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USE OF STEEL IN THE SEISMIC RETROFIT OF HISTORIC OAKLAND CITY HALL

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An Innovative Building–Then and Now

The 19-story Oakland City Hall was a building without precedent when it was completed in 1914. This landmark structure was the tallest building in the western United States, and the first of a genre of public buildings that attempted to combine the features of a modern highrise office tower with those of a traditional grand rotunda building. The resulting "tiered" structure, which was only possible with the advent of riveted steel construction, features a broad "podium" base that houses the ceremonial rotunda, topped by a slender eleven story office building and a five story clock tower (See Figure 1). The perimeter framing is infilled with massive brick, granite, and ornamental terra cotta.

Figure 1: Oakland City Hall

The multiple setbacks in the width of this building gave rise to an ungainly transfer of loading from top to bottom of the 324 feet high building. This difficult transition clearly posed a daunting challenge to the original structural engineer — each successively narrower portion of the building required massive transfer trusses and girders to spread the loads to the broader podium portion below. This challenge was matched by that posed to the seismic retrofit engineer, who had to devise a stiff new structural steel "skeleton" inside the historic building to continuously transfer lateral loads from the top of the clock tower down to the new seismic isolators atop the old concrete mat foundation. Figure 2 shows a cross section of the retrofitted building.

Seismically retrofitting and repairing the building was necessary after the building was heavily damaged during the Loma Prieta earthquake of October 17, 1989. When the retrofit is complete in early 1995, the Oakland City Hall will again set a precedent since it will be the tallest seismically isolated building in the world.

Retrofit Design Approach and Criteria

Because of the archaic nature of the structure and materials, and the need to preserve the historic "fabric" of the building, seismic provisions of the Uniform Building Code (UBC) or other historic building codes could not be directly applied to this building. Therefore a "performance" design approach was used based on the field tested capacities of the infill wall materials and finite element analysis of the steel frame/infill masonry wall, using the unreduced seismic response of the base isolated superstructure. The "performance based" approach was used throughout the design phase. The governing criteria was life safety during a strong earthquake.

