THE SEISMIC RETROFIT OF THE OAKLAND CITY HALL

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Abstract

The historic Oakland City Hall experienced extensive damage in the 1989 Loma Prieta earthquake, and was subsequently the focus of an intensive process of testing, historic evaluation, dynamic structural analysis, and retrofit design using seismic isolation. A comprehensive post-earthquake study done by a team of architects and engineers, and reviewed by FEMA and SHPO, concluded that seismic isolation was the most cost-effective and behavior-effective method to protect the landmark building and its occupants from seismic hazards. The retrofit was completed in 1995. Numerous technical challenges and questions were confronted in the course of this pioneering project. The resolution of these issues, and the features of the seismic design and analysis are discussed.

Introduction

The Oakland City Hall, completed in 1914, was the first high rise government office building in the United States and is listed in the National Register of Historic Places. The primary innovation of the original design was the "stacking" of building segments, each representing a distinct occupancy or function, one atop the other. Such a design with its significant height and numerous setbacks was possible due to the advent of the use of structural steel in building frames. The building was heavily damaged during the Loma Prieta earthquake in October, 1989. Approximately 20% of the building's lateral strength was lost in the north-south direction, and 30% in the east-west direction, primarily due to extensive cracking of the numerous interior hollow clay tile partitions in the office tower. The clocktower at the top of the building "rocked" during the earthquake, resulting in large X cracks in some infill masonry walls and severe damage to several support transfer girders supporting the building's clocktower.

The top of the 18-story building is 324 feet above street level. The structure sits atop a full basement. The lowest and widest portion of the building, known as the podium, is 3 stories tall and contains a central rotunda, council chambers, and administration offices of the Mayor and City Manager. Above the podium is a 10-story office tower. Above the office tower is a 2-story clocktower base supporting a 91 foot high clocktower. The building steps back at each successive portion. These dimensional transitions were facilitated by the clever use of riveted transfer trusses and girders by the original engineers. Refer to Figure 1.

The original structure of the building is a riveted steel frame with infill masonry walls of brick, granite and terra cotta. The clocktower is clad entirely with terra cotta over brick masonry. The building is supported on a continuous concrete mat foundation.

Following the 1989 Loma Prieta earthquake, the building was studied extensively. The process involved testing, historic evaluation, and development of several repair schemes including seismic isolation. The findings of the studies were reviewed by the Federal Emergency Management Agency (FEMA) and the State Historic Preservation Officer (SHPO) as well as other agencies. After cost studies were completed, it was decided that the most behavior effective and economical method to protect this landmark building was to use seismic isolation. A peer review team was retained by the City of Oakland to review the design of the structural repair and seismic isolation, and to conduct a plan check.

The challenges faced during the retrofit evaluation and design include:

- Evaluating the interaction of the riveted steel frames and infill masonry, and evaluation of the acceptable drift limit of this system.
- The need for a dependable assessment of the dynamic modal properties.
- Evaluating and resolving isolator uplift due to the maximum credible earthquake.
- Provision of a continuous path for resistance of lateral loads where such a path did not previously exist, with minimal disruption of historic elements.
- Provision for the jacking and re-support of columns.
- Development of methods to repair and protect historically sensitive, brittle elements of the building.

This paper describes how the above challenges were resolved.

Design Approach

Seismic Performance Design/Analysis Methodology

Due to the archaic nature of the structural system and materials of the building and the need to preserve the historic fabric, it was not possible to directly apply the seismic provisions of the Uniform Building Code (UBC). While other codes, such as the California State Historic Building Code (SHBC) and Uniform Code for Building Conservation (UCBC) generally cover the area of historic structures, there were no specific applicable code provisions in place to guide the analysis or strengthening of rigid framed structures with fenestrated masonry infill. To solve this problem a rigorous design approach was adopted, based on the measured capabilities of the existing materials, and on the anticipated seismic response of the isolated superstructure.

The primary goal of this performance-based approach is identical to that adopted by the UBC; that is, to protect life safety during large earthquakes. The design of the isolation system itself followed the Seismic Isolation Appendix to Chapter 23 of the 1991 UBC.

Earthquake Ground Motion Criteria for Seismic Design

The analysis and design of the seismic repair of the Oakland City Hall superstructure was based on a site-specific design basis earthquake (DBE) response spectrum developed by Dames and Moore (refer to Figure 7). This site spectrum has the same return period (475 years) as the design spectrum required by the UBC for both conventional and isolated buildings. The Seismic Isolation Appendix to Chapter 23 of the 1991 UBC also required that the isolation system be “stable” against a “maximum credible earthquake” (MCE); that is, the strongest ground motion that could reasonably be expected at the site, given the known geological framework. For this purpose, Dames and Moore developed an MCE spectrum for Oakland City Hall that corresponds to a Richter magnitude 7 earthquake on the nearby Hayward Fault. The resulting MCE spectrum was replaced by 1.25 times the 475-year DBE design spectrum, which was higher than the MCE.

