VISCOUS-DAMPER WITH MOTION AMPLIFICATION DEVICE FOR HIGH RISE BUILDING APPLICATIONS

by

R.J. McNamara, C.D. Huang and V. Wan

Viscous-Damper with Motion Amplification Device for High Rise Building Applications

R. J. McNamara, C.D. Huang and V. Wan

McNamara/Salvia Inc., Boston, MA 02138

Abstract

Adding damping by the use of various damping devices has become an accepted method to reduce wind-induced vibrations in tall buildings. An interesting example of a 39-story office tower is presented where large projected accelerations are the result of the vortex shedding of an adjacent existing 52-story building. Viscous dampers and a motion amplification system are utilized to suppress the anticipated accelerations. A description of the damping system and its analytical complexities are discussed. Non-linear analysis of the tower, with time history forcing functions, derived from the wind tunnel is presented. The dichotomy of the wind and seismic design requirements on the damping system is discussed. Cost data for the damper system is also presented.

Introduction

The introduction of various damping devices to dissipate energy and reduce building response to dynamic inputs has become an accepted design approach for high-rise buildings. Now approaches are continually developed by designers as evidenced by the varied applications of tuned mass dampers, sloshing dampers, visco-elastic dampers, friction dampers and viscous dampers. Each system has its own idiosyncrasies and must be evaluated for the particular project and whether the application is being used for the reduction of wind or seismic effects.

This paper presents the results of an investigation to 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 of code loadings and deflection limitations and was tested in a wind tunnel of RWDI facilities in Canada. The building is 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 developed 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 the 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 approach were investigated and evaluated for cost and project 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 form US manufactures. Viscous dampers proved to be the most cost effective and least space intensive on the office tower. An extensive design program was undertaken with various dampers configurations vertically in the tower and with many variations on viscous damper specifications. Since the main intent of the damper installation is to reduce wind motions, the viscous dampers need to provide a large force output at very low displacement levels (±1/8"). In order to insure reliability at this low movement and to keep the number and cost of the damper to minimum, a motion amplification device (MAD) was included in the design. The motion amplification device was used in one direction of the structure, that being the stiffest with the lowest predicted movements.

A motion amplification device called a Toggle Brace Damper system (TBD) was tested in Buffalo by Constaintinou, etc. (1998). Their report demonstrates that on the TBD system is a very useful mechanism to amplify inter-story motion. However, the efficiency of TBD, as reported by McNamara, Huang and

Wan (1999) in companion paper, is largely dependent on various local system design parameters. Improper design parameter selection for TBD may result in malfunction for viscous dampers. From the above parameter study of TBD system, a total 60 viscous dampers are applied to 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. The viscous dampers are then designed for both 100-year return wind and moderate earthquake excitations. (Seismic zone 2, Av = .12g)

Office Building Structural System Description

39-story Office Building consists of three lateral systems at different levels as shown structural elevation diagram. From the 1st to 7th floors and above 34th floor are diagonal braced lateral systems at inner-core. The remaining of floors is a moment frame along perimeter of building. The basic structural lateral system of building is classified as a moment-frame-tube system. The typical floor system is composite metal deck with joist girder system. Typical floor area is 22,500 square feet Total weight of building is about 62,000 ton and supported by 12-6ft diameter drill shafts tied with concrete beams at inner-core and other 22-drilled shafts at perimeter columns. Viscous dampers on E-W direction are diagonally placed in two bays with 19ft length at inner-core on every other floor between 7th floor and 34th floor while TBD systems are assigned on two 31ft bay length along N-S direction at the same level. The damper system layout is shown in Fig.1.

