Appendix E:
Seismic Vulnerability Analysis Study Example

WSDOT

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I-405 BN RR Bridge to Ped Trail Bridge – Seismic Retrofit Analysis

Final Report – Rev. 1

February 12, 2020

405/12
405/45W
405/46E
405/46W
405/47E
405/47W
405/48E
405/48W
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405/56W

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1 Introduction

The purpose of this project is to analyze ten bridges on Interstate Route 405 between Renton and Kirkland to first determine each structure's potential vulnerability to seismic loading and second, to perform a PS&E retrofit design for the deficient substructure elements of each bridge as required by analysis. This report represents the first task of the project: identification of deficient structural elements and proposal of retrofit alternatives and recommendations.

1.1 Project Scope

This task consists of performing an analysis of each bridge to determine if retrofit is required for the upper level seismic event (1,000 year return period). If retrofit is required, retrofit alternatives and recommendations, along with associated costs, are identified. Retrofit items are currently limited to substructure elements above the footings. However, foundations (footings, pilecaps, and piles) have been analyzed and capacity/demand (C/D) ratios are provided for these elements. A site visit was performed on each of the bridges to verify information contained in the as-built drawings. This brief report summarizing the analyses, results, retrofit alternatives and recommendations is submitted to WSDOT Bridge and Structures Office along with supporting calculations.

Three bridges were also analyzed for the lower level seismic event (210 year return period). C/D ratios are provided for the foundation elements for WSDOT to further evaluate the impacts of the lower level seismic event on these structures.

1.2 Analysis and Report Criteria

1.2.1 Design Codes and Guides

The following design codes were used for the seismic analysis:

1. WSDOT Bridge Design Manual, July 2019 (hereinafter referred to as ***WSDOT BDM)***
2. FHWA Seismic Retrofitting Manual for Highway Structures - Part 1 Bridges, 2006. ***(FHWA SRM)***
3. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition incl. 2015 Interim Revisions ***(AASHTO Guide Spec)***
4. AASHTO LRFD Bridge Design Specifications, 8th Edition incl. Interim Revisions ***(AASHTO LRFD)***
5. AASHTO "Manual for Condition Evaluation of Bridges", 3rd Edition
6. CRSI Engineering Data Report Number 48 "Evaluation of Reinforcing Bars in Old Reinforced Concrete Structures"
7. FEMA 356, 2000, Prestandard and Commentary for the Seismic Rehabilitation of Buildings: prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, FEMA publication No. 356 ***(FEMA 356)***

1.2.2 Loading

The seismic analysis was performed for the Extreme Event I load combination, considering DC, DW, and EQ loading effects. Permanent load factors (γ_p) were used. Live loading effects were not considered in the analysis because the presence of live load would not significantly influence the structure's capacity to meet life safety requirements. That, coupled with the high level of structural redundancy in both the bridge superstructure and substructure, provides a rational basis for the exclusion of live loads for the seismic analysis. As illustrated in subsequent sections of this report, any structural items that are not scheduled to be retrofitted, except for foundations, have adequate capacity to force plastic hinging into the retrofitted columns.

1.2.3 Response Spectrum Curves

Horizontal acceleration response spectra curves were generated using the provisions established in the AASHTO Guide Spec, and these curves assume a constant five-percent damping. Parameters required to develop the response spectrum for each bridge have been determined and provided by Shannon & Wilson in the following reports:

- "Geotechnical Report I-405 RR Bridge to Pedestrian Trail Bridge - Seismic Retrofit (Bellevue Grouping), XL5937, I-405, MP 14.68 – 15.06 dated 9/24/2019.
- "Geotechnical Report I-405 RR Bridge to Pedestrian Trail Bridge - Seismic Retrofit (Kirkland Grouping), XL5937, NWR, I-405, MP 19.98 – 20.04 dated 11/6/2019.
- "Geotechnical Report I-405 RR Bridge to Pedestrian Trail Bridge - Seismic Retrofit (Renton Grouping), XL5937, NWR, I-405, MP 1.14 – 1.29 dated 1/16/2020.

Known fault zones have been modeled in the USGS hazard mapping used for this study. Bridge 405/12 is located within 3 miles of the Seattle Fault Zone (SFZ) on the up-thrown block of this south-dipping, north verging reverse fault. Per AASHTO (2017) Section 3.10.2, near fault effects (i.e. directivity) have been considered, however, a site-specific analysis has not been performed. This bridge is categorized as "Essential" and has a period greater than 0.5 seconds.

Similarly, Bridge 405/56E & 405/56W are located within 4 miles of the Southern Whidbey Island Fault Zone (SWIFZ) and the other seven bridges not explicitly named are within 3 miles of the SFZ. Per the geotechnical recommendations provided in these reports, near-fault effects have not been considered for these nine bridges.

1.2.4 Response Spectrum Analysis

Seismic demands -- both forces and displacements -- were determined from a multi-modal response spectrum analysis (RSA) using CSi Bridge software. Enough modes were included to account for at least 90 percent of the total mass. Modal response contributions were combined using the CQC method. Response of the structure was analyzed in two orthogonal horizontal directions and the results were combined according to the SRSS rule. Vertical acceleration effects were not included in this analysis.

The following assumptions were made for all response spectrum models:

1. Members were modeled with frame (beam) elements with 6 degrees of freedom at each joint.
2. The superstructure was modeled as a spine element with 10 equal segments per span. Superstructure frame elements were modeled at the composite neutral axis.
3. Superstructure curvature was neglected for structures that have subtended angles in plan less than 90°. Span lengths of equivalent straight bridges were equal to the arc lengths of the curved bridge.
4. End pier foundation stiffness was modeled with equivalent springs. Passive pressure was included on the end pier backwall/stemwall in accordance with Section 6.2.2.4 of FHWA SRM. Passive pressure resistance capacities were determined from the data provided in the geotechnical reports pertaining to this work.
5. Intermediate piers were modeled as a frame with each column connected to a rigid crossbeam. Column-to-crossbeam joint regions were modeled with rigid links. A rigid link was also used to connect the crossbeam to the superstructure. Piers were skewed from the superstructure spine to the angles shown on the as-built drawings.
6. Columns were modeled with equivalent cracked stiffnesses in accordance with Table 7-1 of the FHWA SRM.
7. Intermediate pier foundation stiffness was modeled with equivalent 6 x 6 springs at each footing in accordance with WSDOT BDM. Pile foundation springs were generated using Ensoft's Group software. Drilled shaft foundation springs were generated using Ensoft's LPile software. Spread footing springs were generated by the FEMA 356 in accordance with the geotechnical reports pertaining to this work.
8. Mass was distributed in accordance with Section 7.3.1 of the FHWA SRM. The inertia of live loads was not included.
9. A constant 5 percent damping coefficient was used for all modes.
10. Each bridge was considered and analyzed as a stand-alone structure.

1.2.5 Frame Displacement C/D Ratios from Pushover Analysis (Method D2)

Pursuant to the requirements included in the request for proposal, each bridge was analyzed using the FHWA SRM Method D2 evaluation only.

The seismic displacement capacity of individual piers was determined using non-linear static pushover analysis in CSi Bridge software utilizing the following procedures:

- The un-factored dead load DC and DW were applied to the model as an initial step. The resulting system displacement from the un-factored DC and DW loading represented the non-seismic displacement demand.

- Incremental lateral forces were then applied to the system. A plastic hinge was assumed to form at either the top or bottom of a column, depending on the type of connection, and was formed when the internal moments reach the idealized plastic moment capacity. Crossbeams were modeled with infinite strength to force plastic hinging into the columns in the pushover models. The sequence of plastic hinging through the system was tracked until an ultimate failure (collapse mechanism) was reached. For this analysis, the collapse of a given pier or frame was defined as the point at which one or more columns of the frame or pier have reached their displacement deformation limit. At that point, the pushover analysis was stopped. The difference between the system displacement at collapse and the non-seismic displacement demand represented the seismic displacement capacity. Internal forces in the crossbeams, at the frame's ultimate displacements, were then scaled up by an overstrength factor and checked against their calculated strength capacities to determine their C/D ratios. If those C/D ratios were determined to be less than 1.0, crossbeam strengthening was included as a recommended retrofit.
- Separate pushover models were used to determine the 'longitudinal' and 'transverse' seismic displacement capacity. Several of the bridges included in this package have very large skews. As such, the use of 'longitudinal' and 'transverse' displacement capacities and demands can become confusing. To clarify the displacements included in this report and the associated C/D Ratio tables and calculations, "longitudinal" refers to the intermediate piers' weak-axis direction and "transverse" relates to the intermediate piers' strong-axis direction.
- The response spectrum bridge models were modified to include plastic hinges at the top and/or bottom of each pier column. The model is then pushed in the longitudinal direction (perpendicular to the centerline of the pier) to determine the seismic displacement capacity of each pier. Each pier is also modeled as a stand-alone space frame and pushed along its 'strong-axis' (parallel to centerline of pier) direction to determine the "transverse" displacement capacity. These values were then compared to the rotated local coordinate axis displacement demands determined from the dynamic elastic analyses and presented as column displacement C/D ratios.

1.2.6 Idealized Plastic Moment Capacity of Reinforced Concrete Members

Plastic moment capacity of ductile concrete elements was determined by Moment-Curvature analysis within CSi Bridge software.

Mander's unconfined and confined concrete models were used to define the concrete stress-strain curves. The following material properties were used:

1. Expected concrete compressive strength is equal to 1.30 times the specified 28-day concrete compressive strength (from the as-built plans or WSDOT supplied materials).
2. Unconfined concrete compressive strain at maximum compressive stress is 0.002.

3. Ultimate unconfined concrete compression (spalling) strain is 0.005.

Park's model was used to define the reinforcing steel stress-strain curves. Strain hardening of the main reinforcing bars was accounted for. Expected reinforcing steel yield strength is equal to about 1.10 times the yield strength (from the as-built plans or WSDOT supplied materials). Where reinforcing bar development lengths did not meet the requirements of the FHWA SRM guidelines, the gross area of steel in the section was reduced by a ratio equal to the development length provided, divided by the development length required. This factor is always less than or equal to 1.0.

Pursuant to the WSDOT BDM, the following material properties were used:

Property	Notation	Bar Size	ASTM A706	ASTM A615 Grade 60	ASTM A615 Grade 40*
Specified minimum yield stress (ksi)	f_y	No. 3 - No. 18	60	60	40
Expected yield stress (ksi)	f_{ye}	No. 3 - No. 18	68	68	48
Expected tensile strength (ksi)	f_{ue}	No. 3 - No. 18	95	95	81
Expected yield strain	ε_{ye}	No. 3 - No. 18	0.0023	0.0023	0.00166
Onset of strain hardening	ε_{sh}	No. 3 - No. 8	0.0150	0.0150	0.0193
		No. 9	0.0125	0.0125	
		No. 10 & No. 11	0.0115	0.0115	
		No. 14	0.0075	0.0075	
		No. 18	0.0050	0.0050	
Reduced ultimate tensile strain	ε_{su}^{R}	No. 4 - No. 10	0.090	0.060	0.090
		No. 11 - No. 18	0.060	0.040	0.060
Ultimate tensile strain	ε_{su}	No. 4 - No. 10	0.120	0.090	0.120
		No. 11 - No. 18	0.090	0.060	0.090

* ASTM A615 Grade 40 is for existing bridges in widening projects.

1.2.7 Retrofit Measures for Structure Components

Retrofit measures selected for discussion and recommendation in this report are each referenced in the FHWA SRM and are generally accepted as providing additional strength and/or displacement capacity. Most don't require specialized construction methodology to complete. Additionally, the retrofit measures discussed are as site-appropriate as possible, relating to aesthetics, potential closures, and construction accessibility.

1.2.8 Cost Estimates of Proposed Conceptual Retrofit Measures

Conceptual level cost estimates for the recommended retrofit work are based upon the use of standard items wherever possible. The WSDOT BDM guidelines for unit costs were used, as well as input from local fabricators and manufacturers. Mobilization is included in the estimates at 20% and a contingency of 25% is added to each estimate due to the preliminary and conceptual nature of the estimate. The stated costs are presented as construction costs of the specific items of work identified for each retrofit.

Engineering, inflation, sales tax, traffic control, utility relocations, inter-agency coordination, permits, right-of-way acquisitions and/or easements and other costs not directly included in the items of work are not included in the estimates.

1.3 Discussion of Analysis Methodologies

1.3.1 Iterating the Structural Models

In terms of structural framing, these bridges fall into one of two categories. The first category is referred to as those with an integral crossbeam, and includes Bridges 405/47W, 405/48W & 405/56E. The structural models for the integral crossbeam use a fixed connection between the superstructure and the crossbeam such that the structural system experiences full frame action in both the longitudinal and transverse directions. The most noteworthy aspect of this modeling is that the superstructure moment and shear capacity must be checked to be sure that the girders have adequate capacity to resist the seismic loads induced by longitudinal demands of either the elastic seismic forces, or those brought about due to plastic hinging of the columns. In both instances, these forces are scaled by an overstrength factor to verify the superstructure is appropriately capacity protected.

The second category is represented by Bridges 405/12, 405/45W, 405/46E, 405/46W, 405/47E, 405/48E, and 405/56W. In this case, an expansion joint at each pier and the superstructure consists of multiple simple spans between supports. The ends of each girder are supported on bearing pads atop the concrete drop-cap. The structural model uses a pin connection between the superstructure and the supports. The connection is modeled such that the superstructure is considered pinned in the longitudinal direction at each support indicating a simple-span configuration, as is appropriate. The elastomeric pads upon which the girders are set provide no real positive connection between the girder and the crossbeam. However, we selected this modeling procedure as a means of conservatively capturing maximum anticipated column drift demands as well as potential moments and shears in the substructure that will occur in the presence of a positive connection between superstructure and crossbeam. We feel this is appropriate because the friction coefficient between rubber and concrete is quite high and will likely be adequate to transmit a substantial amount of shear between the two interfaces. It should be noted however, that this modeling procedure is not conservative for superstructure displacement. As such, great care was taken in the determination of seat width C/D ratios and their implications.

To determine appropriate foundation springs, the geotechnical recommendations and FHWA SRM guidelines were followed to provide initial soil springs for items such as end pier backwalls, spread footing springs, pile footing springs, etc. The overall structural model was then run with these initial springs and then the springs were adjusted to match anticipated forces with their associated displacements.

End pier backwall springs were developed by iterating until the combined longitudinal force at both end piers was less than or equal to the expected soil passive resistance at only one end, since only one end of the bridge will be compressing soil at any given time. The longitudinal displacement was checked to assure that the gap between the end pier backwall and superstructure was closed and that the soil has been engaged.

Spread footings, where present, were checked to make sure that the ultimate soil bearing pressure was not exceeded due to the dynamic model loading.

Pile footings and shafts, where present, were iterated using Group or LPile software to be sure that the forces applied to the foundations and resulting displacements and rotations determined from the RSA analysis matched up with those used in the Group or LPile software models within the acceptable limits established in the WSDOT BDM and FHWA SRM.

1.3.2 Determination of Member Capacities and Frame Displacement Capacity

Member strength capacities were generally determined from current AASHTO LRFD code, modified to exclude strength reduction factors and include ultimate strength characteristics. Shear and moment capacities of reinforced concrete sections in crossbeams and foundations were calculated based on expected member properties. As stated previously, column sections were analyzed directly within the CSi Bridge software to determine moment-curvature capacities, which were used in the pushover analyses.

Column displacement capacities for each pier have been computed using the CSi Bridge software to perform a pushover analysis. At each intermediate pier, two pushover analyses were performed. One pushover has been performed along the pier's 'strong-axis' (along the pier centerline) and another pushover has been performed in the pier's 'weak-axis' (normal to the pier centerline). Performing this analyses in the two orthogonal directions will capture the behavior of the pier.

The table below provides more detail regarding what sections of the reference documents apply to the various capacity calculations associated with these bridges.

