ELSEVIER

Engineering Structures 21 (1999) 603–614

ENGINEERING STRUCTURES

Supplemental damping for seismic strengthening: a case study

Durgesh C. Rai *

Department of Earthquake Engineering, University of Roorkee, Roorkee 247 667, India

Received 4 June 1997; received in revised form 23 December 1997; accepted 29 December 1997

Abstract

This paper illustrates how supplemental damping can be used for seismic strengthening of a landmark old structure (San Francisco City Hall). The objective was to develop a conceptual strengthening scheme for this vertically irregular building which had unusual dynamic properties. The scheme utilizes viscous damping devices (or equivalent energy dissipation devices) to control not only displacements and accelerations to levels which prevent building collapse in a major earthquake, but also to control building damage in the more likely small magnitude earthquakes. The supplemental damping devices together with their support system are independent of the existing structural framing system, with the former significantly reducing the seismic demand on the latter. © 1998 Elsevier Science Ltd. All rights reserved.

Keywords: Energy dissipation devices; Seismic strengthening; Supplemental damping

1. Introduction

Supplemental damping can be used effectively for seismic strengthening and for improving the seismic response of existing non-compliant structures and/or structures damaged in past earthquakes. Many devices have been developed that can be used as part of a primary or secondary system of lateral resistance. An overview of various energy dissipation devices are provided in a state-of-report by Hanson et al. [1]. Traditional seismic-resistant design relies on energy dissipation by the inelastic action in various parts of the structure (e.g., sections of beams near connections in typical moment frame structures) which are suitably designed to provide significant energy dissipation potential. However, this energy dissipation, which is due primarily to material hysteresis requires large plastic deformations in the primary structural members which causes substantial damage to non-structural components as well. Supplemental damping devices intend to dissipate the earthquake-induced energy by acting either in parallel or in series with the primary structural system. As a result, the energy dissipation demand on primary structural members is minimized (or eliminated), thus reducing permanent deformations and damage in structural and non-structural components.

This paper first reviews the simple relations proposed for response reduction factors which can be achieved by increasing the level of damping in the system. Later, a conceptual design for seismic strengthening is developed using supplemental damping devices for a landmark structure, San Francisco City Hall, which was damaged in the 1989 Loma Prieta earthquake. The focus of the study is on identifying the structural weaknesses and dynamic characteristics of the system through simple analyses, and then identifying the location and level of supplemental damping needed to control the response within acceptable limits for a design level earthquake.

2. Effect of viscous damping in reducing response

The seismic response of a structure can be considered as a series of responses to individual earthquake pulses and, for a highly damped system, these individual responses would decay rapidly before they could add up. However, this advantage of added viscous damping is conditional and is available only when the frequencies of input motions are close to frequencies of the system. Moreover, in reality, the earthquake waves have a rather complex distribution of frequencies. Ashour [2] studied the effect of damping in reducing the spectral displace-

* Corresponding author. Tel: + 91 1332 65127; Fax: + 91 1332 76899/73560; E-mail: dcreq@rurkiu.ernet.in

0141-0296/99/$—see front matter © 1998 Elsevier Science Ltd. All rights reserved.
PII: S0141-0296(98)00007-8

ment response of elastic systems by using a set of 15 accelerograms, which included real ground motions as well as synthesized ones. By varying the structural period from 0.5 to 3 s to cover a representative range, he found that mean spectral displacement spectra are simple decaying functions of the critical damping and that they lie in a narrow band for various seismic inputs as shown in Fig. 1. This studies shows, for example, that a 2.5 times reduction in the response can be expected if damping of the system is raised to 50% critical.

Furthermore, Wu and Hanson [3] have shown that the spectral modification for high damping (10–50% critical) can be considered separately from the spectral modification resulting from inelastic response (hysteretic behavior). This means that the code-type design spectrum curves, which incorporate spectral reductions due to inelastic deformations, can be further reduced for high damping. For structures with multiple degree of freedom, the damping coefficient, c, for each storey is first established, and then modal damping for the dominant mode is obtained from those values to be used in determining the reduction factor for a spectral design spectrum. The first mode is usually the dominant mode for seismic response of regular structures. This way of prescribing viscous dissipation devices can cause a dynamic modal coupling, which is usually weak and has an insignificant effect on overall dynamic characteristics. However, a distribution of damping similar to the distribution of storey stiffnesses can be provided to avoid the modal coupling.

It should be noted that the discussion above applies ideally only to those damping dissipation devices which possess linear viscous behavior, such as fluid viscous dampers and visco-elastic material-based devices. For these devices, the maximum earthquake forces are determined by the maximum displacements and velocities in them. Other devices which utilize friction or metal hysteresis for energy dissipation are characterized by equivalent viscous damping, which is amplitude dependent. Therefore, a knowledge of the expected earthquake response is essential to select appropriate equivalent viscous damping. Further details on mechanical damping devices and seismic energy dissipation can be found in the paper by Hanson et al. [1]. The following is a case study in which supplemental damping is the primary method to remedy a building's deficiencies in resisting lateral loads, as evidenced in an earthquake.

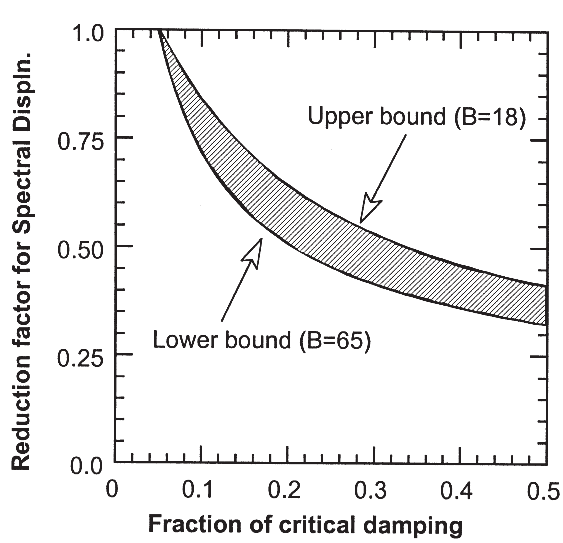

Fig. 1. Spectral displacement reduction factor for various levels of viscous damping (Ashour [2]).

