USE OF MOTION AMPLIFICATION DEVICES AND FLUID DAMPERS TO REDUCE THE WIND INDUCED RESPONSE OF TALL BUILDINGS

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Abstract

Adding damping with various energy dissipating devices has become an accepted method to reduce wind-induced vibrations in tall buildings. An example of a 39-story office tower is presented where large projected accelerations generated by the vortex shedding of an adjacent existing 52-story building are reduced by a passive system composed of viscous dampers and a motion amplification system. A description of the damping system and its analytical complexities are discussed. Non-linear analysis of the tower, using time history forcing functions derived from the wind tunnel is presented. Cost data for the damper system is also presented.

Introduction

The use of energy dissipating devices to reduce building response from dynamic inputs has become an accepted design approach for high-rise buildings. New approaches are continually being developed by designers as evidenced by the varied applications of tuned mass dampers, sloshing dampers, visco-elastic dampers, friction dampers and viscous dampers. Each of these systems has its own idiosyncrasy and which is most appropriate must be evaluated for the particular project under consideration.

This paper presents the results of an investigation of the application of viscous dampers in a high-rise structure located in an urban environment. The structure, a 39-story steel-tube frame was designed using conventional wind engineering methods with code loadings and standard deflection limitations. A model of the tower was tested in a wind tunnel of RWDI facilities in Canada. The building is located within the immediate proximity of a 52-story tower in the center of a coastal downtown urban environment. Wind tunnel results indicted that the structure would experience very high acceleration levels generated by winds coming from a northwestern direction. Detailed investigation into the wind tunnel data indicted that the intense buffeting the tower was experiencing was the result of vortex shedding from the adjacent 52 story existing building. The predicted acceleration levels were double the industry standard for office towers. In order to reduce the projected motion levels, several approaches were investigated and evaluated for cost and planning impact. Tuned mass dampers and sloshing dampers required valuable office space at the top of the tower and proved to be very expensive (although very effective). Viscoelastic dampers were no longer available from US manufactures. Viscous dampers proved to be the most cost effective and least space intrusive on the office tower. An extensive design program was undertaken with various viscous damper configurations vertically and with many variations of viscous damper properties.

Since the main intent of the damper installation is to reduce accelerations resulting from relatively frequent storms, the viscous dampers need to provide a large force output at very low displacement levels (±1/8"). In order to insure reliability at this small movement and to keep the number and cost of the dampers to minimum, a motion amplification device was introduced in the design. The motion amplification device was used in one direction of the structure, that being the stiffest with the lowest predicted inter-story displacements.

The introduction of a motion amplification device to amplify inter-story displacements experienced by the damper was essential to the design reliability. The small inter-story movements normally experienced by frequent storms producing annoying accelerations must be amplified to allow the use of an economical viscous damper and to ensure the reliability of the damper force output.

A motion amplification device called a Toggle Brace Damper system (TBD) was tested by Constaintinou, etc. (1998). Their report demonstrates that the TBD system is a very effective mechanism to amplify inter-story motion. However, the efficiency of TBD, as reported by McNamara, Huang and Wan (1999) is highly dependent on various local system design parameters. Careful design of the TBD is extremely important for the proper performance of the damper system. From the above parameter study of TBD system, a total 60 viscous dampers were used in the 39-story office building to reduce the top floor acceleration into an acceptable range. The viscous dampers in North-South direction use TBD devices. Viscous dampers in the East-West direction use dampers with straight braces. The viscous dampers were then designed for both 100-year return wind and moderate earthquake excitations. (Seismic zone 2, Av = .12g)

Office Building Structural System

The 39-story Office Building consists of three lateral systems at different levels . From the 1st to 7th floors and above 34th floor diagonal bracing is used for the lateral system. Over the remaining of floors the lateral system is a moment frame on the perimeter of building. The typical floor system is composite metal deck with composite joist girders spaced at 10'-0" o.c. Typical floor area is 22,500 square feet. Viscous dampers in E-W direction are straight diagonals placed in two bays of the inner-core on every other floor between 7th floor and 34th floor. The TBD systems are placed in two bays along the N-S direction at the same level as the diagonal dampers. The damper system layout is shown in Fig.1.

