Mitigation of Traffic-Induced Bridge Vibrations through Passive and Semi-Active Control Devices

by

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ABSTRACT

Many of the U.S. bridges today are reaching or have reached their design life and are beginning to deteriorate and are becoming structurally deficient. Much time, effort, money, and resources go into repairing, rehabilitating, or reconstructing these bridges. Therefore, investigation into valid solutions to extending the safe life of these structures is of utmost importance.

A major cause of bridge deterioration is stresses and fatigue induced in the bridge from traffic loading. This paper explores and investigates methods of mitigating traffic-induced bridge vibrations through the integration of control devices to extend the service life of bridges. There are three main classes of structural control devices: passive, semi-active, and active control. Each control scheme has advantages and disadvantages which are discussed in this thesis. To gain a better understanding of both passive and semi-active control strategies, a computer simulation is conducted. The computer simulation allows for a better comparison between passive and semi-active control schemes. The finding from the simulation shows a semi-active control strategy outperforming a passive strategy. The semi-active scheme reduces maximum midspan deflections by 20%, while the passive has a reduction of 12%.

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1 Introduction

1.1 Current Situation

Bridges are an essential part of the transportation infrastructure in the United States. Nearly 600,000 highway bridges are currently in service and being used every day to accommodate personal and freight travel [1]. A large percentage of these bridges (80% of all the interstate bridges) were constructed between 1950 and 1980 [1]. Figure 1 shows the breakdown of interstate bridges built in each decade. Generally, these bridges were designed for a theoretical design life of 50 years. Many of the U.S. bridges are now approaching and surpassing their design life and are structurally deteriorating. These bridges are also often subjected to higher load demands as the average daily traffic (ADT) on the bridges increases and the trucks traveling across become heavier.

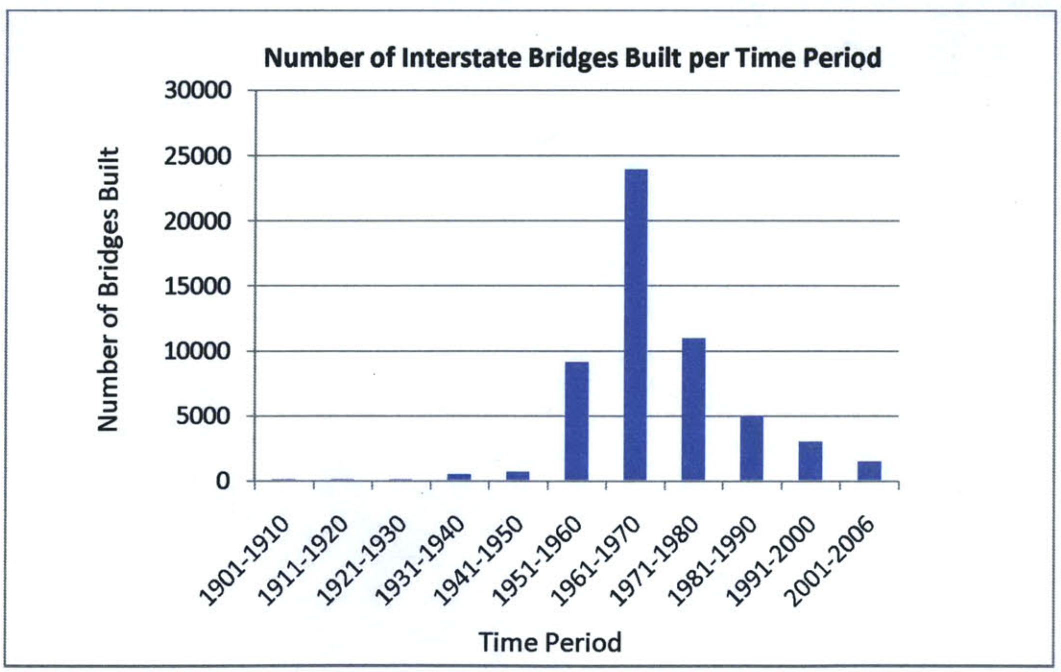


Figure 1: Graph of interstate bridges built per time period [1]

A 2008 *Conditions and Performance Report* [1] released by the Federal Highway Administration reported 27.6% of all bridges are structurally deficient. These structurally inadequate bridges require immediate attention. A large amount of time, effort, money, and resources is currently allocated to repairing, rehabilitating, or replacing these bridges. In a 2007 Report to Congress by the U.S. Department of Transportation [2], it is stated that the current expenditures on rehabilitation of existing bridges is $10.5 billion per year and it is estimated that a total of $65.3 billion would be needed to fix all existing bridge deficiencies through rehabilitation.

There are two basic methods to fixing deficient bridges. One is a complete replacement of the existing deficient bridge. This, however, is a quite time consuming and costly solution and therefore seldom employed as the immediate fix. Rehabilitation through repairs and retrofits is generally a more realistic and viable solution for fixing bridges with structural deficiencies. Therefore, efficient, timely implemented, and cost effective methods to extend the lifespan of existing bridges are of great interest.

Fatigue is one of the major causes of structural deterioration of bridges. A bridge is subjected to constant cyclic loading due to heavy traffic throughout its service life. This repeated loading and unloading of the structure causes structural damage over time due to fatigue in the structural elements. Therefore, a promising way to extend the useful lifespan of a bridge is by reducing the amount of fatigue the structure undergoes.

When a bridge carries heavy traffic, vibrations are induced in the bridge subjecting structural elements to high levels of stress. This stress subjects the bridge to fatigue. Methods to mitigate these vibrations are currently being investigated and implemented as a way to reduce fatigue and extend the service life of bridges. By finding easy, efficient, and effective ways to reduce traffic-induced vibrations in bridges, the number of bridges structural deficient in the U.S. will diminish.

There are three basic methods to reduce the amount of vibrations a bridge is subjected to. One is to reduce the magnitude of the loading (i.e. reduce the weight of the vehicles). This, however, is often not a viable option due to the large percentage of freight traffic that depends on traveling across these bridges. Modifying the structural system is another option. This option is currently the method usually chosen. However, it is generally a quite time consuming and costly solution and greatly impedes and interferes with the current service of the bridge. Finally, integrating a structural control device is an option. This device can usually be retrofitted to the existing bridge fairly easily and quickly and can greatly help reduce bridge vibrations. [3]

Integration of control devices is a promising solution to reduce bridge vibrations effectively and efficiently. By reducing vibrations though control devices, the service life of existing bridges can be greatly increased.

1.2 Objective

The objective of this thesis is to explore and investigate methods of mitigating traffic-induced bridge vibrations through the integration of control devices to extend the service life of bridges. There are three main classes of structural control devices that are currently being researched, developed and implemented: passive, semi-active, and active control. Each control scheme has advantages and disadvantages and will be discussed and explored further here.

In this thesis, an overview of each scheme will be given. In chapter 2, examples of different control devices, along with their advantages and disadvantages will be discussed. In chapter 3, to gain a better understanding of both passive and semi-active control strategies, a computer simulation is conducted. The computer simulation will allow for an added comparison between passive and semi-active control schemes. Based on the research and simulation conducted, a recommended control scheme is suggested for implementation.

2 Control Strategies

As mentioned earlier, there are three basic control strategies which can be implemented to achieve reduction in bridge vibrations: passive control, active control, and semi-active control. Each control scheme has advantages and disadvantages and many factors must go into determining which scheme is the best to implement for a given situation.

Control devices of all three types (passive, active, and semi-active) have been studied, developed, and implemented as a means to mitigate and control vibrations in civil structures. Particular emphasis has been placed on mitigation of vibrations in buildings and bridges due to wind and seismic excitations. The U.S. and Japan have been the leaders in development and full-scale implementation of different control schemes and devices [4]. The benefits and advantages of using such devices have been seen. The research and implementation of these devices have shown great potential as a means to make safer and more economical structures.

Although the majority of attention has been focused on controlling vibrations due to wind and seismic loading in buildings and bridges, investigation into controlling vertical vibrations has also been conducted. Vertical forces (vehicles, trains, pedestrians...) traveling along the bridge induces vertical vibrations (as opposed to lateral vibrations due to wind or earthquake excitations). These frequent vibrations to the structure take a toll on the structural system and shorten the service life of the bridge and therefore potential solutions warrant further investigation and research.

2.1 Passive Control

When a structure is subjected to an excitation without any control scheme, it will respond to the excitation fully. With a passive control scheme, when a structure is subjected to an excitation, the passive control device dissipates energy from the system and the response of the structure is reduced. Figure 2 schematically represents a structure with no control scheme and one controlled by passive energy dissipation (PED).

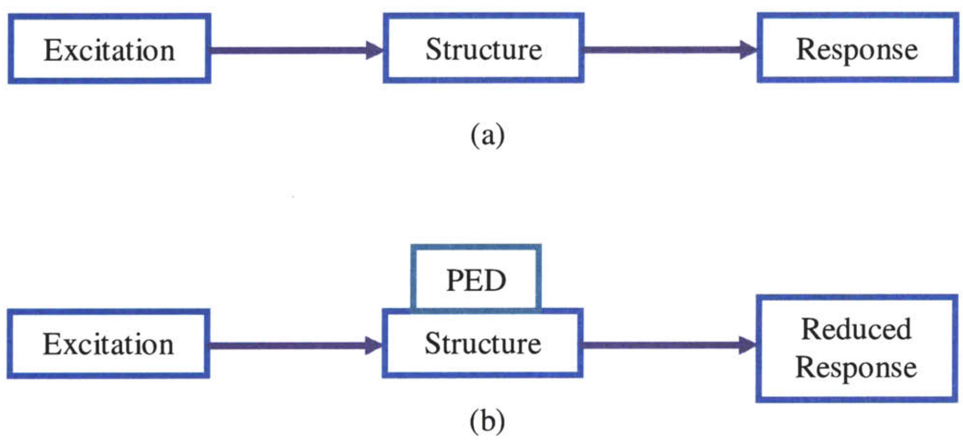


Figure 2: Schematic represent of a structure with (a) no control scheme and (b) passive control

Passive control has been implemented in many civil structures to help control and mitigate vibrations. A passive control scheme dissipates energy in a passive manner; no external power source is required. The parameters of a passive control device are fixed and therefore are only effective for a specific range of excitations.

