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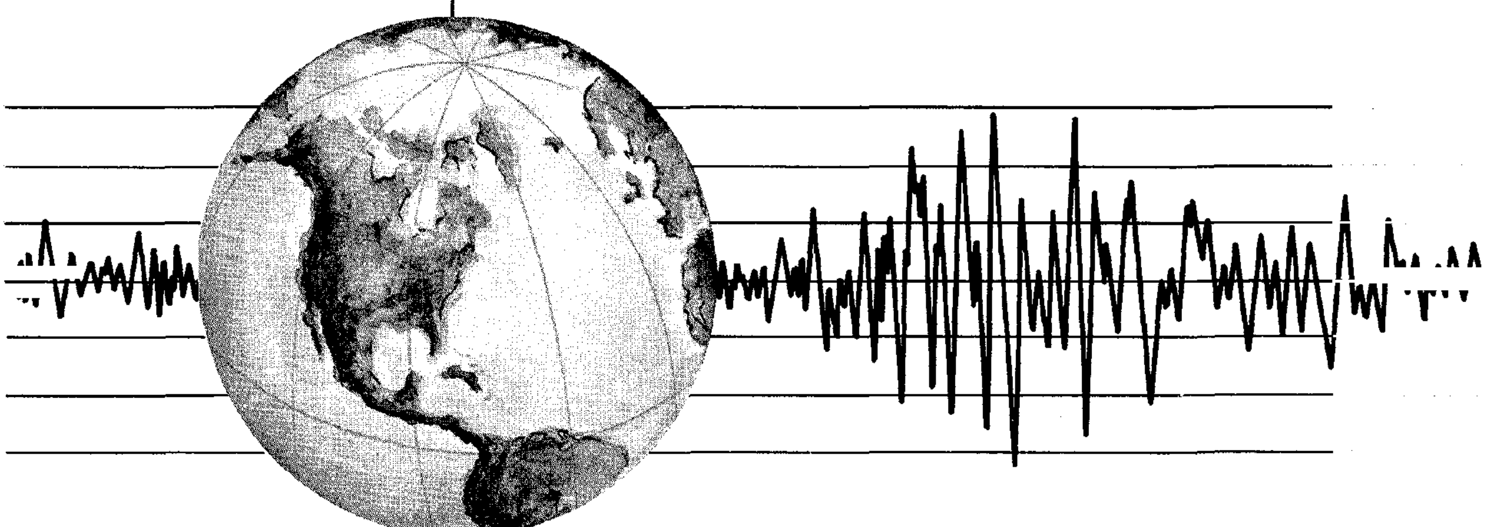
THE ECONOMIC FEASIBILITY OF SEISMIC REHABILITATION OF BUILDINGS BY BASE ISOLATION

by

JAMES M. KELLY

Report to the National Science Foundation and
the Malaysian Rubber Producers Research Association



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For the purpose of demonstrating the economic feasibility of base isolation as a method of rehabilitation, a building in San Francisco was selected for a design study. The exterior of the building constructed in 1912 as a Masonic Hall, is handsome and the interior elegantly furnished. The building has proved to be a difficult one to rehabilitate by base isolation due to its unusual configuration. The one wall of the Masonic Hall that is in contact with an adjacent building must be cut. The unusual structural configuration of the Hall has also rendered analysis of the structural framing system difficult. However, a base isolation rehabilitation scheme has been developed, drawings have been prepared and the cost to implement the scheme has been estimated.

That a practical rehabilitation scheme for an unusually difficult building has been developed indicates that suitable rehabilitation strategies using the concept of base isolation for typical masonry buildings are possible. Substantial building replacement costs will be avoided and the safety of buildings so rehabilitated greatly enhanced.

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by

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Report No. UCB/EERC-83/01

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January 1983

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ABSTRACT

The purpose of the research reported here was to assess the feasibility of the base isolation method for the rehabilitation of existing buildings that do not conform to current seismic code requirements. The number of unreinforced masonry buildings in California is estimated to be as great as 100,000. Many of these buildings will be demolished rather than strengthened due to the problems associated with the conventional procedure which involves adding new structural elements such as shear walls, internal frames or bracing. The economic feasibility of base isolation as a method of rehabilitation has been demonstrated by undertaking a specific project. For this purpose, a building in San Francisco was selected for a design study. The exterior of the building, constructed in 1912 as a Masonic Hall, is handsome and the interior elegantly finished. It must be made to conform to the current San Francisco seismic code and this, under conventional rehabilitation, would be extremely destructive to the interior of the building and extremely costly. The building has proved to be a difficult one to rehabilitate by base isolation due to its unusual configuration. The one wall of the Masonic Hall that is in contact with an adjacent building must be cut. The unusual structural configuration of the Hall has also rendered analysis of the structural framing system difficult. However, a base isolation rehabilitation scheme has been developed, drawings have been prepared and the cost to implement the scheme has been estimated. This estimate is comparable to that for a conventional rehabilitation. That a practical rehabilitation scheme for an unusually difficult building has been developed indicates that suitable rehabilitation strategies using the concept of base isolation for typical masonry buildings are possible. Given the large number of buildings at hazard in seismically active regions of the United States, it is clear that base isolation has been demonstrated to be a viable strategy. Substantial building replacement costs will be avoided and the safety of buildings so rehabilitated greatly enhanced.

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1. INTRODUCTION

Before the 1933 Long Beach earthquake, earthquake resistant design was not required by the building ordinances of any of the large metropolitan areas of California. The 1927 edition of the Uniform Building Code included earthquake design only as an option. Many structures of masonry or lightly reinforced concrete construction were built in this era and are in use today in the San Francisco and Los Angeles metropolitan areas. Although most will eventually be replaced, there are many with architectural or historical significance that justifies their retention and the costs of their rehabilitation.

The damage to a building by an earthquake is caused by the horizontal ground movements that cause the building to vibrate producing accelerations at the higher levels of the building that exceed those at the ground level. It is generally accepted that a building must have a lateral force resisting system that combines both strength and ductility to resist the forces induced by these movements. In a new building these characteristics can readily be provided by using moment resisting frames in steel or reinforced concrete with appropriately detailed beam column connections, but the development of ductility implies some damage. Further, while the structure may be saved by strengthening, higher forces may be transmitted to the nonstructural components and to the contents, and the danger to the occupants may be increased. A different approach to the protection of buildings from earthquake, that of base isolation, has recently become a practical possibility.

Base isolation is a strategy for reducing the effects of earthquake on buildings by the use of a number of possible mechanisms which uncouple the building from the horizontal components of the earthquake ground motion and simultaneously support the vertical weight of the structure. While many base isolation systems have been proposed over the years, until recently none has been considered sufficiently practical to be implemented. With the development of multilayer elastomeric bearings, the concept has become a practical possibility. The bearings for use in aseismic isolation systems for buildings are a development of elastomeric bridge bearings. The vertical stiffness of the bearings is high and the horizontal stiffness low. Many years of experience with bridge bearings have shown that they are equivalently as strong and reliable as reinforced concrete components in bridges. Recognition of the engineering qualities of elastomeric bearings has led to their application in aseismic base isolated in several countries, but not so far in the United States.

Considerable research supported by the National Science Foundation and the Malaysian Rubber Producers' Research Association has been carried out on base isolation at the Earthquake Simulator Laboratory of the Earthquake Engineering Research Center of the University of California, Berkeley, on the large square shaking table at that facility. This research has established that the base isolation of structures is technically feasible. The remaining unanswered questions pertain to its economic feasibility. The projects completed or under way in countries other than the United States suggest that the concept is economical for new buildings.

The purpose of the research reported here was to assess the economic feasibility of the method as a strategy for the rehabilitation of existing buildings that do not conform to current seismic code requirements. The number of masonry buildings in California is estimated to be as great as 100,000. Many of these buildings will be demolished rather than strengthened due to the problems associated with the conventional rehabilitation procedure. This procedure involves adding new structural elements such as shear walls, internal frames or bracings. In many cases, these elements eliminate usable space in the interior of the structure. If base isolation were used, the alterations to the building would be primarily to the foundation. A building in San Francisco has been selected for a design study. The building was constructed in 1912 as a Masonic Hall. The exterior is handsome and the interior elegantly furnished. It is of sufficient architectural merit and in such a location that it could not be demolished without public outcry.

Due to an intended change in use, however, it must be made to conform to the current San Francisco seismic code. Conventional rehabilitation would be extremely destructive to the interior of the building.

The internal structure of the building is unusual and has complicated the analysis of the framing system. In some parts of the building the quality of the concrete is questionable and one of the exterior walls is in contact with that of an adjacent structure. A base isolation rehabilitation scheme has been developed, drawings have been prepared and the cost of the scheme has been estimated. A comparative conventional rehabilitation scheme has also been developed and its cost estimated. The base isolation scheme is superior to the conventional rehabilitation and will provide greater protection than required by code for rehabilitated buildings. In the base isolation scheme the exterior wall in contact with the adjacent building will be cut and a seismic gap provided between the two buildings. In the conventional scheme, this wall will remain, causing an interaction between the contacting buildings and negating to a certain extent the benefits of the rehabilitation procedure. Any conventional scheme would also considerably damage the interior of the building. Restoration of the fine interior to its original condition is an expense not included in the cost estimate. Even when the cost of cutting the common wall is included, the base isolation scheme is not more expensive than the conventional scheme, and if the costs of restoring the interior could be reliably estimated and included, the conventional scheme would cost much more.

2. THE MASONIC HALL

The former Masonic Building was designed in 1911 by the well-known San Francisco architectural firm of Bliss and Faville. In styling it is neo-Italian-Renaissance and although it is seven stories high it is much taller than a seven-story building would ordinarily be. The elegance of the exterior is shown in the Bliss-Faville elevations (Figures 1 through 4) and the fact that the exterior has been well maintained is shown in the recent photograph (Figure 5).

The building was designed for a Masonic order and, as such, consists of two-story reception rooms, banquet rooms, a drill hall with sprung wood floor, and other lodges together with a commandery hall (500-seat theatre), an imposing architectural space a full three stories in height topped by a domed ceiling. The building was abandoned by the Masonic Order (at their structural engineer's request) when they moved to larger modern quarters in 1951 but the drama of the interior remains (Figures 6 and 7). The interior of the building is lavishly eclectic, generally of fine carved wood paneling, parquet and marble floors, and ornate vaulted ribbed ceilings. The building would make an ideal performing arts center. It has a theatre and any of the larger meeting rooms could become recital halls or performance and rehearsal rooms. Many of the meeting rooms have double walls that were originally designed to separate them acoustically from adjacent rooms, and to create an architectural shell within a structural shell. The sound insulation which results makes them even more attractive for this type of use. Some rooms have loft pipe organs—there are six in the building—all of which are functional. It would be essential in any rehabilitation scheme that these architectural details be preserved. The building is within two blocks of the Civic Center, the Opera House and the new Davies Symphony Hall which have become the focal point of the arts in San Francisco.

Given the uniqueness of the building and its potential when rehabilitated, it is clear that any scheme used to bring the structure into conformity with the seismic requirements of the city must be carefully designed to preserve the special character of the building.

3. STRUCTURAL CHARACTERISTICS OF THE MASONIC HALL

The structural system of the building is quite standard for its era although it is uncommon in modern structural engineering practice. The three principal elevations—the north, south and east elevations—all front onto streets and, as such, are architecturally imposing, consisting of a window arch colonade dressed in marble from the ground to the midheight of the second floor. Granite facing continues to the parapet of the building. The Gothic arch windows are repeated again from the fourth floor to the midheight of the fifth floor and are narrower in width. At the attic there is a line of small windows. Above this is a fluted Gothic frieze cantilevered out to form the parapet of the building.

The building has six levels, including an extended basement which is 110 ft. 9 in. (33.76 m) in height, 116 ft. 2 in. (35.41 m) in width and 154 ft. 2 in. (46.99 m) in length. The fenestration between Levels 1 and 2 as shown in the elevations (Figures 1 through 4) produces a structure that has a soft story at this level when compared to the stiffness of the level above. A soft story occurs again between Levels 4 and 5, again due to window openings as shown in the elevations. The interior wood paneled walls are mounted on a metal lath and plaster construction base. These walls are positioned for acoustic reasons so as to allow an access space up to 3 ft. (0.9 m) in width where the interior walls flank the exterior structural walls. Within the core of the building these interior walls are designed to cover the interior columns leaving an access space between these walls. The exterior architectural surfaces are tied to a concrete spandrel wall frame which is cast around a structural steel girder floor and column system that is the principal load-carrying system of the building.

The consulting engineers at the time, Galloway and Markwart, designed the structure with typical concrete encased beams spaced at approximately 7 ft. (2.13 m) on center supporting a 3.5 in. (89 mm) deep concrete slab with about 0.17 sq. in. (110 sq. mm) of square bar reinforcement per foot. This secondary beam system spans an average of 20 ft. (6.1 m) and is carried by girders or built-up girders to the columns. The columns typically are built-up sections with flanges composed of a 7/8 in. (22 mm) cover plate attached to angles in turn attached to a 7/8 in. (22 mm) web plate that forms a column typically 18 in. × 18 in. (457 mm × 457 mm) at the ground floor/basement level, which reduces to 12 in. × 12 in. (305 mm × 305 mm) at Level 6. The columns are supported on a iron base plate cast with stiffening ribs and set on a concrete footing. The built-up column is bolted to the cast iron base. All the connections are riveted and consist of 3/4 in. (19 mm) or 7/8 in. (22 mm) diameter rivets.

The connections are generally of the shear type for beam to girder connections. The moment capacity of the girder to column connections is limited. The moment connection is achieved by angles riveted from the column flanges to the girder flanges. A system of two 32 ft. 6 in. (9.91 m) deep trusses extending from Levels 2 to 4 and spanning some 62 ft. (18.9 m), shown on the longitudinal section (Figure 8), supports the commandery hall (Figure 6) and the ceiling of a large assembly room below. The truss consists of riveted plates and sections encased in concrete for fireproofing. The truss diagonals transfer lateral loads from Levels 4 to 2. The truss is not visible internally as wood paneling walls cover the truss from Level 2 to the underside of Level 4 and form the banquet room.

Above the commandery hall is a dome which has been modified to a lower profile to incorporate a shallow plaster dome ceiling. The exterior wall arch window colonade at Level 1 gives an average concrete column dimension of 5 ft. 6 in. (2.08 m). A deep spandrel wall is above the arches with relatively few window openings up to the underside of the windows on Level 4. At Level 4 the arch windows continue but are narrower and increase the average concrete column width dimension to 12 ft. 3 in. (3.73 m). At the attic, Level 6, the small dormer windows reduce the average concrete column width dimension to 6 ft. 10 in. (2.08 m). The spandrel wall is solid and continues up to the parapet level.

From the basement to the first floor only, the steel columns are fireproofed by concrete, reducing the column dimension to 2 ft. (610 mm) square. The basement at this point extends out under the city pavement. Any shear derived from earthquake forces would have to be transferred at this level through the sidewalk diaphragm to the retaining walls. The rear wall of the building is partially in contact with an adjacent building and is formed of completely cast-in-place concrete with relatively few window openings. These look out onto a light well formed by the two buildings.

Various room functions and the architectural design lead to a variety of floor shapes at each level. There are many additional irregularities in the interior of the building producing a wide variety of column loads. Levels 1, 2, 4 and 7—the roof level—are rigid diaphragms. Levels 3, 5 and 6 are unsupported mezzanines cantilevered from the exterior walls, with varied openings and shapes which make diaphragm action difficult without additional horizontal bracing. New central core walls would be required to collect the horizontal seismic forces generated by these floors. Examples of the many unsupported, randomly braced diaphragms with no lateral resistance are shown in the plan view of Figure 9. There are no lateral resisting elements to restrain the mass associated with these mezzanines except for the piers along one edge. A longitudinal section (north/south) is shown in Figure 8 and a transverse section in Figure 10. A plan of the basement is shown in Figure 11.

The building in its existing condition was modeled dynamically and was found to be unable to withstand a seismic loading greater than 0.05g. The major structural weakness lies in the low strength concrete of the colonade spandrel walls poured around the steel frame. The contribution of the steel frame located on the neutral axis of the columns is negligible. The entire lateral force resistance is provided by the perimeter concrete arch colonade.

4. CONVENTIONAL REHABILITATION SCHEME

A conventional design was developed to bring the Masonic Hall up to code. In view of the poor concrete strength of exterior wall, the design was intended to provide an internally stiff earthquake-resisting system. New shear walls would be located at several places within the building to minimize interference with the open spaces and meeting rooms. Shear walls would be positioned in the central core to absorb the loads from the floating mezzanine floors (Levels 3, 5, and 6) and to restrain the building in the north-south direction. At the unsupported mezzanines the slabs would have to be extended to connect them to the new shear walls. In addition, extensive work would be required at all levels. The central core shear walls would be located in duct spaces or vent shaft areas to minimize damage to the interior wood paneled walls. Other shear walls would be located around the perimeter to support the existing rigid diaphragms formed by Levels 1, 2, 4 and 7, and to tie into the existing spandrel walls. Diaphragm flanges would have to be reinforced to resist high tension and compression forces. The introduction of shear walls would have a major effect on the interior of the building. The locations of these shear walls are shown in Figures 12 through 16. Longitudinal and transverse sections are shown in Figures 17 and 18.

The shear walls would be positioned between the existing steel columns and typically would span one column bay. The existing steel column would form part of the main tension steel or compression steel of the shear wall. In the case of the exterior walls, the existing columns would generally be eccentric to the new shear walls. A new steel member would have to be welded to the existing column and embedded in the new shear wall. This would not only provide steel reinforcement but would also use the existing column dead load forces to counteract overturning. An opening could be cut at each floor level. A connection to the concrete spandrel beam/steel frame system could be cast into the shear wall at that level to act as a general diaphragm collector.

Above the second floor, the existing architectural shell mentioned earlier might have to be removed at various points to allow the new shear walls to be constructed. The cost of this procedure would depend on whether the original architecture was replaced or rebuilt. Reproducing the work of the craftsmen of 1911 would certainly be more expensive than demolishing the existing work and replacing it with modern materials. In the central core adjacent to the elevator shaft the same technique would be adopted by creating a shear wall between existing columns stripped of their concrete fireproofing.

Additional cover plates would be welded to the existing steel column to form a box section that would be needed at the junction where the north/south and east/west shear walls intersect that column. At the other end of this shear wall, a cover plate would be welded to the existing column, again forming a box section. Due to the seismic shear loads acting on the east/west central core, the wall would be three column bays in width for a length of 48 ft. (14.63 m) at the basement (Figure 17) and would be continued at this dimension to Level 3. At this point, the shear wall would be reduced to two bays up to Level 6, and further reduced to one bay to engage the roof diaphragm. The shear walls with a concrete strength of $f_c' = 4,000$ psi (27.58 MPa) have been designed to take the full seismic loads required by the San Francisco building code, which in a shear wall building would be due to a base shear of 0.186g. The concrete floor diaphragms at Levels 3 and 5 would act in the north/south direction and would require additional steel cross bracing under the existing concrete floor to accommodate the resulting shears and chord stresses.

The concrete floor diaphragms at the other levels would require local steel reinforcement to transfer the chord and shear stresses to the new shear walls. Generally the existing steel framing member at the diaphragm boundary would absorb the resulting chord stresses. The 3.5 in. (89 mm) slab can accommodate the horizontal diaphragm shear stresses.

The foundation would contribute a major part to the expense in the conventional rehabilitation. The existing footings would also have to be strengthened not only to resist the seismic forces generated by these shear walls but also to support the additional weight of the walls themselves. Columns would have to be temporarily supported while the existing concrete footings were removed and new footings cast. A major problem was encountered with the central core shear walls in the east/west direction. The new concrete footing would have to be extended to engage the foundations of the adjacent columns and to reduce these pressures. This would have to be done to counter the high soil pressure produced by seismic overturning forces. The shear wall at the west elevation would pose a similar problem in that the existing retaining wall would have to be underpinned in sections during construction of the new footing.

In the conventional scheme the fact that the rear wall of the adjoining building is cast against the rear wall of the Masonic Hall is ignored . Under the present code a 4 in. (102 mm) expansion joint should be provided between two adjacent buildings. When the building was constructed in 1912, however, this was not considered necessary. If the conventional scheme were to provide for this seismic gap, the cost of the rehabilitation would increase significantly. Any rehabilitation scheme that does not provide a seismic gap to a neighboring building will be suspect as seismic loads from that building could compromise the scheme. The connected structures will interact strongly during a seismic attack.

5. BASE ISOLATION REHABILITATION SCHEME

Base isolation is an alternative approach to the rehabilitation of old buildings, and may be cost effective in life safety, structural strengthening and maintenance. The base isolation rehabilitation scheme was designed to protect the structure from an 0.4g peak acceleration earthquake with a response spectrum as given by the ATC-3-06 [1] recommendations (Figure 19). This is a much more severe design criterion than that currently required by the city of San Francisco, but was selected to demonstrate what could be achieved by this approach.

The first step in the design of the base isolation system is to determine the distribution of column loads. This was accomplished by estimating the weight of the various components of the existing building and including an allowance for fixed equipment and furniture. A total of 75 columns support a dead load of 9,000 tons (8,165 tonnes) and a live load of 6,000 tons (5,443 tonnes). The loads carried by individual columns vary widely. The lightest loaded column carries as little as 20 tons (18 tonnes); the heaviest carries in excess of 600 tons (544 tonnes). The distribution of column loads is shown in Figure 20. The load of each column is given in Table 1. The largest practical size of bearing with current technology is around 24 in. (680 mm) square or 28 in. (711 mm) in diameter if circular. A bearing of this size can, depending on the design and the elastomer which is used, carry a load in the range 150-200 tons (136-181 tonnes). For this reason a 24-in. square bearing was taken as the largest bearing to be used and nominally denoted as a 150-ton bearing. A study was carried out to determine the best distribution of bearing sizes and numbers under each column to minimize the number of sizes of bearing that will need to be made. The range is from 50 tons (45 tonnes) nominal load to 150 tons (136 tonnes), and one, two or four bearings are located under each column. Details of the bearing selection and design of the individual sizes are given in reference 2. The design selected uses seven different bearings. Each uses a natural rubber compound with a hardness of 60 IRHD. The maximum difference between the nominal load at a bearing cluster and the estimated column load on the bearing cluster is not more than 10%. Considering the approximate nature of the column load calculation this is completely adequate. In addition to natural rubber bearings a number of teflon bearings will be used to carry light loads below 50 tons (45 tonnes) and to reduce the span lengths for certain slabs.

A plan view and longitudinal and transverse sections of the isolated building are shown in Figures 21 through 23

Since horizontal loads in the existing building are transferred into the pavement diaphragm above the basement level, the basement level columns can carry only vertical load. When the ground floor slab is isolated from the perimeter walls some method of providing lateral stiffness to these columns is needed. The method selected is to introduce a double set of shear walls between Levels 0 and 1. These added shear walls were carried to Level 2 to improve the transfer of horizontal force from the higher levels of the building in the soft story region.

Only that part of the rear wall in contact with the adjacent building need be cut away and a new curtain wall built to provide a seismic gap between the two structures (Figure 25).

The retaining wall at the back of the building must be separated from the floor slab and isolation bearings added. Column piers would have to be added to strengthen the retaining wall so that it could support new bearings under the new rear wall and the existing steel column. A plan and a section of how this will be accomplished are shown in Figure 26. A detailed diagram of the isolation joint at the perimeter of the building is given in Figure 27. The elevator and elevator guide rail would need to be isolated similarly (Figure 28).

An isolation joint would be necessary on the three sides of the building at the sidewalk to allow relative motion of the building and could be provided by sliding steel plates. The isolation joint would, however, require that sidewalk supports be added where the existing sidewalk now frames the building. For both structural schemes the basement of the building must be modified. The conventional scheme would require that the existing footings under the new shear walls be demolished. Larger footings would then be placed to accommodate the increased loads from seismic forces. The base isolation approach would not require larger footings but a structural slab would have to be used to provide base fixity to the columns and ensure uniform movement of all columns under dynamic load.