3. San Francisco City Hall

San Francisco City Hall is a beautiful Renaissance architecture located in the San Francisco Civic Centre National Historic Landmark District. The building was designed in 1913 at the same site where the previous city hall was destroyed in the devastating earthquake of 1906. It is a rectangular five-storey office building, approximately 94 m by 124 m with its central landmark dome rising approximately 90 m above the main floor, as shown in Fig. 2. A schematic of the elevation and the first floor plan is shown in Fig. 3.

3.1. Structural system

The building structure is composed of a complete steel frame with reinforced concrete floors. The granite facade is integrally laid-up with unreinforced brick masonry (URM) walls, and the in-fill partition walls are constructed with hollow clay tiles (HCT). The dome is a multi-tiered truss steel structure supported on four steel column towers. The dome roofing is HCT covered, with lead and copper filling the exterior dome truss. The damage during 1989 Loma Prieta earthquake was primarily concentrated in HCT, in URM walls and in the concrete slabs. The lateral load resistance of the building is mainly provided by the following individual components: (a) complete structural steel frame; (b) URM walls; and (c) HCT in-fill walls. The estimated storey shear capacities of the five storeys of the office building, where most of the building weight is concentrated are shown in Table 1. Clearly, the shear capacity distribution is not what one would expect for a typical building: the upper storeys (especially the main storey) are significantly weak in comparison to the ground storey. For the main storey, the capacity vs. deflection curves are shown in Fig. 4, which also shows the relative contribution of the various components. Much of the ultimate strength of the main storey is provided by brittle construction materials such as URM and HCT, whereas ductile steel frame provides only an eighth of the total shear capacity.

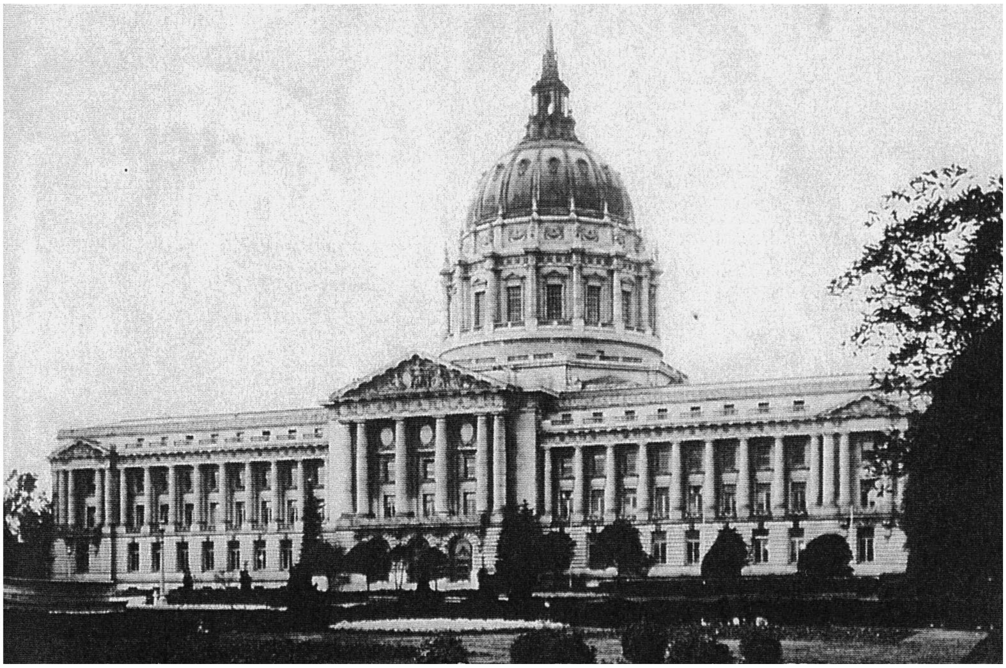

Fig. 2. San Francisco City Hall (Gebhard et al. [4]).

4. Seismic evaluation of the structure

4.1. Structural modeling

For analytical purposes, the structure is modeled as a shear-building (stick model) with five masses lumped at each floor of the office building and four masses to represent the dome structure. The mass, storey stiffnesses and dome stiffnesses are summarized in Fig. 5. The response spectrum as well as time history analyses are performed by SNAP-2DX, a general-purpose computer program for dynamic analysis of two-dimensional structures [5]. Shear force-deformation characteristics of building storeys were modeled by element 7 of the program.

4.2. Dynamic modal properties

To understand the seismic behavior of the structure, it is helpful to study its dynamic characteristics. A modal analysis of the analytical model of the structure is performed: the modal period and mode participation factors of the structure are summarized in Table 2 and mode shapes are shown in Fig. 6. It can be seen in Fig. 6 that the first mode contains mostly office building participation, and that the second, third, and fifth modes contain both dome and building responses. The fourth mode exclusively represents the dome response. Among higher modes (periods of 0.15 s or less), the sixth and ninth modes are mainly the dome response, while the seventh and eighth mode are the office building response. It should be noted that the seventh and eighth modes are very closely spaced with periods of 0.130 and 0.126 s, respectively, and would most likely be excited simultaneously. The next closely spaced modal pairs are the third and fourth mode. Obviously the dynamic characteristics of the structure are influenced significantly by the dome structure.

About 73% of modal mass participates in the first mode, and the next largest modal participation (13%) occurs in the seventh mode. The other significant contributors to the total modal mass are the eighth, third, second and fifth modes, in decreasing order of significance. Clearly, modes which represent only dome response do not contribute significantly to modal mass (less than 0.5%).

4.3. Estimates of seismic demand: response spectrum analysis

A response spectrum analysis of the analytical model of the building was performed using the 1994 Uniform Building Code [6] acceleration spectrum for S_2 type soil site in Seismic Zone 4. The objective was to determine the seismic demands which the structure is supposed to provide in the event of a design level earthquake. A multi-mode spectrum analysis was carried out, which includes the first eight vibration modes of the analytical model. The ninth mode has negligibly small modal participation and can be excluded from calculations. It should be noted that the first seven modes are needed in order to have 90% of the modal mass included in the analysis.

