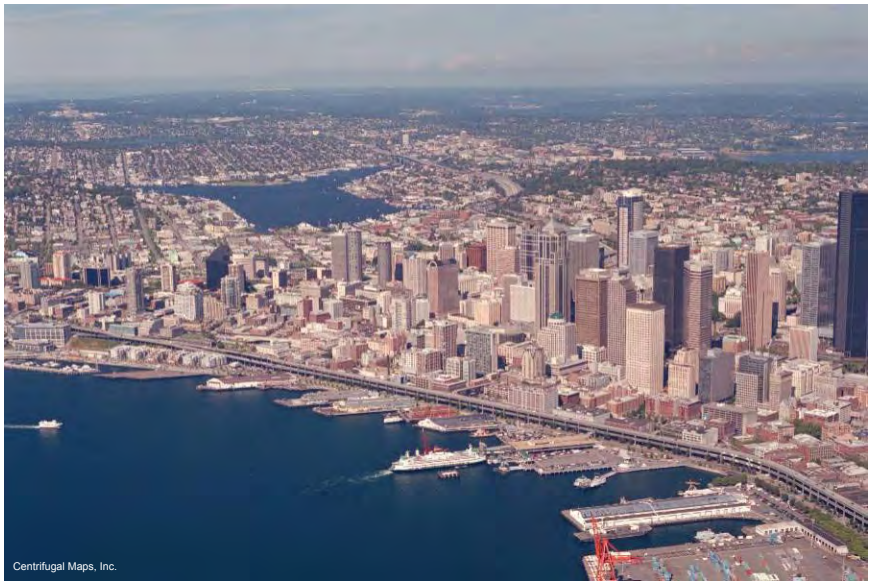
SR 99: ALASKAN WAY VIADUCT & SEAWALL REPLACEMENT PROJECT

REBUILD/RETROFIT 500

500-YEAR DESIGN EARTHQUAKE



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The SR 99: Alaskan Way Viaduct Project is a joint effort between the Washington State Department of Transportation (WSDOT), the City of Seattle, and the Federal Highway Administration (FHWA). To conduct this project, WSDOT contracted with:

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REBUILD/RETROFIT 500

500-YEAR DESIGN EARTHQUAKE

EXECUTIVE SUMMARY

Background

The Alaskan Way Viaduct is the major traffic corridor along the Seattle Waterfront. The facility carries one-quarter of Seattle's north-south traffic through downtown. The Viaduct structure is two miles long from S. Holgate Street to the south portal of the Battery Street Tunnel. The Alaskan Way Seawall is 1.5 miles long and extends from S. Washington Street to Myrtle Edwards Park along Elliott Bay. Deficiencies related to the Alaskan Way Viaduct include seismic vulnerability, deteriorated condition, safety, limited load carrying ability, and remaining useful life.

To address these issues, the **Rebuild 2500 Plan** has been developed. As shown in Figure 1, the **Rebuild 2500 Plan** envisions "rebuilding" the existing Viaduct (mostly double-level) from Holgate Street to Pike Street and "retrofitting" the remaining portion (mostly single-level) from Pike Street to the Battery Street Tunnel.[1] The **Rebuild 2500 Plan** is based on a 2500-year earthquake design standard. This was a proposed American Association of State Highway Transportation Officials (AASHTO) Guide Specification being considered for adoption by the Washington State Department of Transportation (WSDOT). With the **Rebuild 2500 Plan**, the Viaduct would have a life expectancy in excess of 75 years and would be expected not to collapse or cause loss of life following a rare earthquake.

Purpose

The purpose of this memorandum is to assess the potential reduction in project costs and construction impacts resulting from changing the project seismic design criteria to reflect a current code earthquake (500-year return period) instead of the more stringent anticipated code earthquake (2500-year return period). With the 500-year design standard applied, the Viaduct would have a life expectancy approaching 75 years and would be expected not to collapse or cause loss of life following an earthquake that has a return period of 500 years.

[1] For this memorandum, "Rebuild" means replacing most of the existing structure in approximately the same location. "Retrofit" means strengthening the existing structural members and adding new seismic resisting elements.

Plan Descriptions

For this comparison, two alternatives to the **Rebuild 2500 Plan** were considered. One, called the **Rebuild 500 Plan**, envisioned that the section from Holgate Street to Pike Street (mostly double-level) would be rebuilt and the remaining portion from Pike Street to the Battery Street Tunnel (mostly single-level) would be retrofitted using the current 500-year seismic design standard. The other, called the **Retrofit 500 Plan** envisioned that the entire Viaduct from Holgate Street to the Battery Street Tunnel would be retrofitted, again, using the current 500-year design standard.[2] See Figure 1 for a schematic showing the extent and configuration of each plan.

Findings

For this comparison, two locations on the existing Viaduct and Seawall were chosen as being representative of the typical configurations found on the structures. The only variable was the seismic design earthquake standard with either a 2500-year return period (**Rebuild 2500 Plan**) or a 500-year return period (**Rebuild 500 Plan** and **Retrofit 500 Plan**). The results of this analysis are summarized in Table 1, Figure 2, and following:

- The **Rebuild 500 Plan** performance is far superior in a seismic event than the **Retrofit 500 Plan**.
- The **Retrofit 500 Plan** would receive significant damage, require hundreds of millions of dollars and a long time to repair in an event slightly larger than the Nisqually Earthquake.
- The **Rebuild 500 Plan** has the potential to reduce the total project cost by 5 to 15% of the **Rebuild 2500 Plan**.
- The **Rebuild 500 Plan** has the potential to reduce the overall construction duration by one year compared to the **Rebuild 2500 Plan** in a fully funded scenario.
- A rebuild plan allows for a limited funding scenario, but total project cost and duration increase significantly.
- There is little difference in cost or construction impacts between a rebuilt Seawall designed to either the 500-year or the 2500-year seismic design standard.
- Utility impact differences between any rebuild or retrofit plan are minor.

[2] All three plans envision that the section from Pike Street to the Battery Street Tunnel would be "retrofitted". Previous structural analysis has shown retrofitting of this section to be the most cost-effective and practical solution.

Conclusion

One conclusion that can be reached is that the Rebuild 500 Plan is far superior to the Retrofit 500 Plan when seismic performance, aesthetics, cost, and risk are balanced. As a result, the facility as a whole does not lend itself to a retrofit approach. However, the exception is a retrofit of the single-level structures between the Battery Street Tunnel and Pike Street, which do perform well. It is also clear due to the variable site conditions along the corridor, in combination with the significance of the facility, that site specific seismic design criteria, while likely to be less than a 2500-year design criteria, should be used for all Viaduct replacement plans during preliminary engineering.

Table 1: Viaduct Comparison Summary, Rebuild 2500 vs. Retrofit 500 vs. Rebuild 500

DESCRIPTION		DOUBLE-LEVEL VIADUCT			SINGLE-LEVEL VIADUCT		EXISTING VIADUCT
		Rebuild 2500 Nov 2002 Snapshot 2500-Year Earthquake	**Rebuild 500 500-Year Earthquake**	**Retrofit 500 500-Year Earthquake**	**Rebuild 2500 Nov 2002 Snapshot 2500-Year Earthquake**	**Rebuild 500 & Retrofit 500 * 500-Year Earthquake**	
PERFORMANCE	Overall Earthquake Resistance	Excellent	Very Good	Adequate	Excellent	Very Good	Very Poor
	2500-Year Earthquake	Good	Poor	Very Poor	Good	Poor	Very Poor
	500-Year Earthquake	Very Good	Good	Adequate	Very Good	Good	Very Poor
	Moderately Heavy Earthquakes (Stronger Than Nisqually)	Excellent	Very Good	Poor	Excellent	Very Good	Very Poor
	Facility Life Expectancy w/o Earthquake	75 Plus Years	75 Plus Years	75 Years Maximum	75 Plus Years	75 Plus Years	20 Years Maximum Expected
CONSTRUCTION SEQUENCE Duration - Full Funded		10 Year	9 Year	8 Year	10 Year	8 Year	N/A

* - Rebuild 500 is identical to Retrofit 500 for single-level Viaduct

BACKGROUND

The Alaskan Way Viaduct is the major traffic corridor along the Seattle waterfront. It carries approximately one-quarter of Seattle's north–south traffic in downtown. The Alaskan Way Viaduct structure is a six- to eight-lane facility, approximately 11,000 feet long, from S. Holgate Street to the south portal of the Battery Street Tunnel. The Alaskan Way Seawall is approximately 8000 feet long, and extends from S. Washington Street to Myrtle Edwards Park along Elliott Bay.

The Viaduct was constructed in the 1950s and conformed to the design standards of that time. The city of Seattle (City) designed the Viaduct north of Railroad Way. The Washington State Department of Transportation (WSDOT) designed the remainder of the Viaduct to the south a few years later. The structure was designed to seismic criteria that are less than one-third as stringent as the current standards. Although the Nisqually Earthquake on February 28, 2001, with an estimated return period of approximately 170 years, was less than the present design earthquake, localized portions of the Viaduct suffered substantial damage. This damage caused the Viaduct to be closed numerous times for emergency repairs.

The Federal Highway Administration (FHWA), WSDOT, and the City are proposing major improvements to the portion of the SR 99 corridor currently served by the Alaskan Way Viaduct, the Battery Street Tunnel and to the Alaskan Way Seawall. The Battery Street Tunnel is not addressed in this document. A seismic vulnerability analysis has not been done on the Battery Street Tunnel and is recommended. The findings of this future investigation will provide guidance on the seismic retrofit required for the tunnel.

The purpose of the proposed action is to provide a transportation facility and Seawall with improved earthquake resistance that maintains or improves mobility and accessibility for people and goods along the existing Alaskan Way Viaduct corridor.

The project is needed for public safety due to the seismic vulnerability of the Viaduct and Seawall structures as well as traffic safety concerns due to roadway deficiencies. For additional information on the project's seismic vulnerability and traffic safety issues, see "No Action Alternative (Existing Condition)" report by Parsons Brinckerhoff, August 2002.

Screening Criteria were developed to consider numerous design concepts and identify alternatives that best address the purpose and needs of the project. The screening process has led to the Rebuild 2500 Plan for the existing Alaskan Way Viaduct and the Alaskan Way Seawall as one of the alternatives to be considered in the Environmental evaluation process.

INTRODUCTION

Purpose

The In-progress Design Snapshot of the Rebuild Plan, designated herein as the Rebuild 2500 Plan, was submitted November 6, 2002. The Rebuild 2500 Plan showed the structural configuration of a rebuilt and retrofitted Viaduct designed to withstand the earthquake loading of a seismic event with a return period of 2500 years, see the Viaduct Structural Analysis, Design Criteria Section, below. A "rebuilt" Viaduct or Seawall would replace most of the existing structure with a new structure in approximately the same location. A "retrofitted" Viaduct would strengthen the existing structural members and add new seismic resisting elements. The Viaduct may either be rebuilt or retrofitted. The Seawall would only be rebuilt. There is no option under consideration at this time to retrofit the existing Seawall.

In an effort to reduce project costs and construction impacts, portions of the Viaduct and Seawall previously submitted in the Rebuild 2500 Plan have been re-evaluated based upon a reduced earthquake loading resulting from an AASHTO specified seismic event with a return period of 500 years. It is emphasized here that this is the current design standard used by WSDOT, the City, and surrounding counties. This will help to determine the sensitivity of the cost to the design seismic hazard level.

Methodology

Four elements of the project that may be impacted by different earthquake design criteria have been selected for evaluation. These will be investigated to determine the potential reduction in cost, schedule, and constructibility issues by using the current, 500-year earthquake return period instead of the project design criteria of a 2500-year earthquake return period seismic design standard.

The areas under consideration are as follows:

Typical Single-Level, Multi-span,
Viaduct, Bents #25 - #35

A retrofitted configuration will be investigated for comparison with the retrofitted configuration of the Rebuild 2500 Plan single-level Viaduct. This is described beginning on page 9.

Typical Double-Level, Three-Span Viaduct, Bents #103 - #106

Two configurations will be developed for comparison with the Rebuild 2500 Plan double-level Viaduct, a rebuilt section (refer to page 14) and as a retrofitted section (refer to page 17).

Soil Stabilization South of S. King Street

The impact on the extent of soil improvement needed to mitigate lateral spreading will be investigated. The discussion begins on page 21.

Typical Type "A" and Type "B" Seawall

A rebuilt configuration will be investigated for comparison with the Rebuild 2500 Plan Seawall. Refer to the seawall section beginning on page 25.

General extrapolations to the overall Rebuild 2500 Plan will be made based upon the results of the analysis. Potential impacts for a Rebuild 500 Plan and a Retrofit 500 Plan will be considered. The Rebuild 500 Plan would have a rebuilt double-level Viaduct, a retrofit single-level Viaduct, and a rebuilt Seawall. The Retrofit 500 Plan would have a retrofit single- and double-level Viaduct with a rebuilt Seawall. See Figure 1 for the Rebuild 2500 Plan, Rebuild 500 Plan and Retrofit 500 Plan Location Map. The previously developed Rebuild 2500 Plan, In-progress Design Snapshot, dated November 6, 2002, will be used as the basis for comparison.

VIADUCT STRUCTURAL ANALYSIS

Design Criteria

Rebuild 2500 Plan - 2500-Year Return Period Earthquake

The Rebuild 2500 Plan, in contrast with the Rebuild/Retrofit 500 design, uses the project seismic design criteria specified in the "Structural Design Criteria" document (Parsons Brinckerhoff, February 2002). All of the bridge structures are designed in accordance with the "*Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*", Multidisciplinary Center for Earthquake Engineering Research (MCEER), November 2001. It has as its primary objective the development of seismic design provisions that reflect the latest design philosophies and design approaches that would result in highway bridges with a high level of seismic performance. This is a proposed Guide Specification that has not been adopted by either AASHTO or WSDOT to date. The engineering community is currently engaged in discussions regarding its replacement of the present AASHTO seismic design criteria.

The Life Safety Performance Level of this Guide Specification was chosen. Performance Level in seismic design provides a means of communicating risk and potential damage along with the relevant engineering information necessary to develop designs consistent with the stated performance goals. A "no collapse" or "life safety" seismic performance level is intended to protect human life during and following a rare earthquake.

The Guide Specification requires a dual level earthquake design. The "Expected Earthquake" has a return period of approximately 110 years. No significant damage is expected after this event. The rare earthquake or "Maximum Credible Earthquake" has a return period of approximately 2500 years. No collapse is expected after this event, although parts or all of the facility would most likely be unusable for an extended period.

Rebuild/Retrofit 500 - 500-Year Return Period Earthquake

The Rebuild/Retrofit 500 is designed using the present WSDOT earthquake design criteria that requires compliance with Division 1-A, Seismic Design of the AASHTO Standard Specifications for Highway Bridges, 1996, 16th Edition. This uses the USGS Peak Ground Acceleration map to obtain an acceleration coefficient with a 10% probability of exceedance in 50 years or a 500-year return period. The actual calculated return period is 475 years, but this is rounded to 500 for ease of reference. The WSDOT criteria is predicated on the following basic concepts:

- Hazard to life to be minimized
- Bridges may suffer damage but have low probability of collapse due to earthquake motions
- Ground motions used in the design should have low probability of being exceeded during normal lifetime of bridge

The following design parameters were used in the evaluation of the selected area for the single-level Viaduct, Bents #25 through #35:

- Importance Classification (IC): "Other bridges", IC = II
- Seismic Performance Category (SPC): C
- Soil Profile Type: II (stiff clay or deep cohesionless conditions where the soil depth exceeds 200′ and the soil types overlying rock are stable deposits of sands, gravels, or still clays)
- Peak Ground Acceleration (PGA) = 0.33g
- Site Coefficient (S): 1.2
- Peak Spectral Acceleration: 0.83g
- Response Modification Factor (R): R = 5
- See Figure 3 for Soil Profile Type II: AASHTO Response Spectrum Curve

The following design parameters were used in the evaluation of the selected area for the double-level Viaduct, Bents #103 through #106:

- Importance Classification (IC): "Other bridges", IC = II
- Seismic Performance Category (SPC): C
- Soil Profile Type: IV (soft clays or silts greater than 40′ in depth)
- Peak Ground Acceleration (PGA) = 0.33g
- Site Coefficient (S): 2.0
- Peak Spectral Acceleration: 0.66g
- Response Modification Factor (R): R = 5
- See Figure 4 for Soil Profile Type IV: AASHTO Response Spectrum Curve

Single-Level Viaduct Retrofit 500

Description of Existing Structure

The City of Seattle-designed Subsegment 4 was chosen for analysis (see Figure 5, Existing Viaduct Subsegment Key Map). It is located in the area between Lenora Street and Virginia Street, from Bents #25 through #35 (see Figures 6 and 7, Rebuild/Retrofit 500 SED Subsegment 4 Plan and Rebuild/Retrofit 500 SED Subsegment 4 Sections). This is a single-level structure, and is widest at Bents #25, 90.2′ curb-to-curb, and #34, 83.8′ curb-to-curb. The minimum width is at Bent #30, which is 78.0′ curb-to-curb. All existing bents in this subsegment have four columns, except Bents #26 and #27 that have three columns each. Span

lengths are typically 39.8′, except for the 46.0′ span between Bents #26 to #27, and the 38.3′ span between Bents #33 to #34.

Top of roadway elevations, based on the NAVD 88 datum, vary from approximately 80′ to 95′. The structure is on a side-slope, so column heights vary across the bents. The column heights on the east side vary from approximately 10.5′ to 21′ from the top of footing to bottom of cap beam. The column heights on the west side of the structure vary from approximately 35.5′ to 50′. All columns on Bents #25 to #27 are fixed at the bottom. The two east columns on Bents #28 to #30 and Bents #32 to #34 are pinned while the two west columns of these bents are fixed. Only the west column of Bent #31 is fixed. Foundations from Bents #25 to #29 are spread footings and the foundations from Bents #30 to #35 are pile supported.

Non-Linear Ultimate Displacement (Pushover) Analysis

The Non-Linear Ultimate Displacement (pushover) analysis was performed to identify the collapse mechanism of this structure. An initial part of the analysis is to carry out independent collapse analysis of individual bent elements. The bent frame is modeled with GT STRUDL structural software and displacement is incremented, tracking the formation of plastic hinges, shear degradation, joint degradation, and plastic rotation. Serviceability and ultimate (collapse) limit states are related to inelastic rotations of plastic hinges, onset of member or joint shear failure, or other degradation mechanisms.

Bent #30 was selected for a transverse Non-Linear Ultimate Displacement (pushover) analysis because it is located towards the middle of Subsegment 4 and represents a "typical" frame in this location. A concrete strength of 5000 psi was assumed with a maximum compressive strain of 0.004. For the moment curvature analysis, all concrete was assumed to be unconfined.

For Bent #30, a four-column bent, the following was found (also refer to Figures 8 and 9):

- Stage 1 – At a lateral seismic load of 120 kips, a displacement of 1.5" occurs and a plastic hinge forms at the top of the first interior column from the east.
- Stage 2 – At a lateral seismic load of 140 kips, a displacement of 2.0" occurs and a plastic hinge forms at the top of the east exterior column.
- Stage 3 - At a lateral seismic load of 150 kips, a displacement of 2.8" occurs and a plastic hinge forms at the top and bottom of the first interior column from the west.
- Stage 4 - At a lateral seismic load of 160 kips, a displacement of 3.4" occurs and the plastic rotational capacity of the plastic hinge at the top of

the first interior column from the east reaches its maximum limit and it fails in flexure.

Retrofit 500 Design Approach

The Rebuild 2500 Plan would retrofit Subsegment 4 as follows:

- Steel jacket the existing columns to provide ductility
- Strengthen the existing exterior longitudinal beams to HS-25 live load
- Strengthen the existing cap beams at selected locations to HS-25 live load
- Replace the roadway deck and traffic barriers
- Add seven new three column concrete transverse bents tied together with three continuous longitudinal beams to attract lateral load away from the existing structure.

The combination of strengthening the existing structure and the addition of new lateral resisting elements would be adequate to meet the project design criteria previously referenced. The Retrofit 500 of Subsegment 4 would use similar retrofit techniques to the Rebuild 2500 Plan, but there would be fewer and smaller new members.

The combination of strengthening the existing structure and the addition of new lateral resisting elements would be adequate to meet the current WSDOT earthquake design standards. The retrofit of the existing columns in this area would be the same in both the Rebuild 2500 Plan and the Retrofit 500 Plan since the design intent is to limit the structure to the same average lateral spectral displacement of between 3.0″ and 3.5″.

3-D Dynamic Analysis

A 3-D frame of Subsegment 4 was modeled under the retrofitted condition in GT STRUDL. To evaluate the retrofit concept, the flexural capacity of the new columns and shafts were checked assuming 2% reinforcement, and the displacement capacity was checked for the existing columns, assuming they are retrofitted with steel jackets. The retrofit Viaduct structure was modeled using a linear elastic model and elastic forces were converted to design forces using an R-factor of 5 (multi-column bent) as specified in the AASHTO design code.

