NABIH YOUSSEF & ASSOCIATES
Structural Engineers

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Published by Elsevier Science Ltd 1996
Paper No. 2090. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

APPLICATION OF HYBRID DAMPING SYSTEM WITH BASE ISOLATION FOR THE SEISMIC RETROFIT OF LOS ANGELES CITY HALL

Nabih Youssef

The Los Angeles City Hall has a significant historical relevance to the City of Los Angeles and Southern California. It is the first building to exceed the 150 foot height limitation for all privately constructed buildings in Los Angeles. The Los Angeles City Hall was designed prior to the enactment of explicit seismic design requirements. The building does not meet the current code requirements for seismic life safety and has architecturally significant elements that are threatened by future earthquakes. A seismic retrofit of the structure is being undertaken in order to strengthen the building to meet the life safety and damage mitigation objectives of the City of Los Angeles and to maintain the integrity of the building's exterior facade and protect the historic interior fabric. Current plans call for the building to be retrofitted using a hybrid damping system consisting of base isolation bearings and supplemental dampers at the base and top of the building. This scheme provides a level of life safety and damage control that exceeds the level provided by conventional strengthening schemes. Moreover, the analysis results indicates that the use of base isolation bearings in conjunction with viscous dampers is the most effective technique for reducing inter-story drift, story shear and acceleration for the Los Angeles City Hall.

This paper discusses the criteria used to evaluate the performance of the existing building and potential strengthening schemes. The seismic performance of the existing building and various proposed strengthening schemes are discussed. The efficacy of supplementing the damping of the isolation system with viscous dampers is evaluated and discussed. Also, the performance of the isolated building with supplemental damping at the 24th floor is presented. It is shown that the hybrid damping system is the most effective strengthening scheme for controlling the response of the Los Angeles City Hall to earthquake ground motions.

President, Nabih Youssef & Associates, Los Angeles, California, U.S.A.

800 Wilshire Boulevard, Suite 510 • Los Angeles, California 90017 • 213/362-0707 • FAX: 213/688-3018

INTRODUCTION

The Los Angeles City Hall, built in 1926 is a 32 story, steel frame building with riveted connections. The west elevation of the building is shown in Fig. 1. 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 prior to the enactment of explicit seismic design requirements. Therefore, the structure was not specifically designed to resist earthquake generated forces and does not have a distinct seismic force resisting system to provide a consistent and well detailed seismic load path. However, there are a number of building 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 steel wind 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 floor 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 floors of the building. Steel bracing exists at the four corners of the tower from the sub-basement to the 22nd floor. Belt trusses tying the corners of the tower together occur at the ninth and 22nd floors. Fig. 2 shows the steel brace system.

Over the past 65 years, regional earthquakes have caused damage to this building. Masonry infills and concrete walls have suffered cracking. 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. The damage can be attributed to the significant change in the lateral stiffness and strength of the building at this level. The lateral force resisting system of the 23rd and 24th floors consist of the horizontal diaphragm, steel frame and unreinforced masonry walls. The structural components that contribute to the lateral force resisting system of the 25th floor are the horizontal diaphragm, steel frame and concrete walls, piers and spandrels. The diaphragm, steel frame, unreinforced masonry infill walls and belt truss participate in resisting the seismic forces at the 22nd floor.

In order to meet the life safety and damage mitigation objectives of the City of Los Angeles and to maintain the integrity of the building's exterior facade and protect the historic interior fabric from damage, a seismic rehabilitation of the building is currently in progress (Youssef et al., 1994). Various strengthening schemes, such as reinforced concrete shear wall system, reinforced concrete shear walls with steel super brace system, base isolation system with supplemental dampers, etc. have been proposed and evaluated. Evaluation of the analytical results indicate that strengthening the Los Angeles City Hall using base isolation bearings in conjunction with supplemental viscous dampers at the isolation level is the most effective strengthening scheme to satisfy the performance goals of the City of Los Angeles (Youssef et al., 1994, Youssef et al., 1995).

In principle, seismic isolation decouples a building from the damaging effects of ground motion, reducing the energy transmitted to the super-structure. This is achieved by increasing the fundamental period of the isolated building system, such that, high energy ground motions with frequencies in the 3-10 Hz range are effectively filtered by the isolation system. Thus, allowing only relatively low levels of accelerations to be transmitted to the isolated structure. Viscous damping devices provide a mechanism especially designed to dissipate energy. These devices enhance the performance of the structural system by increasing its energy dissipating capacity. In order to reduce the seismic response of the twenty fourth floor, which has experienced significant damage during previous earthquakes, viscous dampers will be installed between the twenty fourth and twenty fifth floors.

In this paper, the criteria used to evaluate the performance of the existing building and potential strengthening schemes is discussed. The seismic performance of the existing building and various proposed strengthening schemes are also discussed. The analysis and design techniques utilized in the evaluation of the hybrid damping system in controlling the seismic response of the building are presented. The results of the analyses conducted to determine the seismic performance of the base isolated building are also presented.

CRITERIA

The performance of the Los Angeles City Hall to earthquakes depends on several factors which are not explicitly considered in code based approaches such as, earthquake intensity, material properties, quality of construction, structural configuration/irregularities, force/deformation characteristics of the lateral force resisting system, etc. 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 City Hall is prohibited by current codes. As such, code based approaches are not appropriate for the evaluation of the Los Angeles City Hall.

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

In order to evaluate the ability of the Los Angeles City Hall to meet the seismic goals, the goals were quantified in engineering criteria. The engineering criteria defines the performance goals in terms of specific analytical limit states. The quantification of the seismic goals to 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.

ANALYSIS PROCEDURE

The performance evaluation of the existing building and the proposed strengthening schemes were based on results from response spectra analyses and linear and nonlinear time history analyses. A set of seven earthquake time history records, appropriate for the site, were used. Various tests were performed to determine the properties of the materials used to construct the existing building. The results of these tests were used to develop computer models of the building. Dynamic tests were performed on the existing building to verify the analytical models.

Earthquake Ground Motions

The analysis and design for the Los Angeles City Hall seismic rehabilitation is based on time history records. These records are scaled to meet the site specific response spectra. Site specific spectra which account for near fault effects and dispersion effects for distant events have been developed. 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. A total of seven time history records, appropriate for the site, have been utilized. Two of the seven records account for the potential near fault activity at the site.

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.

Ambient and forced vibration tests were performed to determine the dynamic properties of the existing building. The ambient vibration test measures the response of the building to vibrations which occur at the building site, such as vehicle traffic, wind, occupants, etc. Forced vibration tests were performed using a forced vibration oscillator to vibrate the building. The 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.

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 masonry. 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.

Computer Models

Linear elastic computer models were developed to asses the performance of the existing and strengthened buildings. The strengthened building model was developed by introducing the structural components, required for the strengthening scheme, into the existing building model. Computer models were also developed to perform nonlinear dynamic analysis for the base isolated building.