There are several different types of passive control devices. Tuned mass dampers (TMDs) and fluid viscous dampers (FVD) are two common devices used to control vertical vibrations in bridges.

2.1.1 Tuned Mass Damper (TMD)

A tuned mass damper (TMD) is a secondary vibration system attached to the main structure at certain locations that dissipates energy of the primary structure. The TMD consists of a comparatively (to that of the bridge) lightweight mass, a spring, and a damper. The frequency of the damper is tuned in such way that when the primary structure experiences a certain vibrational frequency, the damper will resonate out of phase with the motion of the main structure. The damper inertia force acting on the structure dissipates energy. Usually the TMD is tuned to the fundamental frequency of the bridge as the first mode is usually of most concern. A basic schematic of a TMD system is depicted in Figure 3. [5]

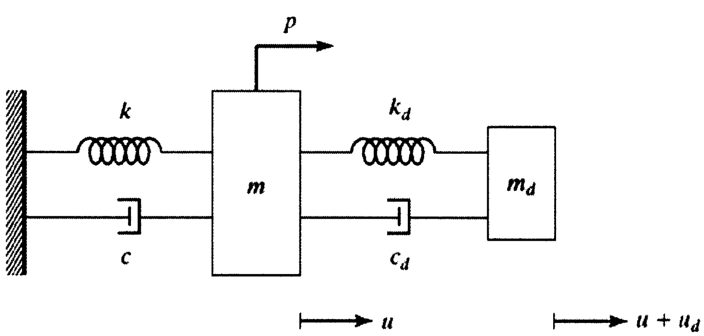


Figure 3: Schematic of a TMD system [5]

Much research and investigation into the use of TMD as a means to suppress vertical vibrations in bridges has been undertaken. Through computer simulations, Shi *et al.* [6] studied the effectiveness of using TMD to mitigate vehicle-induced bridge vibrations. Their finding showed that the TMD was effective at greatly reducing the free vibrations of the bridge. The reduction in maximum dynamic displacement during the forced vibration period (i.e. when the vehicles are on the bridge) was not as great as that found for free vibration. This was attributed to the fact that the forced vibration period is too short and the TMD does not have enough time to respond. Also, free vibration is usually more a single-mode dominated vibration (which is what the TMD is tuned to) than that of the forced vibration, making a TMD better suited for free vibration mitigation. It was also found that the application of a TMD was more effective for short bridges. Short bridges have a large fundamental frequency and therefore free vibrations are more active due to the high multi-axle truck load frequency. A TMD is more effective at suppressing dynamic effects (free vibration) due to several trucks moving in a row rather than when only one truck is traveling across. [6]

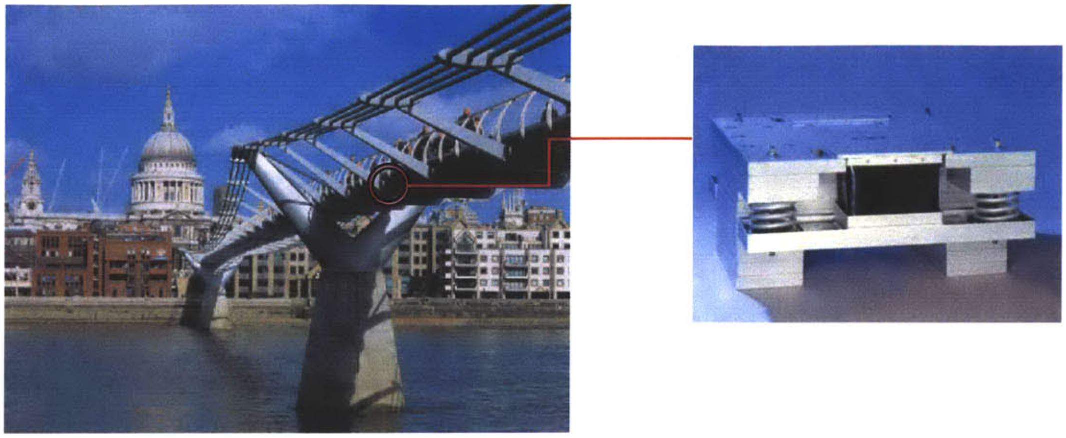
Others have found similar results to that of Shi *et al.* Kumar [7] found similar results when studying, through a finite element approach, the use of TMD for bridge vibration mitigation. Kumar also noted that although the TMD did not reduce displacement significantly during the forced vibration period, it was able to greatly reduce acceleration. Karoumi [8] analytically studied the vibrations of a cable-stayed bridge with a TMD in the middle. The findings of this study also indicated that TMD are more effective at reducing the maximum dynamic response during free vibration as opposed to forced vibration. Kwon *et al.* [9] and Klasztrony [10] analytically studied reducing train-induce vibration through TMD. They found reductions of 21% and 60% respectively in maximum vertical displacements.

Tuned mass dampers have been implemented to help mitigate vertical vibrations in many bridges around the world. Pedestrian bridges, because they are usually light and flexible, are greatly susceptible to vertical vibrations. Therefore, many pedestrian bridges commonly implement TMD to help suppress these vibrations. Although not as common, other bridge types (e.g. highway and train) have also implemented TMD as a means to suppress vertical vibrations. The Germany company GERB has a commercially available TMD that has been incorporated into many bridges around the world [11]. Table 1 lists several different projects where GERB TMD have been utilized. Typical TMD and implementation on bridges are pictured in

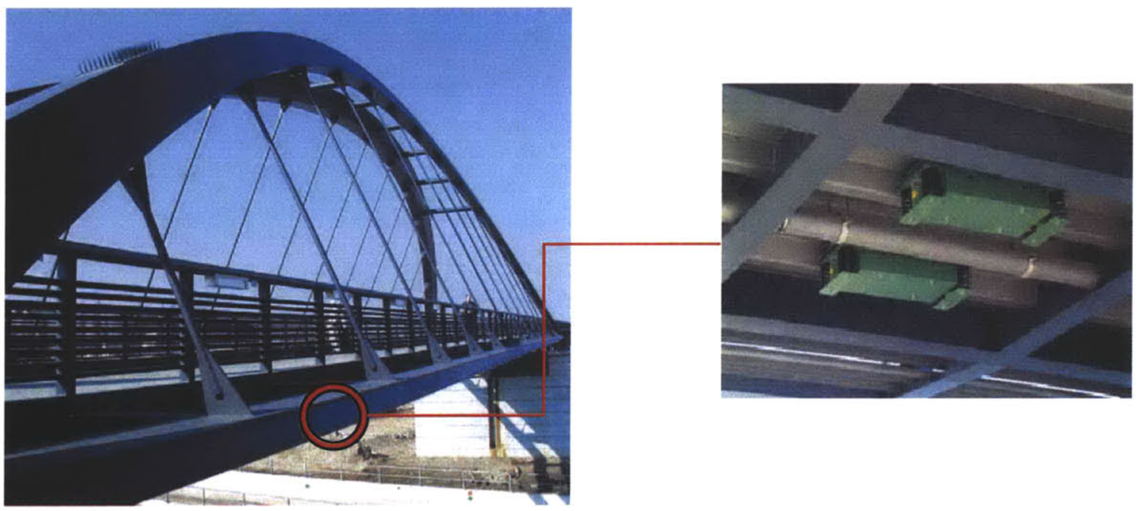
Figure 4.

Table 1: Projects with GERB TMD installed to reduce vertical bridge vibrations [11]

Country	**Project**	**Year**
Denmark	Footbridge	1999
France	Paris, Stade de France, Footbridge	1997
	Paris, Solferino Footbridge	2000
Germany	Hannover Exhibition Center, Bridge	1984
	Dorsten, Footbridge	1990
	Kassel, Footbridge	1998
	Berlin, Schwedter Str., Footbridge	1999
	Berlin, Bundeskanzleramt, Footbridge	2000
	Berlin, Britzer Damm, Footbridge	2001
	Freilassing, Footbridge	2002
Great Britain	Inverness, Kessock Bridge	1989
	London, Millennium Bridge	2001
	Coventry, Footbridge	2003
Iceland	Footbridge	1999
South Korea	Seoul, Sun You Footbridge	2002
Norway	North Trondelag, Bridge	1989
	Mjasundet Bridge	1992
	Bulandet/Vaerlandet, 3 Bridges	2002
Poland	Wroclaw, Footbridge	2003
Switzerland	Rumlang, Footbridge	1992
Thailand	Bangkok, Chao Phya Bridge	1985



(a)



(b)

Figure 4: TMD implemented on bridges (a) Millennium Bridge – London (b) Schwedt Bridge – Berlin, Germany [11]

TMD can provide a valid means of reducing dynamic structural response; there do, however, exist some drawbacks. Although the device weight relative to the primary structure is quite small, the actual weight of the device can be quite substantial. This added weight to the bridge structure can effectively reduce the capacity of the bridge [3]. As with all passive devices, TMD have fixed parameters and are only effective for one forcing frequency. A custom TMD must be designed for each bridge, which makes it a less desirable method for implementation on a large number of bridges across the US.