To get an estimate of the relative frame stiffness between the existing and new frames, the substructure stiffnesses were included. The existing substructure has both spread footings and pile foundations. The stiffnesses of the existing spread footings were modeled based upon by standard formulas used by WSDOT and AASHTO. The stiffnesses of the existing pile supported footings were modeled as the sum of the stiffness from the pile cap plus the stiffness from the piles using the L-PILE software analysis program. The spread footings included translation

springs in both horizontal directions. The pile foundations included only translation springs in both horizontal directions, assuming the pile group fixes the foundations from rotating. For the new shafts, the equivalent depth to fixity was chosen to match displacements based on L-PILE results.

The displacement capacity of the existing columns were calculated assuming they would be retrofitted with 0.5" steel jackets. Because of the variation in column size and reinforcement between the existing columns, the displacement capacity varies for each column. The displacement capacity was checked for four representative columns in each direction: short column – three column bent; long column – three column bent; short column – four column bent; and long column – four column bent. The displacement capacities were controlled by the short columns with the smallest being about 5.5 ".

Analysis Results

Table 2 compares the Rebuild 2500 Plan retrofit design for the 2500-year return period earthquake, the Rebuild 2500 Plan retrofit design for the 500-year return period earthquake, and the Retrofit 500 design for the 500-year return period earthquake. It can be seen by comparison of the different earthquakes on the same structure that average seismic base shear of the 2500-year return period event is about twice the average seismic base shear of the 500-year return period event.

The average distribution of lateral load has the existing structure taking approximately 30% of the base shear while the new structural bent frames attract approximately 70%. The Retrofit 500 limits the average design lateral spectral displacement to just over 3". The maximum calculated design displacement of 3.5" is less than the maximum allowable of 5.5 ".

Table 2: Retrofit 500 Design vs. Retrofit 2500 Design Single-Level Structure

	2500-Year EQ (7 New Bents)	**500-Year EQ**	
		(7 New Bents)*	**(5 New Bents)**
Seismic Reduction Factor	4 (per MCEER)	5 (per AASHTO)	5 (per AASHTO)
Peak Spectral Acceleration	1.61 g (Soil Class D)	0.83 g (Soil Profile II)	0.83 g (Soil Profile II)
Structural Period	0.39 seconds	0.39 seconds	0.76 seconds
Spectral Acceleration Value	1.61 g	0.83 g	0.57 g
Average Seismic Base Shear (based on multimodal analysis results)	1.52 g	0.78 g	0.53 g

Table 2: Retrofit 500 Design vs. Retrofit 2500 Design Single-Level Structure (continued)

	2500-Year EQ (7 New Bents)	500-Year EQ	
		(7 New Bents)*	(5 New Bents)
Average Lateral Spectral Displacement	3.5″	1.2″	3.1″
Average Distribution of Lateral Load	89% to new structure, 11% to existing	92% to new structure, 8% to existing	71% to new structure, 29% to existing
NEW COLUMN SIZES	6′ diameter	6′ diameter	3.5′ diameter
NEW DRILLED SHAFT SIZES	8′ diameter	8′ diameter	5′ diameter

* Shown for direct comparison purposes

Summary

The Retrofit 500 design would use smaller columns, smaller drilled shafts, smaller bent cap beams, 30% fewer new bents, and the same strengthening of the existing structure. The Rebuild 2500 Plan drilled shaft construction would require that the existing deck slab and adjacent longitudinal members be demolished and temporarily shored. The smaller Retrofit 500 drilled shafts would fit between the existing longitudinal beams so that only the existing deck slab would need to be removed during their installation.

The Retrofit 500 Plan for Subsegment 4 would be retrofit as follows (see Figures 6 and 7):

- Steel jacket the existing columns – Similar to Rebuild 2500 Plan
- Strengthen the existing exterior longitudinal beams to HS-25 live load – Similar to Rebuild 2500 Plan
- Strengthen the existing cap beams at selected locations to HS-25 live load– Similar to Rebuild 2500 Plan
- Replace the roadway deck and traffic barriers – Similar to Rebuild 2500 Plan
- Five new three column bents – Rebuild 2500 Plan has seven
- 3.5′ diameter columns – Rebuild 2500 Plan uses 6′ diameter columns
- 5′ diameter drilled shafts– Rebuild 2500 Plan uses 8′ diameter drilled shafts
- 5′W X 6′D bent cap beam – Rebuild 2500 Plan uses 8′W X 8′D bent cap beam
- Three continuous 4′W X 7′D longitudinal cap beams – Similar to Rebuild 2500 Plan

Double-Level Viaduct Rebuild 500

Description of Existing Structure

The three-span unit within the City of Seattle-designed Subsegment 11 was chosen for analysis (see Figure 5, Existing Viaduct Subsegment Key Map). It is located in the vicinity of S. Main Street adjacent to the Pioneer Square Historic District. The existing three-span frame, consisting of Bents #103 through #106, was chosen as representative for the existing double-level Viaduct structure. The structure is composed of a double-level reinforced concrete frame carrying northbound traffic on the upper level and southbound traffic on the lower level. The roadway is a constant width of 40′ curb-to-curb along the length of this frame. Each level of the structure is composed of end spans with lengths of 54.8′ and one interior span of 70.3′; total frame length is 180.0′. Elevations based on the NAVD 88 datum are 71′ and 49′ for the upper and lower roadway surfaces, respectively. Ground elevation in the vicinity is approximately 16′ throughout the footprint of the structure.

The existing frame is supported by reinforced concrete columns founded on concrete piles. End Bents #103 and #106 are composed of two 2.0′x4.0′ rectangular columns supported by 14-pile foundations. Each foundation is shared between adjacent frames and therefore supports two columns in a split-column fashion. Interior Bents #104 and #105 are composed of two 4.0′x4.0′ square columns supported by 20-pile foundations. All piles are reinforced concrete for Subsegment 11. Piles are 14″ diameter and are shown in existing plans to range from 25.0′ to 35.0′ in length providing penetration up to approximately 42.0′ and as little as 32.0′ below the existing ground surface.

Non-Linear Ultimate Displacement (Pushover) Analysis

Non-Linear Ultimate Displacement (pushover) analysis, similar to that described in the Single-Level Viaduct Retrofit 500 Section, was performed on an interior bent in the transverse direction. Based on this analysis, the following transverse load-deflection behavior was assumed for the Seattle designed structures:

For Bent #104, the following was found (also see Figures 10 and 11):

- Stage 1 – At a lateral seismic load of 250 kips, a displacement of 0.5″ occurs and a true hinge forms in the lower bent cap near east column.
- Stage 2 – At a lateral seismic load of 170 kips, a displacement of 1.4″ occurs and a true hinge forms in the top of the east column.
- Stage 3 - At a lateral seismic load of 165 kips, a displacement of 2.2″ occurs and a plastic hinge forms in the upper cap beam at the ¼ point west of the east column.

- Stage 4 - At a lateral seismic load of 165 kips, a displacement of 2.3" occurs and the lower cap beam develops a plastic hinge at the ¼ point west of the east column.
- Stage 5 – At a lateral seismic load of 150 kips, a displacement of 5.1" occurs and a plastic hinge occurs at the mid-height of each lower column. The structure is then rendered unstable and collapse occurs.

Rebuild 500 Design Approach

The Rebuild 2500 Plan would rebuild Subsegment 11 as follows:

- Strengthen the upper existing cap beams to HS-25 live load
- Replace the lower existing cap beams and design to HS-25 live load
- Strengthen the center interior stringers to HS-25 live load
- Replace the existing exterior longitudinal beams
- Replace the roadway deck and traffic barriers
- Replace existing columns
- Pin top of column to upper existing cap beam connection
- Replace existing foundations with new reinforced concrete piles

The combination of strengthening and rebuilding the existing structural elements would be adequate to meet the project design criteria previously referenced. The Rebuild 500 of Subsegment 11 would use similar techniques to the Rebuild 2500 Plan, but with smaller members.

3-D Dynamic Analysis

A 3-D frame of Subsegment 11 was modeled under the rebuilt condition in GT STRUDL. Included in this model were all existing, strengthened, and rebuilt elements along with their masses.

The pile analysis program, L-PILE, was used to determine lateral behavior for new pile foundations. Foundations were then modeled in GT STRUDL using lateral springs. The pile foundations included only translation springs in both horizontal directions assuming the pile group fixes the foundations from rotation.

The behavior for the rebuilt Viaduct was bounded using two types of soil properties: cyclic and liquefied. These soil types were those implemented in the 2500-year analysis and are expected to be similar for a 500-year event. The rebuilt Viaduct structure was modeled using a linear elastic model and elastic forces were converted to design forces with R-factor of five (multiple column bent) as specified in the AASHTO design code.

Analysis Results

Table 3 compares results from the Rebuild 500 design with the Rebuild 2500 design. It can be seen by comparison of the different earthquakes on similar structures that average seismic base shear of the 2500-year return period event is about twice the average seismic base shear of the 500-year return period event. Consequently, the supporting structure of columns and foundations has been reduced from 5′ x 5′ concrete columns supported by twelve 30″ diameter battered piles to 4′ x 4′ concrete columns supported by eight 30″ diameter vertical concrete piles (refer to Figures 12 and 13). Average lateral displacement has also been decreased from 18.9″ with the 2500-year event to 6.6″ with the 500-year event.

Table 3: Rebuild 500 Design vs. Rebuild 2500 Design Double-Level Structure

	Rebuild 2500	**Rebuild 500**
Seismic Reduction Factor	4 (per MCEER)	5 (per AASHTO)
Peak Spectral Acceleration	1.45 g (Soil Class E)	0.66 g (Soil Profile IV)
Structural Period	1.0 second	1.0 second
Spectral Acceleration Value	1.45 g	0.66 g
Average Seismic Base Shear (based on multimodal analysis results)	1.20 g	0.54 g
Average Lateral Spectral Displacement	18.9″	6.6″
Lower Level Columns	Need 5' x 5' columns with 3.5% reinforcement	Need 4' x 4' columns with 1.5% reinforcement
Pile Cap	Need 22.5' x 30' pile cap	Need 22.5' x 22.5' pile cap
Piles/Shaft	Need 12 - 30" diameter battered piles with 4% reinforcement, per pile cap	Need 8 - 30" diameter vertical piles with 4% reinforcement, per pile cap

Summary

The Rebuild 500 design uses smaller columns and fewer piles for the foundation structure when compared with the Rebuild 2500 design. The strengthening in the superstructure remains largely unchanged between the Rebuild 2500 and Rebuild 500 plans because their strength requirements are a combination of deterioration, dead and live loads, and seismic loads. For example, the traffic barriers and selected cap beams require improvement to meet current code standards of impact strength and HS-25 live load, respectively. Existing longitudinal edge beams would be replaced as well to meet the loading demands and seismic detailing requirements. Existing foundations are also to be replaced to satisfy current load requirements.

The Rebuild 500 Plan for Subsegment 11 is as follows (see Figures 12 and 13):

- Strengthen the upper existing cap beams at selected locations to HS-25 live load - similar to Rebuild 2500 Plan
- Replace the lower existing cap beams and design to HS-25 live load - similar to Rebuild 2500 Plan
- Strengthen the center interior stringers to HS-25 live load - similar to Rebuild 2500 Plan
- Replace the lower existing longitudinal edge beams - similar to Rebuild 2500 Plan
- Replace the roadway decks and traffic barriers - similar to Rebuild 2500 Plan
- Replace the existing upper columns with new 4′x4′ columns - Rebuild 2500 Plan uses 4′x5′ columns
- Replace the existing lower columns with new 4′x4′ columns - Rebuild 2500 Plan uses 5′x5′ columns
- Pin top of column to upper existing cap beam connection - similar to Rebuild 2500 Plan
- Replace the existing foundations with new 22.5′x22.5′ pile caps - Rebuild 2500 Plan uses 23.5′x30.0′
- Replace the existing foundations with 8 vertical piles per pile cap - Rebuild 2500 Plan uses 12 battered piles per pile cap

Double-Level Viaduct Retrofit 500

Description of Existing Structure

The Retrofit 500 plan was based on the City of Seattle designed Subsegment 11 as described in the Double-Level Viaduct Rebuilt 500 section.

Non-Linear Ultimate Displacement (pushover) Analysis

As described in the Double-Level Viaduct Rebuilt 500 section, the displacement capacity of Subsegment 11 is limited to 5.1″. At this displacement, the existing Viaduct structure would reach lateral instability and collapse would occur.

Retrofit 500 Design Approach

A different design approach for this Subsegment was used in the Retrofit 500 than the Rebuild 2500. The Retrofit 500 Plan of Subsegment 11 follows:

- Strengthen the existing cap beams at selected locations to HS-25 live load
- Strengthen the center interior stringers to HS-25 live load
- Replace the roadway decks and traffic barriers

- Pin top of column to upper existing cap beam connection
- Strengthen the existing foundation at select locations
- Construct new lateral restraint structure to limit design earthquake lateral displacements to below collapse level

The combination of strengthening the existing structure and the addition of new lateral resisting elements would be adequate to meet the current WSDOT earthquake design standards. Average lateral spectral displacements were limited to below 5.1″ and therefore below the collapse limit estimated by the push analysis. The new frames are designed to perform in a linearly elastic mode under the design earthquake. The existing structure would no longer be unstable at the 5.1″ of displacement due to the addition of the new frames. This is considered a conservative but appropriate limit at this conceptual design level.

3-D Dynamic Analysis

A 3-D frame of Bents #103 to #106 was modeled under the retrofitted condition in GT STRUDL. Included in this model was the existing Viaduct structure with additional stiffness and mass added by retrofitting existing members for a HS-25 live load and current code. Also included were the new lateral resisting elements consisting of the steel moment frame supported by drilled concrete shaft foundations.

L-PILE was used to determine lateral behavior for existing piles and the new drilled concrete shafts. Existing foundations were then modeled in GT STRUDL using lateral springs. The pile foundations included only translation springs in both horizontal directions assuming the pile group fixes the foundations from rotating. Drilled shafts were modeled with beam elements having an equivalent depth to fixity as determined from the L-PILE analysis.

The behavior for the retrofitted Viaduct was bounded using a combination of soil types and existing structural behavior. Two types of soil properties and two scenarios of existing Viaduct behavior combined to produce four unique behaviors. Soil properties of cyclic and liquefied from the 2500-year analysis were used and are expected to be similar for a 500-year event. The existing Viaduct structure was modeled using two linear elastic models. One model had no hinges in the structure and the second model had hinges placed at the locations as prescribed by the Non-Linear Ultimate Displacement (pushover) analysis. All hinges were set as true hinges to produce conservative displacement results and bound the results.

Analysis Results

Table 4 compares results from the Retrofit 500 design with the Rebuild 2500 design. It can be seen by comparison of the different earthquakes on the same structure that average seismic base shear of the 2500-year return period event is about twice the average seismic base shear of the 500-year return period event.

Displacement for the retrofit Viaduct is 5.0″ and is below the design displacement limit to avoid collapse.

Table 4: Analysis Results of Retrofit 500 vs. Rebuild 2500

	Rebuild 2500	**Retrofit 500**
Seismic Reduction Factor	4 (per MCEER)	5 (per AASHTO)
Peak Spectral Acceleration	1.45 g (Soil Class E)	.66 g (Soil Profile IV)
Structural Period	1.0 second	0.79 second
Spectral Acceleration Value	1.45 g	0.66 g
Average Seismic Base Shear (based on multimodal analysis results)	1.20 g	0.63 g
Average Lateral Spectral Displacement	18.9″	5.0″
Average Distribution of Lateral Load	-	71% to new structure, 29% to existing
Lower Level Columns	Need 5' x 5' columns with 3.5% reinforcement	9′ x 9′ Steel Columns
Pile Cap	Need 22.5' x 30' pile cap	-
Piles/Shaft	Need 12- 30" diameter battered piles with 4% reinforcement, per pile cap	Need 12' diameter concrete shaft

The average distribution of lateral load has the existing structure taking approximately 30% of the base shear while the new structural bent frames attract approximately 70%.

Summary

The Retrofit 500 design uses new frame structures composed of steel columns and cross beams founded on drilled concrete shafts to provide the required additional lateral restraint to avoid collapse of the existing Viaduct during a design level earthquake (refer to Figures 14 and 15). Frames would be constructed and attached to the existing superstructure at midspan locations. The extent of the superstructure strengthening has been reduced slightly from the Rebuild 2500 Plan and is limited to strengthening of the existing cap beams, interior longitudinal stringers, and replacement of the existing roadway deck and traffic barriers. The traffic barriers, cap beams, and longitudinal beams would require improvement to meet current WSDOT standards of impact strength and HS-25 live load, respectively.

The Retrofit 500 Plan for Subsegment 11 would be as follows (see Figures 14 and 15):

- Strengthen the existing cap beams at selected locations to HS-25 live load - Similar to the Rebuild 2500 Plan

- Strengthen the center interior stringers to HS-25 live load - Similar to the Rebuild 2500 Plan
- Replace the roadway decks and traffic barriers - Similar to the Rebuild 2500 Plan
- Construct new 12′ diameter drilled shaft foundations - None in the Rebuild 2500 Plan
- Erect new 9′ deep double steel columns and 10′ deep double cross beams, attached to existing Viaduct at upper and lower deck levels - None in the Rebuild 2500 Plan

SOIL STABILIZATION SOUTH OF KING STREET

Design Approach

The depth of liquefaction along the proposed Alaskan Way Viaduct corridor was reevaluated considering a 500-year return period ground motion. The peak ground acceleration (PGA) in soft rock associated with this ground motion level is estimated to be 0.33g (USGS, 1996). A moment magnitude (M_w) 7.5 earthquake was considered the characteristic event for the 500-year return period ground motion. The data sources and analysis methods that were used to obtain the current thickness of potentially liquefiable soils for the 500-year event are similar to those previously presented in Shannon & Wilson's June 21, 2002, Technical Memorandum "Elevation of the Top of Glacially Overridden Soils and Thickness of Liquefiable Soils", Alaskan Way Viaduct Corridor, Seattle, Washington. The technical memorandum "Thickness of Liquefiable Soils 475-Year Return Period Ground Motion, Alaskan Way Viaduct Corridor" (Shannon & Wilson, January 17, 2003) is included as Appendix A.

Ground response analyses were performed for this study using the 1-D equivalent linear program ProShake (EduPro Civil Systems, 1999). For the subsurface conditions north of Royal Brougham Way, the cyclic stress ratio (CSR) equation presented in the June 2002 memo was used to estimate the CSR. For the subsurface conditions south of Royal Brougham Way, where the thickness of the loose Holocene deposits exceed 100' to 200', ground response analyses were performed at representative borings to evaluate an improved estimate of the CSR values. Site-specific shear wave velocity profiles, a required input for the ground response analysis, have not been obtained for the borings drilled during this project. The shear wave velocity profiles assumed for the analyses are based on the results of testing completed for previous studies. Rock input time histories for the ground response analyses were selected from studies previously performed by Shannon & Wilson, Inc. for projects located at nearby sites and scaled to the 500-year return period PGA.

Based on the results of the ground response analyses performed for representative borings located along the alignment, the PGA at the ground surface could vary between 0.10g and 0.22g, depending upon the input time history. The depth of liquefiable soils is highly dependent on this acceleration value.

Findings

Based on the results of the liquefaction analyses described above, two maps showing the estimated thickness of liquefiable soils or liquefaction isopachs for the 500-year ground motion were developed and are presented in the Appendix A figures. The liquefaction isopachs correspond to the thickness of soils where

the factor-of-safety is less than 1.0 (liquefiable soils) for each retrieved sample. The elevation of the bottom of the liquefiable soil layer, $Elev_{liq\ soil}$, at any desired location can be estimated using the following relation:

$$Elev_{liq\ soils} = Elev_{surface} - H_l - H_w$$

where:

$Elev_{surface}$ = the ground surface elevation at the desired location,

H_l = the thickness of liquefiable soils obtained from Figure 2 in Appendix A, and

H_w = the depth to the groundwater table (assume 5′ as a preliminary estimate).

The variability of the input rock time histories and CSR (i.e. soil ground motions) estimated from the ground response analysis results in a thickness of potentially liquefiable soil that varies between the isopachs shown on Figures 1 and 2 in Appendix A. It should be noted that Figure 2 is the same figure (Figure 6) presented in the June 2002 memorandum for the 2500-year earthquake. In our opinion, the isopachs presented on Figures 1 and 2 represent estimates of the lower and upper bounds, respectively, of the thickness of liquefiable soils estimated for the 500-year event. In general, this corresponds to a thickness of liquefiable soils equal to about 60′ and 100′, respectively, below the groundwater table.

Given the extremely close proximity of the alignment to the Seattle Fault, we estimate that lateral spreading displacement for the 500-year event as a result of the liquefaction would be several yards for that portion of the southern alignment that is not protected by a suitable seawall structure. Such lateral spreading would likely impact the existing Viaduct foundations as well as the deep foundations that are proposed to support new aerial structures located along that portion of the southern alignment not protected by a seawall that has been designed to resist lateral spreading forces. The lateral spread displacements would generally be in a westward direction.

During lateral spreading, the deep foundations would likely be subjected to large passive forces applied by the approximate 5′ to 10′ thick layer of nonliquefied crust riding on top of the liquefied soil. Case histories have shown that these passive forces can cause excessive permanent deformation and rotation of the piles (or pile cap) by a relatively shallow nonliquefied soil layer (e.g. Berrill and Yasuda, 2002, Berrill et al. 2001, Hamada, 1992, Youd, 2002, Boulanger, 2002). It is likely that the existing and proposed pile foundations along the southern portions of the alignment cannot be designed to resist this large passive lateral load on the pile cap. Ground improvement measures would likely be required to reduce the potential for damage to the bridge pier due to lateral spreading displacements and the resulting lateral passive forces.

