THE STRUCTURAL DESIGN OF TALL BUILDINGS
Struct. Design Tall Build. **9**, 3–24 (2000)

BUILDING CASE STUDY

LOS ANGELES CITY HALL

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SUMMARY

This paper describes the Los Angeles City Hall building and a unique seismic strengthening program. Four hundred and sixteen high damping rubber bearings, 90 flat sliding bearings and 64 viscous dampers have been installed as part of the seismic rehabilitation. The building is a 460 feet tall steel frame with unreinforced masonry infill. The rehabilitation consists of installing high damping rubber bearings at its base supplemented with nonlinear viscous dampers. This paper describes various aspects of the project including the development of seismic performance goals, identification of inherent seismic deficiencies of the existing building, evaluation of alternative strengthening schemes, the final design process and construction issues. Copyright © 2000 John Wiley & Sons, Ltd.

1. INTRODUCTION

Los Angeles City Hall is perhaps the most famous building in Western United States. It was built in 1926, as a 32-story, 460 feet tall steel frame building with riveted semi-rigid connections. It was the first building to exceed the 150 feet height limitation for all privately constructed buildings in Los Angeles. It is an enduring symbol of Los Angeles and is familiar to many television viewers of the ‘Dragnet’ series. A photograph of the building is shown in Figure 1.

Over the past 65 years, regional earthquakes have caused damage to this building. Masonry infill and concrete walls have cracked. The terra cotta cladding has been cracked, broken or destroyed in portions of the building’s exterior. With every significant earthquake, unanchored masonry debris has been scattered about the building’s interior. At the 24th floor, large cracks in the masonry walls appeared after the 1971 Sylmar Earthquake, the 1987 Whittier Earthquake and the 1994 Northridge Earthquake.

In order to meet the life safety and damage mitigation objectives of the City of Los Angeles, to maintain the integrity of the building’s exterior facade and to protect the historic interior fabric from damage, a seismic rehabilitation of the building was studied, planned and is now currently in progress. Three seismic rehabilitation schemes were evaluated—a reinforced concrete shear wall system, a reinforced concrete shear wall with steel super-brace system and a base isolation system with supplemental dampers. The base isolation system with supplemental damping was determined to be the most effective strengthening scheme based on performance and cost.

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Received November 1999
Accepted December 1999

Figure 1. Aerial view of Los Angeles City Hall

This paper presents a description of the project including the development of seismic performance goals, identification of inherent seismic deficiencies of the existing building, evaluation of the three seismic rehabilitation schemes, the final design process and the current construction status.

2. BUILDING DESCRIPTION

The building has three major structural segments: the podium (sub-basement to second floor), the midrise (third to ninth floors) and the tower (tenth floor to the top of the pyramid). The structural frame of the tower extends from below the tenth floor, through the mid-rise and podium of the building to the matslab foundation beneath the sub-basement floor. Similarly, the mid-rise frame extends from below the third floor, through the podium to the foundation level.

The gravity system of the building consists of a concrete encased steel frame and reinforced concrete slab/pan-joist floor system. The typical beam-to-column connection is a riveted 'wind connection' that utilizes top and bottom seat angles.

The building was designed in the early 1920's prior to the enactment of explicit seismic design requirements. Therefore, the building was not specifically designed to resist seismic forces. Thus, the building does not have a distinct seismic force resisting system to provide a well defined load path. However, there are a number of structural components that, although not specifically designed to resist earthquake forces, participate in resisting these forces.

Lateral load resistance in the existing building is provided by horizontal diaphragms, perforated unreinforced masonry infill walls, lightly reinforced concrete walls and light steel bracing. The unreinforced masonry infill walls provide most of the lateral force resisting capability of the building. The infill walls occur at the perimeter of the ground through 26th floors and around the light courts from the sub-basement to the fifth floor. Reinforced concrete walls occur at the perimeter of the sub-basement and basement floors, at the four corners of the tower below the tenth floor and at the top levels of the tower. The steel bracing occurs at the four corners of the tower from the sub-basement to the 22nd floor. These braces consist of light steel sections (2-Ls $6 \times 3\ 1/2 \times 1/2$) above the mid-rise. The steel bracing was designed to provide lateral stability during erection. Belt trusses tying the corners of the tower together occur at the ninth and 22nd floors.

3. PERFORMANCE CRITERIA

The performance of the Los Angeles City Hall in earthquakes depends on several factors that are not

Table I. Seismic performance objectives

I. Protect and preserve a historic monument after major seismic events	II. Preserve continuity of government functions after major seismic events
1. Insure seismic safety/stability of the structural system	1. Preserve the basic functions of the building
2. Maintain integrity of the building's exterior façade	2. Insure safe means of egress from the building
3. Prevent falling hazards that pose a significant life safety hazard	3. Insure that life safety systems remain operable
4. Protect historic interior fabric of the building from damage	4. Protect emergency telecommunication systems including tower, satellite dishes, etc.
5. Protect interior building contents, historic artwork, ornamental details, etc.	

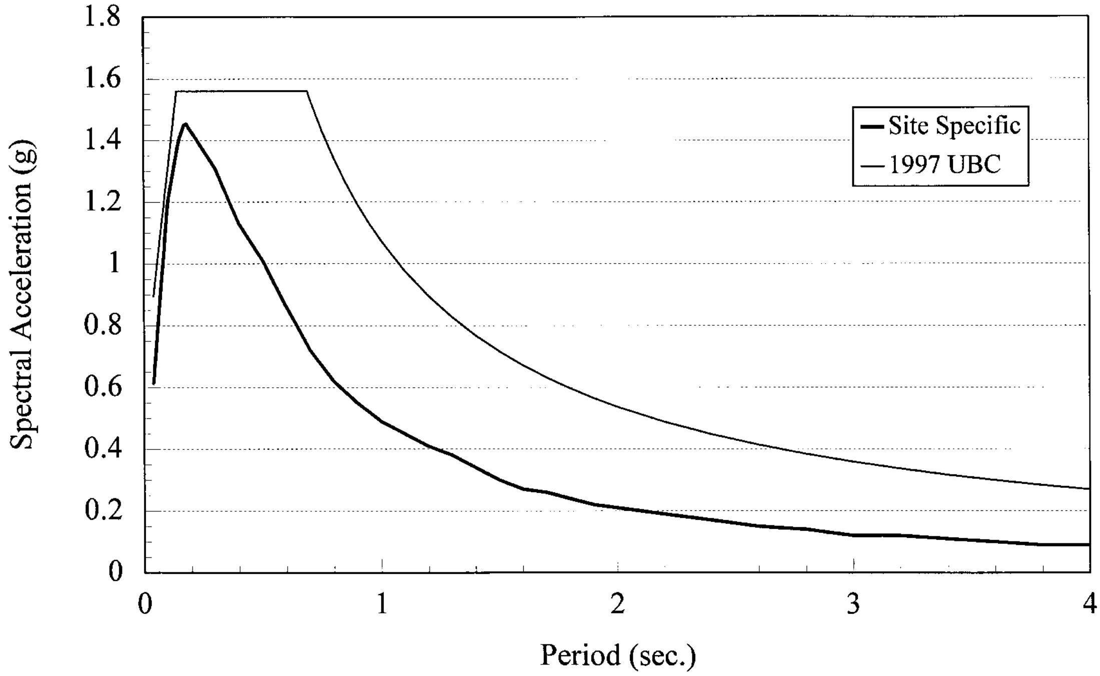


Figure 2. 5% damped site specific spectrum and 1997 UBC spectrum

explicitly considered in code-based approaches, such as earthquake intensity, material properties, quality of construction, structural configuration/irregularities and force/deformation characteristics of the lateral force resisting system. Code techniques are not capable of predicting damage levels or the specific performance of the building. In addition, the structural system and type of construction used for the building is prohibited by current codes. Therefore, a performance-based approach was developed, since a code-based approach was not appropriate for evaluating the building.

Seismic goals were developed to establish performance objectives for the evaluation of the existing building and the design of potential strengthening schemes. The seismic goals were intended to satisfy both the life safety and damage mitigation objectives of the City of Los Angeles for the building and are presented in Table I.

In order to evaluate the ability of the existing building to meet the seismic goals, the goals were quantified as engineering criteria. The engineering criteria defined the performance goals in terms of specific analytical limit states. The quantification of the seismic goals as engineering criteria was based on specific rational analytical limit states, not on simplified code-based approaches, which are inappropriate for this building. These limit states were determined using the latest research data available regarding the seismic performance of existing buildings combined with guidelines developed for life safety protection and damage mitigation.

The limit states were established for interstory drift, inelastic demand ratios for the various structural elements and story accelerations. The interstory drift limits are essential for maintaining global stability in the structural system by limiting the P–Δ effects. In addition, earthquake damage in many structural and non-structural building systems can be directly associated with the interstory drift the building experiences. Inelastic demand ratios (IDRs) represent a measure of the post-yield deformations of the structural members during a given earthquake. In order to insure the safety and

Table II. Earthquake time history records

Ground motion records
1. 1940 Imperial Valley—El Centro
2. 1992 Cape Mendocino—Petrolia
3. 1989 Loma Prieta—Corralitos
4. 1952 Kern County—Taft
5. 1989 Loma Prieta—Hollister
6. 1971 San Fernando—Castaic
7. 1994 Northridge—Pacoima Dam

stability of the structural system, limitations on the allowable IDRs are required. Story accelerations affect the seismic performance of building contents and non-structural systems.

