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USE OF TUNED MASS DAMPERS TO CONTROL EXCESSIVE VIBRATIONS OF PEDESTRIAN BRIDGES

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Abstract. *Pedestrian bridges of are often sensitive to vibrations causing discomfort to the pedestrian and in worst cases may cause resonance and catastrophic failure. Pedestrian bridges with natural frequencies around 3 Hz may experience such behavior. Synchronization is a very important factor in increasing the severity of loading, by which people respond naturally to an oscillating bridge when it has frequencies close to their natural walking or running frequencies. Attempting to increase the natural frequency of a bridge beyond 3 Hz might not be possible or may require heavy structural members way more than what is needed for strength and serviceability limit states. A more robust and economical solution is to use tuned mass dampers (TMD) with natural frequencies tuned to the natural frequency of the bridge. Therefore, if vibrations are excited in a bridge, the TMD also vibrates with a certain phase shift relative to the bridge motion leading to a compensation of inertia loads, that in turn results in a fast decay in the bridge vibration. This study summarizes the modal and time-history analyses conducted to control the vibrations of the pedestrian bridge of Anacortes Terminal Building in Seattle, USA using TMDs. The structural members of the pedestrian bridge were checked using ETABS and STAAD PRO models according to the applicable load combinations (dead load, live load, seismic, and wind) according to the ASCE7, AISC, and IBC Codes (LRFD Approach). The performance of the bridge was significantly improved with TMDs after exciting the bridge by simulated pedestrian activities.*

1 INTRODUCTION

Pedestrian dynamic forces on footbridges may induce significant vibrations causing discomfort and major deterioration of the structural integrity. In many countries including Jordan, the use of footbridges is very common as a method for pedestrian crossing, especially in congested areas or on major roads. However, the pedestrian-induced vibrations on footbridges are not appropriately considered. Besides the potential for discomfort, resonance, in some cases this may cause a pre-mature collapse due to the continuous deteriorations induced by excessive vibrations. Consequently, it is important that this phenomenon is well considered in the design by qualified structural engineers with sound knowledge and experience in structural dynamics and vibrations.

Rainer et al., 1988 [1] found that vertical force pattern one person induces while walking over a flexible structure may create periodic forces with a frequency typically in the range of 1.5 to 4 Hz. Previous research showed that for fewer than about 20 to 25 persons, almost all might synchronize with the footbridge motions [2, 3]. If the lowest natural frequency of a footbridge (or other flexible structure subjected to human loading) is lower than 5 Hz in the vertical and 2.5 Hz in the lateral direction, detailed analysis and comparison with criteria in design codes should be considered (ISO/DIS 10137, 1995). Findings of previous researchers on the control of vibrations of pedestrian bridges can be found in many studies [3-10]. Tuned mass dampers (TMD) can be used to increase damping of a bridge and to reduce the amplification of bridge excitation. A TMD consists of a vibrating mass that is a percentage of the total bridge mass supported by helical springs with parallel viscous dampers. The natural frequency of the TMD is tuned to the natural frequency of the bridge to increase its system damping. Therefore, if vibrations are excited, the TMD will vibrate with a certain phase shift relative to the bridge motion leading to a compensation of inertia loads resulting in a decrease of bridge motion [11].

This article presents the findings of dynamic analysis of a 120 feet-long (36.3 m) pedestrian bridge near the Seattle area. In order to match the top of the bridge deck as close to the main floor of the a building while still allowing vehicles to pass under the bridge, a shallow steel plate girder system is proposed for the bridge main structure. Preliminary analysis indicated that the bridge first natural frequency is close to the human walking and/or running frequencies. This raised the concern of the dynamic loading caused by moving pedestrians for the bridge's vertical and lateral vibrations. The resulting bridge motion will not only reduce the comfort when people cross the bridge but also may in the worst-case cause damage to the bridge. The phenomenon so-called "synchronization" is a very important factor in increasing the severity of loading, by which people respond naturally to an oscillating bridge when it has frequencies close to their natural walking and/or running frequencies. Various structural options were explored; i.e. stiffening and increasing the weight, to reduce or prevent synchronization such as increasing the weight of the steel girders or changing the lateral bracing system, but none of the options prove to be effective and economical. The option of increasing bridge modal damping using commercially available modular TMD was then explored. A numerical verification for the effectiveness of the TMD implementation is provided using a finite element model of the bridge and dynamic loading time history caused by pedestrians.