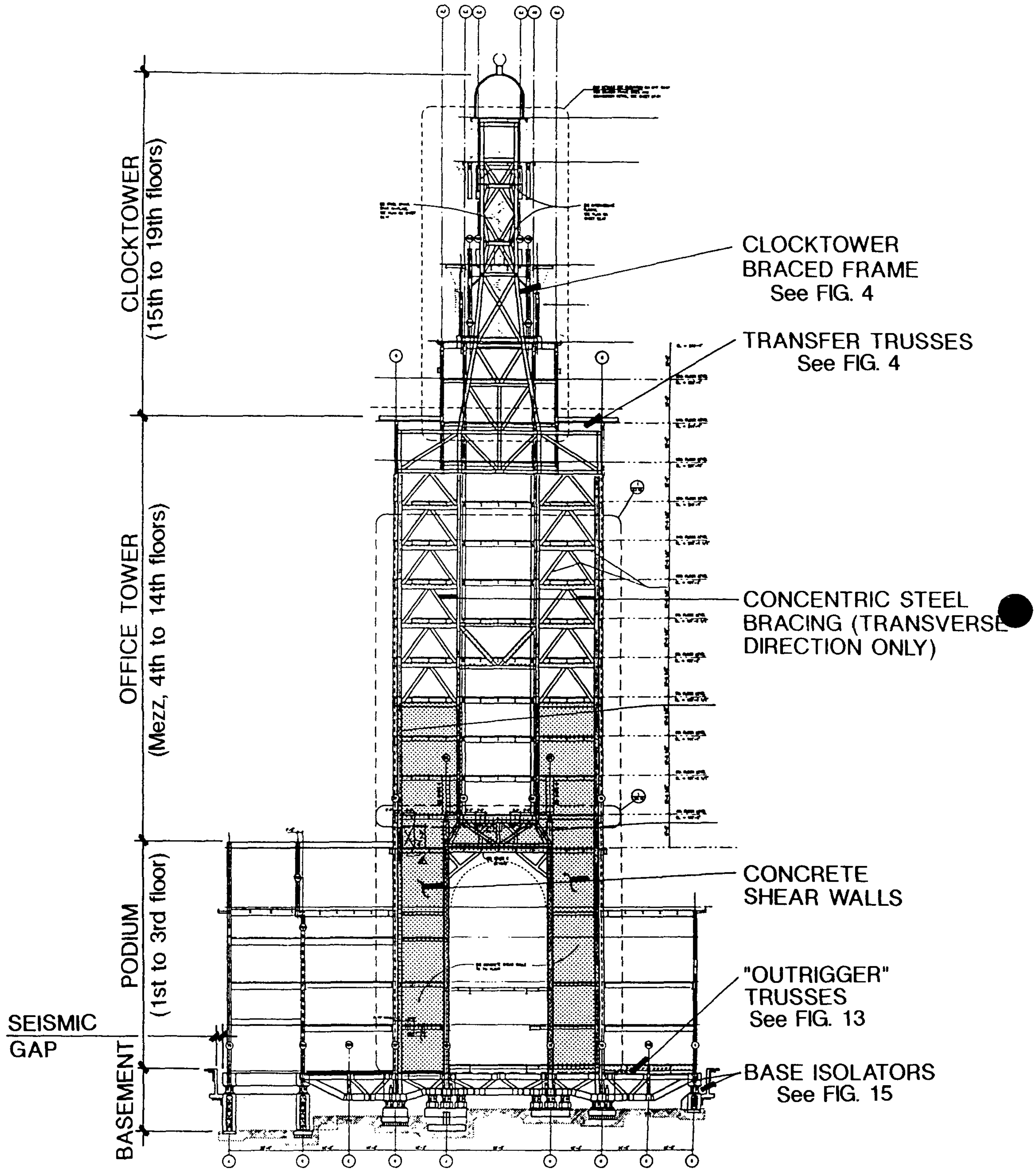


Figure 2: Structural Section of Retrofitted Building

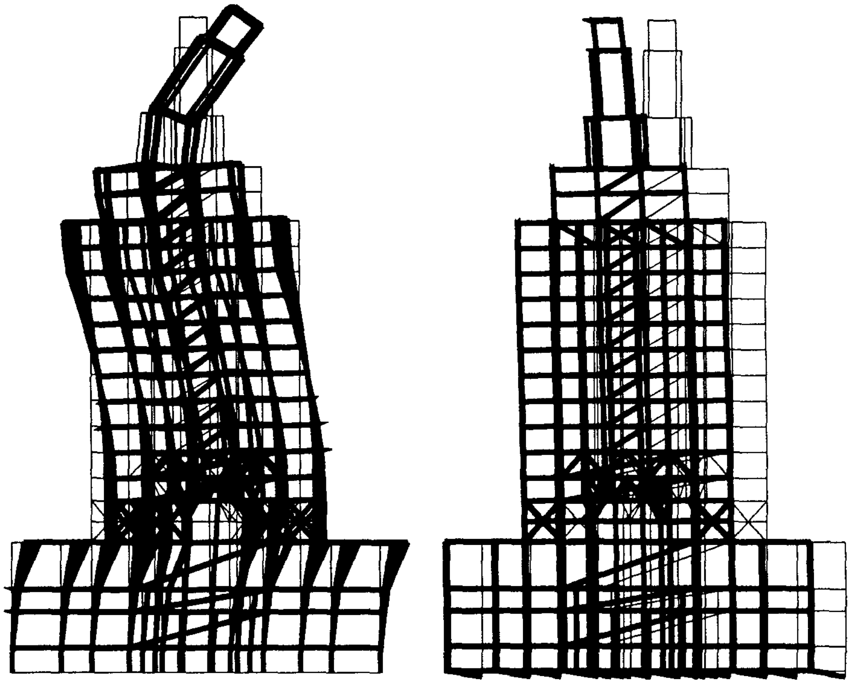
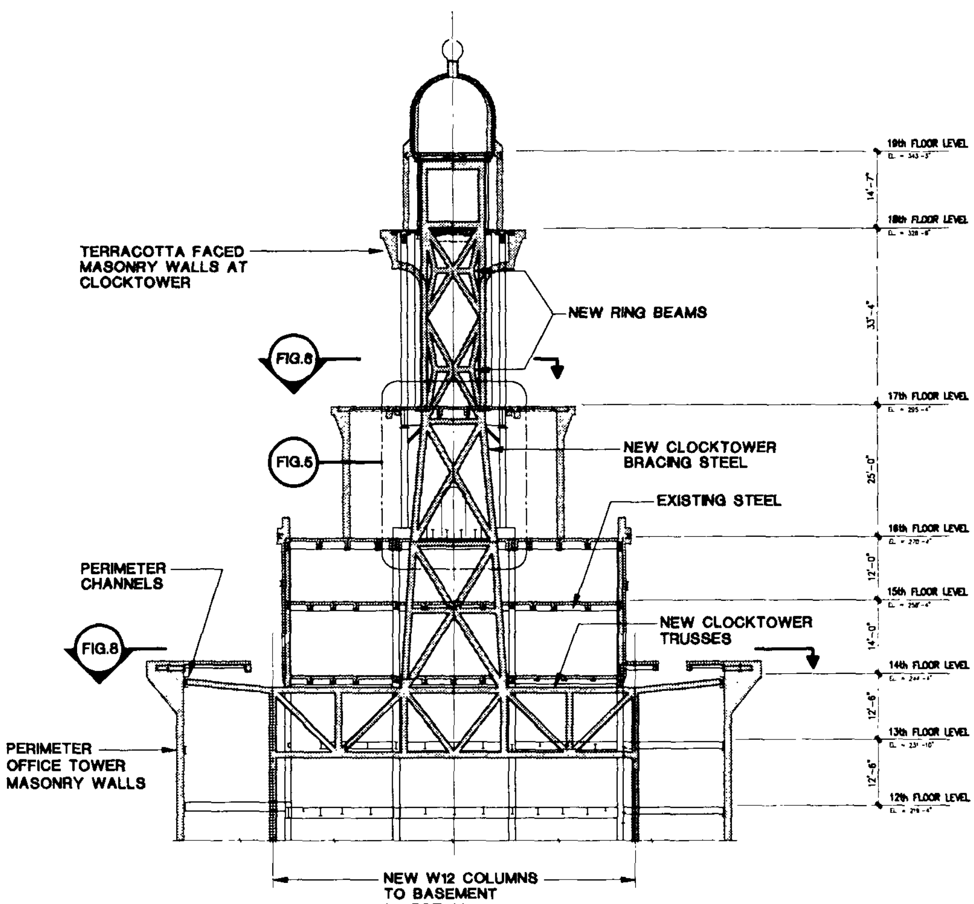


Figure 3: Building Seismic Response Comparison



<u>**CLOCKTOWER BRACING AND SUPPORT TRUSSES**</u>

Figure 4

Site specific response spectra were developed by Dames and Moore for the Oakland City Hall site. A probalistically derived "design basis" earthquake (DBE) with a 475 year return period, and a maximum credible earthquake (MCE) based on a Richter magnitude 7+ on the Hayward fault governed the design.

Seismic Structural Retrofit Concepts

After the Loma Prieta earthquake, both conventional fixed base and seismic isolation concepts were studied. The fixed base concepts involved adding both strength and lateral stiffness to the existing structure by adding concrete shear walls throughout the height of the building. This would have had major impact on the historic interior finishes of the building, and also would have induced higher seismic forces and corresponding larger displacements into the building structure than could be tolerated by the relatively brittle infill walls. Seismically isolating the building, on the other hand, reduced the ultimate seismic acceleration by a factor of more than 3. This significant reduction in lateral forces on the building superstructure results in reduction of building drift (lateral movement) which translates into a major reduction in future damage to the brittle archaic infill wall materials. A further benefit of base isolation is the reduction in the need for shear walls, enabling them to be limited to the central portion of the podium and office tower, thereby preserving historic interior finishes.

