Seismic Strengthening of Existing Buildings

The methods of earthquake damage mitigation — including strengthening buildings and reducing earthquake forces — show a high degree of variation from building to building.

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The strengthening of existing buildings to resist earthquake forces has become over the years an issue of increasing concern and activity not only of engineers but also of the design profession as a whole, including governmental regulators, developers and real estate users. The reason for this concern is a growing awareness of the hazards of earthquakes and the governmental response to those hazards. Earthquakes pose not only a serious threat to life, but also the economic consequences of even a moderate earthquake can be disastrous. Not only can the repairs of buildings and infrastructure be very expensive, but also the cost of the disruption of business and services can have a long-lasting detrimental effect. Governments, both on the local and federal level, are ill-prepared to provide adequate financial aid to stricken communities. Clearly, an approach that addresses the causes of earthquake disasters rather than its effects is in the national interest. It is more cost effective to prevent a large-scale disaster through an earthquake awareness program that leads to the strengthening of hazardous buildings and facilities than to financially support the reconstruction of destroyed or damaged buildings or infrastructure.

This awareness led to the passage of the Earthquake Hazard Reduction Act of 1977 (Public Law 95-124), which directs the President "to establish and maintain an effective earthquake hazard reduction program."[1] The strengthening of existing seismically hazardous buildings is clearly an important element of earthquake hazard reduction.

An earthquake hazards mitigation program has three cornerstones:

- The establishment of the seismic risk;
- The determination of the vulnerability of structures; and,
- The development of hazard mitigating building codes and regulations.

Seismic Risk

The establishment of seismic risk is a complicated and often controversial task. Little action will be taken in earthquake hazard mitigation if the risk of earthquakes is not understood or not taken seriously. Even in California, where most people who have lived there for any length of time have experienced an earthquake, the full severity of a major earthquake does not

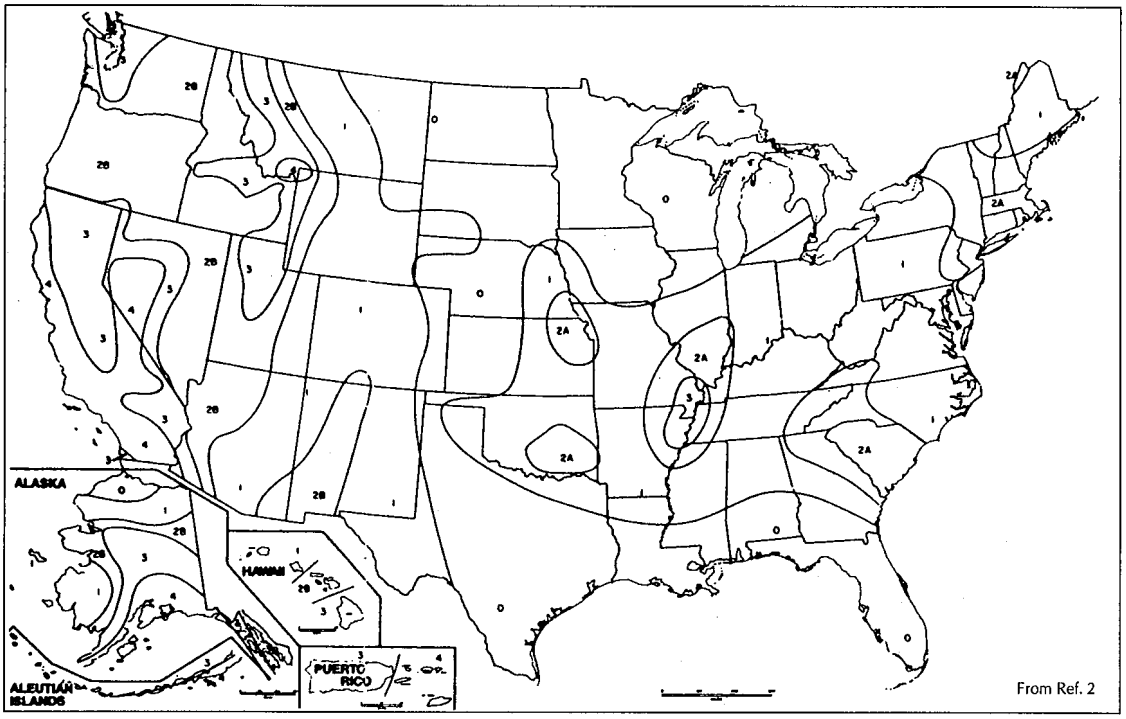


FIGURE 1. A UBC seismic zonation map of the United States.

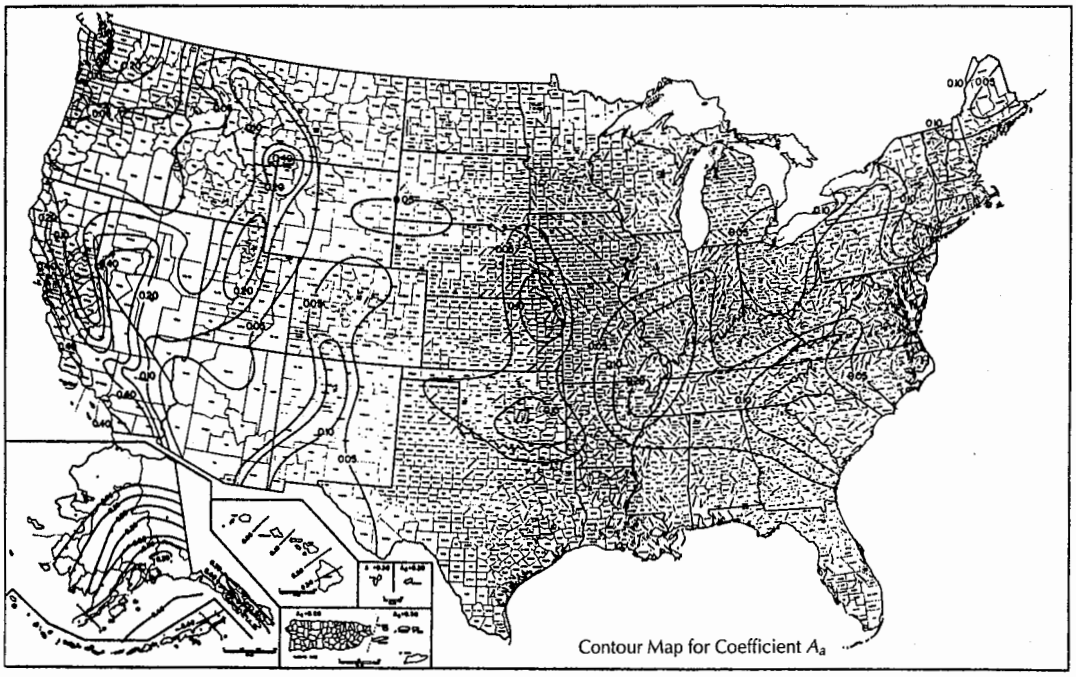
appear to be a driving force. In instances where there is sufficient awareness of the seismic risk, the inconvenience and cost of mitigating earthquake hazard in existing buildings override that awareness. It is therefore not surprising that seismic risk awareness and hazard mitigation is low in areas of lesser seismicity.

The seismic risk of an area is based on its past history of earthquake events as well as on geological observations and deductions. Regions of high seismicity naturally have quite extensive records, but in regions of low seismicity, such as New England, the historical record is sparse. Earthquakes were recorded only if they were of sufficient significance or left traces of their existence. Since ground motion recording devices are relatively recent inventions, quantitative data on historic earthquakes have been matters of deduction. In addition to historic records, seismicity can be determined by geological and seismological exploration to locate existing faults, both active and inactive fault traces, and by the assessment of geologic and tectonic features.

It can be argued that even though the frequency of earthquakes in the New England-New York region may be considerably lower than in California, the seismic risk might well be higher.

The design assumptions behind the *Uniform Building Code* (UBC) [2] and the *National Earthquake Hazard Reduction Provisions* (NEHRP)[3] guidelines are based on a 475-year event, or an event with a ten percent probability of not being exceeded in 50 years. In the case of California, the difference between this design level and the maximum credible earthquake is not great. (The maximum credible earthquake is defined here as the largest earthquake that a particular fault mechanism can generate.) Modern well designed structures in California should survive such an event without collapse even though they may suffer considerable damage.

In regions of low seismicity where very large earthquakes can occur (although they have a low probability of occurrence), the difference between the current design earthquake and the maximum credible event can be very large. Therefore, the rare but possible event, which may have a 1,000-year occurrence, can have a devastating impact on the region.



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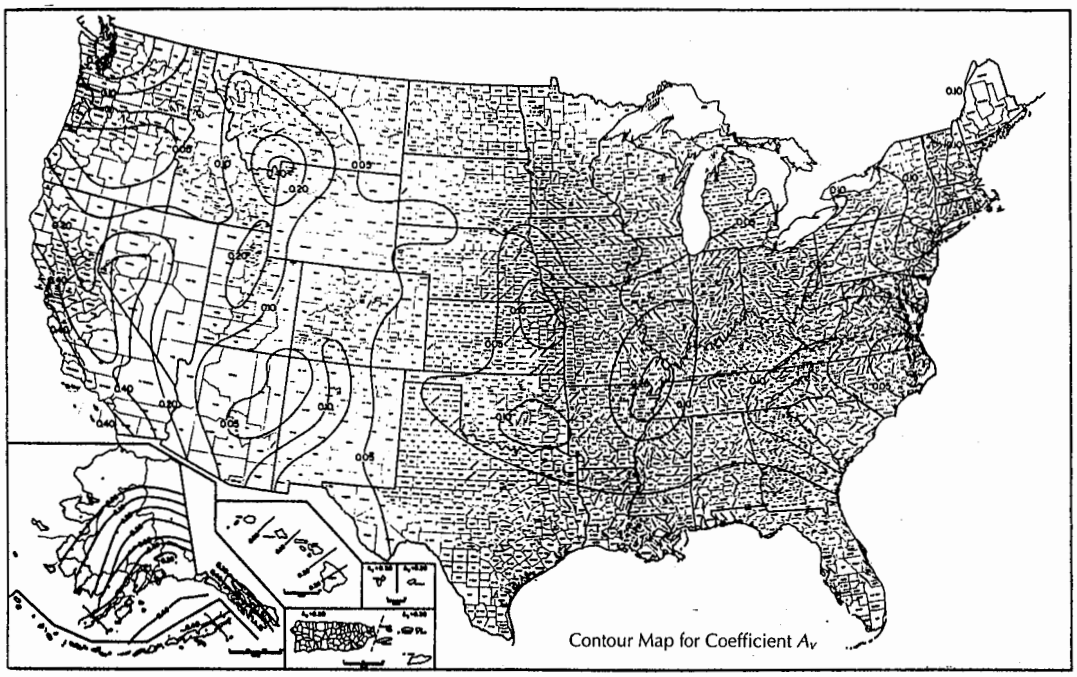

FIGURE 2. NEHRP seismic zonation maps of the United States.

Seismic zonation is an ongoing effort. Currently used zonation maps are included in the UBC (see Figure 1) and NEHRP (see Figure 2). The NEHRP zonation maps are currently under revision (see Figure 3). Earthquake-resistant design is based on an assumption of the seismicity and seismic risk of a specific region. Estimates based on recorded earthquakes carry

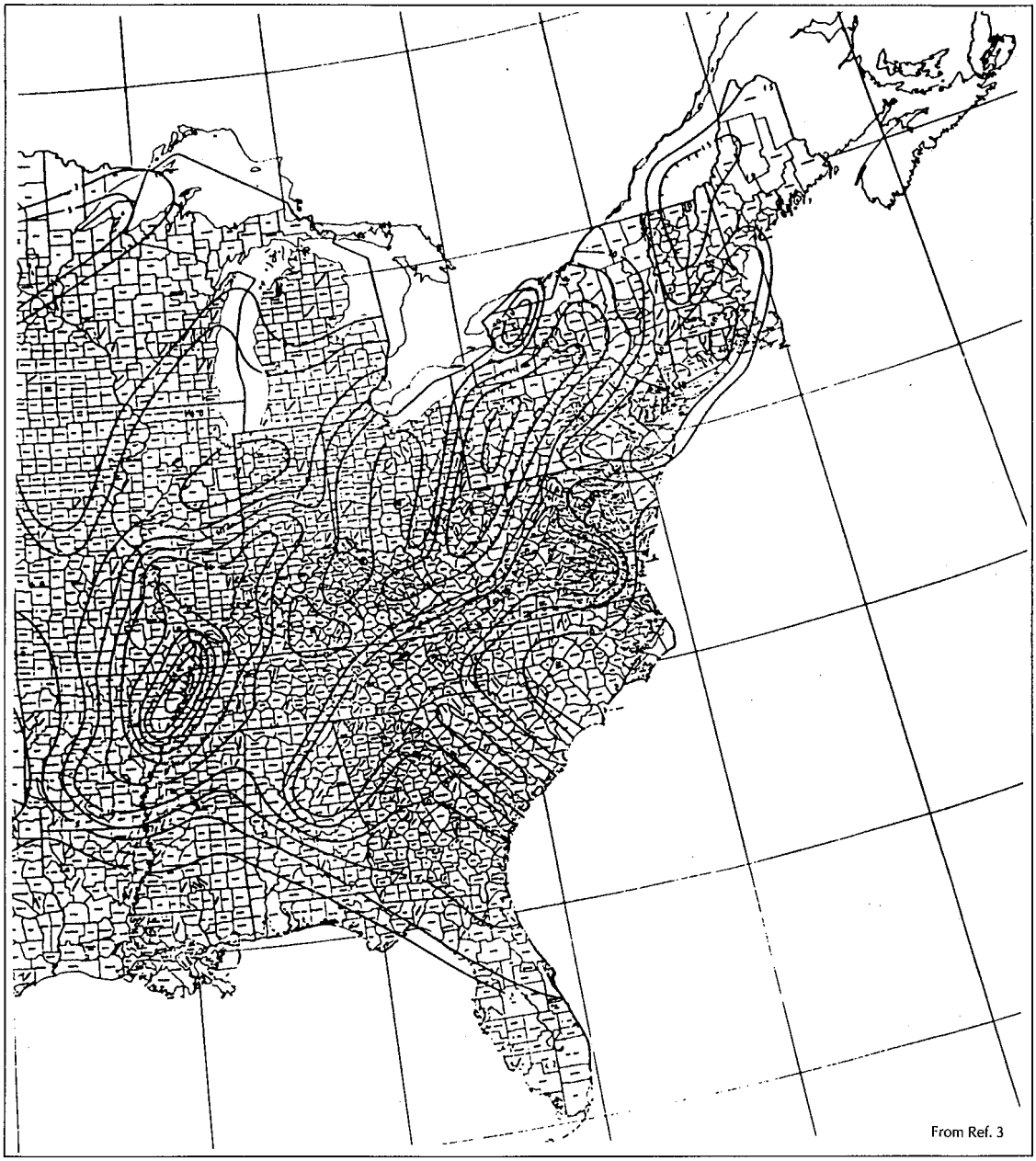

FIGURE 3. Proposed new NEHRP seismic zonation map of the eastern United States.

with them a degree of uncertainty that depends on the completeness of the earthquake catalogs used, the time span of the catalog (which may be less than the average interval between the larger earthquakes) and the lack of knowledge of the magnitude of historic earthquakes. The determination of seismic risk should be given the highest priority in a hazard abatement program. The engineer must know the demand that earthquake forces can place on a structure in order to provide the appropriate design capacity of the structure to resist the demand.