2 STATIC AND MODAL ANALYSES

The footbridge is simply supported with a span length of 120 ft (36.3 m) and a width of 6 ft (1.8 m) at one end and 12 ft (3.6 m) at the other end. Figure 1 shows the top and side views of ETABS model of the footbridge. Static, modal, and time time-history analyses were conducted on the footbridge. In static analysis, the structural members of the pedestrian bridge were analyzed and checked according to the applicable load combinations (dead load, live load, seismic, and wind) provide by the ASCE-2007, AISC, and IBC-2006 Codes (LRFD approach). To check the lateral torsional buckling of the flanges and webs of the main girders, they were meshed into small area elements defined as shell elements. The ETABS model showed that the main girders are adequate in terms of stress limit and lateral torsional buckling. Figures 2 and 3 shows the deformed shape under static loading and verification of the stress ratio in each member.

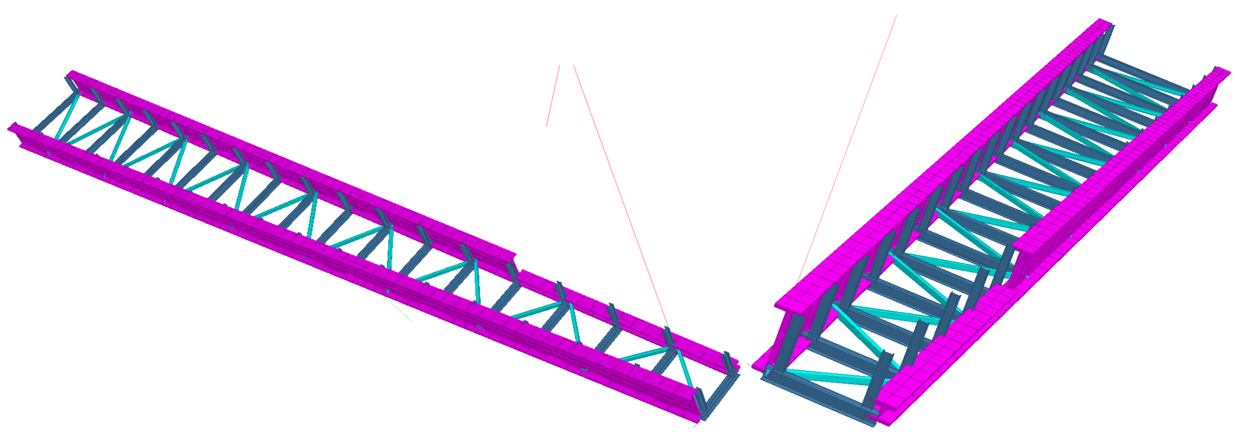

Figure 1. Top and side views of ETABS model of the footbridge

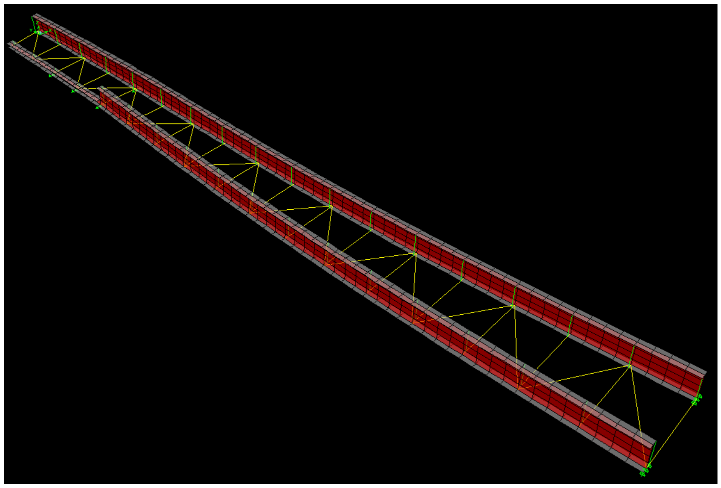

Figure 2. Deformed shape under static loading

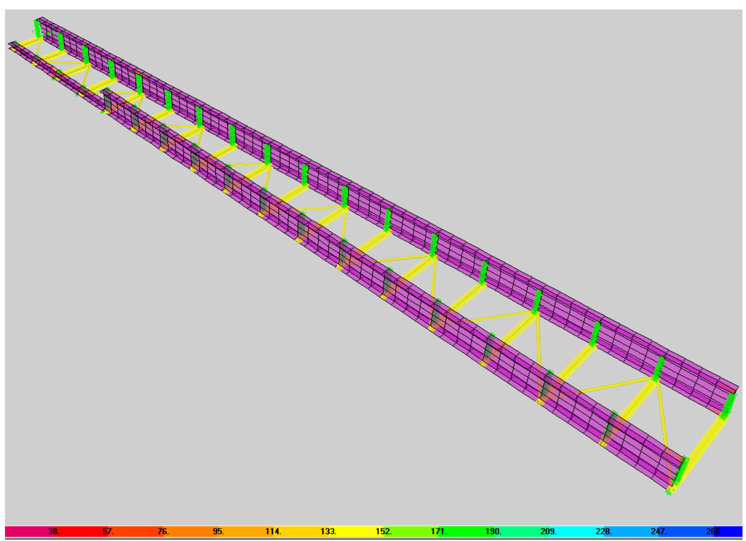

Figure 3. Verification of the stress ratio in each member

Modal analysis was then performed to check the natural frequency of the bridge system and to compare it with the expected frequency from human activities. The self-weight of the designed members in the static analysis plus an additional dead load of 15 psf (728 N/m^2) from the deck were used as a mass source in the modal analysis. It was noticed that the natural frequency of the first mode is 3.68 Hz with a period of 0.271 second as shown n Figure 4. This natural frequency falls within the range of concern of pedestrian induced vibrations; i.e. within 0-4 Hz [1].

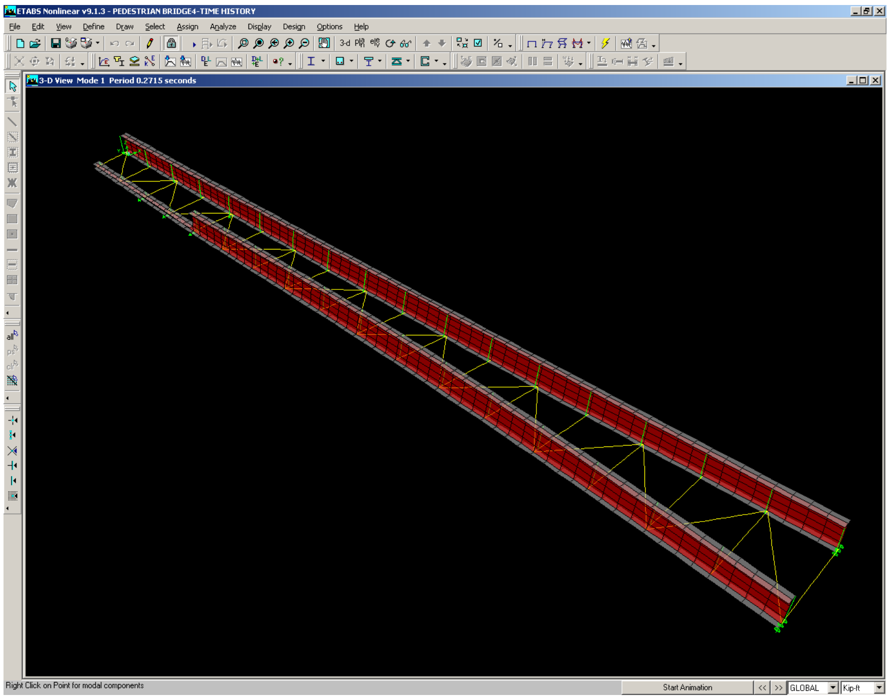

Figure 4. First mode shape (period = 0.271 second)