In the base isolation rehabilitation the new slab would be poured directly over the existing slab using cardboard forms that would remain in place. Teflon pad supports would be provided where the span of the self-supporting slab would be reduced to maintain a reasonable thickness (Figure 29). Where the column loads would be great enough to require more than a single bearing, large steel shearheads (Figure 30) have been designed with built-in jacking points to carry the existing columns in the structure once they were jacked up and cut free of the existing foundations. The shearheads were also designed evenly to distribute the column loads to the bearing cluster once the jacking devices were removed and the column lowered to its final position.

Both the shear and compression stiffnesses of the bearings were designed to be directly proportional to the load from each column to maintain a constant vertical natural frequency throughout the structure. The height of the rubber in the bearings is approximately 8 in. (203 mm). The bearing design will provide an equivalent viscous damping in the isolation mode of 5% which translates to a maximum building displacement of 8 in. (203 mm) based on the ATC-3-06 spectrum with a peak acceleration input of 0.4g. With a sway displacement of 8 in. (203 mm), the maximum shear strain would be 100%, well within the shear strain level that can be sustained by rubber without damage.

The shearheads will provide a future jacking capability under the columns should the bearings require maintenance or should the column loads change significantly.

6. DESIGN AND TESTING OF ISOLATION BEARINGS

The total weight of the structure is approximately 15,700 tons (14,000 tonnes) and there are 75 columns. Each column carries a different vertical load (Figure 20). There is therefore a problem in selecting the bearing sizes to restrict the number that would have to be manufactured. The predicted maximum displacement of 8 in. (203 mm) suggests a total thickness of rubber of approximately 8 in. (203 mm) to restrict the shear strain in the rubber to 100%. It is generally considered desirable to have the minimum plan dimension of the bearings be twice the maximum displacement to accommodate a lateral displacement of 8 in. (203 mm) under the vertical load. This cannot be achieved for some of the more lightly loaded bearings but provided that most of the load is carried by bearings with a minimum plan dimension greater than 16 in. (410 mm) this should be no problem. For ease in manufacturing, the largest possible bearings were taken to be 24 in. (610 mm).

A number of possible schemes were considered and the one selected uses one, two or four bearings to carry the column loads. In this scheme both the horizontal and vertical stiffnesses under the supported columns are proportional to the load for each column. The column loads for which each bearing is to be designed are given in Table 1. The design comprises seven bearings in 60 IRHD rubber with the supported load ranging from 50 tons to 158 tons (45 tonnes to 136 tonnes) in 20% increments. The characteristics of the bearings are given in Table 2 and the numbers required of each bearing are given in Table 3. The buckling load for each bearing type was estimated using the method outlined on reference 3 and the values obtained are given in Table 2. The least safety factor against buckling is for the smallest bearing, 50 tons (45 tonnes), and the greatest for the largest, 150 tons (136 tonnes).

The design of the isolation system finally selected led to a number of bearings of each type being proportioned to have horizontal and vertical stiffnesses appropriate to a working vertical load which varied from a minimum of 50 tons (45 tonnes) to a maximum of 150 tons (136 tonnes). The dynamic analysis of the building when isolated predicted a design maximum displacement for the bearings of 8 in. (203 mm). Two sets of bearings were ordered from a commercial rubber company for testing to verify that the bearings would produce the required stiffnesses and be capable of sustaining the required horizontal displacement under the vertical load. Four of the largest bearings, 150 tons (136 tonnes), and the smallest, 50 tons (45 tonnes), were manufactured.

The 150-ton bearings were roughly 20 in. (508 mm) by 24 in. (610 mm) in plan and 10 in. (250 mm) high. They contained ten layers of rubber 0.75 in. (20 mm) thick and nine steel plates 0.125 in. (3 mm) thick. Two 1 in. (25.4 mm) end plates top and bottom were included. A cross section of one of the bearings sawn through after testing is shown in Figure 31. The 50-ton bearings were 12 in. (305 mm) by 14 in. (356 mm) and 10.9 in. (277 mm) high. They contained eighteen layers of rubber 0.45 in. (11 mm) thick and seventeen steel plates 0.125 in. (3 mm) thick. Two 0.5 in. (12 mm) end plates completed the bearing. The rubber hardness in each bearing was 60 IRHD.

The bearings were tested in a test rig specially designed and constructed for this test. In this test rig four bearings were loaded simultaneously. It is necessary to test the bearings in sets of four. Previous experience with testing smaller bearings had shown that when the bearings are tested in pairs, as is the conventional practice for bridge bearings, the application of a large horizontal load produces large moments at the top and bottom of the bearings which must be resisted by the testing machine. Most commercial testing machines are not capable of resisting these moments and the accuracy of the end conditions and of the load measurement is suspect. This phenomenon persists even when the horizontal load applied to the bearings is absorbed by the test rig and is not reacted to by the testing machine, as was the case in previous tests. The four-bearing test rig applies the horizontal load to the bearings by means of a jack placed between each pair and the end moments are absorbed by large steel wide flange beams top and bottom. The test rig set up in the 4,000,000 lb Southwark-Emery Universal Testing Machine of the University of California at the Richmond Field Station is shown in the photograph (Figure 32).

A 1.5 in. (38 mm) thick steel bearing plate was welded to the cross beams top and bottom and the 150-ton bearings were connected to these plates by threaded studs screwed into holes drilled in the top end plates of the bearings. The same studs were used to connect the bearings to the large steel boxes that were used to apply the horizontal loads generated by the hydraulic jack to each pair of bearings. These steel boxes were fabricated from 2 in. (51 mm) thick steel plate and are roughly 20 in. (508 mm) on side. This method of connecting the bearings to the boxes and to the bearing plates was subsequently found to be a source of failure in the bearings and was not used for the 50-ton bearings. These were merely keyed to the boxes and the bearing plates by short cylindrical shear keys. In the first method of attachment the leading edge of the bearings tends to lift up but is constrained to remain in contact with the adjacent surface. This causes tension in the rubber in that region. In the second the bearing edge is able to lift up and the tension in the rubber is reduced.

The 150-ton bearings were loaded several times with a vertical load equal to and exceeding the working load to verify that the required vertical stiffness had been achieved in the manufacturing process. The test rig with four bearings was then loaded to the vertical working load of 150 tons per bearing (136 tonnes)and horizontal load was applied to the two stacks of bearings by the hydraulic jack while the vertical load was maintained constant. Since the required horizontal displacement was 8 in. (203 mm), the bearings were loaded through several cycles of horizontal displacement of 8 in. (203 mm) to verify the horizontal stiffness and to estimate the damping in the rubber.

The bearings were then tested to determine the maximum displacement to which they could be subjected. At a deflection of 10 in. (254 mm) one of the bearings failed in tension and the test was terminated. A hole, drilled into an end plate to take the locating studs, had gone through the end plate to the rubber layer below. The edge of the hole had a sharp lip of steel that acted as a focus for a crack which spread over the steel and rubber interface and eventually produced the failure. This result made it impossible to determine the stability limit of the bearings but clearly demonstrated that the method used to locate the bearings was unsatisfactory. In the test of the 50-ton bearings the location method was changed. During construction of the bearings the steel plates were held in place by locating mandrels through the bearings. The holes for these mandrels remained after the bearings were vulcanized and were used for cylindrical shear keys which transmitted the horizontal loads to the bearings. In addition to requiring no drilling, with the risk of damaging the bond interface, these shear keys allowed the end plates to lift up.

The 50-ton bearings were similarly loaded vertically several times. Then under a constant vertical load of 50 tons (45 tonnes) each, they were loaded horizontally to 8 in. (203 mm) several times. They were then loaded to a maximum horizontal displacement of 9 in. (230 mm). The appearance of one of the bearings at 9 in. (230 mm) displacement is shown in Figure 33, indicating the uplift at the leading edge of the bearing and the shear key. The 50-tonne bearings were needed for further tests and were unloaded and removed from the rig. Two were subsequently loaded vertically to buckling in a smaller test machine. The buckling load was 310,000 lbs (141 tonnes).

The tests demonstrate that an 8 in. (203 mm) horizontal displacement under vertical load is well within the capabilities of these bearings. The crucial factor is the ratio of the horizontal displacement to the least horizontal plan dimension. Shear strain is not a limiting factor. If larger displacements are needed to satisfy a more severe design spectrum they can be obtained by increasing the plan dimensions of the bearings. The horizontal stiffness can be maintained by decreasing the rubber hardness and thereby reducing the shear modulus of the rubber or by increasing the total thickness of rubber. This may of course reduce the buckling load of the bearings but as shown here the safety factor against buckling is so high that a reduction in buckling load would not be critical.

7. DYNAMIC ANALYSES COMPARISONS

The degree of protection afforded existing buildings by a rehabilitation scheme would normally be assessed by the seismic design requirements of the Uniform Building Code (UBC) [4] of the International Conference of Building Officials. This code assumes that the stresses developed in a building during an earthquake can be satisfactorily estimated by an equivalent static load. Since the maximum stresses in the structure occur at different times and result from the action of several modes, the stresses produced by a static load will only approximate the dynamic stresses. The method leads to a design that performs satisfactorily under earthquake loading for buildings that are reasonably regular and simply configured. A building on a rubber base isolation system cannot be directly analyzed by such a simple procedure. Base isolation achieves its beneficial result through dynamic effects and the presence of the large discontinuity in shear stiffness at the isolation level makes the building structurally unusual.

Two types of analyses have been carried out for the alternative structural forms of the building in its existing and rehabilitated conditions. The existing building and the building as conventionally rehabilitated have been analyzed by the approach recommended by the UBC. The existing building and the building when rehabilitated both conventionally and by base isolation have been analyzed using multi-mode dynamic analysis and the design spectrum recommended in the Tentative Provisions for the Development of Seismic Regulations for Buildings (ATC-3-06) prepared by the Applied Technology Council. For San Francisco this proposed code recommends the ground motion spectrum shown in Figure 19. The spectrum is based on a peak ground acceleration of 0.4g and assumes 5% damping. The spectrum for stiff soil in the range 0.4 sec. period to 3.0 sec. period is based on a constant spectral velocity of 2.0 ft./sec (61 cm/sec). Dynamic analyses were performed and the results compared. All systems were studied using elastic analysis and unreduced stresses. If one or the other of the various analyses were to be used for permit application and the ATC-3-06 recommendations were to be followed, reduction factors would be used. Since such reduction factors depend on the judgment of the design engineer, they were not considered in the comparison. Although the UBC implicitly incorporates reduction factors, the ATC-3-06 requirements are explicit. The stress calculated using the UBC recommended method will be lower than those calculated using the ATC-3-06 method in which reduction factors are not considered.

The building was modeled using the standard computer program TABS 80. The floors were idealized as rigid diaphragms and the beams, columns and walls as frames. The program can perform static (equivalent lateral load) analysis and multi-mode dynamic linear elastic analysis.

The existing structure was first analyzed using the UBC criteria and a base shear coefficient of 0.13. The structural system was found to be severely overstressed. The diagonal in the two story truss was stressed to 174% of allowable stress and the piers at Levels 2 and 3 were stressed to 140% and 130%, respectively. This UBC analysis was based on an assumed f_c'=2,380 psi (16.41 MPa) for the concrete in the piers, which are the important structural elements in this configuration. This concrete strength was obtained from Schmidt hammer results. The fundamental periods of the building in the two directions were roughly 1.0 sec. for this concrete strength.

The existing structure was then analyzed dynamically using the ATC-3-06 spectrum. The base shear in the two directions is given in Table 4. The resulting overstress factors are shown in Tables 5 and 6. The diagonal in the truss is 457% of allowable and the piers at all levels are above 100% allowable stress with a maximum at Level 2 of 406%. In an attempt to determine to what extent the overstress in the existing building could be reduced by stiffening the soft story to Level 2, it was assumed that shear walls up to the second floor level were inserted into certain of the bays in the facades at this level. In this case some improvement was noted as indicated in Tables 5 and 6, but the building remained seriously overstressed.

A full-scale conventional rehabilitation scheme was then designed and analyzed. This scheme, described in detail in an earlier section, would result in a building with periods of 0.48 sec. and 0.49 sec. in the longitudinal and transverse directions, respectively, and a base shear coefficient of 0.722 and 0.589 in these directions. The loads result in serious overstresses in several areas but this would not be considered important as the main load carrying elements would be the new system of shear walls which would not be overstressed when the reduction factors were used. The new system of shear walls would, however, transmit high loads to the two story truss and the diagonal would have to be reinforced. When the UBC approach is used to analyze the conventional rehabilitation scheme, the design base shear coefficient is 0.186 in both directions and the stresses generally below the permissible (Tables 5 and 6). The conventional scheme would be permissible under the UBC.

The base isolation rehabilitation scheme was similarly analyzed for the ATC-3-06 spectrum. The shear stiffnesses used for the bearings were based on a period of 2.0 sec. for the building considered as a rigid body above the isolators. Due to the flexibility of the structure, the periods calculated by the program are slightly greater than this, namely 2.11 and 2.12 sec. in the longitudinal and transverse directions, respectively. The base shear coefficients in the two directions are 0.195 and 0.185, respectively, and the stresses would be everywhere less than allowable. The diagonal member in the truss would slightly exceed the static allowable, but since the member is steel it would be permitted an overstress of 33% above the static level. The stresses calculated for the base isolation scheme are very much less than those calculated for the conventional procedure under comparable conditions, namely ATC-3-06 analysis. The conventional scheme would be acceptable under the current UBC Code. Although the base isolation scheme cannot be assessed under the current code, it is clearly within its limitations.

The various mode shapes and story shear forces generated by the analysis are shown in Figures 34 to 37. Figure 34 shows the mode shapes of the existing building and Figure 35 those when two story high shear walls are inserted into the existing building. The mode shapes for the conventional rehabilitation are shown in Figure 36 and those for the base isolated building in Figure 37. The story shear forces generated by the ATC-3-06 input for the various configurations are shown in Figure 38.

The displacement of the center of mass of the building on the basis of the ATC-3-06 spectrum is 8 in. (203 mm). The torsion of the building on the isolation system might produce corner displacements which would exceed this. Since the analysis of the torsion of the building cannot be adequately performed using the computer program, an analysis was performed in which the building was treated as a rigid body on the isolators and subjected to earthquake input. This analysis is described in detail in reference 5.

An elastomeric rubber bearing designed for a specific vertical load has an identical translational stiffness in each of the two horizontal axes. The center of mass of the superstructure of a base isolated building would coincide with the center of rigidity of the bearing pads, if pads under each of the columns were designed so as to carry precisely the vertical loads and to have the desired lateral stiffness. In practice, this ideal situation can rarely be expected and there is always an eccentricity between the centers of mass and rigidity.

The lateral and torsional motions of the structure are coupled if the centers of mass and rigidity do not coincide. The dynamic response of such a structure is complicated when the natural frequencies of the lower modes are closely spaced. This will inevitably happen when a regularly shaped building is mounted on rubber isolation bearings.

The analytical solution for the motion of a base isolated structure which typically has a small eccentricity of the center of mass of the superstructure with respect to the center of rigidity of the bearing pads is described in reference 5. Such a structure has its first three natural frequencies clustered around the design frequency of the bearings. In the theoretical analysis it is shown that for an undamped system the coupling between lateral and torsional motions can produce a corner displacement which is larger than that for the center of mass. However, this maximum displacement is achieved after a great many cycles of vibration. Damping even at the modest level of 5%, as assumed here for the bearings, has the effect of absorbing the motion before the peak is reached. The result of the study is that for this building, even assuming an eccentricity of 5% of the building length between the stiffness and mass centers of the system, the effect of torsion on the corner motion is negligible.

Throughout the dynamic analysis of the base isolated building, it has been assumed that the rubber will produce an effective damping of 5% of equivalent viscous damping in the isolated mode. In fact, in the shake table experiments [6-8] the damping achieved in the bearings has been higher than this and there are indications that 10% equivalent viscous damping could in some situations be a realistic assumption. At this level the stresses in the structure and the displacements would be reduced. The physical nature of the damping mechanisms in the rubber causes the damping to be frequency independent so that experiments measuring damping at a certain frequency can be used to infer damping values at other frequencies.

8. COST ESTIMATE

The drawings and design details of the two rehabilitation schemes were submitted to an independent firm of consulting engineers and the costs for each were estimated [9]. The estimates by the company suggest that both schemes will have identical costs at $2.4 million.

In the conventional rehabilitation scheme the largest items are the structural concrete at $700,000, reinforcing bars at $200,000, and structural steel at $60,000. Demolition and preparatory work is $160,000 and piling and underpinning is $200,000. Miscellaneous costs, fees, and bonds and a contingency estimate of $223,000 add a further total of $450,000 to complete the $2.4 million total.

In the base isolation scheme the largest single item is in underpinning and shoring the column bases while the bearings are being installed. A total of $520,000 has been allowed for this item, but since it is not a common practice in the contracting industry this may be too high. It is possible that experienced contractors could develop a less costly method to install the bearings. Structural concrete is estimated at $400,000 with reinforcing bars at $160,000 and structural steel at $250,000. This large cost for structural steel in comparison with that for the conventional scheme reflects the use of the steel shearheads which allow the use of four bearings under the most heavily loaded columns. Architectural refinishing is estimated at $70,000. Demolition and preparation is $150,000. Miscellaneous costs and a contingency allowance of $220,000 total $520,000 for an overall total 0f $2.4 million. The costs for each scheme are compared in Table 7.

It should be noted that some of the costs included in the isolation scheme are special only to this building and would not necessarily appear in other cases. There is nearly $0.5 million in additional structural work needed above the isolation system to stiffen the building and to alleviate the very poor quality of concrete in certain areas. Cutting away the wall in contact with the adjacent building, and the special isolation joint under the new wall is an additional cost. The total cost of architectural finishing associated with the isolation system, and demolition, jacking, shoring, and construction of the floor slab and the shearheads is estimated at $1.4 million.

The structural work associated with the conventional scheme is $1.7 million and an additional $0.5 million is assumed for architectural refinishing. However, it is important to note that the architectural refinishing associated with this scheme will not be to the quality of the existing finish. To duplicate the existing finish would cost not less than $2.0 million.

The total floor area of the building is approximately 101,000 sq. ft. so that the isolation system can be estimated at roughly $14 per square foot.

9. CONCLUSIONS

The present study has been carried out to provide a realistic basis for the assessment of an alternative seismic hazard mitigation strategy which, although directed to a particular building in San Francisco, will have application to buildings in any region of high seismicity.

Most of the buildings in the California metropolitan areas which do not meet current seismic requirements are buildings designed prior to the introduction of seismic design codes. These buildings are generally low-rise apartment and commercial buildings which were built without a lateral force resisting system and without ductile capacity. They contain brittle components such as unreinforced masonry partitions and other nonstructural elements with serious seismic hazards. In a study undertaken in 1974 [10] it was estimated that in one limited area of San Francisco there are 2,800 commercial buildings in potentially hazardous condition with a replacement cost in 1974 terms of over one billion dollars. Many of these buildings will not be replaced in the near future but neither will they be rehabilitated, not only because of the costs involved, but mainly because of the uncertainties associated with conventional methods of rehabilitation.

Recent changes in the federal tax treatment of investment real estate contained in the Economic Recovery Act of 1981 have put rehabilitation of older buildings in a more competitive position with respect to new construction. The new law provides for an investment tax credit of one quarter of the rehabilitation costs of buildings of historical or architectural significance. This credit is, in addition, available for the rehabilitation of property leased to governmental agencies and tax exempt organizations. There is thus a strong incentive to develop effective seismic rehabilitation methods for the many older structures in California presently in violation of current seismic code.

If the lateral force resisting system of a building is considered to be inadequate, it may be increased by introducing masonry interior panels between existing columns or by introducing new shear walls of reinforced masonry or reinforced concrete. However, these new structural elements can disrupt the building. Ceilings, wall, and floors may be disturbed and the cost of the disruption and restoration of these areas, which must be considered in addition to the construction cost, is highly uncertain. In many cases the only way to build the strengthening elements is by pneumatically applied reinforced concrete which adds further problems of dust and noise. In view of the high probability of cost overruns, indefinite time scales, and the inconvenience and dislocation that face owners who rehabilitate as opposed to the low probability of an earthquake attack, it is clear that many owners will choose to risk not rehabilitating their buildings. The present study in demonstrating the economic feasibility of a rehabilitation method which is at once structurally superior and much less disruptive should offer an incentive to rehabilitation.

The study carried out here has established that base isolation is a technically feasible strategy for the rehabilitation of existing building. The building studied has proved to be a difficult one to rehabilitate by base isolation due to its unusual configuration. The rear wall in contact with the adjoining building will need to be cut away and isolated. The unusual structural configuration of the Masonic Hall also rendered analysis of the structural framing system difficult. However, a base isolation rehabilitation scheme has been designed, drawings have been prepared, and the cost to complete the scheme has been estimated. This estimate is comparable to the estimate of the cost of a conventional rehabilitation. The fact that a practical isolation scheme with costs similar to conventional rehabilitation for an unusually difficult building has been developed indicates that suitable isolation rehabilitation schemes for typical masonry structures are feasible. A less complicated building could be rehabilitated by base isolation at substantial savings compared to conventional rehabilitation. For buildings with important architectural features, base isolation is the only financially viable approach that will preserve the character of the building. Given the large number of such buildings at hazard in seismic areas of the United States, it is clear that once base isolation has been demonstrated to be a viable strategy, substantial building replacement costs will be avoided and the safety of buildings so rehabilitated greatly increased.