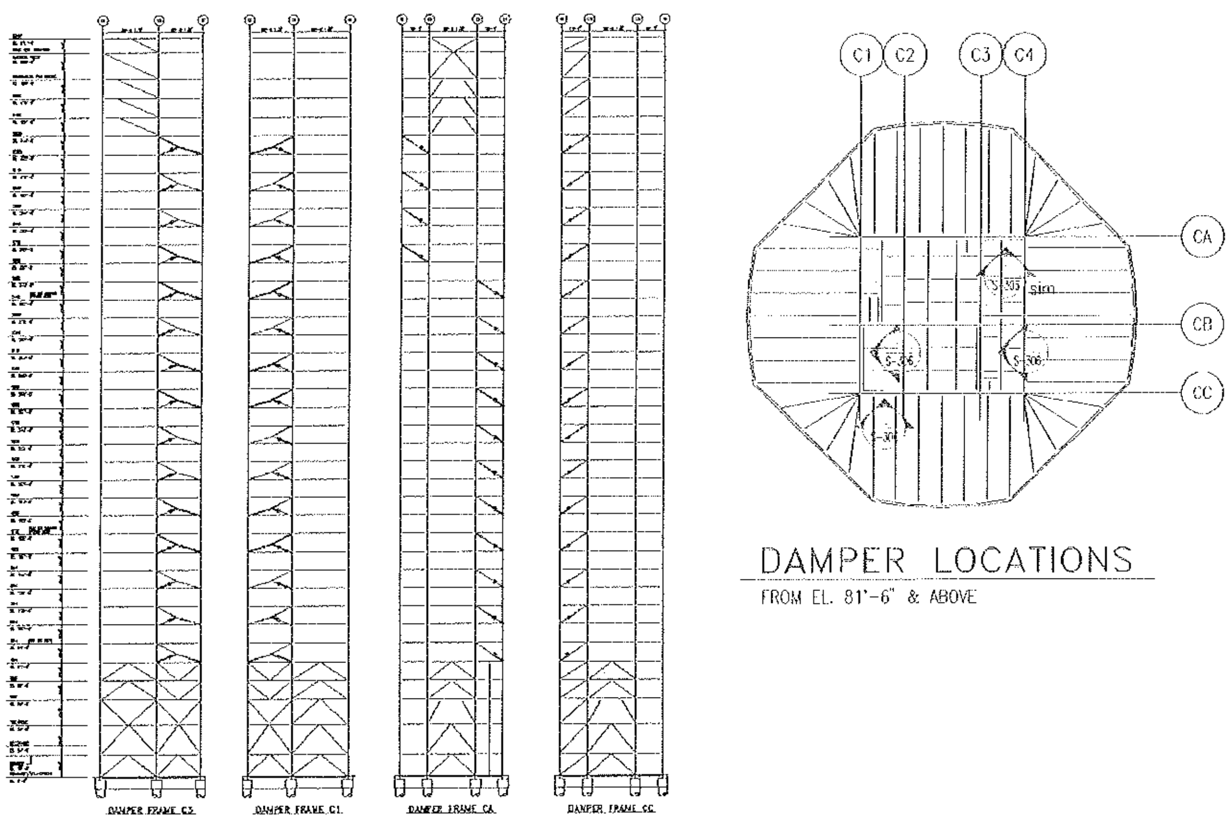

Fig.1 Viscous Damper Elevation and Key Plan

The static lateral analysis and design was conducted by ETAB6.2. The dynamic response and viscous damper design of TBD system were analyzed by SAP2000. The floor masses are lumped at center of mass. 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 bay length (19ft) on X-direction at inner-core where the damper placed is shorter than that of Y-direction (31ft). The overall building stiffness in X-direction is less than that on Y-direction. For cost effective dollar design, the TBD system with the Y-direction was used to magnify the story drift. The damping values (C) are 20 kips-sec/in from 6th floor to 25th floor and 10 kips-sec/in from 26th floor to 35th floor. In the X-direction, damping value (C) are 300 kips-sec/in from 6th floor to 25th floor and 200 kips-sec/in from 26th floor to 35th floor. The damper layout is shown Fig 1. The configuration of TBD devices on the building are described both in table 2 and in Fig,2 and Fig.3. In order to activate viscous damper elements, total 361 full modes are considered in building dynamic analysis. Linear viscous dampers and TBD connection joints are provided and designed by Taylor Devices, Inc.

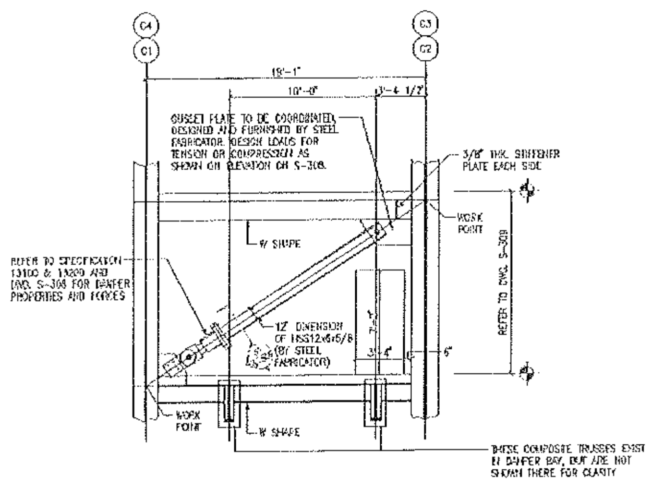

Fig.2 Diagonal Viscous Damper on E-W(X) direction

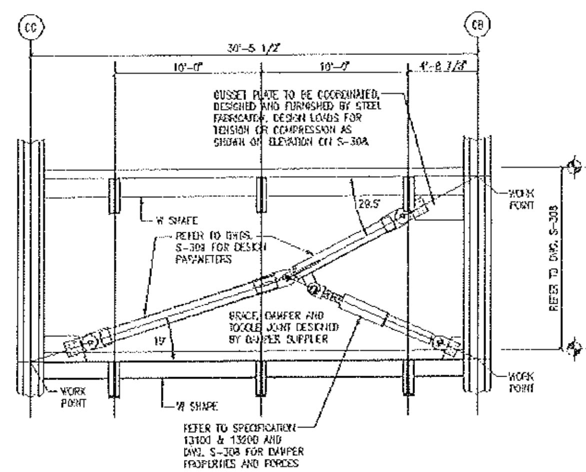

Fig.3 Toggle Brace Damper on N-S(Y) direction

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 (δ_c/Δ) and force amplification (FB/FD) respectively.

