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TUNED MASS DAMPER EFFECTS ON THE DYNAMIC RESPONSE OF PEDESTRIAN SUSPENSION BRIDGES

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ABSTRACT: In many parts of the world, rural communities are separated from basic needs by streams, rivers and frequently flooded pedestrian routes. Low cost and lightweight, pedestrian suspension bridges are a solution to provide a safe travel route. Pedestrian suspension bridges are typically low mass, low stiffness, and marginally damped, resulting in the structure being susceptible to a significant dynamic response to pedestrian loading. Pedestrians often walk at a pace corresponding to suspension bridge modal frequencies, causing bridge resonance and pedestrian discomfort and unease due to the high accelerations, velocities, and displacements. A pedestrian suspension bridge possesses several modal frequencies in the vertical plane, the lateral plane, and combinations of the two. The present parametric study was completed utilizing numerical simulations to determine the dynamic response of selected pedestrian suspension bridges focusing on the influence of tuned mass dampers on the dynamic response. Four geometries and three mass ratios of tuned mass dampers were evaluated for effectiveness. The present study determined that: 1) lateral tuned mass dampers control the vertical dynamic response more effectively than vertical dampers; and 2) vertical dampers control the lateral dynamic response more effectively than lateral dampers.

KEYWORDS: Dynamics; Pedestrian Suspension Bridges; Tuned Mass Dampers.

1 INTRODUCTION

A major pedestrian suspension bridge design challenge is to achieve acceptable serviceability levels, particularly to meet established human comfort levels. Meeting serviceability limits are difficult due to the inherent low stiffness, mass, and damping of pedestrian suspension bridges. Low mass, stiffness, and damping result in large dynamic responses to pedestrian loading. In addition, pedestrian suspension bridges exhibit low modal frequencies that are commonly near pedestrian walking frequencies. Pedestrians can easily excite modal frequencies that will cause the bridge to exceed serviceability limits.

It is widely recognized that modern, slender pedestrian suspension bridges are more susceptible to serviceability failures rather than safety or strength problems [12]. When the walking frequency of pedestrian loading is at or near the pedestrian suspension bridge modal frequencies, serviceability problems typically occur. Typical walking force frequencies in the vertical direction are approximately 2.0 Hz and approximately 1.0 Hz in the lateral direction. The first vertical modal frequency of a typical pedestrian suspension bridge is approximately 0.7 Hz and the first lateral modal frequency is approximately 0.3 Hz [21]. Where a governing design code applies, the bridge modal frequency for each direction must exceed the typical pedestrian stride frequency. This criteria limits the occurrence of a pedestrian exciting any of the bridge modal frequencies. However, where a significant number of pedestrian suspension bridges are constructed, there is no code to govern minimum modal frequencies. Bridges to Prosperity (B2P) pedestrian suspension bridge [3] first vertical modal frequency is in the range of 0.45 Hz to 0.6 Hz, depending on span length. The B2P bridge first lateral modal frequency ranges from 0.28 Hz to 0.73 Hz. These frequencies, and those of other commonly constructed pedestrian suspension bridges, are low modal frequencies, and are likely to respond with large accelerations, velocities, or displacements due to pedestrian loads.

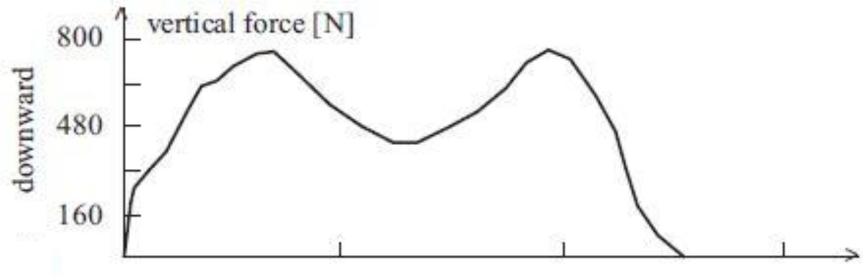
In the present study, tuned mass damper (TMD) effectiveness in mitigating pedestrian suspension bridge dynamic response is evaluated. Two types of tuned mass dampers are evaluated: 1) vertical TMDs; and 2) lateral pendulum-type TMDs. Pedestrian suspension bridge numerical models have been utilized to conduct a parametric study of 40 m, 50 m, 60 m, 70 m, and 80 m bridge spans with several scenarios of damper type and location. The dynamic responses analyzed from the numerical simulations include vertical acceleration, lateral acceleration, vertical velocity, and vertical displacement.

2 PEDESTRIAN WALKING EFFECTS

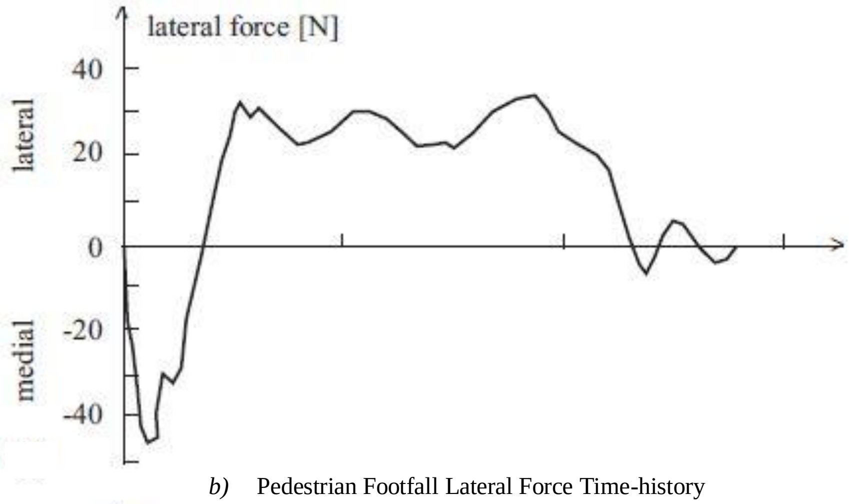
Pedestrians apply footfall forces in the vertical, horizontal, and longitudinal directions as they walk. Vertical forces are largest and, therefore, have been most extensively studied. Lateral and longitudinal footfall forces have not been extensively studied, however, sufficient data is available to include the effect in numerical simulations.

Andriacchi [1] used a force plate to measure single-step footfall forces and reported force time histories as presented in Figure 1. Figure 1a) presents a peak force of approximately 800 N in the vertical direction compared to Figure 1b) where the lateral direction force is approximately 46 N. It can be expected that an increase in walking speed will result in higher peak forces and less contact time [24].

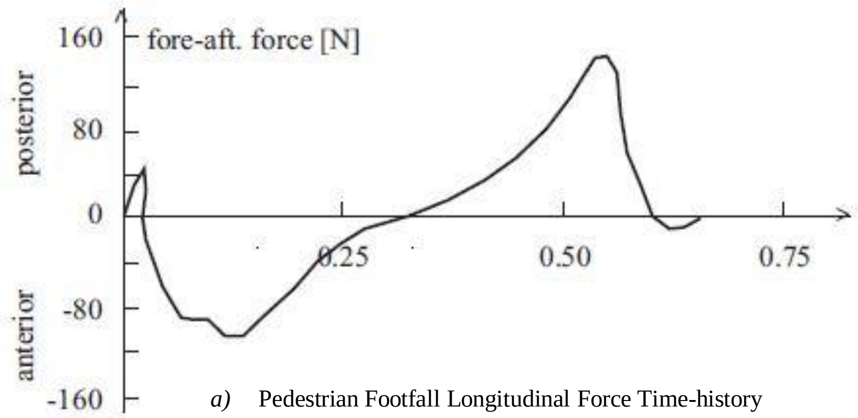
In another extensive pedestrian walking investigation, 505 people were sampled by Matsumoto [15]. Matsumoto [15] concluded that the average walking stride frequency was 2.0 Hz with a standard deviation of 0.173 Hz. A separate investigation observed a walking stride frequency range of 1.6 Hz to 2.4 Hz [2].



c) Pedestrian Footfall Vertical Force Time-history



b) Pedestrian Footfall Lateral Force Time-history



a) Pedestrian Footfall Longitudinal Force Time-history

Figure 1. Typical walking forces (Zivanovic [24] after Andriacchi [1])

Zivanovic [24] observed that the presence of a stationary pedestrian on a structure influences the structure dynamic response through the addition of mass as well as contributes to the overall damping.

Lateral synchronization can also occur where pedestrians widen the distance between left and right footfalls and adjust stride frequency to the bridge frequency in an attempt to maintain balance, increasing lateral force [12] [24]. Lateral synchronization can be significant because pedestrians tend to stride and sway in the same direction as the bridge movement.

3 PARAMETRIC STUDY

To evaluate the range of pedestrian walking effects on the dynamic response of pedestrian suspension bridges with TMDs, a parametric study was designed with the objective of understanding a range of bridge spans, damper types, and mass ratios. The parametric study matrix resulted in sixty-five numerical simulation models including five control models and sixty TMD models. Table 1 presents the different parameter values evaluated.

Table 1. Parameters and Values Included in Simulations

Parameter	Values Evaluated
Bridge Length	40 m, 50 m, 60 m, 70 m, 80 m
TMD Type	Vertical (V), Lateral (L)
Number of TMDs	1L, 2V, 2L and 3V
TMD Mass Ratio	5%, 10%, and 15%

The four damper arrangements: 1) two vertical TMDs (2V); 2) three vertical TMDs (3V); 3) one lateral TMD (1L); and 4) two lateral TMDs (2L) were selected so as to locate TMDs at each antinode of the first four mode shapes to maximize effectiveness. The 2V TMD arrangement is tuned to the mode VA1 where the two antinodes are located at the ¼ and ¾ points of the span as shown in Figure 2. The 3V TMD arrangement is tuned to mode VS2 where the three antinodes are located at center span, ⅛ span, and ⅞ span as shown in Figure 3.