Nonlinear Finite Element Analysis. To accurately model the existing building, the primary steel frame skeleton, steel bracing, concrete and unreinforced masonry infill walls and concrete diaphragms were included in the computer models. In order to accurately assess the dynamic behavior of the building, it was crucial to understand the behavior of the existing unreinforced masonry walls. These walls represent a significant portion of the overall strength and stiffness of the existing structural system. Nonlinear finite element computer analysis was performed on typical URM wall configurations of the building to determine their limit state behavior (Youssef, 1994). Fig. 3 shows the force-deformation behavior of a typical unreinforced masonry infill wall.

The material models used in these analyses account for the biaxial 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 of the typical walls. These walls were modeled by panel elements in the linear elastic model of the building. The effective stiffness of the typical walls were used to adjust the thickness of these panel elements.

Linear elastic computer models of the existing and strengthened building were developed using the structural analysis computer program ETABS (Habibullah, A., 1989). These models were developed to assess the seismic performance of the building and to determine the global dynamic properties of the building.

Nonlinear Dynamic Analysis. Computer models of the base isolated building were developed using the nonlinear dynamic analysis computer programs LPM-BI (Ewing et al., 1987) and 3D-BASIS (Nagarajaiah et al., 1991). In both the 3D-BASIS and LPM-BI computer models the plane of isolation is assumed rigid and isolators and sliders are explicitly modeled as bi-axial elements. The super-structure, in LPM-BI, is modeled using the stiffness matrix determined from ETABS assuming a base isolated condition where the isolators have no lateral stiffness. The 3D-BASIS model uses eigenvalues and eigenvectors from ETABS, assuming a fixed base condition, to model the super-structure. Global damping is provided in the LPM-BI model using Newmark's mass and stiffness proportional damping, whereas 3D-BASIS uses modal damping. Viscous dampers at the isolation plane are explicitly modeled in both programs. The viscous dampers at the twenty fourth floor are lumped into a single damper, between the twenty fourth and twenty fifth floors, for the x and y directions. The 3D-BASIS program is incapable of modeling the viscous dampers at the twenty fourth floor. The LPM-BI computer model was used to determine the nonlinear response of the isolation system. The 3D-BASIS computer model of the base isolated building was developed to verify the results of the LPM-BI analysis.

A linear elastic computer model of the base isolated building was developed using the finite element computer program SAP90 (Wilson et al., 1992). This computer model was used to determine member forces and to evaluate the overturning distribution at the base of the building. Fig. 4 shows the SAP90 model of the building.

EVALUATION OF EXISTING BUILDING

The computer models of the existing building were used to evaluate the seismic performance of the building during an earthquake and to identify any deficiencies in the lateral force resisting system. The model of the existing building was verified by performing an eigen-analysis and comparing the results with the test data obtained from dynamic testing of the building. The periods obtained from eigen-analysis were found to be in good agreement with the results of forced vibration tests conducted on the existing building. The comparison of the periods is shown in Table 1.
To evaluate the performance of the existing building, time history analyses were performed. In this paper, response of the building due to the ground motion from the 1994 Northridge Earthquake, Pacoima Dam Downstream Station, components S5E and S45W, scaled to the site-dependent response spectra is presented.

The maximum story acceleration obtained from the analysis is found to be about 3.3g in the transverse direction and is shown in Fig. 5. At the top of the building, the whipping effect of the tower caused the story acceleration to be 3.3g. At these level of acceleration significant damage to non-braced items can be expected. It should be noted, however, that these results are artificially high since the super-structure is assumed to remain elastic. Yielding of members would be anticipated at significantly lower levels of acceleration, which would increase the hysteretic damping of the structure and consequently would reduce the acceleration response.

Fig. 6 shows that the inter-story drift ratios for the building are between 0.003 and 0.018 in all stories, except the 24th and 25th floors where the maximum drift ratio is 0.020. Significant damage to the exterior masonry walls, terra cotta, partition walls, historic fabrics and all rigid/brittle non-structural systems can be expected to occur at this level of drift. The results of the analyses are in good agreement with the observed damage from past earthquakes.

The results of the analyses performed for the existing building indicate that the Los Angeles City Hall does not satisfy the seismic life safety criteria. Therefore, the seismic response of the existing building must be controlled in order to provide life safety and to reduce the structural damage during an earthquake.

CONTROL OF SEISMIC RESPONSE

The evaluation of the performance of the existing building to seismic events revealed deficiencies in the lateral force resisting system. Various strengthening schemes, such as, reinforced concrete shear walls, reinforced concrete shear walls with base isolation, reinforced concrete shear walls with steel super brace system, etc. were proposed to address these deficiencies.

Proposed Strengthening Schemes. The reinforced concrete (RC) shear wall system consists of adding new RC shear walls to supplement the existing lateral system of the building. The reinforced concrete shear wall with steel super brace system consists of new steel super-braces which couple with new RC shear walls to supplement the existing masonry infill and steel frame system. These proposed conventional schemes increase the building's stiffness, which leads to a higher level of seismic demand placed on the structure. The reinforced concrete shear wall with base isolation scheme consists of base isolating the building in combination with new RC shear walls. Base isolation effectively decouples the building from ground motions, greatly reducing the level of seismic force transferred to the building.

These schemes were evaluated by analyzing the results obtained from response spectrum analyses. Fig. 7 shows the maximum story acceleration for the existing building and the various proposed strengthening schemes. As can be seen from the figure, the RC shear wall and the RC shear wall with steel super brace 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 greatly reduces the accelerations throughout the building. The maximum inter-story drift ratio for the existing building and the various strengthening schemes is shown in Fig. 8. The figure shows that all of the proposed schemes are effective in reducing inter-story drift. The results of these analyses indicate that only the base isolation scheme is capable of reducing inter-story drifts and story accelerations. The other schemes reduce inter-story drifts, while amplifying the acceleration at the top of the building. Based on these results, base isolation was identified as the most effective strengthening approach for the building.

The selected seismic strengthening of the Los Angles City Hall consists of base isolation of the structure in combination with RC shear walls and viscous dampers at the top and at the base of the building. Reinforced concrete walls are added to the existing building to add strength, re-distribute overturning forces and to reduce the effective building period of vibration. In order to reduce the response of the isolated building to large pulse ground motions viscous dampers have been introduced at the isolation plane and top of the tower.

New Concrete Shear Walls

The main contribution of the new concrete shear walls in this strengthening scheme is to provide additional lateral strength necessary to resist the seismic forces in the super-structure.

The design of the new concrete shear walls was based on the strength design procedure (Youssef et al., 1995). Forces in the shear walls, obtained from the analyses, were used without any load factors and were combined with factored dead and live loads. The walls were designed to have nominal strength which would exceed the seismic demand and thus, the inelastic demand ratio of the new concrete shear walls was less than or equal to one. Due to the historic interior fabric and architecturally significant elements of the building, the construction of the new shear walls will consist of poured in place concrete to reduce the possibility of damage to these elements.
Boundary elements were designed without considering any contribution from the existing steel columns at the ends. Boundary elements were provided within the walls adjacent to the existing steel columns. Dowels were provided to transfer shear forces from the new wall boundary elements to the existing steel columns at the ends of the wall. As designed, the composite action of the new boundary element and steel column exceed code requirements.

Base Isolation

Base isolation decouples a building from the damaging effects of ground motion. This is accomplished by increasing the period of the isolated building system, such that, the dominant frequencies of the ground motion are filtered by the isolation system. Thus, reducing the energy transmitted to the super-structure.

