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Multifunctionality of Distributed Sloshing Dampers in Buildings

by

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ABSTRACT

An investigation was made into the feasibility of using Tuned Sloshing Dampers (TSDs) as a means of motion control in American high-rise buildings. TSD behavior, case studies of existing installations, and design methodologies were reviewed. The major obstacle to their wider use – high demand on valuable floor space – was addressed. The preliminary design of a TSD system for a 39 story office building in Boston was proposed to illustrate the necessary characteristics and scale of such a system.

It is shown that the benefits of employing a TSD system – low cost, easy installation, minimal maintenance – can be realized without significant encroachment on valuable rentable office space. This is done by distributing aesthetically pleasing vessels around the habitable floor space at the top of a structure, and concentrating the rest of the vessels on the roof. In this way, no one floor has to be dedicated to housing the TSD vessels.

Thesis Supervisor: Jerome J. Connor
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Personal thanks to:

All the M.Eng'ers, who keep this place interesting
My HPS friends, who gave me many laughs
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The fools, for always being fools, and dragging me out every once in a while
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Mom, Dad, Mer, and PJ for always understanding and loving me
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Jodi, for giving me the most precious gift of all, my first taste of love
Jay, for being the brightest spirit any of us know. I do not forget you are here.

Everyone else with a smile on out there – keep it up!

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1.0 - INTRODUCTION

1.1 -Motivation

One feature common to most of today's so-called "green buildings" is they way they have shifted the paradigm of site-wide water management. Such buildings tend to reject the traditional view that water should be shunted away as efficiently as possible, and only piped in when needed. With a more harmonious and symbiotic relationship to the natural flows of water, developments using constructed wetlands[1], wastewater reuse[2], rainwater collection[3], vegetated swales[4], living machines[5], and roof gardens[6] are able to achieve a wide range of benefits including: savings on water use and discharge fees, improved aesthetic environment, greater microclimate control, higher occupant satisfaction, reduced energy costs, reduced stress on municipal utilities, and reduced environmental impact.

This paper was born out of a desire to apply the principles of sustainable development and green design to the field of structural engineering - specifically to the design of a skyscraper. It began with the idea of extending the functionality of a structural damping device - water-filled Tuned Sloshing Dampers – to include a role in the overall scheme of building and site water management. The original concept involved integrating a rainwater collection scheme, sloshing dampers acting as a primary (perhaps even biological) filtration system, and toilet water supply systems, as shown here:

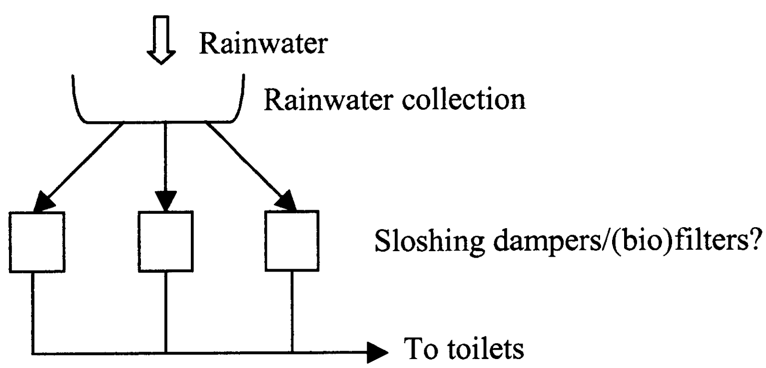

Figure 1.1. Schematic of integrated rainwater collection, sloshing damper, toilet water supply system.

[1] http://www.mcdonoughpartners.com/projects/p_hmiller.html describes nice example of a Herman Miller Factory in Michigan with a constructed wetland on site.

[2] A newly completed high-rise in Oakland makes use of a dual piping system to provide 20,000 gallons per day of recycled water for use in the building's toilets. "Oakland High-Rise To Use Recycled Water", *Civil Engineering,* October 2001, pg. 18.

[3] Refer to the work of Prof. Adhityan Appan, Nanyang Technological University, Singapore. For example, Appan A, Chan HH, and Jih, WH, "Alternative dual-mode working systems for the collection and use of rainwater in high-rise buildings for non-potable purposes" *Proceedings of the 8th International Conference on Rainwater Catchment Systems,* IRCSA/Min of Jihad-e-Sazandegi, Tehran, Iran (1997), pp3-9.

[4] http://www.epa.gov/owm/mtb/vegswale.pdf describes the basics of this alternative for runoff management

[5] See www.livingmachines.com or www.oceanarks.com for details on natural wastewater treatment systems.

[6] William McDonough and Partners have designed green roofs for several high visibility structures such as Chicago City Hall, and Gap Corporate Offices. www.mcdonoughpartners.com

Implementing such a scheme quickly revealed itself to be an enormous and complex scope of work. I decided to focus on a single tangible design example in order to stay as close to reality as possible. Thus I chose to investigate the possibilities of employing Tuned Sloshing Dampers in Boston's newest skyscraper, 111 Huntington Avenue. There are several reasons why I chose this building, including:

- Its dynamic behavior under wind loading seemed especially well suited to the application of Tuned Sloshing Dampers.
- Through the connections of my advisor, Professor Connor, I was able to contact the designer of the building. This proved enormously helpful in obtaining detailed information about the structure.
- Boston's water and sewer system is overburdened and expensive. With the Combined Sewer Overflow problems[7] and high service rates (around $5 per 1000 gal for sewage, and $3 per 1000 gal for water)[8] water usage in this city is becoming both a serious financial and environmental concern. Thus, it is useful and beneficial to at least investigate Green Design possibilities for the city's structures.

I quickly learned that tuned sloshing dampers have never been employed in a building in the United States. Thus it was apparent that the bulk of this project needed to focus on the feasibility of installing even "normal" sloshing dampers into 111 Huntington; the concept of giving them multifunctionality and integrating them into a site-wide water management system cannot be addressed until the feasibility of actually using them is confirmed. As such there is only limited discussion of multifunctional distributed sloshing dampers in the paper. However, it has been useful for me to keep kicking around this idea throughout the development of this thesis, if only to remind me about why I pursued the topic in the first place – an interest in exploring the possibilities of sustainable development.

Sustainability is a vague concept meaning different things to different people. Born out of a natural and well-meaning concern for the viability of a world awash with social, economic, and environmental problems, the burgeoning movement is only just beginning to reach the mainstream consciousness. A sustainable vision of the future undoubtedly is one where economic, environmental and social healths around the world are all strong and secure. Thus, at its core, sustainable development is unabashedly about striving for an elusive utopian vision.

Tangibly, sustainable development must be a local phenomenon. It is about putting to practice, one development at a time, technologies and systems that can simultaneously enhance the economic, environmental, and social well being of a community. Although this is quite challenging, and in many cases, strains the limits of credulity, if planners, designers, and engineers worked with this as their goal, it is certain that we would be moving in a more sustainable direction, slowly, but surely.

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[7] http://web.mit.edu/agfitz/www/11.122/boston.htm

[8] www.bwsc.org

1.2 - Organization

This thesis will investigate the feasibility of implementing distributed Tuned Sloshing Dampers as a motion control system for a high rise building in Boston experiencing excessive acceleration under predicted wind loading. Tuned sloshing dampers are passive damping devices that have been used almost exclusively in Japan. The system consists of tanks of water located throughout a building. These tanks are designed to provide indirect inertial damping, through an out-of-phase hydrodynamic force applied when the structure vibrates and the water sloshes.

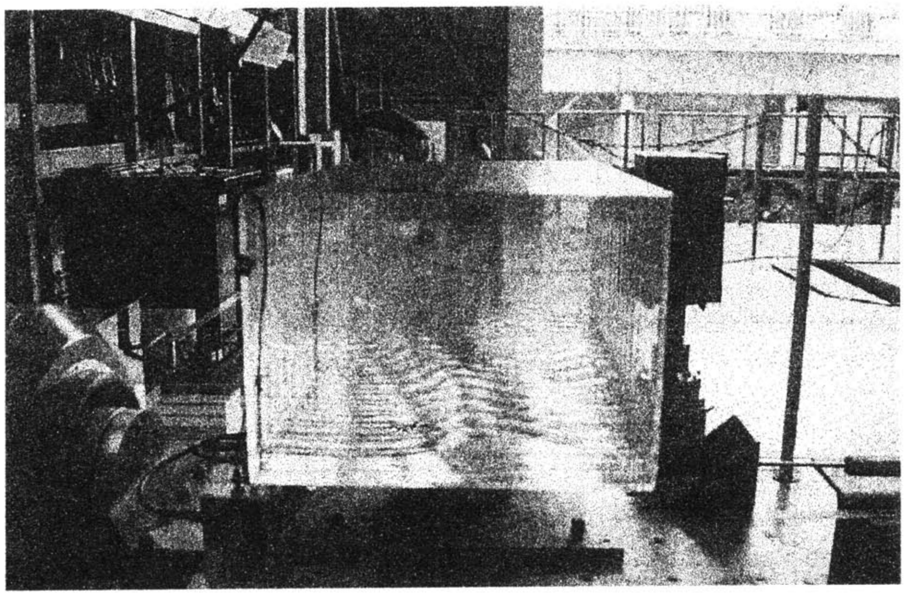

Figure 1.2. Multi-compartment tank of water sloshing with slightly different frequencies.[9]

A brief introduction to the fundamentals of motion based design, wind engineering for high-rise buildings, and structural solutions for excessive motion will be presented in Chapter 2. Chapter 3 will introduce the case study this project is based on, 111 Huntington Avenue in Boston, with a description of the building's actual design and behavior. Chapter 4 will discuss Tuned Sloshing Dampers in detail, through the use of case studies, design methodologies, and experimental observations of behavior. Chapter 5 will discuss the design and behavior of the proposed TSD system for 111 Huntington Avenue, focusing on technical and architectural considerations. This will be compared to a model of the existing building behavior. Finally, the thesis will conclude with a general comparison of Tuned Sloshing Dampers to other specific passive damping devices, and a discussion of the feasibility of their use in American high-rises.

[9] Figure taken from Fujino Y and Sun LM. "Vibration control by Multiple Tuned Liquid Dampers", *Journal of Structural Engineering,* Vol. 119, No. 12, (1993), pg. 3492.

2.0 - MOTION BASED DESIGN

The design of a building must satisfy two requirements, namely safety (strength) and serviceability (stiffness). A structure designed for safety is one whose members are strong enough to avoid collapse, structural failure, major damage and human injury, even during the most severe loadings it will be subjected to in its lifetime. A serviceable structure is one that is stiff enough to prevent the type of motion under service loading that affects the comfort level of its inhabitants. Visual cues that indicate excessive displacement (such as noticing a floor slab that sags in the middle) can sometimes upset the comfort of building users. However, human comfort is generally most affected by acceleration, perception of which depends on exposure time, frequency of vibration, and of course amplitude. Serviceability requirements also deal with minimizing non-structural damage under service loading, because preventing the damage of architectural elements under "normal" story drifts is of the utmost importance.

According to Connor (2001), the "conventional structural design process proportions the structure based on strength requirements, establishes the corresponding stiffness properties, and then checks the various serviceability constraints such as elastic behavior."[10] However, the design of high-rises – especially those made of steel – often follows the opposite process, where members are initially proportioned on the basis of stiffness, and then the corresponding strength properties are checked with the various safety constraints. There are two main reasons why design for motion is so important in high-rises:

(1) As mentioned above, excessive acceleration is the main serviceability concern in any building. Such accelerations are caused by lateral loads such as wind and earthquakes. A building's behavior under lateral load is analogous to a simple cantilever. As such, it is clear that high-rises (longer cantilevers) are naturally more flexible structures than low-rise buildings (shorter cantilevers).[11]

(2) While advances in material science and engineering have caused the strength of steel to double in recent years, its elastic modulus has remained constant. Thus, while structures made of the newer stronger steel can satisfy larger strength requirements, in doing so they are moving more so than before.

Earthquake and wind loadings will impact the same structure in strikingly different ways. Seismic loads on building members are generated when the ground motion causes acceleration of the structure. Recalling that Force = mass x acceleration, it is clear that steel structures, because of their light weight, are generally subjected to smaller seismic forces than their heavier concrete counterparts. The acceleration of the structure as a result of ground acceleration depends on the dynamic properties of the structure itself and the frequency content of the ground movement; when the first few modes of the structure

[10] Connor JJ. *Introduction to Structural Motion Control,* MIT Press, Cambridge MA, (2001), pg. 2.
[11] Connor (2001) states on pg. 4 that, "as the building height increases, the overturning moment and lateral deflection resulting from the lateral loads increase rapidly, requiring additional material over and above that needed for gravity loads alone."

have frequencies that resonate with the ground motion, the ground movement is amplified within the structure, and excessive acceleration can occur. In general, high-rise buildings are not subjected to severe dynamic amplification since their lowest frequencies are often higher than those contained in a seismic excitation.

Unlike seismic loads, wind loads tend to increase as building height increases, due to the strength of the wind at high altitude. With this increased load acting applied on longer cantilevers, it is clear that wind loads affect high-rises much more significantly than low rises. A general design rule of thumb states that the design of steel buildings is earthquake dominant when the building height is under ~300 ft, and wind dominant when the building height is taller than ~300 ft.[12] This is especially true in moderate seismic zones such as Boston.

When the preliminary design of a building is shown to move excessively under predicted wind or earthquake excitation, there are two options to improve the building's behavior. One can either add stiffness to the structure to increase its internal strain energy capacity (thereby reducing overall kinetic energy), or one can add damping to the structure to increase its energy dissipation capacity. Architectural and cost considerations are usually the driving factors behind the structural solution to motion control issues. For example, architectural considerations can sometimes eliminate the possibility of adding bracing to a steel framed building, in which case increasing the rigidity of moment framed connections becomes prohibitively expensive. Indeed it is true that often times employing a damping system is the most cost-effective means of structural control.

[12] Connor, (2001), pg 4.

3.0 – DESIGN OF 111 HUNTINGTON

3.1 – General Information

111 Huntington Avenue is a 39 story office building located in Boston's Back Bay, adjacent to the Prudential Center. This building was completed in late 2001, making it Boston's newest skyscraper, and the city's eighth tallest at a height of 554 ft. With a typical floor area of 22,500 sq. ft., this building will have 850,000 sq. ft. of office space and 63,000 sq. ft. of retail space.

The main structural systems of the building are an exterior steel tube frame, and an inner core. The core and the exterior tube both carry vertical and lateral loads. The inner core provides lateral stiffness on the lowest and highest floors through the use of chevron bracing. Moment frames along the exterior tube provide lateral stiffness for the middle floors.

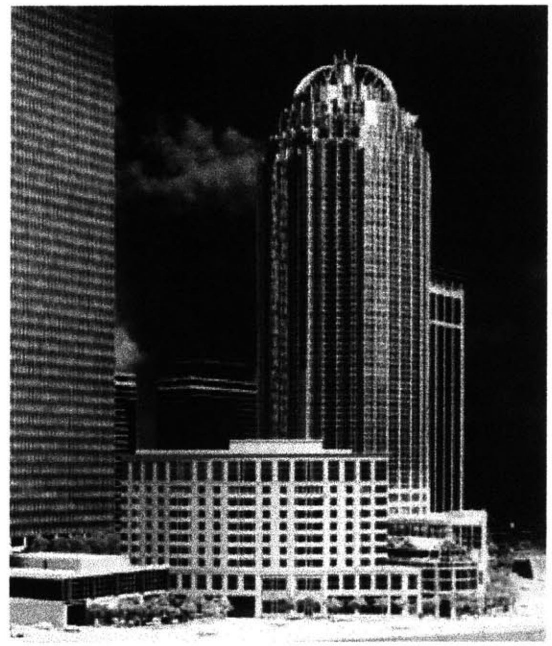

Figure 3.1. Rendering of 111 Huntington, with Prudential Tower at left[13]

High rises in Boston are almost invariably made of steel, due to the skilled and prominent labor force of ironworkers and the established material suppliers in the region. Thus the primary design decision of 111 Huntington – material choice – must have been relatively easy. The use of steel coupled with the design height of 554 ft. make it clear that the building's behavior under wind loading is important to understand.

111 Huntington was initially designed for wind loads by using equivalent static loads specified by code and checking that deflections were within acceptable limits.[14] After this preliminary design, wind tunnel tests were performed on a 1:400 scale model of the proposed building.[15] The test model included all surrounding buildings within a radius of 1600 ft. Here is the designer's description of what the wind tunnel tests revealed:

> "Wind tunnel results indicated that the structure would experience very high acceleration levels developed by winds coming from a northwestern direction. Detailed investigation into the wind tunnel data indicated that the intense buffeting the tower was experiencing was the result of the vortex shedding from the adjacent 52 story existing building [Prudential Center]. The predicted acceleration levels were double the industry standard for office towers."[16]

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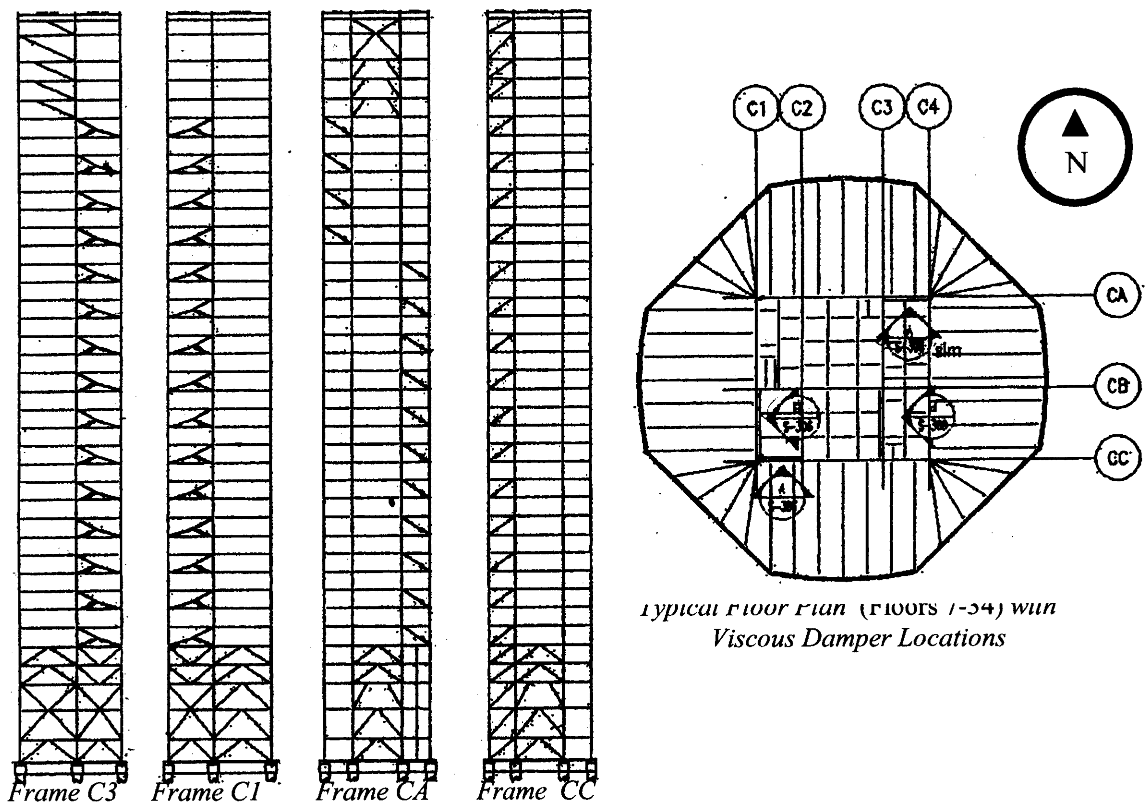
[13] Image taken from http://www.geocities.com/sky_boston/newskyscrapers/111huntington.html.

[14] McNAmara RJ, Huang CD and Wan V. "Viscous-Damper with Motion Amplification Device for High Rise Building Applications", working paper, McNamara/Salvia Inc., Boston, 2001, pg. 1.

[15] Ibid. pg.4.

[16] Ibid. pg.1

When it was discovered that vortex shedding from the adjacent Prudential Center would strike 111 Huntington periodically with a frequency near its natural frequency, the need for a design modification to reduce motion under this resonant condition was made clear.



Figures 3.2 & 3.3. Sections and plan of 111 Huntington showing framing system and damper locations[17]

3.2 - Viscous Damping System

In order to solve the motion problem associated with vortex shedding from the Prudential Center, the designers of 111 Huntington chose to employ an impressive array of 60 viscous dampers. These viscous dampers were placed on the middle level floors whose lateral stiffness comes from moment frames. They were not placed on the chevron braced upper and lower floors (Figures 3.2 & 3.3). In the east west direction, two viscous dampers are placed on every other floor, within 19 ft wide bays in the inner core. In the north-south direction, two toggle-brace-viscous damper systems (discussed below) are deployed on every other floor as well, within 31 ft wide bays.

[17] Figures taken from McNamara *et al.* (2001).

3.2.1 - Toggle-Brace-Damper

Viscous dampers in American high-rises are most commonly employed in diagonal brace and chevron brace configurations.[18] Thus, conventional damper displacements are less than (diagonal case) or equal to (chevron case) the inter-story drift they are subjected to. When story drift is small - as is the case under wind loading - viscous dampers are required to generate a large damping force output under modest displacements. As such, viscous dampers have generally be thought of as more cost-efficient when applied in situations of large story drift (such as seismic applications).

The damping devices employed in 111 Huntington were designed primarily to reduce vibrations caused by wind coming off of the adjacent Prudential Tower. While these winds will cause noticeable and discomforting acceleration of the tower, story drifts due to wind loading will be quite small (± 0.125").[19] Thus, traditional configurations of the dampers were not necessarily cost-efficient. Consider:

u_D = damper relative displacement (along damper axis)
u = story relative displacement (drift)
f = magnification factor
F = horizontal force exerted on frame by damper

$$u_D \equiv f * u \tag{3.1}$$

$$F \equiv f * F_D \tag{3.2}$$

$$F_D = c * \dot{u}_D \tag{3.3}$$

$$F = \frac{u_D}{u} * c * \dot{u}_D \tag{3.4}$$

$$c = \frac{F}{\dot{u}_D} * \left(\frac{u}{u_D}\right) \tag{3.5}$$

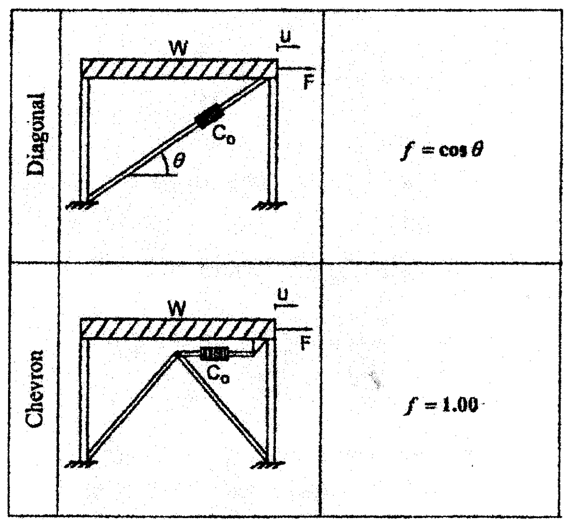


Figure 3.4. Magnification factors for diagonal and chevron oriented dampers[20]

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[18] "The use of these configurations is apparently based on the experience of engineers with bracing systems in steel construction and the fact that experimental research studies utilized only these two configurations for energy dissipation systems." Constantinou MC, Tsopelas P, Hammel W, and Sigaher A. "Toggle-Brace-Damper Seismic Energy Dissipation Systems", *Journal of Structural Engineering,* February 2001, pg. 105.
[19] Ibid. pg. 1.
[20] Figures taken from Constantinou *et al.* (2001).

As can be seen from the above equations, when a certain damping force (F) is required for a given amplitude of story drift, u, small values of u_D (and $\dot{u}_D$) – such as in the case of wind loading - will require the damper coefficient to be large, resulting in a costly and inefficient device. Conversely, when the value u_D (and $\dot{u}_D$) are small, a smaller damping coefficient is sufficient. Therefore, when designing for wind, it is advantageous to devise a scheme where the damper device is subjected to higher amplitudes of motion than the story drift that is moving it.

The results of the wind tunnel tests confirmed that the overall stiffness of the east-west direction of the building was less than that of the north-south direction.[21] With story drifts being smaller in the north-south direction, a more-cost effective use of viscous dampers was employed through what is called a toggle-brace-damper configuration.

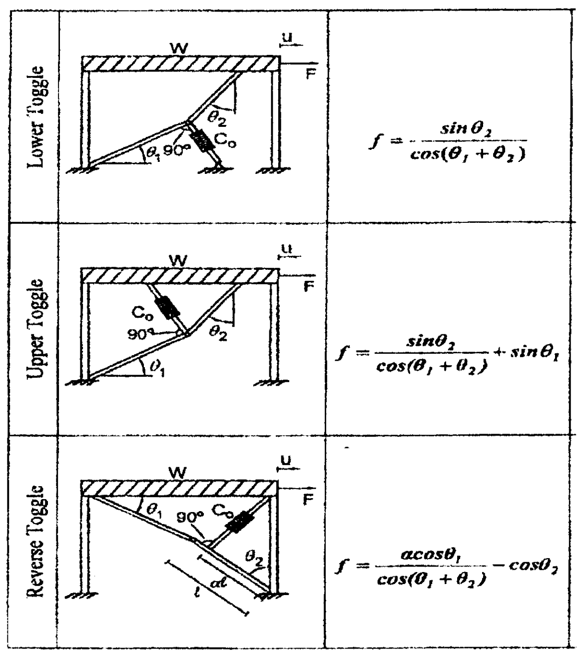


Figure 3.5. Toggle-brace-damper orientations and magnification factors[23]

The toggle-brace-damper configuration creates a damper displacement that is larger than the inter-story drift. Constantinou *et al.* state that while the "amount of magnification depends on the geometry of the toggle braces … practical values are in the range of 2.0 - 5.0."[22] As such, the magnified damper displacement results in a smaller damping coefficient, smaller dampers, and a more cost-efficient motion control scheme. This is readily seen in examining the damping values for the devices employed in 111 Huntington[24]:

North-South Below 25th Floor (Toggle Brace):	20 kip-sec/in
North-South Above 25th Floor (Toggle Brace):	10 kip-sec/in
East-West Below 25th Floor (Diagonal):	300 kip-sec/in
East-West Above 25th Floor (Diagonal):	200 kip-sec/in

[21] McNamaara *et al.* (2001). pg. 3.
[22] Constantinou *et al.* (2001) pg 105.
[23] Figures taken from Constantinou *et al.* (2001).
[24] McNamara *et al.* (2001) pg. 3.

3.2.2 – Effect of Damping System

In computer simulations, the use of 60 viscous dampers greatly improved the dynamic behavior of 111 Huntington. The important design parameter - floor accelerations due to wind loads - were reduced by around 30%.[25] In addition, virtually every other performance criteria for dynamic lateral loads showed improvement: story drifts, story shears, overturning moments, and floor accelerations under both wind and earthquake loading were all reduced by about 20%-35%.[26] It is important to note that while the viscous dampers implemented specifically to improve performance under wind loads, they also lead to the bonus of improved performance under seismic excitation as well. In sum, the addition of the viscous dampers doubled the predicted overall structural damping ratio from about 1.5% to about 3%.[27] The cost of the system was close to $1 million, although the viscous dampers were provided at a discounted price.[28]

Two important factors to consider in the design of a viscous damper system are pointed out by McNamara *et al.*:

[1] "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."[29]

[2] "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 [sic] response reductions predicted analytically can be achieved in the actual installation."[30]

Thus it is important to remember that installing a viscous damper system carries with it the additional cost and design work to strengthen surrounding members. Additionally it is important to keep in mind that analytical models simulating viscous dampers can overstate their actual effect, since flexibility in connections can cause the damper to act "less out-of-phase" with the movement and acceleration of the structure.