The 1L TMD arrangement is tuned to mode LS1 where the antinode is located at center span as shown in Figure 4. The 2L TMD arrangement is tuned to LA1 where the two antinodes are located at the ¼ and ¾ points of the span as shown in Figure 5. The mass, stiffness, and damping values assigned to each damper are shown in Table 3. The total mass of all the dampers in the arrangement is equal to the defined mass ratio. These values were calculated using Equations (1) through (11), as detailed in a later section, and the modal frequencies presented in Table 2.

The parametric study analyzed the self-weight load case, conducted a modal analysis, and a nonlinear, direct-integration, time-history analysis for all sixty-five models. Modal analysis determined frequencies for the five mode shapes of interest. The time-history analysis determined displacements, velocities, and accelerations.

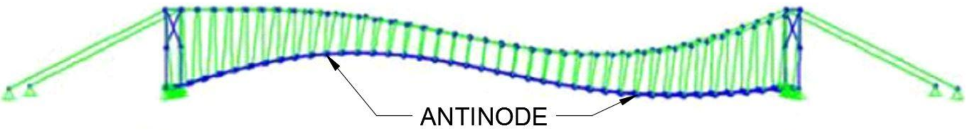


Figure 2. VA1 Antinodes for Elevation View

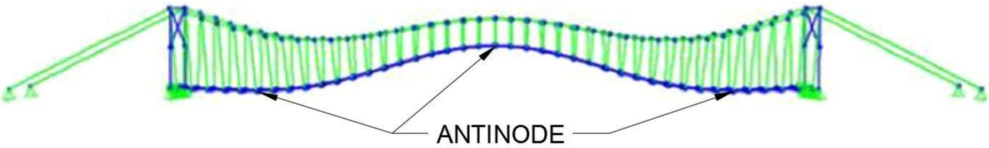


Figure 3. VS2 Antinodes for Elevation View

Figure 4. LS1 Antinodes for Elevation View

Figure 5. LA1 Antinodes for elevation view

Table 2. Modal frequencies for models with no dampers

Mode Shape	40 m Model	50 m Model	60 m Model	70 m Model	80 m Model
VA1	0.60	0.53	0.49	0.45	0.45
VS2	0.99	0.86	0.78	0.71	0.66
LS1	0.73	0.53	0.40	0.31	0.28
LA1	1.39	1.22	1.13	1.04	1.03
TS1	0.84	0.75	0.70	0.63	0.64

Table 3. TMD mass, stiffness, and damping for each simulation

Model Name	Mass (kg)	Stiffness (N/m)	Damping (N-s/m)	Model Name	Mass (kg)	Stiffness (N/m)	Damping (N-s/m)
40-V-2-5	105.5	1427	13.8	60-L-1-5	329.0	1932	175.0
40-V-2-10	211.1	2721	36.8	60-L-1-10	657.9	3606	469.9
40-V-2-15	316.6	3893	63.8	60-L-1-15	986.9	5067	821.6
40-V-3-5	70.4	2684	21.5	60-L-2-5	164.5	1001	63.6
40-V-3-10	140.7	5051	56.0	60-L-2-10	329.0	1932	175.0
40-V-3-15	211.1	7407	100.3	60-L-2-15	493.4	2798	313.2
40-L-1-5	211.1	4128	204.9	70-V-2-5	213.8	1616	15.7
40-L-1-10	422.1	7705	550.2	70-V-2-10	427.6	3079	41.7
40-L-1-15	633.2	10827	962	70-V-2-15	641.3	4407	72.2

Model Name	Mass (kg)	Stiffness (N/m)	Damping (N-s/m)	Model Name	Mass (kg)	Stiffness (N/m)	Damping (N-s/m)
40-L-2-5	105.5	2139	74.4	70-V-3-5	142.5	2797	22.4
40-L-2-10	211.1	4127	204.9	70-V-3-10	285.0	5263	58.4
40-L-2-15	316.6	5979	366.7	70-V-3-15	427.6	7718	104.5
50-V-2-5	137.9	1461	14.2	70-L-1-5	427.6	1508	176.3
50-V-2-10	275.8	2785	37.7	70-L-1-10	855.1	2815	473.4
50-V-2-15	413.7	3985	65.3	70-L-1-15	1282.7	3956	827.6
50-V-3-5	91.9	2647	21.2	70-L-2-5	213.8	782	64.0
50-V-3-10	183.9	4981	55.2	70-L-2-10	427.6	1508	176.3
50-V-3-15	275.8	7304	98.9	70-L-2-15	641.3	2185	315.5
50-L-1-5	275.8	2844	194.4	80-V-2-5	235.5	1797	17.4
50-L-1-10	551.6	5308	522.1	80-V-2-10	470.9	3425	46.4
50-L-1-15	827.4	7459	912.7	80-V-2-15	706.4	4902	80.3
50-L-2-5	137.9	1473	70.6	80-V-3-5	157.0	2662	21.4
50-L-2-10	275.8	2844	194.4	80-V-3-10	313.9	5009	55.5
50-L-2-15	413.7	4119	347.9	80-V-3-15	470.9	7345	99.5
60-V-2-5	164.5	1472	14.3	80-L-1-5	470.9	1355	175.4
60-V-2-10	329.0	2805	38.0	80-L-1-10	941.8	2530	470.9
60-V-2-15	493.4	4014	65.8	80-L-1-15	1412.7	3554	823.3
60-V-3-5	109.7	2597	20.8	80-L-2-5	325.5	702	63.7
60-V-3-10	219.3	4887	54.2	80-L-2-10	470.9	1355	175.4
60-V-3-15	329.0	7166	97.0	80-L-2-15	706.4	1963	313.8

4 PEDESTRIAN MOTION TOLERANCE LEVELS

To evaluate the effectiveness of the TMD arrangements proposed in the present study, it is necessary to establish acceptable dynamic response limits. Many studies, standards, and codes have proposed scales or limits of human vibration and motion tolerance, however, the studies tend to focus on the perceptions of a stationary rather than a moving human subject. This is a significant distinction for establishing motion limits for pedestrian bridges because humans are more tolerant of vibration when in motion themselves and are also more tolerant when they expect a structure to move [24]. For the purposes of the present study the following limits are established on the basis of the available literature:

- European Committee for Standardization [7] human tolerance levels for vertical acceleration are adopted and targeted to 700 mm/s^2.
- Obata [17] maximum velocity of 10 mm/s for human tolerance on a pedestrian suspension bridge.
- Heinemeyer [11] medium comfort level for lateral accelerations is limited at 300 mm/s^2.
- Nakamura [16] 45 mm serviceability limit for lateral displacement.

5 TUNED MASS DAMPERS

TMDs are spring-mass or spring-mass-damper systems that are incorporated into a structure to reduce dynamic response. A TMD is a passive device only effective over a small range of frequencies near the frequency it was tuned to damp. For the present study, the vertical TMDs were designed to respond to the first two vertical modes as determined by modal analyses, VA1 and VS2. The lateral, pendulum TMDs were designed to respond to the first two lateral modes, LS1 and LA1.

The present study adopted the TMD optimization methods proposed by Hartog [10] as follows:

$$f_{opt} = \frac{1}{1+\mu} \tag{1}$$

$$\zeta_{d_{opt}} = \Omega\sqrt{\frac{1.5\mu}{(1+\mu)^3}} \tag{2}$$

from which the optimal damping, c_{opt}, and optimal stiffness, k_{opt}, can be determined:

$$k_{opt} = \frac{m(2\pi f)^2}{(1+\mu)^2} \tag{3}$$

$$\zeta_{d_{opt}} = \frac{c_{opt}}{c_c} = \frac{c_{opt}}{2\omega_a m} \tag{4}$$

$$c_{opt} = 2\zeta_{d_{opt}} f_{opt}\Omega m \tag{5}$$

where: μ is the damper mass to the main structure mass ratio, $\frac{m}{M}$,

ω is the harmonic excitation frequency,

Ω is the main mass natural frequency, $\sqrt{\frac{K}{M}}$,

ω_a is the damper mass natural frequency, $\sqrt{\frac{k}{m}}$,

f is the frequency ratio, $\frac{\omega_a}{\Omega}$,

ζ_d is the TMD damping ratio.

Here, optimum values were based on the assumption of an undamped main mass given the dynamic nature of pedestrian suspension bridges.

Optimal parameters for pendulum tuned mass damper, PTMDs, were calculated for the present study on the basis of Gerges and Vickery [9]:

$$c_a = \frac{c_p^{\theta}}{h^2} + c_d \tag{6}$$

$$k_a = \frac{k_p^\theta}{h^2} + k_s \tag{7}$$

Where: c_a is the equivalent translational damping,
k_a is the equivalent translational stiffness,
c_p^θ is the rotational pendulum inherent damping,
k_p^θ is the gravitational stiffness, $m_a a_g z$,
c_d is the damping of the damper,
k_s is the stiffness of the spring,
h is the distance between the pivot point and the spring/damper attachment point.

