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INFLUENCE OF ENERGY DISSIPATING SYSTEMS ON THE SEISMIC RESPONSE OF CABLE SUPPORTED BRIDGES

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SUMMARY

Friction and viscous dampers were inserted between vertical shafts of the suspension tower in models of the new San Francisco Oakland Bay Bridge main span. Their performance in protecting the bridge under seismic loading was compared to that of the new shear link protection system. The effect of forward directivity was monitored to discover if pulse motion reduced the functionality of the dampers. Friction and viscous dampers each improved upon performance of the shear links in different tower protection configurations. Results suggest forward directivity reduces performance slightly, but bridge tower top restraint has a larger effect on performance.

INTRODUCTION

The new San Francisco Oakland Bay Bridge (SFOBB) East Span was designed by TYLIN [1] and Moffit & Nichol Engineering, and is now under construction. The SFOBB consists of a 3.1-km long reinforced concrete skyway that spans most of the distance between Oakland and Yerba-Buena Island (YBI), a 565-m long signature cable-supported span over the shipping channel off the east shore of YBI, and transition structures at the Oakland and YBI touchdowns. Two designs were originally proposed for the signature span: a concrete cable-stayed bridge (CSB) and a steel self-anchored suspension bridge (SASB). Ultimately the SASB design was chosen to replace the existing steel cantilever truss structure. A rendering of the SFOBB East Span is presented in Figure 1. In 2002, the SASB and CSB were subjects of a study conducted by McDaniel [2] at the University of California at San Diego (UCSD) to study the influence of sacrificial shear links in the bridge tower. The signature tower is composed of multiple shafts with horizontal shear steel shear link members connected between the shafts at various levels throughout their height. These shear links were designed to improve seismic

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performance of the SFOBB by dissipating hysteretic energy and protecting the tower shafts from inelastic behavior under the Safety Evaluation Earthquake (SEE—1,500 year event). As part of his study McDaniel performed nonlinear dynamic analyses of both the CSB and SASB using Ruaumoko3D.

The purpose of this study was to compare the performance of the SASB and CSB suspension towers fitted with shear links to SASB and CSB suspension towers equipped with other energy dissipating devices. In the event of the SEE, the sacrificial shear links would yield and could require replacement. There are, however, several non-sacrificial energy dissipating options that have gained popularity in recent years. Two types of passive dampers were considered for this study. Friction dampers and viscous dampers were inserted between the suspension tower shafts in place of the shear links in the SASB and CSB models, and analyzed using Ruaumoko3D. By comparing the response of the damper fitted models to that of the shear link models, it was possible to determine whether dampers could be made to improve upon the performance levels achieved by sacrificial shear links.

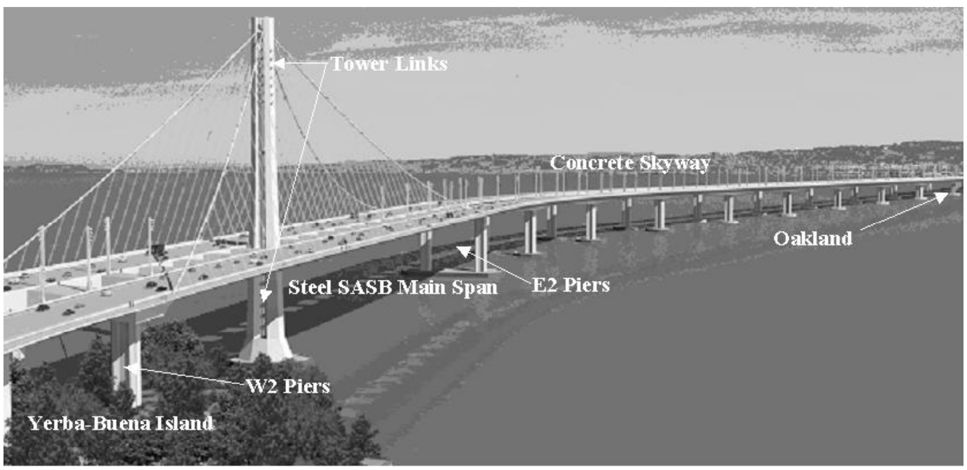

Figure 1: Rendering of the New SFOBB East Span

The nature of ground motions predicted for the new SFOBB East Span location added complexity to the study. Earth Mechanics [3], a geotechnical engineering firm, created six ground motion record sets, three derived from the San Andreas Fault and three from the Hayward Fault. The SFOBB lies between these two major faults at distances of 25-km and 12-km, respectively. The proximity of the seismic source implies that forward directivity effects are important in the ground motions. Forward directivity effects result in ground motion characterized by strong pulses oriented in the fault-normal direction and best seen in the velocity time histories.

Forward Directivity

According to Somerville [4], damage induced by the 1994 Northridge Earthquake elevated concern over the effects of forward directivity. The characteristic large pulse, most visible in velocity and displacement time histories, occurs in ground motions recorded near to the fault plane and oriented in the fault-normal direction. Forward directivity pulses occur because as the fault ruptures in a given direction, the seismic waves that travel in the same direction tend to build up and arrive at the point of measurement simultaneously. The product is a large pulse at the beginning of the record. These pulses are a cause for concern because it is not always clear whether or not a seismic protection system will have the time or capacity to respond. Long-period structures are especially susceptible to damage due to the period of the forward directivity pulse. Displacement response of structures with fundamental periods over 1 second is larger in the event of forward directivity than for records without forward directivity (see Somerville [4]).

Viscous and Friction Dampers

Friction dampers protect structures through hysteretic energy dissipation. Friction dampers often consist of steel members, bolted together through slotted connections such that static friction prevents motion in the joint. When a large enough force is applied, the joint slips and dissipates energy through frictional heat. More complex friction damper devices exist, such as the Pall Device tested by Filiatrault [5], but all friction dampers work in the same basic way. Friction dampers add stiffness to the structure. Until they slip, they act as struts, usually installed across portals in a frame. This could be useful in mitigating forward directivity pulses, since stiffening a structure means shortening its fundamental period. Since forward directivity excitations effect long period structures the most, stiffening a structure could reduce this hazard. At the same time, they provide energy dissipation.