**: Cross area of brace members are 20 in^2; target story drift (Δ) is 0.8 in; damping (c) is 20 k-sec/in and cycle frequency of excitation (ω) is 1.216 rad/sec. For simplicities, ω = 1 rad/sec is used in this section studies.

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 reductions due to the damping increase by viscous dampers is taken into account at static load 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.

The TBD system and viscous dampers are introduced to improve the serviceability of building by raising story drift index and reducing the acceleration at the top floor of building.

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)	3.65sec

Wind Tunnel Test Results and Wind Time History Generation

39-story office building wind tunnel test was carried by RWDI, Ontario. 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, specific time series was generated from the high frequency force-balance wind tunnel tests. Total 45000 data of time series points, which represents 4 hours 10 minutes, are for winds from a 300 degree azimuth with a 10 year return period. This wind azimuth was selected based on its importance on producing high accelerations in both the E-W and N-S directions. For 100 year return time series, a time scale factor 0.837 and load scale factor of 1.426 was used. The time step from 0.333 sec was scaled down to 0.279 sec and time duration down to 3 hour and 30 minutes. In order to avoid a large time consuming nonlinear time history analysis, total 1024 data (5 minutes 41 sec) of time series was adopted in the wind time history analysis where maximum acceleration occurred at 36th floor see Fig. 4 and Fig.5. The building response comparison between 45000 data and truncated 1024 data is listed in table 4.

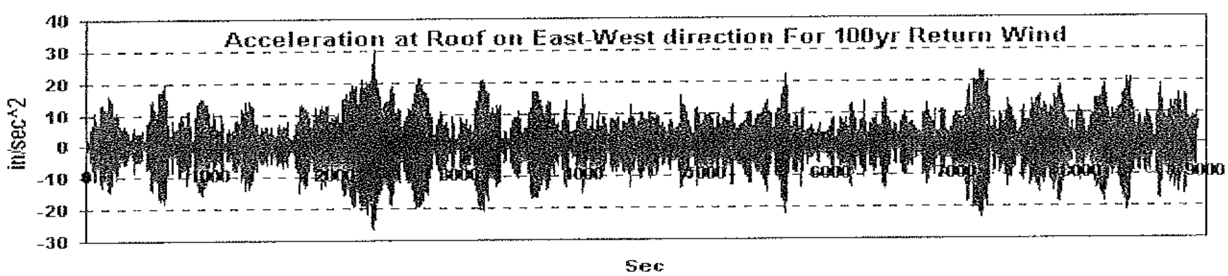

Fig.4 The Build Roof Acceleration Response on (E-W) From Wind Tunnel Test

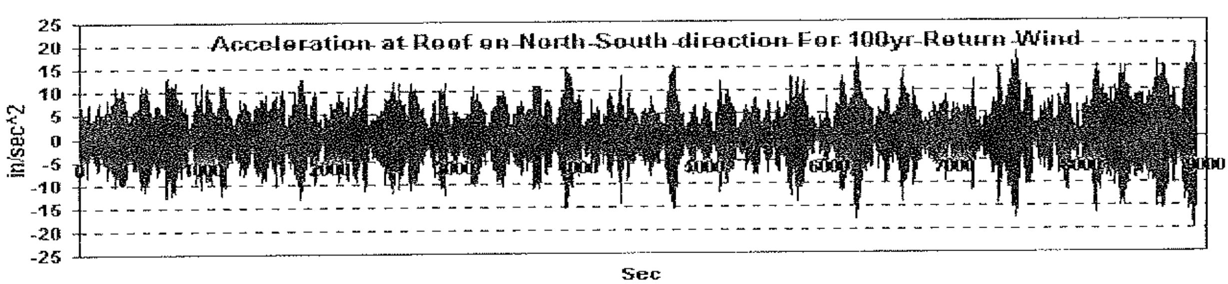

Fig.5 The Build Roof Acceleration Response on (N-S) From Wind Tunnel Test

Table 4 Response Comparison of Truncated Time Series Data

	10 yr		100 yr		Wind Tunnel
Building Response	4.2 hrs (45000 data)	5.7mins (1024 data)	3.5 hrs (45000 data)	4.7mins (1024 data)	100yr 1.5%damp.
36th 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
36th 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	$6.13 \mathrm{X} 10^6$	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 Time History Simulation