[25] McNamara *et al.* (2001) pg. 8.
[26] Ibid. pg. 8.
[27] Ibid. pg. 10.
[28] Personal communication with Robert McNamara, April 30, 2002.
[29] McNamara *et al* (2001), pg. 10.
[30] Ibid. pg. 10.

4.0 – AN ALTERNATIVE: TUNED SLOSHING DAMPERS

A specific structural motion problem has been described above – that of excessive vibrations of 111 Huntington as a result of vortex shedding off of an adjacent building. In the following chapter, background information on Tuned Sloshing Dampers is presented. Reviewing such information is necessary in order to assess the feasibility of using TSDs as an effective alternative to the viscous dampers employed by the designers of 111 Huntington.

At this point it is important to highlight the fact that this paper is in no way a critique of the effectiveness of the current design of 111 Huntington. The designers are a reputable, well-known firm with a history of good works in the field of motion control. This thesis uses the tangible case study of 111 Huntington as a vehicle to explore the issues surrounding a TSD application in an American high-rise building. The intention is to assess the feasibility, and raise awareness, of this option as a motion control possibility in America, not to portray it as the best solution to the design challenges of 111 Huntington.

4.1 – Structural Damping Device Choice

The viscous dampers employed by the designers of 111 Huntington are passive damping devices. Passive damping systems are those that can be installed, left alone, and counted on to use the motion of the structure - not an external power source - to generate vibration-reducing forces and develop damping. Other examples of passive damping systems include base isolation, visco-elastic dampers, hysteretic dampers, tuned mass dampers and sloshing dampers. Passive damping systems are generally preferable to active systems (externally applied force actuators) and semi-active systems (devices with sophisticated controls), because they are simple, relatively inexpensive, and effective. The ability to "power" the structural motion control with energy from the earthquake or wind itself reduces the demand and reliance on external power sources. This is clearly preferable from a quality assurance, environmental, and economic standpoint - so long as the device is effective.

The fundamental design criteria of 111 Huntington is the need to reduce wind vibrations caused by vortex shedding off of the Prudential Center. In the context of this project, the fundamental question became:

> *Is there a way to solve the wind vibration problem - other than installing 60 viscous dampers in the core of the building – in a way that adds to the sustainability of the building (i.e. simultaneously makes it more environmentally, economically, and socially friendly)?*

Inspecting the array of passive damping devices, most of the other forms of damping do not seem to give any distinct advantage over viscous dampers. Base isolation is most effective for seismic motion control, not wind engineering, making it unsuitable. Further, it is not generally appropriate for tall buildings and their large base overturning moments.

Hysteretic dampers, which are often placed diagonally in bay frames like viscous dampers, are more common, easier to procure, and less expensive in Japan. Thus, they provide no distinct advantage over the American-made viscous dampers used in 111 Huntington. According to the designers, visco-elastic dampers "were no longer available from US manufacturers."[31]

In considering tuned mass dampers and sloshing dampers, the designers state:

> "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)."[32]

However, the thought of combining a structural motion solution with a site-wide water management system drove further investigation of these two possibilities.

4.2 – Tuned Mass Dampers

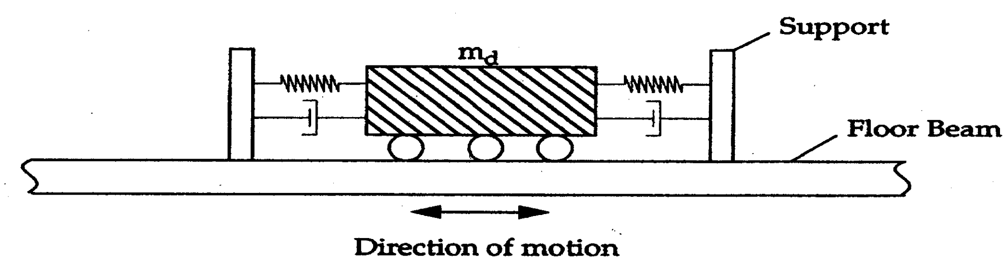


Figure 4.1. Schematic of a Tuned Mass Damper[33]

Tuned mass dampers (TMDs) use the motion of the building to generate inertial forces that are out of phase with the structure, thereby reducing the motion of the overall structure-TMD system. The mass in a TMD is a heavy block of material that rests on a low-friction bearing, while attached to a spring and an energy dissipating viscous damper. The bearing allows the mass to oscillate laterally with a natural frequency:

$$\omega_D = \frac{1}{2\pi}\sqrt{\frac{k_D}{m_D}} \qquad (4.1)$$

where k_D is the stiffness of the spring, and m_D is the mass of the block. The design of a TMD requires tuning this frequency to a particular value to a particular modal frequency of the structure it is deployed in.

[31] McNamara et al. Pg. 1
[32] Ibid. pg.1
[33] Figure taken from Connor (2001), pg. 264.

As shown Figure 4.2, dynamic amplification of a structure is highest when the applied loading resonates with the natural frequency of the structure. A TMD can be employed to eliminate much of the response at this particular frequency; however, it is clearly seen that if any of the design or loading parameters are slightly different than the optimal or assumed values, the TMD can quickly lose its effectiveness. Because of this sensitivity to tuning and loading conditions, TMDs are most effective in reducing the motion of a building whose natural frequency resonates with that of a highly periodic load such as wind. The frequency component of earthquake excitation is a much broader spectrum than wind excitation, though, which renders TMD's unhelpful against the great majority of seismic loading. So while TMD's are effective and behave similarly for small and large amplitudes of motions, they are best suited toward wind engineering.

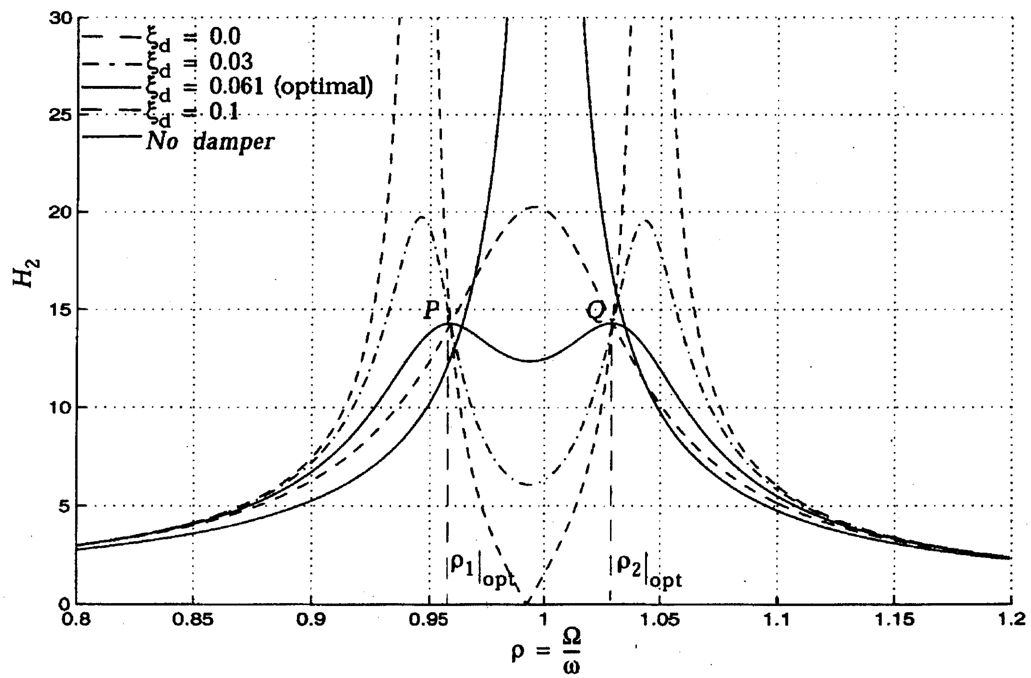


Figure 4.2. Amplification factors (H_2) for periodic and seismic excitation of an SDOF system with TMD[34]

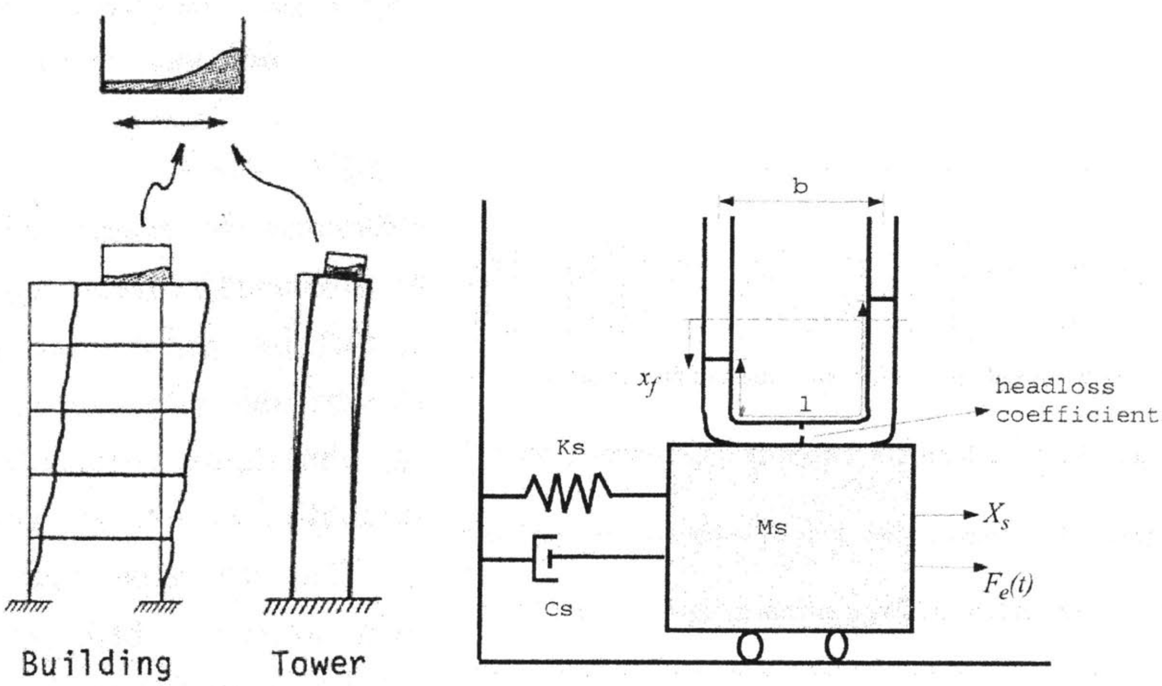
Since 111 Huntington Avenue was designed to be a high-end office building, maximizing office space is of the utmost importance for the economic sustainability of the project. Tuned mass dampers, such as the ones in the Citicorp Building in New York and the John Hancock Tower in Boston, are large (and very expensive) devices. TMD's are located at the point of highest amplitude of motion, which for a high-rise is invariably at the top of the building. Usually weighing about 1% of the building's fundamental modal mass, these systems require an entire floor to house the control and maintenance systems, the mass

[34] Figure taken from Connor (2001), pg. 283.

itself, and the tracks on which it slides. As such, it is no wonder the designers of 111 Huntington passed on the use of a TMD.

It is easy to understand why the designers dismiss the possibility of using tuned sloshing dampers. They have never been used in an American high-rise building and have many similarities to TMD's, devices that are shown above to be unsuitable for use in 111 Huntington. However, as discussed below, sloshing dampers have proven to be effective in reducing the response to wind excitation in Japanese buildings for over a decade. Further, with a little creativity, it might be possible that they can be employed in a way to enhance management of water on site, reduce damping costs, avoid encroachment on "valuable office space at the top of the tower", and even enhance the internal environment in the building itself.

4.3 – Tuned Sloshing Dampers



Figures 4.3 & 4.4. Schematics of Tuned Liquid Damper Applications[35]

[35] Figure 4.3 taken from Fujino Y, Pacheco BM, Chaiseri P, Sun LM. "Parametric Studies on Tuned Liquid Damper (TLD) Using Circular Containers by Free-Oscillation Experiments" *Structural Engineering / Earthquake Engineering,* Vol. 5, No. 2, (1988) pg. 382s Figure 4.4 taken from Yalla S. "Liquid Dampers for Mitigation of Structural Response: Theoretical Development and Experimental Validation" Ph.D. Thesis, University of Notre Dame, (2001), pg. 43.

4.3.1 - General Information

Tuned sloshing dampers (TSDs) are tanks or columns, partially filled with liquid, that are installed onto an existing structural system (Figures 4.3 & 4.4). They impart indirect damping on a structure via out of phase inertial forces, much as a TMD does, while energy is dissipated through internal viscous action of the fluid, friction effects between the fluid and the walls of the container, slamming impacts of the fluid against the wall of the container, contamination of the free surface with floating particles, and wave breaking

As in the case of a TMD, the design of a sloshing damper involves tuning the frequency of wave sloshing to or near the natural frequency of the structure. However, it is important to note that in general the movement of water within a tank is not as "neat" and easy to predict as the lateral oscillation of a mass-spring system. The presence of several different modes of sloshing coupled with non-linear effects such as wave-breaking and slamming impacts can make accurate prediction of sloshing behavior difficult. Indeed it is clear from simply shaking a half-full cup of water that the response modes of a TSD will become progressively more chaotic as the amplitude of excitation increases.

When a TSD is employed in a structure subject to large amplitude excitations such as earthquakes, it is important to consider the effect of the non-linear water sloshing. The presence of several sloshing modes, each with separate frequencies, reduces the inertial forces tuned to dampen structural movement. While wave-breaking and impacts increase the energy dissipation within the vessel, this loss of indirect inertial damping greatly changes the response of the modified structure-damper system. There has been much research concerning the effectiveness of sloshing liquid damping systems for high amplitude excitations; however it is important to realize that such systems act more like chaotic energy dissipaters as opposed to indirect inertial damping systems.[36]

When a TSD is subjected to small amplitude excitations, the non-linearities in its response are largely absent. In such a case, the primary sloshing mode, which can be described analytically, dominates. Thus it is relatively easy to tune a lightly sloshing container of water to the natural frequency of a structure. The amplitudes of motion resulting from wind loading are sufficiently small, such that the fundamental sloshing mode of a TSD is practically the only one that is excited. For this reason, and because the highly periodic nature of wind excitation, TSDs are well suited to reduce structural motion due to wind.

TSDs that employ tanks of water can be broadly classified as either shallow-liquid or deep-liquid systems, depending on the ratio of the liquid depth to the tank length. A ratio

[36] See Modi VJ, Munshi SR. "An Efficient Liquid Sloshing Damper for Vibration Control" *Journal of Fluids and Structures,* Vol. 12, (1998) pp. 1055-1071; Yalla S. (2001); Sun, L M., Fujino Y, Chaiseri P, and Pacheco BM. "Properties of Tuned Liquid Dampers Using a TMD Analogy." *Earthquake Engineering and Structural Dynamics*, Vol. 24, No. 7, (1995) pp. 967-976; Sun LM, Fujino Y, Pacecho BM, and Chaiseri P. "Modelling of Tuned Liquid Damper (TLD)" *Journal of Wind Engineering and Industrial Aerodynamics,* 41-44 (1992) pp. 1883-1894.

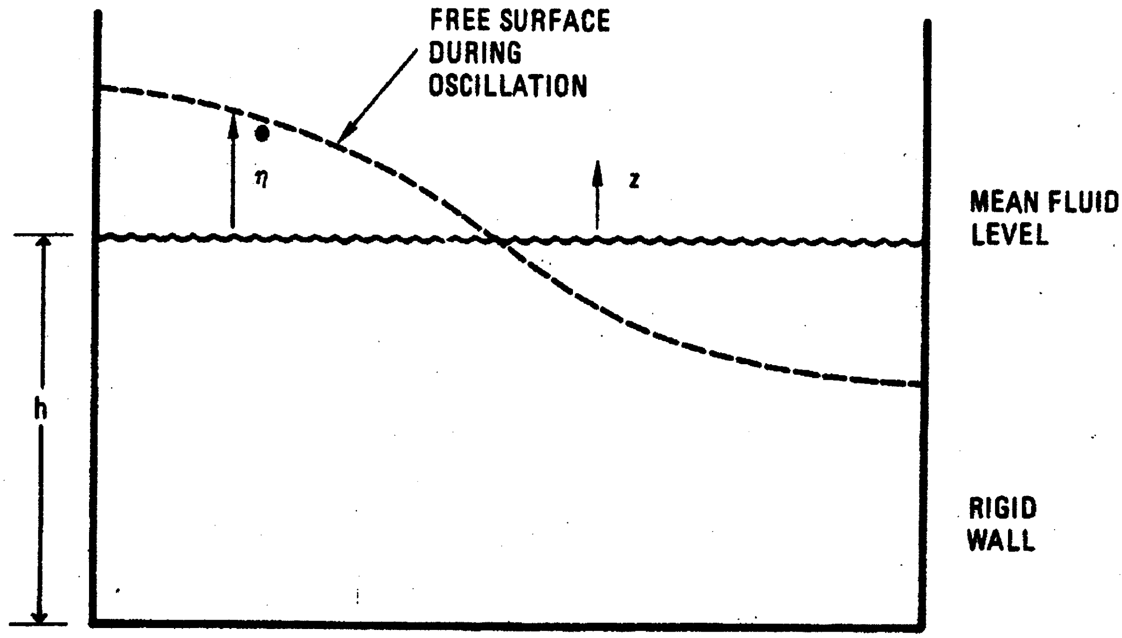


Figure 4.5. Fundamental Sloshing Mode of an Open Tank[37]

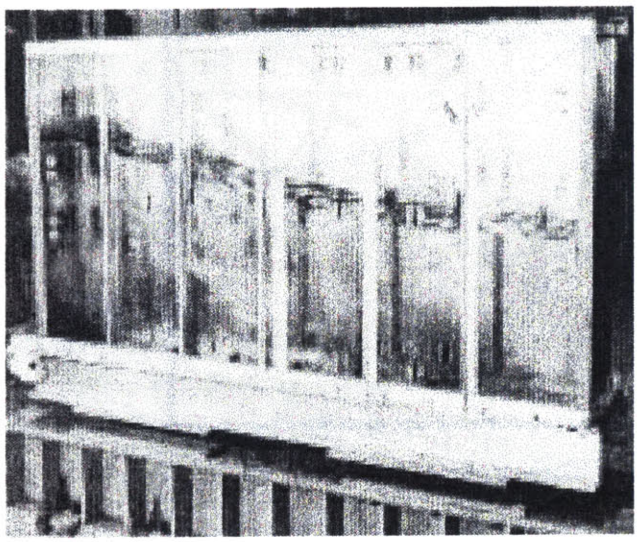
of less than 0.15 is generally regarded as the shallow case. Shallow liquid TSDs provide energy dissipation mainly from viscous action in the fluid, while baffles or screens are commonly used to augment viscous energy dissipation in the deep liquid case.

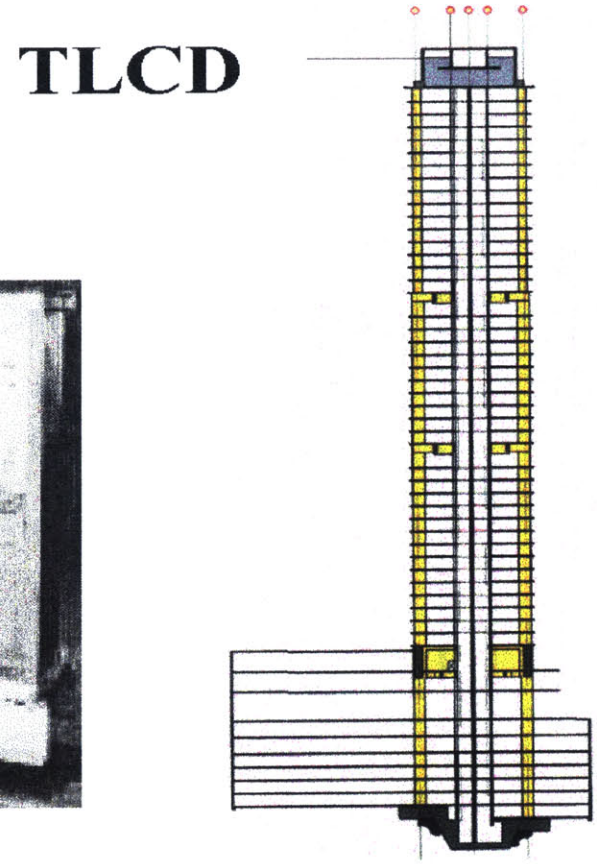
Deep tanks of liquid have several drawbacks to consider before applying them as TSDs. First, a large portion of the liquid below the free surface does not participate in sloshing motion. Not only does this reduce the effective mass providing indirect inertial damping, but it creates a large amount of superfluous dead weight that must be supported. (Of course it is possible to design building supply water tanks as deep-liquid TSDs so the superfluous mass of liquid has a functional purpose.)

TSDs that employ U-tube like tanks of water are generally referred to as Tuned Liquid Column Dampers. As Yalla (2001) states:

> "Tuned liquid column dampers (TLCDs) are a special type of TLD relying on the motion of the column of liquid in a U-tube-like container to counteract the forces acting on the structure, with damping introduced through a valve/orifice in the liquid passage...The damping is amplitude dependent since the valve/orifice constricts the dynamics of the liquid in a non-linear way."[38]

[37] Figure taken from Blevins RD. *Formulas for Natural Frequency and Mode Shapes,* Robert E Krieger Publishing, Malabar FL (1984), pg. 364.
[38] Yalla, (2001), pg. 4.





Figures 4.6 & 4.7. Deep-liquid damper with screen partitions to increase energy dissipation[39] and schematic of Tuned Liquid Column Damper employed in One Wall Centre Project, Vancouver[40]

Much as in the case of the TMD, the amount of water participating in sloshing should be around 1% of the mass of the building in order to achieve adequate inertial damping. Finding a place to put all of this water without overtaking valuable rentable floor space is the primary challenge in designing a TSD system. Deep water systems can be employed in high-rise buildings as water storage tanks or even as swimming pools to give them multifunctionality; however such systems are far from being optimally designed and still take up valuable rental space. Due to the awkward shape of column dampers, they are most likely to be employed on the roof of a building, as shown in Figure 4.7; however such a configuration still takes up a significant and valuable portion of the building height permitted by zoning restrictions. McNamara/Salvia has proposed an innovative installation of TLCDs for a Boston building retrofit project, in which TLCDs would be integrated into (as opposed to simply sitting on top of) the existing roof. It is interesting to note that this proposal was ultimately rejected by developers due to concerns about sloshing water noise.[41] It is important to note that there is much research being performed in the United States on TLCDs.

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[39] Figure taken from Yalla (2001), pg. 11. Originally from MCC Aqua damper literature.

[40] Figure taken from http://www.glotmansimpson.com/projects/onewallcentre/wallcentre.htm

[41] Personal communication. Robert McNamara, McNamara/Salvia Inc., April 30, 2002.

By creating hundreds of shallow liquid layers that all participate entirely in sloshing (at the natural frequency of the building that houses them), the minimal amount of total liquid mass needed to provide sufficient inertial damping can be used. The Japanese have done this in several building applications over the past 15 years, using water as the sloshing liquid. They have employed TSD systems consisting of an array of small, multilayered tanks distributed between and within building floors (Figure 4.8). Such a system provides numerous benefits while avoiding the unnecessary creation of spacious, heavy tanks of water.

The benefits of using a distributed TSD system are:

Low initial cost: In order to create a distributed TSD system, the only material costs are the tanks and the sloshing liquid. The tanks are usually filled with water, due to its low cost, ready availability, and overall effectiveness.

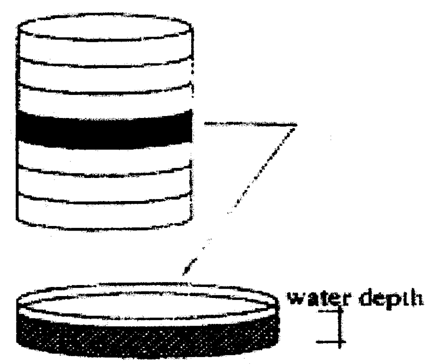


Figure 4.8. Schematic of a multilayered vessel as would be used in a distributed TSD system.[42]

Easy and inexpensive installation: Relatively small empty containers can be transported to their installation point without the use of heavy machinery, and filled with water by hand.

Little to no maintenance: Maintenance of tanks of water is practically unnecessary and, if needed at all, quite simple.

Use in retrofit cases: The ease of installation and small size of distributed TSD tanks render them a quick, easy, minimally invasive, and effective means of retrofitting a building with vibration problems.

Ease of design: Sizing the tanks and amount of water in them to appropriately tune a TSD to the desired frequency requires about a page of calculations. Further, while the TSD is designed for a specific low amplitude, periodic, excitation, there is no limit to the vibration amplitude that the system can handle. In fact, even though the indirect inertial damping of the structure decreases with increased amplitude excitations (once higher sloshing modes appear), the energy dissipation per unit mass of the sloshing water increases.[43] This contrasts with the design of viscous dampers, which can fail catastrophically under excessive amplitudes if not designed for sensible limit states. Designing for these limit states is quite costly and time consuming.

[42] Image taken from Tamura *et. al.* (1995), pg. 611.

[43] Modi and Munshi, (1998), pg. 1064.

Flexibility with regard to installation locations: Because the tanks in a distributed TSD system are of such a manageable size, they are much more easily integrated into buildings in a non-invasive manner than larger tanks or columns. In fact it is quite possible that an architect could design tanks to sit in plain view and add a level of aesthetic quality to the structure. With a distributed system, it may be possible to install an effective TSD without taking up an entire floor or the entire roof. Thinking like this makes the addition of damping devices more of an opportunity to be explored than a burden to be hidden. In addition, smaller tanks will not generate large concentrated forces, meaning that little structural modification will be necessary to design their supports.

Effectiveness in reducing accelerations under wind loading: As described below, existing distributed TSD systems have proven very efficient in increasing the serviceability of buildings and towers under wind excitation, reducing acceleration amplitudes in some cases by up to 60%.

Some level of additional damping under high amplitude earthquake loading: Although TSDs are not efficient damping mechanisms under high amplitude impulsive excitations such as earthquakes, they provide a level of energy dissipation during such an event, due to wave breaking, viscous action, slamming impacts, etc. In that the design of towers and high-rise buildings is generally governed by serviceability requirements under wind, tall steel structures can be efficiently designed for strength under earthquake loading without additional damping requirements, while TSDs added for heightened performance under wind provide a small "bonus" with any additional energy dissipation during an earthquake.

4.3.2 – Case Studies of Distributed Tuned Sloshing Dampers

As mentioned above, the Japanese have pioneered the use of small, distributed sloshing dampers in the use of towers and high-rise buildings. Several papers written in the 1990's detail the design and deployment of 5 of these TSD systems, while providing good insight into their effectiveness at reducing wind vibrations, and the possibilities and obstacles surrounding their wider application.[44][45][46] Four of these cases are analyzed below, first individually, then collectively.

[44] Tamura Y, Fujii K, Ohtsuki T, Wakahara T, and Kohsaka R. "Effectiveness of tuned liquid dampers under wind excitation", *Engineering Structures*. Vol 17, No. 9, (1995), pp. 609-621.
[45] Tamura Y, Kousaka R, and Modi VJ. "Practical Application of Nutation Damper for Suppressing Wind-Induced Vibrations of Airport Towers" *Journal of Wind Engineering and Industrial Aerodynamics,* Vol. 41-44, (1992), pp. 1919-1930.
[46] Wakahara T, Ohyama T, and Fujii K. "Suppression of Wind-Induced Vibration of a Tall Building using Tuned Liquid Damper" *Journal of Wind Engineering and Industrial Aerodynamics,* Vol. 41-44, (1992), pp. 1895-1906.