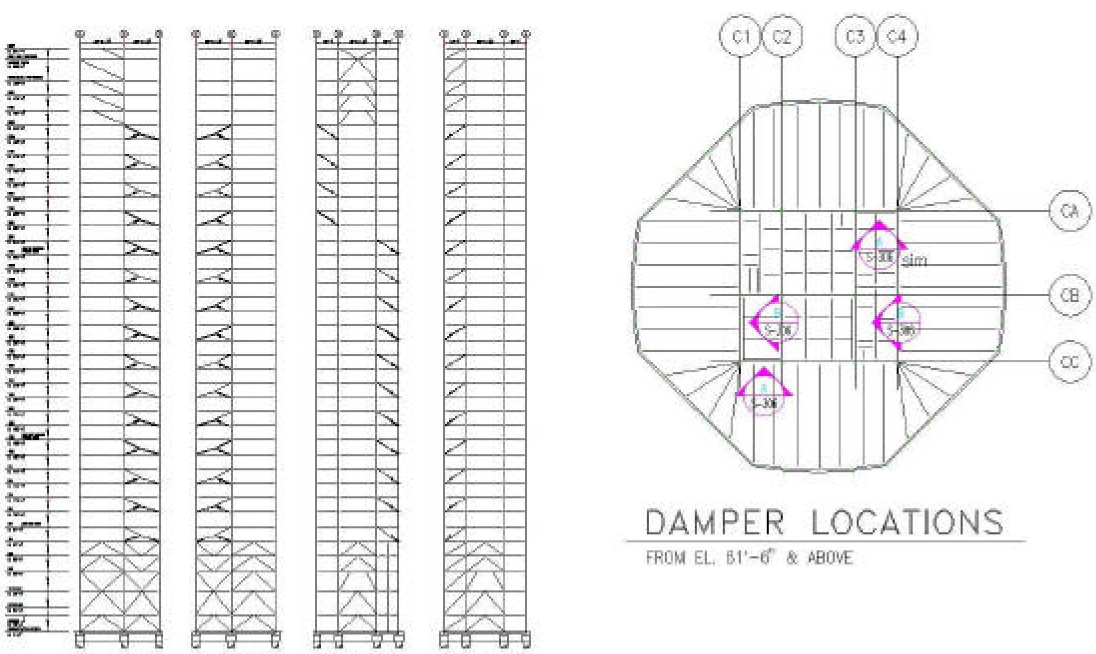

Fig. 1 Viscous Damper Elevation and Key Plan

A static lateral analysis and design was conducted using ETAB6.2. The dynamic response and viscous damper design and the TBD system were analyzed by SAP2000. Simple one story models were used to do parametric studies on the TBD system. The building dynamic properties are tabulated in Table 1.

Table 1 Dynamic Properties of Building for First Six Modes

Mode Shape	**1**	**2**	**3**	**4**	**5**	**6**
Period (sec)	5.26	5.00	3.65	1.92	1.82	1.71
Effective Mass (%)	66.1	62.6	81.2	15.3	12.8	8.5
Direction	X (E-W)	Y (N-S)	Rotation	X (E-W)	Y (N-S)	Rotation

Note: Above dynamic properties obtained from ETAB63-D model

Wind tunnel results indicate average story drifts from 7th floor to 34th floor on E-W (X) direction are larger than the (Y) direction. The overall building stiffness in X-direction is less than that on Y-direction. For cost effective design, a TBD system in the Y-direction was used to magnify the story drift. The damper constant (C) was varied throughout the height of the tower. Linear viscous dampers and the TBD system were designed and manufactured by Taylor Devices, Inc. The damper layout is shown in figure 1. The elevation of the dampers and TBD are shown in figures 2 and 3. Geometric data for the TBD system is given in Table 2.

The design of the viscous damper system can be conceptualized as the damper system providing a set of loads distributed vertically along the height of the tower. These loads are velocity dependent and are applied to the towers lateral force resisting system. The damper loads are out of phase with the displacement response of the tower and represent the mechanisms by which the response is reduced. Maximum damper forces occur at response levels of zero displacement and maximum displacement velocity.

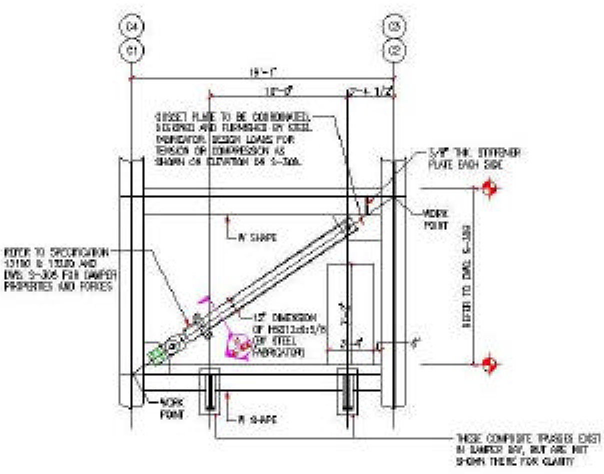

Fig. 2 Diagonal Viscous Damper on E-W (X) Direction

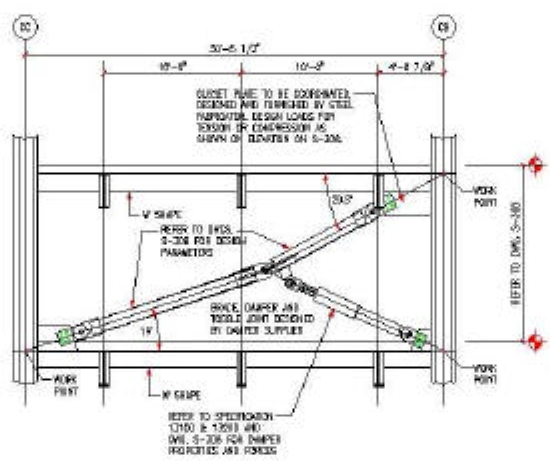

Fig. 3 Toggle Brace Damper on N-S (Y)Direction

Table 2 Toggle Brace Configuration with Story Height 12'-6" **

Bay Length (ft)	Low Brace Angle	Upper Brace Angle	Low Brace Length (ft)	Upper Brace Length (ft)	Amp1*	Amp2*
31'-0"	19°	29.5°	24'-0"	9'-5"	2.9	6.1