Viscous dampers consist of a sealed piston-cylinder arrangement containing a heavy viscous fluid. They improve the response of a structure through viscous energy dissipation. This differs from hysteretic energy dissipation in that force produced by the damper is dependent upon velocity and not displacement. Their use is relatively new in structural engineering (20 years) but they have been developed and used by the military for a century according to Taylor [6]. Presented in Figure 2 is a comparison of the basic hysteretic shapes for viscous dampers and friction dampers. Note that in the friction damper, when displacement is at its largest, damper force is at its largest as well. This is the opposite for viscous dampers. When displacement is zero, damper force is at its largest. The viscous damper acts out of phase with the structure it is protecting. Viscous dampers, therefore, are capable of reducing displacement and base shear at the same time. Also, since viscous dampers have no initial stiffness, they do not cause residual displacement of the structure. Friction dampers can cause residual displacements if they slip, and likewise with shear links if they undergo inelastic deformation. In the case of the CSB and SASB suspension tower shafts, friction dampers and shear links will be restored to their original position by the significantly larger stiffness of the suspension tower shafts, and residual displacement will not be an issue.

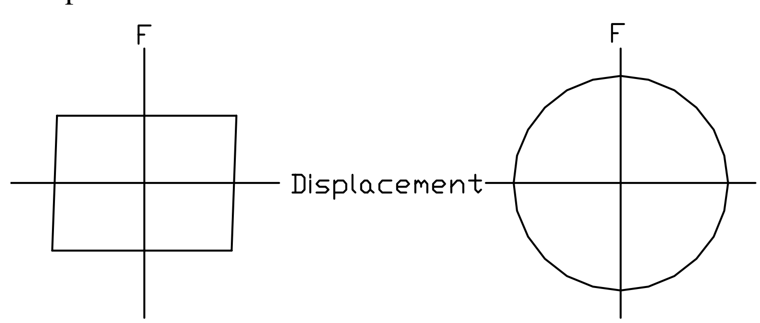

Figure 2: Hysteresis Plots for a friction damper (left) and a viscous damper (right).

Objectives

- Determine if the seismic performance of the SASB and CSB could be improved using viscous dampers or friction dampers along the suspension tower height.
- Determine if the existence of a forward directivity pulse inhibits the effectiveness of viscous dampers or friction dampers in SASB and CSB towers.

METHODOLOGY

The CSB and SASB models for analyses in Ruaumoko3D were borrowed from McDaniel [2] and the UCSD shear link program. Figure 3 presents the SASB and CSB models. The SASB tower is composed of four hollow steel shafts while the CSB tower is made up of two hollow reinforced concrete shafts. To simplify analysis, the SASB model was tested in each of the transverse and longitudinal directions separately. When the transverse direction was being analyzed, the transverse, vertical, and corresponding rotational excitation records were used. The same was done in the CSB analyses, but since there were only links/dampers oriented in the transverse direction, there were only transverse analyses for the CSB model.

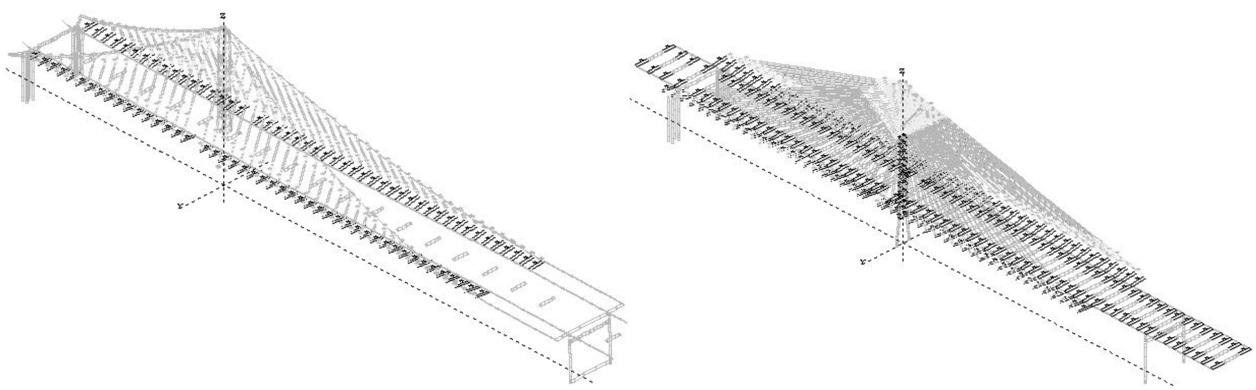

Figure 3: SFOBB SASB (left) and CSB (right) Model in Ruaumoko3D

Excitation Input

Six ground motion record sets had been created by Earth Mechanics [3] for the new SFOBB East Span. For the analyses of the damper-protected models, the governing earthquake scenario derived from the San Andreas Fault (SA_1) and the governing scenario from the Hayward Fault (H_1) were used. Plots of the ground motion displacement time histories for the main pier (tower) for both SA_1 and H_1 appear in Figure 4. There are separate displacement time history input records for each direction at Piers E2 and W2 (labeled in Figure 1). They are similar to the records for the main pier, but differ slightly due to the expected arrival time of excitation and the difference in underlying soils. Note the pulse in the first five seconds of each longitudinal record. This is due to forward directivity motion. Since the longitudinal direction of the bridge is not oriented perfectly in the fault-normal direction, forward directivity affects the transverse bridge response as well, though not as significantly. SA_1 was primarily used in this study. All models were analyzed using SA_1, then a few were selected to be analyzed using H_1.