3 TIME-HISTORY ANALYSIS

The first mode natural frequency of 3.68 Hz was a direct motivation for evaluating the response of the pedestrian bridge under the exposure to unusual pedestrian activities exciting the bridge in a frequency matching the natural frequency of the first mode of the bridge. This issue is important especially when considering the fact that after exciting the bridge, the pedestrians are trying to match their movement with the bridge movement (synchronization), which may significantly increase the amplitude of displacement resulting in a discomfort and long-term deterioration.

Time History Analysis was then performed using ETABS and assuming 8 people are exciting the bridge at the same time. The 8 people each weighing 150 lbs (68 kg) were placed at critical locations to maximize the excitation. The frequency of their steps was set to match the first mode natural frequency of the system (3.68 Hz). At the peaks the loading was amplified to 1.4 times of their static weight. Figure 5 shows the definition of the time-history function. The used (recommended) damping ratio for footbridges is 0.01. The analysis was performed and the response (acceleration) of the bridge at the most critical location was obtained. From the results, it was found that the acceleration of the system at the maximum critical location is 1.894 ft/s^2 (0.57 m/s^2). This is about 5.88%g, which exceeds the maximum recommended limit of 5%g.

The use of stiffer girders to increase the natural frequency of the bridge system up to 5 Hz or more is explored. The practical way to increase the stiffnesses of the girders is to increase the cross sectional area to keep the clearance limitation. However, increasing the cross sectional area will result in significant increase in the weight of the girders. It was shown by modal analysis that if a 2-3 times heavier girders were used, the increase in the first mode natural frequency of the bridge will be very small. It should be noted also that this is a very costly approach and at the same time not satisfying the goal. The total weight of the two main girders (W40x249) is about 30 tons, and if considering the price is $2,500/ton, then the total cost for just doubling the weight is $ 75,000. In addition, increasing the weight of the girders will result in additional construction cost (delivery, erection, etc.). Therefore, it was decided to check what kind of improvement in the system performance would be gained if incorporating two viscous dampers. The goal is to reduce the induced vibrations to acceptable limit. The cost of each viscous damper is $25,000 (both $50,000), which is much lower than the cost of the first option.