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TABLE 1 INDIVIDUAL COLUMN LOADS

Column No.	Load (tons)	Column No.	Load (tons)	Column No.	Load (tons)	Column No.	Load (tons)
1	81	21	48	41	120	61	366
2	201	22	-	42	93	62	365
3	303	23	36	43	450	63	295
4	347	24	56	44	112	64	313
5	400	25	169	45	167	65	361
6	293	26	190	46	56	66	189
7	387	27	407	47	73	67	315
8	387	28	93	48	-	68	164
9	203	29	121	49	48	69	81
10	42	30	242	50	54	70	41
11	54	31	167	51	245	71	-
12	59	32	618	52	137	72	27
13	200	33	572	53	63	73	-
14	26	34	59	54	458	74	19
15	155	35	-	55	128	75	76
16	458	36	68	56	220		
17	41	37	550	57	59		
18	137	38	630	58	103		
19	243	39	202	59	77		
20	54	40	236	60	203		

TABLE 2 DESIGN OF BEARINGS

L (tons)	50	60	70	85
IRHD	60	60	60	60
A (sq. inches)	12.3×12.3	13.6×13.6	14.5×14.5	16.1×16.1
t (inches)	0.44	0.48	0.52	0.57
N (layers)	18	17	15	14
k_h (tons/inch)	1.28	1.52	1.80	2.17
k_v (tons/inch)	250	303	351	432
Buckling (tons)	150	200	250	340

L (tons)	105	125	150
IRHD	60	60	60
A (sq. inches)	18.2×18.2	19.3×19.3	21.2×21.2
t (inches)	0.64	0.70	0.77
N (layers)	13	11	10
k_h (tons/inch)	2.66	3.24	3.90
k_v (tons/inch)	535	619	745
Buckling (tons)	470	600	780

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TABLE 3 TENTATIVE ARRANGEMENT OF BEARINGS

Column No.	Bearings	Actual Load (tons)	Bearing Design Load (tons)	Nominal Difference
1	1×(4)	81	85	+5%
2	2×(5)	201	210	+4%
3	2×(7)	303	300	-1%
4	4×(4)	347	340	-2%
5	4×(5)	400	420	+5%
6	2×(7)	293	300	+2%
7	4×(5)	387	420	+8%
8	4×(5)	387	420	+8%
9	2×(5)	203	210	+3%
10	-	42	-	-
11	1×(1)	54	50	-7%
12	1×(2)	59	60	+2%
13	2×(5)	200	210	+5%
14	-	26	-	-
15	1×(7)	155	150	-3%
16	4×(5)	458	420	-8%
17	-	41	-	-
18	2×(3)	137	140	+2%
19	2×(6)	243	250	+3%
20	1×(1)	54	50	-7%
21	1×(1)	48	50	+4%
22	-	-	-	-
23	-	36	-	-
24	1×(2)	56	60	+7%
25	2×(4)	169	170	+1%
26	4×(1)	190	200	+5%
27	4×(5)	407	420	+3%
28	2×(1)	93	100	+7%
29	1×(6)	121	125	+3%
30	2×(6)	242	250	+3%
31	2×(4)	167	170	+2%
32	4×(7)	618	600	-3%
33	4×(7)	572	600	+5%
34	1×(2)	59	60	+2%
35	-	-	-	-
36	1×(3)	68	70	+3%
37	4×(6)	550	500	-9%
38	4×(7)	630	600	-5%
39	2×(5)	202	210	+4%
40	2×(6)	236	250	+6%
41	1×(6)	120	125	+4%
42	2×(1)	93	100	+7%
43	4×(5)	450	420	-7%
44	1×(5)	112	105	-6%
45	2×(4)	167	170	+2%

TABLE 3 TENTATIVE ARRANGEMENT OF BEARINGS (cont'd)

Column No.	Bearings	Actual Load (tons)	Bearing Design Load (tons)	Nominal Difference
46	1×(2)	56	60	+7%
47	1×(3)	73	70	-4%
48	-	-	-	-
49	1×(1)	48	50	+4%
50	1×(1)	54	50	-7%
51	2×(6)	245	250	+2%
52	2×(3)	137	140	+2%
53	1×(2)	63	60	-5%
54	4×(5)	458	420	-8%
55	1×(6)	128	125	-2%
56	2×(5)	220	210	-4%
57	1×(2)	59	60	-2%
58	1×(5)	103	105	+2%
59	1×(3)	77	70	-9%
60	2×(5)	203	210	+3%
61	4×(4)	366	340	-7%
62	4×(4)	365	340	-7%
63	2×(7)	295	300	+2%
64	2×(7)	313	300	-4%
65	4×(4)	361	340	-6%
66	4×(1)	189	200	+6%
67	2×(7)	315	300	-5%
68	2×(4)	164	170	+4%
69	1×(4)	81	85	+5%
70	-	41	-	-
71	-	-	-	-
72	-	27	-	-
73	-	-	-	-
74	-	19	-	-
75	1×(3)	76	70	-8%

TABLE 4 PERIODS AND BASE SHEAR COEFFICIENTS

SCHEMES	FUNDAMENTAL PERIODS		BASE SHEAR COEFFICIENTS	
	T_x	T_y	C_x	C_y
Scheme 1	1.05	0.98	0.265 (0.130)	0.311 (0.135)
Scheme 2	0.74	0.53	0.359	0.564
Scheme 3	0.49	0.48	0.589 (0.186)	0.722 (0.186)
Scheme 4	2.12	2.11	0.189	0.195

* (UBC Coefficient)

TABLE 5 OVERSTRESS FACTORS (%)

SCHEMES	DIAGONAL IN TRUSS	PIERS OF FRAME 5			
		Level 2	Level 3	Level 5	Level 6
Scheme 1 (UBC)	174	90	130	74	73
Scheme 1 (ATC)	457	232	328	179	174
Scheme 2 (ATC)	355	92	228	291	245
Scheme 3 (UBC)	118	2	56	88	105
Scheme 3 (ATC)	534	10	249	372	445
Scheme 4 (ATC)	108	44	82	80	59

TABLE 6 OVERSTRESS FACTORS (%)

SCHEMES	FRAME A	PIERS OF FRAME B	
	Level 2	Level 2	Level 6
Scheme 1 (UBC)	70	140	82
Scheme 1 (ATC)	176	406	181
Scheme 2 (ATC)	188	--	210
Scheme 3 (UBC)	29	16	85
Scheme 3 (ATC)	112	62	318
Scheme 4 (ATC)	88	_	114

Scheme 1 : Existing Building
Scheme 2 : Existing Building with Shear Walls added at first two levels
Scheme 3 : Conventional Rehabilitation Scheme
Scheme 4 : Isolated Building—Shear Walls added at first two levels

TABLE 7 COST ESTIMATES FOR CONVENTIONAL AND BASE ISOLATION REHABILITATION SCHEMES

Rehabilitation Item	Conventional Scheme ($ thousands)	Base Isolation Scheme ($ thousands)
Demolition	160	150
Piling, Underpinning, Shoring	200	520
Structural Concrete	700	400
Reinforcing Steel	200	160
Structural Steel	60	250
Carpentry	40	40
Architectural Refinishing (Lath, Plaster, Flooring)	470	280
Mechanical/Electrical Systems	120	70
Contingency	150	220
Misc. Costs & Fees	300	300
TOTAL	2,400	2,400

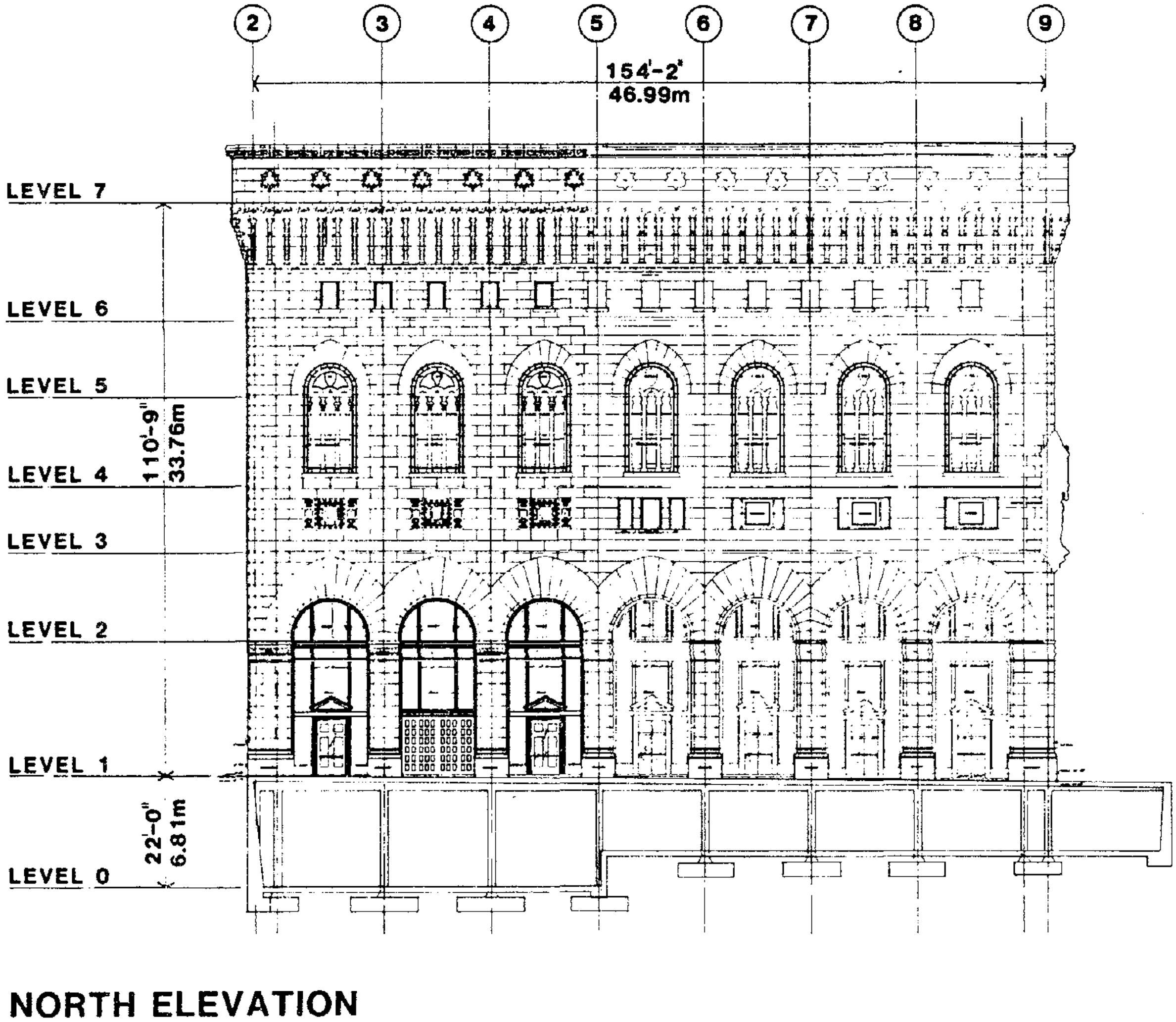

FIGURE 1 NORTH ELEVATION OF THE MASONIC BUILDING

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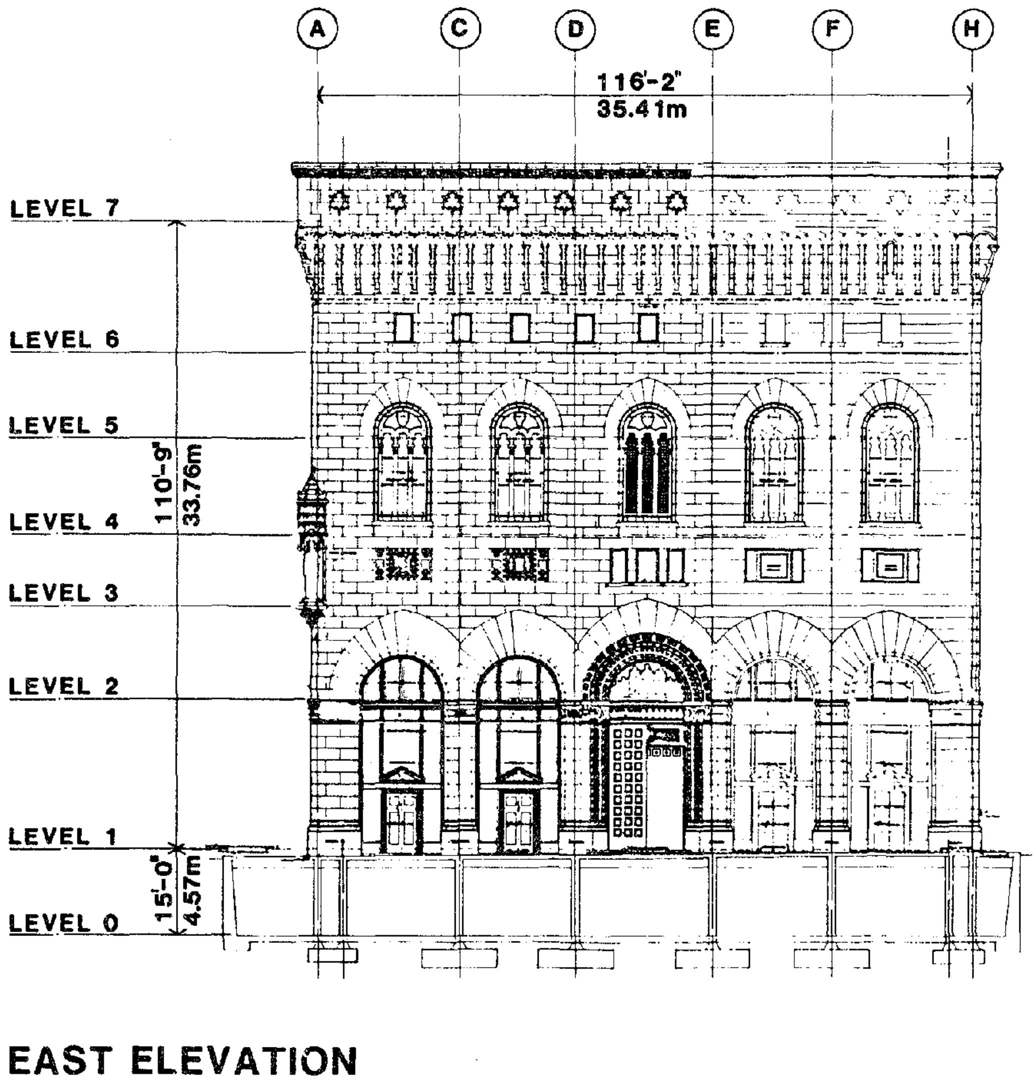
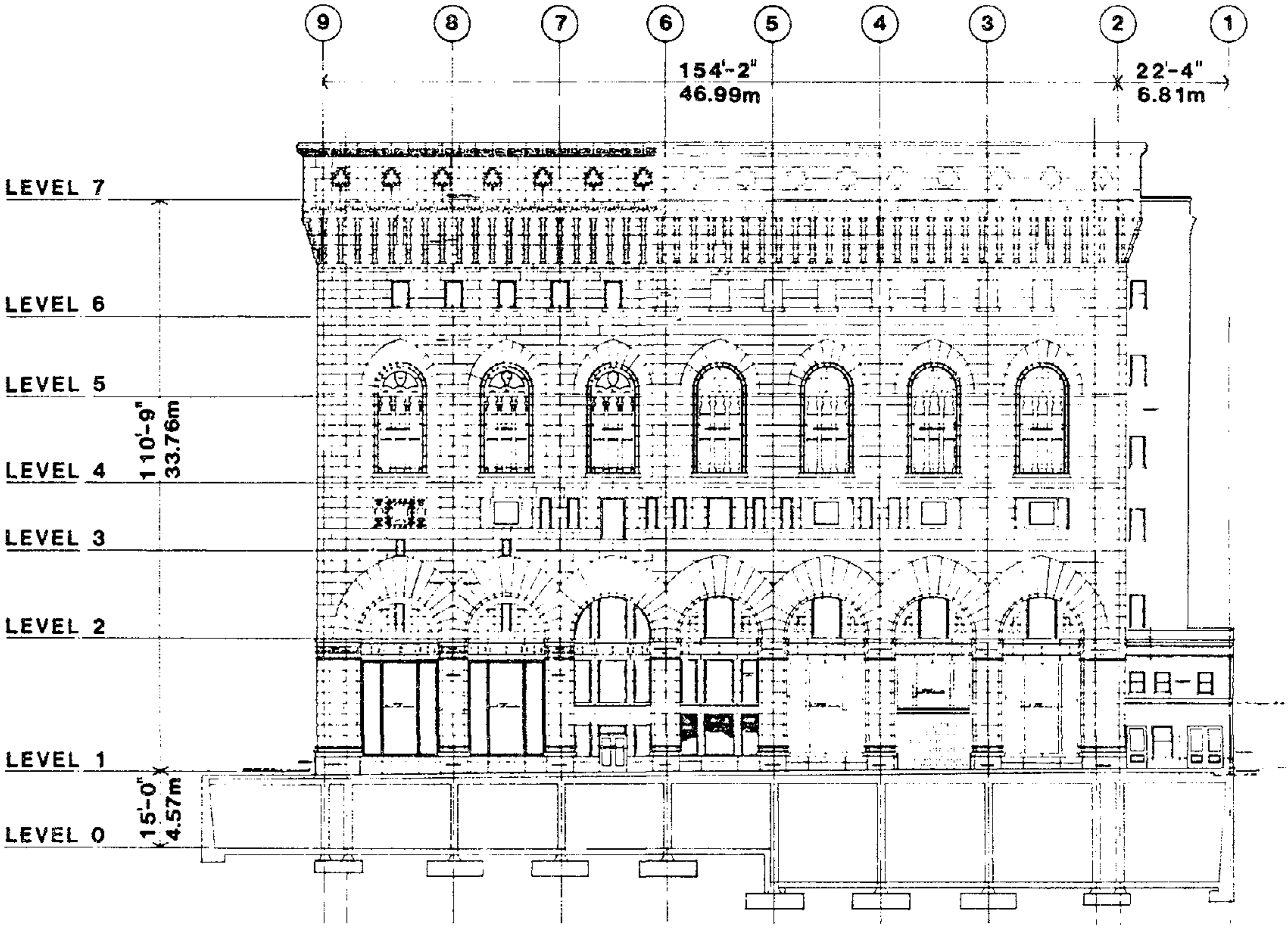


FIGURE 2 EAST ELEVATION OF THE MASONIC BUILDING



SOUTH ELEVATION

FIGURE 3 SOUTH ELEVATION OF THE MASONIC BUILDING

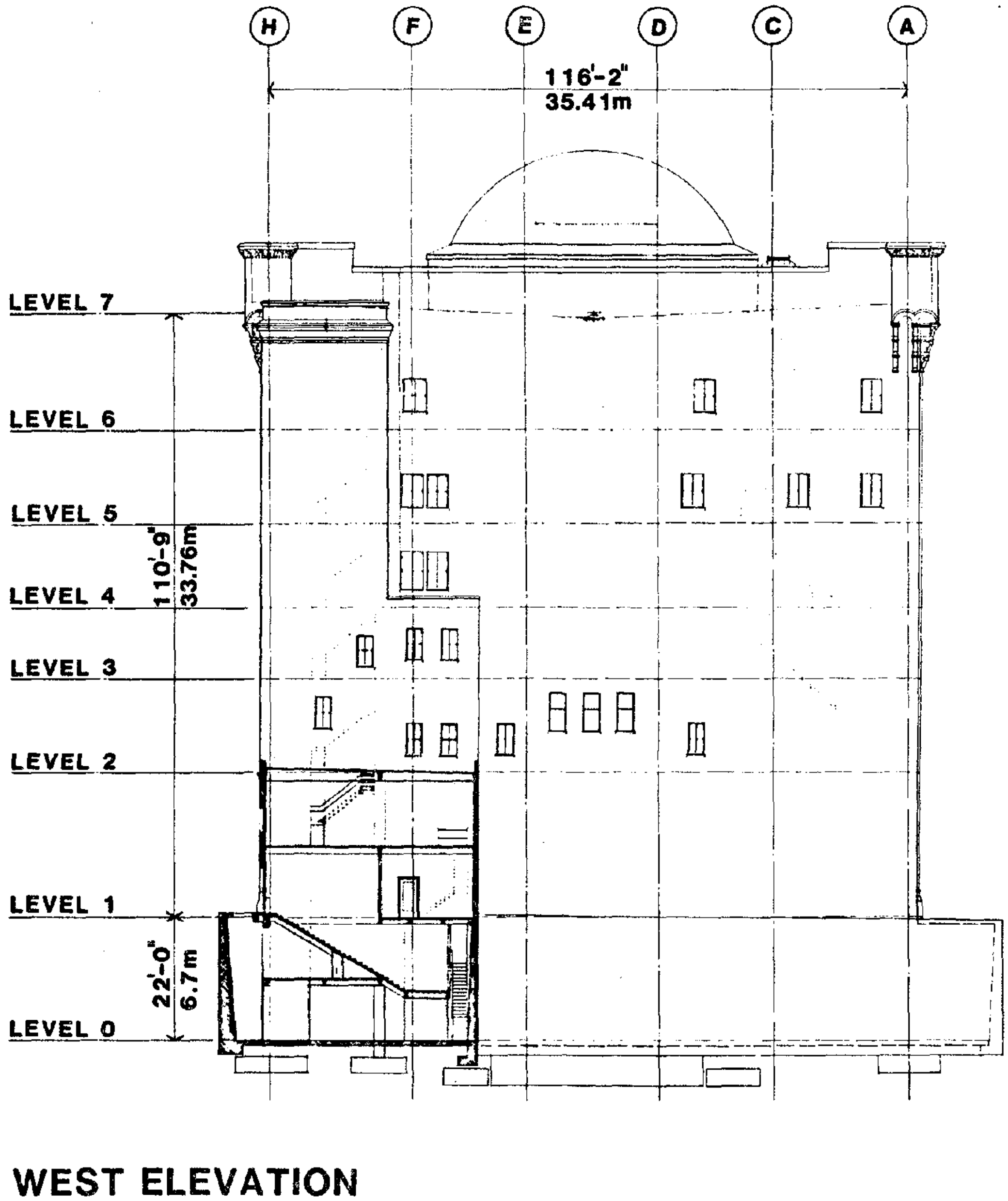


FIGURE 4 WEST ELEVATION OF THE MASONIC BUILDING

FIGURE 5 RECENT PHOTOGRAPH OF THE MASONIC BUILDING FROM VAN NESS AVENUE

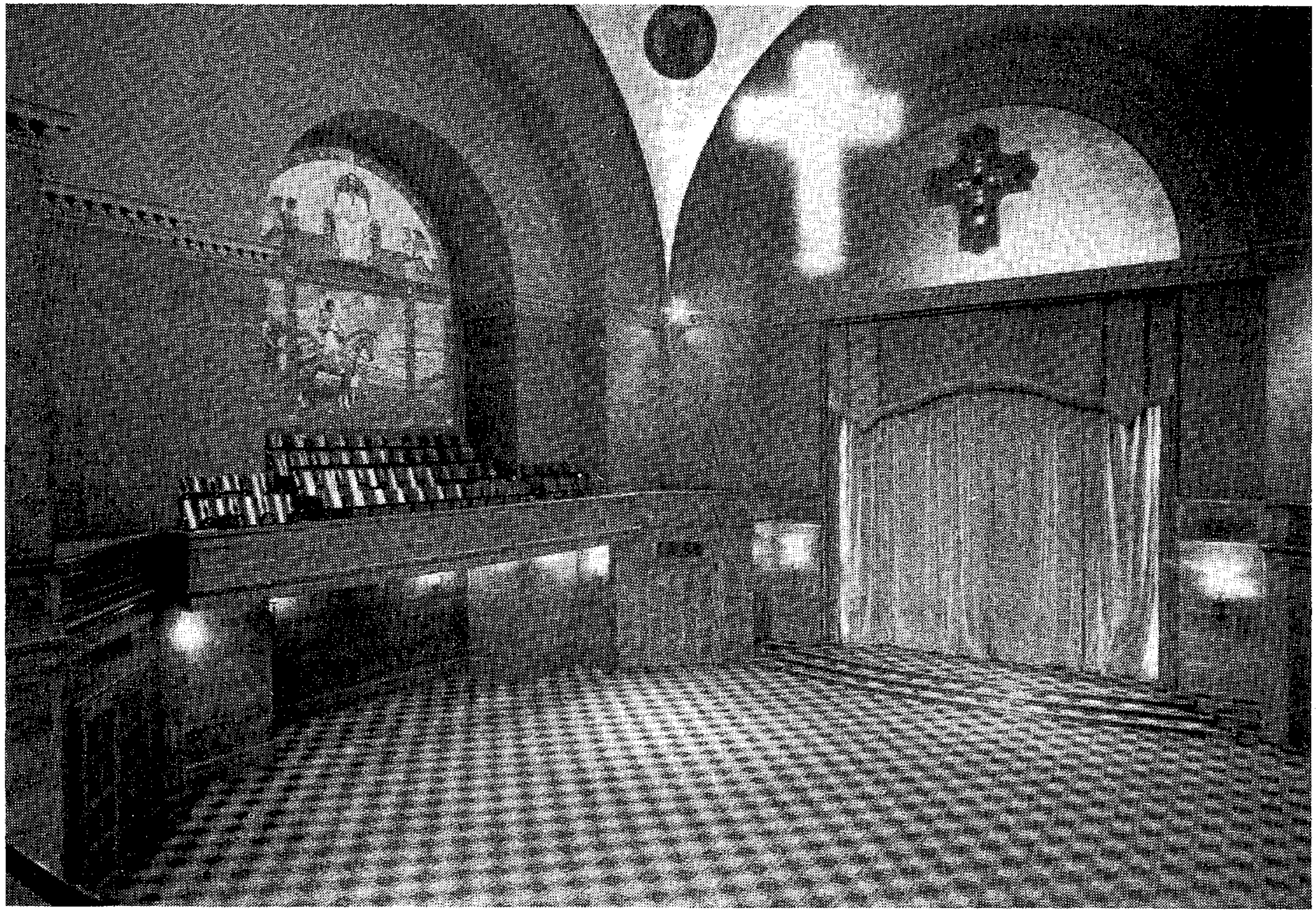
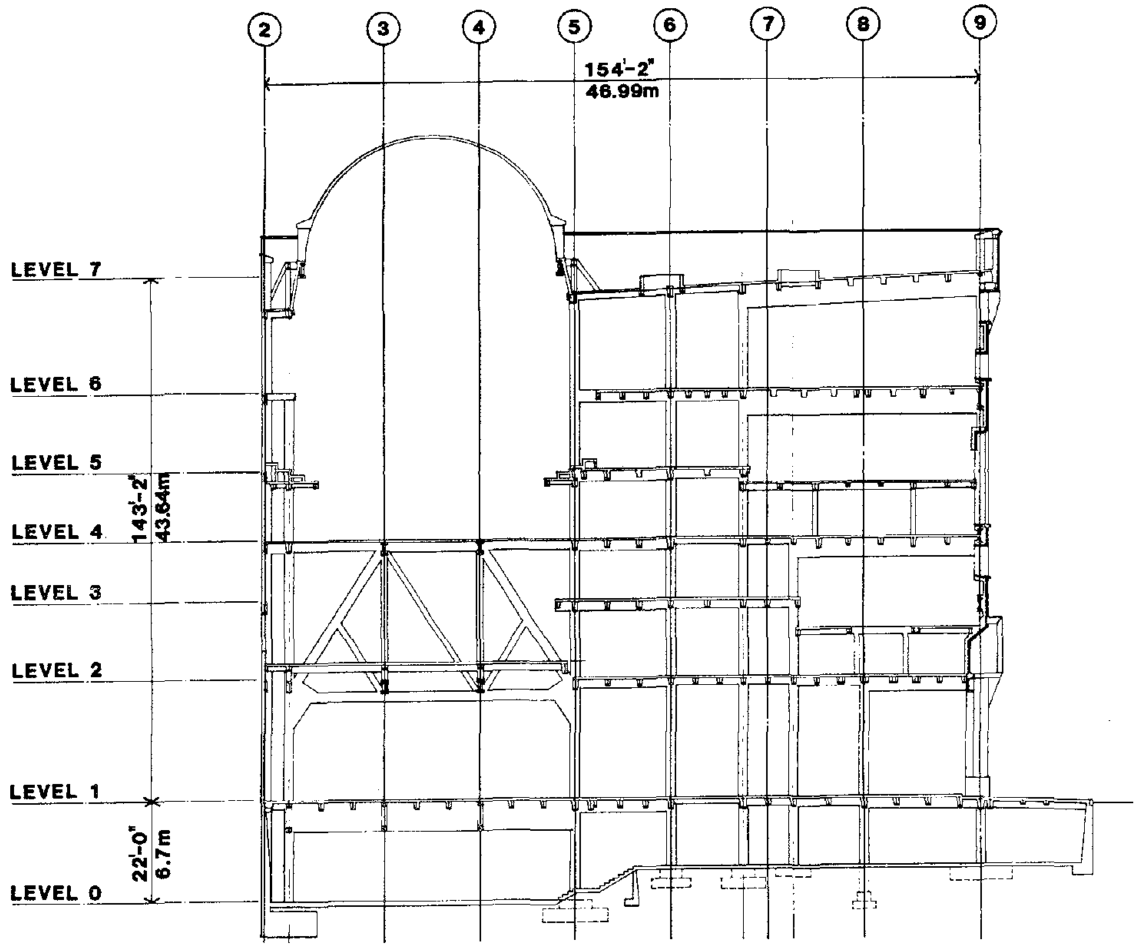