In order to base isolate the building, the superstructure had to be stiffened sufficiently so that its fundamental building period was separated from the isolation period by at least one second, to preclude the chance of dynamic resonance of the isolated building. The stiffening elements added to the building reduced the fundamental period of vibration from 1.56 seconds to 1.26 seconds, allowing seismic isolation to be feasible. Figure 3 shows the difference in the displaced shape of the structure comparing "fixed base" to an "isolated" retrofit design. The isolated building moves more like a rigid body on the isolators, reducing deformations and seismic forces on the superstructure.

Retrofit Structural Systems

Structural steel was used extensively in the retrofit design because of its versatility and strength. Special connection details were developed using slotted bolted erection connections with field welding to provide tolerance to accommodate as built dimensions and to aid in field erection and fit up of steel members.

Clock tower: The tall slender configuration of the clock tower and the "whipping" action at the top of the building during an earthquake make the clock tower particularly vulnerable to earthquake damage. This part of the building suffered the most damage during the Loma Prieta earthquake. Figure 4 shows new steel braced frame constructed of W12 wide flange sections erected inside the clock tower. The frame is designed to resist 100% of the lateral seismic forces in the clock tower and is sufficiently stiff to limit potential damage to the infill brick/terracotta walls, with drift limited to .008 times the clock tower height. The legs of the lower portion of the clock tower steel are inclined to accommodate the larger floor plan at the base of the clock tower and to spread out the high overturning loads. Figure 5 shows the sloped legs and bracing between the 16th and 17th floors.

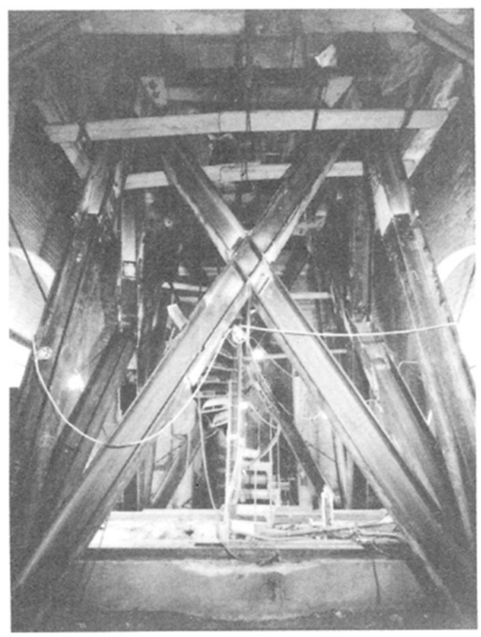
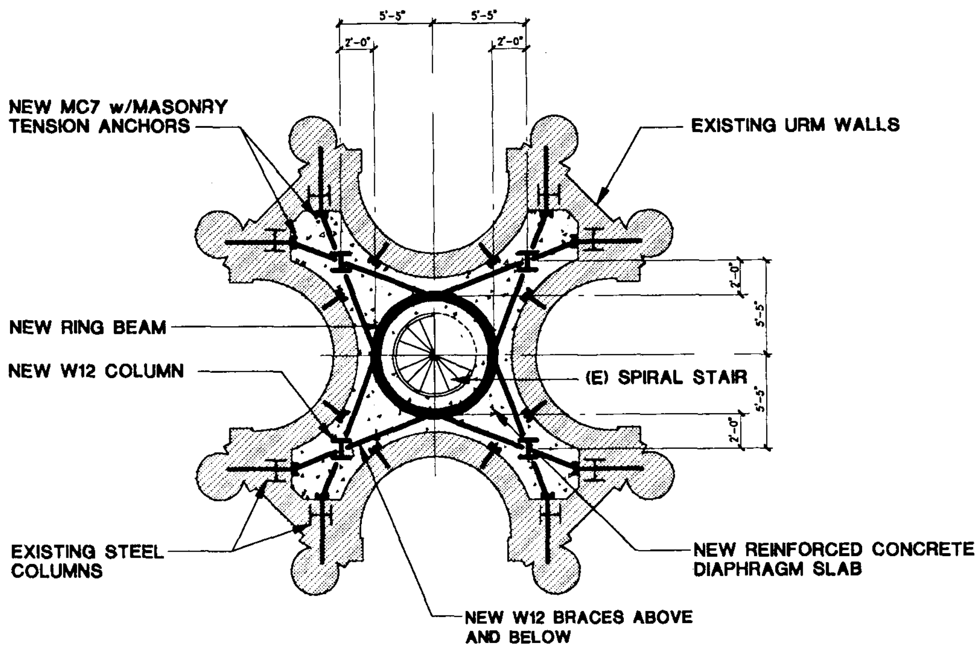


Figure 5: Clocktower Bracing at 16th Floor

The existing clock tower walls, which are concave in plan in the tall story between the 17th and 18th floors, presented a difficult problem for installation of new bracing in this confined space (see Figure 6). This was further complicated by the presence of an historic spiral stair in the center of the tower that could not be removed. The problem was solved by designing concentric "X" bracing on the four faces, with the intersection of each "X" pushed in to clear the concave walls, meeting at two new steel box shaped ring beams erected in segments around the spiral stair. A concrete floor diaphragm was added around each ring beam to further stiffen the ring beam and brace the existing tall unreinforced masonry walls.