Anticipated Seismic Performance

The following seismic behavior of the repaired, isolated building is anticipated in response to a design level earthquake (475 year return period):

- In general, minor yielding is expected in the new concentrically braced steel frames in the office tower. Structural repair to these frames should not be required. No other significant yielding is anticipated for the new or original structure.
- The base of the isolated building is estimated to translate approximately 13 inches near the center of the building and 17 inches near the corners.
- The masonry of the clock tower is expected to crack. Due to the provision of a stiff steel bracing and back-up framing system that is proportioned to resist 100% of the lateral force, the cracking should be repairable.
- The office tower perimeter walls are expected to experience localized cracking in the piers and near the corners of the windows. This cracking should also be repairable.

Assessment of Dynamic Response of the Building

The existing structural system of the building consists of riveted steel frames with infill masonry. Experimental studies have demonstrated that this system can provide significant lateral stiffness and strength until cracks develop, after which the response degrades under repeated cyclic loading (Benjamin and Williams, 1958). Conventional seismic design procedures are generally based on the assumption that, during severe ground shaking, structural members will yield and undergo inelastic deformation. In this manner, the ductility of the structure is utilized to limit the seismic design forces and help the building to survive severe earthquake motions.

In contrast, due to lack of ductility and unreliable post-cracking strength, URM walls, when used as the building’s main lateral resisting system, should be designed to resist the maximum earthquake forces nearly elastically. Such a design approach is generally impractical for conventional fixed-base buildings because of the large seismic force demands. Thus, these elements are not generally recognized by building codes as a viable seismic resisting system.

The use of a seismic isolation system in this building resulted in a significant reduction of the seismic forces that would be imparted to the building during a severe earthquake. Therefore, it was possible to develop the superstructure seismic resisting system by utilizing the existing structural elements. The development of this design concept required:

- A reliable estimate of the strength and deflection capacity of the archaic structural materials
- Analytical techniques to evaluate lateral force resistance and interaction of the steel frames and infill masonry.

The material properties of the infill masonry walls were established by *in-situ* testing of the brick masonry. A series of special tests were designed to measure the force-deflection response of the brick masonry under cyclic compression and shear. Tests were performed on the exterior and middle wythes. Figures 2 and 3 show examples of the measured cyclic compression and cyclic shear force-deflection relationships of the typical brick masonry respectively.

The measured stiffness and strength of individual wythes were combined analytically to establish the response parameters (i.e., cracking stress, maximum stress, initial stiffness, and secant stiffness of the cracked section) of the composite three-wythe wall. Figure 4 shows the envelope of the measured cyclic shear tests and the calculated stiffness of the composite URM wall.

The lateral force resistance response of the building perimeter walls and the interaction of the steel frames and infill panels were studied by detailed finite element models (FEM) of typical multiple pier walls. In order to minimize the boundary effects, the model included three stories of the URM wall; however, only the results from the middle story were used for this study.

In order to determine the wall lateral load resistance and cracking patterns, seismic story forces were applied to the FEM model. At the locations where the calculated strains exceeded the cracking limit of masonry, representative element stiffness was reduced to simulate the opening of cracks. This analysis was used to establish the overall stiffness of the infill masonry wall at different stages of cracking.

The criteria for permissible seismic load in the URM walls was consequently based on a shear strain limit of 0.12% which was shown by tests to preclude severe cracking of brick masonry. Furthermore, at this level of strain, the maximum masonry strength can be utilized and there is a significant margin of safety for the ultimate shear strain (Figure 4).

Assessment of Building Dynamic Properties

In order to assess the seismic response of the building, a series of dynamic modal analyses with site specific response spectra were performed. The modal properties of the building were calculated for the following conditions:

- Existing building structure.
- Fixed-base retrofitted building with supplementary structural shear walls and steel braces.
- Retrofitted building with all seismic isolators at their pre-yield response range (base shear less than 0.5 times building weight).
- Retofitted building with isolators having effective stiffness at the design level lateral displacement.

Table 1 shows a summary of the building fundamental periods and calculated response parameters.

The lateral load-deflection response of the isolation system of this building, which consists of laminated elastomeric bearings with lead cores, exhibits pronounced yielding and change of stiffness. The response of this type of isolation system can significantly influence the seismic force distribution and performance of the superstructure. In order to verify the results of the modal response spectrum analyses and assess the effect of isolator hysteretic behavior, non-linear time history analyses were performed.

For these analyses the superstructure model was simplified to an equivalent stick model with building masses lumped at every floor level. All isolators were modeled explicitly having a bi-linear hysteretic force-deflection response.

For the time history analysis three pairs of ground acceleration records were selected that represent the likely earthquake motion at the building site. The acceleration records were modified to closely match the site specific design spectrum. Each time history analysis was performed under two ground accelerations acting simultaneously in perpendicular directions.

The results of these analyses were compared to the response spectrum results. In general good matching of the peak seismic shear force from different analyses was indicated. Figure 5 shows a comparison of the story shear distribution from response spectrum analysis with the peak responses of time history analysis.