4. EARTHQUAKE GROUND MOTIONS

The analysis and design for the Los Angeles City Hall seismic rehabilitation was performed in 1994–1995 based on time-history records, which were scaled to meet the site-specific response spectra. Site-specific spectra were developed that account for near-fault effects and dispersion effects for distant events. The design basis earthquake (DBE) represents a 10% probability of exceedance in a 50-year time period and the maximum capable earthquake (MCE) represent a 10% probability of exceedance in 100 years. Figure 2 shows a plot of the 5% damped site specific response spectrum and the 1997 UBC design spectrum developed for this project by Dr Marshall Lew of Law Crandall Geotechnical Engineers. Although the site specific spectrum, developed in 1993–1994, is unconservative relative to the 1997 UBC spectrum, contingencies in the isolation system design counteract this effect and will be discussed later.

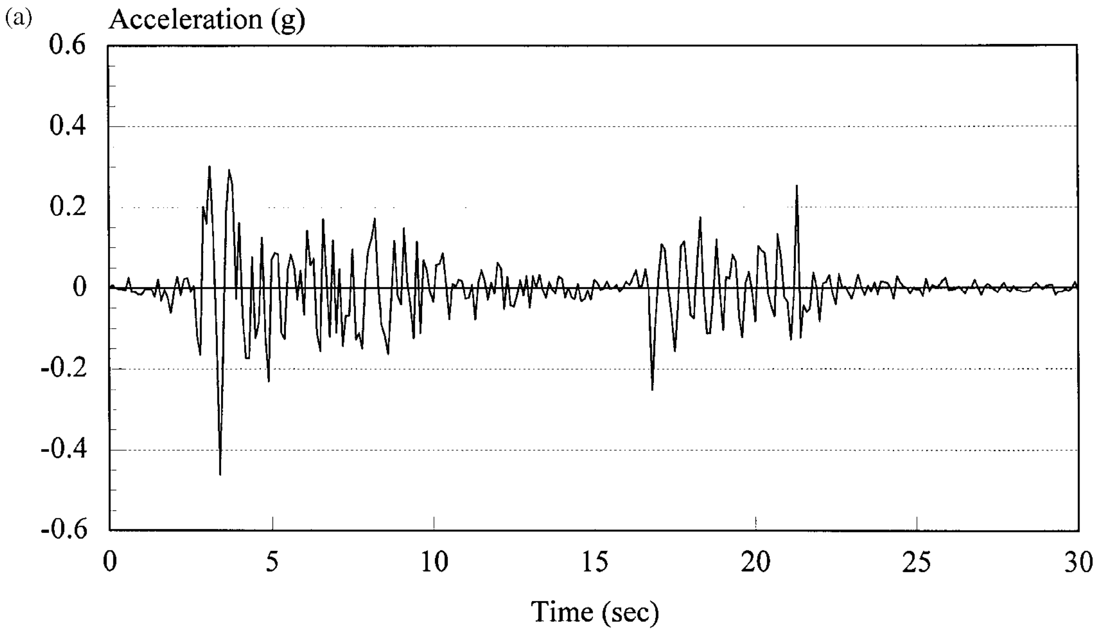
Seven time-history records were used in the analysis and design, and are presented in Table II. The Petrolia ground motion record, which governed the design of the isolation system, is shown in Figure 3. All of the records were amplitude scaled to match the design spectra within the scaling window of 2·5 through 4·0 s. The Hollister and Pacoima Dam records account for the potential near-fault activity at the site. Recorded ground motion from the 1994 Northridge earthquake at sites near the fault rupture provide evidence for design consideration of near-source effects.

5. MATERIAL AND DYNAMIC TESTING

In order to determine the strength and deformation characteristics of the existing building materials and to determine the dynamic characteristics of the existing building, material and dynamic testing have been performed.

A variety of *in situ* tests were performed on the unreinforced masonry. In-place shear tests were performed on the masonry walls to determine the ability of the existing brick and mortar to resist shear stresses. Several flatjack tests were performed to determine the compressive strength and deformability properties of the masonry. A typical stress–strain plot of *in situ* modulus test data is shown in Figure 4. The results of these tests were used in the development of the computer models and in the determination of the strength capacity of the existing building.

Ambient and forced vibration tests were performed Professor Gary C. Hart of UCLA to determine the dynamic, properties of the existing building. The ambient vibration test measured the response of the building to vibrations that occur at the building site due to vehicular traffic, wind, occupants, etc. Forced vibration tests were performed using a forced vibration oscillator to vibrate the building. The



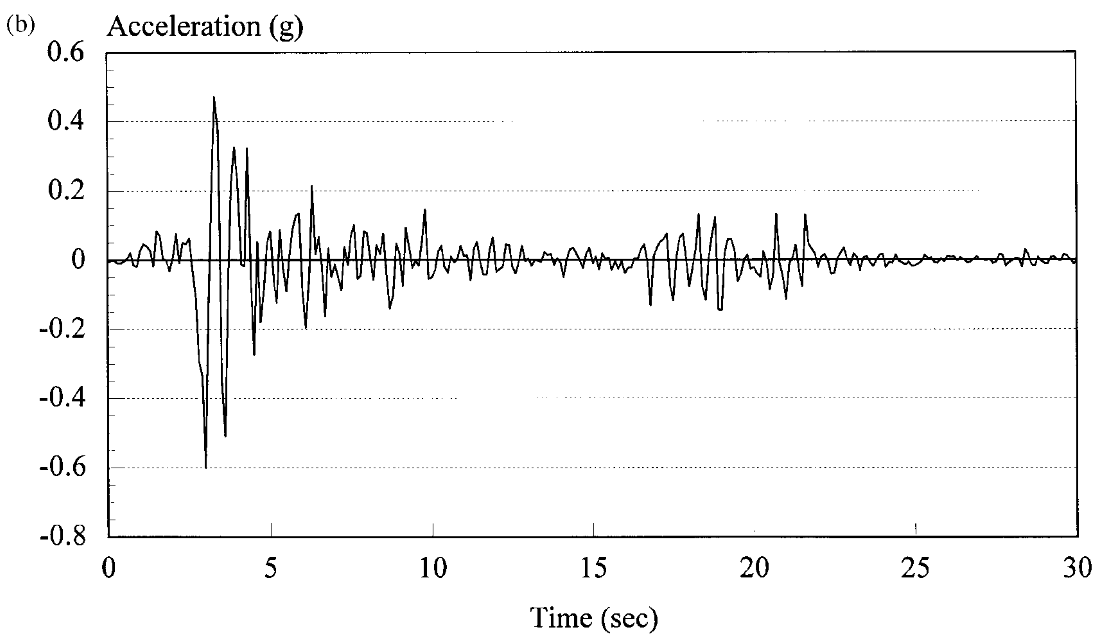


Figure 3. (a) Petrolia ground motion (0 deg. component)—1992 Cape Mendicino earthquake; (b) Petrolia ground motion (90 deg. component)—1992 Cape Mendicino earthquake

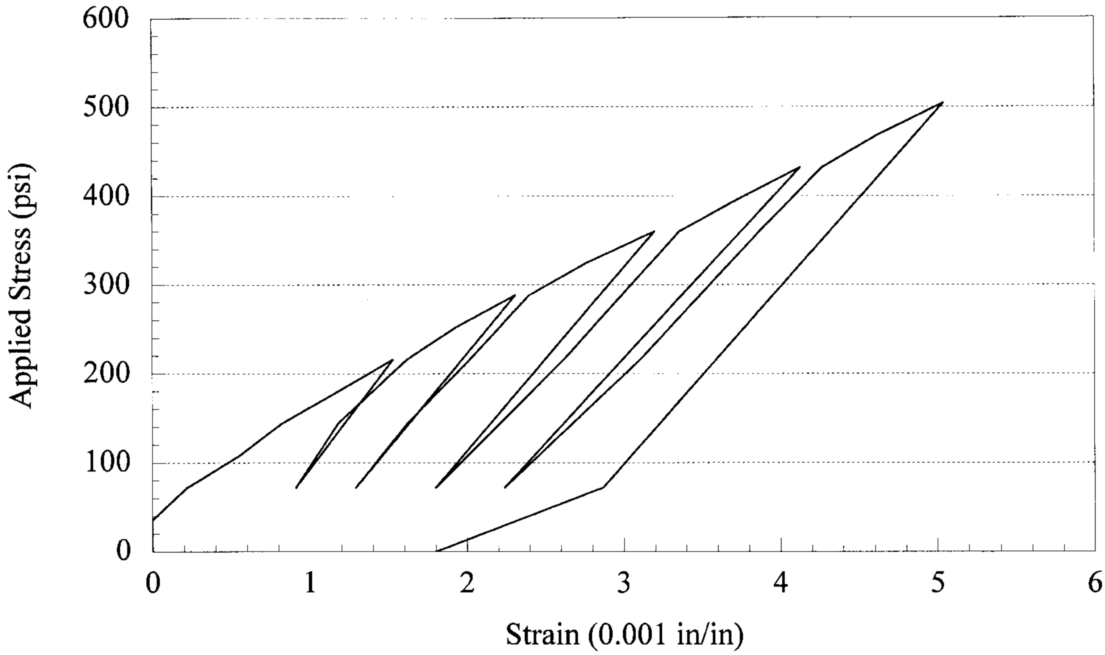


Figure 4. Typical stress–strain plot of *in situ* modulus test results

forced vibration test was used to determine the response of the building to high-level excitations. The results of these tests were used to verify the modeling assumptions made in the development of the computer model of the existing building.