Table 4.1 – Dynamic Properties of Buildings With Distributed TSD Systems

Building	H_S (m)	M_S (kg)	M_{S1} (kg)	f_S (Hz) x-dir	f_S (Hz) y-dir	f_S (Hz) tors	ξ_S (%) x-dir	ξ_S (%) y-dir	Installation date	Cost (US Dollars)
Nagasaki Airport Tower	42.0	0.17×10^6	0.0633×10^6	1.07	1.07	2.07	0.9	0.9	1987	n/a
Yokohama Marine Tower	101.3	0.54×10^6	0.157×10^6	0.55		1.40	0.6		1987	$73,500
Shin-Yokohama Prince Hotel	149.4	26.40×10^6	10.5×10^6	0.31	0.32	0.56	1.0	1.0	1992	$365,000
Tokyo Haneda Airport Tower	77.6	3.24×10^6	0.648×10^6	0.77	0.98	1.40	0.84	1.24	1993	$275,000
Tokyo Narita Airport Tower	87.3	4.14×10^6	n/a	1.0	n/a	n/a	n/a	n/a	1993	n/a
Tokyo Dome Hotel	155.0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	2000	$615,000

Table taken from Tamura et al. (1995), pg 611, Tamura et al. (1992), and Personal Communication with Y. Tamura 05/13/02..

Abbreviations

H_S height of building
M_S total mass of building above ground
M_{S1} fundamental generalized mass of building
f_S fundamental natural frequency of building
ξ_S structural damping ratio of fundamental mode

Table 4.2 – Sloshing Damper System Characteristics and Vessel Dimensions

Building	$D_D \times H_D$ (m x m)	N (vessels)	n (layers)	h_w (m)	f_D (Hz)	m_w (kg)	m_F (kg)	M_D (kg)	ϕ	μ (%)	μ_1 (%)
Nagasaki Airport Tower	0.38 x 0.50	25	7	0.048	1.02	38.1	0.0	0.95×10^3	0.95	0.56	1.5
Yokohama Marine Tower	0.49 x 0.50	39	10	0.021	0.54	39.6	0.0	1.54×10^3	0.98	0.29	0.98
Shin-Yokohama Prince Hotel	2.00 x 2.01	30	9	0.120	0.31	3390.0	0.0	101.7×10^3	0.97	0.39	0.97
Tokyo Haneda Airport Tower	0.60 x 0.125	1404	1	0.053	0.74	14.9	1.2	22.7×10^3	0.96	0.70	3.5
Tokyo Narita Airport Tower	n/a	~1100	1	n/a	~1.00	n/a	n/a	27.0×10^3	n/a	n/a	n/a
Tokyo Dome Hotel	2.20 x 3.50 2.20 x 4.50	22 4	20 13	n/a	n/a	n/a	n/a	$\sim 136 \times 10^3$	n/a	n/a	n/a

Table taken from Tamura et al. (1995), pg 611, Tamura et al. (1992, and Personal Communication with Y. Tamura 05/13/02..

Abbreviations

D_D	diameter of damper vessel	f_D	fundamental sloshing frequency of contained water
H_D	height of damper vessel	m_w	mass of water contained in one vessel
N	number of vessels per building	m_F	mass of floating particles contained in one vessel
n	number of levels per vessel	M_D	total mass of TLD including floating particles
h_w	water depth in each layer of vessel	ϕ	frequency ratio = f_D / f_S
μ	mass ratio = M_D / M_S	μ_1	mass ratio = M_D / M_{S1}

Nagasaki Airport Tower

25 cylindrical multilayered water-containing vessels were employed as sloshing dampers in the 138 ft tall steel Nagasaki Airport Tower for two weeks in March 1987 (Figure 4.9). The vinyl chloride vessels were 0.5m high, 0.38m in diameter, and divided into 7 layers. With each vessel containing about 38 kg of water, the total water mass of the TSD system was 950 kg (0.56% of the total building mass and 1.5% of the fundamental generalized mass).

Measurements of the building's dynamic properties were taken prior to TSD installation, and the vessels were tuned to slosh at a frequency close to building's natural frequency of 1.07 Hz. After installation of the TSDs, recordings of the tower's damped free and forced oscillations highlighted some important aspects of TSD performance and behavior.

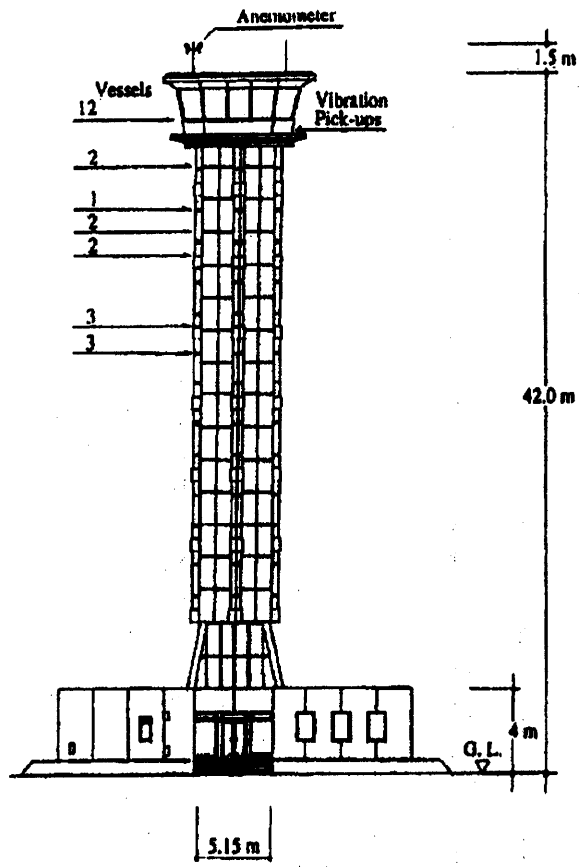


Figure 4.9. Elevation of NAT, with TSD locations.[47]

As can be seen in Figure 4.10, when the number of TSD vessels is increased, there is a sharper initial decay of movement after excitation. However, increasing the amount of water participating in sloshing does not necessarily lead to more efficient damping performance. When large numbers of TSDs were used in the Nagasaki Tower, the presence of beats appeared in the structural response. Beating occurs when the energy dissipation capacity of the water is insufficient, and energy initially transferred to the sloshing water is transferred back to the structure. In order to optimize the damping capabilities of the system, increased energy dissipation within the tanks was required.

Increased energy dissipation can be achieved with the addition of floating particles on the free surface. These particles dissipate energy through friction and collisions. Another way to increase energy dissipation within the tanks is to add protrusions (baffles) that increase friction during sloshing. Because the Nagasaki TSDs were installed for a two-week trial period, neither of these improvements were made in the field. Regardless, with the application of the TSDs, the authors claim a 65% reduction in the RMS displacement

[47] Figure taken from Tamura *et. al* (1995), pg. 610.

of the tower under wind speeds of 20 m/s, as well as increased satisfaction of serviceability requirements.

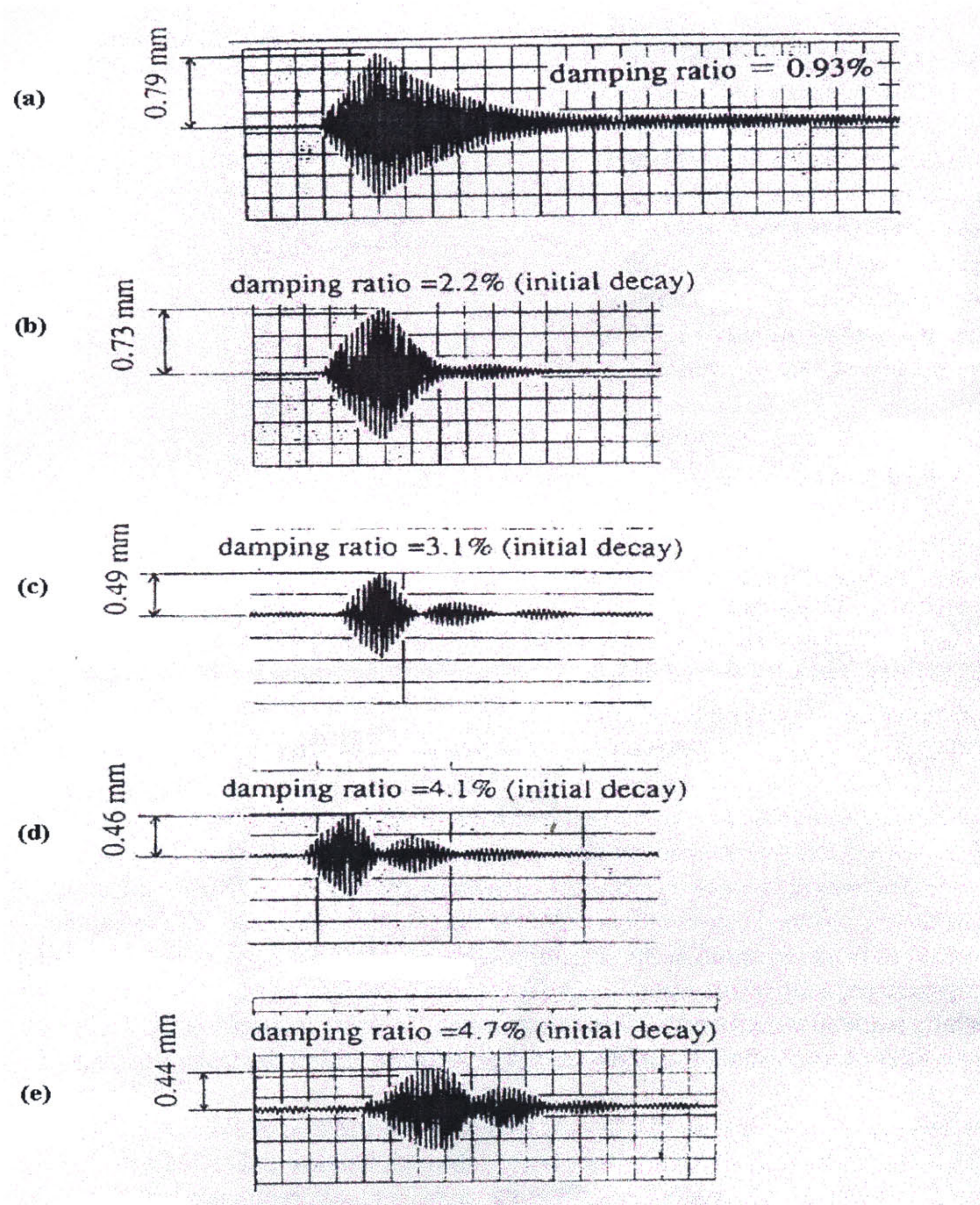


Figure 4.10 – Recording of small damped free oscillations of Nagasaki Airport Tower obtained by "run-down" tests (exciting the building with the movement of a person). Graph (a) shows the case with no TLD vessels. Graph (b) shows the case with 7 TLD vessels. Graph (c) shows the case with 14 TLD vessels. Graph (d) shows the case with 19 TLD vessels. Graph (e) shows the case with 25 TLD vessels.

Yokohama Marine Tower

39 cylindrical multilayered water-containing vessels are employed as sloshing dampers in the 332 ft tall Yokohama Marine Tower, a steel trussed building situated in Yokohama Port (Figure 4.11). The vessels are similar in configuration to those used in the Nagasaki Airport Tower; however, they are made of clear acrylic plastic, have a height of 0.5m, a diameter of 0.49m, and are divided into 10 layers. The TSDs are located in a lighthouse control room at the top of the tower (Figure 4.12).

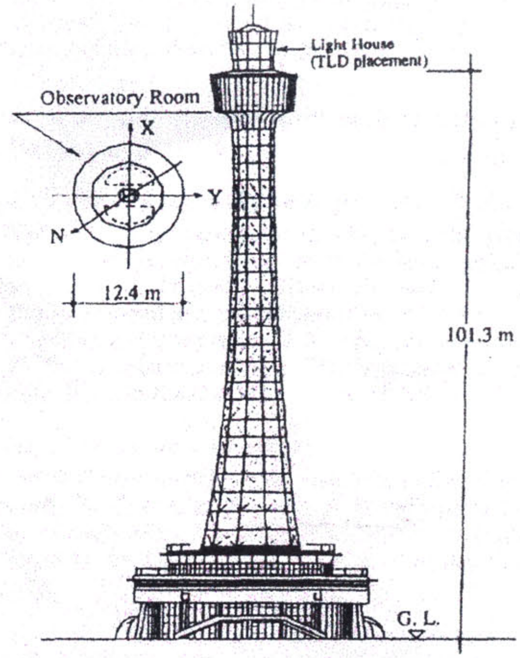


Figure 4.11. Elevation of YMT, with TSD location.[48]

The Yokohama Marine Tower has a symmetric decagonal cross-section, so its dynamic properties are almost identical in either direction, an ideal

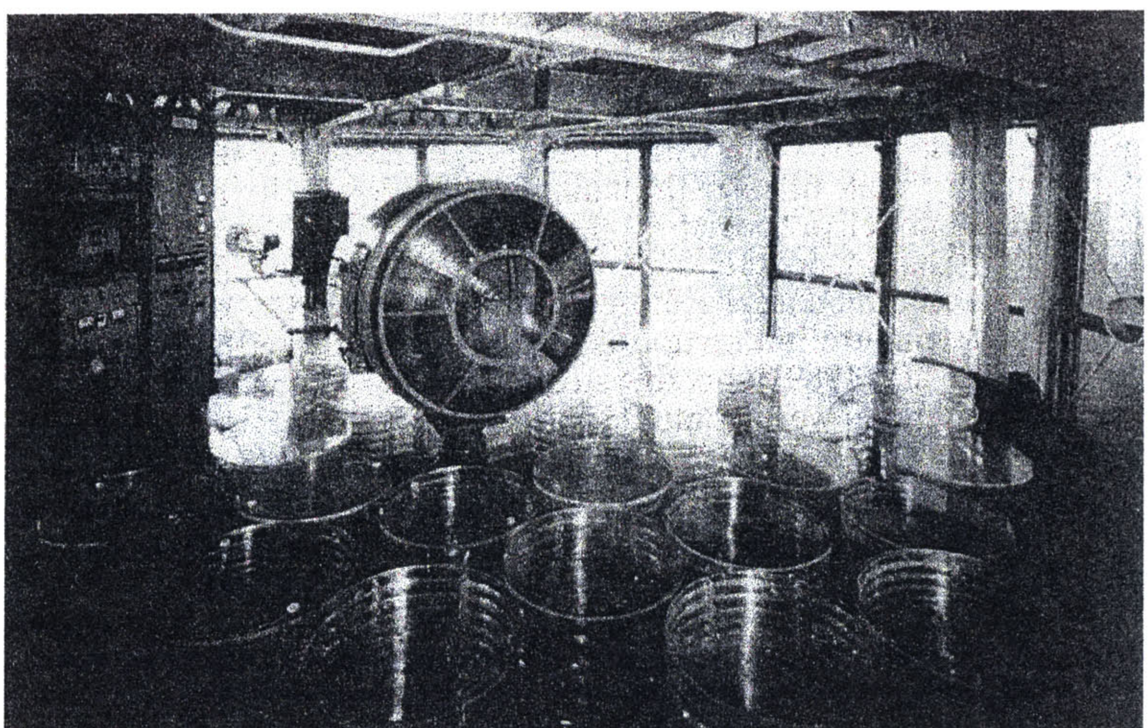


Figure 4.12. Photograph of the TSD installation in the Yokohama Marine Tower.[49]

[48] Tamura *et. al.* (1995), pg. 613.

[49] Figure taken from Fujino Y and Sun LM. (1993), pg. 3483.

case for employing circular sloshing dampers. The dampers were tuned to the building's natural frequency, about 0.55 Hz. As can be seen in Figure 4.13, the presence of the TSDs caused significant reduction (up to 33%) of tower acceleration for a variety of wind speeds. One notices that the TSD effectiveness increases with wind speed (amplitude of excitation) for the range of daily wind speeds tested. This is explained by recognizing that the key factor contributing to damper effectiveness is the amount of water participating in sloshing (and causing inertial damping). For very small excitations, there is little water sloshing and thus little inertial damping. As the amplitude of excitation increases, eventually optimum performance of the damper is achieved, usually in the range of excitations that are caused by wind and felt by humans (between 0.01 and 0.05 m/s^2).[50] It is important to remember, however, that the TSD efficiency starts decreasing again as soon as amplitudes get high enough to cause non-linearities that reduce inertial damping.

To get an idea of the extent to which the dampers increased serviceability, the tower's movement was analyzed for extended time periods. As Tamura *et al.* state:

"According to a 160 min sample with 10 min average windspeeds [of] 15-18 m/s^2, the acceleration of the tower without TLD exceeded the ISO minimum perception range at 0.55 Hz for 36% of the sample length. The acceleration with TLD exceeded [the minimum] only 1%."[51]

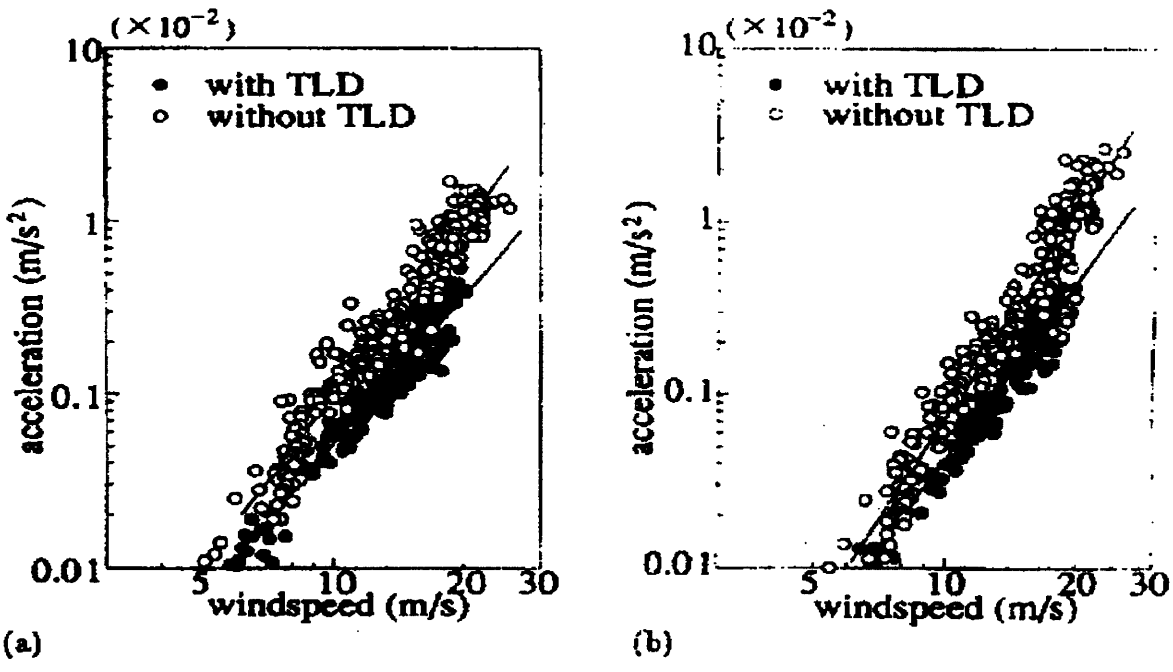


Figure 4.13. Comparison of RMS acceleration responses of Yokohama Marine Tower with and without TSD. Graph (a) shows along-wind response, while Graph (b) shows across-wind response.[52]

[50] This result is replicated in a controlled experiment as described by Sun *et al.* (1995)
[51] Tamura *et al.* (1995), pg 614.
[52] Figure taken from Tamura *et al.* (1995), pg. 614.

Clearly, the liquid vessels have a significant impact on the serviceability of the tower under wind loading. However, as can be seen from Figure 4.12, the issue of finding a suitable location for the TSDs is not trivial. In the case of the Yokohama Marine Tower, the vessels were able to be placed in a little-used control room at the top of the tower, in which they take up a lot of space. It is important to remember that this solution would not generally be available or preferable when considering a TSD application in a commercial high-rise, where floor space is extremely valuable near the top of the tower.

Shin-Yokohama Prince Hotel

The Shin-Yokohama Prince Hotel – an approximately 500 ft tall structure - contains an array of 30 cylindrical fiber reinforced plastic TSD vessels, making it one of the few high-occupancy, high rise buildings containing a TSD system.

One of the interesting features of these vessels is the protrusions seen in Figure 4.15. In addition to reducing swirling motion in the vessel, these protrusions provide increased energy dissipation through friction to reduce the beating effect and increase the efficiency of the structural damping. The effectiveness of these dampers is similar to the previously discussed installations, with a significant reduction in acceleration under wind.

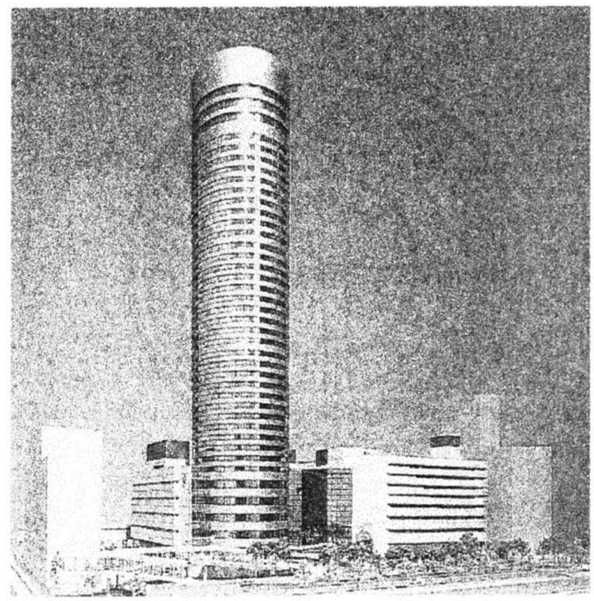


Figure 4.14. Model of the SYPH and surroundings.[53]

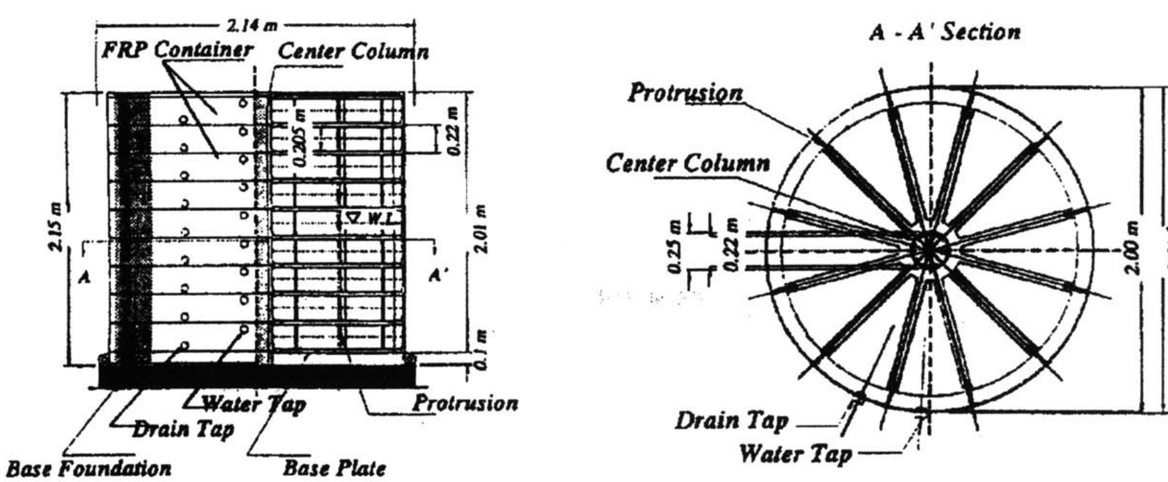
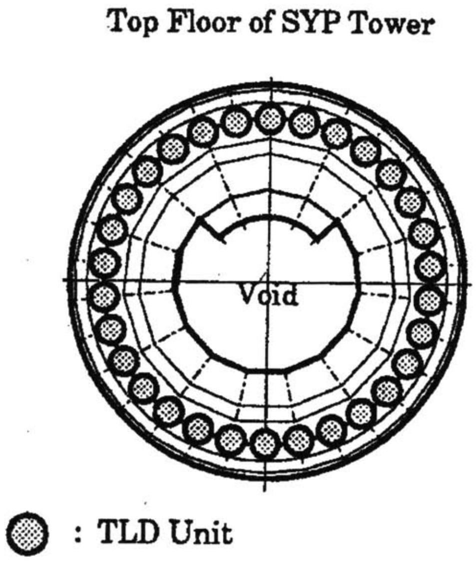
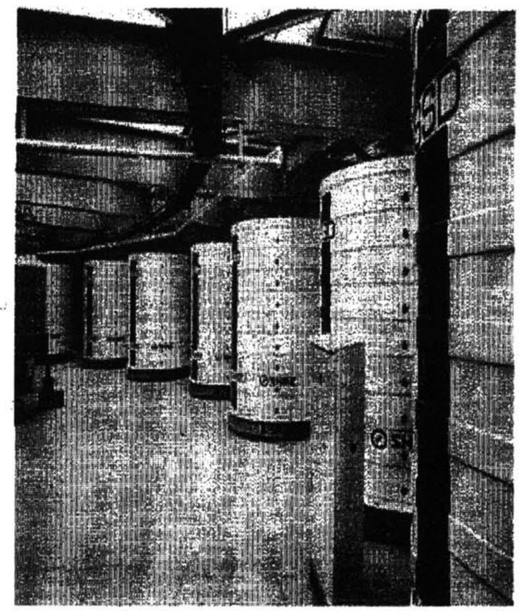


Figure 4.15. Diagram of the TSD vessels employed in SYPH.[54]

[53] Figure taken from Wakahara *et al.* (1992), pg. 1896.

The entire TSD system is installed on the roof floor of the Shin-Yokohama Prince Hotel as shown in Figures 4.16 and 4.17. As can be seen the, 2 meter tall TSD vessels do not take up the entire floor space, but they do effectively eliminate commercial use of the floor. Such an arrangement would not be preferable for an office tower like 111 Huntington; however, it is possible to imagine that a similar number of (perhaps smaller) tanks could be deployed in a habitable floor and actually add to the aesthetic quality of the building, if designed by the right architect!