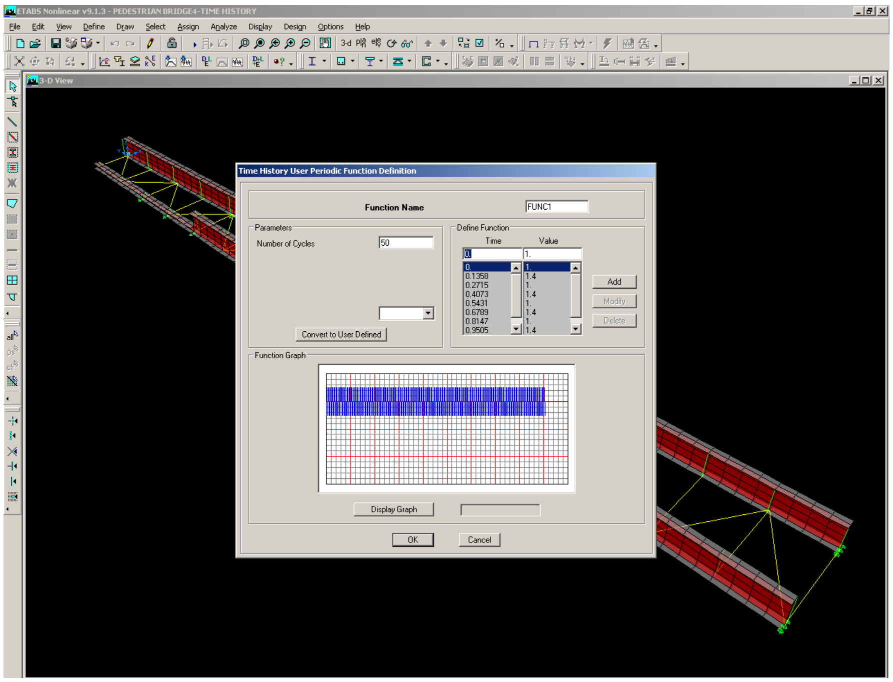

Figure 5. Definition of the time-history function

4 TUNING THE VMD

Using the same ETABS model in the previous step, two viscous dampers were incorporated into the bridge system at the middle of the main-span. The stiffness (*k*) and damping (*c*) of the dampers are tuned (optimized) to have the target performance using a dynamic approach, in which: *M* is the system main mass, *m* is damper mass, μ is the mass ratio = m/M, ω_a is the natural frequency of the damper, $\omega_a^2 = k/m$, Ω_n is the natural frequency of the damper, $\Omega_n^2 = K/M$, *f* is the frequency ratio = ω_a/Ω_n, and c_c is the critical damping = $2m\ \Omega_n$. Table 1 shows the weight of the footbridge members as well as its mass and the practical mass for each damper.

Table 1: Weight of the bridge members and practical mass of each damper

Members	Weight
Two girders (W40x249)	59,262 lb
18 lateral beams (W8x31)	5022 lb
17 diagonal bracing members (HSS4x4x0.375)	3927 lb
36 vertical members (WT7)	3000 lb
Decking system	16065 lb
Total rounded weight	87,000 lb
Total Mass (M) = Total weight/g	2.7 k-s^2/ft
Practical mass for each damper, m = 0.05x(M/2)	0.0675 k-s^2/ft

Note: 1 lb = 4.448 N and 1 k-s^2/ft = 14.7 kN-s^2/m

Once the practical mass of each damper is determined as shown in Table 1, the dampers were then tuned to have the same frequency as the natural frequency of the first mode of vibration of the footbridge (3.68 Hz). The damper stiffness k was then calculated as from $\omega_a^2 = k/m$, which gives k = 0.914 k/ft (13.4 kN/m) for ω_a = 3.68 Hz and m = 0.0675 k-s^2/ft (0.99 kN-s^2/m).

The critical damping was calculated using: $c_c = 2m\ \Omega_n$, which yields c_c = 2x0.0675x3.68 = 0.9936. The optimum damping of the dampers was then obtained using eqn (1), which gives c = 0.10 for μ = 0.025 and c_c = 0.9936. The calculated values for k and c were then employed in the definition of the viscous dampers.

$$\left(\frac{c}{c_c}\right)^2 = \frac{\mu(\mu+3)(1+\sqrt{\mu/(\mu+2)})}{8(1+\mu)} \tag{1}$$

5 RESULTS AFTER INCORPORATING THE TMD

Two TMDs were then defined as linkage elements with c = 0.10 and k = k = 0.914 k/ft (13.4 kN/m), and attached to the girders at the middle of the footbridge as shown in Figure 6. Comparison was then made between the performance of the system before and after adding the TMDs.