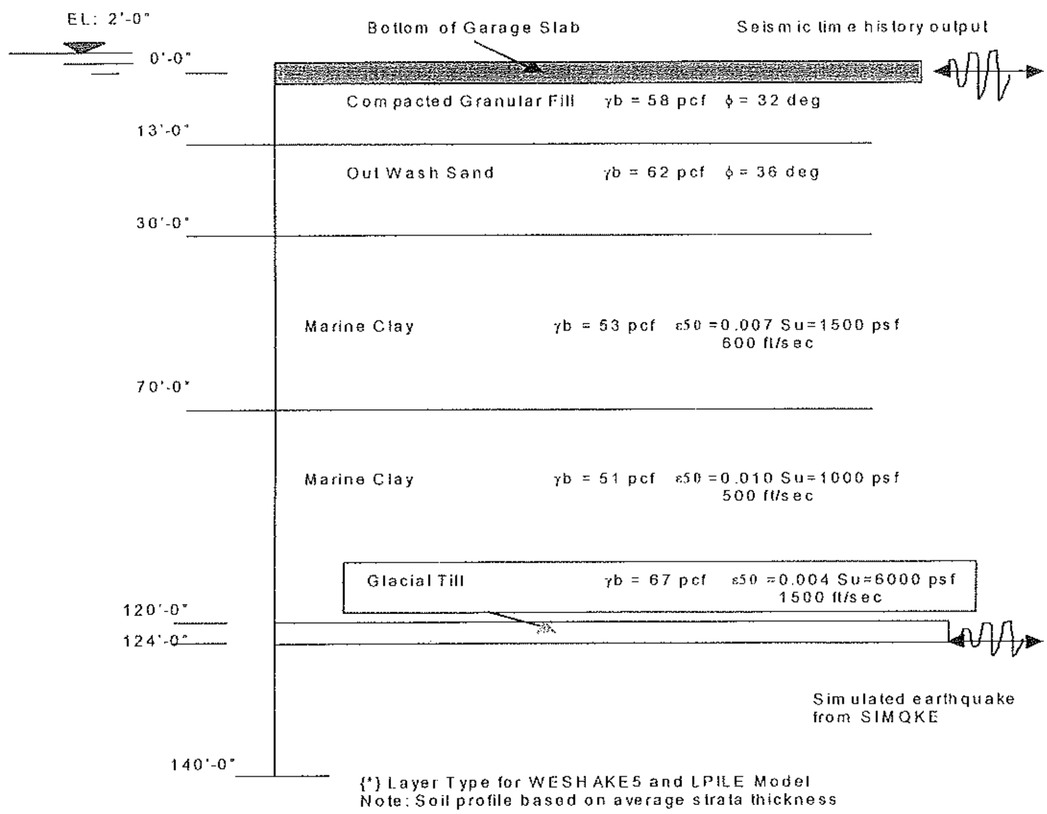

Fig. 6 Soil Profile

The office building is located in Bay Back, Boston a moderate seismic zone according to the MASS State Building Code. The design peak acceleration is 0.12g. Soil profile in this area 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), due to lack of recorded data for the area. 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 m^{th} mode of building. R is modification factor. No response reduction is considered here (R=1), for investigation of initial effects on the dampers. The target pseudo-velocity design spectra (in/sec) for SIMQKE is simply defined as $C_{sm}\ 2\pi\ /\ T_m$. Total duration time for simulated time history is 20 seconds in which 2 seconds rising time and 15 seconds level time. Desired 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.