The damping ratio and frequency of a PTMD with a point mass is:

$$\beta_a = \frac{c_a h^2}{2 m_a L_a^2 \omega_2} \tag{8}$$

where: β_a is the damping ratio for a PTMD,
L_a is the distance between the pivot point and the mass attachment point, and:

$$\omega_2 = \sqrt{\frac{m_a g L_a + k_s h^2}{m_a L^2}} \tag{9}$$

The optimum damping ratio and frequency of a PTMD is:

$$\beta_{a_{opt}} = \sqrt{\frac{\mu + \frac{3\mu^2}{4}}{4 + 6\mu + 2\mu^2}} \tag{10}$$

$$f_{r_{opt}} = \frac{\omega_{2_{opt}}}{\Omega} = \frac{\sqrt{1 + \frac{\mu}{2}}}{1 + \mu} \tag{11}$$

6 NUMERICAL MODEL

Numerical simulation models are based on standard, full-scale, pedestrian suspension bridges provided by B2P Bridge Builder Manual [3] and constructed in SAP2000 (see Figure 6). Tower geometry and tower pipe size vary with each span length. The height of the towers, width of the towers, and deck camber all depend on the span length, the extensive details of which are readily available in the Bridge Builder Manual [3].

Table 4. Global geometric parameters for selected pedestrian suspension bridges

Span (m)	Tower Height (m)	Tower Width (m)	Deck Camber (m)
40	5.0	2.5	1.23
50	6.0	2.8	1.50
60	7.0	3.0	1.77
70	8.0	3.3	2.04
80	8.5	3.4	1.81

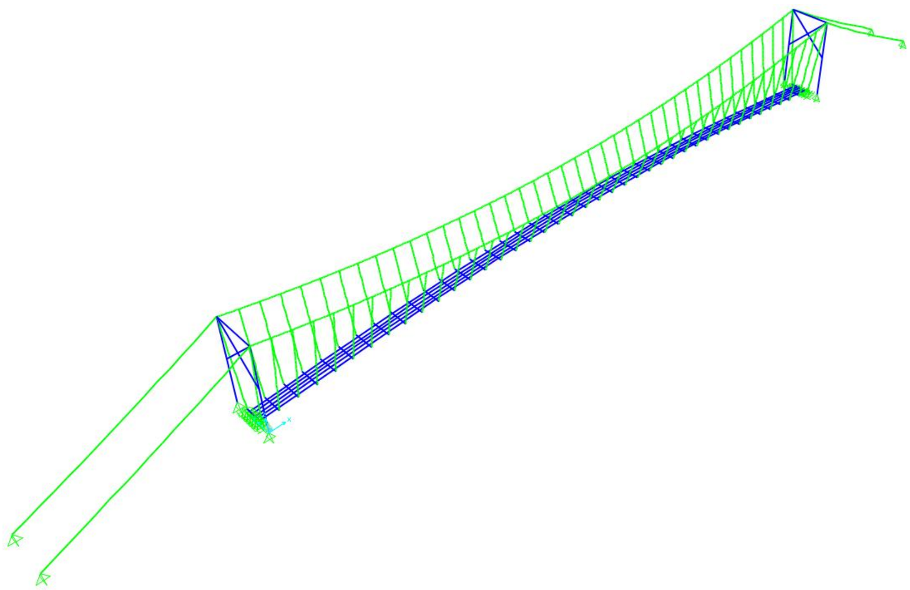


Figure 6. Typical numerical simulation model

6.1 Numerical model design

All numerical models are designed to represent the commonly constructed Bridges to Prosperity bridges for the selected lengths. The main cables and suspenders are cable elements and, therefore, develop tension only. The suspenders are modeled as undeformed cable elements connecting the main suspension cable to the deck crossbeam. The towers, crossbeams, and decking panels are three-dimensional frame elements. The deck nailers are represented as distributed dead load acting along the center of the crossbeam for the length of a standard nailer. Similarly, the deck fence and handrail cable are represented as joint masses at each crossbeam. Figure 7 presents the deck elements and support condition springs at the end of the span. [14]

The boundary conditions for the anchors, the ends of the deck, and the base of the towers are pin or roller connections. The tower pipes are pin connected to the abutment. The deck is supported vertically with rollers at both ends and restrained with 200 kgf/mm (11.2 kip/inch) longitudinal and lateral springs. [14]

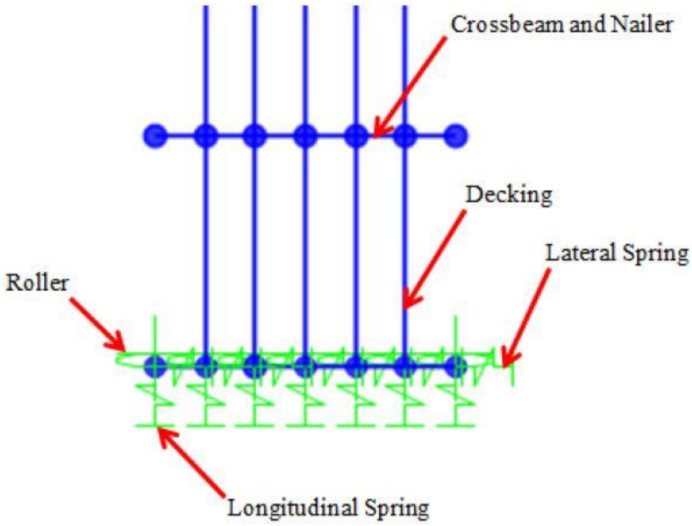


Figure 7. Plan view of deck boundary conditions [14]

6.2 Damper models

Two types of dampers were used: TMDs for vertical dampers and pendulum type TMDs for lateral dampers. The TMDs were modeled as a two-joint linear link where one joint is connected to the center of the crossbeam and the other joint is free. The free joint is defined as 0.5 m (1.64 ft) directly below the center of the crossbeam. The length of the link is arbitrary because the stiffness and damping values of the link are defined per unit length. The link is restricted from rotating in any direction. The stiffness and damping of the link are only defined for the longitudinal direction of the link. The joint mass is assigned to the free end of the link. TMD properties are determined by the equations discussed previously in this paper.

The PTMDs are also modeled as a two-joint linear link with one joint connected to the center of the crossbeam and the other joint free. The free joint is defined as 0.5 m (1.64 ft) directly below the center of the crossbeam, which is also arbitrary because it is just a representation of a pendulum with an equivalent stiffness and damping calculated as show in equations 6 and 7. Stiffness and damping are assigned to allow the equivalent pendulum to swing in any direction (two axes), but not move along its longitudinal axis (third axis).

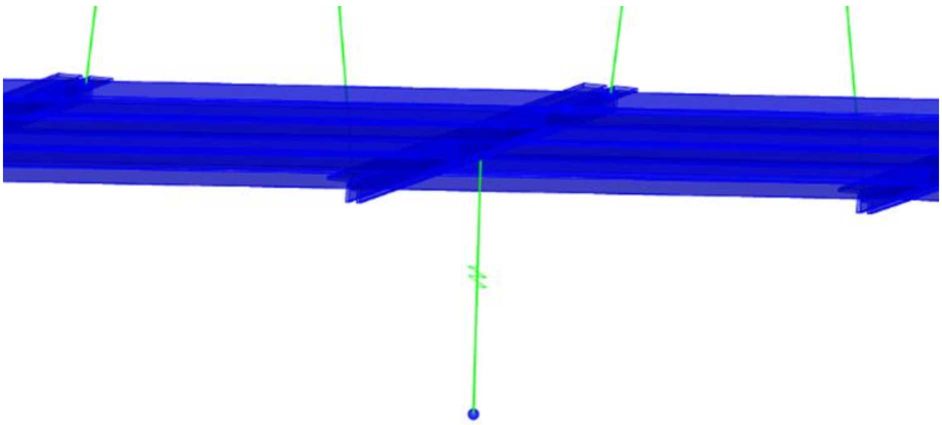


Figure 8. TMD and PTMD modeled in SAP2000

6.3 Pedestrian load simulation

The dynamic pedestrian load is modeled through a time-history load case to simulate a pedestrian in motion with regular footfall forces, both vertically and transversely. A vertical footfall force equal to 81.6 kgf (180 lb) with a transverse force equal to 3.06 kgf (6.7 lb) applied away from the pedestrian center of mass are placed at 0.6 m (1.97 ft) intervals. The transverse forces are applied alternately 10 cm (3.9 inches) from the walking centerline. Footfalls are in contact with the bridge for 0.6 seconds and subsequent steps begin 0.5 seconds after the beginning of the previous footfall to simulate overlap of the applied forces. The simulated footfall loading is applied beginning at the end of the span and continues to midspan.

6.4 Pedestrian walking simulation time-history

Several SAP2000 simulations were conducted to determine the dynamic response of the bridge including dead, modal, and a nonlinear, direct-integration, time-history analysis. A dead load case is defined as nonlinear static to account for the nonlinearity of the cable elements. This case includes the self-weight of all the members, the distributed load of the nailers, and the lumped masses of the fence and railing. The modal analysis starts from the end of the nonlinear dead load analysis and evaluates the mode shapes of the bridge under self-weight, which is required to perform the nonlinear direct-integration time-history analysis. The direct-integration time-history analysis determines the displacements, velocities, and accelerations of the bridge under the pedestrian simulation time-history loading previously detailed. A damping ratio of 1% was used.