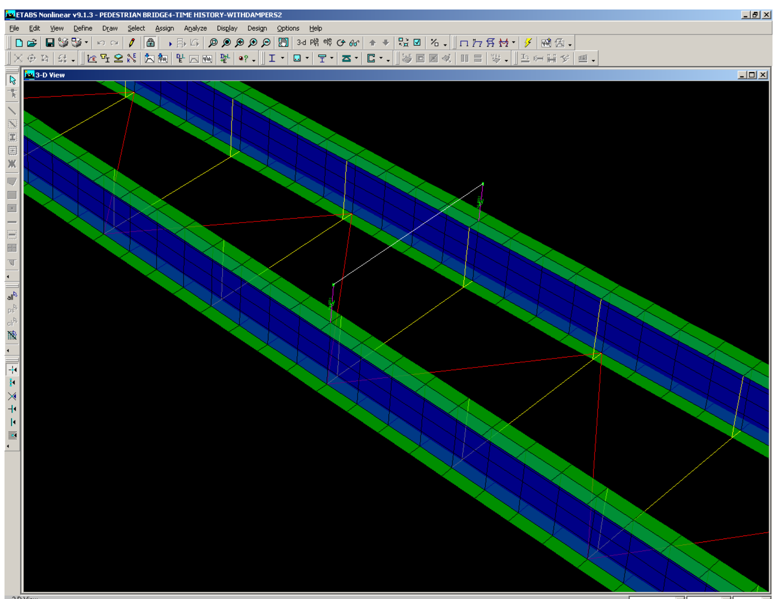

Figure 6. Close-up view shows the two TMD attached to the girders at the middle of the footbridge

The response (acceleration) of the footbridge at the most critical location (at the center) was obtained before and after incorporating the TMD as shown in Figure 7. From the results, it can be seen that the maximum acceleration was decreased after incorporating the TMD from 1.894 ft/s^2 (0.57 m/s^2), which is about 5.88%g to 0.417 ft/s^2 (0.126 m/s^2), which is about 1.3%g, which is way below the recommended limit of 5%g. Figure 8 shows the velocity-time traces before and after the TMD. The maximum velocity was decreased after incorporating the TMD from 0.0805 ft/s (0.024 m/s) to 0.0197 ft/s (0.006 m/s), which is a 75% reduction. Figure 9 shows the displacement-time traces before and after the TMD. The maximum velocity was decreased after incorporating the TMD from 0.0055 ft (0.0017 m) to 0.0029 ft (0.0009 m), which is a 50% reduction.

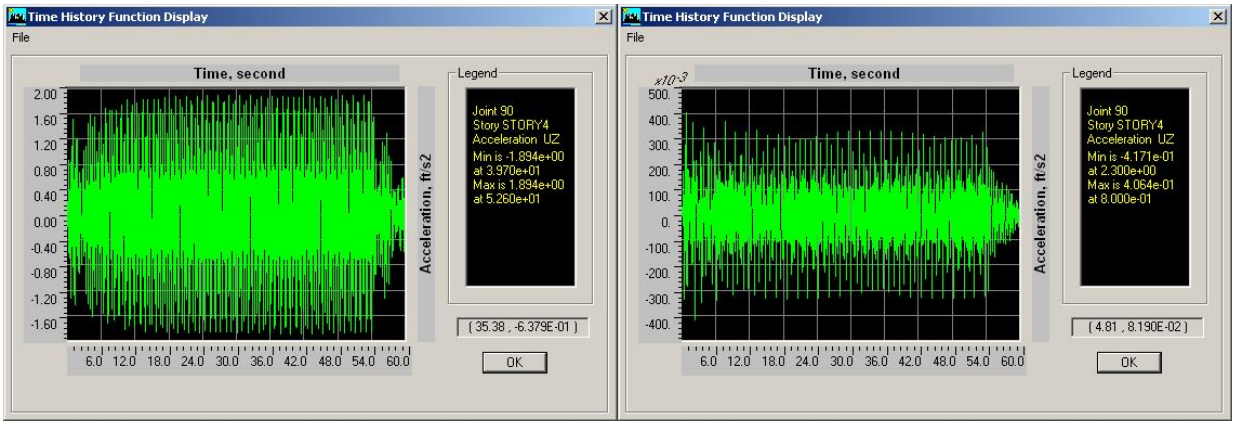

Figure 7. Maximum acceleration-time trace with TMD (1.894 ft/s^2) and without TMD (0.417 ft/s^2)