The objective of the ground improvement at piers along the alignment would be to prevent passive forces from being applied to the pier foundations along the alignment during a lateral spreading event. The soils around the pier could be mitigated against liquefaction for a sufficient distance such that passive forces from the lateral spread would be resisted by the improved ground. The improved ground could also be installed for a sufficient distance around the piers so that if the passive pressures applied by the non-liquefied crust on the pier foundation were greater than the resisting strength of the improved ground, the resulting soil failure surface would not impact the bridge pier.

Ground Improvement Methods

Many methods of ground improvement are available. Installing a grid of deep soil mixed (DSM) or jet grouted columns or vibro-replacement stone columns could be considered suitable methods to reduce the liquefaction potential of soils around deep foundations so that passive forces would not be applied to the pier foundations during a lateral spreading event. DSM and vibro-replacement methods are generally applicable in areas where headroom is not restricted. In limited headroom areas, such as would likely be the case beneath the existing Viaduct, jet grouting could be used in a similar fashion as deep soil mixing.

DSM is a process to treat soil in situ to improve its strength. DSM involves a series of one to four hydraulically driven mixing augers, 18″ to 36″ in diameter, attached to crane-supported leads. As the augers are advanced into the ground, a cement grout is injected under pressure into the soil through the hollow-stem augers. The auger flights penetrate and break loose the soil and lift it to the mixing blades, which blend the soil and cement grout. As the augers continue to advance, the soil and slurry are remixed by additional mixing blades. The percentage of cement grout added per unit volume of soil can be controlled in an attempt to obtain a design strength of improved soil. Adjacent columns would be overlapped to create continuous zones of improved soil. DSM has been used to treat soils to depths greater than 100′.

For this project, overlapped columns could be used to form a grid of cells approximately 15′ to 20′ square. The remaining unimproved soil inside of the cells would be effectively isolated by the strengthened soil such that the untreated soil would not strain sufficiently to liquefy during earthquake shaking. If a grid of DSM cells were utilized, the soil surrounding the piers would not require 100% coverage with DSM. Based on discussions with a specialty DSM contractor, a typical area coverage of DSM to preclude liquefaction would be approximately 35%.

Similar to the DSM alternative, vertical columns of compacted stone could be used to replace and modify the liquefiable subsurface soils. Stone columns are constructed by bottom-feeding stone into a hole that was created by advancing a vibrating probe. Successive lifts of stone are placed and densified with the probe until a column of stone has been formed up to the ground surface. During

placement of the stone, each lift is repeatedly penetrated with the probe to compact it and densify the surrounding soil. When completed, a densified column of stone typically 18″ to 36″ in diameter is left in place. Because the backfilled and compacted stone is relatively dense and relatively pervious, the stone column zone is not liquefiable and improves subsurface drainage. The improved drainage would reduce the potential for pore water pressure build-up that could ultimately lead to liquefaction. The stone columns would be placed in a non-overlapping grid pattern. The spacing and extent of the stone columns would be established to mitigate the liquefaction potential of the surrounding soils and to develop sufficient strength to resist the passive forces applied by the nonliquefied crust on the foundations as a result of lateral spreading. Stone columns can be installed to depths greater than 100′ deep.

Jet grouting is an erosion/replacement system that creates an engineered, in situ soil/cement. Jet grout columns are constructed to form particular geometries for ground improvement solutions. In general, there are no depth limitations for jet grouting. It is anticipated that the geometry required to mitigate liquefaction and lateral spreading along the southern portion of the alignment using jet grouting would be similar to that presented for the DSM technology.

Summary

For conceptual planning purposes, a preliminary amount of ground improvement has been estimated that would be required to mitigate the effects of lateral spreading on the deep foundations. Assuming a 10′ thick nonliquefied soil crust, it is estimated that ground improvement would need to extend approximately 40′ laterally on the north, south, and east sides of the pile caps. Ground improvement would also extend approximately 20′ laterally on the west side of the pile caps. For the existing Viaduct foundations and for new deep foundations proposed to support the aerial structures south of Atlantic Street, the improved ground would extend to a depth slightly greater than the estimated thickness of liquefiable soils as follows:

Table 5: Ground Improvement Depths

	Earthquake Return Period	
	2500-year	500-year
S. Holgate Street to S. Atlantic Street	90 - 110′	50 - 110′*
S. Atlantic Street to S. King Street	50 - 70′	50 - 70′

* Range based upon limited geotechnical testing and analysis. This can be narrowed during Preliminary Engineering.

ALASKAN WAY SEAWALL

An analysis of the existing Alaskan Way Seawall indicates that extensive damage and possible collapse are likely for most of the length of the existing Alaskan Way Seawall in a 500-year return period design level earthquake. The Seawall must be replaced or reconstructed to minimize the risks of damage to Alaskan Way and other facilities that are protected by the wall.

Previous designs for "rebuilding" the Alaskan Way Seawall so that it could withstand larger earthquakes were based on earthquakes with return periods of up to 2500 years. The Rebuild 2500 Plan Seawall involves the installation of a drilled shaft "wall" landside of the existing wall in combination with a substantial amount of ground improvement under the existing relieving platform. This would prevent liquefaction in the immediate vicinity of the drilled shaft wall and thereby minimize the lateral earth loadings, both static and dynamic, that can be applied directly to the wall. The concept is illustrated in Figure 16.

The following sections present the results of additional analysis of two typical sections of the rebuilt seawall for a 500-year earthquake.

Design Criteria

Performance Objectives

For the purpose of this study, it was assumed that the rebuild would ensure "life safety" for both the 2500- and 500-year earthquakes. It should be noted, however, that structural criteria, established to prevent collapse of the wall, results in designs that appear to exhibit relatively small movements. Accordingly, impacts to utilities and other structures behind the wall would be minor and it is expected that Alaskan Way could be returned to service soon after a design level earthquake. However, further analysis work is required to verify the deflections computed for this study.

Loadings

The detailed description and magnitudes for earth pressure loadings on the seawall, for both the 2500-year and 500-year earthquakes, are provided in Figure 7.2 of an appendix (Volume 4 of 7) of the project report "Analysis of the Alaskan Way Seawall" (BERGER/ABAM, January, 2003). The sections selected for this analysis are at Virginia Street for the existing Type A seawall and at Madison Street for the existing Type B seawall. A detailed description of the Type A and Type B seawalls is provided in Volume 1 of the above referenced report.

The loadings for the 2500-year and the 500-year earthquakes are graphically illustrated in Figures 17 and 18 respectively. Cyclical loadings are differentiated

from sustained loadings. Sustained loadings include the loads due to liquefaction. The majority of load on the wall in a large seismic event is a sustained load due to liquefaction. The sustained portion of the load is about the same for both earthquakes, as would be expected, indicating that the reduction in load for the 500-year earthquake versus the 2500-year earthquake is minor. The computed reduction is approximately 15% at Virginia Street and approximately 20% at Madison Street.

Structural Design Criteria

Drilled Shafts

Structural design of the drilled shafts is based upon the AASHTO Standard Specification for Highway Bridges, 16th Edition. The flexural capacity of the proposed 5′ diameter shaft is limited by the amount of reinforcement that can practically be placed within the cross-section. The limiting flexural capacity is approximately 4000 foot-kips.

Improved Soil

Soil improvement under the relieving platform is proposed to be accomplished by jet grouting. This method was selected because it can be accomplished in areas where obstructions are likely to be encountered and provides relatively high strength improvement compared to other methods such as Deep Soil Mixing (DSM).

It is anticipated that an unconfined compressive strength of 500 pounds per square inch (psi) can be obtained along most portions of the waterfront due to the granular nature of most of the deposits. Higher strengths are possible through experimentation with modest increases in the amount of grout used. Exceptions are anticipated to occur at locations that have high proportions of organics. It may be difficult to obtain 500 psi in these area and/or extraordinary measures may be required to obtain it. Because organics comprise a relatively minor portion of the soils along the waterfront, it was assumed that an unconfined compressive strength of 500 psi would be obtained at all locations.

For the purpose of this study, the improved soil was treated as a cohesive soil with an allowable shear capacity (cohesion) of 125 psi. It is anticipated that the ultimate shear strength will be half of the unconfined compressive strength, which would be 250 psi. The 125 psi was selected at this conceptual design level to reflect the fact that a portion of the load is dynamic and the block of unreinforced soil improvement will have little ductility.

Deflections

No specific criteria have been developed with regard to deflections. It is desirable to limit deflections to those that will not cause damage to utilities and/or buildings located behind the wall. The deflection limit that would result

in damage to utilities and structures located behind the wall has not been established but for the purpose of this conceptual design. Lateral (horizontal) deflection of the top of the wall less than 6" is likely to result in little or no damage to major utilities and other structures located behind the wall.

Analysis and Design

Design Approach

The seawall rebuild design, for both the 2500-year earthquake and the 500-year earthquake, was arrived at through a "trial and error" approach to sizing of the basic components of the system. The two primary variables are the spacing of the shafts and the width of the soil improvement (the distance that the soil improvement extends east of the face of the seawall) as illustrated in Figure 19, which presents the rebuild design for the 2500-year earthquake.

The shafts and soil improvements were modeled as an integral unit using a finite element approach. Soil pressures are applied to the landside of the improved soil mass and the resulting shear stresses in the improved soil and moments in the drilled shaft wall, as well as estimates of deflections, are provided by the finite element model. These values are compared with the structural design criteria described above. Subsequently, modifications to the shaft spacing and/or extent of soil improvement are made in an iterative fashion until a solution is found that just meets the criteria.

The analytical approach described above models the soil improvement as a piece of unreinforced concrete and does not address a number of non-linear (elastic-plastic) behaviors that likely occur under the seismic loadings in the improved ground. Despite these shortcomings, the analysis models provide a reasonably accurate indication of the strength of the rebuild, a conservative estimate of its capacity and adequate basis for developing a comparison between the two earthquake magnitudes.

Analysis Results

For the 500-year earthquake, two options were investigated. One option kept the spacing of the drilled shafts the same as for the "2500-year rebuild" and allowed the extent of soil improvement to vary. A second option kept the extent of the soil improvement the same as for "2500-year rebuild" and allowed the drilled shaft spacing to vary. The "500-year rebuild" configurations for the Type A are shown in Figure 20. Configurations for the Type B are shown in Figure 21.

The results of the analysis for a typical Type A seawall are presented in Table 6. Results for the Type B are presented in Table 7. These results indicate that moments in the shafts are controlling the design. Shear stresses in the improved soil are acceptable for the Type B and even lower for the Type A wall. However,

further reductions in soil improvement are not possible because the loads on the shafts rapidly increase as the width of the soil improvement is decreased.

Table 6: Type A Seawall Results

Loading Condition **Width of Jet Grout (ft)** **Shaft Spacing (ft)**	**2500-Year** **40** **10.6**	**500-Year** **35** **10.6**	**500-Year** **40** **16**
% Uplift	75%	80%	75%
Moment in Shaft (ft*k)	3,900	3,900	3,900
Soil Shear Stress (psi)	50	45	60
Deflection (in)	1.50	1.25	1.25

Table 7: Type B Seawall Results

Loading Condition **Width of Jet Grout (ft)** **Shaft Spacing (ft)**	**2500-Year** **62** **10.6**	**500-Year** **58** **10.6**	**500-Year** **62** **13**
% Uplift	80%	85%	75%
Moment in Shaft (ft*k)	4,200	4,100	3,600
Soil Shear Stress (psi)	125	85	130
Deflection (in)	3.75	2.50	2.00

Additional Parametric Studies

An analysis was performed of the Type A and Type B Seawall rebuilds with the shafts removed, using only soil improvement. This analysis demonstrates how a block of improved soil would respond if the shaft is very flexible and the subgrade below the block is very stiff. The results indicate that the shafts could probably be eliminated altogether if the block of improved soil is widened only slightly, although better analysis models that more accurately model the relative stiffness of the soil and shaft are required to verify this result.

Elimination of the shafts and the addition of some jet grout may have additional cost savings to the project. The cost savings could be realized for both the Rebuild 2500 Plan Seawall and the Rebuild/Retrofit 500 Plan Seawall. However, a secondary purpose of the shafts was to provide wave protection to the block of soil improvement by providing a structural anchorage system for fascia panels

spanning between the shafts. Further study is required to determine if a protective wall fascia system can be anchored to the block of improved soil in a durable fashion and can be installed in an environmentally acceptable manner.

Summary

The configurations for the 500-year rebuild resulted in some cost savings over the 2500-year rebuild configuration. Utility relocations, reconstruction of Alaskan Way and allowances for other project contingencies would not change for either type wall. Therefore, the difference between the impacts to the project of the 500-year seawall rebuild compared with the 2500-year seawall rebuild is minimal.

UTILITIES

There is little difference in utility construction impacts between the Rebuild 2500 Plan designed to a 2500-year earthquake return period and either the Rebuild 500 Plan or the Retrofit 500 Plan designed to a 500-year earthquake return period. The utility construction sequence and methodology for the Rebuild 500 Plan is almost identical to the Rebuild 2500 Plan. The same 30′ area west of the existing double-level Viaduct would be required to be kept clear of utilities to accommodate the temporary structure bracing in both Plans. The Seawall utility construction sequence would be essentially the same.

In the Retrofit 500 Plan, there would be some slight differences in construction sequencing and methodology. The most significant difference is with the outfall vaults located at Columbia Street, University Street and Washington Street. In either the Rebuild 2500 Plan or the Rebuild 500 Plan, these vaults would need to be relocated. The Retrofit 500 Plan construction would not require these to be relocated.

The Retrofit 500 drilled shaft foundations also fall outside of the Rebuild 2500 Plan foundation footprint and some shift in utilities would be required to accommodate this. There would be no real difference to utilities between the single-level retrofitted Viaduct that is designed to either the 2500-year or the 500-year earthquake return period.

CONSTRUCTION SEQUENCE

Rebuild 2500 Plan Construction Sequence

The Rebuild 2500 Plan construction sequence process follows the strategy of maintaining current traffic operations and reconstructing the entire facility to the project Structural Design Criteria with a seismic design to a 2500-year return period earthquake (see the technical memorandum "Retrofit/Rebuild Plan," Parsons Brinckerhoff, June 2002). Rebuild 2500 Plan standards are based on AASHTO "life safety" (as opposed to "operational"). Construction staging of the Rebuild 2500 Plan is based on three primary activities: Seawall rebuild, Viaduct rebuild (including ramps), and Battery Street Tunnel upgrade. The primary intent is to rebuild the Seawall first, followed by a Viaduct rebuild schedule. The Battery Street Tunnel upgrade is not considered in this document. The detailed construction sequence is available in the November 6, 2002 Rebuild 2500 Plan In-Progress Design Snapshot submittal. Figure 22 shows the construction sequence flowchart for this plan with a 10-year construction duration.

Rebuild 500 Plan Construction Sequence

This plan closely follows the construction sequence used in the November 6, 2002 Rebuild 2500 Plan In-Progress Design Snapshot submittal. Therefore, each of the plan elements is projected for construction in a manner similar to that described above. Adapting the design criteria from a 2500-year event to a 500-year event creates minor reduction in construction durations for some of the project elements. Figure 23 shows the construction sequence flowchart for this plan, with a nine-year construction duration.

Retrofit 500 Plan Construction Sequence

Comparing the Retrofit 500 Plan with the November 6, 2002 Rebuild 2500 Plan In-Progress Design Snapshot submittal shows little difference in the construction sequence logic. Overall construction duration does not significantly change, but the durations of some individual events do decrease. In the Retrofit 500 Plan the existing structure would be supported with steel columns placed outside the existing concrete columns. The retrofitted structure would be approximately 25' wider in places than the existing Viaduct. Because the Viaduct is in close proximity to a number of buildings and structures, different structural configurations would need to be developed at these select locations. The additional space occupied by this steel could make utility relocations more problematic. These activities do not appear to impact the critical path. Meanwhile the duration of the structural retrofit is expected to be less than the duration of rebuilding the same section. Figure 24 shows the construction sequence flowchart for this plan, with an eight-year duration.

Limited Funding Scenario

This project may be impacted by limited funds. A strategy could be adopted that calls for constructing either the rebuild or retrofit plans using an annual "allowance" to be paid out over a number of years.

Limited funding scenarios are not conducive to design-build practices because one of the advantages of design-build is to account for time as a critical factor. The use of time is by definition not critical when funds are limited. The construction sequence flowcharts for limited funding are applicable for either a Design-Bid-Build or a Design-Build construction methodology.

Risk Reduction Discussion

Under the Limited Funding scenario, the project is broken down into prioritized elements. These elements are proposed to be built in a sequence so that the greatest risk/consequence reduction components are completed first. This would provide the public with the best usage of the taxpayer's money by reducing the highest risk/consequence areas first.

Prioritizing the elements is a complex task. The greatest seismic risk components of the project are not necessarily those areas with the highest consequences resulting from failure. A construction phasing and implementation strategy for risk/consequence reduction under a limited funding scenario was developed by WSDOT, the City, and the Consultant team. It combines the highest technical seismic vulnerability risks with highest social and economic consequences to the public.

All of the Viaduct and Seawall structures were found to have numerous seismic deficiencies, and most likely would fail under a 500-year return period design level earthquake. The prioritization of technical risk/consequence areas, based upon engineering judgment and limited conceptual analysis, is as follows:

- Double-level Viaduct failure in the curve area, adjacent to the historic Pioneer Square District
- Lateral spreading due to the Seawall failure between S. King Street and Virginia Street
- Double-level Viaduct failure in the south end transition area, in liquefaction depths to 100'
- Double-level Viaduct south of S. King Street failure, in liquefaction depths to 100'
- BN Railroad crossing failure at the railroad north downtown tunnel portal
- Double-level Viaduct failure in the north end transition area
- Double-level Viaduct failure north of the Pioneer Square District
- Single-Level Viaduct failure

Failure of any part of the Viaduct and Seawall would have significant adverse consequences. The priority guidelines of social and economic risk/consequence areas, are established for the project under a limited funding scenario as follows:

- City Waterfront between S. King Street and Virginia Street
- BN Railroad crossing at the railroad north downtown tunnel portal
- Seattle City Light Electrical Power hung from the existing Viaduct
- Stadium Area from S. Atlantic Street to S. King Street
- S. Holgate Street to S. Atlantic Street
- Virginia Street to Battery Street

A construction sequence under a limited funding scenario has been developed using a schedule of approximately 25 years to complete and annual expenditures ranging from $75 to $200 million. The lateral spreading due to failure of the Seawall would have the greatest adverse consequence to the public since its failure could destroy not only the Viaduct, but also many waterfront activities and businesses including the WSF; many blocks of downtown Seattle; and the Seattle City Light transmission and distribution systems suspended from the Viaduct. This failure would be mitigated first, followed by the rebuild/retrofit of the most heavily developed downtown areas adjacent to the Viaduct between S. King Street and Pike Street, and the BN Railroad crossing. City density is projected to increase between S. Atlantic Street and S. King Street so this area would be the next rebuild/retrofit priority. This construction is planned to be followed by S. Holgate Street to S. Atlantic Street and then Virginia Street to Battery Street.

The Rebuild 2500 Plan with Partial Seawall, Rebuild 500 Plan with Partial Seawall, and the Retrofit 500 Plan with Partial Seawall construction sequence flowcharts have been developed based upon the above priorities, see Figures 25 to 27. The project construction component sequence to address the risk/consequence priority guidelines previously stated is recommended as follows:

- Temporary Utility Relocations and Rebuild Seawall from S. King Street to Virginia Street
- Permanent Utility Installations from S. King Street to Virginia Street, and all Electrical Power Relocated to Permanent Locations
- Rebuild/Retrofit the Double-level Viaduct from S. King Street to Pike Street and BN Railroad Crossing
- Temporary Utility Relocations and Ground Improvement from S. Holgate Street to S. King Street
- Rebuild/Retrofit the Double-level Viaduct from S. Holgate Street to S. King Street
- Retrofit the Single-Level Viaduct from Pike Street to Battery Street Tunnel

Rebuild 2500 Plan Construction Sequence

The Opinions of Cost section discusses the funding implications of limited funding. The Rebuild 2500 Plan stretches out over approximately 25 years. Project start-up activities such as mitigation, permitting and design are extended because these activities use more funds than would be available on a yearly basis. Rebuilding the Seawall and the structures run consecutively rather than concurrently. Utility relocations, both temporary and permanent are divided into discrete geographic sections, so as to not risk exposing temporarily relocated utilities over too long a period of time, although electrical power is an exception. All electrical power is planned to be removed from the Viaduct to its proposed permanent location in the first utility relocation activity. This plan is inherently inefficient because it requires repeated mobilizations and demobilizations. Figure 25 shows the construction sequence flowchart for this plan.

Rebuild 500 Plan Construction Sequence

The impact of limited funding on the Rebuild 500 Plan is similar to its impact on the Rebuild 2500 Plan. The materials involved and the labor required are less for the Rebuild 500 Plan than for the Rebuild 2500 Plan, but there is no reduction in the overall construction sequence duration. This reduction is recognized in overall average cost reduction per year. The minimum and maximum yearly expenditures are not affected. Figure 26 shows the construction sequence flowchart for this plan.