7 PARAMETRIC STUDY RESULTS

A total of sixty models, each with unique damper arrangements and mass ratio combinations, were analyzed. The sixty parametric models were compared to five control models, one at each span length. First, modal analyses were run to compare the damper models modal frequencies to the control models, walking frequencies, and recommended frequency ranges. Next, nonlinear, direct-integration, time-history analyses were conducted to determine displacements, velocities, and accelerations. The results of the time-history analyses were compared to the control models and human comfort limits. Five mode shapes were included in the evaluation based on the regularity of occurring in pedestrian suspension bridges as presented in Figure 9. VA1 is the first vertical asymmetrical mode. VS2 is the second symmetrical vertical mode. LS1 is the first symmetrical lateral mode. LA1 is the first asymmetrical lateral mode. TS1 is the first symmetrical torsional mode.

The simulation results include vertical displacement, vertical velocity, vertical acceleration, and lateral accleration. Lateral displacements are not presented because none exceed human comfort limits set at 45 mm. Lateral velocity is also not presented because this motion is not discussed in human comfort limits and the simulations did not reveal significant lateral velocities.

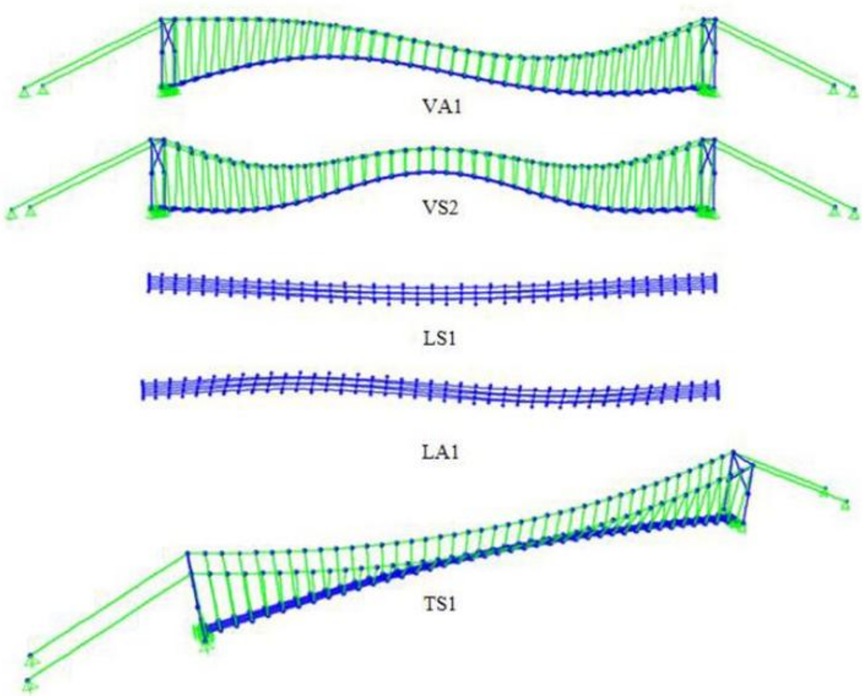


Figure 9. Mode shapes [14]

The time-history simulation data is evaluated for 6 m sections because it takes a pedestrian 5 s to walk 6 m. Evaluating the data in 6 m sections provides a convenient method to evaluate the walker and bystander perceptions. The time-history data is a result of a lone pedestrian walking from the 0 m location to mid-span.

A walker perceives the bridge response in its own proximity while a bystander perceives motion in any section of the bridge at any time during walker travel. The average response experienced by a bystander was calculated by taking the average of all possible responses experienced by a bystander in the second half of the bridge. This is to evaluate how the bridge is responding away from the walker instead of evaluating a number of the same responses the walker feels.

7.1 Discussion of results

The results of the modal analysis and time-history analysis were studied to determine the effect of tuned mass dampers on the modal frequencies and dynamic response.

The results are compared to:

1) control models with no dampers; and
2) human comfort limits.

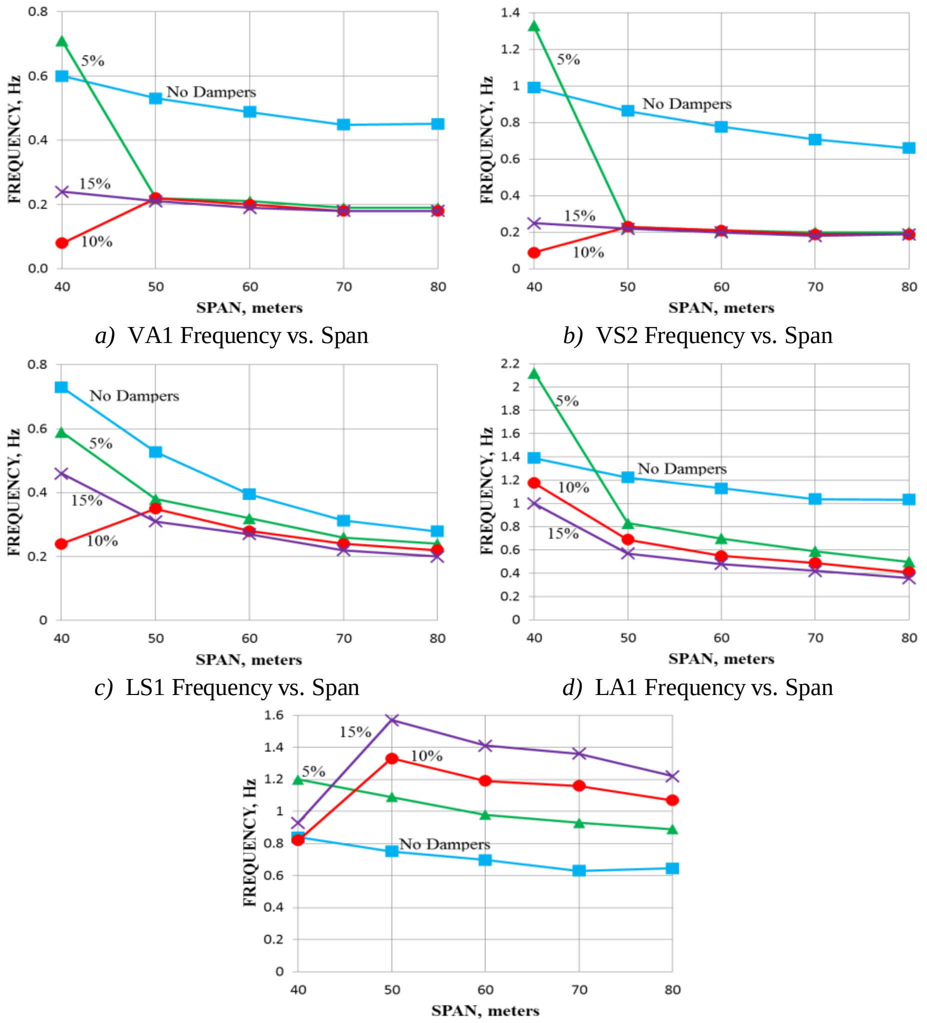
The results are analyzed by damper arrangement:

1) two vertical dampers;
2) three vertical dampers;
3) one lateral damper; and
4) two lateral dampers.

7.2 Two vertical dampers

Modal frequency analysis results for two vertical dampers are presented in Figure 10. VA1 and VS2 frequencies are reduced to approximately 0.2 Hz for all span lengths except the 40 m bridge, and a 50% reduction for the 50 m to 80 m bridges. The 40 m bridge exhibits a wide range of frequencies in the vertical modes. At a 5% mass ratio the vertical frequencies are slightly higher than the control, while 10% and 15% mass ratio results are closer to 0.2 Hz.

It can also be observed that vertical dampers do not significantly affect the LS1 frequencies, but do reduce the LA1 frequencies 15% to 60%. TS1 exhibited a frequency increase above the control for all bridge lengths and mass ratios. Generally the higher mass ratios exhibit larger changes in frequency from the control model and frequencies decrease as the span length increases.



a) VA1 Frequency vs. Span

b) VS2 Frequency vs. Span

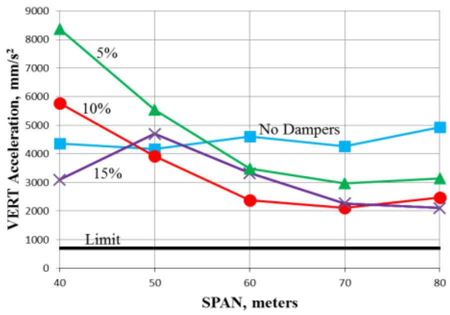
c) LS1 Frequency vs. Span

d) LA1 Frequency vs. Span

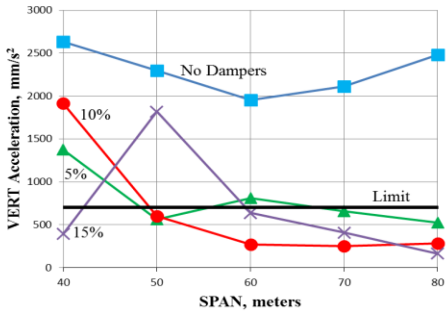
e) TS1 Frequency vs. Span

Figure 10. Modal frequencies with two vertical dampers

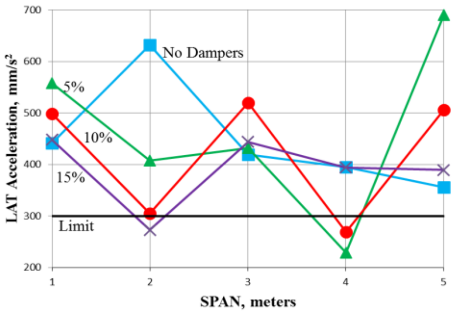
Time-history results are presented for two vertical dampers in Figure 11. Vertical accelerations experienced by the walker are not decreased by vertical dampers for the 40 m or 50 m bridges but are decreased for the 60 and 80 m. A 10% mass ratio performs best for 60 m and 70 m bridges, reducing accelerations by 48%. A 15% mass ratio performs best for the 80 m bridge, however, the acceleration remains three times the comfort limit. Dampers are much more successful for all bridge lengths in reducing the vertical acceleration response experienced by the bystander. At least one mass ratio is successful in lowering the acceleration below the comfort limit for every span length.