* Amp1 and Amp2 are motion amplification factor (d_c/D) and force amplification (FB/FD) respectively

Design Criteria and Static Lateral load

The design criteria for office building are compliant to BOCA 96 and Massachusetts State Building Code. The lateral structural systems are designed to meet AISC strength requirements and seismic provisions for zone 2B. No force reductions due to the damping increase by viscous dampers was taken into account at this design stage. The design coefficients for the equivalent lateral load of BOCA 96 are tabulated in Table 3. Wind design criteria are for 100-year return for strength and 10-year return for serviceability are also shown.

Table 3 Equivalent Lateral Load Design Parameters for BOCA 96

Design Wind Load		Design Earthquake Load	
Wind Speed	90 mph	Seismic Zone	2A
Design Category	B	Peak Acceleration (Av)	0.12g
Importance Factor	1	Reduction Factor (R)	4.5
Aspect Ratio of Depth to Width	3	Soil Factor (S3)	1.5
Aspect Ratio of Depth to Width	1	Building Period (Ta)	3.65 sec

Wind Tunnel Test Results and Wind Time History Loading

The 39-story office building wind tunnel test was carried by RWDI, Ontario, Canada. The tests were conducted on a 1:400 scale model in presence of all surrounding buildings within a full-scale radius of 1600ft. The magnitude of simulated wind speed for a 100 year return period was scaled to correspond to a fastest-mile speed of 94mph at 33 ft (10m) above ground in open terrain, which is consistent with the Massachusetts Building Code and ASCE-93 Standard. In order to perform nonlinear time history required for viscous damper design, a specific time series was generated from the high frequency force-balance wind tunnel results. Response comparisons for various wind force time history studies are shown in Table 4 along with wind tunnel predictions.