FIGURE 6 RECENT PHOTOGRAPH OF A THEATRE IN THE MASONIC BUILDING

FIGURE 7 RECENT PHOTOGRAPH OF A LARGE MEETING ROOM IN THE MASONIC BUILDING



LONGITUDINAL SECTION

FIGURE 8 LONGITUDINAL SECTION

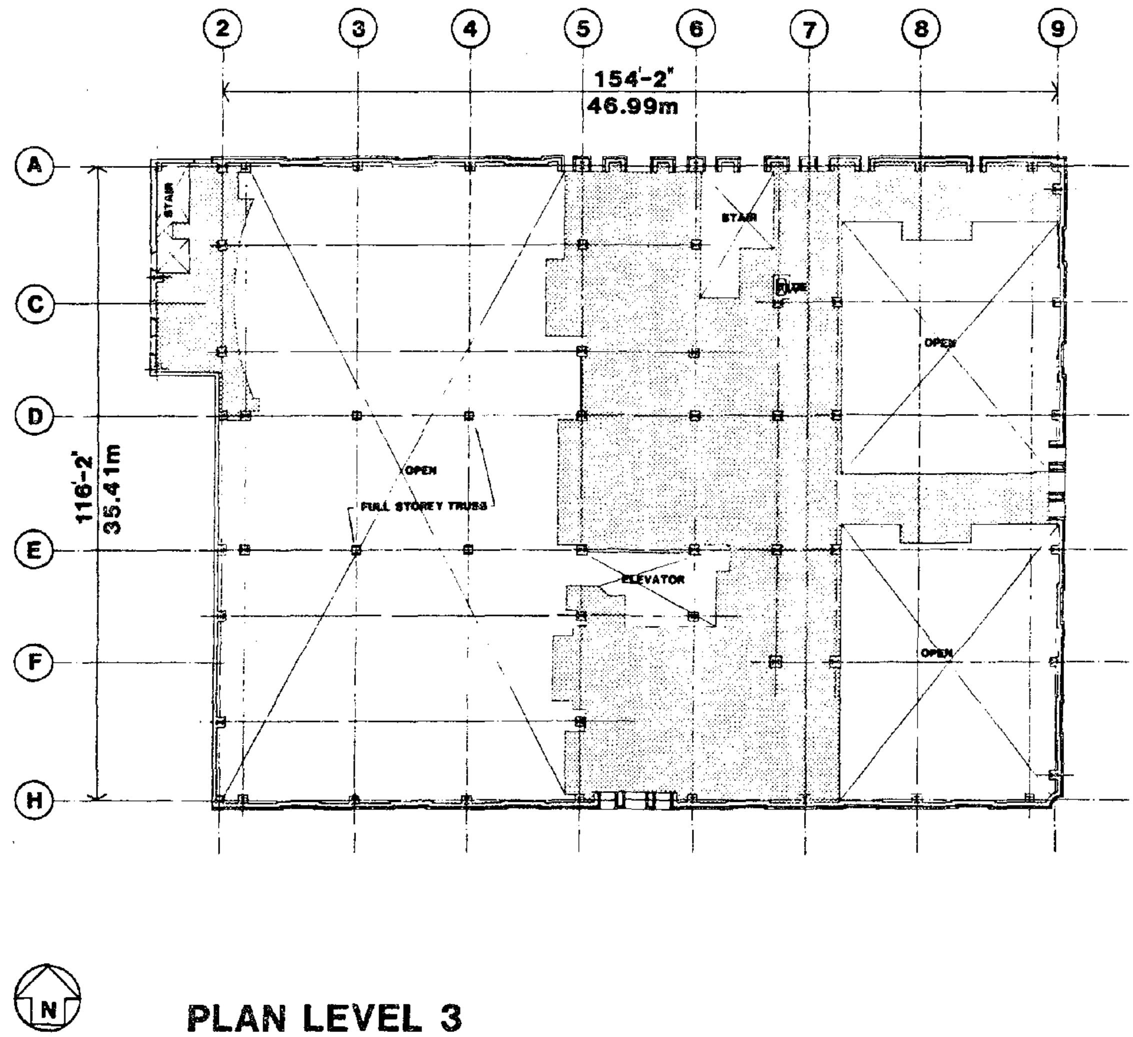


FIGURE 9 A TYPICAL FLOOR PLAN SHOWING MEZZANINE AREAS

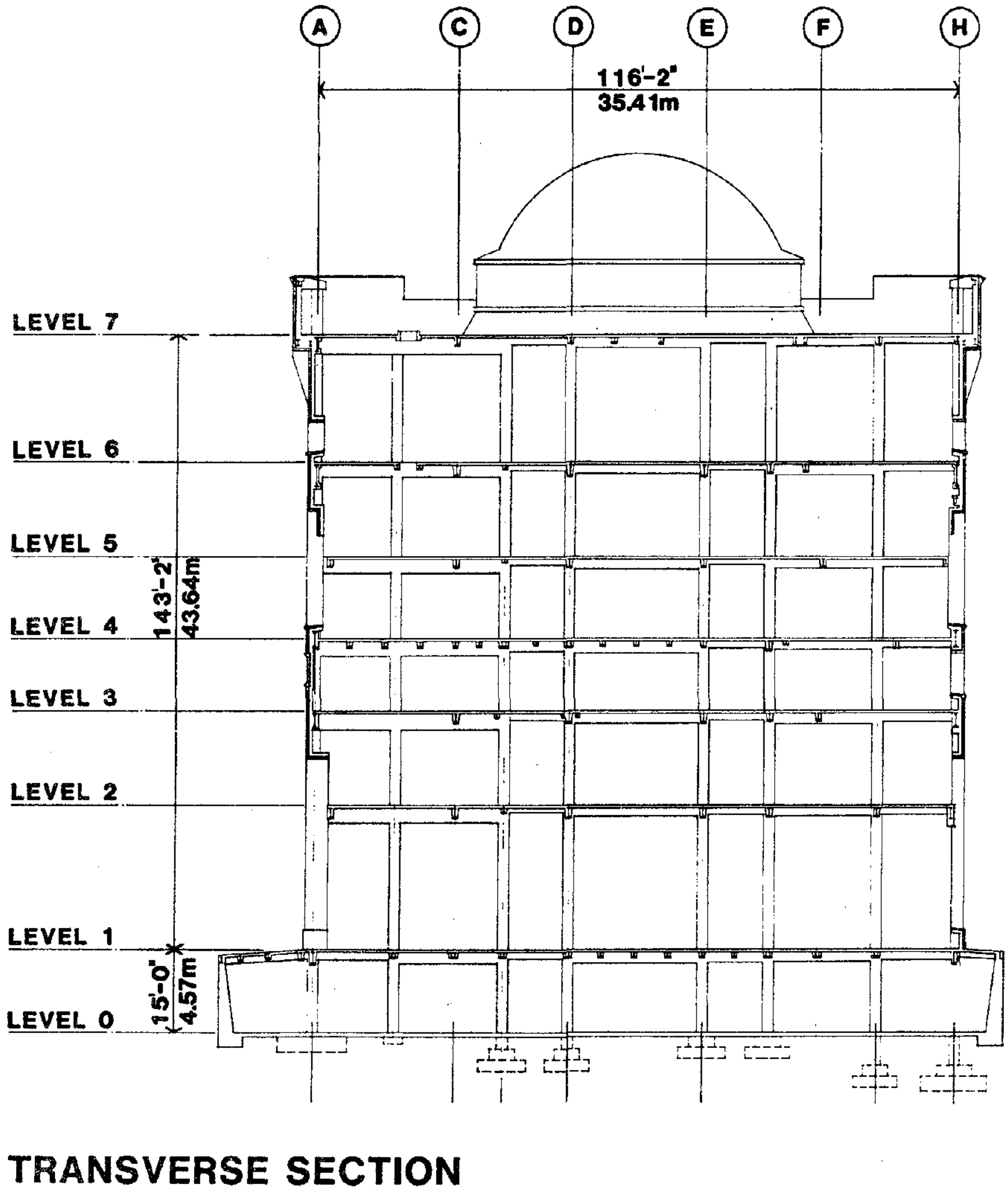


FIGURE 10 TRANSVERSE SECTION

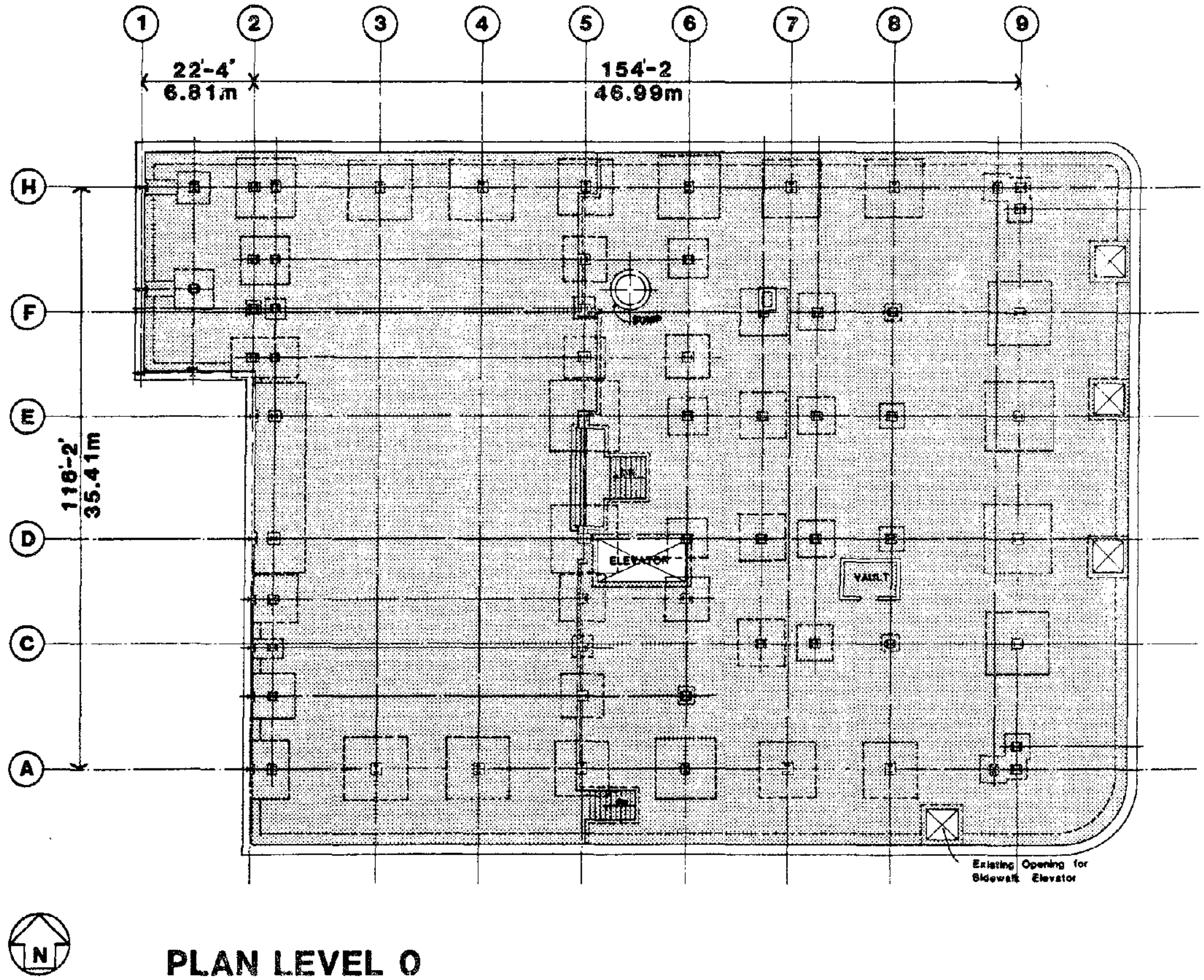


FIGURE 11 PLAN OF BASEMENT SHOWING COLUMN LOCATIONS

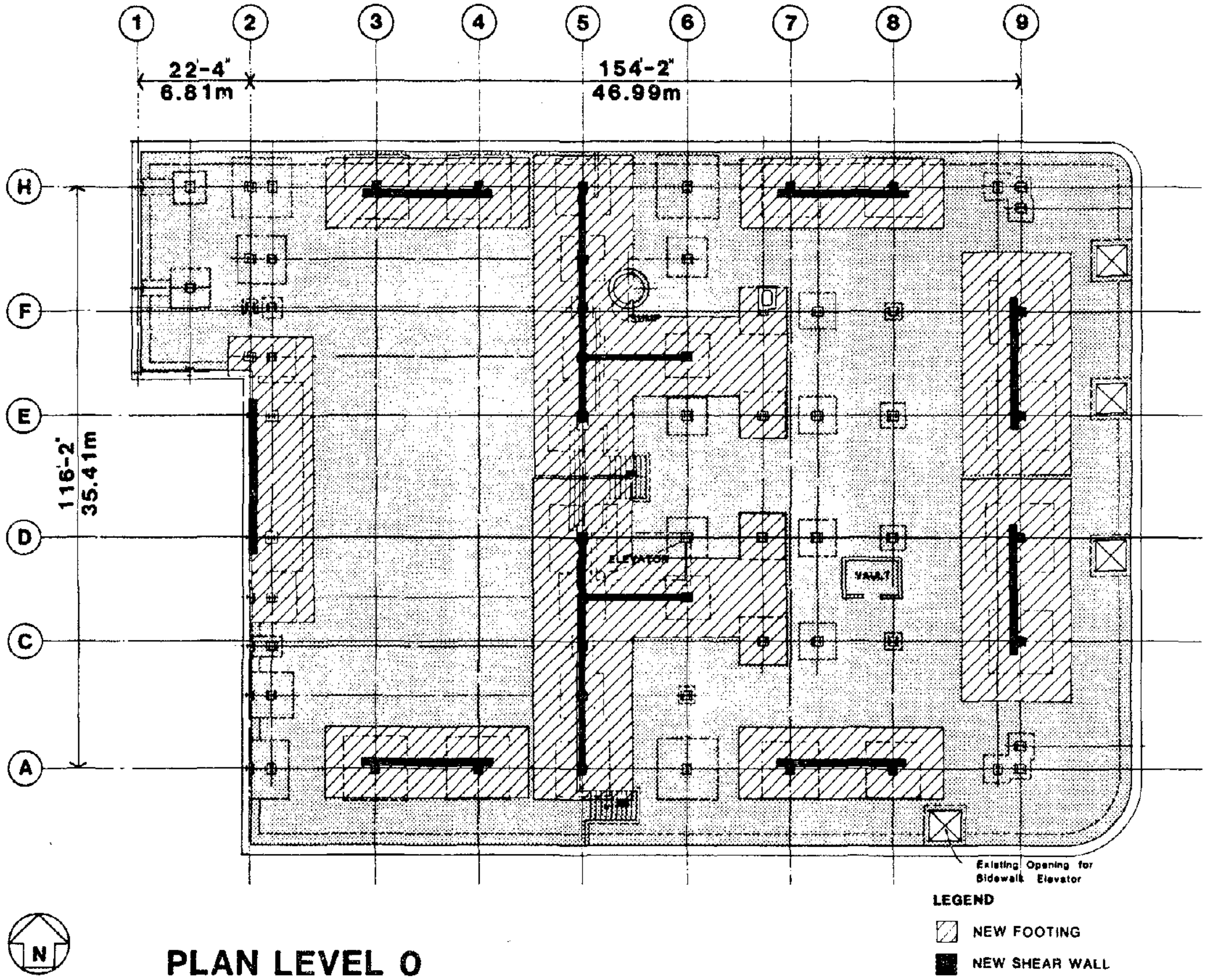


FIGURE 12 PLAN OF BASEMENT SHOWING NEW SHEAR WALL FOR CONVENTIONAL SCHEME SUPERPOSED

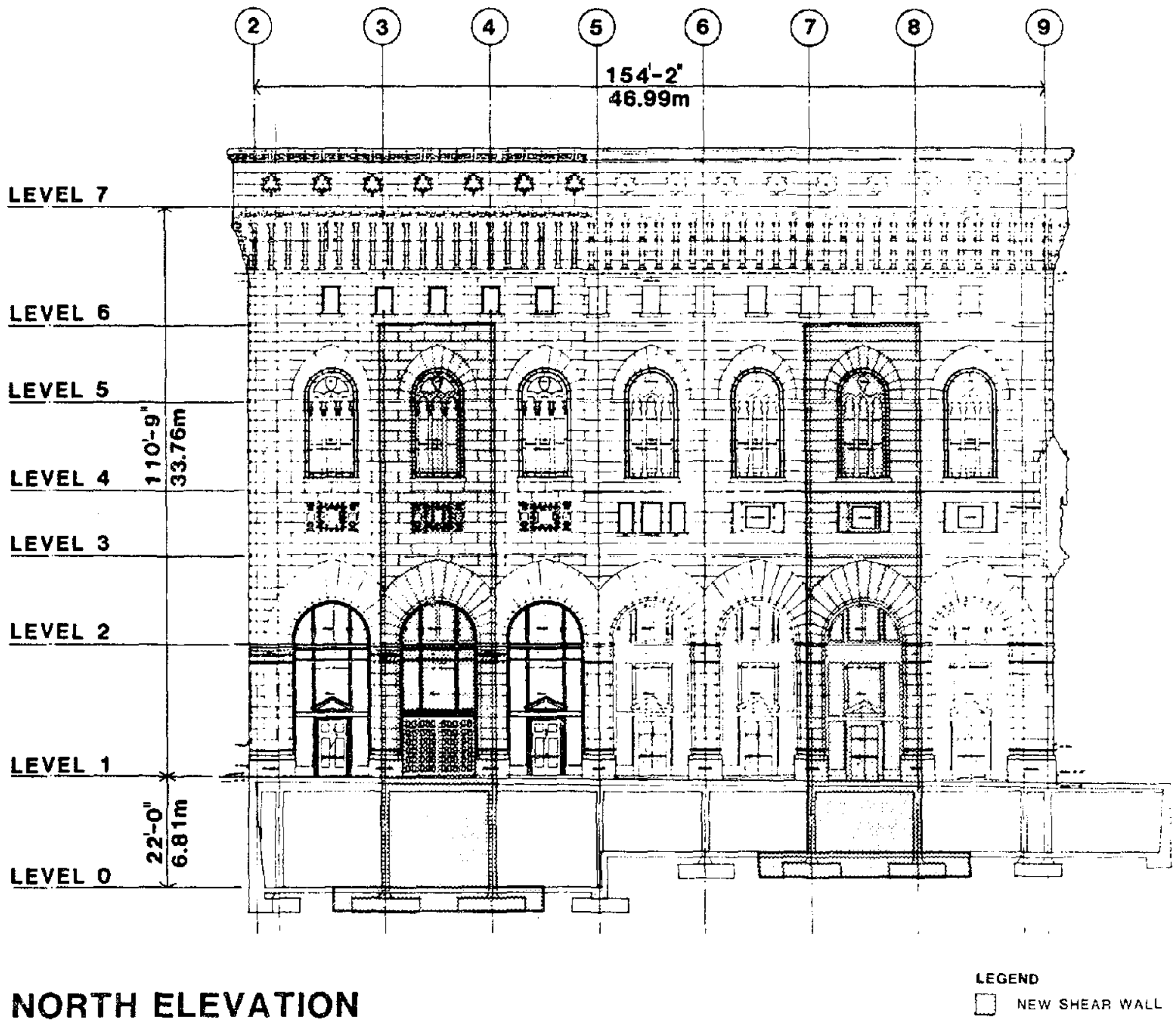


FIGURE 13 NORTH ELEVATION WITH NEW SHEAR WALLS FOR CONVENTIONAL SCHEME SUPERPOSED