In order to make base isolation more effective, it is necessary to shorten the period of the existing structure. This is accomplished by adding concrete shear walls to the perimeter of the tower and to the ends of the mid-rise. The periods of the fixed-base (non-isolated) strengthened building are shown in Table 2.

In order to assess the performance of the base isolated building, non-linear three-dimensional dynamic analyses were performed using the dynamic analysis program LPM-BI. This program is the base isolation version of the LPM/I (Ewing et al., 1987) computer program. The maximum nonlinear response of the isolated building without any supplemental dampers to the 1994 Northridge Earthquake, Pacoima Dam Downstream ground motion are presented in Figs. 5 and 6. Results of the nonlinear time history analysis indicate that a significant reduction in story accelerations and inter-story drift ratios is achieved.

Supplemental Damping

Conventional design practice relies upon the inelastic response of the structure to dissipate seismic energy. This approach requires large inter-story drifts to achieve the necessary hysteretic energy dissipation. Large inter-story drifts may result in substantial damage to historic/brittle non-structural elements. An alternative approach uses supplemental damping devices to increase the energy dissipation capacity of the structure. In this approach, seismic energy is dissipated in devices specifically designed for this purpose, thus reducing the earthquake demand forces in primary structural elements.

The use of mechanical damping devices to reduce the earthquake response of buildings is gaining popularity among structural engineers. A large number of energy dissipation devices have been studied experimentally and a large number of damping devices have been implemented in various structures as part of the earthquake resistant system. These devices fall into three main classes - friction, metallic, and viscous and visco-elastic (Hanson, 1993). Friction and yielding metallic damping devices are characterized as being force limited and highly non-linear. The damping provided by these devices depend on amplitude. The forces developed by viscous and visco-elastic devices are displacement and velocity dependent.

Experimental studies have demonstrated that linear viscous dampers are effective in reducing inter-story drifts (Constantinou et al., 1993). Fluid viscous dampers dissipate energy by allowing fluid to pass through a small orifice at high rates of speed which result in the transformation of mechanical energy into heat. Fluid viscous dampers that are designed to behave as linear viscous devices introduce damping forces which are out-of-phase with drifts and column bending moments.

A large pulse ground motion excitation would result in excessive displacement at the plane of isolation and accelerations in the super-structure. For providing safety against large pulse ground motions, it has been decided to provide fifty two viscous dampers, twenty six in each direction at the isolation plane (7% of the critical damping) and twelve viscous dampers, six in each direction (30% of the critical damping) between the twenty fourth and twenty fifth floors.

In order to evaluate the effectiveness of the viscous dampers nonlinear time history analyses were performed. In these simulations, dampers were introduced at the plane of isolation and between the twenty fourth and twenty fifth floors in the computer model. Linear viscous dampers were evaluated. The damping force of these dampers were modeled as

$$F_d = \text{sign}(V) * C * V^n \qquad (1)$$

where

F_d = Viscous damping force.

V = Relative velocity between the 24th and 25th floor.

C = Viscous damping coefficient.

n = Positive exponent.

For linear viscous damping, n = 1. Nonlinear time history analyses were performed with linear viscous dampers located at the plane of isolation and at the twenty fourth floor. The level of supplemental damping at the plane of isolation was varied from 0% to 7 % of critical. It is shown in Table 3 that significant reduction in displacement at the plane of isolation can be achieved by installing linear viscous dampers at the plane of isolation. The nonlinear responses of the isolated building with viscous dampers at the plane of isolation to the 1994 Northridge Earthquake, Pacoima Dam Downstream ground motion are shown in Figs. 9 and 10. For the purpose of evaluating the effectiveness of dampers at the twenty fourth floor, nonlinear analyses were performed for damping values ranging from 30% to 60% of critical damping. A nonlinear dynamic analysis of the base isolated building with dampers at the plane of isolation and RC shear walls between the 24th and 25th floors was also performed. The responses of the base isolated building with linear viscous dampers at the isolation level and 24th floor to the 1994 Northridge Earthquake, Pacoima Dam Downstream ground motion are presented in Figs. 11 and 12. Also shown in these figures is the response of the isolated building with RC shear walls at the 24th floor.

ANALYSIS OF RESULTS

Figs. 5 and 6 present the story acceleration and inter-story drift ratio of the existing building and the base isolated building without any supplemental damping. The base isolation significantly reduced the story accelerations and inter-story drift ratios of the building. The inter-story drift ratios for the building are generally between 0.001 and 0.002, with a maximum drift ratio of 0.0021 between the 24th and 25th floors. The maximum story acceleration for the base isolated structure between sub-basement and the twenty fourth floor is 0.2g. The story accelerations increase significantly from 0.2g at the twenty fourth floor to a maximum value of 0.4g at the top of the building.

Fig. 9 shows that addition of 7% supplemental damping at the plane of isolation does not help to reduce the whiplash effect of the twenty fourth floor. However, when dampers were added to the twenty fourth floor, the whiplash effect was reduced as can be seen in Fig. 11. Fig. 11 shows that the acceleration gradient above the twenty fourth floor can not be controlled by adding concrete shear walls at that level, rather the level of acceleration increases if concrete shear walls are added. It is shown in Fig. 11 that the level of acceleration can be effectively controlled by installing viscous dampers at the twenty fourth floor.

The introduction of 7% of critical damping at the isolation plane results in significantly reduced displacements at the plane of isolation and base shears. Table 3 shows that the peak displacements at the isolation plane are reduced by 15% to 20% for large pulse ground motions. The table also shows that a reduction of 10% to 20% in maximum base shears can be achieved by installing dampers at the plane of isolation that corresponds to 7% of critical damping. Figs. 7 and 8 show that modest levels of damping added at the isolation plane is effective in reducing the response at the plane of isolation.

Although, higher levels of damping at the isolation level will lead to a substantial reduction of the response, consideration must be given to the effect that the increased damping has on the response of the super-structure. The analysis indicated that the response of the building in the tower increased when higher levels of supplemental damping were added at the plane of isolation. Figs. 9 and 10 show that for 7% damping at the isolation level, a 10% increase in maximum story acceleration occurs between the 9th and 24th floors, while drift ratios increase by 20%. The increased response of the tower is attributed to the coupling of modes that occurs when damping is added at the isolation plane.

The results of the analyses indicate that the combination of viscous dampers at the plane of isolation and the twenty fourth floor reduces the inter-story drifts and story accelerations in the upper level of the building. It is shown in Fig. 12 that the maximum inter-story drift above the 24th floor are reduced by approximately 10%. At this level of drift the masonry infills may experience minor cracking, as indicated by the nonlinear finite element analysis of the masonry infills.

Role of Concrete Shear Walls

Analytical results indicate that a small portion of the seismic forces from the tower is transferred to the perimeter walls of the mid-rise and podium. Thus, the isolation bearings under the tower walls were found to experience large net upward displacements. In order to reduce these displacements, thick concrete outrigger walls were designed for the sub-basement and basement levels. These walls distribute axial loads over a number of bearings when subjected to overturning moment. As the length of these walls increase the axial force in the bearings decrease, thus reducing the net vertical displacements. At columns where net vertical displacements are anticipated, specially designed loose bolt connection details are specified. This prevents damage to the isolation bearing while allowing the column to vertically displace.

At the sub-basement level, two-feet thick, two-story high concrete shear walls were added to carry the existing column loads during installation of the base isolation bearings. Dowels are provided for transferring the full gravity load of the columns to the new walls. After these walls are constructed, the pedestals of the columns will be removed for the installation of the isolation bearings. The gravity loads in the columns will be transferred to the shear walls through the dowels, thus assuring the stability of the structure. After the installation of the isolation bearings is completed a horizontal saw cut will be made in each wall at the sub-basement level. The walls separated from its foundation, will transfer the gravity loads back to the columns and the bearings under them. This installation technique utilizing the concrete shear walls will reduce a considerable amount of installation cost and time.

CONCLUSIONS

Reinforced concrete walls used as a conventional strengthening scheme, are effective in minimizing damage due to drift and inelastic demand on the structural elements. However, concrete shear walls if used as a conventional strengthening scheme, are not effective in controlling the acceleration levels in the building. The large acceleration will result in damage to the exterior facade, historic interior fabric and the emergency telecommunication systems at the top of the building. Moreover, it was found that the number of walls and the thickness required could not be accommodated in the building without sacrificing useable space and the historic interior fabric.

The use of the hybrid damping system has been found to be an effective technique for reducing inter-story drift, story shear and acceleration for the Los Angeles City Hall. Base isolation reduces the amount of energy an earthquake can transmit to the super-structure. The introduction of viscous dampers at the plane of isolation significantly reduces displacements at the isolation plane and thus protects the building against large pulse ground motions. The supplemental damping added to the 24th floor controls the response of the upper levels of the building. The added damping does not affect the response at the plane of isolation. The dampers at the 24th floor essentially adds damping to the higher modes of the building which is decoupled from the isolated mode.

The hybrid damping system consisting of base isolation and supplemental damping at the base and top of the building provides a level of life safety and damage control that exceeds the level provided by conventional schemes. Moreover, the analytical results indicate that the hybrid system is the most effective strengthening scheme for controlling the response of the Los Angeles City Hall to earthquake ground motions.

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Table 1. Existing Building Periods of Vibration.

Mode No.	Dir.	Period (sec.)			
		Ambient	Forced	ETABS	SAP90
1	E - W	2.38	2.50	2.62	2.78
2	N - S	2.08	2.27	2.44	2.52
3	Rot.	1.08	1.19	1.24	1.39

Table 2. Fixed Base Strengthened Building Periods.

Mode Number	Direction	Period (sec.)
1	North-South	1.73
2	East-West	1.71
3	Torsion	0.82

Table 3. Response at Isolation Plane of Isolated Building with 7% Damping at Isolation Level to Large Pulse Ground Motions.

Record	Scale Factor	Max. Displ. % Reduction	Max. Base Shear % Reduction
El Centro	1.69	16	10
James Road	1.0	21	18
Sylmar	1.0	19	9

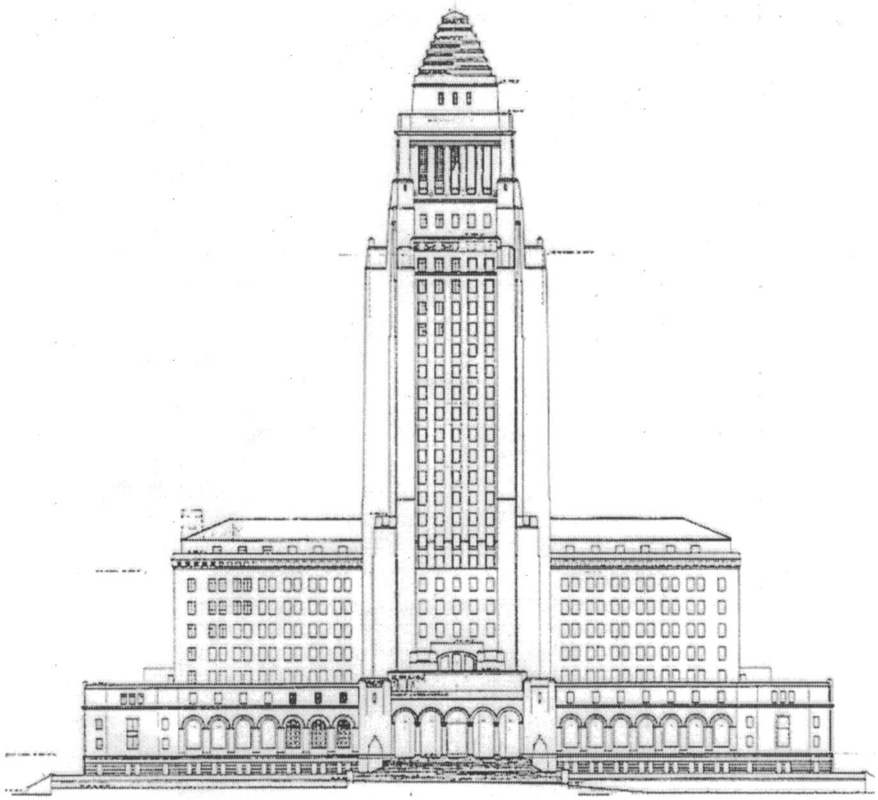

Figure 1. West Elevation of Los Angeles City Hall.

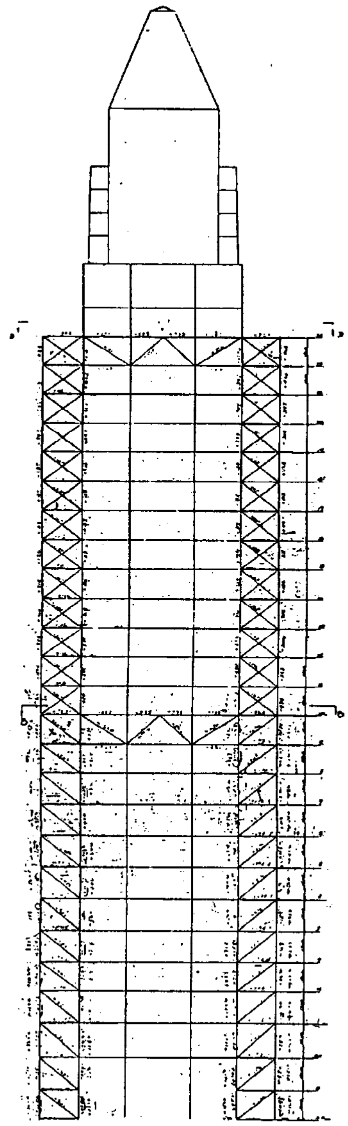

Figure 2. Steel Wind Brace System.

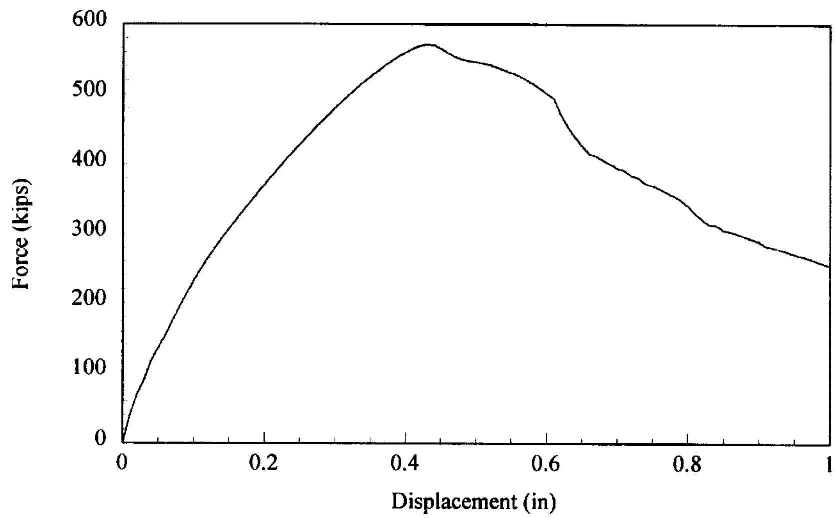

Figure 3. Force-Deformation Behavior of Unreinforced Masonry Infill.

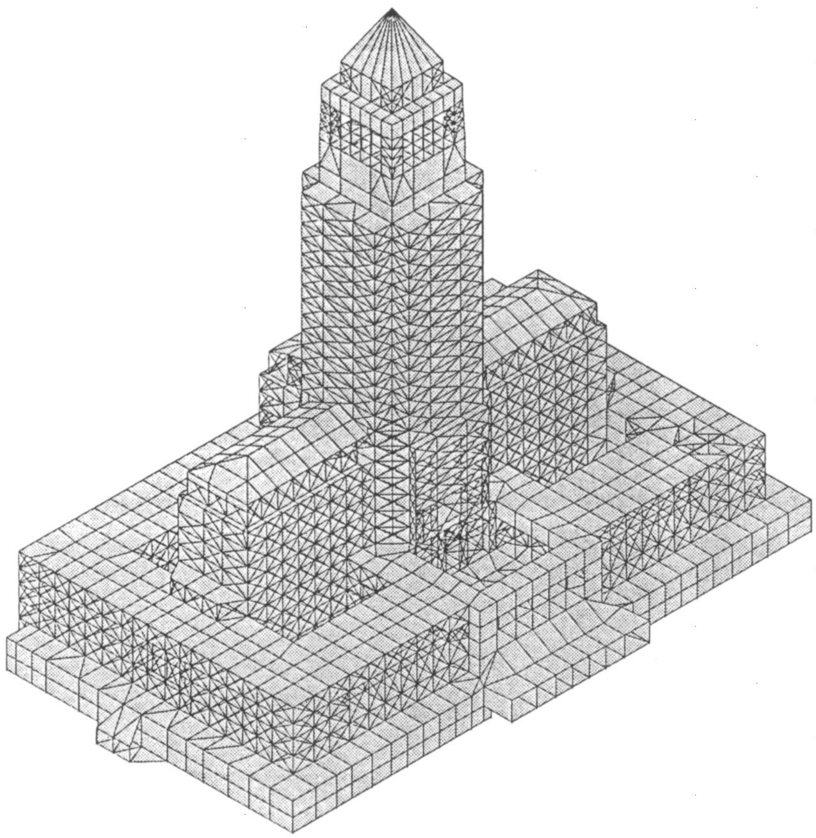

Figure 4. SAP90 Computer Model of Los Angeles City Hall.

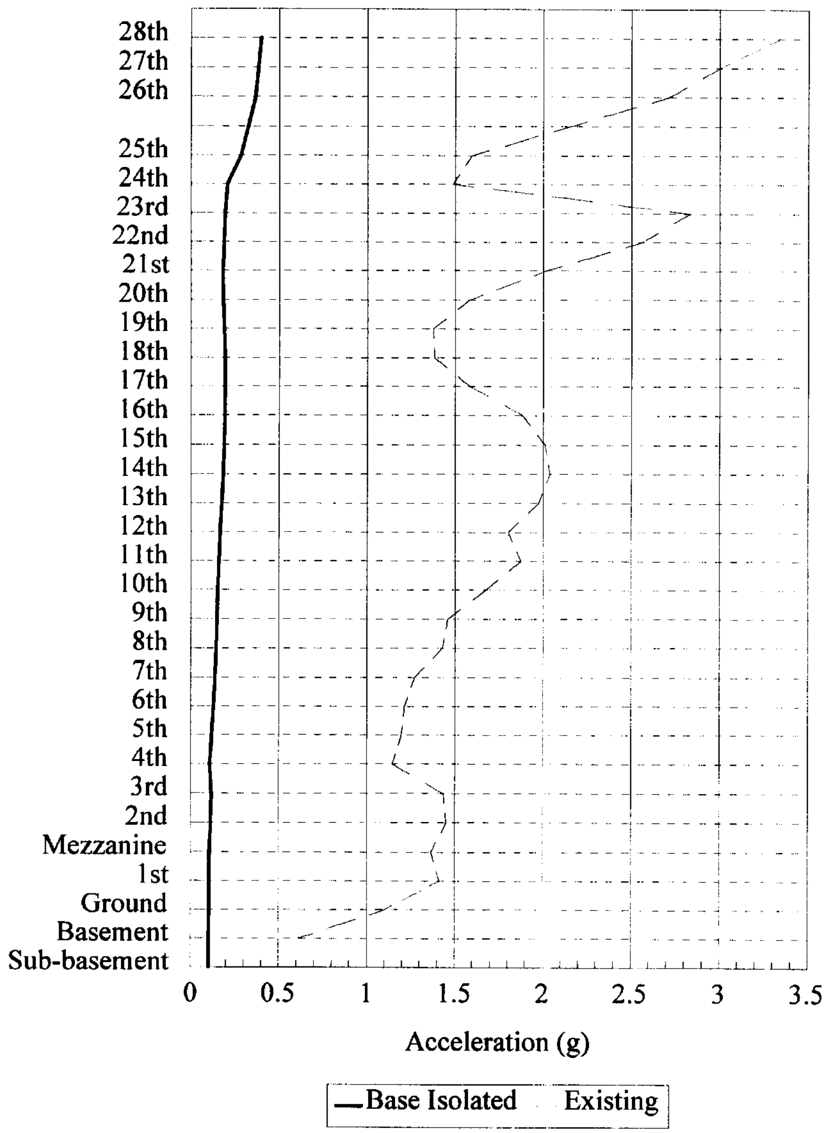

Figure 5. Maximum Story Acceleration of Existing and Base Isolated Buildings.

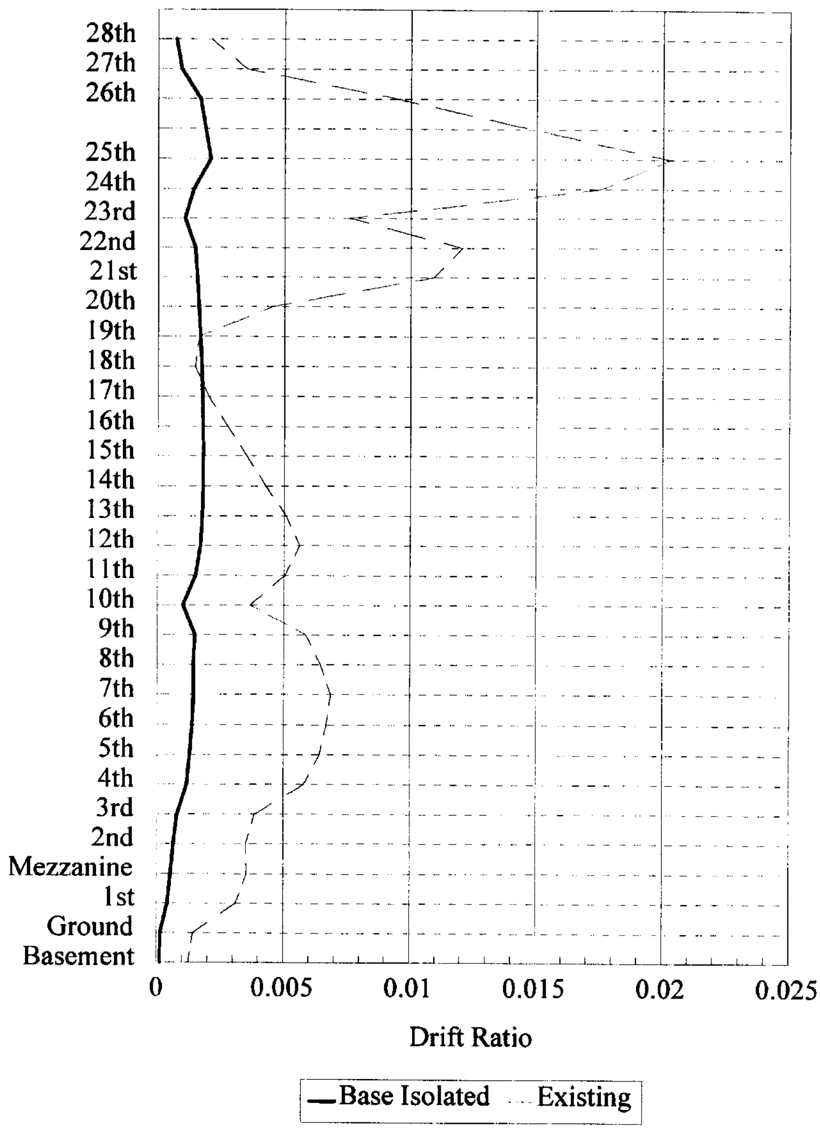

Figure 6. Maximum Inter-Story Drift Ratio of Existing and Base Isolated Buildings.

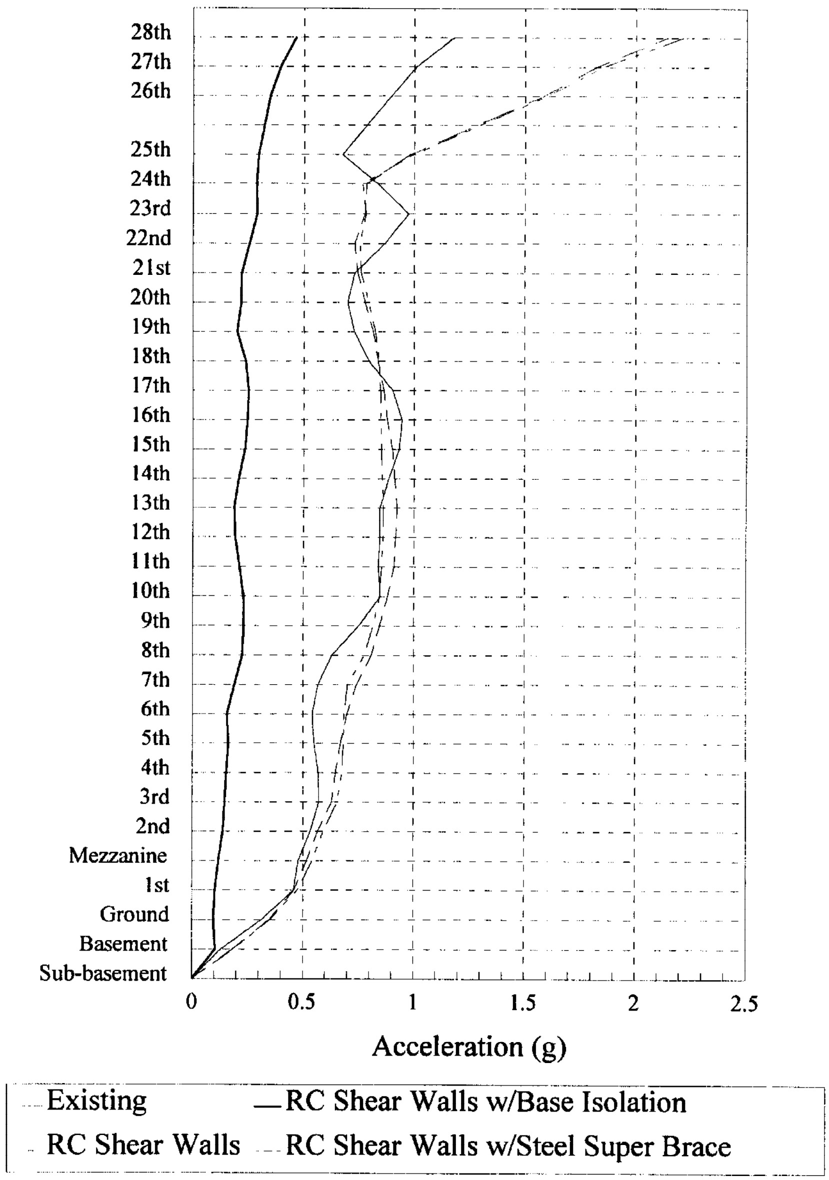

Figure 7. Maximum Story Acceleration of Existing Building and Various Strengthening Schemes.

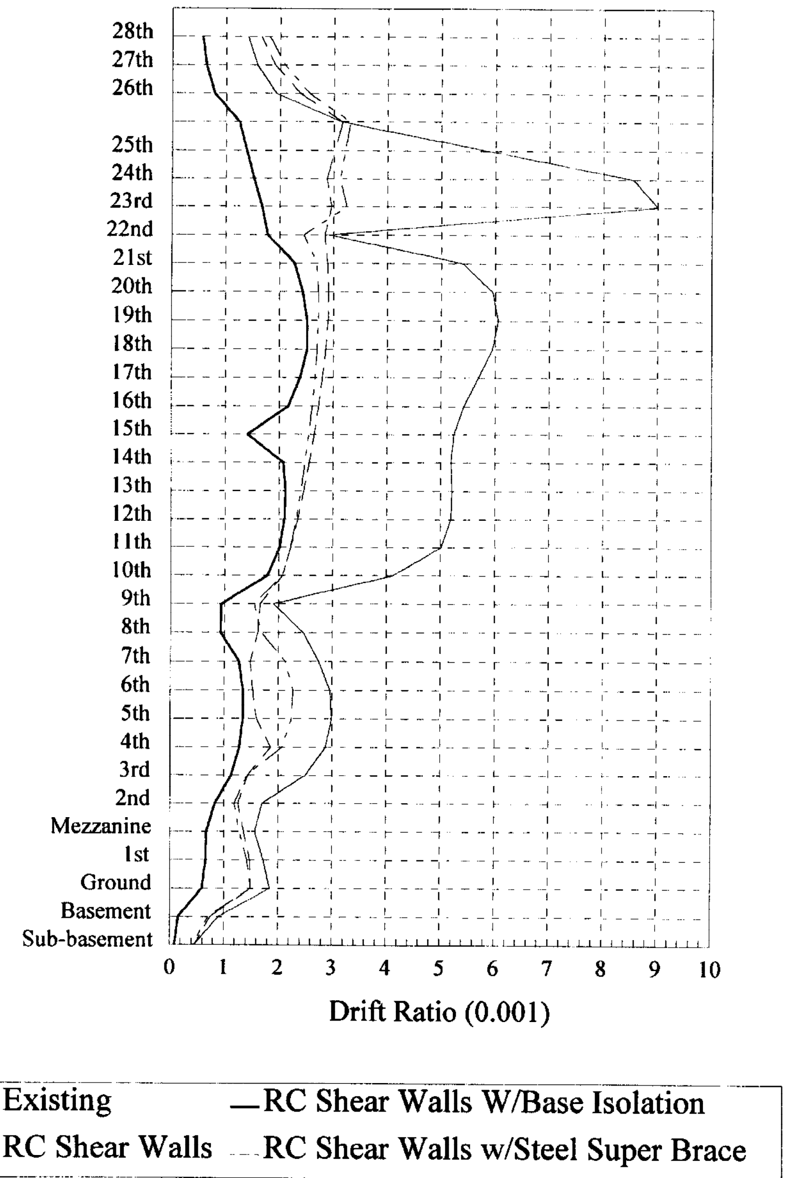

Figure 8. Maximum Inter-Story Drift Ratio of Existing Building and Various Strengthening Schemes.

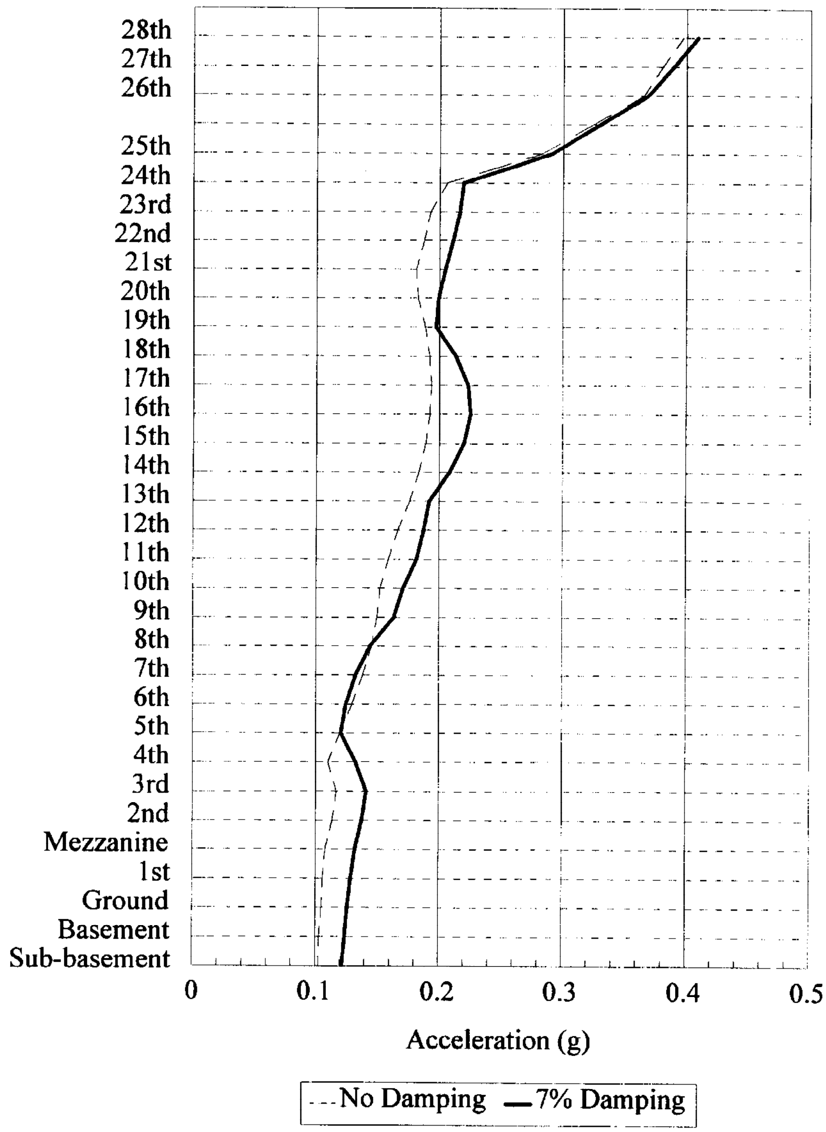

Figure 9. Maximum Story Acceleration of Base Isolated Building with and without 7% Damping at Plane of Isolation.

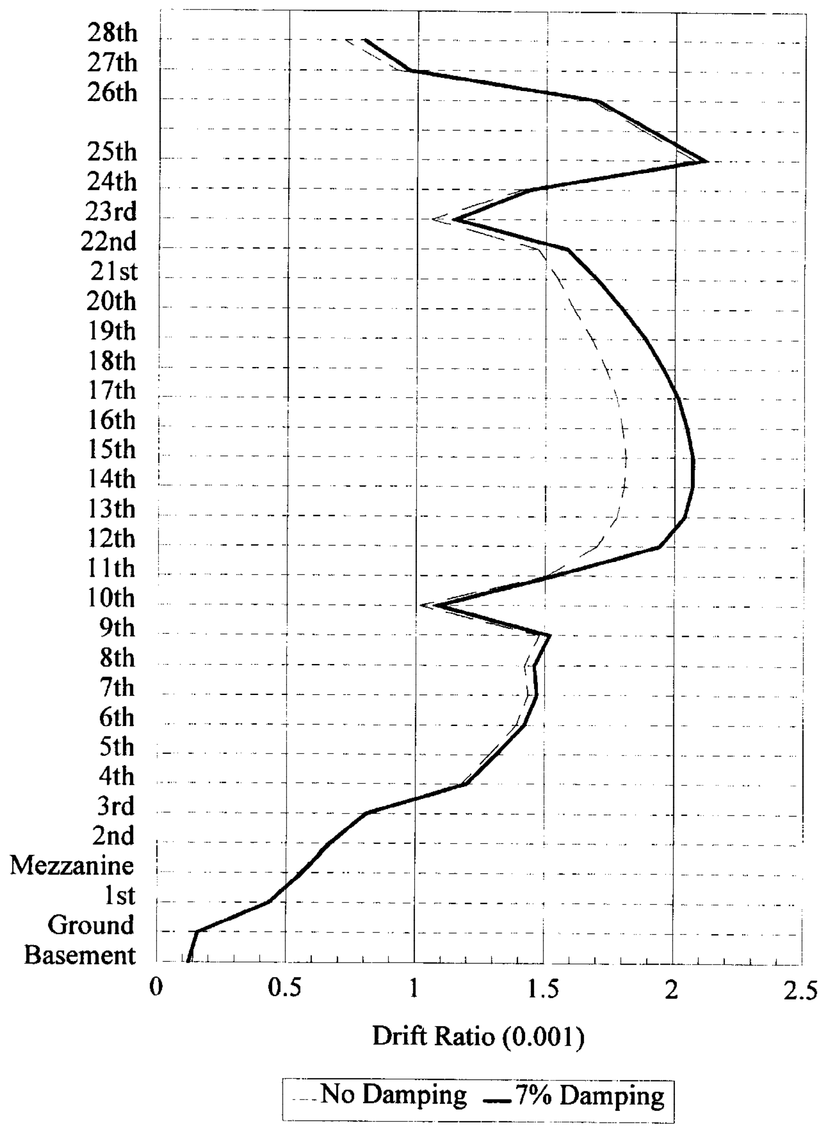

Figure 10. Maximum Inter-Story Drift Ratio of Base Isolated Building with and without 7% Damping at Plane of Isolation.

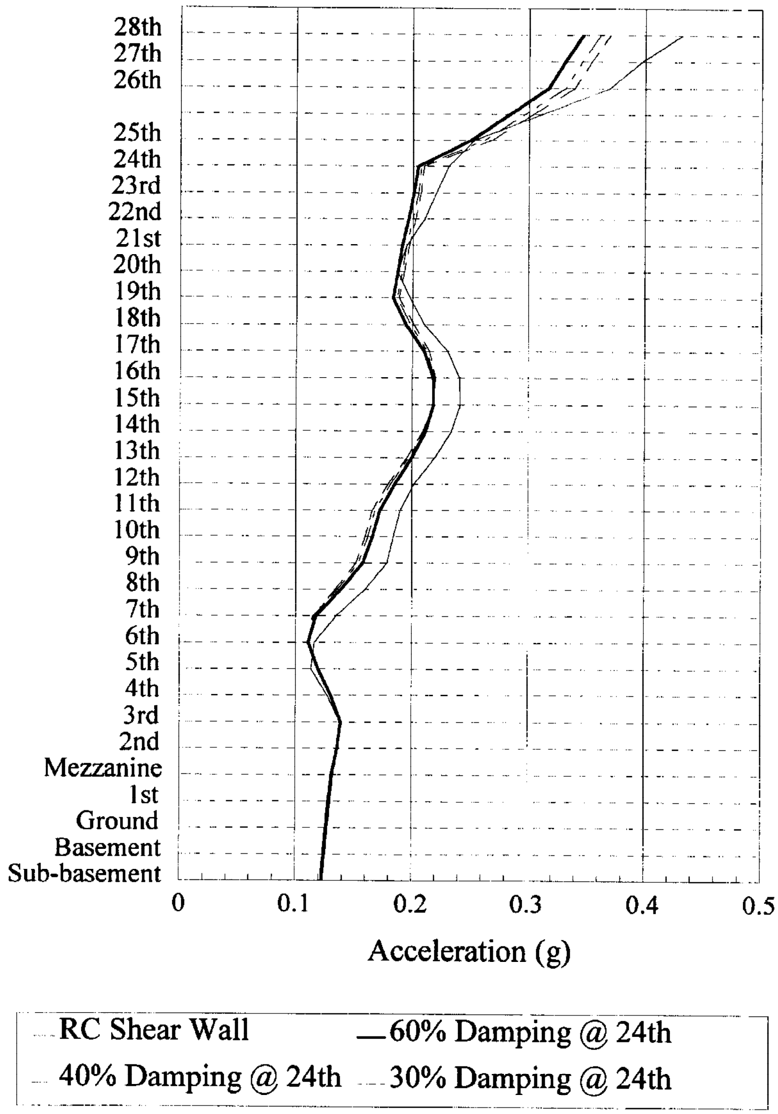

Figure 11. Maximum Story Acceleration of Isolated Building with and without 7% Damping at Plane of Isolation and 30-60% Damping at 24th Floor.

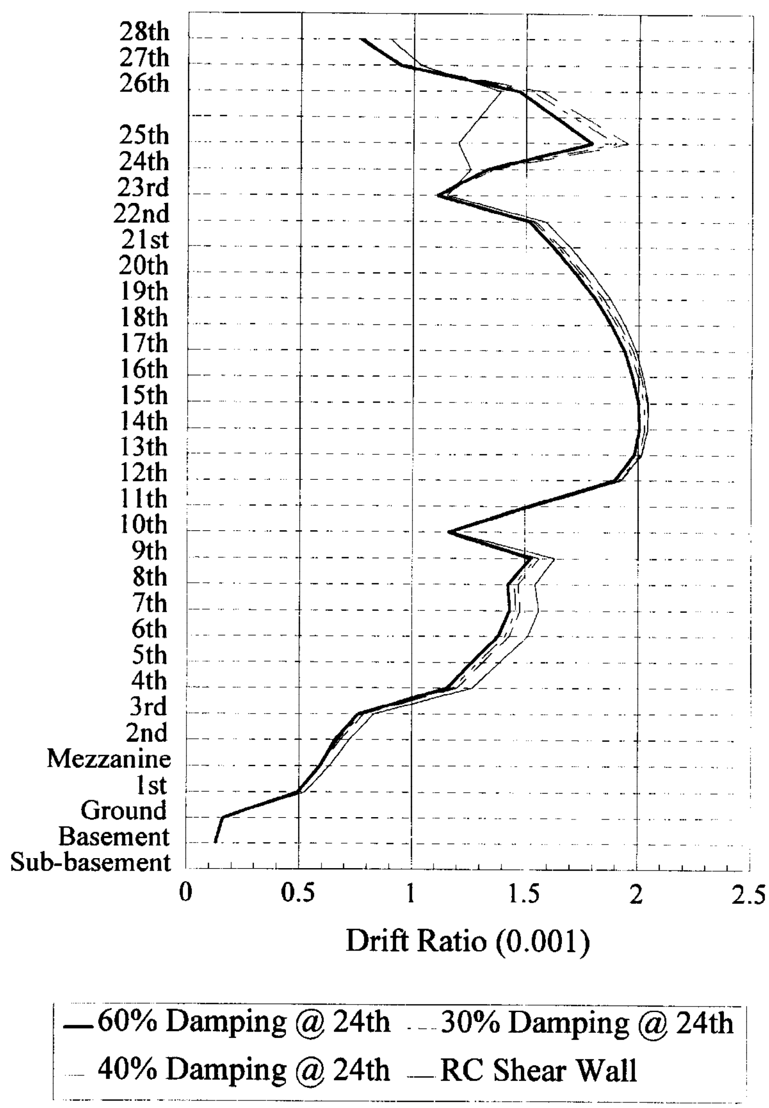

Figure 12. Maximum Inter-Story Drift Ratio of Isolated Building with and without 7% Damping at Plane of Isolation and 30-60% Damping at 24th Floor.