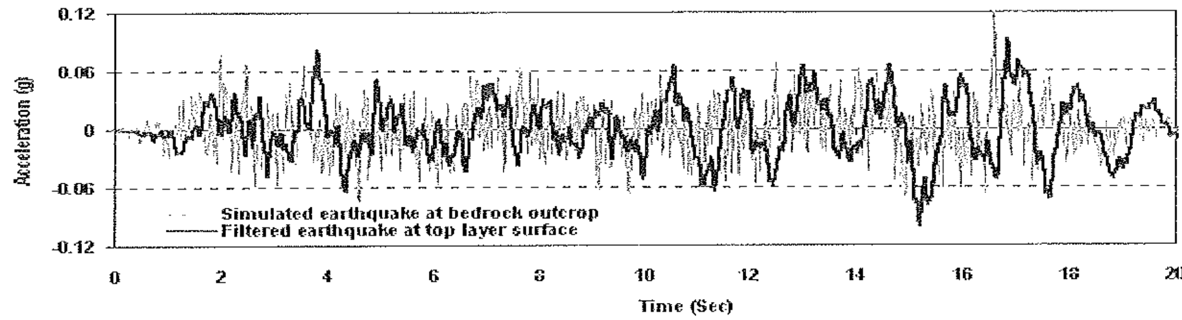

Fig.7 Simulated Earthquake Time History

The site-specific dynamic response of layered soil deposit is estimated by using program WESHAKE5 (Yule and Wahl 1995). The damping and shear modulus for typical soil are classified by numerous researchers and incorporated into program. The soil properties and classification in this project are grouped and shown in soil profile diagram Fig.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 desired response spectra shown in Fig.9. It is found that site-specific period of this building is around 2.5 second. Fig.8 shows the building response comparison of earthquake simulation with a variety of notable case histories of which have different frequency contents. The peak ground acceleration for all time histories is scaled to 0.12g without any response reduction (R=1).

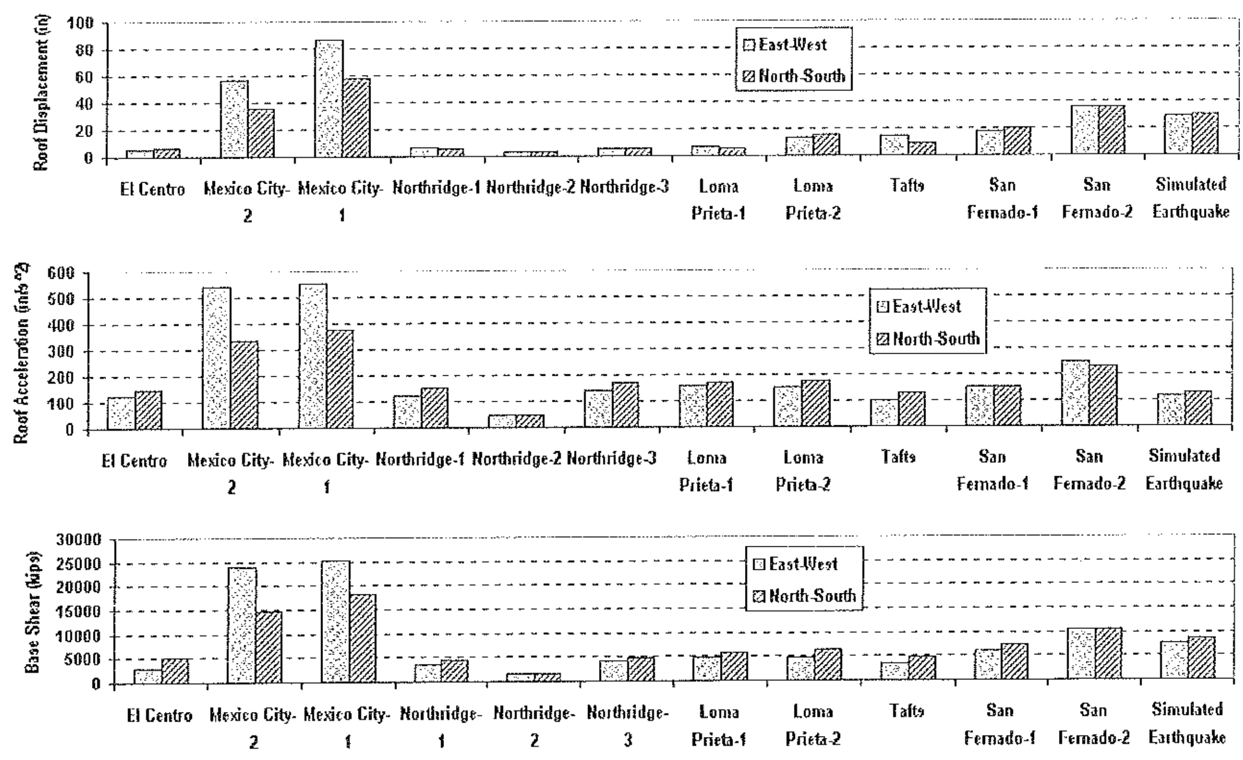

Fig. 8 Building Response under Various History Record
(1% Structural damping and 0.12g peak Acceleration)

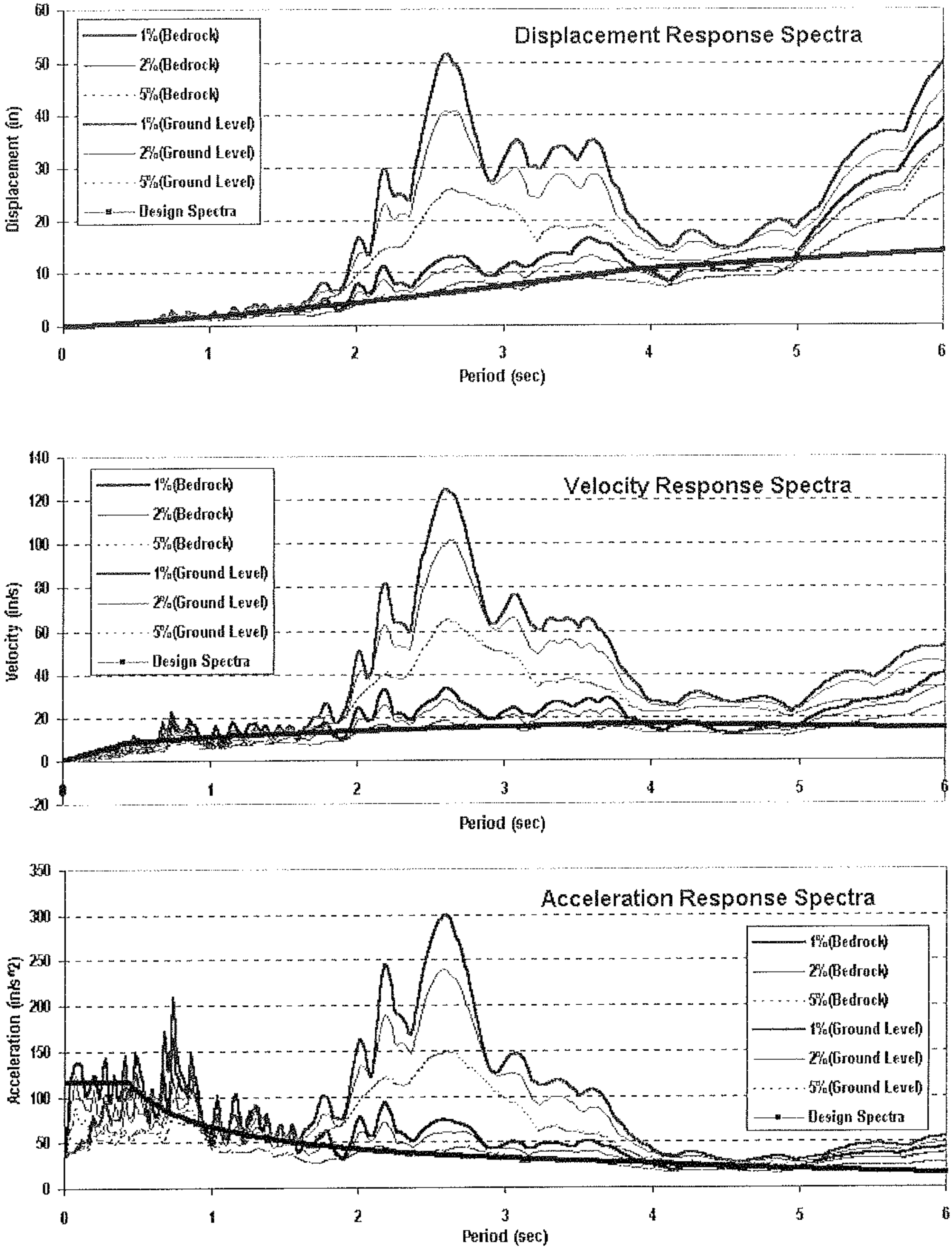

Fig. 9 Comparison of Simulated Time History Response Spectra with Design Spectra

Building Story Drift

High-rise building design is usually governed by stiffness rather than member strength due to its inherent flexibility. This is especially true in moderate seismic zones. Under normal wind conditions, large deflections or story drifts of a building may result in damage of the nonstructural partitions and cladding, overall building stability and comfort of tenants. As stated earlier, the major building lateral structural system is moment frame tube system that is designed to meet strength requirement of current specification. The minimum story drift of building for 100-year return wind tunnel test is about 1/280 in each direction. After sixty viscous dampers are introduced, the deflection and minimum story drift index will be much improved as shown in Fig 10 and 11. The building defection and drift under seismic condition are also and improved.

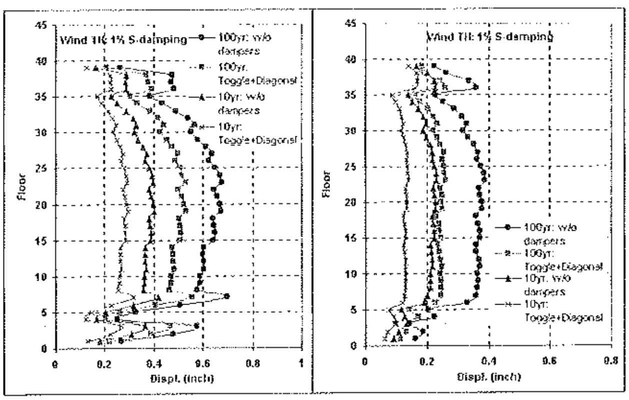
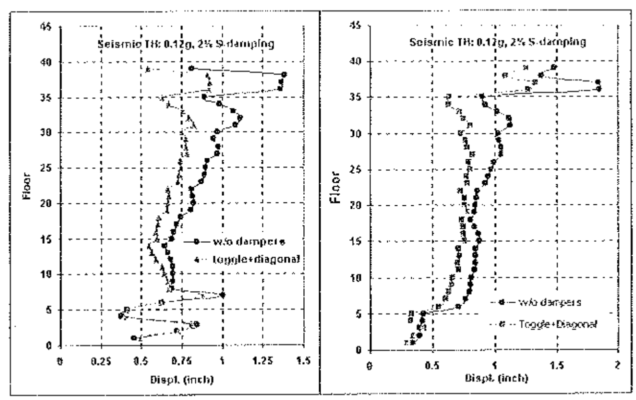