The ambient vibration survey was performed on the Los Angeles City Hall in May 1993. The accelerometers recording the ambient accelerations were placed on the 25th floor of the City Hall building. The layout of the accelerometers on the 25th floor of the City Hall building is shown in Figure 5. A series of samples were recorded during this ambient vibration survey. Power spectral densities of the recorded accelerations were obtained from an on-site spectrum analyzer. The power spectral density functions of the accelerations recorded from Channels 2 and 3 are shown in Figure 6. The fundamental natural periods of vibration of the City Hall building determined from the ambient vibration survey are listed in Table III. The vibration mode type is also identified in Table III. Characteristic periods of vibration of the City Hall building were also measured by inducing motion through the use of a seismic shaker. The fundamental natural periods of vibration of the City Hall building determined from this forced vibration test are also listed in Table III.

6. WIND TUNNEL STUDY

A wind tunnel study of the Los Angeles City Hall was performed by the Hart Consultant Group (Professor Hart with Dr Jon Raggett) to determine static equivalent floor forces and moments for the design of the structural frames. Those were used as input in establishing the yield level of the base isolators. Design wind speeds for return periods of 50 years, 475 years and 1000 years were determined from a statistical analysis of historical wind data, corrected for the specific site, from the Los Angeles Civic Center and a recording station of the South Coast Air Quality Monitoring District. Wind forces and moment time histories on the Los Angeles City Hall building were determined using a 1:240 scale

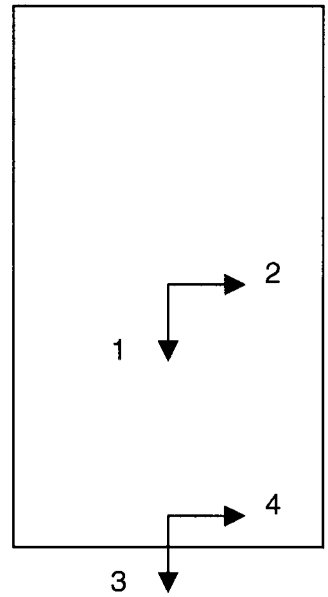


Figure 5. Layout of accelerometers on the 25th floor of the City Hall Building

aeroelastic model of the building and its immediate environment (1200 feet $\times$ 1200 feet) in an atmospheric boundary layer wind tunnel. The model of the City Hall building undergoing testing is shown in Figure 7.

Wind-induced base shear time histories, and a twisting-moment time history about a vertical axis at the building center were measured on the building model. These forces were measured for winds coming from critical directions at 15 degree increments. These forces were computed by combining a

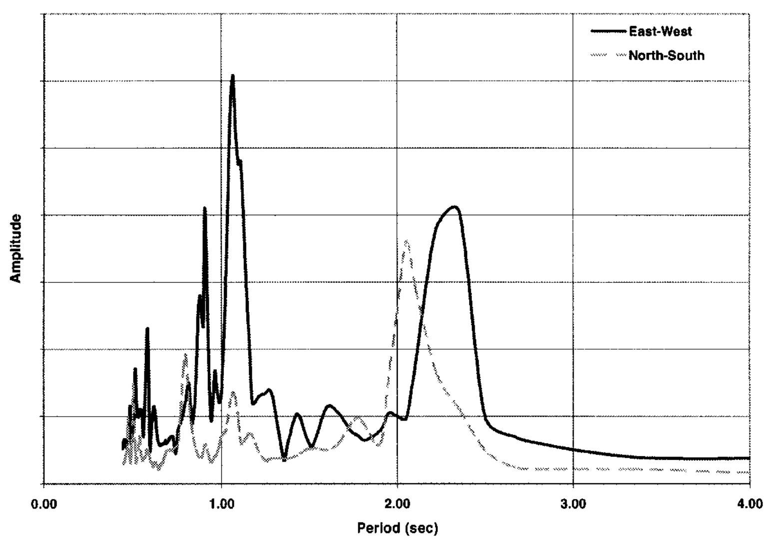


Figure 6. Power spectral density of accelerations from channels 2 and 3

Table III. Natural frequencies of City Hall from ambient and forced vibration tests

Mode of vibration		Period of Vibration (s)	
Direction	Number	Ambient vibrations	Forced vibrations
North–South	1st	2·08	2·50
	2nd	0·78	0·96
	3rd	0·50	—
East–West	1st	2·38	2·27
	2nd	0·89	0·86
	3rd	0·53	—
Torsion	1st	1·09	1·19
	2nd	0·62	0·69
Unknown			0·60
			0·56

time history of wind load data from each direction with a dynamic model of the building. The damping ratio for the analysis was set at 5% to reflect the behavior of the building at these load levels. Therefore, these forces are static equivalent wind forces that incorporate the dynamic response characteristics of the building when subjected to a wind loading time history. The measured force and moment time histories were scaled to full-scale equivalent time histories and were distributed over the