2.1.2 Fluid Viscous Dampers (FVD)

Fluid viscous dampers (FVD) can be used for structural vibration control. Fluid viscous dampers dissipate energy through fluid volume variation and heat loss by forcing fluid through orifices. A schematic diagram of a FVD is pictured in Figure 5. A FVD is comprised of a cylinder with two chambers, a piston rod, a piston head with orifices, and a fluid. As the piston rod moves, the fluid is forced through the orifices of the piston head and moves from one chamber to the next. This action creates a resisting force. The resisting force depends of the velocity of the rod and the viscosity of the fluid. The force can be expressed with the mathematical equation:

$$F = c * v$$

$where$:

$c = damping\ coefficient$

$v = velocity\ of\ piston$

For passive FVD, the damping coefficient, c, is fixed. Frequently, the force is a nonlinear function of velocity:

$$F = c * v^{\alpha}$$

The exponent α is typically a value between 0.1 and 1. Because linear viscous damping is more mathematically convenient to deal with, it is the preferred way to represent the energy dissipation. [5] [12] [13]

The fluid is usually silicone oil. Silicone oil possesses natural qualities which makes it an excellent choice for this application. It is inert, non flammable, non toxic, and long lasting (does not degrade with age). [12] [13]

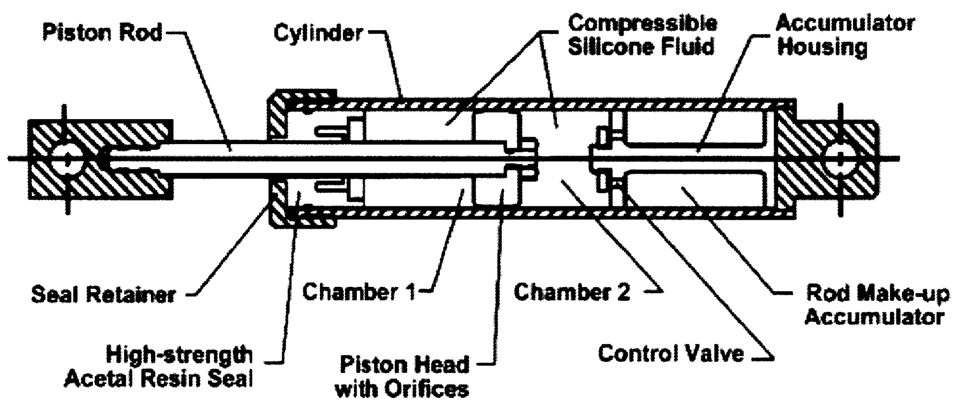
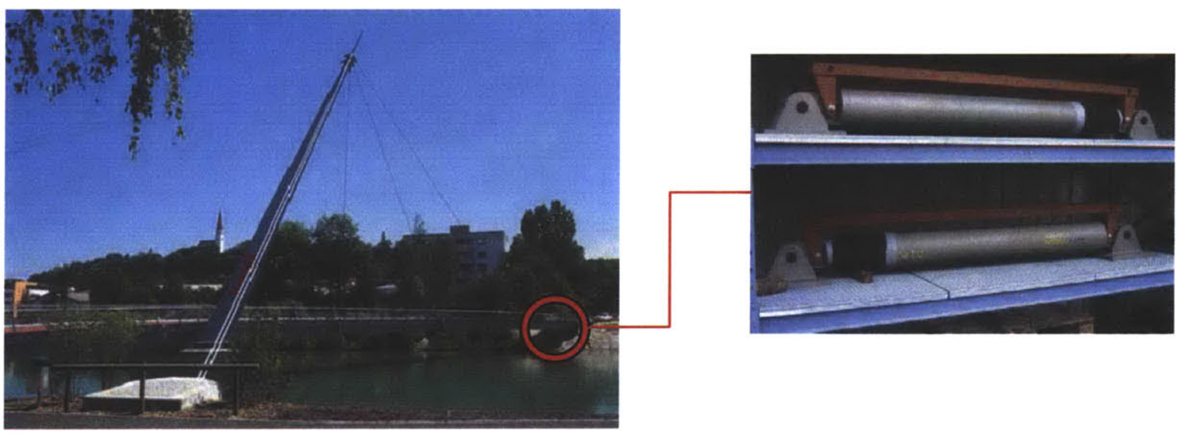


Figure 5: Schematic diagram of a FVD [13]

Studies to investigate the use of FVD as a means to suppress vertical vibrations in bridges have been conducted. Martinez-Rodrigo *et al.* [14] analytically studied mitigation of vibrations of short simply supported railway bridges with the use and implementation of FVD. They proposed and evaluated connecting FVD to the bridge slab and an auxiliary structure (a simply supported beam located below the bridge) to control vertical bridge vibrations. It was found that a reduction of 80.9% for midspan acceleration and 58.8% for midspan displacement could be achieved with this proposed system. [3] [14]

Many bridges around the world have implemented FVD to help suppress unwanted vibrations. FVD are commonly used to mitigate vibrations induced by seismic excitations. However, they have also been incorporated into bridge design to help control vertical vibrations in bridges. For example, a MAURER SOHNE viscous damper was added to the footbridge Traunsteg in Wels, Austria to reduce vertical vibrations, as shown in Figure 6 [15]. The Millennium Footbridge in London was also retrofitted with FVD to help reduce unwanted pedestrian-induced vibrations. FVD were used to control both vertical and horizontal vibrations [12]. Figure 6 shows the added vertical dampers to the Millennium Bridge.



(a)

(b)

Figure 6: Implementation of viscous damper on (a) Traunsteg Bridge- Wels, Austria (b) Millennium Bridge – London [15] [12]

As with TMD, FVD have the disadvantage of possessing fixed properties that must be designed for only one given scenario. This can limit the wide-spread use and implementation of FVD as a means to decrease vertical bridge vibrations in the US.

2.1.3 Other Passive Energy Dissipation Methods and Devices

Other passive energy dissipation methods (although not as common or widely used as TMD or FVD) have been utilized to help suppress vibrations in bridges. Such devices include tuned liquid column dampers (TLCD) and tuned liquid dampers.

A TLCD consists of a U-shaped tube filled with fluid (commonly water). TLCD works on the same principle of a TMD except that with a TLCD, the movement of the liquid in the tube counteracts the movement of the structure. A TLCD is an advantageous device to use because it is easily tuned, has simple construction, and has very low maintenance costs [16]. Similar to a TLCD, a tuned liquid damper consists of a rigid tank filled with liquid whose movement counteracts the movement of the structure. These devices, however, are more efficient and effective at reducing horizontal vibrations rather than vertical. Therefore, they are often implemented in bridges where horizontal vibrations are of more concern than vertical. These devices may provide some reduction in vertical vibration as well. For example, TLCD have also been found to help reduce vertical vibrations [3] [17].

Passive control techniques are a popular and effective method to mitigate vertical vibrations in bridges and are a widely implemented solution. Passive control has limitations. Passive control devices have fixed properties that cannot be modified in real-time. Once a passive device is installed, it cannot be easy changed or modified. Therefore, a reliable estimate of design loads and an accurate understanding of the structure's dynamic response are needed to design and implement an effective passive control strategy. Passive control devices can only be designed to tune out certain specified frequencies and are essentially ineffective for all other dynamic responses the structure may experience.

2.2 Active Control:

Active control is a way to overcome the limitation of parameter fixity that exists with passive control schemes. Active control allows for real-time control and the ability to better control for a wide range of scenarios and excitations. For a given excitation, the active control scheme is able to determine the optimal forces needed to control the response of the structure. The basic configuration of an active control scheme is schematically shown in Figure 7. Three main components make up an active control system: (1) sensors, which measure either external excitations and/or structural response variables; (2) a computer controller, which processes the measured information and computes the necessary force needed based on an optimal control algorithm; and (3) actuators, which are physical devices that produce the required forces needed for optimal control. The actuators usually require a power source to operate. With active control, mechanical energy is directly added to the structural system in order to control the response, as opposed to passive where no energy is added to the system. [18] [5]

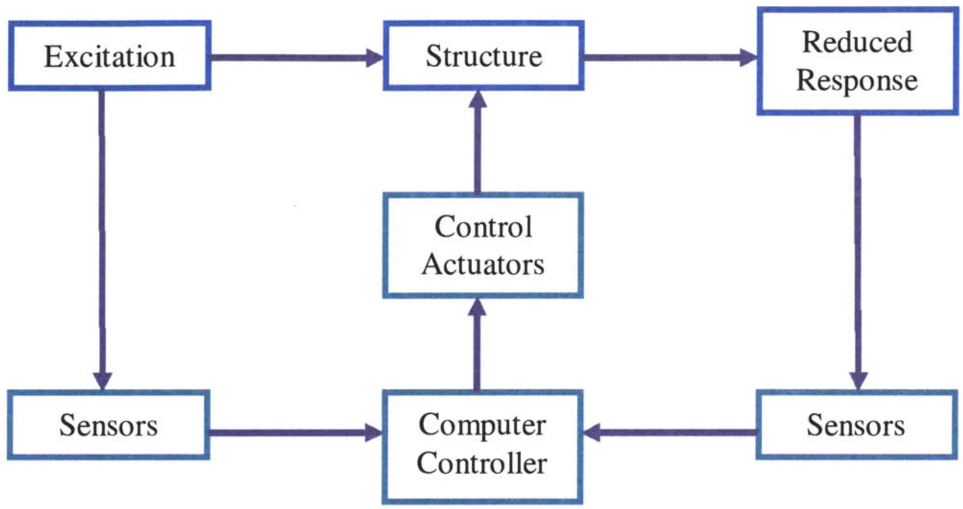


Figure 7: Schematic represent of a structure with active control

When only the structural response variables are measured and used as the input for the computer control system, the control configuration is referred to as feedback control. The structural response is continually being monitored and measured and this information is used to make continual adjustments to the actuator force in order to produce a response as close to the desired response as possible. When only the external excitation is measured and used as the input for determining the needed control forces, the control configuration is referred to as feedforward control. When both quantities are measured and used for the control design, it is called feedback-feedforward control. [18]

The utilization and implementation of active control schemes for seismic control has received a considerable amount of attention and investigation. Although use for seismic control seems to be a more popular interest, the use of active control schemes to reduce traffic-induced bridge vibrations has also been investigated and analyzed. Haji-Hosseinloo *et al.* [19] proposed and evaluated an active control scheme to reduce bridge vibrations under a heavy truck load. The strategy the authors employed (which was of a feedback-feedforward type) was greatly able to control maximum deflections, reducing midspan deflection by nearly 100%. The analysis concluded that the optimal positions of the acting control forces are the first and last one-fourth of the bridge length. For optimal control, the maximum needed control force for each actuator was found to be approximately 18 kips (or 80kN). This is a considerable force and with active control it is always a tradeoff between the power cost and the controlled output. [19]

Although, active control has the potential of suppressing vibrations almost completely, several drawbacks exist in implementing active control schemes. As previously mentioned, there is always a tradeoff between optimal control output and power cost. A large amount of power is often needed to completely suppress vibrations. This can be quite costly and pose problems when there is a power outage. There is also the potential for the system to become unstable due to the fact that energy is added to the system. For a variety of reasons (sensor malfunction, differences between actual bridge properties and response and the assumed properties and response for control design, mistake in control algorithm ...) the actuator can apply an unwanted and undesirable force to the structure, causing the system to become unstable.

