Proposed Retrofit of Alaskan Way Viaduct Using Fluid Viscous Dampers: Preliminary Phase

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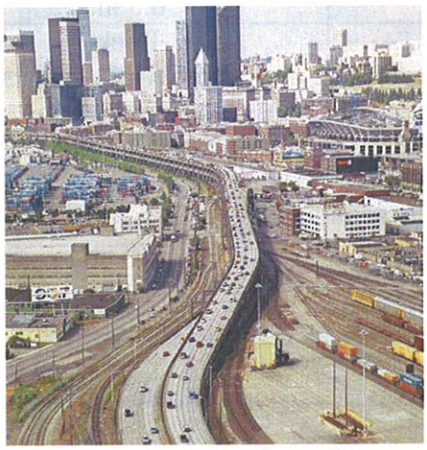
1 Overview

Preliminary time history analysis of a typical three-span frame of the Alaskan Way Viaduct was conducted. The scope of the work was to investigate the response of the superstructure in the existing configuration and after the retrofit with fluid viscous dampers.

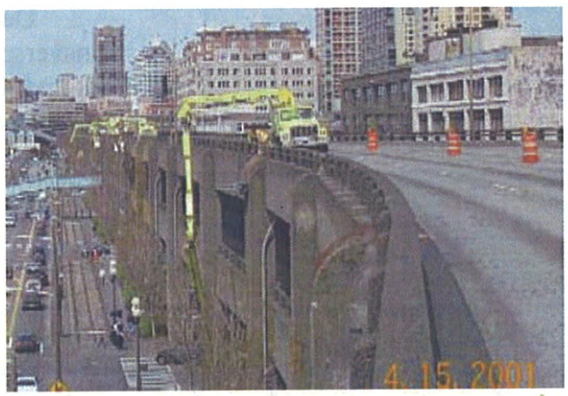
The analysis was limited in scope to one of the frame types and it relied heavily on the findings of previous researchers to determine the capacity of the frame and its failure modes. For assessment, the design criteria and level of seismic input developed by others was used.

2 Description of the structure

The Alaskan Way Viaduct, hereafter referred to as the Viaduct, is a 2.1-mile long two-level elevated structure located along the Seattle seashore. The viaduct, also known as State Route 99 or SR 99, was designed and constructed in the 1950s. Figure 1 presents photographs of the viaduct.



a. Panoramic view



b. Double-deck elevation

Figure 1. Alaskan Way Viaduct (From WSDOT web site)

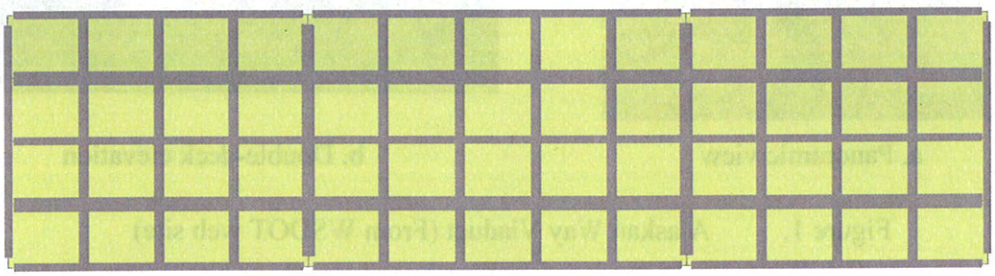
The viaduct is comprised of a suite of different units, including outrigger frames, frames with steel framing. A great portion of the structure is made of three-span reinforced concrete double-deck frames. Frames measure 182 ft long, are 47 ft wide and the deck elevations are 36.5 and 58.5 ft above the top of footing fore the lower and upper decks, respectively. The exterior spans are 56-ft long and the interior span is 70 ft long. To allow for thermal movements, a 1.5-in. wide longitudinal gap is provided between adjacent frames. Typical frames weigh approximately 5,000 kips. All columns are supported on pile supported footings.

Two types of framing were used in construction. The Southern segment was designed by the Seattle Engineering Department (SED), whereas the Northern frames were designed by the Washington State DOT (WSDOT). The two frame types are similar in geometry. The main difference is the load path for the gravity loading from the decks to the columns. SED units use smaller transverse beams and large longitudinal girders to transfer the load to cap beams,

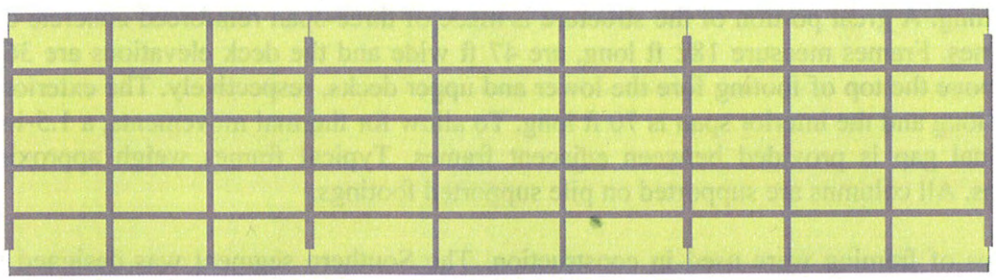
whereas, the WSDOT frames utilize longitudinal stringers and large transverse beams to transfer the load to edge girders. Table 1 summarizes the member sizes for the frames, and Figure 2 shows the gravity framing.

	SED	WSDOT
Exterior columns	48x24	48x24
Interior columns	48x48	48x42
Exterior caps	75x16.5	60x18
Interior caps	75x29	60x18
Edge girder	75x16	88x18
Interior longitudinal beams	Two 38.5-64.5 (haunched)x29 One 19x17 centerline	Four 28.5x13.5
Internal transverse beams	19x20 (three end spans, four mid span)	63.5x15 (three each span)
Deck	6.5 in.	6.5 in.
Gravity framing	Deck transverse stringers longitudinal girders cap beams columns	Deck longitudinal stringers transverse beams exterior girders columns

Table 1. WSDOT and SED framing



a. SED frame



b. WSDOT frame

Figure 2. SED and WSDOT frames

The two types of frames have similar elastic and dynamic properties. However, the reinforcement, structural detailing (splice, connections) differs between the two frames. As such, the expected seismic performance of the frames in the inelastic range and possible modes of failure would also vary between the frames. Since the scope of the analysis in this report is to limit the extent of nonlinear behavior by adding supplementary damping to the system, it would suffice to perform analysis on one of the frame types. As such, for the remainder of this report, the discussion is limited to typical SED frames. Figure 3 present a photograph of a SED framing.