Figures 4.16 & 4.17. Plan view and photograph of TSD vessel layout in top floor of SYPH.[55]

Tokyo Haneda International Tower

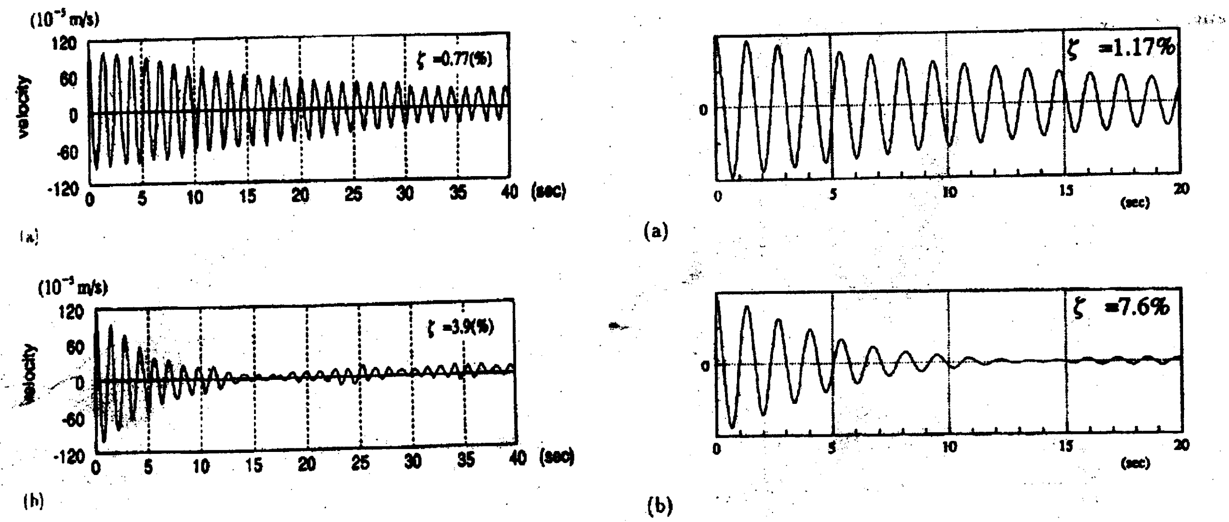
Since 1992, about 1400 small polyethylene TSD vessels have successfully been employed as a wind damping system in the air traffic control tower of Tokyo's Haneda Airport. Much like the other towers, initial performance reports for this TSD system indicated a significantly reduced RMS acceleration of the tower under strong winds (close to 60%). In fact, the designer of the system states:

> "According to the five years inspection of Tokyo International Airport ... there was no [maintenance, noise, other] problem in TLDs and additional 4% damping ratio was confirmed"[56]

One of the interesting features of this system is the presence of hollow cylindrical polyethylene pieces floating on the free surface of the TSD vessels. The addition of these

[54] Figure taken from Tamura *et. al.* (1992), pg. 615.
[55] Figures taken from Tamura *et al.* (1995), pp. 615-616.
[56] Personal Communication with Y. Tamura, May 5, 2002.

particles increases the frictional surface area and leads to collisions, which together result in higher energy dissipation for the vessel. As can be seen in Figures 4.18 & 4.19, the presence of these particles has significantly reduced the beating effect (seen in the case of the Nagasaki Airport Tower) for both human induced and wind induced oscillations. It is also clear that the performance of the dampers improves with increasing amplitude as all more and more water participates in sloshing.



Figures 4.18 & 4.19. The left hand graphs compare the damped free oscillations of the tower under very small oscillations (human run down test) with (a) and without (b) TSD vessels. The right hand graphs compare damped free oscillation components of the tower under wind excitation with (a) and without (b) TSD vessels, as abstracted by the Random Decrement Technique (RDT).[57]

Summary

Several applications of Distributed Tuned Sloshing Dampers were employed in Japanese buildings in the early 1990's. The literature describes their application in three airport towers, one marine tower, and one hotel. Airport towers and marine towers are generally tall, lightweight and flexible structures, whose serviceability is of the utmost importance. In the case of airport towers, the inhabitants are performing sensitive, high-stress work and in the case of marine towers, economic viability results from the ability to provide a pleasurable observation point. TSDs are well suited to provide structural damping for these structures, since the low occupancy density of the structures generally leaves enough space to house the TSD vessels.

The use of sloshing dampers in high-rise buildings is relatively uncommon, because floor space near the top, where TSDs should be placed, is so valuable. Even with the use of distributed sloshing dampers, there remain significant design challenges to effectively

[57] Figures taken from Tamura *et al.* (1995), pg. 618.

integrate enough vessels into a high-rise with minimum encroachment on rentable floor space. A recent application is installed in the Korakuen Dome Hotel in Tokyo.[58] The literature describes a system of 30 TSD vessels located on the top floor of the Shin-Yokohama Prince Hotel. This top floor is dedicated to housing the vessels, which means, importantly, that one floor's worth of revenue has been lost in creating the system. Thus, it is clear that further innovation is necessary to expand the use of TSDs into high-rise buildings. Although they provide a near optimal solution for meeting serviceability requirements under wind, TSDs will not be the optimal solution until effective designs eliminate the perception, and reality that the vessels are taking up too much space.

4.4 – Designing a TSD and Modeling the Structure-Damper Interaction

Numerous researchers have investigated the problem of modeling the behavior of tuned liquid dampers.[59] Because of the non-linearities associated with wave-breaking, the presence of higher harmonic sloshing modes, slamming impacts on container walls and collisions between suspended particles, it is difficult to provide a theoretical model that correctly predicts the behavior of a sloshing damper under high amplitudes of excitation. As stated by Sun *et al.* (1995):

> "Shaking-table experiments, numerical simulations, or analytical solutions for simpler cases may partially provide details of non-linearities, …[however] a simple mechanical model for TLD's is useful in understanding fundamental properties of the TLDs and in their preliminary design." [60]

The simple mechanical model used to both design and understand the basic behavior of TLDs is that of the well-understood Tuned Mass Damper.[61] As can be seen in Figures 4.21 and 4.23, controlled Structure-Damper interaction experiments have confirmed that the TMD analogy accurately predicts the behavior of a TSD for small amplitudes of motion (such as those caused by wind) when non-linearities are negligible. Thus, by applying the TMD design procedure, an effective TSD design can be readily created.

The design procedure for a TMD or TSD involves first defining the following system characteristics:

$\mu = (m_l / m_g)$ = ratio of damper mass to fundamental modal mass
$\phi = (f_D / f_S)$ = ratio of damper frequency to structural natural frequency
ξ = damping ratio of damper itself

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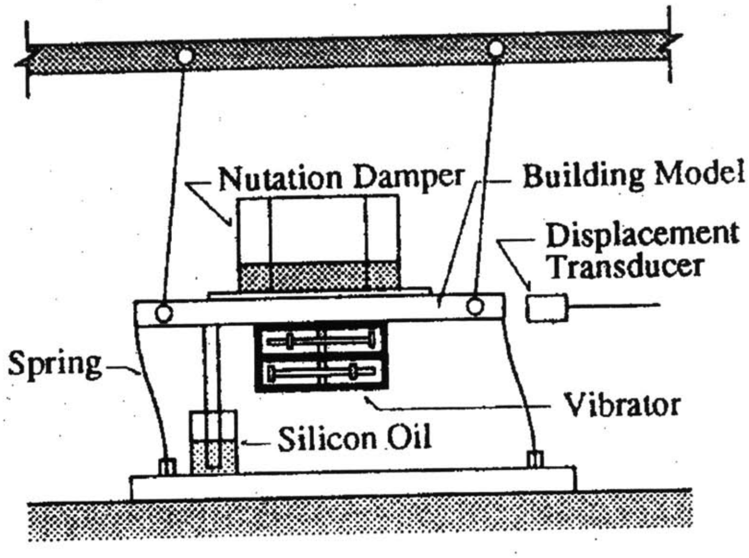
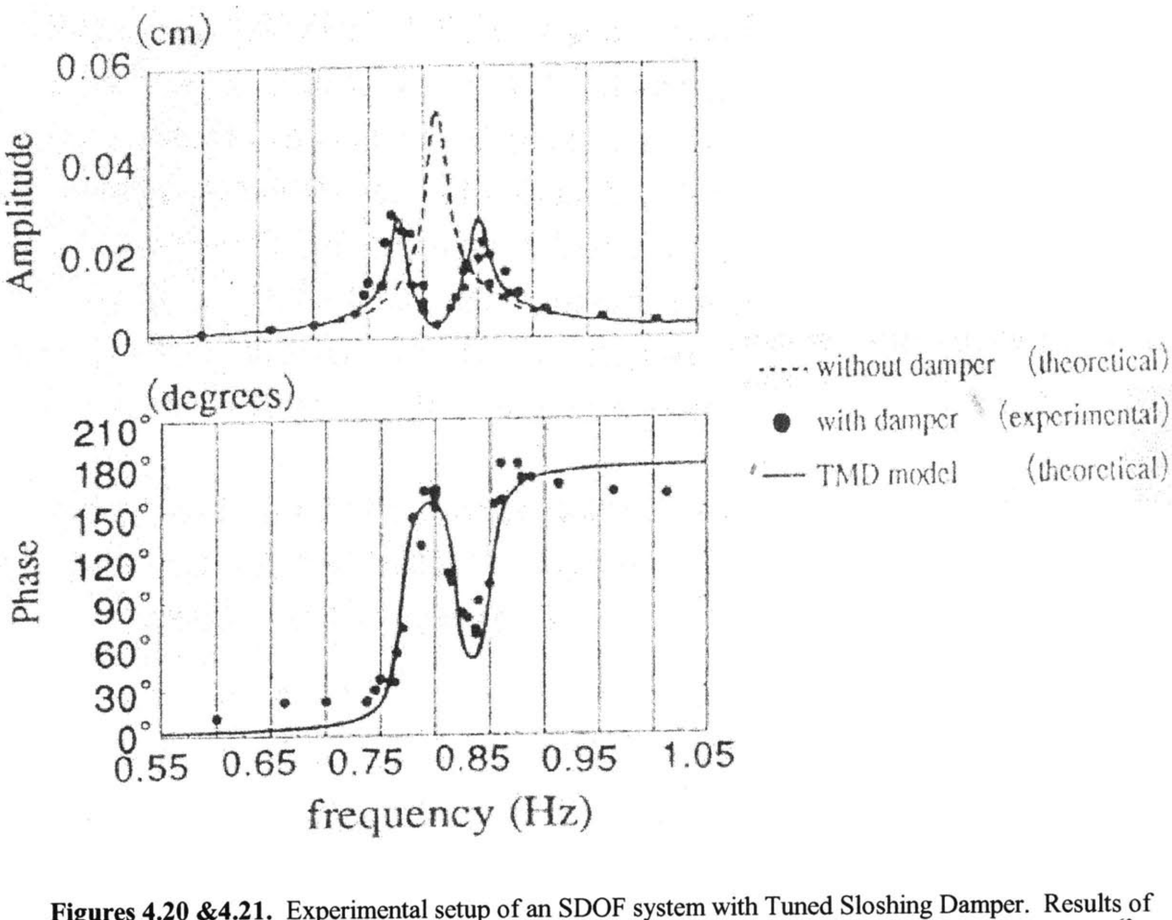
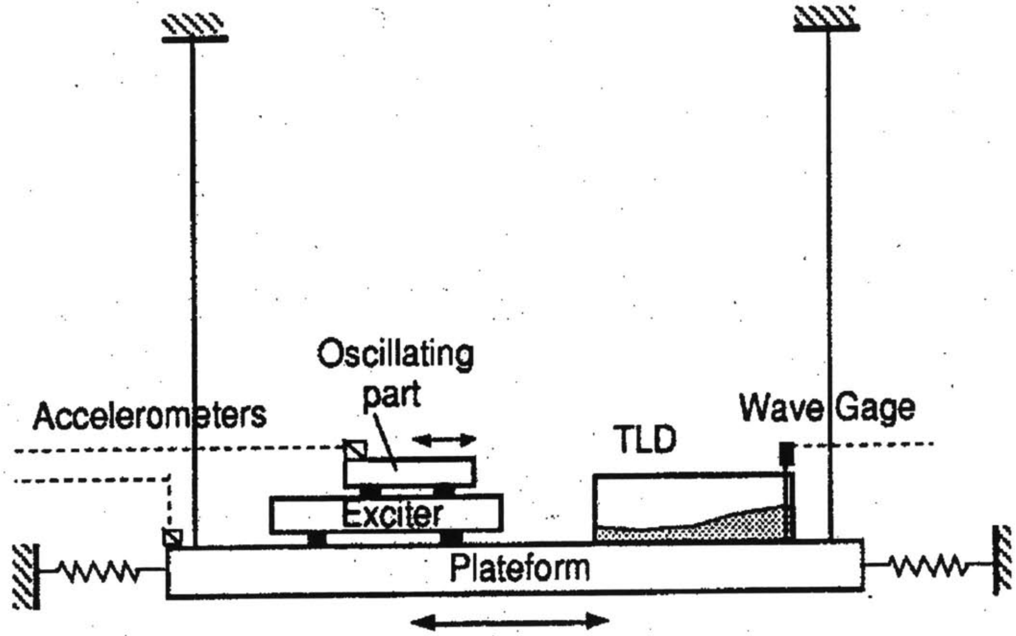
[58] Personal Communication. Y. Tamura, May 5, 2002.
[59] Refer to Fujino Y, Pacheco BM, Chaiseri P, Sun LM. "Parametric Studies on Tuned Liquid Damper (TLD) Using Circular Containers by Free-Oscillation Experiments" *Structural Engineering/Earthquake Engineering,* Vol. 5, No. 2, (1988) pp. 381s-391s.; Sun *et al.* (1992); Sun *et al.* (1995); and Yalla S. (2001).
[60] Sun *et al.* (1995), pg. 967.
[61] See Connor J. (2001), Chapter 4 entitled "Tuned mass damper systems" for an in-depth description of the TMD mechanical model and design procedure.

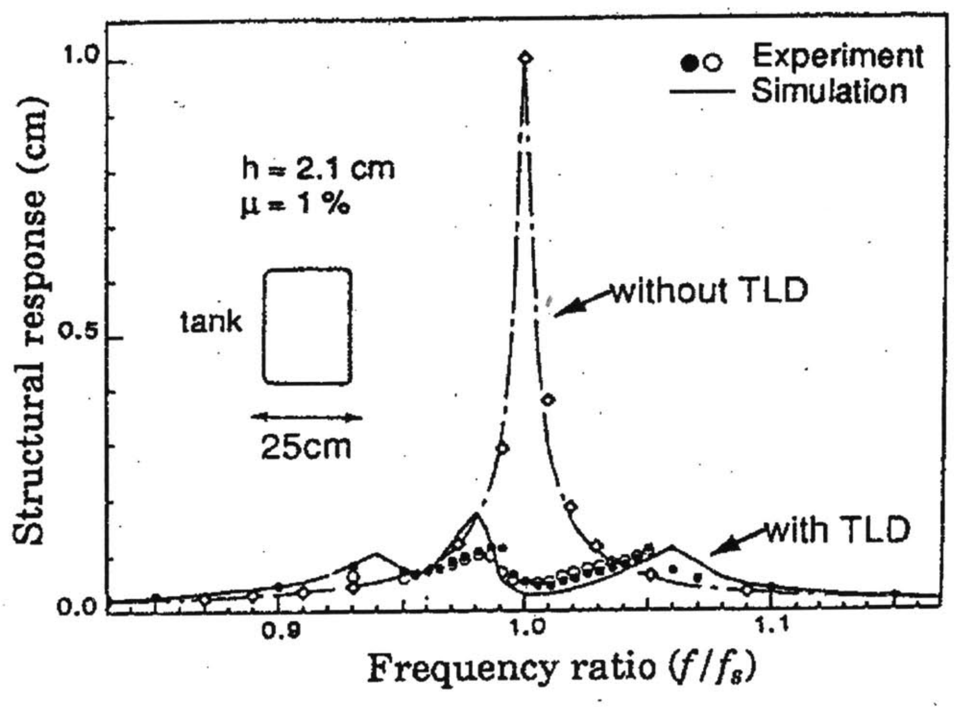




Figures 4.20 &4.21. Experimental setup of an SDOF system with Tuned Sloshing Damper. Results of forced vibration test showing modified structural response. Note the similarity to the TMD model.[62]

[62] Figures taken from Tamura Y, Kousaka R, and Modi VJ. "Practical Application of Nutation Damper for Suppressing Wind-Induced Vibrations of Airport Towers" *Journal of Wind Engineering and Undustrial Aerodynamics,* 41-44 (1992) pgs. 1921 & 1924.





Figures 4.22 &4.23. Experimental setup of an SDOF system with Tuned Sloshing Damper. Results of forced vibration test showing modified structural response. Note the similarity to previous example.[63]

[63] Figures taken from Sun *et al.* (1992) pg. 1890.

4.4.1 - Mass Ratio

Deciding on a mass ratio, μ, is somewhat of a balancing act. As the amount of water mass used increases, the effective damping ratio resulting from inertial movement increases, which is preferable. However, with increasing mass there comes an increase in space requirements, which is not preferable. Generally a mass ratio of about 0.01 can provide an equivalent damping ratio of between 5-10%, while minimizing space requirements. This value is thus commonly used, at least as a starting point, in the design of TSD systems.

4.4.2 - Frequency Ratio

The largest dynamic amplification of structural response occurs when the frequency of the forcing function resonates with the frequency of the structure. The "near optimal" design of a TSD or TMD will reduce the motion during such a condition by oscillating out-of-phase at the resonant frequency (ϕ=1.00). 111 Huntington's natural period is about 5 seconds, while the vortex shedding from the Prudential Center occurs roughly at 5-second intervals as well; thus it is clear that a near-optimally designed system will have a sloshing period of around 5 seconds. Following the rigorous optimal TMD theory (for an undamped structure, which is appropriate for preliminary design since 111 Huntington's inherent damping is around 1%), it can be shown that the optimal frequency ratio for a resonant loading condition is just slightly different than 1.00:[64]

$$\phi\big|_{opt} = \frac{\sqrt{1-0.5\mu}}{1+\mu} \quad \Rightarrow \quad f_D\big|_{opt} = \frac{\sqrt{1-0.5\mu}}{1+\mu} f_S \qquad (4.1)$$

Once the optimal design frequency is determined, a TSD vessel can be designed with the appropriate dimensions and water height such that the first sloshing mode has this desired frequency. In designing multilayered TSD vessels, each layer should be filled between 50-75% with water, leaving enough "head room" to prevent wave interaction with the level ceiling (and accompanying non-linearities).

Standard shaped (i.e. rectangular or cylindrical) vessels are generally used to house the sloshing liquid. Using a standard shaped vessel greatly simplifies the analysis of the predicted low amplitude sloshing motion; as such a problem is commonly described in standard hydrodynamics texts. A standard shaped vessel will also prove to be easily manufacturable and thus inexpensive.

The most important thing to consider when deciding between cylindrical and rectangular TSD vessels is the effect the different boundary geometries have on water sloshing within the tanks. Rectangular tanks can be simultaneously tuned to two different sloshing frequencies - one for each axis - which can correspond to the respective natural frequencies of the two predominant axes of a structure (Figure 4.24). Due to the axisymmetry of cylindrical TSD vessels, they can only be tuned to one fundamental

[64] Connor (2001), Chapter 4.

sloshing frequency. Despite its symmetric exterior appearance, 111 Huntington's "two fundamental frequencies" (5.26 sec in the East-West direction and 5.00 seconds in the North-South direction) are considerably far apart such that the use of bi-directionally tuned rectangular TSD vessels might be appropriate. However, considering that the serviceability concerns are arising from a particular loading coming from a particular direction – and the "two fundamental frequencies" are within 5% of each other – cylindrical TSD vessels can be used if deemed more appropriate for space/aesthetic concerns.

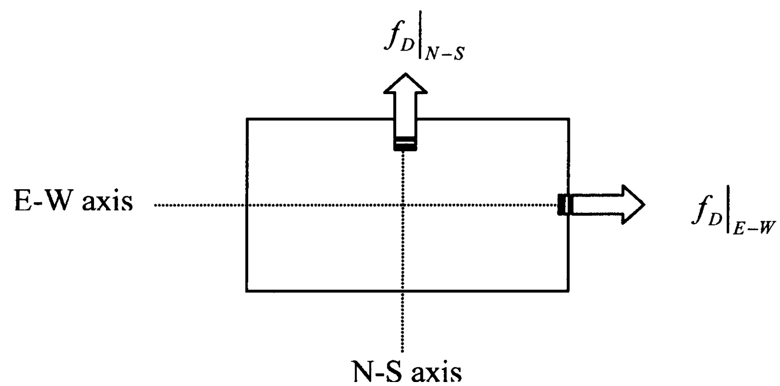


Figure 4.24. Plan view of rectangular TSD vessel, showing first frequency of sloshing along each axis.

For a circular basin, the first frequency of sloshing is:[65]

$$f_D = \frac{1}{2\pi}\sqrt{\frac{\lambda g}{D}\tanh\frac{\lambda h}{D}} \qquad (4.2)$$

where: $\lambda = 3.68$
D = diameter of tank
g = acceleration of gravity
h = height of water in tank

For a rectangular basin, the first frequency of sloshing along either axis is:[66]

$$f_D = \frac{1}{2\pi}\sqrt{\frac{\pi g}{l_{axis}}\tanh\frac{\pi h}{l_{axis}}} \qquad (4.3)$$

where: l_{axis} = length of rectangle side in direction of wave motion
g & h same as above

[65] Blevins (1984), pg. 368.
[66] Ibid. pg. 367.

4.4.3 – Damping Ratio

While indirect inertial damping applied by a TSD reduces much of the structural motion, a certain level of energy dissipation must accompany the mass movement in order to eliminate the beating effect (return of energy from the damper to the structure). Sometimes the frictional and viscous effects of water are not sufficient in terms of energy dissipation, so baffles or floating particles must be added. The optimal damping ratio derived from TMD theory (for an undamped structure) is:[67]

$$\xi_d\big|_{opt} = \sqrt{\frac{\mu(3-\sqrt{0.5\mu})}{8\,(1+\mu)(1-0.5\mu)}} \qquad (4.4)$$

Figure 4.25 summarizes experimental data obtained during the design of TSD vessels for the Shin-Yokohama Prince Hotel. The sloshing behavior of these vessels was investigated with both protrusions and floating particles, in order to obtain the optimal damping ratio of 5%.[68] As can be seen, the damping ratio varies with differing excitation amplitude and particle mass ratios. In order to do an in-depth analysis of the structure-damper interaction (discussed below), the relations between excitation amplitude and damping ratio must be described. For preliminary design, a design excitation amplitude can be used and experiments can determine whether floating particles need to be used.

Equivalent Damping Constant

Particles Mass Ratio μ_P(%)	Equivalent Damping Constant ζ (A_0 : Excitation Amplitude) A_0=1cm/s^2	A_0=5cm/s^2	A_0=20cm/s^2 *
0	0.032	0.058	0.135
2.5	0.042	0.062	0.136
5.0	0.049	0.065	0.132
7.5	0.056	0.067	0.137

* A_0=20cm/s^2 : Wave breaking occurs.

Figure 4.25. Experimental determination of equivalent TSD damping ratios used for design of SYPH.[69]

[67] Connor (2001), pg. 284.
[68] Wakahara *et al.* (1992), pg. 1899.
[69] Ibid. pg. 1988.

4.4.4 – Multi-Frequency Tuned Sloshing Dampers

One of the limiting features of Tuned Mass Dampers and Tuned Sloshing Dampers is their sensitivity to change in the design parameters such as forcing function frequency, damping ratio, and damper natural frequency. A given level of uncertainty (such as in predicting wind), approximations in analytical models, and limitations in manufacturing and construction precision, can all lead to significant differences between predicted and actual loadings conditions and structural properties. This can be of concern if a damper functions sub-optimally at anything but a precise condition.

To increase the flexibility and reduce sensitivity of a TSD system, some Japanese researchers have proposed the use of multi-frequency tuned sloshing damper systems (MTSDs).[70] This is a system in which particular vessels are designed to slosh at one of a range of frequencies near the optimal design frequency (Figure 4.26).

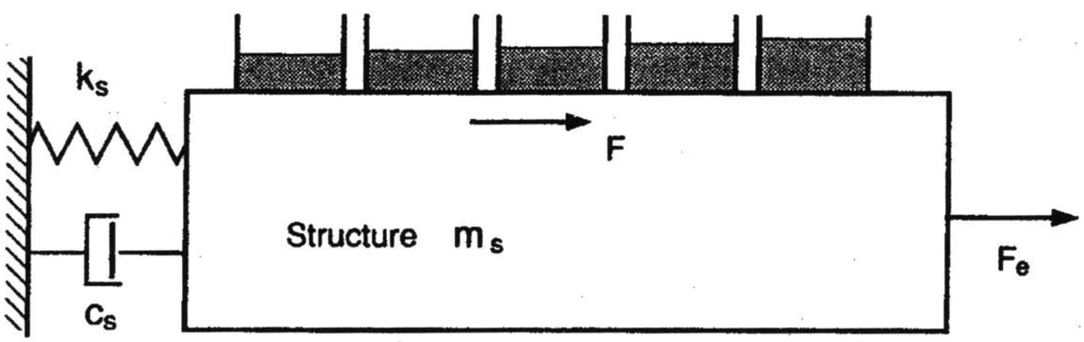


Figure 4.26. Schematic of a Multi-Frequency Tuned Sloshing Damper.[71]

An MTSD system is defined primarily by the frequency bandwidth, ΔR, it covers around the estimated optimal tuning frequency, f_0, where:

$$\Delta R = \frac{f_{lowest} - f_{highest}}{f_0} \qquad (4.5)$$

Experiments have shown that a ΔR ≈ 0.2 gives the best performance; it is large enough to effectively counteract variations in expected conditions, but small enough to keep all the vessels working efficiently in the range of these actual conditions.[72] As the number of differently tuned vessels is increased to "fill up" this frequency bandwidth, the improved performance of the MTSD over a broad range of excitations is seen (Figure 4.28).

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[70] Fujino *et al.* (1993).
[71] Fujino *et al.* (1993), pg. 3484.
[72] Fujino *et al* (1993), pp. 3486-3487.

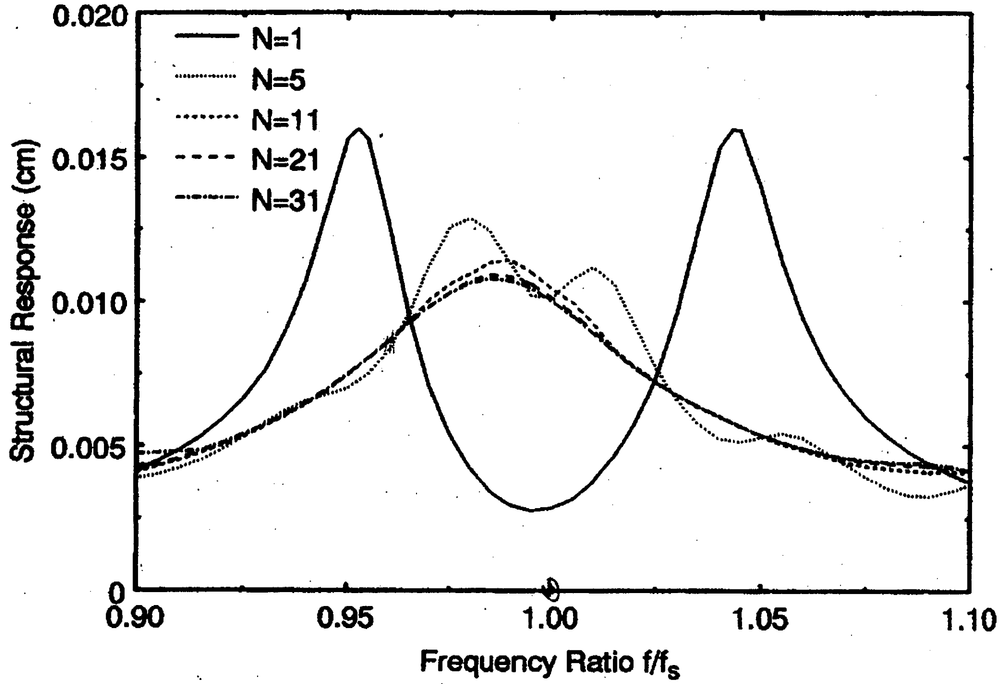


Figure 4.27. Comparison of structural response for single frequency TSD system (N=1), medium density MTSD system (N=5; ΔR=0.2), and high density MTSD system (N=11, 21, 31; ΔR=0.2). Note the increased performance of high-density systems over a broad range of frequencies.

MTSDs show dramatic analytical and experimental performance improvement over single frequency TSDs when there is significant off-tuning between the central (predicted optimal) damper frequency and the structural frequency (Figures 4.29 & 4.30). The off-tuning parameter, $\Delta\gamma$ is defined as:

$$\Delta\gamma = \frac{f_s - f_0}{f_0} \qquad (4.6)$$

where: f_s= natural frequency of the structure

It would be unwise not to expect some degree of off-tuning. As stated by Fujino *et al.*,

> "off-tuning may occur owing to various reasons: the nonlinearity of [the] structure, the change of structural natural frequency due to the change of live load, the error in identifying the natural frequency of the structure and so on."[73]

Thus the use of MTSDs, which do not require significant design or construction changes, has several tangible benefits.