2.3 Semi-Active Control

Semi-active control is a method that seems to combine of the best qualities of both passive and active control strategies. Semi-active control devices possess the capability to adapt, like that of an active control device, but without requiring a large source of power (many can be operated on battery power only). Like passive control, semi-active control does not have the potential to destabilize. Semi-active control devices do not add mechanical energy into the system and therefore there is no threat of destabilization. Unlike passive control devices, the properties of semi-active devices can be controlled to optimally reduce the response of the system for a wide array of dynamic loading conditions. Figure 8 schematically shows the basic configuration of a semi-active control scheme. Semi-active control appears to be a promising and viable means of controlling dynamic responses of civil structures. [18]

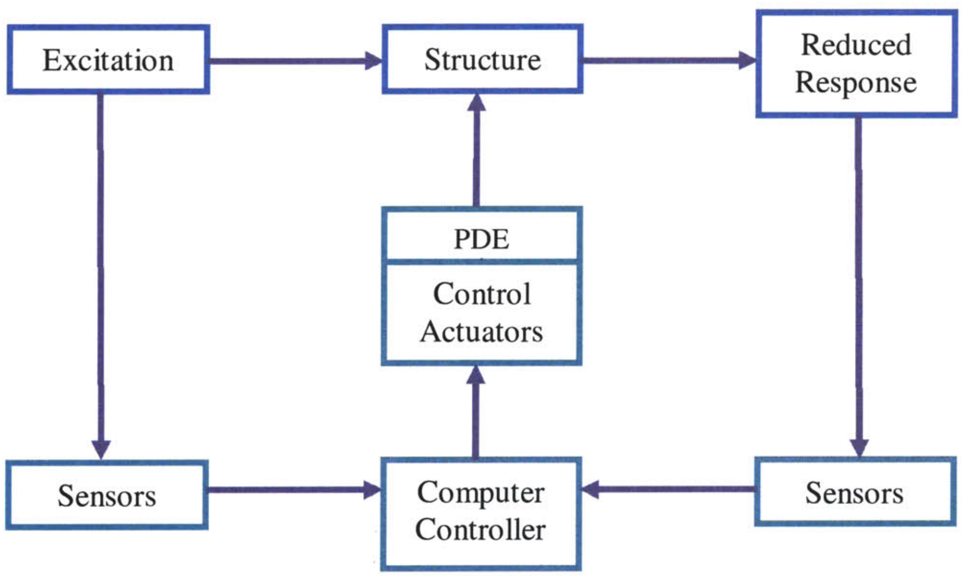


Figure 8: Schematic represent of a structure with semi-active control

The use of semi-active control schemes for mitigation of traffic-induced bridge vibrations has been both analytically, experimentally, and field tested. Patten *et al.* [20] investigated analytically and experimentally the effectiveness of a semi-active vibration absorber on a 40 feet (12.2 meter) single-lane bridge they constructed. A schematic of the bridge with the semi-active actuator is depicted in Figure 9. The results of the study showed a more than 70% reduction in deflection with the semi-active strategy [20]. An analytical study conducted by Christenson showed a reduction of maximum midspan displacement of 50% can be achieved with a semi-active control scheme, while only a 31% reduction was achieved using a passive damper control method [3]. Semi-active control methods for reducing vertical vibration are a promising solution that warrants further investigate and research.

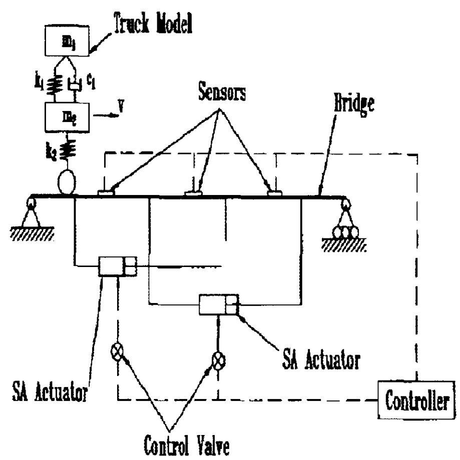


Figure 9: Schematic of the bridge with the semi-active actuators study by [20]

There are several different types of semi-active control devices that are currently being studied, developed, and implemented as a way to help mitigate vibrational effects in structures. Such devices include: smart tuned mass dampers; controllable fluid dampers; and variable-orifice dampers.

2.3.1 Smart Tuned Mass Dampers

Properties of passive tuned mass dampers cannot be modified in real-time and therefore can only offer response reduction for a certain specified frequency for which it is tuned. With semi-active tuned mass dampers (or smart tuned mass dampers), adaptive tuning of the TMD can be achieved through the use of a semi-active variable stiffness system. One such variable stiffness device, the SAIVS, has been developed and studied by Nagarajaiah and Varadarajan [21]. The SAIVS device integrated into a tuned mass damper system is depicted in Figure 10. It is comprised of four spring elements arranged in a plane rhombus configuration with pivot joint at the vertices. A linear electromechanical actuator is used to change the configuration of the rhombus. When the angle θ (the angle of the spring elements with the horizontal) decreases, joints 1 and 2 move closer together casing the stiffness of the SAIVS device to increases. Alternately, when joints 3 and 4 move closer together (when θ is large) the stiffness of the device is decreased. The variable stiffness of the SAIVS device is described by the mathematical equation: [21]

$$k(t) = k_e cos^2(\theta(t))$$

$where:$

$k(t) = time\ varying\ stiffness\ of\ device$

$k_e = constant\ spring\ stiffness\ of\ each\ spring\ element$

$\theta(t)) = time\ varying\ angle\ of\ the\ spring\ elements\ with\ the\ horizontal$

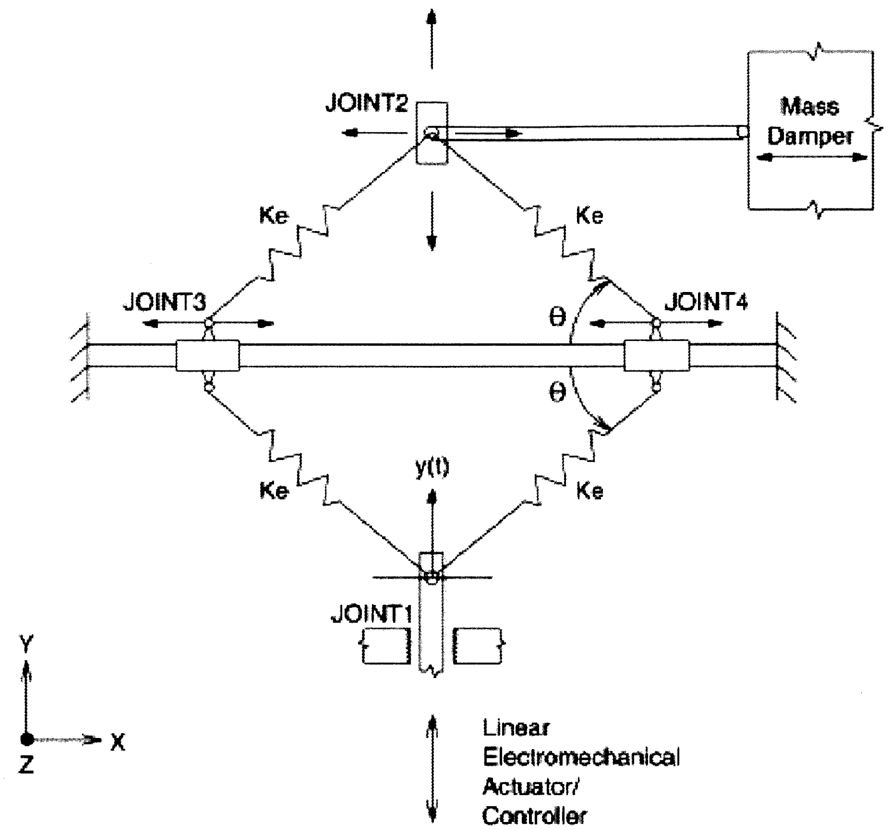


Figure 10: SAIVS device integrated into a tuned mass damper system [21]

2.3.2 Controllable Fluid Damper

Another common semi-active device is a controllable fluid damper which is schematically represented in Figure 11. This device is similar to the passive fluid viscous dampers, except it incorporates a controllable fluid. Two popular and viable fluids currently being studied and developed for use in controllable fluid dampers are electrorheological (ER) fluids and magnetorheological (MR) fluids. When exposed to either an electric (for ER fluids) or magnetic (for MR fluids) field, these two fluids have the ability to change from a free-flowing, linear viscous fluid to a semi-solid with controllable yield strength in milliseconds. By controlling the properties of the fluid, the damping force produced by the device can be controlled and therefore has the ability to control a wide range of dynamic responses. For analysis and design, the material is often idealized as a Bingham solid. A Bingham solid is classified as an ideal plastic solid in parallel with a linear viscous fluid. The total yield stress can be expressed by the equation: [5] [18]

$$\tau = \tau_{y(field)} sgn(\dot{\gamma}) + \eta_p \dot{\gamma}$$

$where:$

$\tau_{y(field)} = yield\ stress\ caused\ by\ the\ applied\ field$

$\dot{\gamma} = shear\ strain\ rate$

$$sgn(\dot{\gamma}) = \begin{bmatrix} 1 & \dot{\gamma} > 0 \\ 0 & \dot{\gamma} = 0 \\ -1 & \dot{\gamma} < 0 \end{bmatrix}$$