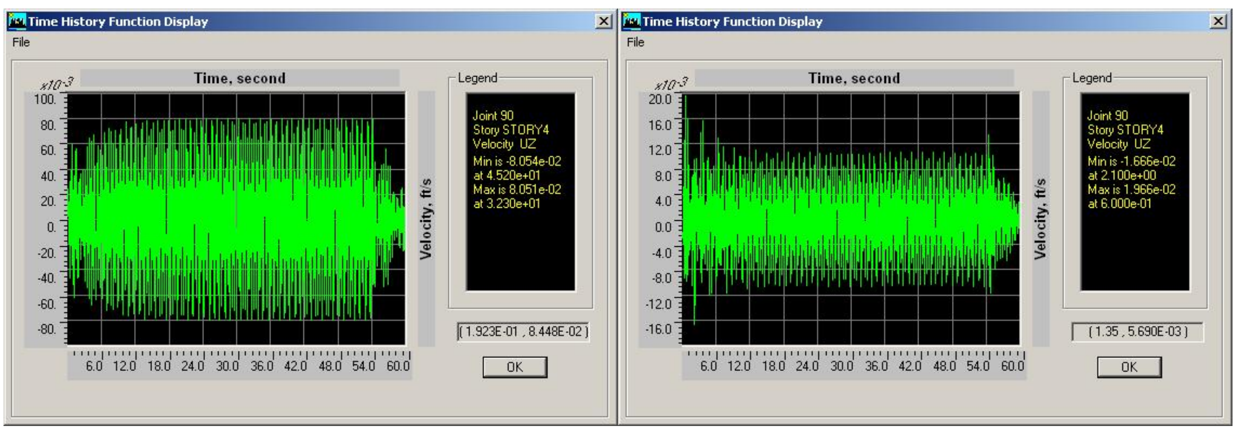

Figure 8. Maximum velocity-time trace with TMD (0.0805 ft/s) and without TMD (0.0197 ft/s)

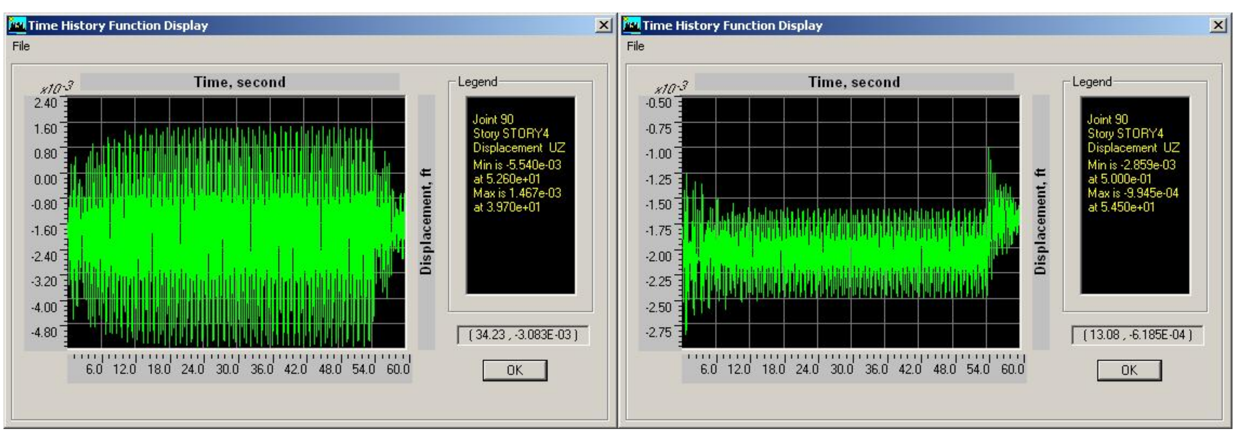

Figure 9. Maximum displacement-time trace with TMD (0.0055 ft) and without TMD (0.0029 ft)

The acceleration, velocity, and displacement-time traces shown in Figures 7-9 clearly show that the incorporated TMD were very effective in controlling the vibrations of the footbridge. Further more the decay in the acceleration after the end of the excitation is obtained as shown in Figure 10. The results show clearly that the decay is faster and more stable after incorporating the TMD. Finally the effectiveness of the damper can be clearly appreciated from Figure 11 that shows the energy dissipated by each damper.

