

## VISCOUS DAMPERS FOR HIGH-RISE BUILDINGS

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### ABSTRACT :

Viscous dampers (VD), when used in high-rise buildings in seismic areas, should reduce the vibrations induced by both strong winds and earthquakes. The optimal behaviour in these two situations is not usually the same, thus the design requirement for VD is often that they should have two different behaviours in the different range of velocities corresponding to wind and earthquake. Recently VD have been applied in three high-rise buildings in Asia, the Taipei 101 in Taipei, Taiwan, and the twin St. Francis Shangri-La Towers in Manila, Philippines. Taipei 101 has been the world's tallest building since 2004 (508m). In this building 8 VD are used to control the motion of the Tuned Mass Damper (TMD) installed at its top. Said TMD has already been put into operation by earthquakes in March 2005 and May 2008 and by many strong typhoons. The St Francis Shangri-La Place is a residential development composed by two towers, each rising to 213m. In this case, 32 VD are installed into the towers structure according to an Arup newly developed and patented configuration. This paper describes the technology of VD installed in said high-rise buildings, and the results of the wide laboratory testing campaign aimed at verifying their behaviour during both wind storms and earthquake.

**KEY WORDS :** viscous dampers, passive control, energy dissipation, high-rise, performance based design, wind

### 1 INTRODUCTION

Viscous dampers (VD) have been used in the last 30 years in major civil structures to mitigate the effects of earthquakes. Their use in high-rise buildings built in seismic areas is a challenge for the designers, since they should reduce the vibrations induced by both strong winds and earthquakes, and the optimal behaviour in these two situations is not usually the same. Consequently, the design requirement for VD to be used in high-rise buildings is often that they should have two different behaviours in the different range of velocities corresponding to wind and earthquake. Recently VD with said behaviour have been applied in three high-rise buildings in Asia, the Taipei 101 in Taipei, Taiwan, and the twin St. Francis Towers in Manila, Philippines.

Taipei 101 has been the world's tallest building since 2004. It has 101 stories above ground and its height is 508m. In this building 8 VD are used to control the motion of the Tuned Mass Damper (TMD) installed at its top, that is the world's largest passive TMD. The VD maximum design force is 1000kN, and the stroke is  $\pm 750$ mm.

The St Francis Shangri-La Place is a residential development composed by two towers, each rising to 60 floors, 213m. In this case, 32 VD - installed into the towers structure according to an Arup newly developed and patented configuration - are required to react in a different way to wind induced vibrations and high velocity (earthquake) motions. Such units are characterized by a maximum design force of 2600kN and a stroke of  $\pm 220$ mm.

This paper describes the technology of VD installed in said high-rise buildings, and the results of the wide testing campaign aimed at verifying their effective behaviour during both wind storms and earthquake.

## 2 TAIPEI 101 – TAIWAN

Wind tunnel testing performed by Rowan Williams Davies & Irwin Inc. (RWDI) determined the wind-induced behaviour of the Taipei Financial Center. This included recommendations of design base bending moments, torsion, effective static floor-by-floor loads, appropriate load combinations, and predictions of the wind-induced accelerations for various return periods. Some modifications to the aerodynamic shape of the tower were explored in the wind tunnel, and were implemented to help reduce the wind-induced loads and motions of the tower. After completing this shape optimization process, the peak total accelerations predicted for the ½ year return period were 0.75% and 0.63% of gravity, where in the first case the effects of typhoons are included and in the second case disregarded. The Taiwanese building codes make recommendations that the ½ year return period acceleration not exceed  $0.05\text{m/s}^2$ , or 0.51% of gravity.

Of the remaining methods available to reduce wind-induced accelerations, a TMD was selected by the design team and the owner. The TMD configuration selected for implementation was a 660 metric tons mass, spherical in shape, suspended on cables as a single stage pendulum, and architecturally-integrated into the theme of the tower (Figure 1). The TMD was designed to be an attraction for visitors to the building.

Extensive analysis was undertaken by Motioneering (the designers of the TMD) to ensure that the TMD would provide the necessary reduction in wind-induced accelerations, including theoretical investigations to design the TMD for controlled behaviour during the design seismic events.

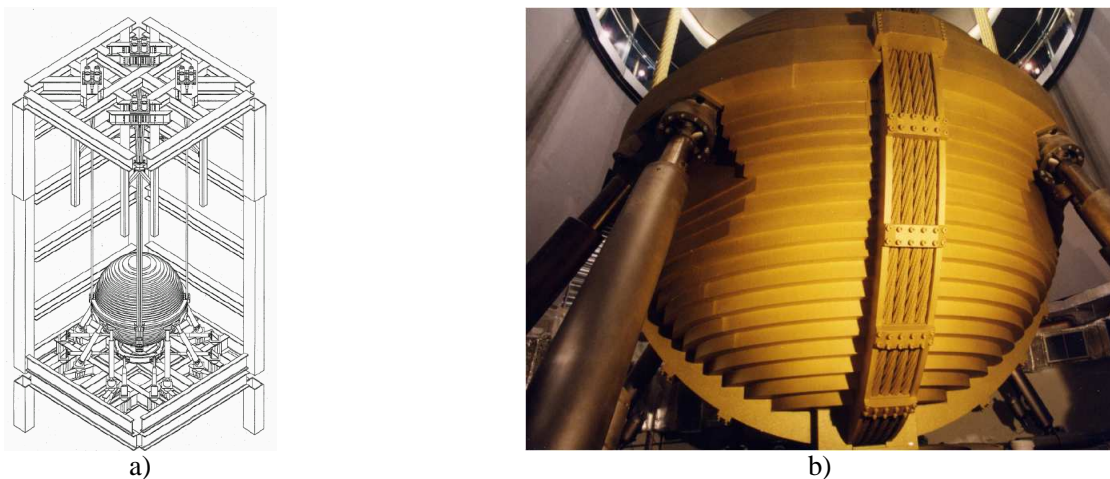


Figure 1 – Taipei 101: TMD general configuration (a) and primary viscous dampers (b).

### 2.1 Dynamic Analysis

Considerable research and in-situ measurement has been performed with the intention of optimizing the vibration mitigating effects of a TMD. Most of this work assumes linearity of excitation, component interaction, and response of the combined building/TMD system; such that the physical manifestation resembles the linear model. These analytical techniques can give moderately accurate results. These calculations are generally carried out in the frequency domain, where such linear assumptions are necessary [Simiu & Scanlon, 1986; Korenev & Reznikov, 1993; Soong & Dargush, 1997]. In 2000 Motioneering developed analysis tools which simulate the time domain response of such a building/TMD system, and not requiring as many simplifying assumptions. The only limitation to the complexity of the dynamic model is the building model itself, derived from software used by the structural engineer to determine the eigen solution for the structure. Every aspect of the TMD is modeled in its full nonlinear nature, from non-linear force/velocity relationships in the VD, to the instantaneous tracking of VD geometry, force saturation levels, and even a secondary VD system which is only active in extreme wind or seismic events.

## 2.2 Wind Induced Behaviour

The main purpose of the TMD design is to reduce the half year-return accelerations to below a criteria of  $0.05\text{m/s}^2$ . For simulations involving wind excitation of the building, it is widely accepted that only the fundamental modes of the building are significant contributors. To save computational effort, the building response is calculated in modal coordinates, while the TMD response is tracked in spatial coordinates. The time domain forcing function is taken from the wind response files, previously measured in the wind tunnel by RWDI. Through appropriate scaling factors many different wind speeds at gradient height, from 36 different azimuths around the building, are simulated in a computer simulation. The resulting response histories are then combined with a probabilistic meteorological model of the winds in the Taipei area, and predictions are made for anticipated acceleration levels as a function of yearly return period. A multi-iterative approach is taken to determine the optimal combination of TMD properties (including VD properties) to attain the target performance level.

## 2.3 Seismic Design

Further to the seismic design information received from Evergreen Consulting Engineers (the local structural engineer), the 195 gal seismic event was used by Motioneering for the design at the damage limitation state (DLS), while the 390 and 500 gal seismic event were used as the basis for design at the ultimate limit state (ULS). For Motioneering's purposes, structural damage (DLS) was defined as any component which would suffer plastic deformation of any sort, while structural safety (ULS) was defined as the preventing the mass block from becoming disconnected from the support cables and the structure in any way. In advance of constructing the TMD, a significant amount of time-domain based analysis was performed to ensure that the above two criteria could be met for all of the design-level seismic events. As construction of the tower proceeded towards completion, and once a set of tower structural properties could be measured, a confirmation study was also undertaken, to provide physical substantiation for the earlier design assumptions during a very strong seismic event (i.e. PGA of 390 and 500 gal). This involved close coordination with the tower structural engineers, as they generated revised nonlinear floor motion time histories based on frequencies measured in the substantially-complete structure.

Table 1 – Peak response quantities for the 6 non-linear seismic analyses.

PGA (gal)	Event file	Max TMD Amplitude (m)	Primary VD (pin to pin)						Secondary VD (pin to pin)						Cable Group (4pc)		
			Max Length (m)	Max Limit (m)	Status	Min Length (m)	Min Limit (m)	Status	Max Length (m)	Max Limit (m)	Status	Min Length (m)	Min Limit (m)	Status	Max Tension (MN)	Max Limit (MN)	Status
390	Spectrum	1.27	3.81	4.18	OK	2.96	2.68	OK	3.26	3.58	OK	2.91	2.58	OK	4.03	17.33	OK
	SE School	1.58	4.04		OK	2.97		OK	3.50		OK	2.72		OK	5.00		OK
	Temple	1.36	3.89		OK	2.95		OK	3.33		OK	2.85		OK	4.65		OK
500	Spectrum	1.40	3.91	4.18	OK	2.94	2.68	OK	3.36	3.58	OK	2.82	2.58	OK	4.75	17.33	OK
	SE School	1.60	4.06		OK	2.93		OK	3.52		OK	2.70		OK	4.67		OK
	Temple	1.65	4.11		OK	2.93		OK	3.55		OK	2.76		OK	4.97		OK

Upon simulating the Tower TMD response to these revised building motions, it was found that the TMD would behave as expected, that is that no contact with the building structural slab systems would occur under any of the six seismic design scenarios. Also, no damage is expected to occur to any of the TMD components. The installed primary and secondary VDs have adequate force and stroke capabilities to replicate the behaviour modeled in the simulations. Table 1 details peak response quantities for the 6 nonlinear simulations.

## 2.4 Viscous Dampers Test Program

To provide the correct level of energy dissipation to the TMD during wind events, as well as survive the earthquake and also not alter the behaviour of the TMD itself, the Primary VD were designed and manufactured by FIP Industriale to provide for a constitutive law characterized by velocity squared exponent ( $F=Cv^2$ ) and a 1000kN

reaction at a 1m/s velocity, being the maximum stroke  $\pm 750$ mm. Furthermore, they were designed to be self-limiting in the axial force that they could exert, by incorporating a Relief Pressure Valve (RPV) system to limit the reaction when subjected to velocities higher than 1m/s typically expected during a major seismic event, being the maximum seismic design velocity equal to 2.3m/s.

A major issue was to design the units in order to react steadily during wind storms that can last up to three hours. For this purpose the units were designed to provide for a 13kW maximum continuous power dissipation being capable to allow for higher power dissipation peaks for short periods (e.g.: 45kW for 1 minute, 25kW for 5 minutes, 17kW for 10 minutes). Since the dampers are subjected to large movements induced by the lateral sway of the TMD mass, their spherical hinges were designed for rotations up to  $\pm 30^\circ$  in all directions.

In order to verify their design capacity, a dynamic full-scale test program has been performed on one unit at FIP Industriale Test Laboratory; furthermore some tests were repeated on all the units for quality control purposes. The test protocol included the following tests: proof pressure test; performance verification test; sinusoidal energy dissipation test and RPV test. The latter three tests are described in the following.

#### *2.4.1 Performance Verification Test*

The aim of the test was both to verify the damper constitutive law and the reaction stability. The test has been performed on every unit imposing 20 continuous sinusoidal cycles of 290 mm amplitude at frequencies producing peak velocities equal to 100, 200, 300 and 400 mm/s. In figure 2a the hysteresis loop graph is reported for the 400 mm/s test. The performance curve for all the units fits the theoretical curve with a maximum deviation of  $-7.11\%$ . The tests results allow to predict that the performance at velocities exceeding 400 mm/s (up to 1000mm/s) will follow the theoretical constitutive law within the required precision of  $\pm 15\%$ . No failure or visible sign of leakage has been determined on all the units during the tests.

#### *2.4.2 Sinusoidal Energy Dissipation Test*

The test was aimed at verifying the capacity of a typical unit to withstand the temperature raise induced by the maximum expected windstorm. This test has been performed sinusoidally cycling one unit at 290mm amplitude displacement at 5 minutes steps producing a 13kW power input. At each step a temperature measurement has been taken on the damper sliding surface in the closest position to the vessel (so to the sealing system). Then three other tests have been performed imposing the required 17kW (10min.), 25kW (5min.) and 45 kW (1min.) to evaluate the temperature increase induced by wind velocity peaks.

As a result, it has been estimated that to a 13kW excitation corresponds an equilibrium temperature of about  $160^\circ\text{C}$ . Should we add to the average power dissipation the effect of the temperature increase induced by wind velocity peaks, the units may reach a maximum operational temperature close to  $200^\circ\text{C}$ , compatible with the temperature resistance of VD components (in particular the sealing system).

#### *2.4.3 Relief Pressure Valve (RPV) Testing*

The damper constitutive law foresees to reach a force equal to 1000kN at 1m/s of velocity. At the internal pressure corresponding to 1000kN design force, a RPV activates, opening a by-pass to the main orificing system thus allowing the most of the flow to pass through the valve itself.

Due to the available testing equipment, a load exceeding 1000kN at a velocity greater than 1m/s was not achievable: so, in order to test the functionality of the valve up to maximum flow rate available, the most of the orifices providing the required Force vs. Velocity constitutive law were plugged. In this way, at a relatively low damper velocity the pressure required for the RPV opening has been reached. Three tests – each one at a different frequency - were carried out imposing three continuous sinusoidal cycles. A typical Force vs Displacement plot is shown in Figure 2. In order to evaluate the damper internal pressure when subjected to the maximum velocity (2.3m/s) and so to the maximum internal flow rate, a power law fitting of the testing results has been performed obtaining a constitutive law of the  $F=Cv^\alpha$  type characterized by a 0.07 velocity exponent.

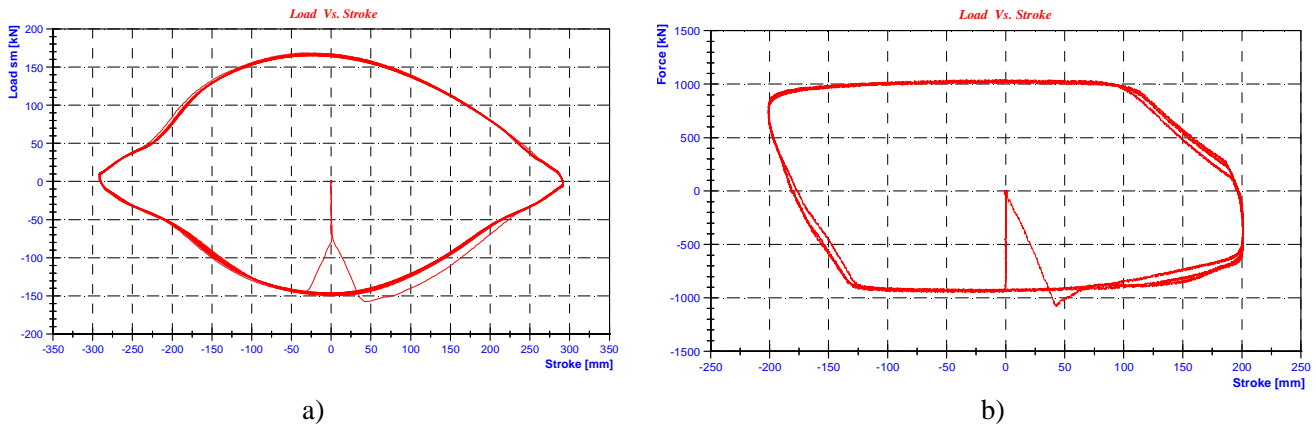


Figure 2 – Results of tests on VD for Taipei 101: a) Typical Hysteresis Loop (peak velocity 400mm/s);  
b) RPV system test (V=154 mm/s)

