Enhancing the Serviceability Performance of Tall Buildings Using Supplemental Damping Systems

Bujar Morava*, Trevor Haskett**, Andrew Smith***

SUMMARY – Over the last two decades, world-wide attention has been directed toward the use of innovative passive and active structural response control systems to mitigate the dynamic effects of wind and seismic loads. Although passive systems provide a limited response control level compared to the active ones, they do not require an external power supply to operate and therefore are still considered as viable and cost effective solutions to control vibrations of buildings or other civil engineering structures. Examples of passive response control systems, also known as supplemental damping systems, include viscoelastic dampers, tuned mass dampers, tuned liquid dampers, viscous dampers and friction dampers.

Supplemental damping systems have been in existence for well over forty years and have been thoroughly researched and tested. Furthermore, their performance has been also validated through full scale monitoring during wind storms or seismic events. A considerable number of supplemental damping system implementations are for wind-induced motion control of skyscrapers. Their implementation has gained much recognition as a workable and highly reliable solution to enhance the serviceability performance of tall buildings and other dynamically sensitive structures.

In this paper, supplemental damping systems that have become increasingly popular will be discussed, with specific attention to their recent notable applications in actual wind-sensitive buildings.

Keywords: supplemental damping systems,tall buildings, serviceability performance, wind-induced motion control, occupant comfort.

1. **Early Applications**

Installation of viscoelastic dampers in the twin towers of World Trade Center in 1969 (Figure 1.a) marks the beginning of the application of innovative technologies to tall building structures to achieve the desired performance in terms of occupant comfort (Mahmoodi, 1969; Mahmoodi et al., 1987; Caldwell, 1986). Approximately ten thousands viscoelastic damper units were installed in each tower, evenly distributed throughout the building from the 7th to the 107th floor (Fanella et al., 2005). As shown in Figure 1.b, damper units were installed between the lower struts of the horizontal trusses and the perimeter columns of the tower. The selection, quantity, shape and location of the dampers was based on the dynamic behavior of the towers under wind loading and on the amount of additional damping required to achieve the performance standards (Mahmoodi et al., 1987). Until their tragic destruction on September 11, 2001, the towers experienced a number of moderate to severe wind storms. The observed performance of both, the dampers and the towers have been reported to be in line with analytical predictions and design objectives (Faschan and Garlock, 2005). Based on the data collected after Hurricane Gloria in 1978, it was found that the total equivalent damping ratio of the towers equipped with viscoelastic dampers was in the range of 2.5% to 3% of critical (Mahmoodi et al., 1987).

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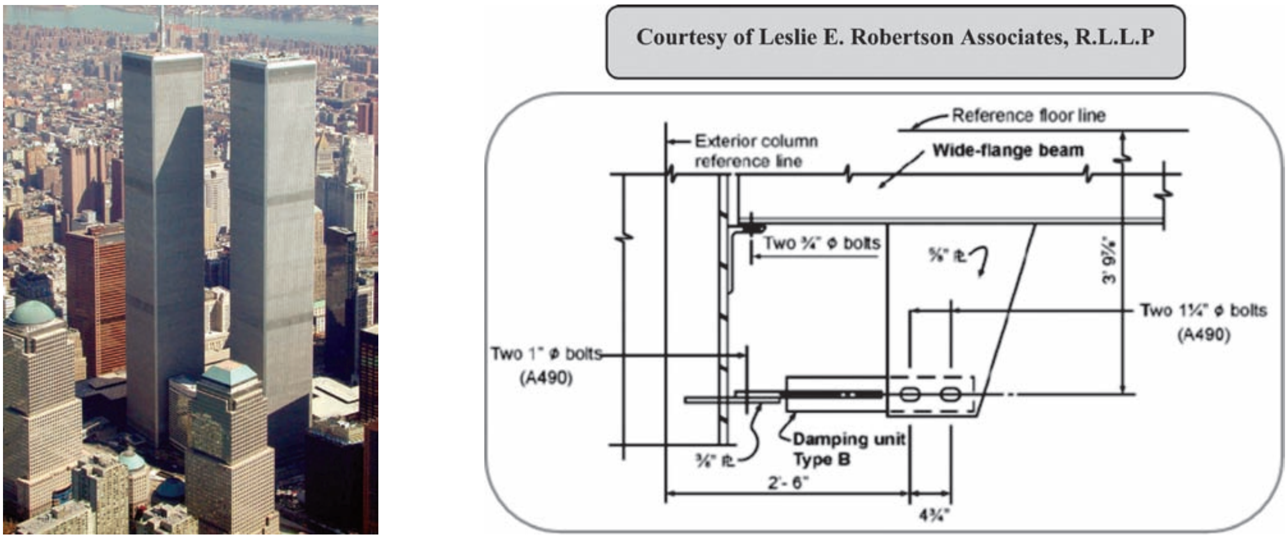
Columbia SeaFirst Building in Seattle, Washington, is also among the first wind-sensitive tall buildings which benefited from the installation of viscoelastic dampers. Two hundred and sixty viscoelastic dampers were installed alongside the main diagonal members in the building core (Keel and Mahmoodi, 1986). The addition of dampers to the structural design scheme of this tower was calculated to increase the damping ratio in the fundamental mode of vibration during frequent wind storms from 0.8% to 6.4% (Mahmoodi and Keel, 1986), hence noticeably improving its performance in terms of occupant comfort.

These early applications of viscoelastic dampers to reduce wind-induced motions in tall buildings were followed by other types of supplemental damping systems such as tuned mass dampers (TMDs), tuned sloshing dampers (TSDs), tuned liquid column dampers (TLCDs), active mass dampers (AMDs), etc. Some recent RWDI/Motioneering implementations of these damping systems in actual tall buildings and other dynamically sensitive structures will be discussed herein.