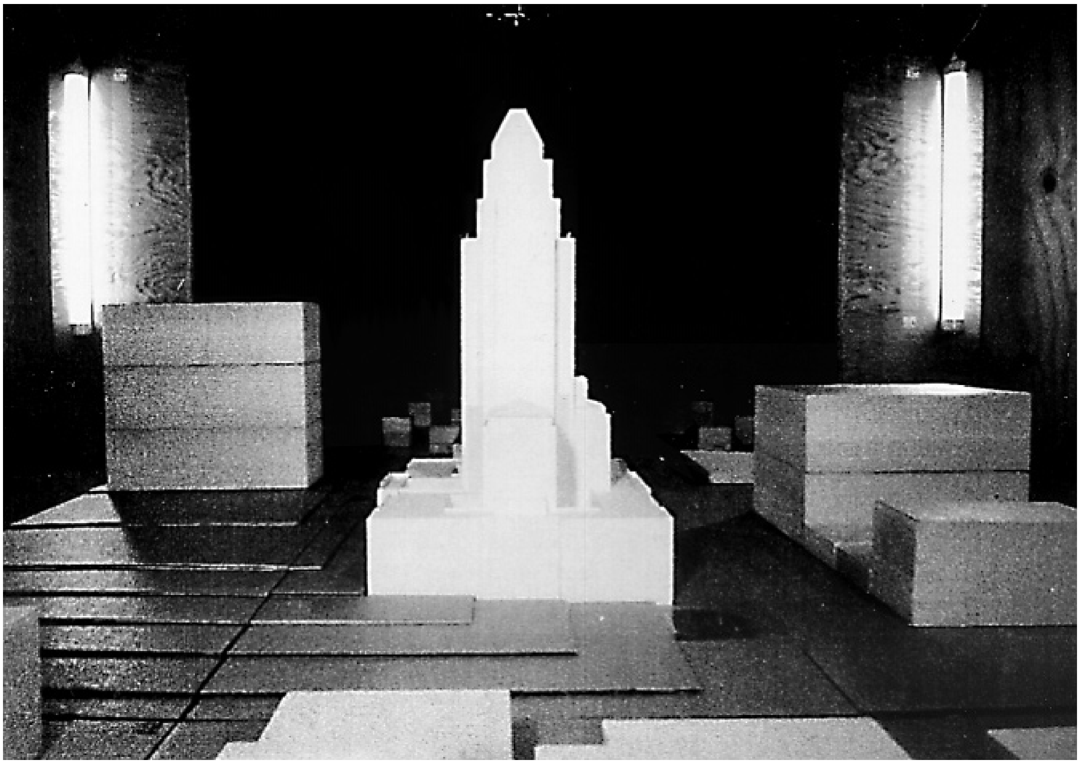


Figure 7. Wind tunnel study

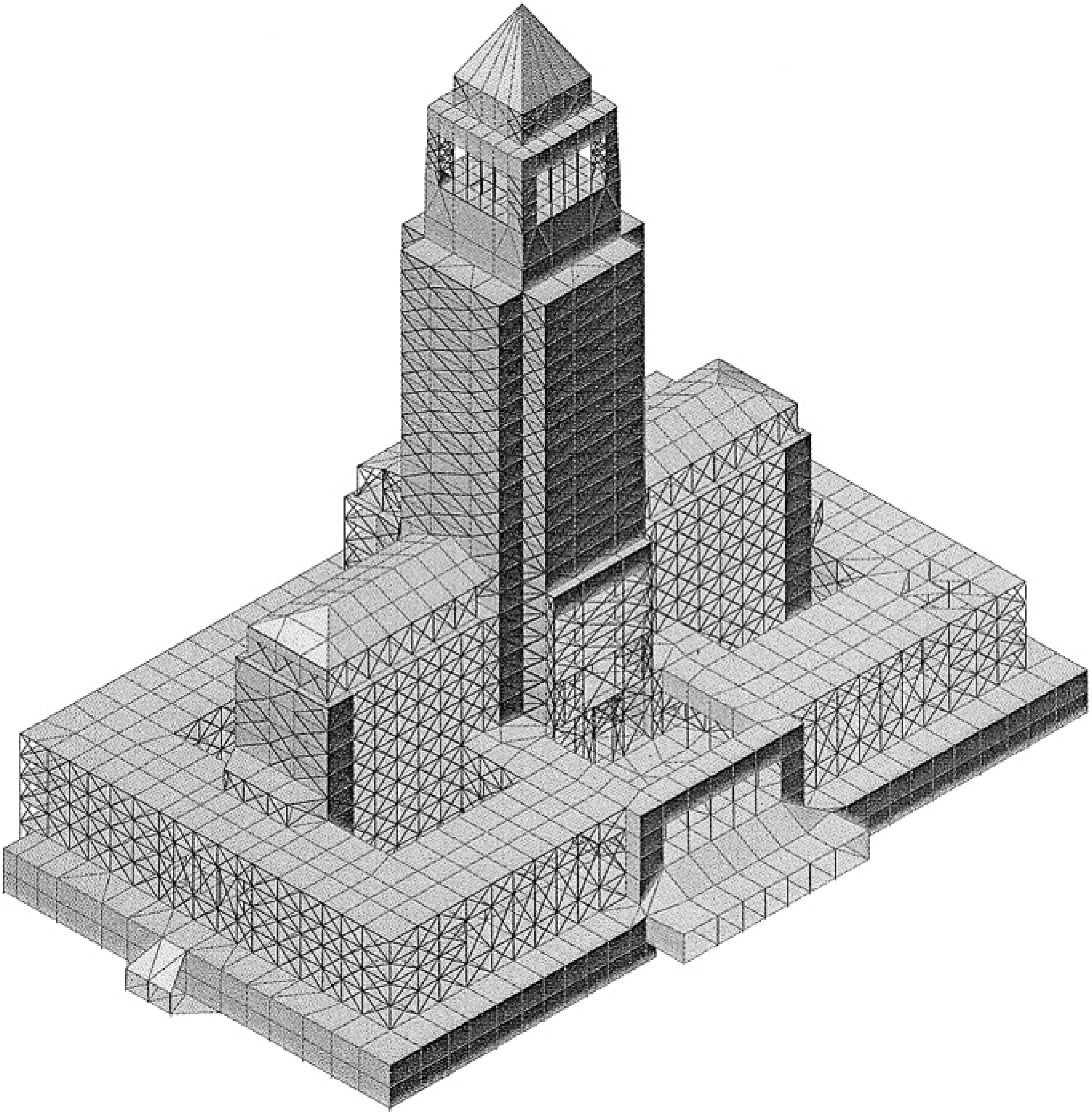


Figure 8. Isometric view of 3-D SAP90 computer model

height of the structure according to the 1991 California Building Code. These forces and twisting-moment time histories were used in theoretical dynamic analysis of the structure. Peak static equivalent floor forces and twisting moments were obtained from the extreme structure displacements computed in the dynamic analysis (Hart *et al*, 1993).[1]

7. COMPUTER MODELS

The project consisted of three phases: Phase I—Evaluation of existing building; Phase II—Development and evaluation of potential strengthening schemes; and Phase III—Final design and analysis. Computer models were developed and/or refined at each phase of the project.

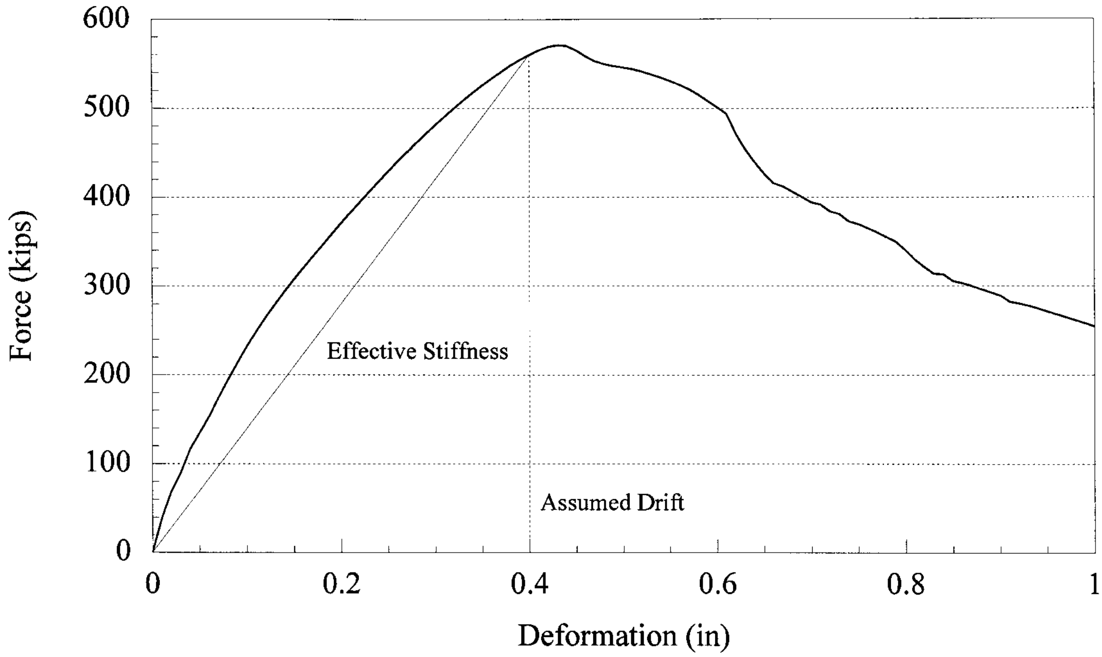


Figure 9. Force–deformation curve of unreinforced masonry infill

In Phase I, linear elastic computer models were developed to assess the global performance of the existing and strengthened buildings (Phases I–III). These models were developed using the structural analysis computer programs ETABS (Habibullah, 1989)[2] and Sap90 (Wilson and Habibullah 1992).[3] A plot of an isometric view of the 3-D SAP90 computer model is shown in Figure 8. Nonlinear finite element models were developed to determine the limit-state behavior of the unreinforced masonry infill. Computer models were also developed to perform nonlinear dynamic analysis for the base-isolated building (Phase III).

The primary steel-frame skeleton, steel bracing, reinforced-concrete walls, unreinforced-masonry infill walls and concrete diaphragms were included in the linear elastic computer models to simulate the behavior of the existing building accurately. In order to assess the dynamic behavior of the building, it was crucial to understand the behavior of the existing unreinforeed masonry walls. These walls represent a significant portion of the overall strength and stiffness of the existing structural system. Nonlinear finite element analysis was performed on typical URM wall configurations of the building to determine their limit-state behavior (Youssef *et al*, 1994).[4] Models of the various wall configurations were developed using the computer program FEM/I (Ewing *et al*, 1990).[5] The material model used by this program accounts for the bi-axial stress states and pre-cracking behavior of masonry. These analyses represent the most comprehensive analytical approach for the evaluation of the limit-state behavior of masonry.

The results from these analyses were used to determine the effective stiffness, strength capacity and effective deformation range of the typical masonry walls. A force–deformation plot from the analysis on a typical URM wall configuration is shown in Figure 9. Frame elements, in a crossbrace configuration, were used to model the masonry walls in the linear elastic model. The section properties of these braces were determined from the results of the nonlinear finite element analyses.

In Phase II, a linear elastic computer model of the base-isolated building was developed using the