### 3 SAINT FRANCIS SHANGRI-LA TOWERS - PHILIPPINES

#### 3.1 Structural form

A tall building in an area of high seismicity and high winds needs to be carefully designed to ensure the adequate balance and stiffness and strength is achieved. Conventional practice is to stiffen a building in order to reduce the dynamic response under wind loading. However, this has the effect of increasing the seismic base shear that is attracted. By adding supplementary damping to the structure, it is possible to reduce the flexural stiffness of the building to minimize seismic base shear, and at the same time control the wind response.

St Francis Shangri-La Place is a development of two residential buildings 210m tall and approximately 38m square in plan located in a region of typhoon winds and high seismicity (Figure 3). Each building has a reinforced concrete core and perimeter columns. For each of the two buildings, 8 outrigger walls are attached to the core approximately half way up the building. Two dampers are attached to the end of each of these outrigger walls, ie a total of 16 dampers per building. This is shown indicatively in figure 4.

#### 3.2 Damping System

As a building undergoes dynamic sway motion, there is relative vertical motion between the perimeter columns and the ends of stiff outrigger elements cantilevering from the core. A damper is inserted across this structural discontinuity, dissipating energy during the cyclic motion, and resulting in an increase in the overall damping of the building. This is shown in Figure 3. Further information is given in [Smith, Willford, 2007].

The use of viscous dampers directly incorporated into the superstructure of the building has a number of significant advantages, when compared with a simple pendulum TMD: 1) they take up less space; 2) they are usually placed away from the top of the building (the most valuable space); 3) they can supply higher levels of damping; 4) they are not as sensitive to a potential variation between the predicted and as-built building frequencies; 5) by using a number of different units, there is inherent redundancy; 6) with strategic positioning to ensure adequate damper stroke is achieved, they can be used over a range of amplitudes from low winds to earthquakes.

Modifying the dynamic response of the superstructure through distributed damping needs to be considered early in the design process, especially since the wind-induced dynamic behaviour of the structure is in itself highly dependent upon the structure's dynamic properties. That is to say, since dynamic property modifications cause

changes in the actual wind load distribution itself, a multi-iterative and integrated design process with the structural engineer is needed. The other conventional method of reducing dynamic response is by stiffening the structure. This can require multiple design iterations (increasing design time), can often be expensive, and leads to reduced floor area. The additional damping achieved in each direction for 100 year wind varied between 5.2% to 11.2% of critical for the two buildings and two principal directions.



Arup Damped Outrigger - Patent Pending

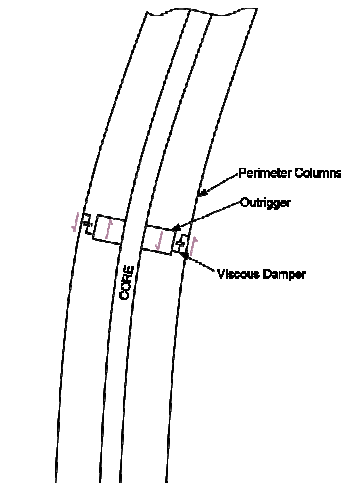


Figure 3 - Architectural rendering of Saint Francis Shangri-La towers and their damping system.

### 3.3 Response of Building with Dampers

#### 3.3.1 Wind Effects

The dampers have two main effects on the wind response. The first is the reduction of the dynamic amplification of the wind loading used for strength design. This is shown in Table 2. The second is to reduce the accelerations felt by the building occupants to acceptable levels. In the case below the target acceleration was 15 milli-g (Table 3).

Table 2 – variation of wind loading with damping for one tower

Method	Assumption	Factored overturning moment (GNm)
Wind tunnel	Code windspeed, intrinsic damping (1.0%)	7.4
Wind tunnel	Code windspeed, damping = 7.5%	4.5

Table 3 – variation of lateral acceleration with damping for 10 year typhoon

Method	Assumption	Peak Lateral Accel. (milli-g)
Wind tunnel	Climate study windspeed, intrinsic damping = 1.0%	25.6
Wind tunnel	Climate study windspeed, Damping = 7.5%	9.4

#### 3.3.2 Seismic Response

In the case of this building, the bending strength of the building was largely governed by the wind loading. A performance based assessment of the seismic effect was made using non-linear response history analysis. This is described in more detail in [Smith, Willford, 2008]. This showed a small decrease in seismic demands on the building, to the point where wind response governed the design.

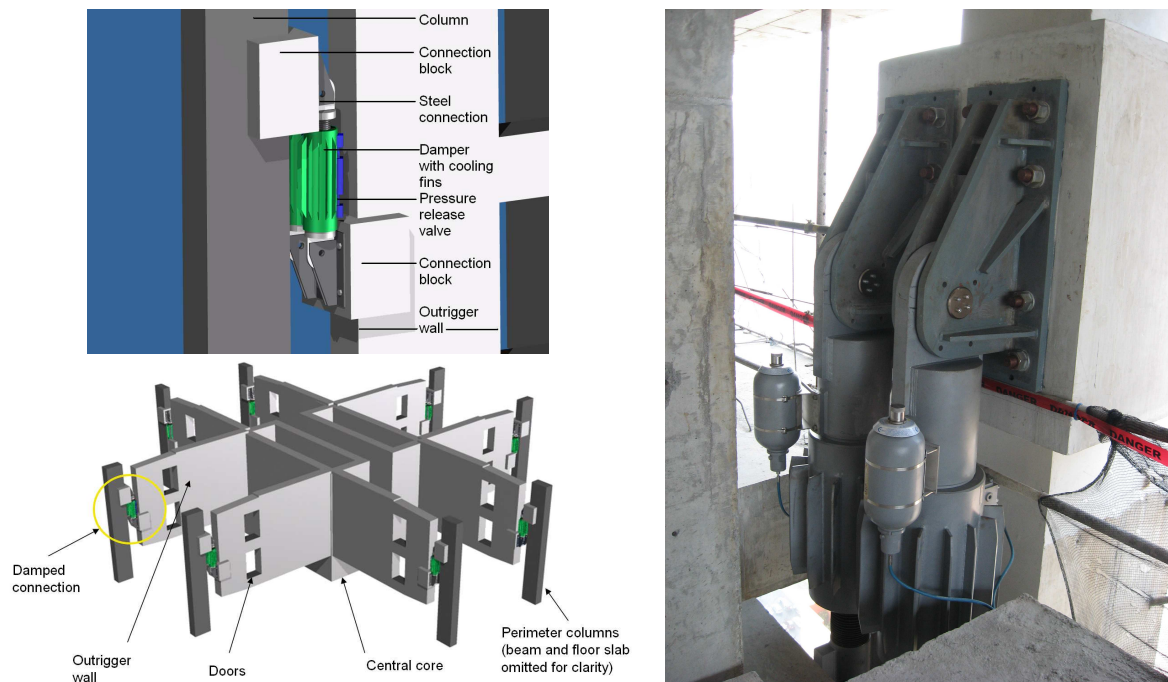


Figure 4 – Dampers conceptual lay-out and as installed into the Saint Francis Shangri-La towers.

### 3.4 Viscous Dampers Test Program

The VD were designed to provide for a constitutive law characterized by velocity squared exponent ( $F=Cv^2$ ) and a 2000kN reaction at a 10mm/s velocity. The units were equipped with a relief pressure valve system to limit the damper reaction when subjected to velocities higher than 10mm/s typically expected during a major seismic event being the maximum seismic design velocity equal to 200mm/s. Maximum stroke is  $\pm 220$ mm.

