0	0	4
1	None	Minor	0.05-0.3	11	2	0	3
2	None	Localized	0.3-1.25	22	0	0	18
3	Not noticeable	Widespread	1.25-3.5	11	2	6	4
4	Minor	Substantial	3.5-7.5	0	11	19	11
5	Substantial	Extensive	7.5-20	22	22	25	11
6	Major	Nearly Total	20-65	0	50	38	41
7	Building Condemned		100	0	4	6	4
8	Collapse		100	0	9	6	7

(Note: DS=Damage State, DR=Damage Ratio)

Table A-12: Modified Mercalli Intensity scale

INTENSITY	CHARACTERISTICS
I.	Not felt, except by a very few under especially favorable circumstances.
II.	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III.	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration like passing of truck.
IV.	During the day, felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rocked noticeably.
V.	Felt by nearly everyone; many awakened. Some dishes, windows, etc, broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI.	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII.	Everyone runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving motorcars.

Table A-12(continued): Modified Mercalli Intensity Scale

INTENSITY	CHARACTERISTICS
VIII.	Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motorcars disturbed.
IX.	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously.
X.	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed over banks.
XI.	Few, if any (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII.	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward in the air.

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Table B-1: List of additional buildings

BUILDING NAME	CITY	COUNTRY
1. Amalfi	Caracas	Venezuela
2. Altamira Apartments	Caracas	Venezuela
3. Atlantic Oil Building	Caracas	Venezuela
4. Bahia Del Mar	Carraballeda	Venezuela
5. Balmoral Apartments	Caracas	Venezuela
6. Balmoral Hotel	Managua	Nicaragua
7. Blue Palace	Caracas	Venezuela
8. Capri Apartments	Caracas	Venezuela
9. Castillete Building	Caracas	Venezuela
10. Coral Building	Caracas	Venezuela
11. Cypres Gardens	Caracas	Venezuela
12. Covent Gardens	Caracas	Venezuela
13. Deco	Caracas	Venezuela
14. Edificio Capri	Caracas	Venezuela
15. Edificio Carlos	Managua	Nicaragua
16. Edificio Roxul	Caracas	Venezuela
17. Guipelia	Caracas	Venezuela
18. IBM	Managua	Nicaragua
19. Immobilaria	Managua	Nicaragua
20. Laguna	Carraballeda	Venezuela
21. Lang	Managua	Nicaragua
22. Le Roc	Caracas	Venezuela
23. Marco Aurelio Building	Caracas	Venezuela
24. Maria Louisa Apartments	Caracas	Venezuela
25. Mijagual	Caracas	Venezuela
26. Mobil Building	Caracas	Venezuela
27. Neveri	Caracas	Venezuela
28. Nobel Building	Caracas	Venezuela
29. Palace Corvin	Caracas	Venezuela
30. Pan American Insurance Co.	Managua	Nicaragua
31. Pasaquire	Caracas	Venezuela
32. Petunia I	Caracas	Venezuela
33. Petunia II	Caracas	Venezuela
34. Plaza I	Caracas	Venezuela
35. Residencias Morgano	Caracas	Venezuela
36. Royal	Caracas	Venezuela
37. San Bosco	Caracas	Venezuela
38. San Jose Building	Caracas	Venezuela
39. Seguro La Protecto	Managua	Nicaragua
40. Sucre Apartments	Caracas	Venezuela
41. Teatro Altamira	Caracas	Venezuela
42. Texaco	Caracas	Venezuela
43. USA Embassy	Caracas	Venezuela
44. Union Building	Caracas	Venezuela

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APPENDIX B

LIST OF ADDITIONAL BUILDINGS

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