2. **Tuned Mass Dampers**

Among the passive damping systems, the most popular concept that has been widely implemented in tall buildings and other wind - or seismically - sensitive structures around the world is the TMD. A TMD is a damped secondary inertial system which consists of a mass attached to the building (generally at a location with maximum motion) through a spring and damping mechanism, usually a viscous damping device (Figure 2). Accurate tuning of the frequency of the TMD to that of the building gene-



a) The World Trade Center, New York

b) Viscoelastic damper installation

Fig. 1. *Twin towers of World Trade Center and viscoelastic damper unit installation.*

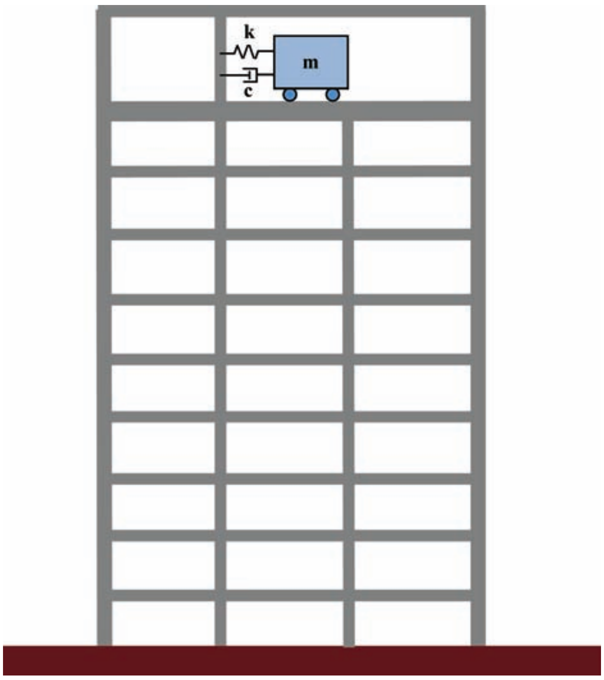


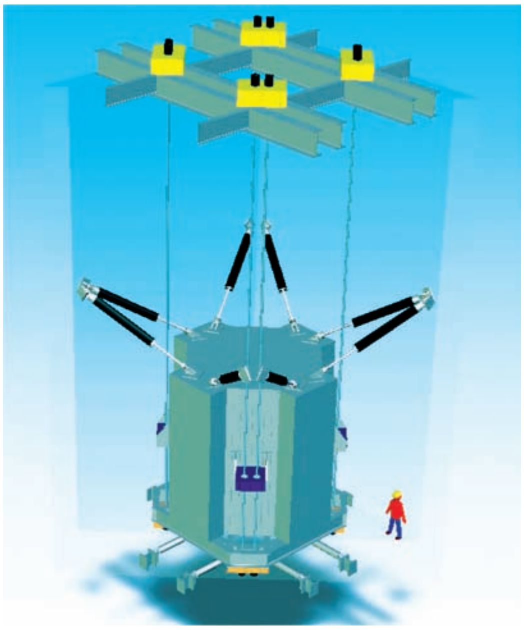
Fig. 2. *Schematic illustration of a building equipped with a TMD.*

rates inertial forces on the TMD mass which counteract the lateral wind or seismic forces acting on the structural frame of the building, resulting in reduced wind- or earthquake-induced motions of the building. The effectiveness of a TMD is determined by its basic design parameters: mass ratio (the ratio of TMD mass to the generalized mass of the building in its target mode of vibration) and TMD mass displacement. Depending on the target performance and the space constraints, a mass ratio in the range of 0.5% to 2.0% is generally specified. Envelope space restrictions do not generally permit the implementation of traditional, simple pendulum type TMDs (Figure 3.a) which often require large vertical space to accommodate the cable length needed to match the tuning frequency. Therefore, installation of alternative, more complex TMD configurations, such as dual-stage pendulums (Figure 3.b), sliding-mass TMDs (Figure 3.c) and articulated mass TMDs (Figure 3.d), have been considered.

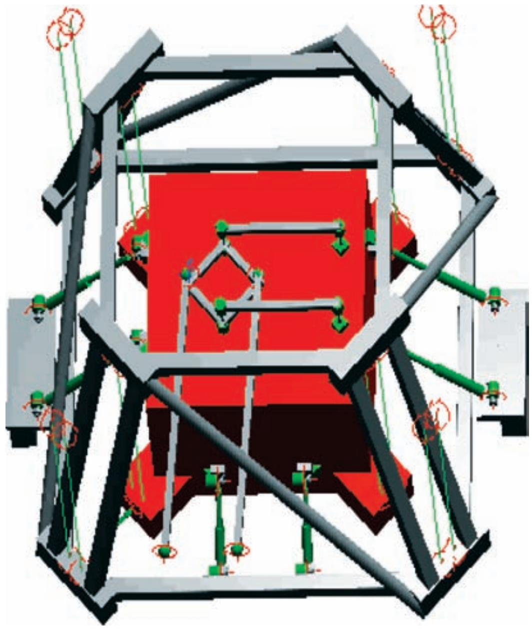
The largest simple-pendulum TMD designed so far to control wind-induced motions of a skyscraper is the 660 tonne ball-shaped tuned mass damper (Figure 4) installed at the top of the Taipei 101 building in Taipei, Taiwan (Haskett, et al., 2003; Joseph, et al., 2006). Eight velocity-squared viscous damping devices (VDDs), designed and manufactured by FIP Industriale S.p.A., Italy, are attached to the TMD mass. The installed length of the VDDs is 3.4 m and their external diameter is 330 mm. The force coefficient of the VDDs is 1000 kN/(m/s)^2 and their estimated maximum continuous power dissipation is 25 kW. This TMD is also the first one designed and constructed as a key architectural and visual element in the building. The architect was able to incorporate this vibration absorber into the architectural scheme of the uppermost occupied floors (Figure 5.a) despite the significant amplitude requirements of the TMD under extreme wind and seismic loading scenarios. From the restaurant and bar, through the center of which the TMD penetrates, patrons will be able to see the golden steel ball slightly swinging, many days of the year, under light winds (Figure 5.b). The TMD is designed to reduce 6-month return period peak accelerations to approximately 5 milli-g (1 milli-g is 1/1000 of gravity acceleration), as required by Taiwanese criteria for occupant comfort (Haskett, et al., 2003).

Since tuning and commissioning of the TMD, several typhoons and earthquakes have hit the Taipei 101 site. The observed performance of both, the damper and the tower have been reported to be in line with analytical predictions and design objectives.