[73] Fujino *et al.* (1993), pp. 3487-3488.

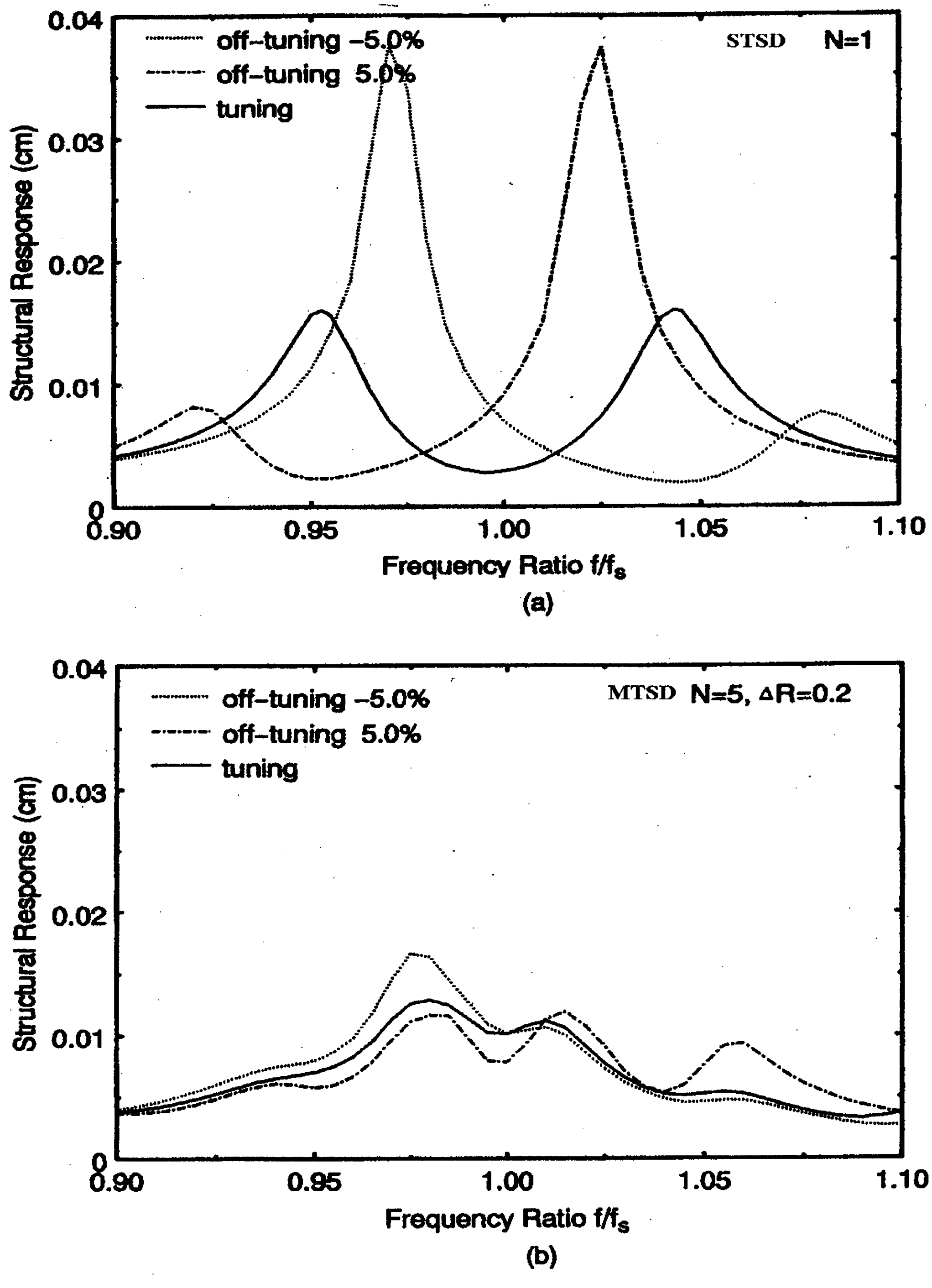


Figure 4.28. Comparison of analytical STSD (a) and MTSD (b) model performance when dampers are off-tuned from natural frequency by ±5.0%.[74]

[74] Figure taken from Fujino *et al*, (1993), pg. 3489.

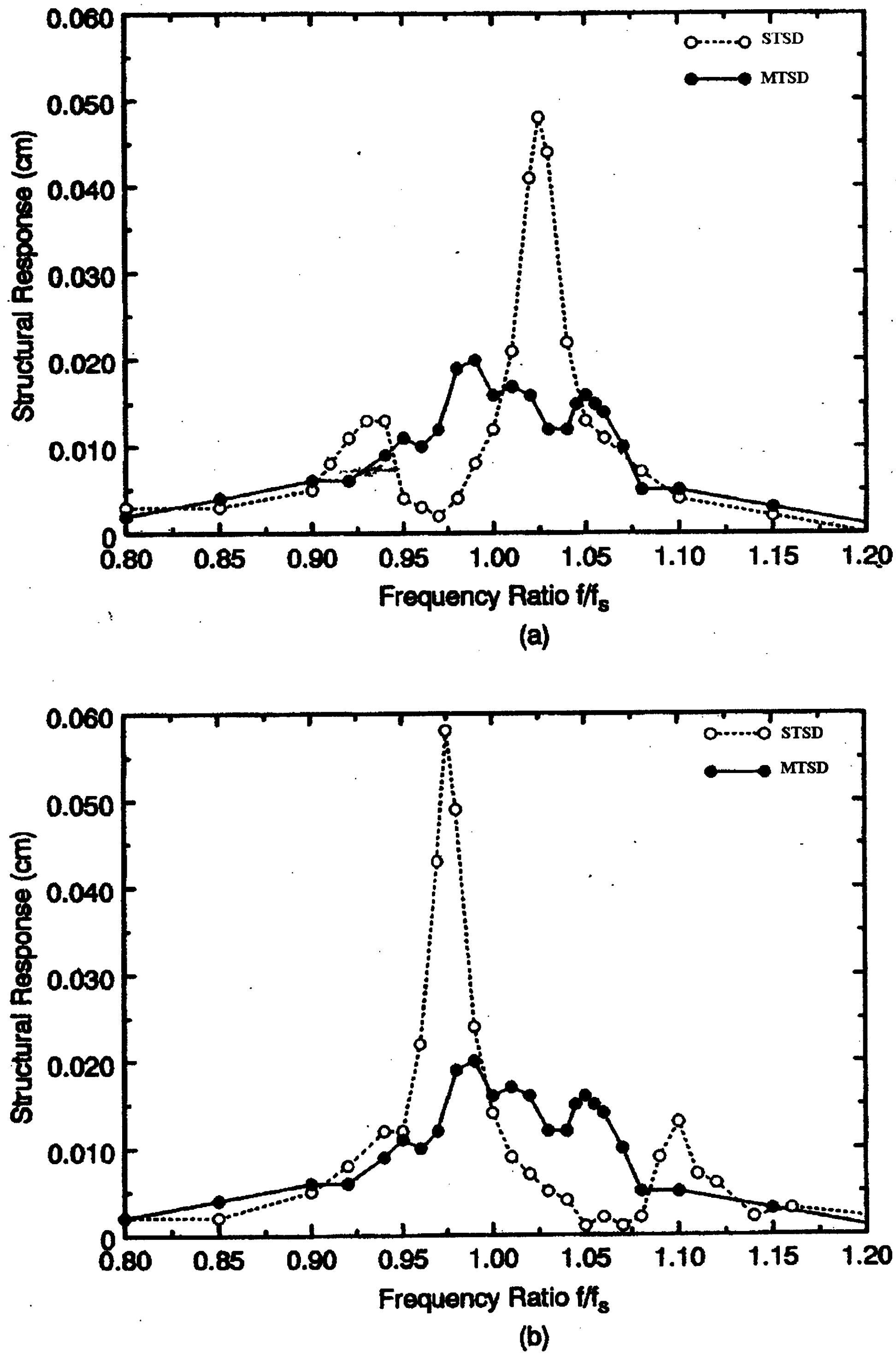


Figure 4.29. Comparison of experimental STSD (a) and MTSD (b) model performance when dampers are off-tuned from natural frequency by ±5.0%.[75]

[75] Figure taken from Fujino *et al,* (1993), pg. 3494.

4.4.5 – Modeling TSD-Structure Interaction

After the TSD system has been preliminarily designed according to the building's dynamic properties, it is important to further investigate the structure-damper system behavior under prescribed loadings, in order to verify the effectiveness of the TSDs. Because TSD and TMD behavior is so similar, it is possible to gain a good understanding of the modified system behavior by utilizing the analytic solutions of TMD theory, or by representing the sloshing dampers as equivalent masses, springs, and viscous dampers in a computer finite element model. Performing hand or computer calculations in accordance with the TMD theory become quite tedious with many degrees of freedom, and when TMD's are applied at several floors. Therefore, use a structural analysis program such as SAP2000 is quite helpful. Such an analysis was performed on a building model based on 111 Huntington (see Section 5, and Appendix A).

For the final design of a TSD system set to be implemented, a more detailed analysis can be performed to capture the true behavior of the sloshing tanks and refine the results of a TMD analysis. The general approach of such numerical solutions is detailed in some of the early papers describing the TSDs discussed above. A brief description of the approach follows below. According to one of the lead designers of these systems, these methods have proven effective and have practically become standard methodologies in Japan.[76]

As stated by Wakahara *et al.*, "the analytical model … can consider only two-dimensional liquid motion [one lateral, and the vertical direction]". Thus, equivalent rectangular tanks must be modeled in place of cylindrical ones, should cylindrical tanks actually be chosen. The dimensions and water height in the equivalent rectangular tank are such that the tuning values μ, ϕ, and ζ are the same as in the actual cylindrical tank.

This method involves applying a time-varying hydrodynamic force on the structure. This force is generated by water sloshing, then transferred from the liquid to the tank, and ultimately transferred from the tank to the structure. A schematic illustrating this approach is shown in Figure 4.31.

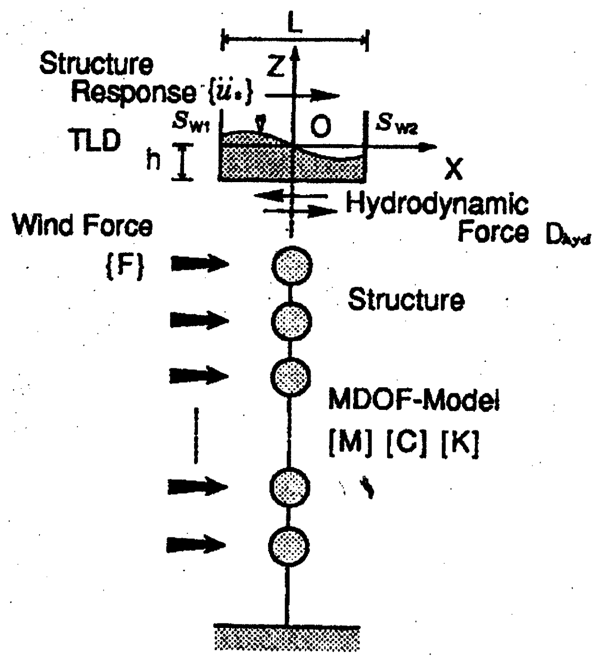


Figure 4.30. Schematic of MDOF model with TSD applied as hydrodynamic force.[77]

[76] "The characteristics and design methods [of TSDs] have already been established, and academic people are not interested in conducting research on them." Y. Tamura, personal communication, May 5, 2002.
[77] Figure taken from Wakahara *et al.* (1992), pg. 1903.

The governing equation of motion for such a system is the standard forced-vibration MDOF equation, with an additional term representing the hydrodynamic force:

$$[M]\{\ddot{u}_S\}+[C]\{\dot{u}_S\}+[K]\{u_S\}=\{F\}+\{\alpha\}D_{hyd} \qquad (4.7)$$

where:

[M]	= mass matrix of structure
[C]	= damping matrix of structure
[K]	= stiffness matrix of structure
$\{u_S\}$	= displacement vector
{F}	= applied wind force
$\{\alpha\}$	= vector consisting of 1 at nodes (floors) where a TSD is located and 0 at all other nodes
D_{hyd}	= hydrodynamic force of TLD

The hydrodynamic force, D_{hyd} is found by integrating the hydrodynamic pressure along the height of the side walls:

$$D_{hyd}=N_D\int_{S_W} p\,dz \qquad (s_W=s_{W1}\cup s_{W2}) \qquad (4.8)$$

where: p = hydrodynamic pressure

Wakahara *et al.* briefly discuss the advanced fluid mechanics description of the system used to determine the hydrodynamic pressure and ultimately the hydrodynamic force. This discussion is reprinted here:

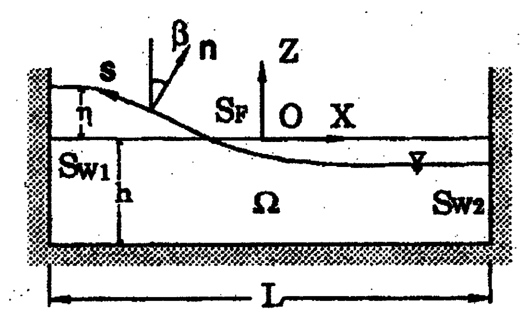


Figure 4.31. Diagram of sloshing basin with variables used to describe fluid motion.[78]

"The fluid is assumed to be incompressible and irrotational, so that the motion can be described by a velocity potential $\phi=\phi(x,z,t)$. The hydrodynamic pressure p can be expressed by using the velocity potential ϕ as:

$$p=-\rho\left[\frac{\partial\phi}{\partial t}+\frac{1}{2}\left(\frac{\partial\phi}{\partial z}\right)^2+\gamma_L\phi+\{\phi\}^T\{\ddot{u}_S\}x+gz\right] \qquad \ldots$$

[78] Figure taken from Wakahara *et al.* (1992), pg. 1903.

where g denotes the gravitational acceleration, ρ the fluid mass density and γ_L the equivalent damping coefficient.

The velocity potential ϕ... can be given as the solution satisfying the following equation:

$$\frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial z^2}=0 \quad (\text{in } \Omega). \qquad \ldots$$

The corresponding boundary conditions at the side wall and the bottom are

$$\frac{\partial\phi}{\partial x}=0 \quad (\text{on } s_{W1}, s_{W2}) \qquad \frac{\partial\phi}{\partial z}=0 \qquad (\text{at } z=-h) \qquad \ldots$$

The kinematic condition of the free surface is

$$\frac{\partial\phi}{\partial n}=\frac{\partial\eta}{\partial t}\cos\beta \quad (\text{on } s_F) \qquad \ldots$$

and the dynamic condition is

$$\frac{\partial\phi}{\partial t}+\frac{1}{2}\left[\left(\frac{\partial\phi}{\partial n}\right)^2+\left(\frac{\partial\phi}{\partial s}\right)^2\right]+\gamma_L\phi+\{\alpha\}^T\{\ddot{u}_S\}x+g\eta=0 \qquad (\text{on } s_F) \quad \ldots$$

which are considered to be nonlinear, where η = the free surface elevation above the still liquid level, s = the unit tangential vector along the boundary, n = the outward unit normal vector, and β = the angle between the vector n and the vertical axis z."[79]

[79] Wakahara *et al.* (1992), pp1902-1903.

5.0 – PROPOSED TSD SYSTEM FOR 111 HUNTINGTON

The following chapter describes the preliminary design of a TSD system for 111 Huntington Avenue, a system whose function is to reduce the response under resonant vortex shedding from the adjacent Prudential Center. The purpose of this exercise was to gain an understanding of what the basic characteristics of an effective system would be, i.e.: number of vessels, total water mass, total area required, vessel layout, vessel dimensions, etc. To do this, the preliminary design methodology described in the above section was used. In order to gauge the relative effectiveness of such a preliminary design, a SAP2000 model of a structure based on 111 Huntington was created. This model was analyzed under several conditions, including:

- No additional damping above the inherent 1.5% originally designed
- Additional damping due to viscous dampers (equivalent damping ratio equal to 3%)
- Addition of Tuned Mass Damper elements representing TSD vessels, preliminarily designed to give an equivalent damping ratio of 6%

It is important to note that these analyses would not be sufficient for final design; however, they provide a level of confirmation that the scope of a preliminarily designed TSD system is sufficient.

5.1 – Preliminary Design of TSD system

In accordance with common practice, it was decided that the TSD system would have a sloshing water mass equal to 1% of the fundamental generalized mass of the building. Because the first modal mass is around 35% of total building mass, the total water required for the system is 434,000 lbs. With a mass ratio of 1%, the system's near-optimal tuning ratio is 0.9875 times the fundamental frequency of the building, or 0.198 Hz.

Cylindrical multilayered vessels were chosen primarily due to personal aesthetic preference. (Rectangular tanks would be applicable as well). If the vessels are to be incorporated into the internal environment of the building, they have to be of a human sized scale, so it was decided that the tanks would be 3 ft. in diameter and about 5 ft. high. In order to slosh at a frequency of 0.198 Hz, water level height in a 3 ft. diameter tank must be 0.48 ft. Thus, with a layer height equal to 1.5 times the water level height, each vessel holds 8 layers of water or about 1500 lbs. Given these dimensions, a total of 290 vessels are required to meet the required sloshing mass of 434,000 lbs.

TSD vessels are most effective at the top of a structure, so it is ideal to place as many vessels on the roof as possible. However, it is also important to consider two things:

(1) The roof is subjected to extreme weather conditions and the tanks can not freeze or they don't work

(2) The roof often serves other functions such being as a place to put mechanical and communications equipment or being a public space for building occupants and visitors looking for a view.

In the preliminary design of the TSD system for 111 Huntington, a large proportion of vessels, 80 out of 290, are placed on the roof. In order to prevent the sloshing water from freezing, a minimal insulated enclosure could be built around the vessels (Figure 5.3), or some anti-freeze solution could be mixed in (though its effect on the sloshing behavior would have to be accounted for). The remaining vessels must be located near the top of the structure to maximize their effectiveness, but they must also be arranged so as to minimize encroachment on valuable rentable floor space. To do so, 21 vessels per floor were arranged on Floors 30-39. Taking up about 150 sq. ft. per floor (out of a total floor area of 22,500 sq. ft.) such a vessel arrangement can certainly be incorporated as an unintrusive aesthetic enhancement by a skilled architect.

The near-optimal damping ratio for the vessels is determined to be about 6%. This is unlikely to be achieved by viscous and friction effects of the water alone, so experimental data would be necessary to determine the appropriate amount of floating particles or an appropriate protrusion arrangement. For the purposes of this study, and based on previous TSD installations, it is assumed that this damping ratio can be achieved.

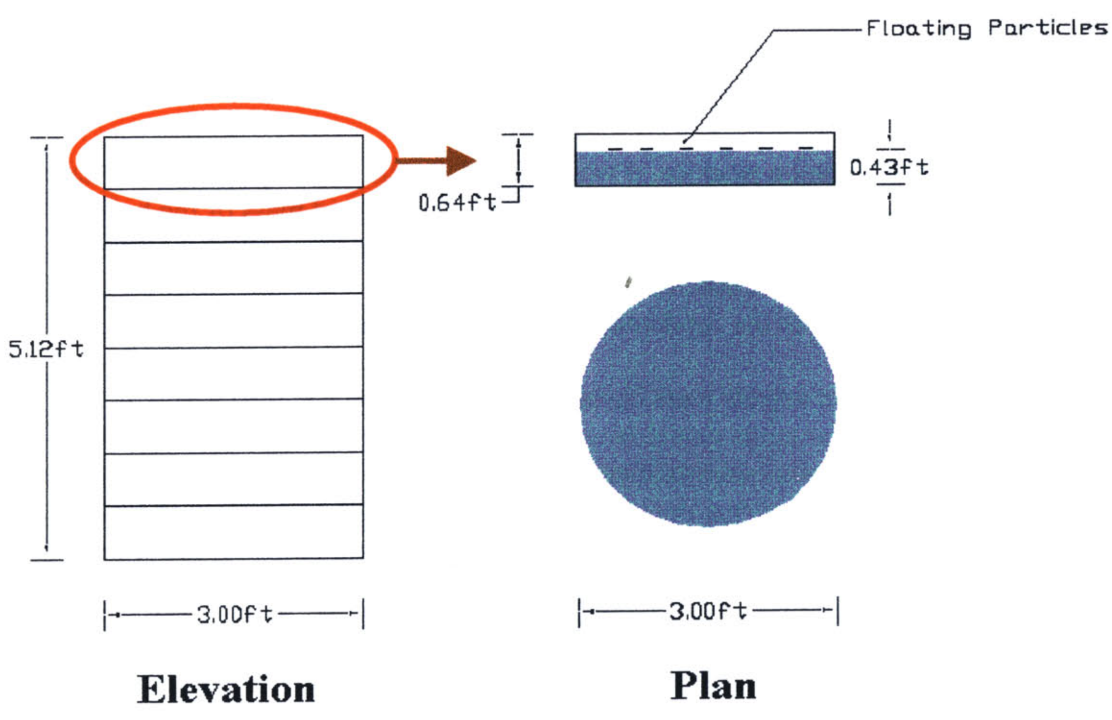


Figure 5.1. Vessel Dimensions for Proposed 111 Huntington TSD System.

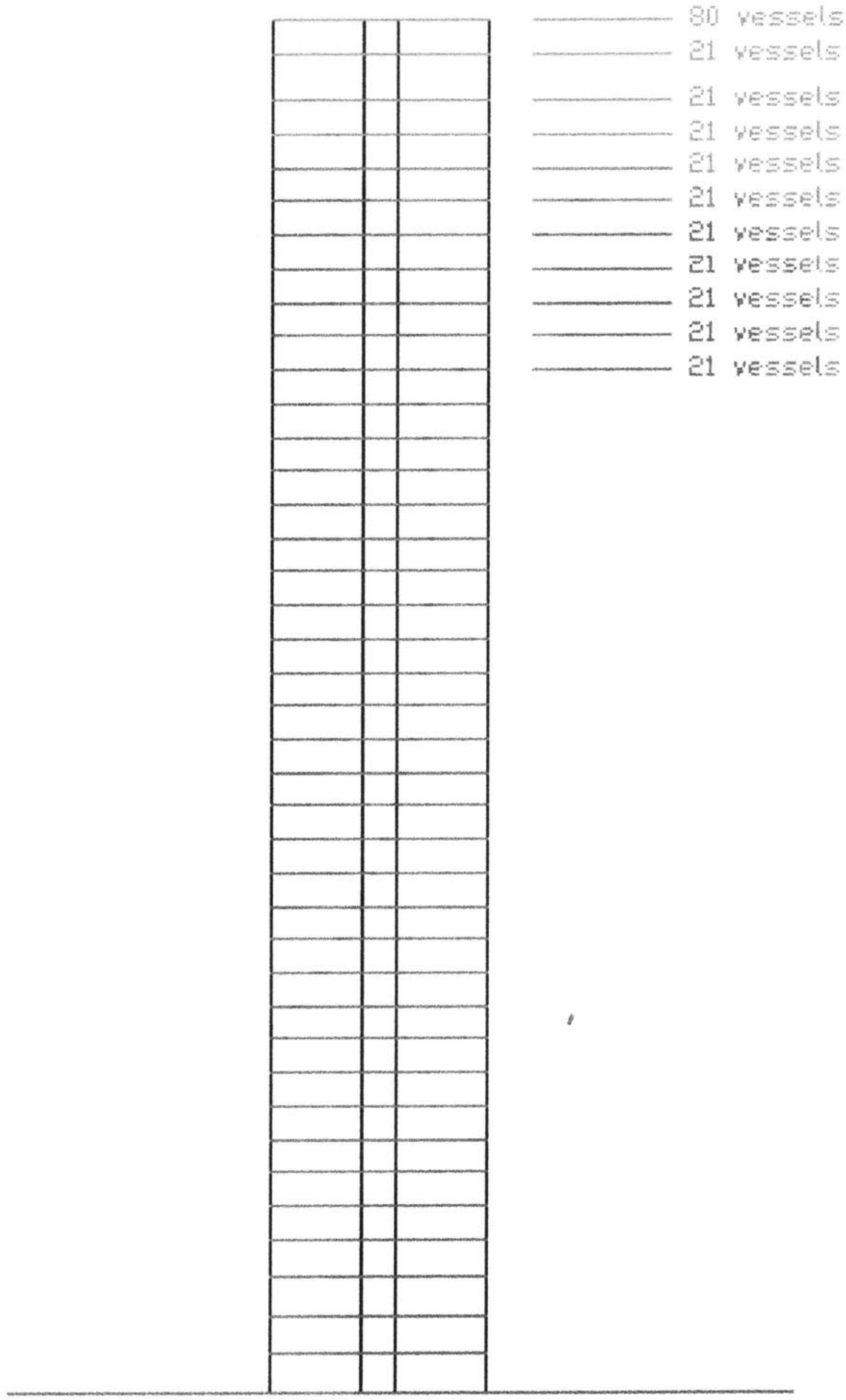


Figure 5.2. Schematic of Proposed Vessel Distribution for 111 Huntington.

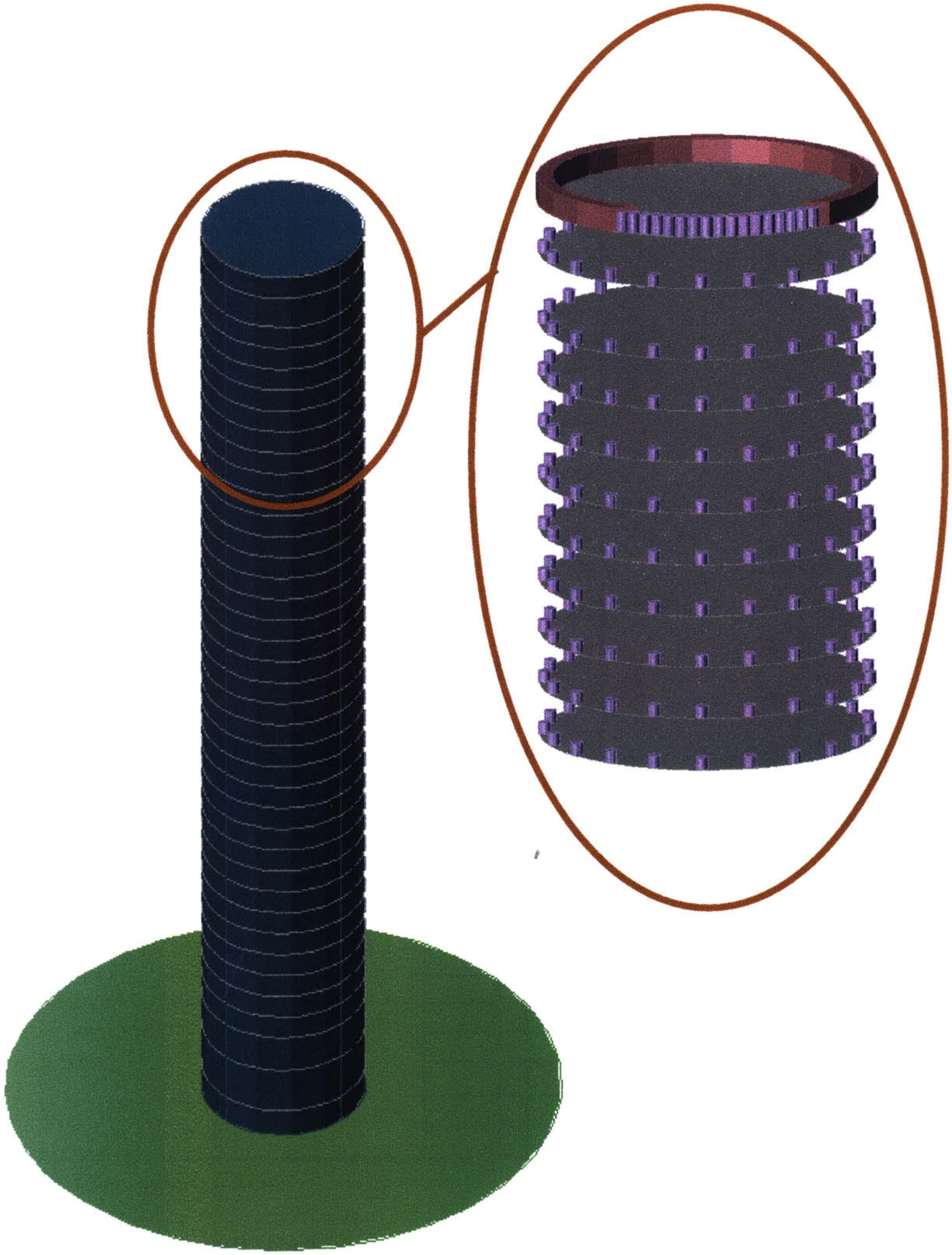


Figure 5.3. Rendering of spatial layout for proposed TSD system Note the insulated enclosure on the roof.