$\eta_p = plastic\ viscosity:$

$defined\ as\ the\ slope\ of\ the\ measured\ shear\ stress\ versus\ shear\ strain\ rate\ data$

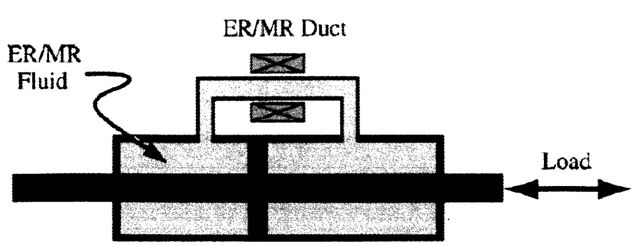


Figure 11: Schematic of controllable fluid damper [4]

For many years, much research and development has focused on ER and MR fluid devices. Currently ER fluids have a few disadvantages that make them less desirable for use in semi-active devices. ER fluids currently have a relatively low yield stress of only 0.4 to 0.5 psi (3.0 to 3.5 kPa) and cannot tolerate common impurities such as water which could easily be introduced during manufacturing or operation. ER fluids also require high voltage (around 4000 V) power supply to operate. [18]

MR fluids may be a better fluid for use in controllable fluid dampers. MR fluids are typically made up of micron-sized, magnetically polarization particles that are dispersed in a carrier medium like mineral or silicone oil. Research has indicated that MR fluid is able to achieve a yield stress that is an order of magnitude larger than that of an ER fluid. MR fluids are not sensitive to impurities, can operate in temperatures ranging from -40 degrees Fahrenheit (-40°Celsius) to 302 degrees Fahrenheit (150°Celsius), and require low voltage (around 12-24V) to operate. [22]

2.3.3 Variable-Orifice Damper

Another semi-active damping device is a variable-orifice damper. This device is similar to a conventional hydraulic fluid damper except that it uses a controllable, electromechanical, variable-orifice valve to alter the resistance to flow. Patten *et al.* [23] developed a variable-orifice damper (referred to in the literature as a hydraulic semi-active vibration absorber -SAVA). A schematic of the device is pictured in Figure 12.

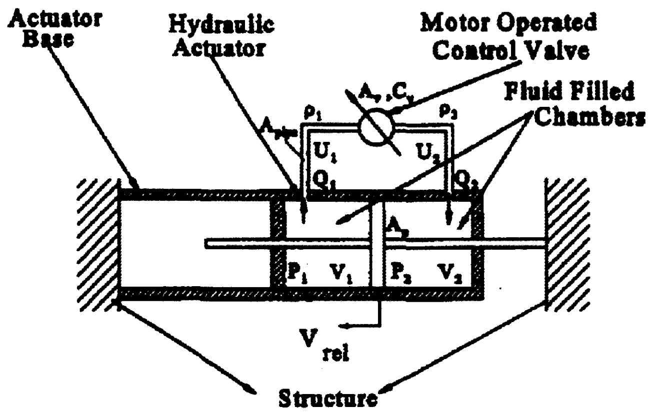


Figure 12: Schematic of variable-orifice damper (SAVA) [23]

The dynamics of the actuator can be expressed with the mathematical formula:

$$\Delta\dot{P} = \alpha A_p V_{rel} - \alpha C_d A_v sgn(\Delta P)\sqrt{\frac{2|\Delta P|}{\rho}}$$

$$\alpha = \frac{\beta(V_1 + V_2)}{V_1 V_2}$$

where:

$A_p = effective\ area\ of\ piston$

$V_{rel} = relative\ velocity\ between\ the\ piston\ and\ the\ cyliner$

$C_d = discharge\ coefficient$

$A_v = orifice\ area\ of\ valve$

$\Delta P = differential\ pressure\ between\ the\ two\ chambers$

$\rho = density\ of\ the\ fluid$

$\beta = bulk\ modulus\ of\ fluid$

$V_1 = volume\ of\ chamber\ 1$

$V_2 = volume\ of\ chamber\ 2$

$$sgn(\Delta P) = \begin{bmatrix} 1 & \Delta P > 0 \\ 0 & \Delta P = 0 \\ -1 & \Delta P < 0 \end{bmatrix}$$

Patten *et al.* [24] conducted a full scale field test of the SAVA device. The device was installed on the Walnut Creek Bridge on Interstate 35 in Oklahoma. The bridge was constructed in 1971 and is subjected to large truck traffic everyday (as there is a quarry located south of the site which supplies much of the gravel used in the Oklahoma City area). The bridge superstructure exhibits visually detectable deflections when trucks cross over, making it a good candidate for the study of the implementation of the SAVA system. [24]

The SAVA system was installed on the bridge in three weeks and traffic was never impeded during construction. Figure 13 shows a picture of the SAVA system installed on the bridge. A full-state control algorithm based on Lyapunov stability theory was implemented to control the SAVA device. Many sensors (including a total of 36 Piezo-resistive accelerometers) were installed on the bridge to gather structural response data. The SAVA system is able to reduce the displacements and velocities of the girders which in turn decreases the maximum stresses in the bridge. The field test conducted by Patten *et al.* [24], indicated that the added SAVA system will be able to increase the safe life of the bridge by roughly 50 years.

Figure 13: SAVA system installed on the Walnut Creek Bridge [22]

Zeng *et al.* [25] conducted a study where, instead of implementing a full-state control algorithm, a bi-state Lyapunov control algorithm was employed for the SAVA system on the Walnut Creek Bridge (this modified system is referred to as SAVAII). The full-order controller requires a complex sensor layout, whereas the modified controller requires only two states and three local sensors: one linear variable differential transformer (LVDT) and two absolute pressure sensors per actuator. A CPU is also installed within the actuator. This allows for all the control devices to be completely installed within the actuator before it is placed on the bridge. The modified controller also reduces the required computational power that a full-order controller requires. The study of the SAVAII system showed that the peak stresses in the bridge were reduced by 33%. The original SAVA system (referred to as SAVAI) with the full-order controller was able to have a 40% reduction, however. Although the SAVAI system provides slightly more stress reduction than the SAVAII system, the SAVAII system, because of the features of the SAVAII system previously mentioned, is much more cost effective and practical for implementation on large structures. [25]

3 Simulation Study: Comparing Passive and Semi-Active Control Systems

Because active control strategies have the potential to become unstable and require a large power source, concerns exist about full-scale implementation and wide use on bridges. Therefore, only passive and semi-active control schemes are investigated in this simulation study. To get a basic comparison between these two control schemes, a relatively simplistic method and model was employed for this simulation study.

3.1 Methodology

A similar approach to that of [3] was employed for this study. A model of typical highway bridge subjected to truck loading was developed. Passive and semi-active control strategies were then applied and studied.

3.2 Bridge Model

The properties for the bridge model were taken from [3] and are based on an existing bridge, the Cromwell Bridge located on Interstate 91 in Connecticut. The Cromwell Bridge spans a total of 213 feet and consists of 3 separate simply supported spans of lengths roughly equal to 75 feet, 75 feet, and 63 feet. The first 75-foot span was selected for modeling. The bridge is approximately 51-foot wide and carries three lanes of highway traffic.

The 75-foot span was modeled as a pin-roller Euler-Bernoulli beam. For an Euler-Bernoulli beam model, only flexural deformations and transverse inertia forces are taken into account and the shear deformations and rotational inertia effects are neglected. The flexural waves of this type of beam can be expressed by the equation:

$$\rho A\,\frac{\partial^2 y}{\partial t^2} + \frac{\partial^2}{\partial x^2}\left(E(x)I(x)\frac{\partial^2 y}{\partial x^2}\right) = p(x,t)$$

$where:$

$x = axial\ coordinate$

$y(x,t) = vertical\ displacement$

$p(x,t) = distibuted\ lateral\ body\ load$

$E = Young's\ modulus$

$\rho = mass\ density$

$A = cross - section\ of\ beam$

$I = moment\ of\ inertia\ of\ cross - section$

For this model, the cross-section and cross-section properties of the bridge are assumed to be constant along the length of the bridge. Therefore the above equation can be simplified to:

$$\rho A\,\ddot{y} + EI\frac{\partial^4 y}{\partial x^4} = p(x,t)$$

The continuous system expressed with the partial differential equation of motion above has infinitely many degrees of freedom and is quite complicated and not convenient to work with mathematically. Therefore, the Galerkin Method can be used to reduce the continuous system into a discrete model with a finite number of degrees of freedom. The Galerkin Method expresses the vertical displacement in terms of trail functions (or assumed modes), $\Psi_j(x)$, and generalized coordinates, $q_j(t)$:

$$y(x,t) = \sum_{j=1}^{DOF} \Psi_j(x) q_j(t) = \boldsymbol{\Psi}(\boldsymbol{x})\boldsymbol{Q}(\boldsymbol{t})$$

$where:$

$\Psi_j(x) = \; the\, jth\, trial\, function$

$\boldsymbol{\Psi} \quad = matrix\, representation$

$q_j(t) = the\, jth\, generalized\, coordinate$

$\mathbf{Q} \quad = matrix\, representation$

$DOF = number\, of\, desired\, degrees\, of\, freedom$

The assumed modes are a function of position, x, along the beam only and the generalized coordinates are a function of time only. The equation of motion for the beam can therefore be written as:

$$\rho A \frac{\partial^2(\boldsymbol{\Psi}(x)\boldsymbol{Q}(t))}{\partial t^2} + EI\frac{\partial^4(\boldsymbol{\Psi}(x)\boldsymbol{Q}(t))}{\partial x^4} = p(x,t)$$

or rewriten as:

$$\rho A \boldsymbol{\Psi}(\mathrm{x})\ddot{\boldsymbol{Q}}(t) + EI\boldsymbol{\Psi}''''(\mathrm{x})\mathbf{Q}(\mathrm{t}) = p(x,t)$$