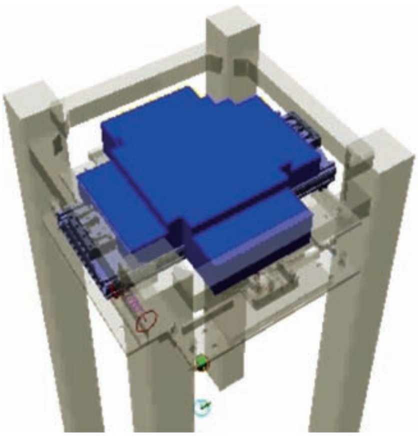
Sample traces of simultaneous TMD and building response as Typhoon Krosa was approaching Taipei (Octo-



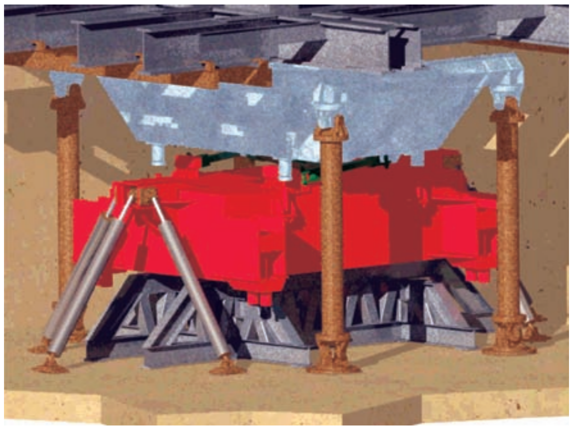
a) Simple pendulum TMD



b) Dual-stage pendulum TMD



c) Sliding-mass TMD



d) Articulated mass TMD

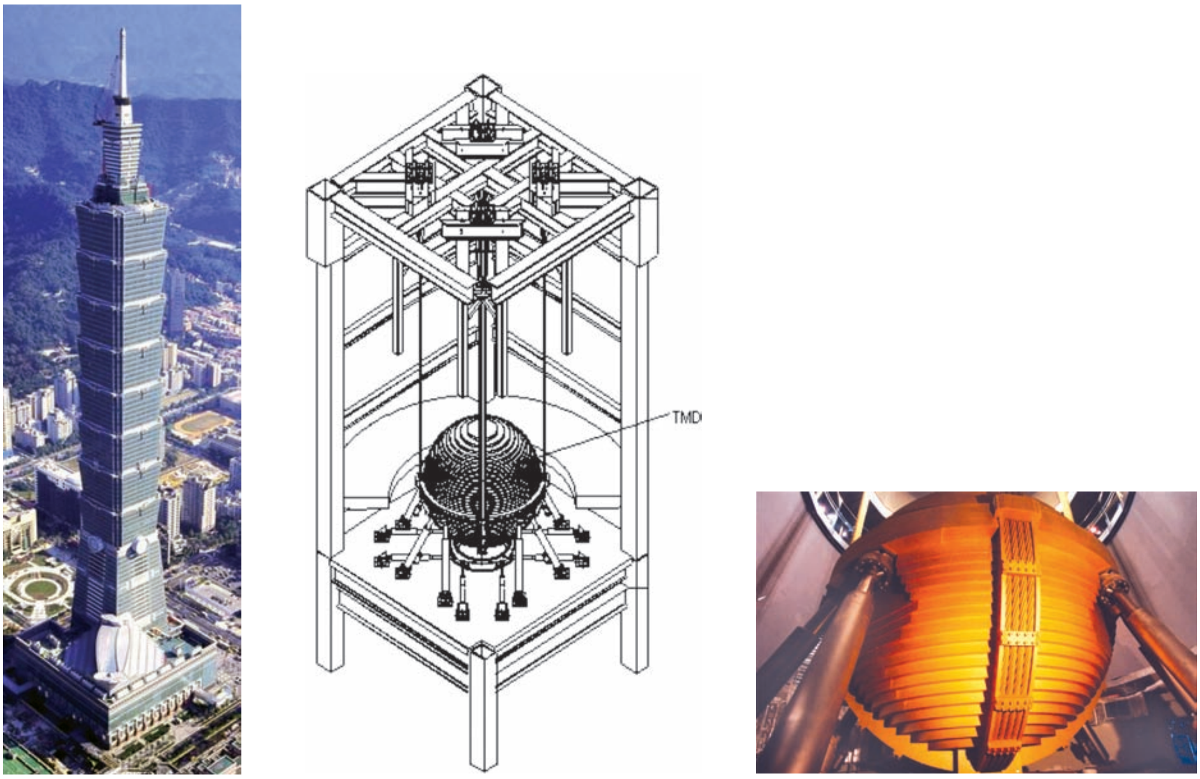
Fig. 3. *Three-dimensional renderings of various types of TMD configurations.*

ber 5-6, 2007) are shown in Figure 6. From this figure it can be seen that building acceleration responses in both directions are in line with the serviceability design target of 5 milli-g.

In some situations, the height of the allocated space for the installation of a supplemental damping system in the building is insufficient to allow a simple type, cable-pendulum TMD to fit. In such a case, to match the tuning frequency of the building a folded pendulum, also known as multi-stage pendulum, can be used. Dual-stage pendulum TMDs have proven to be an effective solution in reducing vertical space requirements while occupying nearly the same horizontal space at the designated floor for supplemental damping system installation. A 550 tonne, dual-stage pendulum TMD similar to the one shown in Figure 3.b was installed in 2001 at the roof level of Trump World Tower in New York City (Figure 7). Detailed wind tunnel studies for this tower predicted 10-year peak acceleration of 27.4 milli-g and therefore implementation of a supplemental damping system was considered as the most effective remedial measure to reduce acceleration responses to acceptable levels for occupant comfort. This TMD is designed to reduce 10-year peak acceleration to approximately 17 milli-g by achieving a total equivalent damping ratio of 5% of critical.

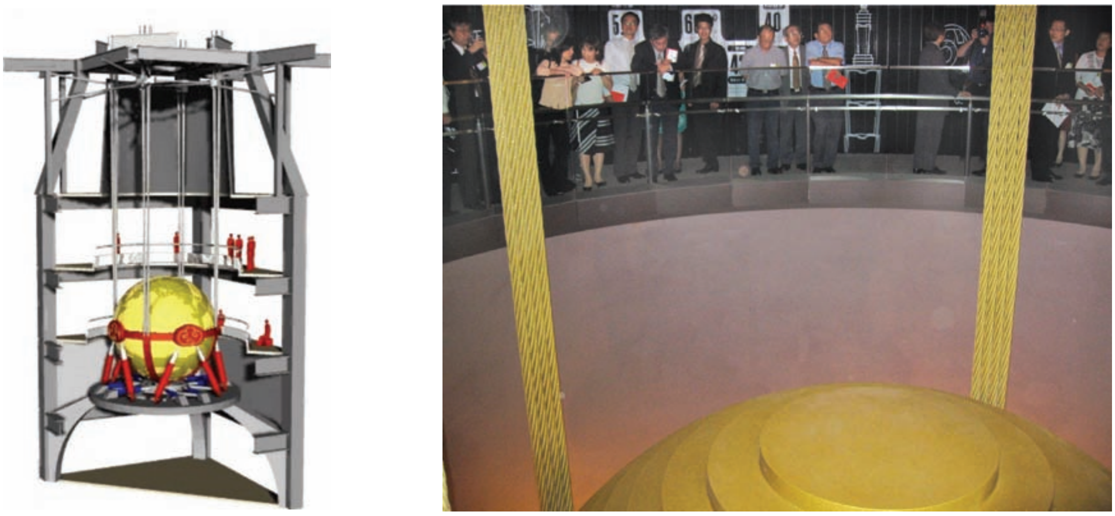
Quite often, the available horizontal and vertical space for a supplemental damping system installation at the top floors is very limited and achieving the desired performance for occupant comfort becomes very challenging. In these situations, considerations have been given to more complex TMD configurations like the one shown in Figure 3.d.