Fig.10 Story Drift on E-W and N-S (Wind Load) Fig.11 Story Drift on E-W and N-S (Seismic Load)

Perception of Motion

Humans are usually sensitive to acceleration and its change rather than displacement and velocity. Past experiments showed that the acceleration perception threshold under cyclic motion was about 5mg. The accelerations became annoying when they reach 20mg. However, that perception and annoyance are depend on ambient motions and activities etc. Occupants in apartments or hotels are more sensitive than in offices. Acceptability of motion perception varies widely. In common practice, the suggested values range from 10~30mg for 10 years return where 10mg for apartments and 30mg for offices. For this office building, the acceleration at the highest occupied (36th floor) level is predicted as high as 41mg.for 10 years return. Dampers reduced the floor accelerations by approximately 35% shown Fig. 12 and Fig.13. Torsion effects are not critical in this building due to the symmetrical layout of building and low period of the torsion oscillation.

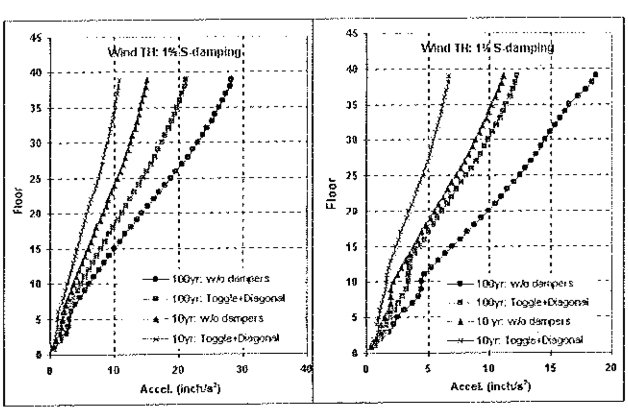
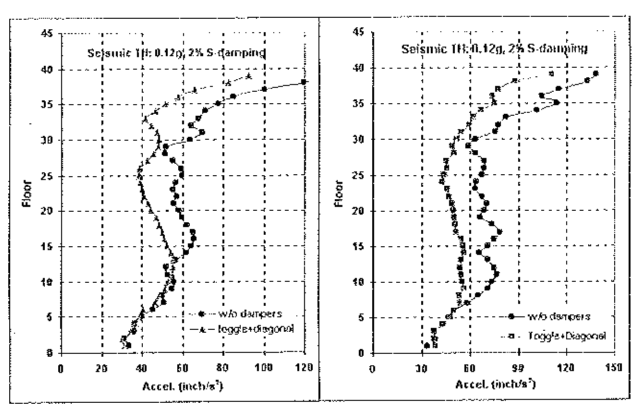

Fig.12 Floor Accel. on E-W and N-S (Wind Load) Fig.13 Floor Accel. on E-W and N-S (Seismic Load)

Results Summary

The wind and seismic effect comparison 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 exerts more pressure at 300 feet and above on E-W direction, but diminished quickly on lower floors. In general, wind load shows more severe influence on the office building than that of earthquake condition.