Table 1 Building Dynamic Response Properties

	Period	Elastic Base Shear
1. Existing Building		
1935 Measurement	1.2 Sec	--
1957 Earthquake	1.2 – 1.3 Sec	--
1990 Ambient test	1.45 Sec	--
1990 Forced vibration test	1.56 Sec	--
Calculated as-is model	1.6 Sec	--
2. Fixed-Base Strengthened Model	1.21 – 1.42 Sec	--
3. Retrofitted Model Before Isolators Yield	1.60 – 1.70 Sec	0.05 W
4. Retrofitted Model with Effective Isolator Displacement	2.85 Sec	0.14 W

The response of a seismically isolated building can vary significantly depending on the intensity of ground motion. During a low level earthquake when the seismic force at the isolation level does not activate isolators, the building effectively responds as a fixed-base structure. Under these circumstances the building is susceptible to all the issues relevant to fixed-base structures, such as amplification of ground motion within the building and higher mode effects. Figure 6 shows comparisons of the lateral drift profiles of the building for both design level and lower level earthquakes. The low level earthquake was arbitrarily defined as that having a base shear just below the trigger force of the isolators. Figure 6 indicates that, in this building, the low level earthquake can induce interstory drifts of the same order of magnitude as the design level event. This load condition was also considered for checking the critical building components.

Lateral Load Path of the Seismically Upgraded Building

The building has four distinct sections, each with its own lateral load resisting system: The clocktower, office tower, podium and basement. Refer to Figure 1.

Clocktower

The clocktower, because of its slender configuration, and inherent overturning problems and damage-prone masonry infill, required a steel braced frame be added inside the clocktower. This braced frame was designed to resist the entire lateral load of the clocktower, not depending on the masonry infill to resist any of the lateral load. The stiffness of the new braced frame was selected to limit the clocktower structure ultimate inter-story drift to 0.008 times the story height to control potential damage to the infill brick/terra cotta cladding.

The base of this new steel braced frame "tower" is supported on a system of six interconnected steel transfer trusses that span 63 feet in the east-west direction and 62 feet in the north-south direction. The top chords of the one-story-deep trusses are located just below the 14th floor and the bottom chords are located just below the 13th floor. These trusses are supported on a system of eight new steel columns that extend down to the basement trusses by way of a new transfer truss system at the mezzanine level near the base of the office tower. The trusses and columns were designed to be stiff enough to limit drift in the clocktower due to overturning, i.e., the trusses "spread out" the reactions from the clocktower overturning moment.

Lateral loads from the base of the new clocktower braced frame are delivered to the exterior walls of the office tower portion of the building by a new horizontally braced diaphragm located just below the 14th floor.

Office Tower

In order to assess the participation of the existing masonry materials in the resistance of future lateral loads, extensive *in-situ* testing was performed on the existing brick infill to determine its strength and stiffness properties (see "Assessment of Dynamic Response of Building" above). It was determined that 100% of the lateral forces could be resisted by the north-south (longitudinal) masonry infill walls in the 10-story office tower portion of the building. In the transverse direction, since the walls are shorter in length, it was determined that supplemental bracing was required in the east-west direction to control potential damage to these walls. Two lines (4 bays) of concentric steel braced frames, supported by the same eight new columns that support the clocktower trusses, were designed to resist approximately 25% of the lateral load in the transverse direction; the remaining 75% would be resisted by the existing infill frame walls. These braced frames extend down to the 7th floor where they transition to concrete shear walls. The new braced frames were designed to be compatible with the stiffness of the existing transverse infill masonry walls. New steel collector beams were added at all floors to deliver lateral loads to the new braced frames and shear walls.

Podium

In order to sufficiently stiffen the podium portion of the building to protect the historic hollow clay tile partitions and plaster, a system of new interior concrete shear walls, located in the core areas, was designed to extend down to the trusses in the basement. The locations of the faces of these new shear walls had to be carefully coordinated with the architectural historic finishes. Steel shear walls were used at some locations to reduce required shear wall proportions where existing exterior windows could not be closed off for historic reasons.

All of the new concrete shear walls are bounded by either new or existing steel columns. These columns are engaged by the new walls to act as boundary steel to resist wall overturning moments. Steel reinforcing plates (up to 4 inches thick) are added between the flanges of the existing columns as needed to provide sufficient boundary steel area.

Basement

The concrete shear walls terminate on new 8.5 feet deep continuous steel “outrigger” trusses in the basement. Typically, double lines of trusses straddle and weld to the existing columns. These trusses are encased in concrete to provide additional stiffness and to tie the double lines of trusses together. The purpose of the trusses is to distribute the building overturning moments over a broad footprint so that the base isolators located beneath the office tower perimeter will not be overloaded nor subjected to any appreciable uplift.

A system of horizontal steel braces forms a “diaphragm” below the first floor to deliver lateral loads to a system of 113 elastomeric isolation bearings, approximately half of which have lead cores.

The existing basement concrete walls have concrete side beams added on both sides so that the wall, after being cut, will span rigidly between base isolators to support the massive stone-clad exterior podium walls.