Detailed wind-induced structural response studies conducted for the Bloomberg Tower in New York City (Figure 8) predicted 10-year peak acceleration of 22.5 milli-g. Traditional structural modification measures of adding stiffness and mass were first investigated but the



Aerial view of the building 3-D rendering of the TMD TMD after completion

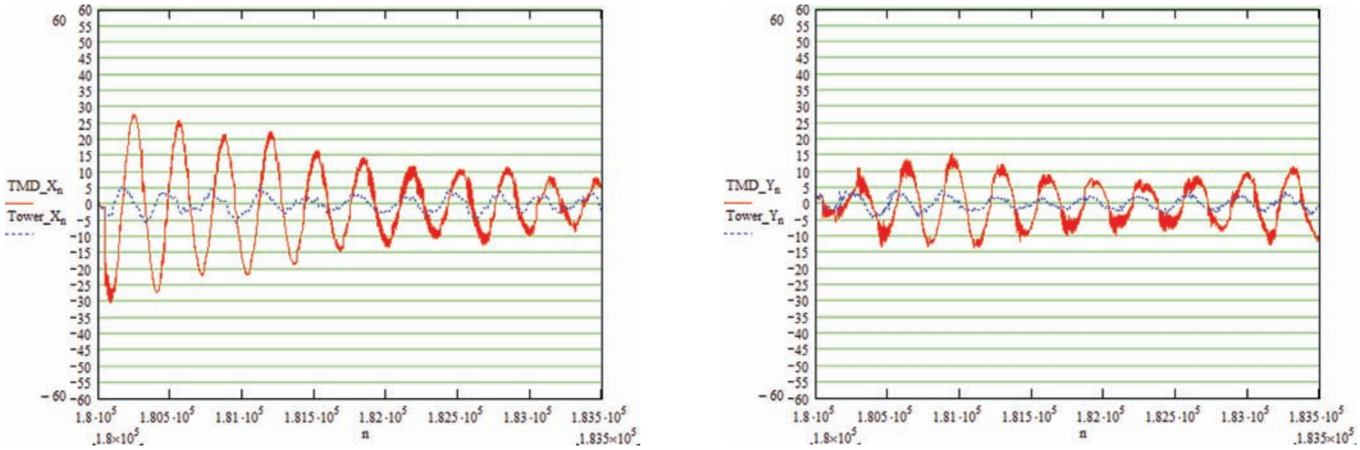
Fig. 4. *The ball-shaped TMD installed in Taipei 101 building.*



a) Architectural design concept b) Implementation

Fig. 5. *Integration of the TMD into architectural design scheme of the uppermost floors.*

results didn't come close to the target performance of 15 milli-g and, moreover, the proposed structural changes were cost-prohibitive. The remaining remedial measure that could be taken was to install a supplementary damping system that could fit in the available limited space of 7.6 m high. If a simple pendulum-type TMD, like that shown in Figure 3.a, were to be employed, a floor to ceiling height of greater than 16 m would be required. In 2004, a uniquely designed articulated mass TMD similar to the one shown in Figure 3.d was installed at the top of the tower (Figure 8). This innovative two-mass system can be tuned to long building periods with limited vertical space for TMD installation. The upper 200 tonne steel block stands on jointed, destabilizing columns. It is linked to the lower 345 tonne steel block suspended from short pendulum cables. As a result, the mass is the sum



X-direction acceleration responses (in milli-g)

Y-direction acceleration responses (in milli-g)

Fig. 6. *Sample traces of TMD and building response during Typhoon Krosa.*

Building view

TMD during construction

TMD after completion

Fig. 7. *The dual-stage TMD installed in Trump World Tower in New York City.*

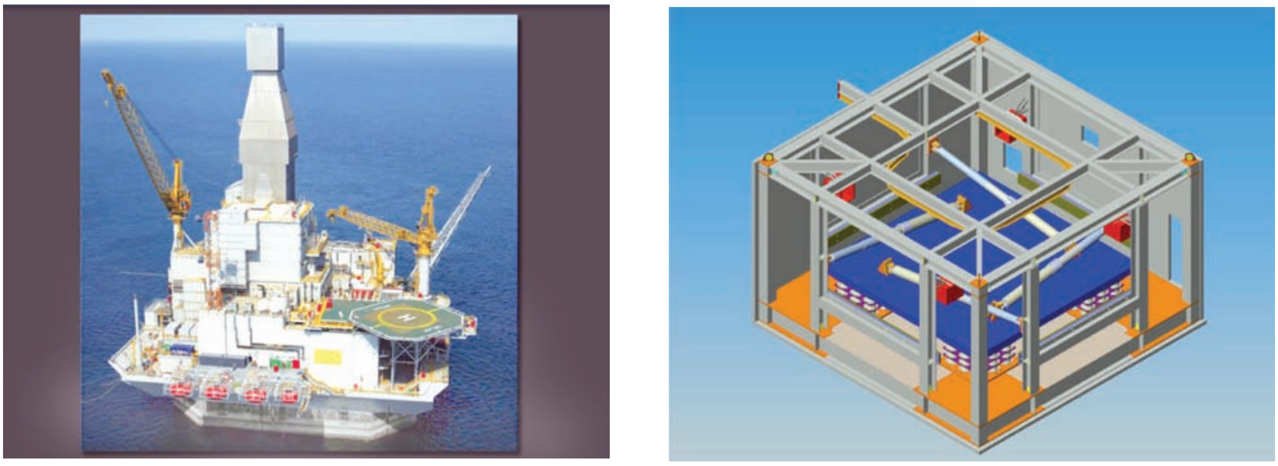
of the two blocks, but the equivalent spring stiffness is established by the difference between the destabilizing effect of the upper mass and the restoring force of the lower mass. The high compound mass coupled with the low spring stiffness results in relatively long periods that can be properly tuned to those of the building. This articulated mass TMD is designed to reduce 10-year peak acceleration to approximately 15 milli-g by generating a total equivalent damping ratio of 4.5% of critical.