5.2 – Assessing the Effectiveness of TSD System

A SAP2000 model of a building based on 111 Huntington was created to test the efficiency of TSDs in reducing the acceleration response to periodic vortex shedding loads. The building was modeled as a 40 degree-of-freedom system and its 2D lateral behavior was analyzed. Each floor was modeled as a lumped mass, and connected to adjacent floors by a single member representing the entire story lateral stiffness. Floor masses were derived from the total mass of the building, which was presented in McNamara *et al.* The designers also presented the first two frequencies of the building (representing the first modes along the east-west, and north-south axes, respectively). With given masses, given frequencies, and theorized mode shapes, the values of story stiffness were determined. The fundamental period of the structure was stated to be 5.26 seconds. The second period was stated to be 5.00 seconds exactly. For simplicity, a model with a fundamental period of 5 seconds was created. The inherent structural damping ratio was stated to be around 1.5% and thus used as the damping ratio for the first mode in the model.

After simulating the dynamic properties of the undamped building, a loading scenario was created to model the effect of vortex shedding off of the Prudential Center. Vortexes are generally periodic and impulsive in nature; thus, the scenario was idealized as a periodic array of impulsive concentrated loads striking the floor masses every 5 seconds (in resonance with the fundamental period of the structure). Because wind pressures vary with altitude and are highest at high altitudes, it is reasonable to assume that the magnitudes of vortex loads follow the same pattern. Vortexes shed off of a building when it starts vibrating laterally under a sustained wind flow. Because the Prudential Center is taller than 111 Huntington, and the significant vortex-inducing lateral oscillations occur near the top of that structure, it is reasonable to assume that vortexes only strike the top 25% of 111 Huntington, or at the top ten floors.

The magnitudes of the impulsive vortex loads were established by creating a condition in the structural model of 111 Huntington that mimicked unserviceable behavior. A target "unserviceable acceleration" was established for the top floor of the model - 0.01 m/s^2, (or 0.033 ft/s^2 or 0.4 in/s^2). This value was established based on the experience of Japanese TSD designers and other wind engineers in assessing human sensitivity to vibrations. The magnitude of the wind loading was adjusted until the response of the structural model met this condition.

With a model of the structure and the dominant load case in place, a comparison could be made between the "undamped" performance (only the inherent 1.5% damping ratio), the performance with a 3% damping ratio (as claimed to be provided by the actual viscous dampers employed), and the performance with a proposed TSD system. This comparison is made on the basis of top floor acceleration, a serviceability requirement that drives the design.

5.2.1 – Top Floor Acceleration: Only Inherent 1.5% Damping

Figure 5.4 shows the top floor acceleration of the "undamped" model of 111 Huntington when subjected to a sustained periodic impulse forcing function for 10 minutes. As can be seen, the maximum top floor acceleration is equal to the target "unserviceable" value of .033 ft/s^2. It is seen that the amplitudes of oscillation level off after about 240 seconds. According to the logarithmic decrement of response, it becomes clear that this is the time it takes for the response to initial impulse to be completely degraded with a damping ratio of 1.5%. Subsequent to this point in time, a sort of equilibrium is reached between the amplitude of oscillation added by impulsive loading, and that dissipated by structural damping. As can be seen in Figure 5.5, during the initial loading of the structure, the response intensifies with each additional impulse of energy striking the structure at 5-second intervals.

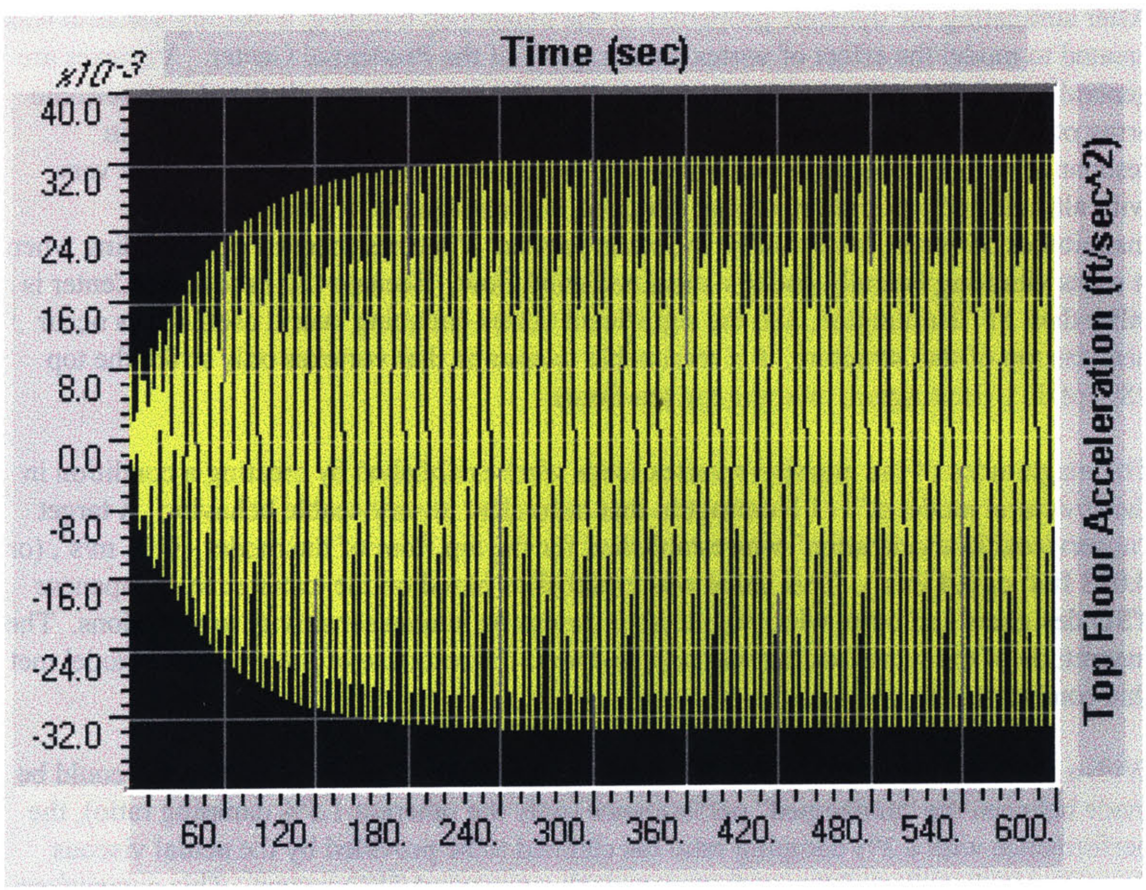


Figure 5.4. Time history of roof acceleration for the "undamped" case of 1.5% inherent damping.

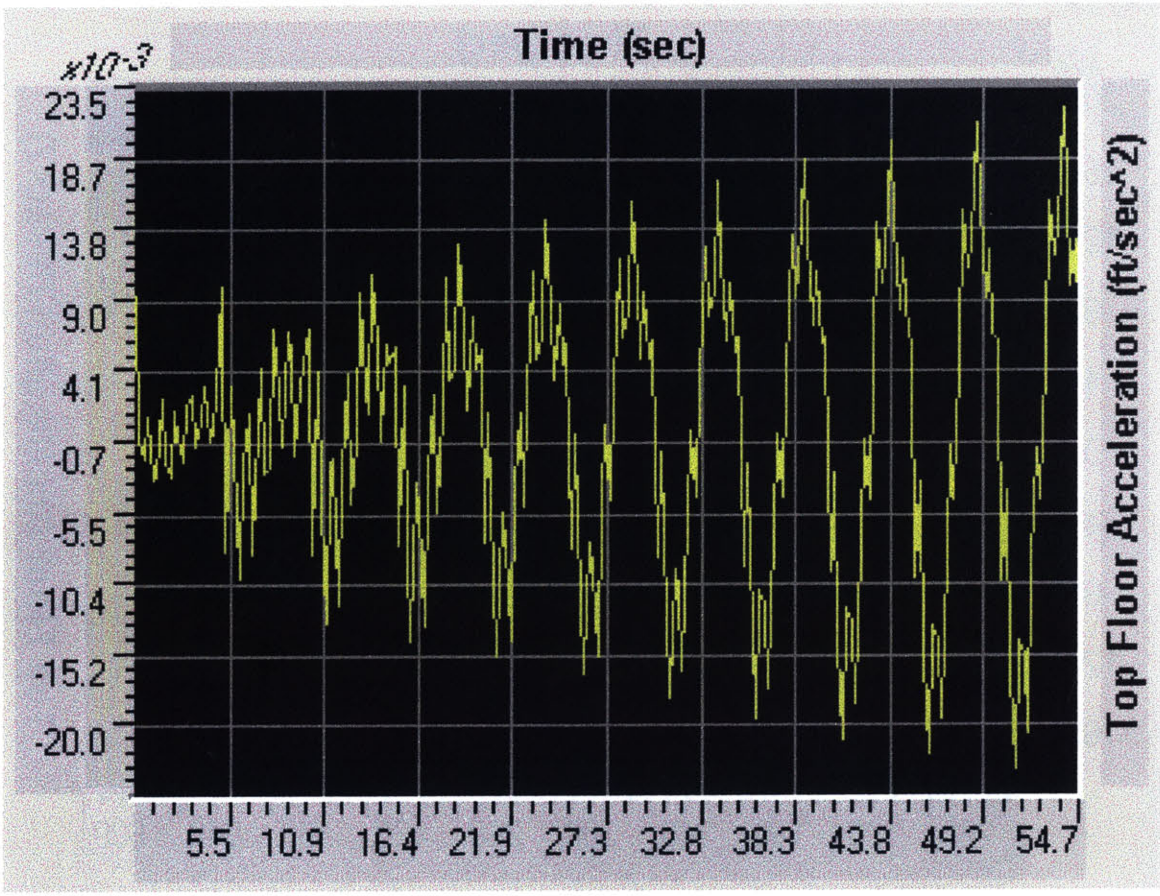


Figure 5.5. Initial amplification of response for "undamped" case.

5.2.2 – Top Floor Acceleration: 3% Structural Damping (Viscous Dampers)

The effect of the viscous dampers designed for 111 Huntington were simulated in SAP2000 by increasing the first modal damping ratio from the inherent 1.5% to the predicted 3%. Naturally, with increased damping came increased dynamic performance. As seen in Figure 5.6, the maximum acceleration of at the top floor of the model was reduced by about 40% to 0.0196 ft/s^2, and the system reached a steady state of vibrations in about half of the time it took the "undamped" case to do so. It is important to again acknowledge that these results only deal with a very specialized and idealized set of conditions, and are not necessarily representative of the structural response to actual wind loading. Nevertheless, by comparing the response of different structures to a highly controlled set of conditions, the broader implications of their performance can be made.

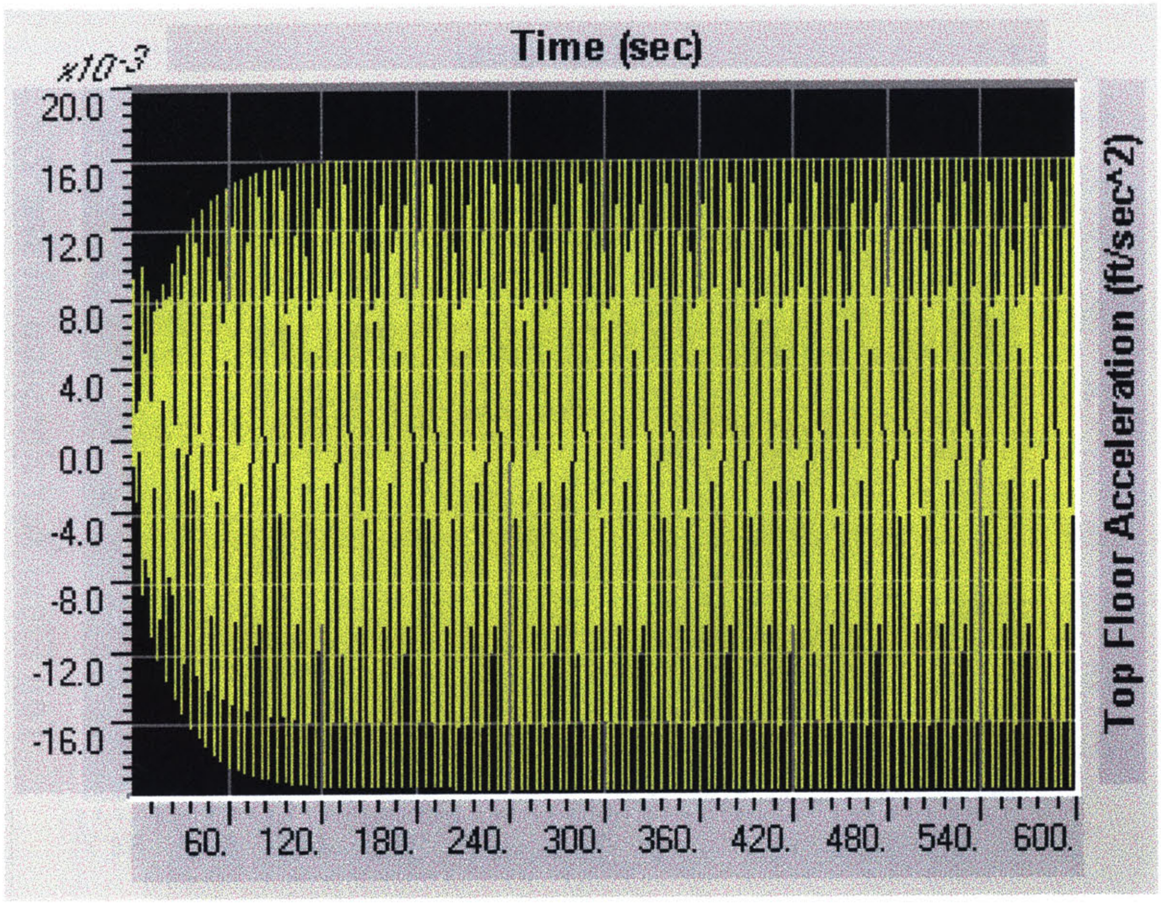


Figure 5.6. Time history of roof acceleration for 3% damping case

5.2.3 – Top Floor Acceleration: TSD Modeled as Equivalent TMD

Once the near-optimal dynamic properties of the sloshing vessels were determined, equivalent tuned mass damper components (masses, springs, viscous dampers) were added to the model of 111 Huntington to simulate their effect. As described above, the TMD analogy does not give a precise description of TSD behavior and efficiency; however it provides a very good approximation. The addition of these elements altered the fundamental period of the structure noticeably, changing it from 5.00 seconds to 5.29 seconds, increasing the modal mass more than the modal stiffness. The TMD (TSD) elements also caused a drastic reduction in structural response to the impulsive vortex loading. Figure 5.7 shows the maximum acceleration at the top floor of the structure to be 0.00788 ft/sec^2 – an almost 75% reduction in response. While the system has been designed to give optimal performance under this load case - providing an equivalent damping ratio much higher than the 3% of the viscous dampers – it is important to remember that TSD's (and TMD's) provide such remarkable performance only for a resonant load case. For 111 Huntington, the periodic vortex shedding is the only dynamic load case which requires additional damping, so TSDs are a prudent choice. Even so, this preliminary analysis provides only an idealized assessment of the TSD behavior; the TSDs will likely not improve general dynamic behavior (even specifically top floor acceleration) by 75%. This analysis does however provide a useful comparison to the undamped and viscous damped cases.

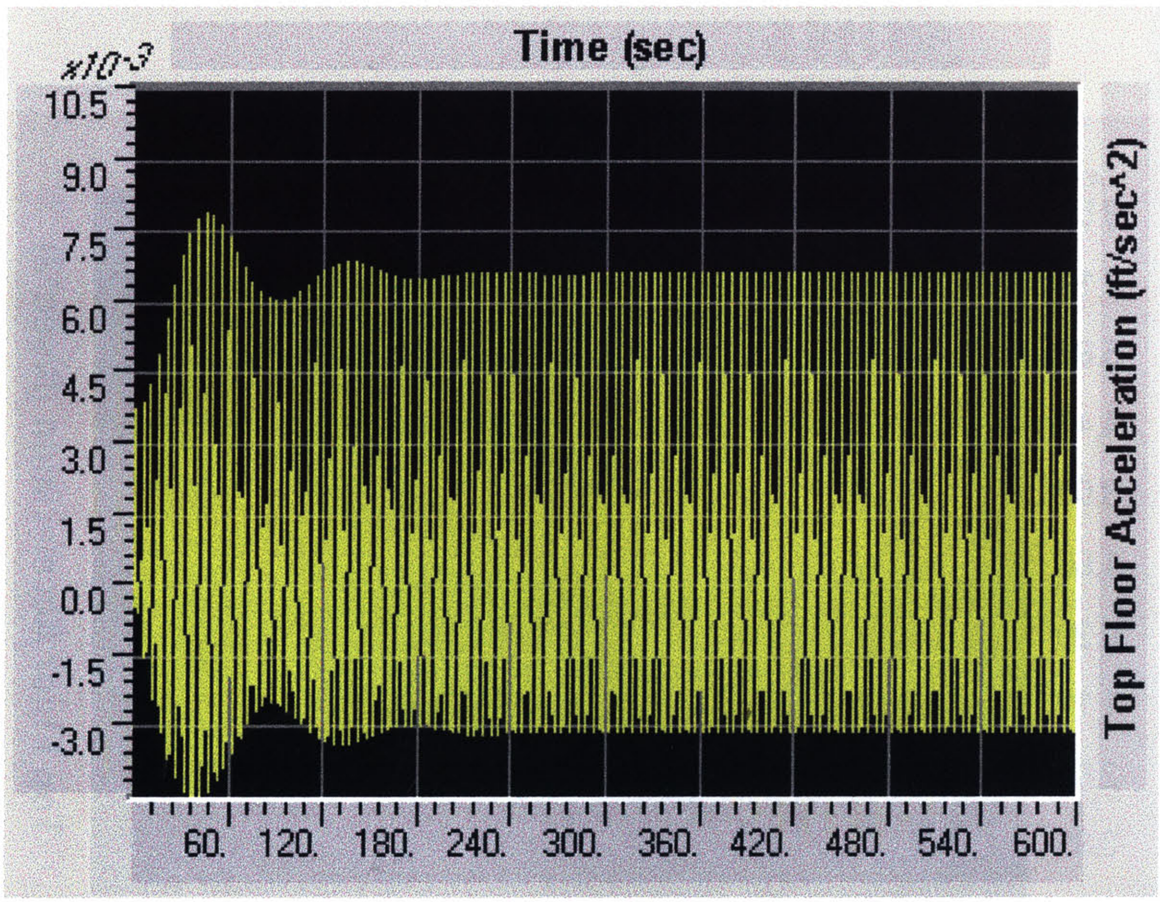


Figure 5.7. Time history of roof acceleration for TSD application.

Figure 5.8 focuses on the first 200 seconds of the roof acceleration displayed in Figure 5.7. This time history displays an initial peak of amplitude corresponding to the initial amplification of the system, followed by a noticeable beat before the oscillations reach a steady level of amplitude. The presence of the initial peak and sharp drop-off at 60 seconds is explained by considering the behavior of the viscous dampers used to model the energy dissipation capacity of the TSDs. As mentioned above, the resistive force of a viscous damper is proportional to the damper velocity. At the initial stages of excitations, the amplitude of damper velocity is small, so the devices are not especially efficient. As the structural response increases with the periodic impulse of input energy, the dampers become more and more effective, and they start drastically reducing the response to each impulse.

As noted above, the beating effect is commonly observed in TSD-structure interactions. In this case, the beating effect is relatively small, with the first (and only noticeable) beat at 150 seconds displaying an amplitude not even 15% greater than the steady level of oscillation eventually reached. This means that although further experimental investigation needs to be done to determine the appropriate level of damping required in the tanks, the preliminarily designed levels provide a good approximation.

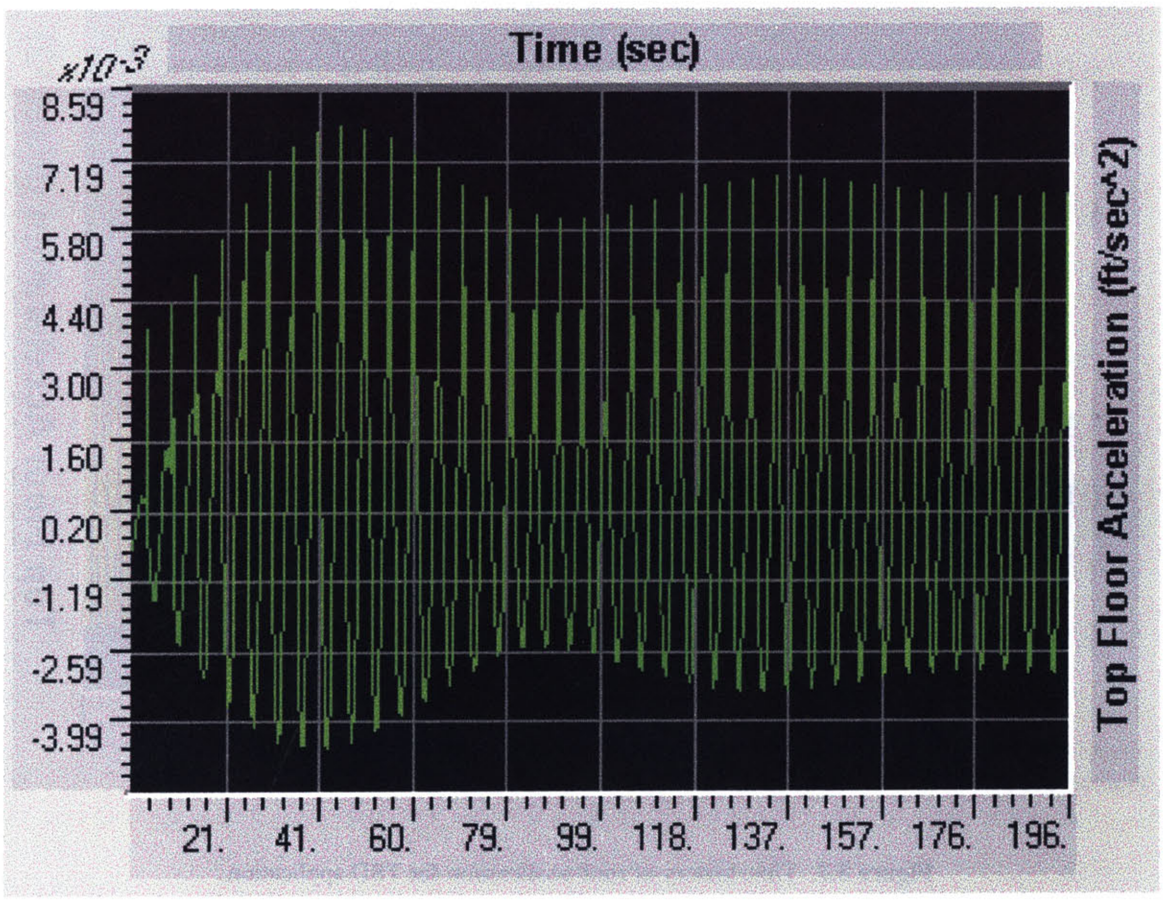


Figure 5.8. Initial amplification and beat of response for TSD case.

5.3 – Other Considerations

Below is a discussion of several other facets of a TSD system that were not employed in the preliminary design and the reasons for their exclusion.

5.3.1 – Multiple Frequency Tuned Sloshing Dampers

The use of Multi Frequency Tuned Sloshing Dampers can be appropriate when there is a level of uncertainty in either the dynamic properties of a building or the frequency content of the applied loading. As was shown in Chapter 4, the use of MFTSDs can largely reduce the sensitivity to off-tuning. It is important to note, however, that similar efficiency under off tuning is observed for the optimally damped case of a single frequency TSD when compared to an MFTSD (refer to Figures 4.2, 4.28, and 4.29). Thus, assuming the optimal damping ratio can be experimentally determined and implemented, the use of an MFTSD system becomes redundant. MFTSDs provide an appealing alternative when the structure's dynamic properties and experimentally derived TSD optimal damping ratio carry a high level of uncertainty. For the purposes of determining the preliminary scope of a TSD system, the decision between an MFTSD system or a single frequency TSD system is insignificant, as the basic properties – vessel number, water mass, dimensions, etc. – will all remain the same.

5.3.2 – Integration Into Site Water Management Scheme

As mentioned in the introduction, the motivation for this paper was to investigate the possibility of incorporating Tuned Sloshing Dampers into a site wide water management scheme. This objective was put on hold to while the challenges of implementing a conventional TSD system in an American high-rise were investigated. The preliminary design of such a system has not been proposed. Nevertheless, some observations about the future potential of such integrated systems were made.

Healthy cities are vital to the world. As centers of economic production, cultural innovation, and governmental jurisdiction, cities are indispensable loci of human society. The existence of these dynamic and compact centers of human activity is also crucial for a healthy and sustainable planet. Having millions of people together into cities reduces human footprint on the remainder of the Earth's surface, which is essential to the preservation of the natural ecosystems that support our lives on this planet.

Nevertheless, cities are not innately benign toward the environment. In fact they decimate and eliminate native ecosystems upon formation, demand intense resource use, and emit large quantities of pollution. This increased stress on the environment reduces its ability to perform for free such priceless and vital services as providing clean air to breathe and water to drink. So while cities are essential to the human condition and greater environmental health, it is imperative to make them more compatible with and less demanding of their surroundings.

In many ways it is inevitable that native ecosystems be drastically altered by the emergence of urban centers. However, development can strive to *replace* more and more of the lost functions of these ecosystems with a built infrastructure that can do similar things. For example, when thousands of acres of trees and soil are removed and/or paved over to create a city, water retention and uptake, air purification, microclimate control, and wildlife habitats are all greatly diminished. In sum, the buildings and pavement that are put in their place create pollution instead of purifying it, shun rain water down storm drains instead of using and retaining it, create harsher, not milder, microclimates, and replace the sounds of nature with the sounds of the city. Specific developments show that this does not have to be so. Indeed it is possible that buildings can contribute to a surrogate, man-made, ecosystem playing a similar role to the trees that they replace.

One of the ways buildings can replace a function of the natural ecosystem they replace is by retaining storm water. Storm water collection is common in places with water shortages, such as Singapore. Although water supply has not historically been an unmanageable problem in Boston, the costs of an ever increasing demand are mounting steadily. The idea of taking a pro-active role in watershed management is gaining popularity. Thus the idea of employing roof gardens and rainwater collection schemes in Boston area buildings is appealing. However, the benefits of integrating a sloshing damper system into such projects are not necessarily clear.