The isolators are supported on a system of existing and new steel/concrete pedestals that are supported on the existing concrete mat foundation. Multiple isolators (up to 4 per group) are used to support individual columns with dead loads in excess of 3300 kips each.

A continuous seismic gap around the building was provided. The isolators are proportioned to move laterally more than 17 inches during an M7 event on the Hayward Fault. The prototype isolator units were tested to approximately 23 inches of displacement, which exceeds the 1991 UBC equivalent static formula requirement of 19 inches.

Resolution of Isolator Uplift due to MCE Forces

The maximum uplift displacements due to the critical cases of an MCE overturning moment were estimated using a step by step, iterative, static ETABS analysis based on the peak dynamic nonlinear time-history ANSR analysis results. The potential for local isolator uplift at the interior columns beneath the office tower perimeter was demonstrated. This problem was resolved by bounding the analytical results and uplift tests on the prototype isolators and their connections.

The critical cases of lateral seismic loads were derived from the ANSR nonlinear time-history analysis. Story forces from the ANSR analysis were applied statically to the ETABS model. Two isolator anchorage conditions were modeled in order to bound the analytical results of the uplift analysis:

- Case 1. Isolators allowed to uplift vertically when net uplift tension occurs.
- Case 2. Isolators bolted top and bottom.

For Case 1, a process of iteration was used to identify the isolators that could go into net tension under the MCE lateral loading. After each analysis step, any isolators showing a tensile load were softened to simulate a doweled bearing condition. The maximum net column uplift displacement using this approach was calculated to be 0.11 inches for the case of uncracked concrete encasement of the basement outrigger trusses, and 0.25 inches for the case of cracked encasement.

Case 2 analysis was intended to provide the maximum tensile force that could be generated if all bearings were bolted. The tensile vertical stiffness was assumed to be equal to the vertical compression stiffness. Results from tests done in Japan on small-scale laminated rubber isolators (Ishida et al. 1991) indicate that vertical stiffness decreases significantly after the bearings reach a tensile stress of about 250 psi and will continue to decrease until failure at roughly 850 psi, corresponding to an axial strain of about 300%. In order to obtain a safe, conservative upper limit on the uplift tension, the isolator forces were calculated using the initial vertical stiffness multiplied by the Case 1 "free" vertical displacement. All isolator anchorage components, including the anchor bolts, were designed to resist the tension forces thus calculated.

The analysis results indicated that the tendency toward isolator uplift would coincide with maximum isolator horizontal displacement. The prototype test program therefore incorporated a combination of ¼-inch vertical upward displacement with the maximum horizontal displacement. The rigidly bolted isolator specimens remained undamaged for the test case, which validated the final selection of conventionally bolted (non-doweled) connections.

Jacking and Re-Support of Existing Columns

Seismic isolation of an existing building such as Oakland City Hall typically involves the complicated task of shoring the existing columns so they may be cut free from the foundation allowing installation of the new isolator bearings. Extensive sequencing notes were developed as part of the shoring design to guide the contractor during the bidding and construction phases.

The sequence notes were intended to help preserve the local and global structural integrity of the building to the extent practicable during construction. To achieve this goal, detail requirements for the following topics were included:

- Temporary lateral bracing was required for the basement level during the period of time between structural demolition and final release of the isolator system. Even partial demolition of the perimeter walls during isolator installation would cause a very weak story condition and put the building basement at risk of being damaged during a moderate or major earthquake if no temporary bracing was provided.
- A symmetric work sequence was required to reduce the possibility of an undesirable torsional response of the structure to an earthquake during the construction period.
- The magnitude of jacking loads and load application points were provided on the drawings. Typically, the jacking points were located on new steel framing and corbels welded to the existing columns. The new corbels also serve as permanent column bases after removal of the existing base plates.
- Vertical column displacement during jacking was limited in the contract documents to prevent damage to the superstructure finishes. This displacement was measured during jacking operations using sensitive instrumentation.
- Submittal and review of the contractors' detailed construction sequence was required to ensure proper interpretation of the design intent.

Protection of Historically Sensitive Brittle Elements

The podium portion of the building (Floors 1, 2 and 3) contains the most prominent public spaces and has the majority of the interior historic finishes. The unreinforced hollow clay tile (HCT) partition walls in the podium required special details to repair existing cracks caused by the Loma Prieta earthquake. Crack repair details were developed to allow the existing HCT to remain without replacing cracked tiles and to require only minimal removal of historic plaster finishes. This was accomplished using a combination of self-tapping anchors, metal lath, epoxy, and cement plaster.

Many of the exterior terra cotta cladding elements were also damaged during the earthquake. The damaged pieces were removed and used to create molds for the casting of new replacement pieces. New attachment details were developed to attach these replacement units.

Conclusions

The use of seismic base isolation as a seismic upgrade strategy dramatically reduced expected seismic force levels in the building and resulted in fewer shear walls than a traditional upgrade would have required. With fewer shear walls, the impact of the upgrade on the historically sensitive interior of this landmark building was significantly reduced. Base isolation proved to be an economically feasible solution when compared to conventional fixed base upgrade schemes. Through a combination of base isolation, extensive testing of the existing masonry infill walls and a comprehensive finite element study of typical wall panels, the majority of seismic lateral forces will be resisted by existing unreinforced masonry infill walls in the office tower portion of the building.