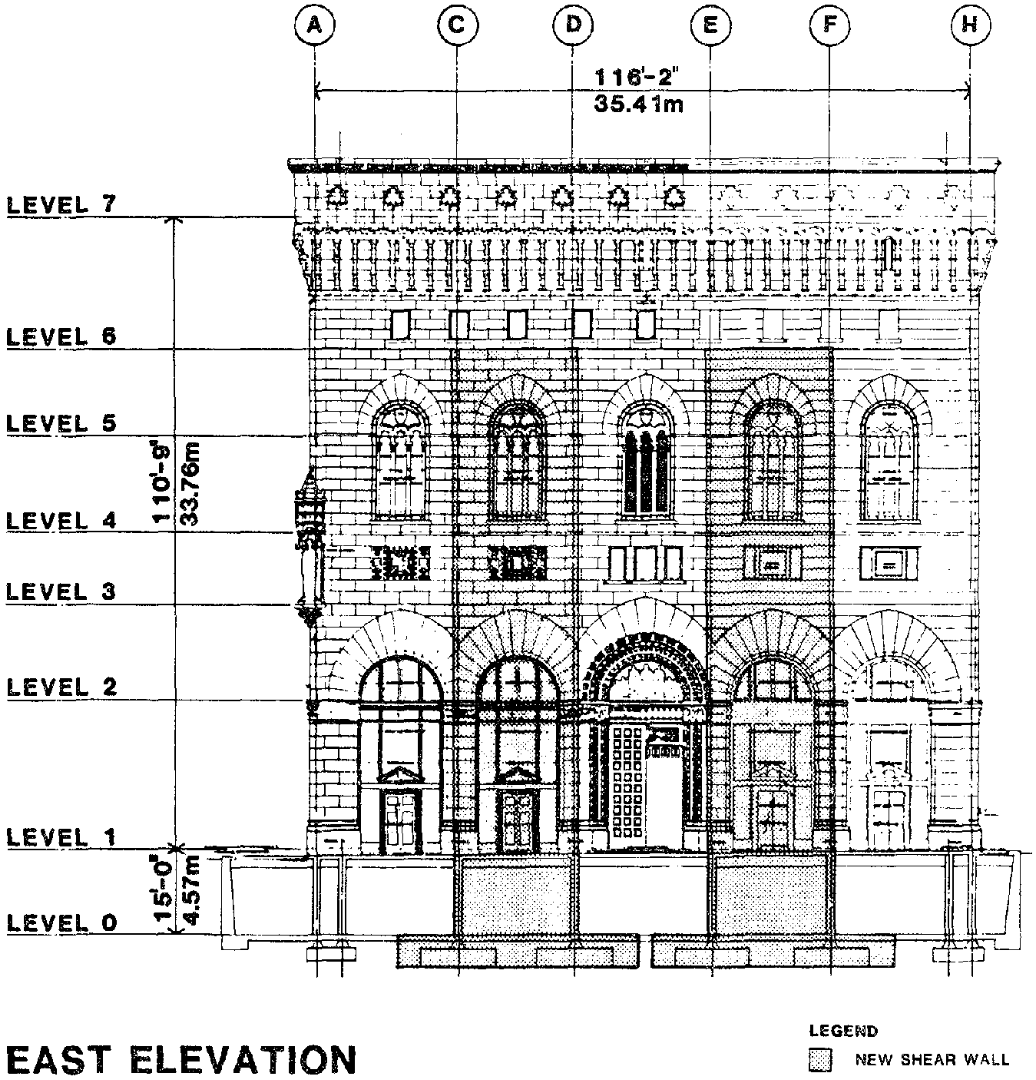


FIGURE 14 EAST ELEVATION WITH NEW SHEAR WALLS FOR CONVENTIONAL SCHEME SUPERPOSED

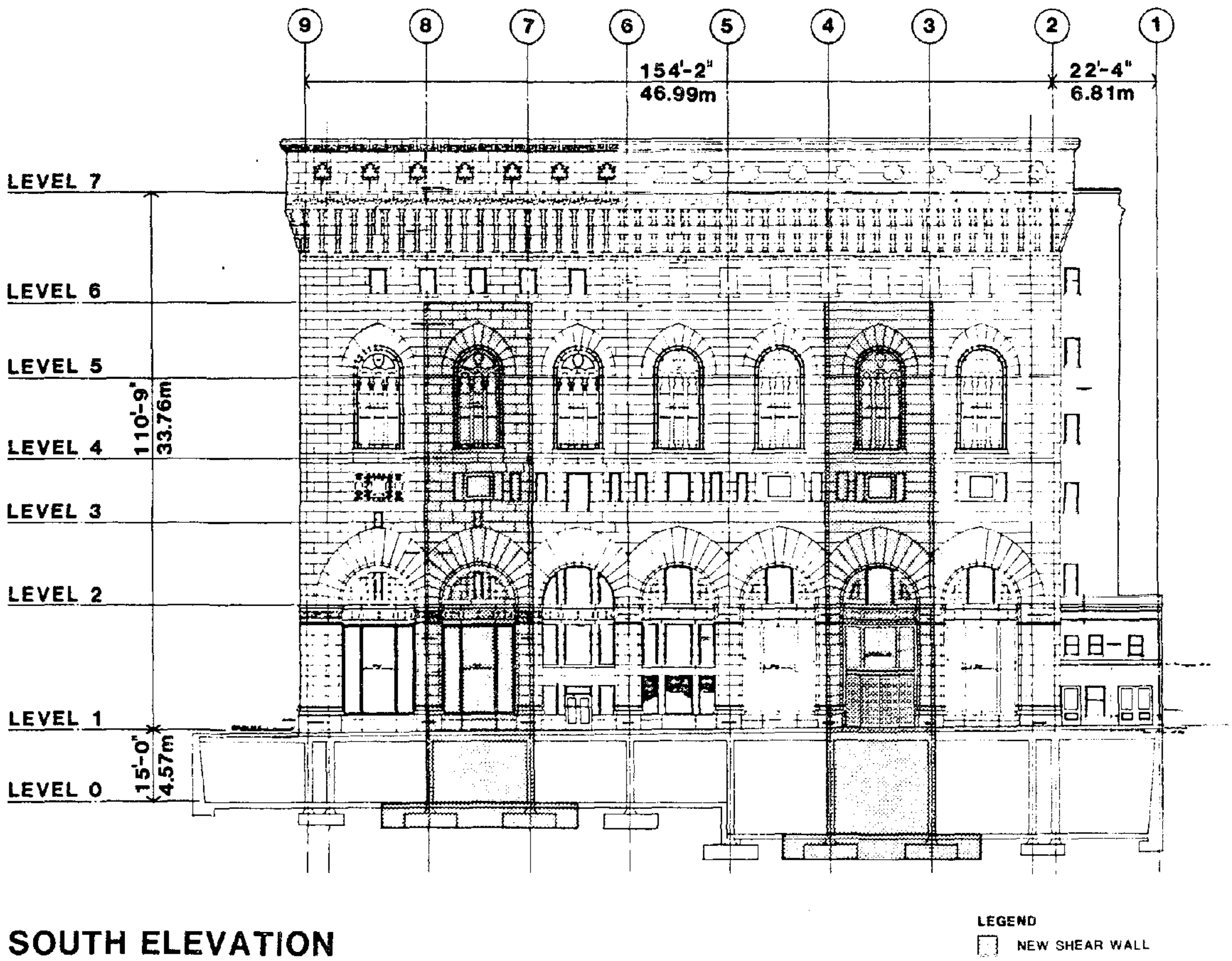


FIGURE 15 SOUTH ELEVATION WITH NEW SHEAR WALLS FOR CONVENTIONAL SCHEME SUPERPOSED

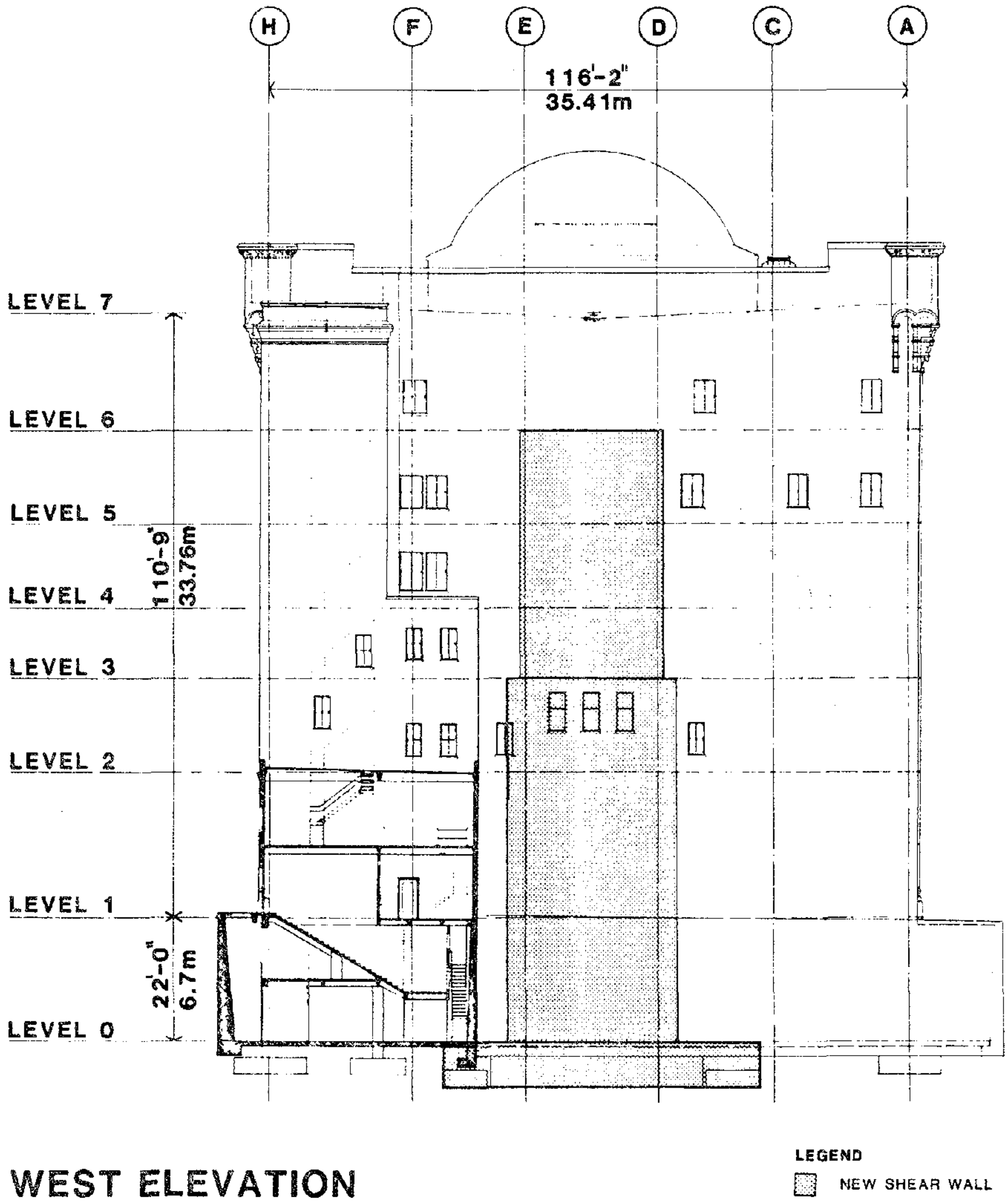


FIGURE 16 WEST ELEVATION WITH NEW SHEAR WALLS FOR CONVENTIONAL SCHEME SUPERPOSED

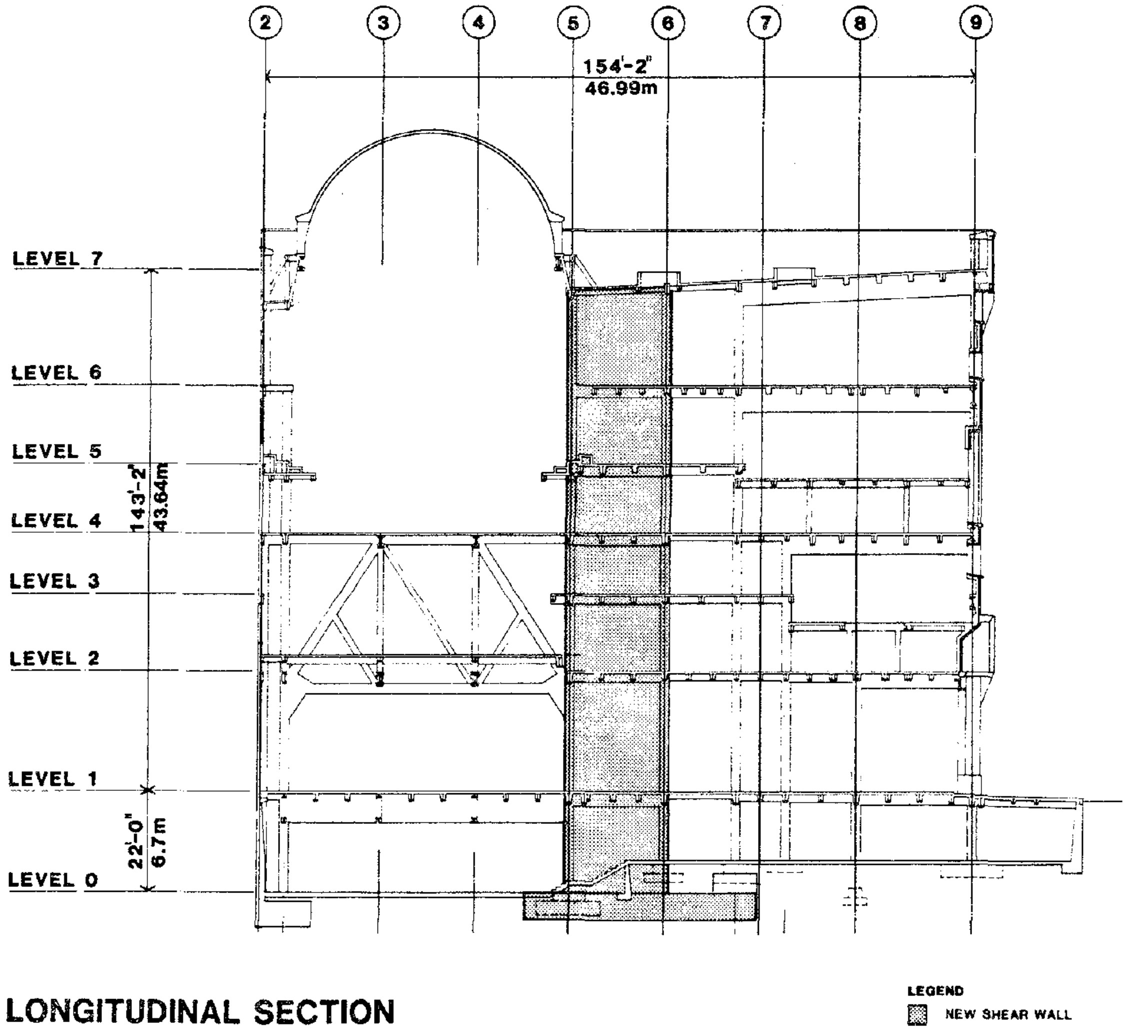


FIGURE 17 LONGITUDINAL SECTION WITH NEW SHEAR WALLS FOR CONVENTIONAL SCHEME SUPERPOSED

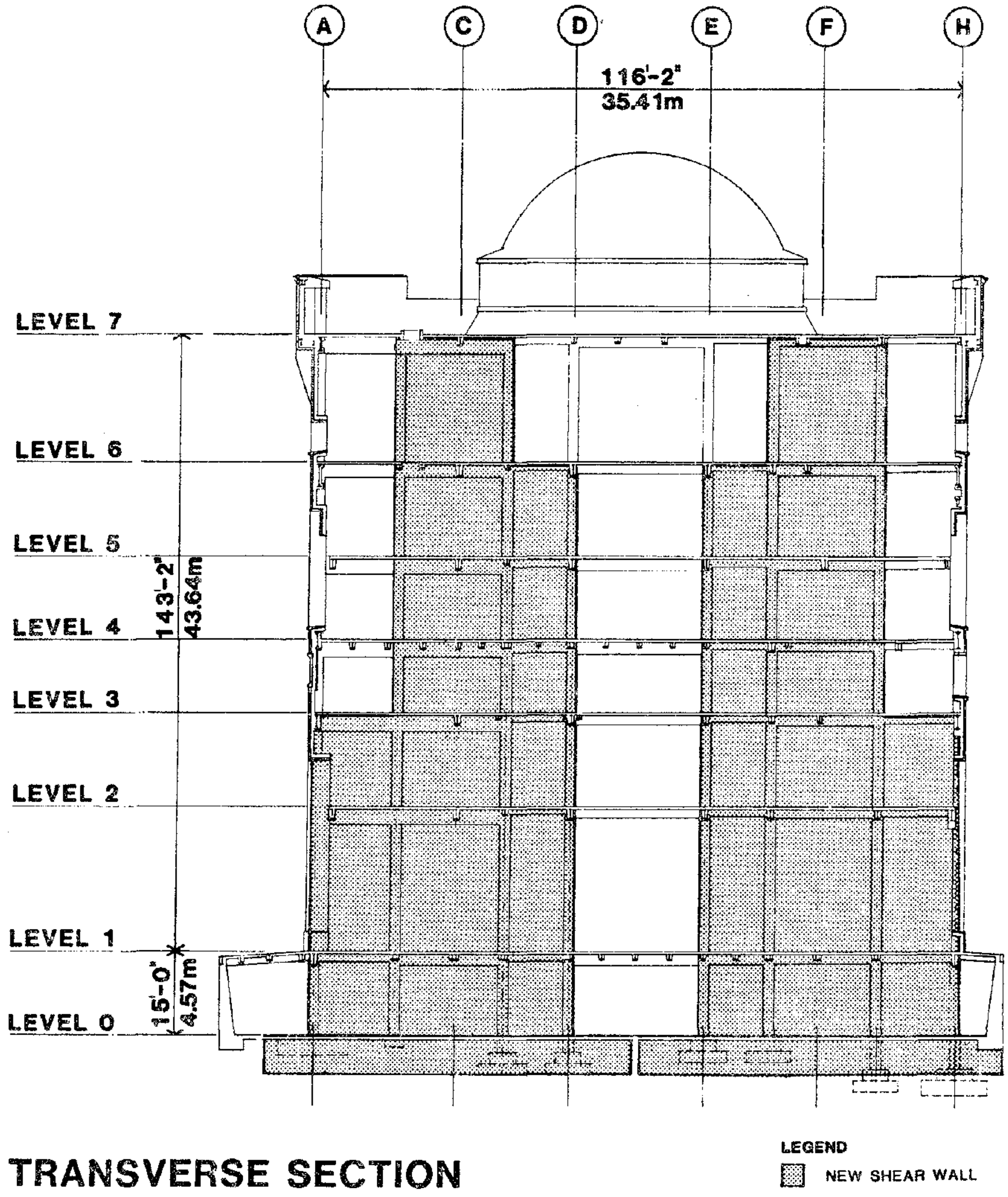


FIGURE 18 TRANSVERSE SECTION WITH NEW SHEAR WALLS FOR CONVENTIONAL SCHEME SUPERPOSED

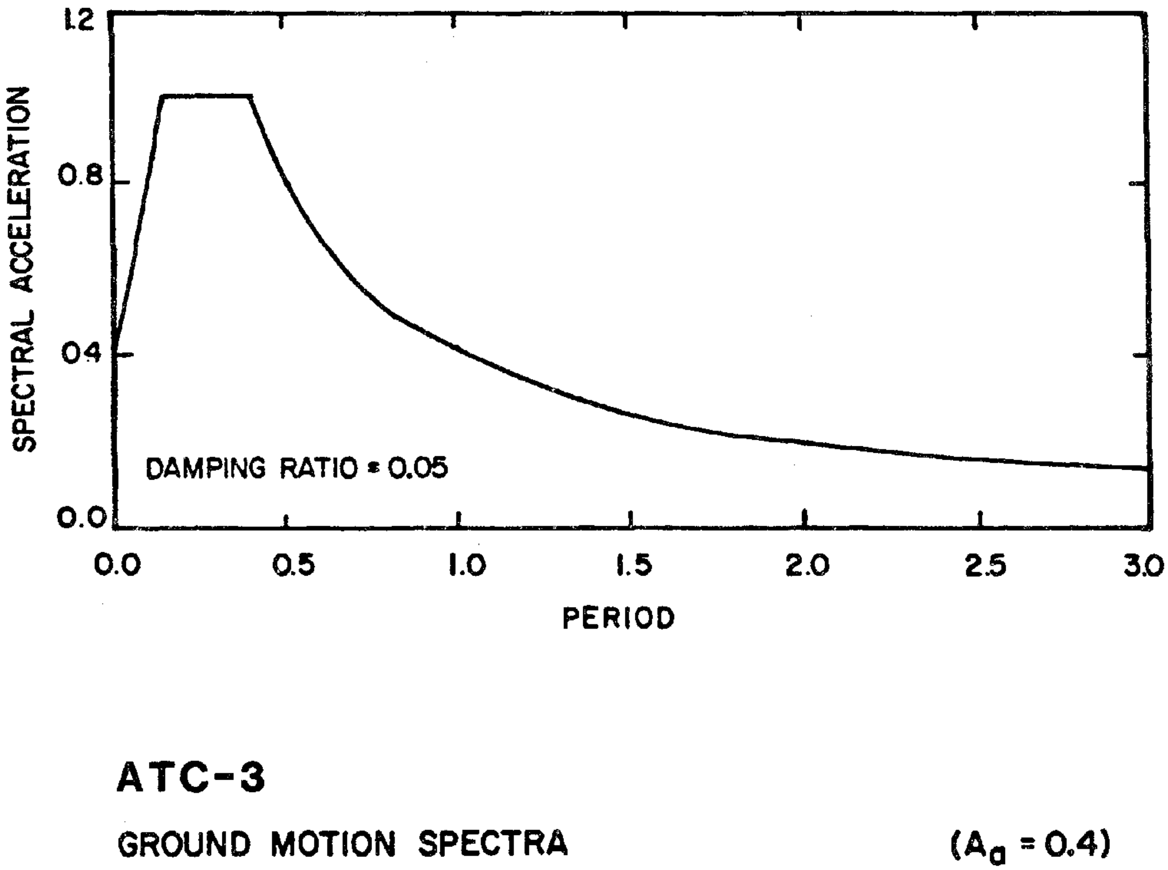


FIGURE 19 ATC-3 GROUND MOTION SPECTRUM USED FOR DYNAMIC ANALYSES