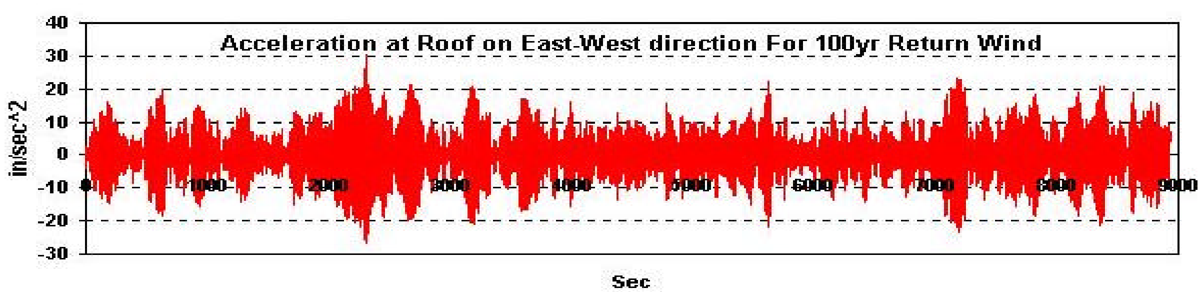

Fig. 4 Roof Acceleration Response on (E-W) From Wind Tunnel Test

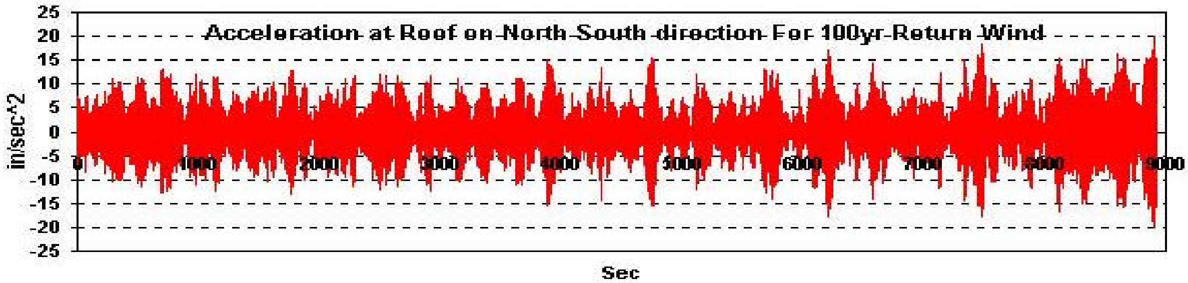

Fig. 5 Roof Acceleration Response on (N-S) From Wind Tunnel Test

Table 4 Response Comparison of Truncated Time Series Data

	10 Year		100 Year		Wind Tunnel
Building Response	4.2 hrs (45000 data)	5.7 mins (1024 data)	3.5 hrs (45000 data)	4.7 mins (1024 data)	100 yr 1.5% damp.
36^{th} x-Accel. (in/s^2)	16.1	14.6	30.6	27.4	NA
y-Accel. (in/s^2)	12.2	10.7	20.2	17.9	NA
36^{th} x-Displ. (in)	13.6	12.5	23.1	20.8	18.0
y-Displ. (in)	8.4	6.8	13.3	11.2	13.1
x-Base Shear (kip)	2738	2630	4374	3907	3541
y-Base Shear (kip)	1832	1699	3118	2903	2844
x-Base Moment (kip-in)	6.97×10^6	6.13×10^6	1.19×10^7	1.03×10^7	1.46×10^7
y-Base Moment (kip-in)	1.10×10^7	1.01×10^7	1.86×10^7	1.67×10^7	1.14×10^7

Earthquake Analysis and Damper Design

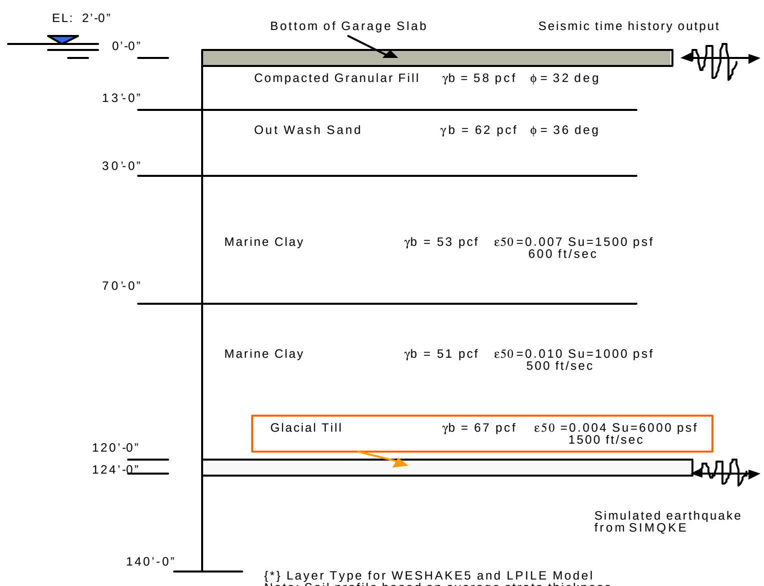

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Fig. 6 Soil Profile

Once the damper system was designed to reduce wind motion, the response of the system must be investigated under expected earthquake motions. Since no ground motion records are available at this site, ground motions to test the design must be simulated.

The office building is located in Bay Back, Boston a moderate seismic zone according to the Massachusetts State Building Code. The design peak ground acceleration is 0.12g. Soil profile is shown in Fig.6. Since time history analysis is required for viscous damper design, artificial earthquake time histories are generated by SIMQKE (Vanmarcke 1976). Design spectra factors used here conform to BOCA 96:

Peak velocity-related acceleration factor (Av):	0.12
Site soil profile properties (S):	1.2
Modal seismic coefficient (C_{sm}):	1.2 Av S / R $T_m^{2/3}$ not over 2.5 Av,
	3 Av S / R $T_m^{4/3}$ for T_m larger than 4 second

T_m is modal period in second of mth mode of building. R is modification factor. No response reduction is considered here (R=1). The target pseudo-velocity design spectra (in/sec) for SIMQKE is simply defined as C_{sm} 2p / T_m. Total duration time for simulated time history is 20 seconds in which 2 seconds rising time and 15 seconds level time. Code maximum ground acceleration is 0.12g. Three cycles are using here to smooth the response spectrum. Three damping ratios (1%, 2% and 5%) are examined.