Retrofit 500 Plan Construction Sequence

Comparing the Retrofit 500 Plan with the Rebuild 2500 Plan shows little difference in the construction sequence logic under a limited funding constraint. Overall construction duration does not significantly change, but the durations of individual events do change. As described previously, the existing structure would be supported with steel columns placed outside the existing concrete columns in the Retrofit 500 Plan. The additional space occupied by this steel could make utility relocations more problematic. Figure 27 shows the construction sequence flowchart for this plan.

Optional Project Elements

From a limited funding and purely cost-saving perspective, there are several elements of the Rebuild 2500 Plan cost estimate submitted as part of the November 6, 2002 Design Snapshot submittal that could be eliminated from the project and not affect the SR 99 Viaduct structural performance. Eliminating these elements would not be consistent with the stated project purpose and need.

These elements are valuable to the project in that they contribute to a more pleasing, safe, and operable roadway after construction, lessen the impact of the project on the downtown Seattle area during construction, or lessen the effects of a design earthquake to the City. These elements are:

- Roadway: Eliminate fire/life safety upgrade to the Battery Street Tunnel
- Eliminate the Seawall Rebuild north of Virginia Street
- Surface Restoration: Construction of a special promenade area along Alaskan Way surface street
- Establish special landscaped areas over and above the street trees already included
- Reduce the number of pedestrian overpasses at Alaskan Way
- Reduce or eliminate minor street improvements to roads adjacent to Alaskan Way
- Special Conditions: Reduction in maintenance of traffic measures
- Reduce funding to support the system Maintenance and Operations Center
- Eliminate the Traffic Service Maintenance Center
- Reduce Flexible Transportation
- Risk: Reduce project unknowns through risk mitigation with a combination of detailed engineering, investigations, and studies.

Combined, these reductions could reduce total project costs by up to 10%.

OPINIONS OF COST

General Discussion – Opinions of Cost Approach

The relative Opinions of Cost developed in this technical memorandum are somewhat unique due to the nature of the project scope for this phase of work. Opinions of cost delivered in this memorandum support the November 6, 2002 Rebuild 2500 Plan In-Progress Design Snapshot Plan submittal as modified below to account for the reduced earthquake design criteria of 500 years. In addition, the conceptual level opinions of cost are further modified to support a construction phasing strategy under a limited project funded scenario.

The following methodology is used in calculating the opinions of cost.

- The November 6 Rebuild 2500 Plan cost opinion and design snapshot serves as the base documents for all work in this memorandum
- Modifications to the double-level portion of the Viaduct that are to be rebuilt are costed by applying the cost to rebuild Subsegment 11 using the 500-year return period earthquake standard to all double-level sections of the structure.
- Modifications to the double-level portion of the Viaduct that are to be retrofitted are costed by applying the cost to retrofit Subsegment 11 using the 500-year return period earthquake standard to all double-level sections of the structure.
- Modifications to the single-level portion of the Viaduct that are to be retrofitted are costed by applying the cost to retrofit Subsegment 4 using the 500-year return period earthquake standard to the single-level sections of the structure.

Risks and uncertainties associated with large capital projects must be accounted for in the cost opinion. The traditional method is to assign percentage mark-ups to the estimated construction cost to account for what is currently unknown in the project. This includes mark-ups to account for unknowns in the design (design contingency), to account for the influence of tying up large amounts of localized and specialized labor and equipment on one project (market conditions), to account for real estate market fluctuations (right-of-way contingency), to account for change orders during construction (project contingency) and to account for overall changes to the project initiated by either the client or the consultant (program reserve).

Risk for this project is developed and analyzed using a WSDOT-approved process called Cost Estimate Validation Process (CEVP). CEVP starts by recognizing that every estimate is a mix of the very likely, the probable, and the maybe. Through an intense workshop process, somewhat resembling Value Engineering, project quantities, costs and risk are established, then processed

through an analytic model grounded in statistics and probability theory, to establish validated cost estimates. The traditional risk method previously described is used when time and/or resources do not permit execution of the CEVP process.

This report identifies cost items and processes which will be included in the preparation of the cost estimates for a plan and/or individual segments. This will facilitate an adequate budget allowance to be established. Items which cannot be accurately quantified will be addressed through addition of risk, either through the addition of contingency factors or during the CEVP. A conceptual level approach will be used, in which typical major cost elements will be multiplied by a quantity to provide an estimate of the total cost of that item. For example, a typical aerial roadway cross-section would include foundation, pier, girder, and superstructure concrete cost elements or components. Each element (i.e. foundation, pier, etc.) will be grouped and costed separately. This method will allow the summary of quantities to be tracked during the follow-on design phases. All costs will be expressed in 2002 dollars, then escalated to mid-year construction dates, and will be developed from WSDOT historical data, from existing Parsons Brinckerhoff data adjusted for Seattle, and from other pertinent systems throughout the country.

Complete Capital Costs are a function of five components: unit costs, base cost, implementation cost, risk cost, and escalation cost. Each cost component is discussed below.

The Unit Cost prices proposed for the various components of the cost estimates have been prepared and are applied to develop the planning and conceptual cost estimates. The unit costs were developed and compiled from recent WSDOT projects, such as I-90 Sunset I/C Modifications, SR 90 and SR 519, whenever appropriate. The Rebuild 2500 Plan is very similar to the I-101/I-280 Interstate Retrofit project in San Francisco, therefore, these unit prices were adjusted to present day and local Seattle rates. All unit costs are referenced in 2002 dollars. The unit costs include contractor's or supplier's total costs (excluding sales tax) along with mark-ups for the general contractor's overhead and profit. Some elements of the project such as mobilization/demobilization, and supplemental condition items (i.e. scheduling tasks, engineer's trailer, etc.) are handled as a percentage of the total estimate based on professional judgment and experience from other similar projects.

The proposed Base Cost segments consist of the following five primary items: roadway; utilities; seawall; surface restoration (including urban design); and special conditions. Additional mark-ups included in the base cost are contractor mobilization and demobilization (mob/demob), supplemental conditions, artwork, sales tax, and right-of-way acquisition costs.

Implementation Costs include EIS preparation, engineering, design services during construction, construction management, project management, agency expenditures, insurance and right-of-way administration costs. For this project a

specialized cost for flexible transportation is included. Occasionally there is an interest in tracking a project without inclusion of risk and escalation factors, simply by adding Base and Implementation costs together. The sum of Base cost and Implementation cost is called Normal cost.

Risk Costs are factored into the Base and Implementation costs, using the CEVP process or percentage mark-ups discussed earlier.

Escalation Cost is factored into the total of the above costs. Escalation for this project is computed at a 3% compound rate to the estimated midpoint of construction.

Rebuild 2500 Plan – Opinions of Cost Approach

Cost estimating for the Rebuild 2500 Plan is performed by creating quantity takeoffs from composite unit prices and follows the basic construction concepts outlined in this document. The project scope is divided into four components: Seawall, Mainline, Ramps and Battery Street Tunnel. The Seawall is a drilled shaft wall with continuous jet grouting, running from King Street to Myrtle Edwards Park. The Mainline consists of temporary lateral bracing, temporary exterior longitudinal beam supports, new foundations, columns, exterior longitudinal beams and decks. There are three different types of ramps, new ramps at S. Atlantic Street and Royal Brougham Way, rebuilt ramps at Columbia Street and Seneca Street, and retrofit ramps at Elliott Avenue and Western Avenue. Finally, the Battery Street Tunnel consists of two portal extensions with jet fans at the portals, improved ventilation and four emergency egress locations. The Rebuild 2500 Plan cost estimate is developed by calculating a Base Cost, consisting of roadway and seawall construction, utilities, surface restoration, special conditions and all associated mobilization/ demobilization, supplemental conditions, artwork, sales tax and right of way. Associated implementation costs for engineering, EIS preparation, design services during construction, construction management, program management, agency costs, insurance, right-of-way costs and flexible transportation costs are added to the Base Cost figure to create the Normal Cost. Risk and escalation are then added to calculate the Project Cost. These estimates of costs are approximate ranges and have not been validated by a CEVP process.

Double-Level Rebuild 500 – Opinions of Cost Approach

The cost estimating concept for the double-level Rebuild 500 concept is to perform a cost estimate on one structural portion of the Rebuild 2500 Plan costed above and apply the results of that estimate on a per lineal foot basis to all double-level portions of the Viaduct. The structural portion chosen is SED Subsegment 11. The percent difference between the Rebuild 500 and the Rebuild 2500 Plan design was applied to all double-level bent unit prices. The per-bent prices were totaled for all double-level bents. The cost savings of basic structural

components of the Rebuild 500 were assessed against the Rebuild 2500 Plan structural components.

Double-Level Retrofit 500 – Opinions of Cost Approach

The smaller design earthquake allows for the double-level Viaduct to be retrofitted instead of rebuild. The structural portion chosen is SED Subsegment 11. The percent difference between the Retrofit 500 and the Rebuild 2500 Plan design is applied to all double-level bent unit prices. The per-bent prices are totaled for all double-level bents. The principles for developing a cost opinion on a double-level rebuilt structure and a double-level retrofitted structure are essentially the same, only the construction approaches are different. The cost savings of basic structural components of the Retrofit 500 were assessed against the Rebuild 2500 Plan structural components.

Single-Level Retrofit 500 – Opinions of Cost Approach

Single-level retrofit costs are performed in the same manner for both the 500-year return period earthquake and 2500-year return period earthquake design plans because the design logic is essentially the same. A retrofit design was used in both. The unit prices are slightly different to account for the smaller structural element used in the Retrofit 500 design. The cost savings of basic structural components of the Retrofit 500 were assessed against the Rebuild 2500 Plan retrofit structural components.

Soil Stabilization South of King Street – Opinions of Cost Approach

Cost estimates for soil stabilization south of King Street are based on a combination of combined cementitious deep soil mixing (CDSM) and jet-grouting at 35% coverage of specified footprints using a volumetric unit price structure. The area south of Atlantic Street has CDSM at a range of depths from 60′ to 100′ outside the Viaduct structure and jet-grouting at a range of depths from 47′ to 87′ inside the structure. Between S. Atlantic Street and S. King Street, the CDSM runs to 60′ depths outside the structure and 47′ depths of jet grouting inside the structure. The potential savings to the ground improvement cost component is based on the minimum 60′/47′ depth of soil improvement throughout the area south of S. King Street.

Alaskan Way Seawall 500 – Opinions of Cost Approach

The Rebuild 2500 Plan design concept assumes the Seawall would be rebuilt using an oscillator drill rig to drill and cast reinforced drilled shafts, followed by drilling holes through the existing pavement to conduct jet grouting soil improvement directly behind the drilled shaft wall. Revised unit prices are

applied to the 500-year return period earthquake standard seawall based on two factors. The first factor is the spacing between the shafts can increase due to the lower seismic load of the 500-year return period earthquake. The second factor is the horizontal distance away from the shaft wall that must be jet grouted can decrease under the 500-year return period earthquake. The overall project cost savings of the 500-year Seawall Rebuild was assessed against the Rebuild 2500 Plan Seawall.

Rebuild 500 and Retrofit 500 Limited Funding Scenarios – Opinions of Cost Approach

A construction phasing and implementation strategy was developed to support a limited annual project funding scenario. Risk reduction is included in this strategy. Assuming an annual expenditure that varies from $75 million to $200 million and making engineering judgments regarding the phasing of various project elements, plans have been developed at a highly conceptual level that stage the project work accordingly. The limited funding plans have a duration of approximately 25 years and significantly increase the overall project cost. These lengthy project durations are extremely dependent upon risk and escalation factors assigned.

Summary

The following list summarizes cost estimates for each of the build scenarios, comparing rebuild and retrofit options at the 500-year return period earthquake standard to the 2500-year return period earthquake standard submitted on November 6, 2002. **These estimates of costs are approximate ranges and have not been validated by a CEVP process.**

- **SPRING 2002 REBUILD 2500 PLAN $3.2 - 3.5B** (per July 2002 CEVP)
- **REFINED REBUILD 2500 PLAN $2.4 – 2.9B** (per November 6, 2002 Design Snapshot)
- **REBUILD 500 PLAN**
  Potential Total Project Cost Reduction Range Based on Refined Rebuild 2500 Plan: 5 to 15%
- **RETROFIT 500 PLAN**
  Potential Total Project Cost Reduction Range Based on Refined Rebuild 2500 Plan: 10 to 20%

SUMMARY OF FINDINGS

Viaduct Structural Analysis

The Rebuild /Retrofit 500 Technical Memorandum has compared various project components under two different earthquake design standards, the 500-year and 2500-year return period. Some of the elements under consideration showed differences between the two design standards, while others did not. Following will provide a comparative overview between the two design criteria standards and the how these standards impact specific project elements.

Design Criteria (MCEER and AASHTO as summarized below)

MCEER Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, November 2001 (2500-Year Return Period Earthquake)

- The Life Safety seismic performance level selected is intended to protect human life during and following a rare earthquake
- The criteria require that two earthquakes be considered with the following performance characteristics:
    - "Expected Earthquake", with a return period of 110 years. No significant damage to the Viaduct would be expected as a result of this event.
    - "Maximum Credible Earthquake", a rare earthquake with a return period of 2500 years. No collapse would be expected as a result of this event, although parts or all of the facility would most likely be unusable for an extended period of time.

AASHTO Standard Specifications for Highway Bridges, 1996, 16th Edition, Division 1-A, Seismic Design (500-Year Return Period Earthquake)

- The criteria is predicated on the concept that hazard to life is to be minimized.
- The criteria require consideration of a single earthquake with the following performance characteristics:
    - Earthquake motions used in the design should have a low probability of being exceeded during the normal lifetime of the bridge
    - Bridges may suffer damage during a design level event, but have a low probability of collapse due to earthquake motions, although parts or all of the facility would most likely be unusable for an extended period of time.

Single-Level Viaduct Retrofit 500

The single-level retrofit configuration of the Viaduct has fewer and smaller retrofit elements than the retrofit from the Rebuild 2500 Plan. A comparison of the Single-Level Viaduct between the 2500-year and 500-year seismic design criteria is found in the Table 2 Viaduct concept comparison matrix.

Double-Level Viaduct Rebuild 500

The double-level rebuild configuration of the Viaduct has smaller column and foundations elements. A comparison of the Double-Level Viaduct between the 2500-year and 500-year seismic design criteria is found in the Table 3 Viaduct concept comparison matrix.

Double-Level Viaduct Retrofit 500

The double-level retrofit configuration reuses the existing columns and foundation system while providing additional lateral resisting elements in the form of large, stiff frames. A comparison of the Double-Level Viaduct between the 2500-year and 500-year seismic design criteria is found in the Table 4 Viaduct concept comparison matrix.

Soil Stabilization South of King Street

- Ground improvement requirements from S. King Street to S. Atlantic Street are the same in both the 2500-year and 500-year return period earthquake events
  - Ground Improvement Depth: 50′–70′
- Ground improvement depth requirements from S. Atlantic Street to S. Holgate Street may be reduced by up to 40′ between the 2500-year and 500-year return period earthquake events.
  - Ground Improvement Depth, 2500 Year Earthquake: 90′-110′
  - Ground Improvement Depth, 500 Year Earthquake: 50′-110′

Alaskan Way Seawall

The configurations for the 500-year rebuild resulted in nominal cost savings over the 2500-year rebuild configuration. Utility relocations, reconstruction of Alaskan Way and allowances for other project contingencies would not change between the 500-year and the 2500-year earthquake event.

Utilities

Retrofit 500 of Single-Level Structure and Rebuild 500 of Double-Level Structure

There is no appreciable difference in construction impacts to utilities between the retrofit of the structure for the 2500-year return period earthquake compared with the 500-year event.

Retrofit 500 of Double-Level Structure

There is little difference in the construction impacts to utilities between the rebuild of the structure for the 2500-year return period earthquake compared with the 500-year event. The Retrofit 500 Plan would not require the relocation of three existing outfall vaults, which would have to be relocated in the Rebuild 500 Plan and Rebuild 2500 Plan. Also, there would be slightly less width in the utility corridor for the final storm-water vaults in the Retrofit 500 Plan compared with the Rebuild 500 and Rebuild 2500 Plans.

Construction Sequence

Full Funding Scenario

- Baseline - Rebuild 2500 Plan Construction: 10-year duration
- Rebuild 500 Plan Construction: 9-year duration
- Retrofit 500 Plan Construction: 8-year duration

Annual construction expenditures are expected to vary between $100 to $500 million from year to year. These values represent the range of yearly expenditures, not an average.

Limited Funding Scenario

The Rebuild 2500 Plan, the Rebuild 500 Plan and the Retrofit 500 Plan are the only plans identified to date that may be built under a limited funding scenario. A strategy could be adopted that calls for construction with an annual "allowance" to be paid out over a number of years.

Order of Limited Funding Work

The following construction sequence was a result of the risk / consequence priority guidelines developed by WSDOT, the City and the Consultant team, from highest to lowest:

- Temporary Utility Relocations and the Rebuilt Seawall from S. King Street to Virginia Street

- Permanent Utility Installations from S. King Street to Virginia Street and all Electrical Power Relocated to Permanent Locations
- Rebuild or Retrofit the Double-Level Viaduct from S. King Street to Pike Street and the BN Railroad Crossing
- Temporary Utility Relocations and Ground Improvements from S. Holgate Street to S. King Street
- Rebuild or Retrofit the Double-Level Viaduct from S. Holgate Street to S. King Street
- Retrofit the Single-Level Viaduct from Pike Street to the Battery Street Tunnel

All limited funding scenarios are based upon 25-year construction durations with annual expenditures that vary from $75 to $200 million. The Limited Funding construction sequence does not include a rebuilt Seawall from Virginia Street to Myrtle Edwards Park, the Battery Street Tunnel upgrade or a Traffic Service Maintenance Center.

Opinions of Cost

The following list summarizes estimates of costs for each of the build scenarios.

- Rebuild 2500 Plan: $ 2.4 to $ 2.9 Billion

Potential Total Project Cost Reduction Range[2]:

- Rebuild 500 Plan: 5% to 15%
- Retrofit 500 Plan: 10% to 20%

In addition, the conceptual level opinions of cost are further modified to support a construction phasing strategy under a limited funding scenario. It is estimated that the total project cost at mid-point of construction could increase by up to 50% of the Fully Funded cost.

The seismic performance compared with the relative cost is shown in Figure 2.