The period and modal participation factors needed for

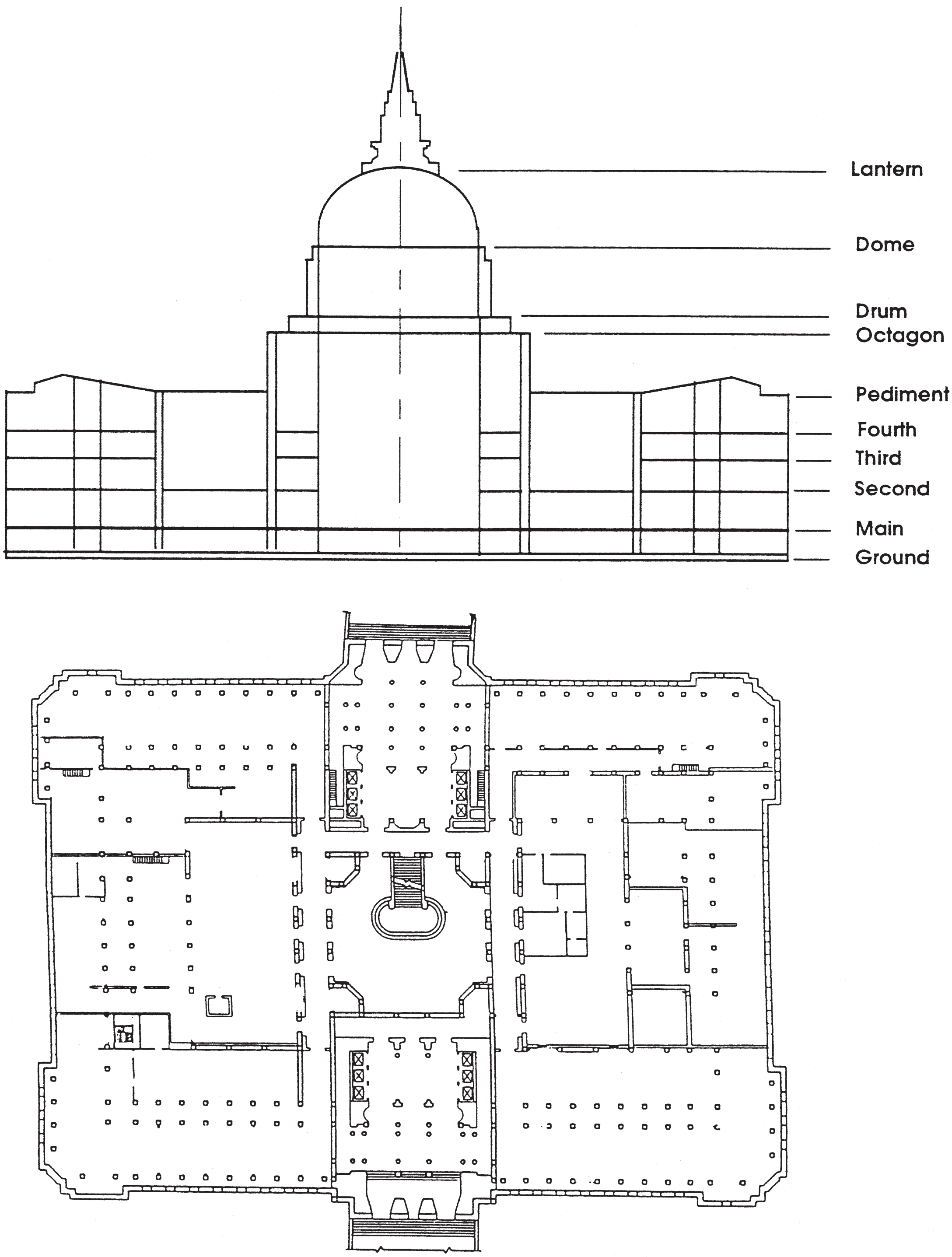

Fig. 3. (a) Elevation and (b) main floor plan of the San Francisco City Hall.

calculations are summarized in Table 2. The spectral accelerations were obtained from the S_2 soil spectrum of the Code normalized to 1.0 g and were then factored down to the effective peak ground acceleration of 0.4 g specified for the Code's Seismic Zone 4. Using these values, modal base shears for each mode were obtained as given in Table 2. These modal shears were derived from the elastic spectra and needed to be corrected for ductility or energy absorbing properties of the type of construction. The base shear was corrected by the procedure recommended in the Code for irregular buildings. All modal contributions were scaled by the ratio of the static design base shear to the total multi-modal base shear V_m. The static design base shear is given by

Table 1
Shear capacities of the office storeys

Storey level	Storey shear capacity (MN)	Percentage of seismic weight
Fourth	64.9	8.8
Third	81.8	11.1
Second	59.2	8.0
Main	57.4	7.8
Ground	272.2	36.9

$$V = \frac{ZIC}{R_w} W \tag{1}$$

where $Z = 0.4$, $I = 1$, and the seismic weight of the structure $W = 737$ MN. The seismic coefficient C for period $T \approx 1$ s (from Table 2 for the fundamental mode) and $S = 1.2$ is equal to 1.5. Taking a value of 6 for R_w, considering the fact that the lateral load system of the structure is a complete steel frame with masonry in-fill walls, the total design base shear V was then calculated to be $0.1W = 73.7$ MN.