The base of the new clock tower steel is supported on a system of six new interconnected one-story deep steel trusses constructed of field welded W12 steel sections spanning 63 feet across the office tower. The top chords



INTERMEDIATE FLOORS BETWEEN 17th & 18th FLOORS

Figure 6

are under the 14th floor and bottom chords under the 13th floor with vertical and diagonal truss web members penetrating through new openings in the 13th floor (see Figures 4 and 7). This floor is now a mechanical floor with new equipment located around the trusses. The trusses transfer overturning forces from the new clock tower bracing to eight new steel W12 columns that extend vertically through the building to new trusses in the basement. The 13th floor trusses and W12 columns were designed to limit drift in the clock tower caused by overturning, using the trusses to spread out the reactions from the clock tower.

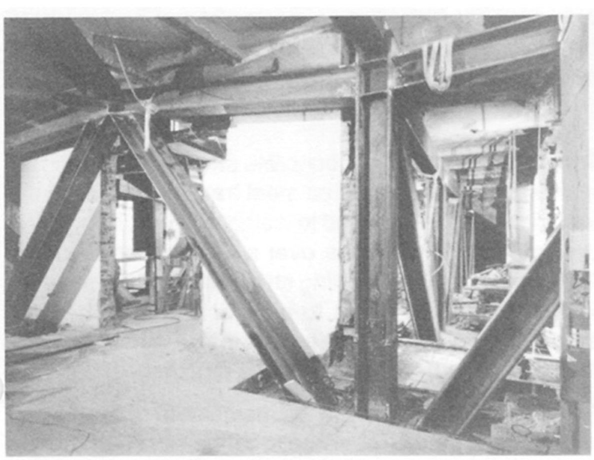
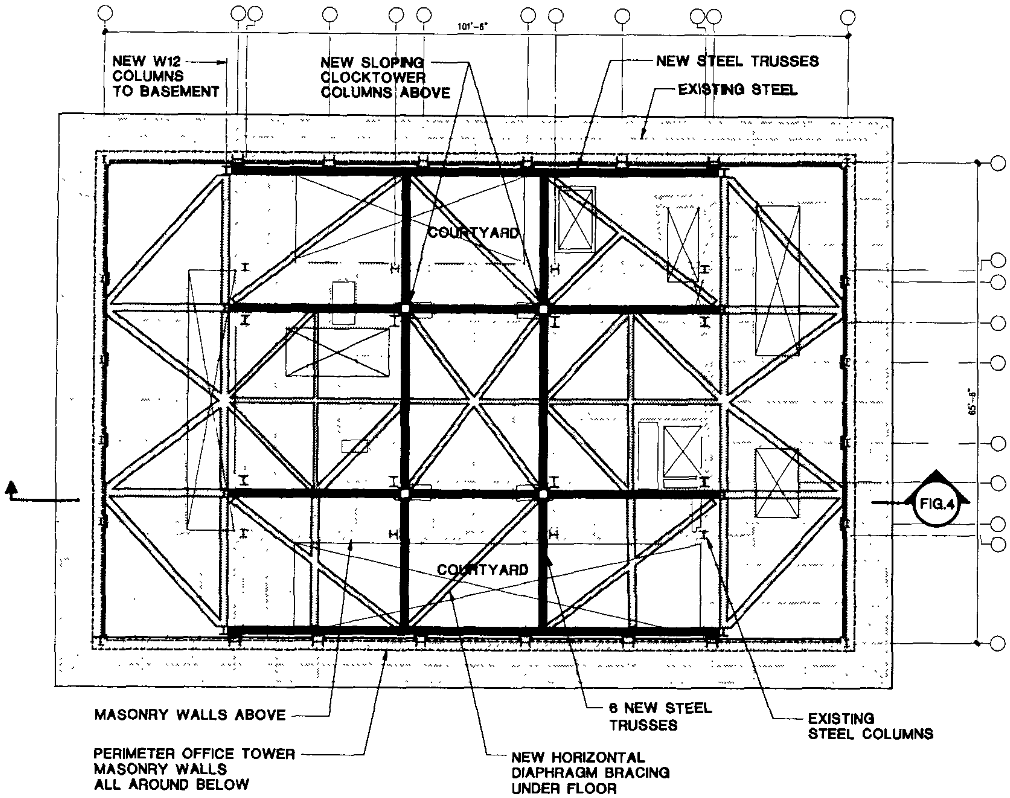


Figure 7: Clocktower Trusses at 13th Floor

Since the 14th floor and adjacent roof have many openings that are not at the same level, horizontal wide flange bracing members were added under the 14th floor in the same plane as the top chords of the trusses. These braces act as a diaphragm so that lateral seismic forces from the clock tower are delivered to the exterior existing office tower walls. See Figure 8 for a plan at the 14th floor showing the trusses, new W12 columns and horizontal bracing.

Office Tower: Lateral loads in the 10 story office tower are resisted by the existing steel frame/infill masonry perimeter walls in the longitudinal direction, and by a combination of the existing steel frame/infill masonry perimeter walls, and two lines of new steel concentric braced frames in the transverse direction. In order to assess the participation of the existing masonry infill for lateral resistance, in situ testing of the infill was performed in conjunction with finite element modeling (FEM) and analysis to determine the steel frame/infill masonry strength and stiffness properties. From the tests and the FEM analysis, it was determined that a shear strain limit of 0.1% would preclude severe cracking and stiffness degradation of the masonry infill. It was also determined that 100% of the longitudinal (north-south) lateral forces and 75% of the transverse (east-west) lateral forces could be resisted by the existing steel frame/infill walls. Two



14th FLOOR FRAMING PLAN

Figure 8

lines of concentric braced frames were added in the transverse direction to resist the remaining 25% in that direction. These braced frames utilize the same eight W12 columns that support the trusses under the clock tower, thus saving steel. These braced frames extend down to the 7th floor where they transition to concrete shear walls. On each braced frame line, the individual braced frames are coupled together with concentric braces at the 9th and 13th floors (see Figure 2).

An average of 24 tons or 5.5 pounds per square foot (PSF) of steel were added to each floor in the office tower portion of the building, including new steel framing for new floor openings and new collector beams. Existing structural steel averaged over 25 PSF.