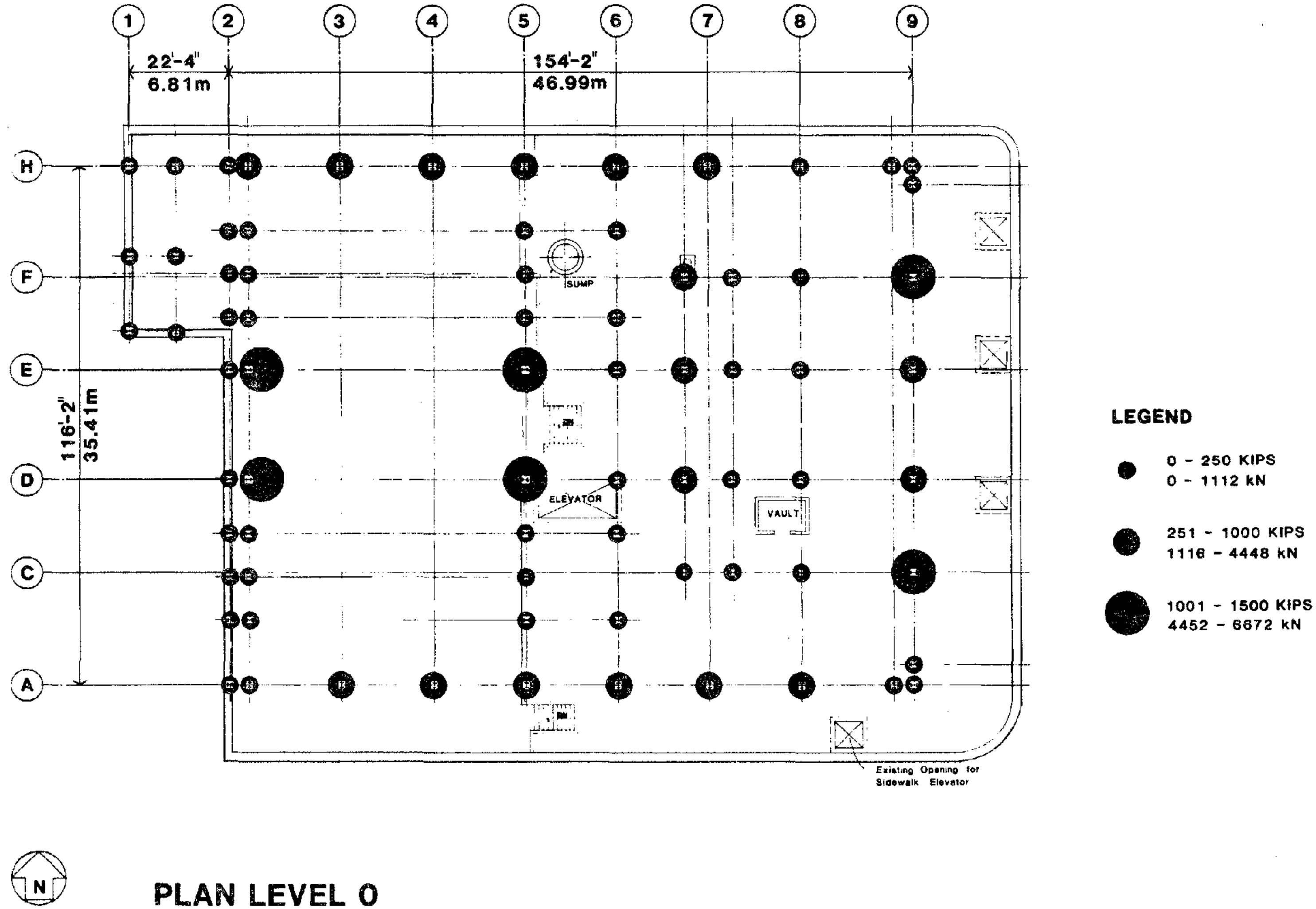
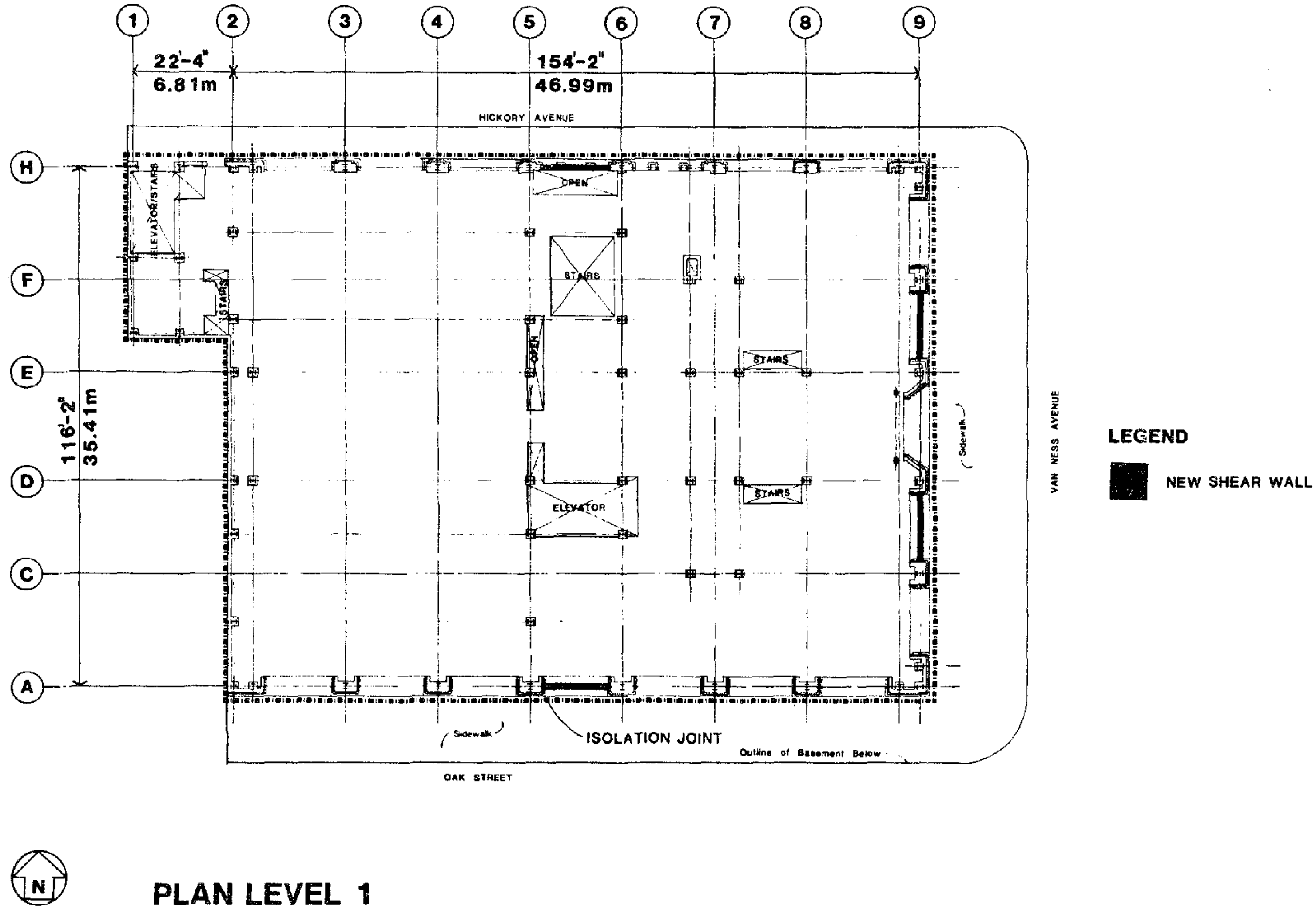


FIGURE 20 BASEMENT PLAN VIEW INDICATING COLUMN LOADS



PLAN LEVEL 1

FIGURE 21 ISOLATION JOINT AND NEW SHEAR WALLS FOR BASE ISOLATION SCHEME

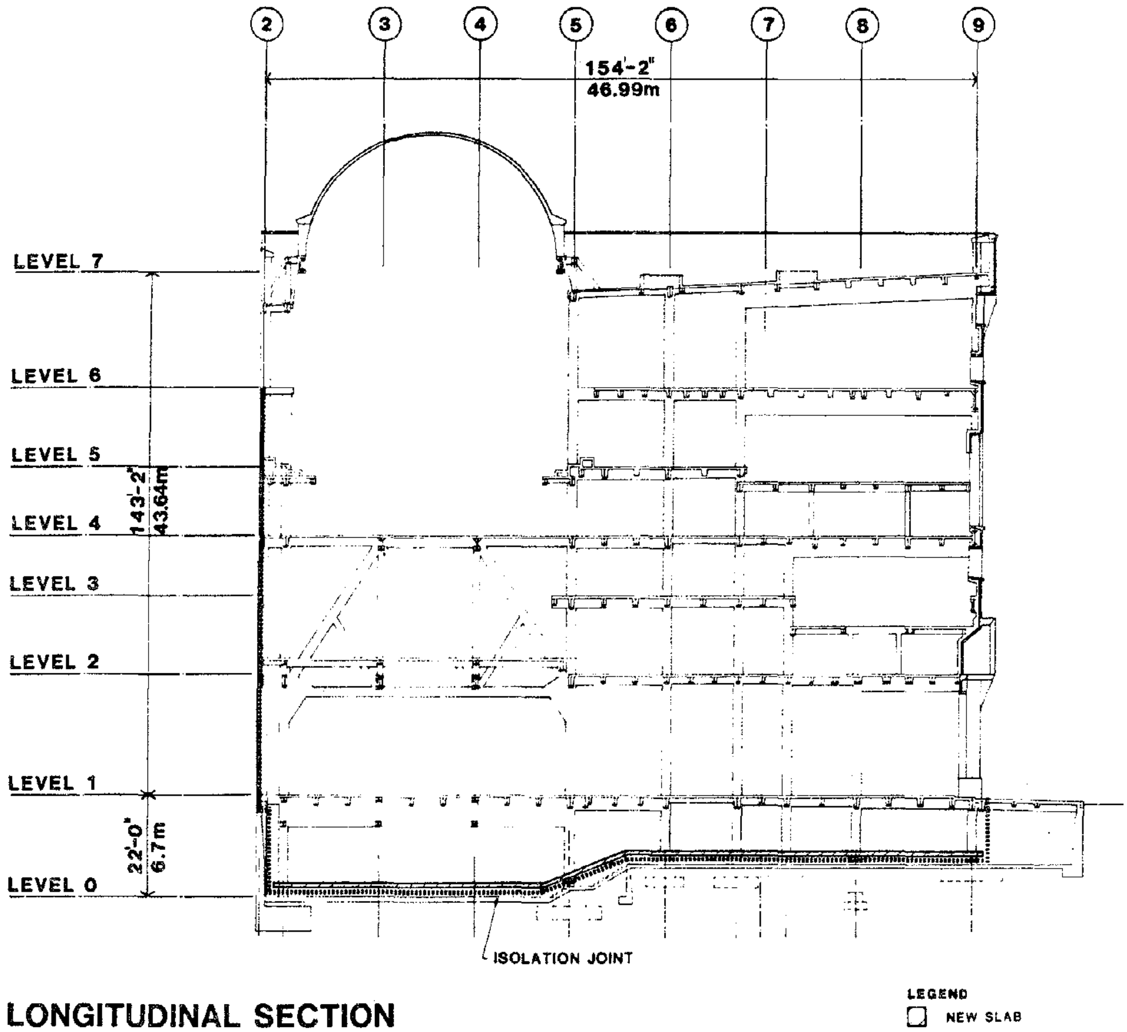


FIGURE 22 LONGITUDINAL SECTION SHOWING ISOLATION JOINT AND NEW SHEAR WALLS FOR BASE ISOLATION SCHEME

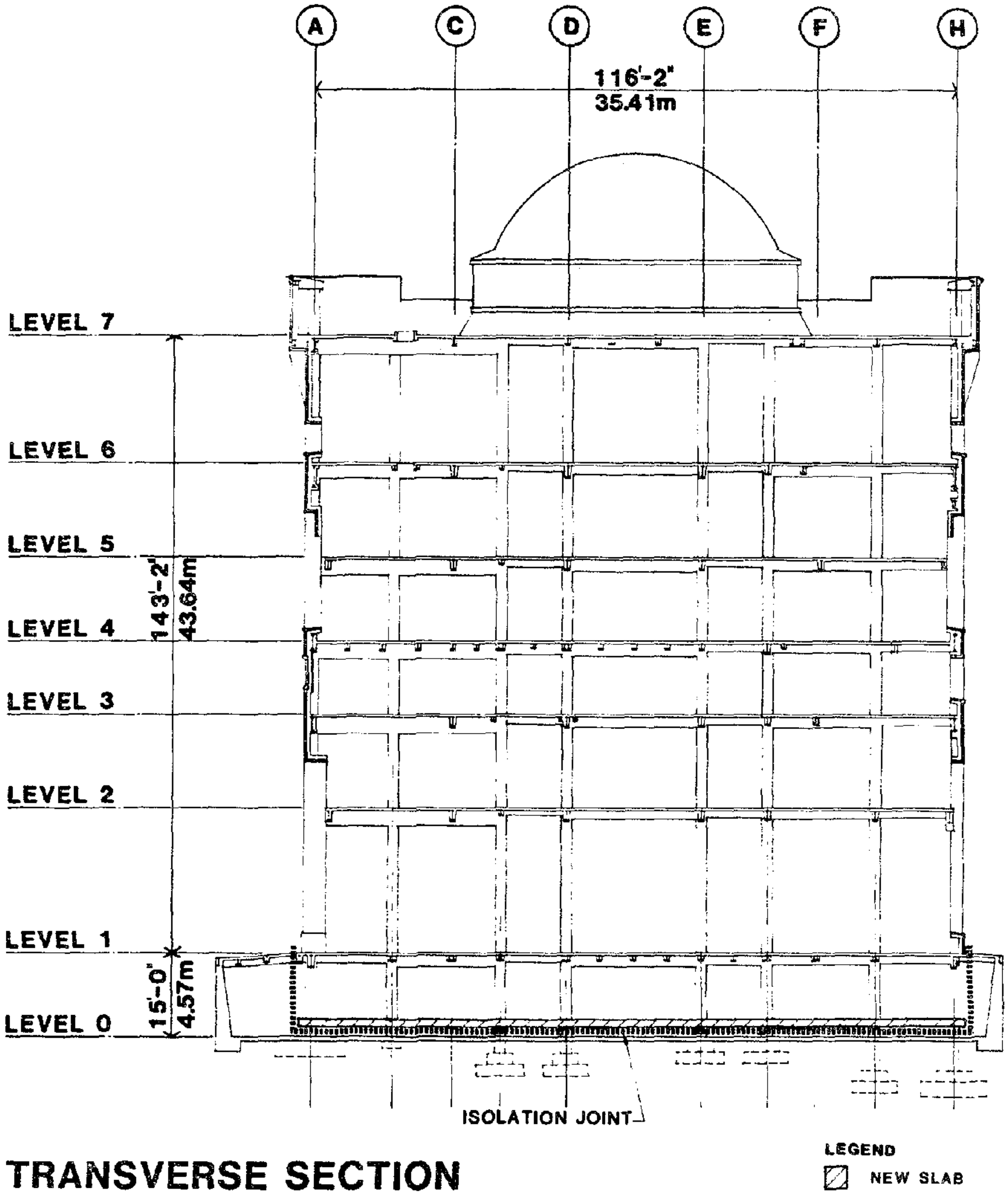


FIGURE 23 TRANSVERSE SECTION SHOWING ISOLATION JOINT AND NEW SHEAR WALLS FOR BASE ISOLATION SCHEME

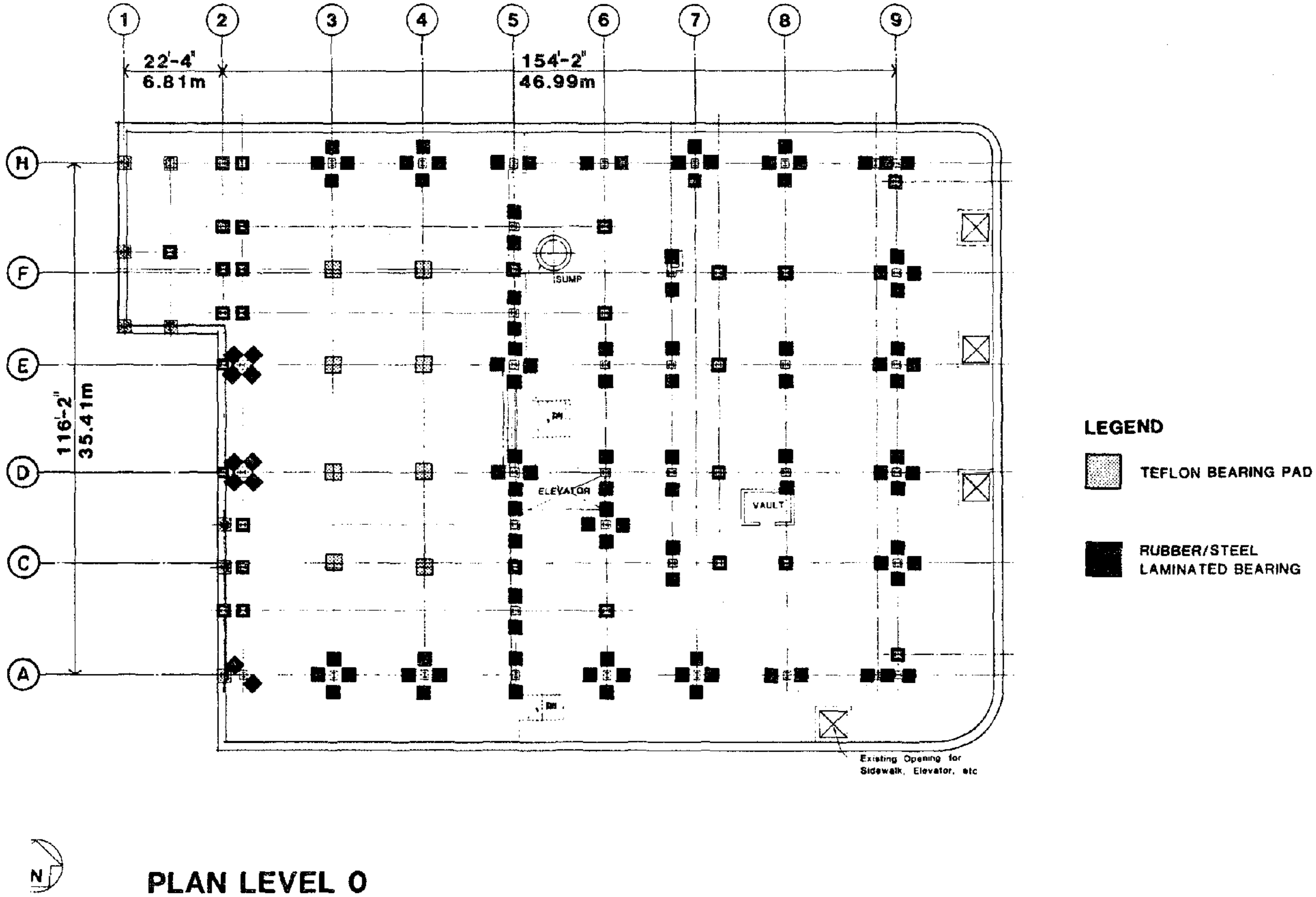


FIGURE 24 BASEMENT PLAN SHOWING LAYOUT OF ISOLATION BEARINGS

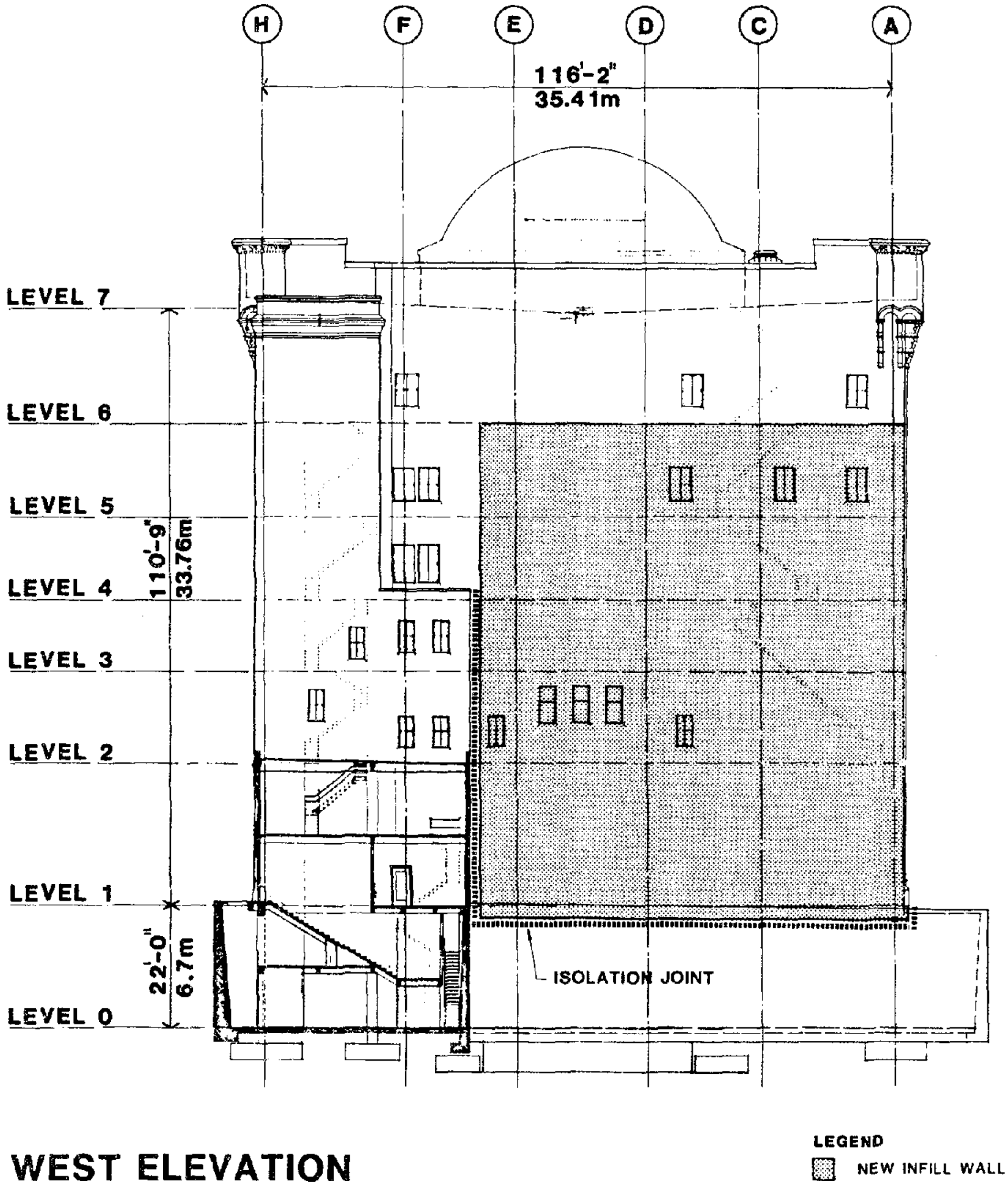
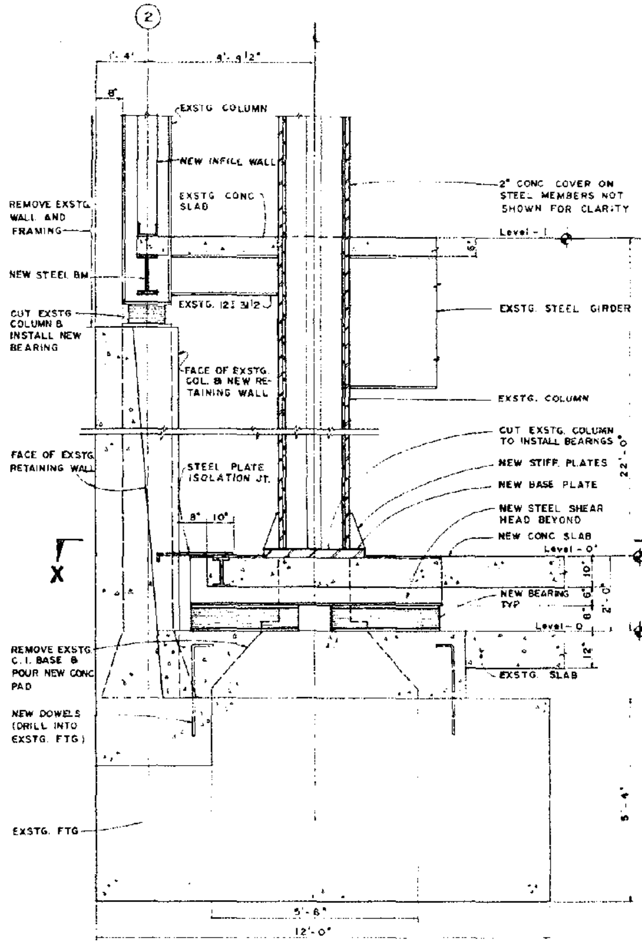
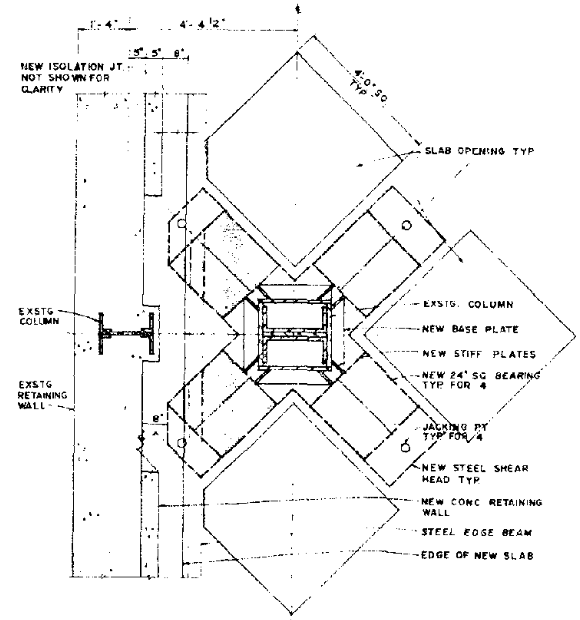