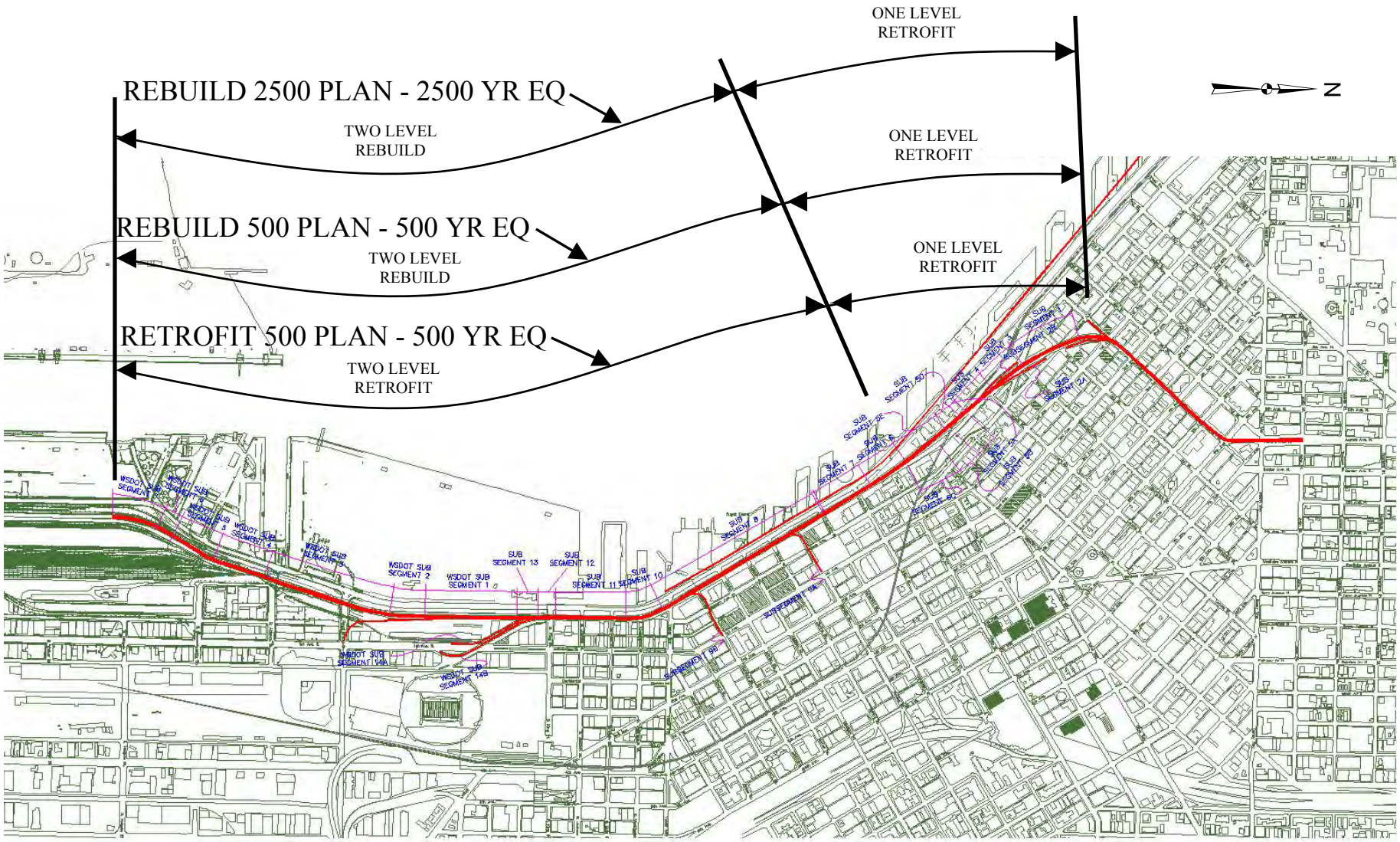
Conclusion

This memorandum provides the technical results of a comparison between the 2500-year and 500-year return period earthquake design standard with various project elements. These findings should provide a solid basis for agency decisions regarding the application of the current verses potential future earthquake design standards. These decisions should be derived by factors other than just technical considerations alone. The project aesthetics, performance,

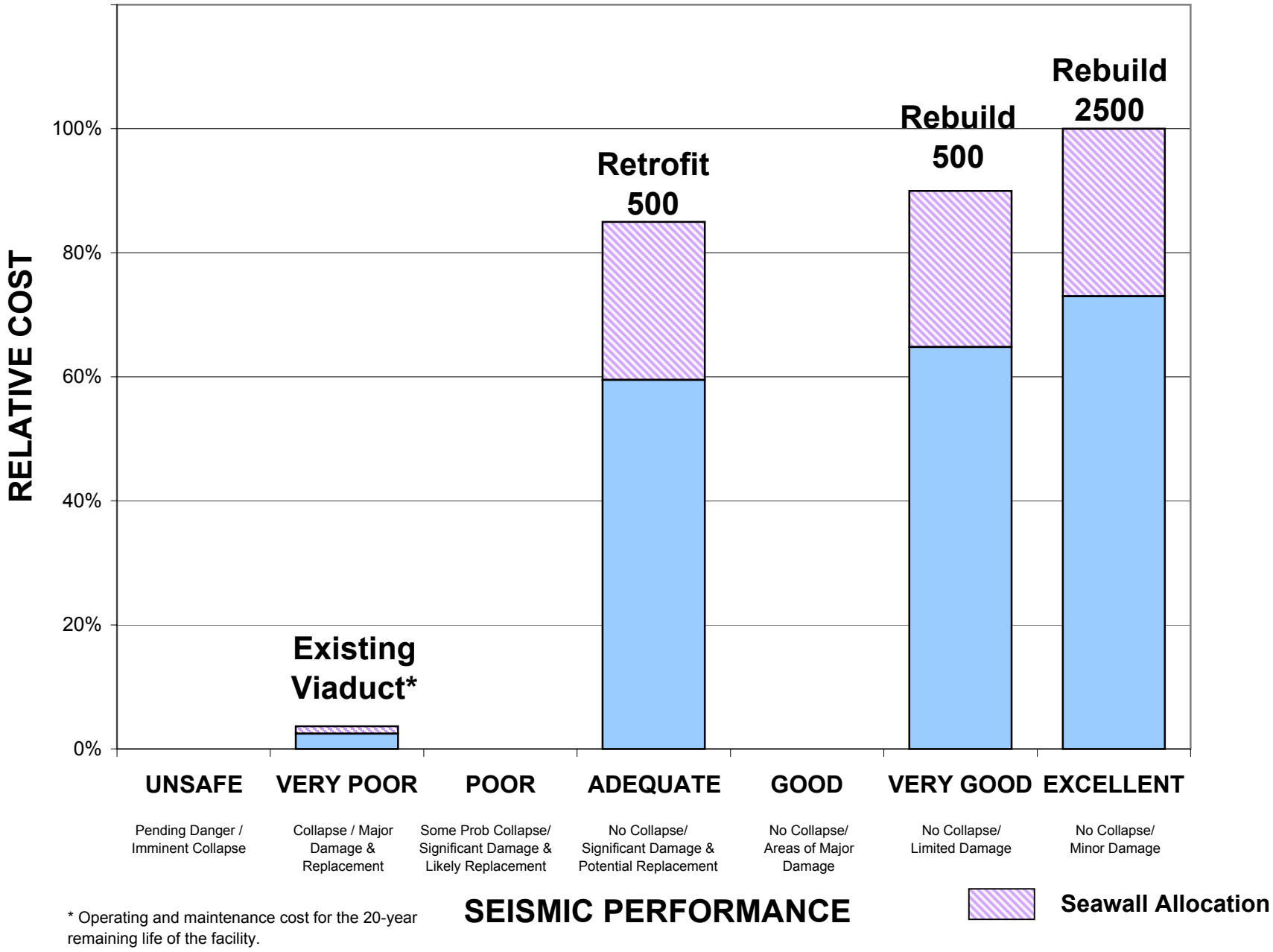
[2] Based upon the Rebuild 2500 Plan Cost range. **These estimates of costs are approximate ranges and have not been validated by the WSDOT Cost Estimate Validation Process (CEVP)**. A CEVP is planned for the current Rebuild 2500 Plan in the future.

constructibility, serviceability, and environmental impacts will all affect policy issues and hence agency decisions.

One conclusion that can be reached is that the Rebuild 500 Plan is far superior to the Retrofit 500 Plan when seismic performance, aesthetics, cost, and risk are balanced. The Retrofit 500 Plan should be dropped from further consideration based upon the technical performance factors.



Rebuild 2500 Plan, Rebuild 500 Plan, and Retrofit 500 Plan Location Map **Figure 1**



* Operating and maintenance cost for the 20-year remaining life of the facility.

Rebuild / Retrofit: Seismic Performance vs. Relative Cost **Figure 2**

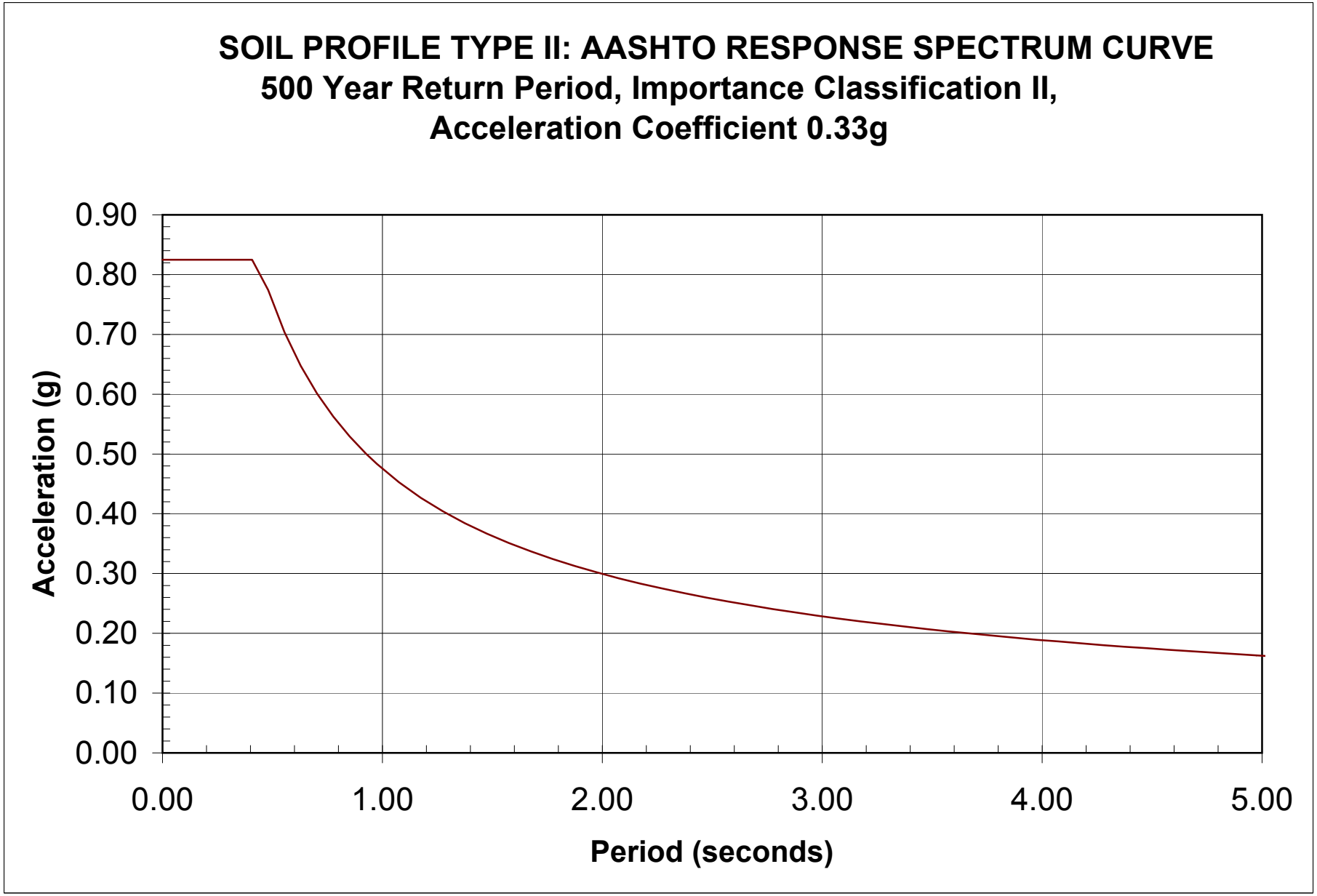

SOIL PROFILE TYPE II: AASHTO RESPONSE SPECTRUM CURVE
500 Year Return Period, Importance Classification II,
Acceleration Coefficient 0.33g
Acceleration (g)
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (seconds)

Figure 3

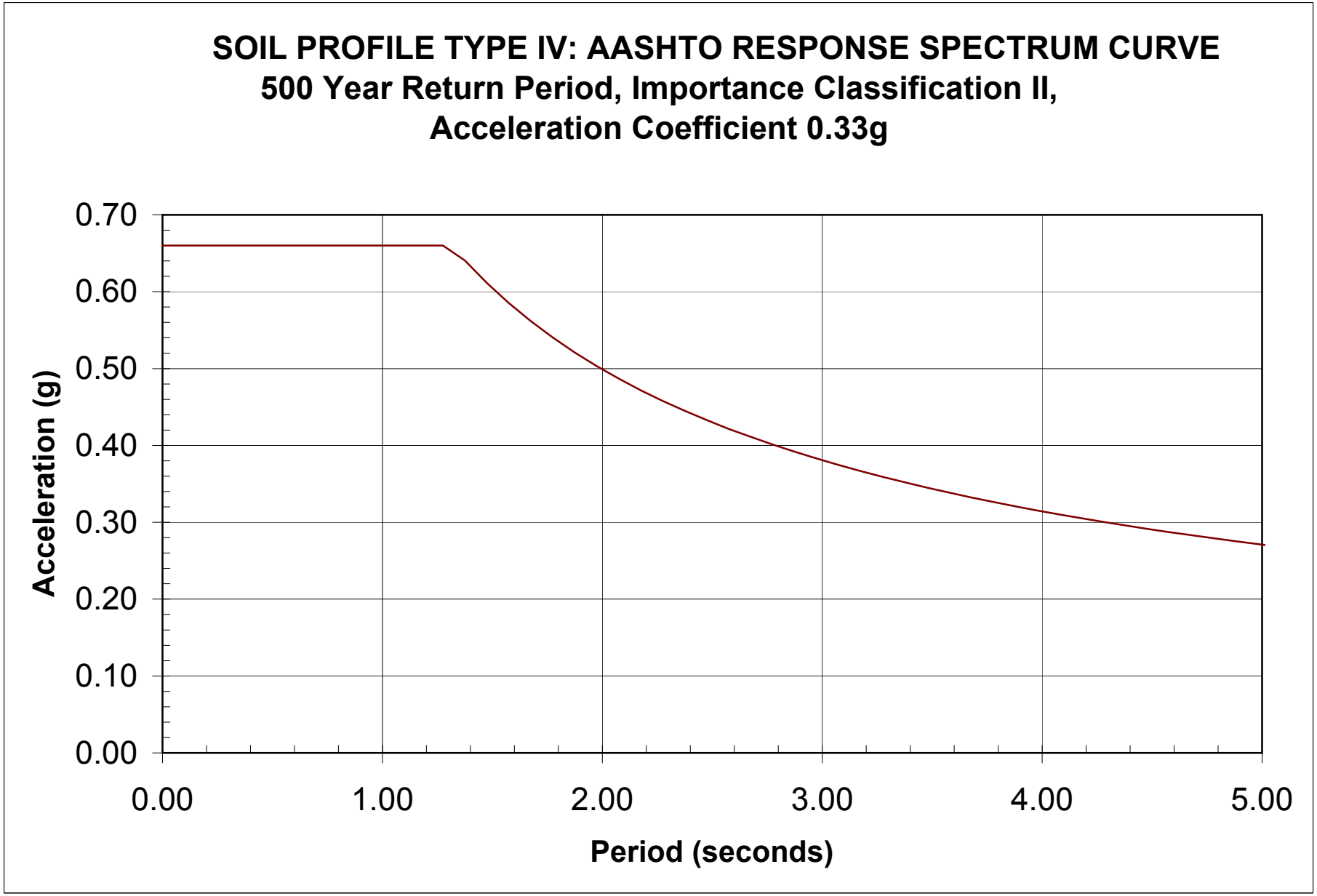
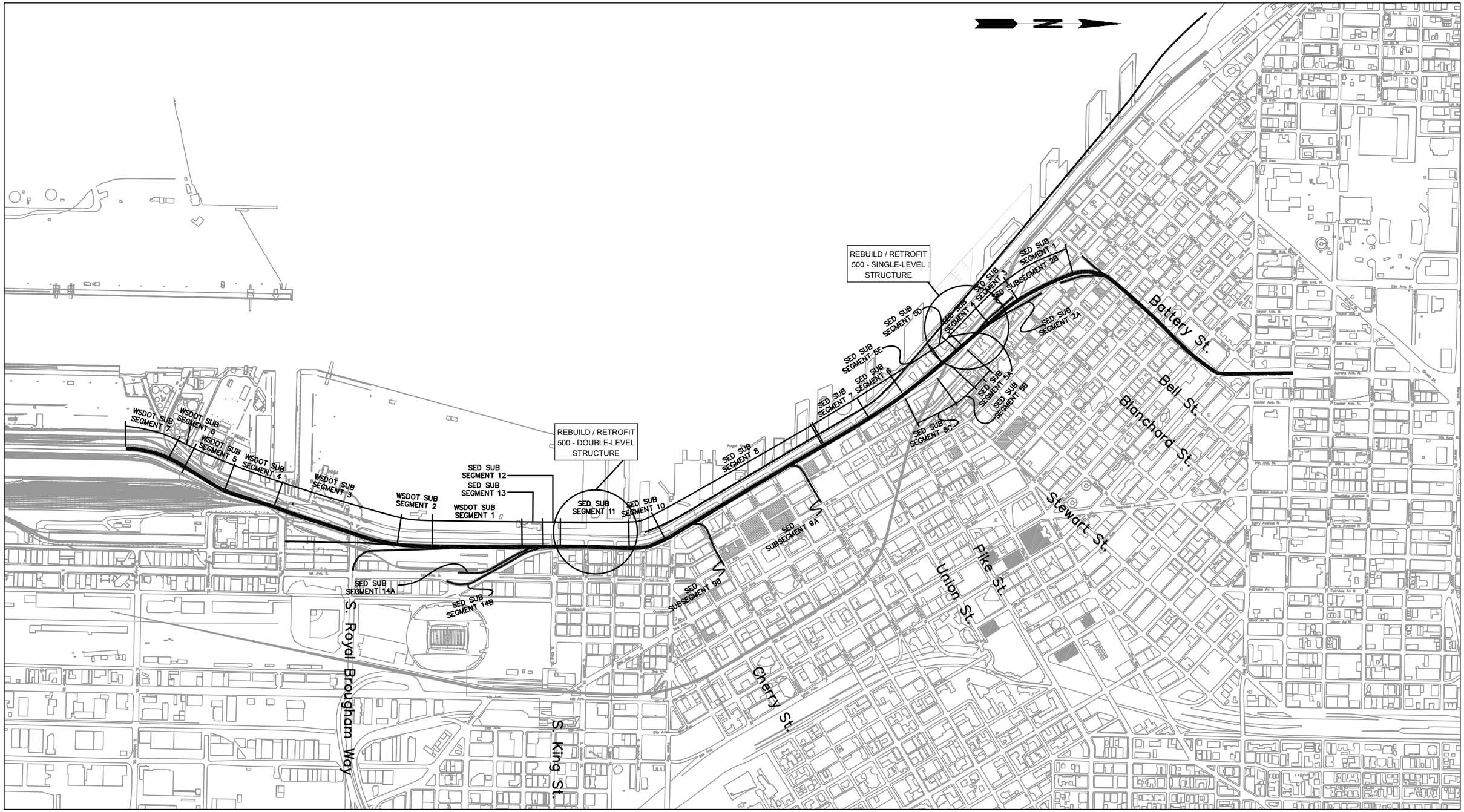

SOIL PROFILE TYPE IV: AASHTO RESPONSE SPECTRUM CURVE
500 Year Return Period, Importance Classification II,
Acceleration Coefficient 0.33g
Acceleration (g)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (seconds)

Figure 4



Existing Viaduct Subsegment Key Map

NTS

NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Engr.						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor						10	WASH			
Designed by										
Checked by										
Detailed by						JOB NUMBER				
Prelim. Design Engr.										
Preliminary Plan by						CONTRACT NO.				
Architect/Specialist	DATE	REVISION	BY	APP'D	DATE					

Washington State Department of Transportation

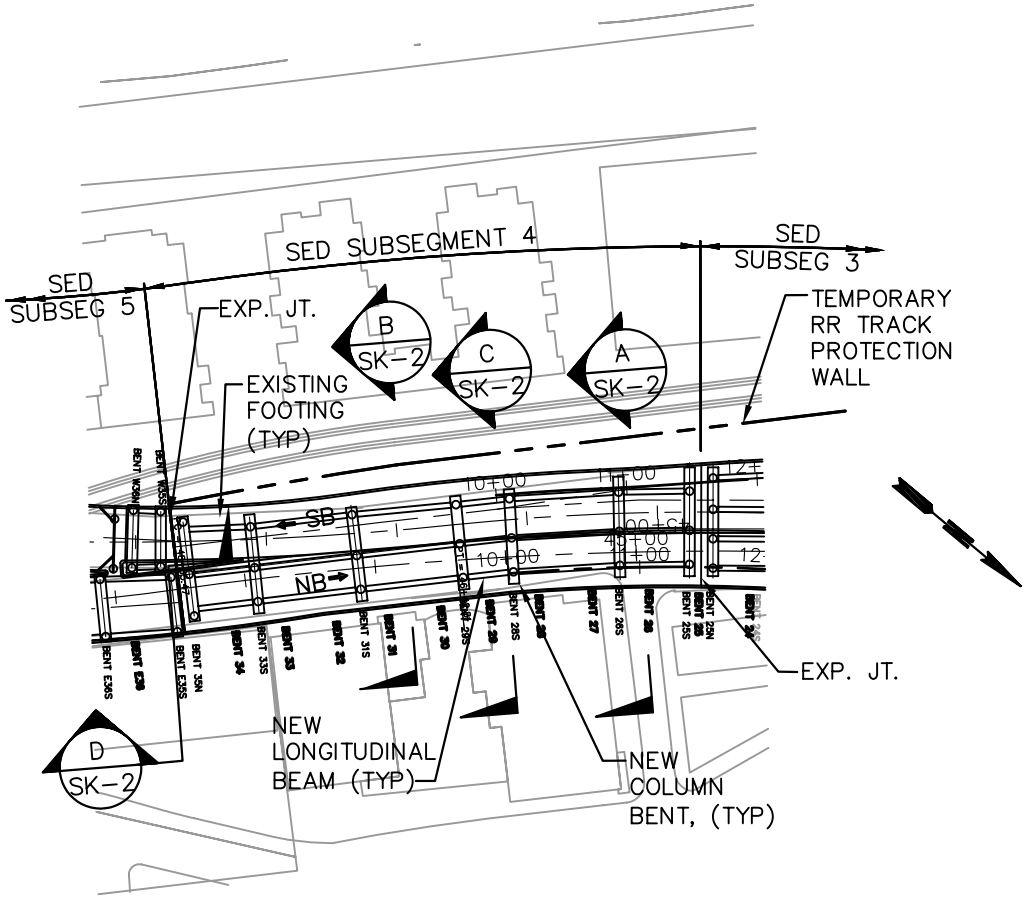
PB PARSONS BRINCKERHOFF

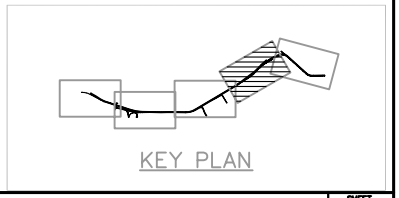
Figure 5

EXISTING VIADUCT SUBSEGMENT KEY MAP

SHEET NO RBLT_01

SHEET OF SHEETS





NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

0 50 100
SCALE IN FEET

Design Project Engr. R. ELLIS						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	CONCEPTUAL DESIGN NOT FOR CONSTRUCTION
Supervisor K. WINTERSTEIN						10	WASH				
Conceptual Design Engr. M. RIGSBY											
Design By						JOB NUMBER Y-7888					
						CONTRACT NO.					
	DATE	REVISION	BY	APP'D	DATE						