By designing stiff concrete encased trusses in the basement, seismic overturning forces from this relatively tall building will be spread out over many base isolators so that they will not be overloaded nor subjected to appreciable uplift.

Acknowlegements

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(Post-Earthquake Studies)

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Razzano Associates
Tennebaum-Manheim Engineers

Geotechnical: Dames & Moore

Peer Review: Wiss, Janney, Elstner Associates

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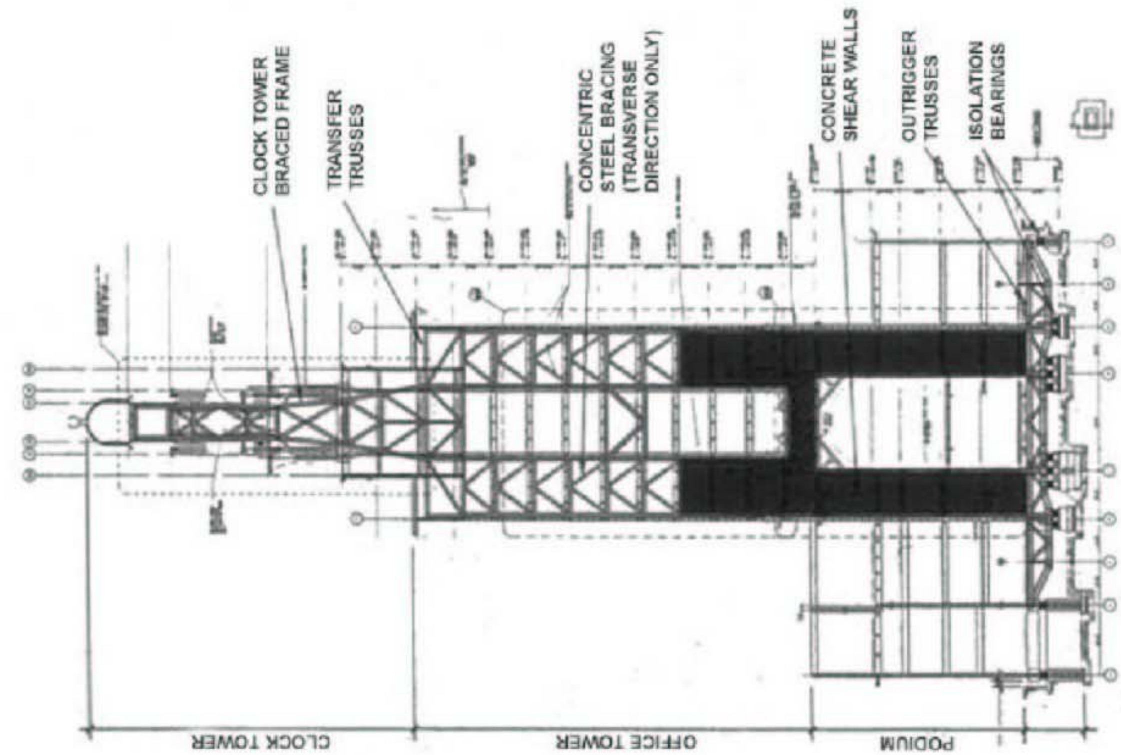