Although the primary purpose of the above discussed TMDs is the enhancement of the wind performance of the buildings, the evaluation of their seismic performance was an important aspect of their design. Detailed time domain earthquake response analyses were conducted to assess the performance of the Structure-TMD system and also determine the seismic forces acting at the Structure-TMD interfaces. For sites where recorded earthquake ground motions are not available, analytically simulated design-response-spectra compatible strong ground motions are used. In contrast to using simple, code-based design spectrum approach, this analysis allows for a more realistic evaluation of the seismic forces required to design these interfaces and the stress level in the other components of the TMD.

A recent application of a TMD where the primary purpose of the installation was the enhancement of the *seismic* performance of a major structure is shown in Figure 9.a. This TMD has been successfully installed on the drilling rig of the Orlan offshore oil-and-gas platform in Far-East Russia (PennWellPetroleum Group, 2007). The purpose of this TMD is to minimize the drilling rig late-

Building view TMD during construction TMD after completion

Fig. 8. *The articulated mass TMD system installed in Bloomberg Tower in New York City.*



a) Orlan platform with the TMD enclosure at the top b) 3D rendering of the TMD and its components

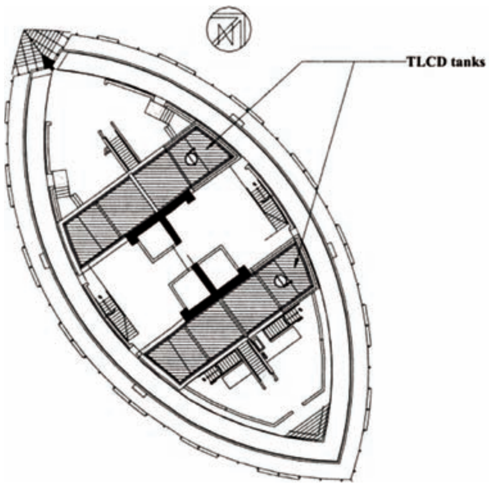
Fig. 9. *The TMD system installed at the top of the Orlan offshore platform.*

ral accelerations during strong seismic events. The TMD, installed at the top of the derrick (Figure 9.a), is a passive system with a 91-tonne mass supported on laminated rubber bearings and connected to viscous damping devices (Figure 9.b) to dissipate the kinetic energy of the rig structure during earthquakes.

3. **Tuned Liquid Dampers**

The idea of applying tuned liquid dampers (TLDs) to reduce vibrations in civil engineering structures began in the 1980s (Bauer, 1984; Modi and Welt, 1987). Since then TLDs, encompassing both TSDs and TLCDs, have become a popular form of supplemental damping system used for mitigation of wind-induced motions. In comparison with tuned mass dampers, the advantages associated with tuned liquid dampers include low initial cost, nearly maintenance free operation and simplicity in frequency tuning. However, there are disadvantages as well which relate to the effectiveness of water mass participation in counteracting the building motion, uncertainties associated with nonlinear behavior effects, lower density of water compared to steel used in TMDs, and generally higher

a) Building view



b) Cross-section

Fig. 10. *Wall Centre tower and its cross section.*

floor space requirements dictated by the water tank geometry to achieve the proper tuning.

The earliest implementations of TLDs for suppressing wind-induced motions of tall buildings and other dynamically sensitive structures have taken place primarily in Japan, in the late 1980s and early 1990s. The list of pioneering buildings and structures equipped with TSDs include Nagasaki Airport Tower, Yokohama Marine Tower, Shin-Yokohama Prince Hotel and Haneda Airport Control Tower (Tamura et al., 1995).

In recent years larger size, deep TSDs and TLCDs have been implemented in the design of several tall buildings in North America, such as Wall Center in Vancouver (Irwin and Breukelman, 2001), Comcast Centre, Philadelphia, and 10 Barclay Street, New York (Morava et al., 2010).

Wall Centre, completed in 2001, is a 48 storey residential tower in Vancouver, BC (Figure 10.a). Wind tunnel studies for the final design of the tower predicted 10-year peak accelerations of 28 milli-g, significantly higher than the acceptable criteria for a residential occupancy. The effects of shape and structural system changes were examined in conjunction with the architects and the structural engineers. However, in the end, when all things were considered by the owners and the design team, it was decided that the approach of increasing the damping was preferable. In discussions with the owner it transpired that a water damper could serve a dual purpose by also providing a large supply of water high up in the tower for fire suppression. Initially, the implementation of a TSD was investigated, but the TLCD was found preferable due to its greater efficiency. The TLCD design turned out to be a remarkably economical solution in this case, especially considering the saved cost of having to install a high capacity water pump and emergency generator in the base of the building, as initially required by fire officials. Two TLCDs, each consisting of a four-storey high, 190,000 liter water tank, extending nearly the full width of the tower in the north-south direction (Figure 10.b), were placed at the top of the 48-storey building. Also a helpful factor was that the wind-tunnel predicted motions of the tower were predominant in one direction only. Therefore only one direction of motion needed to be damped which simplified the design of this damping system.

Figure 11 shows the TLCD design which consisted of two identical U-shaped concrete tanks. Since the building was reinforced-concrete it was relatively easy to incorporate the tanks into the design and to construct them as a simple addition to the main structure. The dimensions of the TLCDs were initially worked out using analytical methods. However, since a number of assumptions are inherent in these methods a scale model was also constructed and tested to validate the TLCD performance. The model behavior satisfactorily agreed with the theoretical results. This TLCD was designed to reduce 10-year peak acceleration to approximately 16 milli-g by achieving a total equivalent damping ratio of 6% of critical.

A TLCD was also designed and implemented in the Comcast Center building in Philadelphia (Figure 12). This is the world's largest TLCD, with a water mass of approximately 1,180 tonnes, or 1,180,000 liters of water. It was designed to achieve a reduction of wind-induced motions of the tower by 30 to 40%.