Washington State Department of Transportation

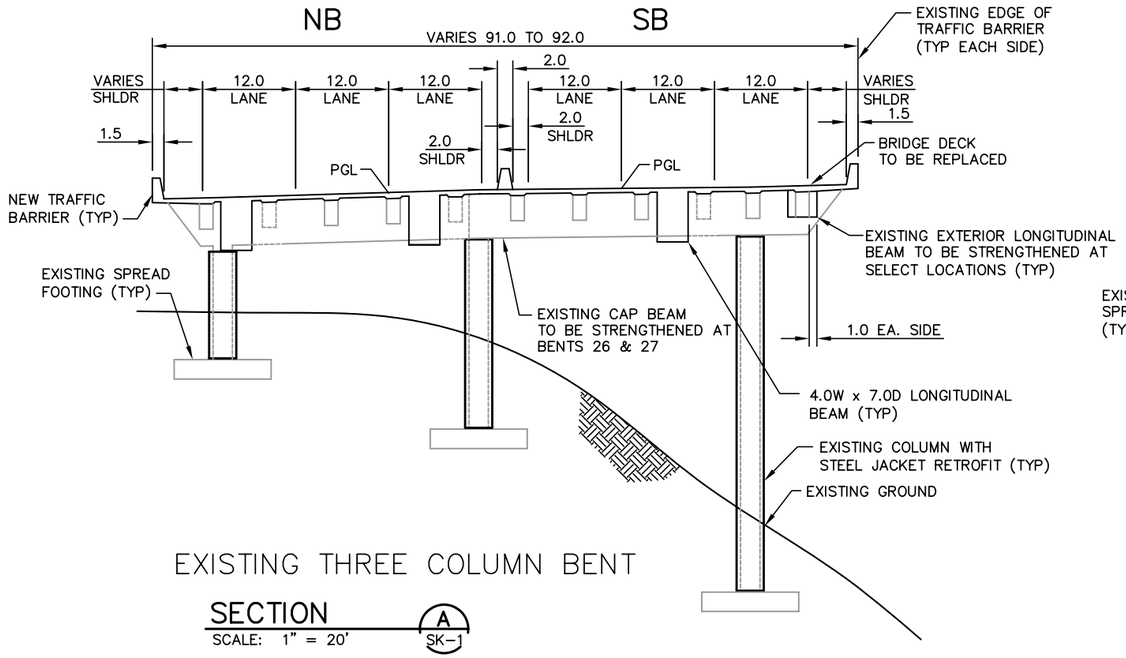
PB PARSONS BRINCKERHOFF

Figure 6

REBUILD/RETROFIT 500
SED SUBSEG 4 PLAN

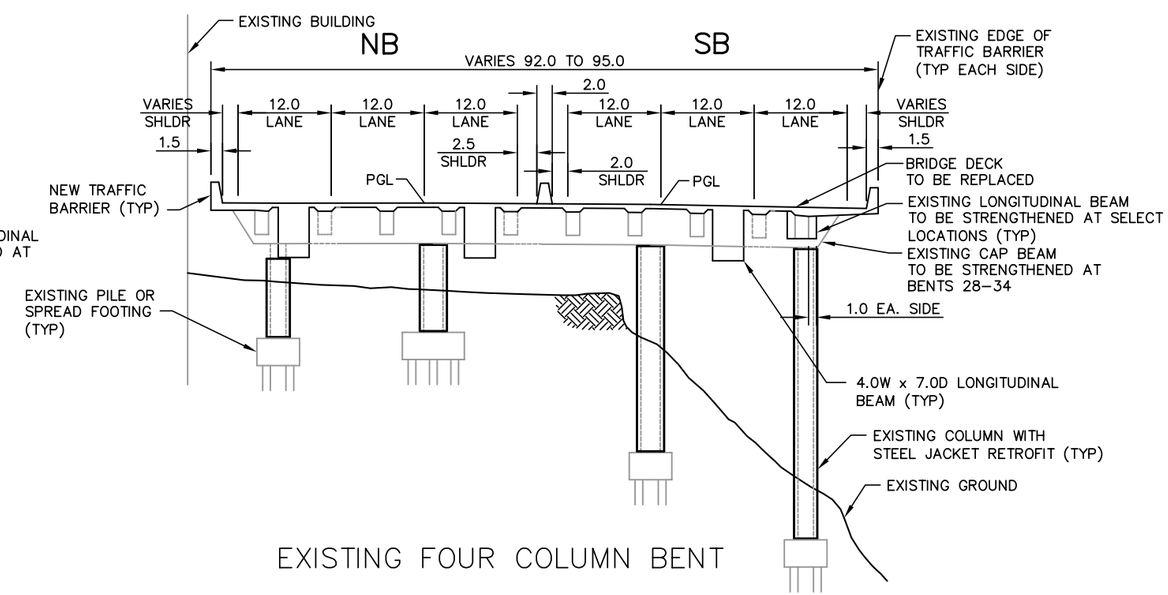
SHEET NO SK-1

SHEET OF SHEETS



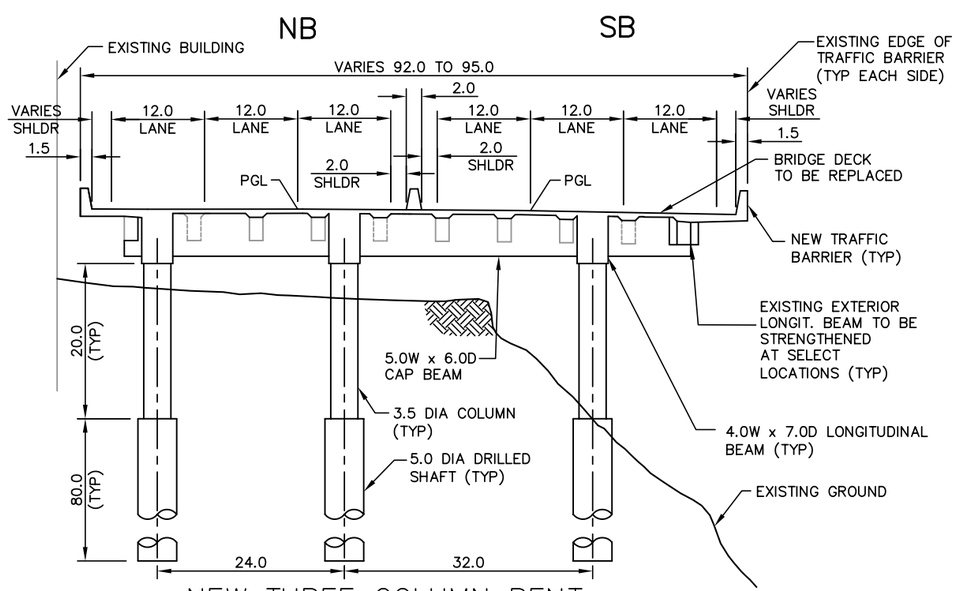
EXISTING THREE COLUMN BENT

SECTION A SK-1
SCALE: 1" = 20'



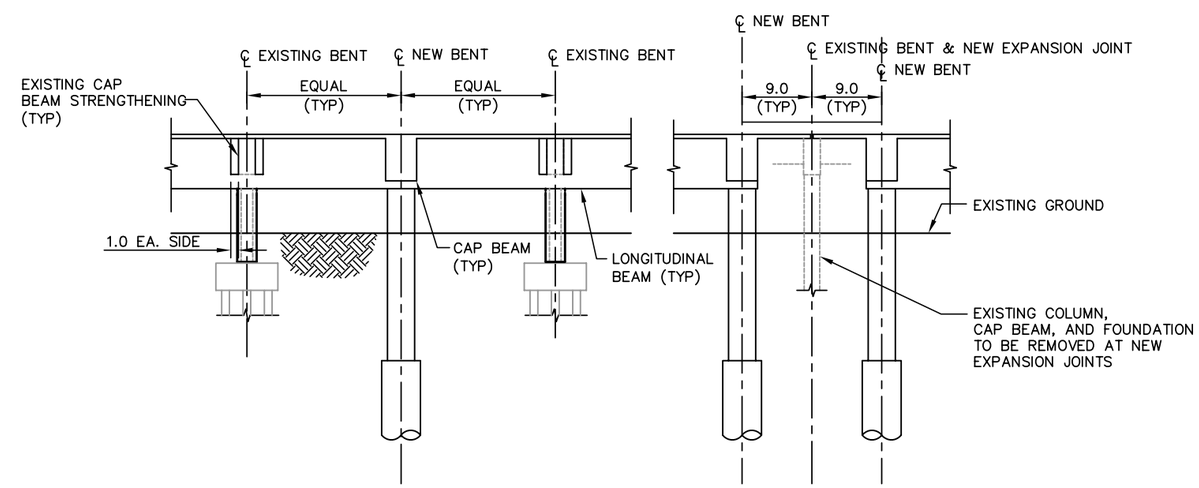
EXISTING FOUR COLUMN BENT

SECTION B SK-1
SCALE: 1" = 20'



NEW THREE COLUMN BENT

SECTION C SK-1
SCALE: 1" = 20'



TYPICAL LONGITUDINAL SECTION

SECTION D SK-1
SCALE: 1" = 20'

NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Engr.						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor						10	WASH			
Designed by										
Checked by										
Detailed by						JOB NUMBER				
Prelim. Design Engr.										
Preliminary Plan by						CONTRACT NO.				
Architect/Specialist	DATE	REVISION	BY	APP'D	DATE					

Washington State Department of Transportation

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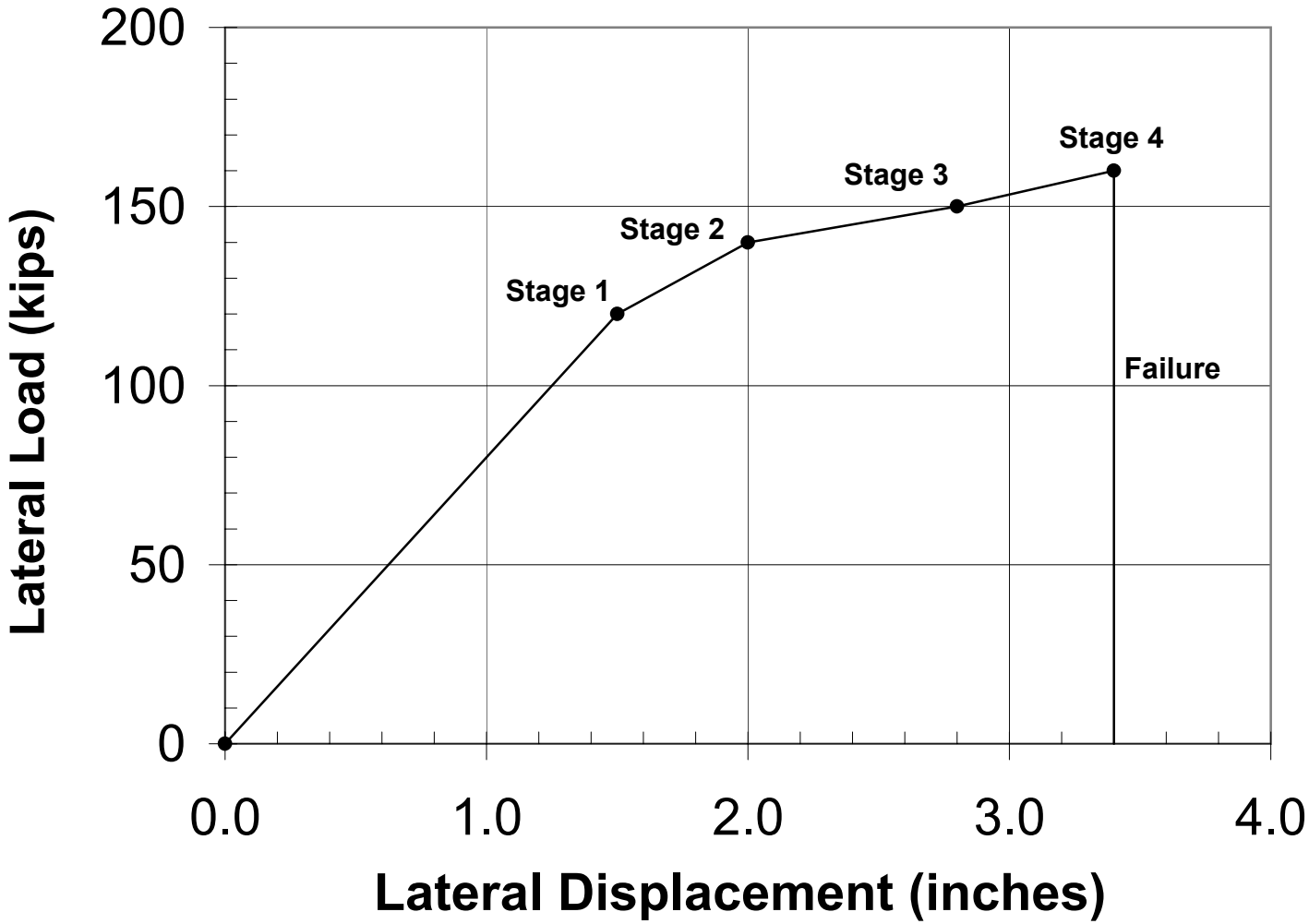
Figure 7

REBUILD/RETROFIT 500
SED SUBSEG 4 SECTIONS

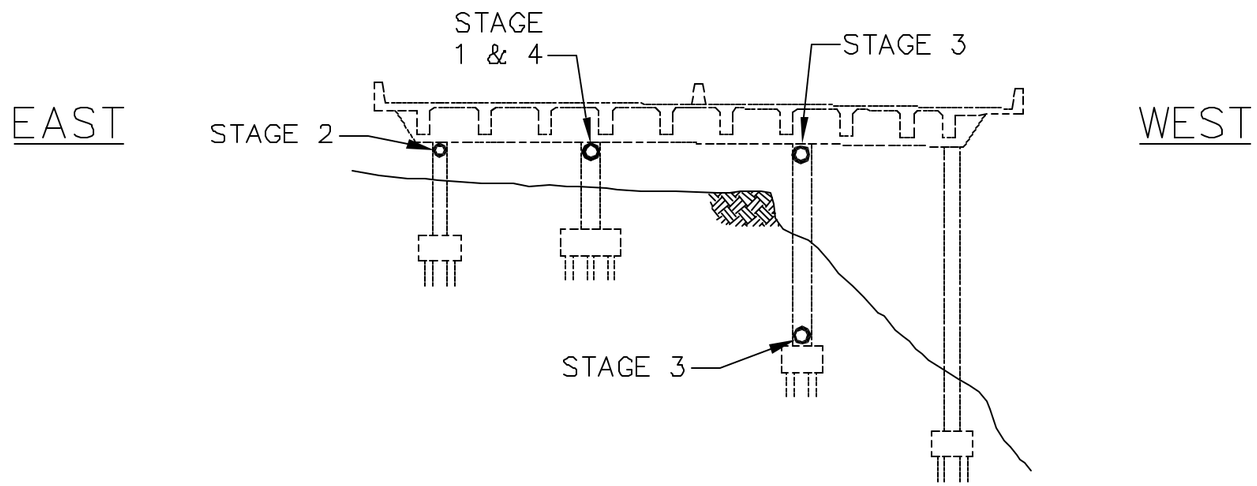
SHEET NO SK-2

SHEET OF SHEETS



NON-LINEAR ULTIMATE DISPLACEMENT (PUSHOVER) ANALYSIS CURVE FOR EXISTING SINGLE-LEVEL AWV

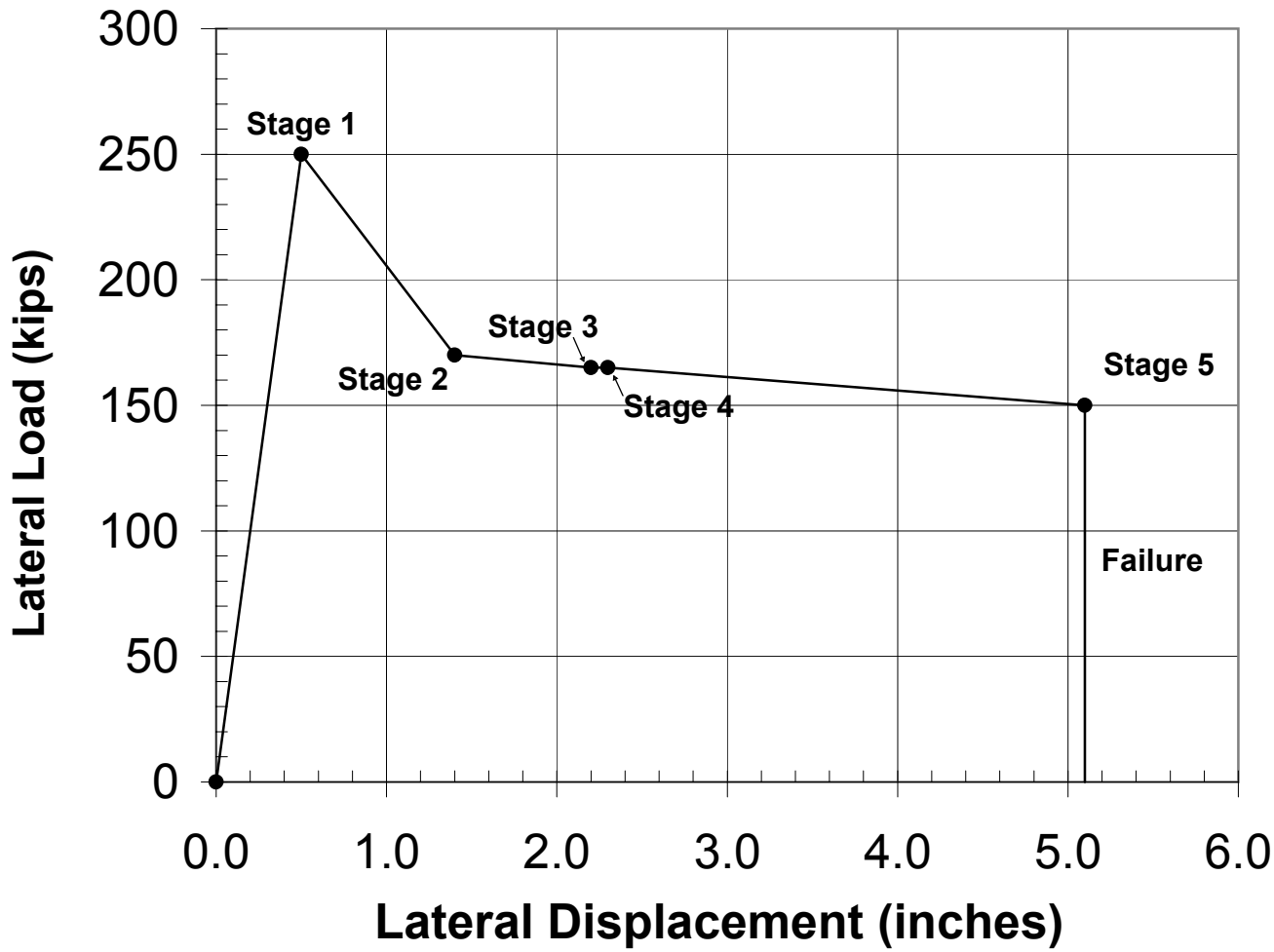
Figure 8



SINGLE LEVEL AWV SEISMIC DEGRADATION
MECHANISM LOCATIONS

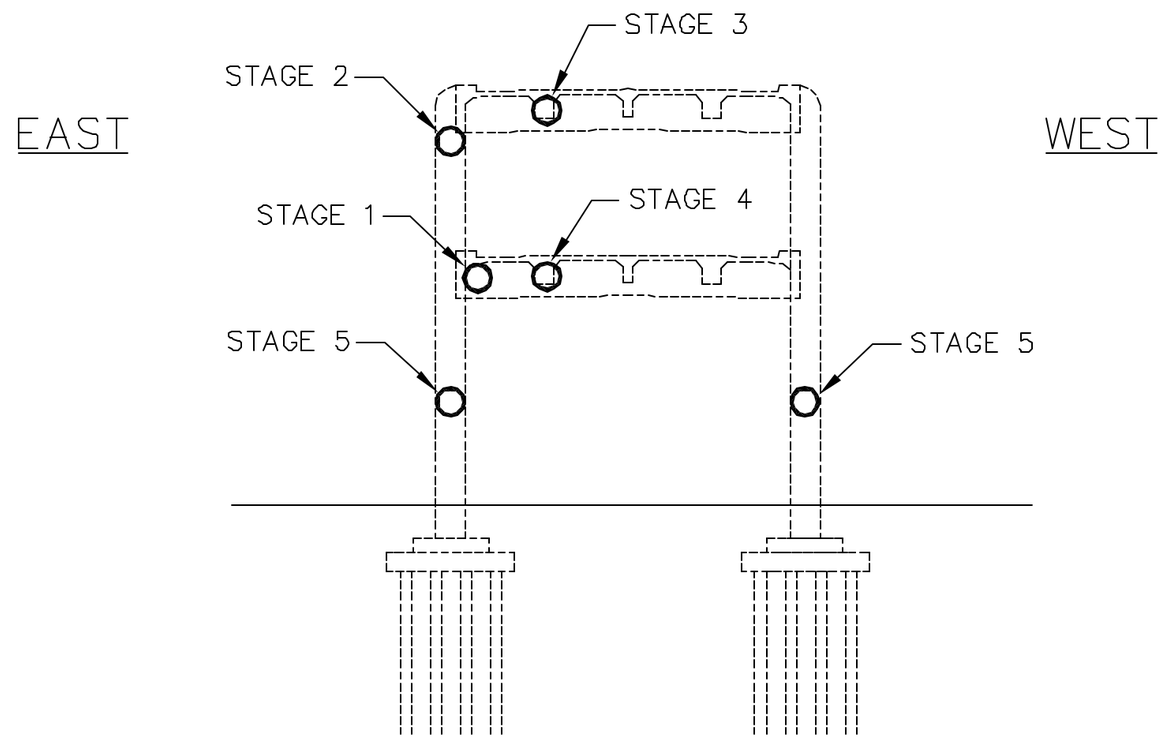
NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Project Engr.	R. ELLIS						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	CONCEPTUAL DESIGN NOT FOR CONSTRUCTION	Washington State Department of Transportation	Figure 9	SHEET NO SK-3
Supervisor	K. WINTERSTEIN						1D	WASH							
Conceptual Design Engr.	M. RIGSBY														
Design By	G. CLARK						JOB NUMBER Y-7888						PARSONS BRINCKERHOFF	REBUILD/RETROFIT 500 SINGLE LEVEL STRUCTURE	SHEET OF SHEETS
							CONTRACT NO.								
		DATE	REVISION	BY	APP'D	DATE									