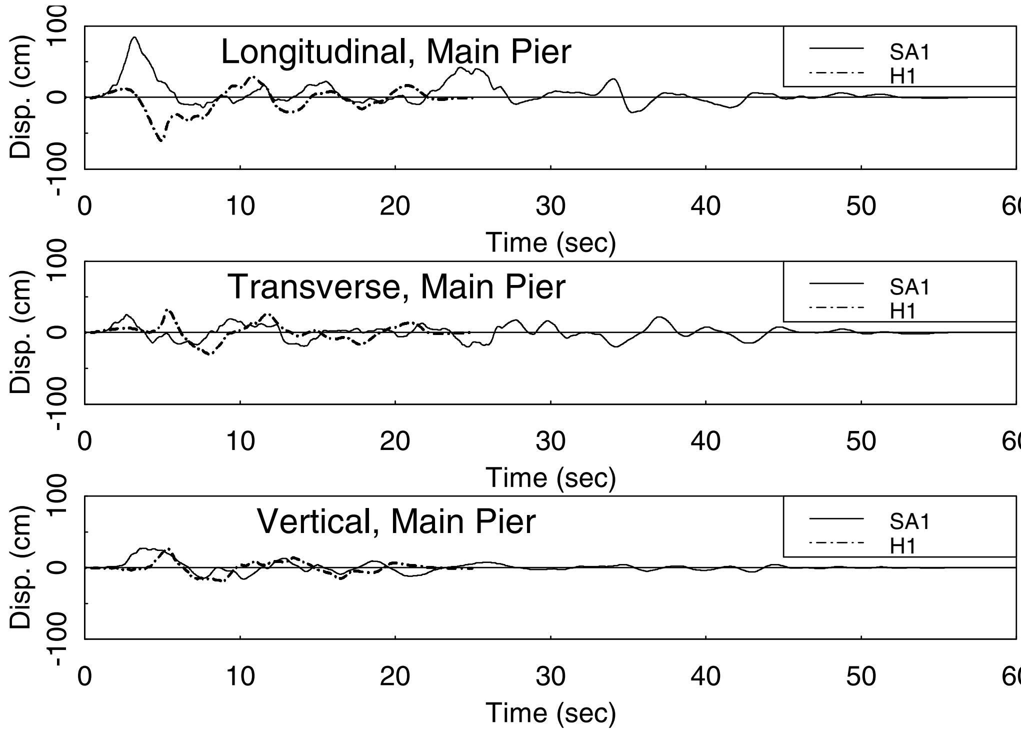

Figure 4: Comparing Displacement Time Histories from SA1 and H1 for the Main Pier

Damper Configurations

In the original SASB shear link model, there were 120 shear links, 6 at each of 20 tower elevations. In the CSB shear link model, there were 20 shear links located in pairs at 10 elevations in the bottom two thirds of the tower height. The top third contained pinned links, designed to remain elastic. For the viscous damper and friction damper studies, the SASB tower was partitioned into bays. The area between each tower shaft in the transverse and longitudinal direction was termed a tower face, such that there were two tower faces in the longitudinal direction and two in the transverse. A bay is best described as a rectangular partition of a tower face. Each face was subdivided into 52 bays along the height of the tower. Each bay could house a damper. In this manner, there could be 208 dampers in the tower, 104 oriented in each of the transverse and longitudinal directions. The CSB tower was divided similarly into 35 bays, although the bays were restricted to the bottom two thirds of the tower. The elastic links in the top third of the CSB tower were left intact.

Dampers performed best if the bay they spanned was close to square, having an aspect ratio of approximately 1.0. Due to the locations of nodes along the tower shafts in the SFOBB models, bays were seldom square. In the SASB, some bays in the longitudinal direction were 2-m wide and 4-m tall, thus having an aspect ratio of 2.0. Aspect ratios were better controlled in the CSB model, where they ranged from 0.9 to 1.3. Locations of nodes along the tower height should be revised to optimize bay aspect ratios in future studies. The current node elevations are sufficient, however, for comparing the damper and shear link options. Dampers were oriented in a zigzag pattern along the height of the tower. This was proven to be more effective than orienting all the dampers parallel to one another. Both orientations are illustrated in Figure 5.

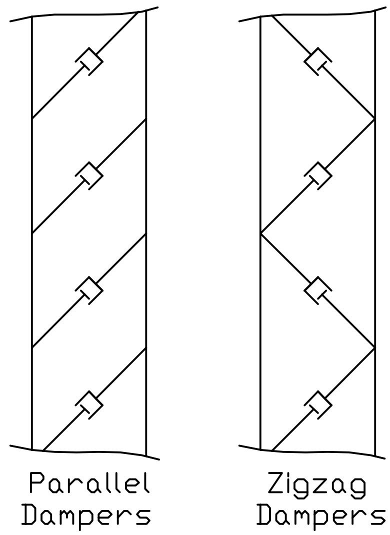

Figure 5: Comparison of Damper Orientations

Two different configurations were tested in the tower bays. Dampers were placed across the diagonal, and they were placed in a toggle-braced configuration. A toggle-braced configuration is illustrated in Figure 6. Toggle-braced configurations magnify the displacement of the damper. A horizontal damper on a chevron brace (also shown in Figure 6 for comparison) has a displacement equal to the relative displacement of its bay. Diagonal dampers (provided the damper is at a 45° angle) have a displacement of 0.707 times the displacement of the horizontal damper. This fraction is called the magnification factor (f). According to Constantinou [7], toggle-braced dampers can have magnification factors over 4.0.