Sloshing dampers require a lot of space. This drawback can be minimized if the vessels are distributed and incorporated into the internal environment of the building. However, if such vessels are going to be on display, they will need to look nice. If storm water were to pass through them, they would become dirty, require lots of maintenance, and uncertainties about the precision of tuning would arise. The original concept to incorporate living systems within the vessels might be too complicated, as well, since maintenance and upkeep of hundreds of "living vessels" could become an all-encompassing task. Again, a lack of precision in keeping an optimal tuning ratio would be prominent if each vessel were filled with soil, plants, algae, etc. At a glance, it seems clear that the capital costs to install piping, and controls for such a system, combined with the increased maintenance costs do not justify the benefits of purifying rainwater with vessels acting as sloshing damper. Despite this, the sloshing damper is an effective, low cost, environmentally friendly means of structural control. Further, such dampers can certainly be used independently, but in parallel, with any water management schemes that aim to make a building "more sustainable".

6.0 - CONCLUSION

This paper has investigated feasibility of using Tuned Sloshing Dampers as a means of motion control in American high-rise buildings. TSDs are effective at reducing the response to highly periodic loadings such as wind, and have demonstrated their ability to improve a structure's serviceability. They are generally less effective against the non-periodic impulsive loads generated by seismic activity. Thus, sloshing dampers are not applicable to every motion control situation; however, there are clear merits in using them to reduce excessive acceleration under wind loadings (Tables 6.1-6.2). Additionally, TSDs are a cost effective way to reduce structural response, generally costing a fraction of the amount of other types of damping systems (Table 6.3).

Table 6.1. Comparison of Three Damping Systems: Different Load Types

	Tuned Sloshing Damper	Tuned Mass Damper	Viscous Damper
Load Type			
Small Amplitude	Very effective	Effective	Effective, though motion amplification device might be required
Large Amplitude	Loss of inertial damping capacity as amplitude increases, though energy dissipation capacity increases	Very effective	Very effective
Periodic Load	Very effective	Very effective	Very effective
Non-Periodic Load	Not very effective	Not very effective	Very effective
Maximum Amplitude Limit	None	Limited by allowable movement of mass, severe loadings must be considered to avoid catastrophic failure	Limited by capacity of damper, severe loadings must be considered to avoid catastrophic failure

Table 6.2. Comparison of Three Damping Systems: Assorted Criteria

	Tuned Sloshing Damper	Tuned Mass Damper	Viscous Damper
Criteria			
Space requirement	Large, but distributed vessels can drastically reduce impact	Very large	Minimal
Retrofit ability	Easy	Difficult	Difficult
Accessibility for maintenance	Easy	Easy	Difficult if framing is hidden

Table 6.3. Comparison of Three Damping Systems: Cost[80]

	Tuned Sloshing Damper	**Tuned Mass Damper**	**Viscous Damper**
Cost			
Design and consulting	Very limited, simple design	Specialized design and consulting required	Design is generally aided by manufacturer. Relatively simple.
Manufacturing	Very inexpensive, tanks and water	Very expensive	Relatively inexpensive
Installation	Very easy, can be done without machinery. Some support structure may be necessary	Local strengthening needed to support large forces, complicated installation	Realtively easy installation, akin to setting framing members
Mechanical components	Some piping might be required	Complicated assortment of bearings, actuators, guide ways, springs, and dampers	Only the damper itself
Electrical components	None	Computer control system	None
Power	None	Some designs require power	None
Maintenance and operation costs	Very limited. Cleaning tanks and changing water	Control system/components need maintenance. Power supply, cooling water, and oil supply are needed	Manufacturer claims no maintenance during service life

TSDs have never been used in the United States, although they have been used in several Japanese structures. One of the main challenges standing in the way of their wider use is the amount of space they take up near the top of a structure, space that is extremely valuable. The preliminary design of a TSD system for a high rise in Boston was proposed to illustrate the necessary characteristics and scale of such a system. By employing vessels in a well-designed distributed system, it is shown that their encroachment on rentable space can be reduced to a financially acceptable level. In doing so, serviceability requirements under low amplitude excitations can be met by a low cost system with minimal maintenance, easy installation, and pleasing aesthetic quality.

[80] Parts of table adapted from Yalla, S. (2001), pg. 156.

APPENDIX A: DEVELOPMENT OF SAP MODEL
DESIGN OF TSD VESSELS
DESIGN OF EQUIVALENT TMD ELEMENTS

Purpose: Determine appropriate values for stiffness (I) to be used in SAP2000 model based on 111 Huntington

Note: SAP Model is 39 DOF system. Each floor is lumped to a single point mass. Masses of each floor are known, fundamental frequency is known.

Notation:

H	height of buidling (ft)
T_1	fundamental period of building (sec)
f_1	fundamental frequency of building (Hz)
ω_1	fundamental angular frequency of building (rad/sec)
g	acceleration of gravity (ft/sec^2)
W	weight of building (lbs)
M	mass of building (lbs-sec^2/ft)
ρ_m	mass per unit length of building (lbs-sec^2/ft^2)
$\phi(x)$	mode shape as a function of building height x

Given Parameters: [from McNamara (2001)]

H	533	ft	
T_1	5.00	sec	(in the North-South direction)
f_1	0.200	Hz	
ω_1	1.26	rad/sec	
g	32.2	ft/sec^2	
W	124000000	lbs	
M	3850932	lbs-sec^2/ft	
ρ_m	7225	lbs-sec^2/ft^2	(assumes uniform mass density)

Key Assumptions:
The fundamental mode shape must be assumed since it is not presented in McNamara's paper. Ordinarily, a linear or parabolic displacement profile would be assumed for the first mode.

111 Huntington's lateral structural system is unique:

Floors 1-6:	Braced Frames
Floors 7-34:	Moment Frames
Floors 35-39:	Braced Frames

Because the top and bottom are stiffer braced frames, it is pprobable that the fundamental mode shape looks more like Figure 1 than Figure 2 or Figure 3. All three possibilities will be investigated initially.

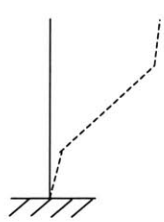


Figure 1

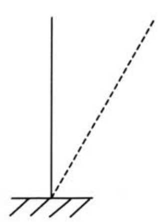


Figure 2

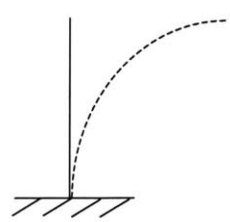


Figure 3

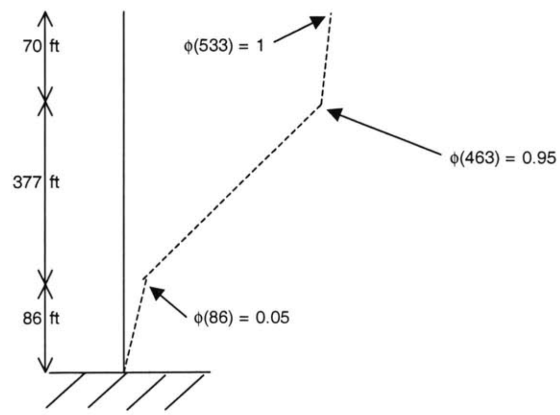
ASSUMING NON-UNIFORM LINEAR MODE SHAPE

Elevation Floor 6:	86 ft	*(lower transition from braced to moment frames)*
Elevation Floor 35:	463 ft	*(upper transition from moment to braced frames)*

Note: Elevations approximated from Figure 3.2

Set $\phi(H)$ =	1.00	*(top of building)*
Set $\phi(0)$ =	0.00	*(bottom of building)*
Assume $\phi(463) = .95[\phi(533)]$ =	0.95	*(due to stiffness of upper braced frames)*
Assume $\phi(86) = .05[\phi(533)]$ =	0.05	*(due to stiffness of lower braced frames)*

Assuming linear displacement profiles, graphical and mathematical representations of the mode shape can be generated



$$\phi(x) = \begin{cases} 0.000581(x) & 0 \le x < 86 \\ 0.00239(x) - 0.156 & 86 \le x < 463 \\ 0.000714(x) + 0.619 & 463 \le x < 533 \end{cases}$$

Now we can find the fundamental generalized mass (1st modal mass) of the building

$$\tilde{m}_1 = \int_0^H \sigma_m [\phi(x)]^2 dx$$

$$\tilde{m}_1 = 134600 \text{ lbs-sec}^2/\text{ft}$$

Note that the 1st modal mass is about 35% of the total mass of the building.

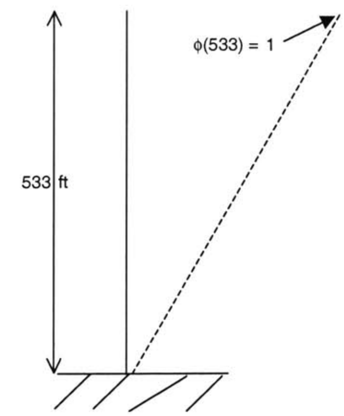
Most likely to represent true mode shape

More difficult and time consuming to model, for only slightly more accurate results

ASSUMING UNIFORM LINEAR MODE SHAPE

Set ϕ(H) =	1.00	*(top of building)*
Set ϕ(0) =	0.00	*(bottom of building)*

Assuming uniform linear mode shape, graphical and mathematical representations of the mode shape can be generated



$$\phi(x) = \frac{x}{H} = \frac{x}{533}$$

Now we can find the fundamental generalized mass (1st modal mass) of the building

$$\tilde{m}_1 = \int_0^H \sigma_m [\phi(x)]^2 dx$$

$\tilde{m}_1$ = 1289000 lbs-sec^2/ft

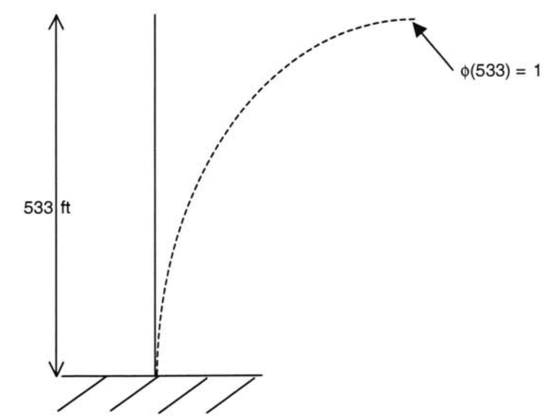
Note that the 1st modal mass is about 33% of the total mass of the building.

Model is easy to analyze
Conservative modeling of effect of Tuned Liquid Dampers placed on upper 5 stores, since massses will be acting at points of "smaller phi" than actual since mode two is more likely

ASSUMING PARABOLIC MODE SHAPE

Set $\phi(H)$ =	1.00	*(top of building)*
Set $\phi(0)$ =	0.00	*(bottom of building)*

Assuming linear displacement profiles, graphical and mathematical representations of the mode shape can be generated



$$\phi(x) = \left(\frac{x}{H}\right)^2 = \left(\frac{x}{533}\right)^2$$

Now we can find the fundamental generalized mass (1st modal mass) of the building

$$\tilde{m}_1 = \int_0^H \sigma_m [\phi(x)]^2 dx$$

$\tilde{m}_1$ = 1284000 lbs-sec^2/ft

Note that the 1st modal mass is about 33% of the total mass of the building.

Conservative
Easy to analyze, building acts like bending beam
Values of I necessary are unreasonable.

FINDING THE BENDING RIGIDITY FOR MODEL OF 111 HUNTINGTIN

Assumptions:
Neglect shear deformation (γ=0)
Building deflects with constant curvature χ
Mode shape (from above):

$$\phi(x) = \left(\frac{x}{H}\right)^2 = \left(\frac{x}{533}\right)^2$$

Governing Equations:

$$u(x,t) = q_B(\cos\ \omega_1 t + \delta)[\phi(x)] = q_B(\cos\ \omega_1 t + \delta)\left(\frac{x}{H}\right)^2 \quad (1)$$

where q_B is the maximum amplitude displacement at the top of the building

$$\gamma = \frac{\partial u}{\partial x} - \beta \quad (2)$$

$$\chi(x,t) = \frac{\partial \beta}{\partial x} = \chi(\cos\ \omega_1 t + \delta) \quad (3)$$

$$V(x,t) = -\rho_m \int_x^H \ddot{u}(x,t)dx \quad (4)$$

where V(x,t) is the shear force

$$M(x,t) = \int_x^H V(x,t)dx = D_B(x)[\chi(x,t)] \quad (5)$$

where M(x,t) is the bending moment

Solving for D_B:

Setting (2) = 0 , and solving:

$$\beta = q_B(\cos\ \omega_1 t + \delta)\left[\frac{2x}{H^2}\right] \quad (6)$$

Solving (3):

$$q_B = \frac{H^2\chi}{2} \quad (7)$$

Substituting (8) and then differentiating (1) twice:

$$\ddot{u}(x,t) = -\frac{H^2\chi\omega_1^2}{2}(\cos\ \omega_1 t + \delta)\left(\frac{x}{H}\right)^2 = -\omega_1^2 u(x,t) \quad (8)$$

Substituting (9) and then solving (4):

$$V(x,t) = \frac{\rho_m \omega_1^2 q_B}{H^2}(\cos\ \omega_1 t + \delta)\left[\frac{H^3}{3} - \frac{x^3}{3}\right] \quad (9)$$

Substituting (10) & (8) and then solving (5):

$$D_B(x) = \frac{\rho_m \omega_1^2}{2}\left[\frac{H^4}{4} - \frac{H^3 x}{3} + \frac{x^4}{12}\right] \quad (10)$$

DISCRETIZING DB FOR MODEL MEMBERS PROPERTIES - USING PRESCRIBED MODE SHAPE

E (steel)= 29000000 psi
D_{Bi} = Value of DB calculated from continuous function for floor (i)
D_{Bi+1} = Value of DB calculated from continuous function for floor (i+1)

Discrete values of I are calculated two ways:
(1) averaging value of DB for top and bottom of story
(2) using value of DB at bottom of story
These two methods give neglieable difference in dyanmic charactistics of SAP Model

Floor Height	Db (lbs-ft^2)	Db (lbs-in^2)
0	1.15E+14	1.65745E+16
15	1.11E+14	1.59525E+16
40	1.04E+14	1.49162E+16
60	9.78E+13	1.40876E+16
73	9.41E+13	1.35497E+16
86	9.04E+13	1.30125E+16
99	8.66E+13	1.24763E+16
112	8.29E+13	1.19415E+16
125	7.92E+13	1.14084E+16
138	7.55E+13	1.08775E+16
151	7.19E+13	1.03493E+16
164	6.82E+13	9.82421E+15
177	6.46E+13	9.30287E+15
190	6.10E+13	8.78589E+15
203	5.75E+13	8.27392E+15
216	5.39E+13	7.76767E+15
229	5.05E+13	7.26791E+15
242	4.71E+13	6.77543E+15
255	4.37E+13	6.29109E+15
268	4.04E+13	5.81577E+15
281	3.72E+13	5.35043E+15
294	3.40E+13	4.89606E+15
307	3.09E+13	4.45369E+15
320	2.79E+13	4.02441E+15
333	2.51E+13	3.60935E+15
346	2.23E+13	3.20969E+15
359	1.96E+13	2.82665E+15
372	1.71E+13	2.46151E+15
385	1.47E+13	2.11558E+15
398	1.24E+13	1.79024E+15
411	1.03E+13	1.48689E+15
424	8.38E+12	1.20699E+15
437	6.61E+12	9.52059E+14
450	5.03E+12	7.23641E+14
463	3.63E+12	5.2334E+14
476	2.45E+12	3.52804E+14
489	1.48E+12	2.13726E+14
502	7.49E+11	1.0785E+14
520	1.35E+11	1.94011E+13
533	5.44E-03	0.783413256

Story	I (in^4) (Dbi+Dbi+1)/(2E)	I (ft^4)	I (in^4) (Dbi)/(E)	I (ft^4)
1	560810585	27045	571533480	27562
2	532219061	25666	550087690	26528
3	500065433	24116	514350431	24805
4	476505429	22980	485780435	23427
5	457968240	22086	467230423	22532
6	439461644	21193	448706057	21639
7	420996346	20303	430217232	20747
8	402584667	19415	411775460	19858
9	384240548	18530	393393874	18972
10	365979547	17649	375087222	18089
11	347818841	16774	356871872	17210
12	329777223	15904	338765809	16337
13	311875107	15040	320788637	15470
14	294134524	14185	302961578	14610
15	276579121	13338	285307470	13759
16	259234165	12502	267850772	12917
17	242126542	11677	250617559	12086
18	225284755	10864	233635526	11267
19	208738923	10066	216933983	10462
20	192520786	9284	200543862	9671
21	176663701	8520	184497710	8897
22	161202644	7774	168829693	8142
23	146174207	7049	153575595	7406
24	131616602	6347	138772819	6692
25	117569658	5670	124460385	6002
26	104074822	5019	110678930	5338
27	91175160	4397	97470713	4701
28	78915355	3806	84879606	4093
29	67341709	3248	72951103	3518
30	56502142	2725	61732315	2977
31	46446191	2240	51271969	2473
32	37225012	1795	41620413	2007
33	28891379	1393	32829611	1583
34	21499684	1037	24953147	1203
35	15105937	728	18046221	870
36	9767765	471	12165652	587
37	5544415	267	7369878	355
38	2193978	106	3718953	179
39	334501	16	669003	32

DISCRETIZING DB FOR MODEL MEMBER PROPERTIES - USING UNIFORM STIFFNESS DISTRIBUTION

The above calculations gives the optimum stiffness distribution tbat will generate the prescribed first mode with a period of 5 second. However it clear that this method gives a somewhat unrealistic distribution of stiffness for a buidilng, since column sizes (stiffness) can not vary so much as to generate order of magnitude differences in stiffness.

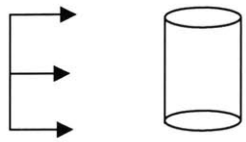
Thus, the assumption of a uniform stiffness distribution was made.
Trial and error within the defined SAP model was used to find the appropriate value for column moment of inertia.
For the given floor masses and elevations, a uniform column moment of inertia equal to **18650 ft^4** gives a fundamental period of 5 second.
This value is reasonable based on a comparison with the values for the optimum stiffness distribution determined above.

INHERENT STRUCTURAL DAMPING

According to McNamara (2001), the inherent structural damping of 111 Huntington is about 1.5%. This value was used as the damping ratio for the first mode.

Generating Static Wind Load:

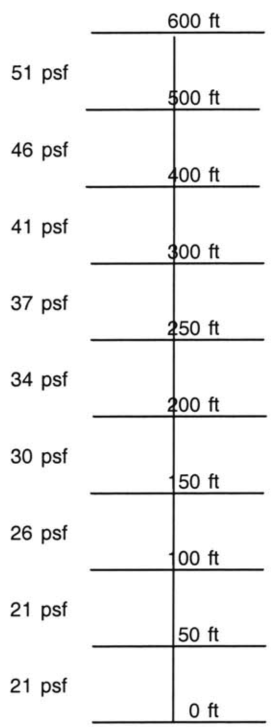
Wind pressure



Assume wind pressure from one side acts uniformly over 1/3 of face of cylinder

Since average floor area is 22,500 sq ft., assume cylinder radius is:	84.65	ft
So for a given height (h) of cylinder, the area subjected to wind pressure is:	177.3 x h	ft^2

Use the Massachusetts Building Code to get wind pressure distribution for a 533 ft tall building
[Table 1611.4 (Zone 3, Exposure B)]



From above pressure distribution we can find the equivalent point static loads on each degree of freedom.

Story #	Story height (i)	Story Elevation	DOF(i)	Exposed height (.5hi +.5hi+1)	Exposed Area	Equivalent Load
1	15	15	1	20	3545.8	74461
2	25	40	2	22.5	3989.0	83769
3	20	60	3	16.5	2925.3	61430
4	13	73	4	13	2304.8	48400
5	13	86	5	13	2304.8	48400
6	13	99	6	13	2304.8	48400
7	13	112	7	13	2304.8	59924
8	13	125	8	13	2304.8	59924
9	13	138	9	13	2304.8	59924
10	13	151	10	13	2304.8	59924
11	13	164	11	13	2304.8	69143
12	13	177	12	13	2304.8	69143
13	13	190	13	13	2304.8	69143
14	13	203	14	13	2304.8	69143
15	13	216	15	13	2304.8	78362
16	13	229	16	13	2304.8	78362
17	13	242	17	13	2304.8	78362
18	13	255	18	13	2304.8	78362
19	13	268	19	13	2304.8	85276
20	13	281	20	13	2304.8	85276
21	13	294	21	13	2304.8	85276
22	13	307	22	13	2304.8	85276
23	13	320	23	13	2304.8	94495
24	13	333	24	13	2304.8	94495
25	13	346	25	13	2304.8	94495
26	13	359	26	13	2304.8	94495
27	13	372	27	13	2304.8	94495
28	13	385	28	13	2304.8	94495
29	13	398	29	13	2304.8	94495
30	13	411	30	13	2304.8	106019
31	13	424	31	13	2304.8	106019
32	13	437	32	13	2304.8	106019
33	13	450	33	13	2304.8	106019
34	13	463	34	13	2304.8	106019
35	13	476	35	13	2304.8	106019
36	13	489	36	13	2304.8	106019
37	13	502	37	15.5	2748.0	126407
38	18	520	38	15.5	2748.0	140147
39	13	533	39	6.5	1152.4	58771

Characterizing Vortex Shedding

Assumptions:

(1) Vortex shedding is modeled as an impulsive periodic function

(2) Because vorteces result from the lateral movement of the Prudential Center, and the bottom of the Prudential will not vibrate much in the wind, consider vortex shedding loads on only the top quarter of 111 Huntington.

(3) The magnitude of the impulsive vortex loads will be determined by trial and error to create a top floor acceleration of (0.01 m/s^2, or 0.033 ft/s^2 or .4 in/s^2) in the undamped structure. This target acceleration does not represent the actual response 111 Huntington exhibited in the wind tunnel tests. Rather it is taken as a representative value of unserviceable vibration for movement with a frequency of ~0.2 Hz. This value was chosen based on the experienceand guidelines of the Japanese TSD designers [Tamura *et al*. (1995)]

(4) The appropriate magnitude for an impulse wind load causing a top floor acceleration of 0.4 in/s^2 was determined by applying a modification factor α to the equivalent static wind loads, meaning the time history vortex load is as follows:

$$v_i(t) = \alpha\left[I(t) * P_i\right]$$

where	$v_i(t)$	= vortex shedding load applied at floor i
	α	= modification factor
	$I(t)$	= periodic unit impulse (T=5 sec)
	P_i	= magnitude of equivalent static wind load at floor i

It was determined by trial and error that $\alpha =$ **0.00915**

Preliminary Information

First Two Fundamental Periods of 111 Huntington:

Mode 1:	East-West Translation		
	T=	5.26 s	For simiplicity's sake, I have modelled 111 Huntington as a structure whose fundamental frequency is 5 seconds.
	f=	0.190 Hz	
Mode 2:	North-South Translation		
	T=	5.00 s	
	f=	0.200 Hz	

Total Mass of Building:	124000000 lbs
Assume 35% of building mass participates in 1st Mode (see above):	43400000 lbs

Mass Ratio

Typical Mass Ratio is 1%, so total mass of sloshing water needed: 434000 lbs

Optimal Tuning Frequency

$$\phi\big|_{opt} = \frac{\sqrt{1-0.5\mu}}{1+\mu} \Rightarrow f_D\big|_{opt} = \frac{\sqrt{1-0.5\mu}}{1+\mu} f_S \quad : \quad 0.198 \text{ Hz}$$

Desired Vessel Dimensions

As discussd above, finding space for enough water mass to provide significant inertial damping is a difficult task
However, it is perhaps an architectural problem, to design the vessels such that they need not be hidden
Certainly, a large number could go on the roof, since that is where they are most effective, and they are out of the way
However, it is important to remember that some sort of enclosure must protect the tanks from the elements and keep the water from freezing.
Architecturally, it is desireable to have the vessels be sized to a human scale, so they are not intimidating if in full view
I am choosing cylindrical vessels because of personal aesthetic preference
For this project, consider the following dimensions (which are somewhat arbritrary).

Diameter		3.00 ft
Area		7.07 ft^2
Height	near	5.00 ft

Determining Water Height and Number of Layers

$$f_D = \frac{1}{2\pi}\sqrt{\frac{\lambda g}{D}\tanh\frac{\lambda h}{D}}$$

where λ=3.68

Assuming:

D=	3.00 ft
fD=	0.20 rad/sec
lambda=	1.84
g=	9.81 ft/sec^2

Then:

h = water height	**0.43** ft
Layer height, choose (h*1.5)	**0.64** ft
# of layers	7.81
round off # of layers	**8.00** layers
New vessel height	**5.12** ft

Optimal Damping Ratio

$$\xi_d\Big|_{opt} = \sqrt{\frac{\mu(3-\sqrt{0.5\mu}}{8\ (1+\mu)(1-0.5\mu)}}\ :\qquad 0.06$$

Assume that with experimental assistance, the appropriate amount of floating particles or protrusions necessary to achieve the optimal damping ratio could be determined (if needed at all).