$where\; [\cdot] indicates\; a\; dervative\; with\; respect\; to\; time, t,$

$and\; ['] indicates\; a\; dervative\; with\; respect\; to\; position, x.$

The equation can then be expressed in terms of a mass matrix, ***M***, a stiffness matrix ***K***, and a load vector ***P(t)***:

$$\boldsymbol{M}\ddot{\boldsymbol{Q}}(t) + \boldsymbol{K}\boldsymbol{Q}(t) = \boldsymbol{P}(t)$$

$where:$

$$\boldsymbol{M} = \rho A \int_0^L \boldsymbol{\Psi}^T(x)\,\boldsymbol{\Psi}(x)\partial x$$

$$\boldsymbol{K} = EI \int_0^L \left(\boldsymbol{\Psi}''^T(x)\right)(\boldsymbol{\Psi}''(x))\partial x$$

$$\boldsymbol{P}(t) = \int_0^L \left(\boldsymbol{\Psi}^T(x)\right)(p(x,t))\partial x$$

$L = length\ of\ bridge$

For this study, the bridge is assumed to have no damping. However, inherent or proportional damping can be used as done in [3]. The damping matrix, $\boldsymbol{C}$, is found by the equation:

$$\boldsymbol{C} = \boldsymbol{\Phi}(x)\bar{C}\boldsymbol{\Phi}^{-1}(x)$$

$where:$

$\boldsymbol{\Phi}(x) = matrix\ mode\ shapes\ obtained\ by\ solving\ the\ eigenvalue\ problem\ of\ \boldsymbol{M}\ and\ \boldsymbol{K}$

$\bar{C} = diagonal\ modal\ damping\ matrix$

Rayleigh damping can also be used. With Rayleigh damping, the damping matrix, $\boldsymbol{C}$, is expressed by the equation:

$$\boldsymbol{C} = a_0\boldsymbol{M} + a_1\boldsymbol{K}$$

$where\ a_0\ and\ \ a_1\ can\ be\ found\ by:$

$$\xi_j = \frac{1}{2}\left[\frac{a_0}{\omega_j} + a_1\omega_j\right]$$

$\xi_j = damping\ ration\ of\ the\ jth\ mode$

$\omega_j = natural\ frequency\ of\ the\ jth\ mode$

The trial functions selected for this study are:

$$\Psi_j(x) = \sin\left(\frac{j\pi x}{L}\right)$$

These functions are the closed-form eigenfunction of a simply supported bending beam. Other choices for trial functions could have been possible. For example, the deflected shape associated with static forces applied at the same location of the control forces could have been implemented. The trial function just needs to satisfy all the boundary conditions. However, the closer the trail functions are to the exact solution the better and more accurate the bridge model will be.

To solve the equation of motion, it is convenient to convert this second-order differential equation into two first-order differential equations by employing a state space formulation. The state space formulation for this system is:

$$\dot{x} = \boldsymbol{A}x + \boldsymbol{B}u$$

$$y = \boldsymbol{C}_{ss}x + \boldsymbol{D}u$$

where:

$$x = \begin{bmatrix} \boldsymbol{Q}(t) \\ \dot{\boldsymbol{Q}}(t) \end{bmatrix}$$

$$\boldsymbol{A} = \begin{bmatrix} \boldsymbol{0} & \boldsymbol{I} \\ -\boldsymbol{M}^{-1}\boldsymbol{K} & -\boldsymbol{M}^{-1}\boldsymbol{C} \end{bmatrix}$$

$$\boldsymbol{B} = \begin{bmatrix} \boldsymbol{0} \\ \boldsymbol{M}^{-1} \end{bmatrix}$$

$\boldsymbol{C}_{ss} = [\boldsymbol{I}]$

$\boldsymbol{D} = [\boldsymbol{0}]$

$u = \boldsymbol{P}(t)$

3.3 Loads

The term $p(x,t)$ in the above equation of motion for an Euler-Bernoulli beam, for this study consist of two major forces: a truck force, f_t , and a control force, f_c.

3.3.1 Truck Loading

The truck load which the bridge is subjected is the same as that in [3]. The truck is assumed to be a 5 axle truck and crosses the bridge at a constant speed, v (equal to 65mph). The relative weight of each axle, λ_k, is a percentage of the total weight of the truck, W_t , and the distance between the first axle and the subsequent axles is d_k. The truck load can be expressed by the equation:

$$f_t(x,t) = W_t \sum_{k=1}^{5} \lambda_k \delta(x - (vt - d_k))$$

$where: \delta(-) is\ the\ Dirac\ delta\ function$

Therefore, the load vector for the truck load, $\boldsymbol{F}_t$, becomes:

$$\boldsymbol{F}_t(t) = \int_0^L \left(\boldsymbol{\Psi}^T(x)\right)(f_t(x,t))\partial x$$

$$\boldsymbol{F}_t(t) = W_t \sum_{k=1}^{5} \lambda_k \boldsymbol{\Psi}^T (vt - d_k)$$

The truck load is only applied while the truck is crossing the bridge. Once the truck has crossed the bridge the truck load becomes zero.

3.3.2 Control Force

For this study, one control force was implemented and applied at the midspan of the bridge. The control force could also be applied at two points by implementing a control device similar to the SAVA that was installed on the Walnut Creek Bridge. This method type was studied by [3].

The expression for the control force is:

$$f_c(x_m, t) = F_d(t)\delta(x - x_m)$$

$where$:

$x_m = midspan\ displacement$

$F_d(t) = force\ produced\ by\ control\ device$

Therefore, the load vector for the control force, $\boldsymbol{F}_c$, is equal to:

$$\boldsymbol{F}_c(t) = \int_0^L \left(\boldsymbol{\Psi}^T(x)\right) (f_c(x_m, t))\partial x$$

$$\boldsymbol{F}_c(t) = F_d(t)\ \boldsymbol{\Psi}^T(x_m)$$

3.4 Passive Control Strategy

A viscous damping device was chosen for the passive control strategy. As previously discussed, the force produced by a fluid viscous damper is equal to:

$$F_d = c * v$$

$where:$

$c = damping\ coefficient$

$v = velocity\ of\ piston$

Because the damper is installed at the midspan of the bridge, the velocity of the piston is equal to the vertical velocity of the bridge at the midspan. The velocity at the midspan is equal to:

$$v_m = \mathbf{\Psi}(x_m)\dot{\boldsymbol{Q}}(t)$$

Therefore the load vector for the damping force is:

$$\boldsymbol{F_c}(t) = c\mathbf{\Psi}(x_m)\dot{\boldsymbol{Q}}(t)\ \boldsymbol{\Psi^T}(x_m)$$

3.5 Semi-Active Strategy

A semi-active control scheme similar to that employed by [3] was implemented for this study. A variable-orifice damper, like that described in chapter 2, was used as the semi-active device. The force from the variable-orifice damper is able to be controlled, and therefore, the required force needed for optimal control can be produced. A saturation rule was implemented for the control strategy. Both a clipped optimal controller and a linear-quadratic-regulator (LQR) were employed. The clipped optimal controller ensured that the semi-active device produces only a dissipative force of the primary structure.

A linear-quadratic-regulator (LQR) was used to find the optimal state-feedback gain matrix, $\boldsymbol{K}_{lqr}$. With the LQR method, the force that decreases the state as much as possible (the optimal state) can be found. The state space formulation is:

$$\dot{x} = \boldsymbol{A}x + \boldsymbol{B}u$$

$where$:

$\boldsymbol{A} = state\ space\ matrix$

$\boldsymbol{B} = state\ space\ matrix$

$x = the\ state\ vector$

$u = input\ force\ introduced\ to\ achieve\ the\ control$

The optimal control force is found by the equation:

$$u = -\boldsymbol{K}_{lqr}x$$

This minimizes the quadratic cost function:

$$J(u) = \int (x^T Q_{lqr} x + u^T R u)$$

The gain matrix, $\boldsymbol{K}_{lqr}$ is equal to:

$$\boldsymbol{K}_{lqr} = \boldsymbol{R}^{-1}\boldsymbol{B}^T\boldsymbol{S}$$

$where$:

$\boldsymbol{R} = weight\ matric$

$\boldsymbol{S} = solution\ of\ the\ Riccati\ equation$: $\boldsymbol{A}^T\boldsymbol{S} + \boldsymbol{S}\boldsymbol{A} - \boldsymbol{S}\boldsymbol{B}\boldsymbol{R}^{-1}\boldsymbol{B}^T\boldsymbol{S} + \boldsymbol{Q} = 0$

With a clipped optimal control, the force produced by the semi-active device is equal to:

$$\begin{cases} -\boldsymbol{K}_{lqr}x & -\boldsymbol{K}_{lqr}xv_m < 0 \\ 0 & otherwise \end{cases}$$

It is worth noting that the semi-active control device in actuality is not capable of instantaneously altering the force it is producing. Therefore, a better representation of the semi-active control scheme would implement a time delay. However, for this preliminary study, no time delay was implemented.

3.6 Results

MATLAB and SIMULINK were used to conduct the simulations. The basic MATLAB script and SIMULINK used can be found in the Appendix. The control device parameters, c_d, $\boldsymbol{Q}_{lqr}$, and $\boldsymbol{R}$ were selected to achieve roughly the same maximum control force for both control strategy.

The goal of this study was to find the best means to reduce the dynamic response of the bridge when subjected to traffic loading. The parameter used to judge and compare the performance of each control type was midspan deflection. Below are the findings of the simulation.

3.6.1 Uncontrolled

The uncontrolled midspan displacement of the bridge (when control force is always equal to zero) is presented below. The peak displacement was found to be 0.23in. Because it was assumed that there is no damping in the bridge itself, when the traffic excitation ends (when the truck as completely crossed the bridge) the bridge goes into a steady state response and never decays.