Figure 3. Photograph of typical SED frames

3 Previous research

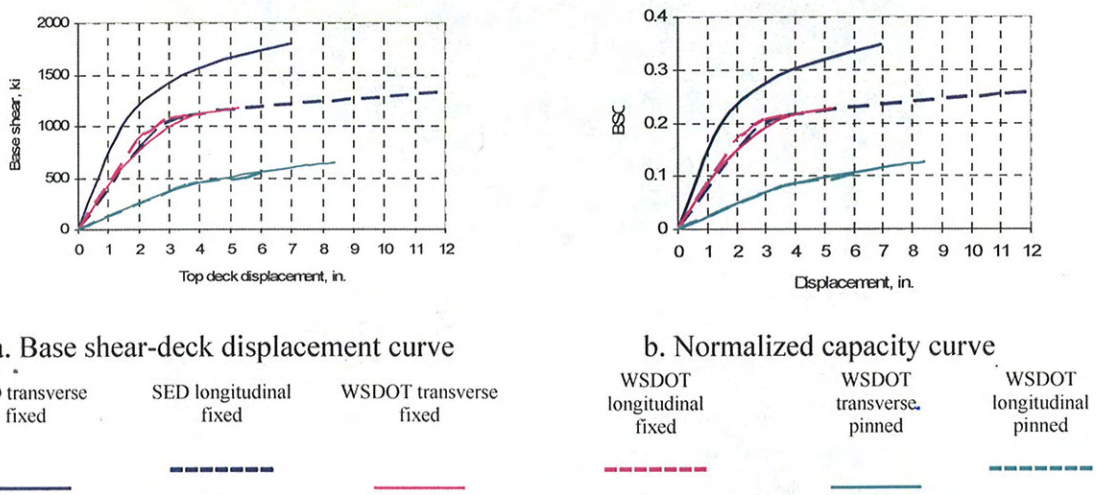
After the collapse of the double-deck Cyprus viaduct during the Loma Prieta earthquake in Oakland, CA, in 1989, WSDOT conducted a comprehensive investigation of the viaduct structure. Both geotechnical and structural evaluations were conducted. Following the Mw 6.8 Nisqually earthquake, WSDOT conducted field investigations, sponsored further analysis and developed design criteria for retrofit/replacement structure. The findings are summarized here.

3.1 Eberhard 1995-a and 1995-b

Eberhard conducted three-dimensional linear dynamic (modal and response spectrum) and two-dimensional nonlinear static analysis of typical SED and WSDOT frames and their longitudinal and transverse bents.

Dynamic analysis showed that for the fixed based frames, the fundamental periods of 0.8 to 0.9 sec. For pinned based frames, the fundamental periods were 1.6 to 2.0 sec. WSDOT and SED frames had similar modal properties. For all cases, the first mode response dominated and had mass participation of close to 100% of total seismic mass. Response spectrum analysis showed that for either fixed or pinned based models, most deformation was considered for the lower level. The drifts at the lower level were three to ten times that of the upper level. As such, soft-story type of response was prominent.

Pushover analysis was conducted to evaluate the capacity of frames. Only ductile flexural hinges were included in the model. Figure 4 presents the normalized pushover curves. The Eberhard 1995a and 1995b data were converted from base shear to base shear coefficient, BSC, (dividing by weight of the frame of 5,200 kips). The pushover curves are shown in Figure 4. Results for SED and WSDOT units in longitudinal and transverse directions are shown in Figure 4. For the SED unit only pinned base condition was evaluated; for WSDOT unit both support conditions were investigated. Since the frames respond primarily in the first mode, the pushover curves can also been considered as spectral displacement-spectral acceleration capacity curves.



a. Base shear-deck displacement curve

b. Normalized capacity curve

Figure 4. Pushover curves, Eberhard 1995a and 1995b

The response is essentially elastic up to a BSC of 0.2g for the fixed support case. The pinned models exhibit elastic response up to approximately 0.1g. Minimal yielding is expected up to a displacement of approximately 2 in. for the fixed based frames and 3 in. for the pinned based frames.

The pushover analysis did not explicitly account for the potential brittle modes of failure. Eberhard 1995a and 1995b conducted determined that the frames would not be able to develop their full flexural capacity because each type of frame was susceptible to some of the following inadequacies: joint shear, column shear, bar pull out, reinforcement splice, or plastic hinge ductility. Eberhart tabulated the likelihood of each type of failure and produced a priority table for the areas of concern.

3.2 TY Lin 2001

After the February 28, 2001, Nisqually Earthquake (Mw 6.8) about 35 miles south of Seattle, WSDOT engineers noted that a structural unit had experienced significant damage. The rest of the viaduct did not experience major structural damage. Immediate repairs were undertaken and WSDOT initiated a committee for the long-term evaluation of the viaduct.

TY Lin 2001 evaluated the complied previous research, and developed retrofit/replacement design guidelines. The governing seismic level was selected as having a 10% probability of exceedance in 50 years (15% in 75 years or 500-year event) and the target performance was selected as collapse prevention at this seismic level. TY Lin 2001 concluded that:

- Nisqually (PGA of approximately 0.19g for stations within 1,000 ft of the viaduct) had a return period of 150 years.
- Estimated the earthquake that would cause collapse of a major portion of the viaduct at the 210-year event.
- Computed a threshold spectral acceleration of 0.26g, for frames with a period of 1.5 sec. This threshold was based on the pushover analysis of a unit 97-100 frame for which the column supports as springs to account for support flexibility.
- Stated that the level of earthquake loading in the design of the WSDOT frames was 0.1g.
- Compared the viaduct and the Cyprus Viaduct and stated that the frame hinges and girder expansion joints in the Cyprus Viaduct contributed to its collapse during the Loma Prieta earthquake..
- Developed response spectrum using USGS maps for the 500-year event and for the 1949 Puget Sound event with a return period of 200 years.
- Given the maintenance issues and that the viaduct's relatively advanced age, recommended replacement.

3.3 PBQD 2002

Parsons Brinckerhoff Quade & Douglas (PBQD 2002) assessed the seismic performance of the viaduct and looked at retrofit and replacement options. They conducted a pushover analysis of a specific frame. The pushover curve is shown in Figure 5. PBQD 2002 does not provide specifics of their analysis. However, from their pushover curve it appears that strain hardening, loss of strength in hinges were included in the model. From the level of BSC, it seems that their model used spring supports—and a period of 1.5 sec—similar to a value used by TY Lin 2001.

Figure 5. Capacity pushover curve for Bent #152, PBQD 2002

PBQD 2002 stated that

- Viaduct response to be evaluated using two levels of earthquake, one with PGA of 0.13g and peak spectral acceleration of 0.33g, and one with PGA of 0.65g and peak spectral acceleration of 1.61g. Although not stated, these values appear to be for site class B (Fa

and Fv equal to unity). The first spectrum is used for functionality (immediate use) and the second for life safety (collapse prevention).

- Viaduct joints can experience brittle failure and there is no method of repairing the knee joints.
- SED frames would sustain damage at spectral acceleration of 0.13g, and structural limit would be reached at 0.25 to 0.30g.
- WSDOT frames would sustain damage at spectral acceleration of 0.10g, and structural limit would be reached at 0.25 to 0.30g.
- Rebar slip starts at a column moment of 5,000 k-ft. At the same time the joint has also reached its principal stress limit of $5\sqrt{f'_c}$. This corresponds to a BSC of 0.24g.
- Initial joint cracking (principal stress of $3.5\sqrt{f'_c}$) starts at BSC level of 0.18g.
- Retrofit/rebuild option must have seismic resilience and energy dissipation capabilities.