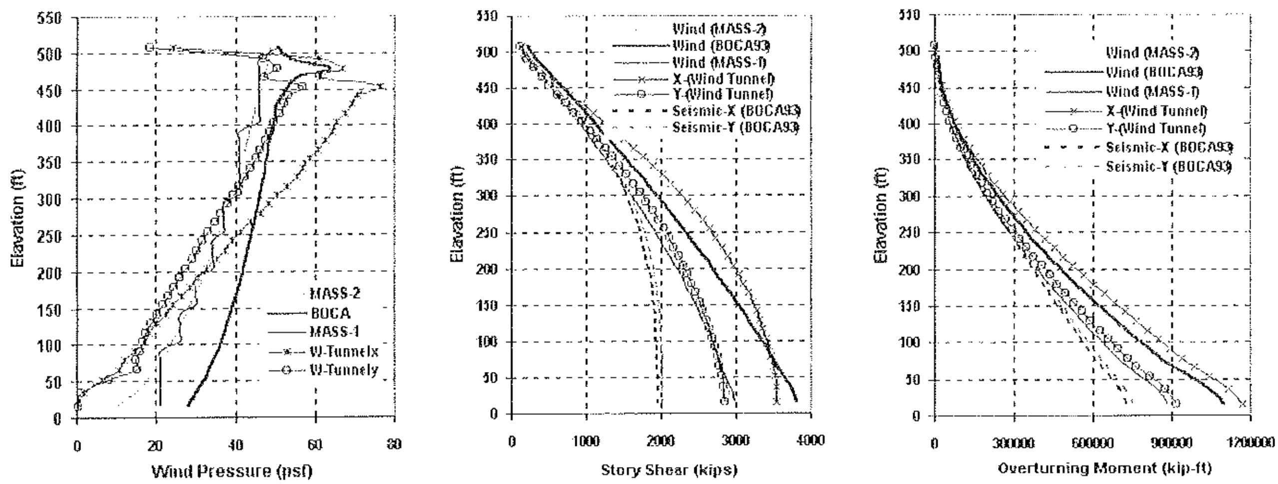

Fig.14 Comparison of Building Behavior for Equivalent Static Lateral Load

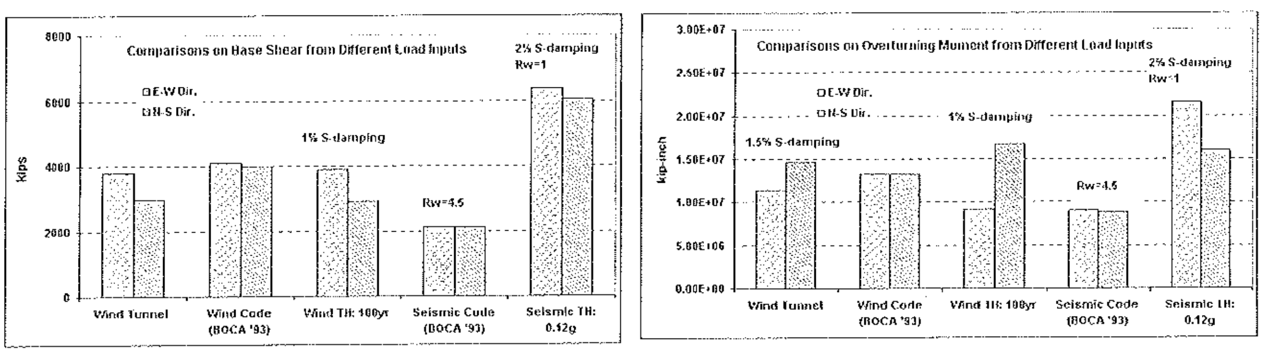

Fig.15 Building base shear and overturning moment comparison for different load condition

The effectiveness of viscous dampers on the office building were summarized in the table 5. From this table, it shows that the viscous dampers will improve the building dynamic behaviors from 20% to 30%. These dampers gave the building additional inherent damping, which are equivalent to entire building structural damping ratio around 3%.

Table 5. Results Summary Table for Time History Analysis

		Wind Load Condition		Seismic Load Condition	
		E-W (X) Dir.	N-S (Y) Dir.	E-W (X) Dir.	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
26^{th}~35^{th}	Max. stroke(in)	0.32	0.80	0.55	2.17
	Max. damper force(k)	60	8	366	66
Overall	Evaluated by energy	1.89%	2.0%	3.56%	3.8%
Damping	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.

Conclusion

The viscous damper system with a motion amplification device proved to be a very cost effective method to reduce wind motions. Cost of the installation, including the motion amplification device, was less than one million dollars. Other aspects from the design are the following:

1. Current modal analytical approaches can produce erroneous results and should not be used for final design.
2. Non-linear time history analysis for both wind and seismic effects is required for damper design.
3. Due consideration of the effects of the local damper forces can have a significant impact on the design of the local surrounding beams, connections and diaphragms.
4. The stiffness of the damper bracing system can have a significant reduction in damper effectiveness. This is especially true for the stiffness of members in the motion amplification system. Large member stiffness' are required to insure the response reductions predicted analytically can be achieved in the actual installation.

Reference

1. Costantinou M.C., Tsopelas P. and Hammel W. (1998), *Testing and Modeling of an Improved Damper Configuration for Stiff Structural Systems*, Technical Report to the Center for Industrial Effectiveness and Taylor Devices, Inc.

2. McNamara, R. J, Huang, C.D. and Wan, V. (1999), *Parametric Study for Motion Amplification Device with Viscous-Damper* , Submitted to 2000 ASCE Congress - Philadelphia

3. Soong, T.T and Dargush G.F. (1996), *Passive Energy Dissipation Systems in Structural Engineering*, Wiley and Sons, London