FIGURE 25 WEST ELEVATION SHOWING NEW CURTAIN WALL AND JOINT

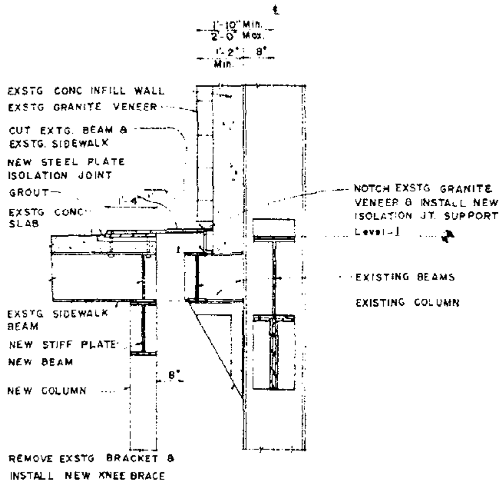


COLUMN BASE DETAIL AT RETAINING WALL

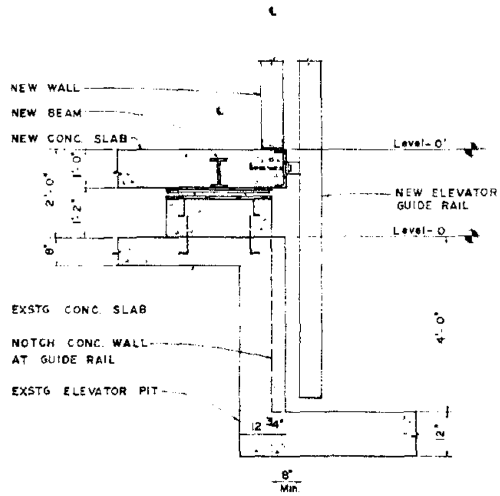


PLAN X-X

FIGURE 26 COLUMN BASE DETAIL AT RETAINING WALL



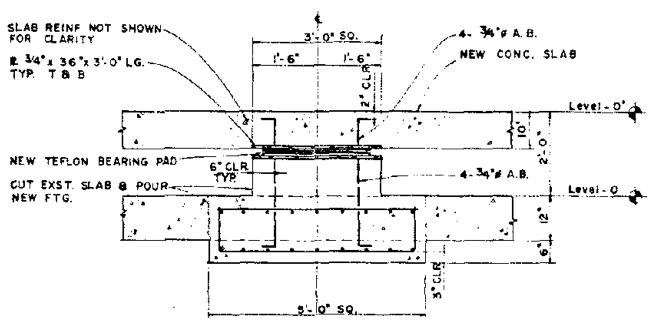
ISOLATION JOINT
AT BLDG. PERIMETER



ISOLATION JOINT
AT ELEVATOR

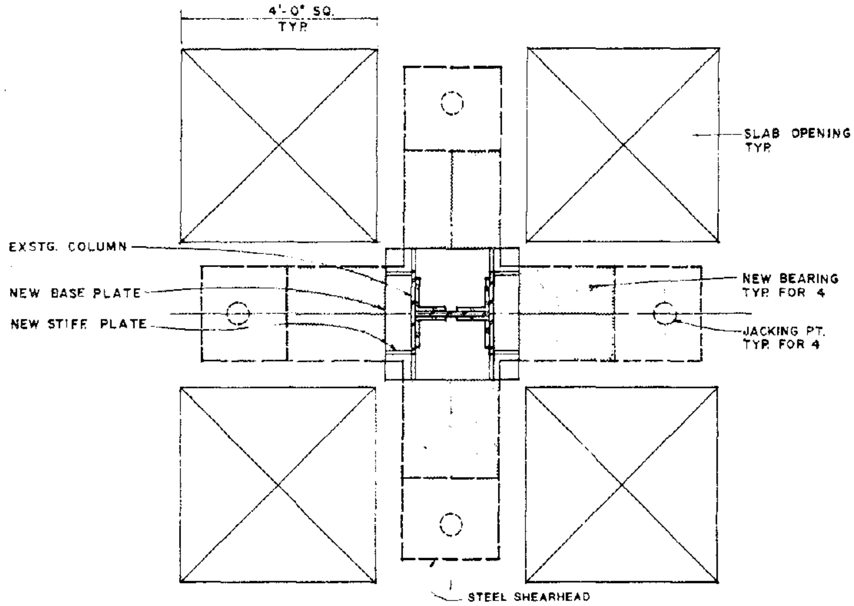
FIGURE 27 ISOLATION JOINT AT BUILDING PERIMETER

FIGURE 28 ISOLATION JOINT AT ELEVATOR

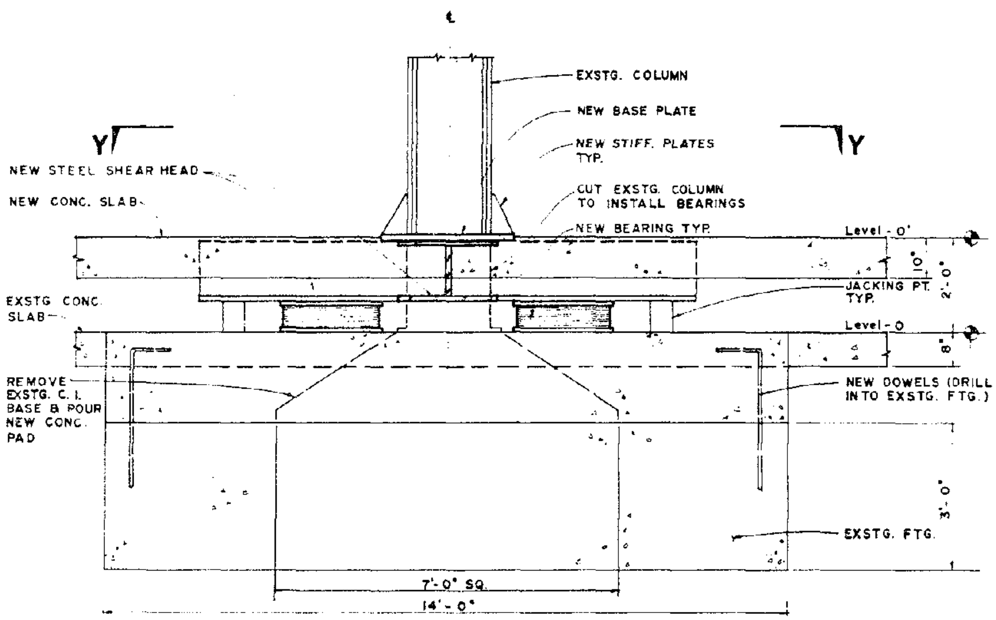


TEFLON BEARING PAD DETAIL

FIGURE 29 TEFLON BEARING PAD DETAIL



PLAN Y-Y



TYPICAL COLUMN BASE DETAIL

FIGURE 30 TYPICAL COLUMN BASE DETAIL

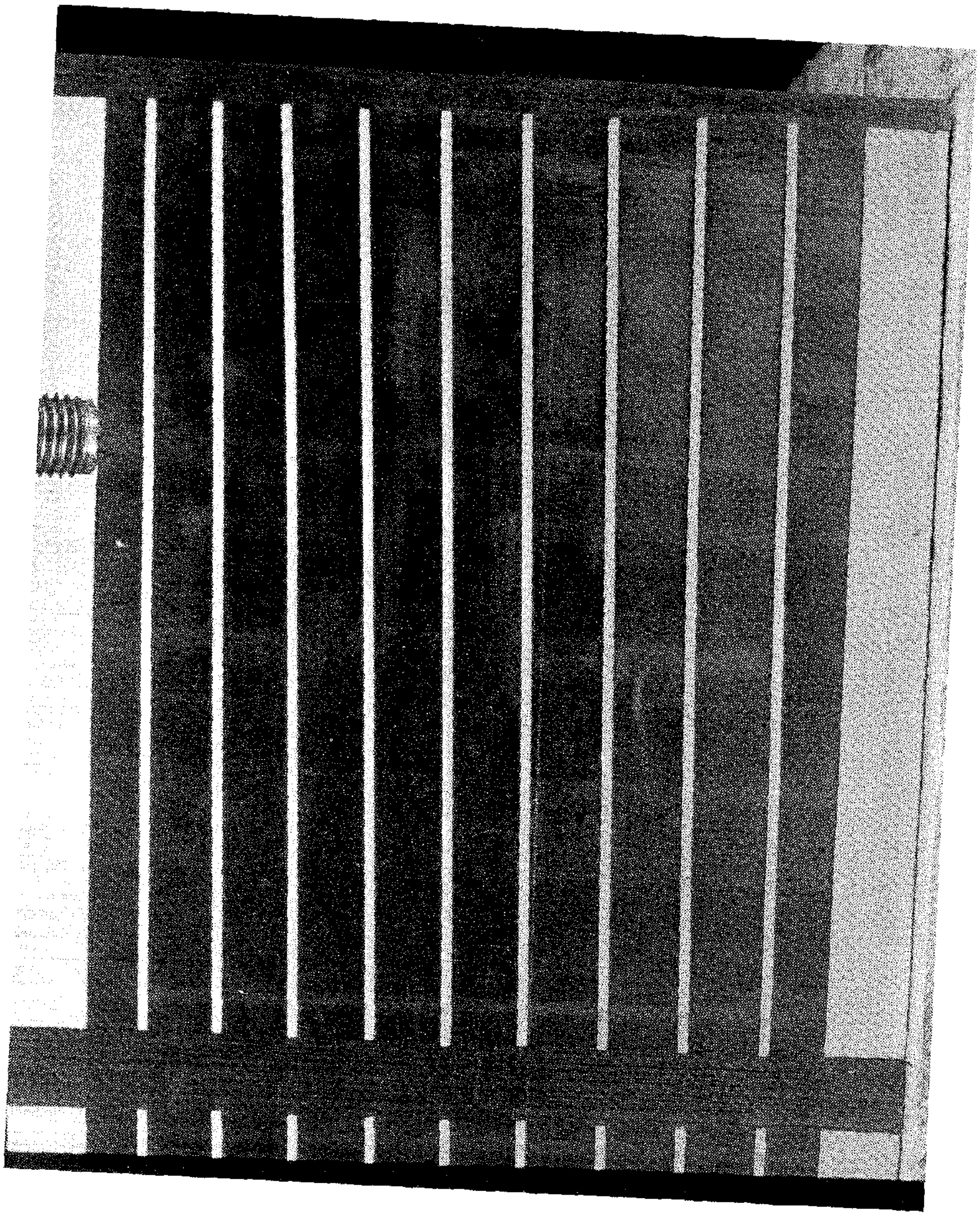


FIGURE 31 CROSS-SECTION OF 150-TON BEARING SAWN THROUGH

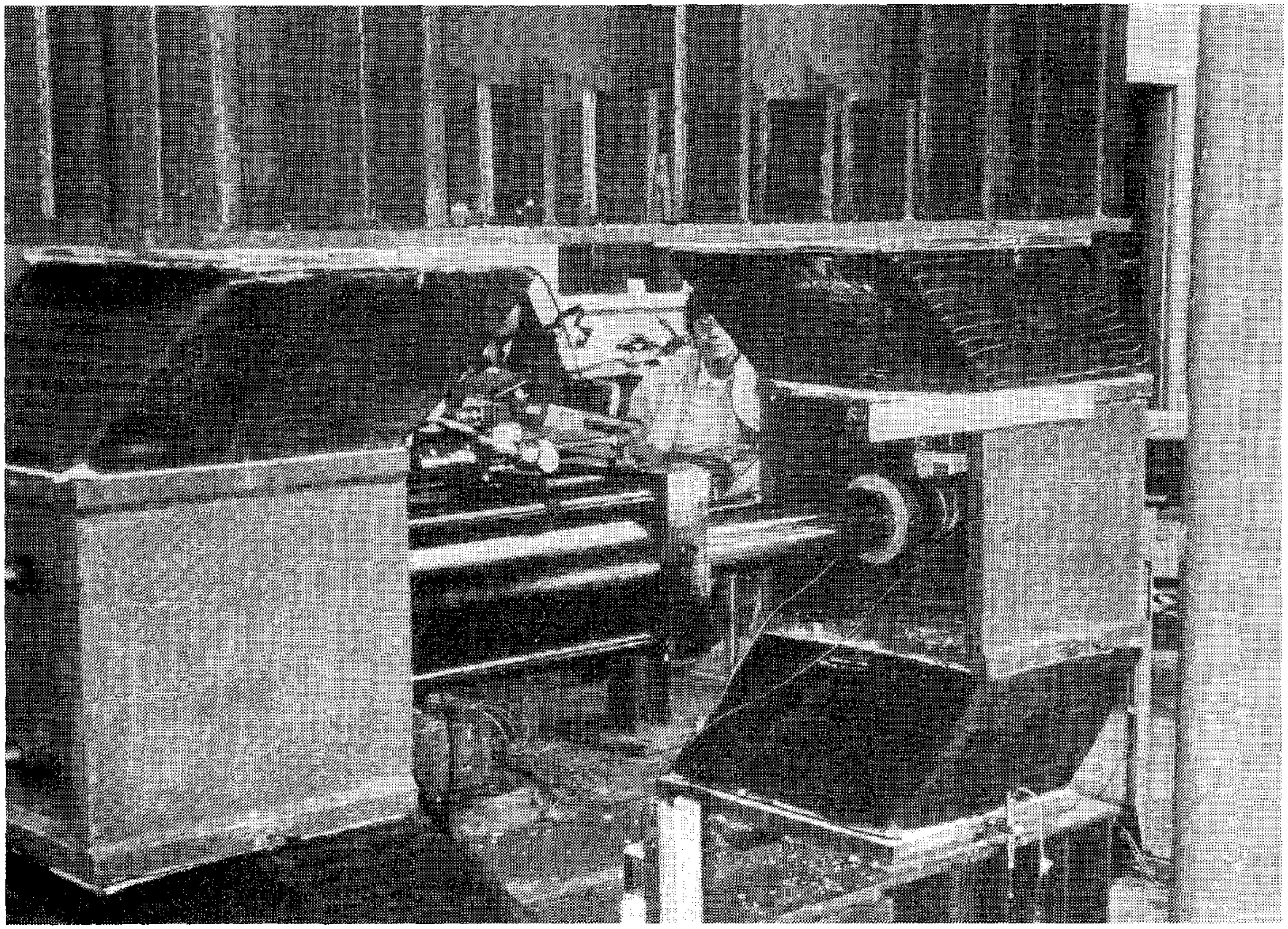


FIGURE 32 TEST RIG FOR SIMULTANEOUS HORIZONTAL AND VERTICAL LOADING OF PROTOTYPE BEARINGS—150-TON BEARINGS UNDER TEST AT A HORIZONTAL DISPLACEMENT OF 10 INCHES

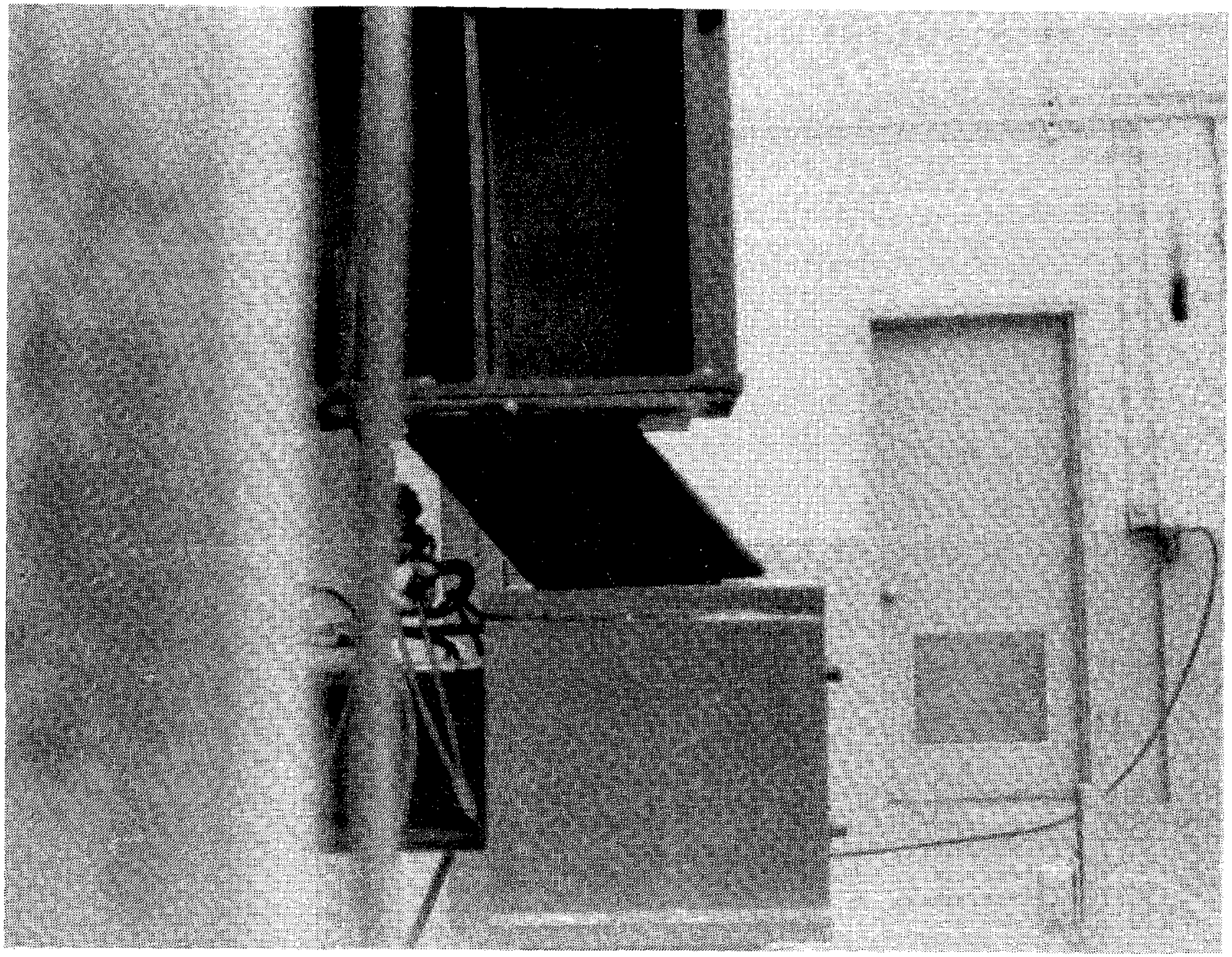
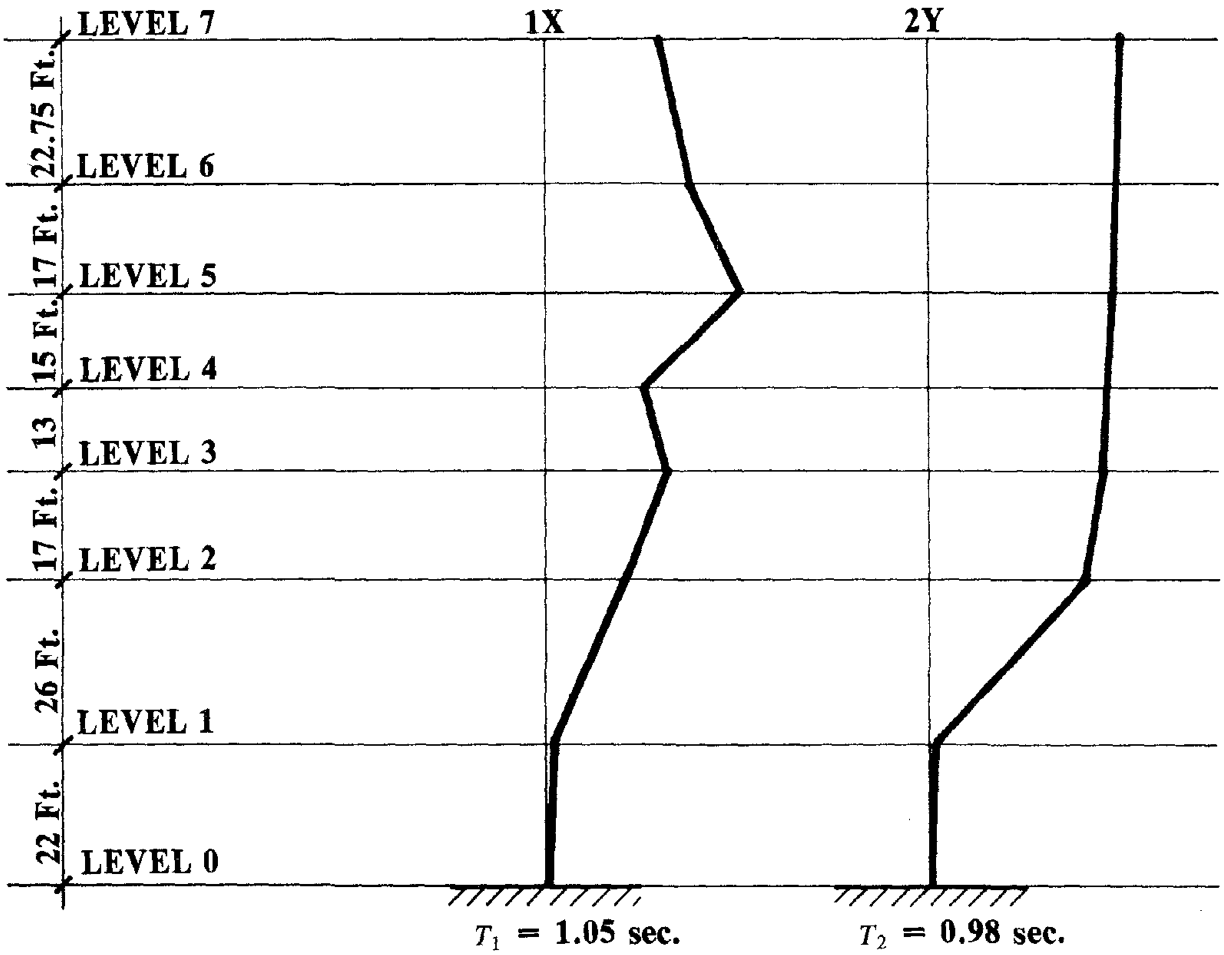


FIGURE 33 50-TON BEARINGS AT A HORIZONTAL DISPLACEMENT OF 9 INCHES—SHEAR KEY VISIBLE AT LIFT-OFF OF THE LEADING EDGE

MODE SHAPES AT CENTER OF MASS

T_1 = **1.05 seconds in X direction**
T_2 = **0.98 seconds in Y direction**

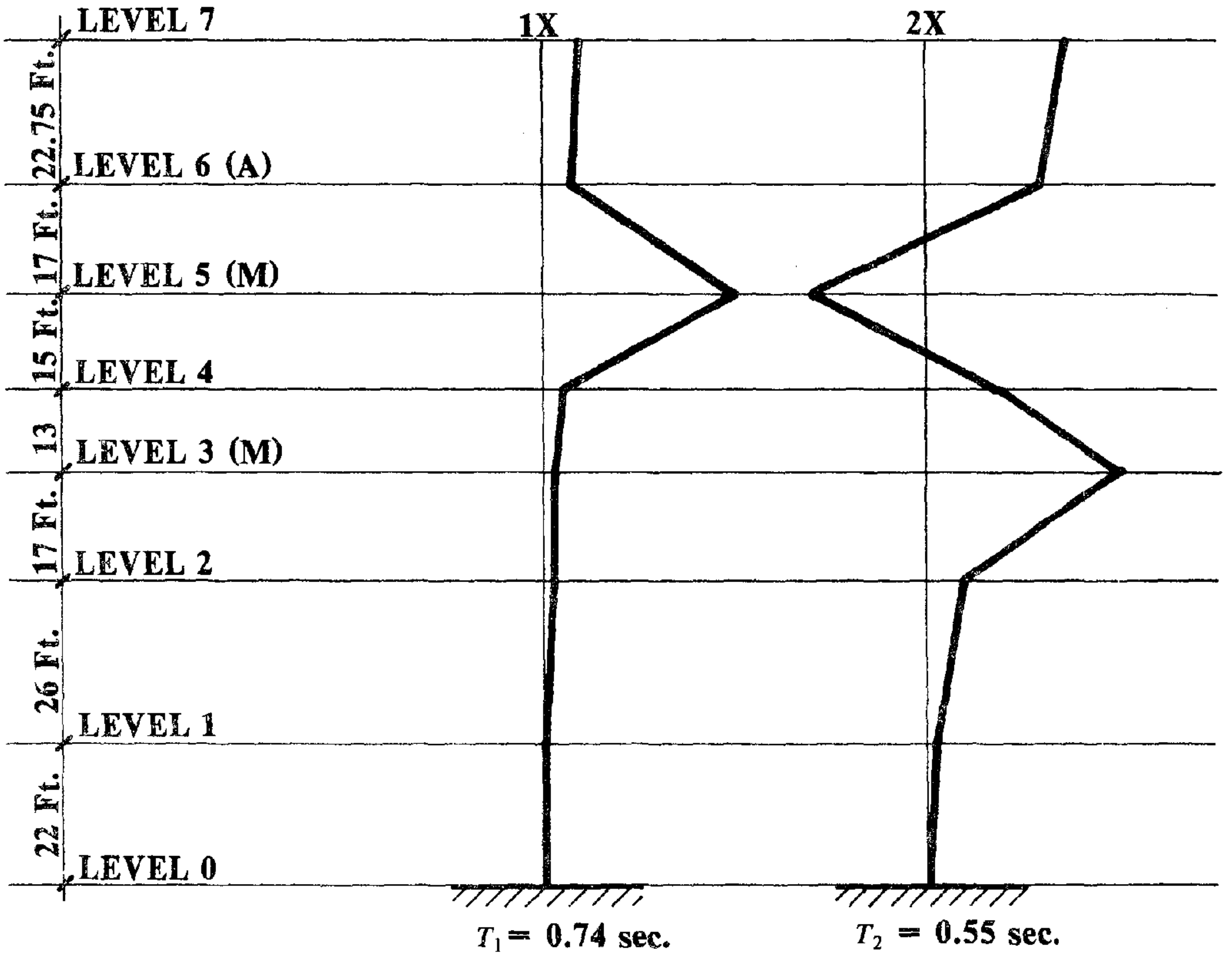


f'_c = **2,380 psi**

FIGURE 34 MODE SHAPES FOR THE MASONIC HALL

MODE SHAPES AT CENTER OF MASS

T_1 = 0.74 seconds in X direction
T_2 = 0.54 seconds in X direction
T_3 = 0.53 seconds in Y direction