3.4 PBQD 2005

Structural design criteria for the viaduct were developed in PBQD 2005. The preliminary design criteria identify two target performance points for the viaduct: 1) immediate functionality for the Expected Earthquake or EE (50% in 75 years or return period of 108 years), and 2) collapse prevention for the Rare Earthquake or RE (3% in 75 years or return period of 2500 years). The seismic hazards have the following characteristics shown in Table 2.

Level	PGA, g	SS, g	S1, g
EE	0.14	0.34	0.14
RE	0.55	1.37	0.66

Table 2. Design response spectra for viaduct

Note that the design response spectra parameters different by what was used by both TY Lin 2001 (10% in 50 years) and from PBQD 2002 (different Ss and PGA for RE event).

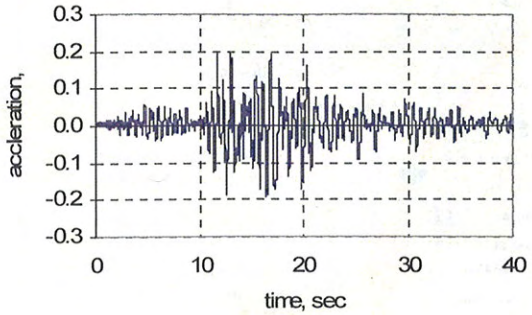
4 Nisqually earthquake

The Mw 6.8 Nisqually earthquake of February 28, 2001, provides a great opportunity to investigate the response of the viaduct to a seismic event. Figure 6 shows a map of the viaduct and the surrounding area. Three stations recorded the strong motion data near the viaduct; see Figure 6 (red dots) and Table 3. (The viaduct coordinates are approximately 47.60 and -122.34 degrees.) Figures 7 and 8 depict a 40-second window of recorded accelerations and the corresponding acceleration response spectrum for the EW and NS components of records at these stations, respectively. Note that the accelerations had strong motion duration of 10 to 20 sec, and the response spectrum was rich in frequency content.

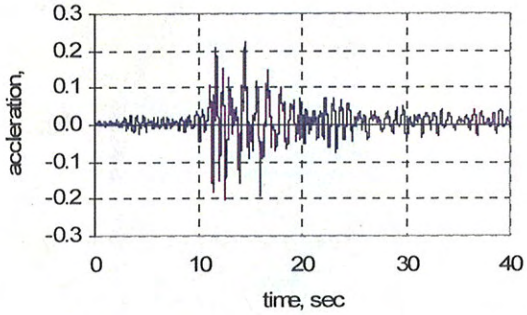
Figure 6. Map of viaduct and recording stations
(redmark indicates location of recording stations)

Station	Latitude	Longitude	PGA NS	PGA EW
NOR	47.6007	-122.3320	0.2	0.22
KDK	47.5951	-122.3336	0.19	0.15
HAR	47.5837	-122.3501	0.21	0.18

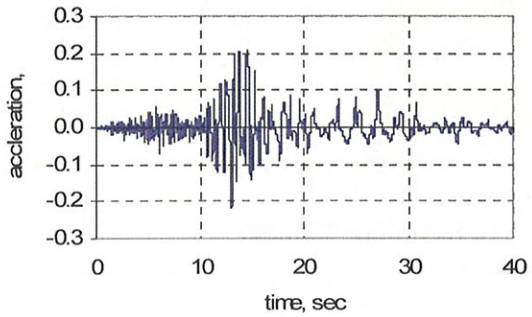
Table 3. Strong motion data station and recorded PGAs