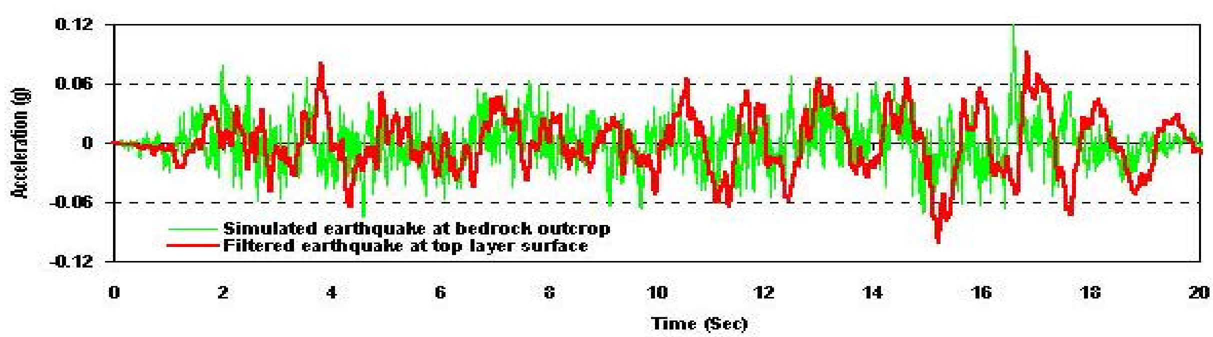

Fig. 7 Simulated Earthquake Time History

The site-specific dynamic response of layered soil deposit is estimated by using program WESHAKE5 (Yule and Wahl 1995). The soil properties and classification are grouped and shown Figure 6. The comparison of simulated and filtered earthquake time history is shown in Fig.7. The response spectra for simulated and filtered earthquake time history are compared with other response spectra shown in Fig.9. It is found that site-specific period of this building is approximately 2.5 second Fig.8 shows the building response comparison of earthquake simulation with a variety of notable case histories which have different frequency contents. The peak ground acceleration for all time histories is scaled to 0.12g. Typically the earthquake simulations produced forces in the viscous dampers which governed the damper ultimate force capacity. This ultimate force becomes part of the damper design specification.

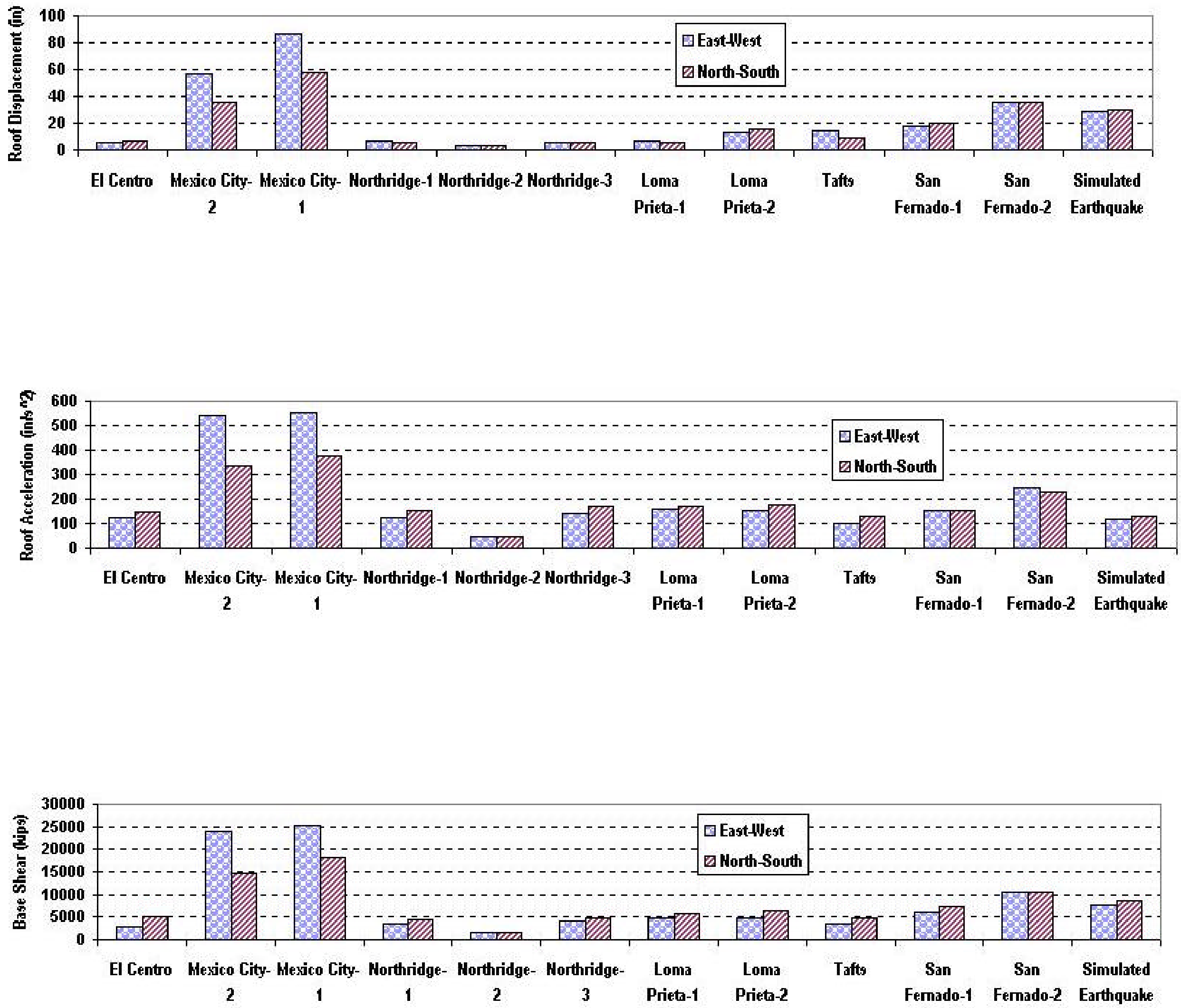

Fig. 8 Building Response Under Various History Record (1% Structural Damping and 0.12g Peak Acceleration)

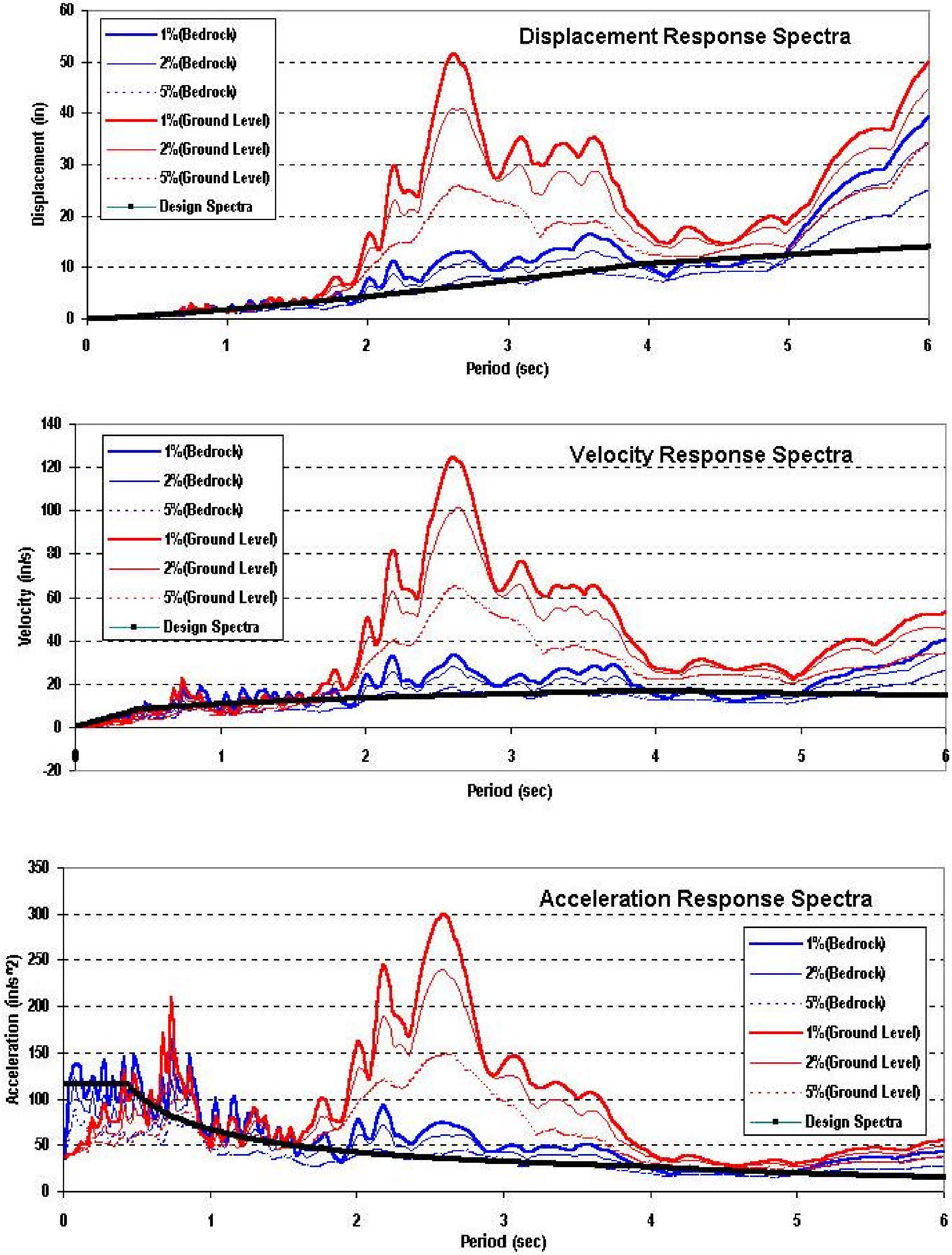

Fig. 9 Comparison of Simulated Time History Response Spectra with Design Spectra

Building Response Reductions

High-rise building design is often governed by member stiffness rather than member strength. This is especially true in moderate seismic zones. Under extreme wind conditions, large deflections or story drifts of a building may result in damage of the nonstructural partitions and cladding. Under smaller storms occupant comfort can control the design. The wind tunnel story drift for 100-year return storm is about 1/280 in each direction. With the introduction of sixty viscous dampers, the deflection and minimum story drift index are much improved as demonstrated in Fig 10. The building defection and drift under seismic loading is also greatly improved as shown in Fig 11.

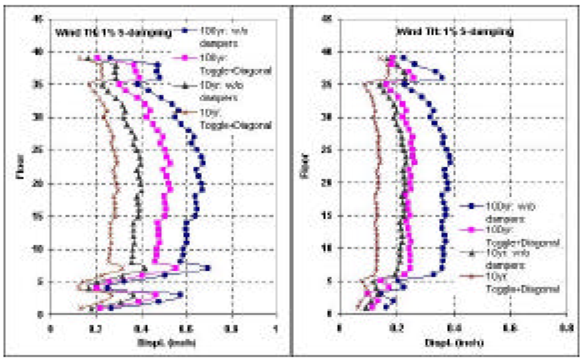

Fig. 10 Story Drift on E-W and N-S (Wind Load)

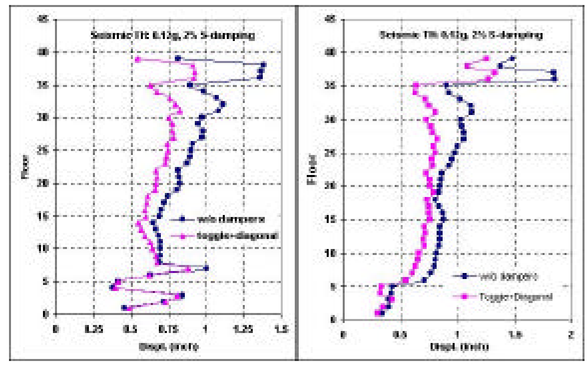

Fig. 11 Story Drift on E-W and N-S (Seismic Load)

Humans are sensitive to acceleration and its change rather than building displacement and velocity. Acceptability of motion perception varies widely. In common practice, the suggested peak values range from 10 milli-g to 30 milli-g for a storm with a return period of 10 years (10mg for apartments and 30mg for offices). For this office building, the acceleration at the highest occupied (36^{th} floor) level is predicted to be 41mg. The introduction of the damper system reduced the floor accelerations by approximately 35% as shown in Fig. 12 and Fig.13.

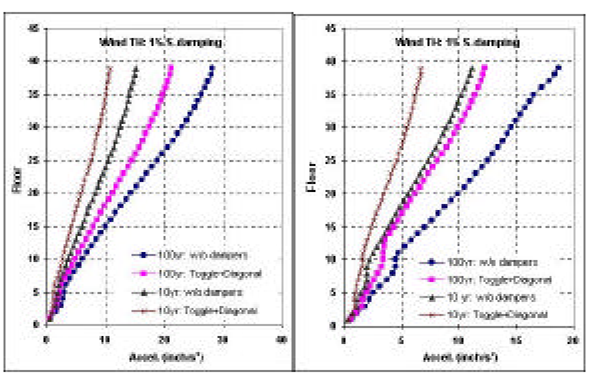

Fig. 12 Floor Accel. on E-W and N-S (Wind Load)

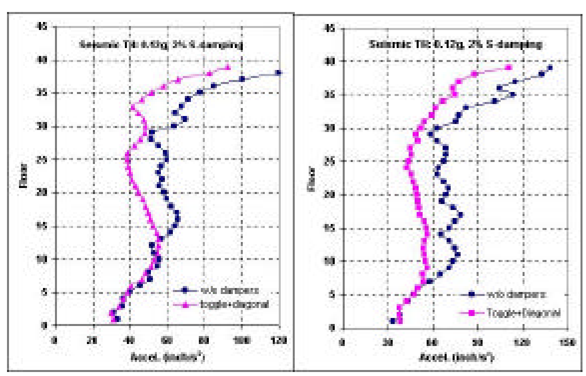

Fig. 13 Floor Accel. on E-W and N-S (Seismic Load)

Results Summary

The equivalent static wind and seismic load effects of Massachusetts State Building Code and National Building Code (BOCA 93) on the office building are plotted on Fig.14 and Fig.15. Under wind load, wind tunnel predicts more pressure at 300 feet and above, but diminished quickly on lower floors. In general, the wind load indicates a more severe design requirements than that of earthquake loads.