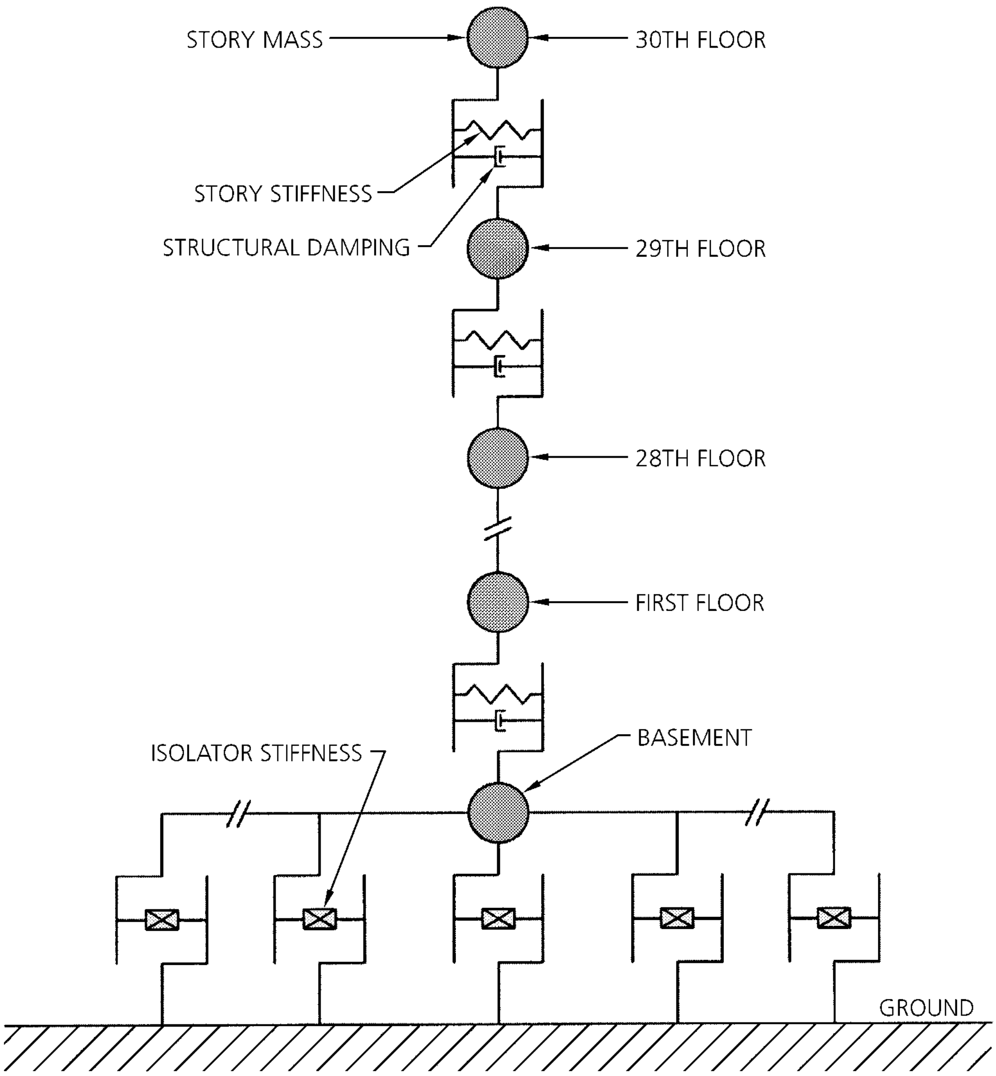


Figure 10. Idealized lumped mass representation of LPM-BI model

computer program ETABS. The isolators were modeled using frame elements with equivalent linear stiffness. Response spectrum analyses were performed to assess the global performance of the isolated building.

In Phase III, nonlinear time history analyses were performed and computer models of the base-isolated building were developed using the nonlinear dynamic analysis computer programs LPM-BI (Kariotis *et al*, 1992)[6] and 3D-BASIS (Nagarajaiah *et al*, 1991).[7] In both the 3D-BASIS and LPM-BI computer models, the plane of isolation is assumed to be rigid, and isolators and sliders are explicitly modeled as biaxial elements. In the LPM-BI model the superstructure is modeled using the stiffness matrix determined from ETABS, assuming a base-isolated condition where the isolators have no lateral stiffness. An idealized lumped mass representation of the LPM-BI model is presented in Figure 10. The 3D-BASIS model uses eigenvalues and eigenvectors from ETABS, assuming a fixed base

Table IV. Existing building periods of vibration

Mode number	Direction	Period (s)			
		Ambient	Forced	ETABS	SAP90
1	East–West	2·38	2·27	2·62	2·78
2	North–South	2·08	2·50	2·44	2·52
3	Torsion	1·08	1·19	1·24	1·39

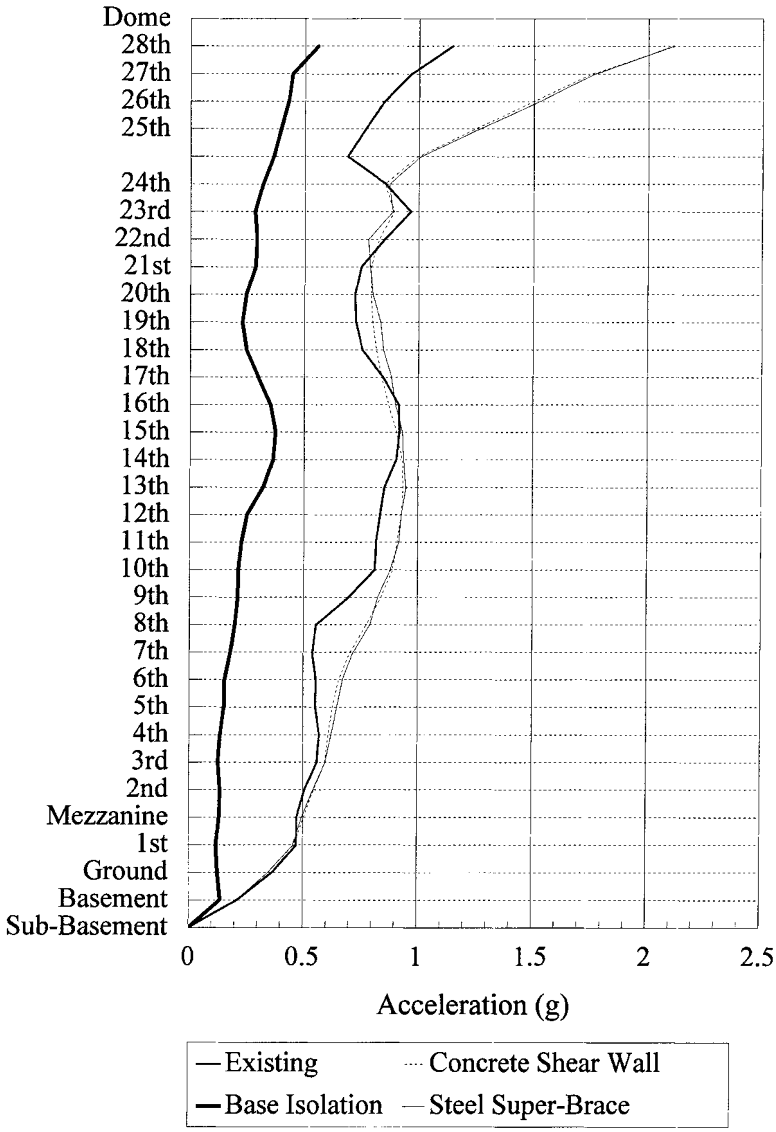


Figure 11. Plot of story accelerations from proposed strengthening schemes

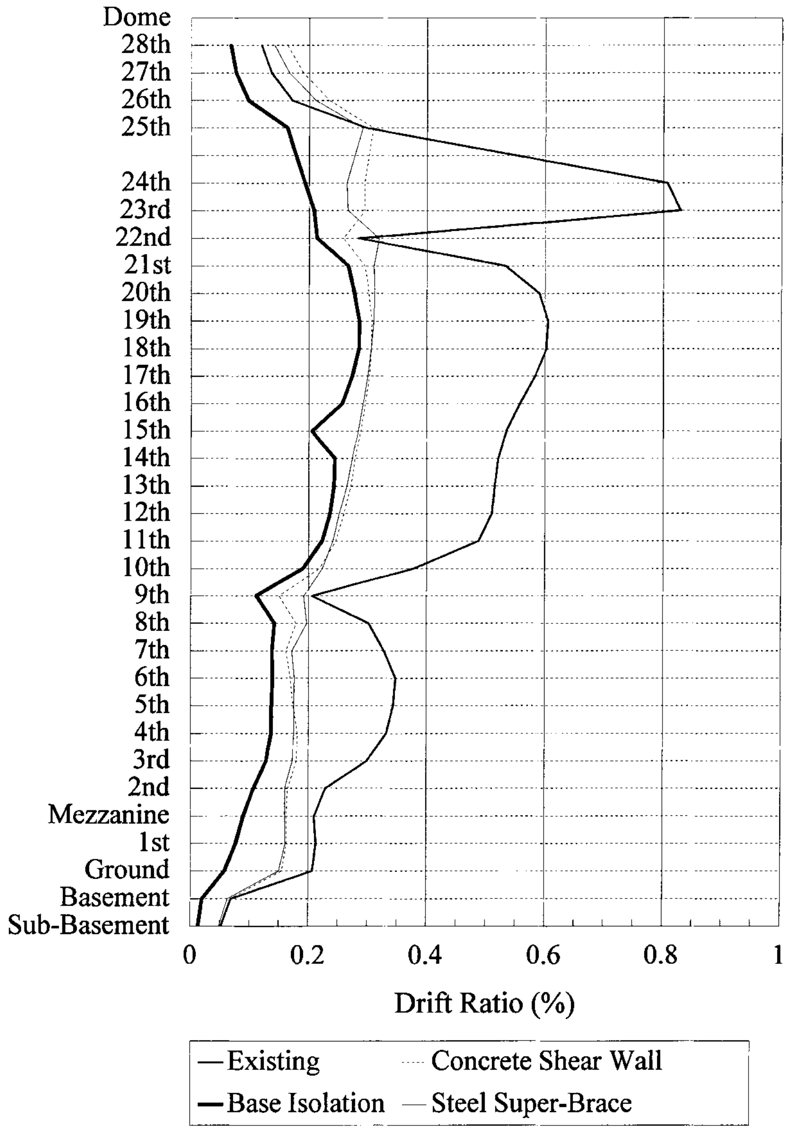


Figure 12. Plot of interstory drifts from proposed strengthening schemes

condition, to model the superstructure. Global damping is provided in the LPM-BI model using Rayleigh's mass and stiffness proportional damping ($C = a_0M + a_1K$), whereas 3D-BASIS uses modal damping.