ELEMENT	FAILURE MODE	REFERENCE SECTION
PILES	SHEAR	Concrete: AASHTO LRFD 5.8.3 Steel: AASHTO LRFD Chapter 6 (modified) Timber: AASHTO LRFD 8.7 (modified)
	COMBINED AXIAL AND BENDING	Concrete: AASHTO LRFD 5.7.4 (modified) Steel: AASHTO LRFD Chapter 6 (modified) Timber: AASHTO LRFD 8.10 (modified)
	CONNECTION TO PILE CAP	Concrete: AASHTO LRFD 5.11.2 (modified) Steel: AASHTO LRFD Chapter 6 (modified) Timber: 10 psi adhesion
	PLUNGING	Geotech parameters supplied
	PULLOUT	Geotech parameters supplied
PILE CAP	MOMENT (+/-)	AASHTO LRFD 5.7.3 & 5.13.3 (modified)
	SHEAR	AASHTO LRFD 5.8.3 & 5.13.3 (modified)
	JOINT SHEAR	AASHTO Guide Spec 6.4.5
SPREAD FOOTING	MOMENT (+/-)	AASHTO Guide Spec 6.3.6
	SHEAR	AASHTO Guide Spec 6.3.7
	OVERTURNING	AASHTO Guide Spec 6.3.4
	SLIDING	AASHTO Guide Spec 6.3.5
	BEARING PRESSURE	AASHTO Guide Spec 6.3.4
COLUMN	DISPLACEMENT	FHWA SRM 7.8.2.1 thru 7.8.2.5 and M-ϕ
	SHEAR	FHWA SRM 7.7.2 & 7.8.2.7
	LAP SPLICE	FHWA SRM 7.7.1.3 & 7.8.2.6
	LONG. BAR DEVELOPMENT	FHWA SRM D.5.1

CROSSBEAM	MOMENT (+/-) HOR. & VERT. AXES	AASHTO LRFD 5.7.3 (modified)
	SHEAR	AASHTO LRFD 5.8.3 (modified)
END PIER STEMWALL	MOMENT (+/-)	AASHTO LRFD 5.7.3 (modified)
	SHEAR	AASHTO LRFD 5.8.3 (modified)
ABUT. CONNECTION	SEAT WIDTH	AASHTO Guide Spec 4.12.3 and FHWA Eqn 5-1b
	SHEAR	AASHTO LRFD 5.7.4 (modified)
ELASTOMERIC BEARING	SEAT WIDTH	AASHTO Guide Spec 4.12.3
GIRDER STOPS	SHEAR	AASHTO LRFD 5.8.4 (modified)
RC OR PC/PS SLAB	MOMENT (+ / -)	AASHTO LRFD 5.6.3 (modified)
	SHEAR	AASHTO LRFD 5.13.3.6 (modified)

1.3.3 Determination of Member Demands

As mentioned previously, the software CSi Bridge was used for the dynamic analysis of each structure. Member elastic demands are determined from this analysis and scaled by an overstrength factor. Member forces in structure components attached, or in a direct load path with the columns were examined on the basis of whether they provide adequate resistance to force plastic hinging of the column. Specific areas of concern for member strength requiring special attention are the following:

- Crossbeam strength (ultimate moment and shear capacities) to resist the loads induced by the plastic overstrength moment capacity of the column
- Pile cap strength (ultimate moment and shear capacities) to resist the loads induced by the plastic overstrength moment capacity of the column, assuming that the piles (or soil for spread footings) are strong enough to resist overturning. In determining pile cap C/D ratios, it is assumed that the piles (or soil for spread footings) have infinite strength.
- Pile strength (ultimate moment and axial capacities) to resist the maximum moment imparted to them by the pile cap. This moment is the lesser of either the ultimate moment capacity of the pile cap, the plastic moment capacity of the column, or the maximum moment determined from the elastic analysis. However, in determining pile C/D ratios, it is assumed that the pile cap has adequate strength. Therefore, the pile demands are assumed to be driven by the column overstrength demand only.
- Pile shear strength to resist the maximum shear imparted to them by the pile cap. This shear is the lesser of either the AASHTO LRFD ultimate shear capacity of the column, the plastic shear in the column, or the maximum shear determined from the elastic analysis.

1.3.4 Columns Deeply Embedded in Fill

Bridge columns that are deeply embedded in fill can have a very different response to seismic forces due to the reduced effective column height and increased rigidity that this situation presents. This is especially troublesome where various piers have substantially greater embedment than other piers within the structure; or where a large variance in embedment exists between columns in a given pier.

The analysis method selected for use on this project was to envelope the effective column heights for intermediate piers. First, we model the bridge in the most flexible condition: as though the fill does not exist. From this model we determine the seismic overstrength shear and displacement demands on the columns. If the column does not have adequate shear or displacement capacity under these conditions, then it must be retrofitted – and further study is not required in this phase. Conversely, if the column has adequate shear and displacement capacity under the flexible model conditions, then the column length is shortened in the model by raising the foundation elevation to three feet below existing grade. This model represents the stiffest condition, in which the pier will attract a greater portion of the seismic load. Again, if the column has adequate shear and displacement capacity to resist the revised seismic demands, then no retrofit is necessary. However, if this condition results in shear or displacement demands that exceed the columns' capacities; then engineering judgment is used to determine if retrofit is necessary by considering the validity of the model and the extent to which the columns are overstressed. Additional fine-tuning of the structure model may be used to aid in this determination if necessary.

For end piers with fully embedded columns, we modeled the columns full length with a lower bound stiffness limit for the rest of the bridge. Retrofitting deeply embedded end piers is generally impractical, and these columns always show flexural and shear vulnerabilities because they have no hinge heights. However, these columns do not represent a collapse mechanism for the bridge if they experience failure since they are surrounded by soil. Therefore, we have elected to report these columns as deficient and not recommend any seismic retrofits.

A more detailed approach to model this additional stiffness is to place lateral springs along the column to represent soil spring stiffness. This is a tedious task and the results of such an analysis are subject to the inherent uncertainty that is associated with soil parameters. Therefore, this latter method was not used.

1.3.5 Brittle Shear, Semi-ductile Shear, and Flexure-limited Rotation

Section 7.8.2.7 of the FHWA SRM reads, "*If the shear strength of the member is less than the shear demand (based on flexural strength) the plastic rotation will be limited. Two limiting cases are: (a) brittle shear, and (b) semi-ductile shear. These cases are based on the shear strength relative to the flexural strength.*"

When the initial shear strength of the member is less than the plastic shear demand, the member is considered to be 'shear-critical' and will fail in a brittle manner with no plastic rotation capacity. This type of failure is considered unacceptable unless the initial shear capacity of the member is enough to resist the seismic loads elastically. Otherwise, the member must be retrofitted to increase its shear capacity.

When the plastic shear demand lies between the initial shear capacity and the final shear capacity of the member, the rotational capacity of the member is limited. If the limited rotational capacity of the member yields a displacement C/D ratio that is less than 1.0, it may be possible to retrofit the member such that the final shear capacity of the member will exceed the plastic shear demand and, thus permit it to act as a ductile member. Provided the flexure-controlled rotational capacity is greater than the demand, the C/D ratio of the retrofitted member would then exceed 1.0.

When the plastic shear demand is less than the final shear capacity of the member, the member is considered ductile and rotational capacity is flexure-controlled.

1.3.6 Columns Constructed with Bridge Widenings

In this group of bridges, the majority of columns added with bridge widenings were constructed with more modern confinement details that greatly enhance the column's ductility. The pushover analyses performed in CSi Bridge software was not always capable of allowing us to determine the "new" columns' displacement capacity because structural instability was initiated (by multiple original column failures) before the analysis reached a deflection capable of causing plastic hinging failures in the widening columns. For purposes of this analysis, the widened columns were assumed to have adequate displacement capacity to resist the design-level demands based on the original columns' C/D ratios all being relatively close to (or greater than) 1.0. This assumption will be validated in the post-retrofit analysis for bridges receiving column jackets because the pushover analysis for those structures will include confined column section parameters for the retrofitted original columns and the pushover analysis will be able to continue to far greater displacements before structural instability halts the analysis.

1.3.7 Restrainer Check Methodology

The current WSDOT BDM (Chapter 4 Section 4.2.10) references FHWA SRM Section 8.4 to outline new longitudinal restrainer design. According to this outline, restrainer design is based on one of two methods. The first method is the 'iterative method,' which determines the effective stiffness and relative displacement at the expansion joint based on an eventual equilibrium with the elongation capacity of the restrainer. The second method is the 'simplified method,' which uses a single step to determine the relative displacement at the expansion joint. This method is applicable as long as the criteria for the ratio of periods and the ratio of displacements are not exceeded.

The structures with existing longitudinal restrainers were examined using the simplified method for this analysis. This methodology (which ignores damping) is based on the premise that, in a two-span simply supported bridge, one span moves away from the adjacent span that was assumed not to move. Based on the stiffness of the substructure elements involved, a period for each span was determined. This period along with the span's mass and stiffness were used to determine the relative displacement of the expansion joint. Since one span was assumed not to move the CQC combination of displacements was not used to determine the relative displacement. This displacement was then compared to the elongation capacity of the restrainer to determine the viability of the existing restrainer gap.

It should be noted that the examined structures do not necessarily fit within the framework described for structures suitable for examination using the simplified method. However, in this phase of work, the simplified method is considered a viable triage check for the existing restrainers to determine if further detailed analysis (using the cumbersome iterative method) is warranted and worthwhile during PS&E design.

1.3.8 Geotechnical Recommendations of Passive Earth Pressure and Sliding Resistance

The geotechnical recommendations of Shannon & Wilson provide passive resistance coefficient values for the soils behind the bridge end piers. Included in this report was a chart which had reduction values if the lateral deflection (relative to the wall height) was inadequate to mobilize the full passive pressure. Displacement values were compared to the wall heights and when they exceeded 5% the percentage of mobilized pressure assumed was 1.0 times the value provided. In the event any values were less than 5%, a reduction was incorporated as suggested by the provided chart.

The sliding resistance or soil friction values were provided by Shannon & Wilson. These soil friction values were used to develop end pier soil springs. This limits the deflection at the end pier and has varying degrees of effect on the deflection at the intermediate piers, depending on the structural framing and connections between the bridge sub- and superstructures.

1.3.9 Soil Liquefaction

According to the geotechnical recommendations of Shannon & Wilson, the potential for soil liquefaction at all but one of the bridge sites is considered low. As such, no structural modeling was performed with severely weakened foundation soil springs that would represent a liquefied case, with one exception. Bridge 405/12 was indicated to be susceptible to liquefaction.

The effects of liquefaction on Bridge 405/12 are anticipated to occur at all piers and include both liquefaction induced settlement on the order of 10 to 15 inches as well as weakened lateral resistance. The effects of liquefaction were modeled in a separate RSA demand analysis that included reduced soil spring stiffness to represent the liquefied soil parameters provided. Those same soil reduced soil spring stiffness parameters were also implemented in separate pushover analyses for this bridge.

1.3.10 Lower-Level vs. Upper-Level Seismic Events

Two earthquake levels – lower-level or Functional Evaluation Earthquake (FEE) and upper-level or Safety Evaluation Earthquake (SEE) -- were considered in this analysis for three bridges (405/12, 405/45W & 405/47W), and the remaining seven bridges were analyzed for only the upper-level event (SEE). The lower-level seismic event (FEE) uses a response spectrum constructed using acceleration coefficients for an event with a 30% probability of exceedance in 75 years (210 year return period). The upper-level seismic event (SEE) uses a response spectrum for an event with a 7% probability of exceedance in 75 years (1000 year return period).

In both instances, the procedure used was similar where an elastic-dynamic analysis procedure for displacement demands, and a Method D2 (pushover) analysis was performed to compute displacement capacities. However, the lower-level analysis is stopped much earlier as the “nominal” limit state is considered to occur when concrete compressive strain reaches 0.003 or the steel reaches a yield strain of 0.0015. These column deformations result in foundation forces which are scaled using an overstrength factor to determine the foundation demands.

Foundation capacities are determined using the same procedure established for the upper-level analysis. Since expected material properties are utilized in that analysis, a C/D ratio that is barely greater than 1.0 indicates that there is a high likelihood that the foundations will not perform elastically during a lower-level event.

2 Result of Analyses

2.1 Bridge No. 405/12

2.1.1 Bridge Description

Bridge 405/12 consists of eight consecutive prestressed concrete I-girder spans. Originally constructed in 1965 as two independent bridge structures – one carrying Northbound traffic and one carrying Southbound traffic. Span 1 is 75 feet, Span 2 is 74 feet, the next 5 spans are each 110 feet and Span 8 is 66 feet. These spans add to a total bridge length of 765’-0” from back-to-back of pavement seats. Each of these original bridges had a roadway width of 33’-0”. All piers are normal to the bridge alignment.

All original spans are simply supported consisting of ten 100 Ft-Series prestressed concrete girders. The intermediate piers have a 4’-0” wide by 3’-3” tall dropped crossbeam supporting the superstructure above. The original crossbeams are supported on two 3’-0” diameter columns that are founded on pile caps supported on piles. At the end piers, the girders each bear on a bearing pad and are simply supported. The end pier stemwalls sit on a reinforced concrete pile cap founded on piles.

The bridge was widened in 1987 by 16 feet to the north, up to 23 feet to the south and the deck was connected between the north and south bridges. Depending on the span, five or six 100 Ft-Series prestressed concrete girders were added to the framing at this time. The intermediate piers were extended – but not connected to each other – and one 4’-0” diameter column was typically added and supported on a 16’-6” by 16’-6” pile cap founded on concrete piles. Several intermediate pier footings deviated from the typical, notably, Bent 5 and 8 footings are each 16’-6” by 20’-3”, Bent 4 Southbound is 15’-6” x 16’-6” and has a clipped corner, and Bent 7 is 16’-6” by 16’-6” with a clipped edge that reduces one side to 13’-4”. There is a column modification at Bent 6 and Bent 2 Northbound that was angled at 20 degrees to maintain railroad clearance. A crash wall was added at this Bent 2 location for railroad protection.

The bridge was widened again in 2009 to the south by 1 foot at the west and 2 feet at the east. For this widening, no additional girder lines were added, however the median barrier and south side barrier were replaced.

2.1.2 Bridge 405/12 C/D Ratios for Method D2 Analysis

Original (1965) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	2150	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	0.22	2005	434	
Pile Axial (k)	0.44	388	170	
Pile Shear (k)	1.42	24	34	
Pile Connection (k)	0.01	219	2	
COLUMN				
Long. Displacement (in)	0.53	16.1	8.5	Column 2 controls
Transv. Displacement (in)	0.41	5.4	2.2	Column 1 controls
Shear (k)	0.56	290	164	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	0.55	1496	821	Positive moment
Shear (k)	0.77	552	426	
PIER 3				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	898	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	0.52	828	434	
Pile Axial (k)	0.72	185	134	
Pile Shear (k)	2.58	13	34	

Pile Connection (k)	**0.03**	98	2	
COLUMN				
Long. Displacement (in)	**0.89**	14.0	12.5	Column 2 controls
Transv. Displacement (in)	1.13	6.0	6.8	Column 1 controls
Shear (k)	**0.84**	196	165	Vf vs.Vp - Column 3 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.49**	1686	821	Positive moment
Shear (k)	**0.51**	426	217	
PIER 4				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	776	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.52**	915	474	
Pile Axial (k)	**0.91**	148	134	
Pile Shear (k)	3.16	11	34	
Pile Connection (k)	**0.04**	63	2	
COLUMN				
Long. Displacement (in)	**0.95**	12.6	12.0	Column 2 controls
Transv. Displacement (in)	**0.96**	6.8	6.5	Column 1 controls
Shear (k)	**0.84**	199	168	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.41**	1984	821	Positive moment
Shear (k)	**0.61**	694	426	
PIER 5				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	781	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.53**	895	474	
Pile Axial (k)	1.19	147	174	
Pile Shear (k)	3.06	11	34	
Pile Connection (k)	**0.04**	65	2	
COLUMN				
Long. Displacement (in)	**0.94**	12.4	11.7	Column 3 controls
Transv. Displacement (in)	**0.88**	7.3	6.4	Column 4 controls
Shear (k)	**0.83**	202	167	Vf vs.Vp - Column 4 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.38**	2147	821	Positive moment
Shear (k)	**0.30**	712	217	
PIER 6				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	916	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.54**	875	474	
Pile Axial (k)	**0.43**	360	154	
Pile Shear (k)	1.01	34	34	
Pile Connection (k)	**0.01**	217	2	
COLUMN				
Long. Displacement (in)	**0.87**	12.8	11.2	Column 2 controls
Transv. Displacement (in)	**0.80**	7.4	6.0	Column 4 controls
Shear (k)	**0.80**	208	166	Vf vs.Vp - Column 4 Semi-Ductile Shear-Controls