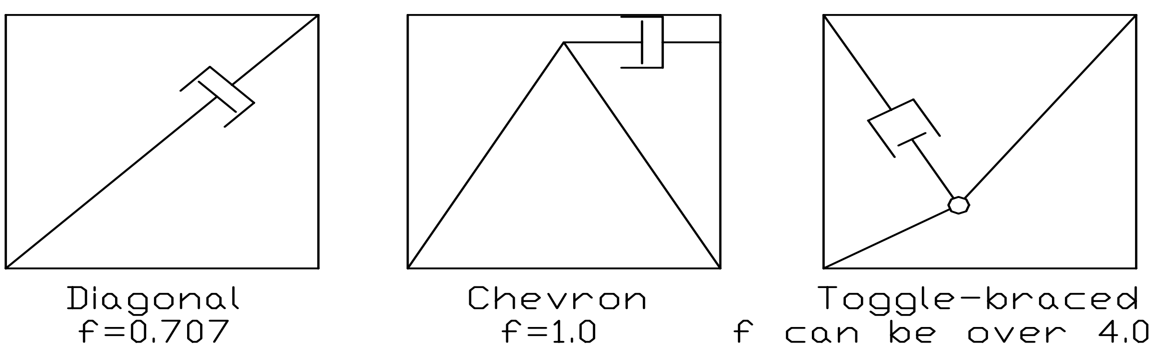

Figure 6: Comparing Damper Configurations and Magnification Factors (f).

In addition to models with dampers in all available bays, models were created with dampers in only the bottom third of the tower and dampers in only the middle third of the tower. This was done in analysis of the CSB models as well. 80 dampers were located in the bottom third and another 80 in the middle third of the tower height. In all, 36 models with dampers were created. Twelve of those were for the SASB transverse direction, 12 for the SASB longitudinal direction, and 12 for the CSB transverse direction. For each group of 12, six models contained viscous dampers and six contained friction dampers. Of each group of 6, three were of the diagonal configuration and three were of the toggle-braced configuration.

Each set of three contained a model with dampers in all available bays, dampers only in the bottom third of the tower, and dampers only in the middle third of the tower. Each of these models was analyzed in Ruaumoko3D and their performances were compared to those of the respective original shear link models.

Basis for Comparison

Several parameters were chosen as the basis for comparison to the original shear link models. The moment demand envelope of the northwest tower shaft was a major basis of comparison. The difference in moment demand for each of the four tower shafts (two in the CSB case) was small so only one was reported. The displacement demand envelope of the tower relative to its base was also compared, along with maximum tower base shear and the fundamental period of the structure. Other comparisons were made such as deck motion and shear in Piers E2 and W2, but variation in these values was small from model to model.

RESULTS

SASB Transverse Models Under SA_1

Of the four models with dampers in all bays created for the SASB transverse study, the diagonal and toggle-braced friction damper models along with the toggle-braced viscous damper model improved upon the performance of the original shear link model, as it was configured in the work by McDaniel [2]. It should be noted that additional shear links could be added to improve the response of the shear link model as well. Shear link locations were mandated by aesthetics and functionality. Toggle-braced systems were set such that magnification factors ranged from 1.0 to 2.0. Although in theory, magnification factors can be higher than 4.0 for toggle-braced dampers, this is dependent upon small relative displacements in the damper bays and bay aspect ratios close to 1.0, neither of which were the case for the SASB transverse models.

Testing models with dampers in only bottom or middle regions of the tower led to the conclusion that dampers in the middle third of the tower were more critical to the performance of the tower than those in the bottom third. This is because the bottom region of the tower was stiffer than the middle region and required less protection. Moment and relative displacement demand envelopes of the tower are presented in Figure 7 for the four models with dampers in all bays, while Figure 8 compares models with all bays damped to those with bottom and middle bays damped with toggle-braced viscous dampers. Displaced tower shapes for selected cases are also shown in Figure 7. It should be noted that tower shafts did not yield even when protected by dampers only in the middle and bottom thirds. If aesthetics or functionality mandated the use of fewer dampers, these partially protected configurations fulfill that important criterion.

Friction dampers increased the maximum base shear demand. For the diagonal and toggle-braced friction damper models, base shear demand values were 38-MN and 41-MN, respectively. The use of viscous dampers in the toggle-braced configuration resulted in a maximum base shear demand of 27-MN. Maximum base shear demand for the original shear link model was 36-MN. Base shear was reduced in the toggle-braced viscous damper model. The fundamental transverse periods of the diagonal and toggle-braced friction damper models were 3.66 and 3.67 seconds, respectively, while the toggle-braced viscous damper model had a fundamental period of 3.95. The fundamental transverse period of the original shear link model was 3.67 seconds. Fundamental periods reveal that the original shear link model and the friction damper models were roughly equal in stiffness while the toggle-braced viscous damper model was less stiff.

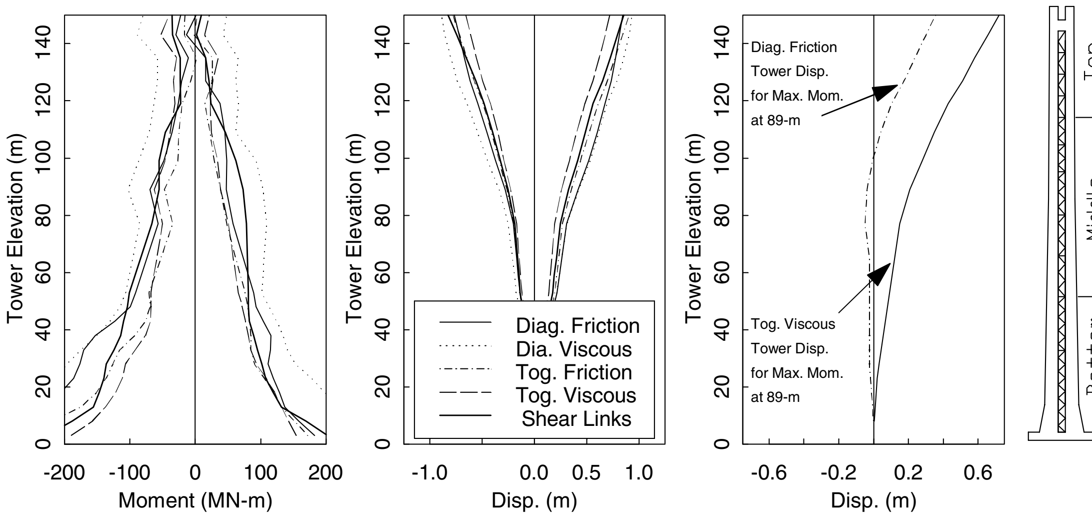

Figure 7: SASB Transverse Subjected to SA_1: Tower Moment and Relative Displacement Demand Envelopes for Models with Dampers in All Bays Compared to the Shear Link Model, and Displaced Shapes of the Diagonal Friction Damper and Toggle-Braced Viscous Damper Model Towers at the Time of Maximum Moment at Specified Elevations

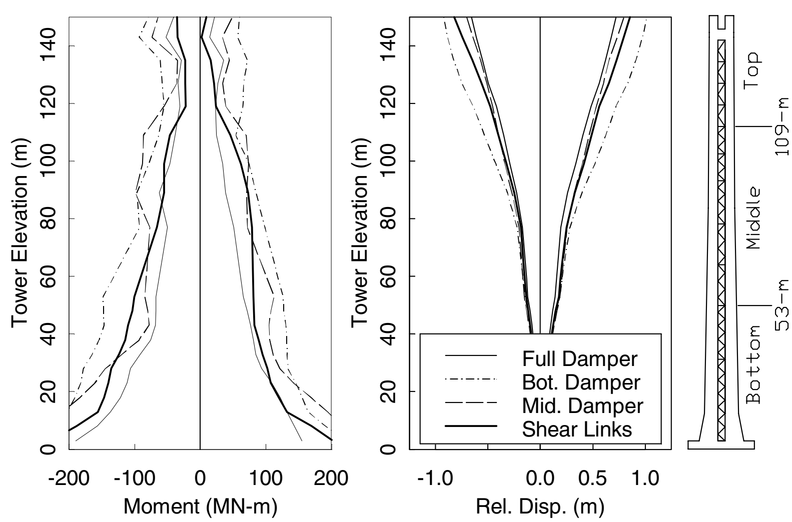

Figure 8: SASB Transverse Tower Moment and Relative Displacement Demand Envelopes for Toggle-Braced Viscous Damper Models with Dampers in All, Bottom and Middle Bays Subjected to SA_1

Upon plotting hysteresis loops for individual dampers, it was found that not all friction dampers slipped in the SASB models. Most diagonal friction dampers slipped, but fewer slipped in the toggle-braced friction damper model. This explains the higher maximum tower base shear demand in the toggle-braced friction damper model. It should be noted that if these dampers do not slip, energy is not being dissipated. Their failure to slip had to do with a modeling issue. There is a large tension force at the top of the SASB tower due to the suspension cable preload. It causes the tower to deflect under static loading. Since dampers were not installed in a computer model after static loading, as they would be in reality, they have high residual loads to begin with. Ruaumoko3D analyses will diverge if the slip force is reached before the onset of dynamic analysis, thus slip forces had to be set high in some locations and weren't always reached. This occurred especially in the top third of the tower. It should also be noted that dampers in this region would be dissipating less energy than dampers in other regions anyway, so this modeling issue is not critical to the overall bridge performance. In addition, energy is still being dissipated by lower dampers, which do slip.

In the diagonal friction damper model, slip forces in the friction dampers were 1, 3, and 8-MN, depending on their location in the tower. The slip forces should be chosen such that reasonable steel sections could be used to construct viscous dampers. Assuming slip force is to be no more than 0.6 times the gross section yield capacity, the required area based on an 8-MN slip force is 0.043-m^2. The largest necessary slip force in the toggle-braced friction damper was 20-MN, due to the modeling issue described above. The section area would have to be 0.107-m^2. Slip forces are even larger in some of the models with only dampers in the bottom and middle. The model with diagonal friction dampers in the middle region had some slip forces of 30-MN.

SASB Longitudinal Models Under SA_1

Moment and relative displacement demand envelopes for the SASB longitudinal tower are displayed in Figure 9. In the longitudinal case, the diagonal friction damper model was the only model to show improvement on the original shear link model with respect to these two demand envelopes. Displaced shape of the diagonal friction damper model tower for the time of maximum moment at the 53-m elevation (5.4 seconds) is shown in the right graph of Figure 9, since it was only at 53-m that its moment demand envelope was larger than that of the original shear link model. The displaced shape in the longitudinal direction is different from the transverse. The tower is partially restrained from translation at the top in the longitudinal direction, and less restrained from translation in the transverse direction. This displacement pattern inhibits damper performance. Note the peaks in the moment demand envelope for the toggle-braced friction damper model at the 89-m and 99-m elevations. These peaks were due to the fact that dampers didn't all slip in this area (due to high slip loads resulting from the modeling issue caused by the suspension cable discussed above). If a friction damper slipped in one location that was next to a damper that did not slip, there was usually a spike of moment discontinuity. The displaced shape for the toggle-braced friction damper model at 89-m is also displayed in the right graph or Figure 9. It also exhibits the same partial restraint at the tower top, although the restraint is not as severe.