The viscous dampers at the isolation plane were explicitly modeled in both programs. The viscous dampers at the 24th floor were lumped into a single damper in the LPM-BI computer model, between the 24th and 25th floors. The 3D-BASIS program was not capable of modeling the viscous dampers at the 24th floor. The LPM-BI computer model was used to determine the nonlinear response of the isolation system. The 3D-BASIS computer model was used to verify the results of the LPM-BI analysis.

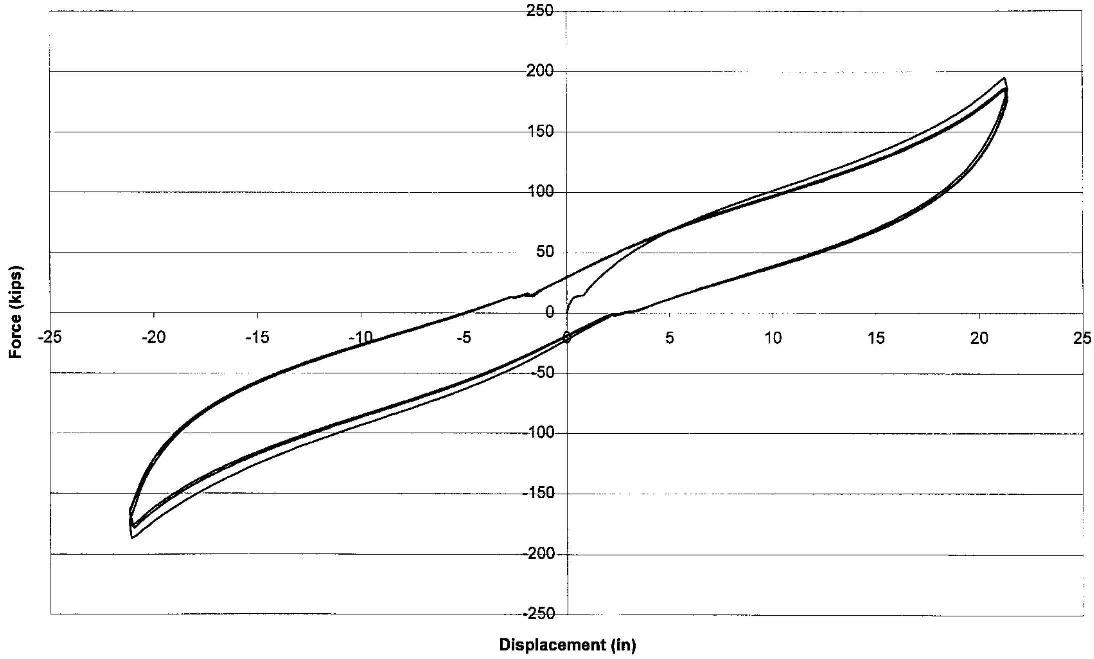


Figure 13. Hystersis loop of 1200 mm prototype bearing

A linear elastic computer model of the base-isolated building was developed using the finite element computer program SAP90. This computer model was used to determine member forces and to evaluate the overturning distribution at the base of the building.

8. EVALUATION OF EXISTING BUILDING

A seismic evaluation of the existing building was performed using the linear elastic computer models. The computer model of the existing building was verified by performing an eigenanalysis and comparing the results with the test data obtained from dynamic testing of the building. The results of the eigenanalysis and dynamic testing are presented in Table IV. The periods obtained from the eigenanalysis were found to be in good agreement with the results of forced-vibration tests.

Response spectrum analyses were performed using the design basis earthquake representing a 10% probability of exceedance in a 50-year time period. Inelastic demand ratios (IDRs) were calculated for the elements of the building. In the tower, where inelastic demands were the highest, the IDRs were in excess of 3·0 for the unreinforced masonry walls. At this level of inelastic demand, significant damage would occur. Typical interstory drift ratios varied between 0·1 and 0·6%. At this level of drift, damage is possible in the exterior masonry walls, terra cotta, partition walls, historic fabrics and all rigid/brittle non-structural systems. The story accelerations were generally between 0·6 and 1·0*g*. At the top of the building, the whipping effect of the tower increased story accelerations to 1·2*g*.

The results of the seismic evaluation indicated that the building in its existing state does not satisfy the seismic life safety criteria established for the project and needs to be strengthened.

9. ENHANCEMENT OF SEISMIC RESPONSE

The evaluation of the structural performance of the existing building to seismic events revealed

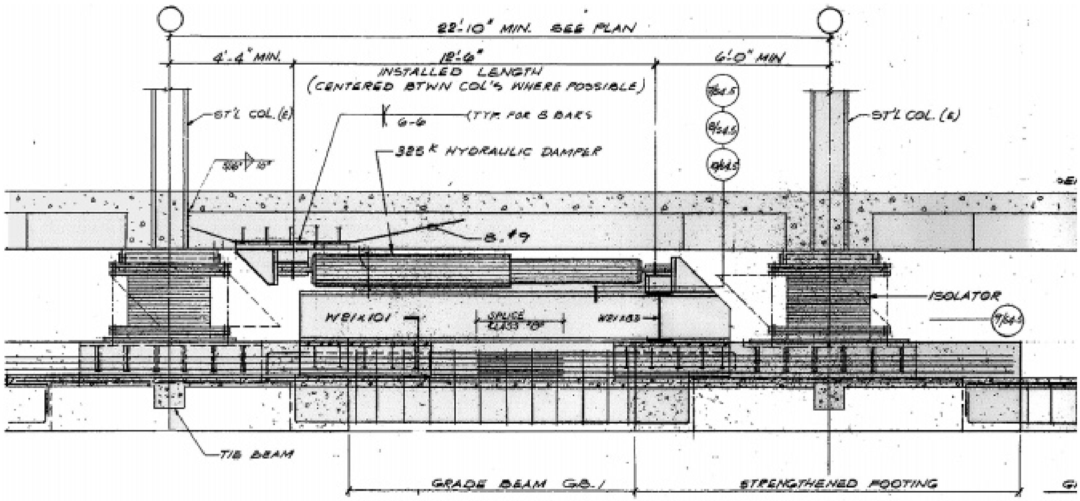


Figure 14. Conceptual detail of viscous damper connection

deficiencies in the lateral-force-resisting system. These deficiencies include insufficient strength of the unreinforced masonry infill, a soft-story at the 24th floor, excessive lateral story accelerations and a poorly-defined lateral-force load path. Various strengthening schemes were proposed to mitigate these deficiencies. These schemes included a reinforced-concrete shear-wall scheme, a base isolation with a