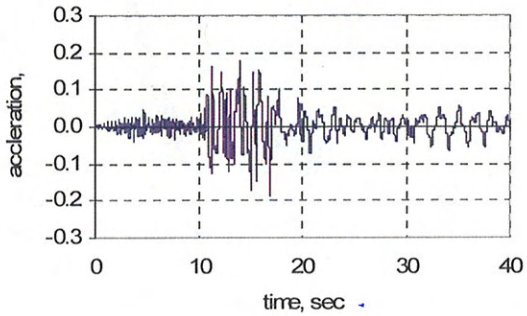
a. Station NOR, zero-degree component



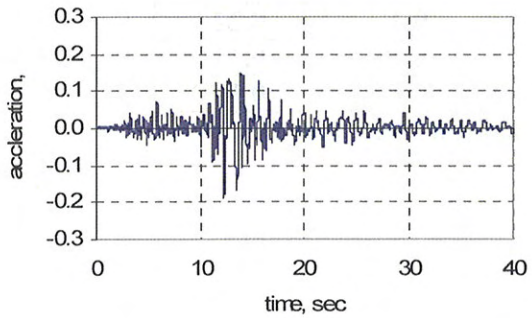
b. Station NOR, ninety-degree component



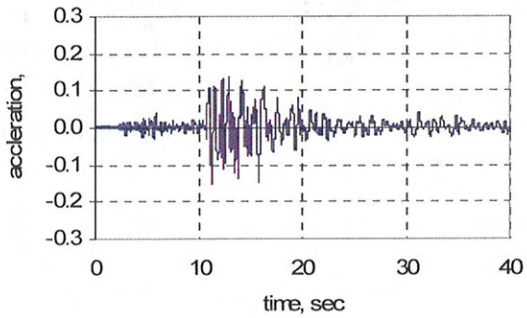
a. Station HAR, zero-degree component



b. Station HAR, ninety-degree component

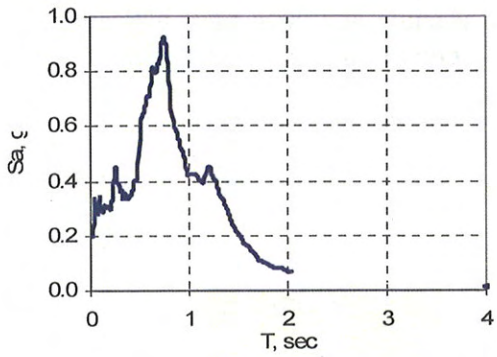


a. Station KDK, zero-degree component

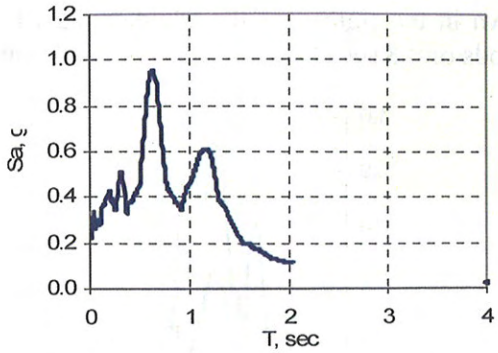


b. Station KDK, ninety-degree component

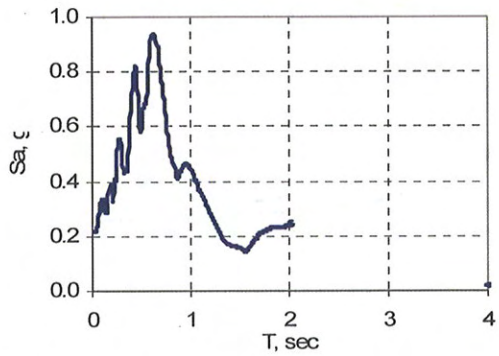
Figure 7. Acceleration records



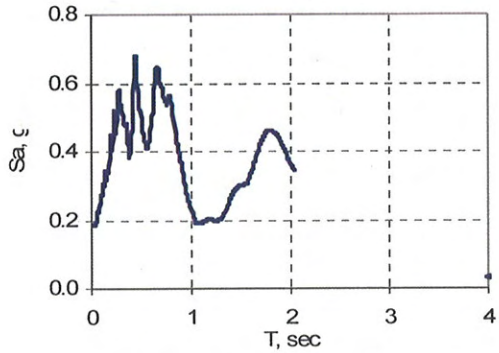
a. Station NOR, zero-degree component



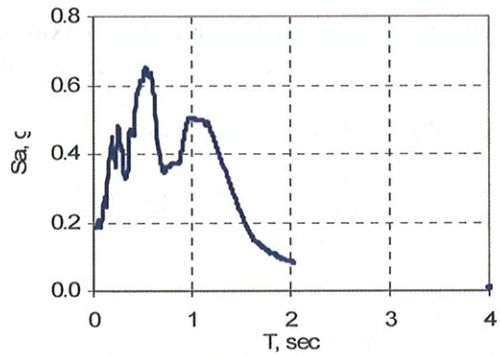
b. Station NOR, ninety-degree component



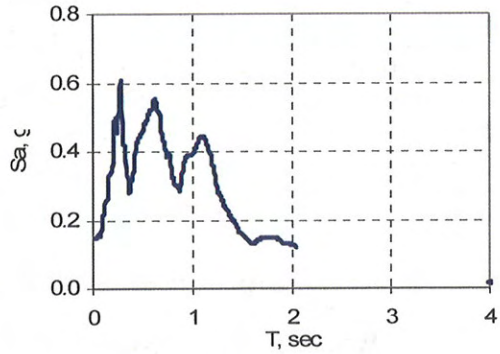
a. Station HAR, zero-degree component



b. Station HAR, ninety-degree component



a. Station KDK, zero-degree component



b. Station KDK, ninety-degree component

Figure 8. Response spectra, 5% damped

Figure 9 presents the average and maximum of spectral accelerations for these six records. Also shown in the figure are the 5%-damped EE response spectum and three lines indicating three periods of 0.8 sec (fixed), 1.8 sec (pinned), and 1.5 sec (spring supported) for a typical frame.

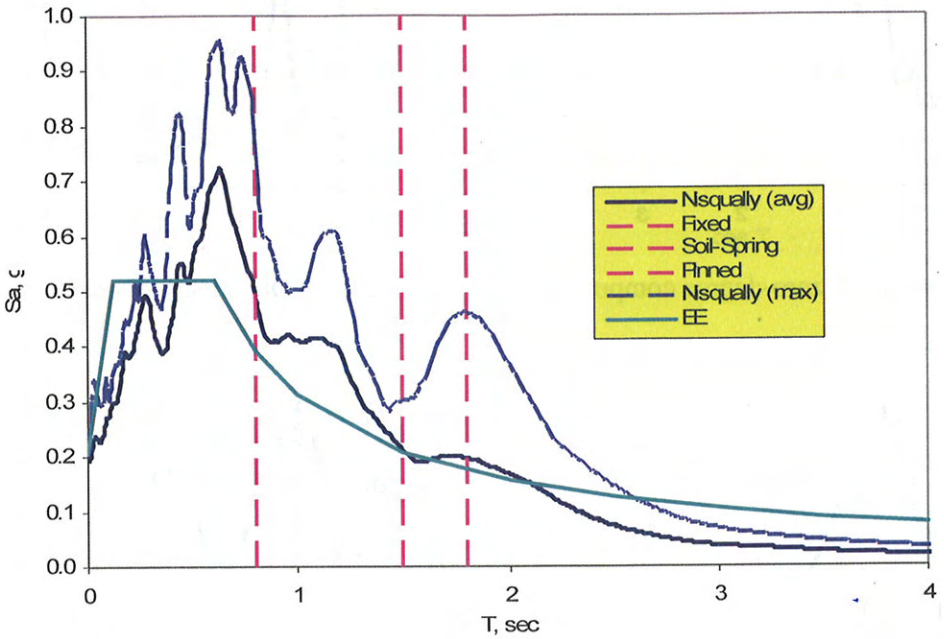


Figure 9. Nisqually and EE spectra

Note the following:

- The average spectrum approximates or exceeds the EE spectrum and the maximum spectrum greatly exceeds the EE spectrum. Thus, it appears that the Nisqually earthquake was a more severe test than the EE event.
- The recorded spectral amplitudes for the periods of interest are listed in Table 4

	T, sec	EE	Nisqually Average	Nisqually Maximum
SED/WSDOT fixed	0.8	0.39	0.49	0.79
Spring-supported	1.5	0.21	0.21	0.30
WSDOT pinned (avg)	1.8	0.16	0.20	0.46

Table 4. Spectral accelerations at target periods

Previous studies (TY Lin 2001 and PBQD 2002) had stated that the viaduct frames—with a period of 1.5—would experience yielding at Sa of 0.1 to 0.13g, would reach their capacity at 0.25-0.3g, and experience bar pull out and joint failure at 0.24g. Entries of Table 4 show that the viaduct did reach most of these target values. However, the structural damage to the viaduct was limited. This is noted by TY Lin 2001. This better-than-expected performance of the viaduct

during the Nisqually earthquake points to the conservatism inherent in the above-mentioned studies.

5 Objective of current study

5.1 General

The objective of the analysis was to assess whether fluid viscous dampers can be used to retrofit the viaduct and meet its performance objectives as defined in PBQD 2006. Figure 10 presents the 5%- and supplementary-damped target response spectra for EE, DBE, and RE events.

Note that if this level of high damping was achieved, the response of the viaduct would then meet the seismic objectives, because the RE acceleration would then equal to or be less than the spectral ordinates computed from the accelerations causing maximum response at that period from the three recording stations during the Nisqually ground motion. The effectiveness of added damping was also noted by PBQD 2002 when they discussed energy dissipation capacity. The only difference being that PBQD 2002 discussed energy dissipated provided by new designed and detailed ductile concrete members.