Figure 1

Photo Figure 1 Oakland City Hall

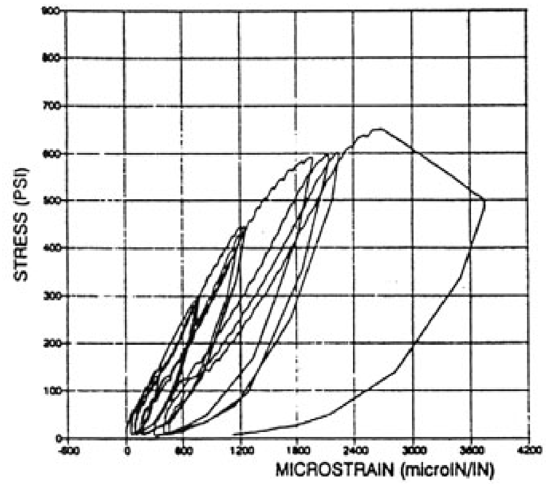


Figure 2 CYCLIC COMPRESSION TEST

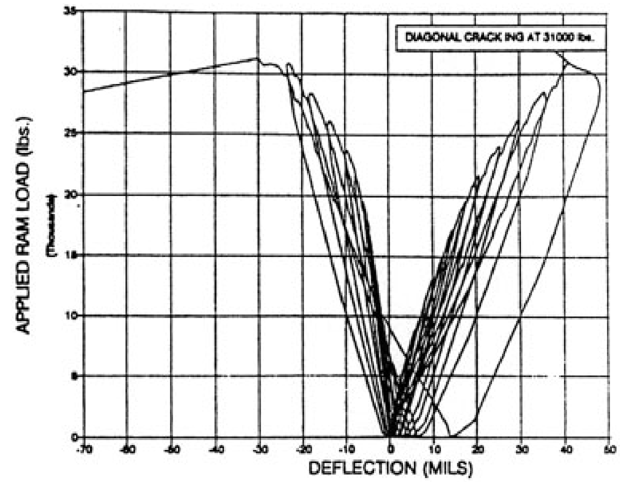


Figure 3 CYCLIC SHEAR TEST

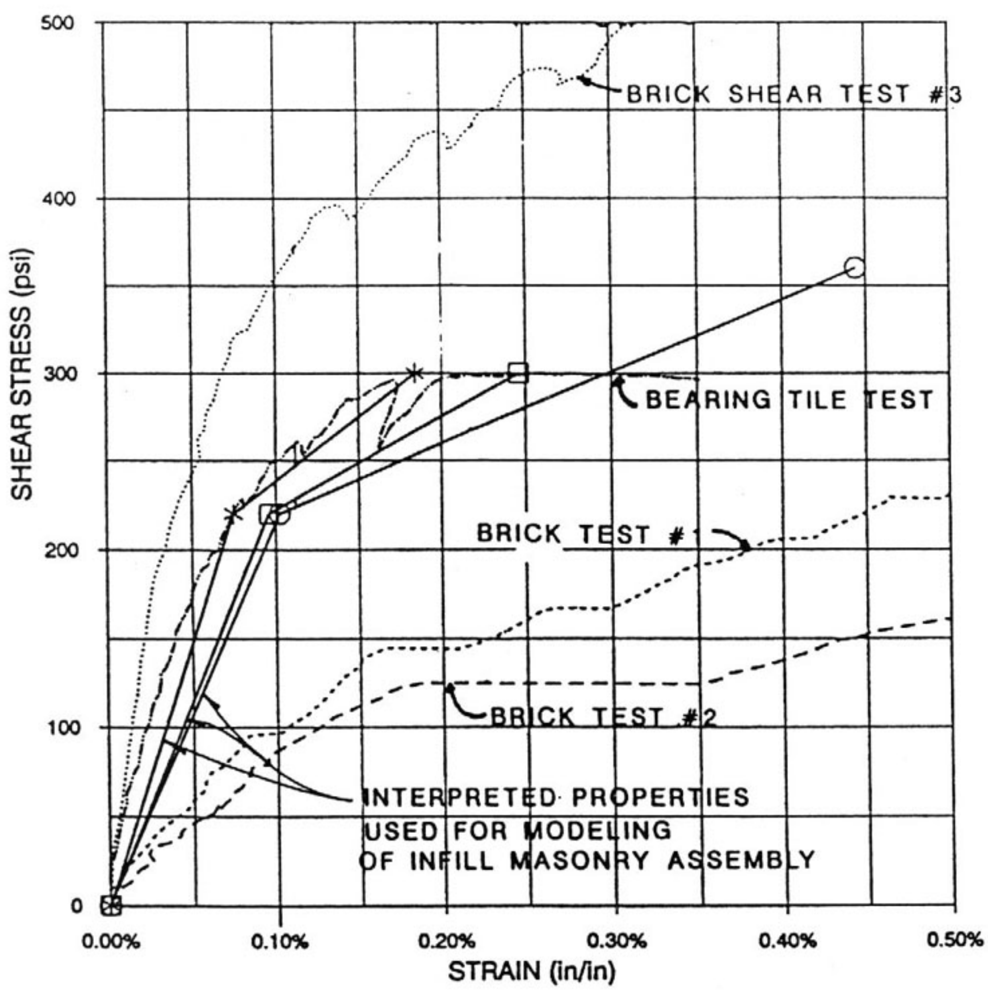


Figure 4

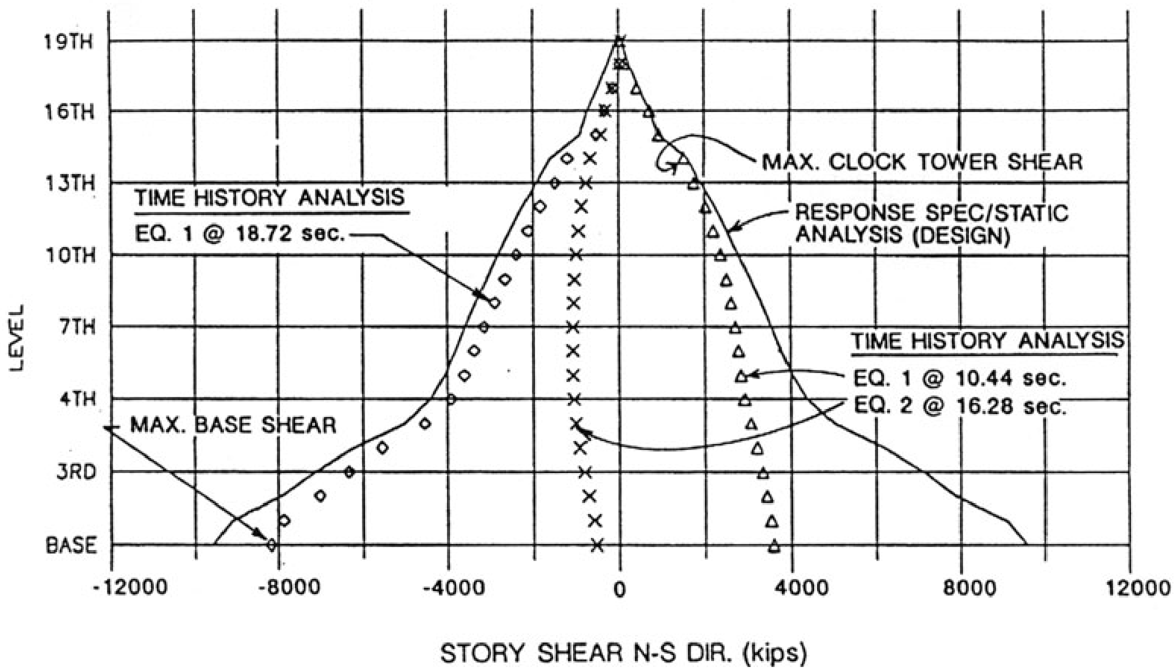

19TH
16TH
13TH
10TH
7TH
4TH
3RD
BASE
LEVEL
-12000
-8000
-4000
0
4000
8000
12000
STORY SHEAR N-S DIR. (kips)
MAX. CLOCK TOWER SHEAR
TIME HISTORY ANALYSIS
EQ. 1 @ 18.72 sec.
RESPONSE SPEC/STATIC
ANALYSIS (DESIGN)
TIME HISTORY ANALYSIS
EQ. 1 @ 10.44 sec.
EQ. 2 @ 16.28 sec.
MAX. BASE SHEAR