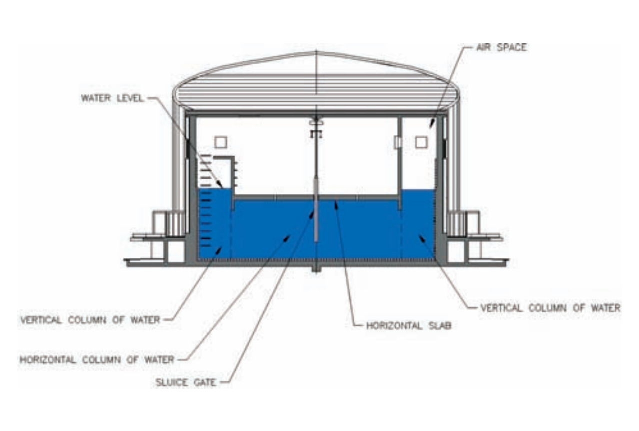


Fig. 11. *TLCD design for Wall Centre tower.*

In 2007, a large size one-directional TSD (Morava et al., 2010) was installed at the roof top level of 10 Barclay Street tower in downtown Manhattan (Figure 13). The height of the tower (H) is 204 m and its typical plan dimensions are D = 19.5 m and B = 45.4 m, resulting thus in a slenderness ratio H/D of 10.5. The North side of the tower sets back at the 8th floor and the floor plate remains the same from the 9th floor up to the 53rd floor. In order to maximize the efficiency of the shear walls, they extend through the full 19.5 m width of the building in the North-South direction, from the 9th floor and above. A combination of reinforced concrete shear walls and flat plates (250 mm thick flat plates up to 34th floor and 305 mm thick flat plates at 35th floor and above) was utilized to sustain lateral wind loads and seismic loads. The building is supported by a caisson foundation which extends down for about 21.3 m to Lower Manhattan bedrock.

A dual system, which combines the advantage of shear walls and frames, was used as a lateral force resisting system of the tower. A finite element model of the tower was used to calculate natural frequencies and mode shapes of the tower. Predicted natural frequencies for the first order sways (Y sway = N-S direction; X sway = E-W direction) and torsional modes of vibration were: fy = 0.191 Hz, fx = 0.206 Hz and ft = 0.235 Hz. These frequencies, along with the corresponding mode shapes, mass distribution along the height of the tower and an assumed damping ratio of 2% of critical were used in the wind tunnel analysis and supplemental damping system detailed design.

Considering the high potential for wind-induced motion issues of the tower due to its significant slenderness ratio and constant floor plate along the height, the design team retained RWDI and Motioneering to address wind and motion control aspects of the project.

Wind tunnel studies performed at the schematic design stage of this slender building predicted peak accelerations at the top occupied floors close to 23 milli-g, considerably higher than the commonly used 10-year criteria of 15 to 18 milli-g for human comfort in a residential tower. The traditional approach of stiffness increase would have produced the desired performance but at the expense of reduced valuable real estate space of the floors and overall cost increase. As an alternative solution, the enhancement of the serviceability performance of the tower through the implementation of a cost-effective TSD was investigated and incorporated in the final detailed design of the tower. This TSD, with a water mass of 170 tonnes, is designed to achieve a total damping ratio of 3.5% of critical. This total damping ratio is required to maintain 10-year peak accelerations within the commonly used comfort criteria of 15 to 18 milli-g.

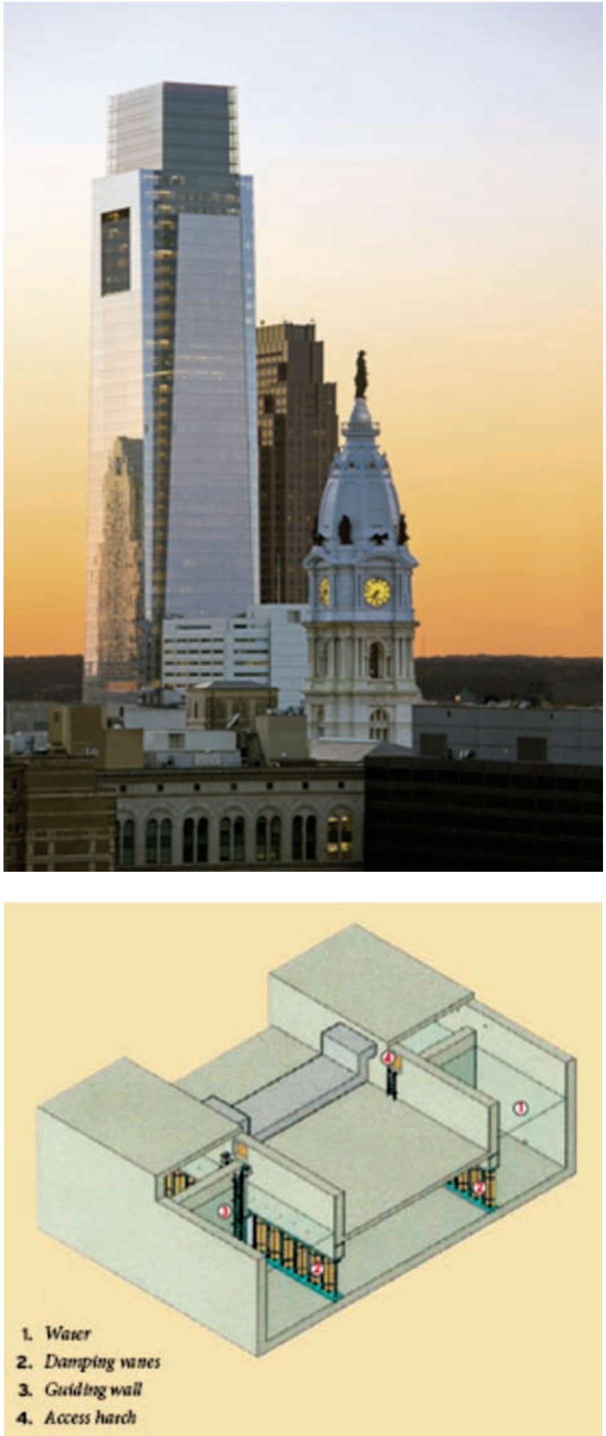
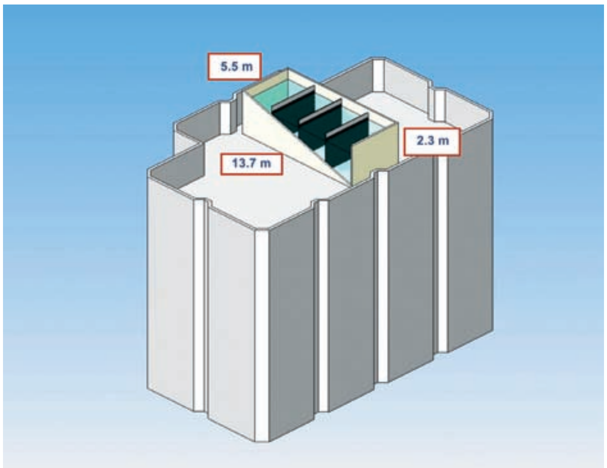


Fig. 12. *Comcast Centre view and a 3-D rendering of the TLCD designed and implemented.*

Building view



b) 3-D rendering of the TSD designed and implemented

Fig. 13. *10 Barclay street tower and a rendering of the TSD installed at the roof top level.*

4. **Concluding Remarks**

In the last two decades, implementation of passive supplemental damping systems has gained considerable recognition around the world as a workable and reliable technology for the mitigation of wind-induced motions in tall buildings. The acceptance of these innovative systems as part of the structural design scheme is based on a well-maintained balance between the serviceability performance targets, construction cost and the complexity of the implementation in an actual building.