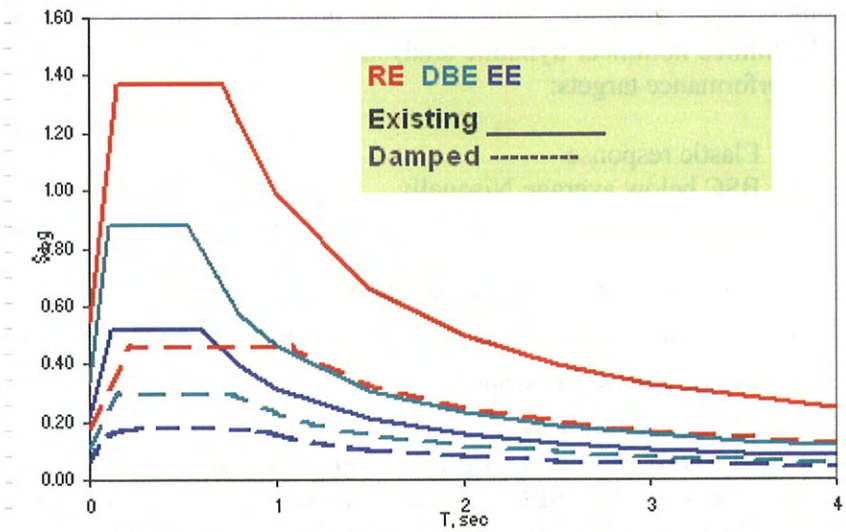


Figure 10. Original and damped response spectra

5.2 Analysis procedure

Limited nonlinear time history analysis was used for evaluation. Nonlinearity is limited to discrete FVD elements. Since the periods of the structure are large enough, the concept of equal displacement is used. This principal implies that the elastic and inelastic structures would have equal displacements, although, the level of force in the yielding structure is smaller; see Figure 11.

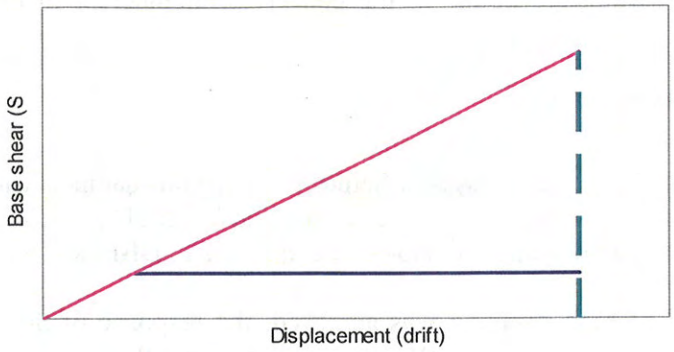


Figure 11. Principal of equal displacement

5.3 Evaluation procedure

Using the concept of equal displacement assumption, the data computed in previous analysis from nonlinear pushover analysis can be used for comparison. All the members were modeled using elastic beam-column elements. However, the nonlinear behavior can be compared accurately using the limited nonlinear dynamic analysis presented herein. Dampers were sized to meet the following performance targets:

- EE level
  a. Elastic response
  b. BSC below average Nisqually

- RE level
  a. Upper deck displacement similar to Nisqually
  b. Joint shear to $5\sqrt{f'_c}$ or lower level column moment of 5,000 k-ft
  c. Lower level column shear

6 Mathematical model of the building

Program SAP2000 (CSI 2006) was used to prepare mathematical models for the building. A typical three-span frame designed by SED was modeled and shown in Figure 12. The member sizes and frame geometry were selected per structural plans.

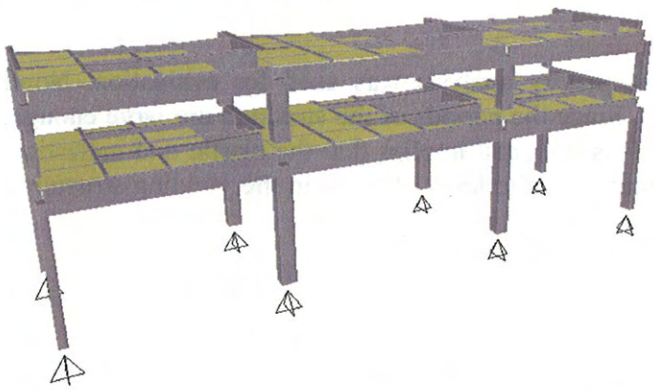


Figure 12. Three-dimensional model of the building

Member centerline dimensions were used in analysis. The joints were modeled as rigid and had dimensions equaling to member sizes framed by beams. Beams and columns were modeled using beam-column elements and decks were modeled using shell elements including both flexural and membrane action. P-Δ was included in analysis. To account for cracking in concrete, Icr/Ig was selected at 0.5 for beams and columns—same value as previous studies.

7 Analysis results

7.1 Dynamic properties

Table 5 presents a comparison of the current and the Eberhard 1995a model. Since the two models have similar mass, stiffness, and dynamic properties, they are dynamically equivalent.

		Eberhard 1995a	Model
	Base	Fixed	Fixed
	W, kips	5180	5160
Longitudinal	T, sec	0.86	0.81
	Participating mass, %	96	98
Transverse	T, sec	0.77	0.74
	Participating mass, %	96	98

Table 5. Properties of the analysis model and Eberhard 1995

The analysis conducted by TY Lin 2001 and PBDQ 2002 had a fundamental period of 1.5 sec in the transverse direction. For the current analysis, the bases of the columns were restrained against translation and rotational springs were placed at the base to simulate the boundary condition of columns. The fundamental periods of this model were 1.67 and 1.52 sec in longitudinal and transverse directions, respectively. As such, it is expected that the behavior of this frame would simulate the previous analysis and the SED frames in the field.

7.2 Seismic loading

Self-weight gravity and seismic loading were applied to the models. Six pairs of two-component accelerations were used in analysis. For each pair, two analyses were conducted. One with the first and second components aligned with the frame's longitudinal and transverse axis, respectively. The other analysis was conducted, with the acceleration components rotated by ninety degrees. The acceleration records used in analysis were comprised of the following.

- Three uncorrected Nisqually acceleration records of three stations—located within 1,000 ft of the viaduct—of Table 3. (The acceleration history and response spectra of the records are shown in Figures 8, and 9, respectively.)
- Three records from accelerations whose spectrum approximately matched the 5%-damped EE, DBE, and RE events, where EE and DBE and RE stand for Rare-, Design Basis-, and Rare - Earthquakes. The definition of these events is presented in Table 6. PBDQ 2005 guidelines were used to obtain the spectral values for EE and RE events, whereas, USGS web site was used to determine spectral ordinates of DBE events. Site Class D was used to compute modified response spectra ordinates. The acceleration response spectra for EE, DBE, and RE for Site Class D are presented in Figure 13.