Even in this case a major design issue was to design the units in order to react steadily during wind storms. For this purpose the units were designed to provide for a 1.2kW maximum continuous power dissipation being capable to allow for higher power dissipation peaks for short periods (e.g.: 100min. at 1.1kW, 5 min. at 2kW, 5 min. at 1.1kW, 5 min. at 2.5kW and the last 65min. again at 1.1kW).

Four units have been subjected to full-scale testing at FIP Industriale Test Laboratory. The test protocol included the following tests: proof pressure test; friction test, power dissipation (wind) test and sinusoidal energy dissipation test.

#### 3.4.1 Friction Test

A 0.1mm/s constant velocity cycle with 10mm amplitude was imposed to evaluate the inherent damper friction. The average measured damper reaction was 4.5kN so about 0.2% of the damper design maximum load capacity.

#### 3.4.2 Power Dissipation (Wind) Test

The dampers were tested imposing sinusoidal movements characterized by 7s period (0.142Hz) according to the required above mentioned sequence aimed at simulating a 180 minutes wind storm. A temperature measurement was taken in the middle of the outer surface of the vessel close to the valve block which resulted to be the most heated surface. The contractual test sequence caused an average temperature increase of about 22°C. After inspection, it has been verified that no visible signs of leakage (or even weepage) or deterioration were detected.

### 3.4.3 Sinusoidal Energy Dissipation (Seismic) Test

The dampers were tested imposing 5 sinusoidal cycles characterized by 7s period (0.142Hz) and 220mm amplitude (197mm/s peak velocity). The maximum reaction was in the range from 2301 and 2541kN, always below the maximum contractual force threshold of 2600kN.

The test caused an average temperature increase of about 17°C recorded immediately after the test (33°C after about 10min) at the same location mentioned on the previous paragraph. After inspection, it has been verified that no visible signs of leakage (or even weepage) or physical deterioration or degradation in performance were observed.

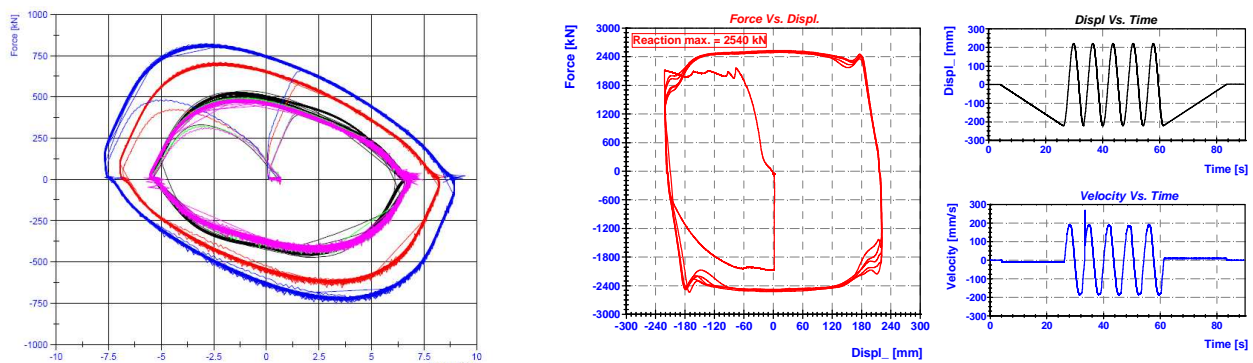


Figure 5 – Typical Wind and Seismic Test Results on a VD of the St. Francis Shangri-La Towers.

## 4 CONCLUSIONS

The two examples described above show that viscous dampers can be effectively used in different configurations to reduce the response of high-rise buildings to wind and earthquake.

The addition of VDs, alongside the use of performance based seismic design, on the St Francis Towers has also enabled a reduced superstructure, making a net saving on the structure of approximately \$4 Millions of USD.

Furthermore, it is interesting to note that the Taipei 101 TMD has already been put into operation by earthquakes in March 2005 and May 2008 (Sichuan Earthquake) and by many strong typhoons (especially throughout an active typhoon season in mid to late 2005). During some of these events, building performance data was obtained via the on-site monitoring system, and the TMD was observed to behave as the design intended. Some occupants of the tower who were present during many of the typhoons have commented that the building motion was barely perceptible with the TMD in operation.

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