12 in. INFILL WALLS, f'_c = 4,000 psi

FIGURE 35 MODE SHAPES FOR THE MASONIC HALL WITH INFILL SHEAR WALLS

MODE SHAPES AT CENTER OF MASS

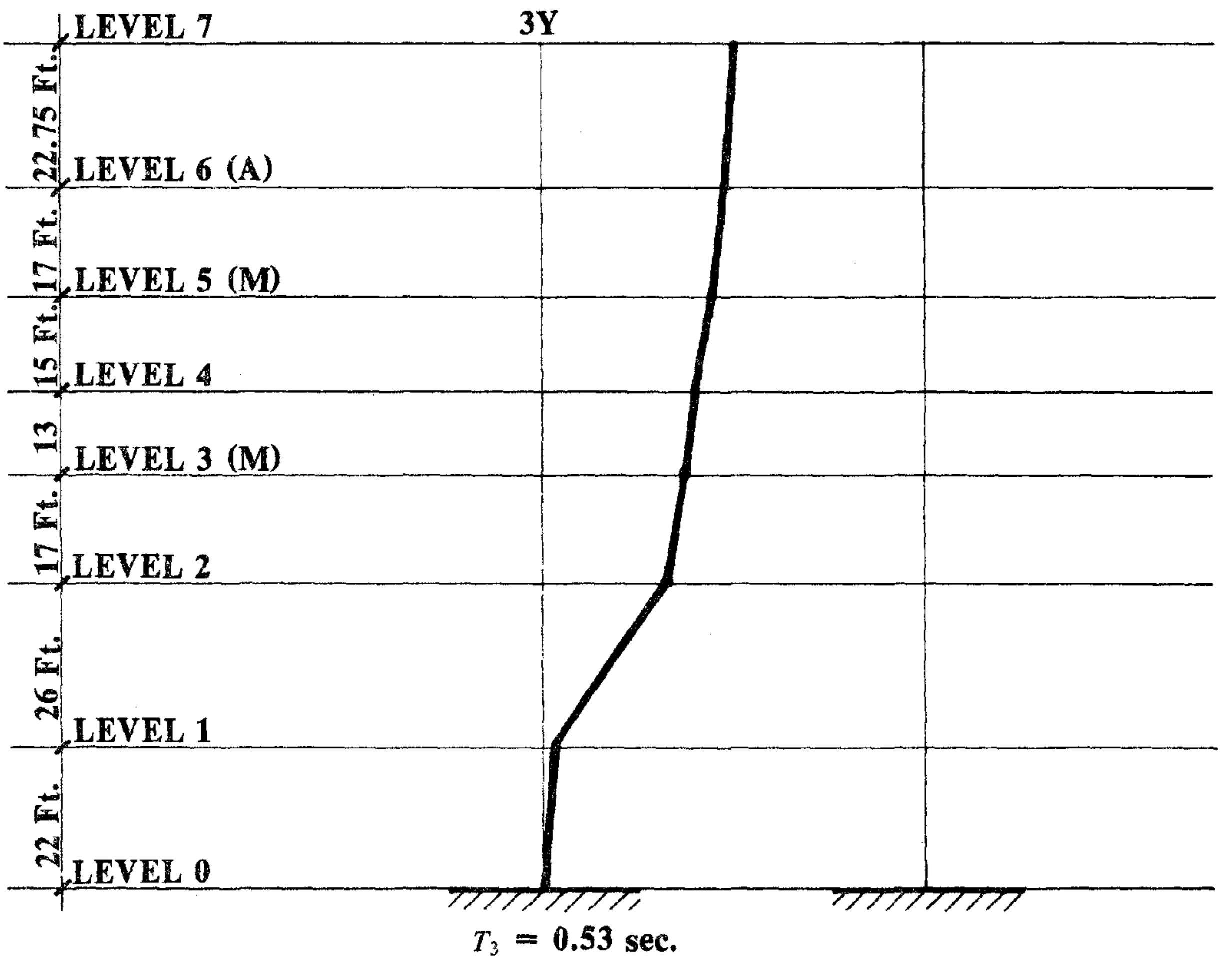


FIGURE 35 MODE SHAPES FOR THE MASONIC HALL WITH INFILL SHEAR WALLS (cont'd)

MODE SHAPES AT CENTER OF MASS

T_1 = 0.49 seconds in X direction
T_2 = 0.43 seconds in Y direction

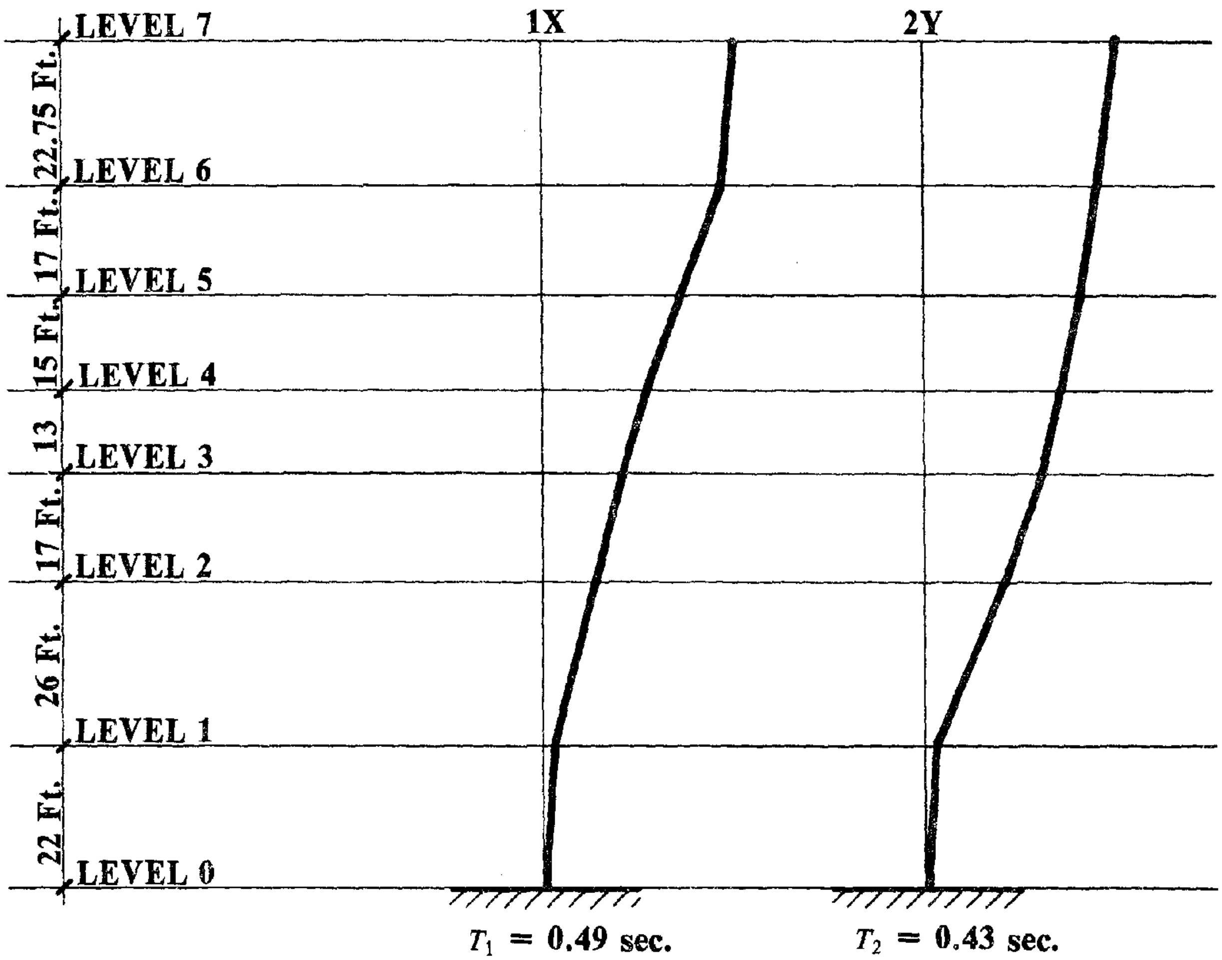


FIGURE 36 MODE SHAPES FOR THE MASONIC HALL USING CONVENTIONAL REHABILITATION PROCEDURES

MODE SHAPES AT CENTER OF MASS

T_1 = 2.12 seconds in X direction
T_2 = 2.11 seconds in Y direction

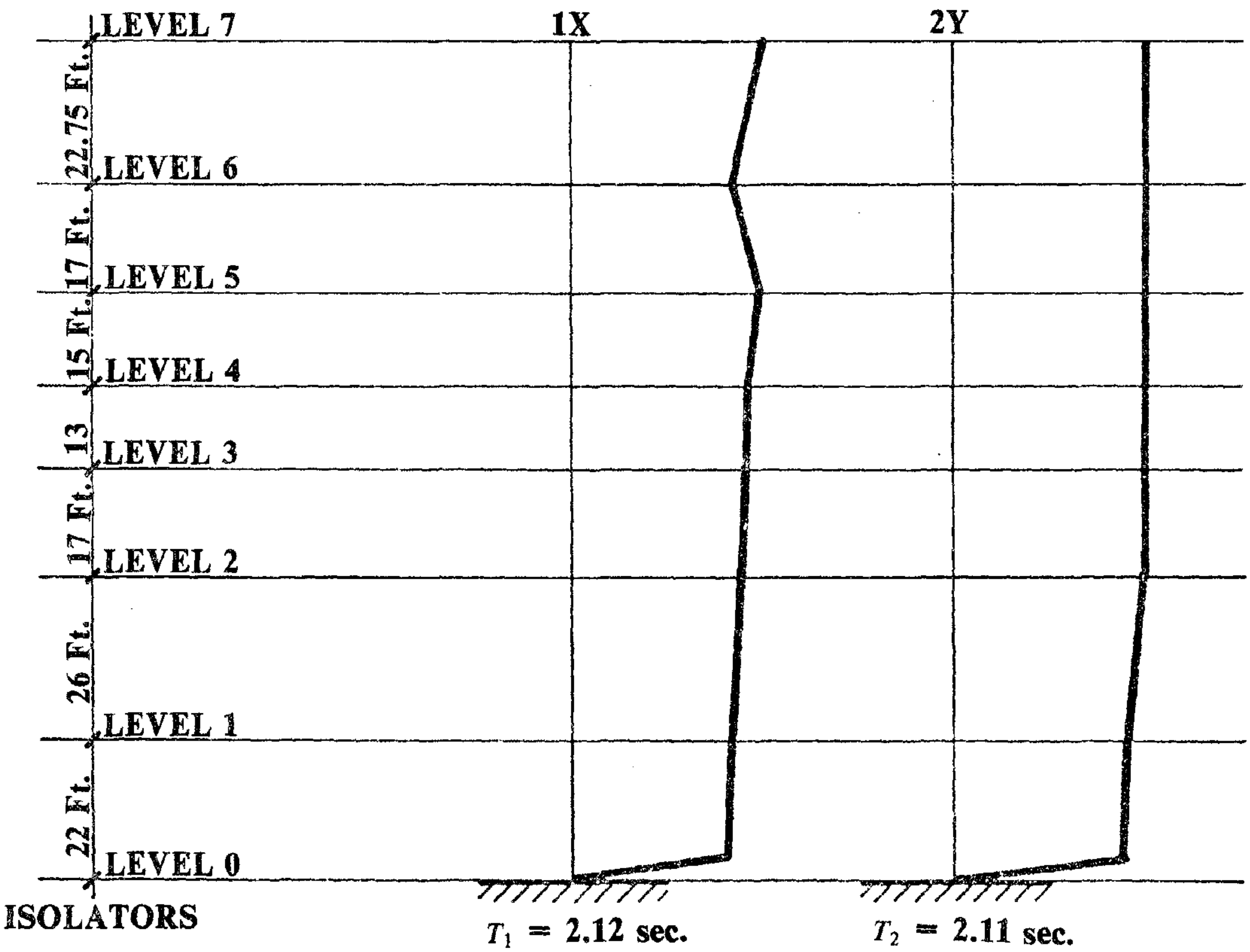


FIGURE 37 MODE SHAPES FOR THE MASONIC HALL USING BASE ISOLATION

BASED ON ATC-3 SPECTRUM

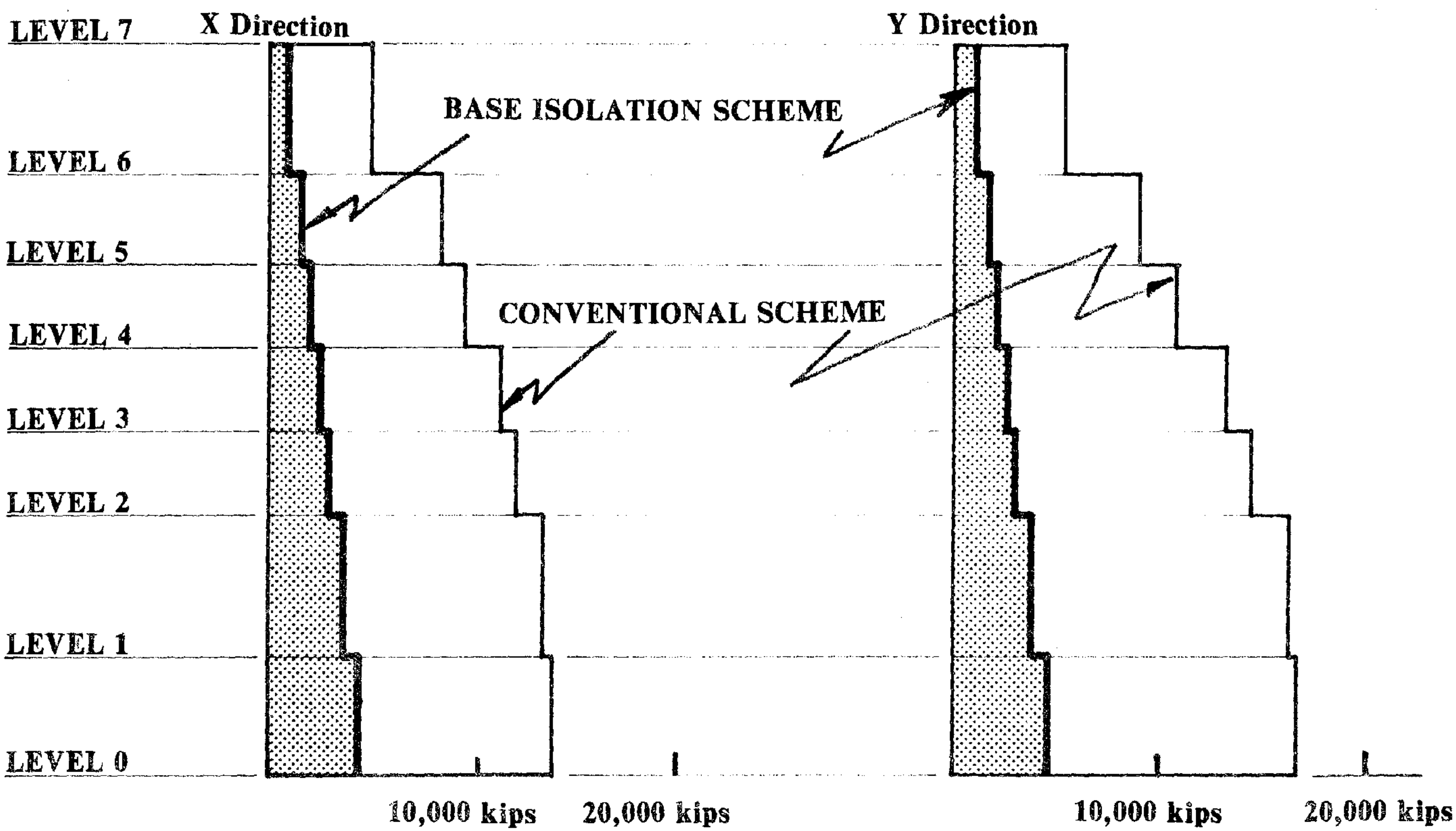


FIGURE 38 STORY SHEARS FOR THE MASONIC HALL

EARTHQUAKE ENGINEERING RESEARCH CENTER REPORTS

NOTE: Numbers in parentheses are Accession Numbers assigned by the National Technical Information Service; these are followed by a price code. Copies of the reports may be ordered from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia, 22161. Accession Numbers should be quoted on orders for reports (PB --- ---) and remittance must accompany each order. Reports without this information were not available at time of printing. The complete list of EERC reports (from EERC 67-1) is available upon request from the Earthquake Engineering Research Center, University of California, Berkeley, 47th Street and Hoffman Boulevard, Richmond, California 94804.

UCB/EERC-77/01 "PLUSH - A Computer Program for Probabilistic Finite Element Analysis of Seismic Soil-Structure Interaction," by M.P. Romo Organista, J. Lysmer and H.B. Seed - 1977 (PB81 177 651)A05

UCB/EERC-77/02 "Soil-Structure Interaction Effects at the Humboldt Bay Power Plant in the Ferndale Earthquake of June 7, 1975," by J.E. Valera, H.B. Seed, C.F. Tsai and J. Lysmer - 1977 (PB 265 795)A04

UCB/EERC-77/03 "Influence of Sample Disturbance on Sand Response to Cyclic Loading," by K. Mori, H.B. Seed and C.K. Chan - 1977 (PB 267 352)A04

UCB/EERC-77/04 "Seismological Studies of Strong Motion Records," by J. Shoja-Taheri - 1977 (PB 269 655)A10

UCB/EERC-77/05 Unassigned

UCB/EERC-77/06 "Developing Methodologies for Evaluating the Earthquake Safety of Existing Buildings," by No. 1 - B. Bresler; No. 2 - B. Bresler, T. Okada and D. Zisling; No. 3 - T. Okada and B. Bresler; No. 4 - V.V. Bertero and B. Bresler - 1977 (PB 267 354)A08

UCB/EERC-77/07 "A Literature Survey - Transverse Strength of Masonry Walls," by Y. Omote, R.L. Mayes, S.W. Chen and R.W. Clough - 1977 (PB 277 933)A07

UCB/EERC-77/08 "DRAIN-TABS: A Computer Program for Inelastic Earthquake Response of Three Dimensional Buildings," by R. Guendelman-Israel and G.H. Powell - 1977 (PB 270 693)A07

UCB/EERC-77/09 "SUBWALL: A Special Purpose Finite Element Computer Program for Practical Elastic Analysis and Design of Structural Walls with Substructure Option," by D.Q. Le, H. Peterson and E.P. Popov - 1977 (PB 270 567)A05

UCB/EERC-77/10 "Experimental Evaluation of Seismic Design Methods for Broad Cylindrical Tanks," by D.P. Clough (PB 272 280)A13

UCB/EERC-77/11 "Earthquake Engineering Research at Berkeley - 1976," - 1977 (PB 273 507)A09

UCB/EERC-77/12 "Automated Design of Earthquake Resistant Multistory Steel Building Frames," by N.D. Walker, Jr. - 1977 (PB 276 526)A09

UCB/EERC-77/13 "Concrete Confined by Rectangular Hoops Subjected to Axial Loads," by J. Vallenas, V.V. Bertero and E.P. Popov - 1977 (PB 275 165)A06

UCB/EERC-77/14 "Seismic Strain Induced in the Ground During Earthquakes," by Y. Sugimura - 1977 (PB 284 201)A04

UCB/EERC-77/15 Unassigned

UCB/EERC-77/16 "Computer Aided Optimum Design of Ductile Reinforced Concrete Moment Resisting Frames," by S.W. Zagajeski and V.V. Bertero - 1977 (PB 280 137)A07

UCB/EERC-77/17 "Earthquake Simulation Testing of a Stepping Frame with Energy-Absorbing Devices," by J.M. Kelly and D.F. Tsztoo - 1977 (PB 273 506)A04

UCB/EERC-77/18 "Inelastic Behavior of Eccentrically Braced Steel Frames under Cyclic Loadings," by C.W. Roeder and E.P. Popov - 1977 (PB 275 526)A15

UCB/EERC-77/19 "A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments," by F.I. Makdisi and H.B. Seed - 1977 (PB 276 820)A04

UCB/EERC-77/20 "The Performance of Earth Dams during Earthquakes," by H.B. Seed, F.I. Makdisi and P. de Alba - 1977 (PB 276 821)A04

UCB/EERC-77/21 "Dynamic Plastic Analysis Using Stress Resultant Finite Element Formulation," by P. Lukkunapvasit and J.M. Kelly - 1977 (PB 275 453)A04

UCB/EERC-77/22 "Preliminary Experimental Study of Seismic Uplift of a Steel Frame," by R.W. Clough and A.A. Huckelbridge 1977 (PB 278 769)A08

UCB/EERC-77/23 "Earthquake Simulator Tests of a Nine-Story Steel Frame with Columns Allowed to Uplift," by A.A. Huckelbridge - 1977 (PB 277 944)A09

UCB/EERC-77/24 "Nonlinear Soil-Structure Interaction of Skew Highway Bridges," by M.-C. Chen and J. Penzien - 1977 (PB 276 176)A07

UCB/EERC-77/25 "Seismic Analysis of an Offshore Structure Supported on Pile Foundations," by D.D.-N. Liou and J. Penzien 1977 (PB 283 180)A06

UCB/EERC-77/26 "Dynamic Stiffness Matrices for Homogeneous Viscoelastic Half-Planes," by G. Dasgupta and A.K. Chopra - 1977 (PB 279 654)A06

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UCB/EERC-77/27 "A Practical Soft Story Earthquake Isolation System," by J.M. Kelly, J.M. Eidinger and C.J. Derham - 1977 (PB 276 814)A07

UCB/EERC-77/28 "Seismic Safety of Existing Buildings and Incentives for Hazard Mitigation in San Francisco: An Exploratory Study," by A.J. Meltsner - 1977 (PB 281 970)A05

UCB/EERC-77/29 "Dynamic Analysis of Electrohydraulic Shaking Tables," by D. Rea, S. Abedi-Hayati and Y. Takahashi 1977 (PB 282 569)A04

UCB/EERC-77/30 "An Approach for Improving Seismic - Resistant Behavior of Reinforced Concrete Interior Joints," by B. Galunic, V.V. Bertero and E.P. Popov - 1977 (PB 290 870)A06

UCB/EERC-78/01 "The Development of Energy-Absorbing Devices for Aseismic Base Isolation Systems," by J.M. Kelly and D.F. Tsztoo - 1978 (PB 284 978)A04

UCB/EERC-78/02 "Effect of Tensile Prestrain on the Cyclic Response of Structural Steel Connections, by J.G. Bouwkamp and A. Mukhopadhyay - 1978

UCB/EERC-78/03 "Experimental Results of an Earthquake Isolation System using Natural Rubber Bearings," by J.M. Eidinger and J.M. Kelly - 1978 (PB 281 686)A04

UCB/EERC-78/04 "Seismic Behavior of Tall Liquid Storage Tanks," by A. Niwa - 1978 (PB 284 017)A14

UCB/EERC-78/05 "Hysteretic Behavior of Reinforced Concrete Columns Subjected to High Axial and Cyclic Shear Forces," by S.W. Zagajeski, V.V. Bertero and J.G. Bouwkamp - 1978 (PB 283 858)A13

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