Figure 5

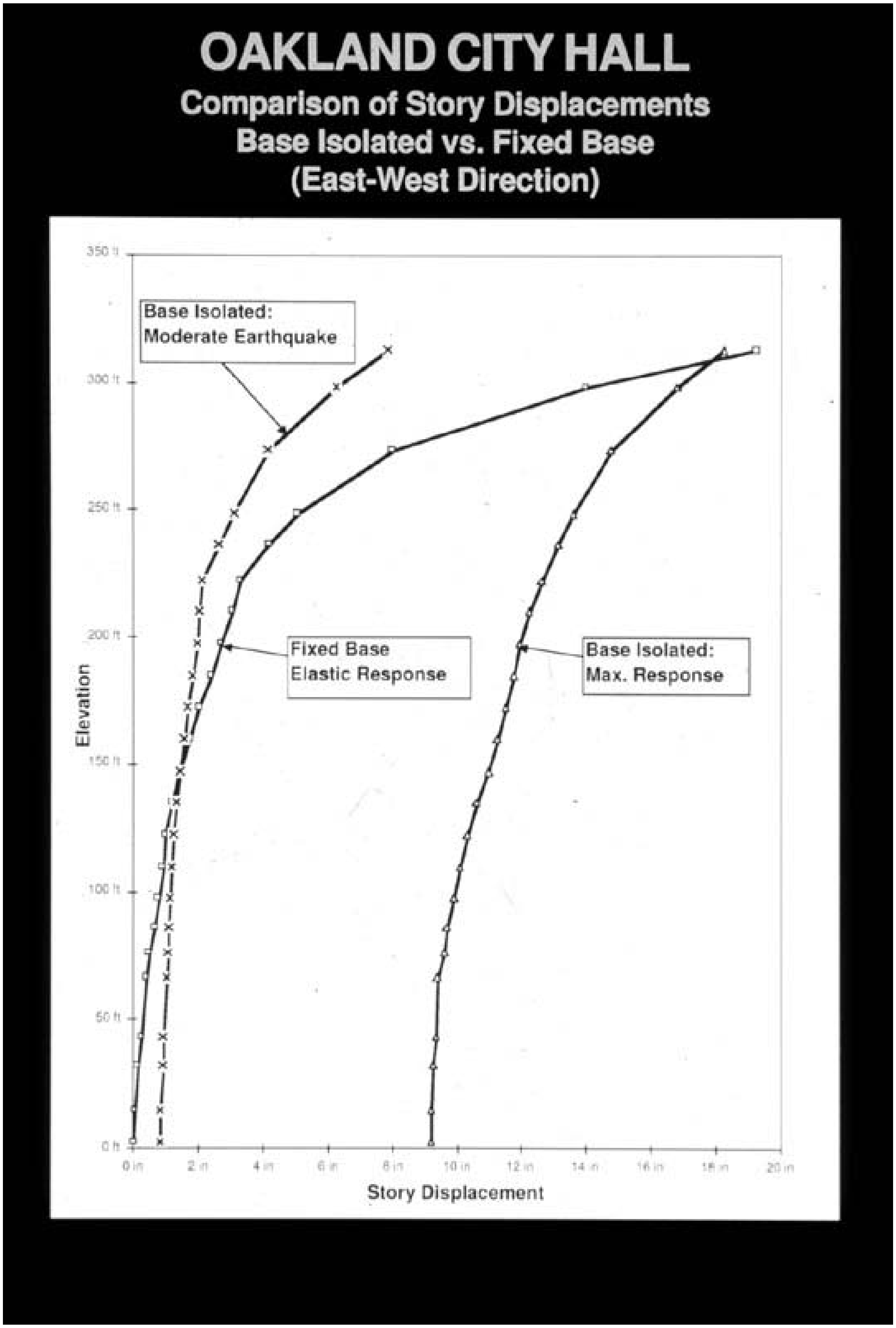
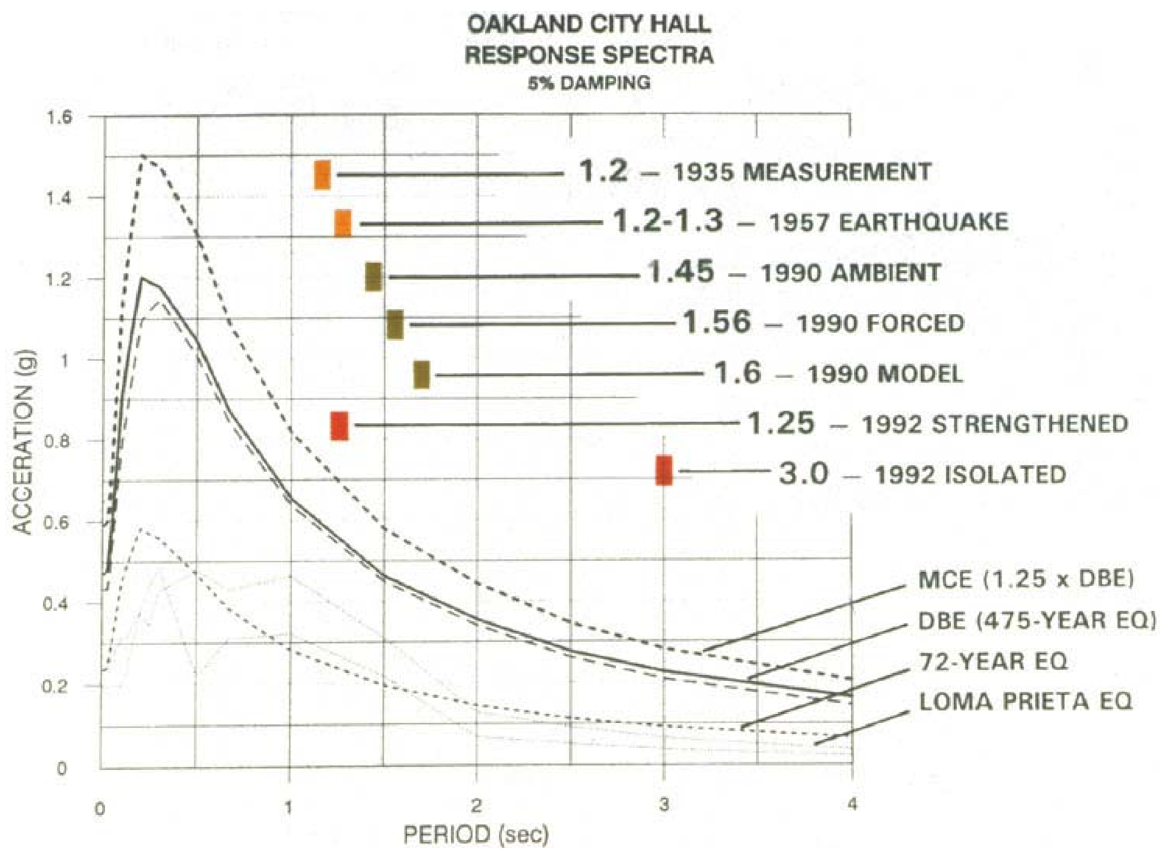

OAKLAND CITY HALL
Comparison of Story Displacements
Base Isolated vs. Fixed Base
(East-West Direction)
Base Isolated:
Moderate Earthquake
Fixed Base
Elastic Response
Base Isolated:
Max. Response
Elevation
Story Displacement
350 ft
300 ft
250 ft
200 ft
150 ft
100 ft
50 ft
0 ft
0 in
2 in
4 in
6 in
8 in
10 in
12 in
14 in
16 in
18 in
20 in

Figure 6



OAKLAND CITY HALL
RESPONSE SPECTRA
5% DAMPING
1.2 – 1935 MEASUREMENT
1.2-1.3 – 1957 EARTHQUAKE
1.45 – 1990 AMBIENT
1.56 – 1990 FORCED
1.6 – 1990 MODEL
1.25 – 1992 STRENGTHENED
3.0 – 1992 ISOLATED
MCE (1.25 x DBE)
DBE (475-YEAR EQ)
72-YEAR EQ
LOMA PRIETA EQ
ACCERATION (g)
PERIOD (sec)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
1
2
3
4

Figure 7