NON-LINEAR ULTIMATE DISPLACEMENT (PUSHOVER) ANALYSIS CURVE FOR EXISTING DOUBLE-LEVEL AWV

Figure 10



DOUBLE LEVEL AWV SEISMIC DEGRADATION MECHANISM LOCATIONS

NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Project Engr. R. ELLIS						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	CONCEPTUAL DESIGN NOT FOR CONSTRUCTION
Supervisor K. WINTERSTEIN						1D	WASH				
Conceptual Design Engr. M. RIGSBY											
Design By G. CLARK						JOB NUMBER Y-7888					
						CONTRACT NO.					
	DATE	REVISION	BY	APP'D	DATE						

Washington State Department of Transportation

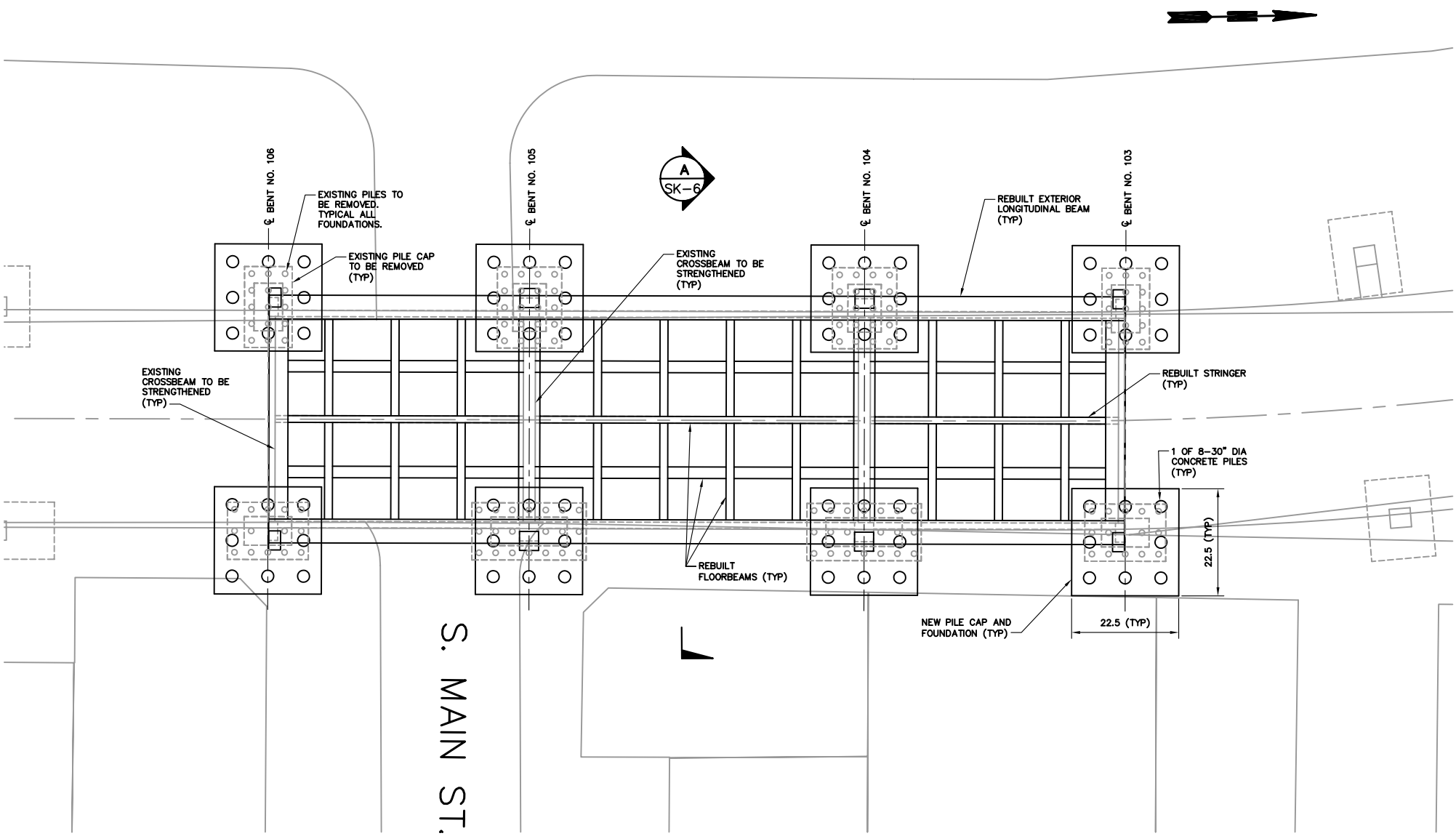
PARSONS BRINCKERHOFF

Figure 11

REBUILD/RETROFIT 500
DOUBLE LEVEL STRUCTURE

SHEET NO SK-4

SHEET OF SHEETS



NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Project Engr.	R. ELLIS						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor	K. WINTERSTEIN						10	WASH			
Conceptual Design Engr.	M. RIGSBY										
Design By	G. CLARK						JOB NUMBER Y-7888				
							CONTRACT NO.				
		DATE	REVISION	BY	APP'D	DATE					

CONCEPTUAL DESIGN
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Washington State Department of Transportation

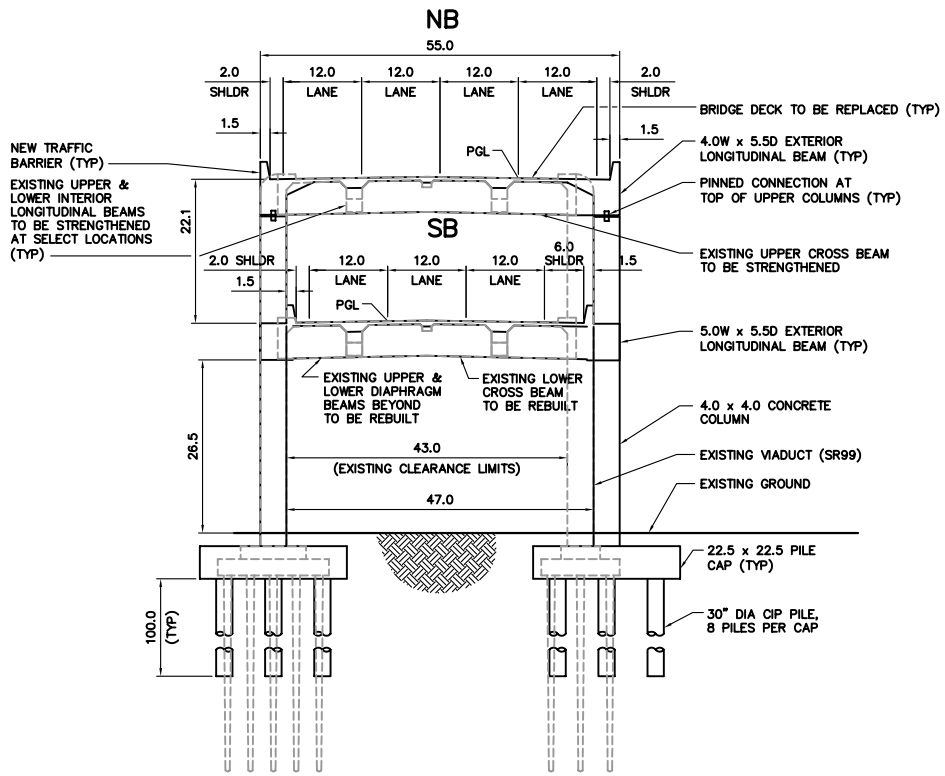
PARSONS BRINCKERHOFF

Figure 12

REBUILD/RETROFIT 500
REBUILD 500 DOUBLE-LEVEL PLAN

SHEET NO SK-5

SHEET OF SHEETS



SECTION A SK-5

SCALE: 1" = 20'

NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Project Engr.	R. ELLIS					REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS	CONCEPTUAL DESIGN NOT FOR CONSTRUCTION
Supervisor	K. WINTERSTEIN					10	WASH				
Conceptual Design Engr.	M. RIGSBY										
Design By	G. CLARK					JOB NUMBER Y-7888					
						CONTRACT NO.					
	DATE	REVISION	BY	APP'D	DATE						

Washington State Department of Transportation

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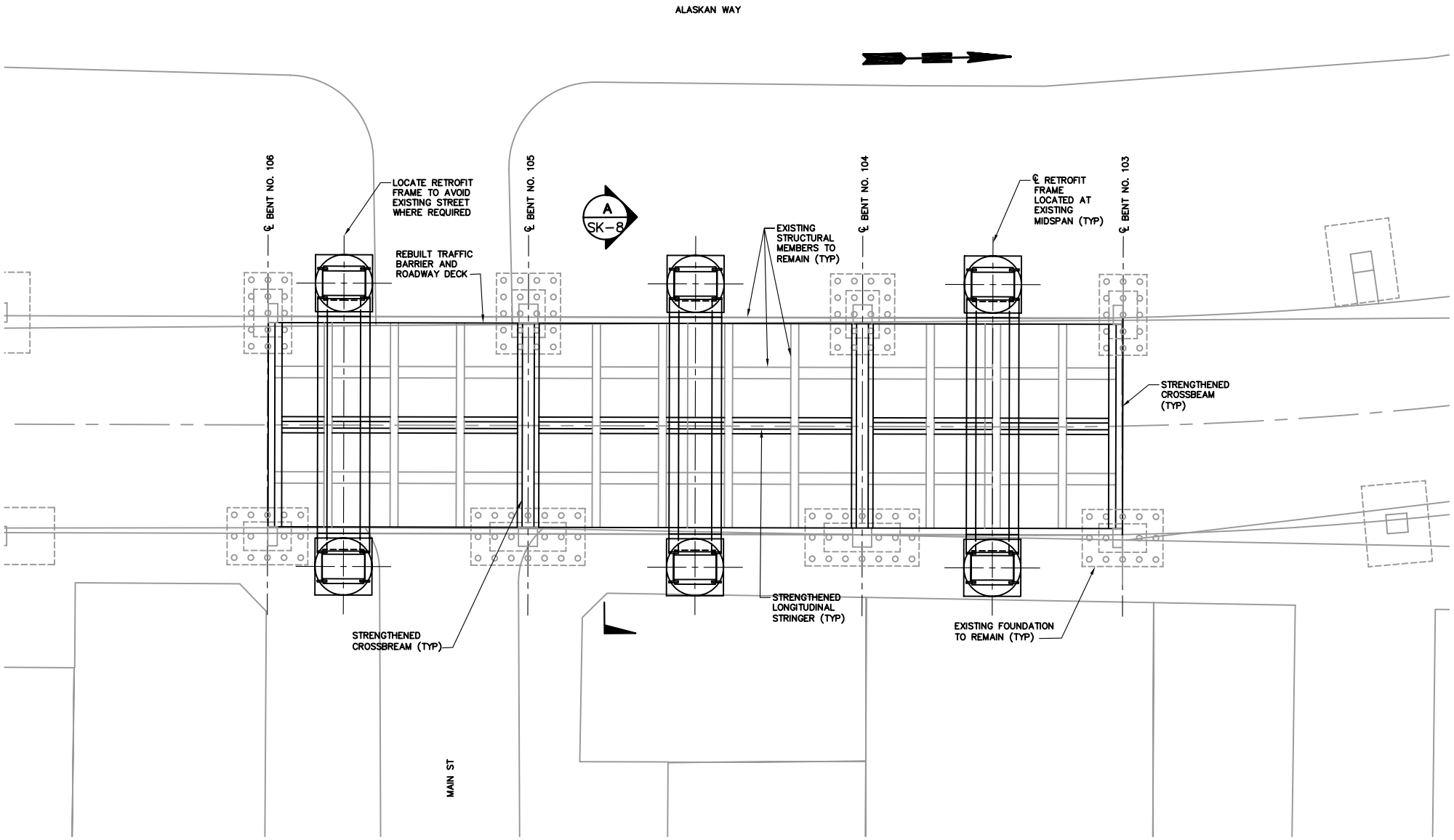
Figure 13

REBUILD/RETROFIT 500
REBUILD 500 DOUBLE-LEVEL SECTION

SHEET NO SK-6

SHEET OF SHEETS



NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Project Engr.	R. ELLIS						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor	K. WINTERSTEIN						10	WASH			
Conceptual Design Engr.	M. RIGSBY										
Design By	G. CLARK						JOB NUMBER Y-7888				
							CONTRACT NO.				
		DATE	REVISION	BY	APP'D	DATE					

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION

Washington State Department of Transportation

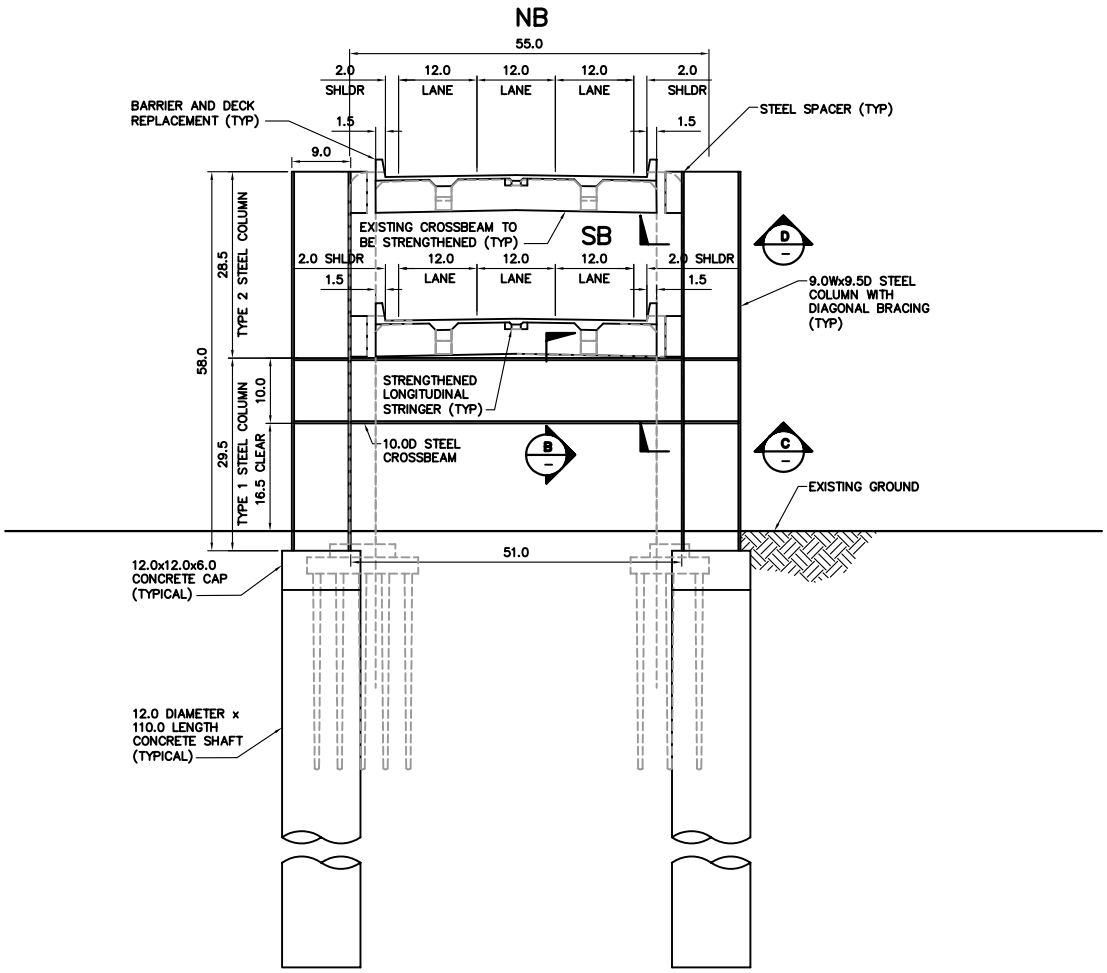
PB PARSONS BRINCKERHOFF

Figure 14

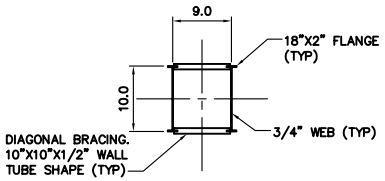
REBUILD/RETROFIT 500
RETROFIT 500 DOUBLE-LEVEL PLAN

SHEET NO SK-7

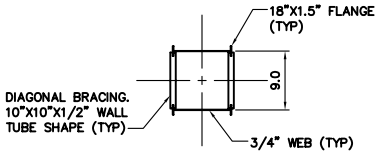
SHEET OF SHEETS



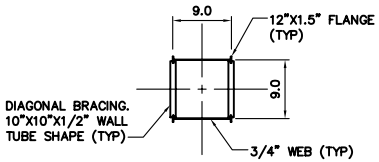
SECTION A SK-7
SCALE: 1" = 20'



SECTION B
SCALE: 1" = 20'



SECTION C
SCALE: 1" = 20'
TYPE 1 STEEL COLUMN



SECTION D
SCALE: 1" = 20'
TYPE 2 STEEL COLUMN

NOTE:
ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.