Vulnerability of Structures

Seismic vulnerability can be determined by a careful evaluation of the structural system and the detailing used. Examples of such an examination are:

- The identification of the lateral load resisting elements, their location and distribution, as well as their consistency over the height of the structure.
- The detailing of the structural elements and their ability to transmit lateral forces to the resisting elements.
- The compatibility of cladding and finishes with the building's anticipated behavior.
- Adjacency to other buildings and the possibility of pounding between structures.
- Signs of previous damage or deterioration of the structure or finishes.

An analysis should quantify the resistance capacity of the structure. An important part of the analysis is the application of a model that takes into consideration all structural elements that participate in the lateral resistance of the building. Elastic and inelastic properties must be taken into consideration. The importance of judgment developed by field observations of the effects of earthquakes on structures is not to be underrated. Much of what has been observed has become the basis of earthquake design provisions.

The program entitled "Learning from Earthquakes," sponsored by the Earthquake Engineering Research Institute (EERI) with the support of the National Seismic Foundation, has provided many engineers with the opportunity to observe and study firsthand the effects of earthquakes, and has provided a valuable library of field observations. Past earthquakes have provided a laboratory where engineers, architects, disaster response personnel and others concerned with the effects of earthquakes can see the failures and success of efforts to mitigate the effects of earthquakes.

Visiting a stricken community can leave an indelible impression. Field observations are of special value in assessing the vulnerability of buildings that were designed prior to the development of modern building codes and design and analysis procedures. The list of lessons learned via these observations is long, but a few salient examples are:

- The performance of unreinforced masonry is consistently poor.
- Walls need to be securely tied to floors and roofs (also true for tilt-up construction).
- Non-ductile concrete frames perform poorly and shear failures of beam-column joints are common.
- Reinforcing bars need to be adequately spliced and anchored.
- Flat slab construction, which has very little tolerance for story drift, performs poorly.
- The P-Δ effect of taller buildings must be considered in design.
- A quality assurance program should be implemented in the field.

By far, the greatest hazard can result from an inappropriate building configuration. Changes in stiffness over the height of the structure, soft stories with large openings at the ground floor, or unusually high stories are hazardous unless they are carefully designed. Re-entrant corners, such as in T-shaped or L-shaped plans, raise stresses and can lead to serious problems.

In evaluating the vulnerability of a building, the geotechnical setting must be considered. Obviously ground failure, settlement or liquefaction has a profound effect on the structure. The 1985 Mexico Earthquake, and to some degree the Loma Prieta Earthquake in the San Francisco Bay Area in 1989, have demonstrated that even distant earthquakes can, under certain soil conditions, amplify ground motions and make buildings in resonance with these motions extremely vulnerable.

A study of the vulnerability of structures to resist earthquakes is a key issue in successful earthquake engineering. It is critical to know where, how and why structures and structural elements fail and which are the weak links. Careful analysis can provide the answers, but the study of past events is invaluable in developing judgmental understanding.

Regulations & Voluntary Hazard Abatement

Building codes and governmental regulations regarding reducing the risk from seismically hazardous buildings fall into two categories: *active* hazard abatement and *passive* hazard abatement programs.

In active programs, a target is typically identified. This target may be a type of building,

such as unreinforced masonry buildings or building components such as parapets or elevators, which are judged to pose a hazard that requires corrective work. Examples of such active programs are the Los Angeles Unreinforced Masonry Strengthening Ordinance, the San Francisco Parapet Safety Program, the State of California Retroactive Elevator Requirements, and the State of California Unreinforced Masonry Abatement Program.

A common denominator of these programs is that time limits are established for compliance with the requirements that allows for progressive corrections in some cases. Generally, the requirements are significantly different from those required for new buildings since they are less stringent than those for new construction.

Passive hazard abatement programs are seismic strengthening requirements that are "triggered" by proposed changes to the building, such as an increase in occupancy load or hazard classification, alterations to the structural system, or alterations that affect a certain percentage of the area of a building or increase the value of a building. Compliance to the levels required for new construction may be difficult to accomplish and extremely expensive, therefore the requirements may be less stringent.

For example, Section 104f of the *San Francisco Building Code* (SFBC) requires the upgrading of a structure to only 75 percent of the code requirement for new buildings.[4] The rationale for that requirement is that at this level *life-safety* (a sufficient probability that persons in and around the structure will not be harmed) should be provided, even though a certain degree of property damage is accepted. The complexity of compliance frequently requires a case-by-case negotiation between the owner and the appropriate governmental agency.

The most common passive hazard abatement program method in codes has been what is called the "25-50 Percent Rule." Basically, this rule states that if the proposed work on a building exceeds 50 percent of its value, full compliance with the code is required. If the work is valued between 25 and 50 percent of the building value, the requirements are lessened and often pertain only to the alterations themselves. If the work is less than 25 percent of the building value, the work must not endanger public safety or extend existing hazards. The philosophy of this rule, which requires an increasing level of seismic upgrading with increasing levels of building alterations or additions, has found its way into many building codes.

Earthquake damage is another "trigger" that leads to mandatory corrective work. For example, in San Francisco, if the damage requiring repair affects more than 30 percent tributary to the vertical or lateral load carrying elements, or of these elements themselves, the entire structure of the building must be brought up to meet the current code requirements.

There is evidence that the amount of voluntary earthquake hazard abatement work is increasing. Although the motives may be manifold, the dominant ones appear to be to protect life-safety, to protect property and to maintain business activity. Examples can be found in the high-tech industry in Silicon Valley, south of San Francisco. IBM has strengthened buildings, braced piping systems, anchored equipment, and has established in-house seismic safety standards. Hewlett-Packard, Intel and Apple are further examples of many companies that have established active seismic hazard abatement programs. It can be conjectured that in this industry enlightened management, expensive equipment, difficult to replace research and intense competition have triggered this activity.

Utilities have also developed their own programs in seismically strengthening their facilities. The Pacific Gas & Electric Company, the East Bay Municipal Utility District, the Southern California Gas Company, and the Los Angeles Department of Water and Power are examples. Private building owners and real estate investors have also become more active in seismic hazard abatement.

Although most of this activity is taking place in California, there are examples of voluntary efforts in other states. The City and County Building in Salt Lake City, Utah, is one of these examples. The trend in voluntary earthquake hazard abatement is encouraging. Aside from the issue of life-safety, there is an increasing understanding that, from a business point of view, the cost of strengthening a building can be far less than its repair or replacement, espe-

cially if life-cycle costs are taken into consideration rather than initial construction costs.

Retrofit Methodologies & Techniques

Conventional Methods. There are a great number of methods and techniques that can be used to strengthen the seismic resistance of existing structures. The selection of the appropriate method is influenced by a great number of factors, such as:

- The structural system used in the original building construction;
- The architectural finishes (both exterior and interior);
- The internal function and use;
- The building configuration and shape; and,
- The building's historic significance.

The most common method is to add earthquake-resisting elements to the structure. These elements can be concrete shear walls, or moment frames or braces installed either externally or internally. A subgroup of these approaches is the addition of external buttresses.

Another method is to replace the existing components of the building with new lateral force resisting elements, such as replacing nonstructural core elements with new seismic resistant elements. The replacement or strengthening of existing structural components is also common. Existing masonry walls can be replaced with reinforced concrete walls, diagonal steel members can be added to steel frames to create braced frames, or wall openings may be closed in order to provide greater lateral strength. Figure 4 presents some of these methods.

Considerations in evaluating which seismic upgrading method to use are (see Figure 5):

The Capacity and Stiffness of the Floor Diaphragms. For instance, if shear walls are provided at the end walls of a rectangular building, diaphragm shears may exceed the capacity of the existing system forcing strengthening or replacement.

Existence or Capacity of Earthquake Force Collectors. Should the resisting elements be installed in the interior of the building, there must be a collector element that can transmit the inertial forces to the resisting elements.

Foundation Capacity. The new resisting elements will introduce additional loads on the footings, especially if the resisting elements (such as concrete shear walls or braced frames) are compact. High overturning forces, both compression or uplift, may require modifications to the foundations.

Structural Compatibility. It is crucial that the stiffness of new resisting elements is compatible with the flexural capacity of the existing structure. If the new element, such as a steel brace, will deflect under design load beyond the flexural capacity of the existing system, such as a masonry wall, the new element cannot protect the building from severe damage or failure.

Base Isolation. During recent years, the use of base isolation has become an accepted method in the seismic rehabilitation of existing buildings. The concept of base isolation is to uncouple the building from the ground by supporting it on isolating devices, making the building respond to the period of the isolator. As can be seen from Figure 6, a period shift from 1 to 2 or 2-½ seconds can reduce the building's response to the ground motion substantially. Clearly, the greater the period shift, the more effective the isolation systems become. However, should the building period coincide with that of the isolator, disastrous harmonic response will occur.

Base isolation is not a cure-all. Even though earthquake ground motion might be dramatically reduced, the reduced ground motion will still be transmitted to the structure. Other considerations — such as configuration, adjacency to other structures, overturning forces, etc. — must still be addressed. The motions of the building, sometimes as much as 18 inches, must be provided for. Utilities, elevators, stairs, etc., have to be designed and detailed to accommodate the motions of the building. Generally, base isolators have no tensile capacity; therefore, column uplift due to overturning forces must be avoided. Although the first cost of an isolator retrofit is generally higher than a conventional retrofit, many factors may make it a more desirable solution, especially true in the case of historic or landmark buildings where

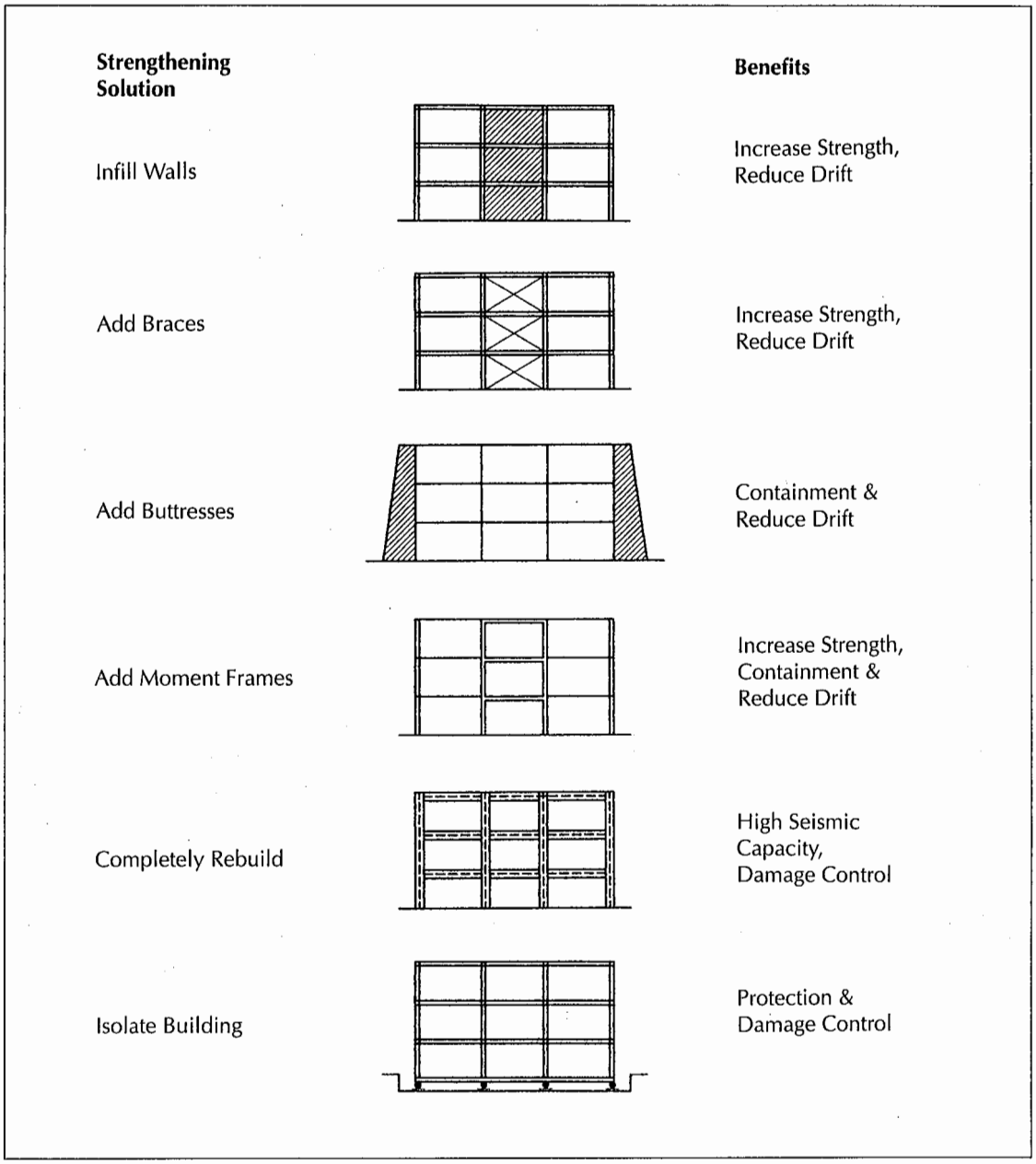

FIGURE 4. Conceptual solutions for the seismic upgrading of existing buildings.

the preservation of the historic fabric of the building is essential.

There are currently a number of isolating devices on the market and more in the development stage. Figure 7 shows three of the most common isolating devices: the high damping rubber isolator, the lead/rubber isolator and the inverted pendulum.

Passive Energy Dissipation. Traditional earthquake design practice permits the use of forces lower than those expected in the elastic design on the premise that, in a well designed and detailed structure, inelastic action will provide sufficient energy absorption to survive the earthquake without collapse. The object of energy dissipation is to introduce devices that are designed to dissipate energy and by doing so reduce the need of the primary structure to do

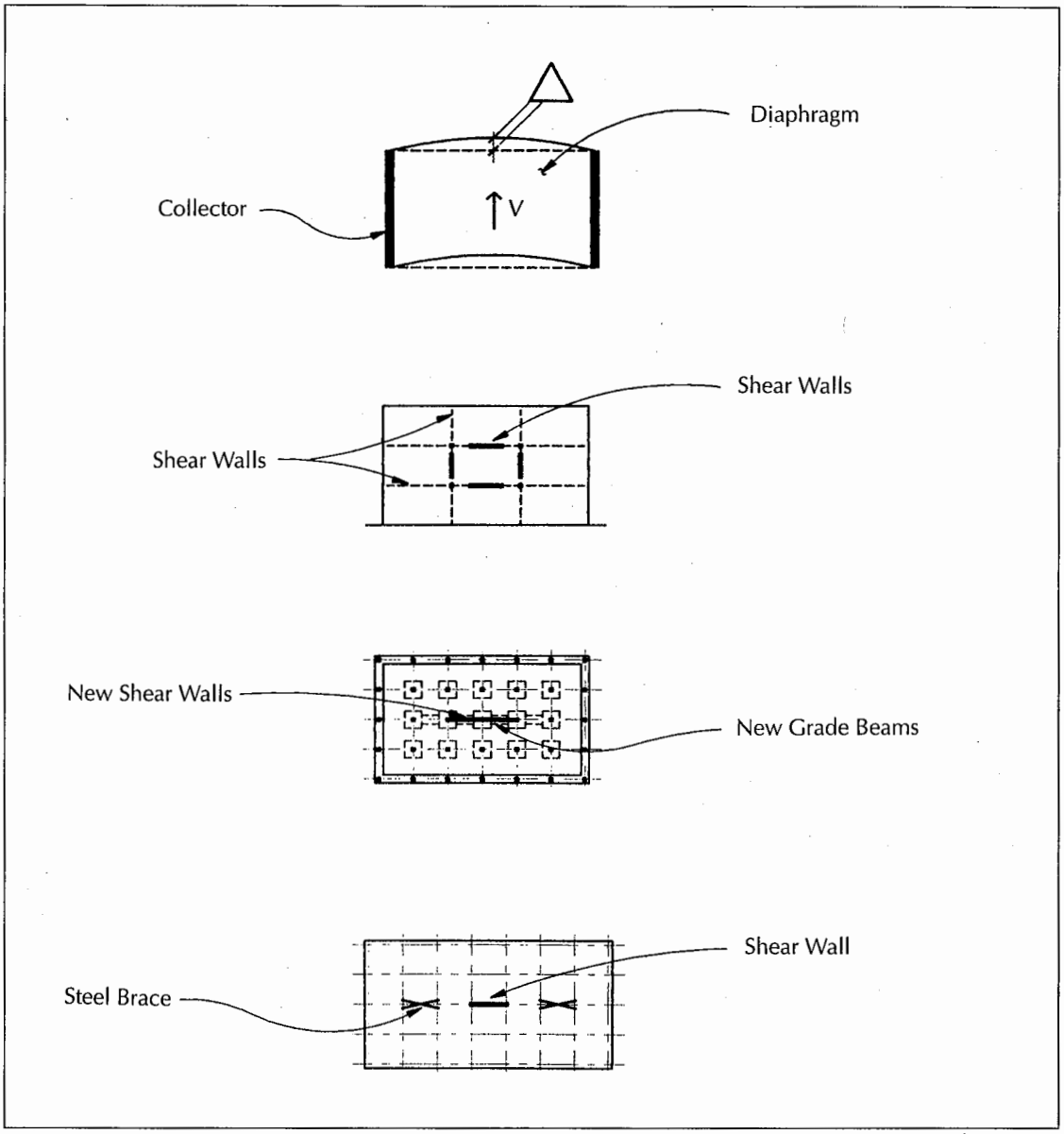

FIGURE 5. The application of different seismic upgrading systems on existing buildings.

so. In other words, energy dissipating devices add damping to a structure in such a way that they significantly reduce response to earthquakes. Some types of dissipation systems are visco-elastic dampers, friction dampers and lead extrusion systems.

Some examples where passive energy dissipation has been applied are:

- A one-story concrete frame building owned by Wells Fargo Bank in San Francisco. This building was damaged as the result of the Loma Prieta Earthquake in 1989. The building, approximately 80 feet square, was retrofitted by installing a steel frame at the four sides of the building. The brace was connected to the building with patented metallic-yield energy dissipators that provided added damping and stiffness.
- A number of buildings in Canada have been retrofitted using friction dampers.

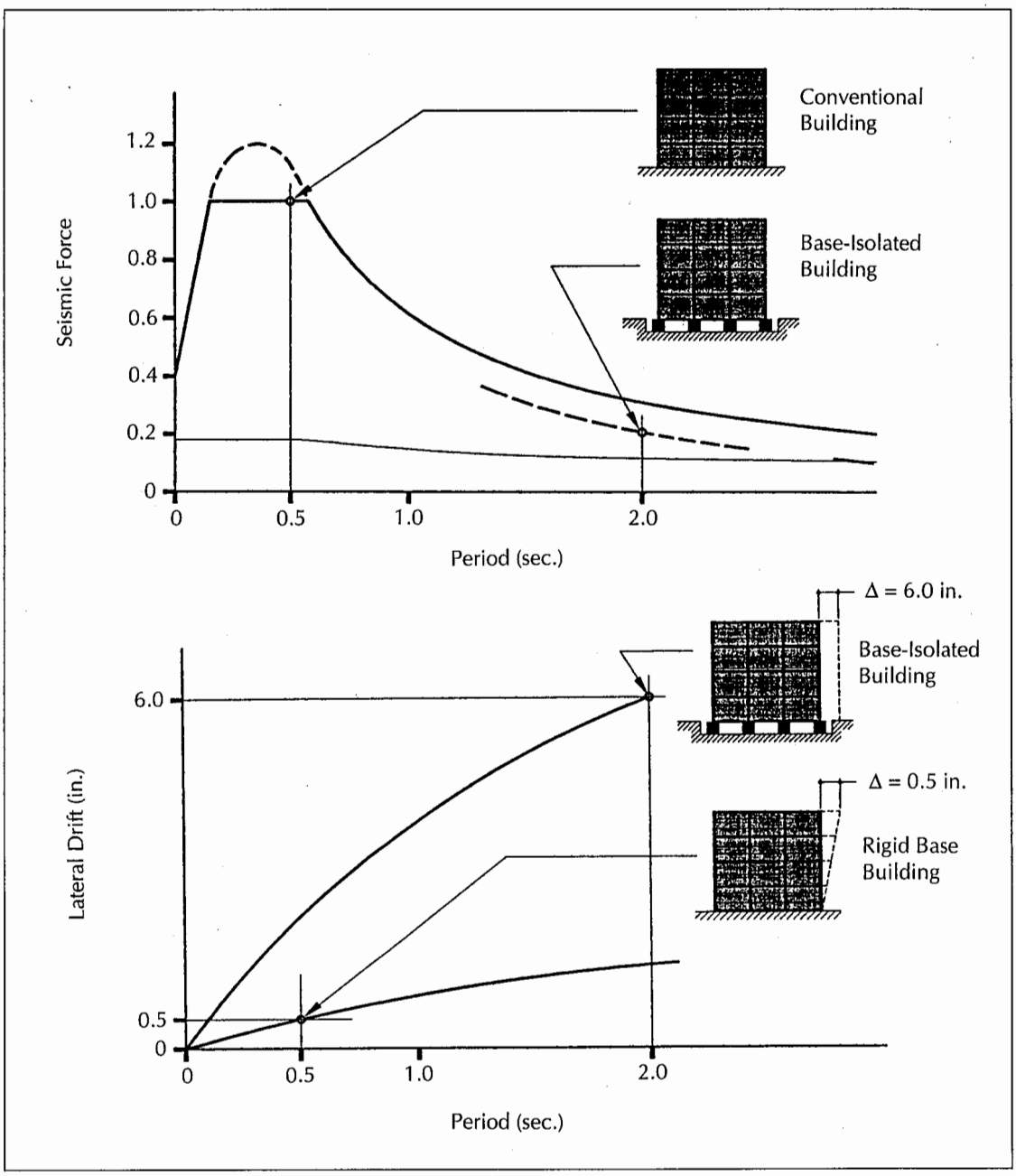

FIGURE 6. Base isolation response.

The devices consist of a series of specially treated steel plates that are clamped together and that are allowed to slip at a predetermined load.

More detail on these types of systems can be found in the proceedings of a seminar recently held by the Applied Technology Council (ATC-17.1).[5]

Active Energy Control. The development of active energy control systems is of fairly recent origin. Though there is much research being carried out in the United States, most of the work and applications have occurred so far in Japan. Generally speaking, the primary focus of active energy control has been on mass dampers.

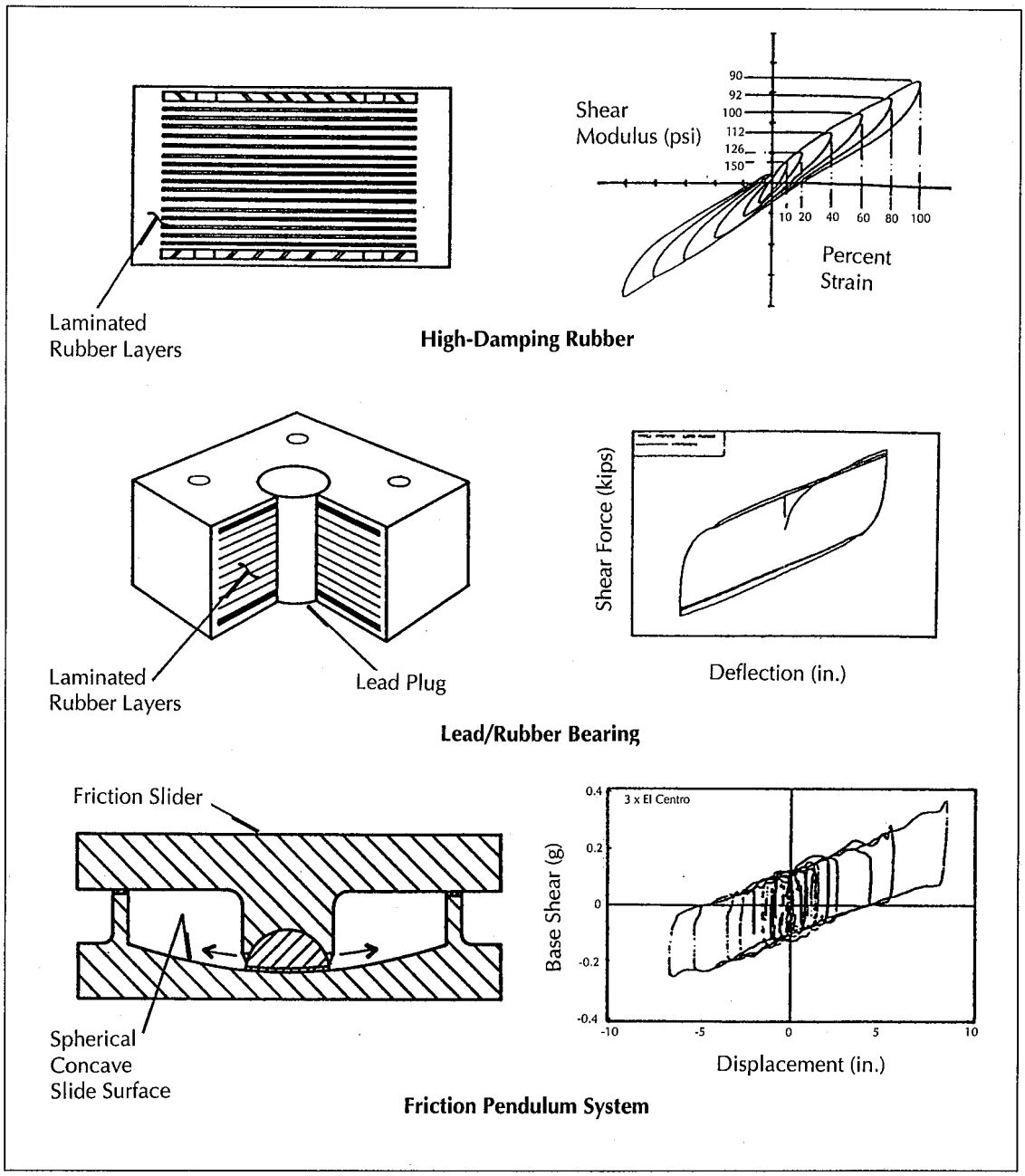

FIGURE 7. Isolation systems and hysteresis curves.

Importance of Case-by-Case Evaluation. The plethora of seismic retrofit methods and techniques should make it clear that there are not fixed solutions to the seismic strengthening of existing buildings. Each building represents a unique problem, the solution of which relies on careful study, ingenuity and experience. Some of the examples that follow have been chosen to demonstrate the great variety of problems and solutions involved in seismic retrofit.

IBM Product Development Center

In the early 1970s, the IBM Corporation in San Jose, California, embarked on a voluntary seismic hazard mitigation program. The motivation no doubt came from a greater awareness

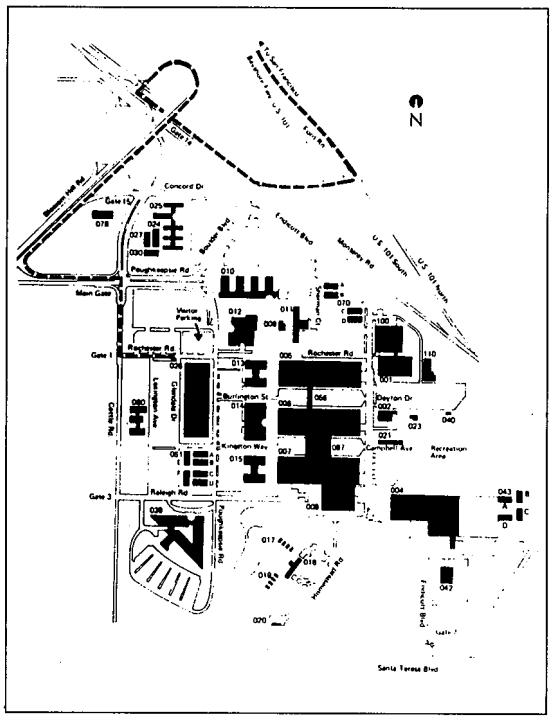

FIGURE 8. Site map of the IBM complex at San Jose.

of the risk of major earthquakes, possibly triggered by the NEHRP program, as well as a concern about life-safety and the post-earthquake operational capability of these important facilities. IBM retained a structural engineering firm to develop a hazard mitigation program. The work was carried out in stages because of the large number of buildings on the site (see Figure 8):

Stage I consisted of the rapid visual evaluation of all buildings. Drawings were reviewed and visual inspection performed. Evaluations were based on the building code used during design, and on the structural system used. Buildings were rated as either good, average or poor. Experience and judgment were major factors in these evaluations.

Stage II established priorities for further engineering studies, in coordination with facilities staff. Performance criteria were developed that set the acceptable design force.

In *Stage III* each building was analyzed for conformance with the design criteria, and a

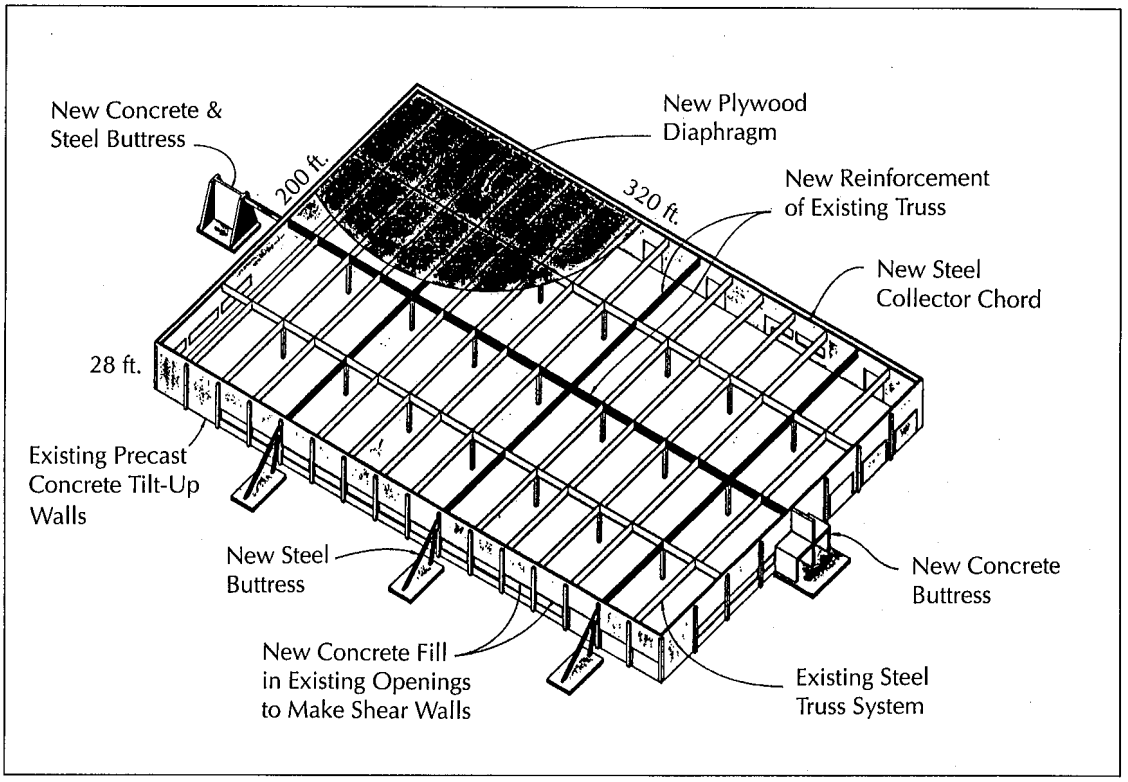

FIGURE 9. Schematic view of the seismic bracing for IBM Building 001.

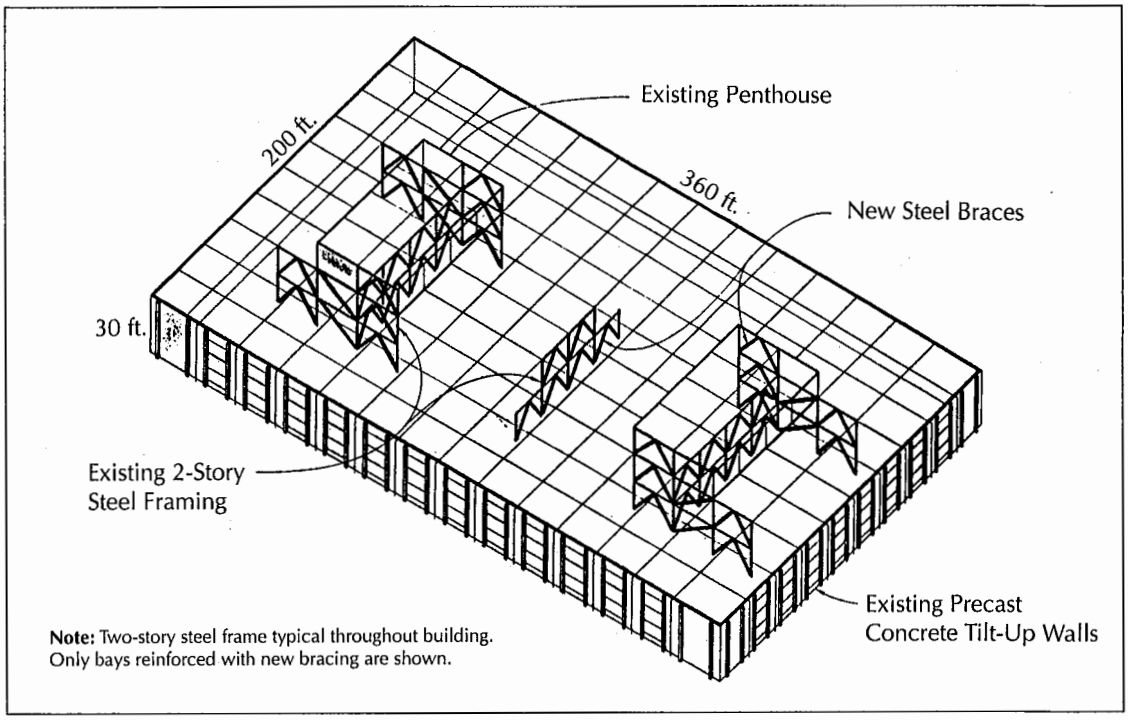

FIGURE 10. Schematic view of the seismic bracing for IBM Building 014.

retrofit concept was developed in accordance with the priorities established in Stage II (see Figures 9 and 10).

In *Stage IV* contract documents were prepared for the retrofit of each building, and bids were taken.

Stage V addressed the building source and process equipment. Criteria were developed for the seismic bracing of all ducts, pipes, mechanical and electrical equipment, and building contents. Because of the magnitude of this task, IBM's facilities management department retained a number of consultants to expedite this work.

IBM developed an in-house manual and trained staff to maintain the required level of earthquake preparedness. Subsequent earthquakes have proven that the work done is effective.

Unites States Geological Survey

The seismic retrofit of Buildings 1 and 2 at the United States Geological Survey (USGS) site in Menlo Park, California, is of note because of the constraints placed on the work. In the case of Building 1, it was to be kept fully occupied during the retrofit construction; in Building 2 (see Figure 11), only limited disruption was permitted. The buildings were upgraded to meet current UBC code requirements.

Building 1 is a light steel frame structure with a wood floor and roof. The adopted retrofit solution was to flank each side of the building with a moment-resisting exposed steel frame acting both in transverse and longitudi-

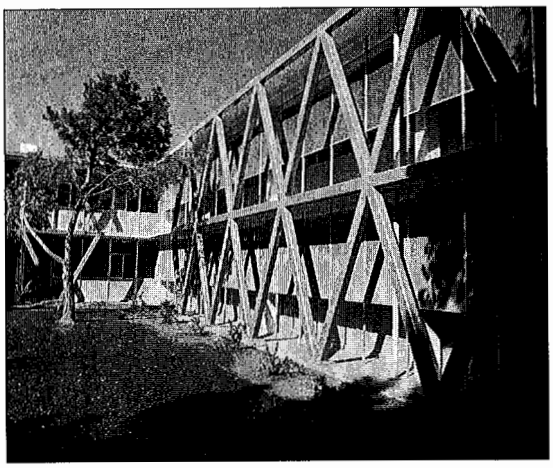

FIGURE 11. A view of USGS Building 2.

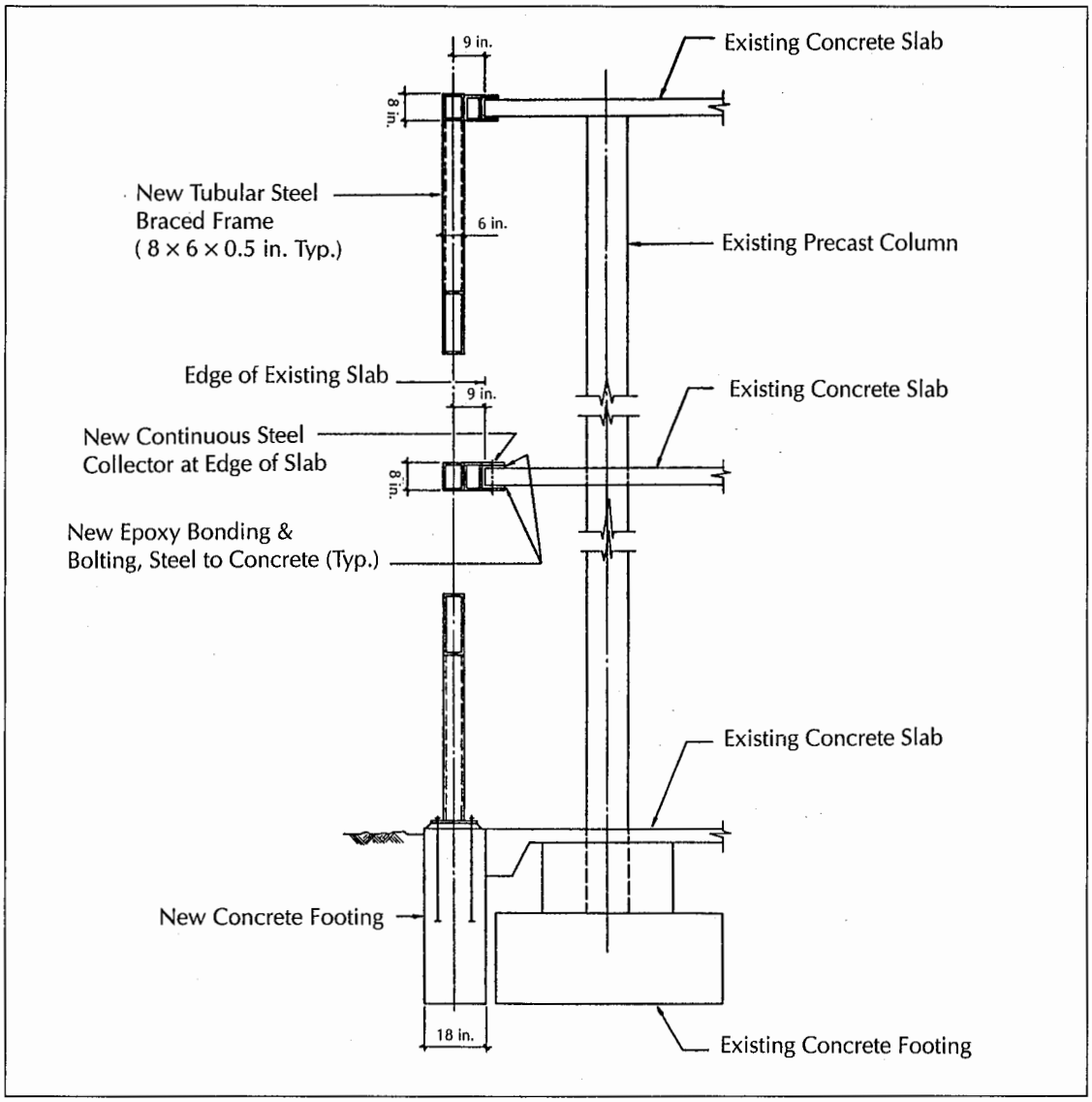

FIGURE 12. A section of the brace frame for USGS Building 2.

nal directions. The existing building was anchored to this new system. The exposed steel frame was designed to support a future horizontal expansion of this building.

Building 2 is a post-tensioned concrete lift slab structure. The major seismic strengthening elements are exposed tubular vertical trusses constructed with new foundations at the face of the building (see Figure 12). These trusses are connected to the existing concrete slabs that cantilever beyond the face of the building. Some new interior concrete shear walls were provided in sections closed off from the rest of the building (see Figures 13 and 14). Off-hour construction was used when needed.

Lawrence Livermore National Laboratories

Building 111, the Central Administration Building at the Lawrence Livermore National Laboratories (LLNL), in Livermore, California, was built in the late 1960s. The seven-story reinforced concrete structure was built in an unsymmetrical cross shaped plan, with a central core housing vertical circulation and utility shafts.

The building's vertical load carrying system

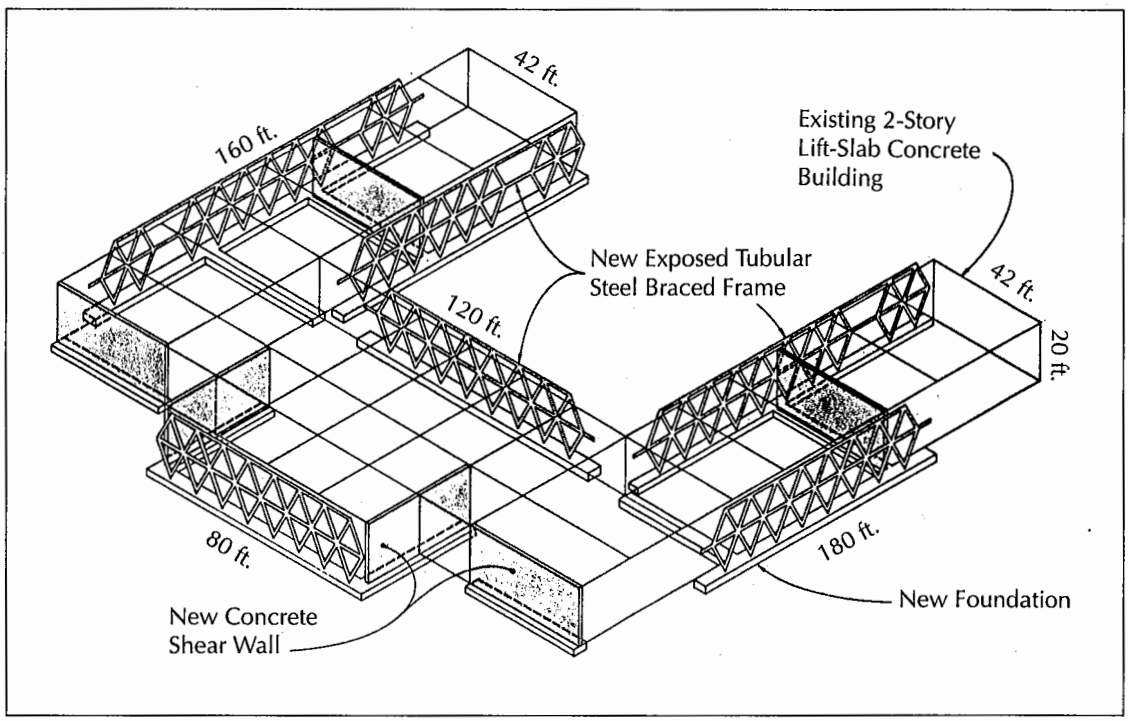

FIGURE 13. A schematic view of the seismic bracing for USGS Building 2.

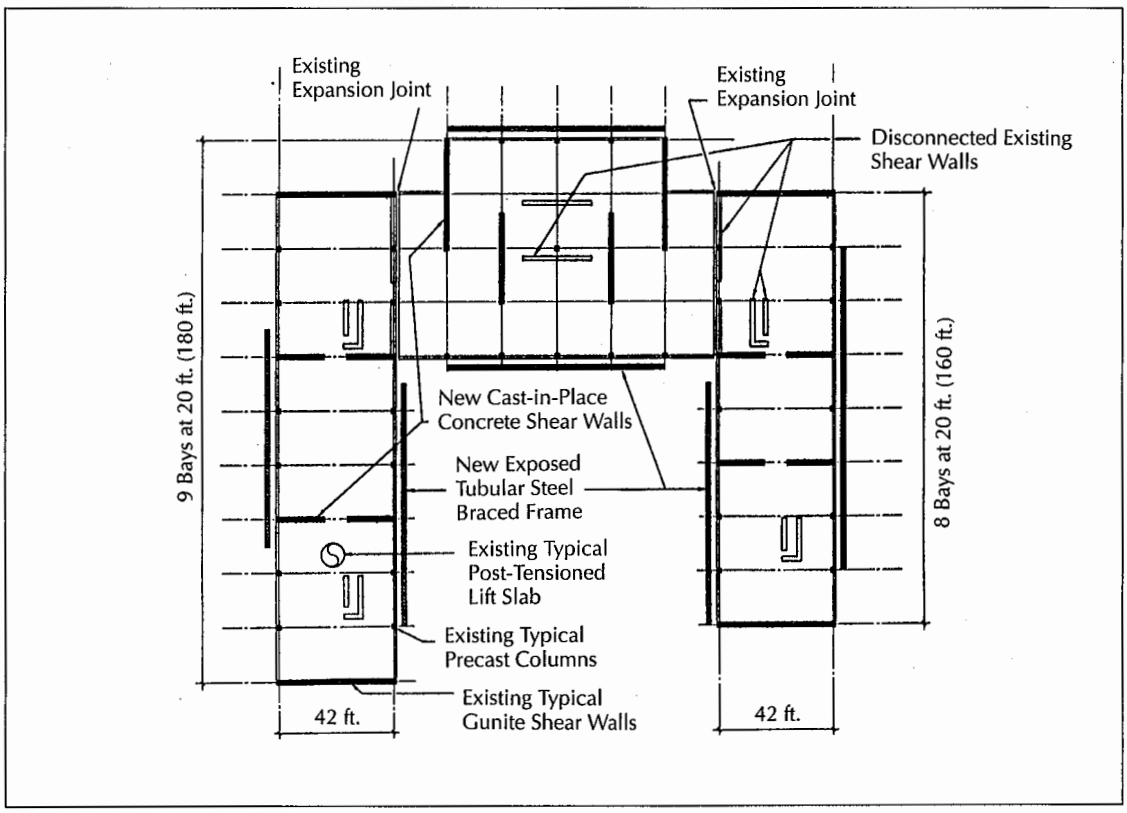

FIGURE 14. The seismic strengthening system for USGS Building 2.

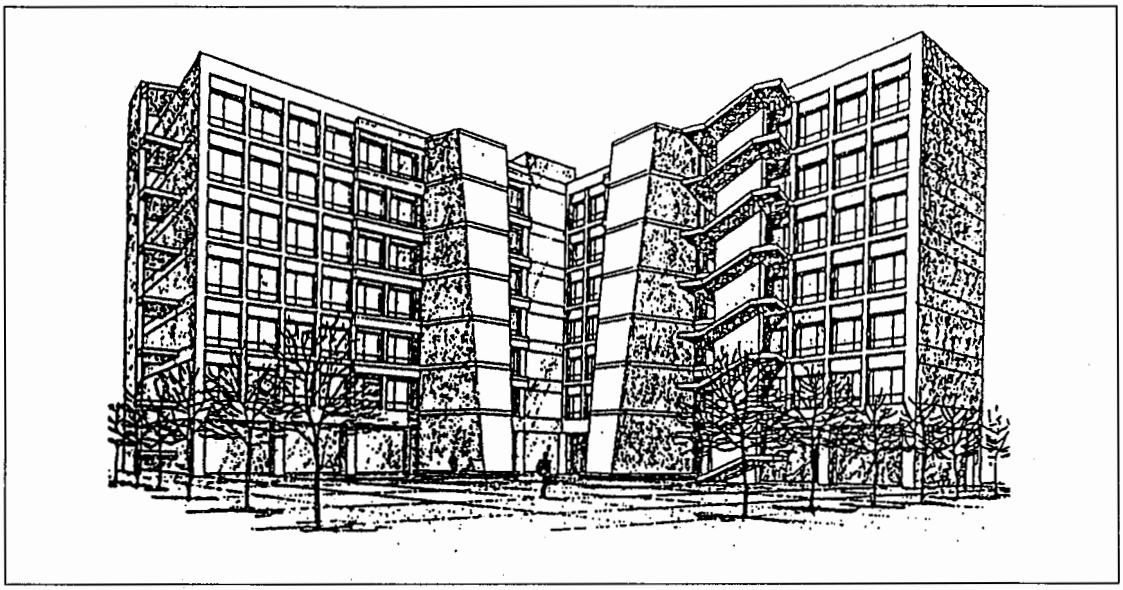

FIGURE 15. LLNL Building 111.

is composed of a 4-½ inch concrete slab supported on reinforced concrete beams spaced ten feet on center spanning across the wings where they are supported, in turn, by reinforced concrete columns. The existing lateral system is composed of reinforced concrete shear walls at the exterior ends of the wings, and shear walls enclosing the core.

The building was subjected to three earthquakes ranging in Richter magnitude from 5.1 to 5.8 in January 1980. Extensive cracking of shear walls, slabs and beam-column connections resulted. Repairs, accompanied by a limited degree of strengthening, were done after the earthquakes.

LLNL commissioned a full seismic upgrade in 1988. The seismic design criteria established for this building were:

- Non-collapse at a 0.5 g, ten percent damped, UBC shaped response spectrum, or UBC 1988 equivalent static approach (whichever is more stringent);
- Dynamic as well as static load analysis for comparison;
- Minimum disruption of occupants during strengthening; and,
- Minimum intrusion into the building during construction because of the sensitive nature of its contents.

The accepted solution to strengthen the building was to add two large tapered reinforced concrete box-section towers at the exterior of the two long building wings, with the

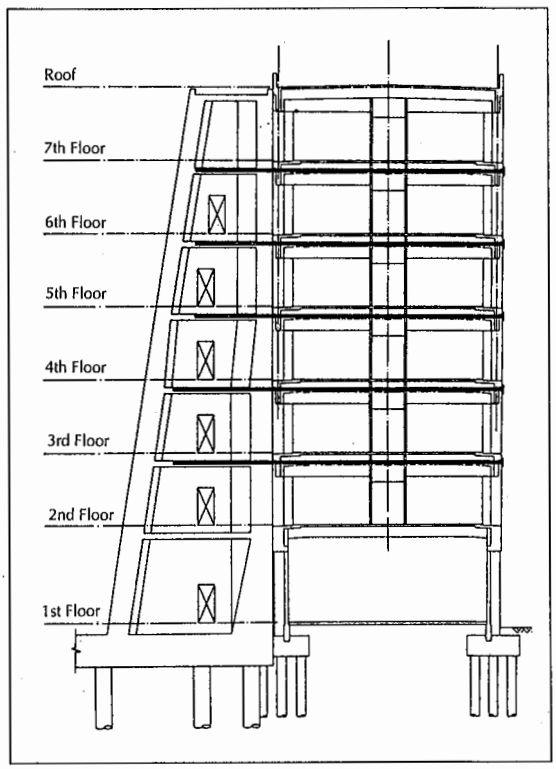

FIGURE 16. Section of LLNL Building 111.

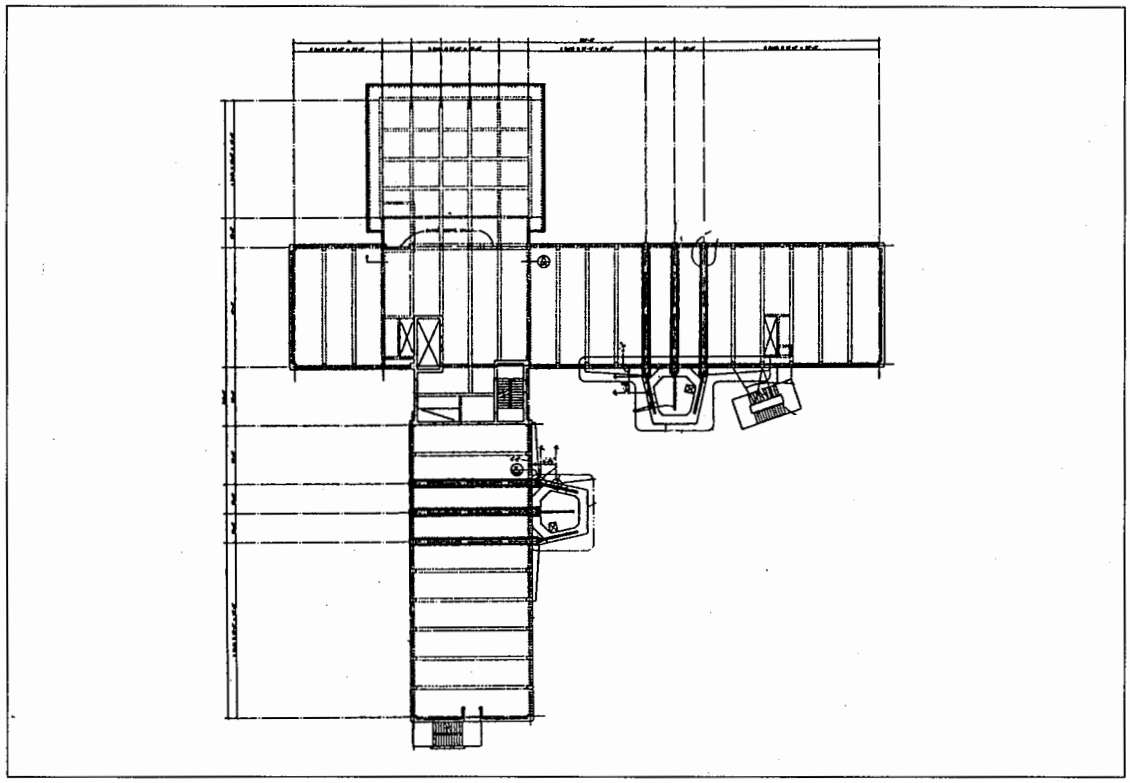

FIGURE 17. Plan view of LLNL Building 111.

towers located to minimize the stresses in the long diaphragms (see Figure 15). The towers rise from a seven-foot thick reinforced concrete foundation mat, which is supported on 38 three-foot diameter drilled reinforced concrete piers extending from 50 to 70 feet below the mat (see Figures 16 and 17).

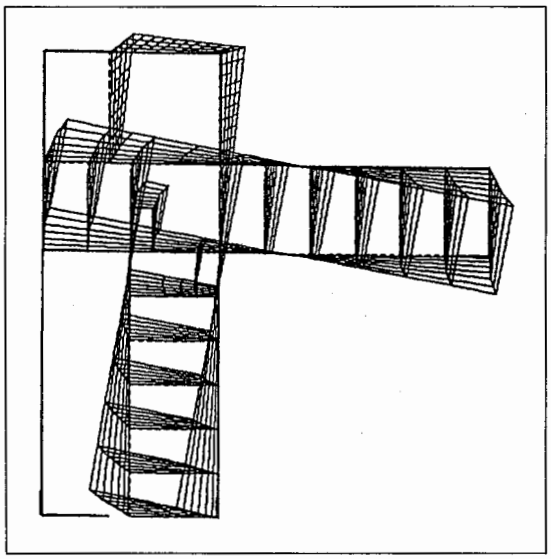

FIGURE 18. Three-dimensional model of LLNL Building 111.

The towers were connected to the building with channel-shaped steel collectors on each side of the floor beams. The collectors extend through the building terminating on a steel bearing plate anchor. The steel channels are epoxy bonded to the side of the reinforced concrete beams. Also, exterior reinforced concrete edge collectors were provided to transfer seismic shear to the towers, and interior 12-inch reinforced concrete shear walls were constructed in isolated locations to reduce the undesired torsional response of the structure.

Extensive analyses were made using four computer models (see Figure 18):

- A three-dimensional model was created (using a three-dimensional finite element program for structures with rigid diaphragms) that incorporated all new and existing structural components.

- A three-dimensional finite element program for modeling frames, shells and asolid elements was used. Foundation springs simulated the vertical and horizontal stiffness of the piers for this structural analysis program (SAP).
- A SAP computer model was also used to assess the effect of vertical accelerations on the structure.
- Lastly, a three-dimensional flexible diaphragm SAP model was developed to assess the effect of the flexibility of the diaphragms on the effectiveness of the newly added towers.

Three variations of the three-dimensional model for structures with rigid diaphragms were employed to simulate various building responses:

- A model excluding the interior collectors served to determine the compression force between the buildings and the tower;
- A model including the interior collectors determined the tensile forces in the collector; and,
- A full model, including the interior collectors and pile foundation, was fixed with a dummy level below the ground to simulate vertical, lateral and rotational stiffness of the pile foundation.

The overall construction cost for this project was below estimates, and construction was completed in 1991.

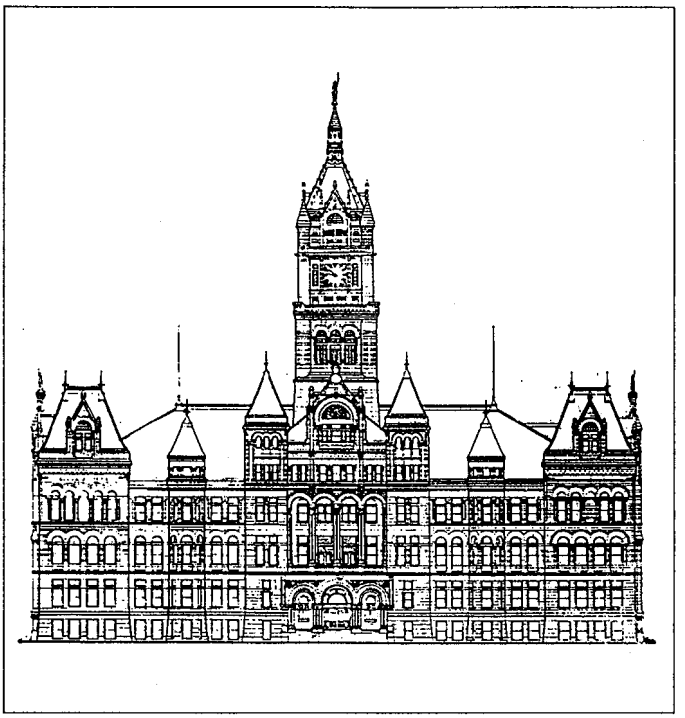

FIGURE 19. City and County Building, Salt Lake City.

City & County Building, Salt Lake City

The City and County Building in Salt Lake City, Utah, constructed in 1894, is a highly ornamented unreinforced brick and sandstone structure. It measures 130 by 270 feet in plan, with five main floors and a 12-story clock tower. This landmark building, with its 240-foot-high tower, dominates the surrounding landscape (see Figures 19 and 20).

The seismic vulnerability of the structure is aggravated by its proximity to the Wasatch Fault Zone. The building has a record of damage from various earthquakes, the largest of which occurred in 1934 with a Richter magnitude of about 6.1.

The building was constructed of unreinforced masonry and sandstone masonry walls with thicknesses of up to 24 inches for interior walls and 36 inches for exterior walls. Foundations are a combination of sandstone plinth and concrete footings. The first and fourth floors are framed with timber beams and planked with either wood floors or concrete topping. The second and third floors are framed with steel beams supporting shallow brick arches, which are covered with stone fill and a concrete topping. Anchorage between floors and walls is minimal. The roof framing is a steeply pitched timber system.

A number of alternative rehabilitation systems for the building were studied in the mid-1980s. The conventional systems envisioned utilizing concrete shear walls, which required the removal of costly architectural finishes such as oak wainscoting and ornamental plaster. In order to minimize the destruction and replacement of finishes, a base isolation system was selected for the retrofit.

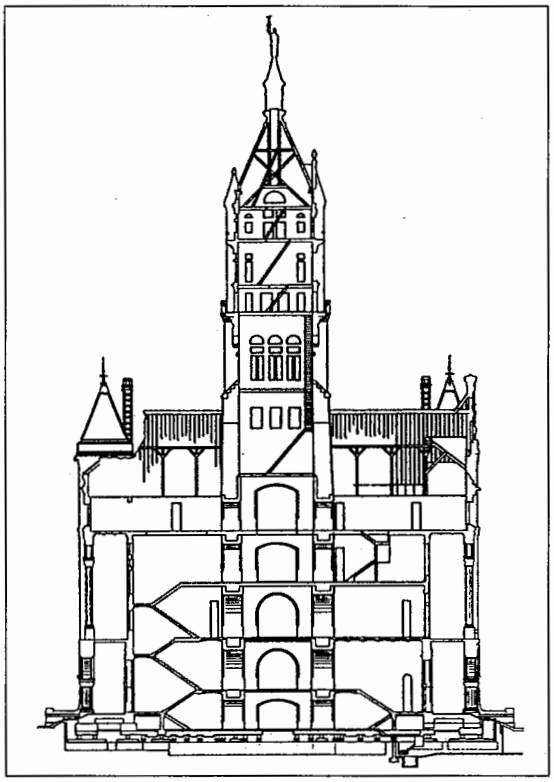

FIGURE 20. Section view of the City and County Building, Salt Lake City.

A site-specific response spectra indicated a free-field ground acceleration of 0.20 g, with a site period of 1.0 to 1.4 seconds. A base-isolation system with an effective fundamental period of 2.0 seconds reduced the base design shear to 0.07 g.

The base-isolation solution required the replacement of the first floor with a concrete slab on a metal deck supported by steel beams. Partial replacement of other floor diaphragms was required to provide continuity. A plywood diaphragm was installed at the ceiling level of the fourth floor, in lieu of a roof diaphragm, to support the tops of the masonry walls. Below the first floor, the masonry walls were "clinched" between pairs of reinforced concrete side beams, tied together by regular cross beams and ducted post-tension rods.

Once the concrete side beams were installed, portions of the masonry walls and plinth under the cross beams were removed, permitting the installation of the base isolators (see Figure 21). Four hundred and forty-seven isolation bearings were placed under the building. The isolators were preloaded to minimize axial shortening of the bearings. After installation, the

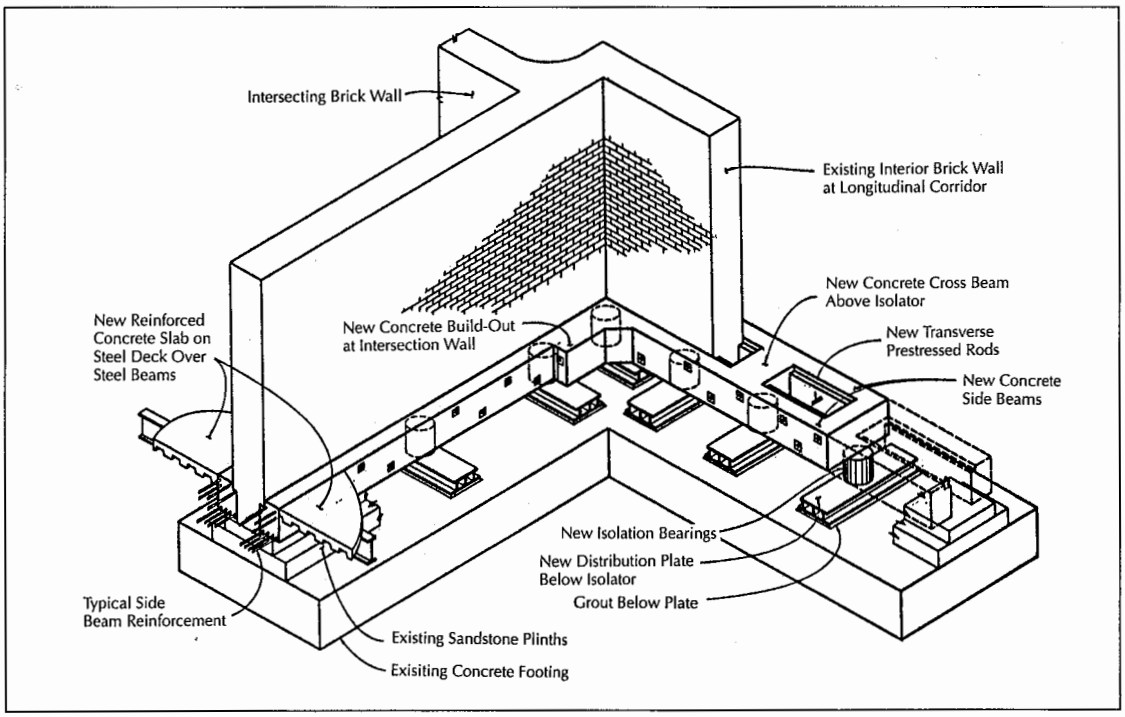

FIGURE 21. An isometric cutaway at an interior wall intersection (Salt Lake City).

balances of masonry walls and plinth stone were removed, permitting the isolators to translate freely.

The computer model for this project combined a linear elastic superstructure with the non-linear base-isolation elements. Shell elements were used to represent the interior masonry walls, and beam and column elements were used to represent the highly fenestrated exterior walls. Diaphragms were modeled using elastic properties that corresponded to the existing or proposed concrete toppings. Information on existing material properties was based on in-plane shear tests and prism test results.

The computer analysis consisted of a response spectrum analysis using a non-linear spectrum to account for the behavior of the bearings. This analysis yielded approximate internal forces and displacements for use in wall and diaphragm checks. The output of this analysis was the basis for the design of the structural elements.

To validate the response spectra analysis and the design, a non-linear time history analysis was done using the complete elastic superstructure model on top of the non-linear bearing elements.

The base isolators were bid on the basis of a performance specification. The low bid provided lead rubber bearings. The seismic rehabilitation was completed in 1989. Cost of the project was $6 million for the structural work, $30 million overall.

Oakland City Hall

The Oakland City Hall is an 18-story structure with full basement. Completed in 1914, it is the first high-rise governmental building in the United States. The structure is steel frame with unreinforced masonry infill. Cladding and ornamentations are terra cotta. A ten-story office tower rises above a three-story podium that houses a central rotunda, council chambers, and administrative offices of the mayor and city manager (see Figures 22 and 23). Above the office tower a two-story transfer system supports a 91-foot-high clock tower. The entire structure rises 324 feet above the street. The building is on the National Register of Historic Places.

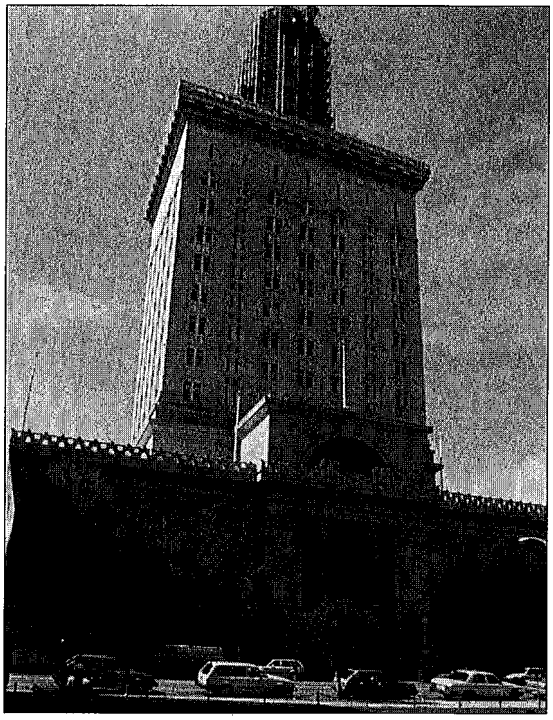

FIGURE 22. Photograph of the Oakland City Hall.

The building suffered extensive damage during the Loma Prieta Earthquake in 1989 and was declared unsafe for occupancy. After numerous studies, which were reviewed by federal and local agencies, it was decided that the building could be rehabilitated using base isolation.

The need to preserve the historic fabric of the building made the direct application of the UBC unfeasible. Rather, a "performance-based" approach was developed, which would protect life-safety during large earthquakes. The design of the superstructure was based on a site-specific response spectrum with an earthquake-return period of 475 years (the same as required by the UBC). The maximum credible earthquake, used to check the stability of the isolation system, corresponded to a magnitude 7.0 event on the nearby Hayward Fault.

The anticipated performance of the retrofitted building permits some minor yielding in the new steel bracing of the office tower. However, any damage caused by such low magnitude movement would not require repairs; and masonry elements would be expected to expe-

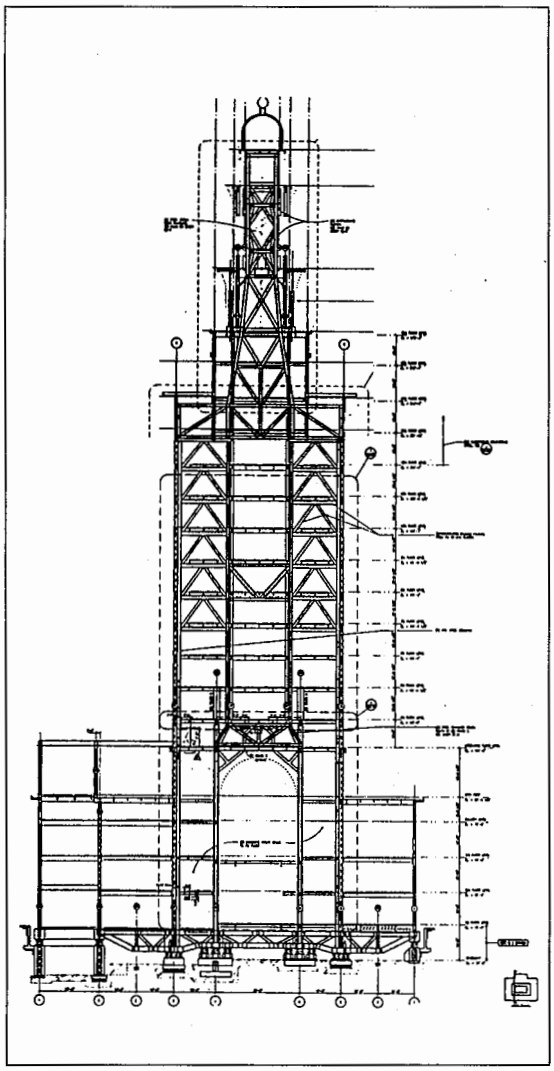

FIGURE 23. Section view of the Oakland City Hall.

rience minor cracks, which would be repairable.

The seismic upgrade called for a braced steel frame in the clock tower, which would resist the entire tributary load without dependence on the existing masonry. Interstory drift of the clocktower then would be limited to 0.008 times story height for the design earthquake.

The clocktower braced frames are supported on a series of one-story deep transfer trusses spanning 62 feet to new steel columns that extend to the basement of the structure. A new diagonally braced horizontal diaphragm transmits clocktower seismic shear forces to the perimeter walls of the office tower. The existing office tower exterior masonry walls are used to resist lateral loads in combination with new vertical steel braced frames, where needed. Stiffness compatibility between the two systems was determined by extensive in-situ dynamic testing and finite element analyses to determine the strength and stiffness of the masonry elements. Ultimate shear strains of the unreinforced masonry walls were limited to 0.001 in/in.

Below the seventh level of the office tower and podium, new concrete shear walls were introduced to resist the lateral forces. The concrete shear walls terminate on new 8-½ foot deep steel "outrigger" trusses in the basement story, which distribute the overturning forces over a broad footprint (see Figure 24). Horizontal steel braces form a diaphragm below the first floor to deliver the lateral loads to a system of 111 lead-rubber isolation bearings. These iso-

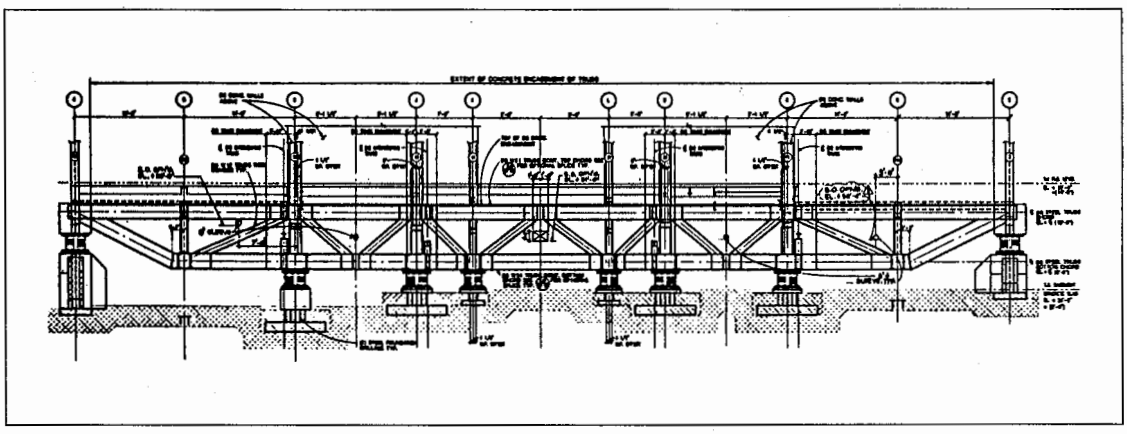

FIGURE 24. Truss section at the Oakland City Hall.

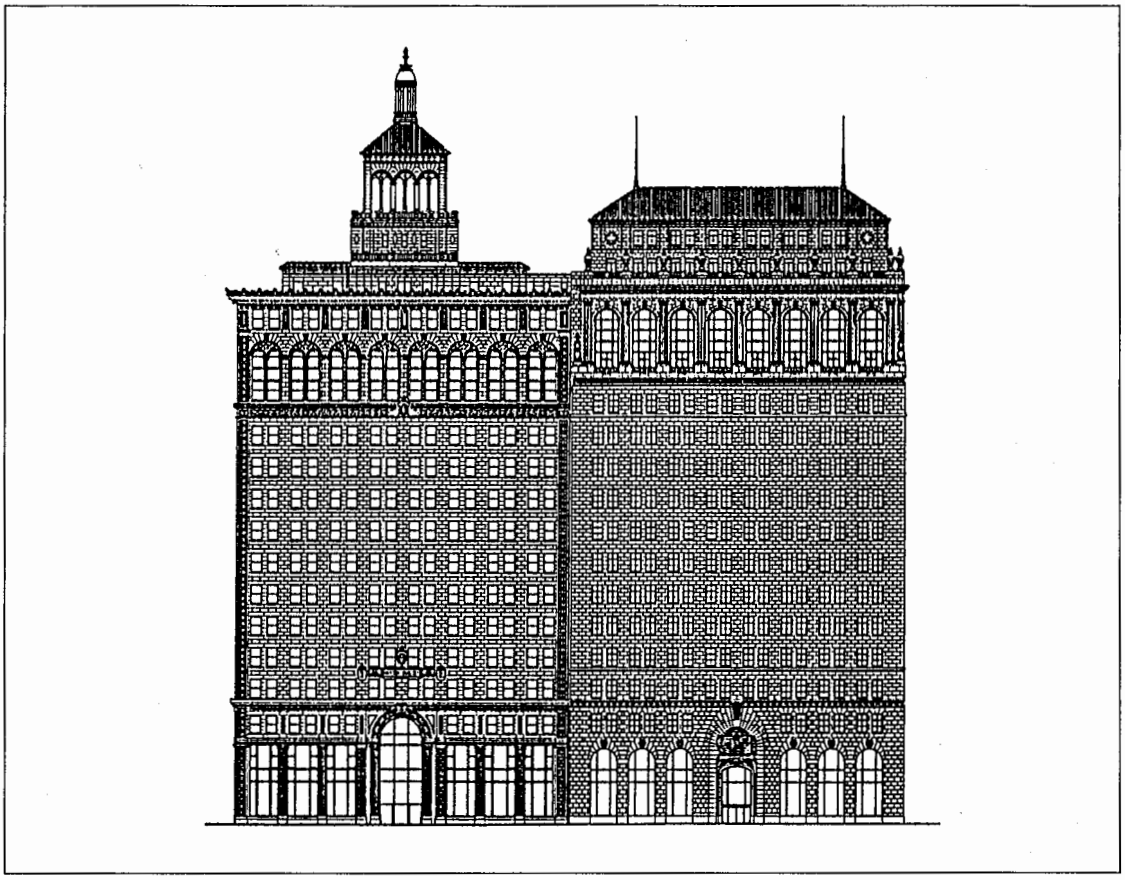

FIGURE 25. North elevation of the PG&E Market Street complex.

lators rest on steel/concrete pedestals that are supported on the existing mat foundation. A gap around the building permits the building to move 17 inches during a magnitude 7.0 event on the Hayward Fault.

In order to assess the seismic response of the building, a series of dynamic modal analyses were performed. In order to verify the results of the modal response analyses, and to assess the effect of the isolator hysteretic behavior, non-linear time history analyses were done. The time history analyses utilized three sets of ground acceleration records that were modified to match the site-specific design spectrum.

Historic hollow clay tile interior finishes are being repaired through a combination of self-tapping anchors, metal lath, epoxy and cement plaster. Damaged exterior terra cotta cladding elements will be replaced using new pieces cast in molds formed from salvaged damaged pieces.

The construction is currently well under way, with completion scheduled for late 1994. The total construction cost is estimated to be $76 million, with structural rehabilitation costs of $19.7 million. The cost of the isolation bearings is $2 million.

Pacific Gas and Electric Company

The Pacific Gas and Electric Company (PG&E) complex occupies one block in the financial district of San Francisco. Located at 215/245 Market Street, the seven-building complex houses the company's corporate headquarters and the company's critical energy management system (see Figure 25).

The October 1989 Loma Prieta Earthquake caused significant disruption and some damage to the buildings within the Market Street complex. The bulk of damage was concentrated in the brittle terra cotta facade of the buildings.

In March 1992, PG&E commissioned the seismic retrofit of four of the seven buildings in the complex. This project included two 17-story buildings built in the 1920s, and one seven-story building and one 14-story building built in the 1940s. The seismic retrofit program calls for complete reconstruction, upgrading the buildings to modern Class A high-rise office standards.

The two 17-story office buildings are historic landmarks. The structures of each are similar: structural steel frames, concrete slabs and unreinforced masonry infill — all bearing on a timber pile foundation system. The masonry infill is finished with glazed terra cotta stone. The two buildings are "L" shaped in plan and form a larger "U" due to their adjacency.

The 1940 annex buildings were constructed using structural steel frames and concrete floor systems with reinforced concrete infill walls as opposed to the unreinforced masonry of their predecessors. The 14-story PG&E annex was designed to resist earthquake forces using concrete wall elements in conjunction with the steel frame. The building supports a terra cotta finish identical to the original high-rise.

PG&E developed seismic performance goals greater than current code standards for the proposed seismic retrofit because this complex houses critical energy management systems. These goals express the owner's expectations for seismic performance after a major earthquake.

The goals that were developed to minimize risk to life were:

- Maintain building stability;
- Minimize falling hazards; and,
- Ensure survival of life-safety systems, stairwells, and egress routes.

The goals to minimize disruption of normal services and operations were:

- Ensure immediate access to critical facilities;
- Maximum of four weeks to restore full operation; and,
- Anticipate an extended period of non-disruptive repair of architectural and minor electrical/mechanical items.

Other criteria and physical constraints affecting the retrofit were:

- Maintain the existing historic character and details;
- Maintain the energy management system facility without interruption;
- Soft soil site;
- Limited separation between adjacent buildings;
- Limited capacity of existing foundations;
- Limited capacity of existing steel frame; and,
- Brittle terra cotta facades.

Reinforced concrete was selected as the material for strengthening the existing steel frame. It was chosen primarily for its ability to achieve a high lateral stiffness that is compatible with the existing masonry and terra cotta materials. The structural system is a dual frame-wall system that is detailed to deform in a ductile manner.

The frame-wall system is located in plan around the inner court walls of the complex and forms a high-rise "U" configuration (see Figures 26, 27 and 28). This configuration provides optimal distribution of overturning forces from the two interconnected buildings to their respective foundations. Additional wall-frames are located at the ends and mid-portion of the main U shape, serving primarily to further distribute overturning forces to the foundation. The location of these new walls was determined by where they would have the least impact on the historic features of the buildings. The inner court walls were not clad with the historically sensitive terra cotta system used for the street side facades.

The re-entrant corners of the wall-frame system include a rectangular core around existing elevator shafts attached to a triangular "pylon" element. The purpose of the pylon is to provide a load path for diaphragm forces at the re-entrant corners of the building without crossing the existing elevator shafts.

The entire wall-frame system is supported along its base by a three- to five-story-high wall-beam that stiffens the lower stories, thus eliminating the soft story configuration present in the existing building.

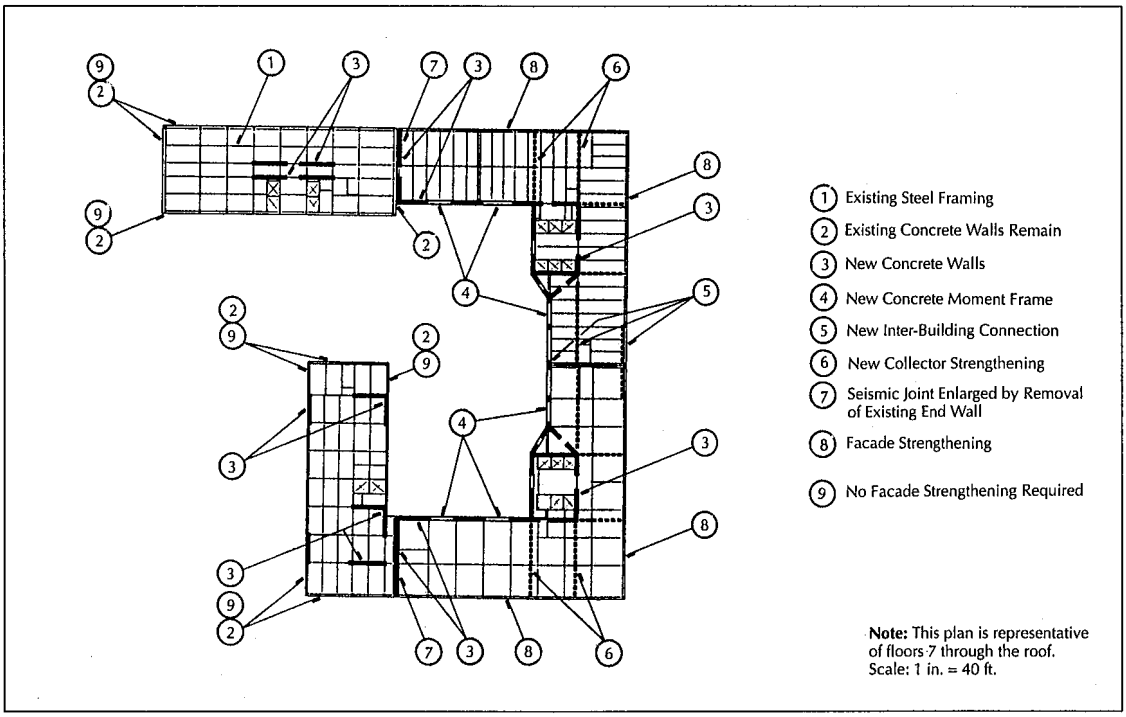

FIGURE 26. Upper level framing plan for the PG&E Market Street complex.

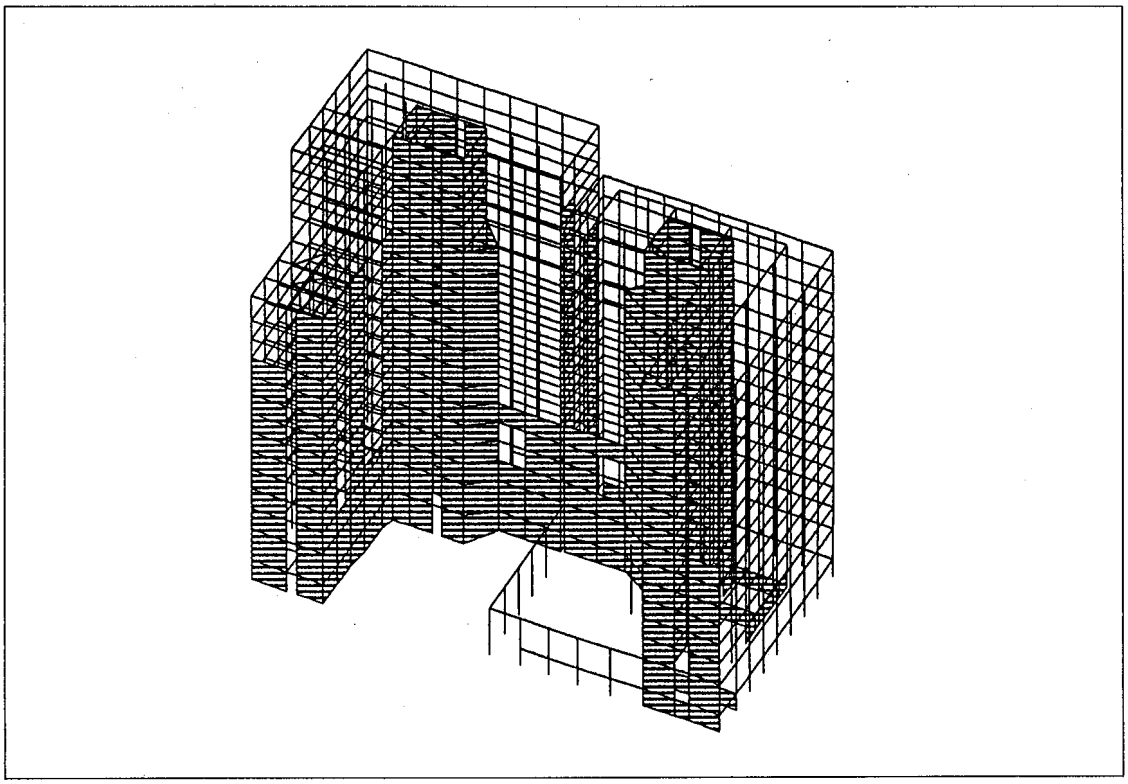

FIGURE 27. A view of the seismic retrofitted PG&E Market Street complex, looking towards the northwest.

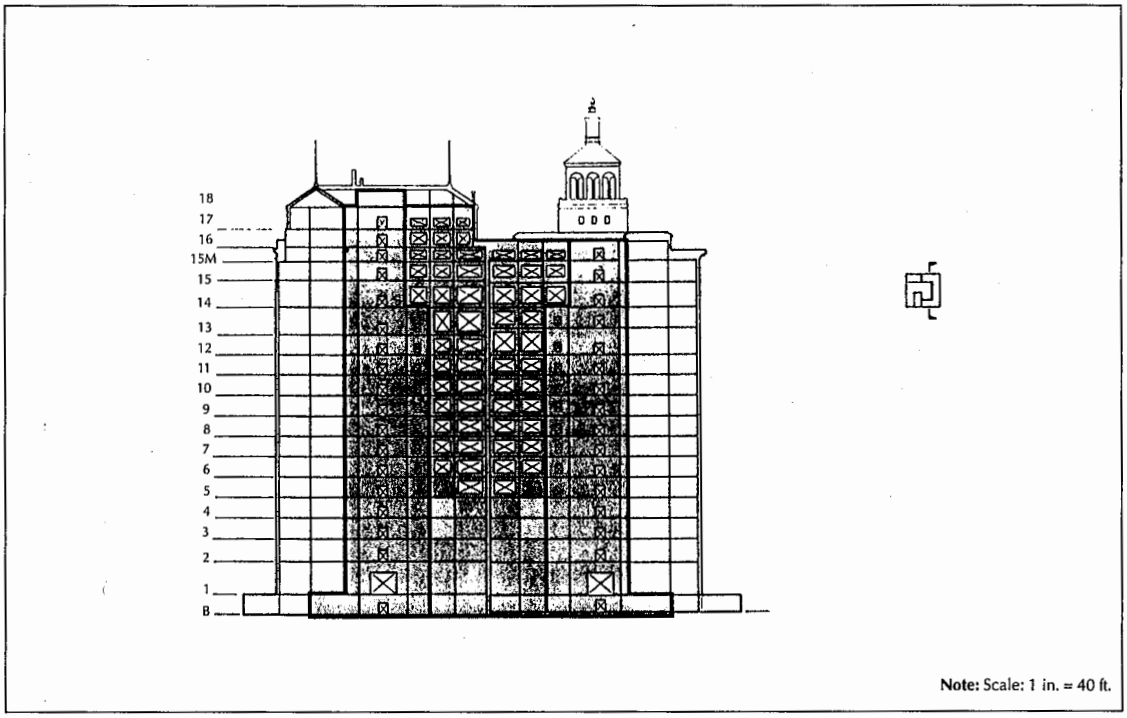

FIGURE 28. South court wall elevation of the PG&E Market Street complex.

The existing floor diaphragms were strengthened by introducing "collector" elements that tie the extreme ends of the building to the new wall-frame system. Separations between the buildings were enlarged by various means in order to minimize the possibility of pounding.

Analysis showed that the masonry/terra cotta piers would begin cracking at drift ratios of about 0.001, whereas a major earthquake would cause drift ratios of between 0.006 and 0.01. A solution that consists of mounting new terra cotta replicas of the originals to a precast concrete panel would have resulted in a loss of historic status for the buildings and, therefore, was not acceptable.

The method devised for isolating the masonry-terra cotta piers from large earthquake deformations was to reinforce the masonry from the inside of the building with a light reinforced concrete skin, and "articulating" the facade by cutting a thin horizontal joint completely through the terra cotta, masonry and concrete at the top and bottom of each pier. The articulation joints allow interstory deformations without inducing the large diagonal tension and compression forces into the masonry piers (see Figures 29 and 30).

An analysis program was established as follows:

- Develop three-dimensional finite element models of each building — modeling flexible foundations, flexible diaphragms, existing structural elements and new structural elements;
- Perform parametric dynamic response spectrum analysis of each building — using unreduced maximum probable and maximum credible site-specific earthquake spectra, and studying the influence of assumptions such as pile stiffness and cracked section properties on forces and displacements;
- Using the detailed three-dimensional finite element models as well as a linear elastic based computer program, perform an approximate non-linear incremental static push-over analysis to collapse (the "capacity-spectrum" approach was used to determine the peak displacements); and,

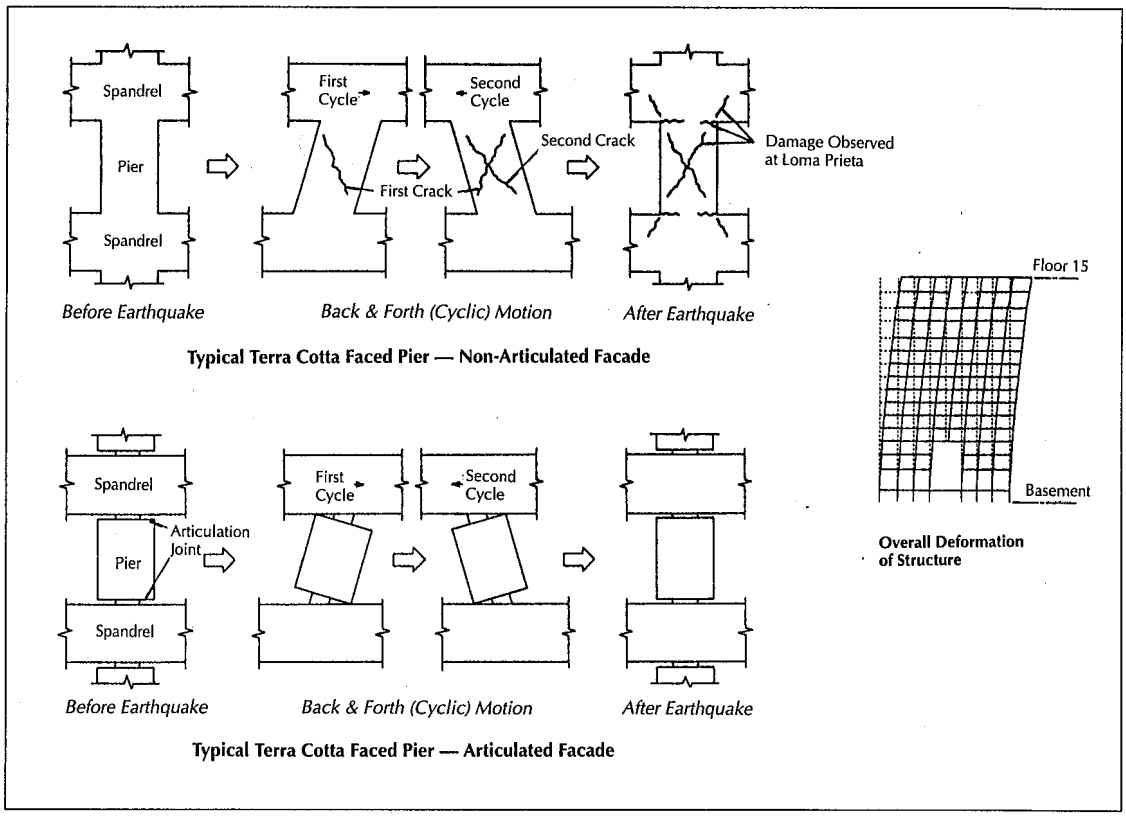

FIGURE 29. Seismic response of the facade for the PG&E Market Street complex.

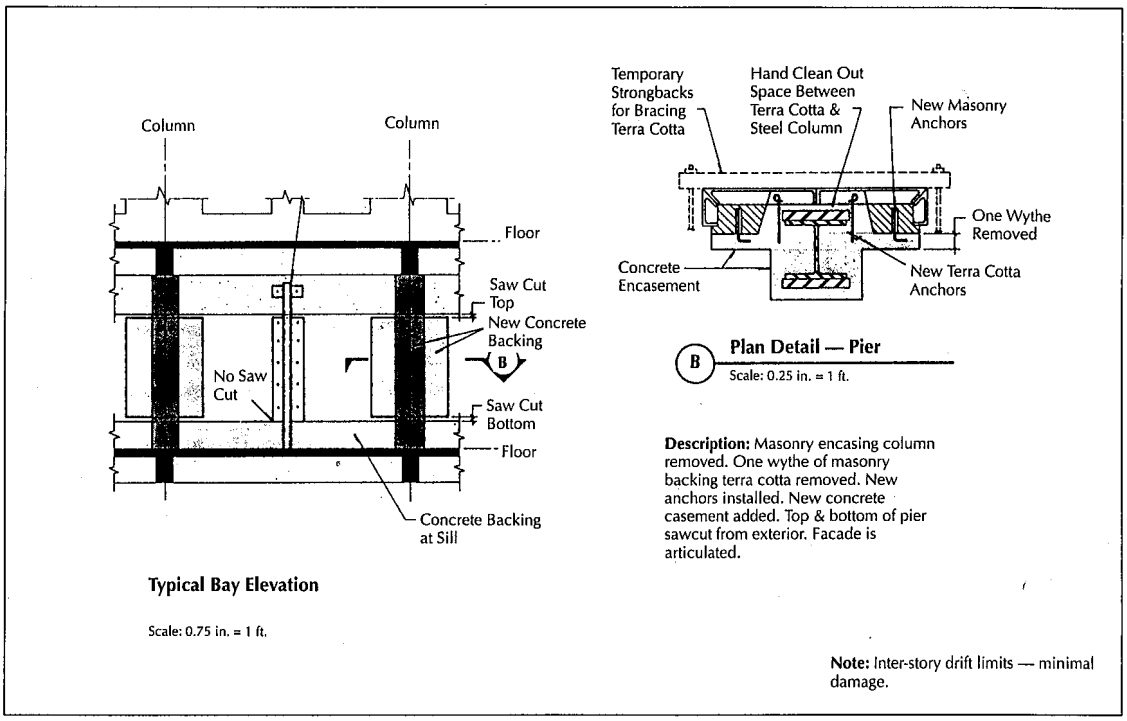

FIGURE 30. Articulated facade details for the PG&E Market Street complex.

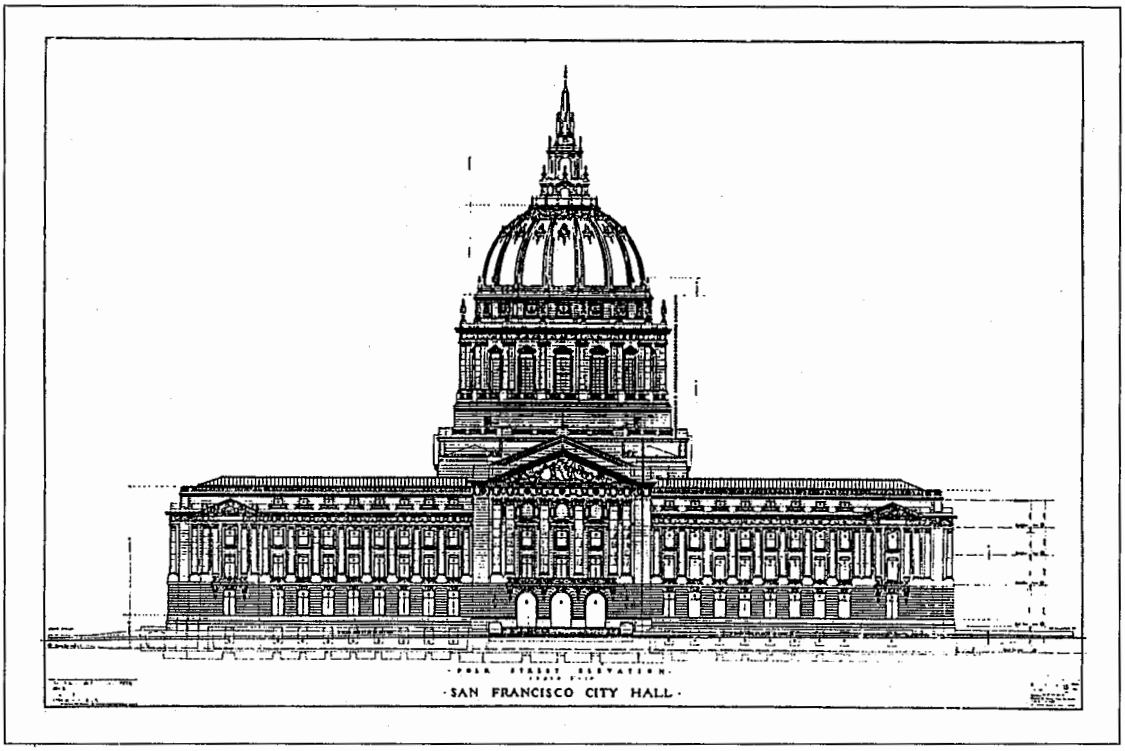

FIGURE 31. Elevation of San Francisco City Hall.

- Determine moment-curvature relations for individual members so that post-elastic deformations could be equated to compressive and tensile strains in the concrete/steel components (this analysis confirmed that the structure could undergo the maximum projected displacement with limited structural damage).

The seismic retrofit of the PG&E Market Street complex is a special case of seismic hazard mitigation. The owner's seismic performance goals surpassed to a significant degree what is normally expected of a structure exposed to a major earthquake. The use of approximate non-linear analysis techniques on structures of this scale is appropriate given the nature of the owner's expectations.

San Francisco City Hall

The San Francisco City Hall is located in the San Francisco Civic Center National Historic Landmark District. Designed in 1913, it replaced the city hall building that was destroyed by the 1906 San Francisco Earthquake.

The five-story building with its rectangular plan occupies approximately two city blocks with dimensions of 309 by 408 feet (see Figures 31 and 32). Its landmark dome rises approximately 300 feet above a central circulation area known as the rotunda. The main entrances to the building open into the rotunda area, which is ornately finished by marble, limestone and cast plaster. The exterior of the building is clad with a facade of ornately detailed granite from the foothills of the Sierra Mountains.

The building structure is composed of a complete steel frame with reinforced concrete slabs. The exterior granite facing is backed with unreinforced brick masonry that was laid-up integrally with the granite. Many of the infill partition walls were constructed with hollow clay tiles covered with cement plaster. The dome is a multi-tiered steel structure supported on four steel column towers located at corners of the rotunda. The exterior dome trusses were infilled with hollow clay tiles covered with lead and copper roofing.

The building was damaged during the 1989 Loma Prieta Earthquake. The damage was con-

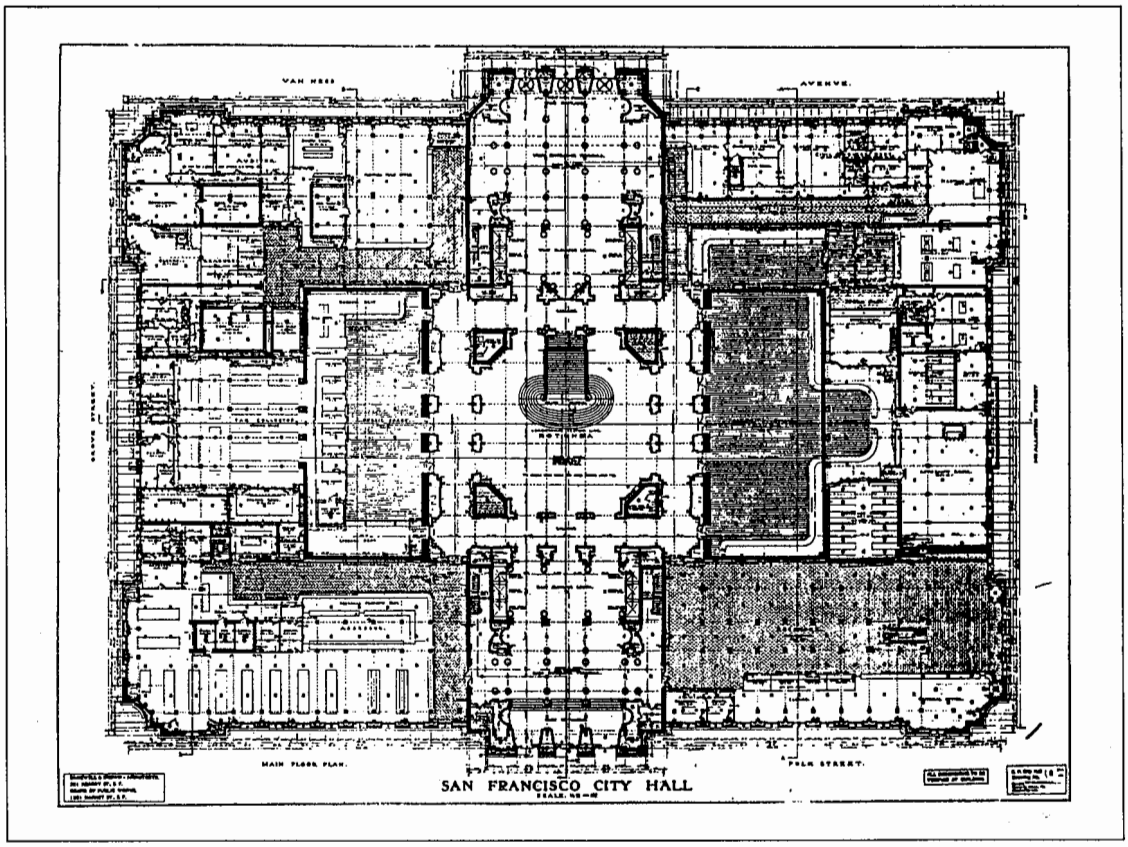

FIGURE 32. Plan of San Francisco City Hall.

centrated in the hollow clay tile and unreinforced masonry walls, and in concrete slabs.

The lateral load resisting system of the city hall building is composed of three major components:

- The unreinforced brick masonry walls;
- The hollow clay tile infill walls; and,
- The steel frame.

The extent of the damage to, and the required repair of, the building was evaluated in relation to requirements of the *San Francisco Building Code* (SFBC) Section 104 for seismic upgrade of the building structure.[4] The extent of the damage and the number of elements requiring repair exceed the threshold measure of a 30 percent tributary area to those affected vertical and lateral load resisting elements. Therefore, seismic upgrading of the structure to meet the current SFBC requirements was warranted.

Dynamic structural analyses were performed that acknowledged the complex interaction between lateral load resisting materials and systems as well as the interaction between the building and the dome. Load versus deformation curves were developed to not only investigate the interaction and response of the unreinforced masonry and hollow clay tile walls, but also to correlate the observed and calculated damage resulting from the Loma Prieta Earthquake.

The seismic strengthening will involve repairs to damaged unreinforced masonry and hollow clay tile walls and the construction of supplementary load resisting systems to stiffen and strengthen the building to withstand severe earthquakes without collapse. The supplemental systems should have a minimal impact on the building's historic aspects. Both fixed-base and base-isolation solutions were studied. Base isolation was chosen because it offered the following advantages:

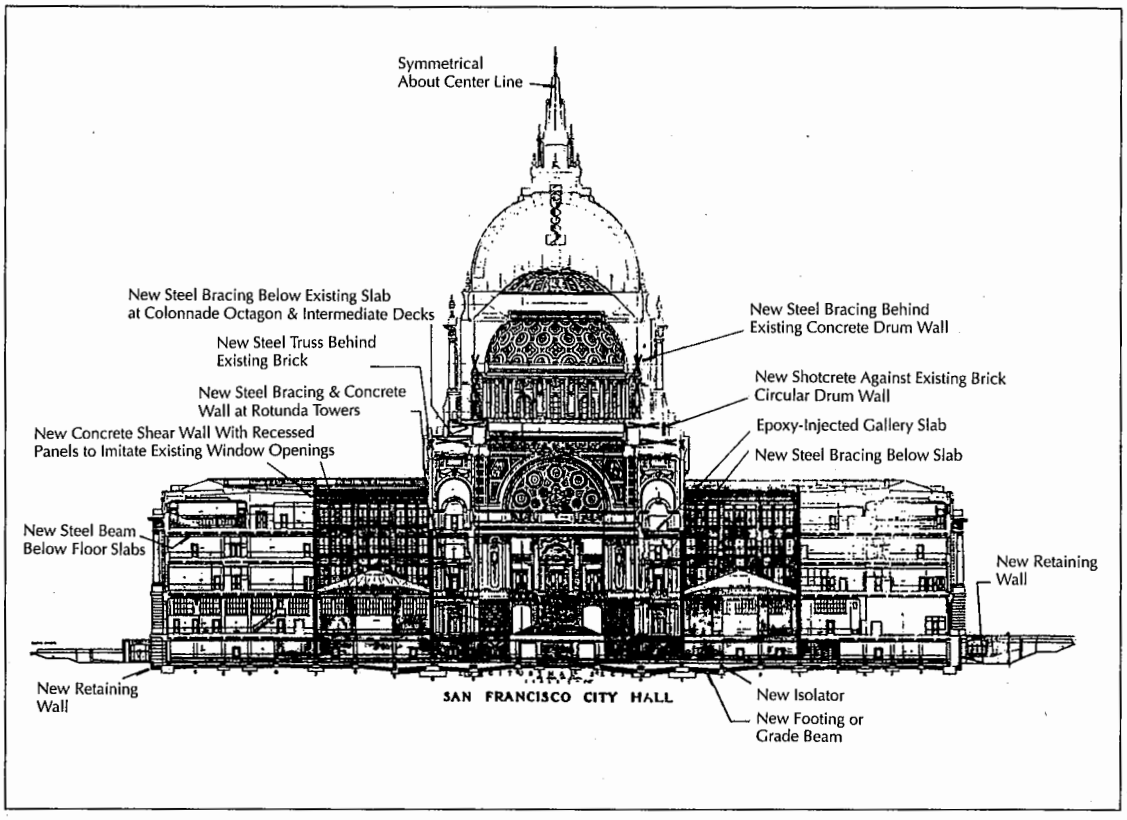

FIGURE 33. Longitudinal building section of San Francisco City Hall showing the base-isolation scheme.

- Significantly reduced seismic response;
- Minimum alteration of interior public spaces;
- Minimum disruption of important historic features;
- Minimum structural work on the dome;
- Minimum damage to architectural finishes during future earthquakes; and,
- Lower cost than a fixed-base solution.

A new ground floor constructed of steel framing supporting metal deck with concrete fill will be located above the isolators to distribute the lateral forces uniformly among the isolators (see Figure 33). New shear walls will be installed at the light courts to stiffen and strengthen the superstructure.

The seismic strengthening of this building is complex both technically and logistically because of the need to relocate personnel and functions now occupying the building. Completion of this project is scheduled for late 1997.

Conclusion

The case studies presented above serve to illustrate the diverse issues that must be taken into consideration in the seismic strengthening of existing buildings. Those examples are intended to demonstrate the great variety of solutions and methods that can be used as well as the complexity of analysis and detailing that may be necessary to accomplish the task.

It is hoped that as awareness of the need to seismically strengthen and preserve existing buildings increases, research and the creative ingenuity of the engineering profession will further enhance the palette of solutions and methods that can be used to accomplish this important task.

ACKNOWLEDGMENTS — *This article was first presented as part of the BSCES Structural Group's 1992 Fall Lecture Series. Dames & Moore performed the site-specific response spectra for the City and*

County Building in Salt Lake City. The Oakland City Hall Building was designed by the architectural firm of Palmer & Hornbostel. The San Francisco City Hall building was designed by the architectural firm of Bakewell and Brown. The structural analysis programs used for LLNL Building 111 were the ETABS three-dimensional finite element program for structures with rigid diaphragms and SAP90, a three-dimensional finite element program for modeling frames, shells and asolid elements.

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