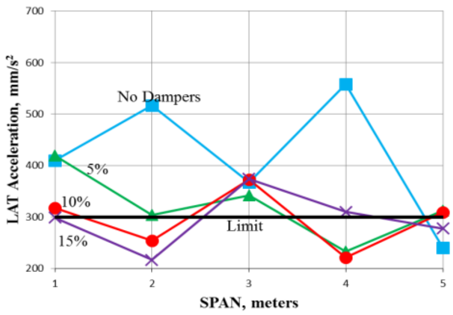
a) Walker Vert Acceleration vs. Span



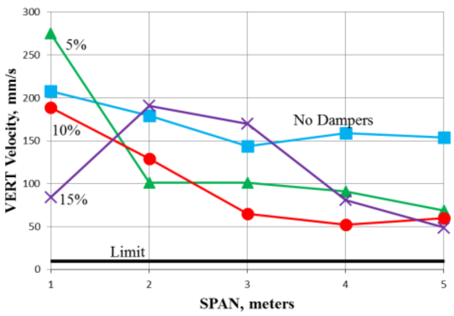
b) Bystander Vert Acceleration vs. Span



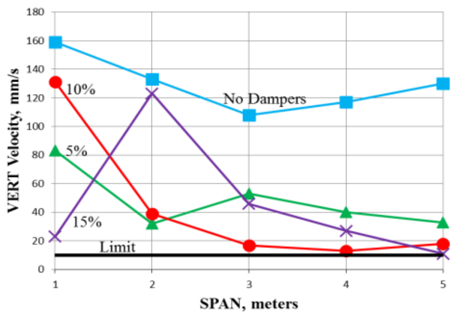
c) Walker Lat Acceleration vs. Span



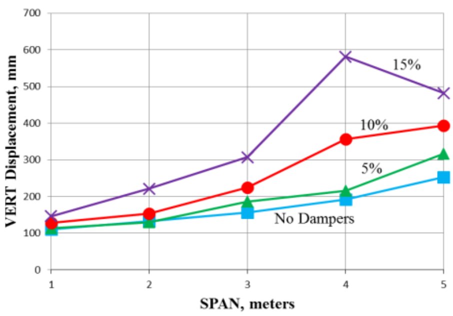
d) Bystander Lat Acceleration vs. Span



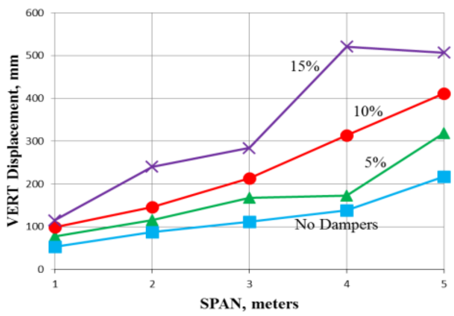
e) Walker Vert Velocity vs. Span



f) Bystander Vert Velocity vs. Span



g) Walker Vert Displacement vs. Span



h) Bystander Vert Displacement vs. Span

Figure 11. Dynamic response of models with two vertical dampers

Vertical dampers reduce lateral accelerations for the 50 m and 70 m bridges with one mass ratio below the comfort limit. A 5% or 10% mass ratio is most successful in reducing frequencies for the 70 m bridge while a 15% mass ratio improves the 50 m bridge.

Vertical velocities are lower than the control bridge in most cases and decrease as the bridge length increases. A 15% mass ratio is the most effective for 40 m and 80 m bridges.

Vertical velocities are not reduced below the comfort limit for either walker or bystander. The most effective configurations reduce walker experienced velocities by 67% and bystander by 92%.

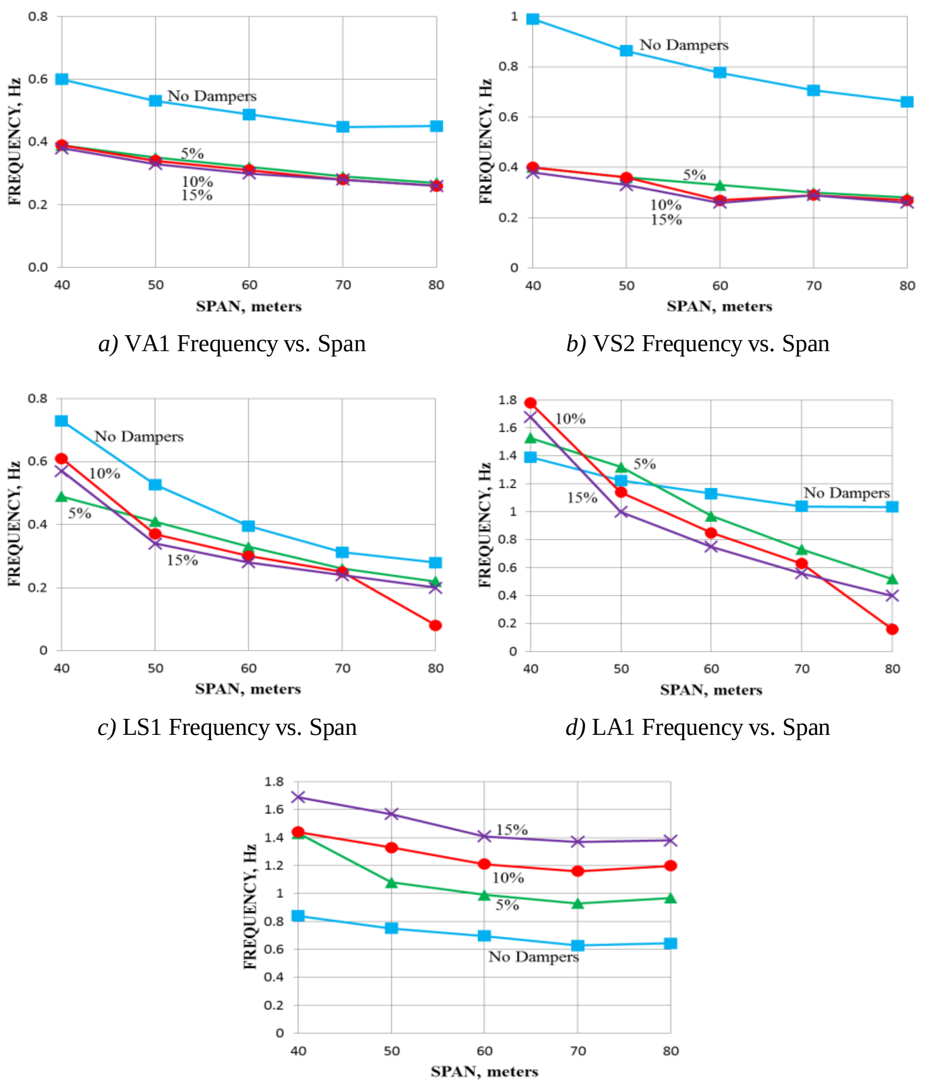
Dampers increase the average displacements experienced by both walker and bystander. A 15% mass ratio increases the displacement experienced by the walker on the 70 m bridge by over 200%. Two vertical dampers slightly reduce the vertical accelerations and velocities, however.

No single mass ratio is significantly better in reducing the vertical accelerations or velocities experienced, however, a 5% mass ratio consistently outperforms a 10% and 15% mass ratio in reducing vertical displacement. Lateral accelerations are controlled well in the 50 m and 70 m bridges and meet comfort limits, but the 40 m, 60 m, and 80 m are not successfully controlled.

7.3 Three vertical dampers

Modal frequency analysis results are presented for three vertical dampers in Figure 12. The three vertical damper arrangement did not reduce the vertical frequencies as much as the two vertical damper arrangement. The 5%, 10%, and 15% mass ratios all result in approximately the same vertical frequencies for VA1 and VS2. VA1 frequencies are reduced by approximately 33% and VS2 frequencies are reduced by approximately 60%. The two vertical damper arrangement reduces the VS2 frequencies more than the three vertical damper arrangement that is tuned to VS2.

The three vertical damper arrangement decreases lateral frequencies as the bridge length increases. LS1 frequencies are reduced by 0.1 Hz to 0.2 Hz. LA1 frequencies increase for 40 m, slightly varied for 50 m, and reduced for 60 m, 70 m, and 80 m. The largest frequency LS1 reduction, 84%, was achieved by the 10% mass ratio for the 80 m span. TS1 frequencies increased for every damper arrangement.



a) VA1 Frequency vs. Span

b) VS2 Frequency vs. Span

c) LS1 Frequency vs. Span

d) LA1 Frequency vs. Span

e) TS1 Frequency vs. Span

Figure 12. Modal frequencies with three vertical dampers

The time-history bridge response with three vertical dampers is presented in Figure 13. Vertical accelerations experienced by the walker are reduced the most for longer span bridges.

A 15% mass ratio is the only ratio that significantly reduces vertical accelerations for the walker for the 40 m and 50 m spans. A 15% mass ratio reduces vertical accelerations experienced by the walker for four out of five spans, in some cases by as much as 50%; however, none of the average walker-experienced accelerations satisfy the comfort limit. Vertical acceleration experienced by the bystander is reduced by 91% for the 50 m span, 10% mass ratio model. With the exception of the 40 m span, 15% mass ratio, 10% and 15% mass ratio reduces the vertical accelerations experienced by the bystander to below the comfort limit.