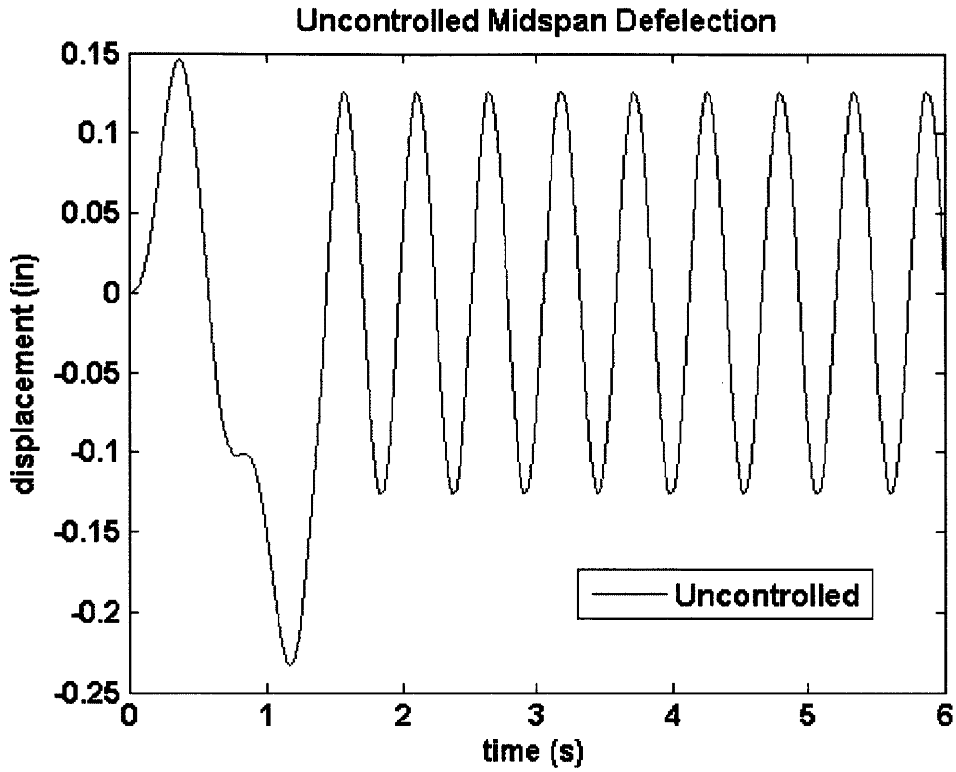
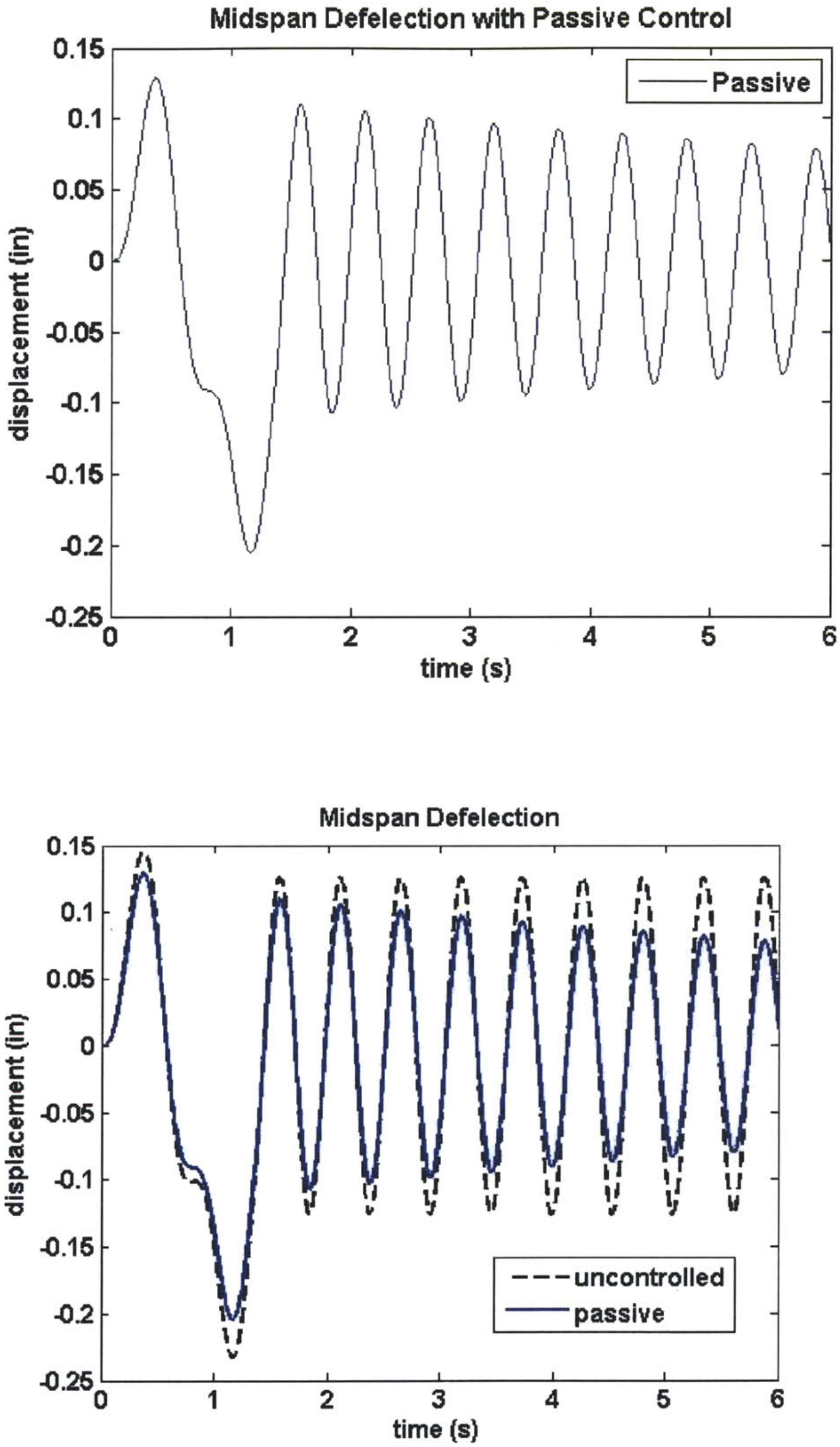


Figure 14: Uncontrolled midspan displacement

3.6.2 Passive

The passive control scheme showed a reduction of 12% in maximum displacement. A graph of displacement versus time is presented in Figure 15. Because the passive device produces a control force as a function of midspan velocity, once the truck excitation ends, the system still experiences damping due to the passive damper. This can be seen by the slow reduction in displacement over time.

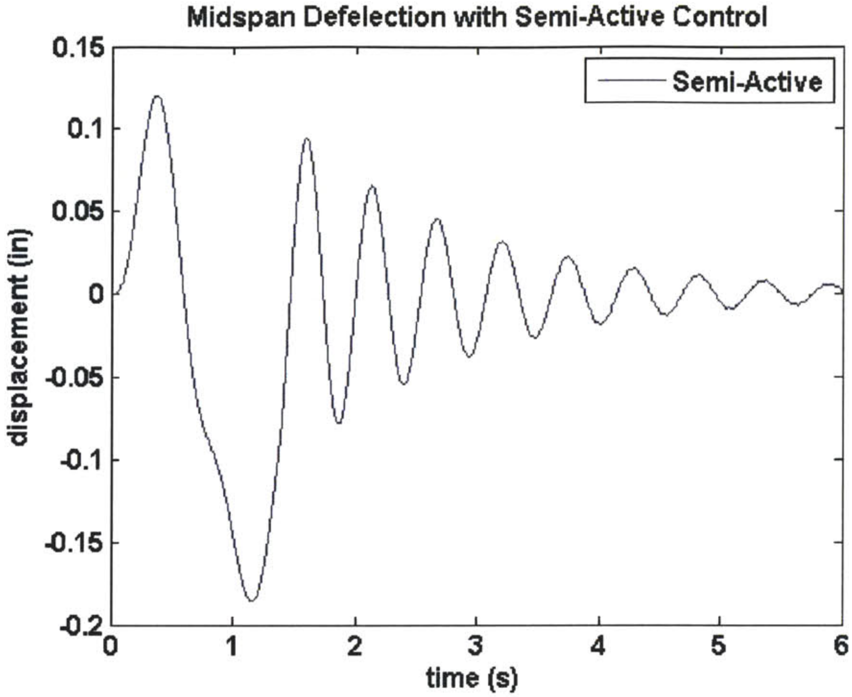


(b)

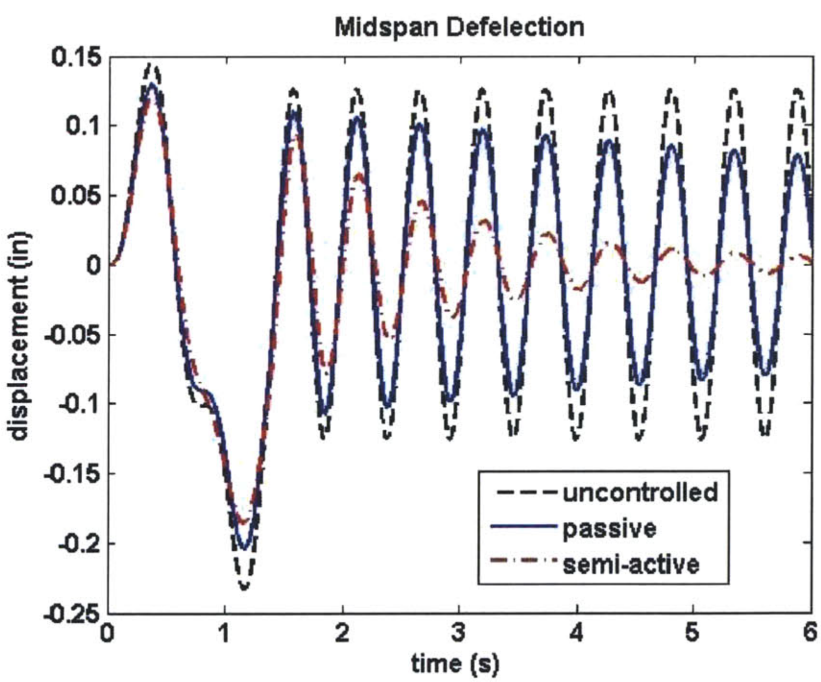
Figure 15: (a) Passive controlled midspan displacement (b) Passive compared to uncontrolled

3.6.3 Semi-Active

A reduction of 20% in maximum displacement is seen with the semi-active control scheme. Figure 16 shows the graph of displacement versus time for the semi-active control strategy. Just like with the passive scheme, because the semi-active device still produces control forces after the truck has crossed the bridge, bridge displacement continues to reduce with time.