CROSSBEAM				
Moment (k-ft)	**0.42**	1977	821	Positive moment
Shear (k)	**0.35**	940	325	
PIER 7				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	839	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.54**	886	474	
Pile Axial (k)	**0.83**	145	120	
Pile Shear (k)	2.71	13	34	
Pile Connection (k)	**0.04**	68	2	
COLUMN				
Long. Displacement (in)	**0.77**	12.4	9.6	Column 2 controls
Transv. Displacement (in)	**0.72**	7.3	5.2	Column 4 controls
Shear (k)	**0.75**	225	169	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.42**	1952	821	Positive moment
Shear (k)	**0.57**	747	426	
PIER 8				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	517	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.64**	680	434	
Pile Axial (k)	**0.84**	146	122	
Pile Shear (k)	2.80	12	34	
Pile Connection (k)	**0.04**	58	2	
COLUMN				
Long. Displacement (in)	**0.70**	12.9	9.1	Column 3 controls
Transv. Displacement (in)	**0.74**	6.7	5.0	Column 1 controls
Shear (k)	**0.92**	176	161	Vf vs.Vp - Column 3 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.48**	1717	821	Positive moment
Shear (k)	**0.66**	650	426	
GIRDER STOPS				
Pier 1 (k)	1.01	2138	2160	
Pier 2 (k)	1.83	2282	4177	
Pier 3 (k)	2.00	2093	4177	
Pier 4 (k)	2.95	1414	4177	
Pier 5 (k)	4.83	864	4177	
Pier 6 (k)	3.56	1175	4177	
Pier 7 (k)	2.93	1427	4177	
Pier 8 (k)	2.40	1740	4177	
Pier 9 (k)	1.58	1369	2160	
SEAT LENGTH				
End Pier 1 (in)	**0.68**	29	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
Pier 2	**0.63**	32	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
Pier 3	**0.61**	33	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
Pier 4	**0.61**	33	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
Pier 5	**0.61**	33	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
Pier 6	**0.63**	32	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
Pier 7	**0.65**	31	20	FHWA Seismic Retrofitting Manual Eq. 5.1b

Pier 8	**0.66**	30	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
End Pier 9 (in)	**0.73**	27	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
SEISMIC RESTRAINER				
Pier 2	**0.33**	6	2	Demand and capacity are in terms of the number of seismic restrainers
Pier 3	**0.40**	5	2	Demand and capacity are in terms of the number of seismic restrainers
Pier 4	**0.67**	6	4	Demand and capacity are in terms of the number of seismic restrainers
Pier 5	**0.67**	6	4	Demand and capacity are in terms of the number of seismic restrainers
Pier 6	**0.50**	8	4	Demand and capacity are in terms of the number of seismic restrainers
Pier 7	**0.44**	9	4	Demand and capacity are in terms of the number of seismic restrainers
Pier 8	**0.29**	7	2	Demand and capacity are in terms of the number of seismic restrainers

Widened (1987) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**0.47**	2868	1352	
Pile Cap Shear (k)	**0.56**	1651	930	
Pile Axial (k)	**0.48**	370	178	
Pile Shear (k)	1.88	22	41	
Pile Connection (k)	**0.01**	192	2	
COLUMN				
Long. Displacement (in)	1.57	10.2	16.0	Column 1 controls
Transv. Displacement (in)	1.18	5.4	6.3	Column 1 controls
Shear (k)	1.05	625	654	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.29**	3956	1155	Positive moment
Shear (k)	**0.63**	344	217	
PIER 3				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**0.79**	1712	1352	
Pile Cap Shear (k)	**0.80**	1160	930	
Pile Axial (k)	**0.72**	197	142	
Pile Shear (k)	1.87	22	41	
Pile Connection (k)	**0.03**	86	2	
COLUMN				
Long. Displacement (in)	2.36	11.4	27.0	Column 1 controls
Transv. Displacement (in)	2.51	6.0	15.0	Column 1 controls
Shear (k)	1.66	427	708	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	0.51	1616	821	Positive moment
Shear (k)	0.90	487	437	
PIER 4				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**0.77**	1766	1352	
Pile Cap Shear (k)	**0.66**	1378	906	

Pile Axial (k)	**0.61**	227	138	
Pile Shear (k)	1.66	24	41	
Pile Connection (k)	**0.03**	90	2	
COLUMN				
Long. Displacement (in)	2.46	11.1	27.3	Column 1 controls
Transv. Displacement (in)	2.26	6.8	15.2	Column 1 controls
Shear (k)	1.61	454	732	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.48**	4666	2245	Positive moment
Shear (k)	**0.76**	574	437	
PIER 5				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	1.04	1526	1592	
Pile Cap Shear (k)	**0.78**	1141	890	
Pile Axial (k)	**0.30**	474	142	
Pile Shear (k)	1.65	25	41	
Pile Connection (k)	**0.02**	106	2	
COLUMN				
Long. Displacement (in)	2.46	10.9	26.8	Column 2 controls
Transv. Displacement (in)	2.08	7.2	15.1	Column 2 controls
Shear (k)	1.58	463	730	Vf vs.Vp - Column 2 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.45**	5808	2594	Positive moment
Shear (k)	**0.63**	689	437	
PIER 6				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**0.70**	1946	1352	
Pile Cap Shear (k)	**0.71**	1268	906	
Pile Axial (k)	**0.66**	217	144	
Pile Shear (k)	1.46	28	41	
Pile Connection (k)	**0.02**	98	2	
COLUMN				
Long. Displacement (in)	2.22	11.3	25.1	Column 2 controls
Transv. Displacement (in)	1.91	7.4	14.2	Column 2 controls
Shear (k)	1.53	480	733	Vf vs.Vp - Column 2 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	0.45	7813	3479	Negative moment
Shear (k)	0.40	820	325	
PIER 7				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**0.82**	1459	1194	
Pile Cap Shear (k)	**0.70**	1198	842	
Pile Axial (k)	**0.58**	223	130	
Pile Shear (k)	1.43	28	41	
Pile Connection (k)	**0.03**	82	2	
COLUMN				
Long. Displacement (in)	1.97	11.7	23.0	Column 2 controls

Transv. Displacement (in)	1.79	7.2	12.9	Column 1 controls
Shear (k)	1.53	468	717	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.48**	5388	2594	Positive moment
Shear (k)	**0.68**	639	437	
PIER 8				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	1.17	1161	1352	
Pile Cap Shear (k)	**0.91**	1228	1112	
Pile Axial (k)	**0.40**	409	162	
Pile Shear (k)	1.41	29	41	
Pile Connection (k)	**0.03**	81	2	
COLUMN				
Long. Displacement (in)	1.64	11.8	19.4	Column 1 controls
Transv. Displacement (in)	1.64	6.7	11.0	Column 1 controls
Shear (k)	1.43	488	700	Vf vs.Vp - Column 2 Semi-Ductile Shear-Controls
CROSSBEAM				
Moment (k-ft)	**0.48**	4703	2245	Positive moment
Shear (k)	**0.74**	593	437	

Original (1965) Foundation Elements - FEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	1621	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.25**	1769	434	
Pile Axial (k)	**0.51**	331	170	
Pile Shear (k)	1.65	21	34	
Pile Connection (k)	**0.01**	165	2	
PIER 3				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	212	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.74**	589	434	
Pile Axial (k)	1.07	125	134	
Pile Shear (k)	5.35	6	34	
Pile Connection (k)	**0.06**	38	2	
PIER 4				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	31	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.75**	629	474	
Pile Axial (k)	1.48	90	134	
Pile Shear (k)	5.66	6	34	
Pile Connection (k)	**0.27**	9	2	
PIER 5				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	107	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.74**	629	466	
Pile Axial (k)	1.85	94	174	

Pile Shear (k)	4.62	7	34	
Pile Connection (k)	**0.19**	13	2	
PIER 6				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	317	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.72**	657	474	
Pile Axial (k)	**0.48**	321	154	
Pile Shear (k)	1.44	24	34	
Pile Connection (k)	**0.01**	177	2	
PIER 7				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	313	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.69**	692	474	
Pile Axial (k)	1.15	104	120	
Pile Shear (k)	3.54	10	34	
Pile Connection (k)	**0.08**	31	2	
PIER 8				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	499	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.68**	641	434	
Pile Axial (k)	**0.90**	135	122	
Pile Shear (k)	3.27	10	34	
Pile Connection (k)	**0.05**	54	2	

Widened (1987) Foundation Elements - FEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**0.49**	2752	1352	
Pile Cap Shear (k)	**0.64**	1456	930	
Pile Axial (k)	**0.54**	328	178	
Pile Shear (k)	2.13	19	41	
Pile Connection (k)	**0.01**	188	2	
PIER 3				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	2.00	2777	5562	
Pile Cap Shear (k)	1.10	825	906	
Pile Axial (k)	**0.90**	158	142	
Pile Shear (k)	3.30	12	41	
Pile Connection (k)	**0.07**	30	2	
PIER 4				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	1.73	3128	5414	
Pile Cap Shear (k)	**0.91**	1000	906	
Pile Axial (k)	**0.80**	173	138	
Pile Shear (k)	2.89	14	41	
Pile Connection (k)	**0.27**	8	2	
PIER 5				
PILE CAP FOOTING				

Pile Cap Moment (k-ft)	1.74	4535	7896	
Pile Cap Shear (k)	1.01	878	890	
Pile Axial (k)	**0.44**	323	142	
Pile Shear (k)	2.46	16	41	
Pile Connection (k)	**0.38**	6	2	
PIER 6				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	1.32	2692	3555	
Pile Cap Shear (k)	**0.92**	986	906	
Pile Axial (k)	**0.82**	177	144	
Pile Shear (k)	2.31	18	41	
Pile Connection (k)	**0.05**	49	2	
PIER 7				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	1.43	3411	4893	
Pile Cap Shear (k)	**0.85**	990	842	
Pile Axial (k)	**0.68**	192	130	
Pile Shear (k)	2.08	20	41	
Pile Connection (k)	**0.05**	42	2	
PIER 8				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	1.89	4298	8115	
Pile Cap Shear (k)	1.12	816	914	
Pile Axial (k)	**0.51**	320	162	
Pile Shear (k)	2.06	20	41	
Pile Connection (k)	**0.08**	29	2	

2.1.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- The as-built plans show the seat length provided at the intermediate and end piers is not adequate to prevent unseating of the end spans.
- The girder stops present at the end and intermediate piers are adequate to achieve good seismic performance, however a girder stop is not present at each girder bay. This is not preferable as it requires significant load to transfer through diaphragms and girder webs.
- Method D2 analysis shows that the original columns have inadequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of the 1987 column at Pier 2 is only marginally adequate for resisting shear resulting from plastic hinging in the column.
- The existing crossbeam is deficient in flexure and shear. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns.

In a capacity protection approach, this behavior is considered unacceptable for bridges.

- The longitudinal restrainers were found to be deficient for the SEE load case. The limiting capacity for them was in the restrainer rods themselves.

- All foundations were found to be vulnerable in both the upper and lower level event. The foundations supporting the 1965 original columns of Piers 2 through 8 lack a top mat of flexural steel and are therefore deficient in bending. They are also typically deficient in shear, pile bending, and pile/cap connection. The 1987 pile caps have higher CD ratios, however, they are still frequently deficient for flexure, shear, pile axial and pile to pile cap connection. There may be additional capacity at the pile to pile cap connection; however, documentation of the pile connection is insufficient. Therefore, only adhesion is assumed for capacity. We are not scoped to discuss upper level retrofit solutions for items below the bottom of the column, but it would be prudent to further examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.1.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around the original 1965 columns. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening. Installation of column jackets will require shoring. The addition of concrete bolsters at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a deficient column displacement C/D ratio in the as-built condition will decrease even further with the introduction of crossbeam bolsters. .
2. The southernmost widened column at Pier 2 is only marginally adequate for shear, thus we recommend it be provided with a steel jacket. The addition of crossbeam bolsters will increase bridge mass and result in increased shear demand. It should be noted that this location is very close to the existing railroad tracks so it will require railroad coordination and approvals during final design.
3. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments near columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the

crossbeam's moment and shear capacity can be increased enough to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width.

4. We recommend adding girder stops at both end piers as it is a low cost addition that helps make sure the bridge response is more predictable and reduces the transverse demand on the adjacent piers.
5. Current WSDOT practice is to provide girder stops in each girder bay to better distribute transverse shear loads amongst the girders. Thus we propose the addition of girder stops at each girder bay at the intermediate piers included with the crossbeam bolsters to provide positive transverse restraint at each girder without relying on the end diaphragms to transfer shear between girders.
6. Even though the existing restrainers are deficient, we do not recommend retrofitting the restrainers. We recommend instead adding crossbeam bolsters to all the piers (as noted above), which function to strengthen the crossbeam capacity and to lengthen all intermediate pier seat widths. Both seat width lengthening and restrainers function to mitigate the same vulnerability – unseating of the superstructure – therefore, only one retrofit method is necessary. Since the deficient seat widths are ameliorated with crossbeam bolsters, the existing restrainers are redundant and need not be retrofitted.
7. Though not a recommended retrofit, we observed erosion at the south end-pier of this bridge. We recommend WSDOT maintenance remedy the source of the erosion, restore the grade and install new slope paving at the north abutment.
8. Foundation retrofits have not been a primary focus for WSDOT and for the lower level event would entail expanding the footings, adding pin piles around the perimeter, and thickening the footings. This likely could be designed to address these foundation deficiencies for the lower level event.

2.1.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1.	Steel Column Jackets	$2,380,000
2.	Reinforced Concrete Bolsters at Piers 2 through 8	$2,000,000
3.	End Pier Girder Stops	$50,000
4.	Miscellaneous (Downspout relocations, etc.)	$50,000

WSDOT has not typically begun to retrofit foundations in this phase of their seismic retrofit program. Therefore, the following construction costs (including 20% Mobilization and 25% Contingency) are being included for planning purposes. Note that these costs assume retrofits for the lower level event, and may increase if WSDOT intends to remove the vulnerabilities for an upper level event.

1. Pin piles and thickened footings $7,200,000

2.2 Bridge No. 405/45W

2.2.1 Bridge Description

Bridge 405/45W consists of three continuous prestressed concrete I-girder spans. Originally constructed in 1966, the center span is the longest measuring 79 feet while the other two spans are 64 feet each to create a total length of 207’-0” from back-to-back of pavement seats. The original roadway width measured 51’-4”. All piers are on a skew that vary slightly and are about 34 degrees measured normal to the bridge alignment line at the centerline of piers for intermediate piers and back of pavement seat for end piers.

All spans are simply supported consisting of ten 80 Ft-Series prestressed concrete girders. The intermediate piers have a 3’-6” wide by 3’-3” tall dropped crossbeam supporting the superstructure above. The original crossbeams sit on three 3’-0” columns that are founded on spread footings. At the end piers, the girders each bear on a bearing pad and are framed integrally into the backwall which has a hinge connection with the stemwall. The stemwall sits on a reinforced concrete spread footing.

The bridge was widened about 21 feet in 1993 to the east adding three W50G prestressed girders to the framing. At this time, the west barrier was replaced. The intermediate piers were extended and one 3’-0” diameter column was added and supported on a 17 feet by 17 feet spread footing. At the end piers, the L-shaped end piers were extended and supported on new spread footings.

2.2.2 Bridge No 405/45W – I-405 over SR520 WB On-ramp Bridge 405/45W C/D Ratios for Method D2 Analysis

Original (1966) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				
Moment (k-ft)	0.85	944	805	Column 2 controls
Shear (k)	0.26	564	149	Column 3 controls
Overturning (k-ft)	0.69	1612	1115	Column 1 controls
Sliding (k)	1.30	147	192	Column 1 controls
COLUMN				
Long. Displacement (in)	1.28	3.54	4.52	Col. 3 controls
Transv. Displacement (in)	1.36	1.66	2.26	Column 3 controls
Shear (k)	0.83	175	146	Vf vs.Vp - Column 3 Semi-Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	0.40	1132	455	Positive moment
Shear (k)	0.53	370	195	
PIER 3				
FOOTING				
Moment (k-ft)	0.84	953	805	Column 2 controls
Shear (k)	0.26	563	149	Column 3 controls
Overturning (k-ft)	0.69	1681	1167	Column 1 controls
Sliding (k)	1.19	171	204	Column 1 controls
COLUMN				
Long. Displacement (in)	1.08	3.27	3.51	Column 1 controls
Transv. Displacement (in)	1.00	1.81	1.81	Column 3 controls
Shear (k)	0.77	163	125	Vf vs.Vp - Column 1 Semi-Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	0.50	906	455	Positive moment
Shear (k)	0.57	342	195	
SEAT LENGTH				
End Pier 1 (in)	1.11	29	32	FHWA Seismic Retrofitting Manual Eq. 5.1b
End Pier 4 (in)	1.11	29	32	FHWA Seismic Retrofitting Manual Eq. 5.1b

Widened (1993) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				
Moment (k-ft)	1.04	2988	3111	
Shear (k)	1.50	632	949	
Overturning (k-ft)	1.14	4980	5686	
Sliding (k)	0.87	412	359	
COLUMN				
Long. Displacement (in)	3.80	3.22	12.23	

Transv. Displacement (in)	4.60	1.65	7.61	
Shear (k)	1.36	410	558	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.32**	3794	1198	Positive moment
Shear (k)	**0.79**	508	401	
PIER 3				
FOOTING				
Moment (k-ft)	1.34	2323	3111	
Shear (k)	1.43	664	949	
Overturning (k-ft)	1.44	4742	6843	
Sliding (k)	1.17	373	437	
COLUMN				
Long. Displacement (in)	4.03	3.31	13.37	
Transv. Displacement (in)	4.66	1.81	8.42	
Shear (k)	1.45	379	550	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.37**	3246	1198	Positive moment
Shear (k)	**0.88**	456	401	
SEAT LENGTH				
End Pier 1 (in)	1.11	29	32	FHWA Seismic Retrofitting Manual Eq. 5.1b
End Pier 4 (in)	1.11	29	32	FHWA Seismic Retrofitting Manual Eq. 5.1b

Original (1966) Foundation Elements - FEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2 FOOTING				
Moment (k-ft)	**0.93**	862	805	Column 1 controls
Shear (k)	**0.28**	531	149	Column 3 controls
Overturning (k-ft)	**0.70**	1518	1067	Column 1 controls
Sliding (k)	1.32	137	181	Column 1 controls
PIER 3 FOOTING				
Moment (k-ft)	**0.92**	874	805	Column 3 controls
Shear (k)	**0.28**	538	149	Column 3 controls
Overturning (k-ft)	**0.70**	1569	1105	Column 1 controls
Sliding (k)	1.19	159	189	Column 1 controls

Widened (1993) Foundation Elements - FEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2 FOOTING				
Moment (k-ft)	2.58	1205	3111	
Shear (k)	2.76	344	949	
Overturning (k-ft)	2.40	2276	5463	
Sliding (k)	1.82	189	343	
PIER 3 FOOTING				
Moment (k-ft)	3.23	929	3002	
Shear (k)	3.45	265	916	

Overturning (k-ft)	2.18	2367	5158	
Sliding (k)	1.73	186	322	

2.2.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- The as-built plans show the seat length provided at the end piers is adequate to prevent unseating of the end spans.
- No girder stops are present at the end piers. This is not preferable as end piers have a lot of lateral resistance that can help 'protect' the interior piers.
- The intermediate pier hinge diaphragms are connected to the crossbeams with 50 #9 bars and the girders are embedded 2 inches into the pier hinge diaphragms with extended strands. This provides adequate connectivity to achieve good seismic performance.
- Method D2 analysis shows that the original columns have marginally adequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of the columns is inadequate for resisting shear resulting from plastic hinging in the column.
- The existing crossbeam is deficient in shear, in flexure. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges.
- At the lower level event, the original spread footing foundations supporting the original columns of Piers 2 and 3 lack the necessary overturning, shear and flexural capacity to resist the column plastic hinging forces. The analysis shows the columns may begin to form a plastic hinge and therefore the foundation C/D ratios are similar to that of the upper level event.
- At the upper level event, the original spread footing foundations supporting the original columns of Piers 2 and 3 lack the necessary overturning capacity to resist the column plastic hinging forces. Additionally, the Pier 2 footings supporting the widening show additional deficiencies for sliding. We are not scoped to discuss upper level retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.2.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around the original 1966 columns. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the

field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening.

The addition of concrete bolsters at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a marginal column displacement C/D ratio in the as-built condition will decrease even further with the introduction of crossbeam bolsters.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments next to columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post-tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment and shear capacity can be increased enough to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width.
3. We recommend adding girder stops at both end piers as it is a low cost addition that helps make sure the bridge response is more predictable and reduces the transverse demand on the adjacent piers.
4. Foundation retrofits have not been a primary focus for WSDOT and for the lower level event would entail expanding the footings, adding pin piles around the perimeter, and thickening the footings. This could be designed to address these foundation deficiencies for the lower level event.

2.2.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

5. Steel Column Jackets on Original Columns	$275,000
6. Reinforced Concrete Bolsters at Piers 2 and 3	$390,000
7. End Pier Girder Stops	$60,000
8. Miscellaneous (Downspout relocations, etc.)	$10,000

WSDOT has not typically begun to retrofit foundations in this phase of their seismic retrofit program. Therefore, the following construction costs (including 20% Mobilization and 25% Contingency) are being included for planning purposes. Note that these costs assume retrofits for the lower level event, and may increase if WSDOT intends to remove the vulnerabilities for an upper level event.

2. Pin piles and thickened footings $450,000

2.3 Bridge No. 405/46E

2.3.1 Bridge Description

Bridge 405/46E consists of four continuous prestressed concrete I-girder spans. Originally constructed in 1966, the spans lengths are 44'-6", 79-0", 59'-0", and 64'-0" from south to north to create a total length of 246'-6" from back-to-back of pavement seats. The original roadway width varied between 61'-1" and 63'-6". All piers are on a skew that vary slightly and are about 10 degrees measured normal to the bridge alignment line at the centerline of piers for intermediate piers and back of pavement seat for end piers.

All spans are simply supported and consisting of 80 Ft-Series prestressed concrete girders. Twelve girders are in the first span and eleven girders are in the remaining spans. The intermediate piers have a 4'-0" wide by 3'-3" tall dropped crossbeam supporting the superstructure above. The original crossbeams sit on three 3'-0" columns that are founded on spread footings. The end piers are L-shaped and supported on spread footings.

The bridge was widened about 19 feet in 1993 to the west adding three W50G prestressed girders to the framing. At this time, the east barrier was replaced. The intermediate piers were extended and one 3'-0" diameter column was added and supported on a 17 feet by 17 feet spread footing. At the end piers, the L-shaped end piers were extended and supported on new spread footings.

Bridge No 405/46E – I-405 over SR520

2.3.2 Bridge 405/46E C/D Ratios for Method D2 Analysis

Original (1966) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				
Moment (k-ft)	1.06	1074	1133	Column 1 controls
Shear (k)	1.48	201	298	Column 1 controls
Overturning (k-ft)	**0.89**	1878	1666	Column 3 controls
Sliding (k)	1.27	167	212	Column 3 controls
COLUMN				
Long. Displacement (in)	1.16	4.1	4.7	Column 1 controls
Transv. Displacement (in)	1.94	1.2	2.3	Column 1 controls
Shear (k)	**0.76**	165	126	Semi-Ductile Shear
CROSSBEAM				
Moment (k-ft)	**0.37**	2368	885	
Shear (k)	**0.68**	612	417	
PIER 3				
FOOTING				
Moment (k-ft)	1.21	935	1133	Column 1 controls
Shear (k)	1.83	163	298	Column 1 controls

Overturning (k-ft)	**0.99**	1883	1873	Column 3 controls
Sliding (k)	1.71	142	244	Column 3 controls
COLUMN				
Long. Displacement (in)	1.59	4.1	6.5	Column 1 controls
Transv. Displacement (in)	1.25	2.6	3.2	Column 1 controls
Shear (k)	**0.87**	145	127	Semi-Ductile Shear
CROSSBEAM				
Moment (k-ft)	**0.40**	2240	885	
Shear (k)	**0.63**	667	417	
PIER 4				
FOOTING				
Moment (k-ft)	1.19	949	1133	Column 1 controls
Shear (k)	1.95	153	298	Column 1 controls
Overturning (k-ft)	**0.97**	1808	1754	Column 3 controls
Sliding (k)	1.70	133	225	Column 3 controls
COLUMN				
Long. Displacement (in)	1.74	4.1	7.2	Column 1 controls
Transv. Displacement (in)	1.22	2.9	3.5	Column 1 controls
Shear (k)	**0.90**	147	134	Semi-Ductile Shear
CROSSBEAM				
Moment (k-ft)	**0.40**	2218	885	
Shear (k)	**0.61**	684	417	
GIRDER STOPS				
Pier 2 (k)	2.29	758	1736	
Pier 3 (k)	2.59	671	1736	
Pier 4 (k)	2.15	808	1736	
SEAT LENGTH				
End Pier 1 (in)	1.49	18.5	27.5	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
Pier 2	1.21	21.2	22.4	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
Pier 3	1.06	20.9	22.4	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
Pier 4	1.07	20.9	22.4	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
End Pier 4 (in)	1.31	21.0	27.5	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)

Widened (1993) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				
Moment (k-ft)	**0.75**	3379	2531	
Shear (k)	2.31	392	905	
Overturning (k-ft)	**0.96**	4890	4702	
Sliding (k)	**0.90**	392	352	
COLUMN				
Long. Displacement (in)	3.03	3.7	11.1	
Transv. Displacement (in)	4.21	1.2	4.9	
Shear (k)	1.02	483	492	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.23**	3542	800	

Shear (k)	**0.63**	359	226	
PIER 3				
FOOTING				
Moment (k-ft)	1.05	2406	2531	
Shear (k)	2.26	401	905	
Overturning (k-ft)	1.15	5543	6375	
Sliding (k)	1.01	401	405	
COLUMN				
Long. Displacement (in)	4.19	3.8	16.0	
Transv. Displacement (in)	2.88	2.6	7.4	
Shear (k)	1.30	395	513	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.18**	4385	800	
Shear (k)	**0.56**	406	226	
PIER 4				
FOOTING				
Moment (k-ft)	1.08	2348	2531	
Shear (k)	2.29	395	905	
Overturning (k-ft)	1.11	5498	6112	
Sliding (k)	**0.98**	395	387	
COLUMN				
Long. Displacement (in)	4.45	3.8	16.9	
Transv. Displacement (in)	2.58	2.9	7.4	
Shear (k)	1.31	388	510	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.16**	4922	800	
Shear (k)	**0.52**	435	226	
GIRDER STOPS				
Pier 2 (k)	2.29	758	1736	
Pier 3 (k)	2.59	671	1736	
Pier 4 (k)	2.15	808	1736	
SEAT LENGTH				
End Pier 1 (in)	1.51	18.2	27.5	AASHTO LRFD Seismic Spec. (4.12.3-1)
Pier 2 (in)	1.23	19.8	22.4	AASHTO LRFD Seismic Spec. (4.12.3-1)
Pier 3 (in)	1.13	19.9	22.4	AASHTO LRFD Seismic Spec. (4.12.3-1)
Pier 4 (in)	1.12	20.0	22.4	AASHTO LRFD Seismic Spec. (4.12.3-1)
End Pier 5 (in)	1.38	20.0	27.5	AASHTO LRFD Seismic Spec. (4.12.3-1)

2.3.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- The as-built plans show the seat length provided at the intermediate and end piers is adequate to prevent unseating of the end spans.

- No girder stops are present at the end piers. This is not preferable as end piers have a lot of lateral resistance that can help ‘protect’ the interior piers.
- The girder stops present at the intermediate piers are adequate to achieve good seismic performance, however a girder stop is not present at each girder bay. This is not preferable as it requires significant load to transfer through diaphragms and girder webs.
- Method D2 analysis shows that the original columns have marginally adequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of the 1966 columns is inadequate for resisting shear resulting from plastic hinging in the column.
- The existing crossbeam is both deficient in shear and flexure. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges.
- The original spread footing foundations supporting the original columns of Piers 2, 3 and 4 lack the necessary overturning capacity to resist the column plastic hinging forces. Additionally, footings supporting the widening show additional deficiencies for flexure, overturning and sliding. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.3.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around the original 1966 columns. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening. Installation of column jackets will require shoring.

   The addition of concrete bolsters at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a deficient column displacement C/D ratio in the as-built condition will decrease even further with the introduction of crossbeam bolsters.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments near columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the

length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment and shear capacity can be increased enough to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width. Pier 3 may require an altered concept as there may limited clearance to the SR520 travel way.

3. We recommend adding girder stops at both end piers as it is a low cost addition that helps make sure the bridge response is more predictable and reduces the transverse demand on the adjacent piers.
4. Current WSDOT practice is to provide girder stops in each girder bay to better distribute transverse shear loads amongst the girders. Thus we propose the addition of girder stops at each girder bay at the intermediate piers included with the crossbeam bolsters to provide positive transverse restraint at each girder without relying on the end diaphragms to transfer shear between girders.

2.3.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1.	Steel Column Jackets on Original Columns	$475,000
2.	Reinforced Concrete Bolsters w/ Girder Stops at Piers 2, 3 and 4	$480,000
3.	End Pier Girder Stops	$65,000
4.	Miscellaneous (Downspout relocations, etc.)	$20,000

2.4 Bridge No. 405/46W

2.4.1 Bridge Description

Bridge 405/46W consists of four continuous prestressed concrete I-girder spans. Originally constructed in 1966, the spans lengths are 46'-3", 73-0", 61'-0", and 60'-6" from south to north to create a total length of 240'-9" from back-to-back of pavement seats. The original roadway width varied between 65'-6" and 67'-9". All piers are on a skew that vary slightly and are about 16 degrees measured normal to the bridge alignment line at the centerline of piers for intermediate piers and back of pavement seat for end piers.

All spans are simply supported and consisting of twelve 80 Ft-Series prestressed concrete girders. The intermediate piers have a 4'-0" wide by 3'-3" tall dropped crossbeam supporting the superstructure above. The original crossbeams sit on three 3'-0" columns that are founded on spread footings. The end piers are L-shaped and supported on spread footings.

The bridge was widened about 19 feet in 1993 to the east adding three W50G prestressed girders to the framing. At this time, the west barrier was replaced. The intermediate piers were extended and one 3'-0" diameter column was added and supported on a 16 feet by 16 feet spread footing. At the end piers, the L-shaped end piers were extended and supported on new spread footings.

Bridge No 405/46W – I-405 over SR 520

2.4.2 Bridge 405/46W C/D Ratios for Method D2 Analysis

Original (1966) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				
Moment (k-ft)	1.74	890	1546	Column 1 controls
Shear (k)	**0.59**	542	320	Column 3 controls
Overturning (k-ft)	1.06	1817	1932	Column 1 controls
Sliding (k)	1.40	166	232	Column 1 controls
COLUMN				
Long. Displacement (in)	**0.84**	3.8	3.2	Column 3 controls
Transv. Displacement (in)	1.30	1.6	2.1	Column 3 controls
Shear (k)	**0.73**	157	115	Semi-Ductile Shear
CROSSBEAM				
Moment (k-ft)	**0.52**	1709	885	
Shear (k)	**0.39**	584	230	
PIER 3				
FOOTING				
Moment (k-ft)	1.84	539	1546	Column 1 controls
Shear (k)	**0.62**	515	320	Column 3 controls

Overturning (k-ft)	1.15	1811	2090	Column 1 controls
Sliding (k)	1.81	140	254	Column 1 controls
COLUMN				
Long. Displacement (in)	1.16	3.8	4.4	Column 3 controls
Transv. Displacement (in)	1.33	2.9	2.2	Column 3 controls
Shear (k)	**0.87**	164	142	Semi-Ductile Shear
CROSSBEAM				
Moment (k-ft)	**0.52**	1698	885	
Shear (k)	**0.38**	608	230	
PIER 4				
FOOTING				
Moment (k-ft)	1.84	842	1546	Column 1 controls
Shear (k)	**0.63**	509	320	Column 3 controls
Overturning (k-ft)	1.12	1751	1961	Column 1 controls
Sliding (k)	1.77	133	237	Column 1 controls
COLUMN				
Long. Displacement (in)	1.19	3.9	4.6	Column 3 controls
Transv. Displacement (in)	1.16	2.7	3.1	Column 3 controls
Shear (k)	**0.87**	158	138	Semi-Ductile Shear
CROSSBEAM				
Moment (k-ft)	**0.41**	2158	885	
Shear (k)	**0.36**	640	230	
GIRDER STOPS				
Pier 2 (k)	2.66	729	1941	
Pier 3 (k)	3.57	544	1941	
Pier 4 (k)	2.77	700	1941	
SEAT LENGTH				
End Pier 1 (in)	1.5	19	28	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
Pier 2 (in)	1.02	22.4	22.9	
Pier 3 (in)	1.03	22.2	22.9	
Pier 4 (in)	1.03	22.2	22.9	
End Pier 5 (in)	1.35	20.8	28.1	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)

Widened (1993) Structure Members - SEE

ITEM	**C/D RATIO**	**DEMAND**	**CAPACITY**	**NOTES:**
PIER 2				
FOOTING				
Moment (k-ft)	**0.74**	3195	2351	
Shear (k)	1.50	611	914	
Overturning (k-ft)	**0.73**	4980	3645	
Sliding (k)	**0.55**	437	240	
COLUMN				
Long. Displacement (in)	2.54	3.6	9.2	
Transv. Displacement (in)	3.56	1.6	5.7	
Shear (k)	1.23	546	669	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.29**	3340	954	
Shear (k)	**0.53**	445	238	

PIER 3				
FOOTING				
Moment (k-ft)	**0.82**	2865	2351	
Shear (k)	1.48	620	914	
Overturning (k-ft)	**0.88**	4630	4060	
Sliding (k)	**0.76**	352	269	
COLUMN				
Long. Displacement (in)	3.19	3.7	11.9	
Transv. Displacement (in)	3.33	2.2	7.3	
Shear (k)	1.49	444	664	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.24**	3155	766	
Shear (k)	**0.51**	463	238	
PIER 4				
FOOTING				
Moment (k-ft)	**0.67**	3504	2351	
Shear (k)	1.88	487	914	
Overturning (k-ft)	**0.71**	5230	3726	
Sliding (k)	**0.64**	386	245	
COLUMN				
Long. Displacement (in)	3.40	3.8	12.8	
Transv. Displacement (in)	2.84	2.7	7.6	
Shear (k)	1.56	421	658	Ductile Shear, therefore will not control
CROSSBEAM				
Moment (k-ft)	**0.18**	4228	766	
Shear (k)	**0.51**	465	238	
GIRDER STOPS				
Pier 2 (k)	2.66	729	1941	
Pier 3 (k)	3.57	544	1941	
Pier 4 (k)	2.77	700	1941	
SEAT LENGTH				
End Pier 1 (in)	1.52	18.5	28.1	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
Pier 2 (in)	1.10	20.8	22.9	
Pier 3 (in)	1.09	21.1	22.9	
Pier 4 (in)	1.09	21.1	22.9	
End Pier 5 (in)	1.40	20.0	28.1	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)

2.4.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- The as-built plans show the seat length provided at the end piers is adequate to prevent unseating of the end spans.
- No girder stops are present at the end piers. This is not preferable as end piers have a lot of lateral resistance that can help ‘protect’ the interior piers.
- The girder stops present at the intermediate piers are adequate to achieve good seismic performance, however a girder stop is not present at each girder bay.

This is not preferable as it requires significant load to transfer through diaphragms and girder webs.

- Method D2 analysis shows that the original columns have marginally adequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of the 1966 columns is inadequate for resisting shear resulting from plastic hinging in the column.
- The existing crossbeam is both deficient in shear and flexure. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges.
- The foundations were found to be adequate for this structure. However, the additional column jackets may cause a vulnerability as the existing foundations may not be adequate to capacity protect the foundation elements. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.
- The original spread footing foundations supporting the original columns of Piers 2, 3, and 4 lack the necessary shear capacity to resist the column plastic hinging forces. Additionally, the footings supporting the 1993 widening are adequate for shear but have additional deficiencies for flexure, overturning and sliding. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.4.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of column jackets around the original 1966 columns. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening. Installation of column jackets will require shoring.

   The addition of concrete bolsters at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a deficient column displacement C/D ratio in the as-built condition will decrease even further with the introduction of crossbeam bolsters.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments near columns and the

amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment and shear capacity can be increased enough to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width. Pier 3 may require an altered concept as there may limited clearance to the 520 travel way.

3. We recommend adding girder stops at both end piers as it is a low cost addition that helps make sure the bridge response is more predictable and reduces the transverse demand on the adjacent piers.
4. Current WSDOT practice is to provide girder stops in each girder bay to better distribute transverse shear loads amongst the girders. Thus we propose the addition of girder stops at each girder bay at the intermediate piers included with the crossbeam bolsters to provide positive transverse restraint at each girder without relying on the end diaphragms to transfer shear between girders.

2.4.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1.	Steel Column Jackets on Original Columns	$460,000
2.	Reinforced Concrete Bolsters w/ Girder Stops at Piers 2, 3 and 4	$495,000
3.	End Pier Girder Stops	$70,000
4.	Miscellaneous (Downspout relocations, etc.)	$20,000

2.5 Bridge No. 405/47E

2.5.1 Bridge Description

Bridge 405/47E consists of three continuous prestressed concrete I-girder spans. The center span is the longest measuring 58 feet while the other two spans are 51 feet each. The spans vary in width from 77 feet to 80 feet. All spans are composed of a total of fifteen girders spaced evenly; twelve of the fifteen girders were part of the original 1965 construction while the other three girders were added on as part of the 1993 widening. The I-girders are made continuous for live load over the piers. The deck is 7.25-inch-thick over the widened section with no overlay, while the old deck is 5.75-inch-thick and has a 1.5-inch latex modified concrete (LMC) overlay making the deck the same thickness over the entire bridge. A standard 32-inch shape F traffic barrier is in placed on both sides of the bridge.

The bridge is supported by skewed end piers and intermediate piers, the skew is constant along the entire bridge at 12 degrees. The end piers are seat-type end piers consisting of a crossbeam that is integral with the girders. Each girder sits on a bearing pad and contains no longitudinal restrainers. In the transverse direction a combination of wingwalls and girder stops is employed to limit lateral displacements. The original bridge does not use girder stops and has only the wing walls at the ends. The widened portion uses girder stops between each of the added girders, no girder stops were added to the old section during the widening. A wingwall at the end of the widened portion is also in place. The intermediate piers consist of four columns and an integral crossbeam. Three of the four columns were part of the original 1965 bridge and the other column was added during the widening. Each column is supported on a spread footing ranging in depth below ground surface from 5 to 10 feet. The girders frame into an integral diaphragm at the piers. The intermediate pier diaphragms have two different connections to the crossbeam; for the old section of bridge the diaphragm is integral with the crossbeams, and for the new section the diaphragm is hinged to the crossbeam. The hinge used is the typical WSDOT "Intermediate Hinge Diaphragm." A light pole and mast arm located between girders G and H should be considered if retrofits are chosen for this bridge.

Bridge No 405/47E – I-405 over Northup Way

2.5.2 Bridge 405/47E C/D Ratios for Method D2 Analysis

Original (1965) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				

Moment (k-ft)	**-**	1420	-	Footings do not contain top steel.
Shear (k)	**0.78**	326	252	Columns 2 & 4 controls
Overturning (k-ft)	2.09	848	1773	Columns 2 & 4 controls
Sliding (k)	1.64	69	114	Column 3 controls
COLUMN				
Long. Displacement (in)	**0.38**	4.4	1.7	Column 4 Controls
Transv. Displacement (in)	**0.32**	5.2	1.7	Column 4 Controls
Shear (k)	1.55	51.3	79.3	Vf vs Vp - Ductile Shear-controlled at column 4
CROSSBEAM				
Moment (k-ft)	**0.49**	3853	1905	Negative Moment
Shear (k)	**0.75**	442	330	
PIER 3				
FOOTING				
Moment (k-ft)	**-**	1420	-	Footings do not contain top steel.
Shear (k)	**0.78**	322	252	Columns 2 & 4 controls
Overturning (k-ft)	2.12	829	1754	Columns 2 & 4 controls
Sliding (k)	1.67	65	108	Column 3 controls
COLUMN				
Long. Displacement (in)	**0.38**	4.5	1.7	Column 4 Controls
Transv. Displacement (in)	**0.34**	5.0	1.7	Column 4 Controls
Shear (k)	1.55	51	79	Vf vs Vp - Ductile Shear-controlled at column 4
CROSSBEAM				
Moment (k-ft)	**0.45**	4274	1905	Negative Moment
Shear (k)	**0.75**	440	330	
GIRDER STOP				
End Pier 1 (k)	**0.79**	562	444	
End Pier 4 (k)	**0.90**	494	444	
SEAT LENGTH				
End Pier 1 (in)	1.33	24	32	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
End Pier 4 (in)	1.33	24	32	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)

Widened (1993) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				
Moment (k-ft)	**0.63**	2700	1703	
Shear (k)	1.34	529	710	
Overturning (k-ft)	1.29	2659	3423	
Sliding (k)	**0.68**	212	145	
COLUMN				
Long. Displacement (in)	1.88	4.9	9.1	
Transv. Displacement (in)	1.84	5.0	9.1	

Shear (k)	4.11	120	493	Vf vs Vp - Ductile Shear-controlled at column 1
CROSSBEAM				
Moment (k-ft)	**0.13**	2438	320	Positive Moment
Shear (k)	1.81	185	335	
PIER 3				
FOOTING				
Moment (k-ft)	**0.61**	2811	1703	
Shear (k)	1.32	537	710	
Overturning (k-ft)	1.36	2632	3569	
Sliding (k)	**0.75**	201	152	
COLUMN				
Long. Displacement (in)	1.90	4.8	1.7	
Transv. Displacement (in)	1.71	5.3	1.7	
Shear (k)	4.11	120	493	Vf vs Vp - Ductile Shear-controlled at column 1
CROSSBEAM				
Moment (k-ft)	**0.17**	1827	320	Positive Moment
Shear (k)	1.82	184	320	
SEAT LENGTH				
End Pier 1 (in)	1.33	24	32	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)
End Pier 4 (in)	1.33	24	32	AASHTO LRFD Seismic Guide Spec. (4.12.3-1)

2.5.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- The as-built plans show the seat length provided at the end piers is adequate to prevent unseating of the end spans.
- The girder stops are only present at the end piers in the widening. Common practice is to place these between every girder to evenly distribute the load. Girder stops at the end piers provide a lot of lateral resistance that can help 'protect' the interior piers.
- Method D2 analysis shows that the original columns have inadequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of the 1965 columns is inadequate for resisting shear resulting from plastic hinging in the column.
- The existing crossbeam is deficient in flexure and shear. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges.

- The original spread footing foundations supporting the original columns of Piers 2 and 3 lack top flexural steel and are therefore deficient in bending. These original footings are also deficient for shear. The footings supporting the 1993 widening are deficient in bending and sliding. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.5.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around the original columns that are seismically deficient. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening. Installation of column jackets will require shoring.

   The addition of concrete bolsters at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a deficient column displacement C/D ratio in the as-built condition will decrease even further with the introduction of crossbeam bolsters.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments next to columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment and shear capacity can be increased enough to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width.

3. We recommend adding girder stops at both end piers as it is a low cost addition that helps make sure the bridge response is more predictable and reduces the transverse demand on the adjacent piers.

4. Current WSDOT practice is to provide girder stops in each girder bay to better distribute transverse shear loads amongst the girders. Thus we propose the addition of girder stops at each girder bay at the intermediate piers included with the crossbeam bolsters to provide positive transverse restraint at each girder without relying on the end diaphragms to transfer shear between girders.

2.5.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1.	Steel Column Jackets on Original Columns	$360,000
2.	Reinforced Concrete Bolsters w/ Girder Stops at Piers 2 and 3	$325,000
3.	End Pier Girder Stops	$60,000

2.6 Bridge No. 405/47W

2.6.1 Bridge Description

Bridge 405/47W consists of three continuous cast-in-place T-beam spans. The center span is the longest measuring 58 feet while the other two spans are 44 feet each. The spans are all uniform in width at 70 feet. All spans are composed of a total of ten T-beams with varying spacing. The original bridge portion from 1953 had four T-beams, four others were added on the 1965 widening (three to the west side of the structure and one to the east side) along with one additional column to the west. In the 1992 widening the T-beam added on the east side of the structure was removed and three other T-beams and a column were added to that side (east side of structure). The deck over the three girders on the west of the structure from 1965 was formed with a ½ inch longitudinal gap between itself and the main deck. The end piers were cast continuous with the existing, but the pier crossbeams and the intermediate diaphragms are not. This makes two separate bridges connected at the end piers only. The deck is 7.50-inch thick over the 1992 widened section with no overlay, while the rest of the deck is 6-inch thick and has a 1.5-inch latex modified concrete (LMC) overlay making the deck the same thickness across the entire bridge. A standard 32-inch shape F traffic barrier is in place on each side of the bridge.

The bridge is supported by skewed end piers and skewed intermediate piers; the skew is constant along the entire bridge at 15 degrees. The end piers are integral with the superstructure through a crossbeam / pile cap. Each end pier has thirteen driven precast concrete piles with a 13-inch diameter each and two drilled shafts with a 3 feet diameter each. The intermediate piers are also integral with the superstructure. The piers each have four square columns two built along with the original bridge and two added during the subsequent widenings. The original two columns are supported on a 9 feet X 9 feet square spread footing each, the column added in the 1965 widening is supported on a group of driven piles with a pile cap. The 1992 column is supported on a 6 feet diameter drilled shaft.

The T-Beams have a parabolic depth tapering along each span, with a maximum depth of 5 feet and a minimum depth of 2 feet 10 inches. The bridge also contains intermediate crossbeams that follow the same skew angle as the end piers and intermediate piers. The bridge has electrical utilities running underneath.

Bridge No 405/47W – I-405 over Northup Way

2.6.2 Bridge 405/47W C/D Ratios for Method D2 Analysis

Original (1953 and 1965) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
SPREAD FOOTING				
Moment (k-ft)	-	929	-	Footings do not contain top steel.
Shear (k)	**0.65**	437	284	Column 2 controls
Overturning (k-ft)	**0.87**	1788	1559	Column 2 controls
Sliding (k)	**0.98**	102	99	Column 2 controls
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	656	-	no top steel on cap
Pile Cap Shear (k)	**0.92**	761	703	face perpendicular to longitudinal direction
Pile Axial (k)	**0.23**	308	70	
Pile Shear (k)	**4.46**	250	1117	
Pile Connection (k)	**0.02**	221	4	
COLUMN				
Long. Displacement (in)	**0.61**	3.6	2.2	Column 1 controls
Transv. Displacement (in)	**0.72**	3.1	2.2	Column 1 controls
Shear (k)	**0.69**	245	169	Vf vs Vp - Semi-Ductile Shear Controlled at Column 1
CROSSBEAM				
Moment (k-ft)	**0.44**	1595	695	controlling at face of col 3
Shear (k)	**0.67**	517	346	controlling at face of col 3

PIER 3				
SPREAD FOOTING				
Moment (k-ft)	-	929	-	Footings do not contain top steel.
Shear (k)	0.65	437	284	Column 2 controls
Overturning (k-ft)	0.87	1788	1559	Column 2 controls
Sliding (k)	0.98	102	99	Column 2 controls
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	656	-	no top steel on cap
Pile Cap Shear (k)	0.92	761	703	face perpendicular to longitudinal direction
Pile Axial (k)	0.23	308	70	
Pile Shear (k)	4.46	250	1117	
Pile Connection (k)	0.02	221	4	
COLUMN				
Long. Displacement (in)	0.61	3.6	2.2	Column 1 controls
Transv. Displacement (in)	0.63	3.5	2.2	Column 1 controls
Shear (k)	0.69	245	169	Vf vs Vp - Semi-Ductile Shear Controlled at Column 1
CROSSBEAM				
Moment (k-ft)	0.44	1595	695	controlling at face of col 3
Shear (k)	0.67	517	346	controlling at face of col 3
SUPERSTRUCTURE				
1953 Const. Moment (k-ft)	0.80	575	458	9 ft away from CL of support
1965 Const. Moment (k-ft)	0.30	1606	480	at face of support

Widened (1992) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
DRILLED SHAFT				
Moment (k-ft)	1.68	4243	7133	
Shear (k)	5.15	375	1928	
Axial (k)	5.39	520	2800	
COLUMN				
Long. Displacement (in)	4.82	1.6	7.7	
Transv. Displacement (in)	4.05	1.9	7.7	
Shear (k)	2.53	313	792	Vf vs Vp - Ductile Shear Controlled at Column 4
CROSSBEAM				
Moment (k-ft)	0.45	4736	2121	positive moment (bottom steel)
Shear (k)	1.12	484	602	
PIER 3				
DRILLED SHAFT				
Moment (k-ft)	1.68	4243	7133	
Shear (k)	5.15	375	1928	
Axial (k)	5.39	520	2800	
COLUMN				

Long. Displacement (in)	4.82	1.6	7.7	
Transv. Displacement (in)	3.79	2.0	7.7	
Shear (k)	2.53	313	792	Vf vs Vp - Ductile Shear Controlled at Column 4
CROSSBEAM				
Moment (k-ft)	**0.45**	4736	2121	positive moment (bottom steel)
Shear (k)	1.12	484	602	

Original (1953 and 1965) Structure Members - FEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
SPREAD FOOTING				
Moment (k-ft)	-	824	-	Footings do not contain top steel.
Shear (k)	1.10	258	524	Column 2 controls
Overturning (k-ft)	2.27	687	1559	Column 2 controls
Sliding (k)	4.00	25	99	Column 2 controls
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	425	-	No top steel on pile cap
Pile Cap Shear (k)	1.42	494	703	face perpendicular to longitudinal direction
Pile Axial (k)	**0.41**	170	70	
Pile Shear (k)	10.36	108	1117	
Pile Connection (k)	**0.03**	113	4	
PIER 3				
SPREAD FOOTING				
Moment (k-ft)	-	829	-	Footings do not contain top steel.
Shear (k)	1.05	271	284	Column 2 controls
Overturning (k-ft)	2.05	762	1559	Column 2 controls
Sliding (k)	3.60	28	99	Column 2 controls
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	-	423	-	No top steel on pile cap
Pile Cap Shear (k)	1.42	495	703	face perpendicular to longitudinal direction controls
Pile Axial (k)	**0.39**	178	70	
Pile Shear (k)	10.36	108	1117	
Pile Connection (k)	**0.03**	121	4	

Widened (1992) Structure Members - FEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
DRILLED SHAFT				
Moment (k-ft)	21.45	767	16462	Calculated using P-M curve
Shear (k)	19.51	99	1928	
Axial (k)	7.30	383	2800	
PIER 3				
DRILLED SHAFT				
Moment (k-ft)	25.91	635	16462	Calculated using P-M curve
Shear (k)	17.04	113	1928	

Axial (k)	7.13	393	2800	

2.6.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- Method D2 analysis shows that the original columns have inadequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of the 1953 and 1965 columns is inadequate for resisting shear resulting from plastic hinging in the column.

- The existing crossbeam is deficient in flexure and shear. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges.

- The superstructure has a significant moment deficiency. This vulnerability is present at the ends of the bridge where it is integrally connected to the end piers and also appears near Piers 2 and 3 where the moment reinforcing has been dropped off as it is presumably no longer required for the strength and/or service load cases.

- At the lower level event, the foundations supporting the 1953 and 1965 original columns of Piers 2 and 3 lack top flexural steel and are therefore deficient in bending. The 1965 column footings were also found to be deficient in pile axial and pile to pile cap connection. There may be additional capacity at the pile to pile cap connection; however, documentation of the pile connection is insufficient. Therefore, only adhesion is assumed for capacity. The 1992 widening is supported on drilled shafts which were found to have adequate capacity.
- At the upper level event, the foundations supporting the 1953 and 1965 original columns of Piers 2 and 3 lack top flexural steel and are therefore deficient in bending. These footings are also deficient for shear, overturning and sliding while the 1965 piles were found to have additional deficiencies. The 1992 widening was supported on drilled shafts which were found to have adequate capacity. We are not scoped to discuss upper level retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies

2.6.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around the 1953 and 1962 columns that are seismically deficient. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening. Installation of column jackets will require shoring.

   The addition of concrete bolsters at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a deficient column displacement C/D ratio in the as-built condition will decrease even further with the introduction of crossbeam bolsters.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments next to columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment and shear capacity can be increased to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width.
3. We recommend strengthening the superstructure using fiber reinforced polymer (FRP) to provide additional moment capacity for the girders. FRP on the bottom of the girders (traditionally thought of as the positive moment region) will provide an alternative load path that will allow for adequate moment redistribution. The analysis shows negative moment deficiency as well, however, strengthening the superstructure for negative moment resistance is more difficult.
4. Foundation retrofits have not been a primary focus for WSDOT and for the lower level event would entail expanding the footings, adding pin piles around the perimeter, and thickening the footings. This could be designed to address these foundation deficiencies for the lower level event.

2.6.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1. Steel Column Jackets on 1953 & 1965 Columns	$875,000
2. Crossbeam Strengthening w/ Girder Stops at Piers 2 and 3	$400,000
3. FRP Superstructure Strengthening	$450,000
4. Miscellaneous (Utility relocations, etc.)	$40,000

WSDOT has not typically begun to retrofit foundations in this phase of their seismic retrofit program. Therefore, the following construction costs (including 20% Mobilization and 25% Contingency) are being included for planning purposes. Note that these costs assume retrofits for the lower level event, and may increase if WSDOT intends to remove the vulnerabilities for an upper level event.

1. Pin piles and thickened footings $450,000

2.7 Bridge No. 405/48E

2.7.1 Bridge Description

Bridge 405/48E consists of four continuous prestressed concrete I-girder spans. Originally constructed in 1965, the spans lengths are 82’-0”, 90-6”, 58’-6”, and 62’-0” from south to north to create a total length of 293’-0” from back-to-back of pavement seats. The original roadway width varied between 60’-6” and 64’-3”. All piers are on a skew that is about 39 degrees measured normal to the bridge alignment line at the centerline of piers for intermediate piers and back of pavement seat for end piers.

All spans are simply supported and consisting of nine original prestressed concrete girders. The intermediate piers have a 3’-4” wide by 3’-4” tall dropped crossbeam supporting the superstructure above. The original crossbeams sit on four 3’-0” columns. Piers 2 and 4 are founded on spread footings. Pier 3 columns are integral with a retaining wall and extend to the top of the continuous spread footing supporting the wall.This retaining wall supports the west side of 115th Ave NE. The end piers are supported on spread footings. Pier 2 has a partial height crash wall between the original columns as there were once railroad tracks under the bridge between Piers 2 and 3. These tracks have since been abandoned.

The bridge was widened about 19.5 feet in 1993 to the west adding three W58G prestressed girders to the framing. At this time, the east barrier was replaced. The intermediate piers were extended and supported by one additional column (3’-4” diameter for Pier 2 and Pier 3 and 3’-0” diameter for Pier 4). The new Pier 2 and Pier 3 columns were founded on a drilled shaft, and Pier 4 is founded on a 15 feet by 15 feet spread footing. The retaining wall was extended to the west at Pier 3. At the end piers, the end piers were extended and supported on new spread footings. The crash wall was extended between the original westernmost column and the new Pier 2 column.

Bridge No 405/48E – I-405 over 115th Ave NE

2.7.2 Bridge 405/48E C/D Ratios for Method D2 Analysis

Original (1965) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
FOOTING				
Moment (k-ft)	1.67	153	256	
Shear (k)	**0.99**	323	318	Column 4 controls
Overturning (k-ft)	2.64	1272	3363	Column 4 controls
Sliding (k)	1.74	305	531	Column 3 controls
COLUMN				
Long. Displacement (in)	2.82	4.34	12.24	Column 3 controls
Transv. Displacement (in)	1.51	1.38	2.08	Column 1 controls
Shear (k)	**0.42**	322	136	Column 4 controls
CROSSBEAM				
Moment (k-ft)	**0.69**	3096	2145	Negative Moment
Shear (k)	**0.83**	453	374	
PIER 3				
FOOTING				
Moment (k-ft)	**0.17**	2406	413	Column 4 controls

Shear (k)	**0.21**	588	126	Column 2 controls
Overturning (k-ft)	**0.28**	1707	486	Column 1 controls
Sliding (k)	**0.17**	939	161	Column 1 controls
COLUMN				
Long. Displacement (in)	4.03	4.21	16.94	Column 2 controls
Transv. Displacement (in)	**0.36**	1.23	0.44	Column 2 controls
Shear (k)	**0.40**	342	138	Column 4 controls
CROSSBEAM				
Moment (k-ft)	**0.43**	5030	2145	Negative Moment
Shear (k)	**0.47**	711	335	
PIER 4				
FOOTING				
Moment (k-ft)	**0.82**	1314	1077	Column 3 controls
Shear (k)	**0.48**	493	238	Column 4 controls
Overturning (k-ft)	**0.78**	1996	1559	Column 1 controls
Sliding (k)	1.60	159	254	Column 1 controls
COLUMN				
Long. Displacement (in)	1.64	4.21	6.90	Column 3 controls
Transv. Displacement (in)	1.80	1.64	2.95	Column 1 controls
Shear (k)	**0.86**	164	141	Column 1 controls
CROSSBEAM				
Moment (k-ft)	**0.74**	2886	2145	Negative Moment
Shear (k)	**0.71**	456	326	
SEAT LENGTH				
End Pier 1 (in)	**0.45**	44	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
End Pier 4 (in)	**0.45**	44	20	FHWA Seismic Retrofitting Manual Eq. 5.1b

Widened Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
PIER 2				
SHAFT				
Moment (unitless)	2.19	0.46	1.00	Unitless as this was computed as $P_u/P_r + M_u/M_r$
Shear (k)	23.20	204	4723	
Axial (k)	**0.92**	825	760	
COLUMN				
Long. Displacement (in)	3.77	4.27	16.09	
Transv. Displacement (in)	4.48	1.37	6.12	
Shear (k)	**0.97**	677	654	
CROSSBEAM				
Moment (k-ft)	**0.25**	2845	699	Positive Moment
Shear (k)	1.00	553	553	
PIER 3				
SHAFT				
Moment (unitless)	1.62	0.62	1.00	Unitless as this was computed as $P_u/P_r + M_u/M_r$
Shear (k)	8.53	554	4723	
Axial (k)	**0.40**	1879	760	

COLUMN				
Long. Displacement (in)	2.76	4.17	11.50	
Transv. Displacement (in)	1.00	1.16	1.17	
Shear (k)	**0.68**	1019	693	
CROSSBEAM				
Moment (k-ft)	**0.11**	6224	699	Positive Moment
Shear (k)	**0.72**	553	768	
PIER 4				
FOOTING				
Moment (k-ft)	**0.57**	3055	1734	
Shear (k)	1.01	491	496	
Overturning (k-ft)	**0.67**	4050	2726	
Sliding (k)	**0.97**	303	293	
COLUMN				
Long. Displacement (in)	3.44	4.21	14.48	
Transv. Displacement (in)	3.53	1.63	5.75	
Shear (k)	1.12	435	487	
CROSSBEAM				
Moment (k-ft)	**0.24**	2967	699	Positive Moment
Shear (k)	1.06	447	474	
SEAT LENGTH				
End Pier 1 (in)	**0.52**	39	20	FHWA Seismic Retrofitting Manual Eq. 5.1b
End Pier 4 (in)	**0.52**	39	20	FHWA Seismic Retrofitting Manual Eq. 5.1b

2.7.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- The as-built plans show a provided seat length at the end piers that is less than that required, based on the AASHTO Guide Spec. However the displacements determined from the RSA analysis indicate that anticipated displacements at the end piers are well within the limits of the available seat widths. Thus, we do not feel that seat extensions at the end piers are warranted.
- No girder stops are present at the end piers. This is not preferable as end piers have a lot of lateral resistance that can help 'protect' the interior piers.
- The intermediate pier hinge diaphragms are connected to the crossbeams with #9 and #10 or #9 and #11 bars – depending on the pier – and the girders are embedded 2 inches into the pier hinge diaphragms with extended strands. This provides adequate connectivity to achieve good seismic performance.
- Method D2 analysis shows that the original columns have inadequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake at Pier 3. The results also found the shear capacity of original columns is inadequate for resisting shear resulting from plastic hinging in the column. The column added at Pier 3 to support the widening is marginally adequate for displacement capacity and all widening columns had at least a

slight deficiency for shear. These results are expected at Pier 3 as the retaining wall causes restraint for part of the height in the transverse section thus shortening the effective column height.

- The existing crossbeam is both deficient in shear and flexure. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges.

- The foundations supporting the original columns of Piers 2 and 4 lack the necessary capacity to resist the column plastic hinging forces, and the more recent spread footing foundations show a slight vulnerability to moment, sliding and overturning. The drilled shafts at Piers 2 & 3 were found to have vulnerability for axial capacity. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.7.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around all of the Pier 2, Pier 3, and Pier 4 columns that are seismically deficient. At Pier 3, only the portion of column above the retaining wall should be jacketed unless it is found during the post-retrofit design phase that this detail is not adequate to remove the vulnerability. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening. Installation of column jackets will require shoring.

   The addition of concrete bolsters at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a deficient column displacement C/D ratio in the as-built condition will decrease even further with the introduction of crossbeam bolsters.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments next to columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam’s moment and shear capacity can be increased to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement.

However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width.

3. Current WSDOT practice is to provide girder stops in each girder bay to better distribute transverse shear loads amongst the girders. Thus we propose the addition of girder stops at each girder bay at the intermediate piers included with the crossbeam bolsters to provide positive transverse restraint at each girder without relying on the end diaphragms to transfer shear between girders.
4. Add girder stops in each girder bay at the end piers to better use the stiffness of the abutments and distribute transverse shear loads amongst the girders. Thus we propose the addition of girder stops at each girder bay at the end piers.
5. The partial height crash walls located at Pier 2 should be removed to increase the effective height of these columns and obtain more predictable seismic performance.

2.7.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1.	Steel Column Jackets on All Columns	$725,000
2.	Reinforced Concrete Bolsters w/ Girder Stops at Piers 2-4	$730,000
3.	End Pier Girder Stops	$33,000
4.	Crash Wall Removal	$35,000
5.	Miscellaneous (Utility relocations, etc.)	$40,000

2.8 Bridge No. 405/48W

2.8.1 Bridge Description

Bridge 405/48W consists of three continuous cast-in-place concrete T-beam spans. Originally constructed in 1954, the center span is the longest measuring 80 feet while the other two spans are 62 feet each to create a total length of 204’-0” from back-to-back of pavement seats. The original roadway width measured 31’-0”. All piers are on a skew that is about 41 degrees measured normal to the bridge alignment line at the centerline of piers for intermediate piers and back of pavement seat for end piers.

All spans are continuous and T-beam girders are haunched varying in height from 3’-6” to 7’-0”. The intermediate piers are integral and have a 2’-4” wide by 7’-6” tall crossbeam supporting the superstructure. The original crossbeams sit on two 3’-6” square columns that are founded on 12-feet square spread footings. At the end piers, the girders frame into an integral crossbeam. This crossbeam bears on four 2’-1” by 3’-4” concrete columns that are founded on a combined spread footing.

The bridge was widened about 19.25 feet in 1965 to the west adding three girder lines. At this time, the east edge was also expanded adding one additional girder to support a new barrier along this edge. The intermediate piers were extended and supported on one 3’-6” square column. The Pier 2 footing is connected to a 12’-4” x 12’-9” pile cap which in turn is founded on sixteen 13-inch concrete piles. The Pier 3 footing is supported on a 12’-4” by 15’-0” spread footing. At the end piers, the girders frame into a crossbeam that bears on 18-inch diameter concrete piles.

The bridge was again widened about 14.5 feet in 1993 to the east adding three girder lines. The intermediate piers were extended and supported on one 3’-6” square column. The Pier 2 and Pier 3 columns were then founded on a 6’-0” diameter drilled shaft. At the end piers, the girders frame into a crossbeam that is supported on 36-inch diameter concrete piles.

Bridge No 405/48W – I-405 over 115th Ave NE

2.8.2 Bridge 405/48W C/D Ratios for Method D2 Analysis

Original (1954 & 1965) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
Pier 1				
FOOTING				
Moment (k-ft)	**0.26**	1665	435	
Shear (k)	4.07	296	1207	
Overturning (k-ft)	4.00	2780	11120	
Sliding (k)	21.27	24	508	

COLUMN				
Long. Displacement (in)	6.77	3.3	22.5	Column 1 controls
Transv. Displacement (in)	4.21	3.9	16.2	Column 1 controls
Shear (k)	1.27	1457	838	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.13**	1500	194	Negative Moment
Shear (k)	**0.44**	234	104	
Pier 2				
PILE CAP FOOTING				
Pile Cap Moment (k-ft)	**-**	738	-	No top mat of reinforcement in the pile cap
Pile Cap Shear (k)	**0.38**	1378	527	
Pile Axial (k)	**0.45**	156	70	
Pile Shear (k)	**0.18**	246	44	
Pile Connection (k)	**0.05**	51	2	
COLUMN				
Long. Displacement (in)	5.24	2.37	12.41	Column 2 controls
Transv. Displacement (in)	1.18	2.61	3.07	Column 1 controls
Shear (k)	**0.35**	489	171	Vf vs.Vp – Brittle Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.09**	10339	906	Negative Moment
Shear (k)	**0.15**	2066	302	
Pier 3				
FOOTING				
Moment (k-ft)	**0.52**	2343	1223	Column 1 controls
Shear (k)	**0.33**	1115	363	Column 1 controls
Overturning (k-ft)	1.04	4054	4223	Column 1 controls
Sliding (k)	2.19	262	573	Column 1 controls
COLUMN				
Long. Displacement (in)	5.32	2.37	12.62	Column 2 controls
Transv. Displacement (in)	1.31	2.39	3.14	Column 1 controls
Shear (k)	**0.35**	482	171	Vf vs.Vp – Brittle Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.14**	6398	906	Negative Moment
Shear (k)	**0.15**	2022	302	
SUPERSTRUCTURE				
Moment (k-ft)	**0.64**	10388	9321	Negative Moment
Shear (k)	**0.76**	1645	1746	
Pier 4				
FOOTING				
Moment (k-ft)	**0.34**	1292	436	
Shear (k)	3.07	394	1207	
Overturning (k-ft)	3.09	2889	8917	
Sliding (k)	5.07	286	1449	
COLUMN				
Long. Displacement (in)	4.61	2.38	10.94	Column 3 controls
Transv. Displacement (in)	2.91	2.39	6.95	Column 1 controls
Shear (k)	**0.79**	112	88	Vf vs.Vp – Semi-ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.13**	1500	194	Negative Moment
Shear (k)	**0.44**	234	104	

SUPERSTRUCTURE				
Moment (k-ft)	**0.35**	20980	10371	Negative Moment
Shear (k)	**0.76**	1645	1746	

Widened (1993) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
Pier 1				
CROSSBEAM				
Moment (k-ft)	**0.79**	1498	1180	Negative Moment
Shear (k)	2.66	175	467	
Pier 2				
SHAFT				
Moment (unitless)	1.35	0.74	1	Unitless as this was computed as $P_u/P_r + M_u/M_r$
Shear (k)	5.13	431	2213	
Axial (k)	2.87	1044	3000	
COLUMN				
Long. Displacement (in)	5.72	2.38	13.58	
Transv. Displacement (in)	1.85	2.64	4.87	
Shear (k)	**0.49**	678	330	Vf vs.Vp –Brittle Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.09**	10461	906	Negative Moment
Shear (k)	**0.56**	1922	1073	
Pier 3				
SHAFT				
Moment (unitless)	1.03	0.97	1	Unitless as this was computed as $P_u/P_r + M_u/M_r$
Shear (k)	3.35	596	1996	
Axial (k)	2.51	1194	3000	
COLUMN				
Long. Displacement (in)	3.52	2.37	8.34	
Transv. Displacement (in)	1.25	2.41	3.01	
Shear (k)	**0.35**	935	330	Vf vs.Vp –Brittle Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.24**	12452	2950	Positive Moment
Shear (k)	**0.55**	1940	1073	
Pier 4				
CROSSBEAM				
Moment (k-ft)	**0.79**	1498	1180	Negative Moment
Shear (k)	2.66	175	467	
SUPERSTRUCTURE				
Moment (k-ft)	**0.35**	20980	10371	Negative Moment
Shear (k)	**0.76**	1645	1746	

2.8.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- Method D2 analysis shows that the original columns have marginally adequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of all columns is inadequate for resisting shear resulting from plastic hinging in the column
- The existing crossbeams are deficient in shear, and very deficient in flexure. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges. These crossbeams are integral, therefore, adding additional capacity can be challenging.
- The superstructure has a negative moment deficiency. This vulnerability is present at the ends of the bridge where it is integrally connected to the end piers and also appears near Piers 2 and 3 where the moment reinforcing has been dropped off as it is presumably no longer required for the strength and/or service load cases.
- The superstructure positive moment C/D ratios are slightly deficient at the supports where the structure was not detailed to resist the plastic hinging forces in the columns.
- The foundations supporting the 1954 and 1965 columns of Piers 2 and 3 lack the necessary capacity to resist the column plastic hinging forces. The 1965 Pier 2 foundation was supported on piles which are inadequate for the upper level seismic as is the Pier 3 spread footing foundation. The 1993 widening was supported on drilled shafts at the intermediate pier and does not have these same vulnerabilities. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.8.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around the intermediate pier columns. At Pier 3, proximity to the guardrail is a concern for the square columns and will be a challenge to work through in final design. At this juncture, steel column jackets are assumed to be both adequate and able to meet required clearances for roadway design. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the

column jacket so that the existing column can flex without causing unwanted strengthening.

The addition of concrete crossbeam strengthening at the intermediate piers, noted below, will likely increase the displacement demands of the structure by increasing the mass of the bridge that is excited by seismic response. Therefore, a marginal column displacement C/D ratio in the as-built condition will decrease even further with the crossbeam strengthening.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments next to columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment capacity can be increased to resist plastic hinging moments and shears of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. This will carry additional challenge as these crossbeams are integral with the cast-in-place superstructure.
3. We recommend strengthening the superstructure using fiber reinforced polymer (FRP) to provide additional positive moment capacity for the girders. FRP on the bottom of the girders (traditionally thought of as the positive moment region) will provide an alternative load path that will allow for adequate moment redistribution. The analysis shows negative moment deficiency as well, however, strengthening the superstructure for negative moment resistance is more difficult.

2.8.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1.	Steel Column Jackets on 1954 & 1965 Columns	$625,000
2.	Crossbeam strengthening	$1,400,000
3.	FRP on girders in Spans 1, 2, and 3	$540,000
4.	Miscellaneous (Downspout relocations, etc.)	$20,000

2.9 Bridge No. 405/56E

2.9.1 Bridge Description

Bridge 405/56E consists of three continuous cast-in-place concrete T-beam spans. Originally constructed in 1956, the center span is the longest measuring 76 feet while the other two spans are 59'-1" each to create a total length of 194'-2" from back-to-back of pavement seats. The original roadway width measured 28'-0". All piers are on a skew that is about 33 degrees measured normal to the bridge alignment line at the centerline of piers for intermediate piers and back of pavement seat for end piers.

All spans are continuous and T-beam girders are haunched varying in height from 3'-3" to 6'-0". The intermediate piers are integral and have a 2'-1" wide by 6'-6" tall crossbeam supporting the superstructure. The original crossbeams sit on two 3'-3" square columns that are founded on 10 feet square spread footings. At the end piers, the girders frame into an integral crossbeam. This crossbeam bears on four 1'-10" by 3'-0" concrete columns that are founded on a combined spread footing.

The bridge was widened to the east in 1965 by about 32 feet at Pier 1 and 46 feet at Pier 4 adding five girder lines. At this time, the west was also expanded adding one girder line and about 3 feet of roadway width. The intermediate piers were extended to the east and supported on two 3'-3" square columns which are founded on a combined spread footings. At the end piers, the girders frame into an end crossbeam that is supported on concrete piles. There were five piles added to Pier 1 and 4 piles added to Pier 4. Pier 2 and Pier 3 received a partial height crash wall between the original columns and connected to the new columns. These crash walls were added as this bridge crossed over railroad tracks between Piers 2 and 3. These tracks have since been abandoned.

The bridge was again widened to the east again in 1989 by about 20 feet at Pier 1 and 14.5 feet at Pier 4 adding two girder lines. The intermediate piers were extended and supported on one 3'-3" square columns. The Pier 2 column is founded on a 17'-0" by 17'-9" footing and the Pier 3 column is founded on 18 feet square footings. At the end piers, the girders frame into an end crossbeam that is supported on fifteen 16-inch diameter concrete piles. There were five piles added at Pier 1 and four piles added at Pier 4. Pier 2 and Pier 3 received a partial height wall between the new columns and extended to the original crash wall.

The bridge was again widened to the west again in 1994 by about 16 feet adding three girder lines. The intermediate piers were extended and supported on two 3'-3" square columns. The Pier 2 and Pier 3 columns were then founded on a combined spread footing. At the end piers, the girders frame into an end crossbeam that is supported on 16-inch diameter concrete piles. There were four piles added at Pier 1 and four piles added at Pier 4. Pier 2 and Pier 3 received a partial height wall between the new columns and extended to the original crash wall.

Bridge No 405/56E – I-405 over Pedestrian Trail

2.9.2 Bridge 405/56E C/D Ratios for Method D2 Analysis

Original (1956 & 1965) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
Pier 1				
FOOTING				
Moment (k-ft)	5.93	21	125	
Shear (k)	4.06	255	1034	
Overturning (k-ft)	1.85	931	1722	
Sliding (k)	5.64	58	326	
COLUMN				
Long. Displacement (in)	3.96	2.1	8.3	
Transv. Displacement (in)	3.26	2.4	7.8	
Shear (k)	1.17	87	102	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.11**	2287	259	
Shear (k)	**0.27**	250	67	
Pier 2				
FOOTING				

Moment (k-ft)	**0.13**	1566	205	Col.1, 1965 construction controls
Shear (k)	**0.61**	435	267	Col.1, 1965 construction controls
Overturning (k-ft)	**0.83**	2482	2064	e_c is out of footing
Sliding (k)	1.48	216	319	Col.1, 1965 construction controls
COLUMN				
Long. Displacement (in)	17.95	2.1	37.0	
Transv. Displacement (in)	1.61	2.4	3.9	
Shear (k)	1.93	82	158	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	1.18	-353	-415	
Shear (k)	14.87	156	2314	
Pier 3				
FOOTING				
Moment (k-ft)	**0.08**	2872	228	Col. 2,1956 construction controls
Shear (k)	**0.54**	495	267	Col. 1,1965 construction controls
Overturning (k-ft)	**0.64**	3951	2511	ec is out of footing
Sliding (k)	1.28	282	360	Col. 2,1956 construction controls
COLUMN				
Long. Displacement (in)	16.48	2.1	34.7	
Transv. Displacement (in)	1.43	2.3	3.3	
Shear (k)	2.65	86	229	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	1.18	351	415	
Shear (k)	16.12	14	224	
Pier 4				
FOOTING				
Moment (k-ft)	2.58	48	125	1965 construction controls
Shear (k)	3.55	446	1587	1965 construction controls
Overturning (k-ft)	1.54	2253	3476	1965 construction controls
Sliding (k)	8.09	79	640	1965 construction controls
COLUMN				
Long. Displacement (in)	4.06	2.1	8.5	
Transv. Displacement (in)	3.38	2.2	7.3	
Shear (k)	1.16	88	102	Vf vs.Vp - Semi-Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.11**	2287	259	
Shear (k)	**0.39**	173	67	
SUPERSTRUCTURE				
Moment (k-ft)	**0.39**	24038	9307	
Shear (k)	1.19	1505	1792	

Widened (1989 & 1994) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
Pier 1				

CROSSBEAM				
Moment (k-ft)	**0.15**	1566	240	
Shear (k)	**0.62**	253	156	
Pier 2				
FOOTING				
Moment (k-ft)	**0.38**	1817	698	1994 Construction controls
Shear (k)	2.05	372	764	1994 Construction controls
Overturning (k-ft)	1.32	3339	4412	1994 Construction controls
Sliding (k)	2.91	157	458	1994 Construction controls
COLUMN				
Long. Displacement (in)	13.29	2.2	29.2	
Transv. Displacement (in)	2.53	2.4	6.1	
Shear (k)	1.60	103	164	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	1.35	1695	2288	
Shear (k)	**0.96**	254	245	
Pier 3				
FOOTING				
Moment (k-ft)	**0.31**	2380	739	1994 Construction controls
Shear (k)	1.40	578	809	1994 Construction controls
Overturning (k-ft)	1.42	4001	5694	1994 Construction controls
Sliding (k)	2.96	200	593	1994 Construction controls
COLUMN				
Long. Displacement (in)	11.59	2.2	25.2	
Transv. Displacement (in)	2.44	2.3	5.7	
Shear (k)	2.45	120	295	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	3.00	369	1108	
Shear (k)	25.06	10	254	
Pier 4				
CROSSBEAM				
Moment (k-ft)	**0.30**	809	240	
Shear (k)	**0.52**	209	110	
SUPERSTRUCTURE				
Moment (k-ft)	**0.39**	24038	9307	
Shear (k)	1.19	1505	1792	

2.9.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- Method D2 analysis shows that the original columns on intermediate piers have adequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of columns is adequate for resisting shear resulting from plastic hinging in the column.

- End pier columns are embedded in fill embankment. Results shown do not account for the embankment fill, and using engineering judgment, these columns have a shear deficiency in the upper level event. However, the ability to retrofit would be very complicated and costly, and have a tremendous impact to the traffic carried by this bridge. If these columns were to fail, the risk of collapse is very low as they are encapsulated in embankment fill. No mention of this occurrence was made in the geotechnical report.
- The existing crossbeams are deficient in both shear and flexure. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges. These crossbeams are integral, therefore, adding additional capacity can be challenging.
- The superstructure has a negative moment deficiency. This vulnerability is present at the ends of the bridge where it is integrally connected to the end piers and also appears near Piers 2 and 3 where the moment reinforcing has been dropped off as it is presumably no longer required for the strength and/or service load cases. This deficiency can be very challenging to retrofit.
- The superstructure positive moment C/D ratios are much better, though they are still slightly deficient at the supports where the structure was not detailed to resist the plastic hinging forces in the columns.
- The foundations supporting the original columns of Piers 2 and 3 lack the necessary capacity to resist the column plastic hinging forces, and the more recent spread footing foundations show a slight vulnerability to flexure. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.9.4 Recommendations for Retrofit

Based upon our evaluation of the structure and the listed deficiencies we recommend the following retrofit measures:

1. Addition of steel column jackets around the 1956 and 1965 columns. This stretch of I-405 has been identified by the State as a primary disaster response route between Joint Base Lewis-McChord (Lakewood, WA) and Paine Field (Everett, WA). This lifeline route has been established to allow emergency supplies and responders be flown into the air fields and transported along the corridor where they are needed. Based on this identification, the bridges along this stretch of I-405 have been moved to the top of the seismic retrofit priority list. Therefore, it seems appropriate to err on the side of caution and take a slightly conservative approach and add column jackets. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening.

2. Crossbeam strengthening to resist demands that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments next to columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment and shear capacity can be increased to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. This will carry additional challenge as these crossbeams are integral with the cast-in-place superstructure.
3. We recommend strengthening the superstructure using fiber reinforced polymer (FRP) to provide additional positive moment capacity for the girders. FRP on the bottom (traditionally thought of as the positive moment region) of the girders will provide an alternative load path that will allow for adequate moment redistribution. The analysis shows negative moment deficiency as well, however, strengthening the superstructure for negative moment resistance is more difficult.
4. The partial height crash walls located at Pier 2 and Pier 3 should be removed to increase the effective height of these columns and obtain more predictable seismic performance.

2.9.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

1.	Steel Column Jackets on 1956 & 1965 Columns	$750,000
2.	Crossbeam strengthening	$1,700,000
3.	FRP on girders in Spans 1, 2, and 3	$230,000
4.	Crash Wall Removal	$80,000
5.	Miscellaneous (Downspout relocations, etc.)	$20,000

2.10 Bridge No. 405/56W

2.10.1 Bridge Description

Bridge 405/56W consists of three continuous prestressed concrete I-girder spans. Originally constructed in 1970, the spans lengths are 85'-10", 78-0", and 79'-3" from south to north to create a total length of 243'-1" from back-to-back of pavement seats. The original roadway width measured 62'-0". All piers are on a skew that is about 33 degrees measured normal to the bridge alignment line at the centerline of piers for intermediate piers and back of pavement seat for end piers.

All spans are simply supported and consisting of nine original prestressed concrete girders. The intermediate piers have a 3'-6" wide by 3'-6" tall dropped crossbeam supporting the superstructure above. The original crossbeams sit on four 3'-0" columns that are founded on 9'-6" square spread footings. The end piers are L-shaped and supported on spread footings.

The bridge was widened about 16 feet in 1994 to the east adding two W50G prestressed girders to the framing. At this time, the west barrier was replaced. The intermediate piers were extended and one 3'-0" diameter column was added and supported on a 13 feet by 18 feet spread footing. At the end piers, the girders were connected using a semi-integral connection and supported on four 14-inch diameter concrete piles.

Bridge No 405/56W – I-405 over Pedestrian Trail

2.10.2 Bridge 405/56W C/D Ratios for Method D2 Analysis

Original (1970) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
Pier 1				
SHEAR BLOCKS				
Girder Stop (k)	11.1	435	4807	
SEAT LENGTH				
End Pier 1 (in)	**0.59**	34	20	FHWA Seismic Retrofit Manual Eqn. 5-1b
Pier 2				
FOOTING				
Moment (k-ft)	**0.67**	2202	1485	Col. 4 Controls
Shear (k)	**0.53**	747	393	Col. 4 Controls

Overturning (k-ft)	**0.83**	2956	2452	Col. 2 Controls
Sliding (k)	1.73	274	475	Col. 2 Controls
COLUMN				
Long. Displacement (in)	2.17	4.94	10.73	Col. 4 Controls
Transv. Displacement (in)	1.17	1.73	2.02	Col. 4 Controls
Shear (k)	**0.75**	249	188	Vf vs.Vp - Col. 4 Semi-ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.49**	5274	2608	Positive Moment
Shear (k)	**0.62**	584	364	
Pier 3				
FOOTING				
Moment (k-ft)	**0.62**	2397	1485	Col. 4 Controls
Shear (k)	**0.54**	728	393	Col. 4 Controls
Overturning (k-ft)	**0.57**	2937	1687	Col. 1 Controls
Sliding (k)	**0.81**	248	201	Col. 1 Controls
COLUMN				
Long. Displacement (in)	2.10	4.76	9.99	Col. 4 Controls
Transv. Displacement (in)	1.41	1.26	1.78	Col. 4 Controls
Shear (k)	**0.69**	269	186	Vf vs.Vp - Col. 4 Semi-ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.49**	5340	2608	Positive Moment
Shear (k)	**0.63**	575	364	
Pier 4				
SHEAR BLOCKS				
Girder Stop (k)	13.4	359	4807	
SEAT LENGTH				
End Pier 4 (in)	**0.59**	34	20	FHWA Seismic Retrofit Manual Eqn. 5-1b

Widened (1994) Structure Members - SEE

ITEM	C/D RATIO	DEMAND	CAPACITY	NOTES:
Pier 1				
SHEAR BLOCKS				
Girder Stop (k)	11.1	435	4807	
SEAT LENGTH				
End Pier 1 (in)	**0.60**	33	20	FHWA Seismic Retrofit Manual Eqn. 5-1b
Pier 2				
FOOTING				
Moment (k-ft)	**0.74**	6611	4889	
Shear (k)	1.09	877	959	
Overturning (k-ft)	**0.70**	9061	6308	
Sliding (k)	1.91	465	888	
COLUMN				
Long. Displacement (in)	5.58	5.52	30.77	
Transv. Displacement (in)	4.38	1.74	7.63	
Shear (k)	1.55	421	653	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				