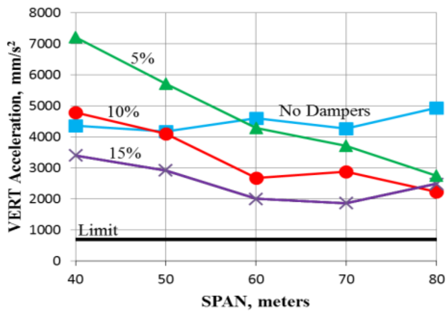
Lateral accelerations experienced by the bystander are reduced to below the comfort limit for all damping ratios. Lateral accelerations experienced by the walker are reduced below the comfort limit for the 70 m span only. The 40 m and 50 m spans respond with a 12% to 41% reduction in lateral accelerations experienced by the walker. The 80 m span responds with an increase in lateral accelerations experienced by the walker.

Vertical velocities experienced by the walker are reduced for all damper arrangements, however, none are below the comfort limit. A 10% mass ratio results in the largest reduction for 40 m and 80 m spans and a 15% mass ratio results in the largest reduction for all other spans.

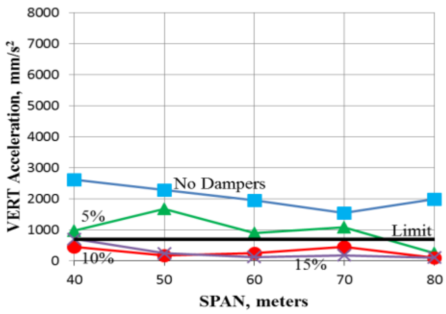
Vertical velocities experienced by the bystander are reduced for all damper arrangements, but the 10% and 15% mass ratios outperform the 5% mass ratio. A 15% mass ratio reduces the average vertical velocity experienced by the bystander to a level below the comfort limit for 60 m, 70 m, and 80 m spans. A 10% mass ratio is also below the comfort limit for the 80 m span.

Vertical displacements experienced by the walker and bystander are increased for a majority of the damper arrangements. The higher the mass ratio, the greater the increase in displacement.

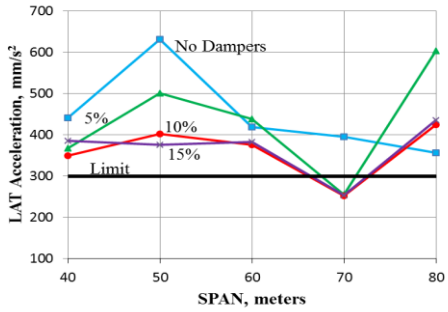
A walker experiences an average vertical displacement of 900 mm on the 80 m span with 15% mass ratio dampers, which is a 2½ fold increase. Overall, the three vertical damper arrangement performs better than the two damper arrangement. A 15% mass ratio does not provide the best velocity and acceleration response control in all arrangements, but always results in the largest vertical displacements.



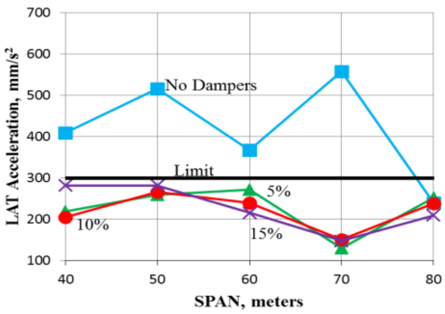
a) Walker Vert Acceleration vs. Span



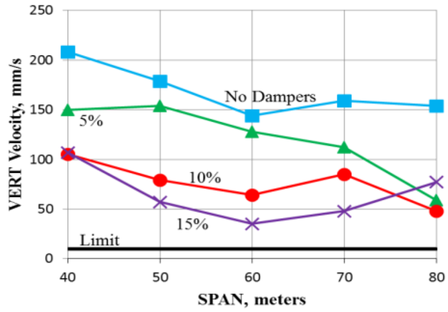
b) Bystander Vert Acceleration vs. Span



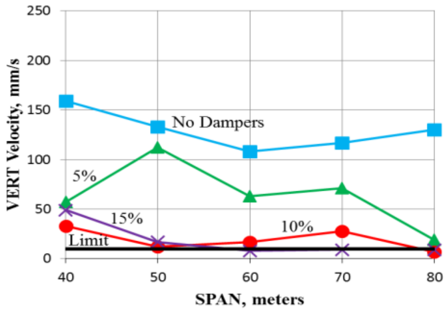
c) Walker Lat Acceleration vs. Span



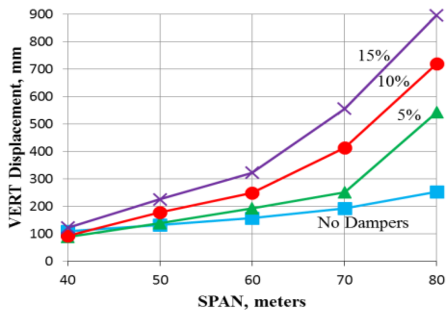
d) Bystander Lat Acceleration vs. Span



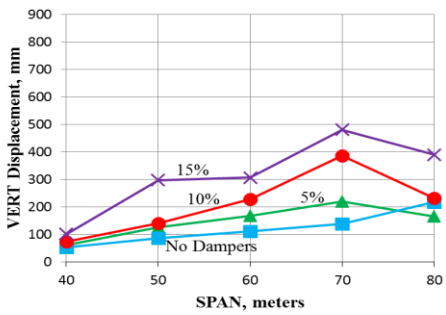
e) Walker Vert Velocity vs. Span



f) Bystander Vert Velocity vs. Span



g) Walker Vert Displacement vs. Span

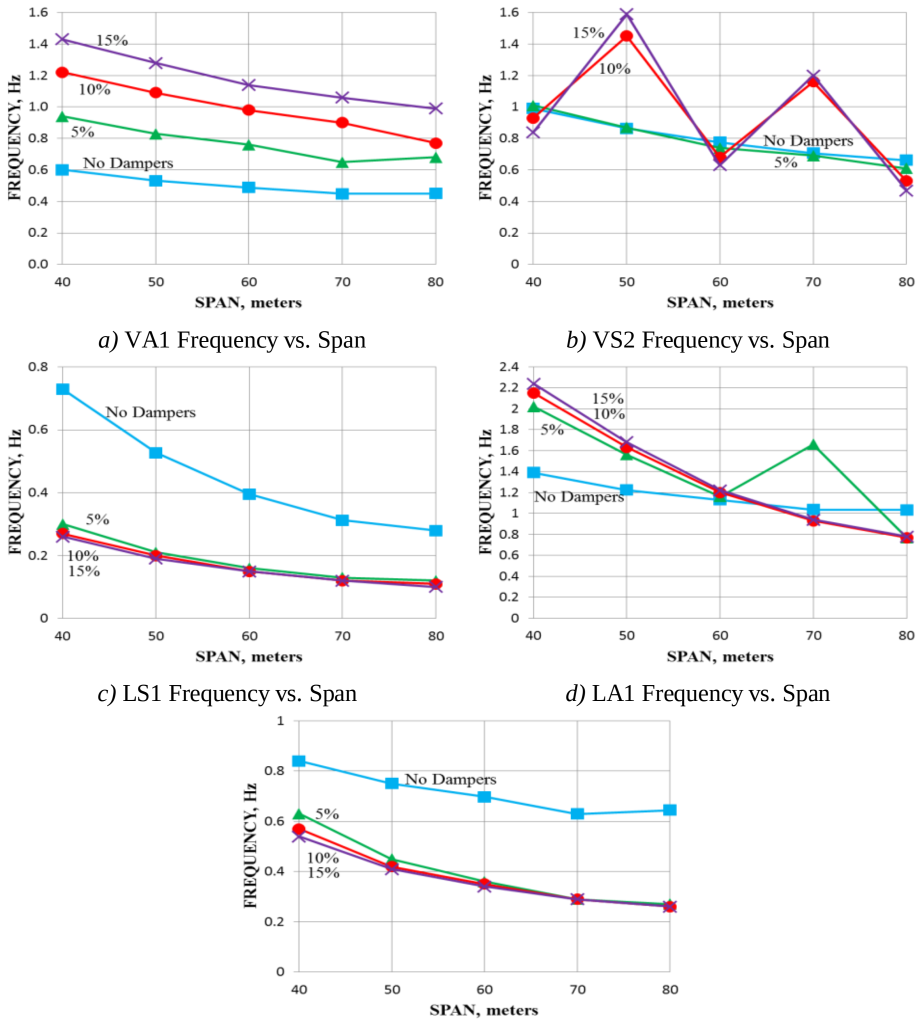


h) Bystander Vert Displacement vs. Span

Figure 13. Dynamic response with three vertical dampers

7.4 Single lateral damper

Results of a dynamic modal analysis with one lateral damper are presented in Figure 14. This lateral damper arrangement is tuned to LS1, resulting in the dynamic response being reduced to the same relative magnitude for each span. A similar result is achieved for TS1 response. These responses are to be expected as the lateral dampers are located at the antinodes of these modes and are tuned to the modal frequency, reducing the LS1 frequencies by 57% to 63% and TS1 frequencies by 21% to 60%.



a) VA1 Frequency vs. Span

b) VS2 Frequency vs. Span

c) LS1 Frequency vs. Span

d) LA1 Frequency vs. Span

e) TS1 Frequency vs. Span

Figure 14. Modal frequencies with one lateral damper

VA1 modal frequencies increased with the addition of a lateral damper and the greater the mass ratio, the greater the frequency increase. VS2 modal frequencies are slightly affected by a 5% mass ratio and the 10% and 15% mass ratio frequencies are also slightly lower for the 40 m, 60 m, and 80 m spans, but increase from 63% to 84% for 50 m and 70 m.

The dynamic response results of a time-history analysis with a pedestrian and one lateral damper are presented in Figure 15. Vertical accelerations experienced by the walker differ across span lengths and mass ratios with no clear trend and no average acceleration below the comfort limit.

A 15% mass ratio reduces accelerations by over 50% for the 40 m, 50 m, and 80 m spans, but increases the vertical acceleration experienced by the walker for the 70 m span.

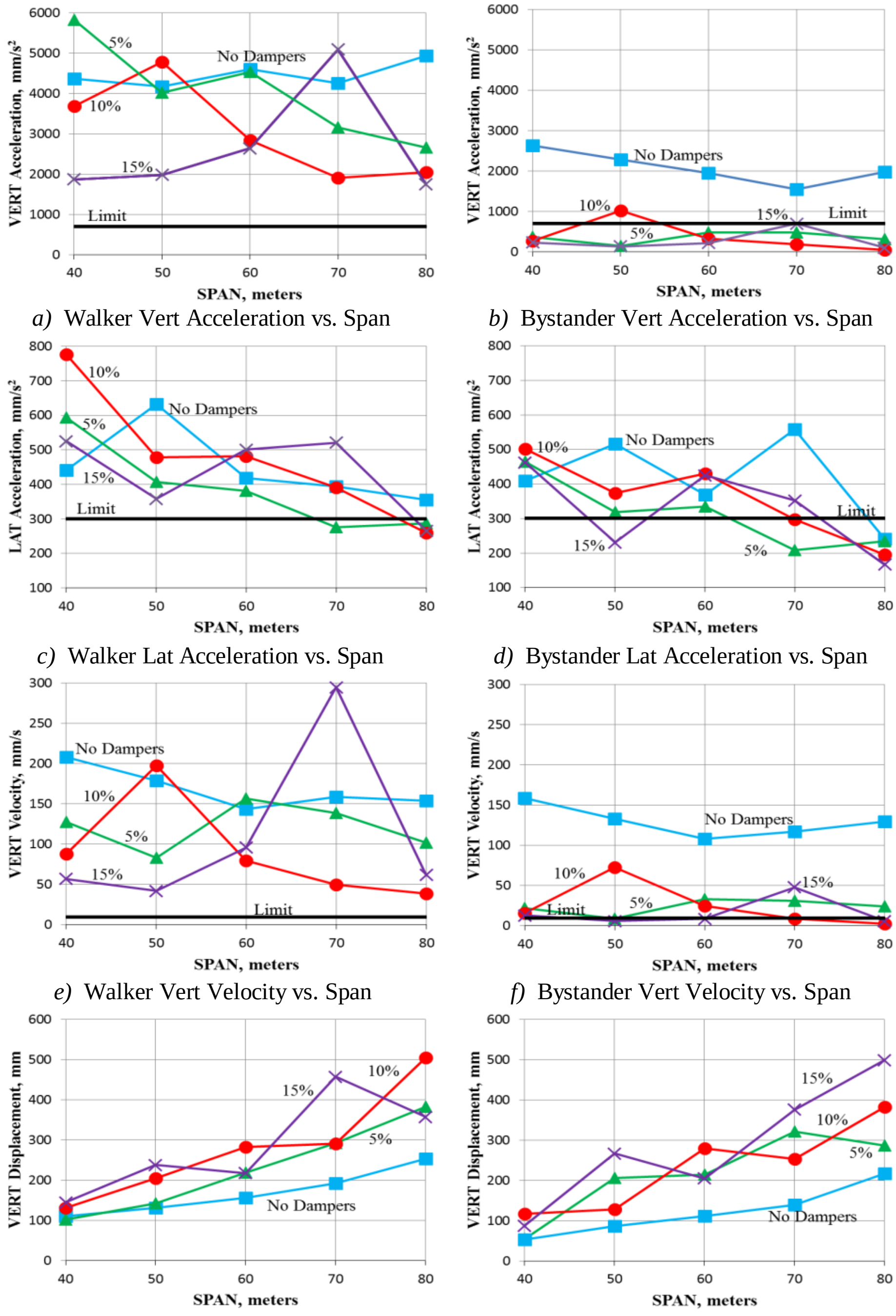
A 5% mass ratio increases vertical acceleration for the 40 m span, has little effect on the 50 m and 60 m spans, and decreases the acceleration for the 70 m and 80 m spans. The 80 m span exhibits a significant acceleration decrease for each mass ratio.

All vertical accelerations experienced by the bystander are below the comfort limit except for the 50 m span, 10% mass ratio and the 70 m span, 15% mass ratio. These configurations responded with increased vertical acceleration and increased vertical velocity experienced by the walker. Vertical displacements experienced by the walker and the bystander are increased for all configurations as compared to the control. A 5% mass ratio results in the smallest increase in vertical displacements on average.

Lateral accelerations experienced by the walker increase for the 40 m span for all mass ratios. A 5% mass ratio decreases lateral accelerations experienced by the walker as the span increases.

A 5% mass ratio is the most effective for a 60 m span, achieving a 9% reduction, and on the 70 m span, achieving a 30% reduction which lowers the lateral accelerations below the comfort limit.

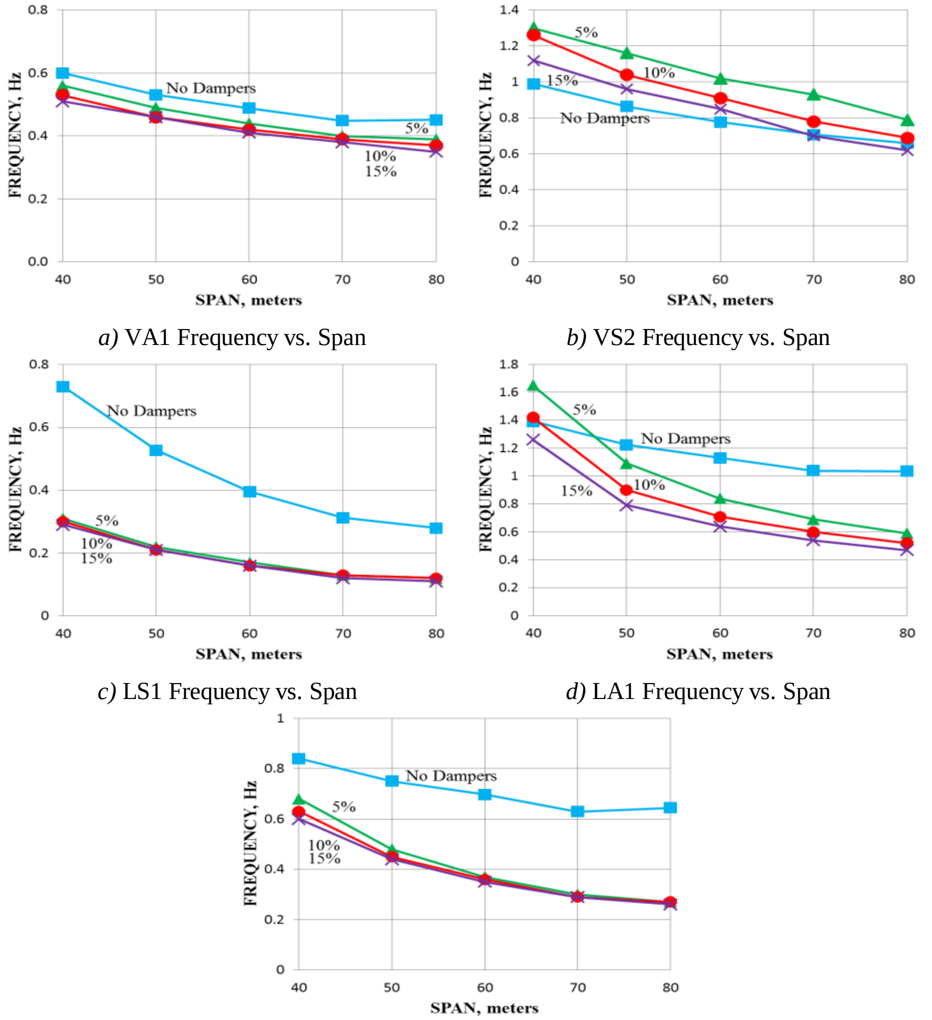
A 10% mass ratio decreases the walker experienced lateral accelerations for the 50 m and 80 m spans. All configurations are successful in reducing the lateral acceleration experienced by the walker below the comfort level for the 80 m span. Lateral accelerations experienced by the bystander are reduced most effectively for 50 m and 70 m spans.



a) Walker Vert Acceleration vs. Span
b) Bystander Vert Acceleration vs. Span
c) Walker Lat Acceleration vs. Span
d) Bystander Lat Acceleration vs. Span
e) Walker Vert Velocity vs. Span
f) Bystander Vert Velocity vs. Span
g) Walker Vert Displacement vs. Span
h) Bystander Vert Displacement vs. Span