Determining Number and Location of Vessels

Total weight per vessel	1495.90	lbs
Total weight required	434000	lbs
Total number of vessels	290.13	ft^3

In determining location of dampers, minimize the amount in the habitable top ten floors, maximize amount at roof
As seen on the attached drawing, 80 vessels can be comfortably placed on the roof
This requires that 21 be placed on each of the top ten floors which is reasonable since this means only 150 sq ft per floor is required

Modeling as Equivalent Tuned Mass Dampers

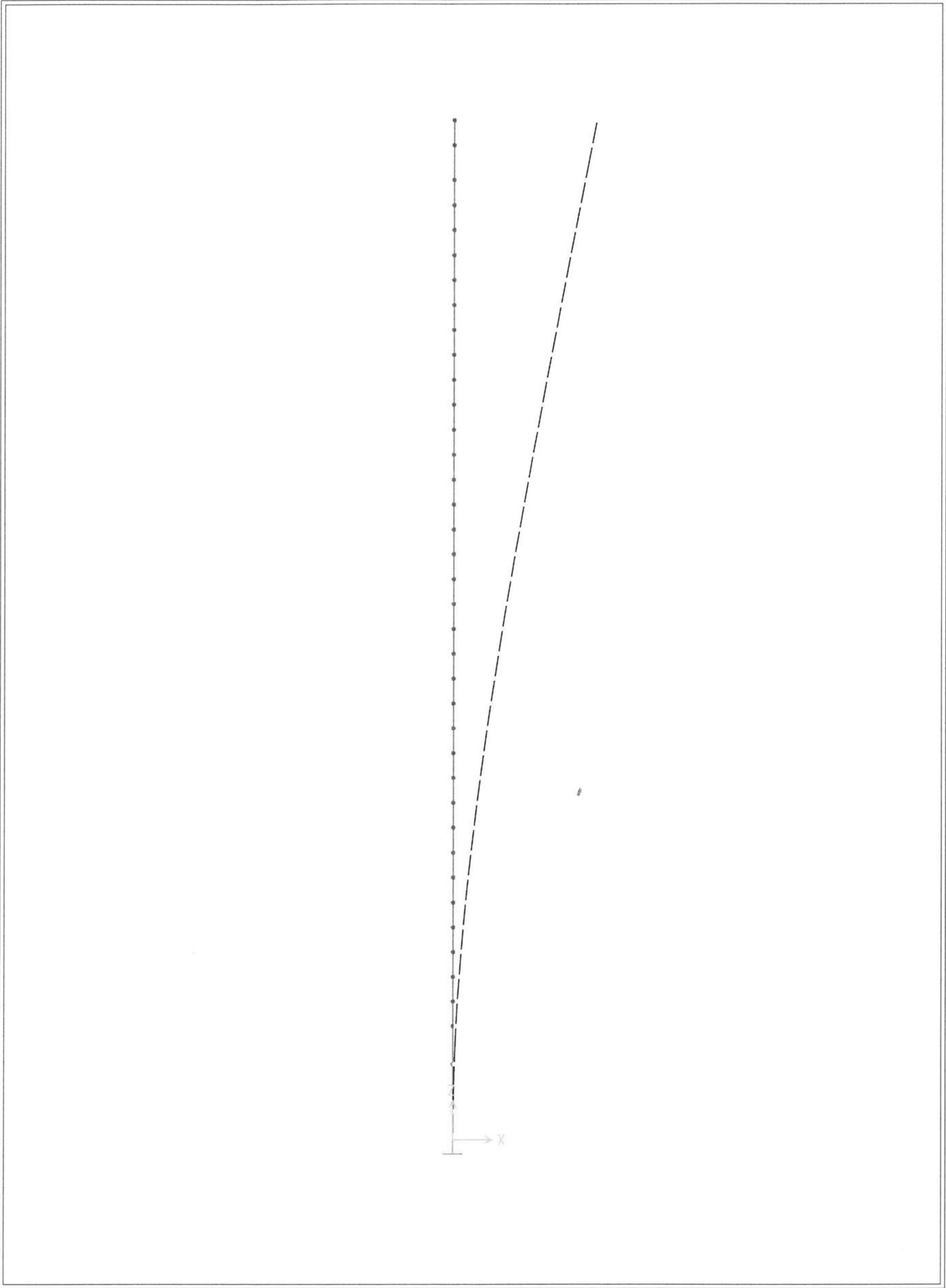
In order to model the structure-damper interacion, mass-spring-damper elements representing equivalent Tuned Mass Dampers will be added to a SAP model based on 111 Huntington. The following determines the appropriate stiffness and damping constants for these members

TSD	TMD
ω_D	ω_D
m_D	m_D
ξ_D	ξ_D
n/a	$k = \omega_D^2 m_D$
n/a	$c = 2\ \omega_D\ m_D\ \xi_D$

Floor #	Vessels per floor	Water mass per vessel (lbs-s^2/ft)	Water mass per floor (lbs-s^2/ft)	Sloshing Ang Frequency (rad/sec)_	Equivalent k (lbs/ft)	Equivalent c (lbs-s/ft)
Roof	80.0	46.5	3716.5	1.24	5724.5	562.1
39	21.0	46.5	975.6	1.24	1502.7	147.6
38	21.0	46.5	975.6	1.24	1502.7	147.6
37	21.0	46.5	975.6	1.24	1502.7	147.6
36	21.0	46.5	975.6	1.24	1502.7	147.6
35	21.0	46.5	975.6	1.24	1502.7	147.6
34	21.0	46.5	975.6	1.24	1502.7	147.6
33	21.0	46.5	975.6	1.24	1502.7	147.6
32	21.0	46.5	975.6	1.24	1502.7	147.6
31	21.0	46.5	975.6	1.24	1502.7	147.6
30	21.0	46.5	975.6	1.24	1502.7	147.6

APPENDIX B: SAP2000 INPUT FILE: UNDAMPED CASE
SAP2000 INPUT FILE: EQUIVALENT TMD CASE

X



SAP2000 v7.40 - File:nodampers - Mode 1 Period 5.0029 seconds - lb-ft Units

```
; File W:\FenwayPark\marc\sapthesis\sap\nodampers.s2k saved 5/11/02 20:46:43 in
lb-ft

SYSTEM
  DOF=UX,UZ,RY  LENGTH=FT  FORCE=LB  PAGE=SECTIONS

JOINT
  1  X=0  Y=0  Z=0
  2  X=0  Y=0  Z=15
  3  X=0  Y=0  Z=40
  4  X=0  Y=0  Z=60
  5  X=0  Y=0  Z=73
  6  X=0  Y=0  Z=86
  7  X=0  Y=0  Z=99
  8  X=0  Y=0  Z=112
  9  X=0  Y=0  Z=125
  10  X=0  Y=0  Z=138
  11  X=0  Y=0  Z=151
  12  X=0  Y=0  Z=164
  13  X=0  Y=0  Z=177
  14  X=0  Y=0  Z=190
  15  X=0  Y=0  Z=203
  16  X=0  Y=0  Z=216
  17  X=0  Y=0  Z=229
  18  X=0  Y=0  Z=242
  19  X=0  Y=0  Z=255
  20  X=0  Y=0  Z=268
  21  X=0  Y=0  Z=281
  22  X=0  Y=0  Z=294
  23  X=0  Y=0  Z=307
  24  X=0  Y=0  Z=320
  25  X=0  Y=0  Z=333
  26  X=0  Y=0  Z=346
  27  X=0  Y=0  Z=359
  28  X=0  Y=0  Z=372
  29  X=0  Y=0  Z=385
  30  X=0  Y=0  Z=398
  31  X=0  Y=0  Z=411
  32  X=0  Y=0  Z=424
  33  X=0  Y=0  Z=437
  34  X=0  Y=0  Z=450
  35  X=0  Y=0  Z=463
  36  X=0  Y=0  Z=476
  37  X=0  Y=0  Z=489
  38  X=0  Y=0  Z=502
  39  X=0  Y=0  Z=520
  40  X=0  Y=0  Z=533

RESTRAINT
  ADD=1  DOF=U1,U2,U3,R1,R2,R3

PATTERN
  NAME=DEFAULT

MASS
  ADD=1  U1=96273  R2=96273
  ADD=2  U1=96273  R2=96273
```

```
    ADD=3  U1=96273  R2=96273
    ADD=4  U1=96273  R2=96273
    ADD=5  U1=96273  R2=96273
    ADD=6  U1=96273  R2=96273
    ADD=7  U1=96273  R2=96273
    ADD=8  U1=96273  R2=96273
    ADD=9  U1=96273  R2=96273
    ADD=10  U1=96273  R2=96273
    ADD=11  U1=96273  R2=96273
    ADD=12  U1=96273  R2=96273
    ADD=13  U1=96273  R2=96273
    ADD=14  U1=96273  R2=96273
    ADD=15  U1=96273  R2=96273
    ADD=16  U1=96273  R2=96273
    ADD=17  U1=96273  R2=96273
    ADD=18  U1=96273  R2=96273
    ADD=19  U1=96273  R2=96273
    ADD=20  U1=96273  R2=96273
    ADD=21  U1=96273  R2=96273
    ADD=22  U1=96273  R2=96273
    ADD=23  U1=96273  R2=96273
    ADD=24  U1=96273  R2=96273
    ADD=25  U1=96273  R2=96273
    ADD=26  U1=96273  R2=96273
    ADD=27  U1=96273  R2=96273
    ADD=28  U1=96273  R2=96273
    ADD=29  U1=96273  R2=96273
    ADD=30  U1=96273  R2=96273
    ADD=31  U1=96273  R2=96273
    ADD=32  U1=96273  R2=96273
    ADD=33  U1=96273  R2=96273
    ADD=34  U1=96273  R2=96273
    ADD=35  U1=96273  R2=96273
    ADD=36  U1=96273  R2=96273
    ADD=37  U1=96273  R2=96273
    ADD=38  U1=96273  R2=96273
    ADD=39  U1=96273  R2=96273
    ADD=40  U1=96273  R2=96273

MATERIAL
  NAME=STEEL  IDES=S  M=7.323999E-04  W=.283
    T=0  E=4.176E+09  U=.3  A=.0000065  FY=36000
  NAME=CONC  IDES=C  M=.0002246  W=8.679999E-02
    T=0  E=3600000  U=.2  A=.0000055
  NAME=OTHER  IDES=N  M=2.246377E-04  W=.0868
    T=0  E=3600000  U=.2  A=.0000055

FRAME SECTION
  NAME=FSEC1 MAT=STEEL SH=R T=18,10 A=180 J=3916.671 I=18650,20000 AS=150,150

FRAME
  1  J=1,2  SEC=FSEC1  NSEG=4  ANG=0
  2  J=2,3  SEC=FSEC1  NSEG=4  ANG=0
  3  J=3,4  SEC=FSEC1  NSEG=4  ANG=0
  4  J=4,5  SEC=FSEC1  NSEG=4  ANG=0
  5  J=5,6  SEC=FSEC1  NSEG=4  ANG=0
  6  J=6,7  SEC=FSEC1  NSEG=4  ANG=0
```

```
    7  J=7,8   SEC=FSEC1   NSEG=4   ANG=0
    8  J=8,9   SEC=FSEC1   NSEG=4   ANG=0
    9  J=9,10   SEC=FSEC1   NSEG=4   ANG=0
    10  J=10,11   SEC=FSEC1   NSEG=4   ANG=0
    11  J=11,12   SEC=FSEC1   NSEG=4   ANG=0
    12  J=12,13   SEC=FSEC1   NSEG=4   ANG=0
    13  J=13,14   SEC=FSEC1   NSEG=4   ANG=0
    14  J=14,15   SEC=FSEC1   NSEG=4   ANG=0
    15  J=15,16   SEC=FSEC1   NSEG=4   ANG=0
    16  J=16,17   SEC=FSEC1   NSEG=4   ANG=0
    17  J=17,18   SEC=FSEC1   NSEG=4   ANG=0
    18  J=18,19   SEC=FSEC1   NSEG=4   ANG=0
    19  J=19,20   SEC=FSEC1   NSEG=4   ANG=0
    20  J=20,21   SEC=FSEC1   NSEG=4   ANG=0
    21  J=21,22   SEC=FSEC1   NSEG=4   ANG=0
    22  J=22,23   SEC=FSEC1   NSEG=4   ANG=0
    23  J=23,24   SEC=FSEC1   NSEG=4   ANG=0
    24  J=24,25   SEC=FSEC1   NSEG=4   ANG=0
    25  J=25,26   SEC=FSEC1   NSEG=4   ANG=0
    26  J=26,27   SEC=FSEC1   NSEG=4   ANG=0
    27  J=27,28   SEC=FSEC1   NSEG=4   ANG=0
    28  J=28,29   SEC=FSEC1   NSEG=4   ANG=0
    29  J=29,30   SEC=FSEC1   NSEG=4   ANG=0
    30  J=30,31   SEC=FSEC1   NSEG=4   ANG=0
    31  J=31,32   SEC=FSEC1   NSEG=4   ANG=0
    32  J=32,33   SEC=FSEC1   NSEG=4   ANG=0
    33  J=33,34   SEC=FSEC1   NSEG=4   ANG=0
    34  J=34,35   SEC=FSEC1   NSEG=4   ANG=0
    35  J=35,36   SEC=FSEC1   NSEG=4   ANG=0
    36  J=36,37   SEC=FSEC1   NSEG=4   ANG=0
    37  J=37,38   SEC=FSEC1   NSEG=4   ANG=0
    38  J=38,39   SEC=FSEC1   NSEG=4   ANG=0
    39  J=39,40   SEC=FSEC1   NSEG=4   ANG=0

LOAD
  NAME=WIND  CSYS=0
    TYPE=FORCE
      ADD=30  UX=94495
      ADD=31  UX=106019
      ADD=32  UX=106019
      ADD=33  UX=106019
      ADD=34  UX=106019
      ADD=35  UX=106019
      ADD=36  UX=106019
      ADD=37  UX=106019
      ADD=38  UX=126407
      ADD=39  UX=140147
      ADD=40  UX=58771

MODE
  TYPE=EIGEN  N=10  TOL=.00001

FUNCTION
  NAME=FUNC2 DT=0 NPL=1 PRINT=Y FILE=wind2.txt

HISTORY
```

```
  NAME=HIST1 TYPE=NON NSTEP=2400 DT=.25 DAMP=.015 FTOL=.00001 ETOL=.00001
  DTMAX=0 ENVE=Y
    LOAD=WIND FUNC=FUNC2 SF=.00915 AT=0
```

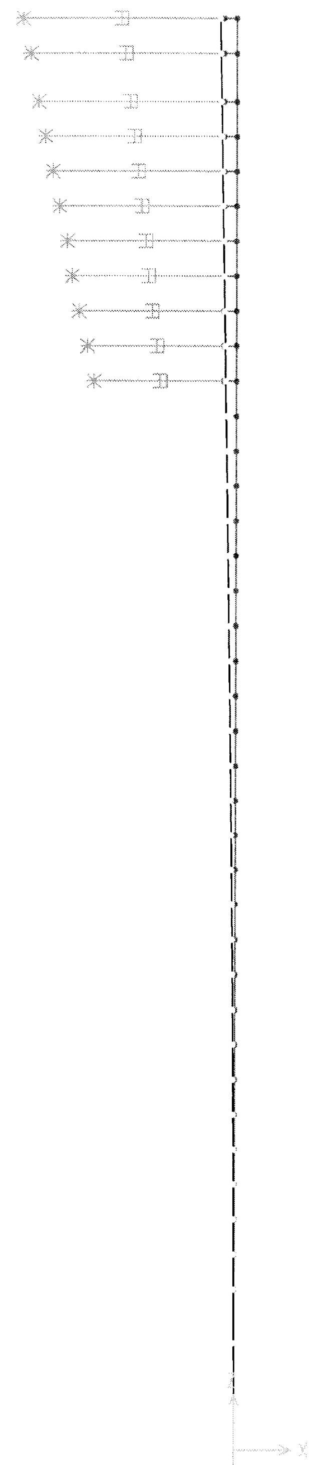
Change to .03 to simulate effect of viscous dampers

```
OUTPUT
; No Output Requested

END

; The following data is used for graphics, design and pushover analysis.
; If changes are made to the analysis data above, then the following data
; should be checked for consistency.
SAP2000 V7.40 SUPPLEMENTAL DATA
  GRID GLOBAL  X "1"  -120
  GRID GLOBAL  X "2"  -72
  GRID GLOBAL  X "3"  -24
  GRID GLOBAL  X "4"  24
  GRID GLOBAL  X "5"  72
  GRID GLOBAL  X "6"  120
  GRID GLOBAL  Y "7"  -120
  GRID GLOBAL  Y "8"  -72
  GRID GLOBAL  Y "9"  -24
  GRID GLOBAL  Y "10"  24
  GRID GLOBAL  Y "11"  72
  GRID GLOBAL  Y "12"  120
  GRID GLOBAL  Z "13"  0
  GRID GLOBAL  Z "14"  48
  GRID GLOBAL  Z "15"  96
  GRID GLOBAL  Z "16"  144
  MATERIAL STEEL FY 36000
  MATERIAL CONC FYREBAR 60000 FYSHEAR 40000 FC 4000 FCSHEAR 4000
  STATICLOAD WIND TYPE DEAD
END SUPPLEMENTAL DATA
```

SAP2000 v7.40 - File:tld - X-Z Plane @ Y=0 - lb-ft Units



SAP2000 v7.40 - File:tld - Mode 1 Period 5.2921 seconds - lb-ft Units

```
; File W:\FenwayPark\marc\sapthesis\sap\tld.s2k saved 5/11/02 20:46:24 in lb-ft

SYSTEM
  DOF=UX,UZ,RY  LENGTH=FT  FORCE=LB  PAGE=SECTIONS

JOINT
  1  X=0  Y=0  Z=0
  2  X=0  Y=0  Z=15
  3  X=0  Y=0  Z=40
  4  X=0  Y=0  Z=60
  5  X=0  Y=0  Z=73
  6  X=0  Y=0  Z=86
  7  X=0  Y=0  Z=99
  8  X=0  Y=0  Z=112
  9  X=0  Y=0  Z=125
  10  X=0  Y=0  Z=138
  11  X=0  Y=0  Z=151
  12  X=0  Y=0  Z=164
  13  X=0  Y=0  Z=177
  14  X=0  Y=0  Z=190
  15  X=0  Y=0  Z=203
  16  X=0  Y=0  Z=216
  17  X=0  Y=0  Z=229
  18  X=0  Y=0  Z=242
  19  X=0  Y=0  Z=255
  20  X=0  Y=0  Z=268
  21  X=0  Y=0  Z=281
  22  X=0  Y=0  Z=294
  23  X=0  Y=0  Z=307
  24  X=0  Y=0  Z=320
  25  X=0  Y=0  Z=333
  26  X=0  Y=0  Z=346
  27  X=0  Y=0  Z=359
  28  X=0  Y=0  Z=372
  29  X=0  Y=0  Z=385
  30  X=0  Y=0  Z=398
  31  X=0  Y=0  Z=411
  32  X=0  Y=0  Z=424
  33  X=0  Y=0  Z=437
  34  X=0  Y=0  Z=450
  35  X=0  Y=0  Z=463
  36  X=0  Y=0  Z=476
  37  X=0  Y=0  Z=489
  38  X=0  Y=0  Z=502
  39  X=0  Y=0  Z=520
  40  X=0  Y=0  Z=533
  3001  X=-5  Y=0  Z=398
  3101  X=-5  Y=0  Z=411
  3201  X=-5  Y=0  Z=424
  3301  X=-5  Y=0  Z=437
  3401  X=-5  Y=0  Z=450
  3501  X=-5  Y=0  Z=463
  3601  X=-5  Y=0  Z=476
  3701  X=-5  Y=0  Z=489
  3801  X=-5  Y=0  Z=502
```

```
  3901  X=-5  Y=0  Z=520
  4001  X=-5  Y=0  Z=533

RESTRAINT
  ADD=1  DOF=U1,U2,U3,R1,R2,R3
  ADD=3001  DOF=U2,U3,R1,R2,R3
  ADD=3101  DOF=U2,U3,R1,R2,R3
  ADD=3201  DOF=U2,U3,R1,R2,R3
  ADD=3301  DOF=U2,U3,R1,R2,R3
  ADD=3401  DOF=U2,U3,R1,R2,R3
  ADD=3501  DOF=U2,U3,R1,R2,R3
  ADD=3601  DOF=U2,U3,R1,R2,R3
  ADD=3701  DOF=U2,U3,R1,R2,R3
  ADD=3801  DOF=U2,U3,R1,R2,R3
  ADD=3901  DOF=U2,U3,R1,R2,R3
  ADD=4001  DOF=U2,U3,R1,R2,R3

PATTERN
  NAME=DEFAULT

MASS
  ADD=1  U1=96273  R2=96273
  ADD=2  U1=96273  R2=96273
  ADD=3  U1=96273  R2=96273
  ADD=4  U1=96273  R2=96273
  ADD=5  U1=96273  R2=96273
  ADD=6  U1=96273  R2=96273
  ADD=7  U1=96273  R2=96273
  ADD=8  U1=96273  R2=96273
  ADD=9  U1=96273  R2=96273
  ADD=10  U1=96273  R2=96273
  ADD=11  U1=96273  R2=96273
  ADD=12  U1=96273  R2=96273
  ADD=13  U1=96273  R2=96273
  ADD=14  U1=96273  R2=96273
  ADD=15  U1=96273  R2=96273
  ADD=16  U1=96273  R2=96273
  ADD=17  U1=96273  R2=96273
  ADD=18  U1=96273  R2=96273
  ADD=19  U1=96273  R2=96273
  ADD=20  U1=96273  R2=96273
  ADD=21  U1=96273  R2=96273
  ADD=22  U1=96273  R2=96273
  ADD=23  U1=96273  R2=96273
  ADD=24  U1=96273  R2=96273
  ADD=25  U1=96273  R2=96273
  ADD=26  U1=96273  R2=96273
  ADD=27  U1=96273  R2=96273
  ADD=28  U1=96273  R2=96273
  ADD=29  U1=96273  R2=96273
  ADD=30  U1=96273  R2=96273
  ADD=31  U1=96273  R2=96273
  ADD=32  U1=96273  R2=96273
  ADD=33  U1=96273  R2=96273
  ADD=34  U1=96273  R2=96273
```

```
  ADD=35  U1=96273  R2=96273
  ADD=36  U1=96273  R2=96273
  ADD=37  U1=96273  R2=96273
  ADD=38  U1=96273  R2=96273
  ADD=39  U1=96273  R2=96273
  ADD=40  U1=96273  R2=96273
  ADD=3001  U1=975.6
  ADD=3101  U1=975.6
  ADD=3201  U1=975.6
  ADD=3301  U1=975.6
  ADD=3401  U1=975.6
  ADD=3501  U1=975.6
  ADD=3601  U1=975.6
  ADD=3701  U1=975.6
  ADD=3801  U1=975.6
  ADD=3901  U1=975.6
  ADD=4001  U1=3716.5

MATERIAL
  NAME=STEEL  IDES=S  M=7.323999E-04  W=.283
    T=0  E=4.176E+09  U=.3  A=.0000065  FY=36000
  NAME=CONC  IDES=C  M=.0002246  W=8.679999E-02
    T=0  E=3600000  U=.2  A=.0000055
  NAME=OTHER  IDES=N  M=2.246377E-04  W=.0868
    T=0  E=3600000  U=.2  A=.0000055

FRAME SECTION
  NAME=FSEC1 MAT=STEEL SH=R T=18,10 A=180 J=3916.671 I=18650,20000 AS=150,150

NLPROP
  NAME=D40    TYPE=Damper
    DOF=U1  KE=0  CE=562.1
  NAME=S40    TYPE=Damper
    DOF=U1  KE=5724.5  CE=0
  NAME=DREST    TYPE=Damper
    DOF=U1  KE=0  CE=147.6
  NAME=SREST    TYPE=Damper
    DOF=U1  KE=1502.7  CE=0

FRAME
  1  J=1,2  SEC=FSEC1  NSEG=4  ANG=0
  2  J=2,3  SEC=FSEC1  NSEG=4  ANG=0
  3  J=3,4  SEC=FSEC1  NSEG=4  ANG=0
  4  J=4,5  SEC=FSEC1  NSEG=4  ANG=0
  5  J=5,6  SEC=FSEC1  NSEG=4  ANG=0
  6  J=6,7  SEC=FSEC1  NSEG=4  ANG=0
  7  J=7,8  SEC=FSEC1  NSEG=4  ANG=0
  8  J=8,9  SEC=FSEC1  NSEG=4  ANG=0
  9  J=9,10  SEC=FSEC1  NSEG=4  ANG=0
  10  J=10,11  SEC=FSEC1  NSEG=4  ANG=0
  11  J=11,12  SEC=FSEC1  NSEG=4  ANG=0
  12  J=12,13  SEC=FSEC1  NSEG=4  ANG=0
  13  J=13,14  SEC=FSEC1  NSEG=4  ANG=0
  14  J=14,15  SEC=FSEC1  NSEG=4  ANG=0
  15  J=15,16  SEC=FSEC1  NSEG=4  ANG=0
```

```
   16   J=16,17   SEC=FSEC1   NSEG=4   ANG=0
   17   J=17,18   SEC=FSEC1   NSEG=4   ANG=0
   18   J=18,19   SEC=FSEC1   NSEG=4   ANG=0
   19   J=19,20   SEC=FSEC1   NSEG=4   ANG=0
   20   J=20,21   SEC=FSEC1   NSEG=4   ANG=0
   21   J=21,22   SEC=FSEC1   NSEG=4   ANG=0
   22   J=22,23   SEC=FSEC1   NSEG=4   ANG=0
   23   J=23,24   SEC=FSEC1   NSEG=4   ANG=0
   24   J=24,25   SEC=FSEC1   NSEG=4   ANG=0
   25   J=25,26   SEC=FSEC1   NSEG=4   ANG=0
   26   J=26,27   SEC=FSEC1   NSEG=4   ANG=0
   27   J=27,28   SEC=FSEC1   NSEG=4   ANG=0
   28   J=28,29   SEC=FSEC1   NSEG=4   ANG=0
   29   J=29,30   SEC=FSEC1   NSEG=4   ANG=0
   30   J=30,31   SEC=FSEC1   NSEG=4   ANG=0
   31   J=31,32   SEC=FSEC1   NSEG=4   ANG=0
   32   J=32,33   SEC=FSEC1   NSEG=4   ANG=0
   33   J=33,34   SEC=FSEC1   NSEG=4   ANG=0
   34   J=34,35   SEC=FSEC1   NSEG=4   ANG=0
   35   J=35,36   SEC=FSEC1   NSEG=4   ANG=0
   36   J=36,37   SEC=FSEC1   NSEG=4   ANG=0
   37   J=37,38   SEC=FSEC1   NSEG=4   ANG=0
   38   J=38,39   SEC=FSEC1   NSEG=4   ANG=0
   39   J=39,40   SEC=FSEC1   NSEG=4   ANG=0

NLLINK
   3001   J=30,3001   NLP=DREST   ANG=0
   3002   J=30,3001   NLP=SREST   ANG=0
   3101   J=31,3101   NLP=DREST   ANG=0
   3102   J=31,3101   NLP=SREST   ANG=0
   3201   J=32,3201   NLP=DREST   ANG=0
   3202   J=32,3201   NLP=SREST   ANG=0
   3301   J=33,3301   NLP=DREST   ANG=0
   3302   J=33,3301   NLP=SREST   ANG=0
   3401   J=34,3401   NLP=DREST   ANG=0
   3402   J=34,3401   NLP=SREST   ANG=0
   3501   J=35,3501   NLP=DREST   ANG=0
   3502   J=35,3501   NLP=SREST   ANG=0
   3601   J=36,3601   NLP=DREST   ANG=0
   3602   J=36,3601   NLP=SREST   ANG=0
   3701   J=37,3701   NLP=DREST   ANG=0
   3702   J=37,3701   NLP=SREST   ANG=0
   3801   J=38,3801   NLP=DREST   ANG=0
   3802   J=38,3801   NLP=SREST   ANG=0
   3901   J=39,3901   NLP=DREST   ANG=0
   3902   J=39,3901   NLP=SREST   ANG=0
   4001   J=40,4001   NLP=D40   ANG=0
   4002   J=40,4001   NLP=S40   ANG=0

LOAD
  NAME=WIND  CSYS=0
    TYPE=FORCE
      ADD=30  UX=94495
      ADD=31  UX=106019
      ADD=32  UX=106019
```

```
       ADD=33  UX=106019
       ADD=34  UX=106019
       ADD=35  UX=106019
       ADD=36  UX=106019
       ADD=37  UX=106019
       ADD=38  UX=126407
       ADD=39  UX=140147
       ADD=40  UX=58771

MODE
  TYPE=EIGEN  N=1  TOL=.00001

FUNCTION
  NAME=FUNC2 NPL=1 PRINT=Y FILE=wind2.txt

HISTORY
  NAME=HIST1 TYPE=NON NSTEP=2400 DT=.25 DAMP=.015 FTOL=.00001 ETOL=.00001
DTMAX=0 ENVE=Y
    LOAD=WIND FUNC=FUNC2 SF=.00915 AT=0

OUTPUT
; No Output Requested

END

; The following data is used for graphics, design and pushover analysis.
; If changes are made to the analysis data above, then the following data
; should be checked for consistency.
SAP2000 V7.40 SUPPLEMENTAL DATA
  GRID GLOBAL  X "1"  -120
  GRID GLOBAL  X "2"  -72
  GRID GLOBAL  X "3"  -24
  GRID GLOBAL  X "4"  24
  GRID GLOBAL  X "5"  72
  GRID GLOBAL  X "6"  120
  GRID GLOBAL  Y "7"  -120
  GRID GLOBAL  Y "8"  -72
  GRID GLOBAL  Y "9"  -24
  GRID GLOBAL  Y "10"  24
  GRID GLOBAL  Y "11"  72
  GRID GLOBAL  Y "12"  120
  GRID GLOBAL  Z "13"  0
  GRID GLOBAL  Z "14"  48
  GRID GLOBAL  Z "15"  96
  GRID GLOBAL  Z "16"  144
  MATERIAL STEEL FY 36000
  MATERIAL CONC FYREBAR 60000 FYSHEAR 40000 FC 4000 FCSHEAR 4000
  STATICLOAD WIND TYPE DEAD
END SUPPLEMENTAL DATA
```