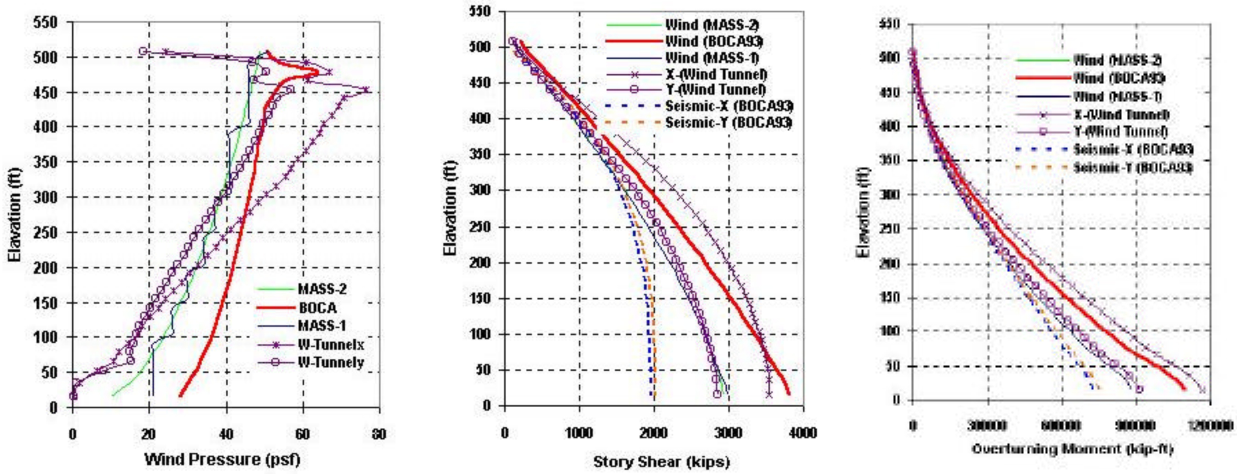

Fig. 14 Comparison of Building Behavior for Equivalent Static Lateral Load

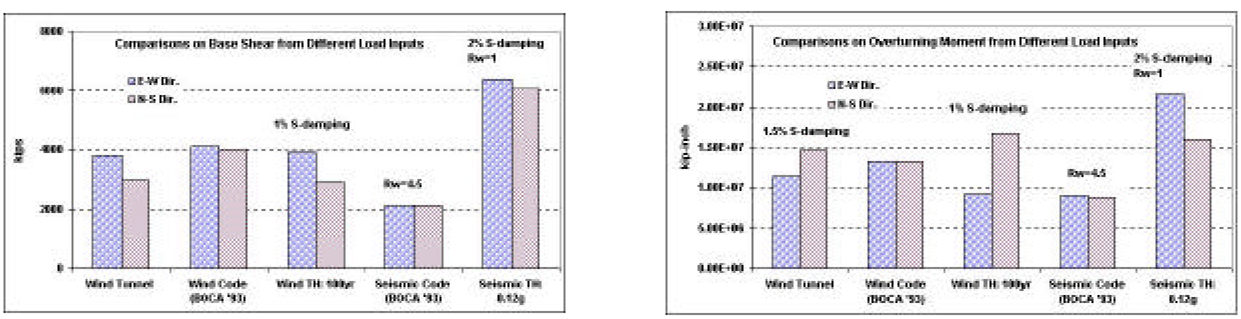

Fig. 15 Building Base Shear and Overturning Moment Comparison for Different Load Condition

The effectiveness of viscous dampers on the office building were summarized in the Table 5. As seen from this table, viscous dampers will improve the building dynamic behavior from 20% to 30%. These dampers gave the building with an additional inherent damping, equivalent to entire building structural damping ratio of approximately 3%-4%. This is a significant increase above the assumed 1% structural damping for the tower without the dampers.

Table 5 Results Summary Table for Time History Analysis

		Wind Load Condition E-W (X) Dir.	Wind Load Condition N-S (Y) Dir.	Seismic Load Condition E-W (X) Dir.	Seismic Load Condition N-S (Y) Dir.
Response without dampers	accel. at 37th Flr. (in/s^2)	27.4	17.9	95.1	112.1
	displ. at 37th Flr. (in/s)	20.8	11.2	23.5	26.2
	Base Shear (kips)	3907	2903	6387	6057
Response with dampers	accel. at 37th Flr. (in/s^2)	20.6	12.0	76.3	77.8
	displ. at 37th Flr. (in/s)	16.4	7.3	21.9	23.0
	Base Shear (kips)	3172	2038	5852	5246
6th ~ 15th	Max. stroke (in)	0.37	0.77	0.59	1.87
	Max. damper force (k)	113	18	409	81
16th ~ 25th	Max. stroke (in)	0.36	0.82	0.43	2.12
	Max. damper force (k)	108	17	326	80
26th ~ 35th	Max. stroke (in)	0.32	0.80	0.55	2.17
	Max. damper force (k)	60	8	366	66
Overall Damping	Evaluated by energy	1.89%	2.0%	3.56%	3.8%
	Evaluated by accel.	1.94%	3.08%	3.56%	4.58%

Note: 1% and 2% internal modal damping included for wind and seismic condition respectively

Conclusions

The viscous damper system with a motion amplification device proved to be a very cost effective method to reduce accelerations resulting from the buffeting by vortex shedding from winds off of an adjacent tower. Other interesting aspects of the design are the following:

1. Current modal analytical approaches can produce erroneous results and should not be used for final design for systems with motion applification devices.

2. Non-linear time history analysis for both wind and seismic effects is required for damper system design.

3. Due consideration of the effects of the local damper forces must be considered. These forces can have a significant impact on the design of the local beams, connections and diaphragms.

4. The stiffness of the damper bracing system has an important role in the overall damper effectiveness. This is especially true for the stiffness of members in a motion amplification system. Large member stiffness' are required to insure that the response reductions predicted analytically can be achieved in the actual installation.

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