Figure 15. Dynamic response with one lateral damper

7.5 Two lateral dampers



a) VA1 Frequency vs. Span

b) VS2 Frequency vs. Span

c) LS1 Frequency vs. Span

d) LA1 Frequency vs. Span

e) TS1 Frequency vs. Span

Figure 16. Modal frequencies with two lateral dampers

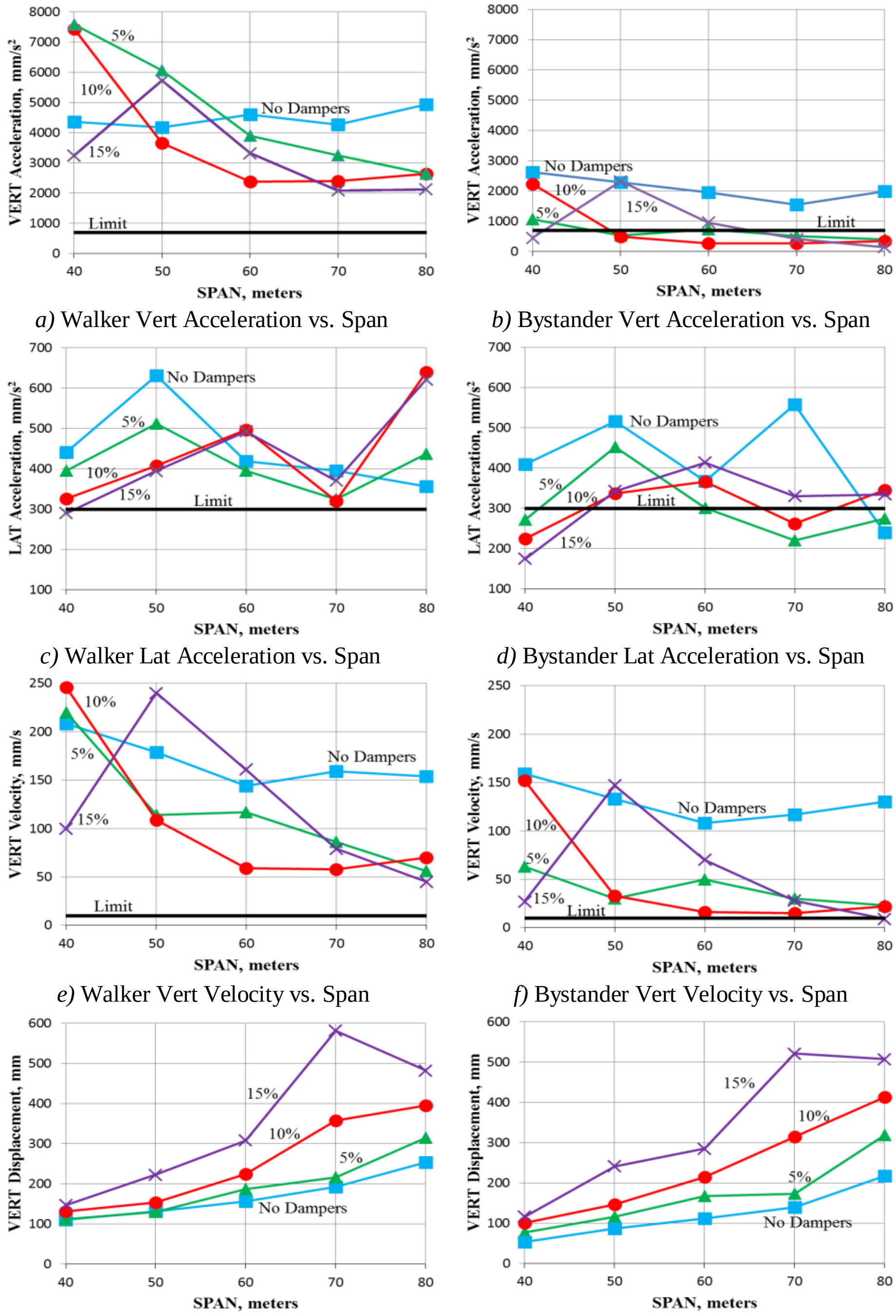
The modal analysis results for two lateral dampers are presented in Figure 16. The two lateral damper arrangement does not significantly affect the vertical frequencies. VA1 modal frequencies are decreased by less than 0.2 Hz and VS2 modal frequencies are slightly increased. The two lateral damper arrangement primarily affects the lateral and torsional modes. LS1 modal frequencies are reduced by 56% to 59% for all mass ratios for the 40 m span and 57% to 61% for all mass ratios for the 80 m span. Damped LA1 modal frequencies are similar to the control model for the 40 m span, but exhibit an increasingly large reduction as the span length increases. A 15% mass ratio reduces the frequency of LA1 for the 80 m span by 54%, the largest reduction for any of the two lateral damper models on LA1. All damped TS1 modal frequencies are reduced with little dependency on mass ratio. Reductions range from 15% to 60%, with the largest reductions occurring in the longer span lengths.

The dynamic responses derived from a time-history analysis with two lateral dampers are presented in Figure 17. Vertical accelerations experienced by the walker are reduced for the 60 m, 70 m, and 80 m spans, but do not satisfy the comfort limit. The 40 m span experiences a 70% increase in vertical acceleration for the 5% and 10% mass ratio models. Vertical accelerations experienced by the bystander are reduced in all damper arrangements except a 15% mass ratio on the 50 m span. The bystander-experienced vertical accelerations are reduced below the comfort limit for at least one mass ratio model on each span length. A 10% mass ratio is most effective across span lengths and reduces the vertical acceleration experienced by the bystander to 266 mm/s2 for the 60 m model, a reduction of 86%.

Lateral accelerations experienced by the walker are not greatly affected on the 60 m and 70 m spans, but are increased on the 80 m span. The 40 m and 50 m span damper models decrease the walker-experienced lateral accelerations by as much 38%. The only arrangement that meets the human comfort limit for walker-experienced lateral accelerations is the 15% mass ratio 40 m span with a lateral acceleration of 290 mm/s2, a 34% reduction from the control model. Lateral accelerations experienced by the bystander are decreased on the 40 m, 50 m, and 70 m spans. All of the 40 m two lateral damper arrangements reduce the bystander-experienced lateral accelerations below the comfort limit. A 5% and 10% mass ratio meets the comfort limit on the 70 m span.

The vertical velocities are reduced most effectively by a 10% mass ratio, with the largest reduction occurring on the 70 m span at 64%. A 15% mass ratio is the only model that reduces the vertical velocities experienced by the walker on the 40 m span. None of the two damper arrangements reduce vertical velocities experienced by the walker below the comfort limit. A 15% mass ratio on the 80 m span reduces the average vertical velocity experienced by the bystander below the comfort limit. Although the vertical velocities experienced by the bystander were not reduced below the comfort limit, the 10% mass ratio model was able to affect a reduction of 75% to 85% for the 50 m, 60 m, 70 m, and 80 m spans.

Vertical displacement experienced by the walker increased for all damper arrangements, most significantly for higher mass ratios. This is also true for vertical displacements experienced by the bystander. The largest increase occurs as a result of a 15% mass ratio on a 70 m span, with a 203% increase on the average walker displacement and a 275% increase on the average bystander displacement.



a) Walker Vert Acceleration vs. Span
b) Bystander Vert Acceleration vs. Span
c) Walker Lat Acceleration vs. Span
d) Bystander Lat Acceleration vs. Span
e) Walker Vert Velocity vs. Span
f) Bystander Vert Velocity vs. Span
g) Walker Vert Displacement vs. Span
h) Bystander Vert Displacement vs. Span

Figure 17. Dynamic response with two lateral dampers

8 CONCLUSIONS

Pedestrian suspension bridges are generally more susceptible to serviceability issues than to strength or safety issues. Many rural communities are in need of pedestrian suspension bridges to access essential needs year round. Common pedestrian suspension bridge designs that can be easily adapted to site conditions, are economical, and simple to construct will inherently be low mass and flexible. This results in a bridge that experiences larger dynamic responses than pedestrian suspension bridges constructed with a high level of resources.

TMDs can effectively and economically be installed on suspension bridges with serviceability issues and can dramatically affect the response of a bridge. The present parametric study was conducted to determine if TMDs improve the common pedestrian suspension bridge employed in developing countries and rural areas. Sixty models with unique damper arrangements were analyzed against control models and human comfort limits to determine the effect of the dampers. A modal analysis and time-history analysis were conducted to analyze the model modal frequencies, vertical displacements, vertical velocities, vertical accelerations, and lateral accelerations.

The results of the parametric study support the following conclusions:

1. Addition of TMDs results in a wide range of dynamic response changes that make it difficult for a TMD to synchronize to the single frequency.
2. Modal frequencies are not a clear forecaster of the dynamic response. Many of the lateral damper arrangements have nearly identical lateral modal frequency magnitudes, however, the lateral responses varied significantly.
3. Adding mass reduces the velocity and acceleration responses experienced by the walker and bystander, but increases displacement.
4. Higher mass ratios did not consistently reduce acceleration or velocity responses, however, higher mass ratios always result in larger displacements.
5. Damper arrangements with a TMD located at center span reduced the dynamic response most effectively.
6. Human comfort limits were very rarely achieved for the walker. Some arrangements reduced dynamic response by over 80%, but were not below the comfort limit.
7. The two lateral damper, 15% mass ratio arrangement most effectively reduces the dynamic response of the 40 m model.
8. The one lateral damper, 15% mass ratio arrangement most effectively reduces the dynamic response of the 50 m model.
9. The three vertical damper, 15% mass ratio arrangement most effectively reduces the dynamic response of the 60 m model.
10. The three vertical damper, 15% mass ratio arrangement most effectively reduces the dynamic response of the 70 m model.
11. The one lateral damper, 15% mass ratio arrangement most effectively reduces the dynamic response of the 80 m model.

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