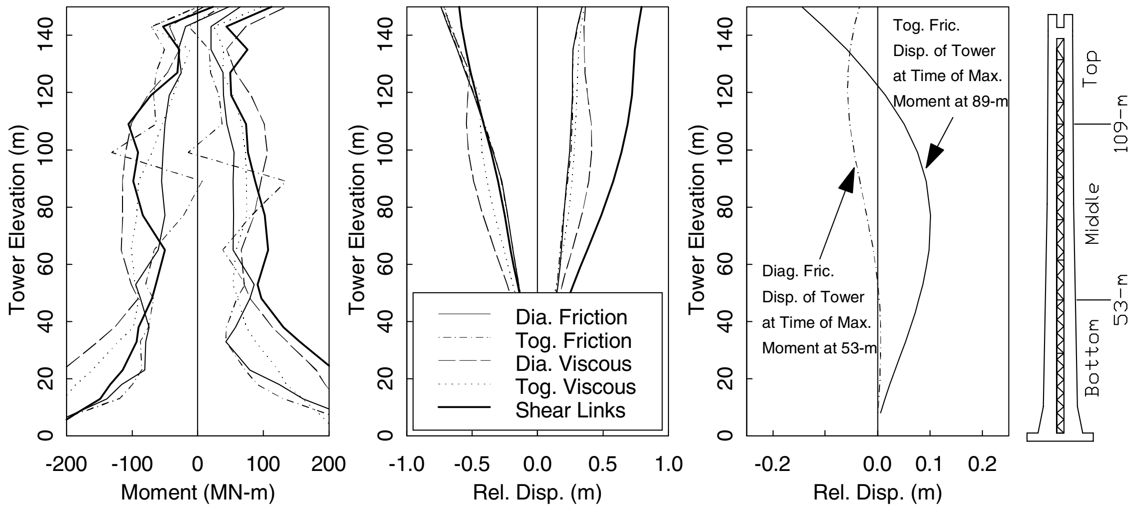

Figure 9: SASB Longitudinal Subjected to SA_1: Tower Moment and Relative Displacement Demand Envelopes for Models with Dampers in All Bays Compared to the Shear Link Model, and Displaced Shapes of the Diagonal and Toggle-braced Friction Damper Model Towers at the Time of Maximum Moment at Specified Elevations

Models with dampers in only the bottom and middle regions of the suspension tower behaved similarly to how they had in the SASB transverse study, although it was not as clear whether the models with dampers in the middle or bottom region had the better response. Tower moment demand envelopes revealed that both had about the same maximum moment, just at different locations. Moment demand is reduced more in the bottom of the tower if dampers are placed in that region and more in the middle of the tower if dampers are placed there. It's a matter of what portion of the tower is more important to protect. Most likely, the middle region would be chosen for protection due to its lower capacity as a result of the tapered shape of the tower shafts.

As evident in the tower moment demand envelope for the toggle-braced friction damper model, fewer friction dampers slipped in the longitudinal models than in the SASB transverse. This is confirmed by the hysteresis plots, which show few friction dampers slipping. There are several possible reasons for this reduced response. First, geometry was not as conducive to dampers as in the transverse. Tower shafts were only 2-m apart in the longitudinal, where they were 3-m apart in the transverse. Because of the locations of tower shaft nodes in the original model, bays in both directions were made to be the same height. That meant higher aspect ratios for the longitudinal bays. Also, the cable at the top is connected at a shallower angle in the longitudinal direction than it is in the transverse direction. Since the cable angle is shallow and the far ends of the cable are fixed to E2 and W2, the tower is partially restrained from translation in the longitudinal direction at the top. This seemed to inhibit the damper response. This restraint is illustrated by comparing the displaced shapes of the longitudinal and transverse SASB models in the right-hand plots of Figures 7 and 9.

Base shear values for the friction damper models were significantly larger than they had been for the original shear link model. Toggle-braced and diagonal friction damper models had maximum tower base shear demands of 63-MN and 60-MN, respectively. Toggle-braced and diagonal friction damper models had base shear demands of only 29-MN and 28-MN, respectively. The base shear demand of the shear link model was 41-MN. The fundamental period was decreased from 3.98 seconds in the shear link model to 3.14 seconds in the diagonal friction damper model for the SASB longitudinal direction. Although it was stated earlier that period reduction would improve response, the reduction in spectral displacement of 13-cm between a 3.14 second period and a 3.98 second period isn't significant (see Figure 10).

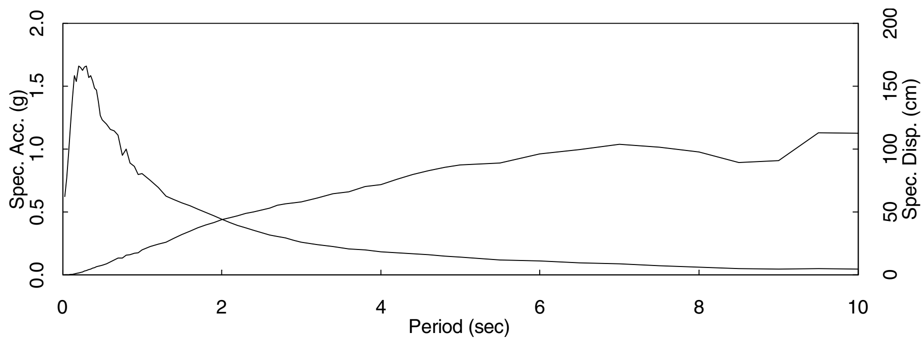

Figure 10: Spectral Graphs: Pseudo Acceleration and Displacement versus Period for the Main Pier in the Longitudinal Direction

Along with the determination of which models would improve performance, another objective of this research was to determine if the forward directivity pulse inhibited the protection provided by either the viscous damper or friction damper systems. There is evidence that dampers under-performed in the SASB longitudinal direction where the pulse was most prevalent, but it is not definitive as to whether or not this was the result of forward directivity. The geometric complications and modeling issues could also be to blame. A single degree-of-freedom study on simple portal bays protected with diagonal viscous dampers was done in conjunction with this global bridge study. The bays were subjected to nine earthquake time history records, both with and without forward directivity pulses. Upon comparing results, there was no obvious reduction in performance for those bays subjected to motions with forward directivity characteristics. From this single degree-of-freedom study it could be deduced that the under-performance of the SASB longitudinal model was due more to geometry and modeling, and less to forward directivity. Note also that the transverse ground motion had forward directivity characteristics as well (although less prominent).

CSB Models Under SA_1

Figure 11 presents envelopes and selected displaced shapes for the CSB towers with dampers in all bays of the transverse direction. A peak of high moment is visible in the friction damper models around the 119-m elevation. This is the elevation above which there are elastic links and below which there are friction dampers. The peak, therefore, is due to the large change in overall tower stiffness. Displaced shapes are plotted for the time of maximum tower moment at these peaks. Viscous dampers appear to improve upon the moment demand of the shear link model except in one place just under the 100-m elevation. Toggle-braced dampers do not show significant improvement over diagonal dampers. This is because of large displacements of the tower. Some diagonal dampers actually displace 20-cm or more. According to Constantinou [7], toggle-braced configurations are most effective for smaller displacements (10 to 20-mm).