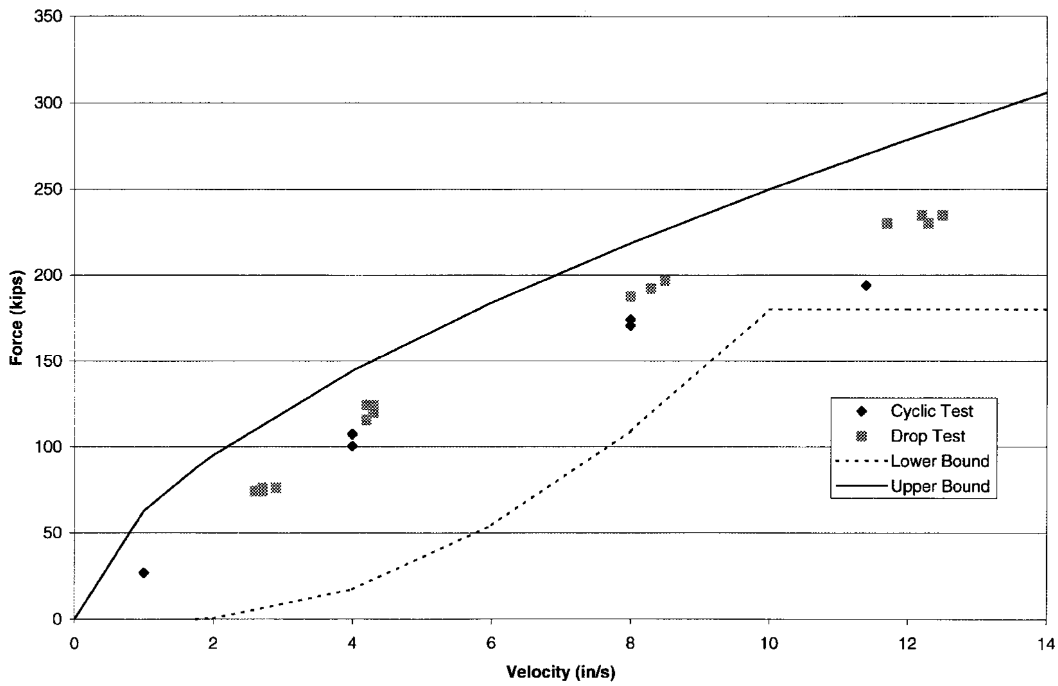


Figure 15. Plot of force–velocity results from prototype damper test

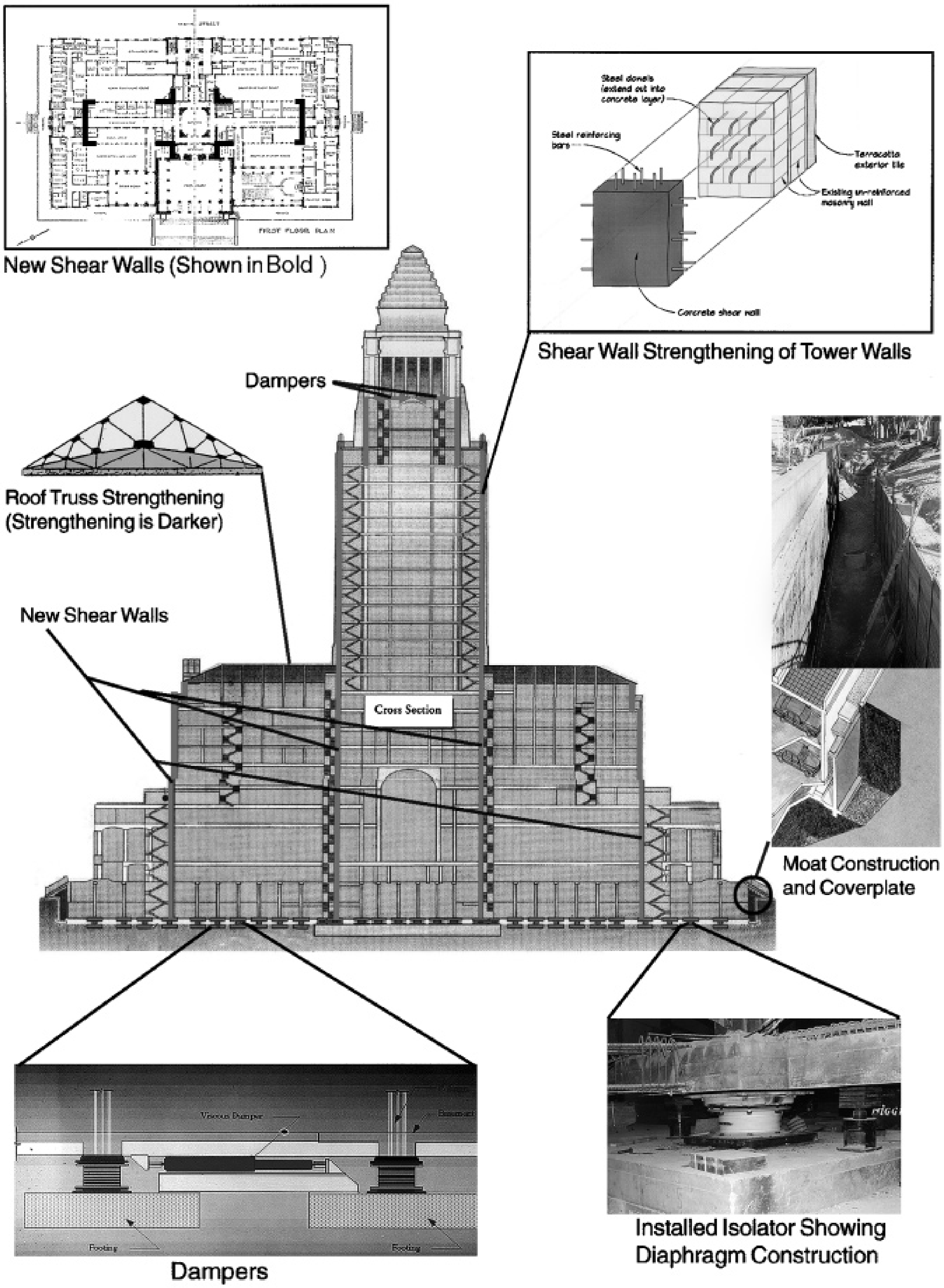


Figure 16. LA City Hall seismic rehabilitation

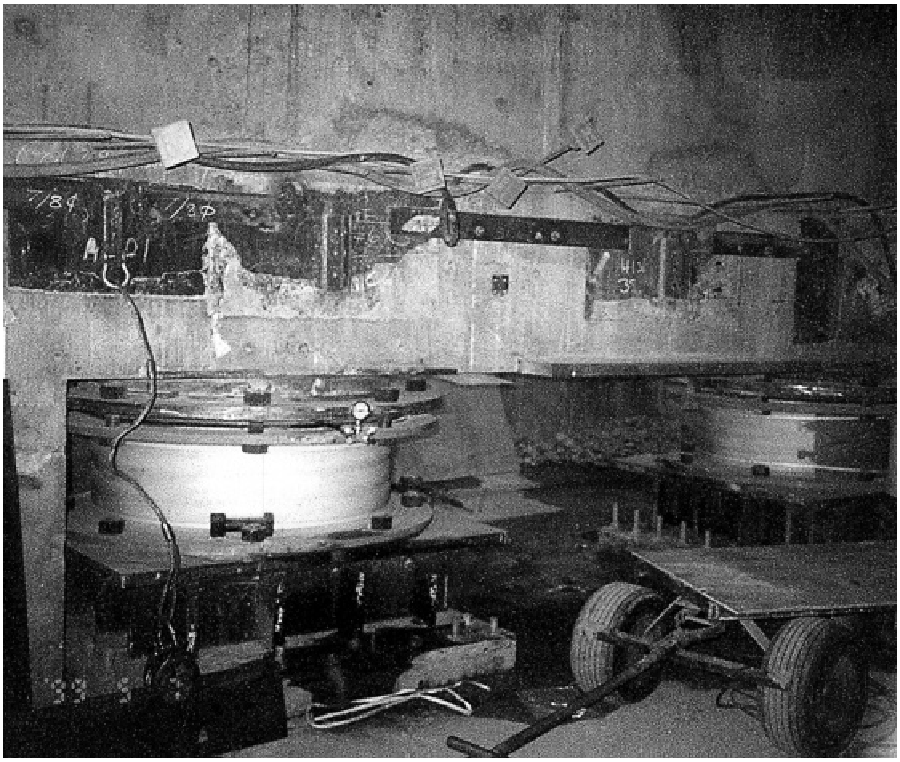


Figure 17. Photograph of HDR bearings installed under tower walls

limited reinforced concrete shear wall scheme, and a reinforced concrete shear wall with steel superbrace scheme.

The reinforced-concrete (RC) shear-wall system consists of adding new RC shear walls to supplement the existing lateral system of the building. The RC shear-wall with steel super-brace system consists of new steel super-braces coupled with new RC shear walls to supplement the existing masonry infill and steel frame system. These proposed conventional schemes increase the building's strength and stiffness, which results in a higher level of seismic demand on the structure.

The base-isolation with limited RC shear-wall scheme consists of seismically isolating the building at the base and adding new RC shear walls. Base isolation effectively decouples the building from ground motions, greatly reducing the level of seismic force transferred to the superstructure. The RC walls add strength, re-distribute seismic overturning forces, stiffen the superstructure increasing the effectiveness of the isolation system, and improve the lateral force load path.

These schemes were evaluated by analysing the results obtained from response spectrum analyses. The results indicated that the conventional strengthening schemes do not significantly reduce the maximum story acceleration in the building and actually amplify the accelerations at the top of the building. The base-isolation scheme significantly reduced the accelerations throughout the height of the building.

Figure 11 compares the story accelerations from the three schemes. All of the proposed strengthening schemes reduced the interstory drift. A comparison of the interstory drifts from the different schemes is shown in Figure 12. The results of these analyses indicated that only the base-isolation scheme was effective in reducing interstory drifts and story accelerations.