Podium: The podium portion of the building (1st floor to 3rd floor) contains significant historic spaces, and required stiffening to seismically protect the historic hollow clay tile partitions. A system of new interior concrete shear walls, located in the core areas, was designed to extend down to new trusses in the basement. Existing 12 by 12 inch "H" shaped riveted steel columns are located at the vertical edges of the new shear walls. Steel plates up to 4 inches thick were welded to these columns to provide sufficient steel area to resist the added seismic overturning forces induced at the ends of the shear walls. The added plates create "box shaped" columns.

Above the 3rd floor, crescent shaped historic windows could not be closed in with concrete shear walls because of the need to light important interior 3rd floor spaces. To solve this problem, steel shear walls made from plates up to 2 inches thick were designed with half circle opening to accommodate the windows. These steel shear walls are set within and connected above and below to the concrete shear walls using shear studs and welded rebar dowels.

Basement: The podium concrete shear walls terminate on new 8 foot deep doubled steel transfer trusses in the basement. These are used to distribute the building overturning moment reactions over a broad base footprint. Trusses straddle the existing steel columns and are connected to them by welds to new jacking corbels. Concrete encases these trusses to provide additional stiffness and to tie the double trusses together.

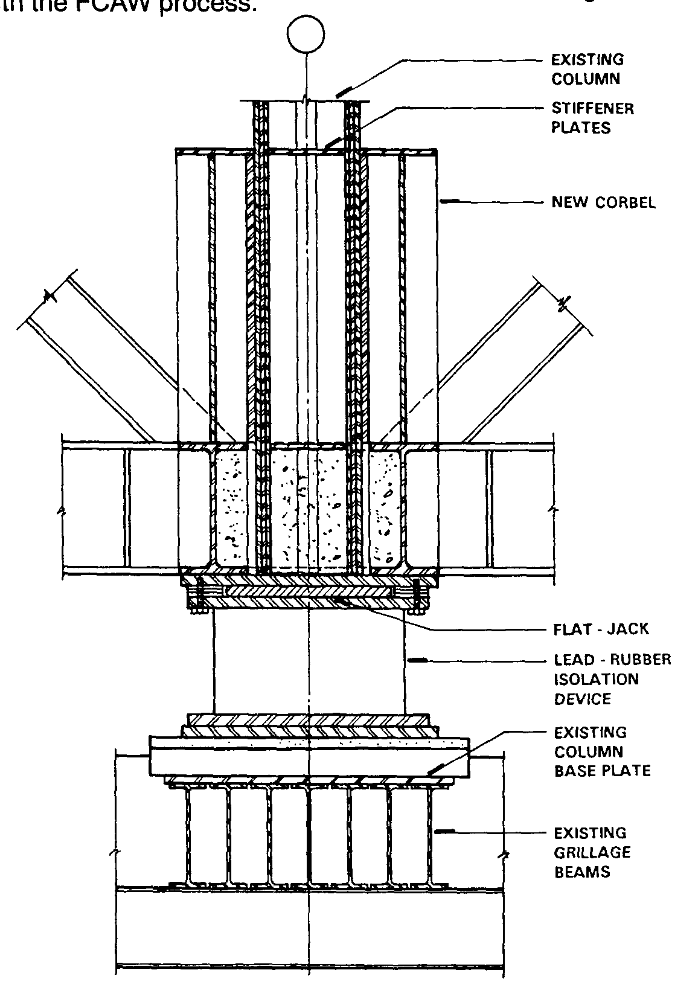
The retrofit design required the attachment of new, heavy vertical steel plates, or corbels, to the bases of the riveted columns to make it possible to lift the entire dead load reaction with hydraulic jacks as shown in Figure 9. Such lifting is required in order to cut out the bottom section of columns to install the isolators.

The weldability of each column was demonstrated by welding a 6 inch long piece of A36 bar to the riveted angle flange of the column and simply bending it over at a 90 degree angle. The test bar folded over in a ductile manner for each column with neither the weld nor the base metal cracking or failing.

Prior to attaching the corbels, fillet welds were added to connect the riveted column angles to the plates, using the FCAW process. This welding revealed an unexpected setback: the original fabricator had coated the original faying surfaces of the column components with a tar like substance prior to riveting them together. The welding heated the tar and caused it to expand into the new weld metal, forming unacceptable porosity in the weld. This problem was resolved by placing a small stringer pass at the seam using the less heat intensive SMAW process, then following through with the FCAW process.

Formation of lamellar tearing in the old column flanges was another problem. The propagation of this tearing, which occurred at two locations, stopped before it reached the first row of flange rivets. The tearing was repaired by backgouging and rewelding, and the problem was eliminated by resequencing the placement of the welds to minimize shrinkage stresses and by using stringer passes instead of "weaving."

A horizontal two way grid of paired A572 grade 50 W24 girders was used throughout the basement to tie the reinforced column bases together into a new diaphragm and to serve as jacking beams. The basement plan is shown in Figure 10. These girders were welded to the new corbels to provide lifting capacity and to flexurally stiffen the column bases against seismic rotation. After the erection of these girders, the columns were jacked to relieve their load, and their bases were cut and removed to make room for the new isolators. The rectangular bays created by the new girder grid were infilled with tubular steel diagonal bracing to provide in-plane