(a)



(b)

Figure 16: (a) Semi-active controlled midspan displacement (b) Semi-active compared to passive and uncontrolled

Figure 17 shows the magnitude of the control force for both the passive and semi-active control devices. As can be seen from the graph, the forces for both devices are of the same magnitude. The semi-active device does require a slightly larger control force compared to that of the passive device initially but then requires less with time.

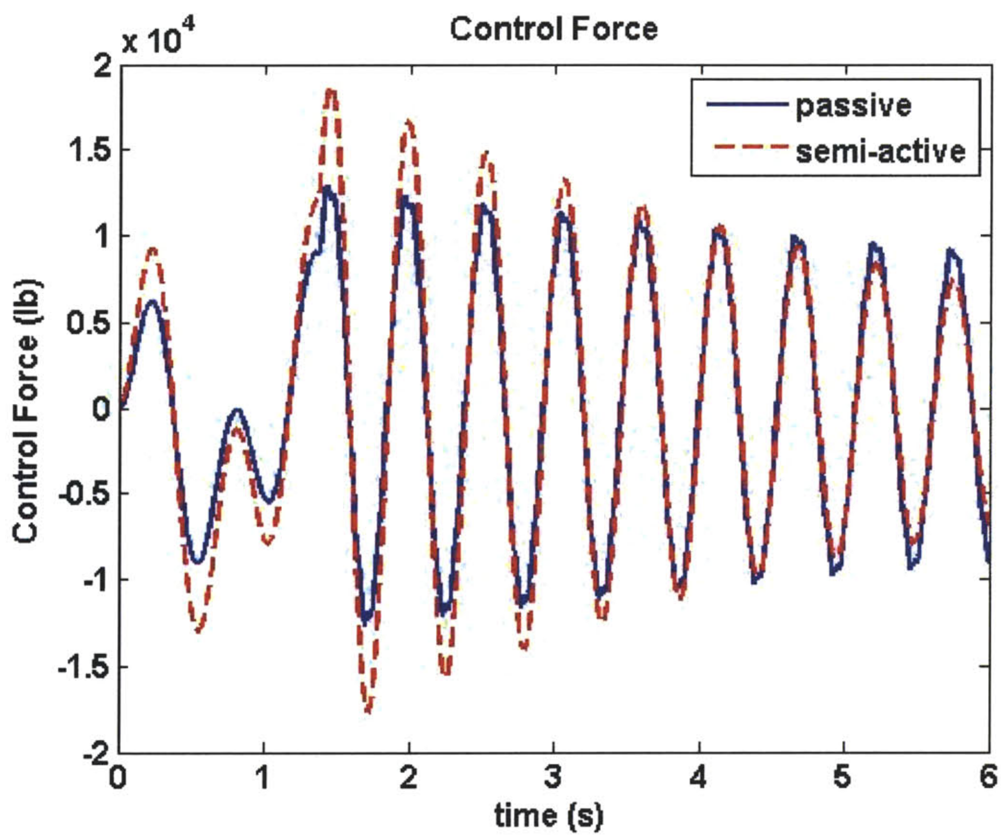


Figure 17: Control forces produced by passive and semi-active control devices

3.7 Discussion and Future Work

A simplified method and model was employed for this simulation study. This basic simulation allows for a good first look at the relative performance of the two control schemes. The results of the study show that the semi-active control scheme outperformed the passive strategy. The semi-active method reduced maximum deflection by 20%, whereas the passive reduction was 12%. The semi-active deflection also decays faster with time. From this initial investigation, the implementation of a semi-active control system would be more effective at suppression traffic-induced bridge vibrations.

There are several limitations to this study and further investigation would give better insight in the comparison of passive and control schemes. The model used could be improved. The model for the truck load could be improved by taking into account road roughness and modeling the truck as system with its own mass, spring, and damping properties. For this study, the truck speed is assumed to be constant. This, however, is not always the case. Vehicles traveling across the bridge may in actuality be accelerating or braking, which could change the system response and dynamics. A better representation of vehicle loading and bridge response (such as that presented by J.H. Lin in [26]) would allow for a better and more accurate understanding of how a passive and semi-active system could improve bridge vibrations. In future work, exploring control force location would be interesting. For example, one could investigate placing the control forces at the first and last quarters of the bridge length like Haji-Hosseinloo *et al.* [19] proposed for active control schemes. For this study, the bridge properties were held constant. It would be interesting to explore how the two control systems perform for a variety of different bridge types. This would help to evaluate the validity of wide spread implementation of these control schemes on bridges in the US.

4 Conclusion

Many of the U.S. bridges today are reaching or have reached their design life and are beginning to deteriorate and are becoming structurally deficient. A major cause of this deterioration is due to stresses and fatigue induced in the bridge from traffic loading. Much time, effort, money, and resources go into repairing, rehabilitating, or reconstructing these bridges. Therefore, investigation into valid solutions to extending the safe life of these structures is of utmost importance.

Although there are several possible solutions, one that has been gaining more interest is the integration of structural control device to suppress vertical traffic-induced bridge vibrations. The implementation of control devices poses several advantages over other solutions. Retrofitting control devices is an efficient and cost effective approach to extending the useful life of a bridge. By implementing control devices, a new bridge does not need to be constructed, which saves time and money. As with the SAVA device installed on the Walnut Creek Bridge, control devices can also often be retrofitted to the existing bridge with little to no impedance to bridge traffic.

There are three primary control methods: passive, active, and semi-active. All have their advantages and disadvantages. Based on the research and simulation conducted in this thesis, the semi-active control method is recommended for wide spread implementation. Semi-active devices are able to effectively control the bridge response without the need of a large power source and are stable, unlike active control systems. Semi-active schemes have also been shown to outperform passive systems at reducing peak deflection of bridge subjected to truck loading. The simulation conducted in this thesis found a 20% reduction with a semi-active scheme whereas the passive had a 12% reduction.

Implementation of control devices on bridges shows great potential and further investigation and study of all three control strategies as a mean to reduce traffic-induced bridge vibrations and extend the life of bridges should continue to be conducted.

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6 Appendix

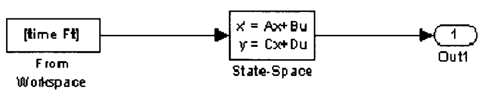
6.1 MATLAB Script

```
clear all; clc
close all;

rhoA = 1.7025*1000; %lb/ft
E = 29000*12^2*1000; %psf
I = 3.7497*10^5/(12^4); %ft4
L = 75; %ft span of bridge
v = 95.33; %ft/s = 65mph speed of truck
W = 61.84*1000; %lb total weight of truck
lambda = [0.1727 0.2222 0.2144 0.2044 0.1863]; %relative weigth of each axle
d = [0 17.42 21.75 51.5 55.83];%ft distance between first axle and subsequent
axles

dof=20;

syms x

b=1

for i=1:dof
    PSI1(i)=b*sin(i*pi*x/L);
end


M=vpa((rhoA*(int((PSI1'*PSI1),x,0,L))),4);
DPSI1=diff(PSI1,x,2);
K=E*I*int(DPSI1'*DPSI1,x,0,L);

[phi, om2] = eig(M\K);
xi = 0*eye(dof); %assume undamped i.e. C=0
xi = diag(xi);
om=sqrt(diag(om2));
Mbar = phi\M*phi;
Mj=diag(M);
Cbarj=2*Mj.*om.*xi;
Cbar = diag(Cbarj);
C=phi*Cbar/phi;

cd=10000;


s=zeros(dof,1);

step=0.01;
last=6;

time=[0:step:last]';

Ft = zeros(1,dof);
for t = step:step:last;
```

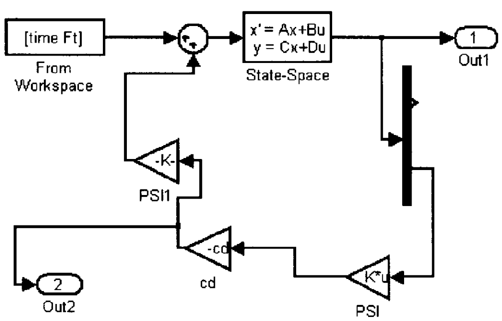
```
    for j=1:length(d)
       if 0<=v*t-d(j)<=L %&& (L/2+d(j)-v*t)<=L
           s=lambda(j)*subs(PSI1',x,(v*t-d(j)))+s;
        end

    end
        if v*t-d(5)>L
            s=zeros(dof, 1);
        end

   Ft=[Ft; W*s'];
end


M=double(M);
K=double(K);
C=double(C);

%for state space
A = [zeros(dof) eye(dof); -M\K -M\C];
Css = eye(2*dof);
Dss = zeros(2*dof,dof);
B = [zeros(dof); inv(M)];

PSI = subs(PSI1,x,(L/2));

simulation_time=last; %seconds

[t1,f1,q1,p1]=sim('thesis21.mdl',simulation_time);

Q1 = q1(:,1:dof)';

y1=PSI*Q1;
figure
plot(t1,y1')
legend('Passive')
title('Midspan Defelection with Passive Con-
trol','fontsize',12,'fontweight','bold')
xlabel('time (s)','fontsize',12,'fontweight','bold');
ylabel('displacement (in)','fontsize',12,'fontweight','bold')
set(gca,'fontsize',12,'FontWeight','bold')
```

6.2 SIMULINK

Uncontrolled:



Passive:



Semi-Active: