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UNIVERSITY OF CALIFORNIA, SAN DIEGO

Seismic Bridge Response Modification due to Degradation of Viscous Dampers
Performance

A Thesis submitted in partial satisfaction of the requirements for the degree

Master of Science

in

Structural Engineering

by

Francesco Graziotti

Committee in charge:

Francesco Lanza di Scalea, Chair
P. Benson Shing
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Gianmario Benzoni

2010

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University of California, San Diego

2010

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ABSTRACT OF THE THESIS

Seismic Bridge Response Modification due to Degradation of Viscous Dampers Performance

by

Francesco Graziotti

Master of Science in Structural Engineering

University of California, San Diego, 2010

Professor Francesco Lanza di Scalea, Chair

The goal of this thesis is to analyze the variation of a seismic response of a bridge in case of degradation of installed viscous fluid dampers.

The study was conducted with nonlinear time-history analyses of a detailed three-dimensional FE model of the Vincent Thomas Bridge, provided by Caltrans. Three different type of excitations were used (two white noises and a real measured earthquake). Such numerical model, including cables, suspenders, suspended structure, towers, cable bents and anchorages, reflects the state of the structure after the last retrofit phase, when dampers and fuses were installed and towers were stiffened.

A preliminary validation of the numerical model of the bridge, aimed to ensuring the reliability of the FE model, has been carried out by comparing the numerical response with recorded signals.

The degraded performance of the dampers was simulated through the use of gap, spring and viscous elements and validated against experimental results of real devices. The parametric study was intended to investigate the effects of progressive degradation of the energy dissipators on the bridge structural performance, both under service load conditions and seismic excitations. Results indicated a significant level of relative displacement experienced by the devices during daily loading conditions, potentially resulting in a premature degradation due to wear of the internal components of the units. For this reason an alternative device was proposed at least in conceptual terms, in order to decompose the dampers during service loading and to engage them only in case of seismic event.

A solution to the early degradation of the damper devices has been proposed.

I. Introduction

The goal of this work is to analyze the effects of the performance degradation of viscous damper installed on a bridge structure. The structures of the Vincent Thomas bridge, in Los Angeles area was utilized as a case study for this research. The specific structure, subjected to seismic retrofit in recent times, represent an ideal example of viscous damper application. Due to its original configuration of suspended bridge, the retrofit interventions as well as the increasingly damaging traffic loads, the bridge experiences significant movements in service conditions. The relative displacements across the installed viscous dampers are significant and continuous as documented by a series of recorded ambient vibration records. A number of dampers on the real structure were also found to have suffered performance degradation and recently replaced. After research conducted on the damaged/degraded damper behavior (Benzoni et al., 2008) at the Caltrans SRMD Testing Laboratory of the University of California San Diego, appears that a possible leak of the fluid in a damper may vary the response of the device and may create a gap in the force/displacement response. Numerical analysis of the structure with simulated damaged dampers were conducted. The attention was focused to the most representative parameter of damper response, the energy dissipated per cycle. Previous researchers evaluated the effects of damper performance variation simulated as a modification of the viscous coefficient C_m

but the analyses were limited to the global bridge response (Benzoni,2005; Cendron, 2008).

In this work a parametric study has been conducted to evaluate the effects of the damper characteristics on the structural performance considering also the range of forces and displacements associated with the devices when installed on the real structure. The damaged configuration of the dampers has been modeled by a gap (viscous and spring element), validated against laboratory test results of the actual devices removed from the bridge. The parametric analysis has been designed by investigation of several parameter combinations able to reproduce conditions of silicon fluid leakage of increasing severity.

A recent forensic analysis of the conditions of damaged units removed from the Vincent Thomas bridge, confirmed the existence of excessive wear experienced by devices during service loads. For this reason a conceptual solution was investigated in chapter 6, in order to release the dampers during the normal service motions and connect them in case of seismic event. The proposed approach suggests a valuable distribution of contribution between the dampers and a new connecting element that protects the dampers from excessive wear.

II. The Vincent Thomas Bridge

The Vincent Thomas bridge is located on Route 47 (P.M. 0.86) in Los Angeles County. It is a cable-suspension bridge, 6062 ft long, consisting of a main suspended span of approximately 1500 ft, two suspended side spans of 506 ft each, ten spans in the San Pedro Approach of approximately 1838 ft total length, and ten spans in the Terminal Island Approach of approximately 1712 ft total length. The roadway width between curbs is typically 52 ft, and accommodates four lanes of traffic. The clear height of the navigation channel is approximately 185 ft. The Vincent Thomas Bridge is a cable-suspension structure retrofitted in different stages, and lately equipped with 48 viscous dampers. The sensitivity of the bridge seismic response to the variation of the damper characteristics will be analyzed.



Figure 1. The Vincent Thomas bridge

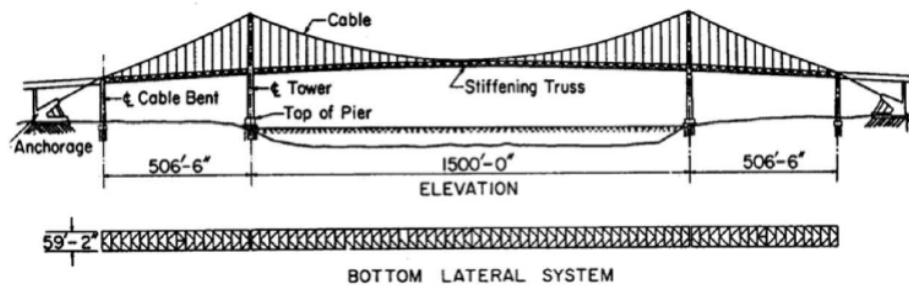


Figure 2. Vincent Thomas bridge - lateral view

The design of the bridge was completed by Caltrans in 1960. The substructure contract was completed in 1962 while the superstructure contract was completed in early 1964.

Stage 1 seismic upgrading in the form of cable restrainers, shear keys abutment seat extenders and girder lateral supports was completed in 1980. Modifications to the vertical cross frames, and the lateral bracings near the bents, and inclusion of a full length cat-walk were also made in 1980 as part of the seismic upgrading contract. New elevators at Bents 9 and 15 were added in 1992.

Twenty-six seismic sensors were installed on the bridge to record ambient and seismic behavior.

2.1 Main Spans

The suspended structure consists of two stiffening trusses spaced 59 feet apart, floor trusses with 31 feet center to center spacing and a lower lateral bracing system (K-truss type). The stiffening truss is 15 feet deep while the floor truss is 10 feet deep. The top edge of the roadway is 3 feet below the top edge of the stiffening trusses. A lightweight concrete deck is supported on rolled beam stringers, spaced transversely at 7 feet, which are in turn supported by the transverse floor trusses. The suspended spans are supported by the main cables through suspenders at each floor truss location.

The towers are 335 feet high and are supported on steel HP pile foundations. The towers consist of two shafts which are braced together through five struts. The tower shafts, with a cellular cross-section, consist of four welded box sections (3/4 inch plates) that are field bolted with 1 inch diameter high strength bolts. The cross-section of each shaft tapers toward the top. Each tower shaft rests on a 3 inch thick base plate and is anchored to the underlying concrete footing by thirty nine 2.5 inch (in diameter) by 25 feet long prestressed rods.

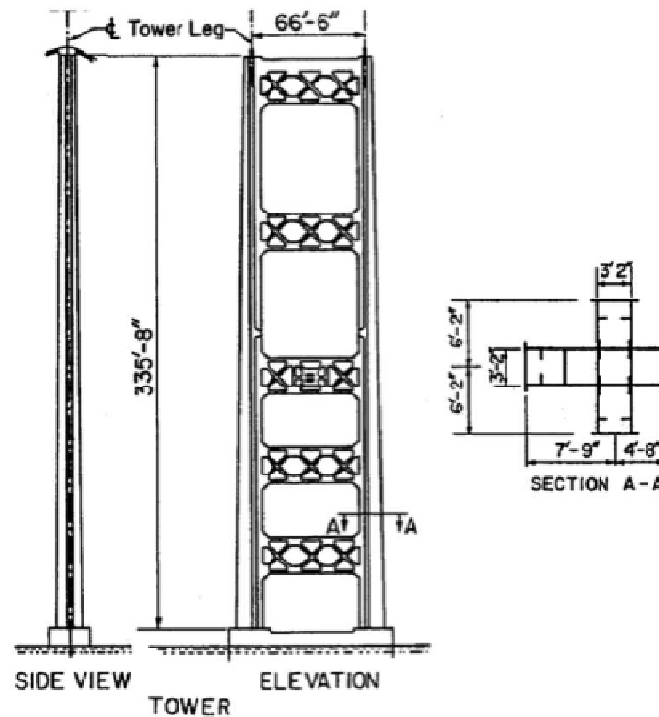


Figure 3. Vincent Thomas bridge - tower

The cable in the suspended spans consists of 4,028 cold-drawn galvanized, 6 gauge steel wires providing 121.5 in² of area including galvanizing. The suspenders are made of small diameter, high strength wires laid up into rope. The cable saddles are centered on the top of tower legs. This causes the cable to spread at the tower tops where the frictional resistance between the cable and the saddle to prevent the cable from slipping through the saddle.

The main span and the side spans are connected to the towers through truss links that provide vertical support and wind shoes that provide transverse support only. The end of each side span is supported on a reinforced concrete cable bent through truss links

that provide vertical support and wind shoes that provide longitudinal as well as transverse support. The cable bents also support the last spans of the approach structures.

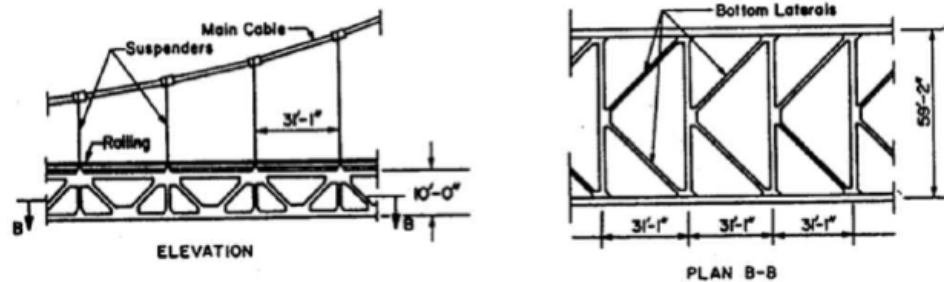


Figure 4. Vincent Thomas bridge deck

At the cable bents, the cables are deviated by a cable saddle towards the anchorages where they are attached to eye-bars embedded in the anchorages. These are massive reinforced concrete structures that are supported on heavily battered pile foundations.

2.2 Approach Spans

2.2.1 Deck and Girders

The approach superstructure consists generally of four rows of steel girders ranging in span from 132 ft to 230 ft. Two spans adjacent to the San Pedro abutment consist of five rows of girders. The built-up, stiffened plate girders are composite with the 8-3/8" thick lightweight reinforced concrete deck. All spans are simple spans, except the two spans adjacent to both cable bents, which consist of continuous haunched girders.

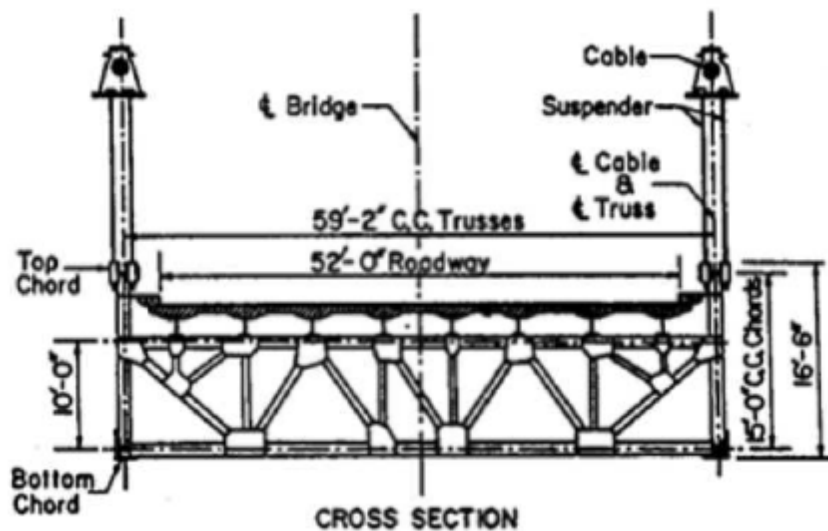


Figure 5. Vincent Thomas bridge - transversal deck view

2.2.2 Cross Frames, Bracings and Bearings

End frames and intermediate cross frames consisting of structural tee-section members spaced 20 to 25 feet on center, are provided between girders. Tee-section bottom chord lateral bracing is provided between the interior girders along the approach spans. In general, the connections are made with 3/8" gusset plates and 7/8" high strength bolts.

The vertical frames were modified after construction to accommodate a walkway which runs the entire length of the approach spans.

Girders are supported at each end by built-up steel bearings approximately 2'0" in height. In general, the end of the girder nearest the abutment has a fixed bearing while the other end is supported on rocker type expansion bearing. The continuous spans adjacent to the cable bents have a fixed bearing in the center and expansion bearings at the ends.

2.2.3 Typical Bents and Cable Bents

The superstructure is supported on normal weight reinforced concrete bents which range in height from approximately 36' to 120' above bottom of footing. All bents are two-column bents except Bent 2 which has three columns. Bents 2,3,4 and 5 on the San Pedro approach and Bents 19, 20 and 21 on the Terminal approach have 7' x 4'4" solid columns; the remaining bents have 12' x 6' hollow columns. All bents have 8' deep solid rectangular bent caps. Concrete shear keys which provide lateral restraint to the girders were constructed as part of the seismic retrofit contract. These are cast on the top of the bent caps and secured with dowels drilled and bonded into the bent caps.

The cable bents (Bents 11 and 12) are approximately 142' tall frames, each with two 16' x 8' hollow normal weight concrete columns and a 10' x 8' solid concrete bent cap. There are no intermediate braces. 8' x 3' concrete pedestals rise approximately 16'-6" above the girder seats on each end of the bent cap to serve as seats for the cable saddles.

Steel brackets are embedded in the top of the bent concrete to support the truss links and to resist the forces from the wind shoes.

2.2.4 Bent Footings and Piles

All bent footings are supported on piles. Footings for the typical bents are of normal weight concrete and range in size from 14' x 10' to 26' x 18' thicknesses range from 3'6" to 4'-9". There is no top reinforcement mat in these footings. The cable bent footings are continuous between the columns with an upstand spine beam. The cable bent footings do have a top reinforcement mat.

The San Pedro Approach bent footings are supported on 14-BP-89 steel piles. These piles are embedded 6' into the bottom of the footing and do not have any hairpins or other type of anchorage. Both cable bent footings are supported on three rows of 14BP-117 steel piles, the outer row being battered 1:6. In this case, however, the outer rows of battered piles have #11 hairpins connecting the tops of the piles to the footings.

The Terminal Island Approach bent footings are supported on 16" octagonal reinforced concrete piles. These are embedded 3" into the bottom of the footings and have 8#9 dowels embedded 2'-6" into the concrete.

2.2.5 Cable Restrainers

3/4" diameter cable restrainers are provided to longitudinally interconnect the deck segments at each typical bent. These are looped cables connected by tumbuckles at the ends. The total length of each loop is approximately 100'. Three banks of looped cables are provided at each bent. The interior banks consist of eleven cables while the exterior banks are eight cables each. The cables are looped around cross beams which were retrofitted during the seismic upgrading contract and which span between the deck girders.

The looped cable restrainers are not present either at the abutments or at Bents 10, 11, 12 and 13. Vertical restrainers are provided at these locations as hold-down devices.

These vertical restrainers consist of 1-1/4" diameter high-strength bolts cast into new concrete pedestals constructed on either side of the existing deck girders.

2.3 Modification History

As written before the bridge has had some retrofit to upgrade its capacity against seismic events. A brief chronology is now present:

1964 Bridge completion

1980 Stage 1 seismic upgrading (cable restrainers, shear keys abutment seat extenders and girder lateral supports, modifications to the vertical cross frames, and the lateral bracings near the bents, inclusion of a full length cat-walk)

1987 Whittier Earthquake

1992 Added new elevators at Bents 9 and 15

1994 Northridge Earthquake

1998 Seismic retrofit (installation of dampers between the stiffening trusses and the towers of the bridge, stiffening of the bridge towers, installation of structural fuses in the side spans of the bridge)

2003 Ambient vibration records (used in this work)

2.4 Finite Element Model of the Structure

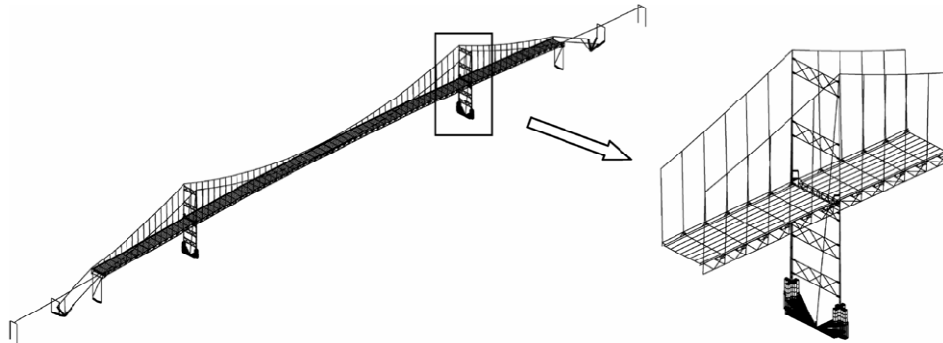


Figure 6. Vincent Thomas bridge ADINA model

The study will be conducted with of nonlinear time-history analyses of a detailed 3D FE model of VTB (see Figure 6) developed in the structural analysis software ADINA (ADINA R&D Inc., 2002) provided by Caltrans. Such numerical model, including cables, suspenders, suspended structure, towers, cable bents and anchorages, reflects the state of the structure after the last retrofit phase, when dampers and fuses were installed, and towers were stiffened. The static and the time-history analyses of the bridge were both geometrically nonlinear (large displacement analyses) to account for the geometric stiffness of the cables and suspenders. An exhaustive description of the FE model can be found in Ingham et al.

This FE model is composed of 3D linear elastic (tension-only) truss elements to represent the main suspension cables and suspender cables, 3D linear elastic membrane and shell elements to model the reinforced-concrete bridge deck and stringers supporting

the deck on the floor trusses, and beam-column elements to model the stiffening trusses, the lateral braces between the stiffening trusses, and the tower shafts.

The floor trusses were modeled with 3D elastic beam-column and trusses elements. This FE model consists of approximately 8,900 nodes and 9,400 elements, resulting in approximately 22,000 degrees of freedom (DOFs).

2.4.1 The Dampers in numerical model

Several type of dampers were used in the retrofit of the bridge. They provide a diffuse viscous damping connecting some structural components. Dampers are located between central deck and towers, side decks and towers, in cable bent.



Figure 7. Bridge Dampers detail on real structure

In particular location are:

- 1) Mid Span Center
- 2) Mid Span Quarter
- 3) Tower
- 4) Fuses
- 5) Cable Bent

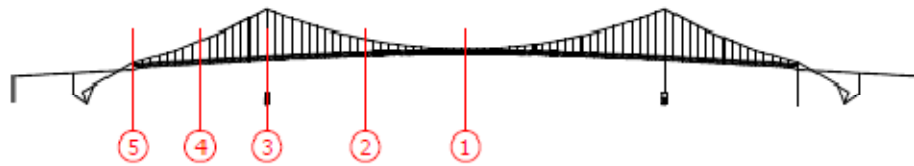


Figure 8. Dampers locations on the structure

2.4.2 Dampers numerical parameters

The damping force is defined as

$$F_d = C \cdot \dot{x}^\alpha$$

The location and the current characteristics of the dampers are described in table

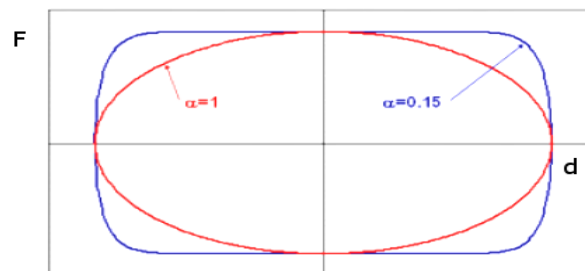
1. These parameters are used in the original Caltrans file.

Table 1. Dampers parameters

	Total #	C [kips's/in]	α [-]
Side span to cable bent top chord dampers	4	0.1	1
Side span to cable bent bottom chord dampers	4	5	1
Side span to tower	8	2.5	1
Main span to tower	8	4	1
Side span hinge damper at fuses	4	100	0.5

It is noted that the dampers “side span to cable bent top chord dampers” are not present in the technical drawings of the bridge; they are characterized by a very low C constant compared to the others. The dampers at fuses location are characterized as nonlinear dampers with an extremely high C constant and low α to simulate a rigid plastic behavior.

The influence of α factor on the hysteretic response is represented in figure 9. Since the exponent α is close to 0, non linear fluid viscous dampers react with an almost constant force over a large range of velocities.

**Figure 9.** Influence of α factor on damper response

2.4.3 Analysis of natural elastic frequencies and modes of the bridge

These are the results of the natural frequency analysis on the model of the bridge. In a modal analysis the influence of the damping is not consider. This first analysis just give the possibility to understand the principal mode-shapes of the structure and their elastic undamped periods.

A graphic representation of the modes will follow the table. For each mode the participation factor in each direction is printed in the table. In bolt the modes that regards the deck.

Table 2. Modal analysis output

MODE NUMBER	FREQUENCY (Hz)	NATURAL PERIOD (s)	X-MODAL PART. FACTOR	Y-MODAL PART. FACTOR	Z-MODAL PART. FACTOR
1	0.13	7.69	-4.44E-04	4.96E+00	8.85E-04
2	0.18	5.56	-4.31E+00	1.62E-03	9.79E-05
3	0.22	4.63	2.59E+00	1.16E-01	-6.46E-04
4	0.23	4.42	5.63E-01	-2.51E-02	-2.44E+00
5	0.23	4.42	-2.54E+00	1.08E-01	-4.89E-01
6	0.23	4.35	3.00E+00	-3.08E-04	7.20E-03
7	0.36	2.78	-4.27E-01	-1.90E-03	1.34E-02
8	0.36	2.75	3.22E-03	1.99E-04	-1.79E-01
9	0.41	2.44	-9.12E-02	1.60E-01	-6.89E-03
10	0.41	2.42	-3.28E+00	-4.32E-01	-9.55E-02
11	0.43	2.33	-3.31E+00	4.41E-01	1.65E-01
12	0.46	2.19	8.72E-02	1.67E-02	-5.74E+00
13	0.50	2.01	6.28E-03	1.27E+00	-3.95E-03
14	0.57	1.76	-9.10E-02	4.27E-03	-3.56E-03

The following plots represent the deformed shapes of the modes regarding the deck. The first mode has a period of approximately 7.7 s and is characterized by a transversal displacement of the deck. The second mode (5.6 s) presents a longitudinal motion of the deck associated to an anti-symmetrical vertical translation.

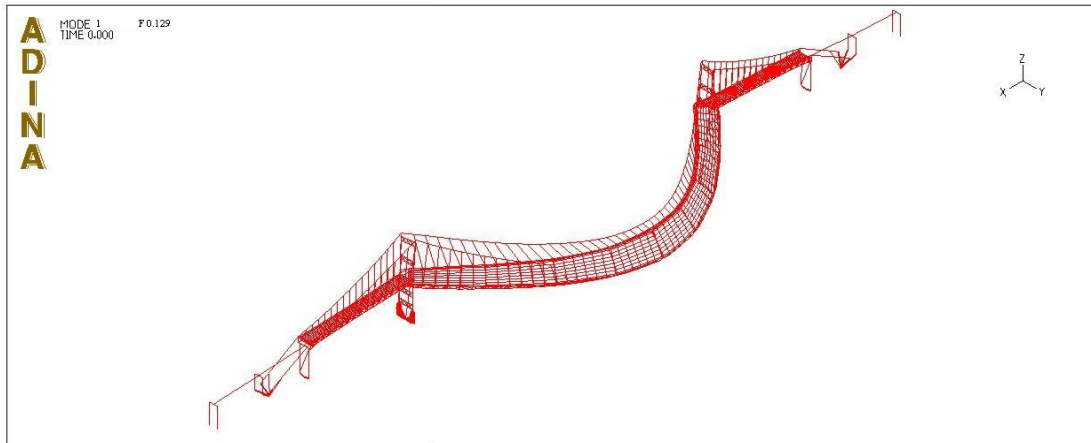


Figure 10. 1 Modal shape ($\omega_n = 0.129$ Hz)

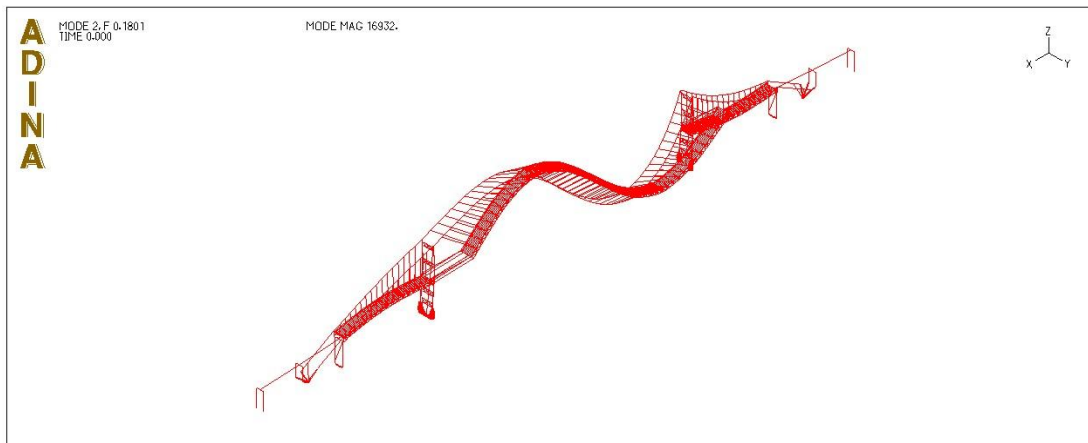


Figure 11. 2 Modal shape ($\omega_n = 0.18$ Hz)

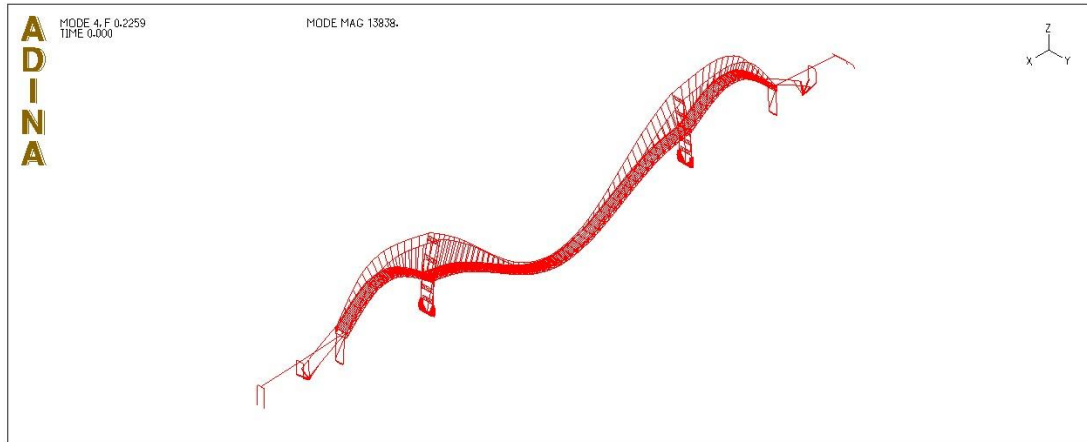


Figure 12. 4 Modal shape ($\omega_n = 0.226$ Hz)

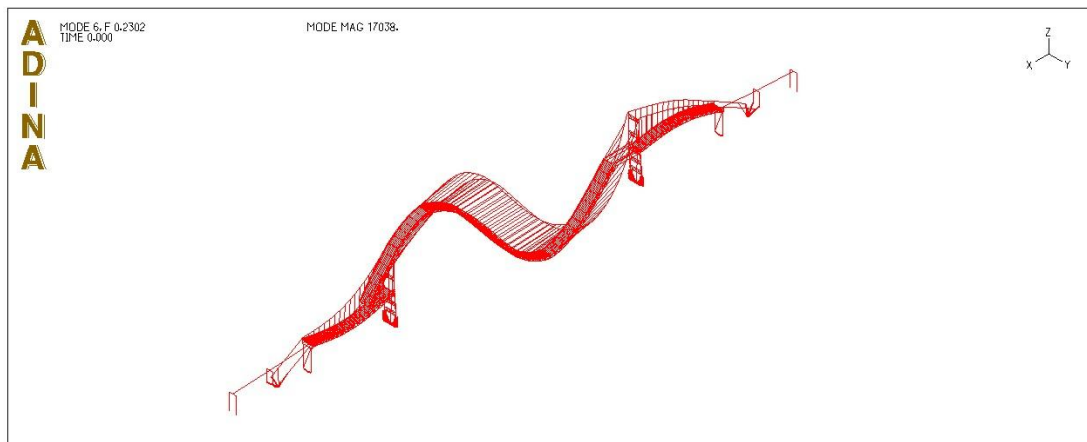


Figure 13. 6 Modal shape ($\omega_n = 0.23$ Hz)

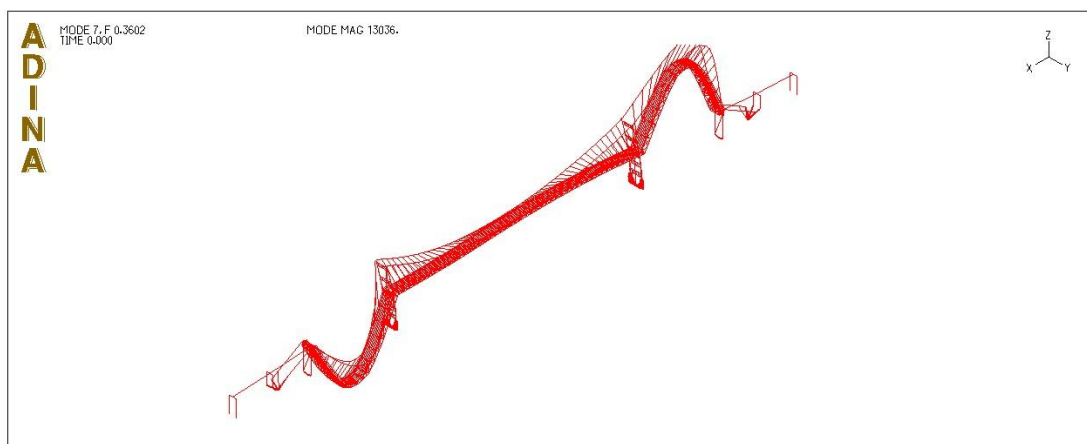


Figure 14. 7 Modal shape ($\omega_n = 0.36$ Hz)

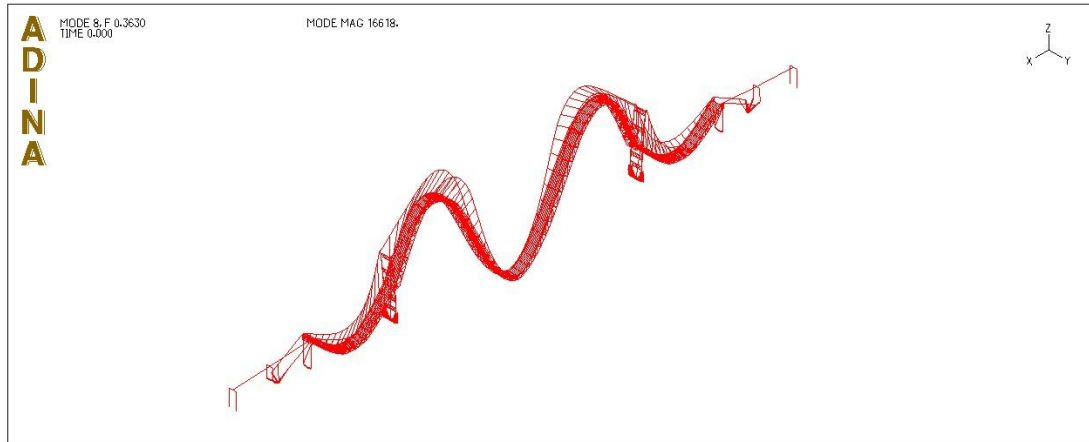


Figure 15. 8 Modal shape ($\omega_n = 0.363$ Hz)

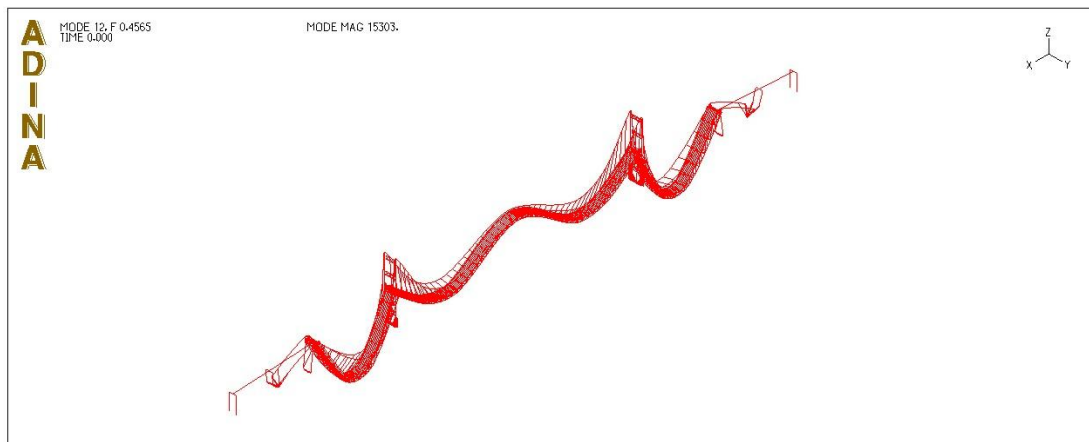


Figure 16. 12 Modal shape ($\omega_n = 0.456$ Hz)

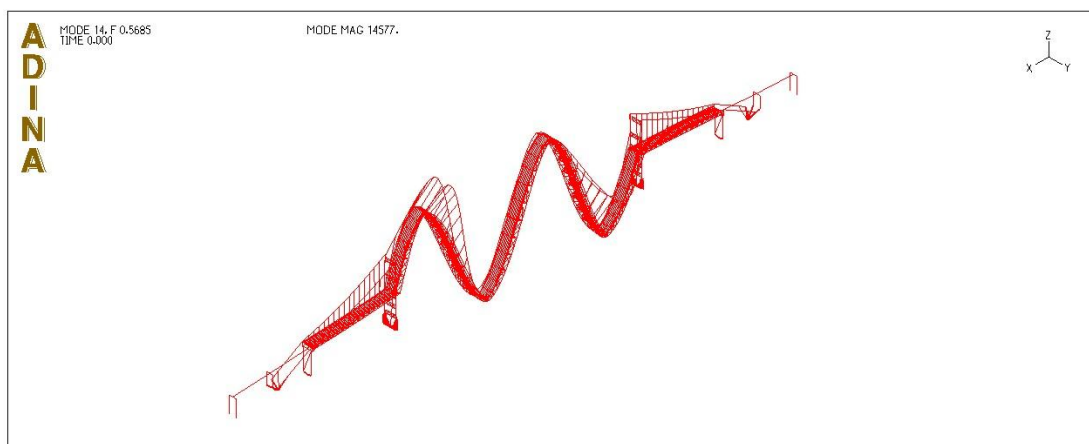


Figure 17. 14 Modal shape ($\omega_n = 0.56$ Hz)

III. Introduction to Seismic Passive Energy Dissipation for Structures

In seismic design of structures, the design forces are generally calculated using an elastic response spectrum. To account for energy dissipation through inelastic action, a response modification factor R , is used in the codes to reduce the calculated elastic forces. The philosophy in permitting inelastic action is that during severe earthquakes, the structure can sustain damage without collapse due to the ductility of members and redundant load paths. The inelastic action, while contributing to substantial energy dissipation, often results in significant damage to the structural members. In addition, the hysteretic behavior of the members degrades with repeated inelastic cycles. Furthermore, the inelastic action necessitates large inter-story drifts which usually result in substantial damage to non-structural elements such as in-fill walls, partitions, doorways, windows, and ceilings.

Energy dissipation devices can absorb a portion of earthquake-induced energy in the structure and minimize the energy dissipation demand on the primary structural members such as beams, columns, or walls. These devices can substantially reduce the inter-story drifts and congruently nonstructural damage. In addition, lower accelerations and smaller shear forces lead to lower ductility demands in the structural components. Passive energy dissipation systems (also known as passive control devices) have been developed to achieve the above objectives. These systems include a range of materials and devices for enhancing damping, stiffness, and strength. In general, they are

characterized by their capability to dissipate energy either by conversion of kinetic energy to heat or by transfer of energy among different modes of vibration. The first category, referred to as passive dampers, includes supplemental devices which operate on principles such as frictional sliding (friction dampers), yielding of metals (hysteretic and dampers), phase transformation in metals (shape memory alloys), deformation of viscoelastic solids (viscoelastic dampers), and fluid orificing (fluid dampers). The second category, referred to as tuned systems, includes supplemental devices which act as vibration absorbers such as tuned mass dampers, tuned liquid dampers, and tuned liquid column dampers.

3.1 Effect of Supplemental Damping on Seismic Response

The advantages of damping in structural components have long been recognized and accepted. Although the nature of the energy dissipation through damping is not fully understood for a structural system, inherent equivalent viscous damping in the range of two to five percent of critical has been generally accepted in practice and used in dynamic response analysis and design.

3.1.1 Influence on maximum displacement

For a single-degree-of-freedom structure undergoing free or forced vibration, an increase in the damping ratio of the system ξ results in a reduction in the vibration amplitude. This can be observed from the equation of motion of a damped single-degree-of-freedom (SDOF) system with natural frequency ω . Under free vibration, the displacement of the oscillator $x(t)$ is given as

$$x(t) = e^{-\xi\omega t} \left[x_0 \cos(\omega_d t) + \frac{\dot{x}_0 + x_0 \xi \omega}{\omega_d} \sin(\omega_d t) \right] \quad (1)$$

where x_0 and $\dot{x}_0$ are the initial displacement and velocity and ω_d is the damped vibration frequency ($\omega_d = \omega\sqrt{1-\xi^2}$). For a SDOF system with stiffness k under a harmonic excitation $P\sin\hat{\omega}t$, the displacement is given (Clough & Penzien, 1995) as

$$\frac{x(t)}{P/k} = \frac{1}{[1-(\hat{\omega}/\omega)^2]^2 + (2\xi\hat{\omega}/\omega)^2} [(1 - (\hat{\omega}/\omega)^2)\sin\hat{\omega}t - 2\xi\hat{\omega}/\omega \cos\hat{\omega}t] \quad (2)$$

Equation (2) indicates that a significant reduction in the displacement amplitude can be expected with large values of ξ when the excitation frequency $\hat{\omega}$ is close to the natural frequency ω of the system ($\hat{\omega}/\omega \approx 0.8 \sim 1.2$) as show in figure 18. If the excitation frequency is not close to that of the system, supplement viscous damping will not significantly impact the response.

For earthquake ground motions, the spectral displacement SD is an important parameter in estimating the maximum displacement and the base shear in the structure. The effect of original and supplemental damping on SD has been studied extensively. Ashour and Hanson (1987) proposed a relationship which describes the decrease in the elastic SD with the increase in ξ . They used SDOF structures with natural period $T=0.5s$ to $3.0s$ (with increments of $0.5s$), and damping ratios of 0, 2, 5, 10, 20, 30, 50, 75, 100 (critical damped), 125 and 150 percent. The excitations considered of three real and twelve artificial accelerograms. The computed SD for each natural period was normalized to the SD for zero and for 5 percent damping ratios for each earthquake

record. The results of their statistical analysis let to the introduction of a reduction factor r_f which for normalization to zero damping is given as

$$r_f = \sqrt{\frac{1-e^{-\xi B}}{\xi B}} \quad (3)$$

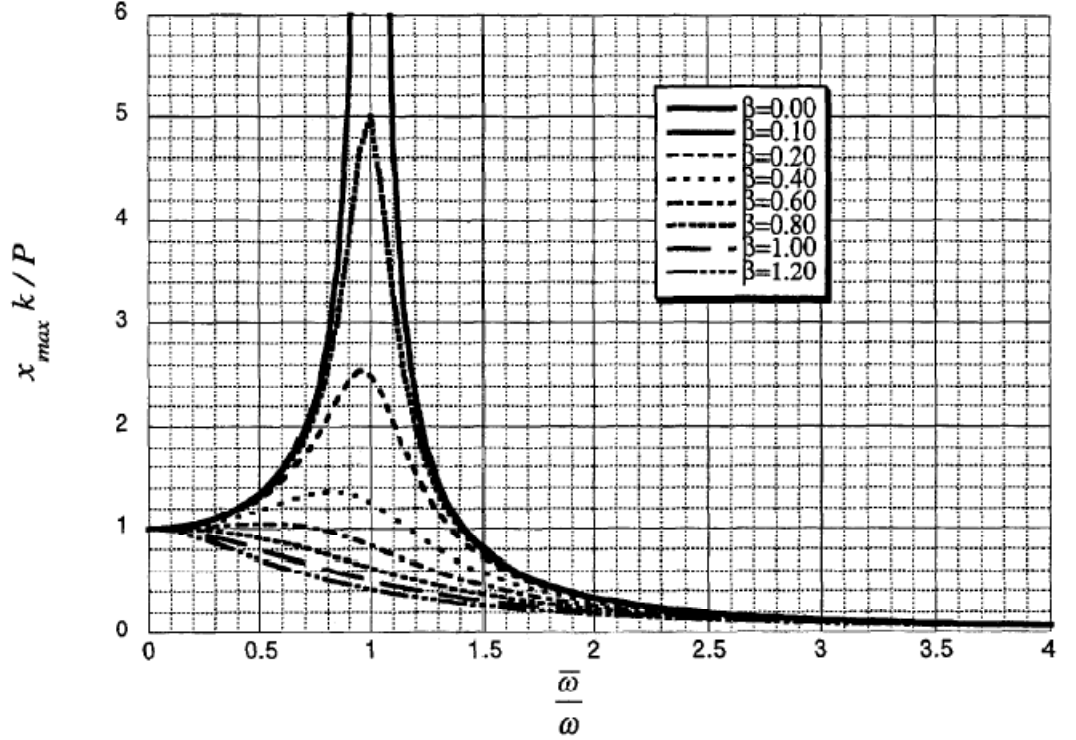


Figure 18. Peak response of a SDOF structure with different damping ratios to harmonic loading

and for normalization to five percent damping as

$$r_f = \sqrt{0.05 \frac{1-e^{-\xi B}}{\xi(1-e^{-0.05B})}} \quad (4)$$

where B is a parameter that ranges from 24 (upper bound) to 140 (lower bound) for zero damping and from 18 (upper bound) to 65 (lower bound) for five percent

damping. It is evident from the equations that an increase in the damping ratio reduces the displacement response significantly.

Because earthquake resistant design codes use reduction factors (R_w) to account for inelastic action, it is important to consider the effect of increased damping on the inelastic response of SDOF systems. Wu and Hanson (1989) studied response of SDOF structures with supplemental damping devices. The study considered linear SDOF systems with periods ranging from 0.1 s to 3.0 s with increments of 0.1 s and damping ratios of zero, 2, 5, 10, 20, 40, 60, 80, 100, and 120 percent. The structures were subjected to a set of 72 horizontal components of accelerograms from 36 stations in the western United States. These records include a wide range of earthquake magnitudes (5.2 to 7.7), epicentral distances (6 km to 127 km), peak ground accelerations (0.044 g to 1.172 g), and two soil conditions (rock and alluvium). The relative displacement and absolute acceleration response ratios are computed as the ratio of the peak response of the structure with higher damping ratios to the peak response with damping ratio of 0, 2 and 5 percent.

The mean response ratios for the 72 records are presented in figures 19 to 21. The figures show that increasing the supplemental damping results in further reductions in the displacement response. While higher damping ratios (greater than approximately 40 percent) provide further reductions in the displacement response, the additional reduction is not significant and may adversely affect the absolute acceleration response, especially for structures with longer periods (i.e. flexible structures as suspended bridge).

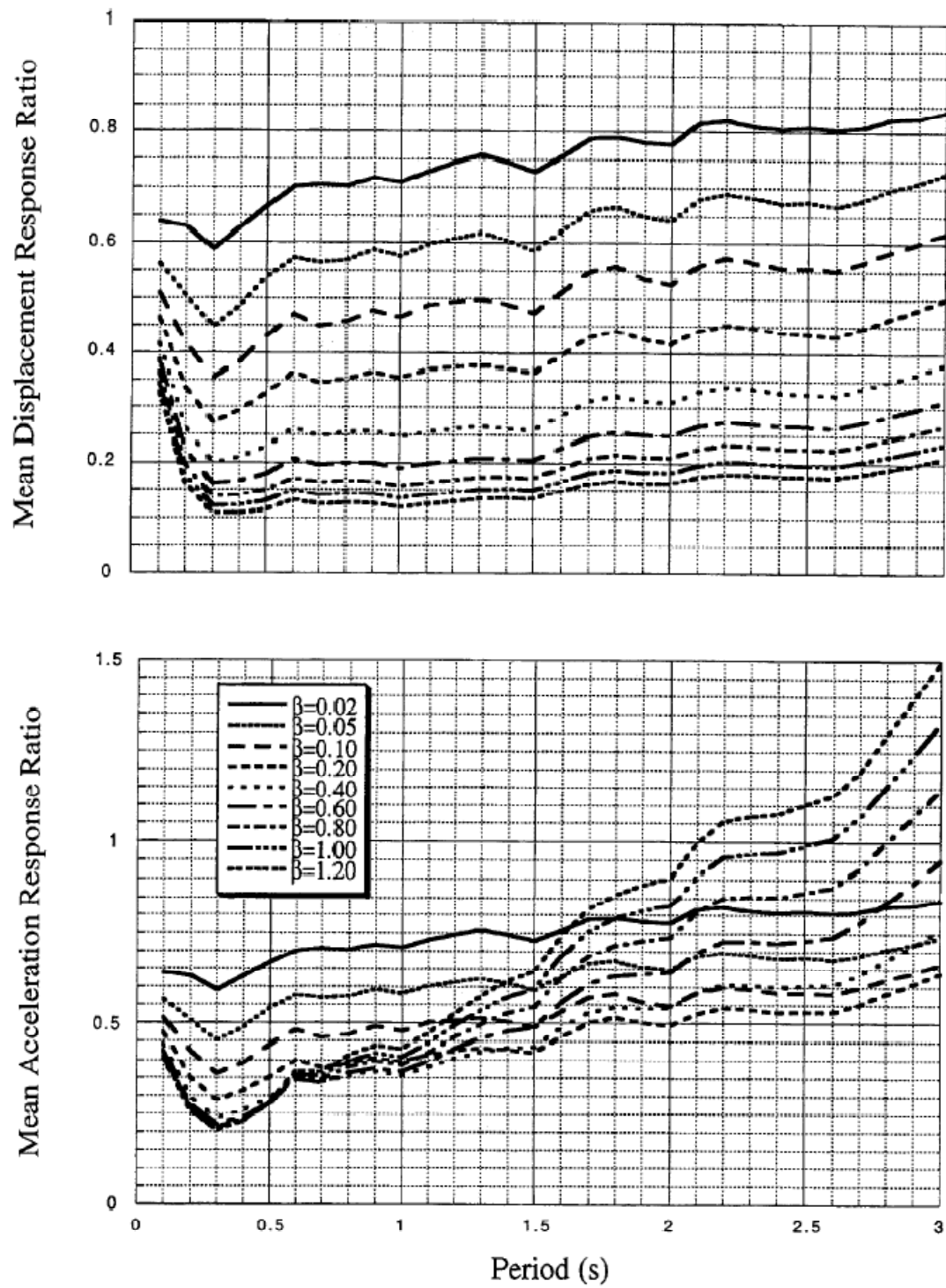


Figure 19. Mean response ratios for SDOF structures with supplemental damping ($\xi_0=0$)

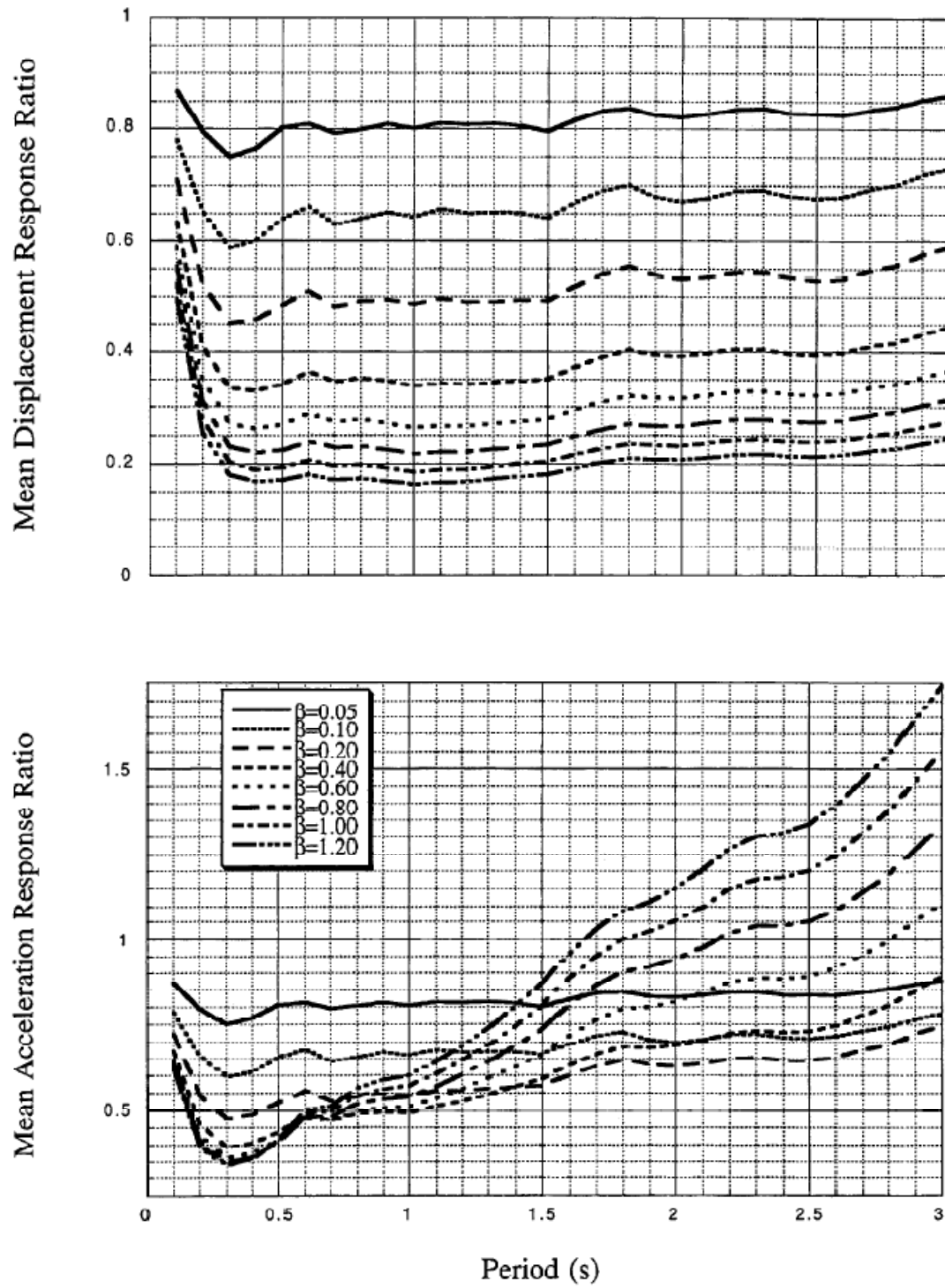


Figure 20. Mean response ratios for SDOF structures with supplemental damping ($\xi_0=0.02$)

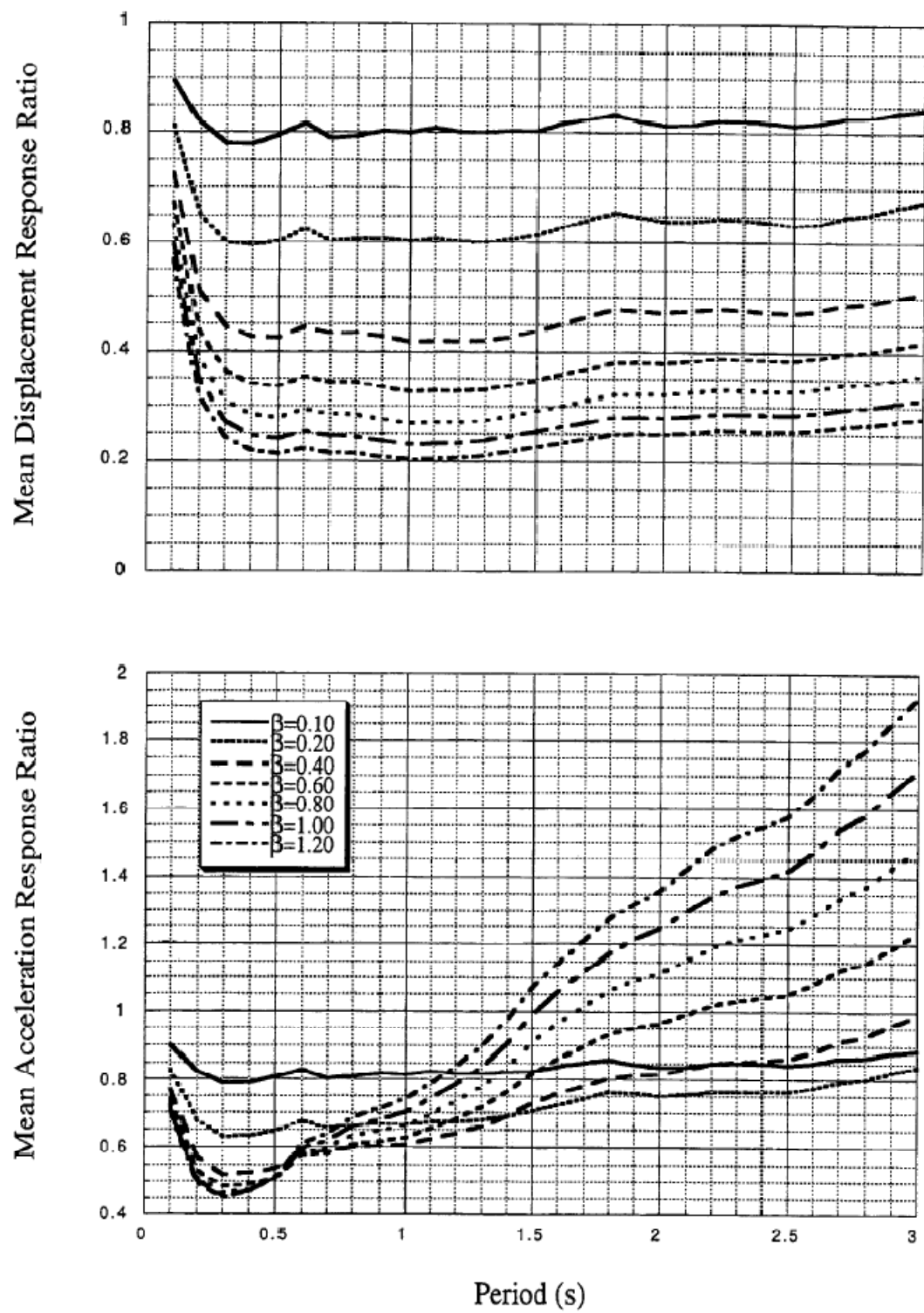


Figure 21. Mean response ratios for SDOF structures with supplemental damping ($\xi_0 = 0.05$)

3.1.2 Influence on dissipated energy

In order to investigate the influence of the damping on the absorbed energy in structure with different natural periods, a parametric analysis over the two variables was conducted (ξ and T).

Results show that add a viscous damper reduce the displacements and the acceleration as well. This is caused by the absorption of energy provided by the device. Assume as example a structure similar to the one represented in figure 2 and consider to increase the viscosity of the device, till reach an extremely high C ($\xi = \left(\frac{C}{2M\omega_n}\right)$). The device will lock and transfer all the inertia force as shear in the column. Extreme high C not only do not produce any vantage in the structure, but can also obstruct the work of a rubber isolator.

A Matlab program was written to perform several time histories with different combination of K and C to find the optimum point. The model implemented is represented above in figure 22:

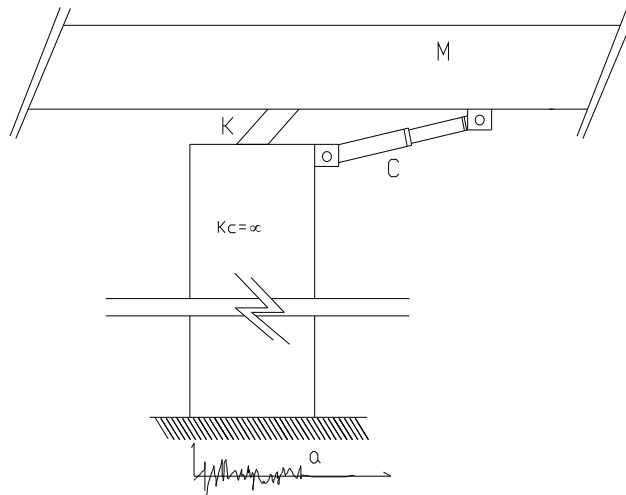


Figure 22. Simple SDOF structure with added damper

The integration algorithm to solve the equation of motion used is Newmark constant acceleration method ($\gamma=0.5$; $\beta=0.25$).

A scheme of the iteration is now presented from Chopra:

NEWMARK'S METHOD: LINEAR SYSTEMS	
Special cases	
(1) Average acceleration method ($\gamma = \frac{1}{2}, \beta = \frac{1}{4}$)	
(2) Linear acceleration method ($\gamma = \frac{1}{2}, \beta = \frac{1}{6}$)	
1.0 Initial calculations	
1.1	$\ddot{u}_0 = \frac{p_0 - c\dot{u}_0 - ku_0}{m}$.
1.2	Select Δt .
1.3	$\hat{k} = k + \frac{\gamma}{\beta \Delta t} c + \frac{1}{\beta (\Delta t)^2} m$.
1.4	$a = \frac{1}{\beta \Delta t} m + \frac{\gamma}{\beta} c$; and $b = \frac{1}{2\beta} m + \Delta t \left(\frac{\gamma}{2\beta} - 1 \right) c$.
2.0 Calculations for each time step, i	
2.1	$\Delta \hat{p}_i = \Delta p_i + a\dot{u}_i + b\ddot{u}_i$.
2.2	$\Delta u_i = \frac{\Delta \hat{p}_i}{\hat{k}}$.
2.3	$\Delta \dot{u}_i = \frac{\gamma}{\beta \Delta t} \Delta u_i - \frac{\gamma}{\beta} \dot{u}_i + \Delta t \left(1 - \frac{\gamma}{2\beta} \right) \ddot{u}_i$.
2.4	$\Delta \ddot{u}_i = \frac{1}{\beta (\Delta t)^2} \Delta u_i - \frac{1}{\beta \Delta t} \dot{u}_i - \frac{1}{2\beta} \ddot{u}_i$.
2.5	$u_{i+1} = u_i + \Delta u_i, \dot{u}_{i+1} = \dot{u}_i + \Delta \dot{u}_i, \ddot{u}_{i+1} = \ddot{u}_i + \Delta \ddot{u}_i$.
3.0 Repetition for the next time step. Replace i by $i + 1$ and implement steps 2.1 to 2.5 for the next time step.	

For every combination of ξ ——— and K the value of shear on the column and relative displacement was plotted. They are presented in the surface plotted below.

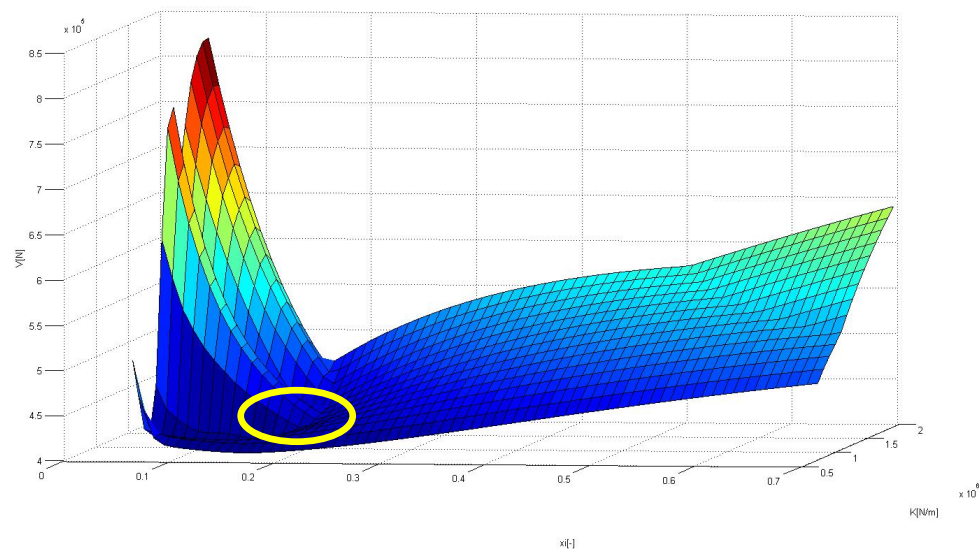


Figure 23. Shear - Viscous Damping - Stiffness relation

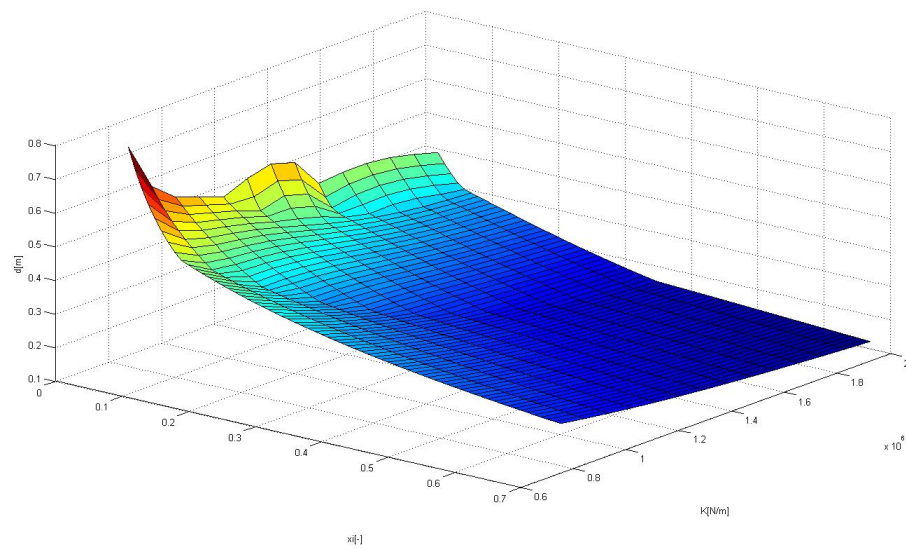


Figure 24. Relative displacement - Viscous Damping - Stiffness relation

In figure 23 is possible to detect a relative surface minimum in the 3d function. These point represent the optimum C-K combination in terms of transferred shear. Is possible to see that the best damping is below the critical damping ($\xi = 1$).

The energy dissipated in one cycle of vibration by viscous damping force is:

$$\begin{aligned}
 E_D &= \int f_D du = \int_0^{2\pi/\omega} (c\dot{u})\dot{u} dt = \int_0^{2\pi/\omega} (c\dot{u}^2) dt = \\
 &= c \int_0^{2\pi/\omega} [\omega u_0 \cos(\omega t - \varphi)]^2 dt = \pi c \omega u_0^2 = 2\pi \xi \frac{\omega}{\omega_n} k u_0^2 \quad (5)
 \end{aligned}$$

The energy dissipated is proportional to the square of the amplitude of motion.

For every combination of $\xi = \left(\frac{c}{2M\omega_n}\right)$ and K the value of energy dissipated by viscous damping force was plotted. It is presented in the surface plotted below.

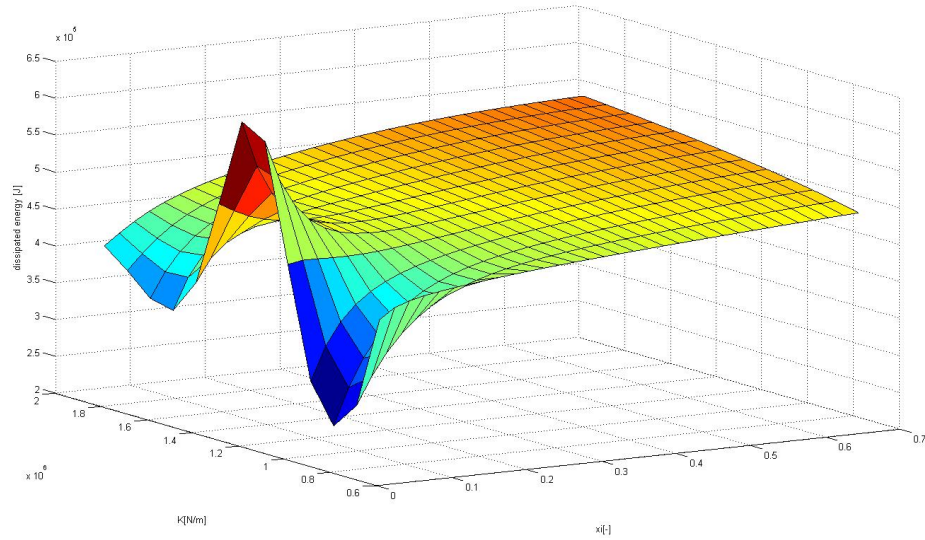


Figure 25. Energy dissipated - Viscous Damping - Stiffness relation

The optimum combination that minimize the force transmitted from the deck to the column do not correspond to the maximum level of energy dissipated.

Other analysis were conducted in order to verify if the function Dissipated Energy vs. ξ is always an increasing function as seems from the previous plot.

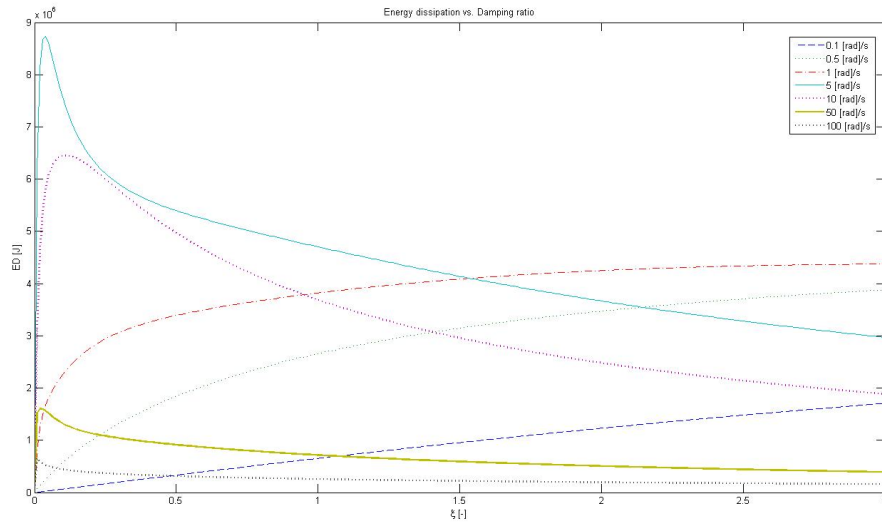


Figure 26. Energy dissipated - Viscous Damping - Stiffness relation

The ξ corresponding to the maximum dissipation of energy could be less than the critical damping. In particular the higher is the frequency the lower is the value of ξ that at maximum dissipated energy. For frequency higher than circa 1 Hz the maximum value of dissipated energy is less than the lower frequency one. Further studies could be conducted on a damping model that increase the damping with the stiffness (for example Rayleigh model).

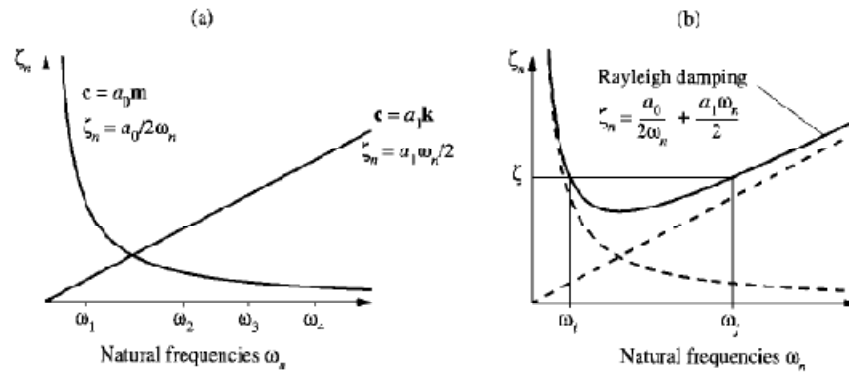


Figure 27. Variation of modal damping ratios with natural frequency:

a) mass-proportional damping and stiffness-proportional damping; b) Rayleigh damping (Chopra 2001)

The result could be that for higher modes of a structure the dissipated energy goes to zero due to the increased frequency and damping ratio. In particular condition is possible that a constant damping model dissipate more than a higher values frequency-dependant one. Sounds reasonable that the first modes dissipates more energy in a dynamic response of a structure.

3.2 Passive Damping Devices

From the previous section, it is apparent that damping in structures can significantly reduce the displacement and acceleration responses, and decrease the shear forces, along the height of buildings. The use of supplemental passive dampers in buildings is desirable for the following reasons:

1. Supplemental dampers can provide the building with additional stiffness and damping to reduce the response.

2. Energy dissipation in buildings can be confined mainly to supplemental dampers.

3. Damage to the building can be limited to supplemental dampers which are easier to replace than structural components and do not affect the gravity load-resisting system

Passive energy dissipation devices are used extensively in other areas of vibration control such as shock absorbers for vehicles, vibration isolators for equipment, pipe restraints, and shock isolation devices for mitigation of blast effects. In the last two decades, much effort has been directed towards applying passive energy dissipation techniques to seismic applications. Many of the devices that have emerged for passive control were first developed as damping devices for seismic base isolation systems. Several passive damping devices have been suggested and used for wind and earthquake loads. The devices are categorized according to how they operate. Following is a brief discussion of the applications of each device:

3.2.2 Friction Dampers

In the case of friction dampers, the design philosophy to enhance the structural performance is to provide a way for the structure to yield without damaging the existing structural members: seismic energy is dissipated by mean of friction, i.e. by making steel plates sliding one against the other, while bolts hold the steel plates together providing the normal component of the friction force. Sliding plates are fixed to the cross braces and then clamped together. At a given sliding load, the plates begin to slide and dissipate energy. Varying the sliding load will alter the seismic energy attracted by the structure.

Incorporating the braces adds initial lateral stiffness to the system, thus lowering the natural period of the structure and providing a margin over which the structure can shift its period if resonance is encountered: any time the current structural period attracts seismic energy enough to activate the friction dampers, the resonance phenomenon can be avoided by a period shift. When in fact at the low braced period the structure attracts large amounts of seismic energy, the structure begins to soften as the friction dampers begin to slip and dissipate energy: the reduced lateral stiffness of the structure, due to the dampers slippage, causes the desired period shift. If the braced natural period is moved far from the un-braced natural period, the structure will have a sufficient ability to soften.

Advantages and disadvantages of friction dampers and environmental effects

These devices possess good characteristics of structural behavior. Some of their *advantages* are the listed below:

- high capacity of energy dissipation; compared to devices based on yielding of metals, friction dissipators possess a great capability of absorbing energy
- This characteristic disappear with the wearing of the sliding surfaces
- behaviour is not seriously affected by the amplitude, the frequency contents or the number of cycles of the driving force
- controllable friction force (through the pre-stressing normal force).
- not affected by fatigue effects; the materials are low maintenance or even maintenance free
- perform well in various environmental conditions such as temperature.

- the damper design is straightforward and low tech: the design does not require
- expensive engineering design costs or testing prior to implementation

Some potentially relevant *disadvantages* exhibited by friction dissipators are

- the energy dissipated per cycle is only proportional to the maximum displacement instead of the square of the same displacement, as in the case of viscous damping: this can be relevant for sudden pulses and for inputs stronger than those expected
- moreover, resonance peaks can not be properly cut
- durability is also a controversial issue, mostly due to the high sensitivity of the
- coefficient of friction to the conditions of the sliding surfaces
- high frequencies can be introduced in the response, due to the frequent and sudden
- changes in the sticking-sliding conditions. The dynamic highly non-linear behavior of friction dissipators makes their numerical simulation very difficult. This situation has arisen some controversial issues, such as the possible introduction of high frequencies into the structural response, as well as the lack of studies of these devices when subjected to near-fault pulses. Environmental effects might alter the frictional characteristic of the sliding interface.

Critical conditions to be assessed in a design situation are:

- localized heating of the contacting materials during slippage: on occasions, these thermal effects may alter the frictional response by causing material softening or by promoting oxidation. However, for the type of sliding systems typically

encountered with friction dampers, system response will be barely sensitive to the relatively small variations in ambient temperature.

- atmospheric moisture and contaminants: physic-chemical processes may be triggered by atmospheric moisture or contaminants, occurring at the material interfaces. These processes may change the physical and chemical character of the surfaces, thus significantly affecting the frictional response.
- formation of oxide layers or scale on the exposed surfaces.
- crevice corrosion (cathodic/anodic effect between exposed and inaccessible regions) and bimetallic corrosion: in aggressive environments, corrosion may be a problem. It is necessary to rely on physical testing to determine the extent of corrosion expected in a given situation and to find out the potential effects on the frictional characteristics of a sliding system.

Friction damper typologies: slotted-bolted connections

The simplest form of friction dampers are the Slotted-bolted connections introduced at the end of conventional bracing members (fig. 28). It is important to ensure that the slippage of the device occurs before the compressed braces buckle or yield. Each connection incorporates a symmetric shear splice with slotted holes in the connecting plates extending from the bracing member: the slot length has to accommodate the maximum slip anticipated from the design earthquake. Disc spring washer can be used in the bolting assembly to accommodate the possible variation in the plate thickness due to the wear at the contact surfaces and to the temperature rise resulting from friction heating. Tests results performed by Pall et al. (1982) show that sliding connections can

exhibit a very high energy dissipation capability under extreme loading conditions, provided that appropriate materials and bolt clamping forces are used.

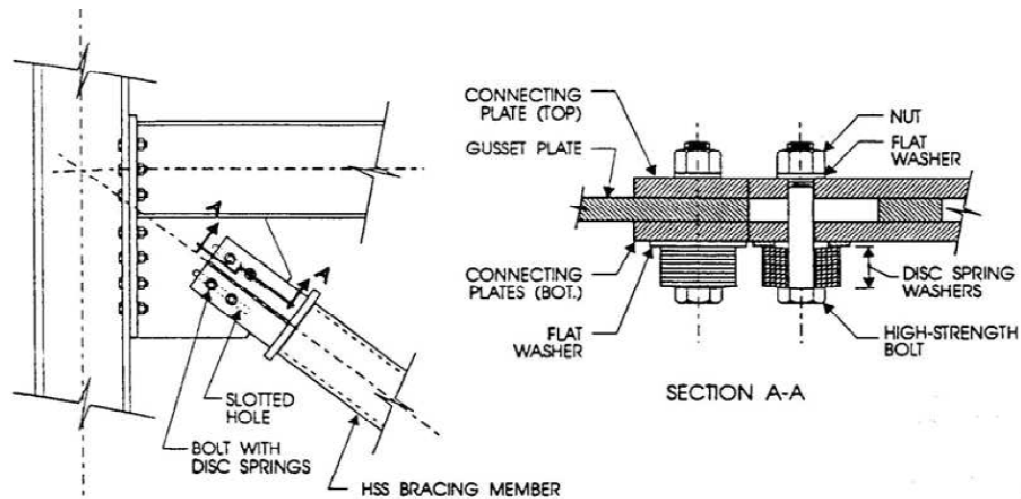


Figure 28. Slotted Bolted Connection Assemblage

Friction damper typologies: Pall devices

The Pall Device consists of diagonal brace elements with a friction interface at their intersection, connected together by horizontal and vertical link elements (fig. 29, left). These link arms ensure that when the load applied to the device via the braces initiate the slip on the tension diagonal, then the compression diagonal will also slip of an equal amount in the opposite direction (fig. 29, right). The normal force on the sliding interface, responsible for the friction resistance, is achieved through a bolt at the intersection of the diagonal arms. Utilization of this type of geometric deformation in the cross bracing of a building frame (or in multi-bent bridge pier or deck) is a way to permit substantial controlled energy dissipation (Tyler, 1985), as shown by the typical hysteretic loop of a Pall Device in fig. 29.

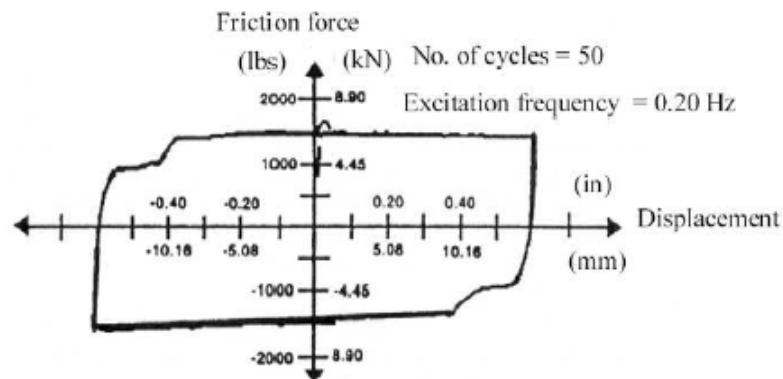
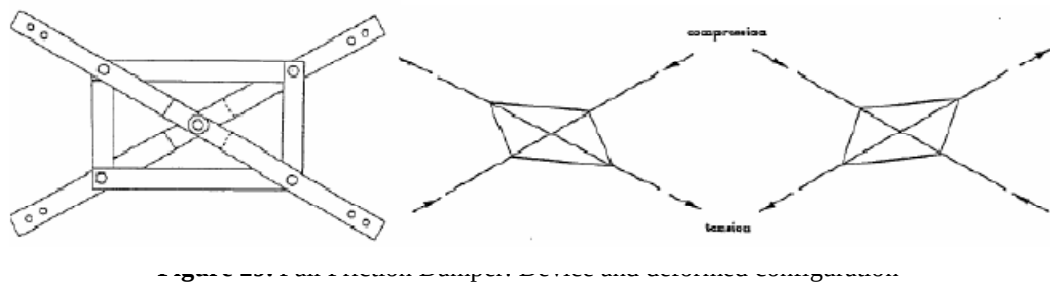


Figure 30. Hysteretic loop of a Pall Friction Damper

Friction damper typologies: Somitomo devices

The Sumitomo Device was designed and developed by Sumitomo Metal Industries, Ltd., Japan, originally as a shock absorber in railway rolling stock. It is a cylindrical device with friction pads that slide directly on the inner surface of the steel casing of the device (fig. 31). The friction devices might be attached to the underside of the floor beams and connected to chevron brace assemblages. The Sumitomo dampers exhibited outstanding behavior: their hysteretic behavior is extremely regular and repeatable (fig. 32). The devices show almost no variation in slip load during earthquake

motion; their force- displacement response is known to be quite independent of loading frequency, amplitude, number of loading cycles, and temperature.

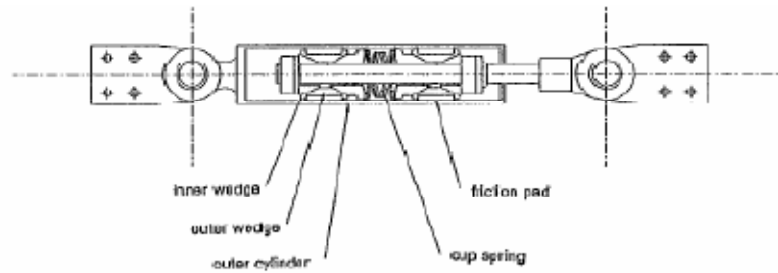


Figure 31. Section of the Sumitomo friction damper

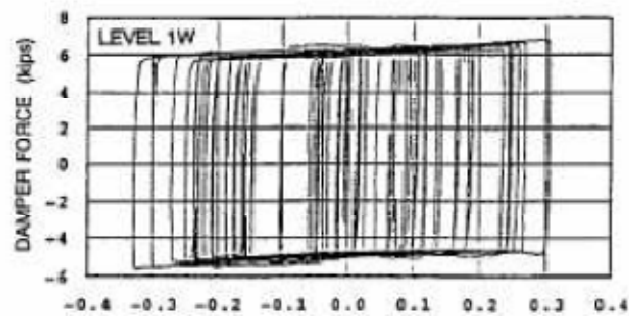


Figure 32. Typical Sumitomo damper hysteresis loop (Aiken 1993)

3.2.3 Steel hysteretic dampers

Hysteretic dampers originated in New Zealand in the early 1970's, and were first used in the U.S. in the early 1980's. Hysteretic dampers dissipate energy by flexural, shear or extensional deformation of the metal in the inelastic range. Typically, mild steel plates with triangular or hourglass shapes are used.

These devices are able to sustain repeated cycles of stable yielding, avoiding premature failure. Further, they are reliable, maintenance free, not sensitive to temperature variations and not subjected to aging. In continuous span bridges, they may be located either in one position (e.g. one abutment) to allow free movements of the bridge (in this case they must be designed for very large forces), or distributed in several locations to allow thermal movements of the structure (normally associated to hydraulic shock transmission units). The steel used for these devices must be characterized by a very high elongation at failure and a very low hardening, in order to grant a very high low-cycle fatigue life with negligible performance decay after many cycles. There are three types of metallic damper, according to their working principle:

- uniform moment bending beam with transverse loading arms;
- tapered-cantilever bending beam;
- torsional beam with transverse loading arms.

Several devices developed in the early 1980's showed some limits: reduced capacity to resist yield cycles without breaking, characteristic degradation after first cycles with progressive reduction of the yield force up to failure, asymmetry of the load-displacement cycles with stiffness variations in tension and compression and difficulty to provide uniform response in any direction.

New devices overcoming these limits have been developed. They are based on the combination of C-shaped elementary energy dissipators. Tests on these devices have shown long cyclic life, almost no cycle deterioration before failure and very good dissipation thanks to the almost square shape of the hysteresis loops.

These devices may constitute either the dissipative part of a seismic isolation system of the bridge deck, or they may simply act as dampers by themselves. Then they can be arranged to be a part of unidirectional or multidirectional bridge bearings. The conceptual design of the single damper unit is based on optimization criteria, i.e.:

- An optimized shape allows almost constant strain range across each section: in this way, the diffusion of plasticization is uniform over the most of the volume, and, by preventing localization and concentration of deformation, extended low-cycle fatigue life is obtained.
- Particular design arrangements neutralize the effects of geometry changes, that otherwise can cause strain hardening or softening behaviour and/or asymmetrization of the hysteresis cycles at large displacements: the dissipation effectiveness is improved, and large displacements and damping of response in all directions are allowed.

Hysteretic damper typologies: E-shaped devices

The E-shaped Device is shown in fig. 33: it can be viewed as a symmetric one storey, two-bays portal frame, hinged at the base (fig. 34). The device is forced to deform anti-symmetrically in the elasto-plastic range: the legs are designed to act essentially as lever arms, deforming elastically, while the energy dissipation occurs only in the transverse beam, where the desired uniform plasticization is ensured by the acting constant moment.

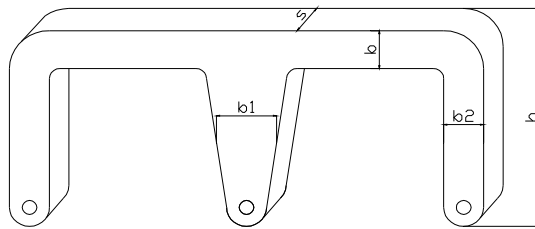


Figure 33. Typical E-shaped hysteretic damper layout

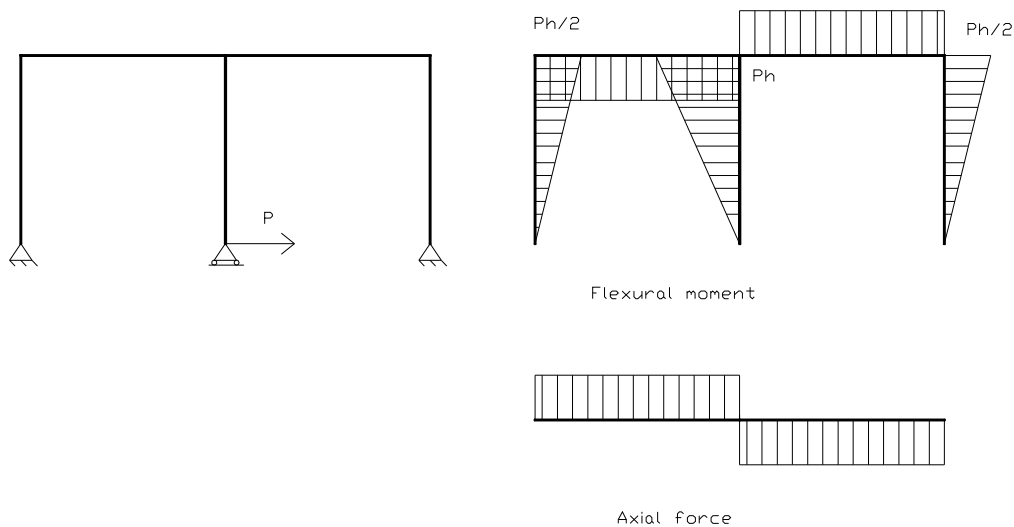


Figure 34. Static Scheme of an E-shaped hysteretic damper

The fact that both moment and axial force have opposite sign in the two parts of the transverse beam allows the neutralization of the geometry changes effects and permit to avoid the progressive accumulation of axial strain, that is the main source of deterioration at increasing number of cycles in those devices in which moment alternates and axial force does not: E-shaped Hysteretic Dampers are characterized by a high repetition of hysteresis cycles (>50), very low degradation after 50 cycles and very high dissipating efficiency ($>70\%$).

Hysteretic damper typologies: C-shaped and EDU devices

C-shaped elements grant very high energy dissipation, very high fatigue resistance and allow the realisation of multidirectional devices (fig. 35). A typical C-shaped damper has a semicircular shape (fig. 35, left), with constant radius r , while depth varies in order to ensure uniform plasticization through each section.



Figure 35. C-shaped device (left) and EDU device (right)

The EDU device

The EDU Device is a multi-composed device constituted by C-shaped elementary energy dissipators, combined in order to be forced to deform anti-symmetrically, i.e. for each compressed one, another is in tension; the radial symmetry allows uniform behavior under earthquake loading acting in any direction. This device can be coupled with hydraulic shock transmitters in parallel.

Different design requirements, different responses in longitudinal and transverse directions or unidirectional devices can be easily met by a suitable arrangement of the elementary dissipators or combining elementary dissipators with different stiffness.

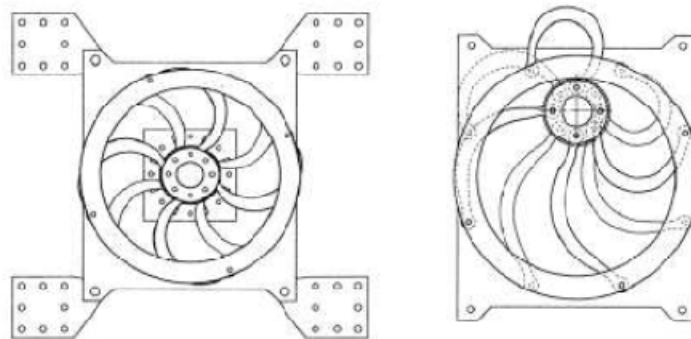


Figure 36. EDU Device: device and deformed configuration

The EDU device has been tested by Marioni (1996) with a real earthquake of 7.4 magnitude with PGA of 0.8g; the device proved to be able of dissipating much more energy than any other system and fulfilled European standards for in-service conditions. It showed self-recentering properties for thermal effects and small earthquakes. Due to its conceptual simplicity, the EDU device has low costs.

Hysteretic damper typologies: ADAS and TADAS elements

The Bechtel Added Damping and Stiffness (ADAS) device is another example of a hysteretic damper. The X-plate constituting the ADAS is was developed from the triangular plate devices born in New Zealand and firstly employed as piping support elements. ADAS elements are designed to dissipate energy through the flexural yielding deformation of mild-steel plates: they consist of multiple X-shaped mild steel plates configured in parallel between top and bottom boundary connections (fig. 37). A rectangular plate, when plastically deformed in double curvature, will yield only at its ends, giving a plastic concentration undesirable both in terms of the amount of energy that can be absorbed and by its inherent lack of stability and repeatability in the plastic range: the particular advantage of an X-plate is that, when deformed in double curvature, the plate deformation is uniform across its height, and once pushed into its plastic regime, the yielding is distributed and contemporary at all sections.

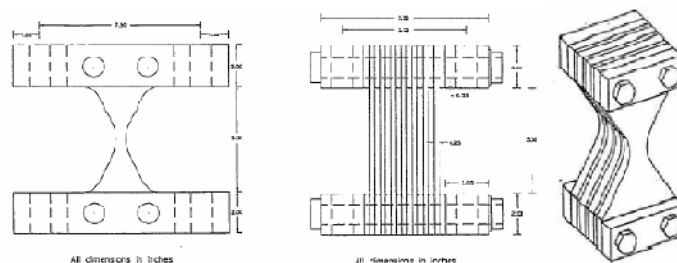


Figure 37. Added Damping and Stiffness (ADAS) element

A typical hysteresis loop from one test of a 7-plate element is shown in fig. 38. The primary factors characterizing the ADAS element behavior are the device elastic stiffness, the yield strength, and the yield displacement. ADAS elements are capable of

sustaining more than 100 loading cycles with a displacement ductility of 3, proving stable response and no degradation; they can safely be designed for displacement ductility ranges up to about 10.

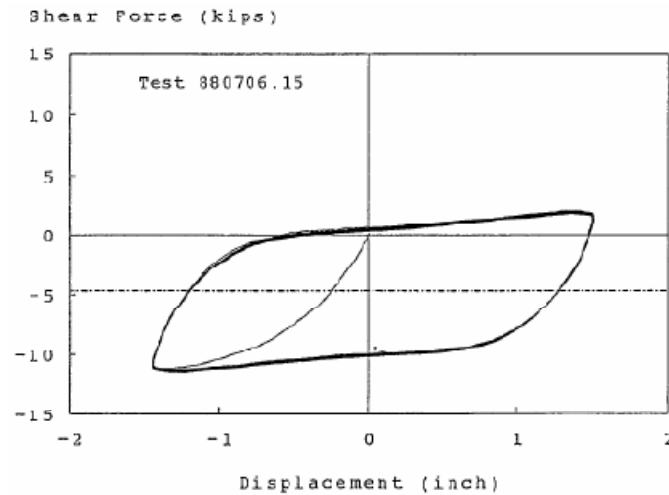


Figure 38. 7-Plate ADAS element hysteretic behavior

One practical configuration for installing ADAS devices in a structure is in conjunction with a chevron brace assemblage, designed using capacity principles, based on an ADAS element strength of at least twice the device yield strength. The performance of an ADAS element is influenced by the degree of restraint at its extremities, and the design of these connection details must be aware of this factor: experimental tests indicated the importance of rigid boundary connections for successful performance of ADAS elements. Possible shortcomings with X-shape ADAS are that the stiffness of the device is very sensitive to the tightness of the bolts and generally much less than that predicted by assuming both ends fixed; moreover, the flexural

behavior might be weakened when the device is subjected to axial loads. Triangular ADAS (TADAS) devices were developed to avoid these inconveniences.

TADAS elements use triangular steel plates instead of X-shape plates, with boundary conditions of welding at bottom and bolting at top (fig. 39). Stiffness varies linearly along the height, as well as moment does, implying constant curvature, and avoiding curvature concentration. Experimental Load-deformation relationships do not show significant stiffness or strength degrading. The TADAS device can be modelled as bilinear elasto-plastic, and it can sustain a large number of yielding reversals (fig. 39).

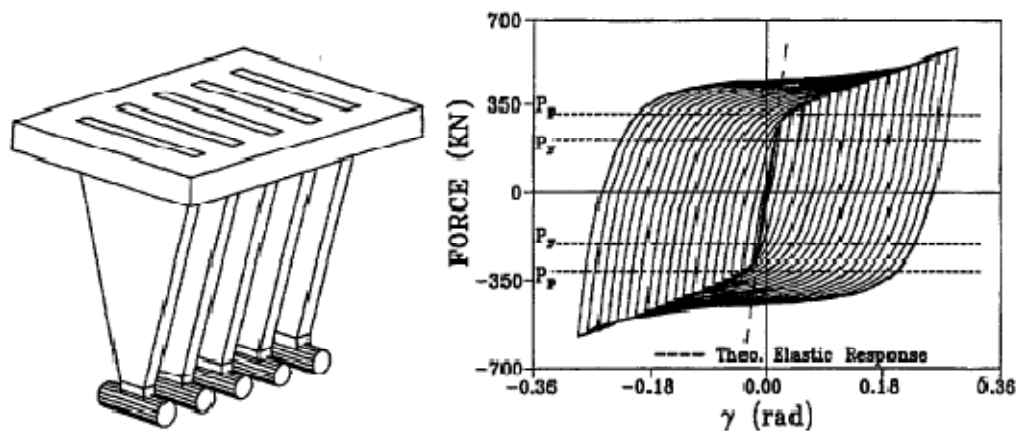


Figure 39. Hysteresis loops for T-ADAS devices (Tsai and Hong, 1992)

Hysteretic damper typologies: lead extrusion devices

The Lead Extrusion Dampers (LEDs) take the advantage of the extrusion of lead through orifices. Fig. 40 illustrates two types of lead extrusion dampers: the constricted tube, that forces the extrusion of the lead through a constricted tube, and the bulged shaft, that uses a bulged shaft through a lead cylinder. The main advantages of these

devices are due to the lead properties: the hysteretic behavior is essentially rectangular, stable and unaffected by number of load cycles (fig. 41), allowing to maximize the energy dissipation, it is unaffected by any environmental factor and fatigue is not a major concern, because strain rate has a minor effect and aging effects are insignificant.

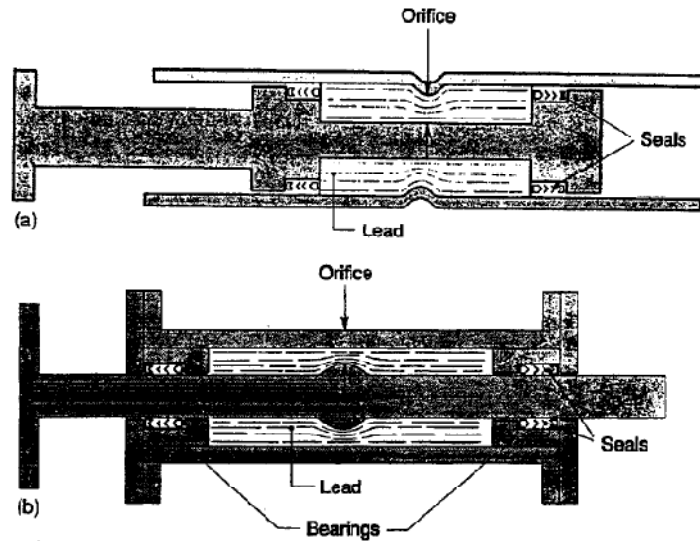


Figure 40. Longitudinal section of lead extrusion dampers (a) constricted-tube type and (b) bulged-shaft type (Skinner et al., 1993)

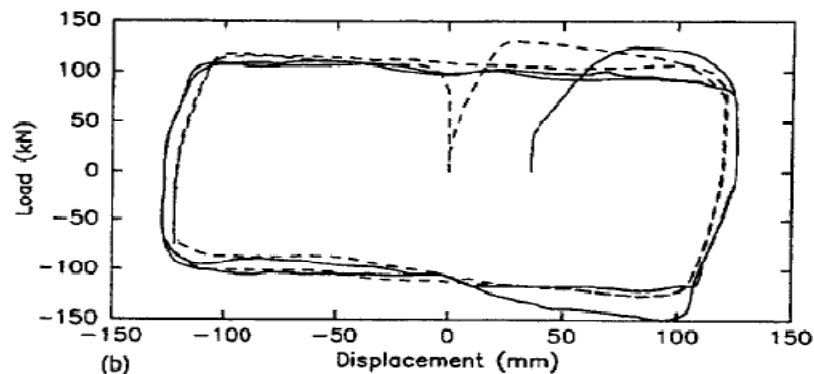


Figure 41. Hysteresis loops of LEDs (Robinson and Cousins, 1987)

3.2.4 Viscous and Viscoelastic dampers

Viscous Dampers

Linear devices produce damping forces proportional to the velocity of the damper deformation, greatly attenuating the higher-mode seismic response, that is only relatively reduced by high isolator damping. Hydraulic dampers utilize viscosity properties of a fluid to improve structural resistance against the earthquake. They are generally used as shock transmitters, able to allow slow movements (in service conditions) without valuable resistance, and stiffly react to dynamic actions. It should be possible to develop effective velocity dampers, of the adequate linearity, by using the properties of high-viscosity silicone liquids: a double-acting piston drives the silicon fluid cyclically through a parallel set of tubular orifices (fig. 42), giving high fluid shears and hence the required velocity-damping forces. By using a sufficient working volume of silicon to limit the temperature rise to 40°C during a design-level earthquake, the corresponding reduction in damper force is limited to about 25%. Shortcomings are the increasing of silicon volume with temperature (10%/100°C) and its tendency to cavitate under negative pressure.

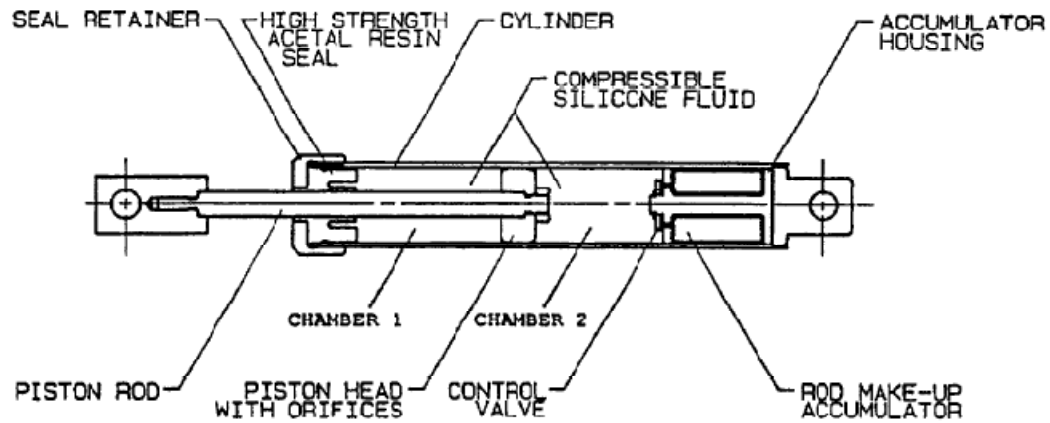


Figure 11. Construction of fluid pressure sampler (Constantine and Sydnor, 1967)

The force generated by the device can be described by the following:

$$F = CV^{\alpha} + A$$

where F is the force applied to the piston, V is the piston velocity, C , A and α are constants depending on the fluid and circuit properties; α may range between 0.1 and 2, according to the type of valves. Force-displacements plot for devices with different values of α subject to sinusoidal input are elliptical shaped.

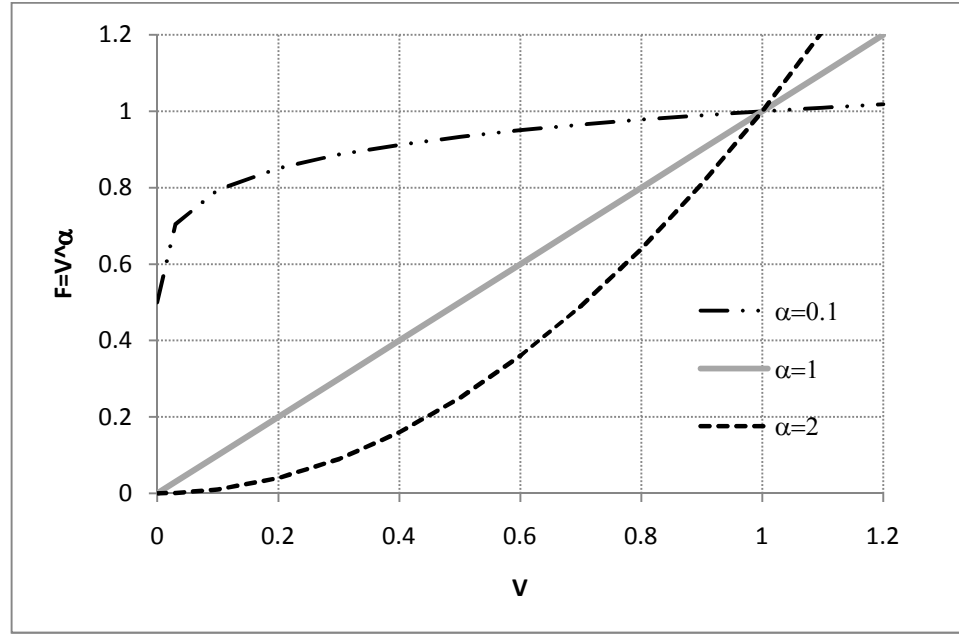


Figure 43. Force-velocity type dependence for different values of the parameter α

Fig. 43 illustrates the dependence of the force on the velocity, for different values of α . In case of low α the dissipated energy per cycle is maximised. When energy dissipation is required, $\alpha \leq 2$ is preferred in order to increase the hysteretic area; in this case they are called Viscous Dampers (VD), for which a reference value of α is generally 0.1. The parameter α higher than 2 is preferred when the difference of force at low and high velocities shall be maximized, in order to react stiffly as soon as the velocity increases, while allowing slow movements due to thermal variations, creep and shrinkage, and becoming rigid in case of dynamic actions (braking force and earthquake), or when energy dissipation is not required; in this case they are called Shock Transmission Devices (STD) or Hydraulic Couplers.

The *Taylor device* is a fluid damper incorporating a stainless steel piston with a bronze orifice head and an accumulator; the device is filled with silicon oil (fig. 42). The force generated by the fluid inertial damper is due to the pressure differential across the piston head. Due to the fluid compressibility, the volume reduction following the flow develops a restoring spring-like force, generally prevented by the use of an accumulator: test results indicate a cut off frequency of 4Hz, under which no stiffness is produced. This means that higher modes, with frequencies larger than the cut-off threshold, might be affected by the elastic component. The damping constant of the device is not importantly affected by the temperature. Analytical results also showed that modelling these dampers as simple linear viscous elements yields predicted responses in good agreement with experimental results; the purely viscous nature is evident in fig. 44.

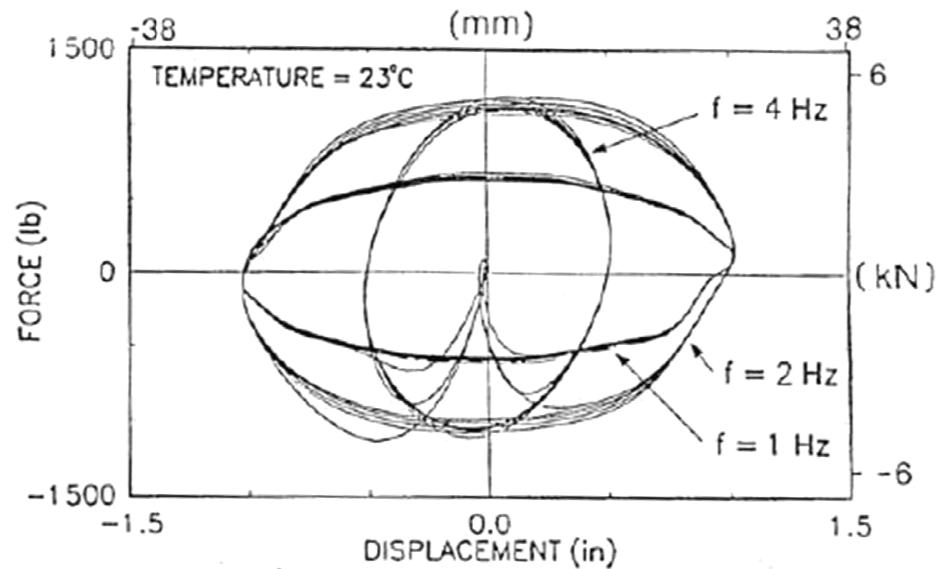


Figure 44. Experimental hysteresis loop of a Taylor fluid damper at various frequencies and temp.

Shock Transmission devices

Shock Transmission Devices (STD) can closely approximate the ideal parameters such that low velocity displacements are allowed with negligible resistance, withstanding high seismic loads with minimal deformation: they have an α value approaching 2. The oil filled cylinder is divided into two chambers by a piston and fixed to the structure normally through spherical hinges (allowing rotations up to $\pm 3^\circ$ in all directions), in such a way that the relative movement of the structure causes the piston to move inside the cylinder, allowing the oil flow from one chamber to the other through a hydraulic circuit. The oil flow through the circuit is practically independent of the external temperature: the constant performance level of the device is provided thanks to a design based on a turbulent oil flow practically independent from the viscosity of the fluid and therefore from its temperature. When the device dimensions are very large (it can reach 870mm of diameter and 2900mm of length in exceptional cases) an external hydraulic circuit and an external accumulator are preferred to the internal ones employed in smaller devices, to allow easy access, to avoid any interference between the flexural bending of the device due to own weight and to the behavior of the circuit, and to avoid overpressure in the cylinder due to the oil thermal expansion.

Viscoelastic dampers

Viscoelastic (VE) dampers have been used as energy dissipation devices in structures where the damper undergoes shear deformations. As their name implies, viscoelastic materials exhibit combined features of elastic solid and viscous liquid when deformed, i.e. they return to their original shape after each cycle of deformation and

dissipate a certain amount of energy as heat. Mahmoodi (1969) described the characteristics of a constrained double-layer viscoelastic shear damper and indicated that it can be effective in reducing the dynamic response of structures. These dampers, made of bonded viscoelastic layers (acrylic polymers), have been developed by 3M company and used to control wind-induced vibrations in buildings. The 3M dampers are known to have a stable behavior with good aging properties and resistance to environmental pollutants. The extension of VE shear dampers to seismic applications is more recent. For seismic applications, more effective use of VE materials is required since larger damping ratios than use for wind are usually required.

A typical VE shear damper consists of viscoelastic layers bonded to steel plates, fig. 45. When mounted to a structure, shear deformations and consequently energy dissipations take place when relative motions occur between the center plate and the outer steel flanges. To understand their behavior under a sinusoidal load with a frequency $\hat{\omega}$, the shear stress $\tau(t)$ can be expressed in terms of the peak shear strain γ_0 and peak shear stress τ_0 as (Zhang et al., 1989)

$$\tau(t) = \gamma_0 [G'(\hat{\omega}) \sin \hat{\omega} t + G''(\hat{\omega}) \cos \hat{\omega} t] \quad (6)$$

where $G'(\hat{\omega}) = \tau_0 \cos \delta / \gamma_0$, $G''(\hat{\omega}) = \tau_0 \sin \delta / \gamma_0$, and δ is the phase angle between the shear stress and the shear strain. Equation (6) can be rewritten as

$$\tau(t) = G'(\hat{\omega}) \gamma(t) \pm G''(\hat{\omega}) \sqrt{\gamma_0^2 - \gamma(t)^2} \quad (7)$$

which defines an elliptical stress-strain relationship similar to that shown in fig. 45. The area of the ellipse indicates the energy dissipated by viscoelastic material per

unit volume and per cycle of oscillation. From equation (7), it can be seen that the in-phase term $G'(\gamma(t))$ represents the elastic stiffness component, and the out-of-phase term $G''(\gamma(t))$ the damping component. Rewriting the equation (7) as

$$\tau(t) = G'(\gamma(t)) \dot{\gamma}(t) + G''(\gamma(t)) \gamma(t) \quad (8)$$

and comparing it with the equation of a SDOF system, the equivalent damping ratio of the VE material is obtained as

$$\zeta = \frac{G''(\gamma(t))}{2G'(\gamma(t))} \quad (9)$$

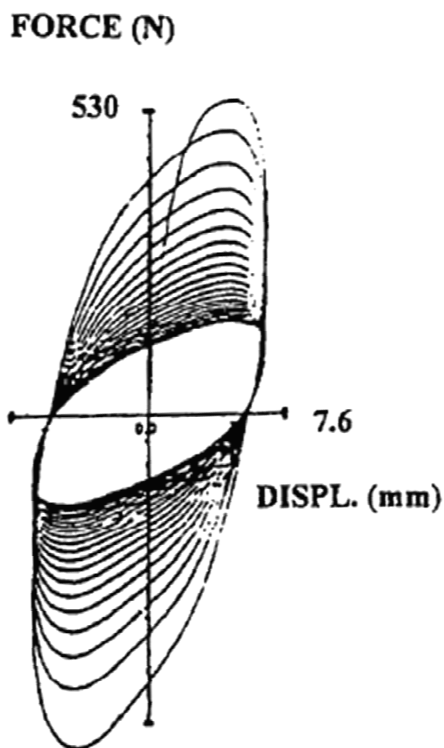


Figure 45. Elliptical force-displacement loops for VE dampers under cyclic

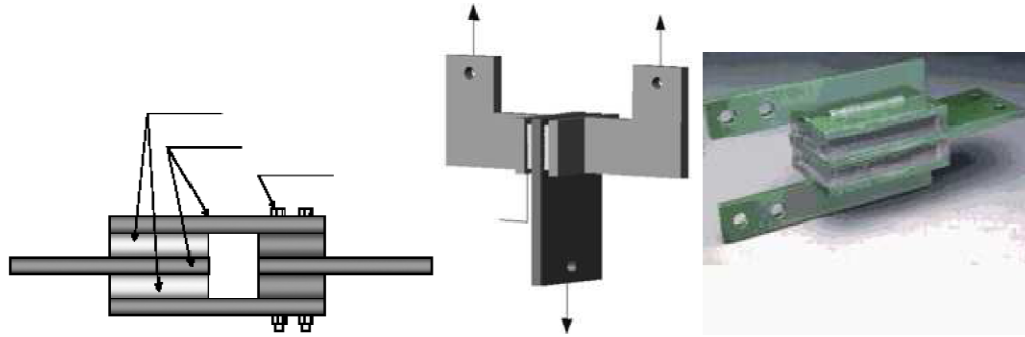


Figure 46. VE damper part of bracing member: typical scheme (front and 3D views) and picture

3.2.4 Device typologies: *self centering dampers*

Generally dampers do not limit the residual displacements of the structure after a seismic event. Some recently developed damper systems incorporate re-centering capabilities, characterized by the so-called flag-shaped hysteretic loop. The main advantage of the self-centring behavior consists in reducing permanent offsets when the structure deforms inelastically. A number of different devices have been developed, among these:

- Shape Memory Alloys Dampers (SMA);
- Energy Dissipating Restraint (EDR);
- The Friction Spring Seismic Dampers;
- The Post-Tensioned Energy Dissipating (PTED) steel connections.

Self-centering damper typologies: shape memory alloys dampers

Shape-Memory Alloys (SMAs) have the capacity to undergo large strains and subsequently to recover their initial configuration. The basis for this behavior is that,

rather than deforming in the usual manner of metals, shape-memory alloys undergo transformations from the austenitic to the martensitic crystal phase. Sasaki (1989) studied the suitability of Nitinol for energy dissipation under seismic-type loading, and investigated flexural, torsional, shear, and axial modes of deformation. Witting (1992) have studied of shape memory alloys for energy dissipation applications in structures. A Nitinol energy dissipator has the particular advantages of being mechanically very simple and reliable.

The use of the SMA restrainers in multi-span simply supported bridges at the hinges and abutments can provide an effective alternative to conventional restrainer systems: the SMA restrainers can be designed to provide sufficient stiffness and damping to limit the relative hinge displacement. The SMA restrainers may be connected from pier cap to the bottom flange of the girder beam in a manner similar to typical cable restrainers, as shown in fig. 47. The restrainers are typically used in a tension only manner, with a thermal gap to limit the engaging of the restrainer during thermal cycles. If adequate lateral bracing could be provided, the restrainers can be made to act in both tension and compression.

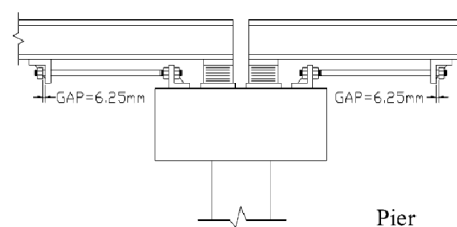


Figure 47. Configuration of shape memory alloy restrainer bar used in multi-span simply supported bridges

Des Roches and Delemont (2002) investigated the effectiveness of the SMA restrainer bars in bridges through an analytical study of a multi-span simply supported bridge. The results showed that the SMA restrainers reduce relative hinge displacements at the abutment much more effectively than conventional steel cable restrainers. The large elastic strain range of the SMA devices allows them to undergo large deformations while remaining elastic and due to their superelastic properties, they are able to maintain their effective stiffness for repeated cycles, differently from conventional restrainer cables once yielded. In addition, the superelastic properties of the SMA restrainers results in energy dissipation at the hinges.

Finally, evaluation of the multi-span simply supported bridge subjected to near-field ground motion showed that the SMA restrainer bars are extremely effective for limiting the response of bridge decks to near-field ground motion: the large pulses from the near- field record produced early yielding in conventional cable restrainers, thus reducing their effectiveness and resulting in large relative hinge displacements for the reminder of the response history. The SMA restrainer is able to resist repeated large cycles of deformation while remaining elastic. In addition, the increased stiffness of the SMA restrainers at large strains provides additional protection from unseating as the relative hinge opening approaches the critical value.

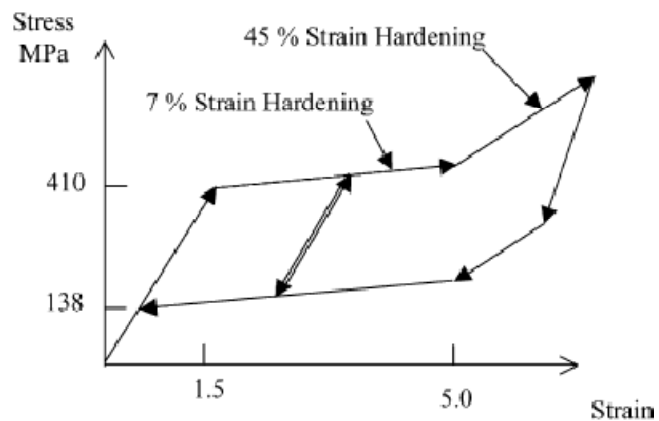


Figure 48. Scheme of hysteretic behavior model of SMA restrainer

Self-centering damper typologies: the energy dissipating restraint

Fluor Daniel, Inc., has developed and tested a type of friction device, called Energy Dissipating Restraint (EDR), originally used as a seismic restraint for the support of piping systems in nuclear power plants. The mechanism of the EDR consists of sliding friction wedges with a stop located at the end of the range of motion. A complete detail for this device can be seen in fig. 49. The main components of the device are the internal spring, the steel compression wedges, the Bronze friction wedges sliding on a steel barrel, the stops at both ends of the internal spring, and the external cylinder. Full description of the EDR mechanical behavior and detailed diagrams of the device are given by Nims (1993).

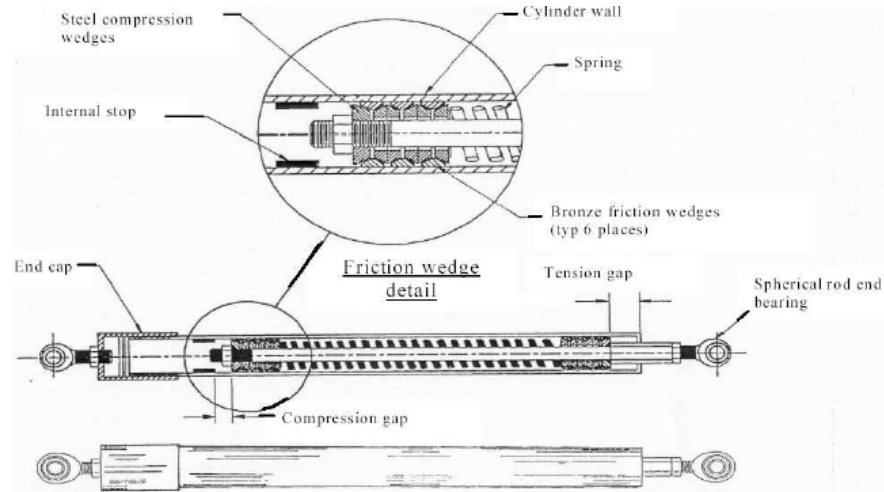


Figure 49. External and internal views of the EDR (Nims et al., 1993)

Characteristic features of the device are its self-centering capability and the developed frictional force proportional to the displacement. In operation, the compressive force in the spring acting on the compression and friction wedges causes a normal force on the cylinder wall. This normal force is proportional to the force in the spring. The normal force and the coefficient of friction between the bronze friction wedges and the steel cylinder wall determine the slip force in the device. In the EDR, two types of behavior are combined: linear stiffness and friction. Different combinations in between are possible, developing several different types of hysteresis loops, in dependence on the spring constant of the core, the initial slip load, the configuration of the core, and the gap size. Two typical hysteresis loops for different adjustments of the device are shown in fig. 50. The most interesting behavior consists in possessing fat “S-shaped” loops and self-centering properties. The friction force dissipating the energy is proportional to the displacement and the internal preload of the DR. The proportionality

between the dissipated energy and the displacement makes the EDR effective at low levels of seismic excitation or for wind loads while also being effective at high seismic levels.

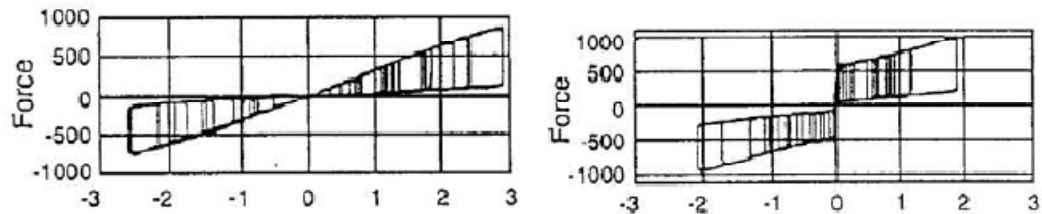


Figure 50. Hysteresis loop shapes (lb-in units, Richter et al., 1990) for EDR tested with different adjustments; left: no gap, no preload; right: no gap, some preload

Self-centering damper typologies: the friction spring seismic damper

The SHAPIA seismic damper, also known as Friction Spring Damper, uses a ring spring to dissipate earthquake-induced energy (Kar and Rainer 1995, 1996; Kar et al. 1996). A section through a typical ring spring assembly (fig. 51) consists of outer and inner rings with tapered mating surfaces. As the spring column is loaded in compression, the axial displacement is accompanied by sliding of the rings on the conical friction surfaces: the outer rings are subjected to circumferential tension (hoop stress), and the inner rings experience compression.

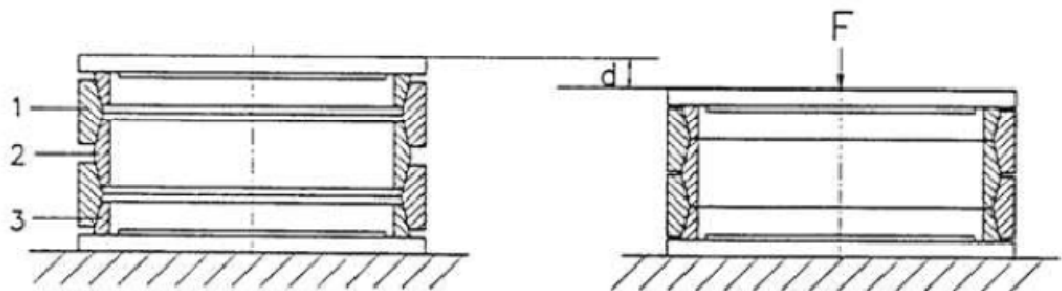


Figure 51. Friction spring details; 1) outer ring; 2) inner ring; 3) inner half ring

At the time of assembly and fabrication of the damper, special lubricant is applied at the tapered surfaces and at the external surfaces of the ring stack as a unique treatment for lifelong operation. The damper housing is virtually hermetically sealed to prevent any access of contaminants and to preserve and protect the lubricant. Some pre-compression by means of a centrally located tie-bar may be needed to align the rings axially as a column stack. The fabrication and assembly details are designed to ensure that the friction springs themselves are always in axial compression whenever the damper unit is subjected to either tension or compression. Fig. 52 shows a diagrammatic view of the prototype unit. The spring stack is retained at its ends by the flanges of a pair of cups. The damper carries no external load.

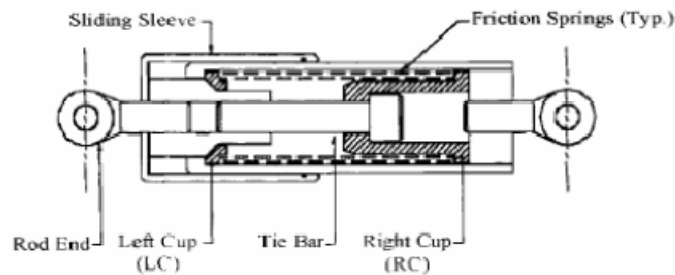


Figure 52. Diagrammatic view of seismic damper

During the process of loading and unloading, it offers spring effects together with damping. It is also strongly self-centering, provided that no plastic deformations have occurred: the ring springs are designed to remain elastic during a seismic event so that no repair or replacement should be required, and the structure should thus be protected against aftershocks and future earthquakes. This friction-based seismic damper is designed to display a symmetrical flag-shaped hysteresis diagram stable and repeatable

(fig. 53). Five different physical parameters define the hysteretic behavior of the SHAPIA damper: an elastic stiffness K_0 , a loading slip stiffness $r_L K_0$, an unloading slip stiffness $r_U K_0$, a slip force F_s , and a residual re-centering force F_c . The maximum forces reached upon loading $F_{\max L}$ and unloading $F_{\max U}$ are also shown.

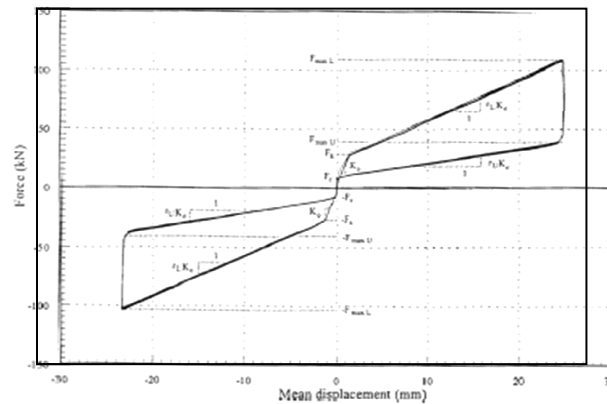


Figure 53. Experimental force-displacement hysteresis loops of seismic damper

Self-centering damper typologies: the post-tensioned energy dissipating steel connections

Moment-resisting connections using post-tensioning concepts were developed for precast concrete construction and recently extended to steel Moment Resisting Frames: Christopoulos et al. (2002) demonstrated that the performance of these connections is excellent under simulated seismic loading, due to their capacity of ensuring small residual drifts through self-centring capabilities, even when significant transient inelastic deformations occurred during the seismic response.

The post-tensioned energy dissipating (PTED) connection incorporates high strength steel post-tensioned bars designed to remain elastic, and confined energy-dissipating bars designed to yield both in tension and in compression. Fig. 54 illustrates the implementation of the PTED steel connection on a steel frame, together with its deformed configuration. Fig. 54 shows a moment-rotation relationship obtained experimentally from a large scale PTED connection (Christopoulos et al., 2002), in which the self-centering capacity and energy dissipation characteristics are evident.

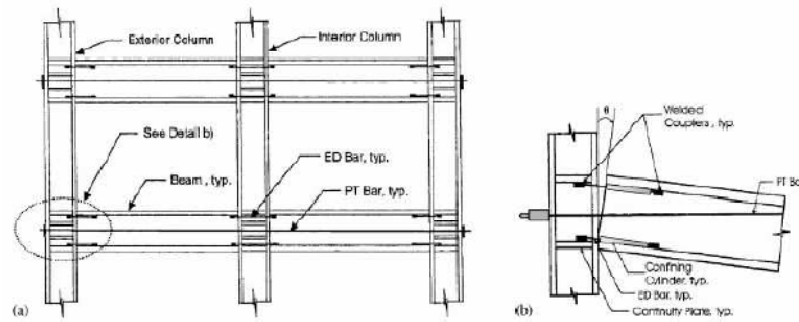


Figure 54. PTED steel connection: (a) steel frame with PTED connections; (b) deformed configuration of exterior PTED connection

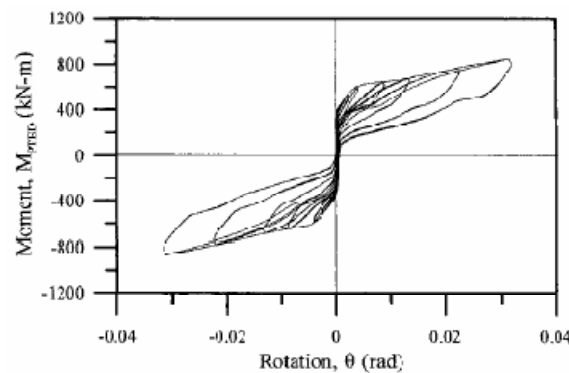


Figure 55. Experimental moment-rotation curve of PTED connection

Fig. 55 shows an idealization of the flag-shaped hysteretic behavior of a PTED connection: the overall response of the connection can be decomposed into the non-linear elastic contribution from the PT bars and the bilinear elasto-plastic hysteretic contribution provided by the ED bars.

Other studies were conducted on external hysteretic devices (Chou et al., 2009; Restrepo et al., 2002; Pampanin et al., 2006). The test gives very good result considering that this type of device is easy to replace after a seismic event.

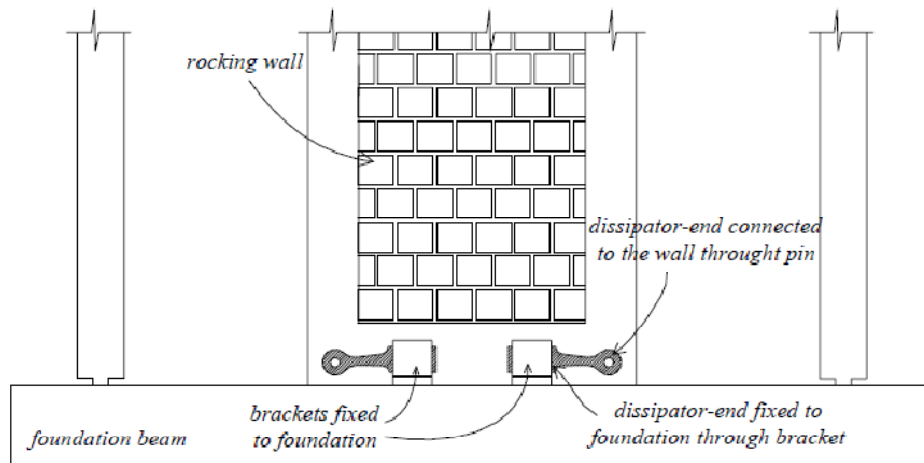


Figure 56. External hysteretic device on a shear-wall (Restrepo 2002)

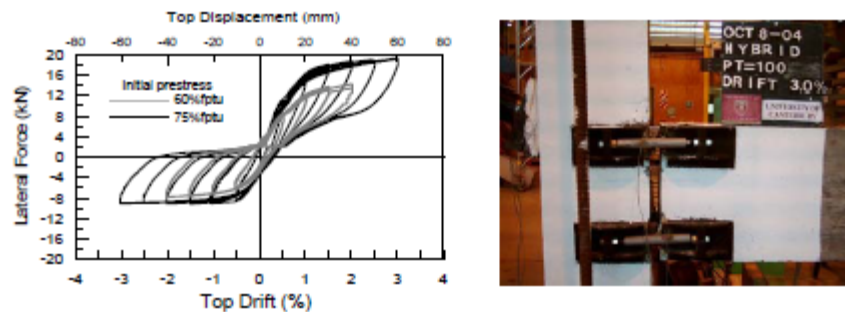


Figure 57. External hysteretic device on a beam column joint (Pampanin 2006)

IV. Viscous Fluid Dampers, Modelling and Calibration of a Damaged Device

4.1 Introduction

Metallic, friction and viscoelastic dampers all utilize the action of solids to enhance the performance of structures subjected to transient environmental disturbances.

However, as will be discussed in the present chapter, fluids can also be effectively employed in order to achieve the desired level of passive control. In fact, the concept of a fluid damper for general shock and vibration mitigation is well-known. One prominent example is, of course, the automotive shock absorber.

Significant effort has been directed in recent years toward the development of viscous fluid dampers for structural applications, primarily through the conversion of technology from the military and heavy industry.

One straightforward design approach is patterned directly after the classical dashpot. In this case, dissipation occurs via conversion of mechanical energy to heat as a piston deforms a thick, highly viscous substance, such as a silicon gel. Fig. 58a depicts a particular damper manufactured by GERB Vibration Control, which can be designed to provide vibration control in piping networks or for use as components in seismic base

isolation systems. As indicated in the diagram, ribs and other geometric details are sometimes included in the piston design to enhance performance.

While these devices could also be deployed within the superstructure, an alternative, and perhaps more effective, design concept involves the development of the viscous damping wall (VDW) illustrated in fig. 58b.

In this design, developed by Sumitomo Construction Company, the piston is simply a steel plate constrained to move in its plane within a narrow rectangular steel container filled with a viscous fluid. For typical installation in a frame bay, the piston is attached to the upper floor, while the container is fixed to the lower floor. Relative interstory motion shears the fluid and thus provides energy dissipation. By incorporating a sufficient number of VDW panels within the structural frame, a significant increase in the damping level can be achieved (Miyazaki and Mitsusaka, 1992).

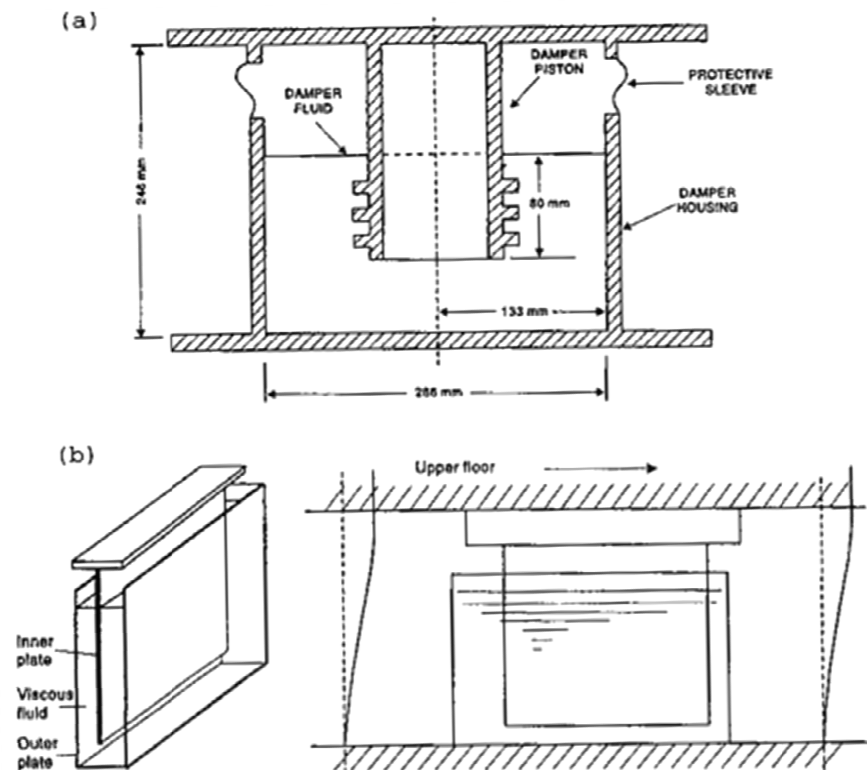


Figure 5t

Figure 5t: (a) Cross-section of a damper, (b) Schematic of a viscous Damping Wall

Both of the devices discussed above accomplish their objectives through the deformation of a viscous fluid residing in an open container. In order to maximize the energy dissipation density of these devices, one must employ materials with large viscosities. Typically, this leads to the selection of materials that exhibit both frequency and temperature dependent behavior.

There is, however, another class of fluid dampers that rely instead upon the flow of fluids within a closed container. In these designs, the piston acts now, not simply to deform the fluid locally, but rather, to force the fluid to pass through small orifices. As a result, extremely high levels of energy dissipation density are possible. However, a

correspondingly high level of sophistication is required for proper internal design of the dampers unit.

A typical Taylor Devices fluid damper for seismic application is illustrated in fig 59a. This cylindrical device contains a compressible silicone oil which is forced to flow via the action of a stainless steel piston rod with a bronze head. The head a state-of-the-art fluidic control orifice design with a passive bimetallic thermostat to compensate for the change in volume due to rod positioning. High strength seals are required to maintain closure.

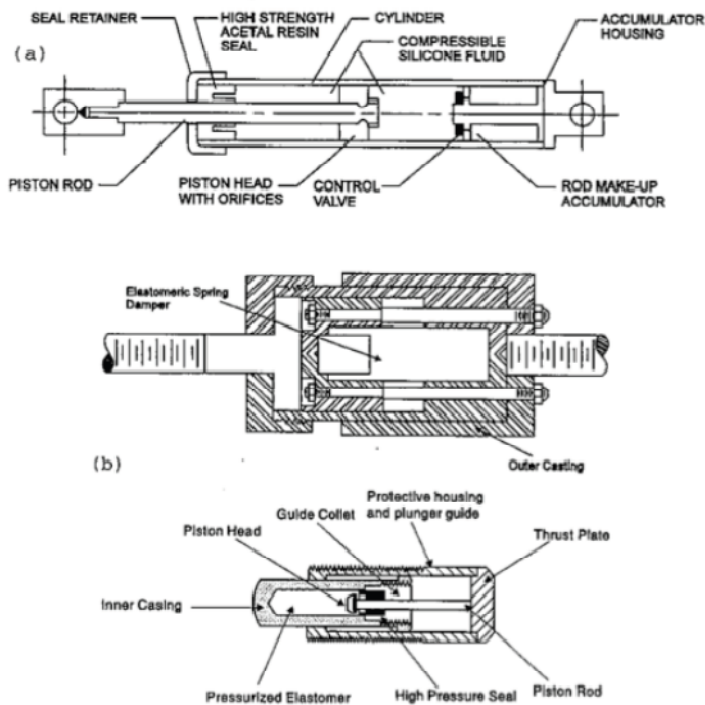


Figure 59. Viscous Liquid Dampers; a) Taylor Fluid Damper, b) Jarret Elastomeric Spring Damper (Constantinou and Symans, 1992)

These uniaxial devices, which were originally developed for military and harsh industrial environments, have recently found application in seismic base isolation systems as well as for supplemental damping during seismic and wind-induced vibration.

Another fluid damper featuring orifice flow to achieve energy dissipation is shown in fig. 59b. This device, manufactured by Jarret, utilizes a pressurized compressible silicone-based elastomer to provide additional structural stiffness and damping. Recent scale model experiments suggest that the device is suitable for application in aseismic design.

Although the four dampers illustrated in figs 58 and 59 all depend upon the flow of fluids to achieve energy dissipation, each device is distinct in terms of geometric design and material performance. In order to obtain a better understanding of their anticipated behavior within a structural system, one can examine the internal flow fields from a continuum fluids dynamics viewpoint.

4.2 Elements of a typical viscous damper

The following elements form a typical viscous damper as shown in fig. 59a.

Piston rod

The piston rod is machined from high alloy steel stainless steel and then highly polished. This high polish provides long life for the seal. The piston rod is designed for rigidity as it must resist compression buckling and must not flex under load, which would injure the seal.

Fluid

Structural applications require a fluid that is fire-resistant, nontoxic, thermally stable and that will not degrade with age. Under current OSHA (Occupational Safety & Health) guidelines this means a flash point of at least 200°F. Silicone fluid is often used as it has a flash point over 650°F and is cosmetically inert, completely nontoxic and one of the most thermally stable fluids available.

Seal

The seal must provide a service life of at least 35 years without replacement. As dampers often sit for long periods without use, the seal must not exhibit long-term sticking or allow fluid seepage. The dynamic seal is made from high-strength structural polymer to eliminate sticking or compression set during long periods of inactivity. Acceptable materials include Teflon, stabilized nylon and members of the acetyl resin family. Dynamic seals made from structural polymers do not age, degrade or cold flow over time.

Piston head

The piston head attaches to the piston rod and effectively divides the cylinder into two separate pressure chambers. This space between the outside diameter of the piston and in the inside diameter of the cylinder forms the orifice. Very often the piston head is made from a different material than the cylinder to provide thermal compensation. As the temperature rises the annulus between the piston head and the cylinder shrinks to compensate for thinning of the fluid.

Accumulator

The damper shown in Figure 59a uses an internal accumulator to make up for the change in volume as the rod strokes. This accumulator is either a block of closed-cell plastic foam or a movable pressurized piston, or a rubber bladder. The accumulator also accommodates thermal expansion of the silicone fluid.

4.3 Analytical Modeling

4.3.1 Maxwell Model

As the name indicates, visco-elasticity is a generalization of elasticity and viscosity. The elastic element could be modeled as a spring and the damper as a fluid-dynamic piston. If these two elements are combined together in series we have the Maxwell model like in fig. 60 , where K is the elastic constant of the spring and C is the viscosity. When a force F is applied to the model, the total elongation is equal to the sum of the elongation of the elastic and viscous element.

$$u = u_e + u_v$$

where u_e is the elongation of the element spring and u_v is the one of the viscous element.

Derivating the former expression by the time, we get as a result:

$$\frac{du}{dt} = \frac{du_e}{dt} + \frac{du_v}{dt}$$

Because the force F is the same in the two elements, applying the constitutive equations (keeping in mind the derivatives) we have:

$$\frac{du}{dt} = \frac{1}{K} \frac{dF}{dt} + \frac{F}{C}$$

The response u , therefore in Maxwell model depends just on the force F .

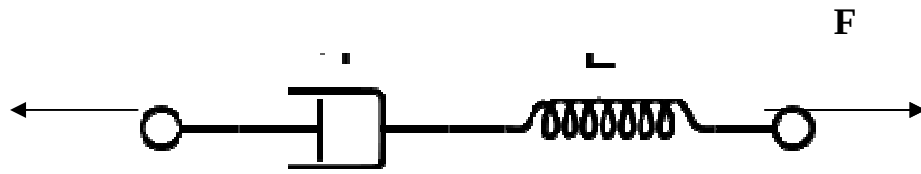


Figure 60. Schematic representation Maxwell model

4.3.2 Voigt Model

If the two element are combined in parallel, the resultant model is known as Voigt model as in fig. 61 . When a force is applied to this model, the sum of the forces in the spring and in the piston has to be equal to the one the one that has been applied, that is:

where F_e is the force in the spring and F_v is the force in the piston. If the displacements in the two elements are equal we will have the following relation between force and displacements:

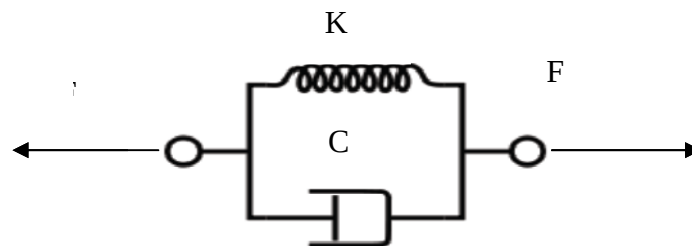


Figure 61. Schematic representation of Voigt model

4.4 Model used by Caltrans in Adina

The model used by Caltrans in the 3D finite element model of Vincent Thomas Bridge to describe the behavior of the dampers is a combination of both Voigt and Maxwell models; it consists of an elastic beam in series with a non-linear spring made by a dashpot in parallel with a non-linear spring. The non-linearity of the spring is given by the possibility of plugging in a non-linear Force-Relative displacement law for the spring, and an α coefficient for the dashpot which can differ from one.

However, from the analyses it has turned out that the elastic beam is basically non influent in the damper's behavior, providing its stiffness ($K = 5000$ Kip/in and its shortness 1"), much bigger than the other element in series (the non linear spring).

It's more likely that it has been used to make the model more stable. The use of this coupled model will be necessary once modeling the gap (to simulate a damage). The gap will be localized in the Maxwell element as a non linear spring.

All the dampers have the same configuration. They just differ because of the viscous coefficient C 's value.

The task that follows consists in comparing the results gotten in the SRMD laboratory in terms of reaction forces with the one gotten by finite element analyses with ADINA.

4.5 Model used in this work

4.5.1 *Experimental tests on damaged dampers*

After research conducted on the damaged/degraded damper behavior (Benzoni et al., 2008) at the Caltrans SRMD Testing Laboratory of the University of California San Diego, appears that a possible leak of the fluid in a damper may vary the response of the device. One of the main objectives of that research program was the characterization of the performance of viscous dampers under different conditions of possible degradation of their basic response characteristics. As described in the previous paragraph, the F.E. model allows to introduce changes in the two main damper performance parameters C and α . However, no other information are available in literature about the experimental response of viscous dampers in damaged/degraded configurations that can provide physical significance to the artificial variations of the two parameters C and α . For this reason an experimental phase was completed to study the response of one viscous damper in conditions that can be realistically representing damaged stages of increasing severity. It is clear that the experimental program, limited to a single device unit, cannot be considered inclusive of the subject but it represents an important set of data for several reasons. It allows, in fact, the validation of a numerical model of the damper performance that can be used in F.E. simulations to analyze the impact of damper defects to the overall structural performance. In these terms the data are critical for this research because they allow to create realistic variations of damper behavior that are targets of the health monitoring procedure. In addition, these experimental results

represent a first data sample for the process of definition of maps of deterioration at the device level. The use of deterioration maps (schematically shown in Figure 62) in concert with time maps (shown on the right) are of paramount importance for a predictive approach using which estimates can be made of remaining life of the structure affected by device performance degradation, as well as of an optimized schedule for inspection of the device conditions.

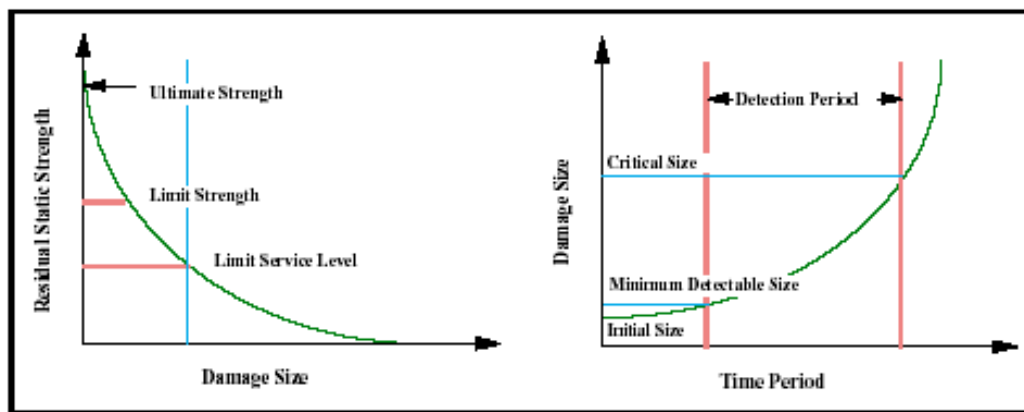


Figure 62. Example of 2D maps of deterioration

A viscous damper unit (Figure 63) was made available by F.I.P. Industriale (Italy) to complete this preliminary program. The different levels of degradation were artificially created to simulate leakage of damper, for instance associated to a damage of the damper seals. The experimental tests have been carried out for different level of leakage (0.4, 0.8, 1.2, 1.6 liters) and at different frequencies (0.01, 0.28, 0.56, 0.84, 1.11 Hz) of the imposed sinusoidal motion. Tests were completed at the Caltrans SRMD Testing Laboratory of the University of California San Diego.

The damper basic characteristics are:

Stroke: ± 275 mm

Design capacity: 670 kN

Damping Exponent: 0.15

Tests were performed imposing a sinusoidal motion of constant peak amplitude equal to 100 mm. Three main cycles were completed for each test. In order to minimize the inertia effects at the beginning and end of the test, entrance and exit ramps were introduced resulting in a total number of four cycles. In the data analysis process only the central three cycles were considered. In Table 3 peak forces and displacements are reported for the tests at different frequency and for different amount of oil removed.

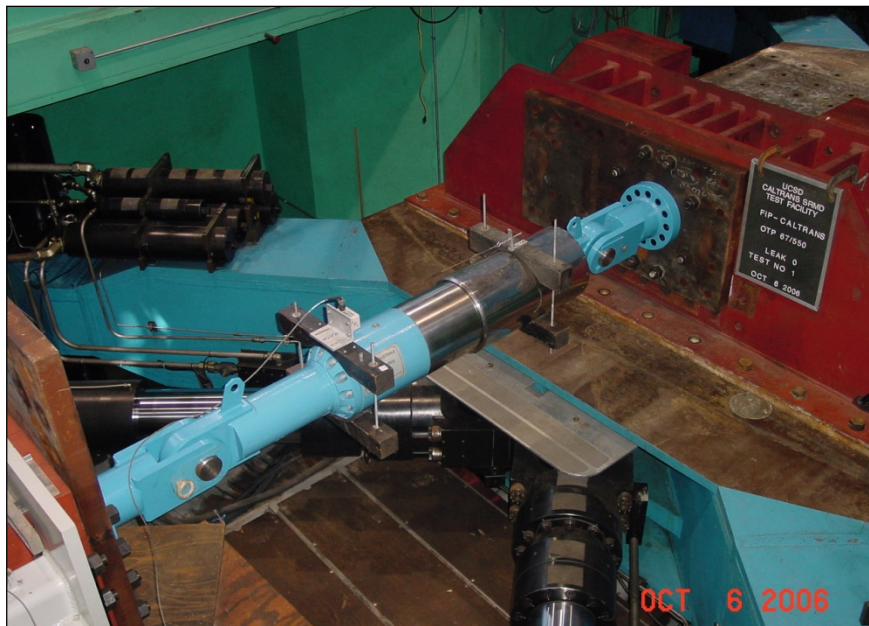


Figure 63. FIP Damper unit

Table 3. Dampers FIP: Test Summary

TEST	Frequency (Hz)	F_{max} (kN)	F_{min} (kN)	D_{max} (mm)	D_{min} (mm)
FIP Leakage 0 Test 1	0.01	290.1413	-273.0191	99.9595	-99.5549
FIP Leakage 0 Test 2	0.28	465.4184	-443.0362	99.2253	-100.0793
FIP Leakage 0 Test 3	0.56	520.5636	-497.1634	99.4500	-100.2292
FIP Leakage 0 Test 4	0.84	556.5353	-533.1351	100.0494	-101.8475
FIP Leakage 0 Test 5	1.11	592.8463	-568.7674	101.8025	-103.4208
FIP Leakage 04 Test 1	0.01	291.8380	-269.4559	99.3601	-100.0644
FIP Leakage 04 Test 2	0.28	463.3822	-432.6859	99.4650	-99.9894
FIP Leakage 04 Test 3	0.56	511.9101	-482.7408	100.0194	-100.3191
FIP Leakage 04 Test 4	0.84	545.5062	-514.9796	100.7087	-101.7126
FIP Leakage 04 Test 5	1.11	583.3444	-557.0597	101.7426	-103.4358
FIP Leakage 08 Test 1	0.01	290.3109	-270.8133	99.2852	-100.1992
FIP Leakage 08 Test 2	0.28	461.3461	-433.5343	99.2552	-100.0794
FIP Leakage 08 Test 3	0.56	509.1952	-480.7047	100.0044	-100.1992
FIP Leakage 08 Test 4	0.84	543.4701	-514.6402	100.9035	-101.6377
FIP Leakage 08 Test 5	1.11	585.8895	-559.6048	102.0722	-103.2410
FIP Leakage 1.2 Test 11	0.01	287.5961	-265.2140	100.5738	-100.3790
FIP Leakage 1.2 Test 12	0.01	288.6142	-271.4920	99.4650	-99.7347
FIP Leakage 1.2 Test 2	0.28	460.1583	-434.8917	98.9705	-100.1243
FIP Leakage 1.2 Test 3	0.56	511.0617	-484.4376	100.409	-100.4390
FIP Leakage 1.2 Test 4	0.84	541.0946	-517.0157	100.8435	-101.6077
FIP Leakage 1.2 Test 5	1.11	580.7992	-552.9874	102.1921	-103.3759
FIP Leakage 1.6 Test 1	0.01	288.6142	-269.2862	99.5699	-99.6598
FIP Leakage 1.6 Test 2	0.28	460.1583	-432.6859	99.1803	-100.1543
FIP Leakage 1.6 Test 3	0.56	512.5888	-483.4195	100.3041	-100.6338
FIP Leakage 1.6 Test 4	0.84	542.4520	-518.0338	100.6787	-101.4579
FIP Leakage 1.6 Test 5	1.11	576.5573	-552.1390	101.6527	-103.3759

A sample of performance results are shown in Figure 64, 65 and 66, for the condition of 0.8 liters of fluid removed from the damper and an excitation of 0.28 Hz.

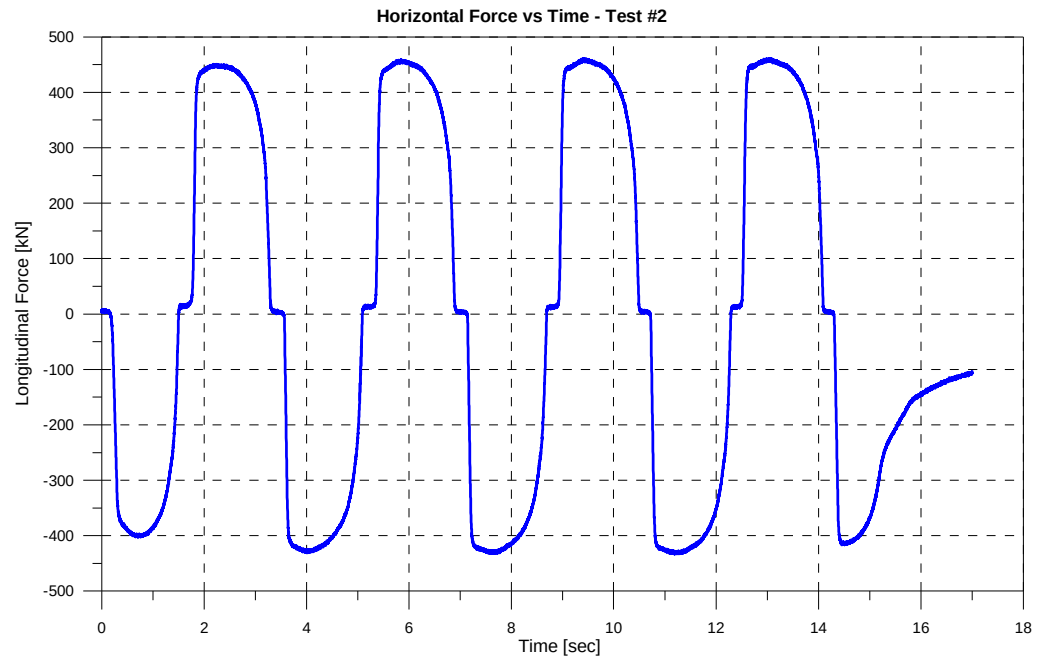


Figure 64. Test #2 Leakage = 0.8 liters: Longitudinal force (Benzoni et al.,2008)

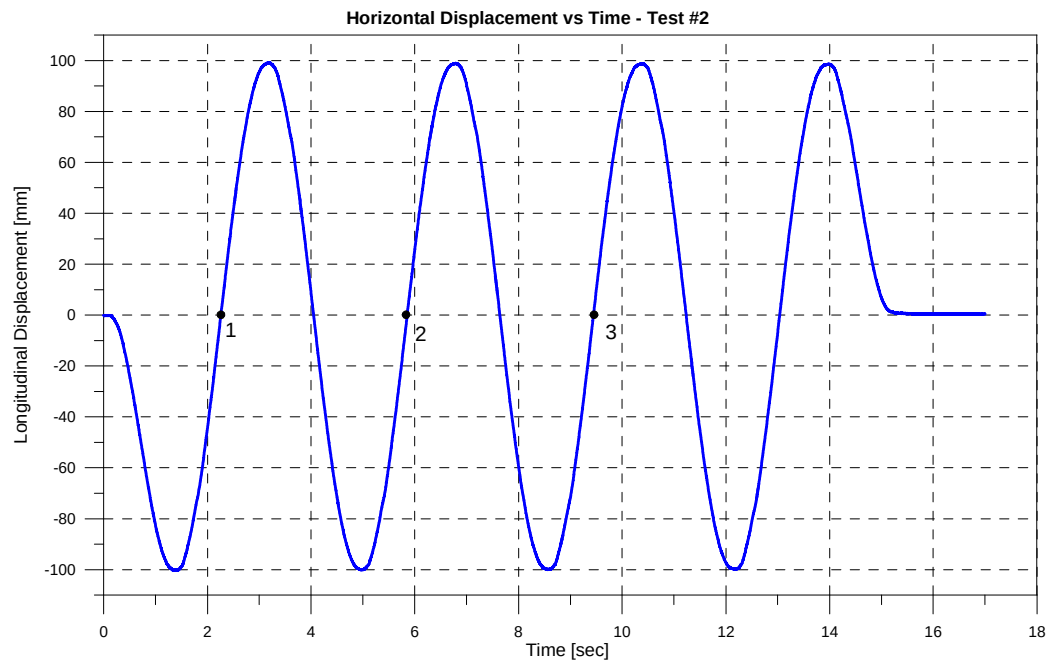


Figure 65. Test #2 Leakage = 0.8 liters: Longitudinal displacement (Benzoni et al.,2008)

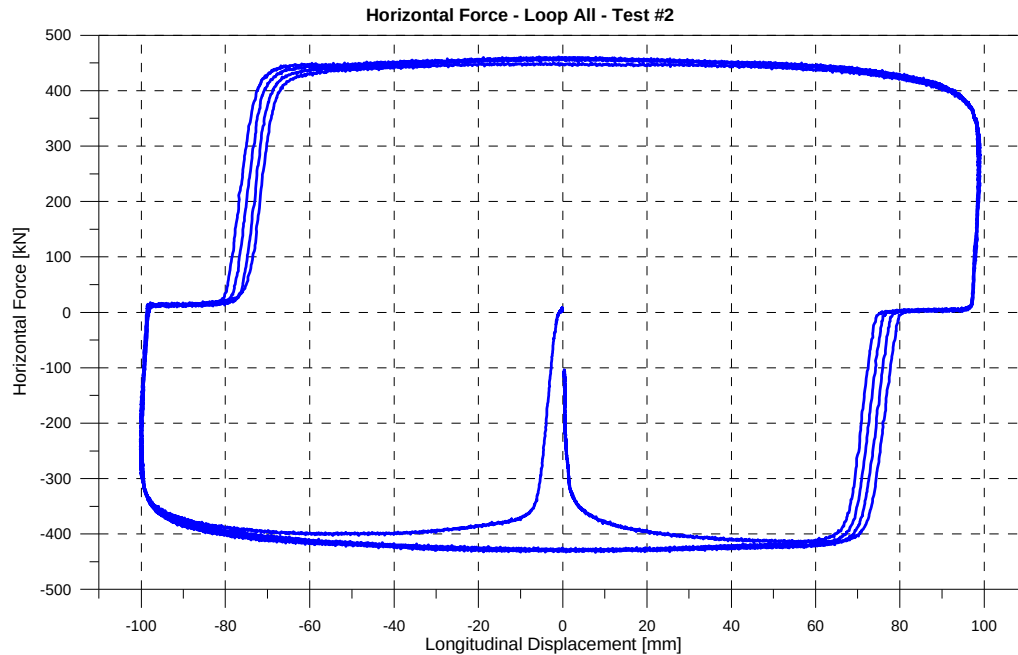


Figure 66. Test #2 Leakage = 0.8 liters: Force vs displacement (Benzoni et al., 2008)

The effect of the removal of a portion of the viscous material is clearly visible in Figure 66 at displacement reversal. The damper is able to describe a portion of the assign displacement time history without developing any reaction force, describing the flat portions of the force-displacement loop. When the displacement of the internal piston is sufficient to generate again a level of pressure in the fluid, the force trend is restored. The length of the “zero force” segments depends on the amount of fluid removed.

The performance of the damper, in its original conditions, is accurately modeled by the Maxwell constitutive law, characterized by a linear spring in series to a non-linear dash-pot element (Infanti et al, 2002). The first element represents the elasticity of the

system mainly due to the compressibility of the silicon fluid, while the second elements accounts for the damping properties. The constitutive equation is:

$$F = C \left(\dot{x} - \frac{Kx}{K} \right)^\alpha \quad (1)$$

where x represent the displacement, K the elastic stiffness of the spring and α the exponent that characterize the non-linearity of the response.

In order to simulate the performance of the damper in damaged conditions, consistently with the response of Figure 66, a model was obtained with a dash-pot element in series with a gap element (Figure 67).

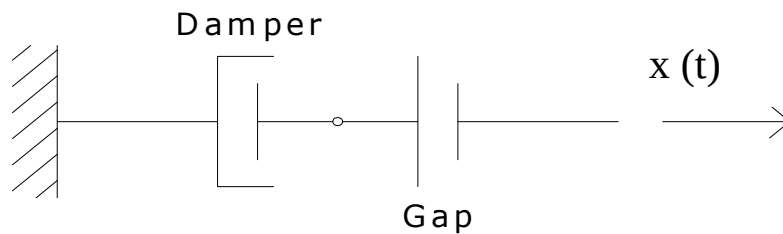


Figure 67. Model of Damper with leakage

The force-velocity relationship can be written:

$$F = C \dot{x} + K(x - d_g) \quad (2)$$

Where:

K = gap stiffness;

x = displacement;

d_g = length of gap.

The second part of the equation is equal to zero when $x < d_g$. Figure 68 shows the contribution of the two components (the dash-pot on the left and the gap element on the right) to the force-displacement response.

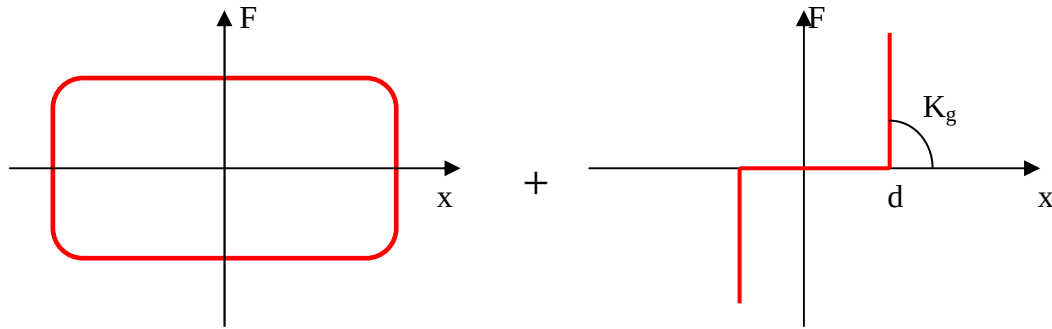


Figure 68. Model of the damaged damper

The variation of the gap length as a function of the leakage for all the tests is reported in Figure 69. In general the lengths variation is linear with the increment of leakage.

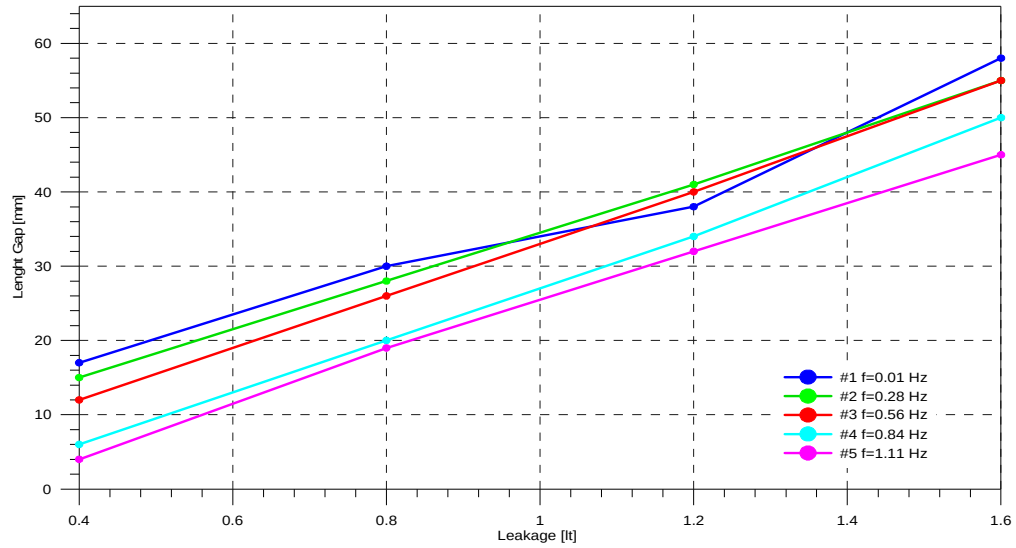


Figure 69. Variation of Length Gap with leakage for the different test

The model results are presented in Figure 70 to Figure 73. The experimental results and the model performance for Test #5, a different level of leakage are compared.

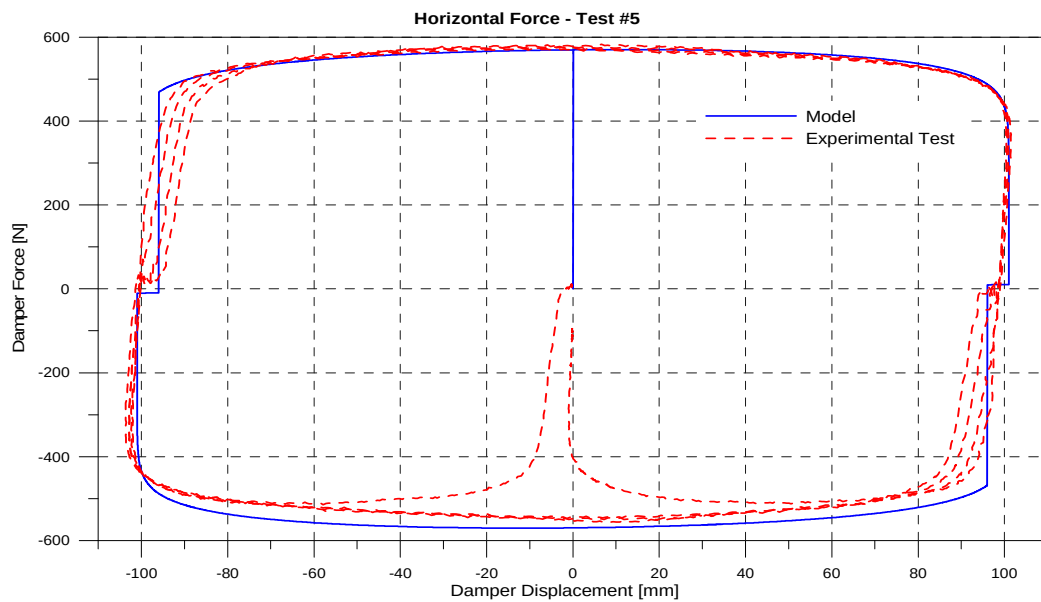


Figure 70. Test#5 Horizontal Force Leakage 0.4 liters: comparison between the theoretical model and test

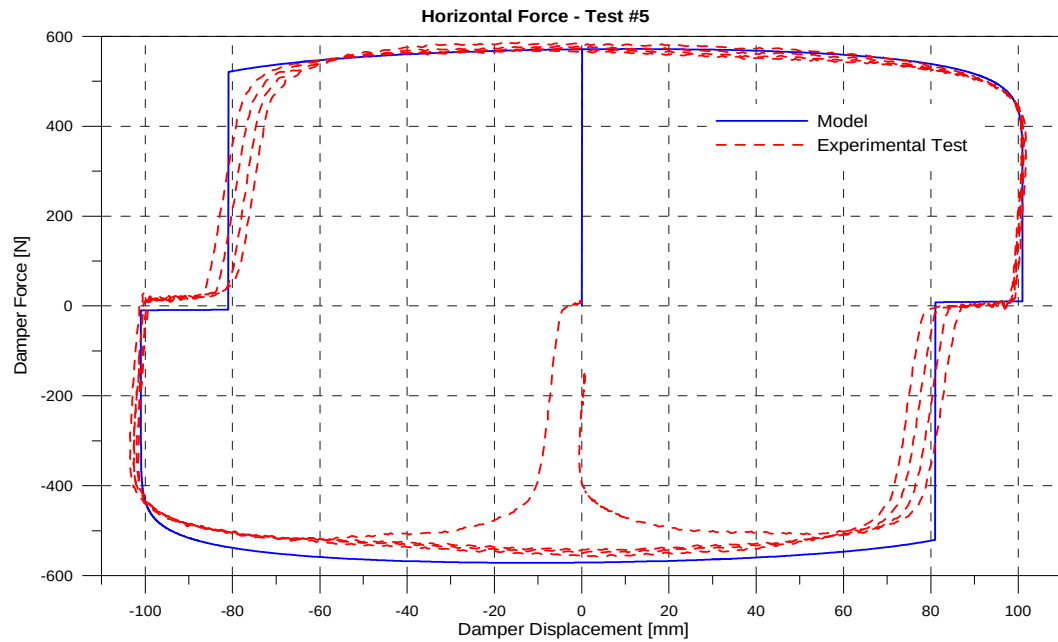


Figure 71. Test#5 Horizontal Force Leakage 0.8 liters: comparison between the model theoretical and test

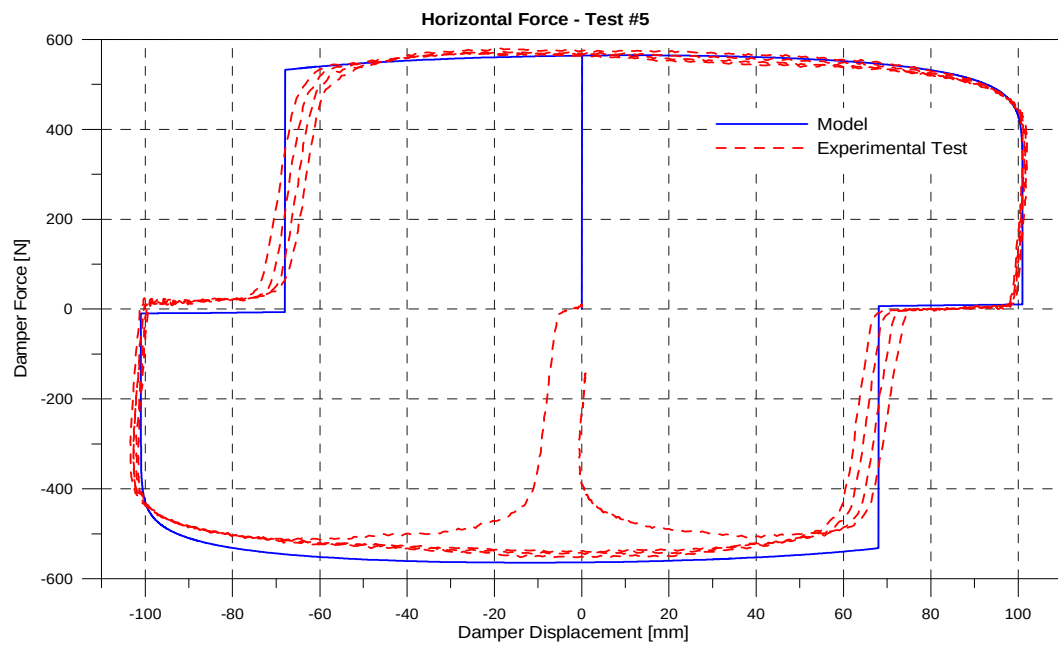


Figure 72. Test#5 Horizontal Force Leakage 1.2 liters: comparison between the theoretical model and test

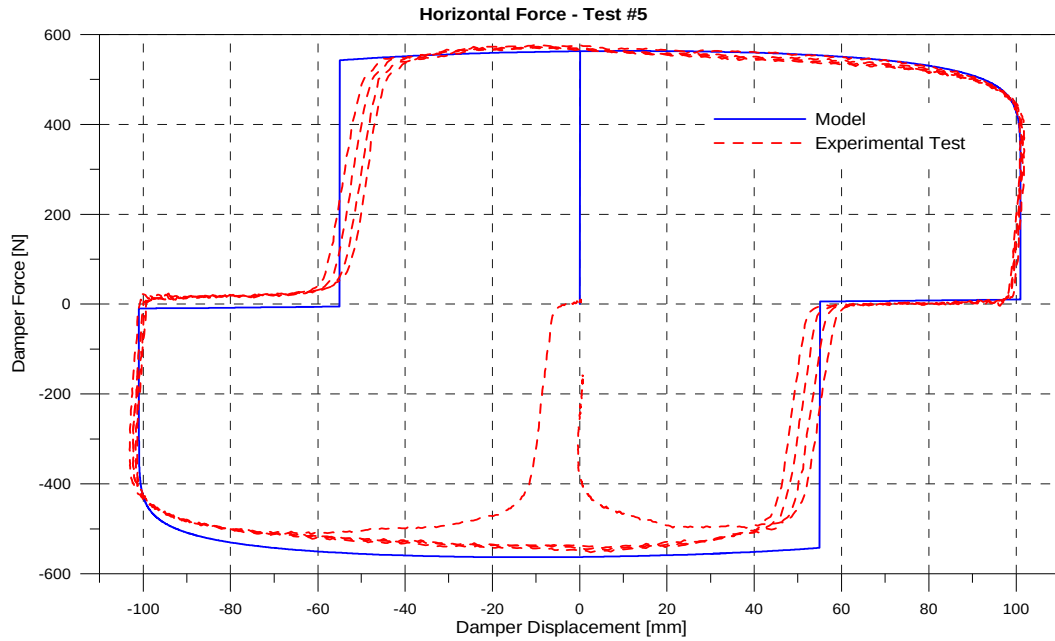


Figure 73. Test#5 Horizontal Force Leakage 1.6 liters: comparison between the theoretical model and test

In previous research (Benzoni et al., 2008) the attention was focused to the most representative parameter of damper response, the energy dissipated per cycle (*EDC*). The energy dissipated for every cycle was calculated for all the tests using the following equation:

$$EDC = \int F_d(x(t)) dx \quad (3)$$

The obtained *EDC* values are reported in Table 4.

Table 4. Energy dissipated per cycle

TEST	EDC (kN-mm) Cycle 1	EDC (kN-mm) Cycle 2	EDC (kN-mm) Cycle 3
FIP Leakage 0 Test 1	103722	103781	103516
FIP Leakage 0 Test 2	168039	169621	170537
FIP Leakage 0 Test 3	188167	189887	190842
FIP Leakage 0 Test 4	205062	205406	206007
FIP Leakage 0 Test 5	219445	220109	219621
FIP Leakage 04 Test 1	96534	96834	97030
FIP Leakage 04 Test 2	155876	158640	160776
FIP Leakage 04 Test 3	175892	178785	180896
FIP Leakage 04 Test 4	191964	195029	197433
FIP Leakage 04 Test 5	207885	209132	210478
FIP Leakage 08 Test 1	90444	90686	90791
FIP Leakage 08 Test 2	145773	148315	150392
FIP Leakage 08 Test 3	163853	166487	168553
FIP Leakage 08 Test 4	179064	181216	183147
FIP Leakage 08 Test 5	192512	192483	193742
FIP Leakage 1.2 Test 11	86141	86483	86615
FIP Leakage 1.2 Test 12	83660	83587	83966
FIP Leakage 1.2 Test 2	134959	137098	138706
FIP Leakage 1.2 Test 3	152336	154010	155784
FIP Leakage 1.2 Test 4	165760	167487	169179
FIP Leakage 1.2 Test 5	179212	179549	180570
FIP Leakage 1.6 Test 1	75846	75966	76224
FIP Leakage 1.6 Test 2	122471	124387	125972
FIP Leakage 1.6 Test 3	138705	140478	141823
FIP Leakage 1.6 Test 4	151051	152125	153628
FIP Leakage 1.6 Test 5	162410	163131	164444

The percentage variation of *EDC*, obtained experimentally and from the numerical model, is compared in Figure 74 for the tests at 1.11 Hz and at different levels of leakage. Details about the other tests are presented in the SRMD report No. 2007/09 (Benzoni et. al, 2007). It is visible that the effect of an increased removal of viscous

fluid produces a linearly proportional effect on the damper response. The performance of the model is also sufficiently accurate to simulate the effects obtained by tests.

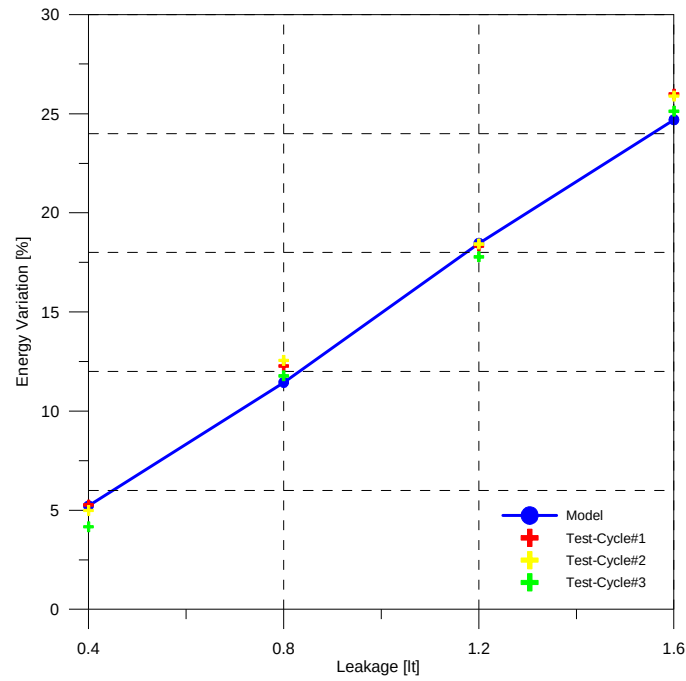


Figure 74. Test#5 ($f=1.11\text{Hz}$): Energy variation model and experimental data

4.5.2 Disassembly of a damaged damper

Figures 75 show different stages of the disassembly of the damper.



Figure 75. Disassembly of the Damper

As shown in figure 75 (3) fluid was removed from the damper at the appropriate location prior to removal of the piston head. This fluid was dark and was seen to have a high viscosity with visible agglomerates. Fluid collected after significant draining and collected from within the bearing area showed a sludge like consistency and had particles of bronze and steel in addition to the PTFE and graphite. Figure 76 shows photomicrographs of two samples of the fluid collected between glass slides. The size of particles/agglomerates can be easily seen in both micrographs and clearly shows the addition of foreign matter.

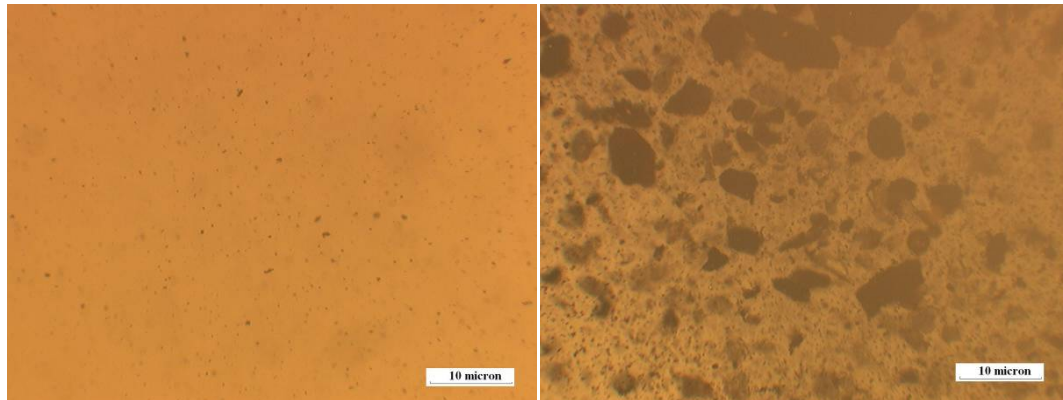


Figure 76. Photomicrographs of fluid

It should be noted that while PTFE is often used as an additive in hydraulic fluid to decrease friction and wear, research has shown that it can agglomerate fairly easily and the solid particles can accumulate at inlets acting as a blockage/dam which results in decrease in overall lubrication. This may well have happened over time in the piston area and in the bearing.

It should be noted that the O-ring from the port itself showed significant fraying and had a sticky consistency suggesting degradation of the material (Fig. 77). Microhardness tests showed changes of over 50% in areas.



Figure 77. Degraded O-Ring

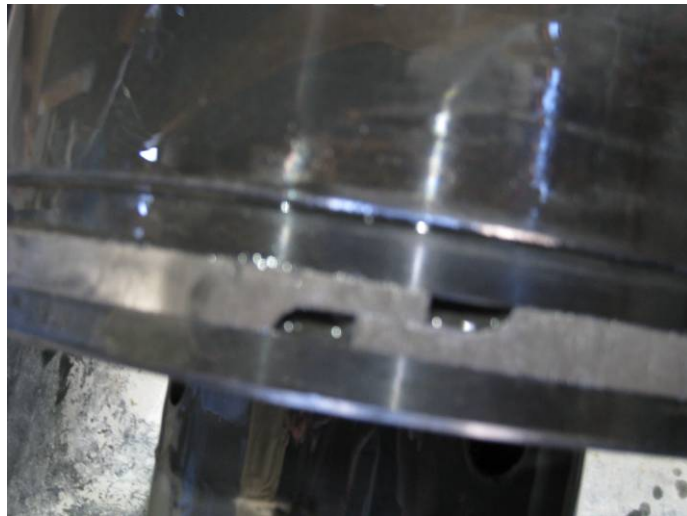


Figure 78. Gasket

The positioning of the outside teflon gasket is shown in fig. 78. There appeared to have been uneven compression on this with the width of the material ranging from 6.8 mm to 7.13 mm. Microhardness tests showed changes of 25-37% in some areas. Removal of the gasket showed the collected of sludge within the groove with small particles of teflon which may have been abraded through motion. A few bronze particles were also noted which was surprising since the bronze is actually only found within the bearing itself indicating movement of abraded particles out of the bearing retainer block.

It should be noted that the backup urethane O-ring was found to be frayed in areas and compressed to an oval cross-section in some areas. Thickness was also diminished by as much as 50% in some areas which indicated substantial abrasion.

The teflon back-up rings showed deterioration at both ends with substantial decrease in thickness (upto 60% in some areas coincident with the scratches in the bearing area itself (see fig. 79).



Figure 79. View of Inside of Bearing Showing Scratches From Abrasion as well as a Vertical Seam

The bronze seal backup ring seen in Figure 6 shows deterioration in the form of pitting abrasion at the left side of the figure. In addition the surface open to the cavity showed significant abrasion related loss of section in a circumferential arc of 127.5 mm length centered above the vertical seam seen in fig. 79. It is interesting to note from fig. 79 that the abrasion caused loss of section both vertical next to the seam and circumferentially about 1/3rd of the depth from the surface. Sludge was again removed from below each of the seals and seal retainers indicating leakage. In all cases the overall composition was similar to that of the sludge described earlier. It should also be noted that most of the damage was restricted throughout the bearing cavity to a circumferential arc of about 185 mm around the seam indicating greater contact between the rod and this area, either directly or through contact with abraded particles.

4.5.3 Creation of a model of damaged damper

In previous works regarding the behavior of the Vincent Thomas Bridge with damaged viscous dampers (Benzoni et al., 2005 and 2008), in order to implement the peculiar response of the damaged damper into the F.E. model of the bridge, the attention was focused to the most representative parameter of damper response, the energy dissipated per cycle (*EDC*), changing C as consequence.

In this work we modify the F.E. model of the bridge in order to simulate the entire behaviors of the degraded dampers. This allow to evaluate if the gap create some discontinuity or some undesired peaks in the internal force in the neighborhood of the damaged device.

In order to calibrate the damaged device an Adina F.E. model of damper was created. As written before the model consist in 2 elements: a nonlinear spring simulating the gap and a Voigt model spring simulating the undamaged damper. The only parameters that we changed in the model were the stiffness K_g and the viscosity C_g of the nonlinear spring simulating the gap. The theoretical wanted behavior as show in fig. 68 should have an infinite stiffness (infinite flexible in the gap, rigid elsewhere). This create peaks in the response not measured in test experiments. Fig. 80 shows this problem.

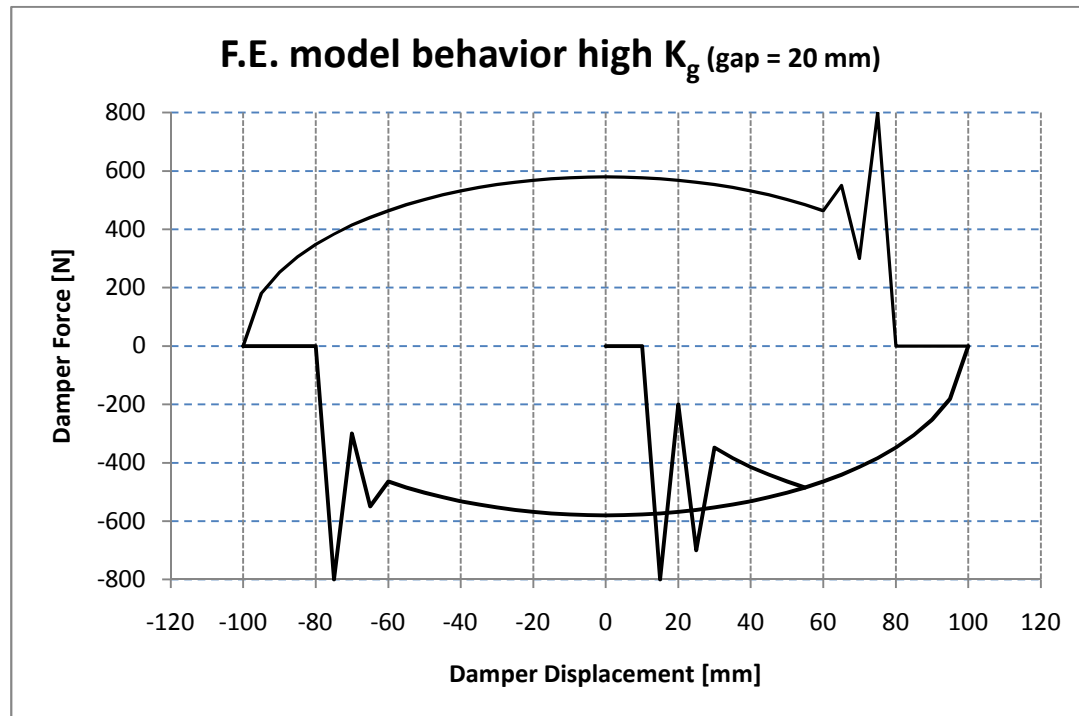


Figure 80. Horizontal Force gap 20mm: undesired peaks due to non-calibrated K_g

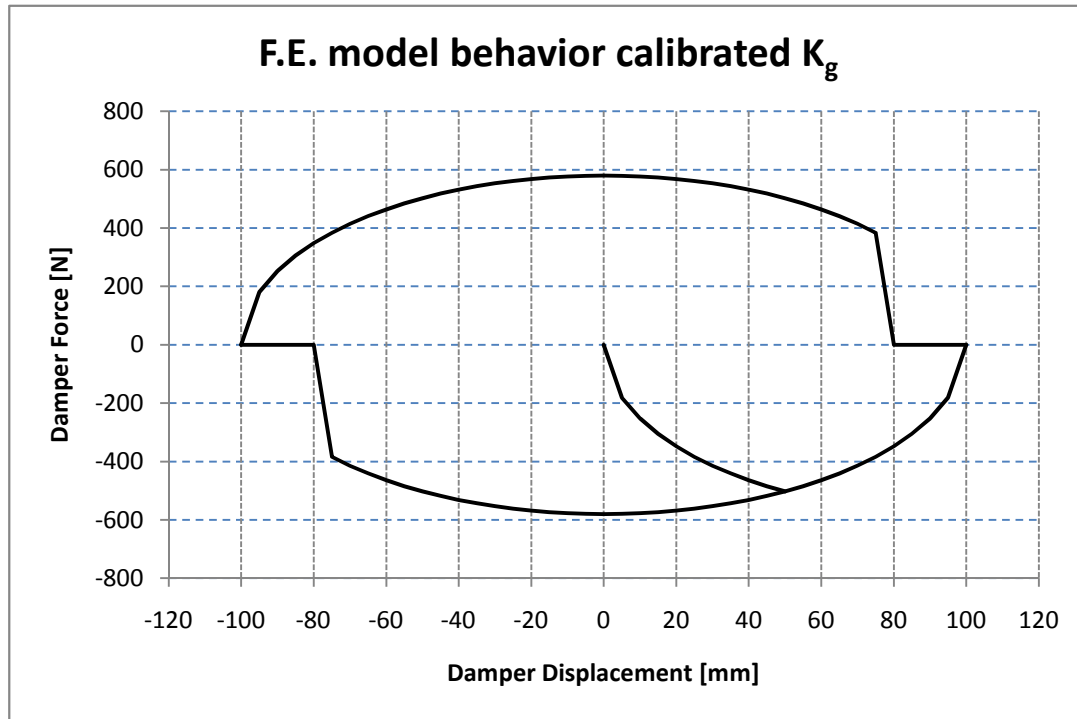
After several analysis a stiffness K_g factor was found for all the dampers that we want to modify in the F.E. model of the Vincent Thomas bridge. All the experimental tests was performed in displacement control (sine function $f = 0.2\text{Hz}$; $A = 100\text{ mm}$). The modeled damaged damper is able to capture with sufficient approximation the behavior of experimental tests, with all the different parameters of the various dampers.

It was decided to perform a parametric analysis considering possible leakages only in the tower damper devices. It was not considered a possible damage in the cables dampers. In particular, the calibrates value are summarized in next table:

Table 5. Dampers parameters with gap stiffness

	Total #	C [kips's/in]	α [-]	K_g [kips/in]
Main span to tower	8	4	1	16000
Side span to tower	8	2.5	1	10000

Fig. 81 shows as example the hysteretic loop of a calibrated damaged damper system. Those parameters are used in the complete F.E. model of the bridge to simulate the behavior of the damaged device on the entire structure.

**Figure 81.** Horizontal Force gap 20mm: calibrated K_g

V. Parametric study of the dampers damages influence on the bridge response

The calibrated models of the damaged dampers were used to perform a parametric study on the influence of the gap on the response of the entire structure. The damage was concentrated in the dampers linked to the towers. Different configurations have been considered: variation in gap length, symmetrical and nonsymmetrical damage distribution, different ground motion input.

Parameterization of the damage in dampers

As seen before the damage in dampers could be a gap in their force -deformation response (see ch. 3). Artificial gaps were inserted in the model in order to simulate these damages. The first parameterization has been conducted producing a progressive damage, equal to 25%, 50%, 75% and 100% of the maximum operative work length of the device. Maximum gap is considered equal to the maximum relative displacement of two extremes points of a particular damper. The structural response of a 100% damaged damper is so equal to the response of a non working damper (C and $K = 0$).

The damages was considered in two type of dampers. In particular the ones connected the tower with the main and side spans, called MAIN TO TOWER and SIDE TO TOWER dampers.

In the next figure is showed, as an example, the maximum relative displacement in main to tower damaged dampers during simulated Northridge earthquake. An analysis with a full damaged damper was conducted. Maximum elongation obtained is 4.6". Other calculated reference gaps are: 1.2" (25%), 2.3" (50%), 3.5" (75%).

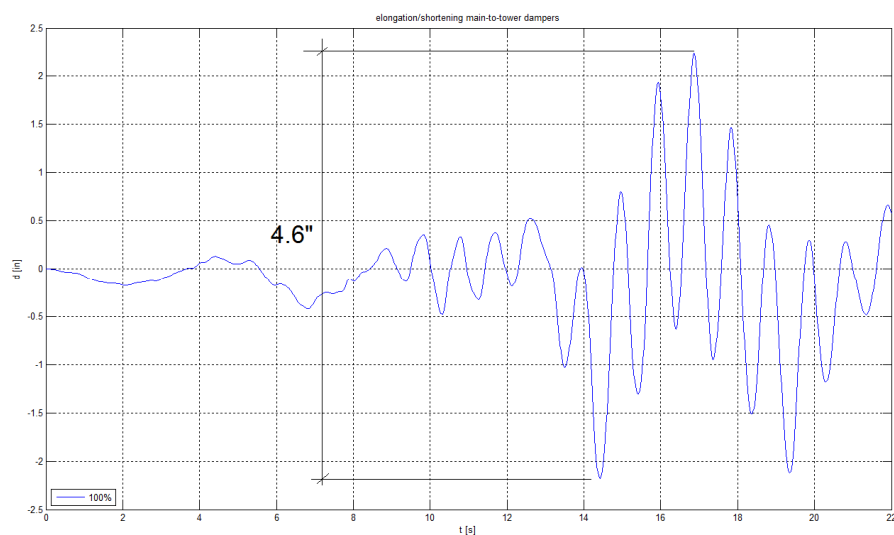


Figure 82. Working range of main to tower damper during Northridge earthquake

The same thing was calculated for side to tower. The maximum relative displacement is 2.2" and consequently calculated gaps are: 0.6" (25%), 1.1" (50%), 1.7" (75%).

The same procedure was conducted for all the analysis, and the damage parameterization selected as function of the non damaged working deformation of the dampers.

5.1 Ambient vibration analysis

5.1.1 Introduction

The ambient vibration analysis has been completed by a white noise input appropriately scaled to produce the same effects recorded by the seismic sensors installed on the bridge during normal traffic condition.

Right now, twenty-six seismic sensors are installed on the bridge to record ambient and seismic behavior. Fig. 78 shows the layout of the location of all 26 sensors allocated on the bridge. A summary of the sensor numbering system and measurement directions is presented in table 6. An enlarged view of sensor locations is presented in fig. 83.

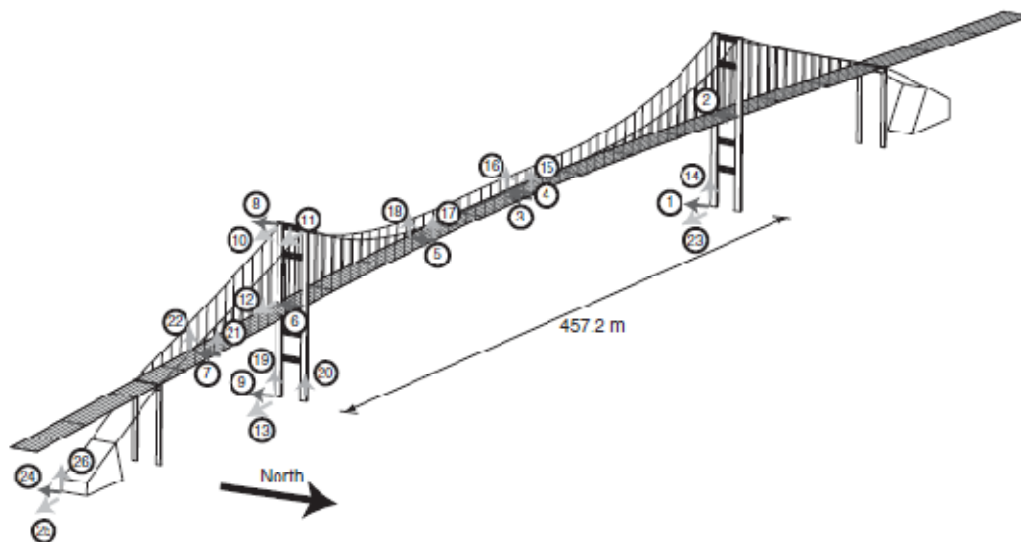
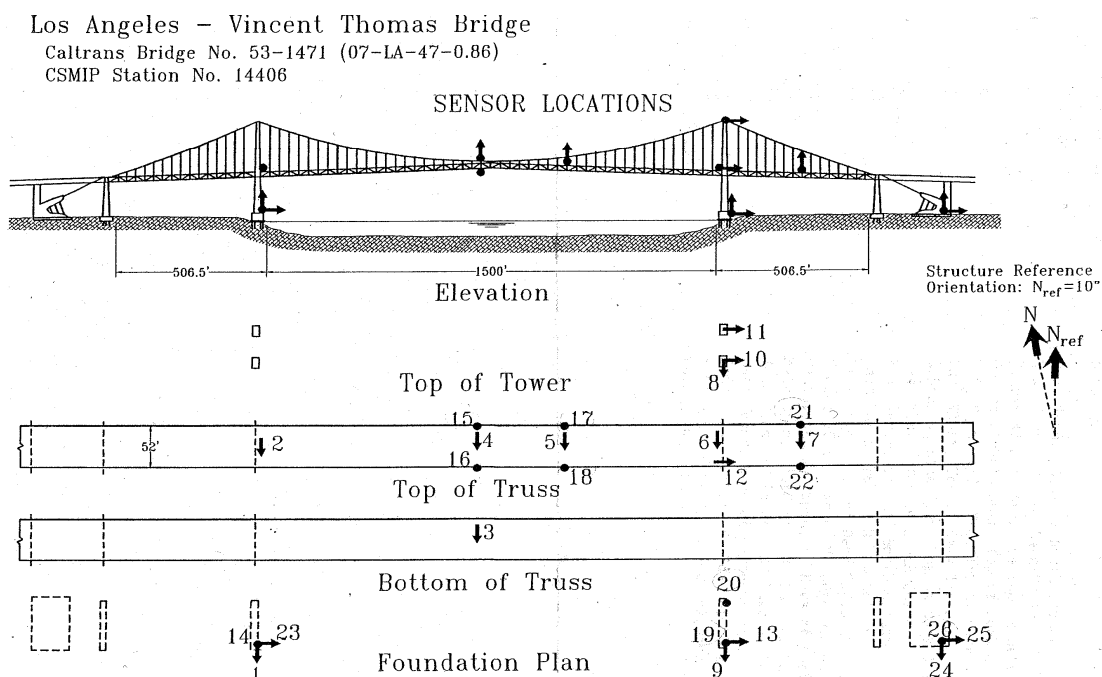


Figure 83. Accelerometer locations for the instrumental network (Smyth et al., 2003)

Table 6. Sensor localization (Smyth et al., 2003)

Location	Sensor	Sensor direction
Tower base	14, 19, 20	Vertical
	1, 9	lateral
	13, 23	longitudinal
Anchorage	26	vertical
	24	lateral
	25	longitudinal
Truss top, i.e. deck	15, 16, 17, 18, 21, 22	Vertical
	2, 4, 5, 6, 7	Lateral
	12	Longitudinal
Truss bottom	3	Lateral
Tower	8	Lateral
	10, 11	Longitudinal

**Figure 84.** Sensor localization (Smyth et al., 2003)

5.1.2 Description of the recorded Data

On April 18th, May 28th, June 1st of 2003 and December 3rd of 2006, four sets of acceleration data were collected to measure the response of the bridge due to ambient vibration. These data sets were collected by the instrumentation system on the bridge.

It has been chosen, like reference set, the data recorded on December 3rd of 2006. During that day the sensors monitored the following max displacements:

- 0.40 in, channel number 12;
- 1.50 in, channel number 4;
- 4.00 in, channel number 15.

That's measures represent the displacements relatively the x, y and z axis of the model.

A ground motion was calibrated to produce a similar displacement time history of the deck. The white noise input is shown in fig. 85.

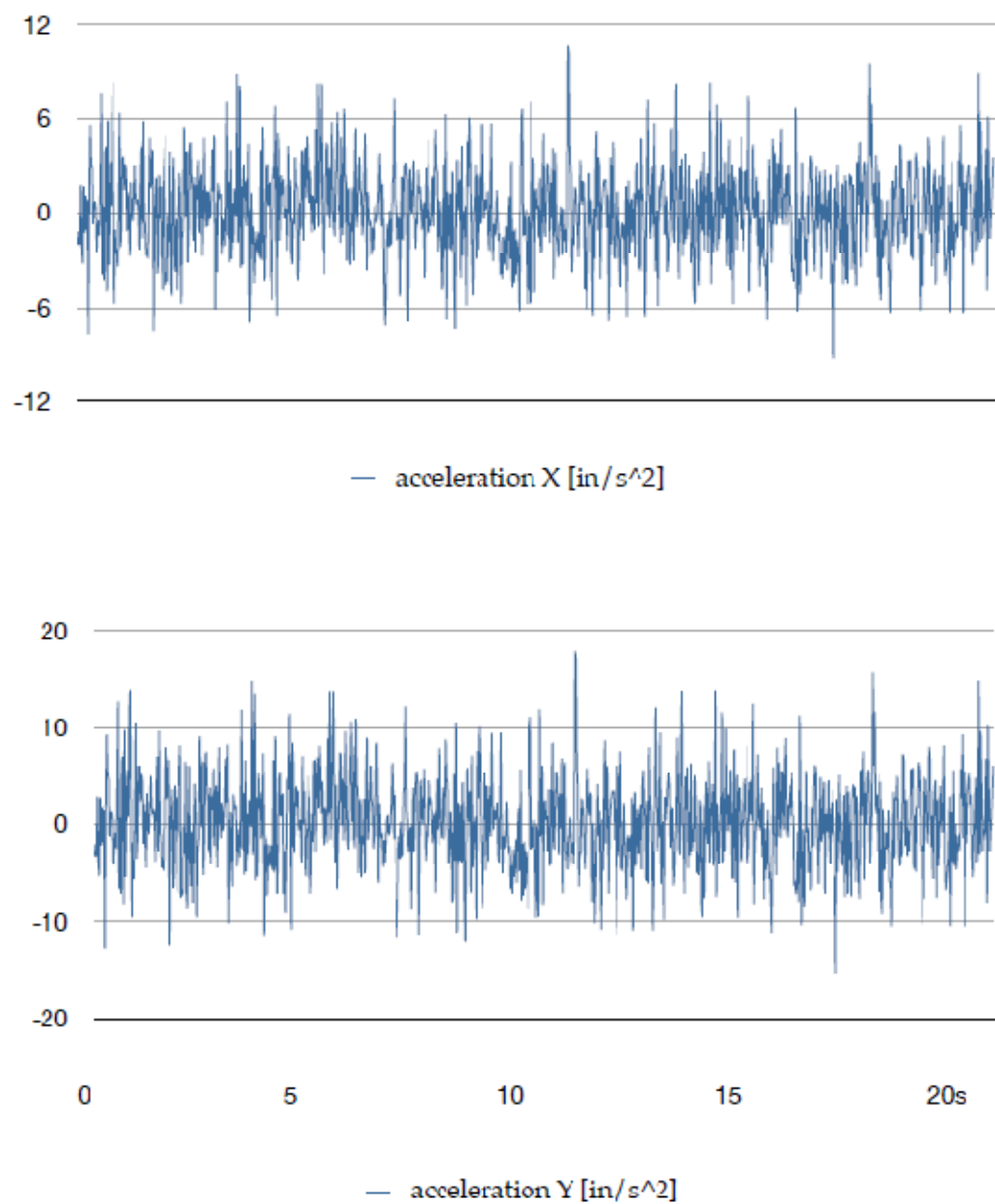


Figure 85. X,Y acceleration time history input for ambient vibration analysis

5.1.3 Parameterization and results of the analysis, Symmetric Damage

As seen before the damage in dampers has been modeled like a gap in their force-deformation response. Artificial gaps have been inserted in the model in order to simulate these damages. The first parameterization has been conducted producing a progressive damage, equal to 25%, 50%, 75% and 100% of the maximum operative gap. Maximum gap is considered equal to the maximum relative displacement of two extremes points of a particular damper. The structural response of a 100% damaged damper is so equal to the response of a not working damper of two type of damper which connected the tower with the main and side spans, called MAIN TO TOWER and SIDE TO TOWER dampers.

The maximum relative displacement in main to tower damaged dampers during simulated ambient vibration. Considering 0.98" as completely damaged damper, other calculated gaps are: 0.24" (25%), 0.50" (50%), 0.74" (75%).

The same thing was calculated for side to tower. The maximum relative displacement is 3.76" and consequently calculated gaps are: 0.975" (25%), 1.88" (50%), 2.82" (75%).

Effect on structural displacements

The selected check points are the following:

- mid points of main span and side spans (298, 232, 2164);
- top of east tower (11392);
- based of the foundation's east tower (10201);
- anchorage element (7121).



Figure 86. Bridge check points

From the analysis we obtained the follow results:

Table 7. Maximum displacements symmetrical damage

<i>maximum x - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	-0,81	-0,84	-0,83	-0,83	-0,86
298	mid main span	-0,18	-0,19	-0,24	-0,26	-0,37
2164	mid side span W	1,88	1,90	1,91	1,92	1,92
7121	cable bent W	-0,06	-0,06	-0,06	-0,06	-0,06
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	0,84	0,85	0,85	0,85	0,85

<i>maximum y - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	0,27	0,28	0,28	0,28	0,28
298	mid main span	-1,65	-1,65	-1,66	-1,67	-1,67
2164	mid side span W	0,41	0,41	0,41	0,41	0,41
7121	cable bent W	0,15	0,15	0,15	0,15	0,15
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	0,12	0,12	0,12	0,12	0,12

<i>maximum z - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	-7,47	-7,54	-7,55	-7,56	-7,58
298	mid main span	5,03	5,04	5,05	5,06	5,07
2164	mid side span W	-7,57	-7,63	-7,66	-7,67	-7,69
7121	cable bent W	-0,05	-0,05	-0,05	-0,05	-0,05
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	0,07	0,07	0,07	0,07	0,07

These results are summarized in fig. 87, representing the maximum longitudinal displacement of the various check points of the structure.

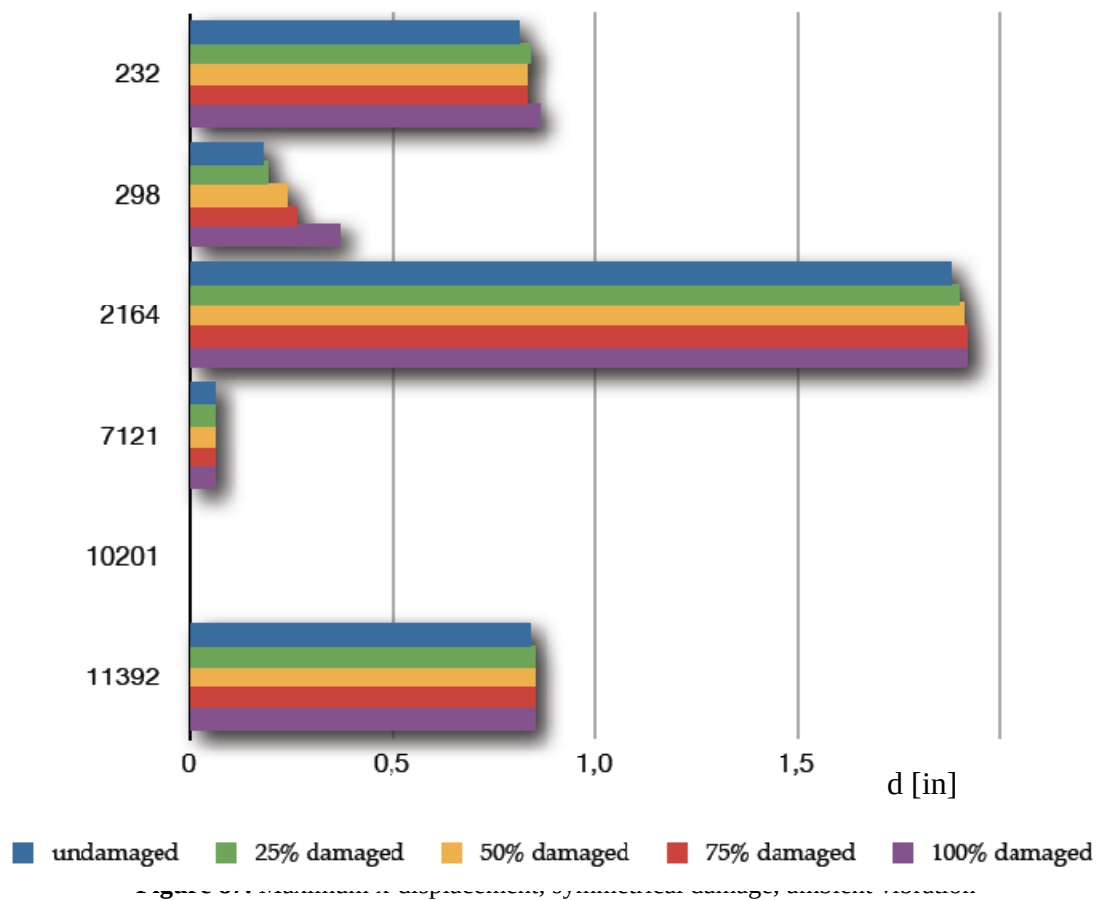


Fig. 87 shows that the higher variation in the displacement response is in the longitudinal displacement of the main span (point 298). Following are shown the displacement time histories of the points 11392 and 298, computed applying a progressive damage.

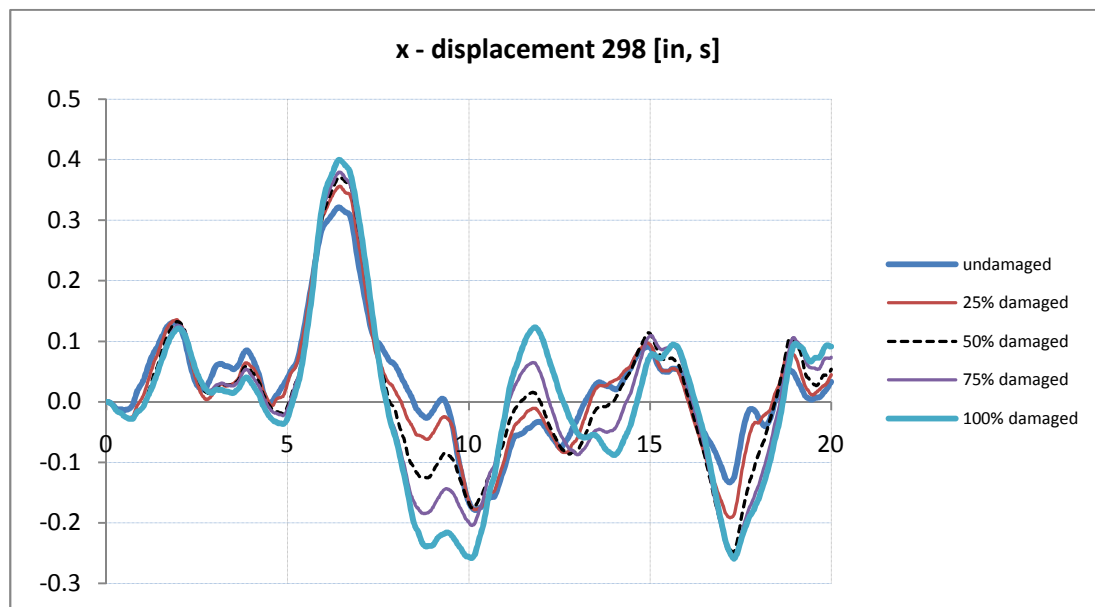


Figure 88. Longitudinal displ. midpoint main span at different dampers damages, ambient vibration

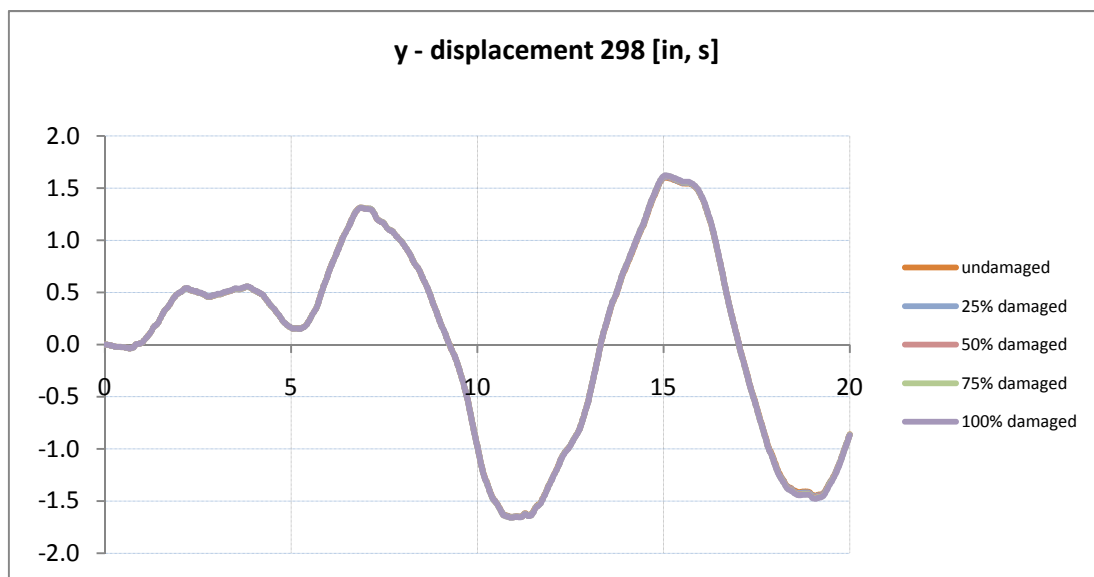


Figure 89. Transversal displ. midpoint main span at different dampers damages, ambient vibration

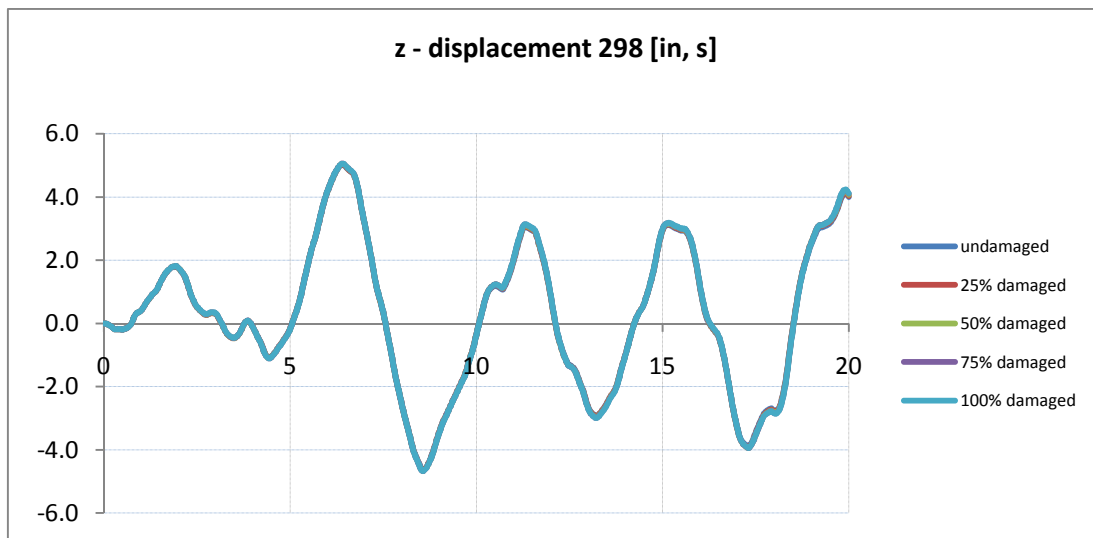


Figure 90. Vertical displ. midpoint main span at different dampers damages, ambient vibration

From the analysis is showed up that appreciable differences are just long the longitudinal direction of the bridge and properly at the main span. This direction is the direction where the dampers are oriented. The fact that only this direction of motion is interested to the damage is useful to understand that the most influenced modes shape are 2 and 6 (fig. 11 and 13).

The other components of displacement, didn't show up interesting changes.

Maximum forces and stress in the elements do not present any relevant change.

Check dampers deformations

The elongation/shortening of the two types of dampers were calculated by the relative displacements of the extreme nodes:

- MAIN to TOWER - nodes : 250 and 10144;
- SIDE to TOWER - nodes: 1848 and 10341.

In the next plots is possible to see that there is a significant increase when a gap damage is present.

In particular the full main to tower damper deformation is 2 times the deformation of the same undamaged damper. The deformation of damaged side to tower damper is around 120% the undamaged one.

Time histories of the elongation/shortening of two types of damper considered are plotted in next page:

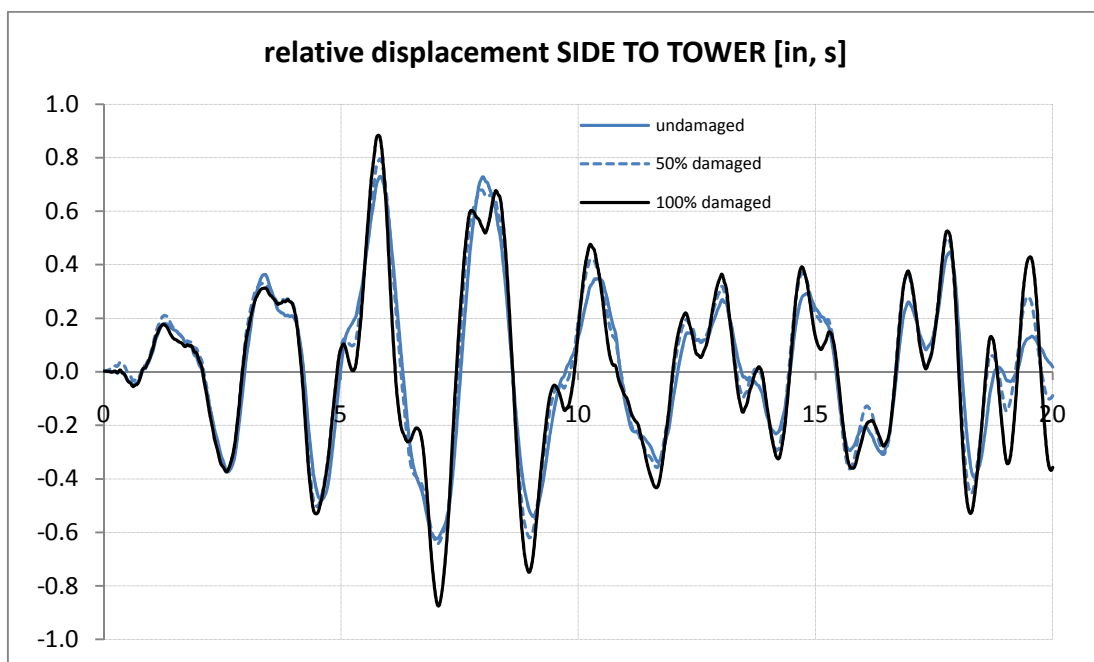


Figure 91. Damper deformations, Side to Tower

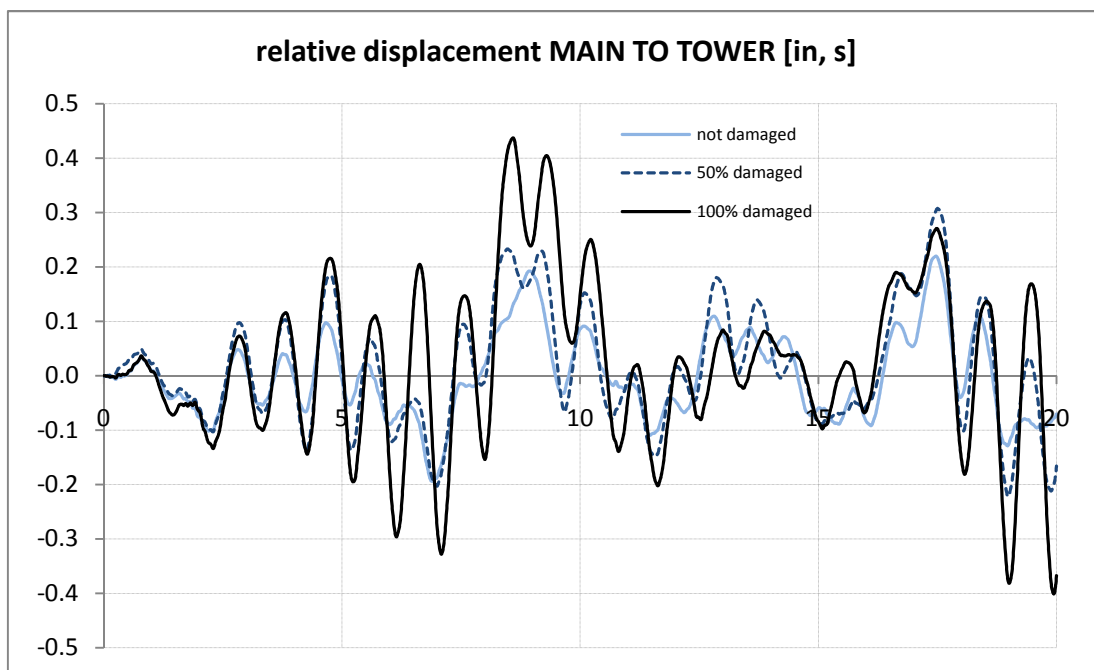


Figure 92. Damper deformations, Main to Tower

From the above charts is possible claim that the deterioration of the dampers produce an increase of the displacement of themselves. In particular in the main to tower dampers increase their work length of more than 120% (from 0.2" to 0.45").

Check dampers maximum forces

The damper forces have been calculated using the monitor element located next to the damper element. The forces were computed in the main span to tower dampers and in the side span to tower damper of both sides.

The results are summarized in next figures and table and commented after.

Table 8. Max damper forces symmetrical damage

DAMPER FORCES - ASYMMETRIC DAMAGE [kips]					
	0%	25%	50%	75%	100%
MAIN TO TOWER - EAST SIDE	3.88	5.28	5.49	6.45	1.13
MAIN TO TOWER - WEST SIDE	1.81	2.01	2.06	2.22	0.34
SIDE TO TOWER - EAST SIDE	4.65	5.15	4.79	3.19	1.47
SIDE TO TOWER - WEST SIDE	2.98	1.66	1.58	1.64	1.72

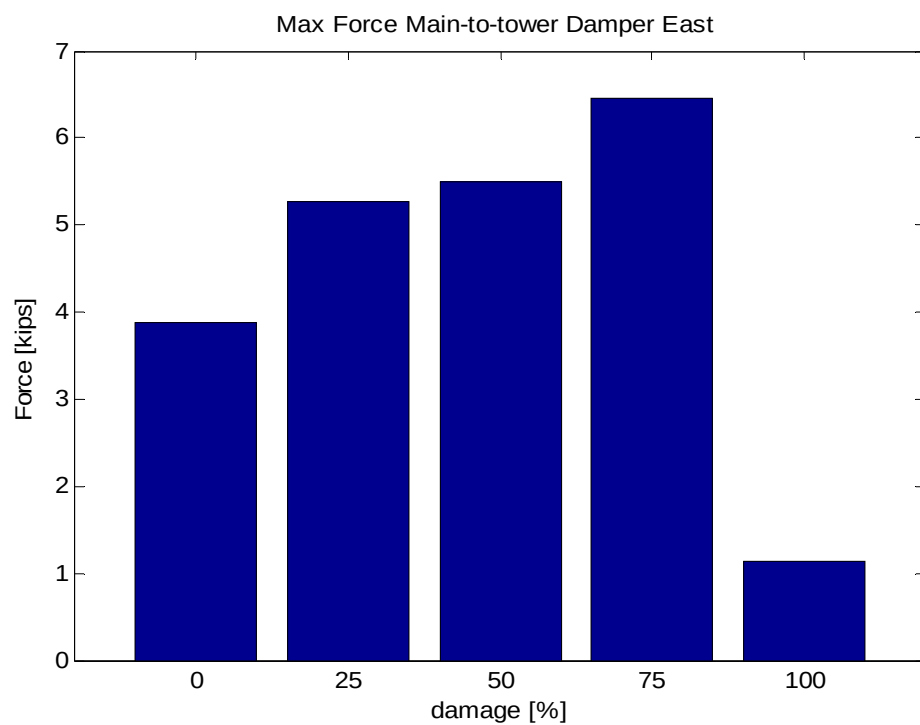


Figure 93. Peak force Main to tower damper East, ambient vibration, sym. damage

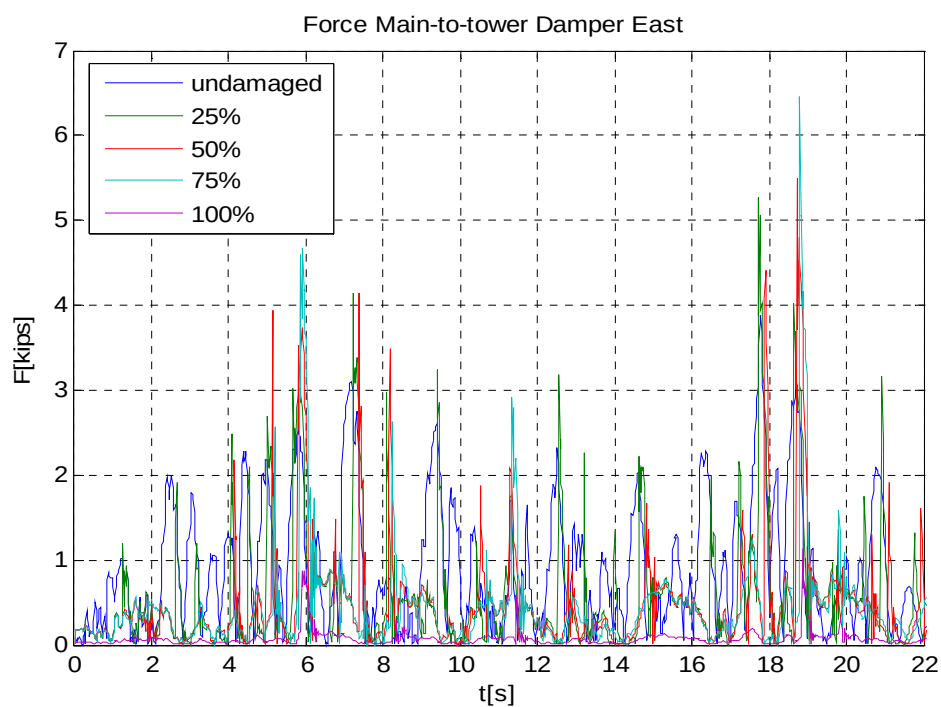


Figure 94. Time histories force Main to tower damper East, ambient vibration, sym. damage

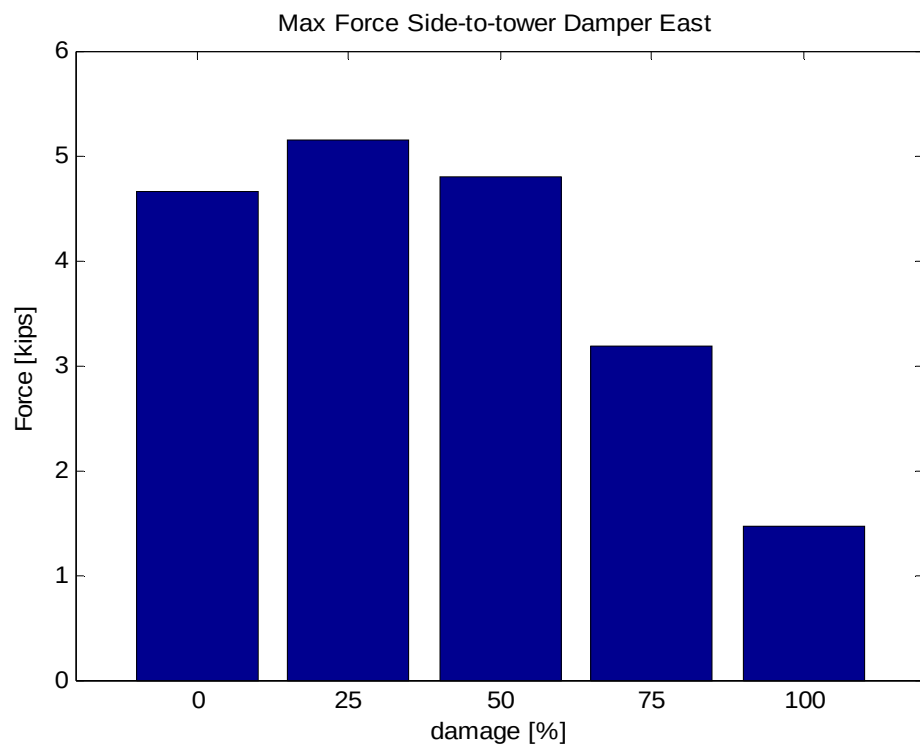


Figure 95. Peak force Side to tower damper East, ambient vibration

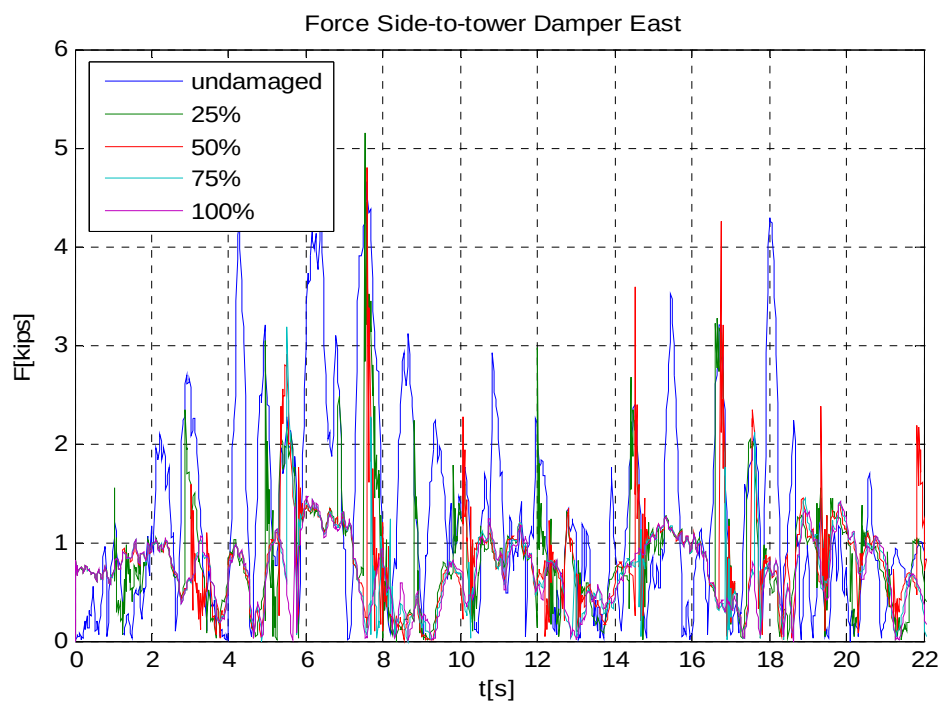


Figure 96. Time histories force Side to tower damper East, ambient vibration, sym. damage

The results show that the peak force could be higher when a damper is damaged with a gap. The reason is that the shortening velocity of the damper could be higher if the movement of the deck is not contrasted by the viscous damping force. As soon as the damper begin to work it receives an impulsive force necessary to decelerate the structure. This phenomenon was not taken in account during the lab tests that were conducted in displacement control (constant angular velocity). Fig. 97 shows exactly what happen when the velocity is sufficient to create this unwanted peak.

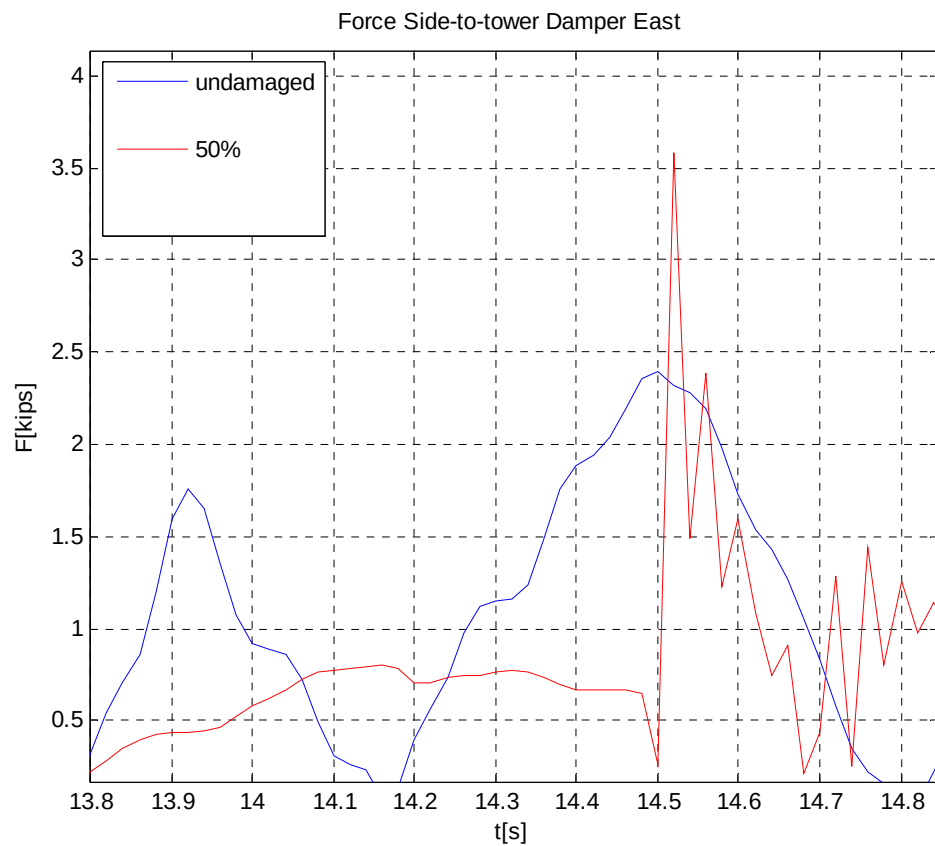


Figure 97. Peak in the 50% damaged damper, ambient vibration, sym. damage

This result was confirmed also computing the relative velocity of the ends of the damper (integrating the acceleration and taking the derivative of the displacement). The "impact" velocity in presence of a gap could be higher than the maximum velocity on a non damaged damper. C and α remain constant, so:

$$\begin{aligned} \text{if} \quad & \dot{x}_{dam} > \dot{x}_{und} \\ \Rightarrow \quad & F_{dam} = C \cdot \dot{x}_{dam}^{\alpha} > F_{dam} = C \cdot \dot{x}_{und}^{\alpha} \end{aligned}$$

This impulse in force, not considered during the design of the bridge could be a problem in the neighborhood of the damaged device. A possible solution will be presented in following chapters.

Frequencies and mode shapes analysis

As seen in chapter 1 it has been performed an elastic modal analysis of the FEM model. An elastic modal analysis is not able to compute the influence of a velocity dependent stiffness in the structure (as concentrated dampers for example).

For this reason the non linear natural frequencies were calculated by a Matlab program performing FFT analysis.

In the program were implemented the following subroutines:

- i. Input: check points acceleration time history;
- ii. improve frequency accuracy;
- iii. compute displacement time history;
- iv. filter;
- v. compute and plot non linear natural frequencies;
- vi. compute and plot non linear mode shapes.

i. Input: check points acceleration time history

The check points acceleration time history were imported by ADINA. Convenient check points have been chosen to represent the linear mode shapes function.

For example to calculate the first non linear mode of the deck's bridge, the y-direction of the main span mid point (#298) was chosen; to calculate the fourth non linear mode of the deck's bridge, the z-direction of point 298 was chosen (fig. 10 and

12). The effective correspondence between a particular frequency and its modeshape was confirmed following procedure described in next pages.

ii. Improve frequency accuracy

The time history length influence the accuracy of the frequency computation. After a FFT the minimum appreciable variation in frequency is:

$$\Delta f = 1/T$$

where T is the length of the time histories.

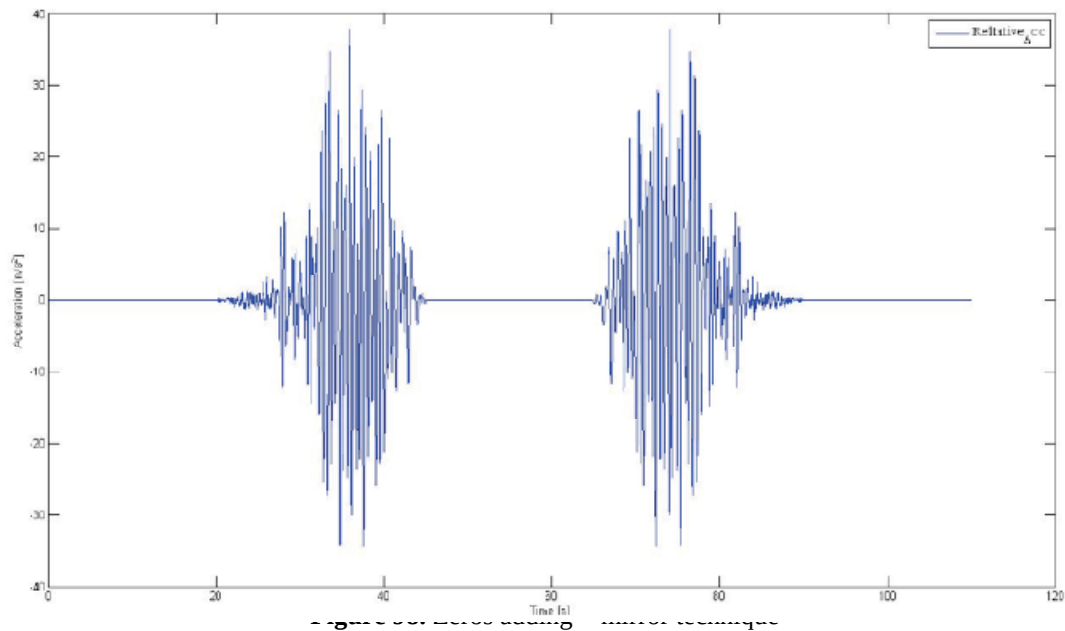
The differences between the frequencies that we want to compute were less than the original Δf . A zero adding technique and mirroring were adopted.

When zeros are added to a data sequence, no change takes place in the filter shape. However, because of the nature of the FFT computational formulas, the spacing of the estimates is based on the augmented record length and becomes

$$\Delta f' = 1/(N + N_z) h$$

where N_z represent the number of added zeros. Thus the main effect of making $N_z = N$, for example, is to halve the spacing and obtain twice the number of estimates for a given record length. To improving the frequency accuracy another strategy done has been make the double length of the record by mirror of itself.

Following the representation of these steps where the three horizontal line confirm the implementation of the zero adding technique.



In signal engineering, side lobes are the lobes of the far field radiation pattern that are not the main beam, where the terms "beam" and "lobe" are synonyms. The power density in the side lobes is generally much less than that in the main beam. It is generally desirable to minimize the sidelobe level (SLL), which is measured in decibels relative to the peak of the main beam.

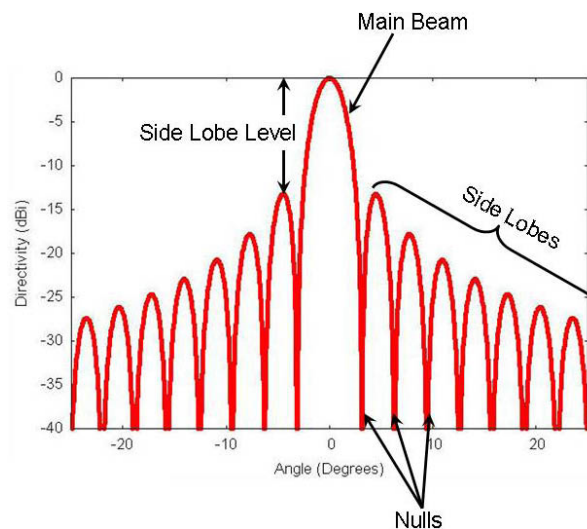


Figure 99. Side lobe effect

To minimize the undesirable side lobe effect produced in the interface zone between the digital signal with the zero adding an Tukey procedure has been implemented.

The Tukey window, also known as the *tapered cosine window*, can be regarded as a cosine lobe of width $\frac{\alpha N}{2}$ that is convolved with a rectangle window of width $(1 - \frac{\alpha}{2})N$. At $\alpha = 1$ it becomes rectangular, and at $\alpha = 0$ it becomes a Hann window.

$$\omega_0(n) = \begin{cases} \frac{1 + \cos\left(\pi \frac{|n| - \frac{\alpha N}{2}}{(1 - \frac{\alpha}{2})\frac{N}{2}}\right)}{2} & \text{when } \frac{\alpha N}{2} \leq |n| \leq \frac{N}{2} \\ 1 & \text{when } 0 \leq |n| \leq \frac{\alpha N}{2} \end{cases}$$

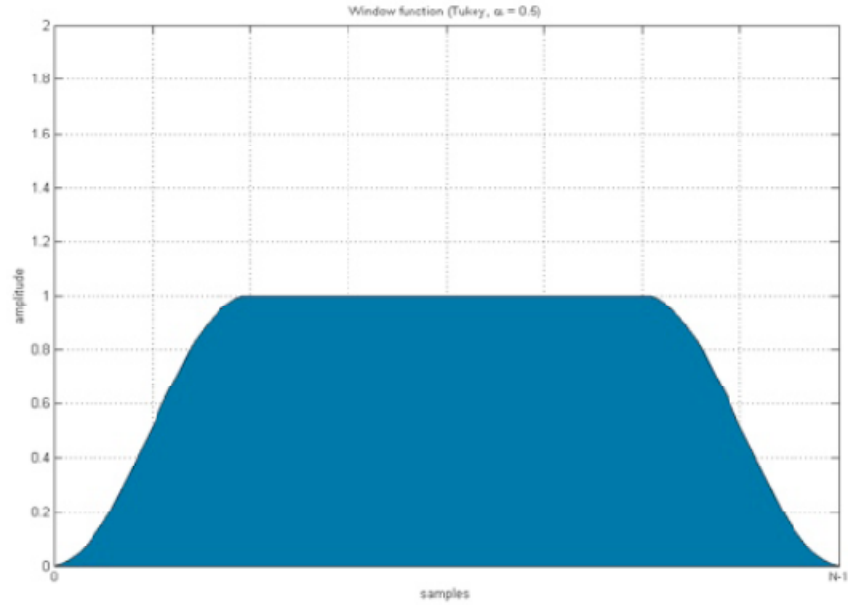


Figure 100. Tukey procedure function

After these 3 modifications the signal is ready to be integrate, filtered and transformed in a frequency domain to evaluate the frequency response.

iii. *Computation of displacement time history*

The displacements of time history were calculated by twice cumulative trapezoidal numerical integration.

iv. *Filtering*

A band-pass filter is a device that passes frequencies within a certain range and rejects (attenuates) frequencies outside that range. These filters can also be created by combining a low-pass filter with a high-pass filter.

An ideal bandpass filter would have a completely flat passband (e.g. with no gain/attenuation throughout) and would completely attenuate all frequencies outside the passband. Additionally, the transition out of the passband would be instantaneous in frequency. In practice, no bandpass filter is ideal. The filter does not attenuate all frequencies outside the desired frequency range completely; in particular, there is a region just outside the intended passband where frequencies are attenuated, but not rejected. This is known as the filter roll-off, and it is usually expressed in dB of attenuation per octave or decade of frequency. Generally, the design of a filter seeks to make the roll-off as narrow as possible, thus allowing the filter to perform as close as possible to its intended design. Often, this is achieved at the expense of pass-band or stop-band *ripple*.

The bandwidth of the filter is simply the difference between the upper and lower cutoff frequencies. The shape factor is the ratio of bandwidths measured using two different attenuation values to determine the cutoff frequency, e.g., a shape factor of 2:1

at 30/3 dB means the bandwidth measured between frequencies at 30 dB attenuation is twice that measured between frequencies at 3 dB attenuation.

Following an example of band-pass filter.

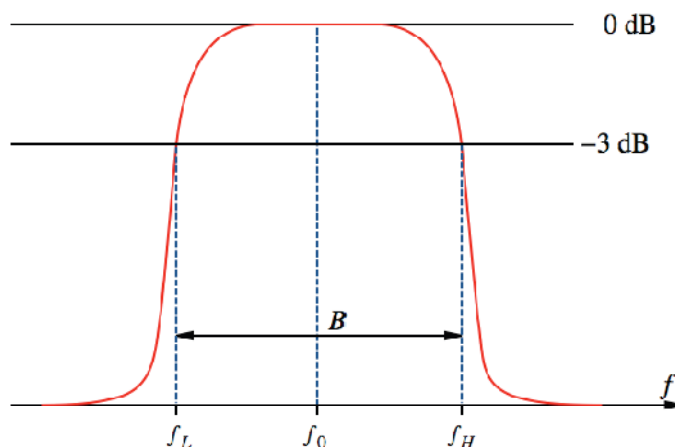


Figure 101. Pass-band filter, example

where the B length has been defined equally to 0.1 Hz and where the f_0 is the awaited frequency.

v. *Compute and plot of a non linear natural frequency*

To compute non linear natural frequencies the FFT has been used.

A fast Fourier transform (FFT) is an efficient algorithm to compute the discrete Fourier transform (DFT) and its inverse. There are many distinct FFT algorithms involving a wide range of mathematics, from simple complex-number arithmetic to group theory and number theory.

A DFT decomposes a sequence of values into components of different frequencies. This operation is useful in many fields but computing it directly from the definition is often too slow to be practical. An FFT is a way to compute the same result more quickly. The difference in speed can be substantial, especially for long data sets where N may be in the thousands or millions—in practice, the computation time can be reduced by several orders of magnitude in such cases, and the improvement is roughly proportional to $N/\log(N)$. This huge improvement made many DFT-based algorithms practical; FFTs are of great importance to a wide variety of applications, from digital signal processing and solving partial differential equations to algorithms for quick multiplication of large integers.

Superimposing on the same plot the transform of the filtered signal calculated with different gaps in the dampers is possible to determine if there are any differences in the frequency response of the structure. The same analysis were conducted for all the significant modes and for all the damaged configuration used in this work.

Following the frequencies computed by Matlab program.

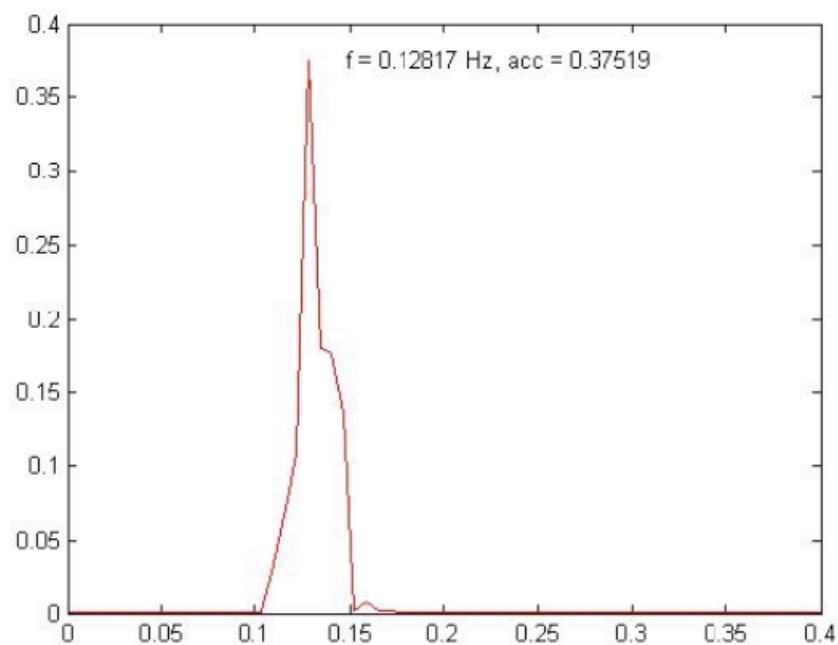


Figure 102. FFT point 298, y direction, filtered for mode 1

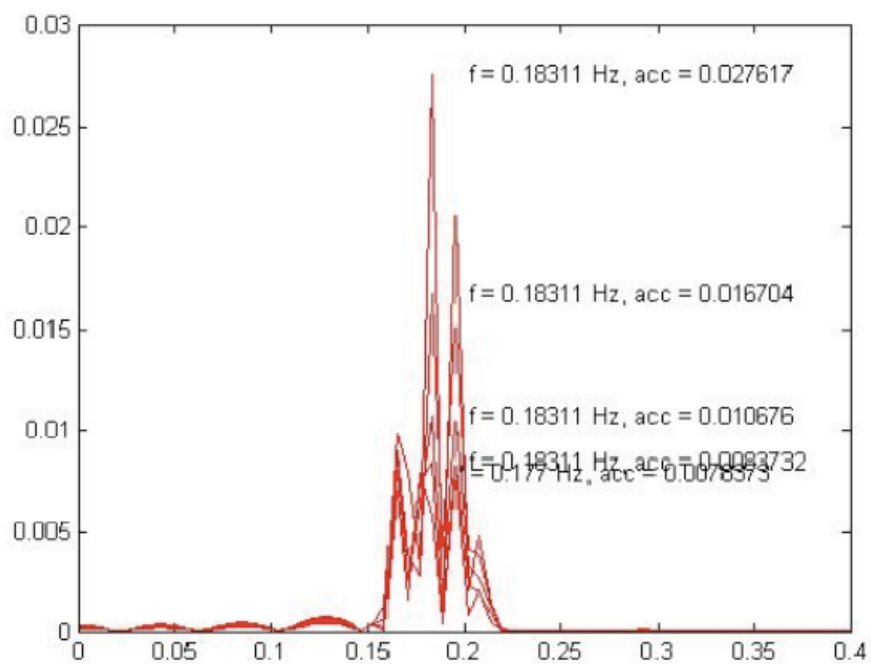


Figure 103. FFT point 298, y direction, filtered for mode 2

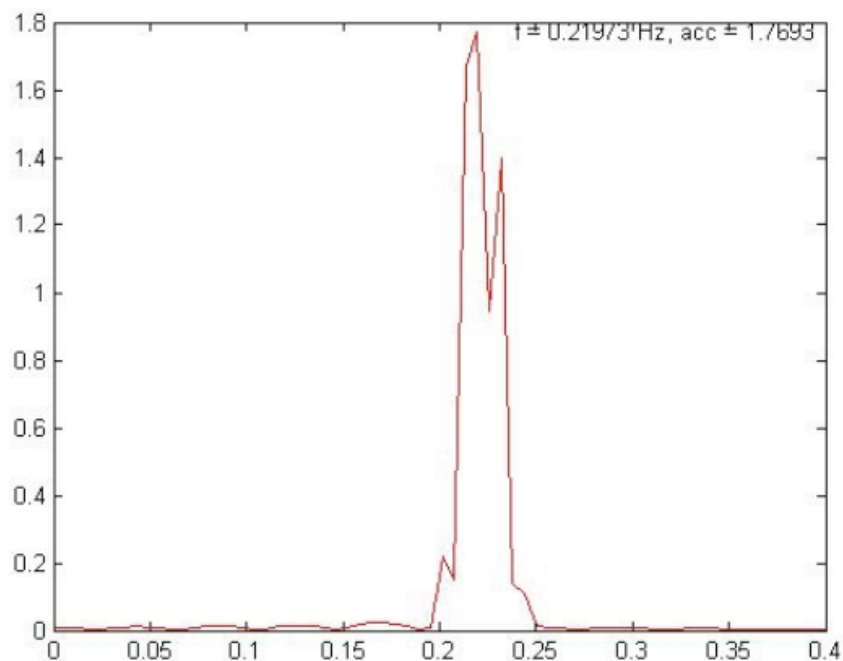


Figure 104. FFT point 298, z direction, filtered for mode 4

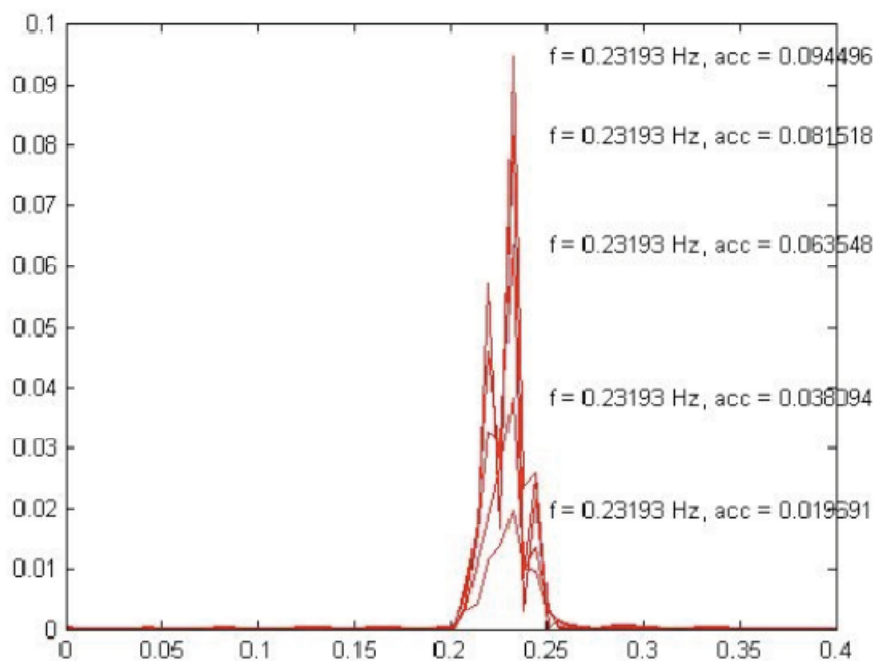


Figure 105. FFT point 298, x direction, filtered for mode 6

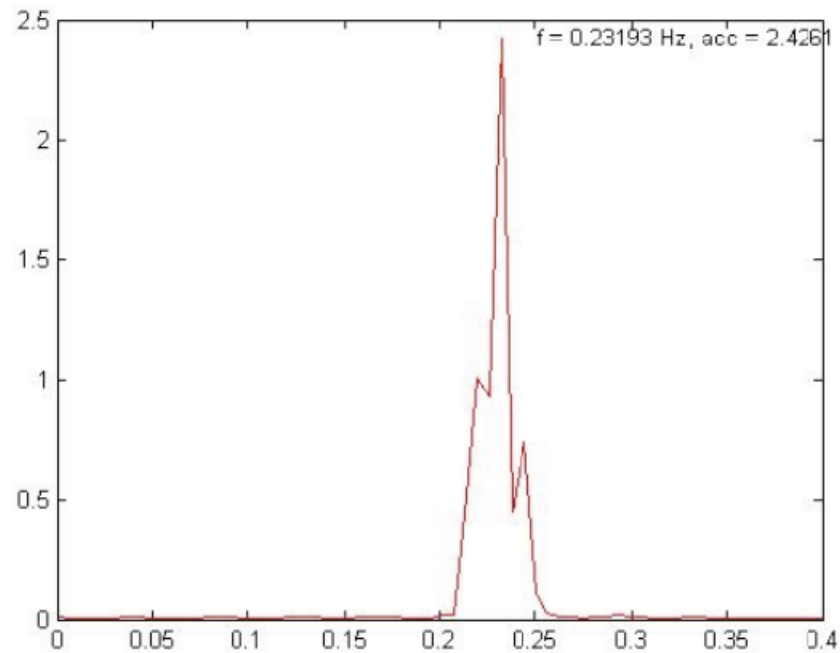


Figure 106. FFT point 298, z direction, filtered for mode 6

From the above charts is possible to appreciate a variation of the frequency belongs to the second mode long the longitudinal direction. The frequency belongs to the undamaged condition pass from 0.183Hz to 0.177Hz, is possible recognize a limited increase of the flexibility of the bridge. Consulting table 2 is possible to see that the non linear natural frequencies are very similar to the same ones computed with a modal analysis (as expected).

vi. Compute and plot of a non linear mode shapes

Goal of this subroutine is to verify if the frequencies calculated in the previous step match their own natural mode shapes. The reason for this check is because is not to

be excluded a priori that a different mode shape could be associated to a frequency previously associated to another shape.

Best way to notice this is to make an animation of the filtered displacement of the deck. To create this video of the non linear mode shape 5 equally spaced deck points has been chosen. To represent the modal shape associated to a particular frequency it has been applied to the points displacement time histories a bandpass filter.

Two screen shots of the mode shape animation of the deck have been reported below:

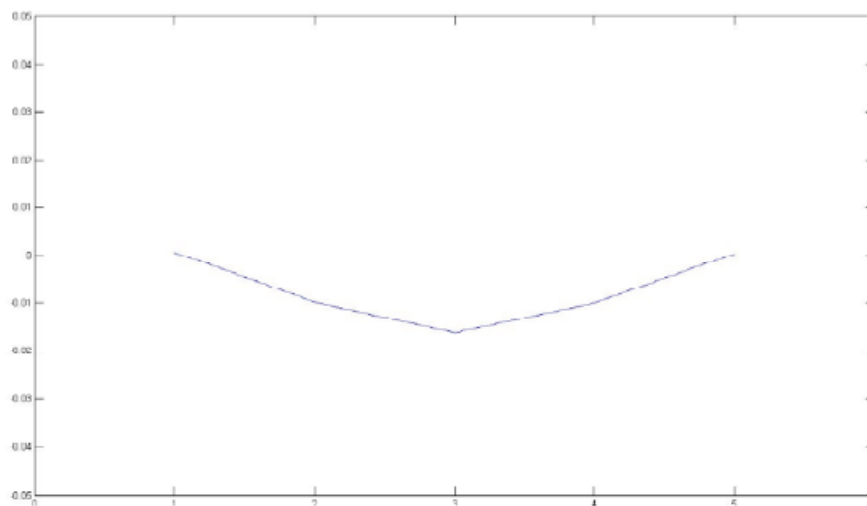


Figure 107. Screenshot of 1st mode, y direction

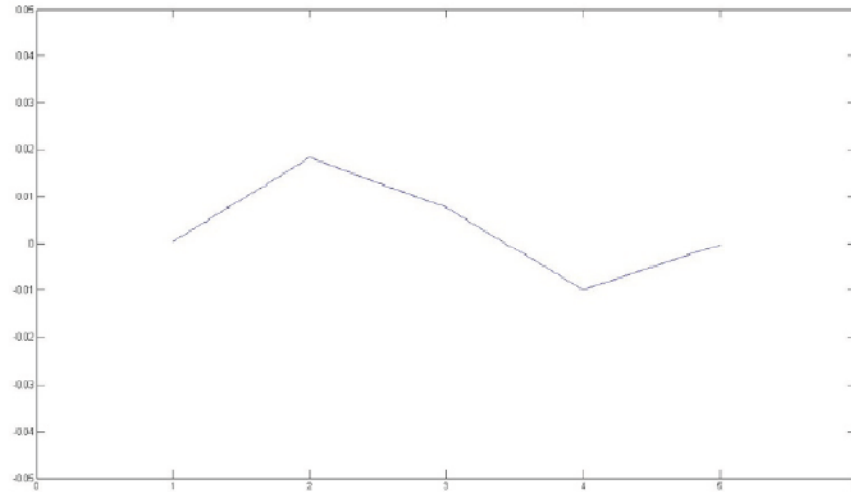


Figure 108. Screenshot of 2nd mode, z direction

The non linear mode shapes match correctly the non linear frequencies computed.

5.1.4 Parameterization and results of the analysis, Asymmetric Damage

The second parameterization has been conducted producing a progressive damage of the bridge's west side, equal to 25%, 50%, 75% and 100%, of two type of damper which connected the tower with the main and side spans, called MAIN TO TOWER and SIDE to TOWER dampers.

The damage by gap has been set just in the WEST side of the bridge, the gap's lengths are the same of the previous symmetric analysis.

Effect on structural displacements

The selected check points are the following:

- mid points of main span and side spans (298, 232, 2164);
- top of east tower (11392);
- based of the foundation's east tower (10201);
- anchorage element (7121).

From the analysis we obtained the following results:

Table 9. Maximum displacements asymmetrical damage, ambient vibration analysis

<i>maximum x - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged W	50% damaged W	75% damaged W	100% damaged W
232	mid side span E	-0,81	-0,81	-0,81	-0,81	-0,81
298	mid main span	-0,18	-0,20	-0,21	-0,23	-0,42
2164	mid side span W	1,88	1,89	1,9	1,9	1,9
7121	cable bent W	-0,06	-0,06	-0,06	-0,06	-0,06
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	0,84	0,84	0,84	0,84	0,83
<i>maximum y - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged W	50% damaged W	75% damaged W	100% damaged W
232	mid side span E	0,27	0,27	0,27	0,27	0,27
298	mid main span	-1,65	-1,65	-1,65	-1,65	-1,66
2164	mid side span W	0,41	0,41	0,41	0,41	0,41
7121	cable bent W	0,15	0,15	0,15	0,15	0,15
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	0,12	0,12	0,12	0,12	0,12
<i>maximum z - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged W	50% damaged W	75% damaged W	100% damaged W
232	mid side span E	-7,47	-7,48	-7,49	-7,49	-7,53
298	mid main span	5,05	5,03	5,04	5,03	5,05
2164	mid side span W	-7,27	-7,29	-7,29	-7,29	-7,8
7121	cable bent W	-0,05	-0,05	-0,05	-0,05	-0,05
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	0,07	0,07	0,07	0,07	0,07

The maximum displacement are very similar to the maximum displacement computed in the symmetrical analysis. The input results too low to create a difference in the global response between the various configurations.

Values of all the other global parameters controlled in section 4.1.3 appear to be similar.

Check dampers maximum forces

The damper forces have been calculated using the monitor element located next to the damper element. The forces were computed in the main span to tower dampers and in the side span to tower damper of both sides.

The results are summarized in next figures and table and commented after.

Table 10. Max damper forces asymmetrical damage

DAMPER FORCES - ASYMMETRIC DAMAGE [kips]					
	0%	25%	50%	75%	100%
<i>MAIN TO TOWER - EAST SIDE</i>	3.88	3.88	3.87	3.86	0.4
<i>MAIN TO TOWER - WEST SIDE</i>	1.81	1.78	1.83	1.88	0.2
<i>SIDE TO TOWER - EAST SIDE</i>	4.65	4.62	4.62	4.61	0.5
<i>SIDE TO TOWER - WEST SIDE</i>	2.98	2.48	2.50	2.51	0.22

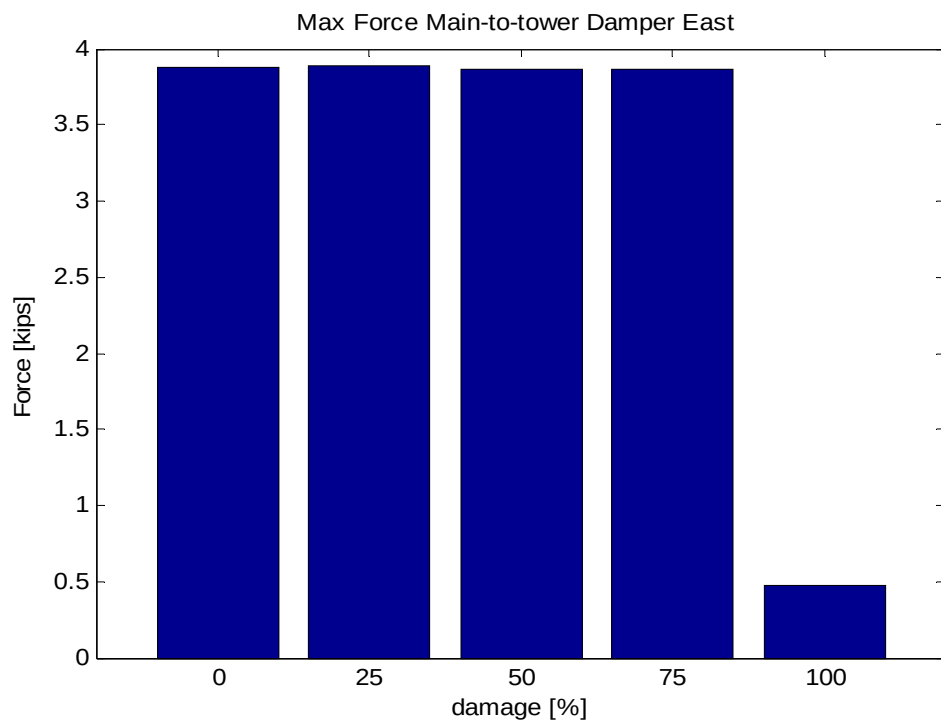


Figure 109. Peak force Main to tower undamaged damper East , ambient vibration, asym. damage

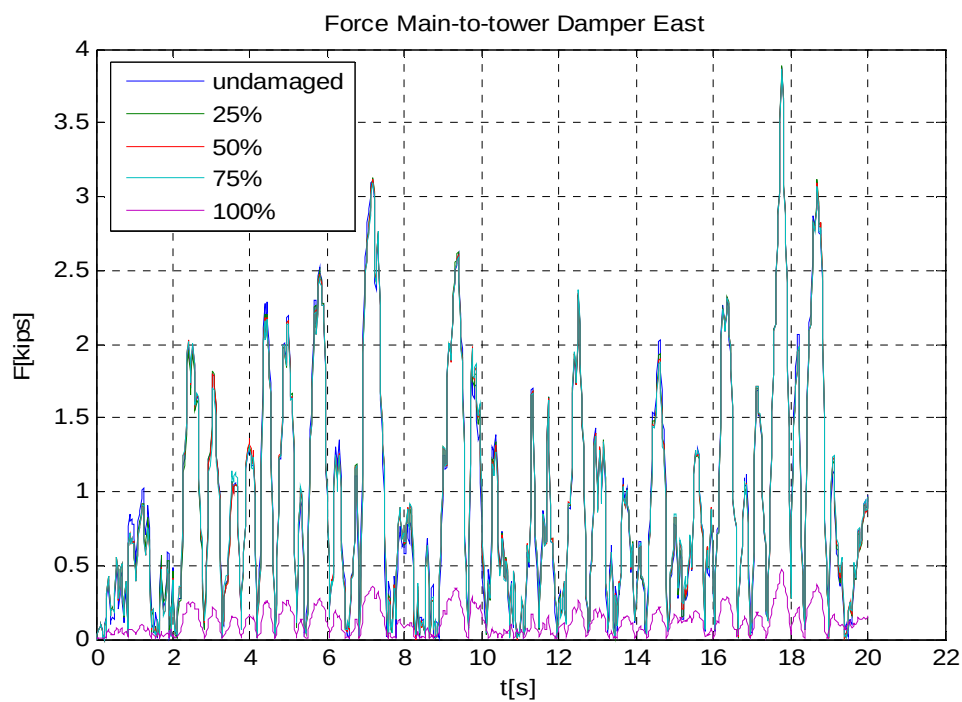


Figure 110. Time histories force Main to tower undamaged damper East, ambient vibration, asym.

damage

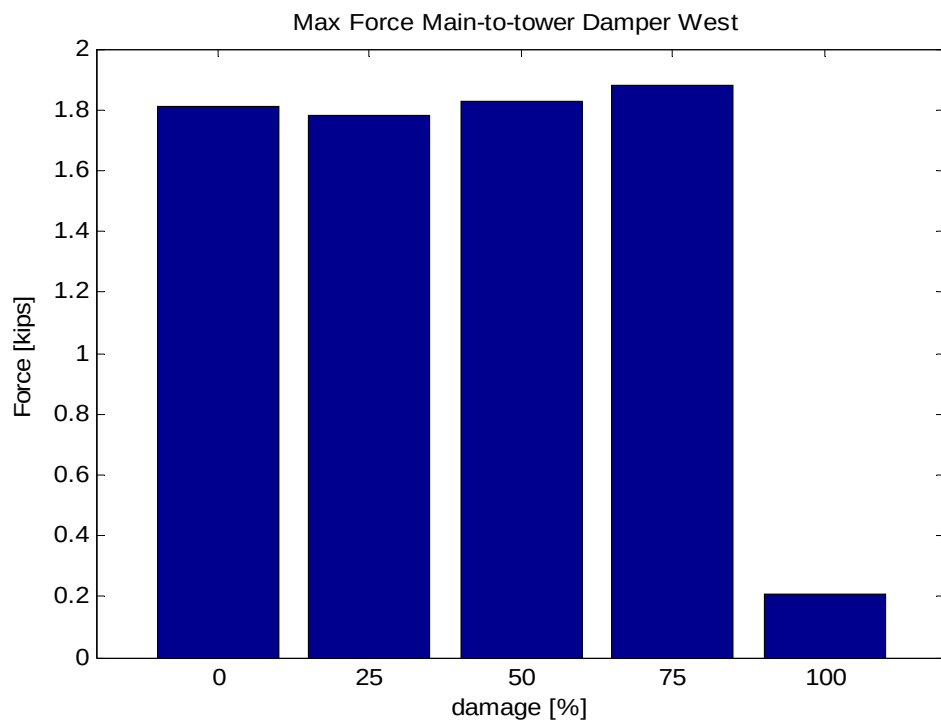


Figure 111. Peak force Main to tower damaged damper West , ambient vibration, asym. damage

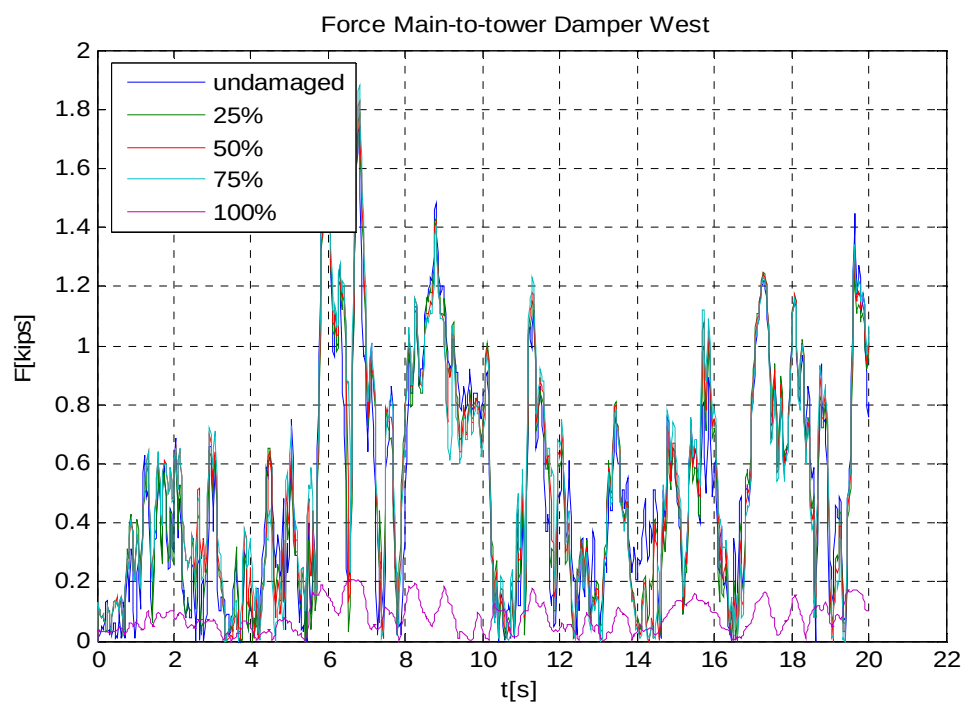


Figure 112. Time histories force Main to tower undamaged damper East, ambient vibration, asym.
damage

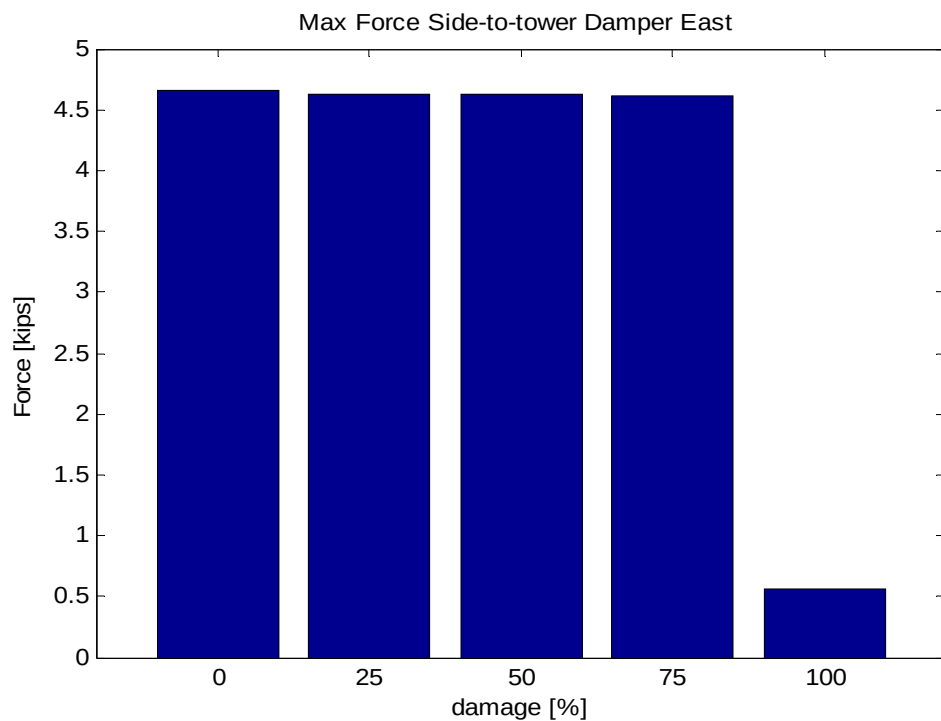


Figure 113. Peak force Side to tower undamaged damper East , ambient vibration, asym. damage

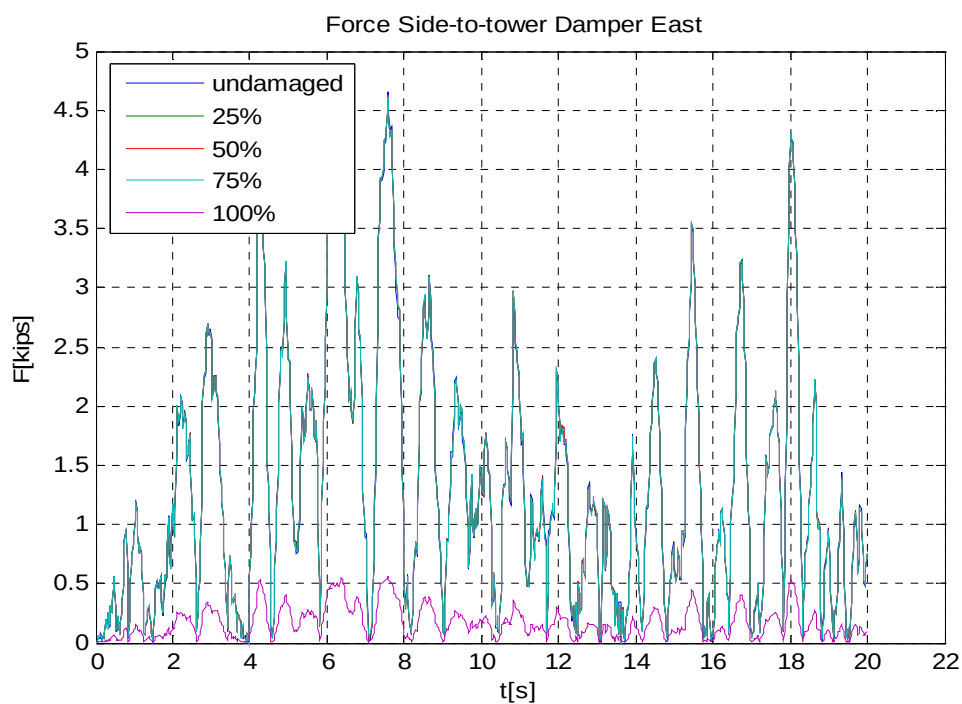


Figure 114 .Time histories force Side to tower undamaged damper East, ambient vibration, asym. damage

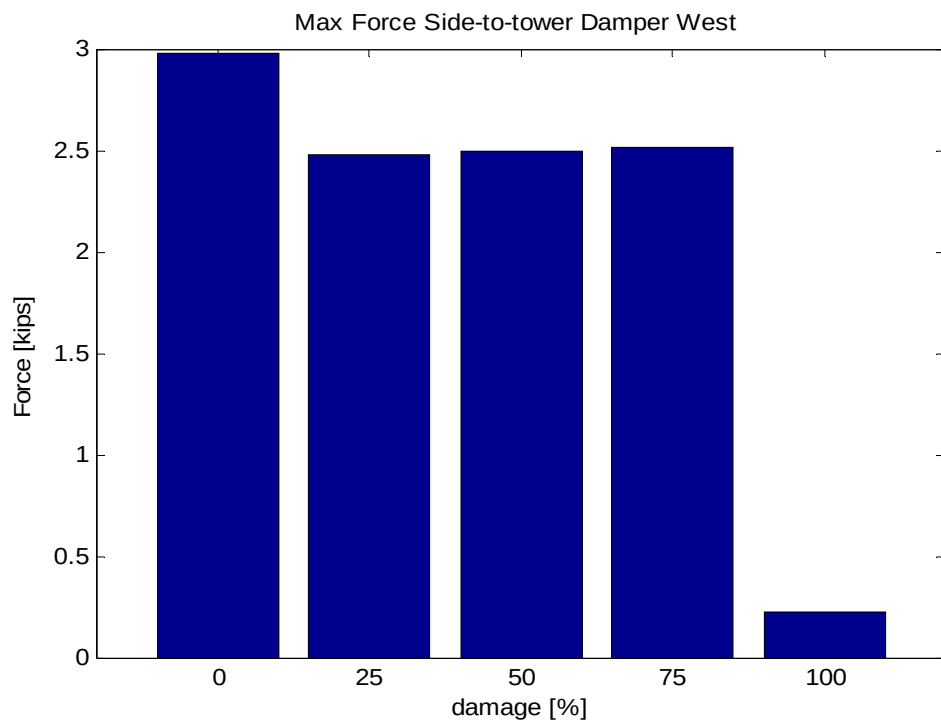


Figure 115. Peak force Side to tower damaged damper West , ambient vibration, asym. damage

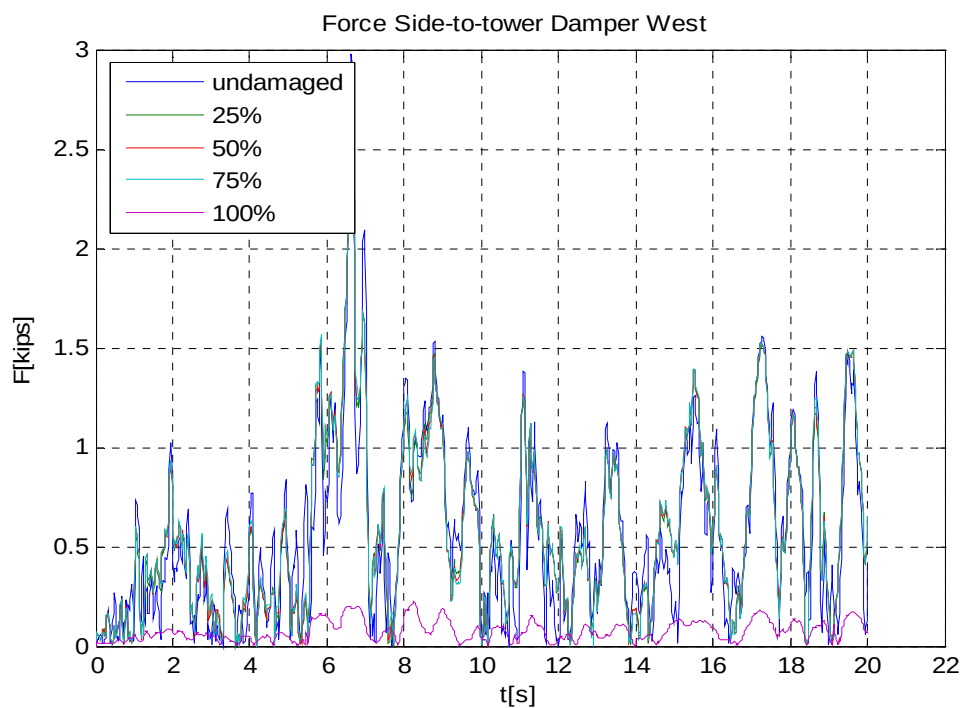


Figure 116. Time histories force Side to tower undamaged damper East, ambient vibration, asym. damage

Results show that during ambient vibration if the damage is localized just in one side of the bridge, the undamaged damper on the other side of the deck are able to damp the movement of the decks (main and side ones) and prevent the peak force phenomenon.

Frequencies and global behavior of the structure do not change.

5.2 Strong white noise analysis

5.2.1 Description of the input data

Ambient vibration analysis shows that there is not a appreciable difference in the global behavior of the structure is a damage happen in one or more dampers. In order to verify if this is true also during a severe earthquake the previous white noise has been modified conveniently to try to produce an earthquake damage on the bridge. The acceleration peaks are 1.5g on x direction and 1.0g on y and z directions.

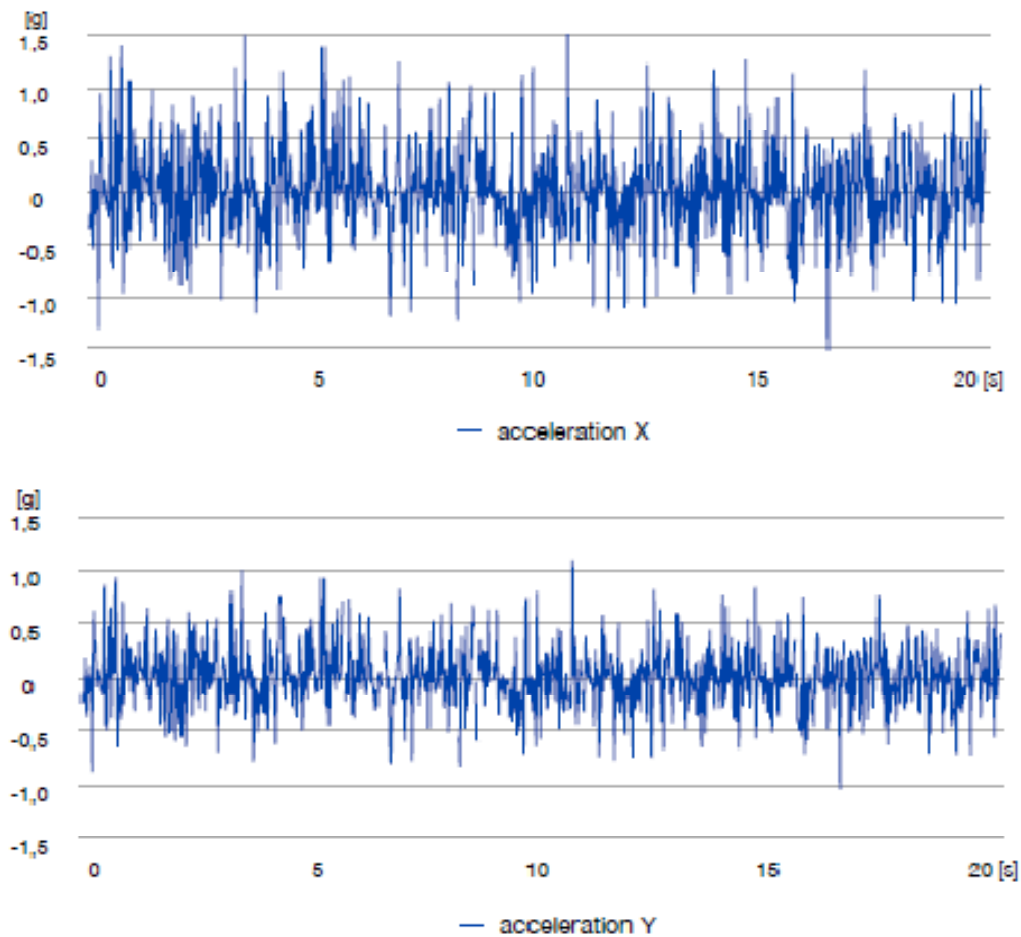


Figure 117. X,Y acceleration time history input for strong white noise analysis

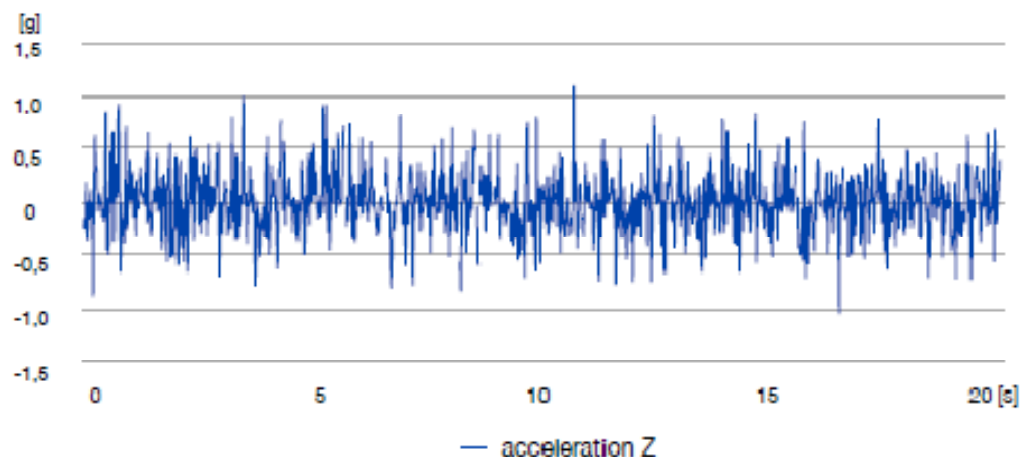


Figure 118. Z acceleration time history input for strong white noise analysis

5.2.2 Parameterization and results of the analysis, Symmetric Damage

As seen before the damage in dampers has been modeled like a gap in their force-deformation response. Artificial gaps has been inserted in the model in order to simulate these damages. The first parameterization has been conducted producing a progressive damage, equal to 25%, 50%, 75% and 100% of the maximum operative gap. Maximum gap is considered equal to the maximum relative displacement of two extremes points of a particular damper. The structural response of a 100% damaged damper is so equal to the response of a not working damper. The damage was localized in two type of damper which connected the tower with the main and side spans, called MAIN TO TOWER and SIDE TO TOWER dampers.

The maximum relative displacement in main to tower damaged dampers during simulated ambient vibration. Considering 18" as completely damaged damper, other calculated gaps are: 4.5" (25%), 9.0" (50%), 13.5" (75%).

The same thing was calculated for side to tower. The maximum relative displacement is 6.0" and consequently calculated gaps are: 1.5" (25%), 3" (50%), 4.5" (75%).

Effect on structural displacements

The selected check points are the following:

- mid points of main span and side spans (298, 232, 2164);
- top of east tower (11392);
- base of the foundation's east tower (10201);
- anchorage element (7121).

From the analysis we obtained the follow results [in]:

Table 11. Maximum displacements symmetrical damage, strong w.n. analysis

<i>maximum x - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	8,48	8,99	8,19	8,11	8,52
298	mid main span	19,58	22,96	25,27	26,32	28,50
2164	mid side span W	8,58	8,93	8,20	8,56	9,09
7121	cable bent W	-0,45	-0,45	-0,45	-0,45	-0,45
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	2,93	2,95	3,31	3,49	3,49
<i>maximum y - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	-5,63	-6,56	-6,69	-6,74	-6,75
298	mid main span	-38,33	-38,5	-38,65	-38,75	-38,86
2164	mid side span W	-7,44	-7,51	-7,42	-7,46	-7,48
7121	cable bent W	0,89	0,90	0,89	0,89	0,89
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	3,15	3,19	3,25	3,23	3,24
<i>maximum z - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	-15,69	-15,88	-15,05	-14,90	-14,74
298	mid main span	17,00	17,26	17,43	17,63	17,83
2164	mid side span W	-21,43	-21,12	-21,69	-21,74	-21,74
7121	cable bent W	-0,33	-0,33	-0,33	-0,33	-0,33
10201	base tower W	0,00	0,00	0,00	0,00	0,00
11392	top tower W	0,20	0,20	0,20	0,20	0,20

Longitudinal displacement of the deck show appreciable variation between the various damage configurations. Is possible to observe some variations of displacement in the vertical direction in the same points (232, 298).

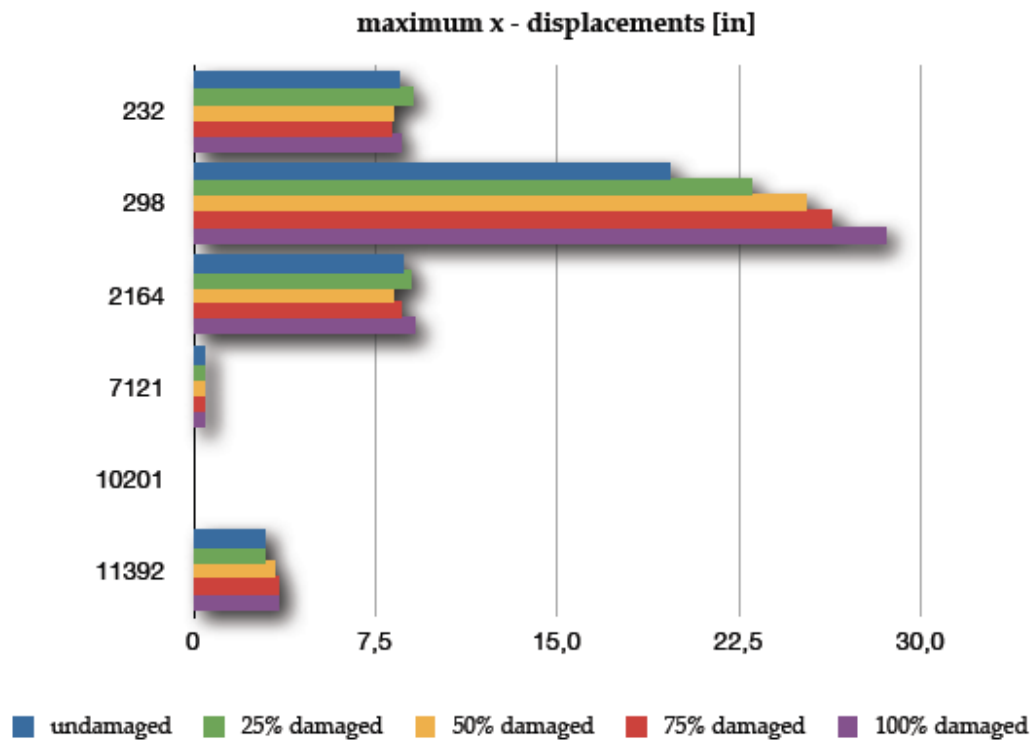


Figure 119. Maximum x-displacement, symmetrical damage, strong w.n. analysis

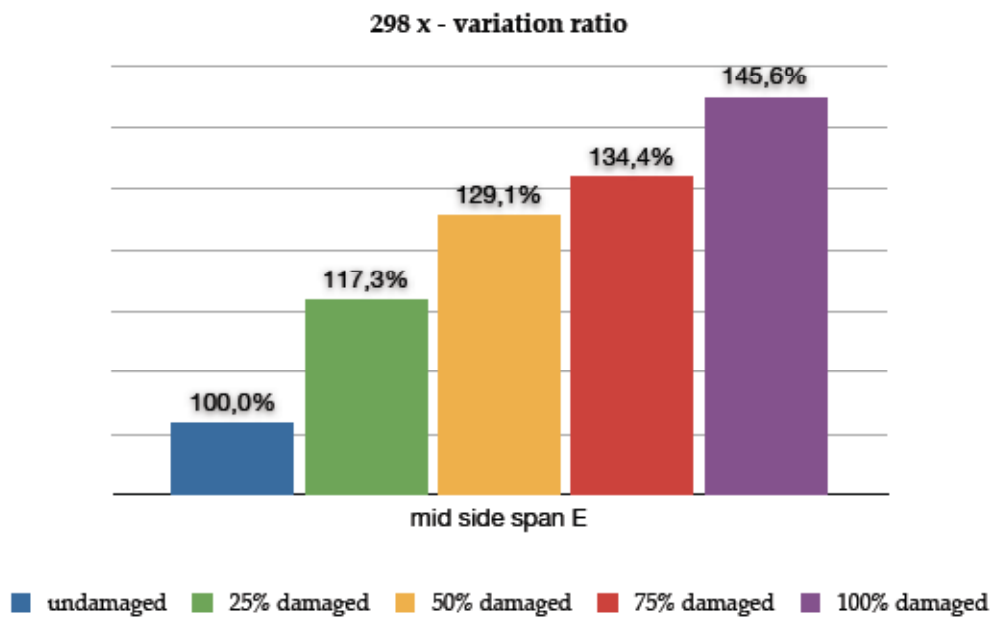


Figure 120. Percent variation x-displacement #298, symmetrical damage, strong w.n. analysis

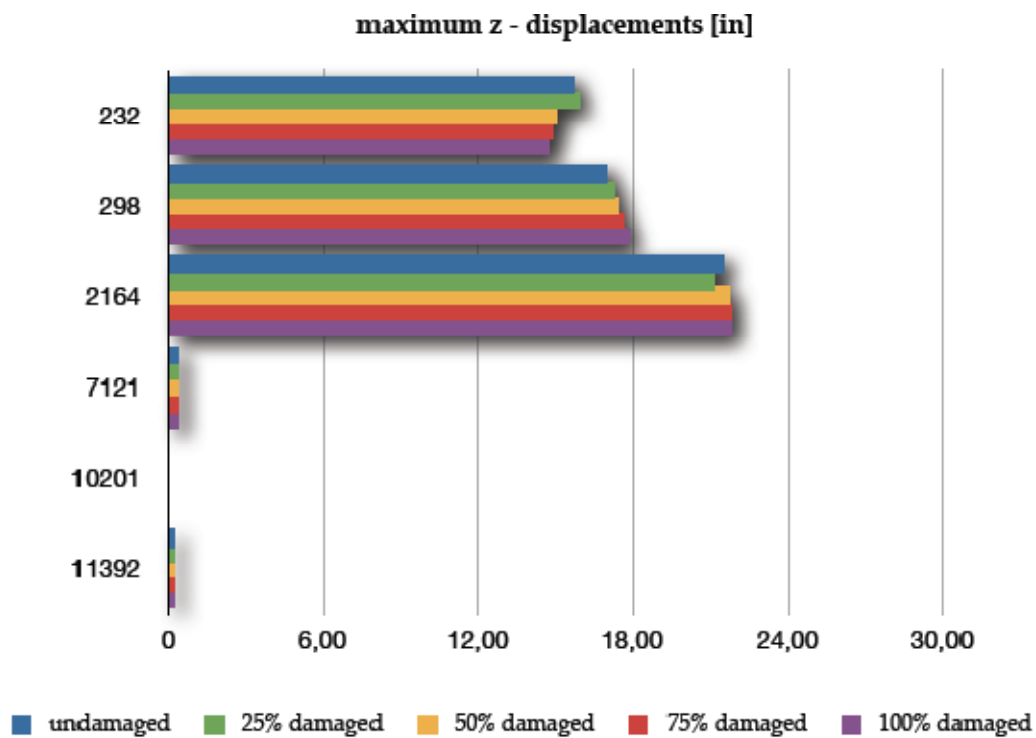


Figure 121. Maximum z-displacement, symmetrical damage, strong w.n. analysis

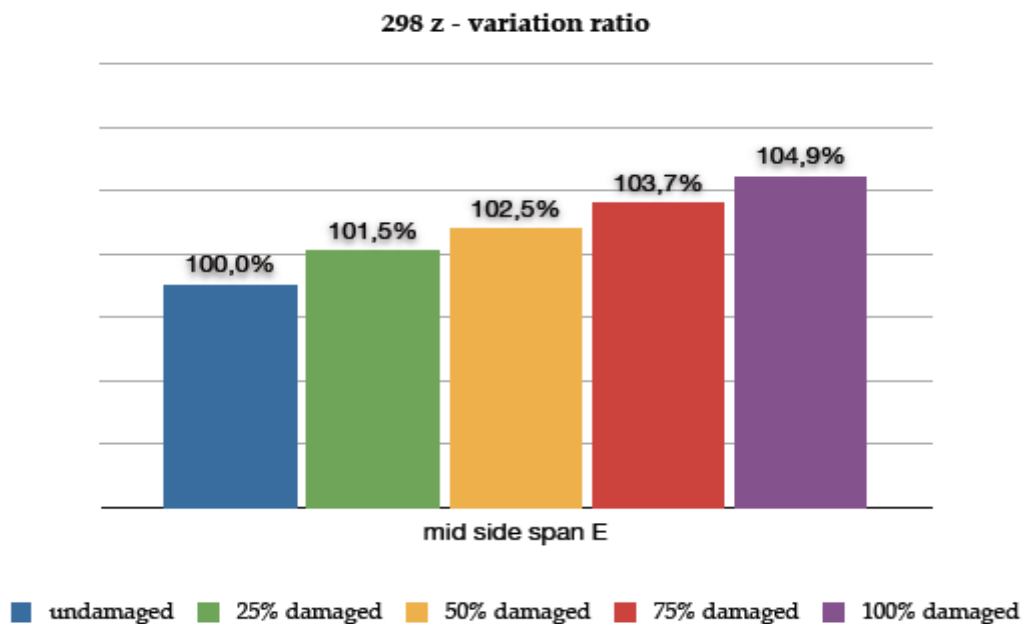


Figure 122. Percent variation z-displacement #298, symmetrical damage, strong w.n. analysis

The displacements of the mid main span increase more than the other check points of the structure increasing the seismic input.

This point increases more than the other increasing the damage in the dampers (+46% with 100% damaged dampers). This could be a confirmation that the dampers could modify only the longitudinal response of the main deck.

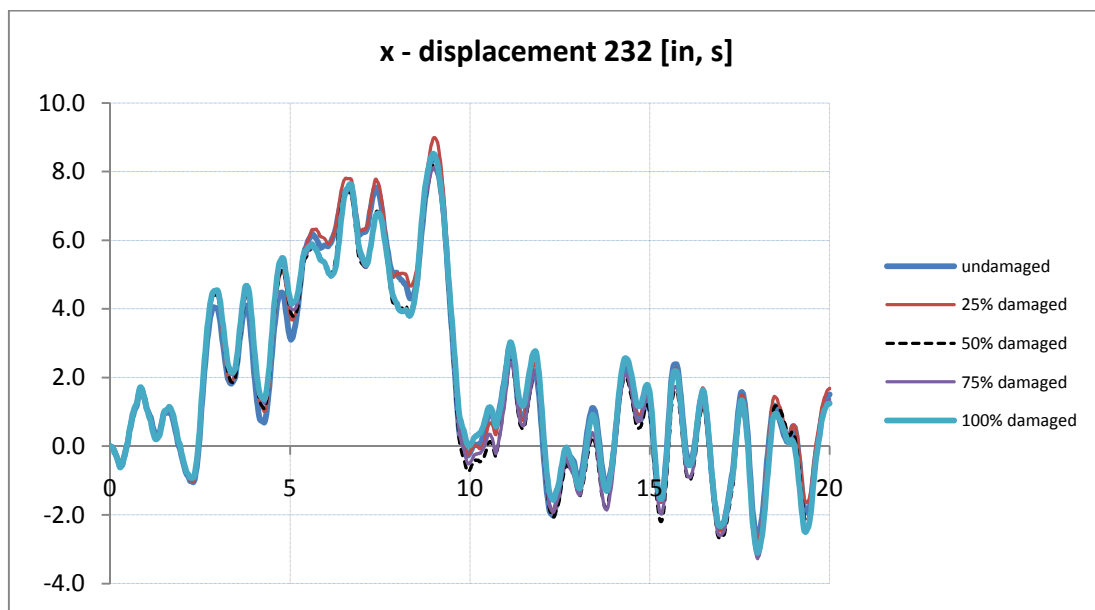


Figure 123. Longitudinal displ. midpoint side span at different damper damages, strong white noise

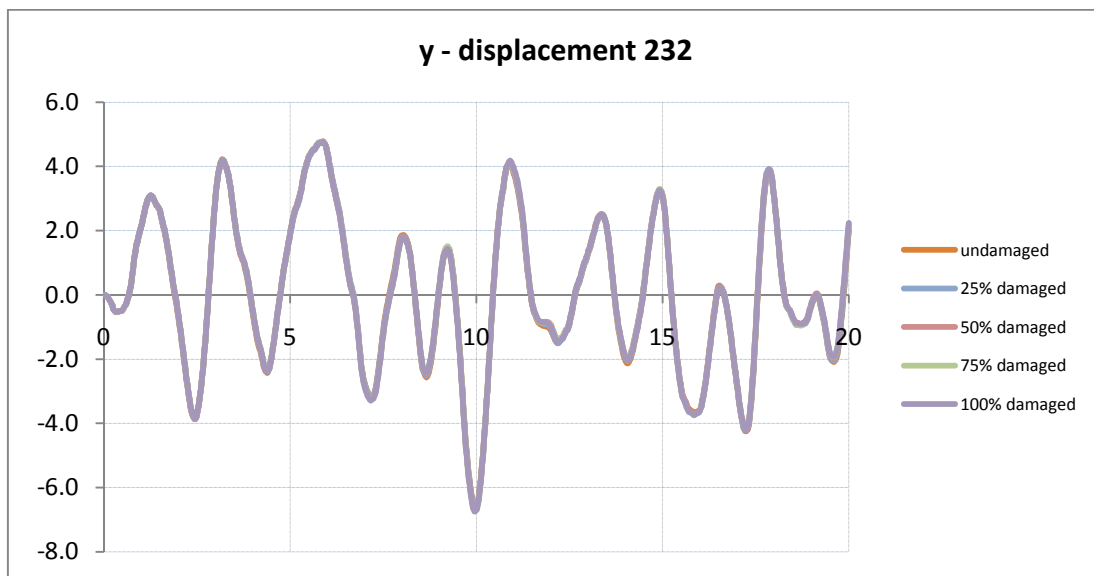


Figure 124. Transversal displ. midpoint side span at different dampers damages, strong white noise

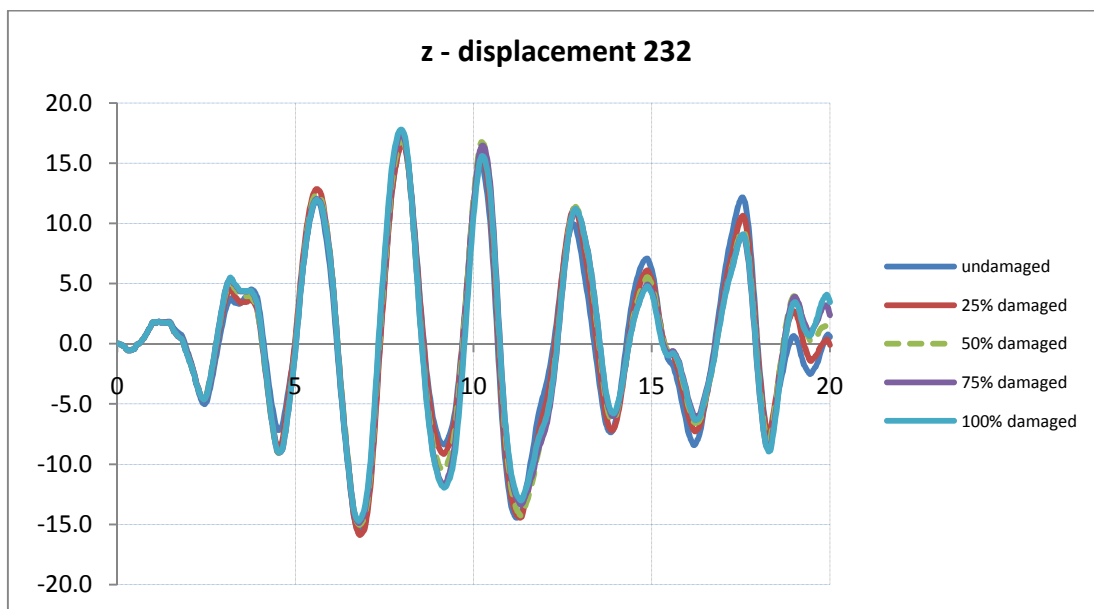


Figure 125. Vertical displ. midpoint side span at different dampers damages, strong white noise

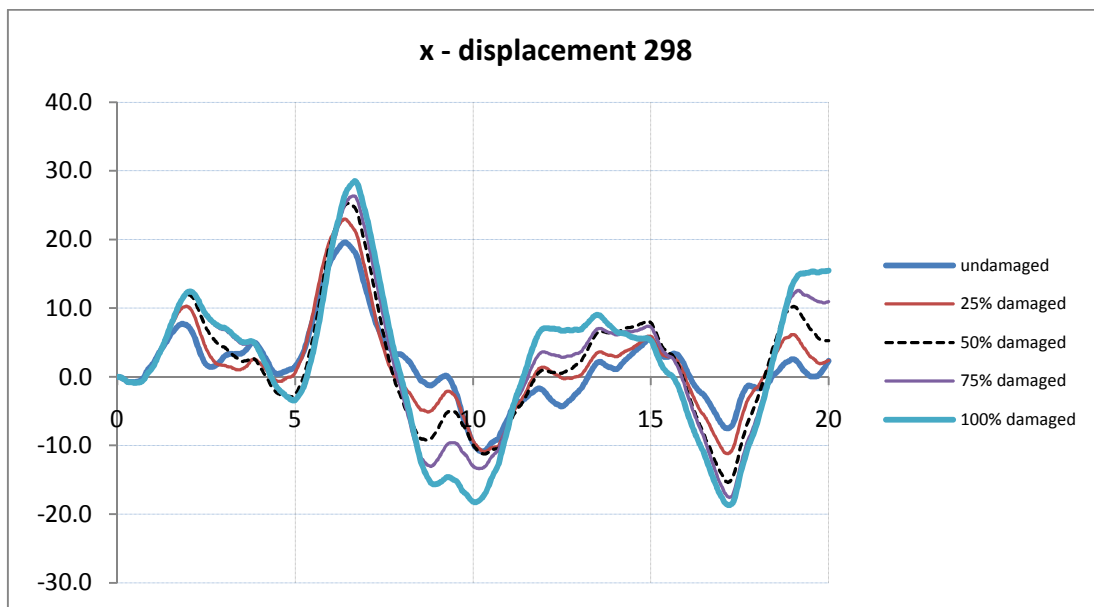


Figure 126. Longitudinal displ. midpoint main span at different dampers damages, strong white noise

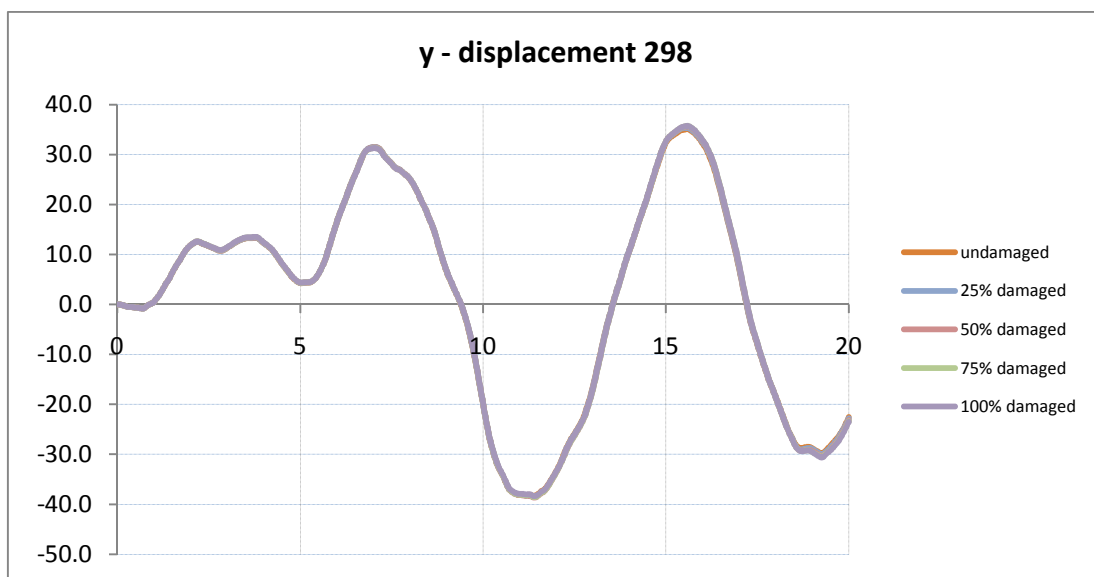


Figure 127. Transversal displ. midpoint main span at different dampers damages, strong white noise

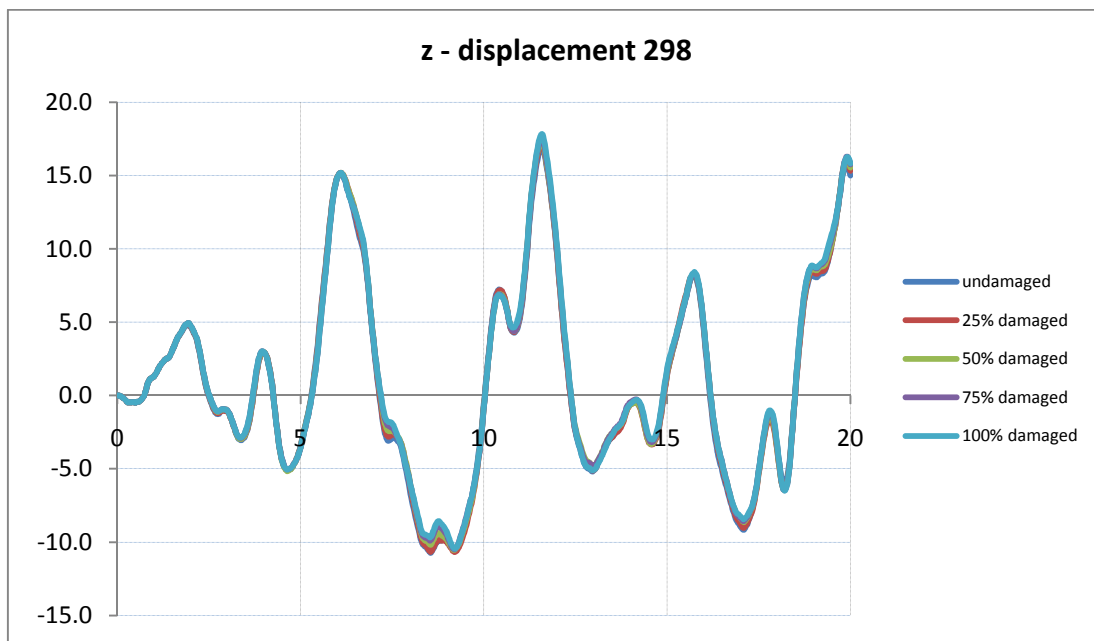


Figure 128. Vertical displ. midpoint main span at different dampers damages, strong white noise

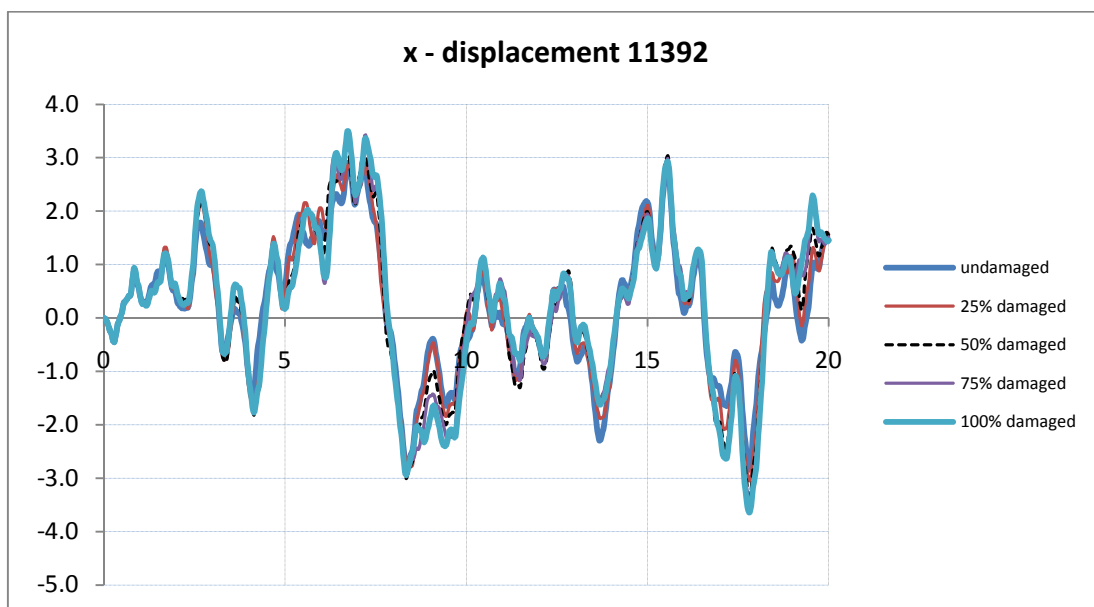


Figure 129. X displ. top tower at different dampers damages, strong white noise

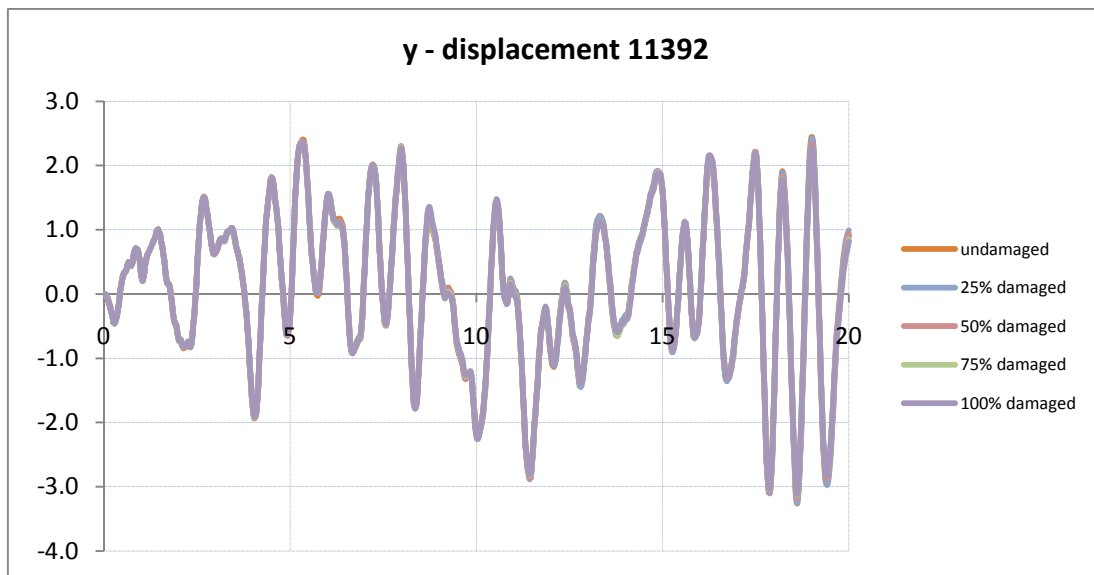


Figure 130. Y displ. top tower at different dampers damages, strong white noise

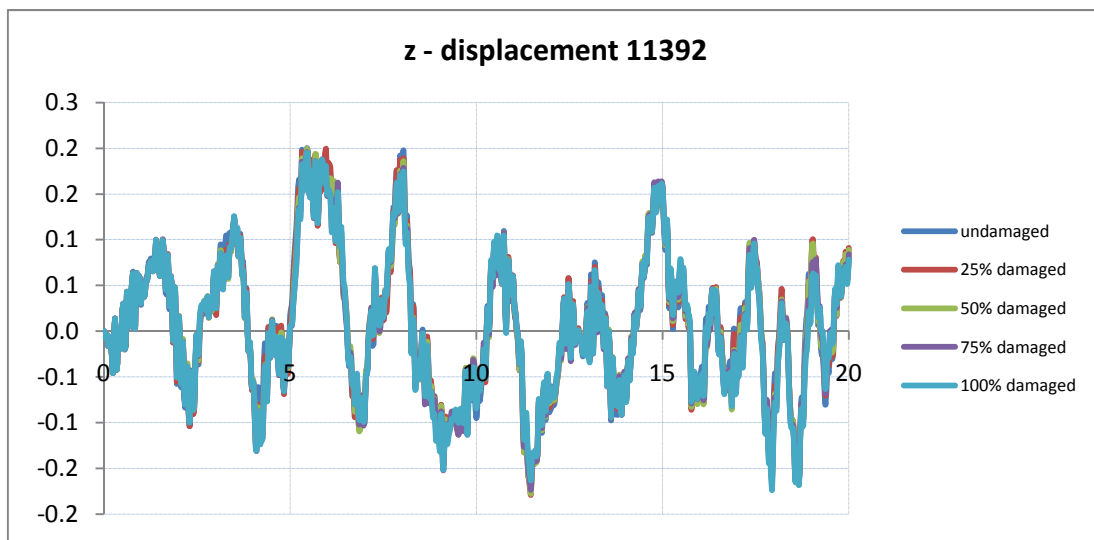


Figure 131. Z displ. top tower at different dampers damages, strong white noise

Is possible to note, as in the previous chapter, that the degradation of the dampers is felt just on the longitudinal direction of the bridge's deck.

Check dampers maximum forces

The damper forces have been calculated using the monitor element located next to the damper element. The forces were computed in the main span to tower dampers and in the side span to tower damper of both sides.

The results are summarized in next figures and table and commented after.

Table 12. Max damper forces symmetrical damage, strong white noise

DAMPER FORCES - ASYMMETRIC DAMAGE [kips]					
	0%	25%	50%	75%	100%
<i>MAIN TO TOWER - EAST SIDE</i>	101.4	159.3	213.5	158.3	33.3
<i>MAIN TO TOWER - WEST SIDE</i>	99.2	150.3	214.8	143.9	37.9
<i>SIDE TO TOWER - EAST SIDE</i>	25.2	66.8	70.8	54.4	12.1
<i>SIDE TO TOWER - WEST SIDE</i>	26.3	73.8	72.2	69.4	13.0

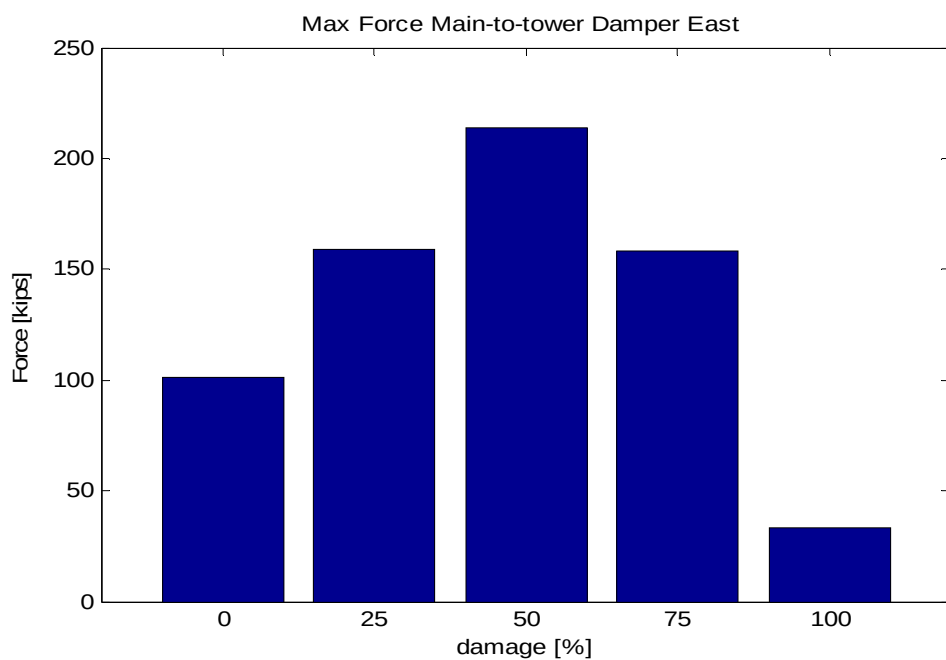


Figure 132. Peak force Main to tower damper East, strong white noise, sym. damage

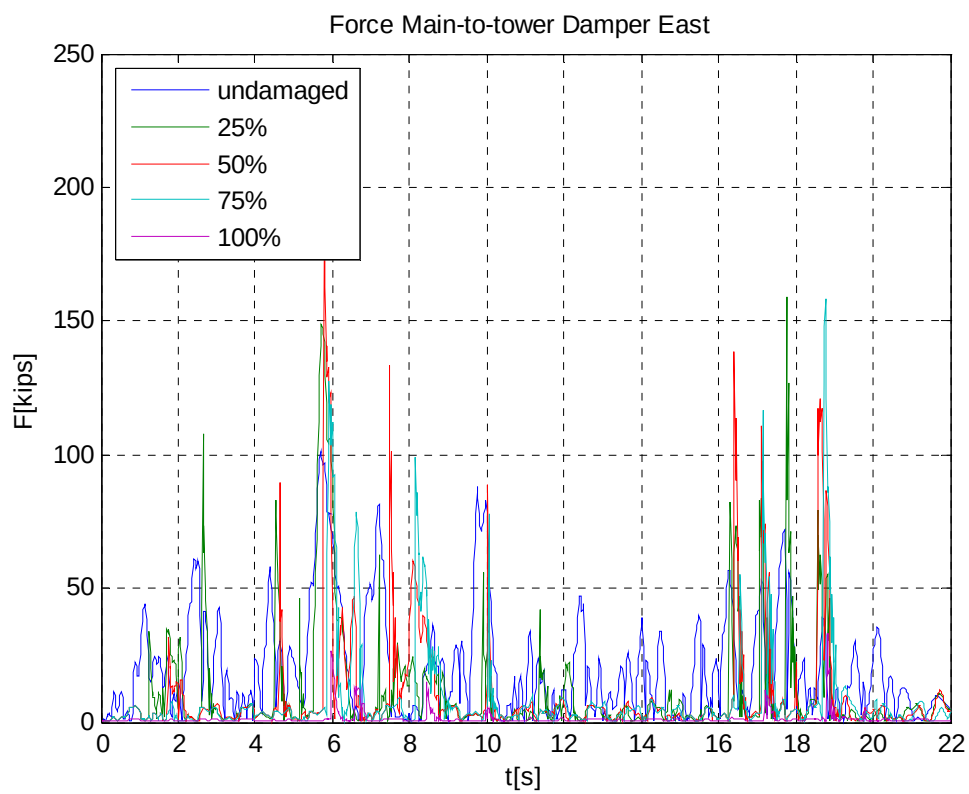


Figure 133. Time histories force Main to tower damper East, strong white noise, sym. damage

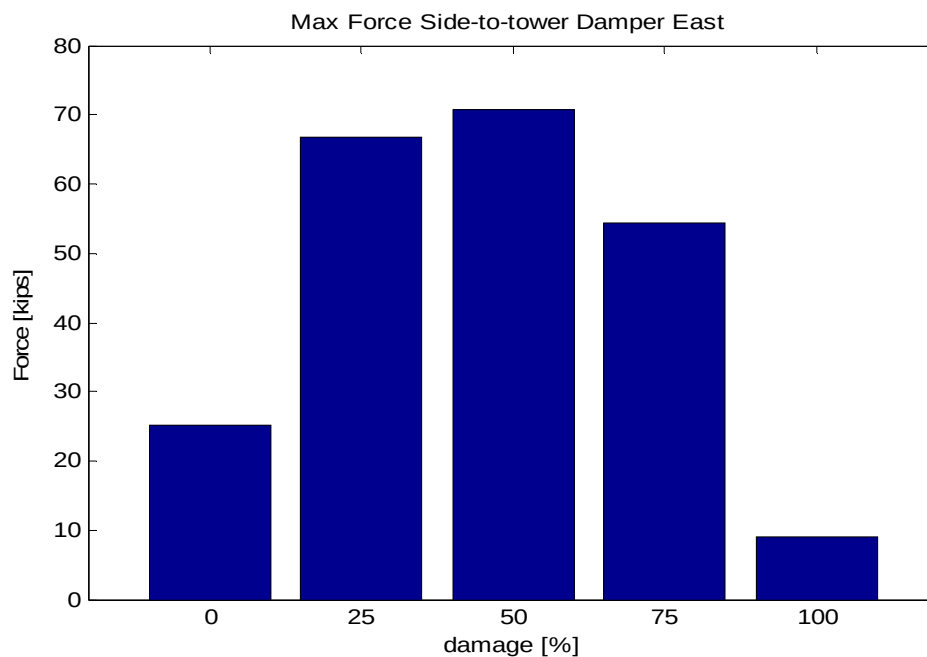


Figure 134. Peak force Side to tower damper East, strong white noise

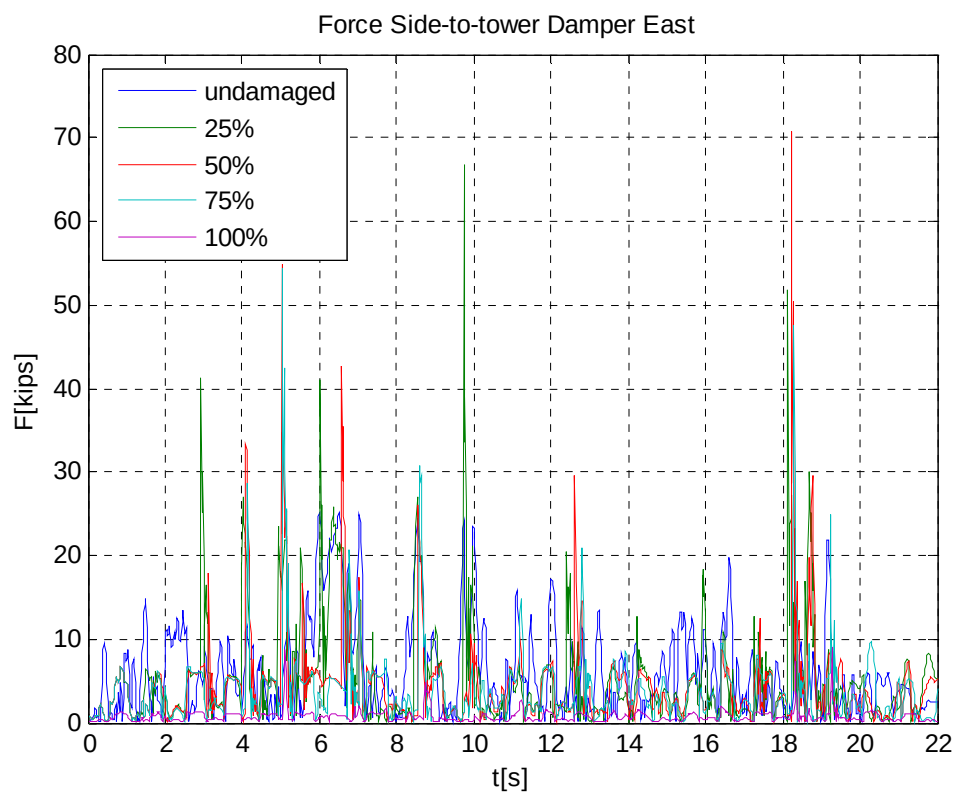


Figure 135. Time histories force Side to tower damper East, strong white noise, sym. damage

The results show that the peak force could be higher when a damper is damaged with a gap. The reason is that the shortening velocity of the damper could be higher while the movement of the deck is not contrasted by the viscous damping force. At the end of the gap the damper receives an impulsive force necessary to decelerate the structure. This phenomenon was not taken in account during the lab tests that were conducted in displacement control (constant angular velocity). Fig. 97 shows exactly what happen when the velocity is sufficient to create this unwanted peak.

Higher is the seismic input, stronger is the peck of the impulsive force in the dampers, in this case the force in side to tower damper with all the dampers 50% damaged is 280% the completely working damper's force.

Frequencies and mode shapes analysis

As seen in chapter 1 it has been performed an elastic modal analysis of the FEM model. An elastic modal analysis is not able to compute the influence of a velocity dependent stiffness in the structure (as concentrated dampers for example).

Following the frequencies computed by Matlab program.

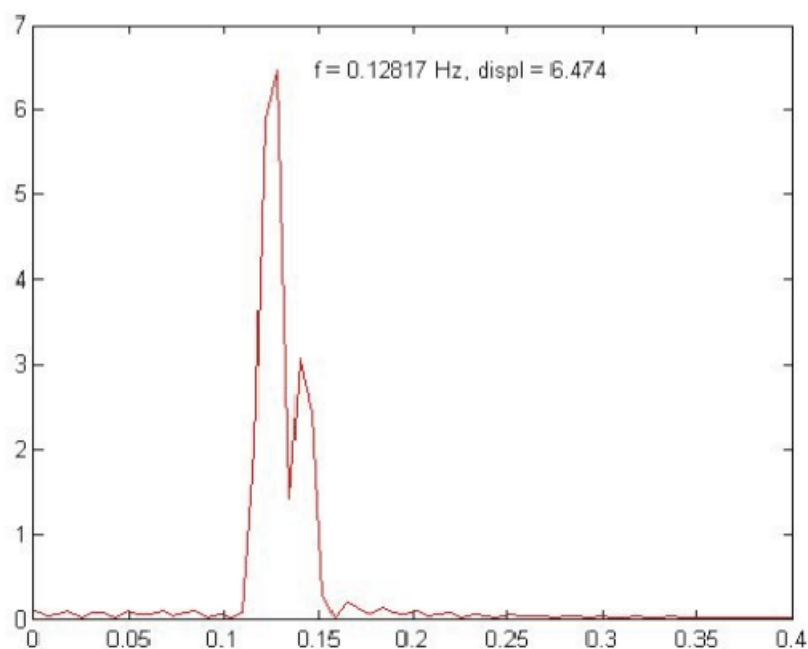


Figure 136. FFT point 298, y direction, filtered for mode 1, strong white noise

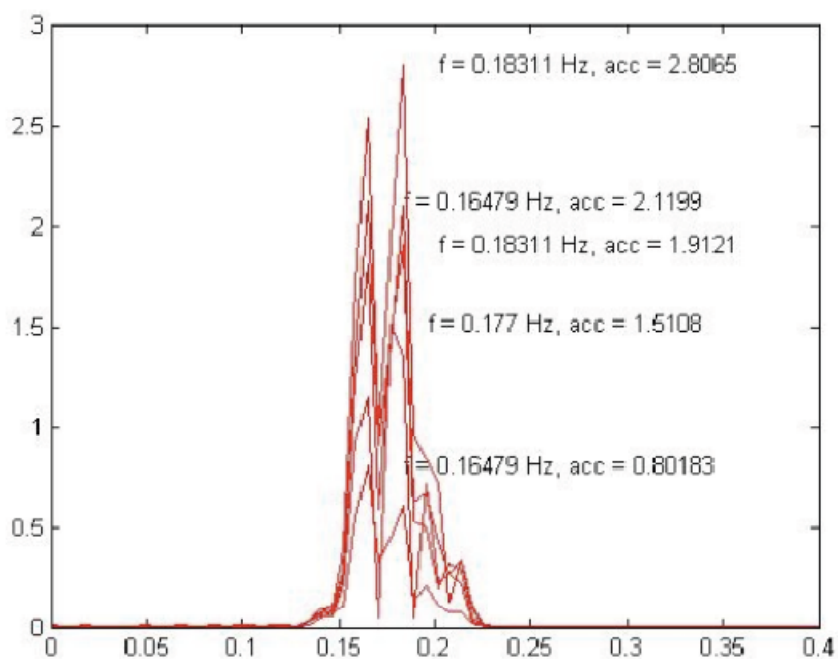


Figure 137. FFT point 298, y direction, filtered for mode 2, strong white noise

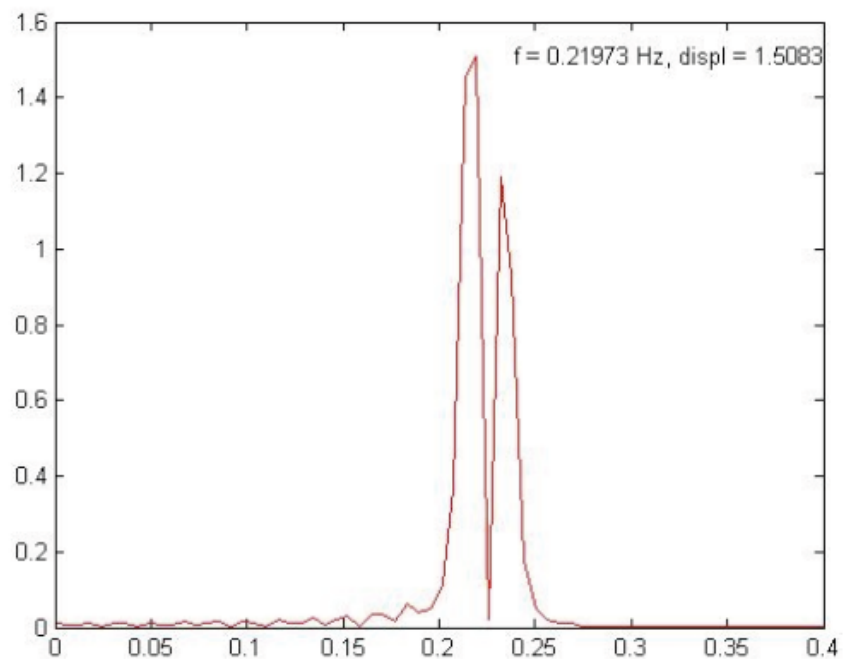


Figure 138. FFT point 298, z direction, filtered for mode 4, strong white noise

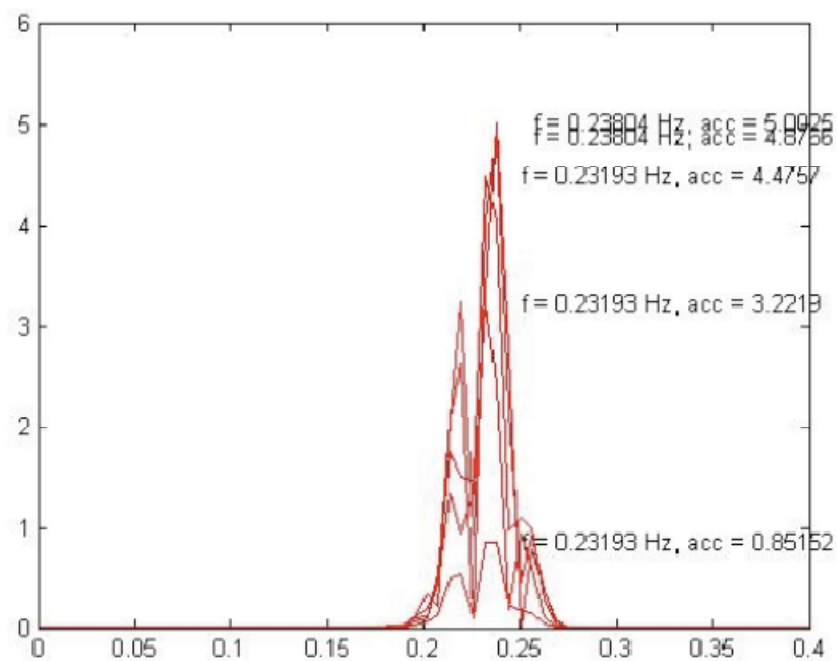


Figure 139. FFT point 298, x direction, filtered for mode 6, strong white noise

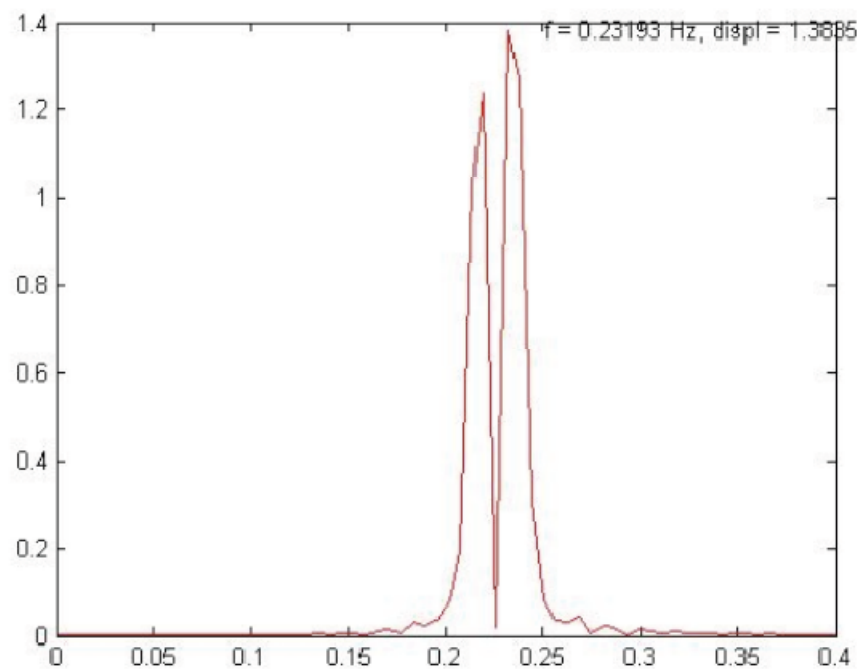


Figure 140. FFT point 298, z direction, filtered for mode 6, strong white noise

From the above charts is possible to appreciate a variation of the frequency belongs to the second mode long the longitudinal direction. The frequency belongs to the undamaged condition pass from 0.183Hz to 0.164Hz, is possible recognize a limited increase of the flexibility of the bridge. Consulting table 2 is possible to see that the non linear natural frequencies are very similar to the same ones computed with a modal analysis (as expected).

5.2.3 Parameterization and results of the analysis, Asymmetric Damage

The second parameterization has been conducted producing a progressive damage of the bridge's west side, equal to 25%, 50%, 75% and 100%, of two type of damper which connected the tower with the main and side spans, called MAIN TO TOWER and SIDE to TOWER dampers.

The damage by gap has been set just in the WEST side of the bridge, the gap's lengths are the same of the previous symmetric analysis.

Effect on structural displacements

The selected check points are the following:

- mid points of main span and side spans (298, 232, 2164);
- top of east tower (11392);
- based of the foundation's east tower (10201);
- anchorage element (7121).

From the analysis we obtained the follow results:

Table 13. Maximum displacements asymmetrical damage, strong white noise analysis

nodal point	nodal identification	<i>maximum x - displacement</i>				
		undamaged	25% damaged W	50% damaged W	75% damaged W	100% damaged W
232	mid side span E	8.84	8.50	8.5	8.36	8.36
298	mid main span	19.60	20.60	21.4	21.6	21.2
2164	mid side span W	8.60	8.90	9	9	9.4
7121	cable bent W	0.50	0.50	0.50	0.50	1.20
10201	base tower W	0.00	0.00	0.00	0.00	0.00
11392	top tower W	2.90	2.90	2.9	2.9	2.8

nodal point	nodal identification	undamaged	25% damaged W	50% damaged W	75% damaged W	100% damaged W
232	mid side span E	6.6	6.6	6.6	6.6	6.7
298	mid main span	38.3	38.3	38.4	38.5	38.4
2164	mid side span W	7.4	7.5	7.5	7.5	7.5
7121	cable bent W	0.9	0.9	0.90	0.9	0.9
10201	base tower W	0.00	0.00	0.00	0.00	0.00
11392	top tower W	3.2	3.2	3.2	3.2	3.2

<i>maximum z - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged W	50% damaged W	75% damaged W	100% damaged W
232	mid side span E	15.7	15.5	15.4	15.40	15.5
298	mid main span	17	16.8	16.9	16.90	16.8
2164	mid side span W	21.4	21.14	21	20.90	20.7
7121	cable bent W	0.33	0.33	0.33	0.33	0.33
10201	base tower W	0.00	0.00	0.00	0.00	0.00
11392	top tower W	0.20	0.20	0.20	0.20	0.20

The maximum displacement are very similar to the maximum displacement computed in the symmetrical analysis. The input results too low to create a difference in the global response between the various configurations.

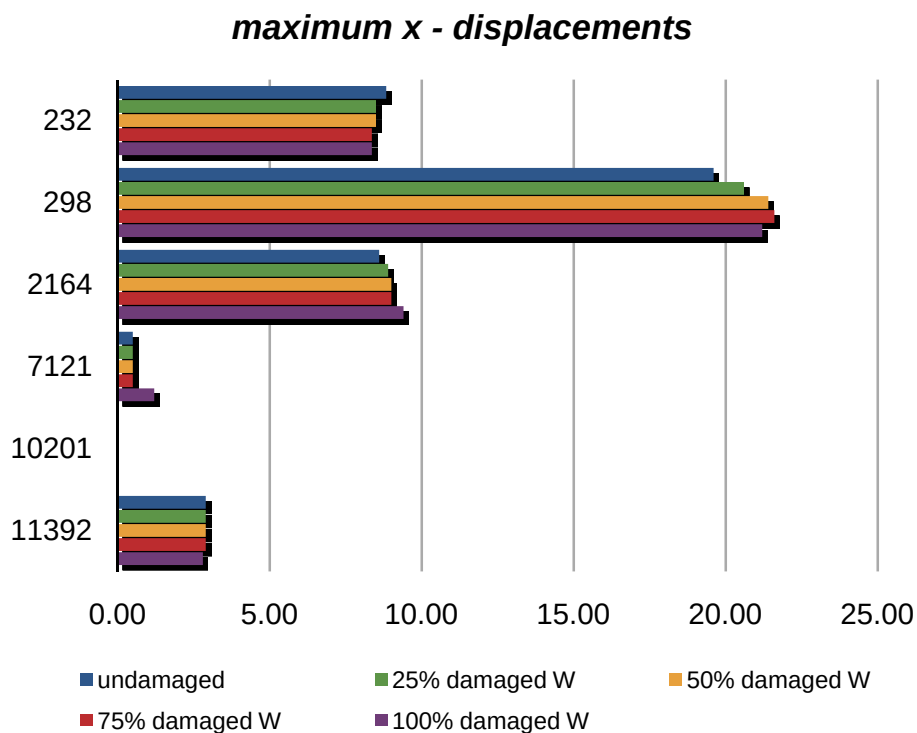


Figure 141. Maximum x-displacement, asymmetrical damage, strong w.n. analysis

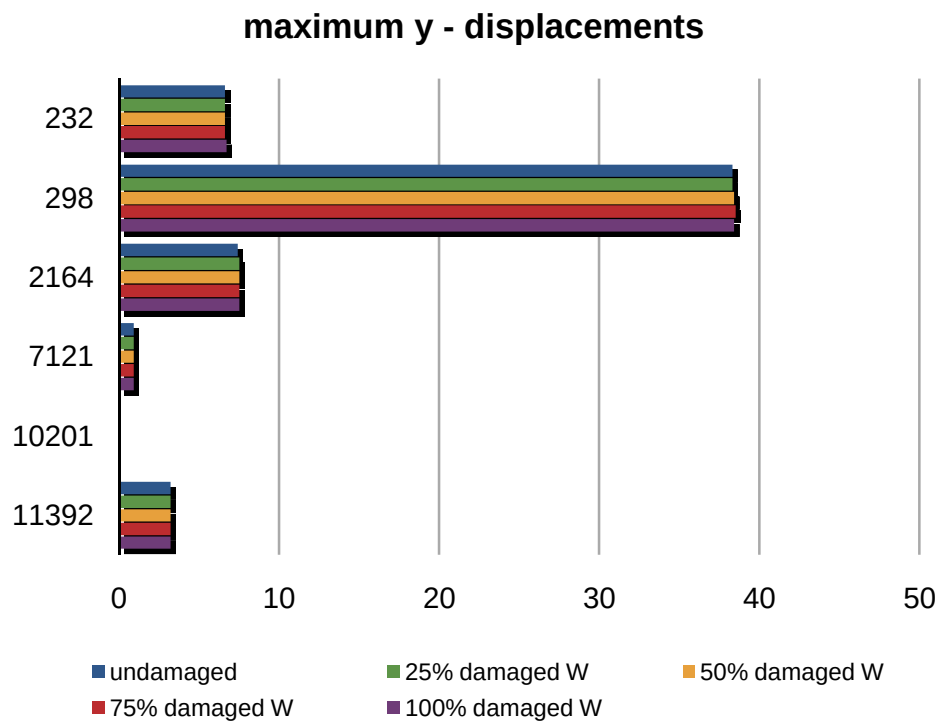


Figure 142. Maximum y-displacement, asymmetrical damage, strong w.n. analysis

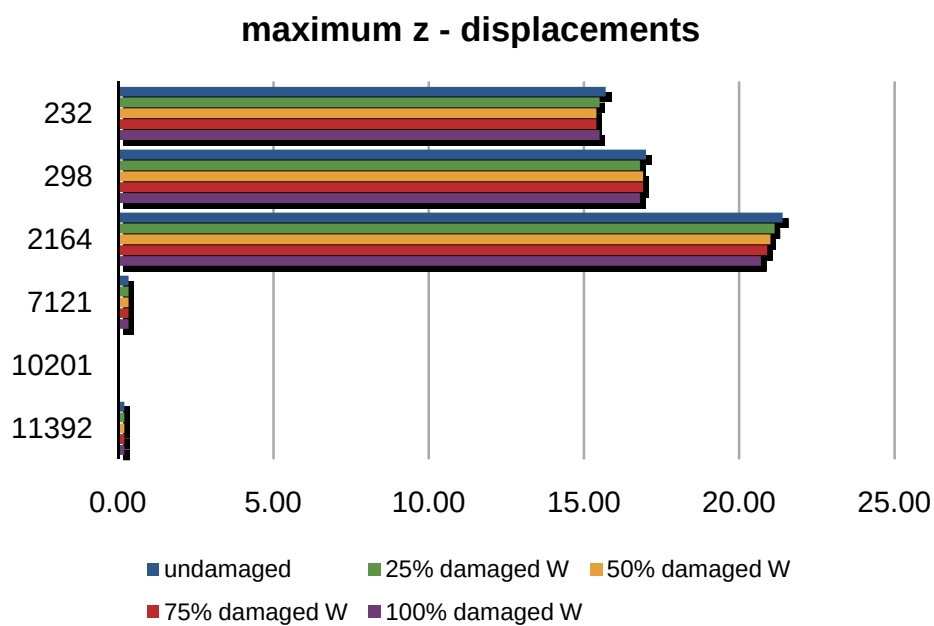


Figure 143. Maximum z-displacement, asymmetrical damage, strong w.n. analysis

Values of all the other global parameters controlled in section 4.2.2 appear to be similar.

Check dampers maximum forces

The damper forces have been calculated using the monitor element located next to the damper element. The forces were computed in the main span to tower dampers and in the side span to tower damper of both sides.

The results are summarized in next figures and table and commented after.

Table 14. Max damper forces asymmetrical damage

DAMPER FORCES - ASYMMETRIC DAMAGE [kips]					
	0%	25%	50%	75%	100%
<i>MAIN TO TOWER - EAST SIDE</i>	101.4	106.5	106.5	106.5	14
<i>MAIN TO TOWER - WEST SIDE</i>	99.2	103.8	76	59.9	25.1
<i>SIDE TO TOWER - EAST SIDE</i>	25.2	26.0	26.0	25.3	26.3
<i>SIDE TO TOWER - WEST SIDE</i>	26.3	27.3	21.7	22.0	14.6

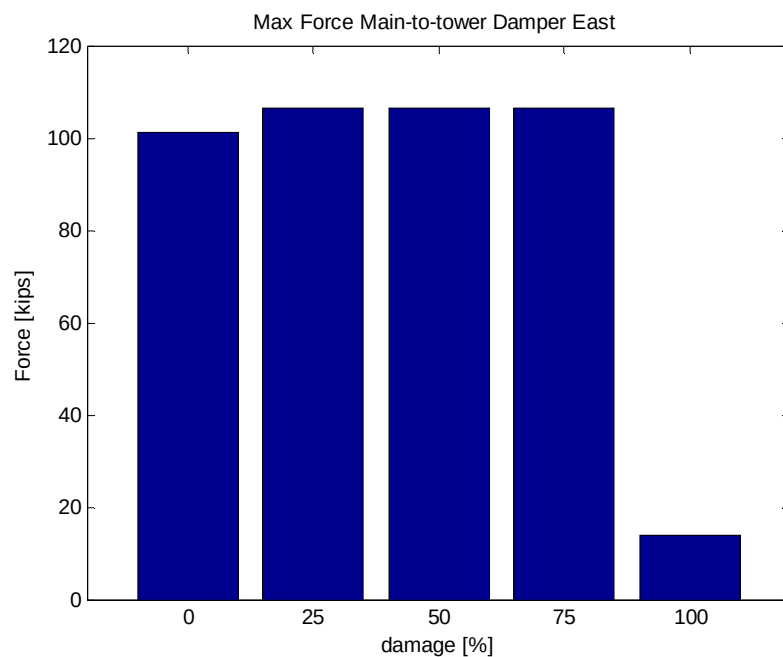


Figure 144. Peak force Main to tower undamaged damper East, strong w.n., asym. damage

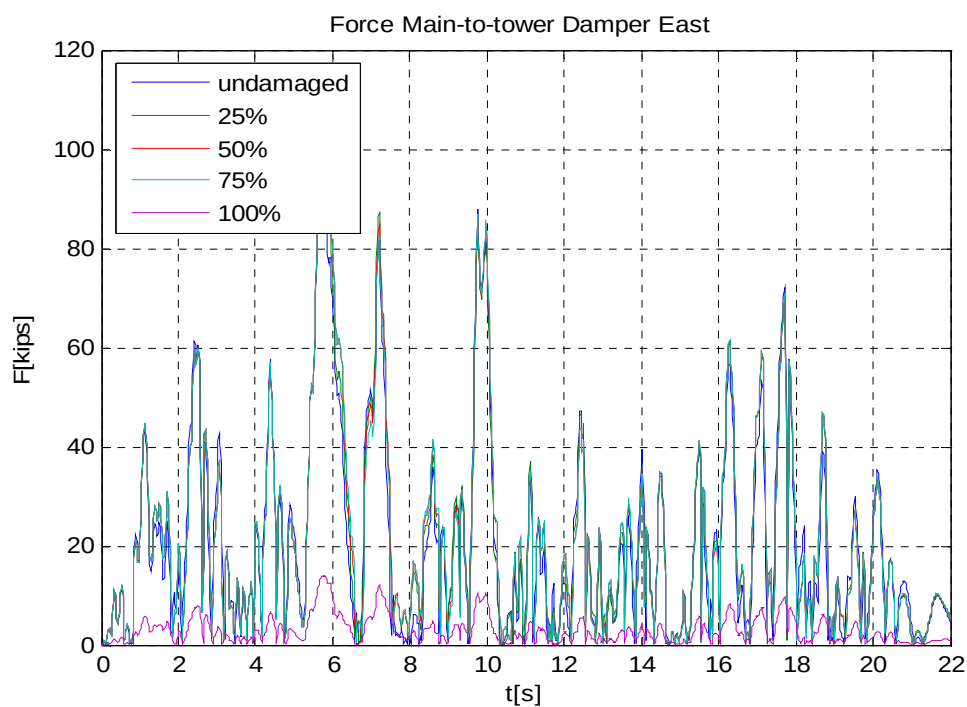


Figure 145. Time histories force Main to tower undamaged damper East, strong w.n., asym. damage

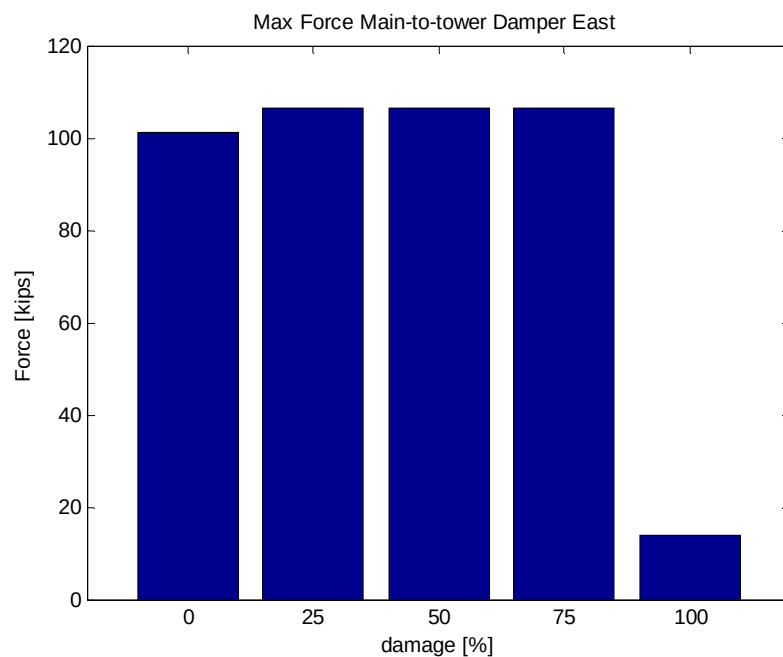


Figure 146. Peak force Main to tower damaged damper West , strong w.n., asym. damage

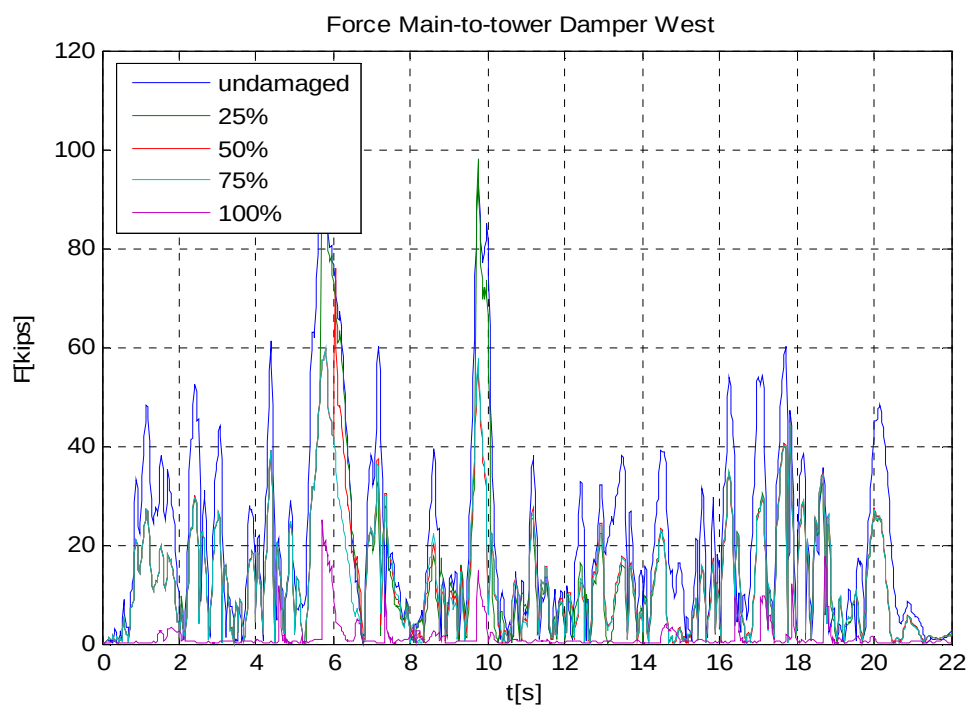


Figure 147. Time histories force Main to tower undamaged damper East, strong w.n., asym. damage

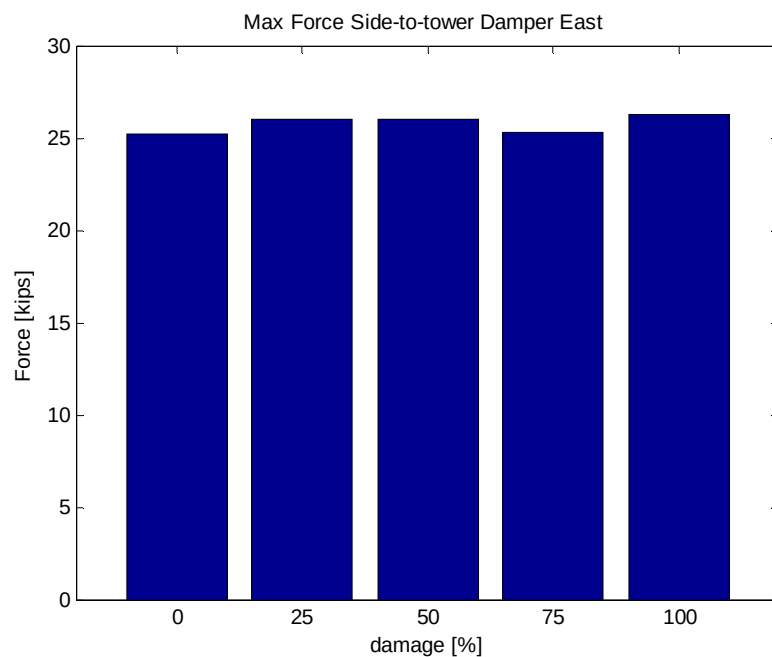


Figure 148. Peak force Side to tower undamaged damper East , strong w.n., asym. damage

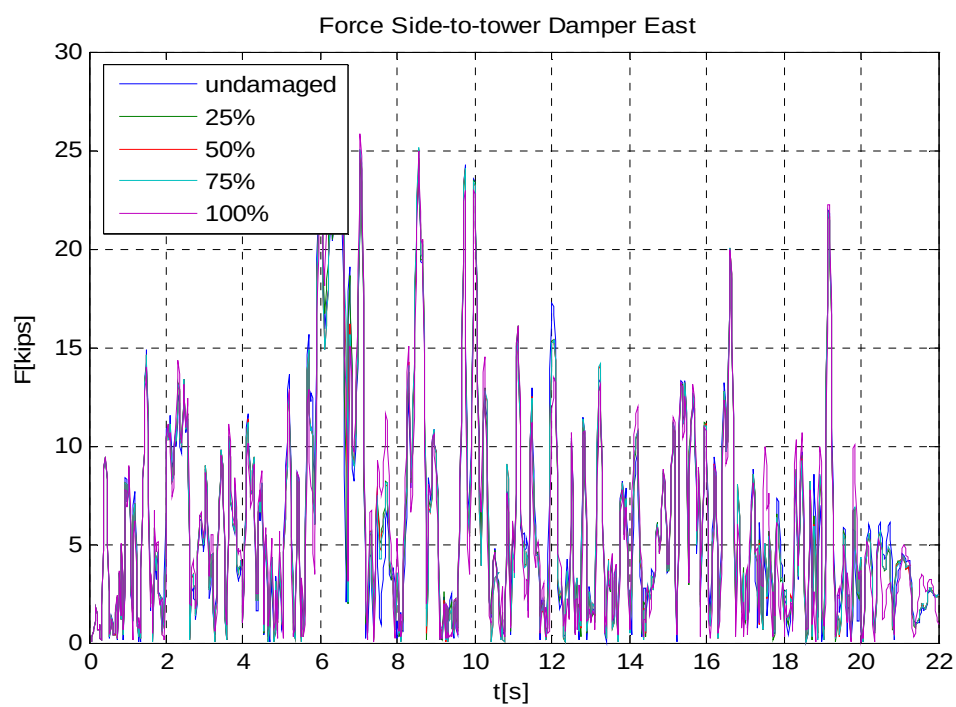


Figure 149. Time histories force Side to tower undamaged damper East, strong w.n., asym. damage

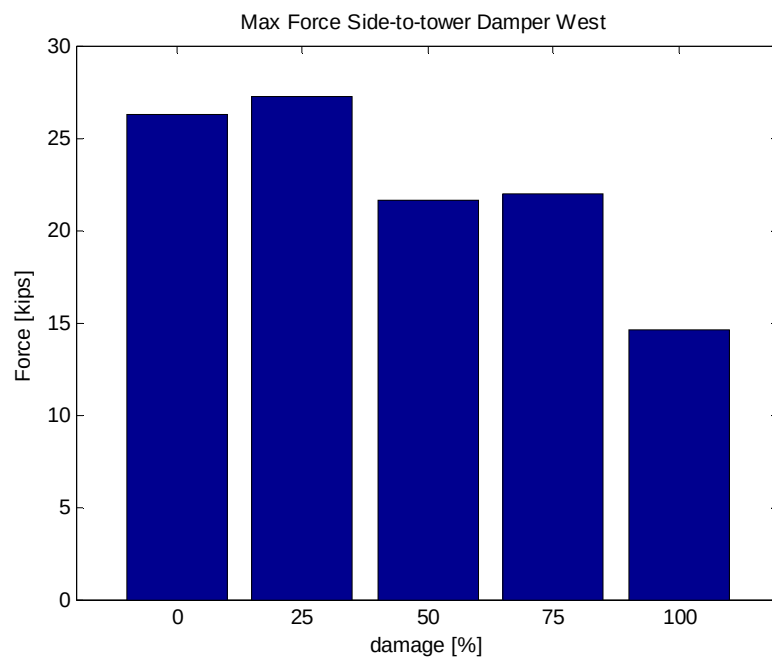


Figure 150. Peak force Side to tower damaged damper West , strong w.n., asym. damage

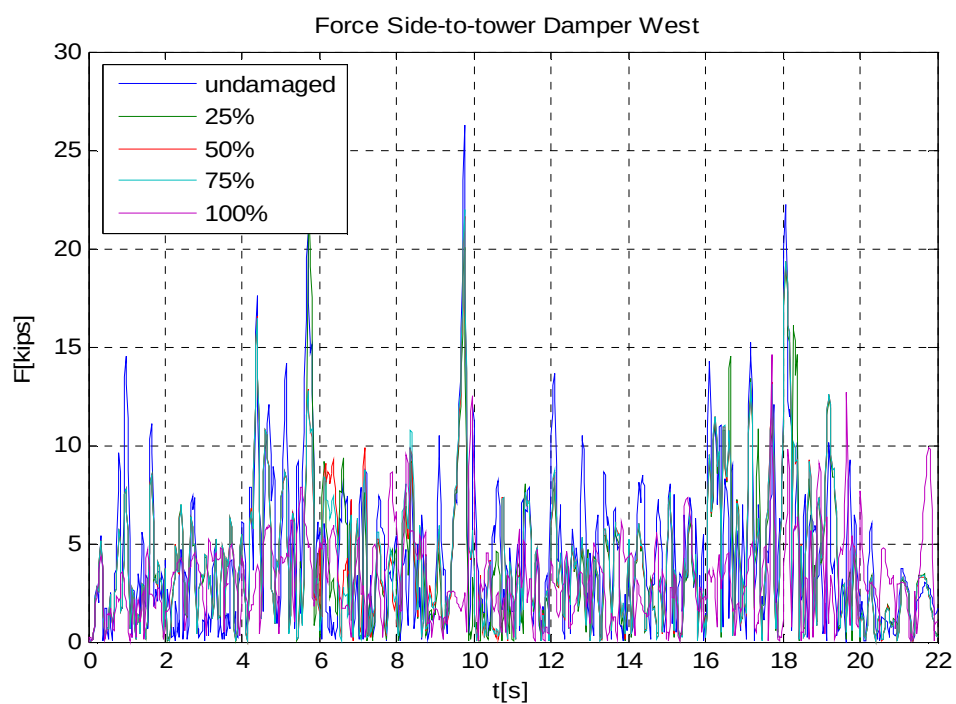


Figure 151. Time histories force Side to tower undamaged damper East, strong w.n., asym. damage

Results show that during ambient vibration if the damage is localized just in one side of the bridge, the undamaged damper on the other side of the deck are able to damp the movement of the decks (main and side ones) and prevent the peak force phenomenon.

Frequencies and global behavior of the structure do not change.

5.2 Recorded Northridge seismic event analysis

5.2.1 Introduction

The instrumentation present on the bridge recorded the ground motion acceleration and the response at particular points of the structure (see ch. 4.1) during 1994 Northridge earthquake ($M=6.7$).

The distance from the epicenter was approximately 60 km, as shown in fig. 152.

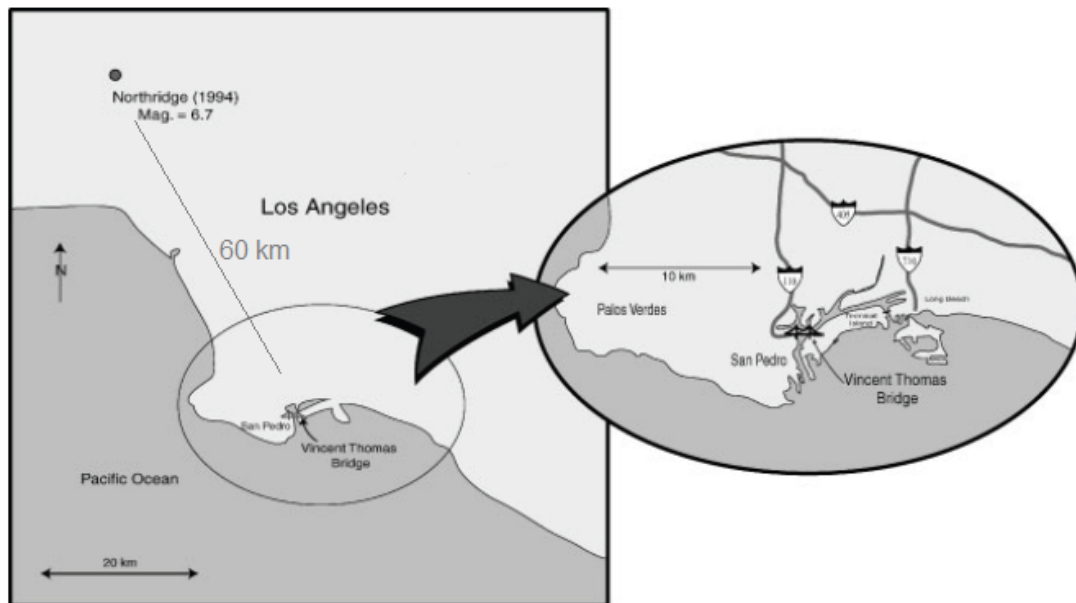


Figure 152. Location of the Vincent Thomas Bridge with respect to the 1994 Northridge earthquake

In order to simulate that particular event the acceleration time history measured at the bases of the columns was applied to numerical model. That data have slightly higher values of acceleration compared to the nearest USC recording station situated in Terminal Island (# 82). Next figures shows first 20 seconds of acceleration time history

and pseudo acceleration spectrum of the direction of higher values of acceleration (X direction, sensor # 13).

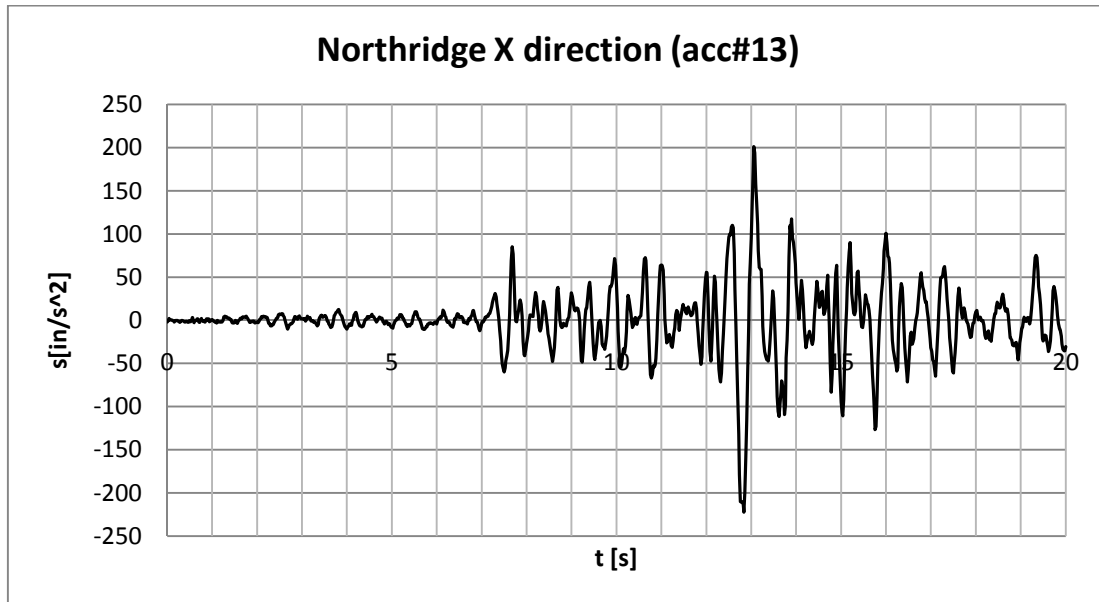


Figure 153. Northridge input time-history X

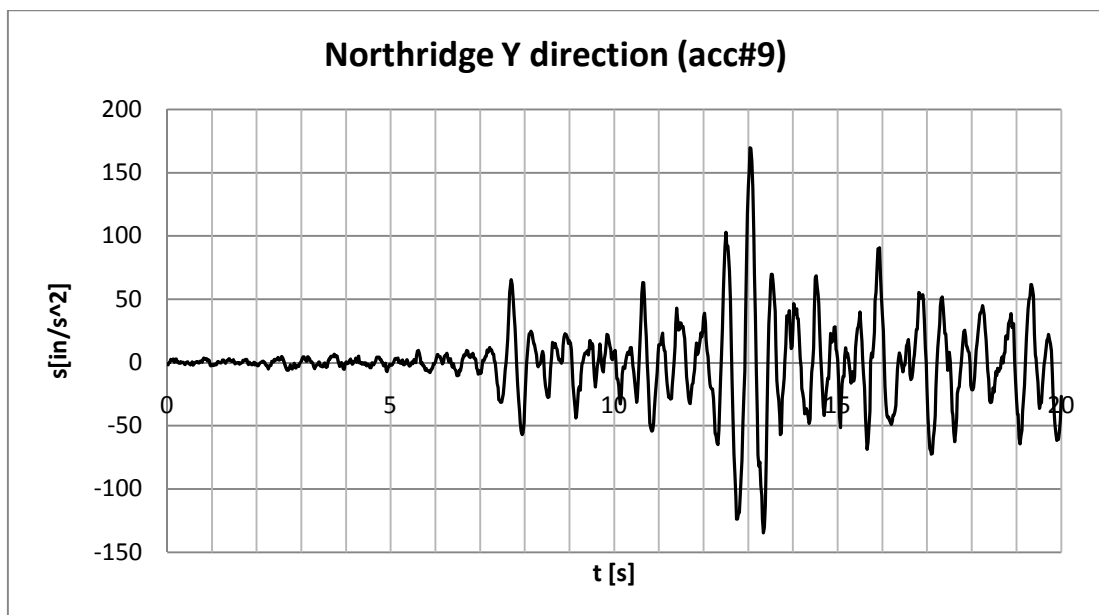


Figure 154. Northridge input time-history Y

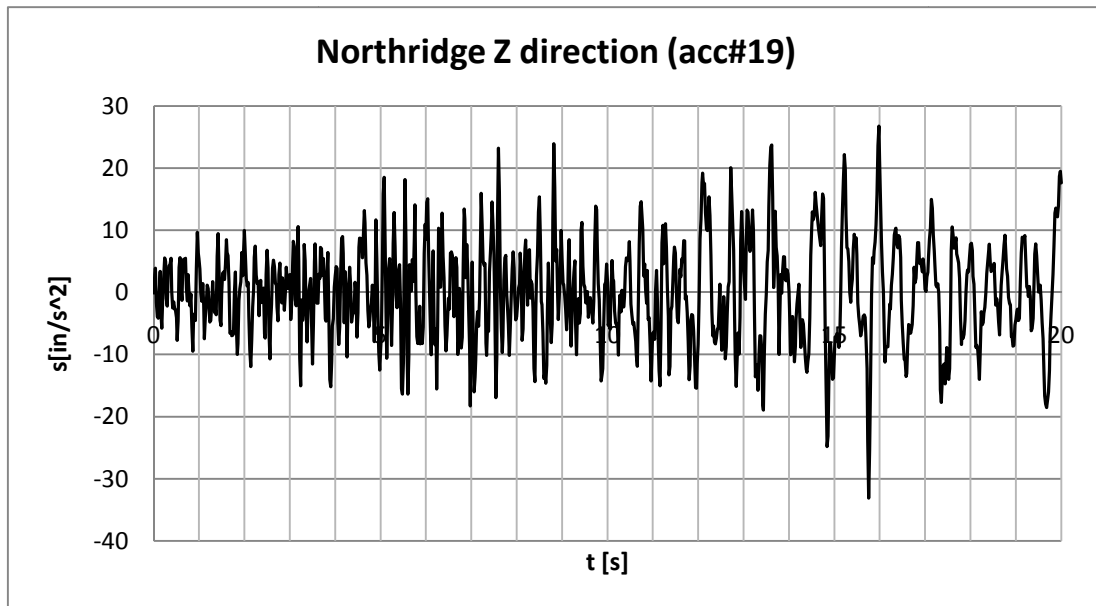


Figure 155. Northridge input time-history Z

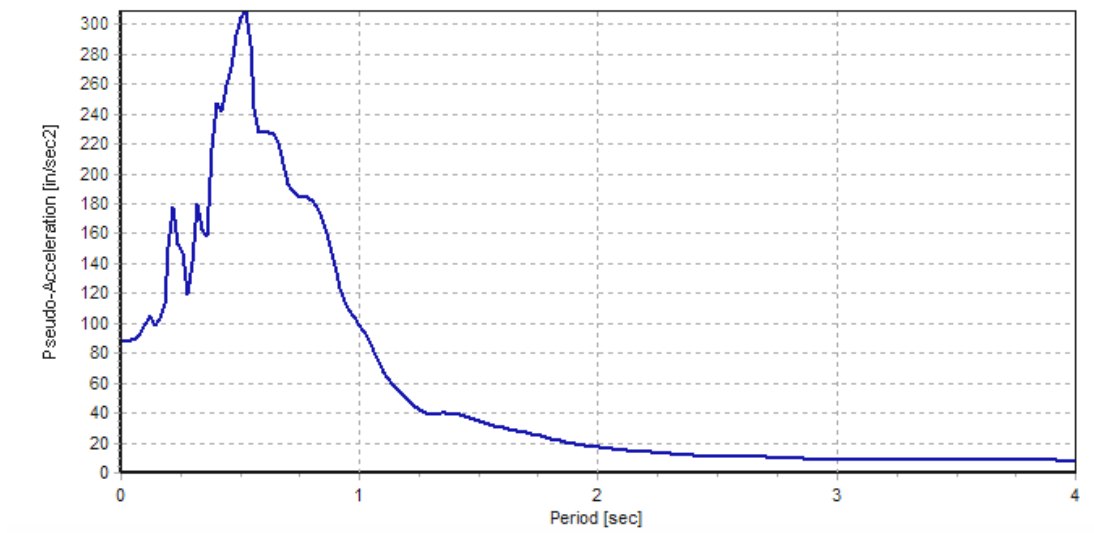


Figure 156. Pseudo-acceleration spectra (5% damping) of Northridge input time-history X

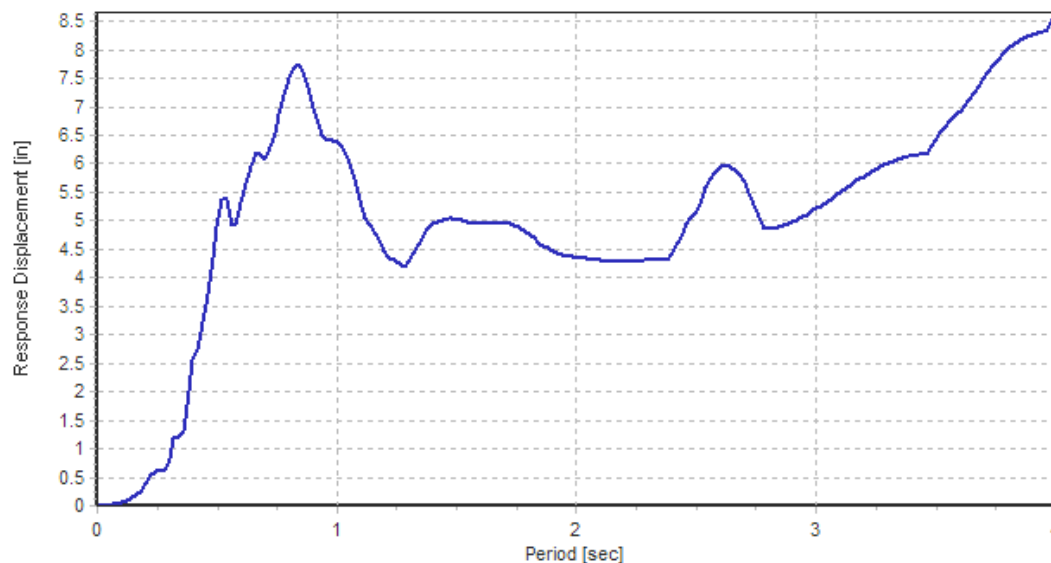


Figure 157. Displacement spectra (5% damping) of Northridge input time-history X

5.2.2 Parameterization and results of the analysis, Symmetric Damage

As seen before the damage in dampers could be a gap in their force -deformation response (see ch. 3). Artificial gaps were inserted in the model in order to simulate these damages. The first parameterization has been conducted producing a progressive damage, equal to 25%, 50%, 75% and 100% of the maximum operative gap. Maximum gap is considered equal to the maximum relative displacement of two extremes points of a particular damper. The structural response of a 100% damaged damper is so equal to the response of a non working damper (C and $K = 0$).

The damages were considered in two types of dampers. In particular the ones connected the tower with the main and side spans, called MAIN TO TOWER and SIDE TO TOWER dampers.

In the next figure is showed, as an example, the maximum relative displacement in main to tower damaged dampers during simulated Northridge earthquake. An analysis with a full damaged damper was conducted. Maximum elongation obtained is 4.6". Other calculated reference gaps are: 1.2" (25%), 2.3" (50%), 3.5" (75%).

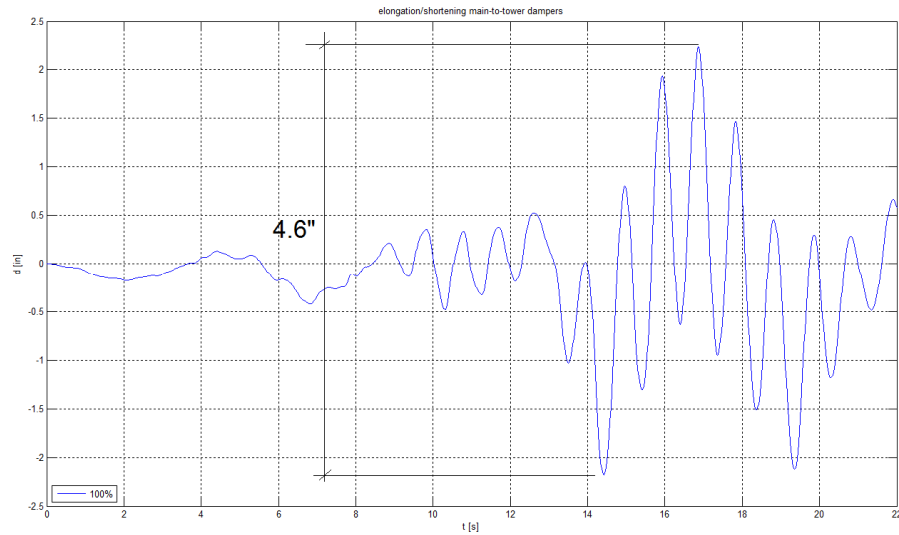


Figure 158. Working range of main to tower damper during Northridge earthquake

The same thing was calculated for side to tower. The maximum relative displacement is 2.2" and consequently calculated gaps are: 0.6" (25%), 1.1" (50%), 1.7" (75%).

Effect on structural displacements

The selected check points are the same controlled in ambient vibration analysis (fig. 86):

- mid points of main span and side spans (298, 232, 2164);
- top of east tower (11392);
- foundation base east tower (10201);
- anchorage element (7121).

In the graphs represented in next figures are represented the absolute maximum displacements of the check points at different level of damage (gap length).

Table 15. Maximum displacements symmetrical damage

<i>maximum x - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	1.1158	1.1965	1.2517	1.2521	1.1556
298	mid main span	0.6991	0.7956	0.9867	1.075	1.1092
2164	mid side span W	0.959	1.0213	1.0372	1.0345	1.0595
7121	cable bent W	0.1278	0.1278	0.1284	0.1288	0.1288
10201	base tower W	0.0006	0.001	0.0012	0.0013	0.0014
11392	top tower W	0.5574	0.5897	0.5689	0.5844	0.5937
<i>maximum y - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	0.5793	0.588	0.5806	0.587	0.5853
298	mid main span	1.7296	1.7348	1.7442	1.7512	1.7501
2164	mid side span W	0.5692	0.5701	0.5738	0.5726	0.576
7121	cable bent W	0.1688	0.1688	0.1688	0.1688	0.1687
10201	base tower W	0.0015	0.0015	0.0015	0.0015	0.0015
11392	top tower W	0.7949	0.793	0.793	0.793	0.7924
<i>maximum z - displacement</i>						
nodal point	nodal identification	undamaged	25% damaged	50% damaged	75% damaged	100% damaged
232	mid side span E	0.615	0.5882	0.626	0.6265	0.5829
298	mid main span	0.8169	0.8531	0.8561	0.853	0.8417
2164	mid side span W	0.6725	0.7321	0.7521	0.7298	0.7659
7121	cable bent W	0.1046	0.1047	0.1049	0.1049	0.1048
10201	base tower W	0.0002	0.0002	0.0002	0.0002	0.0002
11392	top tower W	0.0263	0.0253	0.025	0.025	0.0255

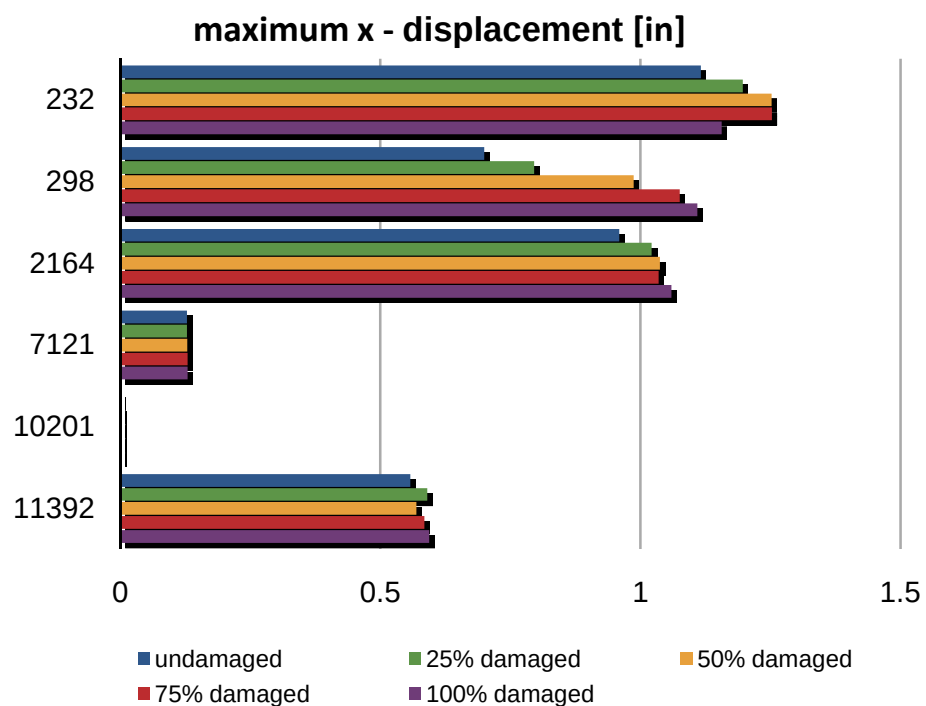


Figure 159. Maximum displacement in x direction at different level of damage

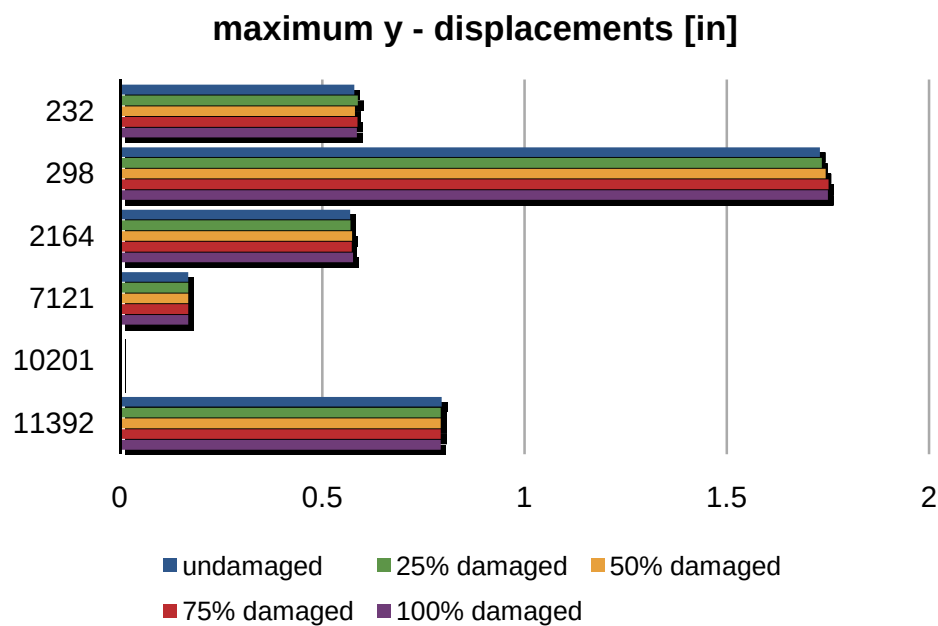


Figure 160. Maximum displacement in y direction at different level of damage

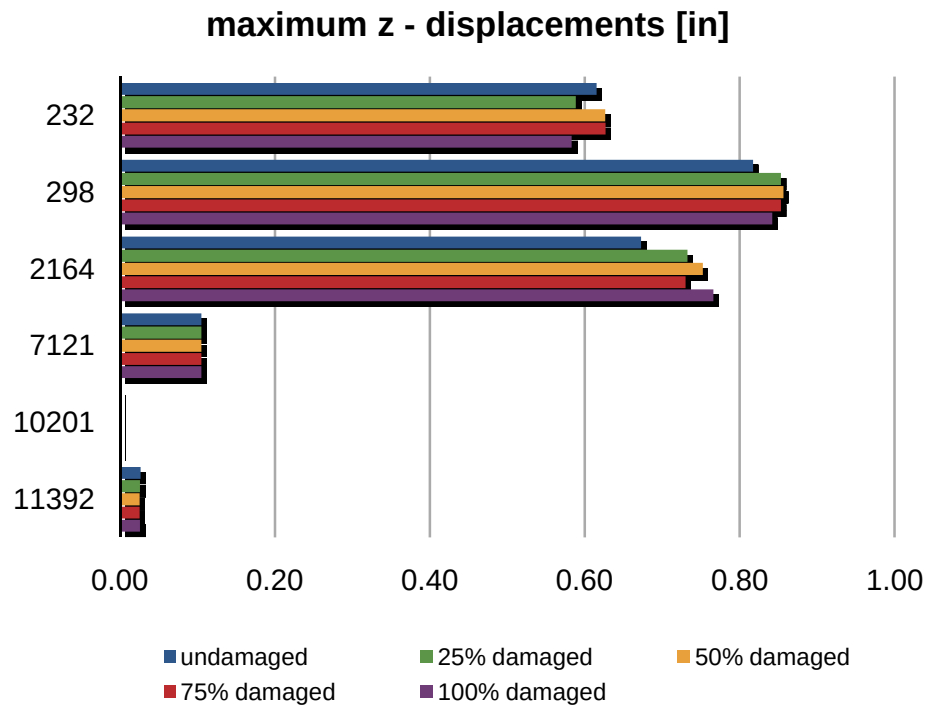


Figure 161. Maximum displacement in z direction at different level of damage

From the graphs is possible to see that the only consistent variation is in the longitudinal displacement of central mid-span. The presence of a gap in all the dampers increase up to 60% the range of displacement during an event as Northridge earthquake.

An increment of 15% has been calculated in the vertical displacement of side-span. The maximum top tower displacement seems not to be influenced (max 5% of variation) of by the presence of a damage in the dampers.

The following time histories represent the displacement at different level of damage. As we can see in the next plots the significant difference in displacement due to damages in dampers are only along x direction. Small differences can be seen in vertical

displacement of mid secondary span (point #232) and in the x displacement of the top of the towers (point #11392).

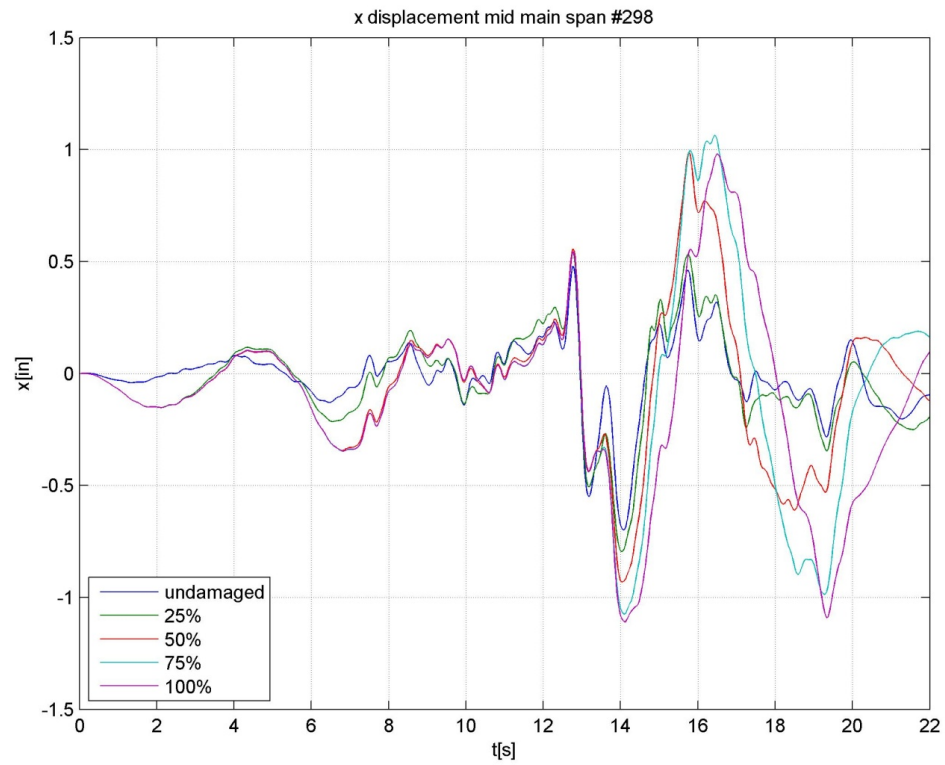


Figure 162. Time histories of displacement in x direction of mid main span

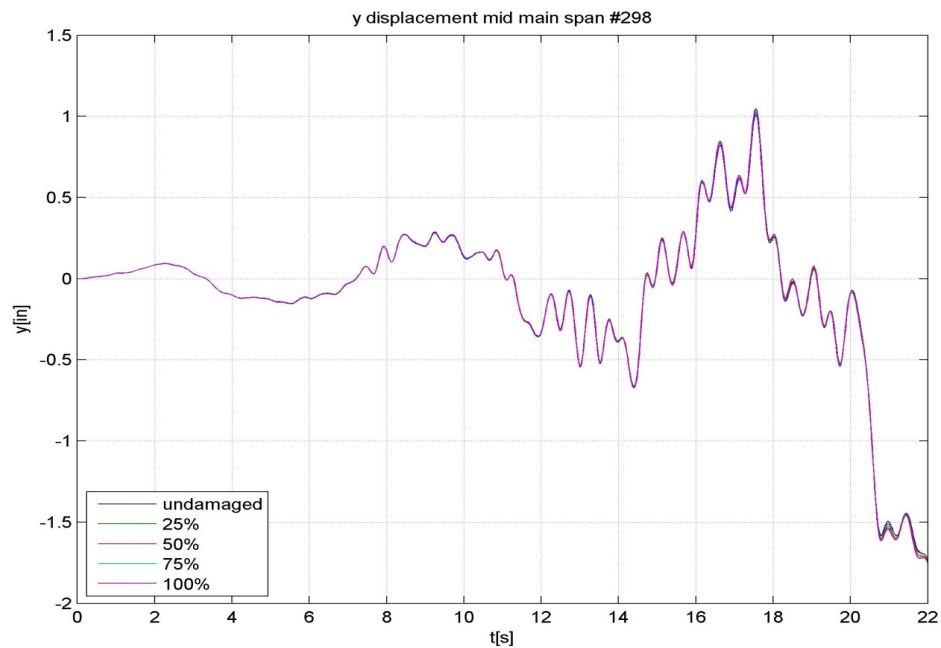


Figure 163. Time histories of displacement in y direction of mid main span

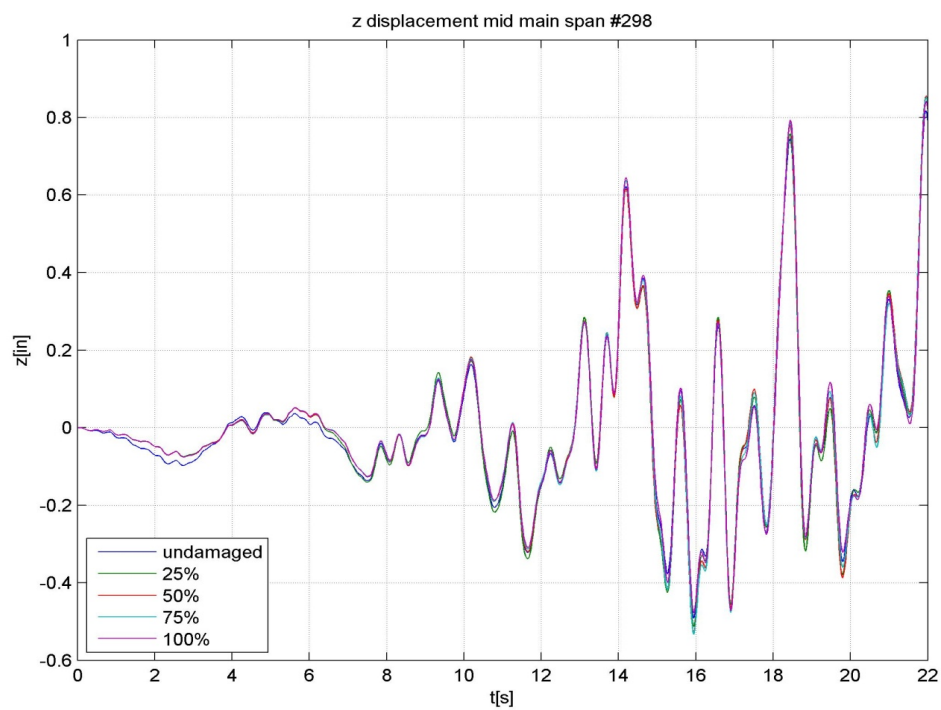


Figure 164. Time histories of displacement in z direction of mid main span

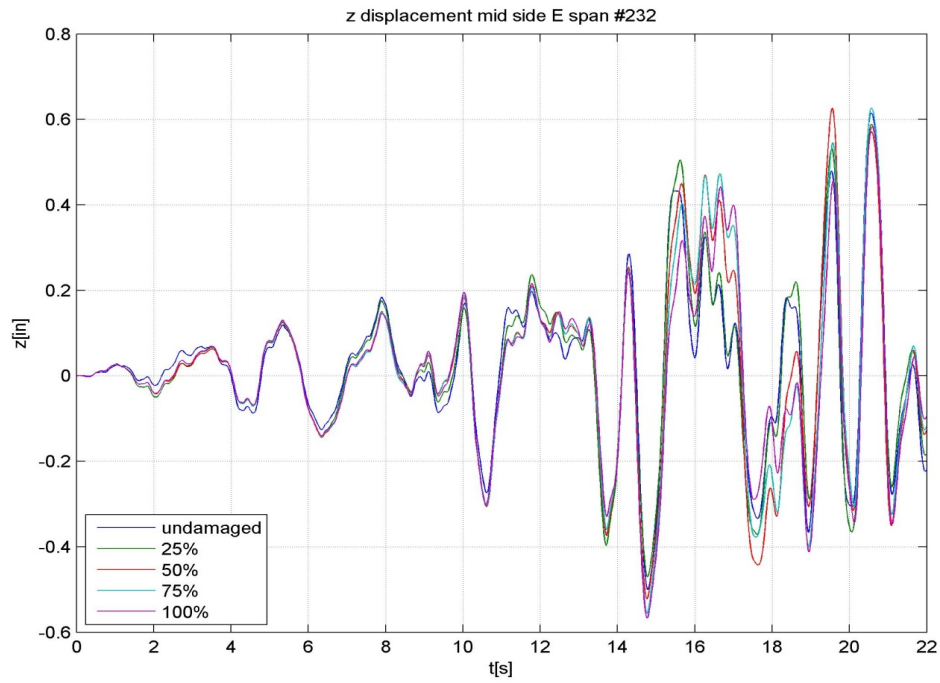


Figure 165. Time histories of displacement in z direction of mid side span

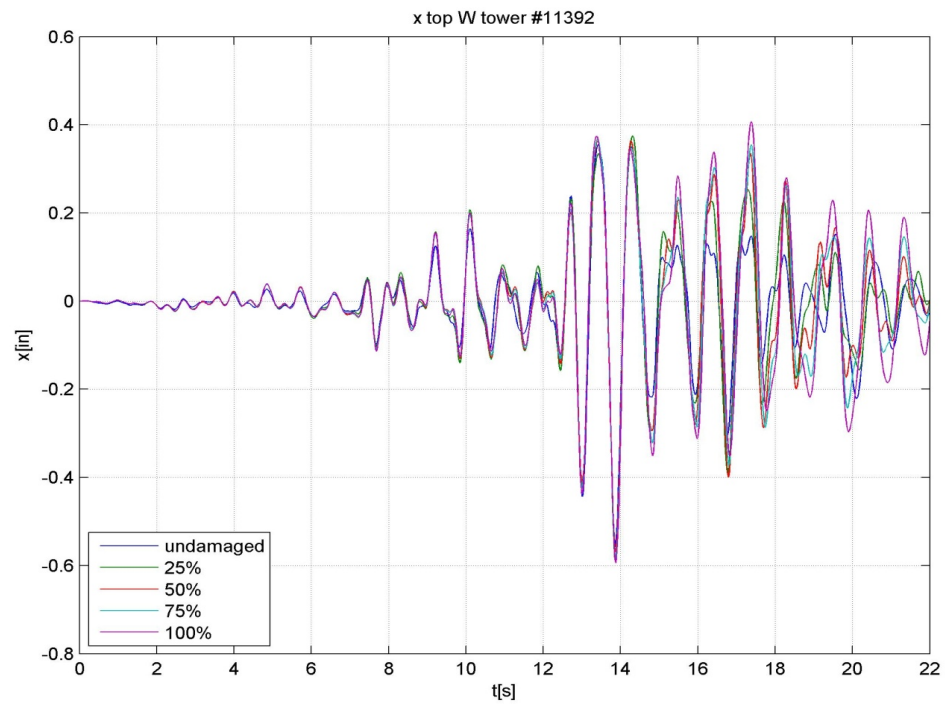


Figure 166. Time histories of displacement in x direction of top tower

A fundamental consideration is that the maximum displacements of various points of the bridge due to a nondestructive event as Northridge earthquake have the same magnitude of displacements due to normal use of the structure (white noise). The greater registered (or calculated) increment append in x-displacement of mid main span (from 0.42" to 1.1" for 100% damaged).

Check dampers deformations

The elongation/shortening of the two types of dampers were calculated by the relative displacements of the extreme nodes:

- MAIN to TOWER - nodes : 250 and 10144;
- SIDE to TOWER - nodes: 1848 and 10341.

In the next plots is possible to see that there is a significant increase when a gap damage is present.

In particular the full main to tower damper deformation is 4.5 times the deformation of the same undamaged damper. The deformation of damaged side to tower damper is around 2 times the undamaged one.

Time histories of the elongation/shortening of two types of damper considered are now plotted:

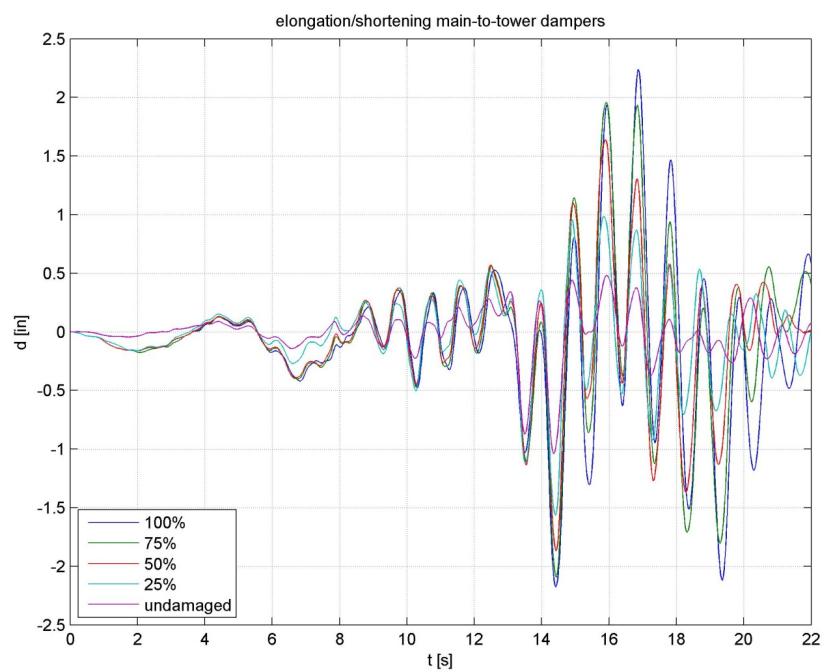


Figure 167. Time histories of deformation main-to-tower damper with different damages

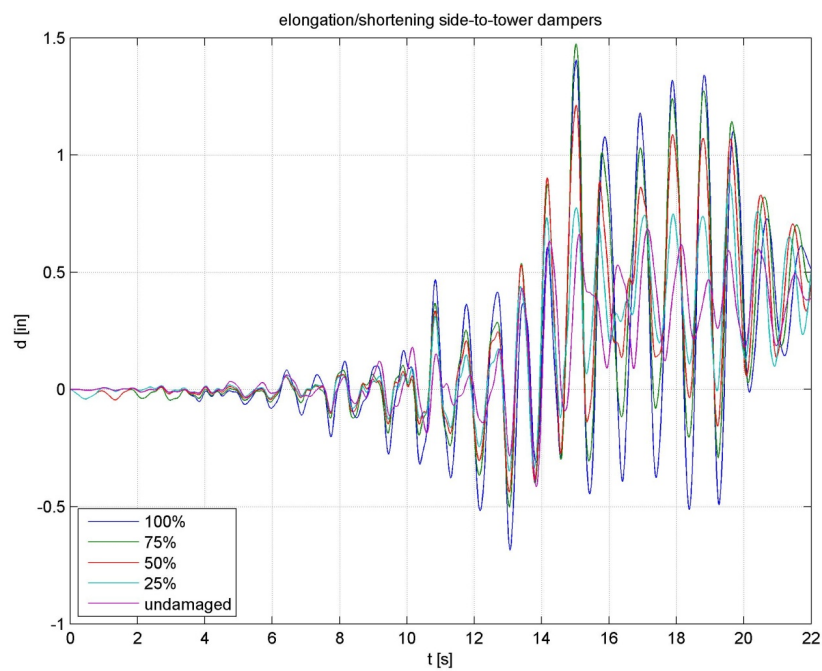


Figure 168. Time histories of deformation side-to-tower damper with different damages

Following graphs represent the maximum elongation (or shortening) of the two types of damper (with different level of damage). The diagonal line represent the gap length so the red part of the bar identify the effective working stroke of each damper. Is possible to see how the stroke (and consequently the energy dissipated) reduces when the gap increase:

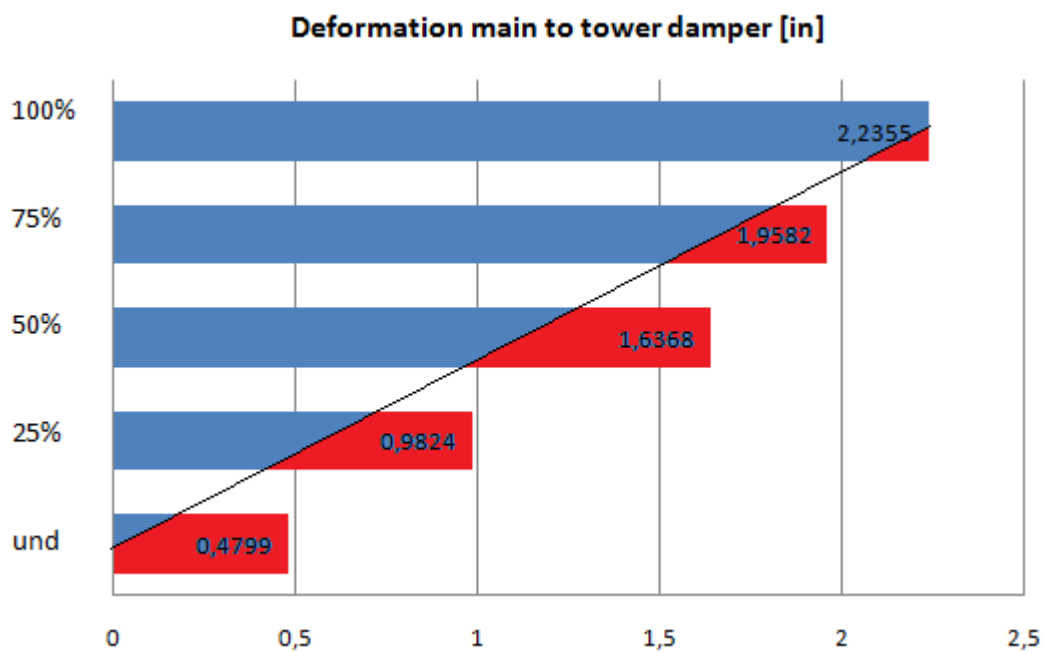
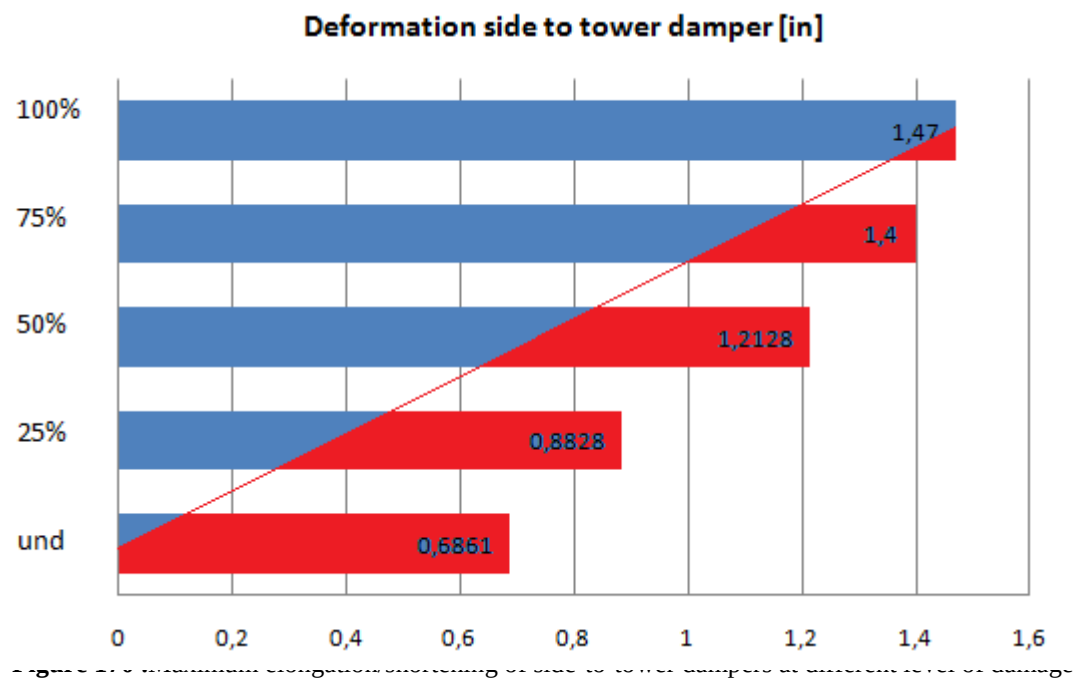


Figure 169. Maximum elongation/shortening of main-to-tower dampers at different level of damage



A discussion over the real benefits of the type of retrofit adopted for this structure could be open. The dampers are working for several cycles during normal operation of the bridge. A non destructive event as Northridge excites the dampers more than normal traffic condition. Anyway, considering the Adina model as a valid one the dampers shortening during Northridge event appears to be 0.68". This small deformation create a very small variation in the energy absorption of the entire structure, with the results that these retrofit doesn't work very well for normal traffic condition and non destructive earthquake.

Check dampers maximum forces

The damper forces have been calculated using the monitor element located next to the damper element. The forces were computed in the main span to tower dampers and in the side span to tower damper of both sides.

The results are summarized in next figures and table and commented after.

Table 16. Max damper forces symmetrical damage, Northridge

DAMPER FORCES - ASYMMETRIC DAMAGE [kips]					
	0%	25%	50%	75%	100%
<i>MAIN TO TOWER - EAST SIDE</i>	22.3	43.9	30.9	22.8	2
<i>MAIN TO TOWER - WEST SIDE</i>	21	36.7	32.7	35.9	1.5
<i>SIDE TO TOWER - EAST SIDE</i>	10.3	11.6	16.3	12.7	1
<i>SIDE TO TOWER - WEST SIDE</i>	10.6	16.7	20.7	20.7	1.5

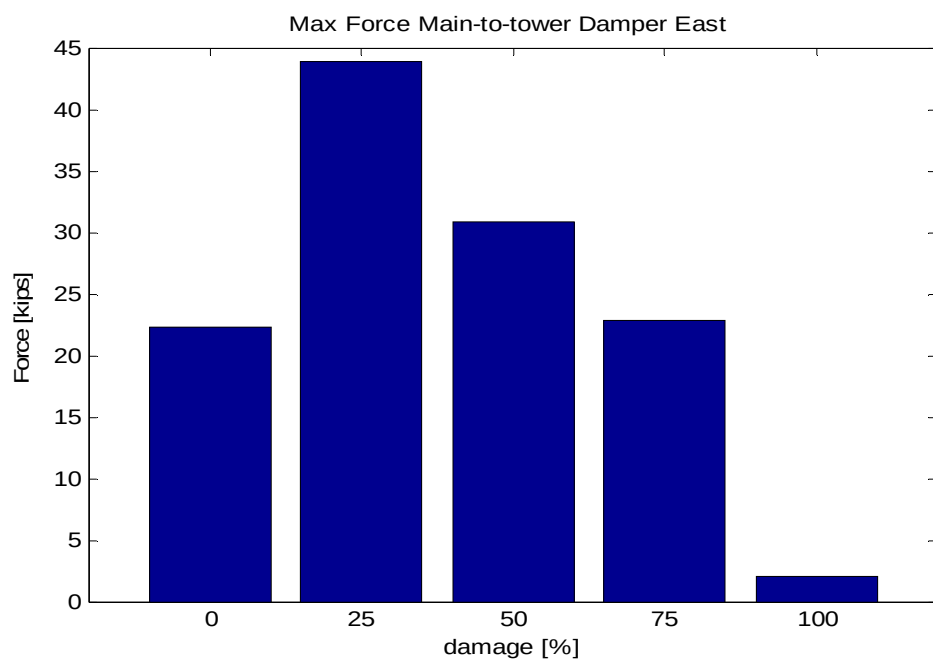


Figure 171. Peak force Main to tower damper East, Northridge, sym. damage

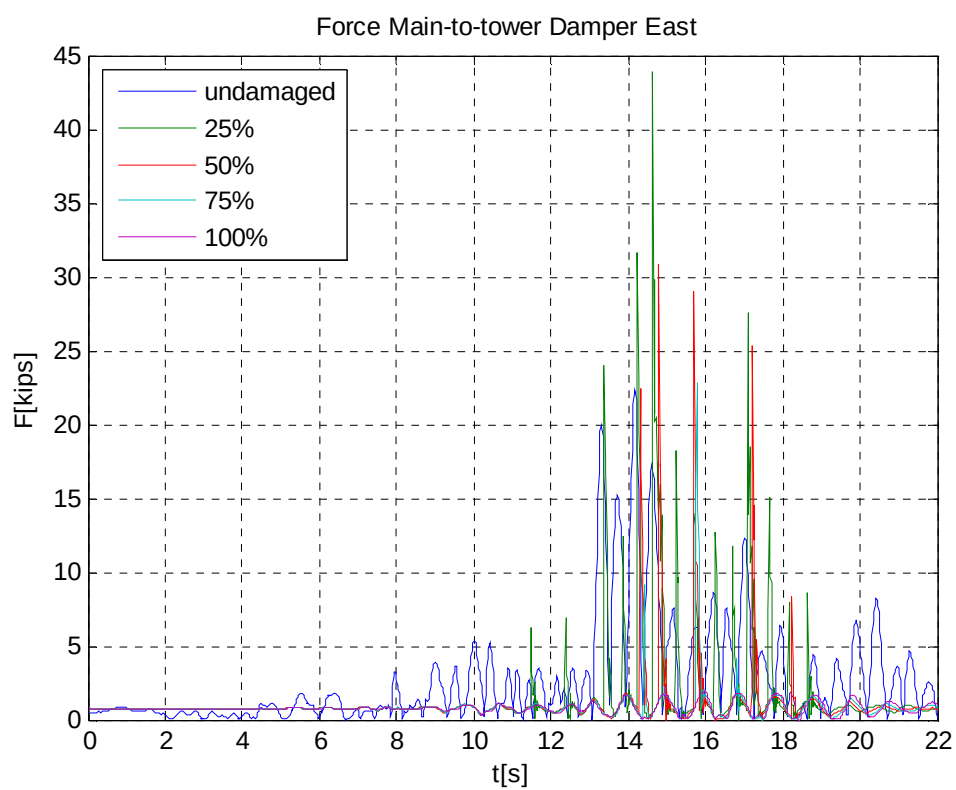


Figure 172. Time histories force Main to tower damper East, Northridge, sym. damage

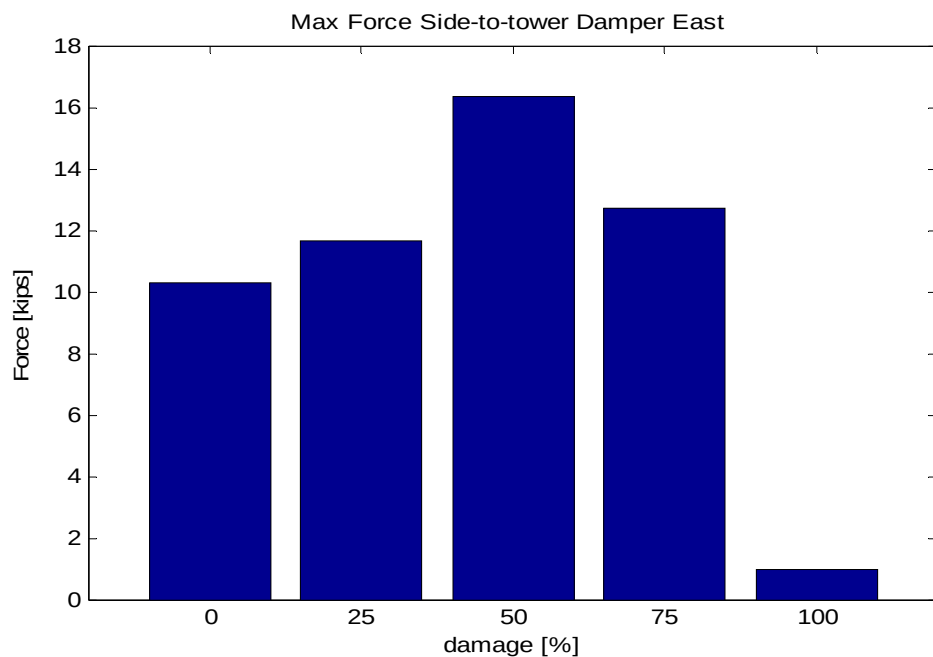


Figure 173. Peak force Side to tower damper East, Northridge

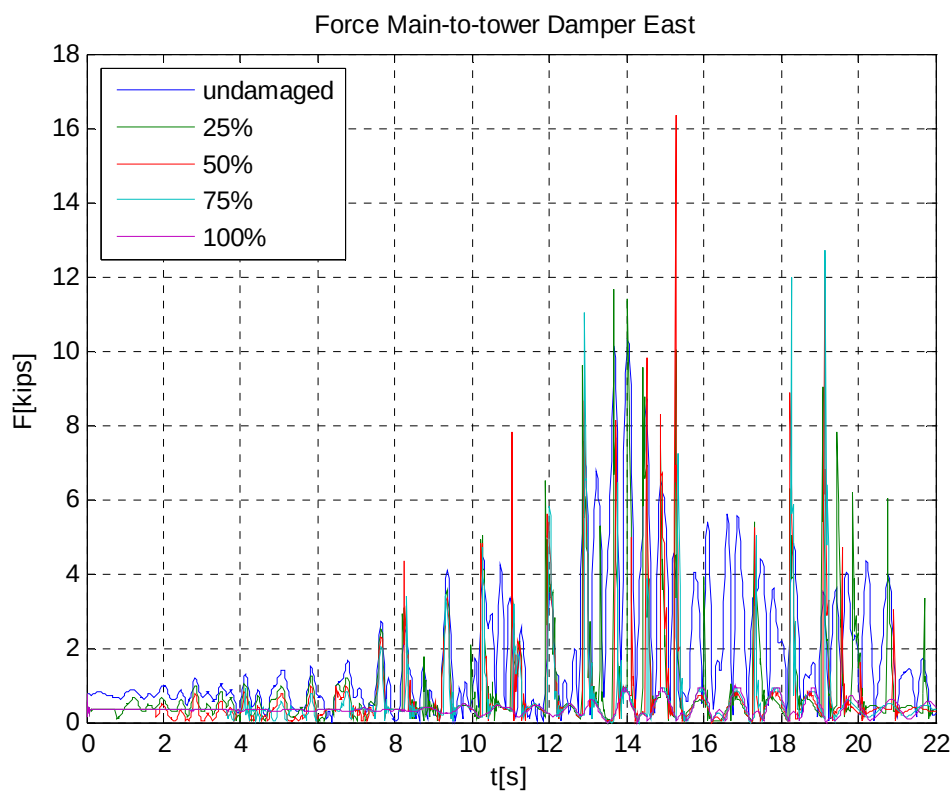


Figure 174. Time histories force Side to tower damper East, Northridge, sym. damage

Results show that during an event as Northridge one, a gap of 0.6" in the Main to Tower damper could increase the force in the device itself of 100%.

Frequencies and global behavior of the structure do not change.

VI. Possible solutions and future developments

Previous chapters show that normal traffic conditions produce a continuative work of the dampers. This high number of cycles could damage some components of the devices and cause some leakage of the viscous liquid. This damage produce a gap in the hysteretic device response and consequently could change the global behavior of the bridge. It was observed that a damaged device could cause higher impact forces in the neighborhood of the damper with gap.

A possible solution is to let the damper not to work for the normal serviceability condition and start to damp the structure only during a seismic event.

6.1 Rigid-brittle fuses in parallel to the dampers

The first and more straight forward idea is to couple to the damper some rigid/brittle element. This solution allow the damper not to work till the force in the fuse is sufficient to brake it.

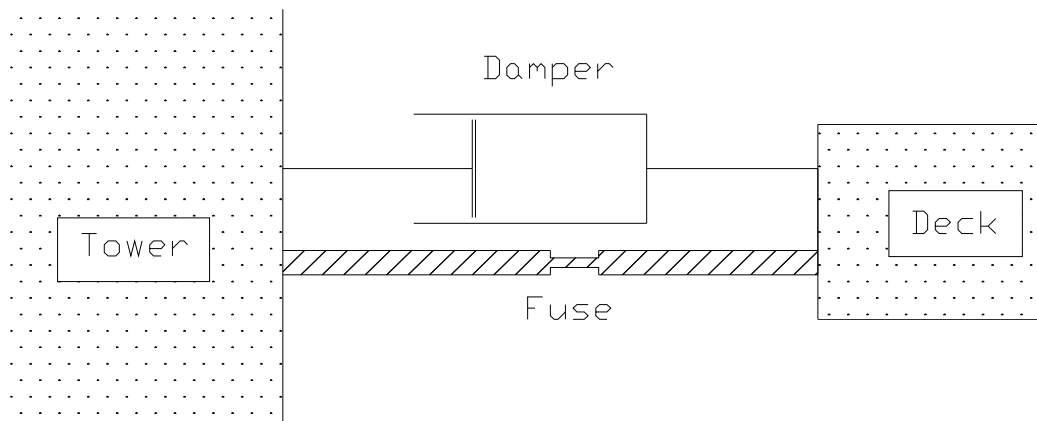


Figure 175. Damper with rigid-brittle fuse scheme

In order to define a ultimate strength of the fuse an ambient vibration analysis whit rigid link substituting the dampers was performed. Are now plotted the time histories of the forces in the fuses during a serviceability vibration.

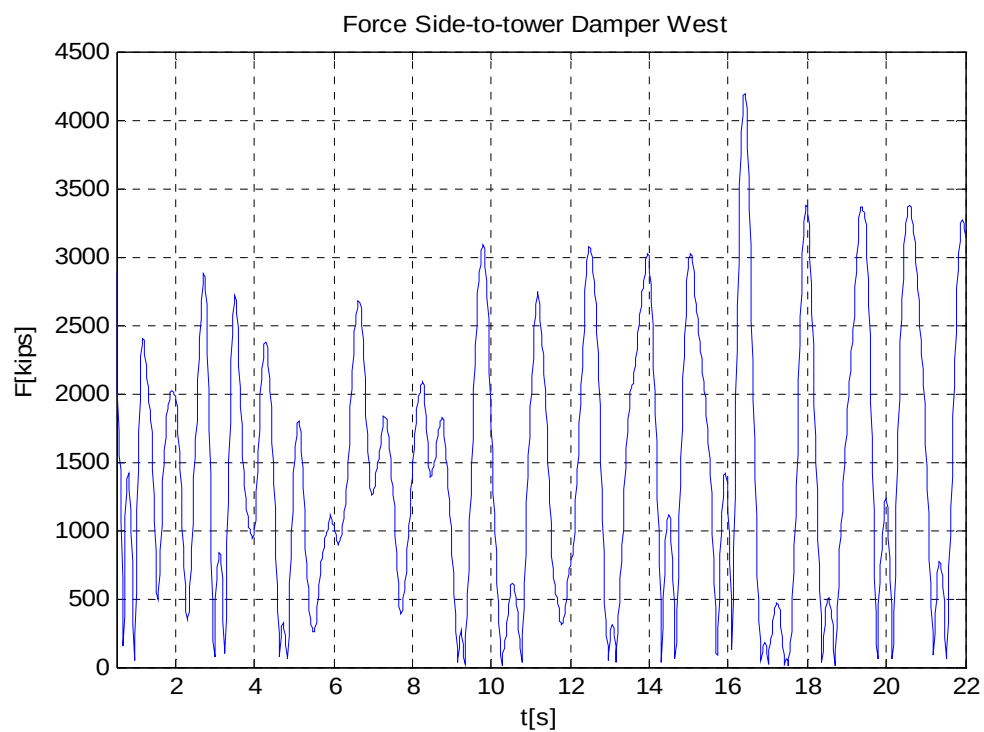


Figure 176. Force in damper's fuse side to tower west, during ambient vibration

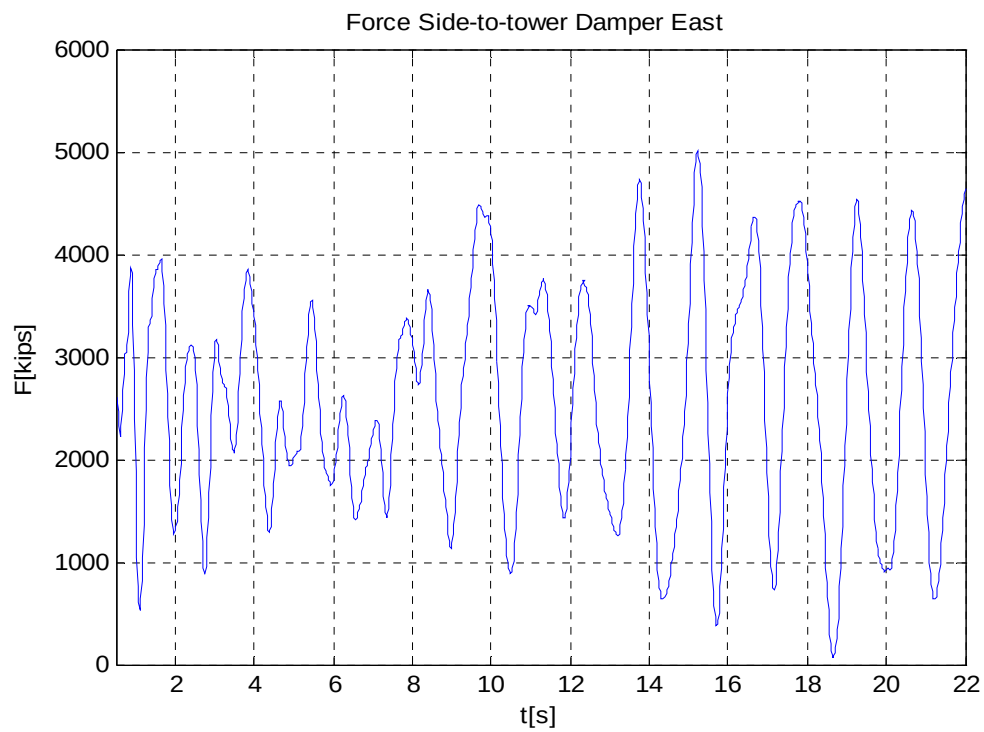


Figure 177. Force in damper's fuse side to tower east, during ambient vibration

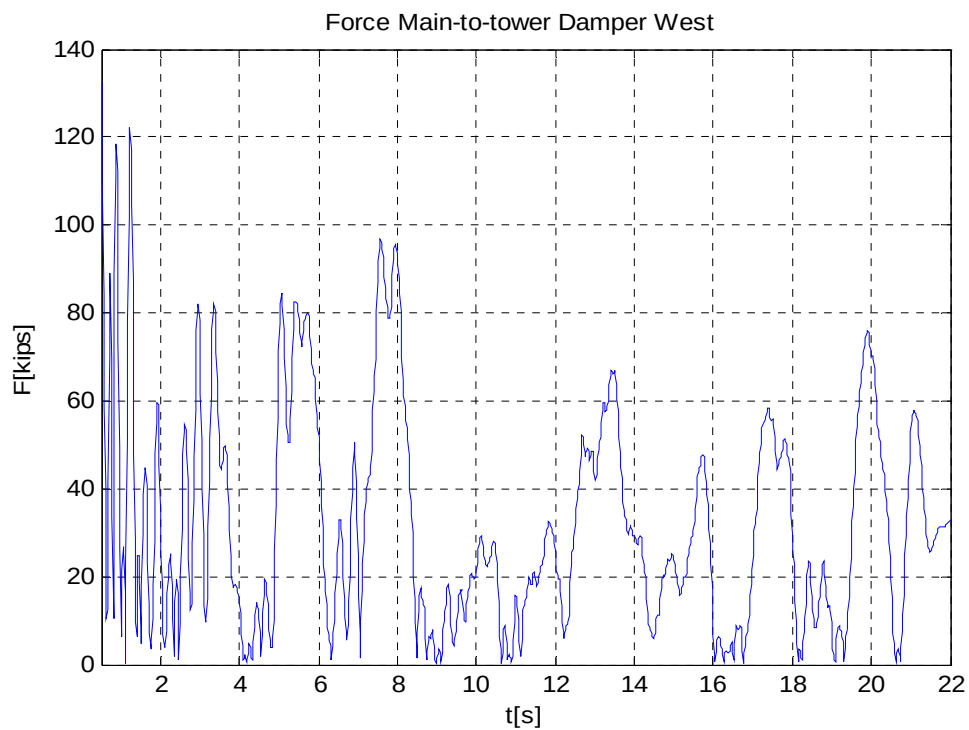


Figure 178. Force in damper's fuse main to tower west, during ambient vibration

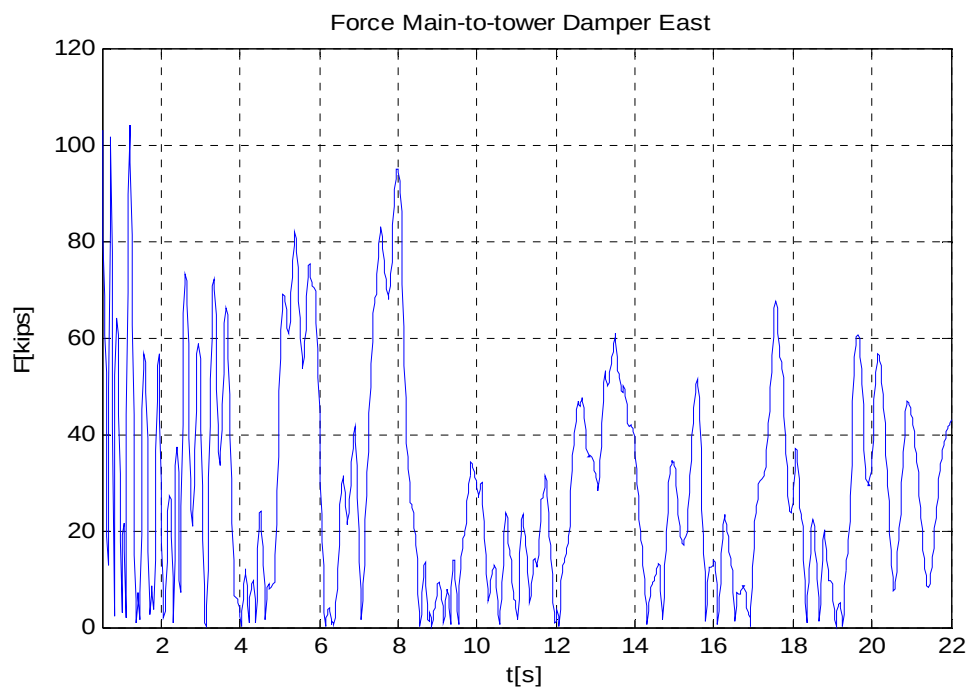


Figure 179. Force in damper's fuse main to tower east, during ambient vibration

Table 17 summarize the result plotted in previous figures.

Table 17. Max force in damper or fuses during ambient vibration and Northridge event

DAMPER or FUSE MAX FORCES [kips]		
	<i>amb.vibration (fuse)</i>	<i>Northridge (und. damper)</i>
<i>MAIN TO TOWER - EAST SIDE</i>	105	22.3
<i>MAIN TO TOWER - WEST SIDE</i>	121	21
<i>SIDE TO TOWER - EAST SIDE</i>	5000	10.3
<i>SIDE TO TOWER - WEST SIDE</i>	4250	10.6

During a serviceability ambient vibration the fuses have to resist to a force that is higher than the force in the correspondent dampers during a non destructive seismic event as Northridge. This suggests that the tower will be not able to take this stress because the deck was not designed to be linked to the tower. As is possible to see from the analysis (see fig. 11) mode 2 would completely change frequency. This changes the global behavior of the bridge, something that is generally discouraged in a retrofit like this.

6.2 Decoupling devices in series to the dampers

Demonstrated that rigid-brittle fuses in series to the dampers is not a good solution to prevent leakage of the damper devices, another solution is now proposed. The final goal is exactly the same, let the damper to remain not working during serviceability conditions and impose them to work during a certain level of seismic event.

6.2.1 Scheme and ambient vibration analysis

The proposed device has to be linked in series with the damper, the device isolates the damper from the deck in serviceability condition and link the damper to the deck as soon as a certain relative displacement is reached.

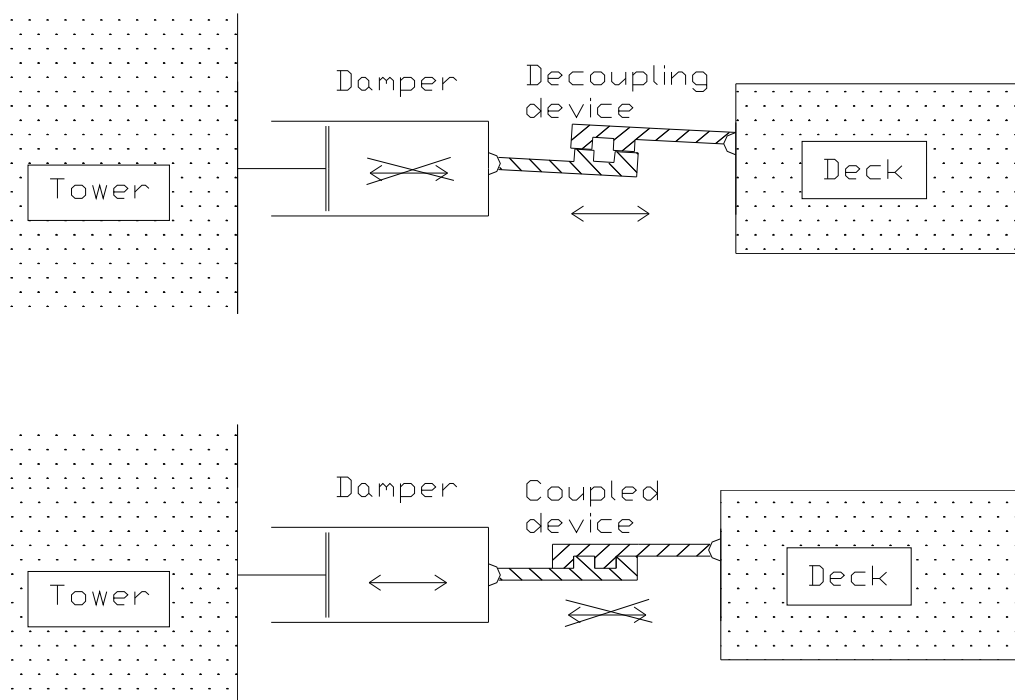


Figure 180. Damper with decoupling device scheme

The disadvantage of this solution is a little increment in the longitudinal deck's displacement during ambient vibration. Next graph shows the difference between the response of the deck in the actual non-damaged configuration and after the decoupling device retrofit.

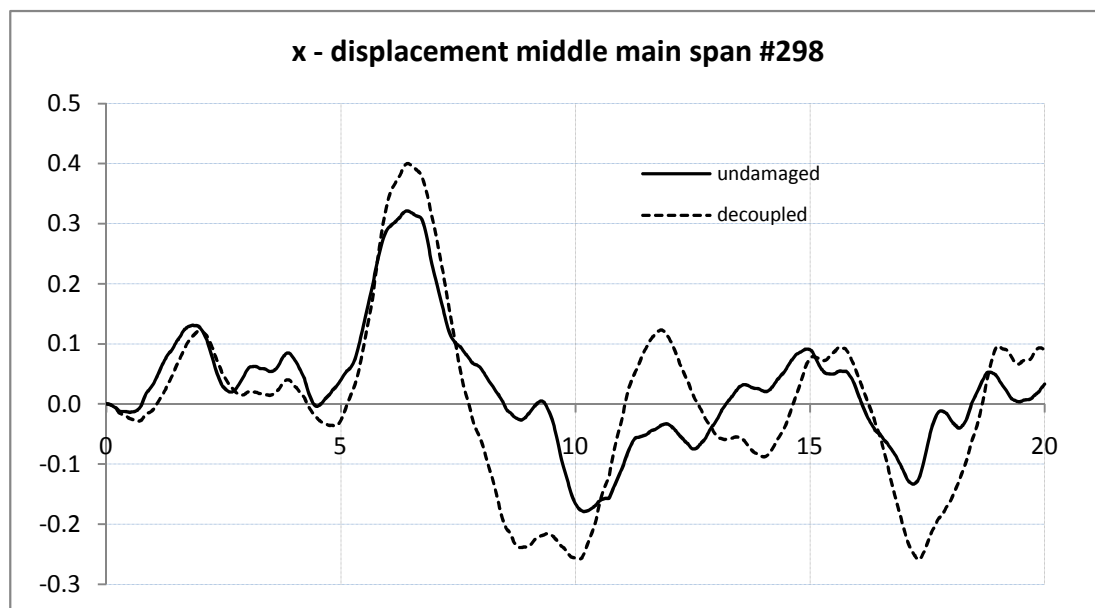


Figure 181. Longitudinal displ. midpoint main span with and without decoupling device, amb. vib.

As written before the main difference is in the longitudinal displacement of the deck. Decoupling the damper from the deck there is an increment of 20% in this displacement (from 0.32" to 0.4"). Next table summarize all the results.

Table 18. Maximum displacements with and without decoupling device, amb. vibration

<i>X maximum displacement [in]</i>			
Node point	Position	Undamaged	Decoupled
232	Mid side span E	0.81	0.86
298	Mid main span	0.32	0.4
2164	Mid side span W	1.88	1.92
7121	Cable bent W	0.06	0.06
10201	Base tower W	0	0
11392	Top tower W	0.12	0.12
<i>Y maximum displacement [in]</i>			
Node point	Position	Undamaged	Decoupled
232	Mid side span E	0.27	0.28
298	Mid main span	1.65	1.67
2164	Mid side span W	0.41	0.41
7121	Cable bent W	0.15	0.15
10201	Base tower W	0	0
11392	Top tower W	0.12	0.12
<i>Z maximum displacement [in]</i>			
Node point	Position	Undamaged	Decoupled
232	Mid side span E	7.47	7.58
298	Mid main span	5.03	5.07
2164	Mid side span W	7.57	7.69
7121	Cable bent W	0.05	0.05
10201	Base tower W	0	0
11392	Top tower W	0.07	0.07

6.2.2 Decoupling device gear teeth wide design

In order to design the decoupling device a certain level of free gap has to be defined. Starting from the deformations of the dampers during a serviceability vibration and a non destructive earthquake is possible to define a optimum length for the two devices' gear teeth.

The next figures show the free damper shortening/elongation during an ambient vibration.

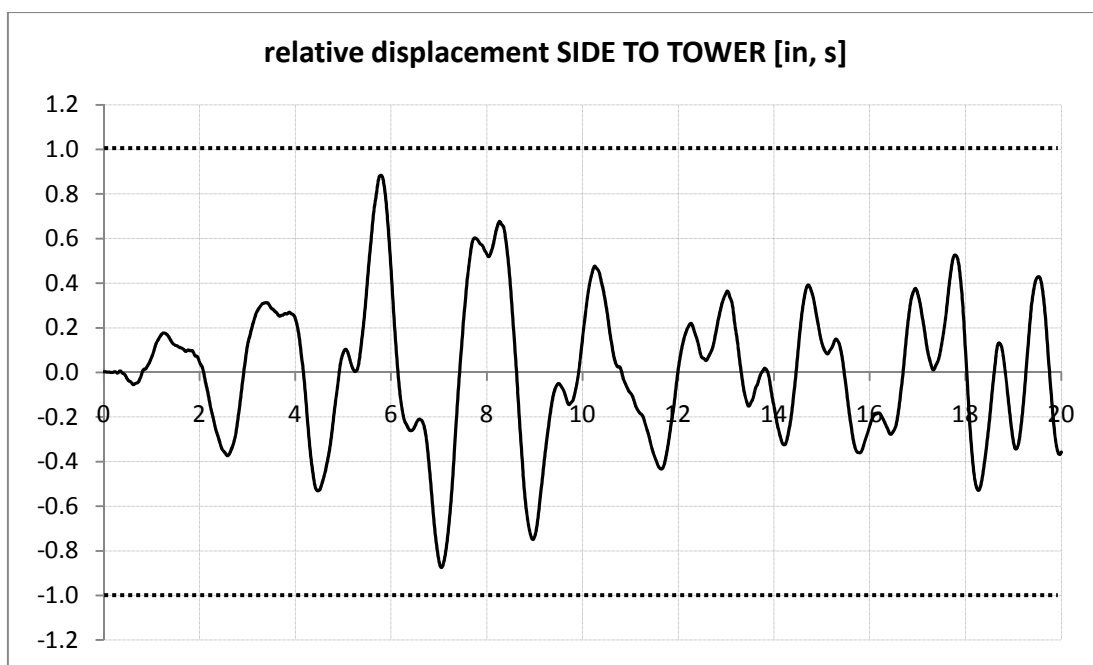


Figure 182. Damper deformations, Side to Tower, decoupled dampers

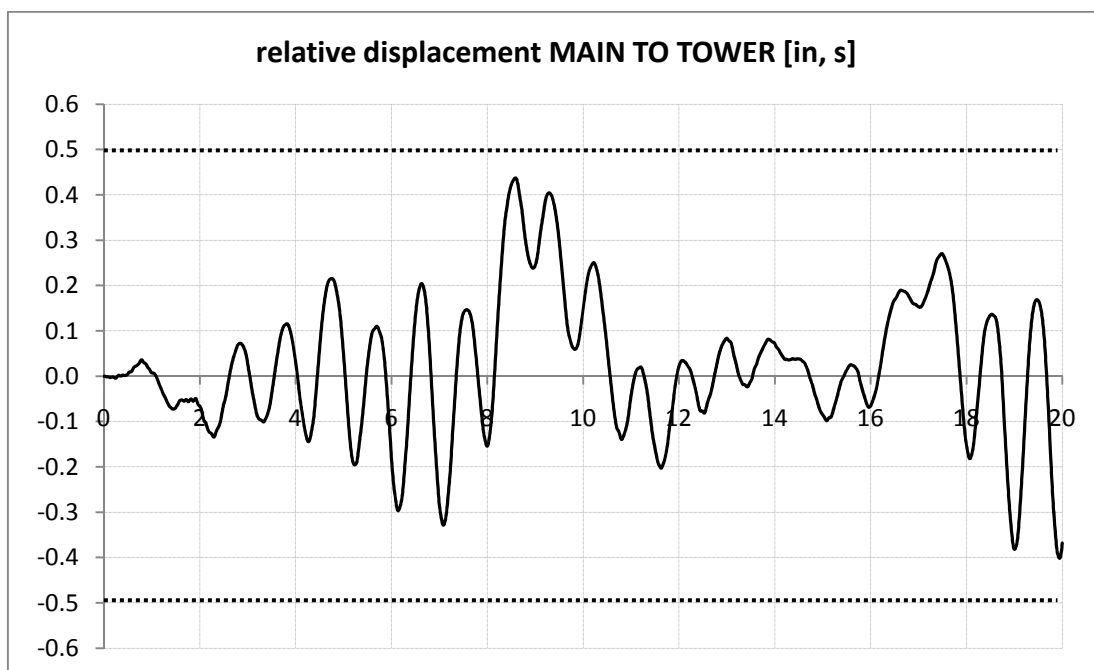


Figure 183. Damper deformations, Main to Tower, decoupled dampers

The dotted lines represent the maximum allowed free relative displacement before the dampers begin to contribute to the structure response. In particular $\pm 1"$ and $\pm \frac{1}{2}"$ for the side and main to tower respectively.

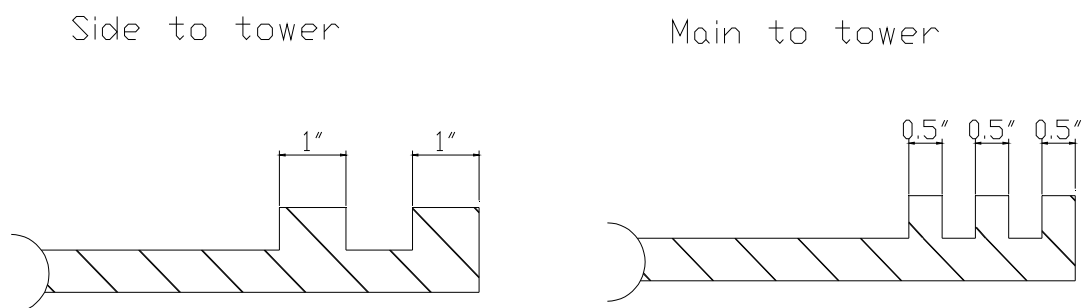


Figure 184. Side and Main to tower decoupling device gear teeth wide

6.2.3 Seismic event verification

A verification on the response of the structure during a recorded earthquake was performed with the same Northridge input used in ch. 4.2.

It was not possible to model directly the nonlinear element in Adina. A manual restart analysis was performed. An analysis with not working dampers was run till one set of damper reached the maximum free deformation (fig. 185 and 186). After that the analysis was restart with the full working damper properties. During the Northridge event the device would be activated after 14.5 sec. (side to tower) and 13.5 sec. (main to tower).

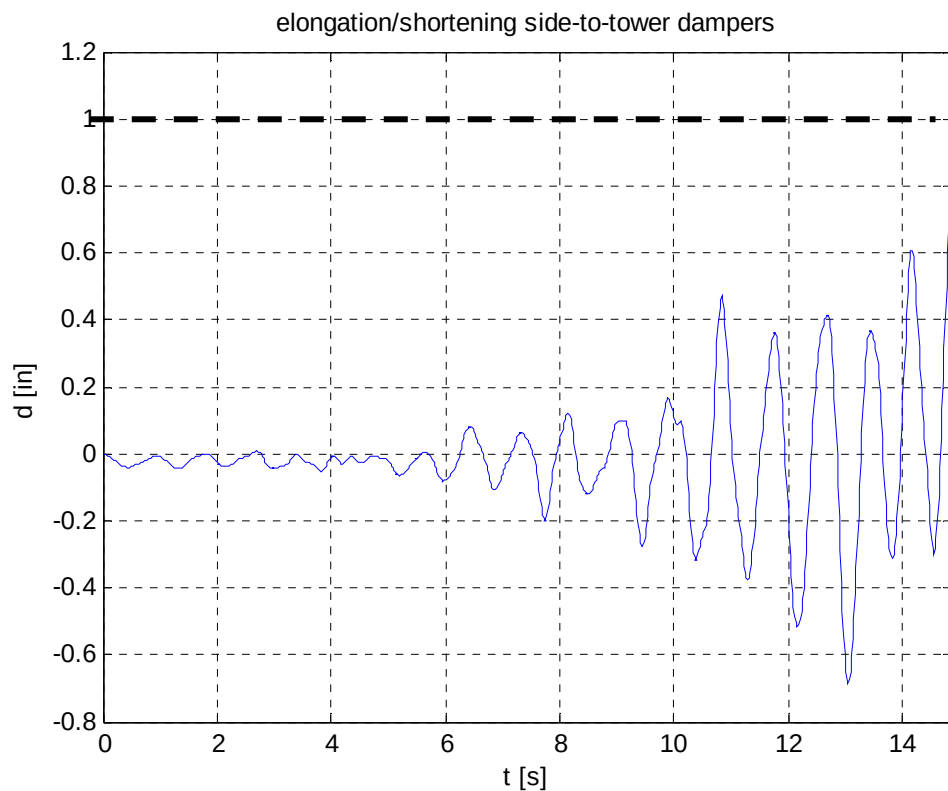


Figure 185. Time histories of deformation side-to-tower damper with device decoupled

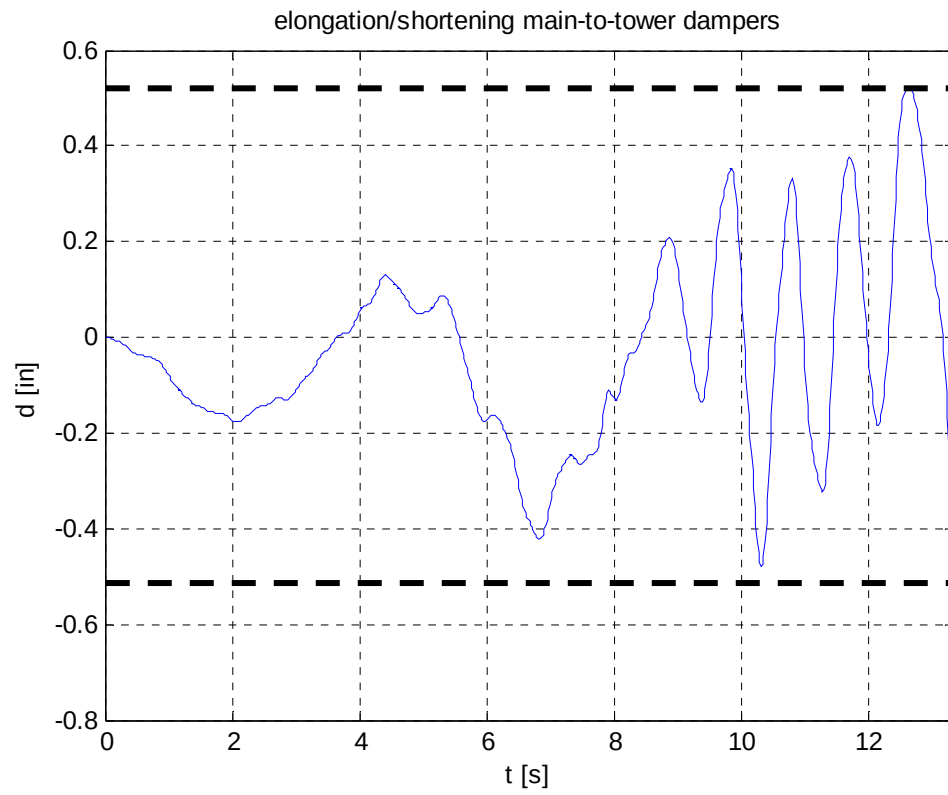


Figure 186. Time histories of deformation main-to-tower damper with device decoupled

The next figures represents the elongation of the dampers during all the event. Is possible to observe that the dampers are beginning to deform after the locking of the devices.

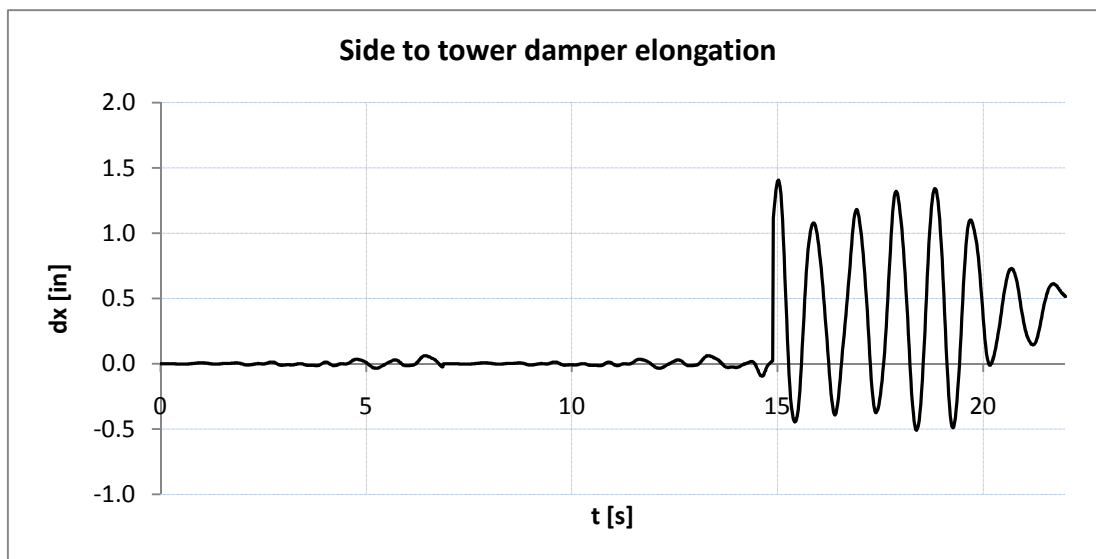


Figure 187. Time histories of deformation side-to-tower damper with working device

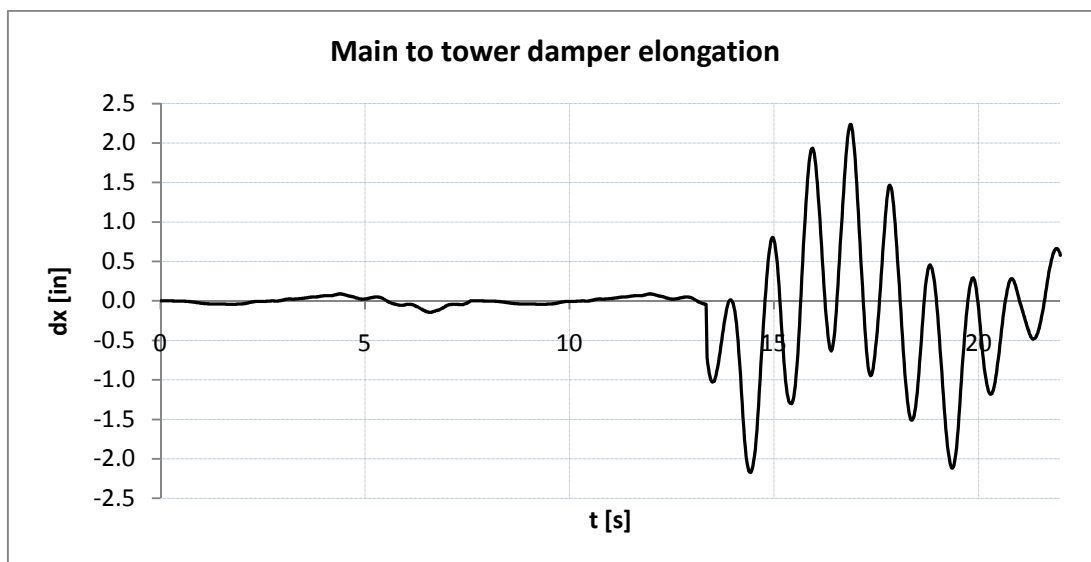


Figure 188. Time histories of deformation main-to-tower damper with working device

And the same consideration can be done for the force in the devices.

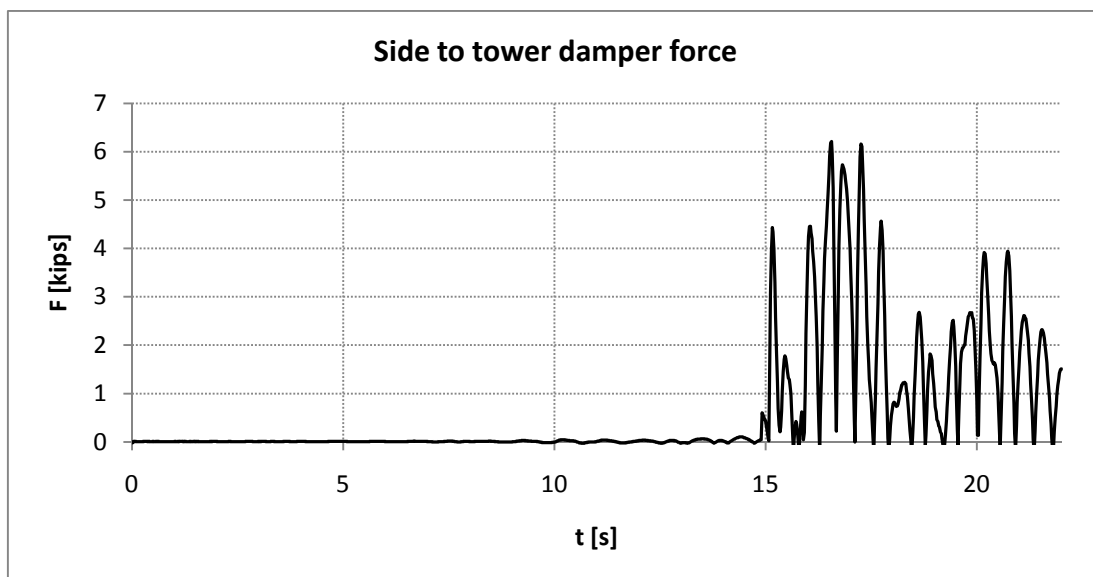


Figure 189. Time histories force Side to tower damper East, working device

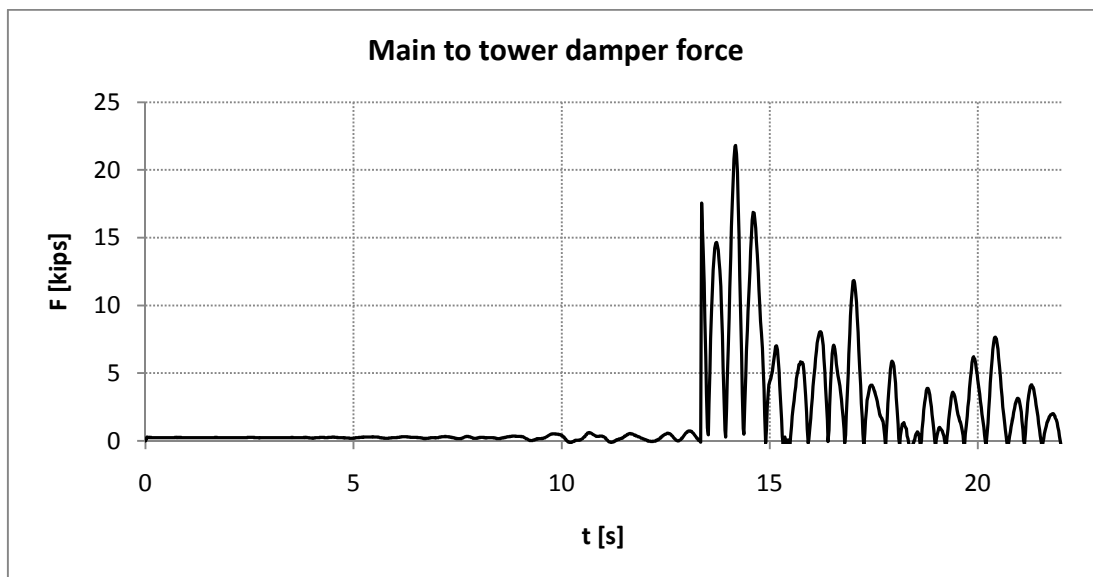


Figure 190. Time histories force Main to tower damper East, working device

The impulse force results less than the force acting during the full operation of the dampers. The element in the neighborhood of the device are so able to resist to this force. The same result was found in ch. 4.2, considering the first peaks referred to gaps similar to the artificial one of the proposed devices.

The main advantage of the device is that there is an important reduction of the global displacements in the entire structure (2 decks and towers). The following graphs represent the time histories of the main check points displacement. The black line is positioned in correspondence to the side to tower lock and the gray one in correspondence to the main to tower lock.

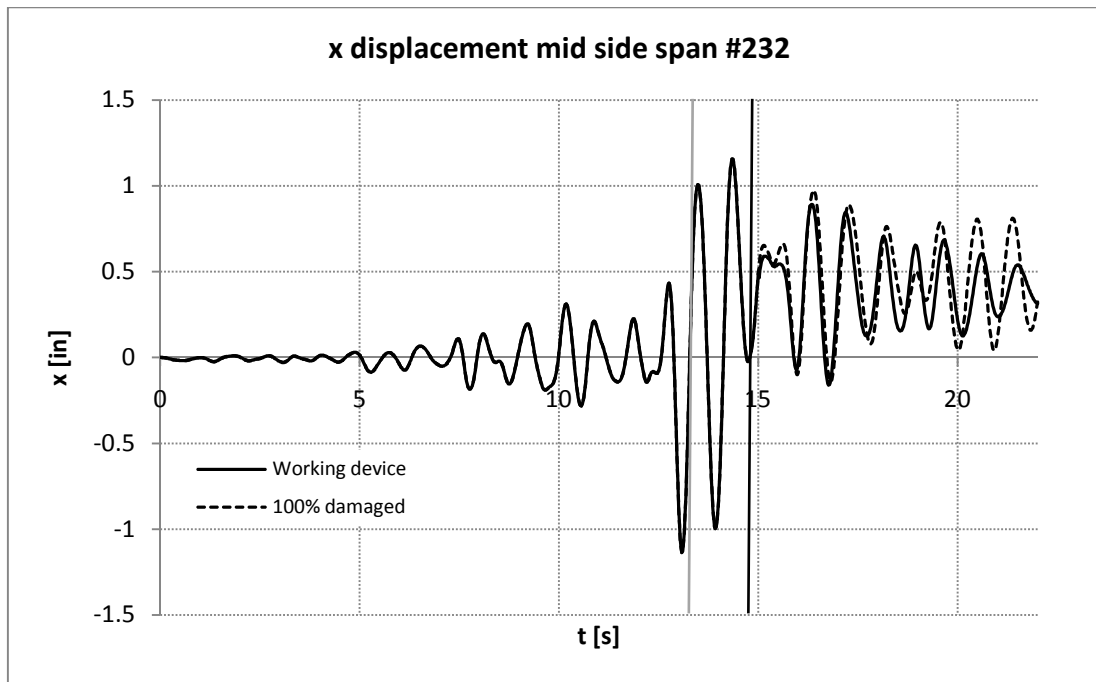


Figure 191. Time histories of displacement in x direction of mid side span

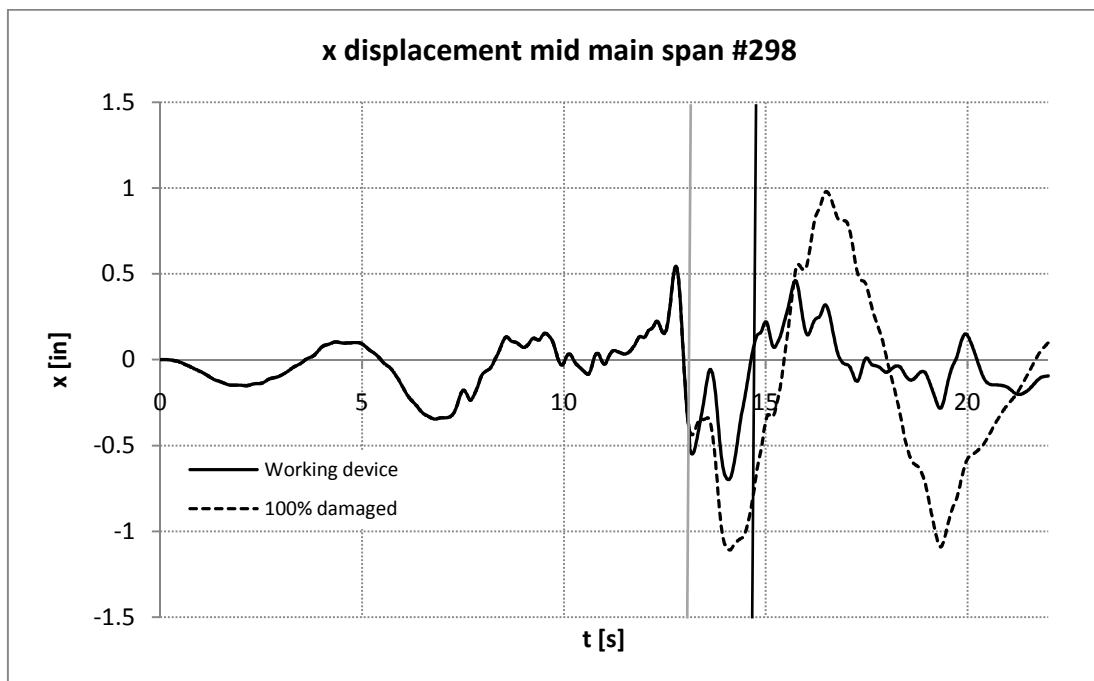


Figure 192. Time histories of displacement in x direction of mid main span

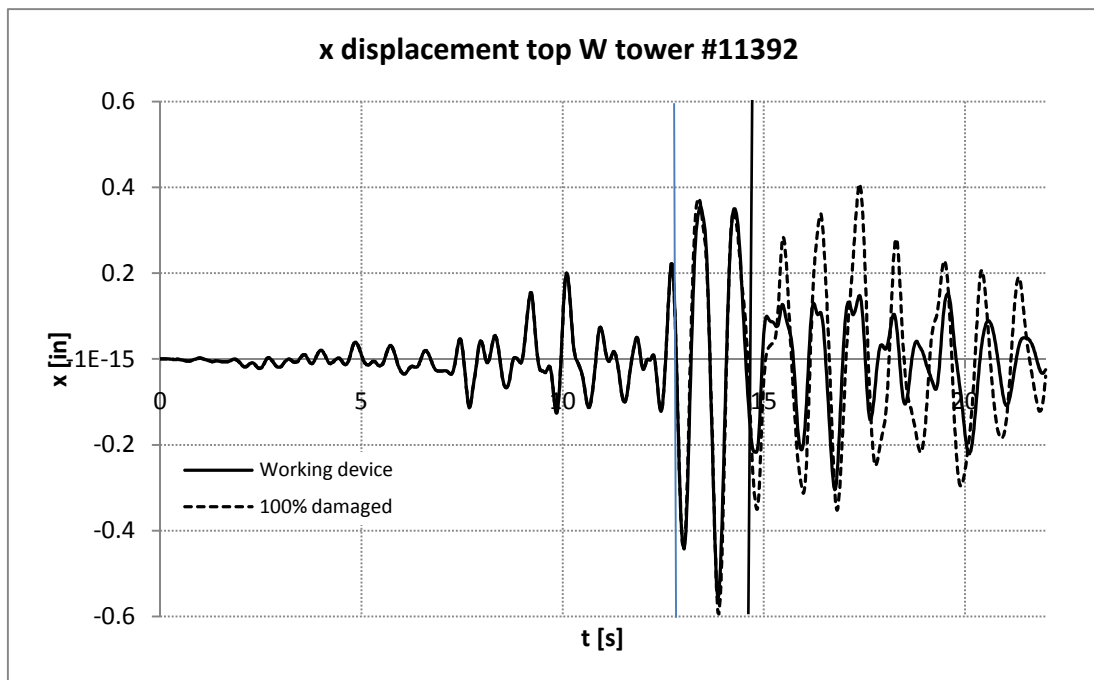


Figure 193. Time histories of displacement in x direction of top W tower

From the previous chart appears that each deck change its behavior right after the locking of the device related to the dampers directly linked to the deck. The following tables summarize the results.

Table 19. Difference of behavior during ambient vibration with or without decoupling devices

<i>Serviceability vibration</i>	Full working dampers	Decoupling device	Comments
X disp. mid side span [in]	0.81	0.86	+ 6%
X disp. mid main span [in]	0.32	0.4	+ 20%
X disp. tower [in]	0.12	0.12	-
Damper usage	$\pm 0.5''$ continuative working	Not working	Advantage preventing damage of the dampers

Table 20. Difference of behavior during Northridge with or without decoupling devices

<i>Northridge</i>	Damaged dampers	Decoupling device	Comments
X disp. mid side span [in]	1.2	1.2	-
X disp. mid main span [in]	1	0.6	- 40%
X disp. tower [in]	0.6	0.58	- 3%
Damper force [kips]	45	23	- 50%

The most important results using decoupling devices are:

- dampers are not in use during ambient vibration (no degradation of the dampers);
- 20% increment of longitudinal displacement of mid main span compared to full operational dampers with ambient vibration;
- 40% reduction of longitudinal displacement of mid main span compared to damaged dampers during non destructive event;
- 50% reduction of impulsive force compared to damaged dampers.

Further analysis focused on the stresses of the devices' neighborhood would be suggested after an actual design of these components.

6.3 Change in dampers properties

In order to verify the possibility to obtain a better performance of the bridge, others analysis were performed.

Given

$$F = C \cdot v^{\alpha}$$

the two parameters that governs a damper behavior are C and α .

The dampers modified in these analysis were only the main to tower ones (originally $C=4$, $\alpha =1$). From a free vibration analysis was possible to calculate the equivalent viscous damping ratio (ξ) associated to mode 2 (longitudinal movement of the main deck). The decay of motion as a logarithmic decrement indicates an original ξ value around 6%. From fig. 26, considering the frequency of the second mode (0.18 Hz), is possible to see the a higher value of energy would be dissipated incrementing ξ (or C).

Four different analysis were conducted (Northridge input), changing the parameters of the main to tower damper in order to understand the influence of these two parameters on the structural displacement of the main deck and on the force on the dampers as well.

It was decided to change the parameter α because on the market is not uncommon to find devices with low α value (less than 0.2).

I. Analysis: $C=8$, $\alpha=1$

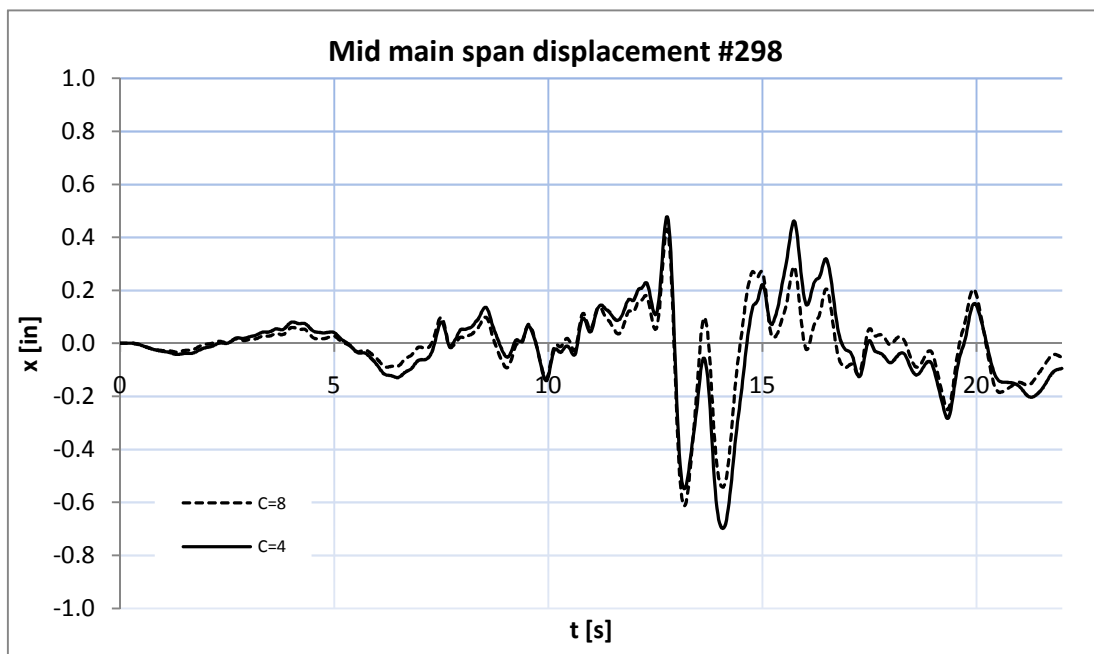


Figure 194. Displacement time histories mid main deck, $C=8$, $\alpha=1$

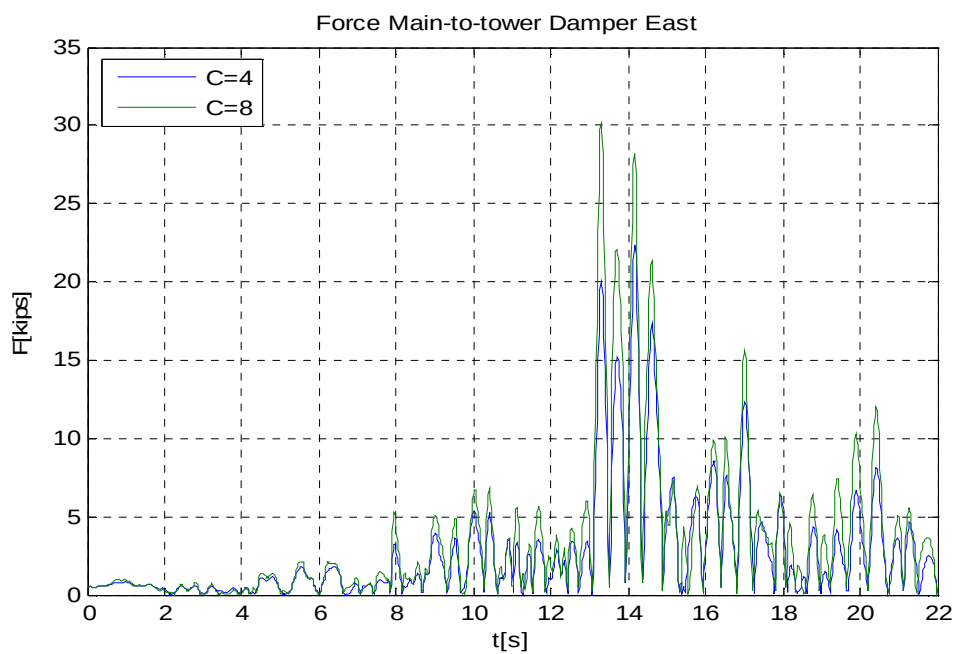


Figure 195. Force in main to tower dampers, $C=8$, $\alpha=1$

II. Analysis: $C=4$, $\alpha=0.25$

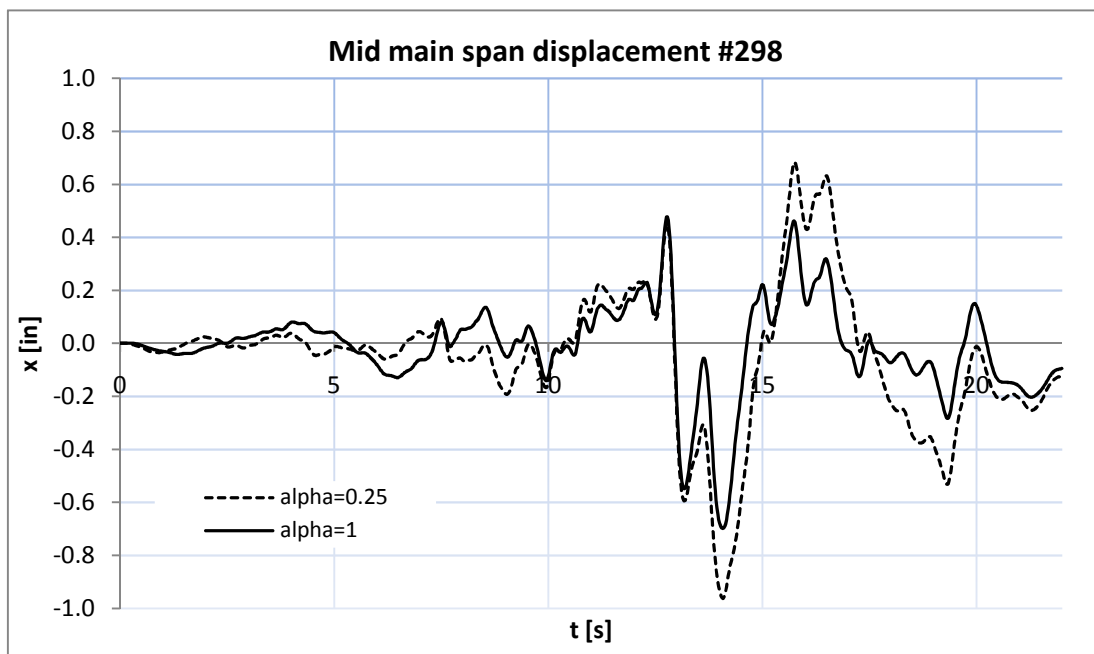


Figure 196. Displacement time histories mid main deck, $C=4$, $\alpha=0.25$

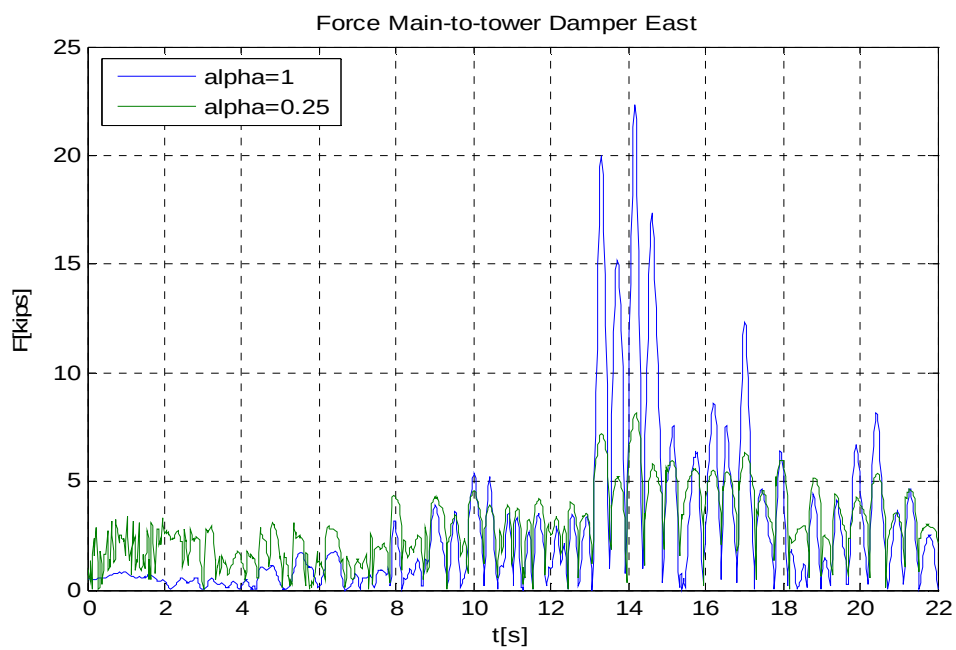


Figure 197. Force in main to tower dampers, $C=4$, $\alpha=0.25$

III. Analysis: $C=8$, $\alpha=0.25$

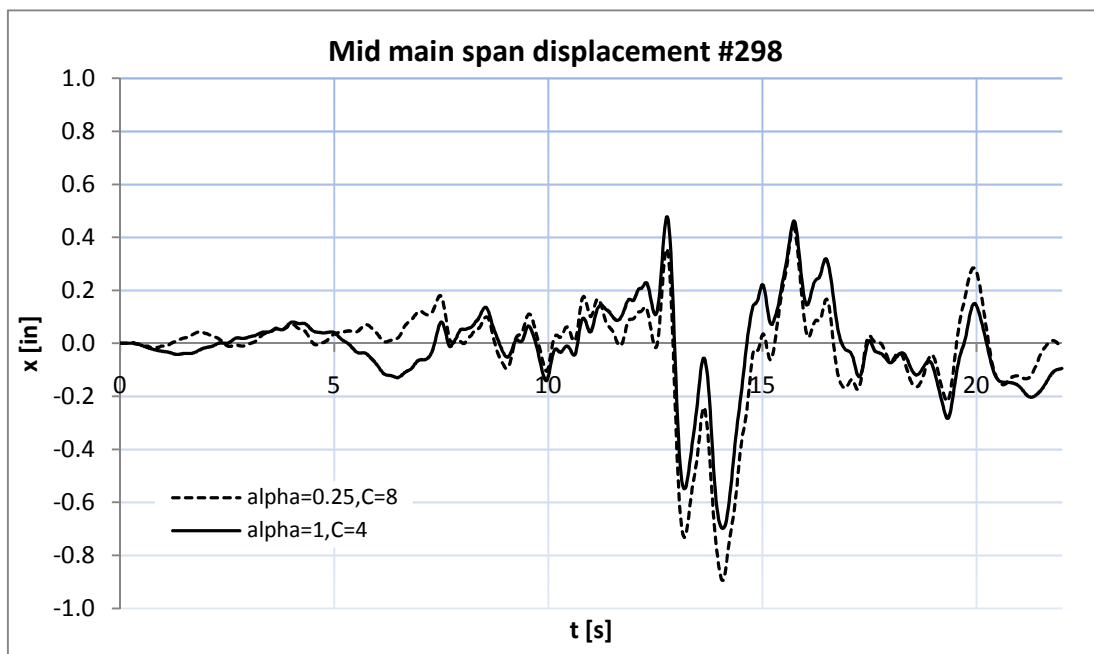


Figure 198. Displacement time histories mid main deck, $C=8$, $\alpha=0.25$

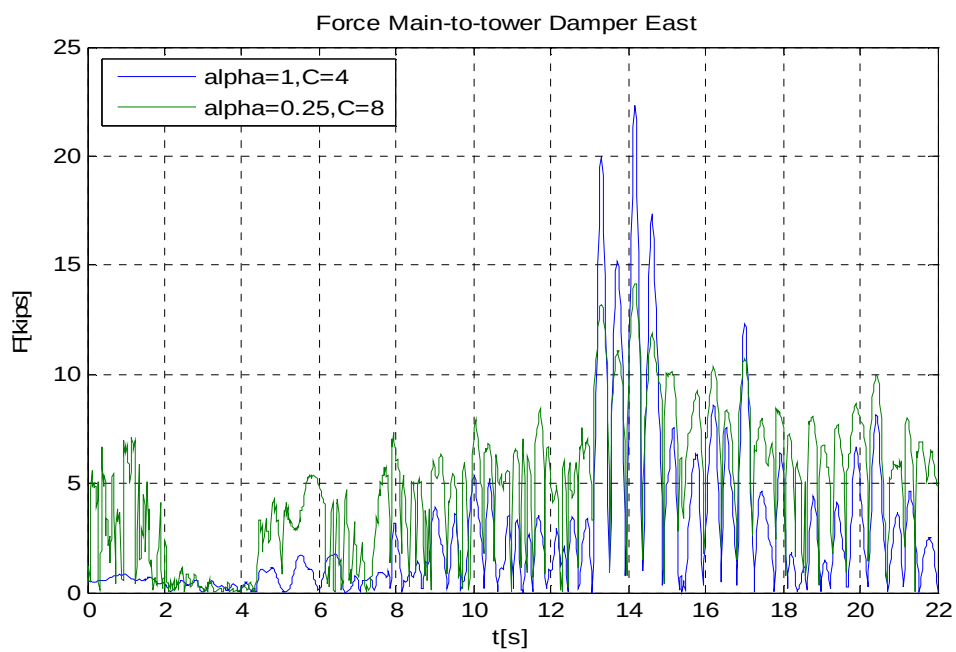


Figure 199. Force in main to tower dampers, $C=8$, $\alpha=0.25$

IV. Analysis: $C=16$, $\alpha=0.25$

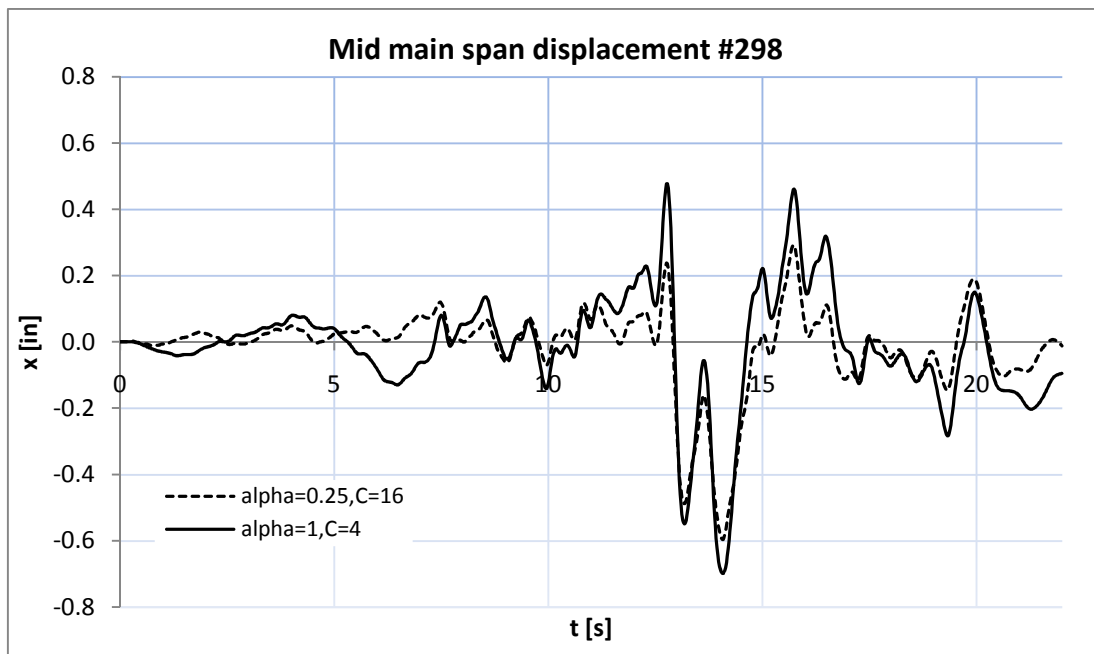


Figure 200. Displacement time histories mid main deck, $C=16$, $\alpha=0.25$

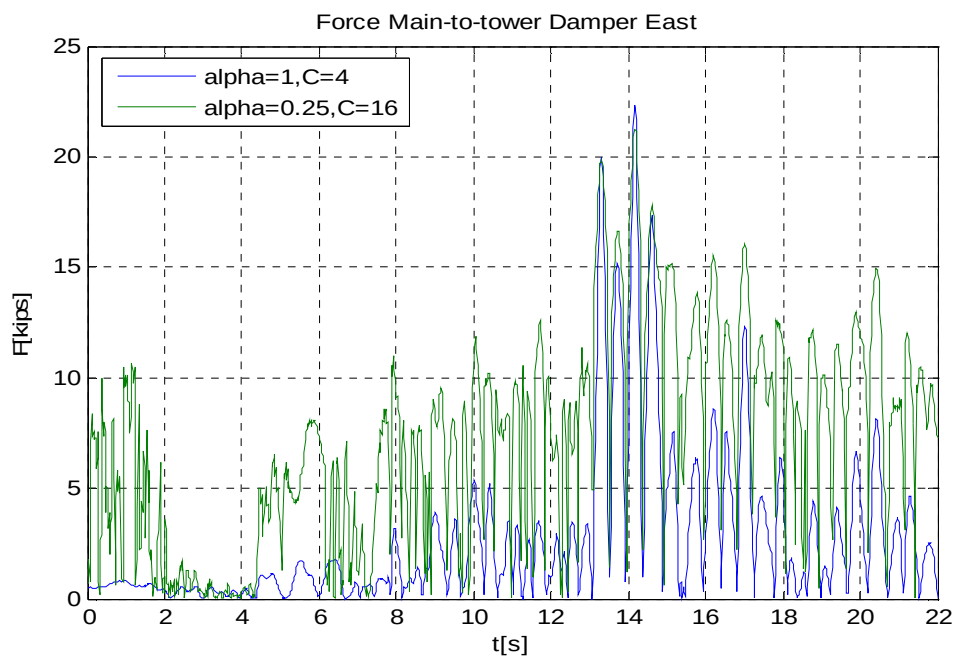


Figure 201. Force in main to tower dampers, $C=16$, $\alpha=0.25$

The previous graphs show that incrementing C there is a decrement in the displacement and an increment in the force in the dampers, as expected. An opposite behavior is obtainable decreasing α .

From analysis number four is possible to understand that changing both parameters to find the optimum solution is possible to change the behavior of the deck during the entire event, but seems not possible to decrease the maximum displacement and the maximum force in the same time.

VII. Conclusions

In order to simulate a damage in the viscous damper devices of Vincent Thomas Bridge, a non linear gap element has been introduced in the original Caltrans Adina FE model of the bridge.

From the parametric analysis it is possible to deduce that the degradation of the viscous dampers substantially impacts the structural response of the bridge, in terms of displacements, along the longitudinal direction of the bridge. In particular a variation of 25% of the longitudinal displacement of the main deck was calculated under ambient vibration condition. The same increments during a simulated seismic event were 45% and 70% for strong white noise and Northridge analysis respectively. No significant variations have been detected in the transverse and vertical directions, associated with reduction of the damper performance.

A study on the change of frequencies shows that there are no important changes linked to the damper degradation.

A potentially dangerous phenomenon has been found in case of damaged damper in the structure. If all the devices linked to the tower were damaged (with a gap wider than $1/2''$) impulsive forces be experienced by the dissipators and transmitted to adjacent structural elements. This force could be high and further analysis will be necessary to understand if the structure is actually able to resist these impulsive forces. This

phenomenon was not detected in case of non-symmetric distribution of damage on the structure.

The dampers are working continuously with several cycles during normal operation of the bridge. This is likely to be the cause of the early degradation of the dampers devices, due to wear of the internal components. In this work a device was proposed to be linked in series with the damper. The device isolates the damper from the deck in serviceability conditions and engages the damper to the deck as soon as a certain relative displacement is reached. The solution appears to be functional. The damper will not experience relative displacement during normal condition and will perform as designed during a seismic event. Providing supplemental damping capabilities as well as limiting the level of displacements.

Analysis were conducted in order to verify if it will be possible to obtain a better performance of the bridge substituting the device with more efficient ones. The results is that, not changing the force admissible in the devices appears not to be possible to decrease the global displacement of the decks.

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