The most probable uncorrected value of base shear V_m was calculated from the modal base shears by the complete quadrature combination (CQC) method. Using cross-modal coefficients corresponding to 5% damping for each mode, V_m was calculated to be 346.3 MN. Therefore, the correction factor $= V/V_m = 73.7/346.3 = 0.213$. The modal base shears were then corrected as shown in Table 2. Finally, the modal forces and storey shears for each mode were computed separately and then combined by the CQC method, as shown in Table 5. It

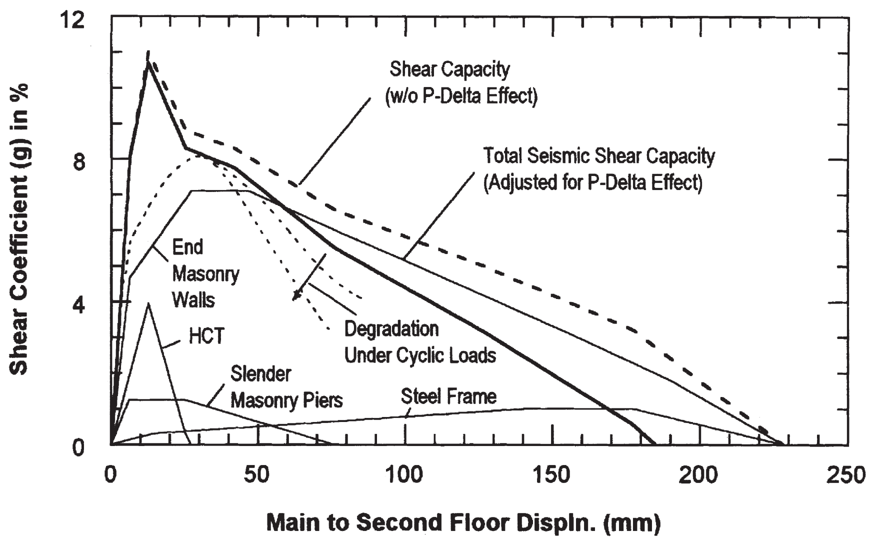

Fig. 4. Estimated storey shear capacity of the existing building in the main floor, N–S direction (Forrel/Elsseser).

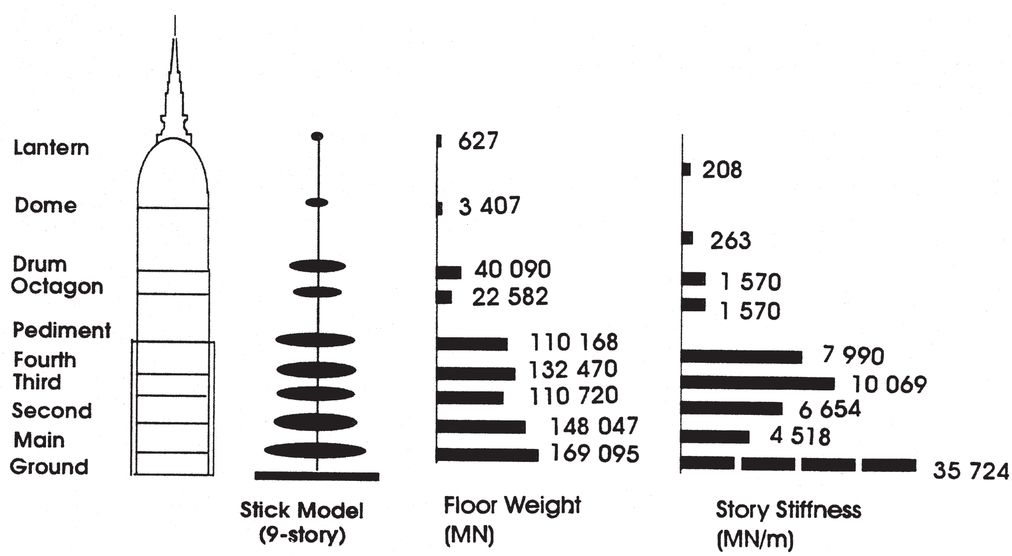

Fig. 5. Stick model of the San Francisco City Hall used in dynamic analysis.

Table 2
Multi-modal response spectrum analysis

	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8
Participation factor	7.407	1.874	1.808	0.415	0.774	0.258	3.107	1.735
Modal mass percentage	73.0	4.67	4.34	0.23	0.80	0.09	12.85	4.01
Period (s)	0.977	0.480	0.271	0.240	0.180	0.153	0.130	0.126
Spectral acceleration (g)	0.60	1.0	1.0	1.0	1.0	1.0	0.92	0.90
Modal base shear (MN)	322.5	34.5	32.0	1.7	5.9	0.6	87.0	26.8
Corrected base shear (MN)	68.6	7.3	6.8	0.36	1.3	0.14	18.3	5.7

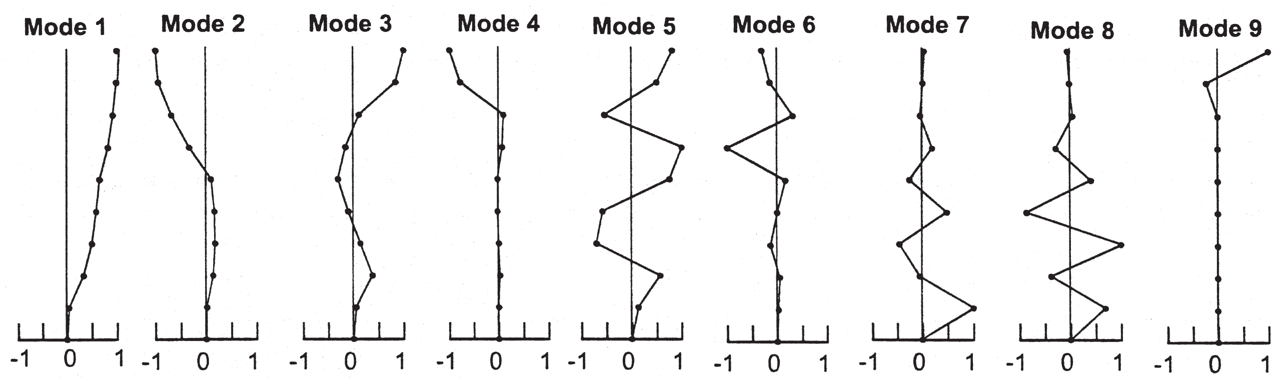

Fig. 6. Mode shapes of a stick model of the San Francisco City Hall.

should be noted that the corrected modal force quantities are at working stress level because the basis for the modal correction is the UBC design base shear, which is based on the working load. Therefore, before the modal storey shears are compared with the estimated capacities, they need to be multiplied by a factor of 1.5 (typical load factor for lateral loads) to be brought up to the ultimate level. The demand capacity ratio (DCR) is then obtained by dividing the ultimate modal storey shear by the estimated storey shear capacity of the existing building as shown in Table 3. It is clear that for the second and the third storeys, the DCR is larger than unity and shows the vulnerability to design level earthquakes. This observation is more serious considering that a majority of shear capacity is due to non-ductile materials, thereby underlining the need for seismic strengthening.

Table 3
Estimated demand capacity ratios (response spectrum analysis)

Floor level	Modal floor forces (MN)	Modal storey[a] shears (MN)	Estimated shear capacity (MN)	DCR = demand/capacity[b]
Lantern	0.22			
Dome	1.11	0.22		
Drum	9.83	1.33		
Octagon	4.37	11.01		
Pediment	17.24	15.07		
4th floor	17.92	29.51	64.9	0.68
3rd floor	13.36	45.75	81.8	0.84
2nd floor	16.71	57.17	59.2	1.45
Main floor	27.7	68.03	57.4	1.78
Ground		73.7	272.2	0.41

[a]Modal storey shears are at working stress level.
[b]Modal storey shears were multiplied by factor 1.5 to calculate the shear demand at the ultimate.

5. Selection and distribution of supplemental damping

The response spectrum analysis and modal dynamic characteristics of the structure indicate that the seismic demand on the second and third storeys need to be reduced and also the vibrations of the tower (or dome) structure need to be damped out. With these objectives in mind, the amount of supplemental viscous damping in the second and third storeys was chosen to be 350 and 175 MN s/m and the three storeys of tower structure, the pediment, octagon and drum storeys were each provided with 17.5 MN s/m. This distribution of supplemental viscous damping results in a non-classical structural damping matrix C for the analytical model, i.e. the generalized damping matrix $\phi^{\mathrm{T}}C\phi$ is not diagonal, where ϕ is the mode shape matrix. Neglecting the cross-modal components, a rough estimate of the amount of damping in each mode that will result from this distribution of supplemental damping can be obtained by the diagonal elements of the matrix $\phi^{\mathrm{T}}C\phi$. These modal damping ratios and the damping matrix C are shown in Table 4.

Table 4
Supplemental damping properties

Structure supplemental damping matrix[a]	$C = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 17.5 & -17.5 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & -17.5 & 35.0 & -17.5 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & -17.5 & 35.0 & -17.5 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & -17.5 & 17.5 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 175.0 & -175.0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & -175.0 & 525.0 & -350.0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & -350.0 & 350.0 \end{bmatrix}$ MN s/m

	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	Mode 9
Supplemental modal damping (fraction of critical)	0.143	0.127	0.303	0.766	0.231	0.238	0.184	0.221	0.091

[a]In addition to the supplemental damping, a 5% stiffness proportional damping is assumed in the response analysis of the analytical building.

It can be seen that about 13–30% of critical damping is present for modes representing office building response. For the fourth mode, which is almost exclusively dome response, the damping is very large. Referring to Fig. 1, it can be seen that this amount of modal damping can reduce the building response by about 25–50%. As will be shown later, with this selection and distribution of supplemental damping, the office and dome response can be well controlled in a design level earthquake.

6. Dynamic response with supplemental damping

The dynamic (time history) response of the analytical model of San Francisco City Hall, with supplemental damping under a selected ground motion, is presented in this section. This response is compared with the building response without supplemental damping to demonstrate the efficacy of damping to control the response.

The earthquake record used for the analysis was the 20 s length of the N–S component of the El Centro (18 May 1940) earthquake accelerogram with the acceleration intensity increased by about 1.1 to give a peak ground acceleration (PGA) of 0.347 g. This adjusted accelerogram results in an acceleration response spectra which nearly matches the 1994 Uniform Building Code (soil profile S_2) elastic design forces for a building of the same significant periods as the study building as shown in Fig. 7. The calculated acceleration spectra assumed a critical damping of 5%, therefore, the analytical model included a 5% stiffness proportional damping, in addition to the supplemental damping. In contrast, a mass proportional damping results in a very small effective damping for higher modes, which are significant for deflections of the tower structure of the building.

It should be noted that the elastic response can not be obtained accurately using modal superposition methods because the structure damping matrix C does not satisfy the orthogonality condition, i.e. the modal equations are coupled by the generalized damping forces. Therefore, integration of original coupled equations (of motion) of the system is carried out by step-by-step methods.

6.1. Time history response

In Fig. 8, typical floor displacement time histories for the second floor and dome top, and the storey shears for the second and third storeys show a major response between 2 and 3 s of the ground motion, followed immediately by two to three cycles of lower, but significant, response. Another cycle of significant response is observed at a later time around 12 s of the motion. It is recognized that different earthquake accelerograms will generate somewhat different responses, but this accelerogram record equals or exceeds the Code design spectral responses.

6.2. Response envelopes

The maximum relative storey displacements, viscous damper forces, storey shears resisted by the steel frame, and in-fill walls (i.e. the ‘spring’ component) are shown in Table 5. It should be noted that a significant amount of forces develop in viscous damping devices, which the devices and the supporting structure should be able to resist safely. These envelope response quantities are

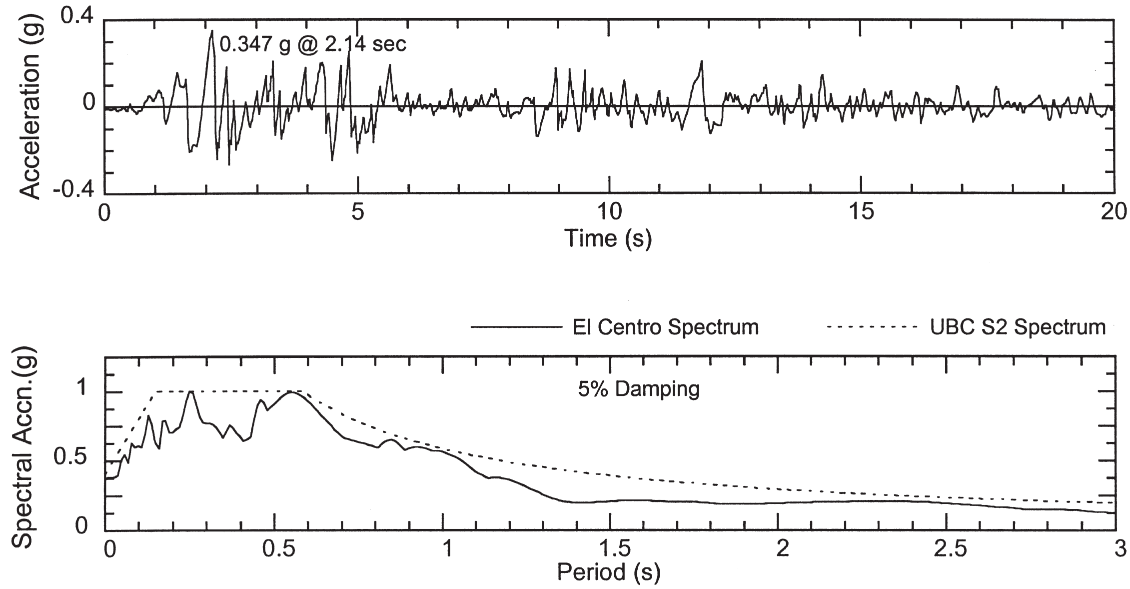

Fig. 7. (a) El Centro accelerogram (PGA = 0.347 g) and (b) its acceleration response spectrum compared with UBC-S_2 design spectrum.

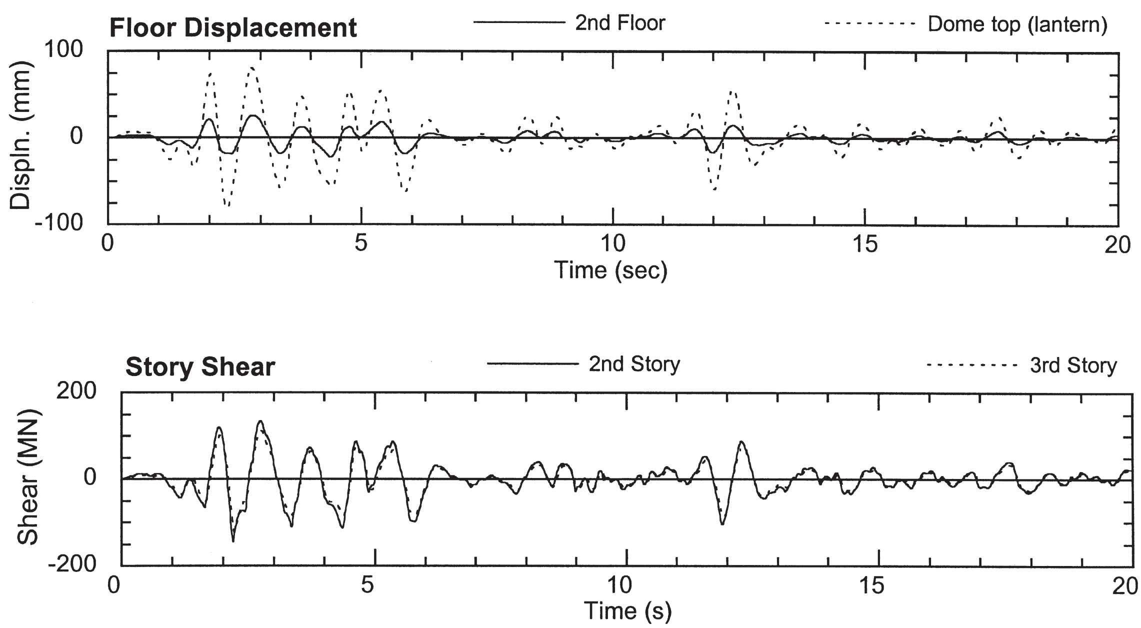

Fig. 8. Typical time history response of the analytical model with supplemental damping. (a) Second floor and dome top displacements and (b) second and third storey shears.

compared with the building without supplemental damping in Fig. 9. The relative storey drifts are reduced with supplemental damping, and the most noticeable reduction is about 40% for the 'soft' main storey. Additionally, there is a significant reduction in the shear resisted by the existing building components, especially in the office storeys with supplemental damping. It should be noted that these reductions compare very well to those given in Fig. 1 for the modal damping ratios shown in Table 4 resulting from the supplemental damping.

The storey shear strength ratios, defined as the ratio of the elastic demand to the shear capacities, are shown in Table 6 for the structure with and without supplemental damping. These values are reported for the five storeys of the office building for which the shear capacity data was available. It can be seen that supplemental damping reduced the storey strength ratio in the main floor level to 1.68 from a value of 2.76 for the existing structure. Similarly, in the second floor, the storey strength ratio was reduced from 2.24 to 1.65. All other levels of the office building remained nearly elastic with supplemental damping.

Table 5
Envelope response of time history analysis

Floor level	Relative floor displacements (mm)	Elastic storey shear[a] (MN)	Viscous damper force (MN)
Lantern	1.6		
Dome	5.4	0.33	-
Drum	12.5	1.4	0.89
Octagon	17.5	19.6	2.1
Pediment	7.1	27.4	2.9
Fourth floor	8.5	56.9	-
Third floor	14.3	85.8	-
Second floor	21.4	95.1	26.9
Main floor	3.9	96.8	73.4
Ground		139.3	-

[a]The storey shear is resisted by the existing structural frame (spring component) and does not include shear resisted by viscous dampers.

Table 6
Storey strength ratio (time history analysis)

Storey level	Estimated storey shear capacity (MN)	Without supplemental damping		With supplemental damping	
		Elastic storey shear (MN)	Storey strength ratio	Elastic storey shear (MN)	Storey strength ratio
Fourth	64.9	64.6	1.00	56.9	0.88
Third	81.8	104.3	1.27	85.8	1.05
Second	59.2	132.7	2.24	95.1	1.65
Main	57.4	158.4	2.76	96.8	1.68
Ground	272.2	178.3	0.65	139.3	0.51

Table 7
Estimated inelastic dynamic response

Floor level	Sum of weights above (MN)	Elastic storey shear (MN)	Elastic floor accelerations (g)	Inelastic floor accelerations (g)
Lantern	-	-	0.53	0.30
Dome	0.627	0.33	0.35	0.18
Drum	4.03	1.4	0.44	0.25
Octagon	44.1	19.6	0.41	0.23
Pediment	66.7	27.4	0.32	0.20
Fourth floor	176.9	56.9	0.28	0.16
Third floor	309.1	85.8	0.23	0.13
Second floor	419.8	95.1	0.17	0.10
Main floor	567.9	96.8	0.19	0.19
Ground	737.0	139.3		

6.3. Estimation of inelastic building response

The maximum elastic floor acceleration can be estimated by dividing the maximum storey shear forces by the sum of weights of all the floors above that level. The true accelerations will be smaller because some of the structural members exceed their elastic limit during an earthquake. These accelerations above a given level can be reduced by the amount of expected elastic storey strength reduction. The reduced accelerations for expected inelastic building response with supplemental damping are estimated by dividing the elastic storey shears by the maximum strength ratio which occurs at any level below the particular floor. These values are summarized in Table 7. From this elastic dynamic analysis it can be concluded that the addition of supplemental damping in the tower, main and second floor level of the building can satisfactorily meet and exceed the Code requirements.

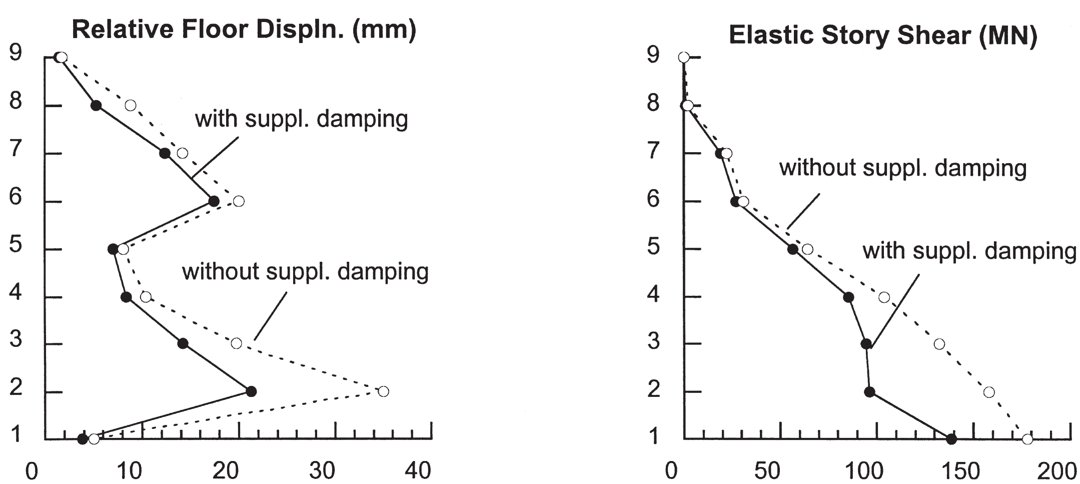

Fig. 9. Effect of supplemental damping on the envelope values of relative floor displacement and storey shear resisted by the existing structural frame.

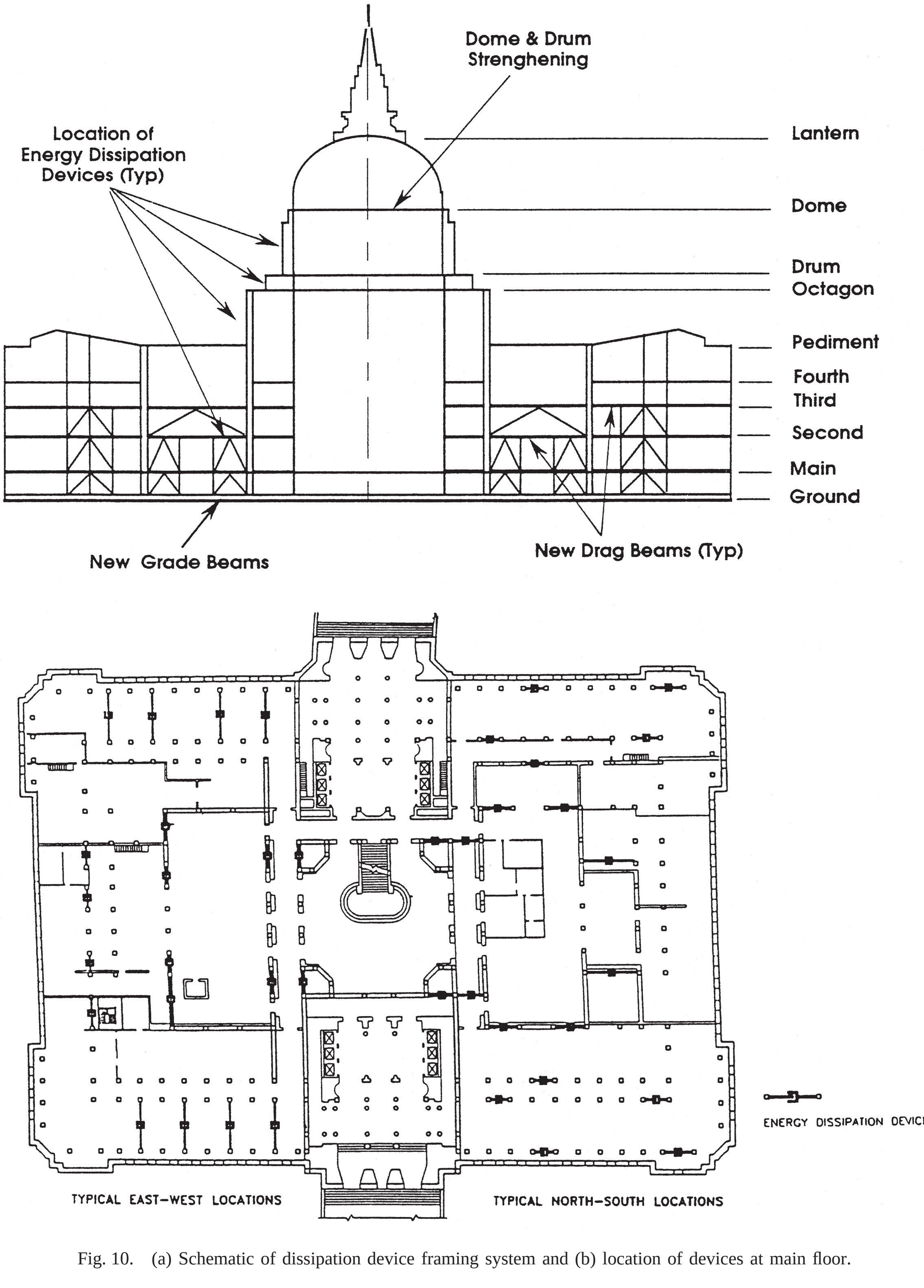

Fig. 10. (a) Schematic of dissipation device framing system and (b) location of devices at main floor.