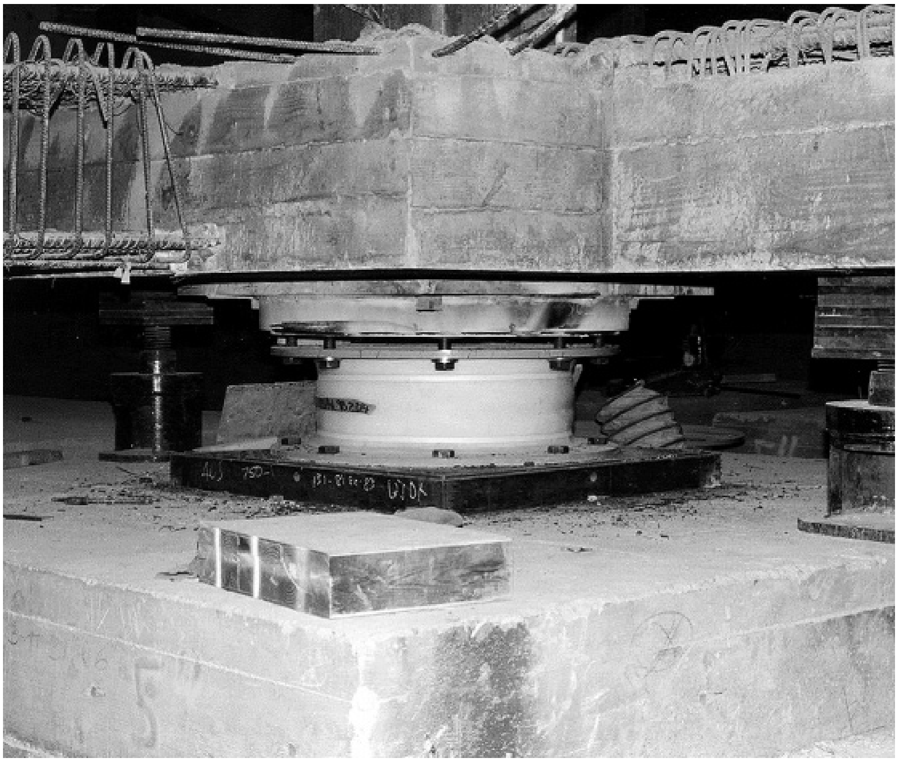


Figure 18. Photograph of HDR bearing installed under stand-alone column

Story accelerations are an indicator of building performance as non-structural elements and building contents are sensitive to variations in acceleration levels. In terms of the seismic goals established for this project, story acceleration has a direct effect on the prevention of failing hazards, insurance of safe means of egress, integrity of the exterior facade, and the protection of the historic interior fabric, emergency telecommunication systems and building contents. As the only proposed strengthening scheme to mitigate the excessive story accelerations in the building, the base-isolation scheme was determined to be the most effective.

10. ISOLATION SYSTEM

The plane of isolation is located just below the basement level and above the existing foundation. The isolation system was designed per the 1994 UBC provisions for seismic isolated structures. It is noted that the components of the isolation system were designed prior to the introduction of viscous dampers into the system. Thus, the loading conditions (maximum displacement, maximum shear, and maximum axial force) used in the design of the isolator units exceed those from the final (hybrid) system. This 11 "over design" provides an additional level of safety to the building.

The isolation system consists of 416 high-damping rubber (HDR) bearings, varying in size from 29·5 to 51·2 in. diameter, and 90 flat sliding bearings. The HDR bearings varying in height from 9·6 to 16·6 in. The sliding bearings are composed of a teflon pad mounted on a natural rubber bearing, and a stainless-steel sliding plate. The overall height of the sliding bearing, excluding the sliding plate, is 4·8 in. The sliding bearings support less than 10% of the total building weight.

An uplift criterion was developed that limits the maximum uplift displacement to 0·5 in, and specifies the number of isolators that can uplift at any given instant in time. The HDR bearings have a very limited tension capacity and cannot tolerate a tension displacement of 0·5 in. without sustaining damage. A loose-bolt connection detail was developed that allows uplift to occur without loading the bearing in tension.

A rigorous test regime was developed for the prototype test phase. The results from these tests were used to determine the bearing properties used in the analysis and design of the isolation system and superstructure. The hystersis loop of a 1200-mm diameter prototype bearing is shown in Figure 13. Bearing stability at a maximum lateral displacement of 21 in. was also verified by tests.

11. SUPPLEMENTAL DAMPING

Near-source ground motions are known to manifest themselves as short-duration, large-amplitude (large-pulse) motions. This type of ground motion results in excessive displacements for base-isolated structures. To provide safety against this type of ground excitation, viscous dampers were introduced into the base-isolation strengthening scheme.

The viscous dampers are located at the plane of isolation and between the 24th and 25th floors. The dampers at the plane of isolation increase the energy dissipation capacity of the isolation system and thus, reduces the energy transmitted to the super-structure. The dampers in the tower add damping to the higher modes of the building response and reduce the whiplash (due to the large mass of the pyramid) at the top of the tower.

Fifty-two viscous dampers, 26 in each direction, will be installed at the plane of isolation. These dampers bridge the plane of isolation, i.e. one end will be connected to the foundation and the other to the underside of the basement slab. Figure 14 shows a conceptual detail of the viscous damper connection. The dampers have a mid-stroke length of 143 in., a force-rated capacity of 300 kips at 50 in/sec, and a stroke of ± 21 in.

Twelve viscous dampers, six in each direction, will be installed between the 24th and 25th floor. These dampers have a force-rated capacity of 225 kips at 10 in. s^{-1}, a stroke of ± 4 in. and a mid-stroke length of 46 in.

Full-scale prototype 225 kip dampers were cyclic-load and drop tested to verify the damper properties in the prototype test phase. The results from the cyclic-load and drop tests were well correlated. The results are shown in Figure 15.

At the conclusion of the design phase, the scaling of the ground motions used in the design was revisited. This was done to allay ‘post-Northridge’ concerns regarding the efficacy of base isolation to large-pulse ground motions. Selected ground motions were scaled to 1997 UBC code levels. Nonlinear time-history analyses were performed using these records. The results of the analyses indicated that the maximum displacement at the plane of isolation was less than the maximum design displacement. The viscous dampers at the isolation plane substantially reduce the displacements at this level. Also, the conscious ‘over design’ of the isolation system at the early stage of the project ensures that the integrity of the system will not be compromised at this level of loading.

12. STRUCTURAL STRENGTHENING

The new RC shear walls add strength and stiffness to the superstructure, redistribute seismic overturning forces, and improve the lateral-force load path (Youssef *et al*, 1995).[8] The new walls are located along the perimeter walls of the tower extending to the basement and along the walls at the North and South ends of the mid-rise extending to the basement. The walls under the tower are 24 in. thick at the basement level. These walls redistribute the seismic overturning forces developed in the tower and reduce the net uplift experienced by the isolators under the walls. Figure 16 graphically illustrates the various components of the strengthening scheme.

The existing basement diaphragm will be demolished and a new 8 in. thick concrete diaphragm will be constructed. Demolition of the existing diaphragm provides access to the foundation and simplifies the installation of the isolators. The new diaphragm system ties all of the isolators together and ensures proper force transfer between the superstructure and isolation system.

A new horizontal brace diaphragm will be added at the roof of the mid-rise to facilitate a transfer of seismic forces from the tower to the new RC walls at the North and South ends of the mid-rise. This diaphragm couples the new tower walls to end walls of the mid-rise providing an additional load path for the seismic forces and reduces the overturning demand at the base of the tower.

Limited diaphragm strengthening will occur at various levels of the building to improve load transfer between structural elements.

The existing foundation system will be strengthened, and individual footings will be strengthened and tied together by a network of tie beams.

13. CONSTRUCTIBILITY

The new 24 in. thick concrete walls at the basement level will also facilitate the installation of the HDR bearings located under these walls. Initially, these new walls will extend to the foundation. Dowels will be provided for the transfer of the gravity loads from the columns to the walls. After construction of these walls, the pedestals of the columns will be removed. The gravity loads will then be transferred to the shear walls through the dowels. The bearings will then be installed under the columns. A horizontal saw cut must be made in each wall at the basement level, to separate the wall from its foundation. The loads will then be transferred back to the columns and bearings. This technique reduces the installation time considerably. Figure 17 is a photograph of HDR bearings installed under the tower wall.

The spread footings located under the stand-alone steel columns will be enlarged and strengthened. Reinforced concrete lifting blocks and beams will be built approximately 3 feet above the top of the footings. The existing steel columns will be stripped of its existing concrete cover and will be encased in RC. Hydraulic jacks will be placed between the lifting block and footing. These jacks will be pressurized to transfer the vertical loads from the column. The section of the steel column between the lifting block and footing will be removed and an isolation bearing installed with a flatjack. The flatjack will then be inflated to a specified axial load, relieving the load on the hydraulic jacks. Figure 18 is a photograph of an HDR bearing installed under a stand-alone column.

To ensure stability of the building during installation of the bearings the columns will be laterally braced until the new basement diaphragm is constructed. The building will be laterally braced by the perimeter and tower walls until all bearings are installed. These walls will be horizontally saw-cut below the basement level to ‘release’ the building and activate the isolation system.

14. CONCLUSION

The development of a performance based approach appropriate for this project was presented. Seismic goals were established to satisfy the desired performance objectives of life safety and damage mitigation. These goals were quantified as specific analytical limit states.

The results of the seismic evaluation of the existing building revealed deficiencies in the lateral force resisting system, including insufficient strength of URM infill, a soft-story condition at the 24th floor, excessive story accelerations and a poorly defined load path.

Base isolation supplemented with viscous dampers was determined to be the optimum strengthening scheme based on the performance criteria adopted for this project. This scheme was the only one that was effective in reducing interstory drifts and story accelerations. Viscous dampers were introduced at the plane of isolation to provide safety against large pulse (near source) ground motions. Additional dampers are located at the 24th floor where the soft-story condition exists. These dampers reduce the whiplash effect (increased story accelerations) at the top of the tower. This final scheme addresses the existing structural deficiencies of the building and satisfies the desired seismic performance objectives of the City of Los Angeles.

The hybrid base isolation and supplemental damping scheme provides a level of life safety and damage control that exceeds the level provided by conventional schemes.

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