Moment (k-ft)	**0.43**	3151	1344	Positive Moment
Shear (k)	1.33	460	610	
Pier 3				
FOOTING				
Moment (k-ft)	**0.73**	6670	4889	
Shear (k)	1.12	855	959	
Overturning (k-ft)	**0.61**	9128	5568	
Sliding (k)	**0.98**	493	482	
COLUMN				
Long. Displacement (in)	5.29	5.34	28.25	Col. 2 in Pier 2 Controls
Transv. Displacement (in)	5.28	1.27	6.72	Col. 4 in Pier 3 Controls
Shear (k)	1.43	457	655	Vf vs.Vp - Ductile Shear-Controlled
CROSSBEAM				
Moment (k-ft)	**0.45**	2968	1344	Positive Moment
Shear (k)	1.36	450	610	
Pier 4				
SHEAR BLOCKS				
Girder Stop (k)	13.4	359	4807	
End Pier 4 (in)	**0.60**	33	20	FHWA Seismic Retrofit Manual Eqn. 5-1b

2.10.3 Conclusions

Detailed seismic analysis performed using Method D2 of the FHWA SRM combined with our knowledge and understanding of bridge seismic response gives way to the following conclusions:

- The as-built plans show a provided seat length at the end piers that is less than that required, based on the AASHTO Guide Spec. However the displacements determined from the RSA analysis indicate that anticipated displacements at the end piers are well within the limits of the available seat widths. Thus, we do not feel that seat extensions at the end piers are warranted.
- The intermediate pier hinge diaphragms are connected to the crossbeams with 79 #11 and 20 #9 bars and the girders are embedded 2 inches into the pier hinge diaphragms with extended strands. This provides adequate connectivity to achieve good seismic performance.
- Method D2 analysis shows that the original columns have adequate displacement capacity (ductility) to accommodate the expected lateral deflections during the design earthquake. The results also found the shear capacity of the columns is inadequate for resisting shear resulting from plastic hinging in the column.
- The existing crossbeam is deficient in both shear and flexure. This would indicate that the existing crossbeams are not capable of forcing plastic hinging in the columns. In a capacity protection approach, this behavior is considered unacceptable for bridges.
- The foundations supporting the original columns of Piers 2 and 3 lack the necessary capacity to resist the column plastic hinging forces, and the more

recent spread footing foundations show a vulnerability to flexure, overturning and sliding. We are not scoped to discuss retrofit solutions for items below the bottom of the column, but it would be prudent to examine this location in the event a future phase of seismic retrofit is undertaken to address foundation deficiencies.

2.10.4 Recommendations for Retrofit

1. Addition of steel column jackets around the original 1970 columns. The steel column jackets are fabricated to encase the column, with a split seam that is welded in the field, and then the annular space between column and jacket is pumped full of grout. WSDOT has typical details for this construction that have been used extensively. Care must be taken to leave a space at the top and bottom of the column jacket so that the existing column can flex without causing unwanted strengthening.
2. Crossbeam strengthening to resist moments that occur when the pier is pushed in the transverse direction. This is a common deficiency in older bridge crossbeam details, where seismic displacements create positive moments next to columns and the amount of continuous reinforcement in the bottom of the crossbeam is minimal. Strengthening is achieved by either the application of post-tensioning along the length of the crossbeam, enlarging the dimensions of the crossbeam and adding additional mild reinforcement, or a combination of mild reinforcement and post tensioning. Based upon our experience with similar bridges, we anticipate that the crossbeam's moment capacity can be increased to resist plastic hinging moments of the columns by enlarging the section and adding mild reinforcement. However, post-tensioning may be required following detailed analysis during the PS&E phase of this project. Installation of the crossbeam bolsters also provides additional seat width.
3. Though not a recommended retrofit, we observed significant erosion at the north end-pier of this bridge. We recommend WSDOT maintenance remedy the source of the erosion, restore the grade and install new slope paving at the north abutment.
4. The partial height crash walls located at Pier 2 and Pier 3 should be removed to increase the effective height of these columns and obtain more predictable seismic performance.

2.10.5 Cost Estimate of Conceptual Retrofit Measures

The anticipated structural construction costs (including 20% Mobilization and 25% Contingency) for the aforementioned retrofit items are as follows:

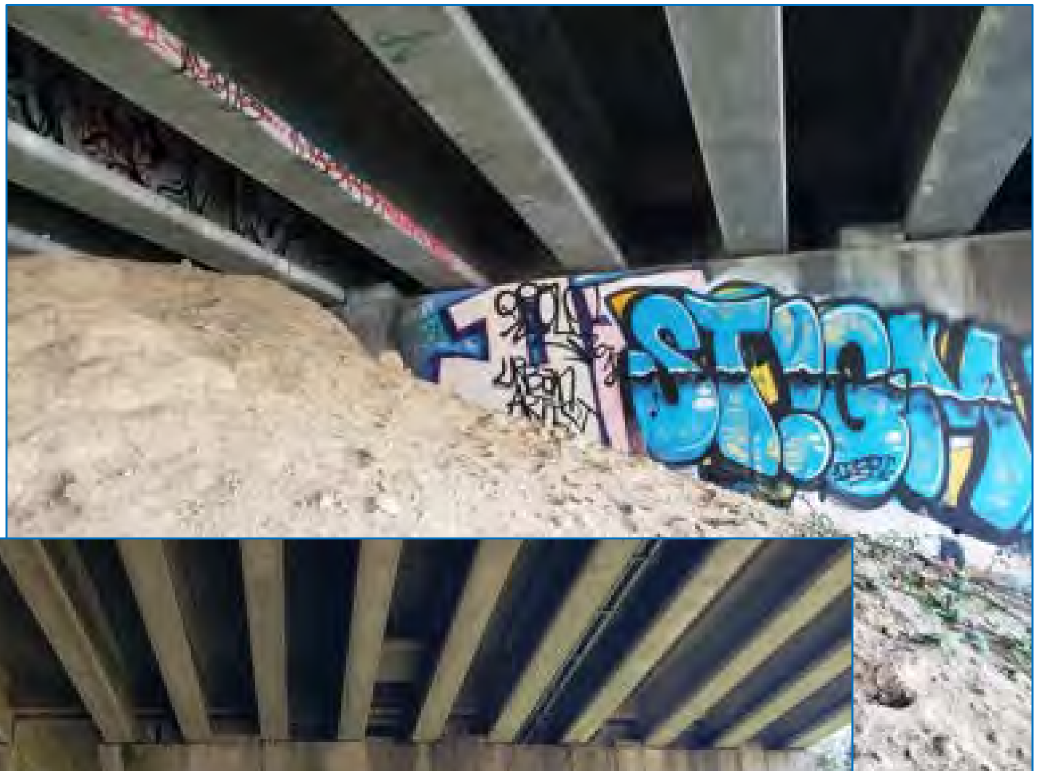
1.	Steel Column Jackets on Original Columns	$320,000
2.	Reinforced Concrete Bolsters	$475,000
3.	Crash Wall Removal	$70,000
4.	Miscellaneous (Downspout relocations, etc.)	$20,000

3 Additional Photos

Bridge 405/12

ABOVE: Erosion noted at end pier

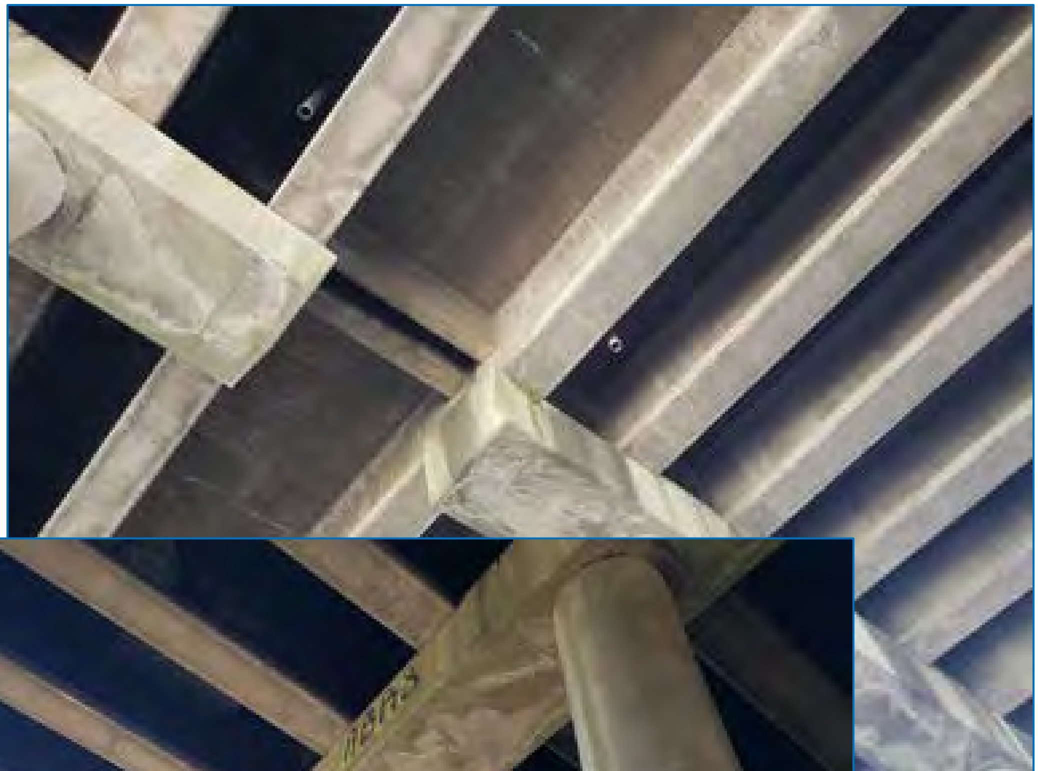
LEFT: Typical expansion joints over intermediate piers

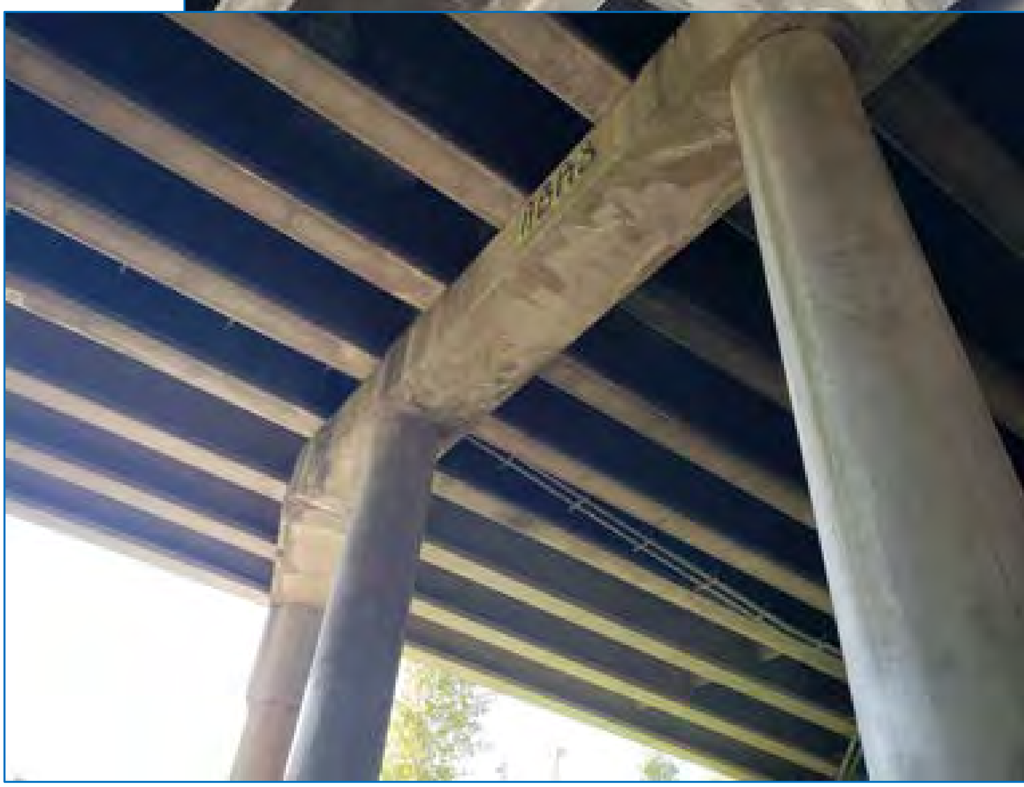


Bridge 405/12

UPPER: Some shear keys present at end piers

LOWER: Some shear keys present at intermediate piers





Bridge 405/12

UPPER: Original crossbeams not connected

LOWER: Crossbeam angled at Pier 2

Bridge 405/12

UPPER: Pier 2 Crashwall and presence of utilities

LOWER: Pier 6 straddle pier and close proximity to roadway

Bridge 405/45W

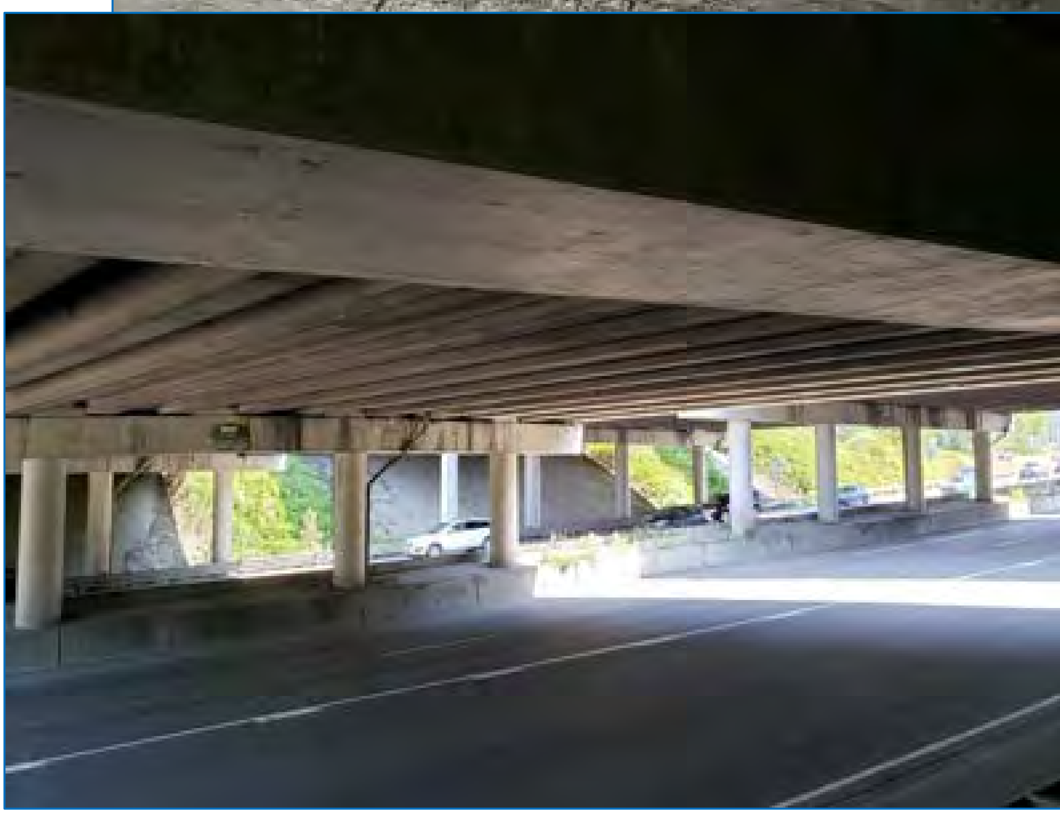
ABOVE: Columns embedded in slope paving

LEFT: Utilities on columns and capbeam

Bridge 405/45W

ABOVE: No shear keys at end piers

LEFT: Utility removed



Bridges 405/46E/W

UPPER: Some shear keys at end piers

LOWER: Utilities present and proximity of intermediate pier to traffic lanes

Bridges 405/46E/W

UPPER: Columns embedded in slope paving

LEFT: Proximity to existing guardrail

Bridges 405/47E

UPPER: No girders stops at end piers

LOWER: Typical hinge detail at intermediate bents

Bridges 405/47W

UPPER: Varying crossbeam depth in intermediate piers (due to widening)

LEFT: Longitudinal expansion joint at widening

Bridges 405/48 E/W

UPPER: Proximity of columns to existing guardrail

LOWER: Pedestrian trail under center bridge span

Bridges 405/48E

UPPER: Integral retaining wall

LOWER: Crash wall at Pier 2

Bridges 405/56E

UPPER: Columns embedded in soil at end piers

LOWER: Crash walls at intermediate piers and trail beneath center span

Bridges 405/56W

UPPER: Crash walls at intermediate piers

LOWER: Erosion noted at end pier

4 Geotechnical Reports

The Geotechnical Reports were completed by others and are attached for reference only.