When displacement of the diagonal damper is large, toggle-braces are only capable of small magnification factors, in this case, as low as 0.6.

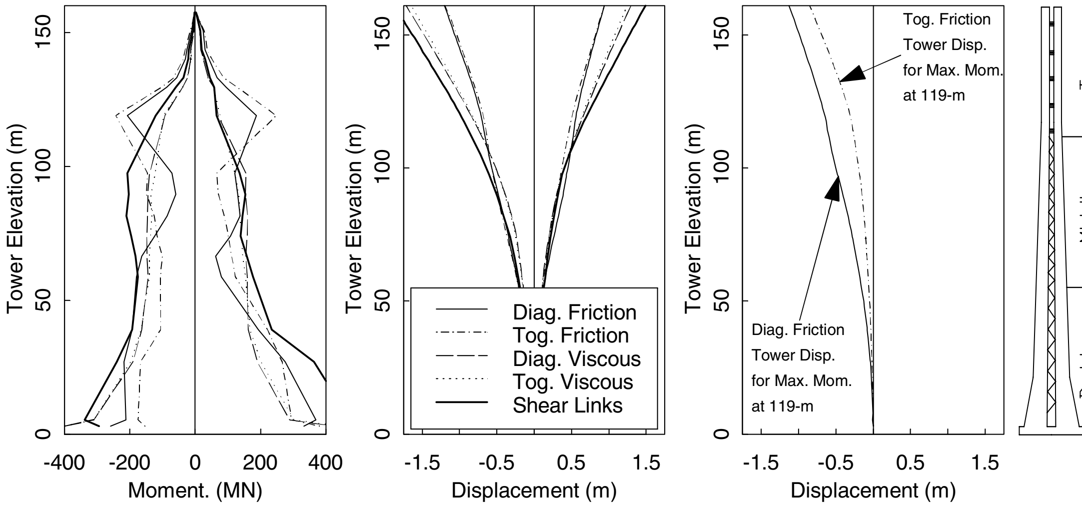

Figure 11: CSB Subjected to SA_1: Tower Moment and Relative Displacement Demand Envelopes for Models with Dampers in All Bays Compared to the Shear Link Model, and Displaced Shapes of the Diagonal and Toggle-Braced Friction Damper Model Towers at the Time of Maximum Moment at Specified Elevations

Note that the demands in the CSB model are higher than those of the SASB models. Capacities are also higher due to the larger tower shafts. Hysteretic behavior was improved for the CSB model as well. Displacement values in the tower were high, so diagonal dampers worked well. The cables were attached throughout the top third of the tower reducing the effect of the cable preload, thus the modeling problem that occurred in the SASB studies was absent from the CSB study. Friction dampers were all made to slip. Viscous dampers actually had to be limited. The damper force is based upon the coefficient of damping, C. For the SASB models, C had been 100 MN-s/m. Dampers available from Taylor Devices, Inc. [7] are limited to 8896-kN of damper force and this C-value had never reached that damper force in the SASB models. In the CSB model, however, C had to be limited to 30 and 50 MN-s/m to meet the force limit. Had the limit been ignored (by assuming that each modeled damper represented several actual dampers), viscous damper models could have been even more effective.

Models with dampers in the bottom and middle regions showed the same results as they had in the SASB models. None of them improved upon the protection provided by the original shear link configuration, but dampers in the middle region appeared to be more critical in the protection of the tower. This again is probably due to the difference in stiffness in the bottom and middle regions.

The results of maximum tower base shear were consistent with those seen in the SASB model as well. Base shear was higher for the friction damper models, around 100-MN for both, and lower for the viscous damper models. Viscous dampers didn't reduce base shear as well as they had in the SASB model though. The shear link model had a maximum base shear of 86-MN, and the diagonal and toggle-braced viscous damper models had base shear values of 85 and 82-MN, respectively. Upon running a viscous damper model with higher C-values though, base shear was reduced significantly. The base shears in the viscous damper models are high because the tower shafts are stiff and the C-values are limited. Fundamental periods support the maximum base shear findings. They are higher than the shear link model period for friction damper models, and slightly lower for the viscous damper models.

Models Under H_1

As a final step in this study, fourteen models were analyzed under the governing Hayward Fault rupture scenario, H_1. From the comparative plot of time histories in Figure 5, it can be seen that H_1 and SA_1 differ, both in duration and shape. The affect of running models under each of these was not so different though. The 14 models run under H_1 had responses close in comparison to those run under SA_1. Figure 12 presents comparative plots of moment and relative displacement demands for two of these models under each of the earthquake ground motions.