Every tall building or dynamically sensitive structure has unique characteristics and constraints that must be carefully investigated in order to determine which type of supplemental damping system can best achieve the target performance in terms of cost, construction schedule, implementation etc. The up-front assessment should consider a wide range of factors that are critical in achieving the desired performance, such as:

- The type of the external dynamic excitation (i.e. vortex shedding, buffeting, seismic, etc.)
- Dynamic behavior of the structure
- Damper – structure interaction
- Available space
- Level of performance required based on the occupancy or usage (i.e. high end condominiums, hotel, office, rental, pedestrian bridge, sky-bridge etc.)
- Construction method
- Construction schedule
- Maintenance requirements

Careful consideration of the above factors at early design stages is key to a successful implementation of a supplemental damping system and to the achievement of the desired occupant comfort level.

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Miglioramento della risposta in servizio di edifici alti attraverso l'uso di sistemi smorzanti

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1. **Introduzione**

Strutture flessibili possono riscontrare livelli eccessivi di vibrazioni sotto l'azione del vento e del sisma, con problemi per il comfort degli occupanti e per la manutenzione dell'edificio. Per garantire prestazioni funzionali vi sono varie soluzioni, tra queste sta avendo diffusione l'utilizzo di dispositivi di controllo passivo e attivo.

I sistemi passivi hanno potenzialità di controllo piu ridotte rispetto a quelli attivi, ma non richiedendo una fonte di energia esterna, sono una buona ed economica soluzione ed è per questo che negli ultimi decenni il loro impiego si è diffuso in tutto il mondo. Questi sistemi sono stati ampiamente testati ed è oramai accertata la loro efficienza nella riduzione degli effetti dinamici.

Esempi di sistemi passivi comprendono, tra gli altri, smorzatori viscoelastici, smorzatori viscosi, smorzatori a massa accordata (Tuned Mass dampers), smorzatori a liquido e smorzatori viscosi.

2. **Prime applicazioni**

L'inizio dell'applicazione di tecnologie innovative ad alti edifici risale al 1969 con l'installazione di smorzatori viscoelastici nelle Twin Towers del World Trade Center a New York. Circa 10.000 smorzatori viscoelastici furono installati in ognuna delle torri. Distribuiti uniformemente tra il 7 ed il 107 piano collegavano la parte inferiore degli impalcati e le colonne perimetrali (vedi Fig. 1b). Le caratteristiche dei dispositivi ed il loro posizionamento fu giustificato dal comportamento delle torri in presenza di vento e, fino alla loro tragica distruzione del 11/09/2001, hanno sopportato egregiamente condizioni di vento medio ed elevato. Gli obbiettivi progettuali in termini di comportamento delle torri e dei dispositivi smorzanti è stato documentato da Faschan (Faschan e Garlock, 2005) e da Mahmoodi (Mahmoodi et al., 1987). I dati raccolti durante l'uragano Gloria del 1978 indicano un rapporto di smorzamento equivalente delle torri nell'ordine del 2.5% - 3% del critico.

Il Columbia SeaFirst building di Seattle rappresenta un altro esempio di pionieristiche applicazioni di smorzatori viscoelastici per edifici alti, sensibili alle azioni del vento. Un numero di 260 smorzatori è stato installato a fianco dei diagonali principali del vano centrale. L'idea progettuale era di aumentare il termine di smorzamento associato al principale modo vibrazionale da 0.8% a 6.4% (Mahmoodi and Keel, 1986) migliorando sensibilmente il comfort per gli occupanti.

A queste applicazioni iniziali, con dissipatori viscoelastici, ha fatto seguito l'impiego di altri tipi di sistemi supplementari più complessi ed articolati, come gli smorzatori a massa accordata (Tuned Mass Damper) (TMD), smorzatori a liquido (Tuned Sloshing Damper) (TSD), smorzatori a colonna liquida (Tuned Liquid Column Damper) (TLCD), smorzatori a massa attiva (Active Mass Damper) (AMD) ecc.

3. **Il sistema TMD**

Il sistema TMD è il sistema che ad oggi ha avuto maggiore diffusione. Trattasi di un sistema di smorzamento inerziale (massa, rigidezza, smorzamento) posizionato nel fabbricato, generalmente nella zona di massima oscillazione (Fig. 2). L'accordo tra le frequenze del TMD e dell'edificio genera forze d'inerzia sul sistema smorzante che si oppongono alle azioni indotte dal sisma e dal vento sulla struttura dell'edificio. L'efficacia di un TDM è determinata dai suoi parametri fondamentali: rapporto tra la massa del TMD e la generalizzata massa dell'edificio nel suo modo di vibrazione di riferimento e spostamento della massa del TMD. Un tipico rapporto di massa varia tra lo 0.5% e il 2.0%. Raramente le dimensioni degli edifici permettono l'uso si sistemi a pendolo semplice (Fig. 3.a) che richiedono grandi altezze disponibili per alloggiare la lunghezza dei cavi necessari a raggiungere il desiderato valore di frequenza. Per questo motivo vengono impiegati sistemi più complessi come quelli a pendolo a doppio stadio (Fig. 3.b) a masse scivolanti (Fig. 3.b) o a masse articolate (Fig. 3.d).