7. Implementation of the strengthening scheme

It should be noted that the study does not attempt to select the best energy dissipation device for providing the supplemental damping assumed to modify the response of the building. Furthermore, the analysis assumed a viscous behavior for energy dissipation devices used to provide supplemental damping. Therefore, caution should be used for the application of devices which do not have truly viscous behavior and for which equivalent viscous damping values are prescribed.

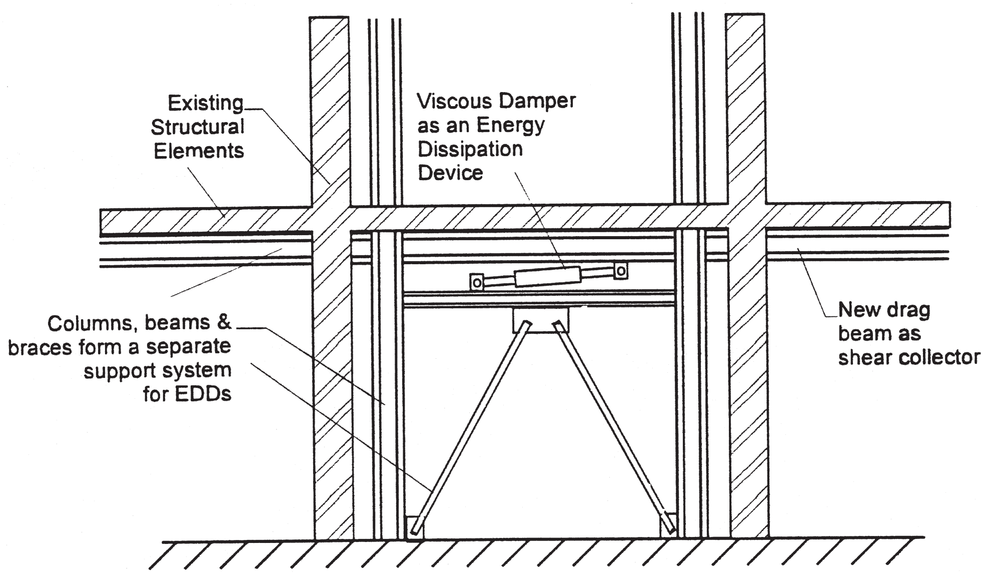

Fig. 11. Schematic of a typical arrangement of damper and supporting structure.

A support system for the energy dissipation devices, which is independent of the existing structural frame of the building, will be provided at the main and second floor. This independent system has to transfer the forces in the devices through the ground floor level to the new foundations. Forty dissipation devices are provided for the main floor level, each with the capacity to carry a maximum dissipation force of about 2000 kN, in both the N–S and E–W directions. Similarly, for the second floor, 20 such devices with a force capacity of about 1400 kN are installed. Fig. 10 shows a typical locations of energy dissipation devices on the main floor level and the device support framing system. At most locations drag members are installed in the spans adjacent to the device spans to provide relatively uniform floor diaphragm shear force transfer. A schematic showing the arrangement of a viscous type damper supported on a braced frame system is shown in Fig. 11. The braced frame system is independent of the existing lateral system and is designed to carry the forces from dampers.

In the tower structure, the energy dissipation devices are placed in combination with a steel truss in the circumferential direction at three levels. Although the analytical model does not accurately model the details of the local tower behavior and, consequently, the best characteristics of the energy dissipation devices, the simple model does show that the tower response can be controlled effectively with supplemental damping.

8. Conclusion

Supplemental damping can be used effectively to control the response of structures in earthquake-type lateral loads. As illustrated in the case study, the scheme can be successfully used in seismic strengthening of old noncompliant structures, in which supplemental damping will significantly reduce the seismic demand on the existing lateral frames. For the building with supplemental damping, dynamic response to a design level earthquake results in lower values of storey drifts and storey shears. At the conceptual stage, simple dynamic analyses can be performed first to determine the weaknesses of the existing structure and then to identify the location and distribution of supplemental damping devices. The reduction factors for high damping can be used to estimate the level of damping needed to achieve the desired level of response control. One of the advantages of using energy dissipation devices is the ease of tailoring their strength and damping characteristics to the requirements of particular storeys.

Acknowledgements

This work was carried out at the Department of Civil Engineering, University of Michigan, Ann Arbor, MI, USA, where the author was a doctoral student. The guidance and help from Prof. Robert D. Hanson is gratefully acknowledged. Relevant data about the City Hall were provided by Forrel/Elsseser Engineers.

References

[1] Hanson RD, Aiken ID, Nims DK, Richter PJ, Bachman RE. State-of-the-art and state-of-the-practice in seismic energy dissipation, ATC-17-1, ATC, March 1993: pp. 449–471.
[2] Ashour SA. Elastic seismic response of buildings with supplemental damping. Ph.D. Dissertation, Department of Civil Engineering, University of Michigan, Ann Arbor, Michigan, 1987.

[3] Wu J, Hanson RD. Inelastic response spectra with high damping. J. Struct. Engrg., ASCE 1989;115(6):1412–31.
[4] Gebhard D, Montgomery R, Winter R, Woodbridge J, Woodbridge S. A Guide to Architecture in San Francisco & Northern California, Peregrin Smith Inc., Santa Barbara, 1973: p. 513.
[5] Rai DC, Goel SC, Firmansjah J. SNAP-2DX: a general purpose computer program for nonlinear analysis. UMCEE Report no. 96–21, University of Michigan, MI, August 1996.
[6] ICBO, Uniform Building Code, 4th ed. In International Conference of Building Officials, Whittier, CA, 1994.