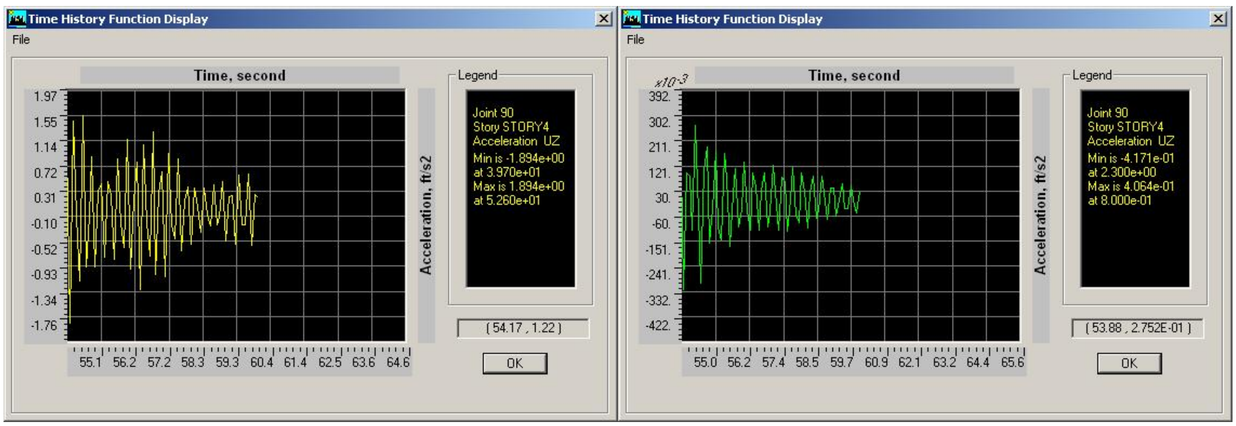

Figure 10. Decay in the acceleration after the end of the excitation with and without TMD

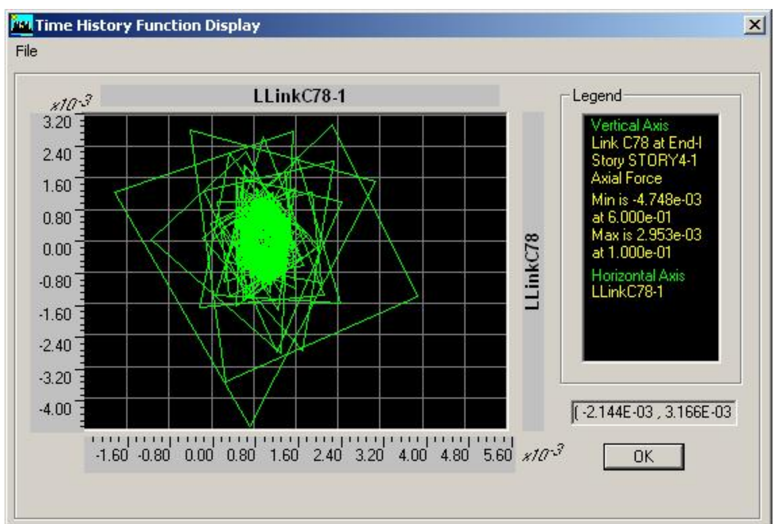

Figure 11. Energy dissipation by the TMD

6 CONCLUSIONS

The use of TMD showed effectiveness in controlling pedestrian-induced vibrations. The maximum acceleration, velocity, and displacements were substantially decreased when TMD were incorporated in the structural system of the footbridge. In addition, the decay in the acceleration was faster and steadier.

The energy dissipated by the TMDs reflects the dampers' effectiveness in controlling the vibrations of the footbridge and perhaps reducing long-term vibration-induced deteriorations in footbridges.

For the studied footbridge herein, the use of TMD was more economical than attempting to increase the stiffness of the system to control its vibrations. This provides a feasible solution in some cases for controlling potential excessive vibrations of new and existing footbridges.

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