ISOLATION BEARING ASSEMBLY

Figure 9

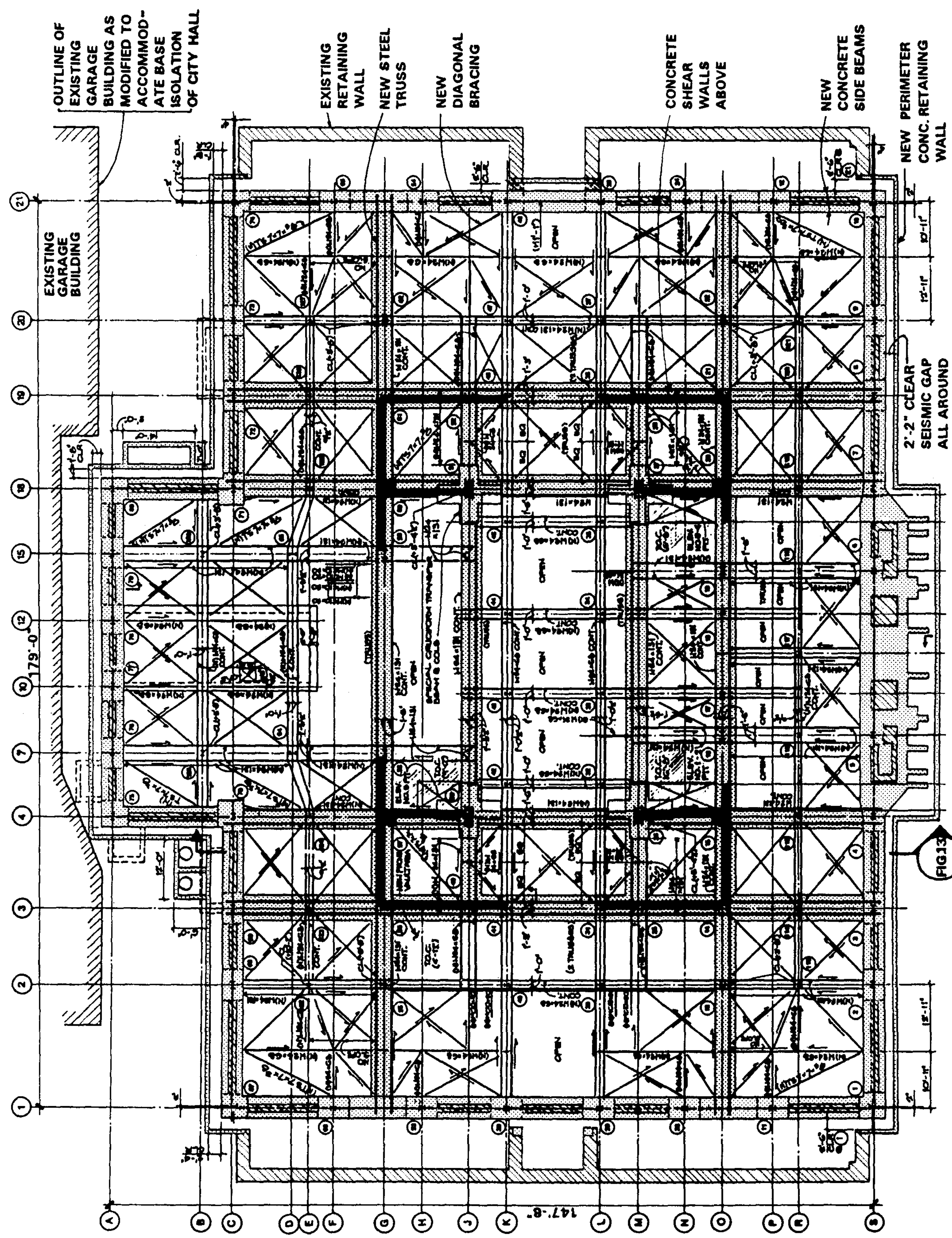
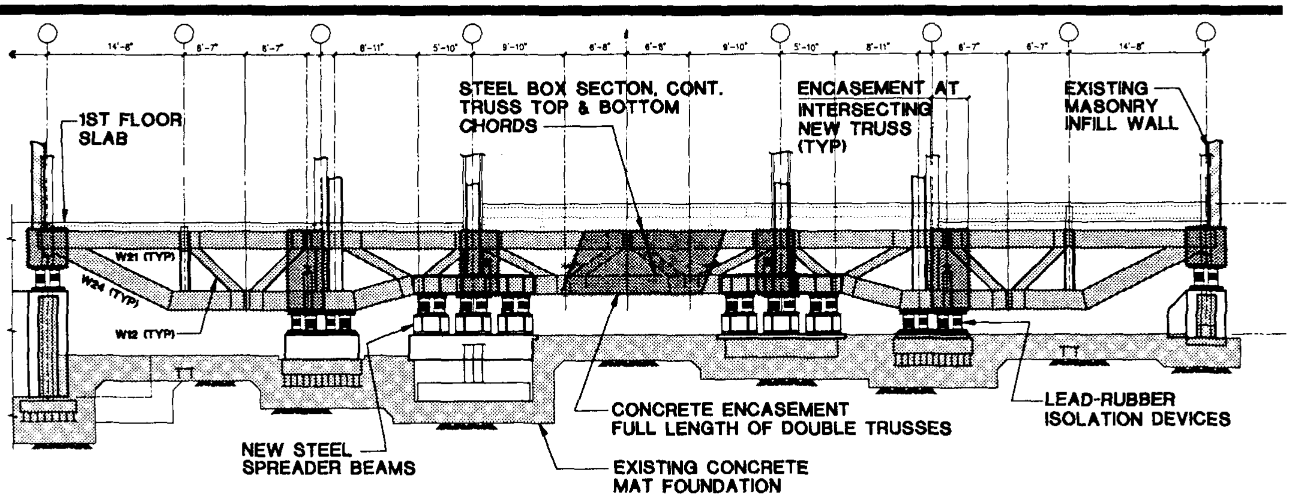


Figure 10: **NEW STEEL FRAMING PLAN ABOVE ISOLATORS IN BASEMENT**



OUTRIGGER TRUSS ELEVATION

Figure 13

stiffness for the new basement diaphragm. The new diaphragm of girders and tubular bracing slopes upward to the perimeter elements, which were positioned at a higher elevation to avoid the need for deep perimeter retaining walls.

Figure 11 shows modifications to the perimeter basement walls to accommodate a new continuous reinforced concrete tie beam (not shown) placed just above the perimeter isolators. This concrete beam encases the structural steel corbel attachments on the existing riveted columns. The ends of the outrigger trusses are connected to the concrete beam to help control uplift. (Refer to the section on uplift control below).

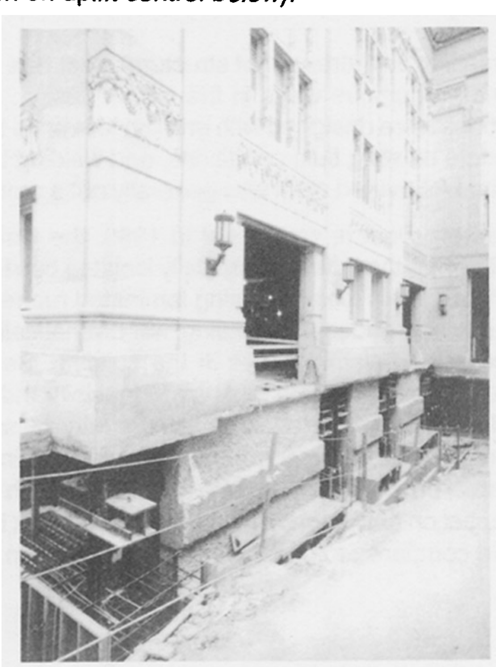


Figure 11: Modifications to Perimeter Walls

The W24 framing was not sufficient for lifting the bases of the four largest columns. These 24-inch square riveted box columns each had dead load reaction of approximately 4,000,000 lbs. In order to spread the load of each column over four large isolation bearings, a four legged "cruciform" welded box beam was fabricated using A572 grade 50 plates with thicknesses of up to 2.5 inches. The isolators beneath this assembly were supported by new box "spreader" beams, similar to the cruciform beams, which spread the 4,000,000 lb. reaction over the existing foundation. Figure 12 shows the jacking arrangement. Special measures were taken by the fabricator to minimize warpage of the connecting plates of the box beam elements during welding. Some warpage of the bearing surfaces was inevitable, and this was resolved using epoxy injection techniques to provide for solid plate-to-plate bearing.

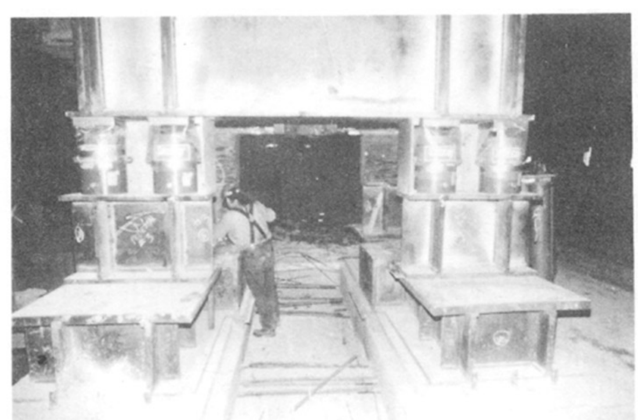


Figure 12: Column Jacking for Bearing Installation

The need for two to four isolators at individual columns required the use of very large baseplates to spread the load to the foundation mat. This caused a concern about grouting operations and the potential of voids forming beneath the plate. The contractor took special precautions to effectively place and consolidate the grout to provide for a solid bearing interface with the foundation. Their efforts were validated using full width lateral cores beneath several baseplates to verify that the grout had no voids.

Solving the Problem of Isolator Uplift with "Outrigger" Trusses

Even with reduced lateral loading due to seismic isolation, the 324 feet height of the building creates large seismic overturning forces, which cause localized uplift forces to develop at certain isolators. To control this potential uplift, a two-way series of 8 foot deep trusses was designed to span the entire basement, acting like outriggers on select column lines. Figures 13 and 14 show the trusses, which were constructed using A572 grade 50 plate box and W24 chords, with W12 web elements. They were erected in 15' to 20' long segments that were field connected using complete penetration butt joint welds. The trusses were then encased with reinforced concrete to increase their stiffness. The trusses will limit the uplift occurring at the isolators to 0.25 inch during a maximum credible earthquake. To validate this approach, the prototype isolators were successfully tested in combined shear and tension with vertical displacement (stretch) of 0.25 inch.

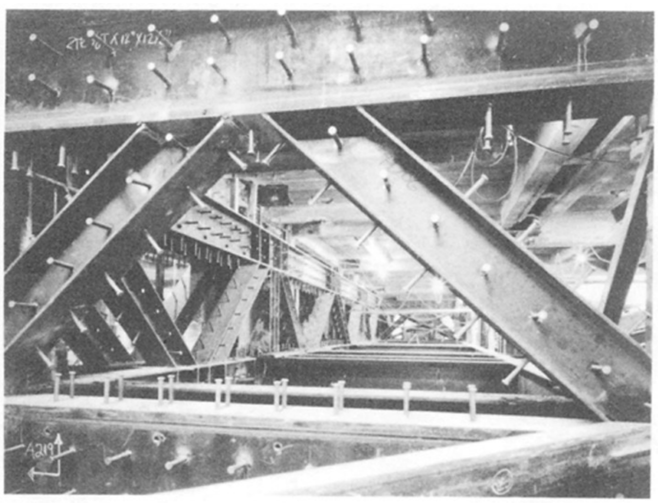


Figure 14: Trusses

Isolation Bearings

Of the 113 laminated steel and rubber seismic isolators used in the building, 36 have lead cores. Figure 15 shows the layout of the isolators in the basement. Figure 16 is a photo of the 4 isolators installed under one of the large columns mentioned earlier. The isolators range in diameter from 29 inches to 37 inches, and are approximately 19 inches high. The isolators were manufactured by Dynamic Isolation Systems Inc. at their plant in Wellington, New Zealand. The calculated ultimate seismic base shear for the isolated building is about 13% G, and the first mode period of the isolated building is 3.2 seconds with lateral displacement of 13 inches.