Il più grande sistema a pendolo semplice realizzato è quello posto in sommità del Tapei 101 a Tapei Taiwan, dove una massa di 660 ton, contornata da 8 smorzatori viscosi, forniti dall'azienda italiana FIP Industriale (lunghi 3,4 m e del diametro di 330 mm) e stata installata in un vano ai piani piu alti, costituendo anche elemento di interesse architettonico e visivo (Fig. 4 e Fig. 5). il TMD è progettato per ridurre le accelerazioni di picco per un periodo di ritorno di 6 mesi a circa 5 milli-g come richiesto dalla normativa di Taiwan (Haskett et al., 2003). Registrazioni della risposta del TMD e dell'edificio durante il tifone Krosa (Ottobre 5-6, 2007) sono mostrate in Fig. 6 con la conferma che i livelli di accelerazione sono contenuti all'interno del target progettuale di 5 milli-g

Un sistema a pendolo a doppio stadio da 550 ton come quello di Fig. 3.b è stato installato nel 2001 sulla Trump World Tower a New York (Fig. 7). Accurate indagini in galleria del vento hanno stimato accelerazioni di picco, per un periodo di ritorno di 10 anni, nell'ordine di 27.4 milli-g. Il sistema è progettato per ridurre questi livelli massimi a 17 milli-g corrispondenti ad uno smorzamento del 5% rispetto al valore critico.

Studi approfonditi della Bloomberg Tower a New York (Fig. 8) hanno indicato che mitigare gli effetti del vento (accel. di picco di 22.5 milli-g), con interventi tradizionali, presentava costi proibitivi. La soluzione proposta considera l'impiego di un sistema TMD in uno spazio di altezza limitata (7,6 m) con due masse articolate, la superiore di 200 ton, sorretta da colonne di acciaio incernierate, ed a cui è sospesa una ulteriore massa di acciaio del peso di 360 ton. L'effetto risultante è una massa pari alla somma delle due masse ed una rigidezza equivalente a quella data dalla differenza tra l'effetto destabilizzante della massa superiore e la forza di ritorno della massa inferiore. L'alta massa e la bassa rigidezza genera un periodo relativamente lungo, che è stato opportunamente accordato a quello dell'edificio.

Sebbene lo scopo primario dei sistemi di cui sopra, sia il miglioramento del comportamento degli edifici rispetto all'azione del vento, anche la valutazione della risposta all'azione sismica è un aspetto importante della loro progettazione e viene svolta con una analisi nel dominio del tempo, ottenendo oltre al comportamento generale del sistema edificio-TMD, le forze agenti all'interfaccia dei due sistemi.

Una applicazione recente di un sistema TMD, per migliorare il comportamento al sisma di una grande struttura, è quello realizzato sulla piattaforma petrolifera offshore Orlan, nel mare di Okhotsk nella Russia orientale, per minimizzare l'accelerazione laterale sulla perforatrice, durante i numerosi sismi che interessano l'area. Il TMD è installato alla sommità del derrick ed ha una massa di 91 ton posata su appoggi in gomma e collegata a smorzatori viscosi (Fig. 9.b) a dissipare l'energia cinetica della scossa.

4. **Smorzatori a liquido**

Questo tipo di dispositivi, ed anche i TSD, basati sull'impiego di grandi masse d'acqua, sono stati utilizzati a partire dalla fine del 1980 in Giappone, nella Nagasaki Airport tower, nella Yokohama Marine Tower, nel Shin-yokohama prince Hotel e nella Haneda Airport Control Tower (Bauer, 1984, Modi e Welt, 1987, Tamura et al., 1995). Rispetto ai TMD i vantaggi introdotti da queste soluzioni includono il ridotto costo iniziale, la scarsissima manutenzione richiesta e la semplicità di accordo delle frequenze. Svantaggi riguardano invece l'efficacia del sistema, le incertezze associate a fenomeni non lineari, la bassa densità dell'acqua rispetto all'acciaio utilizzato nei TMD e la generale richiesta di grandi volumi per l'installazione.

Grandi sistemi TSDs e TLCDs sono stati recentemente montati in diversi edifici nel nord America, tra questi il Wall Center di Vancouver (Irwin e Breukelman, 2001), il Comcast Centre a Philadelphia ed la Barclay Tower a New York (Morava et al., 2010).

Il Wall Center di Vancouver, completato nel 2001, è un edificio residenziale di 48 piani (Fig. 10). In fase di progettazione gli studi nella galleria del vento ed i vari calcoli analitici portavano a prevedere una accelerazione di picco nel periodo di ritorno di 10 anni, pari a 28 milli-g, ritenuta troppo alta per il confort abitativo; era quindi necessario incrementare lo smorzamento. Nella discussione del team progettuale vennero valutate varie soluzioni, optando per un sistema TLCD, che oltre ad una grande efficienza, permetteva di disporre di una grande quantità d'acqua ai piani alti dell'edificio, necessaria per la sicurezza antincendio, facendo quindi risparmiare il costo di pompe e generatori d'emergenza che si sarebbero dovuti installare alla base dell'edificio. Sono stati realizzati alla sommità dell'edificio (Fig. 10.b e Fig. 11) due serbatoi d'acqua da 190.000 litri alti 4 piani. Il sistema ha ridotto a 16 milli-g il valore dell'accelerazione di picco con il 6% di smorzamento rispetto al critico.

Un grande sistema TLCD, con una massa d'acqua di 1.180.000 litri, è stato progettato ed installato sul Comcast Center a Philadelphia (Fig. 12); un edificio di 58 piani per 297 m di altezza.

Sulla Barclay Street Tower a NYC, nel 2007 è stato utilizzato alla sommità un sistema TSD (Fig. 13) con una massa d'acqua di 170.000 litri. La torre, alta 204 m ha dimensioni in pianta di 19.5 m per 45.4 m che risultano in un rapporto di snellezza di 10.5. Una combinazione di pareti a taglio in c.a. e piastre è stato utilizzato per sostenere le azioni del vento e sismiche. Analisi in galleria del vento hanno fornito una stima dei picchi di accelerazione in sommità prossimi a 23 milli-g rispetto ai valori comunemente accettati di 15-18 milli-g. per periodi di ritorno di 10 anni. Il sistema adottato consente di ridurre i picchi di accelerazione entro i limiti accettabili con uno smorzamento di 3.5% del valore critico.

Concludendo possiamo dire che ogni edificio alto o struttura dinamicamente sensibile, ha caratteristiche uniche che devono essere attentamente analizzate, per determinare quale tipo di sistema supplementare di smorzamento possa essere impiegato per migliorarne le condizioni di utilizzo e durabilità a costi contenuti. Fattori da considerare includono:

Il tipo di eccitazione dinamica
Il comportamento dinamico della struttura
L'interazione smorzatore-struttura
lo spazio disponibile
I livelli di risposta accettabili basati sull'uso e sull'utenza
Metodologie di costruzione
Tempi di costruzione
Requisiti di manutenzione

L'esperienza indica che un buon livello di confort si raggiunge quando l'edificio reagisce alle azioni dinamiche con una accelerazione di picco pari a 15-18 milli-g con periodo di ritorno di 10 anni.