Performance level	Probability of occurrence in 75 years	Return interval, years	PGA, g	Ss, g	S1, g
EE	50%	100	0.14	0.52	0.31
DBE	15%	500	0.33	0.88	0.46
RE (MCE)	3%	2,500	0.55	1.37	1.0

Table 6. Seismic design performance levels

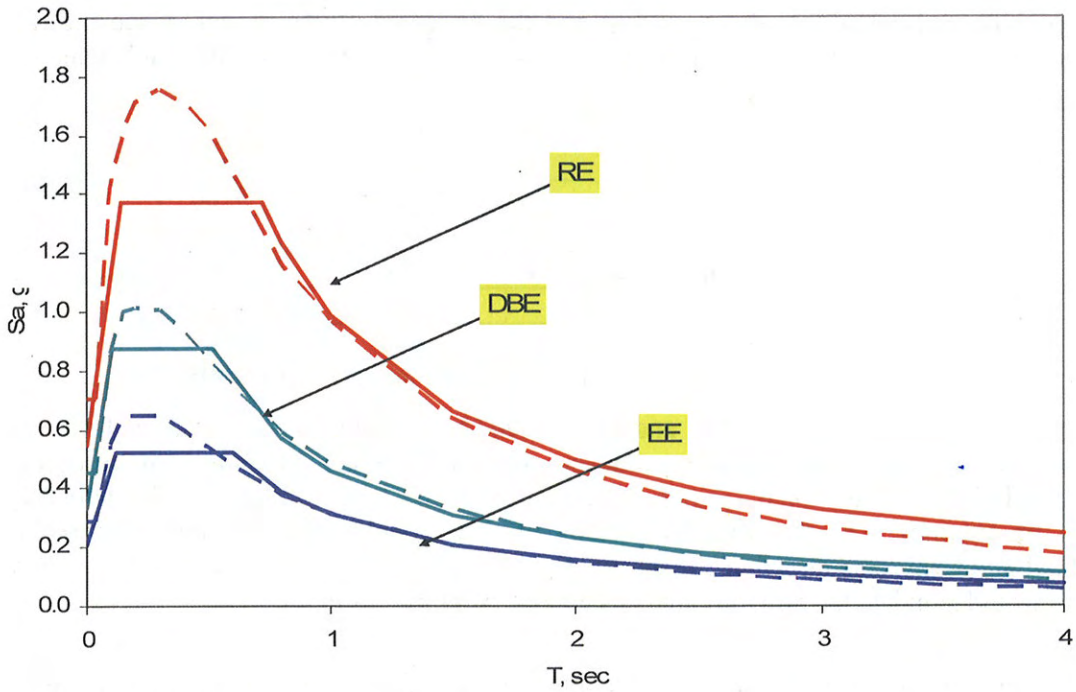


Figure 13. Target and analysis spectra

7.3 Response of the existing SED frame

Table 7 summarizes the maximum computed displacement at the upper deck level of the existing frame for each of the records. The computed response from the Nisqually records is similar to the EE level. As such:

- It is expected that the viaduct will experience minimal damage during the EE event, based on the 2001 Nisqually earthquake.
- The response of existing frame is not satisfactory for the RE event.
- If the viaduct displacement at the RE level is reduced to the levels experienced during the Nisqually event, then the structural damage will be limited during the rare event. This reduction in viaduct response—for example, from 15 in. to 4 in. in upper deck displacement—will be archived by adding supplementary FVDs to the frames to reach the target spectra of Figure 10.

Record	x-component, in. (longitudinal)	y-component, in. (transverse)
EE	4.6	4.4
DBE	7.3	7.0
RE	14.9	14.1
NOR	4.2	4.4
HAR	4.4	3.2
KDK	3.8	4.6

1. Maximum of two-component analysis
2. Response from one of the HAR component was large and discarded (possibly due to a peak in spectra matching frame period)

Table 7. Maximum computed upper deck displacement, existing frame

7.4 Fluid Viscous Dampers

Although there are several methods to add supplementary damping to the viaduct, Fluid Viscous Dampers (FVDs) are selected. The supplementary damping would serve to reduce the seismic demand on the viaduct to the target level. FVDs are selected because the force in them is mainly proportional to velocity. As such, they do not stiffen the structure (and add more seismic load) or increase load on the connecting frames or foundation by a significant amount. FVDs were initially developed for the defense and aerospace industry. They are reliable, efficient, and robust. FVDs have been extensively and successfully used for hundreds of seismic retrofit of buildings and bridges worldwide. Figure 14a present the application of FVDs for seismic of Immunex Central Utility Plant located in Seattle waterfront north of Alaskan Way Viaduct. Figure 14b depicts the FVD installed as part of the seismic retrofit of the SF-Oakland Bay Bridge.

a. Immunex Central Utility Plant

b. SF-Oakland Bay Bridge

Figure 14. Application of FVDs for seismic retrofit

7.5 Seismic retrofit

The mathematical model of the typical SED frame was modified by adding FVDs. Since the drifts are largest at the lower level—nearly 90% of component of displacement occurs here, resulting in soft-story behavior—dampers need only be added at this level. Two dampers were added in each direction; see Figure 16. In the longitudinal directions, FVDs were placed along the interior spans, whereas, in the transverse direction, FVDs were placed along the interior bents. Limited nonlinear analysis was then conducted.

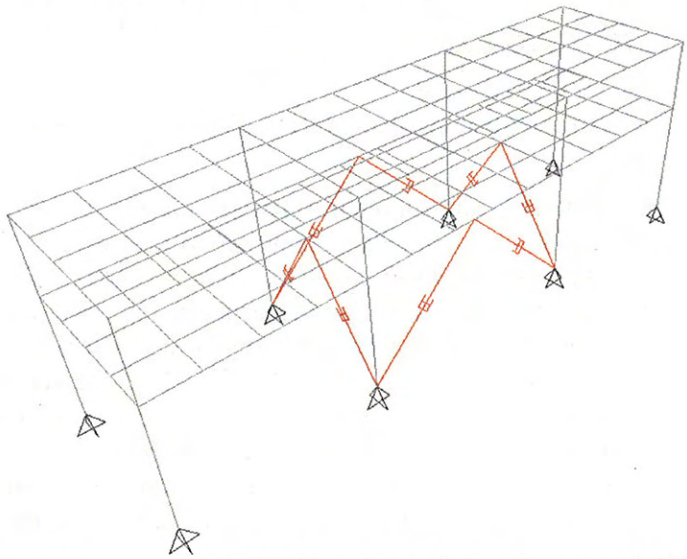


Figure 15. Isometric view of the viaduct frame with FVDs

7.6 Response of retrofitted SED frame

Table 8 presents the computed upper deck displacements for the retrofitted SED frame. Note that the maximum displacement for the RE level is now similar or less that the values that would not cause major structural damage or collapse of the frame. A comparison of upper deck displacement at performance seismic events is presented in Table 9. The addition of dampers has reduced the deck displacement from over 14 in. to less than 4 in. The entries of this table were obtained by averaging the x- (longitudinal) and y- (transverse) components of response at each performance level. The addition of the dampers has reduced the response by approximately 80%.

Record	x-component, in.	y-component, in.
EE	0.8	1.3
DBE	1.3	1.9
RE	2.6	3.9

Table 8. Maximum computed upper deck displacement, retrofitted frame

Record	Existing, in.	Damped, in.	% reduction
EE	4.5	1.0	78
DBE	7.2	1.6	
RE	14.5	3.3	

Table 9. Maximum computed upper deck displacement, retrofitted frame

Note that when FVDs are added to the viaduct, the upper deck displacements are limited to:

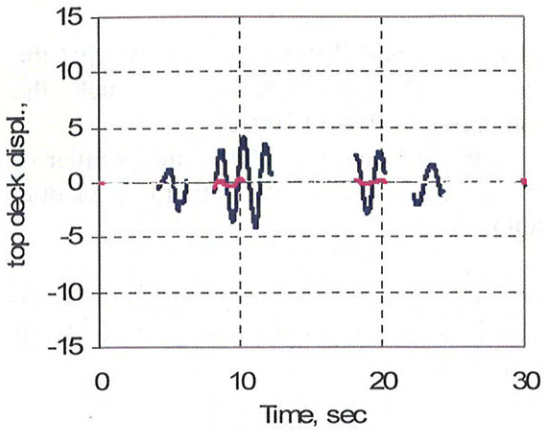
- EE level to 1.3 in. Previous research by TY Lin 2001 and PBQD 2002 shows that the viaduct frames would remain essentially elastic at this displacement. As such, the retrofitted viaduct would meet the first design objective of PBQD 2005.
- RE to 3.9 in. The viaduct experienced the Nisqually earthquake in 2001 under similar or larger displacements without major structural damage. As such, the retrofitted viaduct would meet the second design objective of PBQD 2005.

In the longitudinal direction, there is a nominal 1.5-in. wide gap between adjacent frames. To mitigate pounding problem, shock-transmission units, such as the one shown in Figure 16 can be used to limit the differential movement between the viaduct frames.

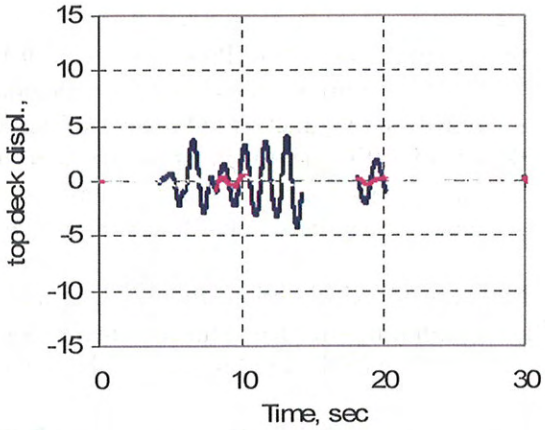
Figure 16. Shock transmission unit (Taiwan High Speed Rail viaduct)

Figure 17 presents the computed x- and y- components of the top deck displacement for the existing and damped frames. The results are shown for the 100-year, 500-year, and 250-year events. The addition of dampers has reduced the computed displacements significantly. To achieve such reduction, the seismic energy must be dissipated. Figure 18 presents the hysteresis response of dampers during the RE event. Note the dissipated energy for the dampers.

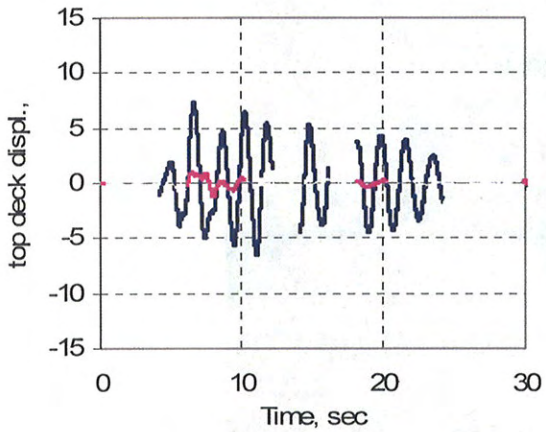
In new construction, state DOTs design and detail concrete members to behavior in a ductile fashion during seismic events. This entails the yielding of members. Such yielding (typically in columns) dissipates the input seismic energy. In the analysis presented here, similar effect was accomplished by resisting the seismic energy via the nonlinear response of the dampers. Figure 19 shows the component of input seismic energy for the RE level excitation. Dampers disperse almost the entirety of the seismic energy.



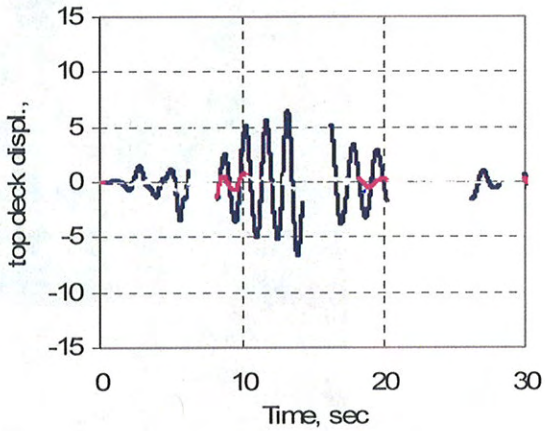
a. 100-year event, x-direction



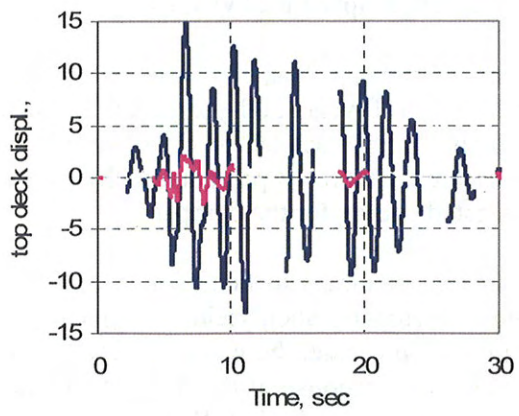
b. 100-year event, y-direction



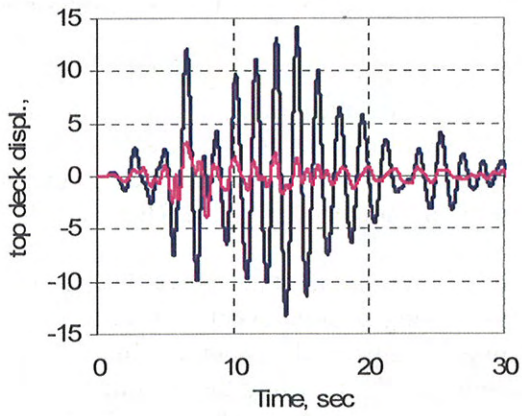
c. 500-year event, x-direction



d. 500-year event, y-direction

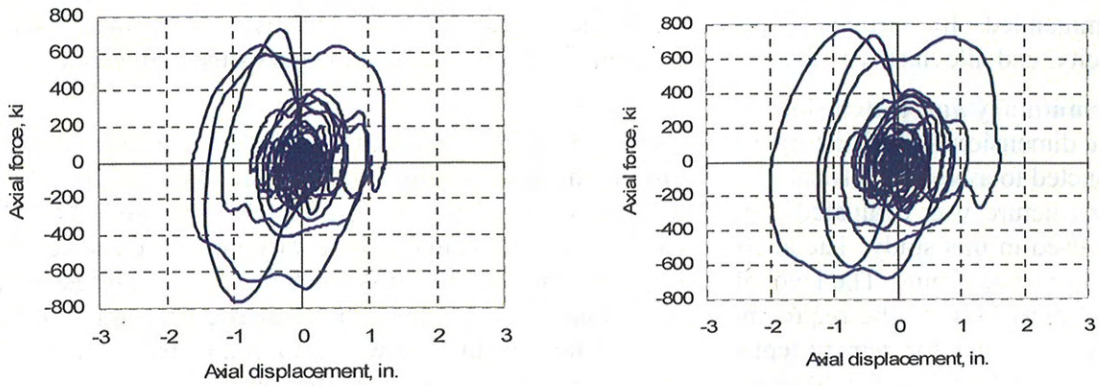


e. 2500-year event, x-direction



f. 2500-year event, y-direction

Figure 17. Upper deck displacement history (Existing _____ Damped _____)



a. FVD in x-direction b. FVD in y-direction

Figure 18. Damper hysteretic response, RE level

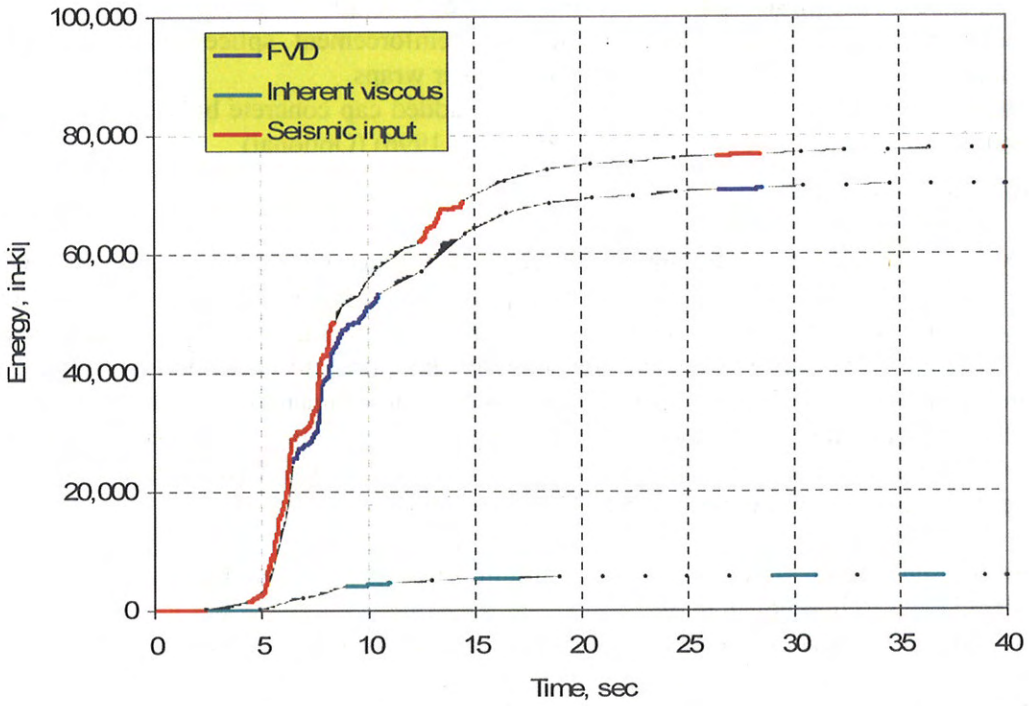


Figure 19. Components of input seismic energy, RE level

The maximum computed lower-level column bending moment and shear are approximately 4,600 k-ft and 150 kips, respectively. (The computed values for the exterior frames are smaller.) TY Lin 2001 and PBQD 2002 wrote that if reinforcement slip and joint principal limit state are reached at column bending moment of 5,000 k-ft.

To enhance seismic performance of the viaduct columns and joints, additional measures are recommended. In particular, jacketing of the columns would increase confinement, shear capacity, and alleviate reinforcement development problem due to the inadequate lap splices.

8 Summary and conclusions

Three dimensional time-history analysis of a typical SED frame in the Alaskan Way Viaduct was conducted to assess the efficacy of retrofitting the frames with dampers. Only the response of the superstructure was evaluated. The design guidelines developed by PBQD 2005 and WSDOT were used in this study. The analysis showed that the dampers materially improved the seismic response of the frame. The level of deck displacement was reduced such that the retrofitted frame would meet both of the requirements (functionality at EE level and collapse prevention at RE event) proposed for retrofit/replacement. Limited evaluation was also conducted to ascertain whether brittle failure would be expected and it was found not to be the case.

Based on the above findings, a retrofit measure that meets the Department's requirement consist of:

- Add Fluid Viscous Dampers to interior bays of viaduct frames.
- Add shock-transmission units between the frames in longitudinal direction.
- Increase shear capacity, confinement, and reinforcement splice length of existing columns by wrapping the columns in ductile fiber wraps.
- Increase the shear capacity of joints by either added cap concrete bolsters and drill-and-bond reinforcement or by prestressing (Priestley 1996) (Optional)

9 Future investigations

The analysis reported herein, was limited in scope. The objective of the investigation was to assess whether FVDs can be used to limit the superstructure displacements to meet the design criteria's performance objectives. Geotechnical evaluation as related to the foundations and condition of underlying soil was beyond the scope of the work presented here. The evaluation was limited to structural assessment. No attempt was made to evaluate the remaining service life, local traffic and population forecast and adequacy of the number of lanes for the future development plans. No analysis of maintenance costs was performed. Benefit-to-cost analysis to determine the merits of replacement vs. retrofit was not conducted.

The results presented here are conceptual and preliminary. It is suggested that the following steps be considered prior to implementation:

- Include the geotechnical data for the frame foundation. Assess substructure condition and investigate soil improvements.
- Investigate the response of a typical WSDOT frame.
- Perform material testing to determine the in-situ concrete and reinforcement properties.
- Conduct independent nonlinear static analysis to determine the response of a typical two-dimensional bay. Include flexural, shear, foundation, and joint nonlinear limit states.
- Refined nonlinear behavior: Develop three two-component site specific acceleration histories to be used for analysis, incorporate soil-structure interaction.
- Verification studies: Conduct comprehensive nonlinear time history analysis incorporating material and damper nonlinearity.

- Proof of concept testing: Construct a scaled model test frame and seismically conduct tests of existing and retrofitted frames at EE and RE levels.

10 References

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