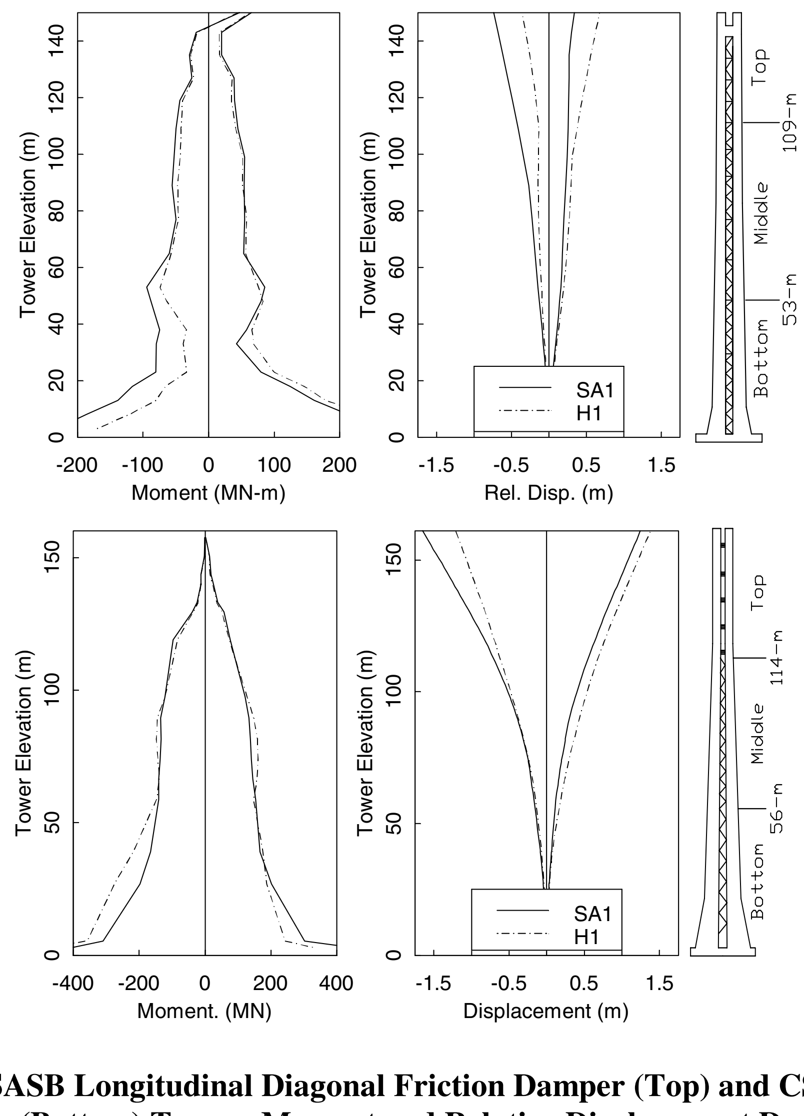

Figure 12: SASB Longitudinal Diagonal Friction Damper (Top) and CSB Diagonal Viscous Damper (Bottom) Towers Moment and Relative Displacement Demand Envelopes Comparing SA_1 and H_1

CONCLUSIONS

The main goal of the passive energy dissipation systems was to protect the suspension tower shafts from inelastic response under the SEE. All of the damper protection systems created for this study were successful in that respect. No single passive damping configuration improved upon the original shear link model in all of the cases. Toggle-braced dampers worked well in the SASB transverse direction because higher magnification factors could be achieved there than in the other model-types. Of the diagonal damper models in the SASB transverse direction, the friction damper model performed the best, showing improved performance over the original shear link model. Even tower base shear was kept close to the same level as it had been in the shear link model. This usually wasn't the case when friction dampers were added. Friction damper models also had periods close to those of the shear link model. If a viscous damper with a larger C-value were used or viscous dampers were placed in a toggle-braced (or maybe even a chevron) configuration, they could be effectively used as well.

In the longitudinal direction, the response of the damper-protected SASB tower wasn't as good. Fewer friction damper slip forces were reached. This is due to a combination of a modeling issue, since dampers could only be added to the computer model before static loading, and tower fixity due to the suspension cable restraint, which causes a different motion at the tower top than witnessed in the transverse direction. Also, forward directivity is more prominent in the ground motion record for this direction than in the transverse. The diagonal friction damper model provided the best protection of the SASB longitudinal tower, according to the tower moment and relative displacement demand envelopes. It was the only damper model that improved upon the protection provided by the shear links. The maximum tower base shear value was increased though, from 41-MN in the shear link model to 60-MN in the diagonal friction damper model. Friction dampers reduced the fundamental period of the SASB longitudinal model, but not significantly.

Diagonal friction dampers worked well in the CSB models. Slip forces were limited to 7-MN, easily achievable with a modest brace cross-section. Friction damper models did have a few downsides in the CSB. They would cause a spike of high moment demand at the 119-m elevation due to the change in stiffness at that level. They also caused very high shear values at the tower base. Consequently, viscous dampers performed the best in the CSB tower. They reduced moment and relative displacement demand while keeping base shear levels close to the value it had been for the CSB shear link model. The downside was that their fundamental period was 4.4 seconds, compared to 3.9 in the diagonal friction damper model. Still, that increase is small when considering it only results in a spectral displacement difference of approximately 8-cm. Toggle-braced dampers did not work well in the CSB due to large displacements in the tower bays. Displacements here were even larger than those experienced in the SASB longitudinal models. Toggle-braced configurations could only be set to magnification factors between 0.6 and 0.9. Chevron configurations would at least yield magnification factors of 1.0.

Models with dampers in only the middle and bottom regions were successful in preventing yielding of the CSB and SASB tower shafts, although, in the case of friction damper models, slip forces would often have to be set so high that very large members (as large as 0.161-m^2) would be needed for friction damper construction. Even so, running these models provided evidence that dampers are more critical in the middle section of the tower then they are in the bottom third. This was the case for the SASB in both directions and for the CSB.

It was also a goal of this study to decide whether forward directivity caused adverse response in the damper-protected towers. The forward directivity pulse was most prominent for the longitudinal direction, which was also the direction that showed the poorest performance. It cannot, however, be presumed that the forward directivity pulse was completely responsible for this underperformance since geometric and modeling issues were also influential in that direction, and single degree-of-freedom analyses have shown that forward directivity has only minimal influence on passive damper performance.

None of the models created for this study are the optimized solution. In the future, combinations of dampers should be studied. Viscous dampers could be put in the areas where friction dampers don't slip as much. Toggle-braced configurations could be used in combination with diagonal dampers in the same tower. Diagonal dampers would be optimal for bays that displace more, and toggle-braced dampers could be installed in the bays where displacement is minimal. The chevron configuration should also be considered. It could actually take the place of all diagonal dampers, increasing of displacement magnification by a small amount. An in-depth economic analysis should also be performed, comparing cost of maintenance, materials and construction for each configuration attempted.

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