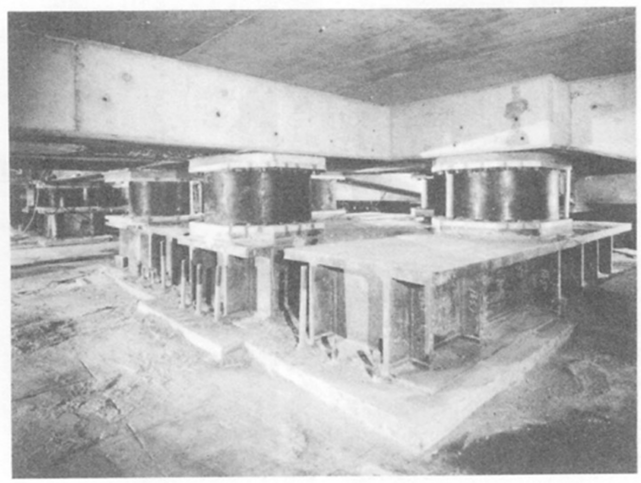
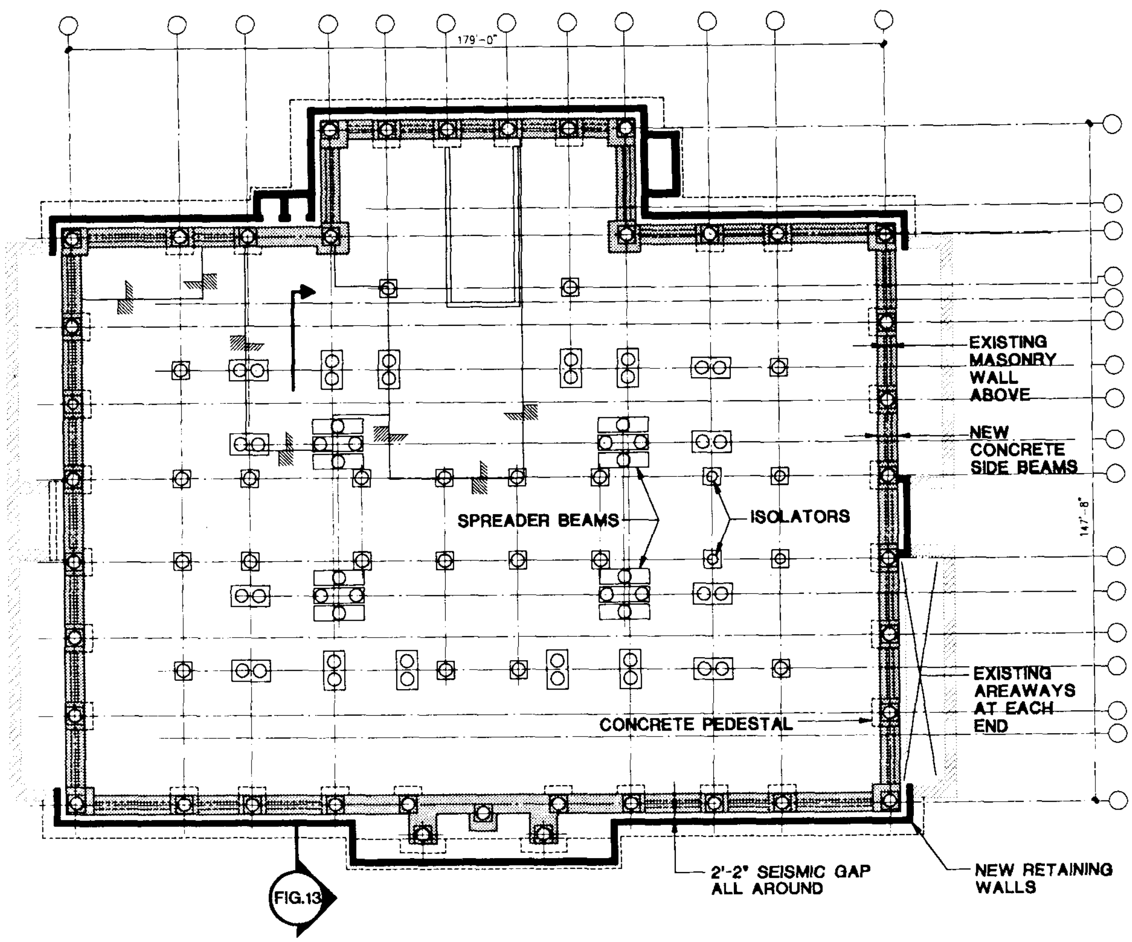


Figure 16: Installed Isolators

Summary

The versatility and strength of structural steel was used in a variety of unique ways in the retrofit design. Steel connections were designed with enough tolerance to accommodate existing field conditions, and welding to the old original A9 riveted steel was generally not a problem.

When retrofit work is completed in 1995, the Oakland City Hall will be the tallest seismically isolated building in the world. Seismic isolation using laminated rubber isolations devices under the building will dramatically reduce expected seismic forces in the building. Seismic isolation proved to be both technically feasible and economical. The use of isolation devices required less retrofit work in the building superstructure resulting in savings in steel bracing and concrete shear walls, with minimum impact on the historic interior of this landmark building when compared to conventional retrofit design.



FOUNDATION PLAN & SEISMIC ISOLATOR LAYOUT

Figure 15

Acknowledgments

Owner: City of Oakland, California

Architect: VBN/Willis/Carey Co. Associated Architects, Oakland, CA

Project Manager: Turner Construction Company, San Francisco, CA

Contractors:
General Contractor:
Overaa/Miller, Richmond, CA
Steel Subcontractor:
Bostrom Bergen Metal Products, Oakland, CA
Base Isolator Bearing Installation:
Sheedy Company, San Francisco, CA
Base Isolator Bearing Supplier:
Dynamic Isolation Systems, Inc., Berkeley, CA

Photo Credits: Robert Canfield, San Francisco, CA (Figures 5, 7, 11, 12, and 16)

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