Design Project Engr.	R. ELLIS						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor	K. WINTERSTEIN						10	WASH			
Conceptual Design Engr.	M. RIGSBY										
Design By	G. CLARK						JOB NUMBER Y-7888				
							CONTRACT NO.				
		DATE	REVISION	BY	APP'D	DATE					

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION

Washington State Department of Transportation

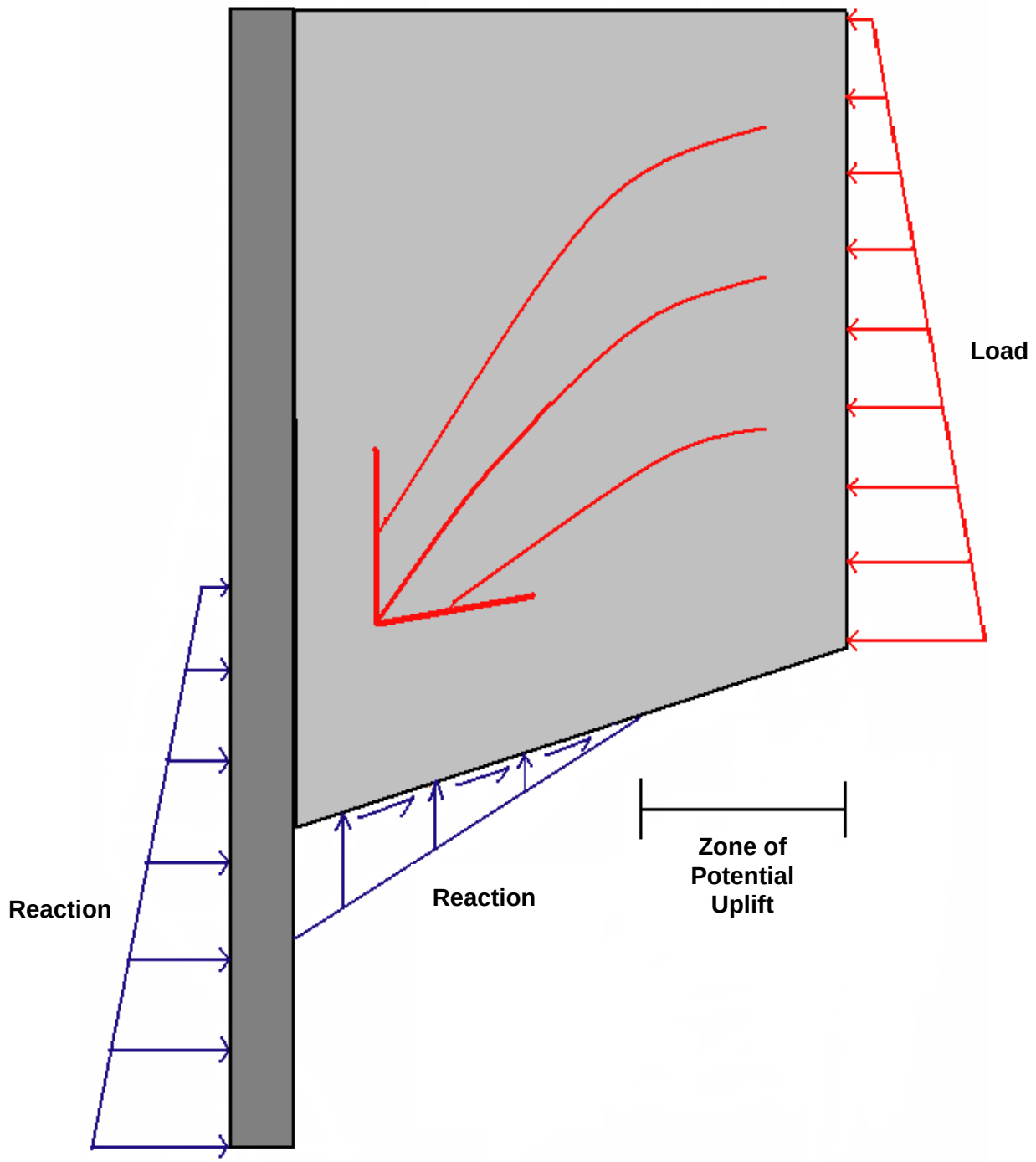
PARSONS BRINCKERHOFF

Figure 15

REBUILD/RETROFIT 500
RETROFIT 500 DOUBLE-LEVEL SECTIONS

SHEET NO SK-8

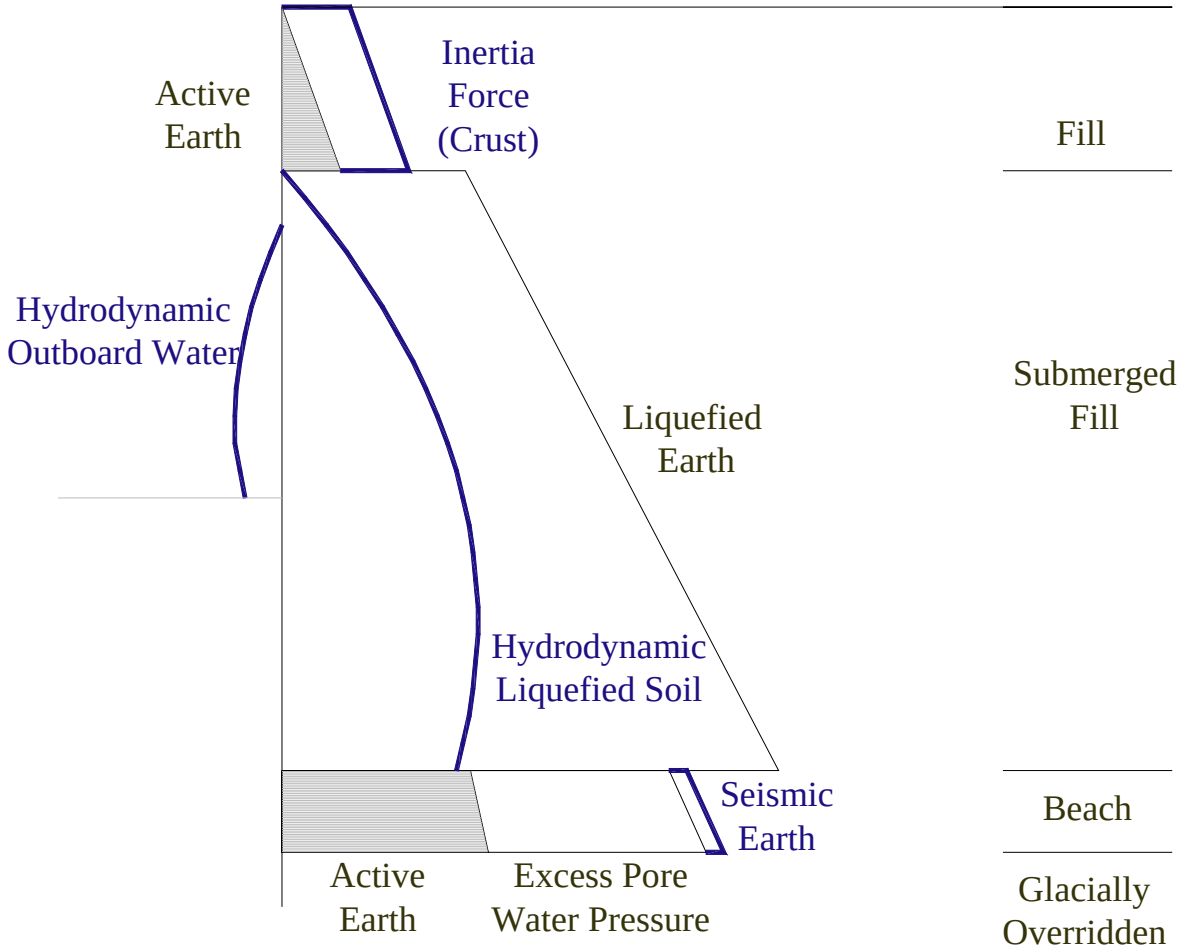
SHEET OF SHEETS



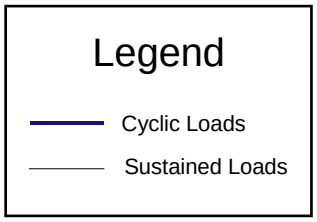
Load Reactions on Seawall

Figure 16

Rebuild/Retrofit 500



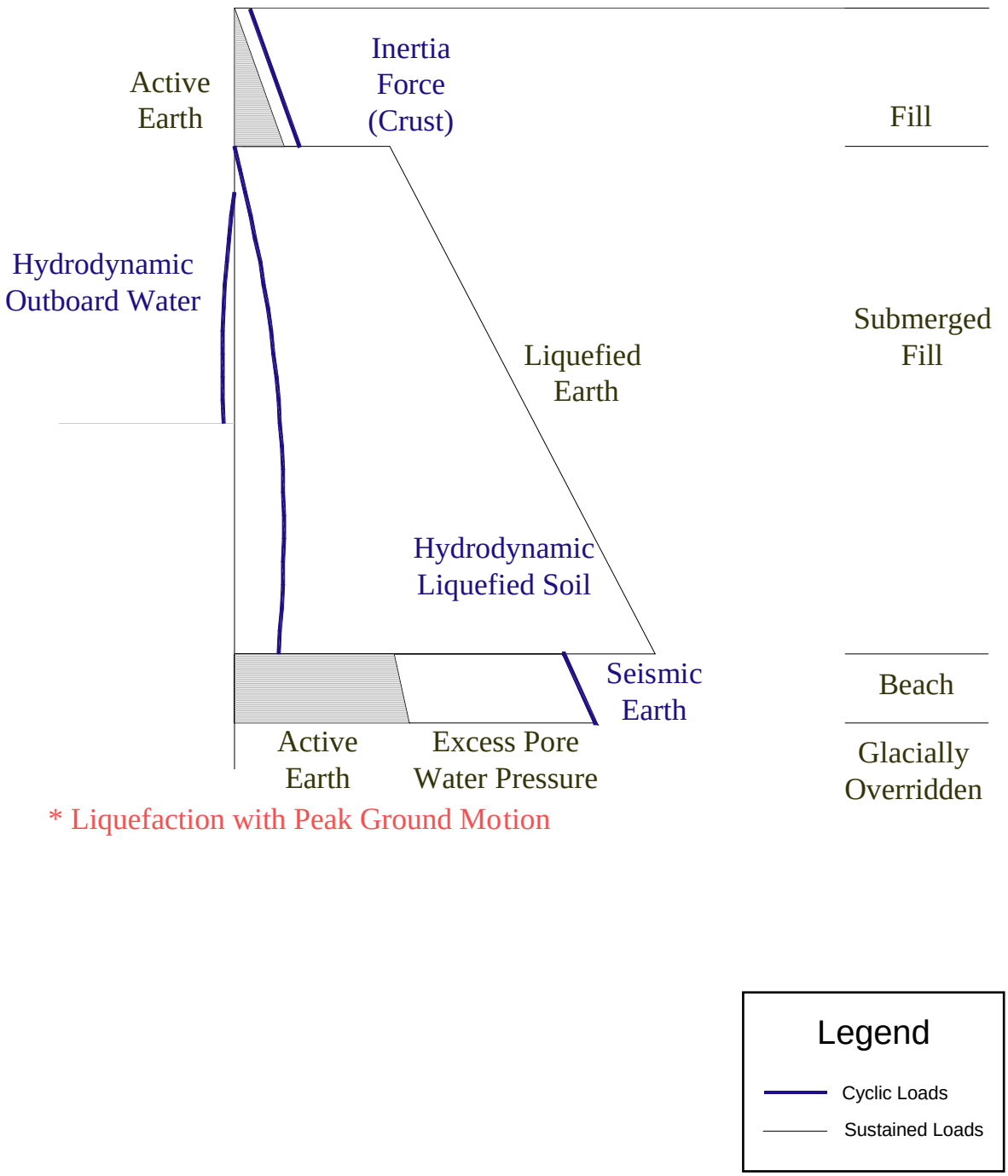
* Liquefaction with Peak Ground Motion



2500-Year Earthquake
Soil Loads on Seawall

Figure 17

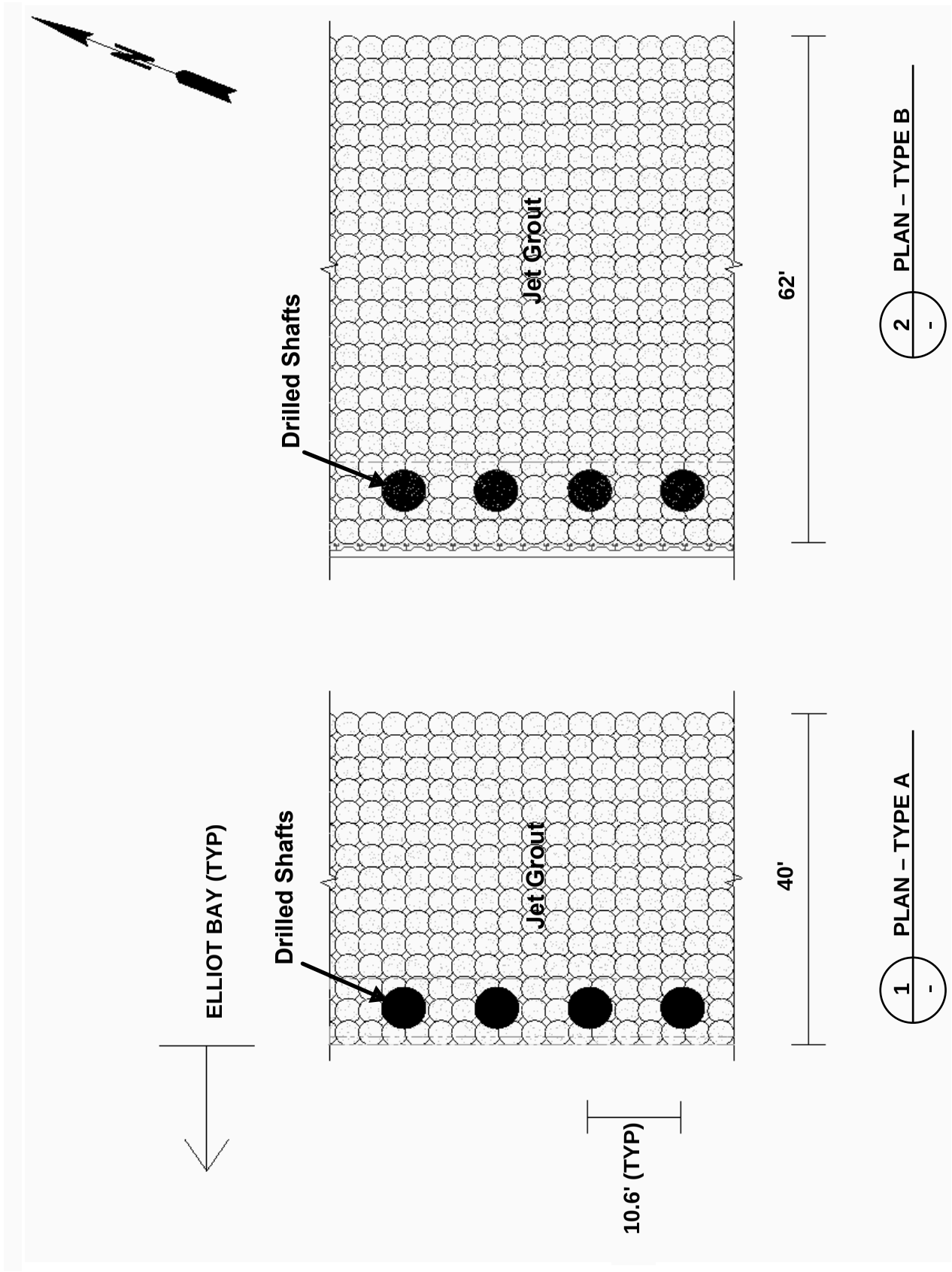
Rebuild/Retrofit 500



500-Year Earthquake
Soil Loads on Seawall

Figure 18

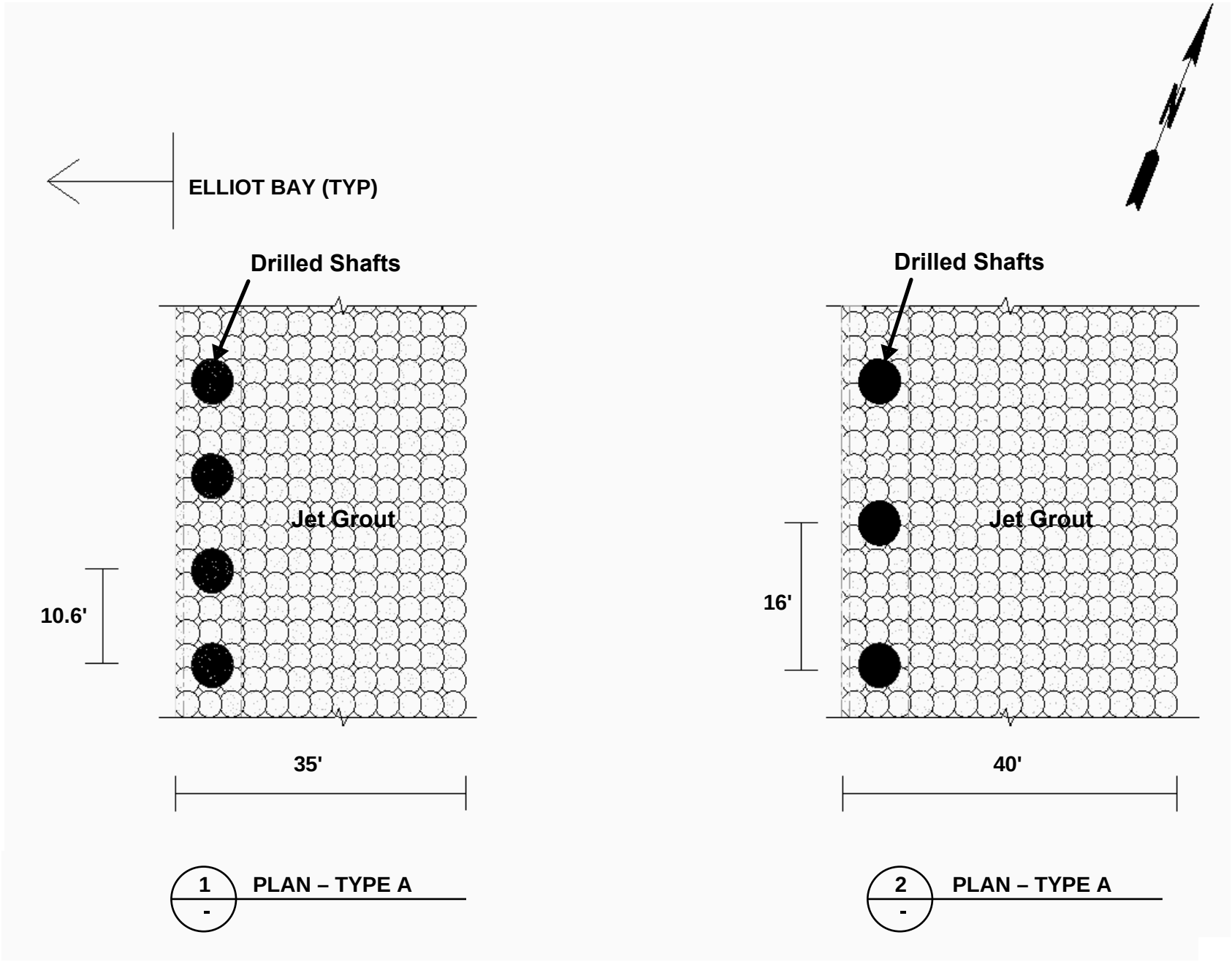
Rebuild/Retrofit 500



2500-Year Earthquake
Seawall Rebuild Concepts

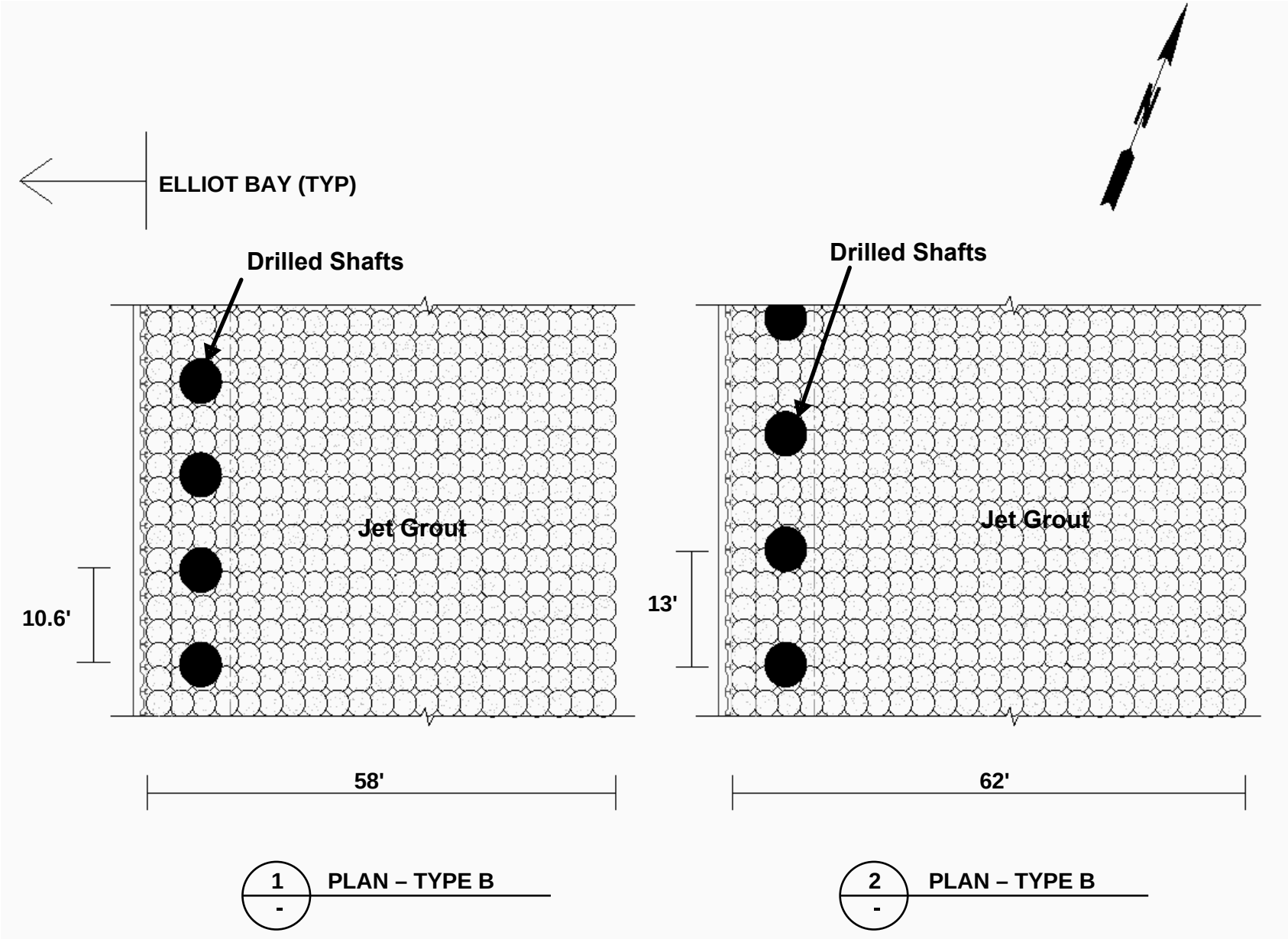
Figure 19

Rebuild/Retrofit 500



500-Year Earthquake
Type A Seawall Rebuild Concepts

Figure 20
Rebuild/Retrofit 500

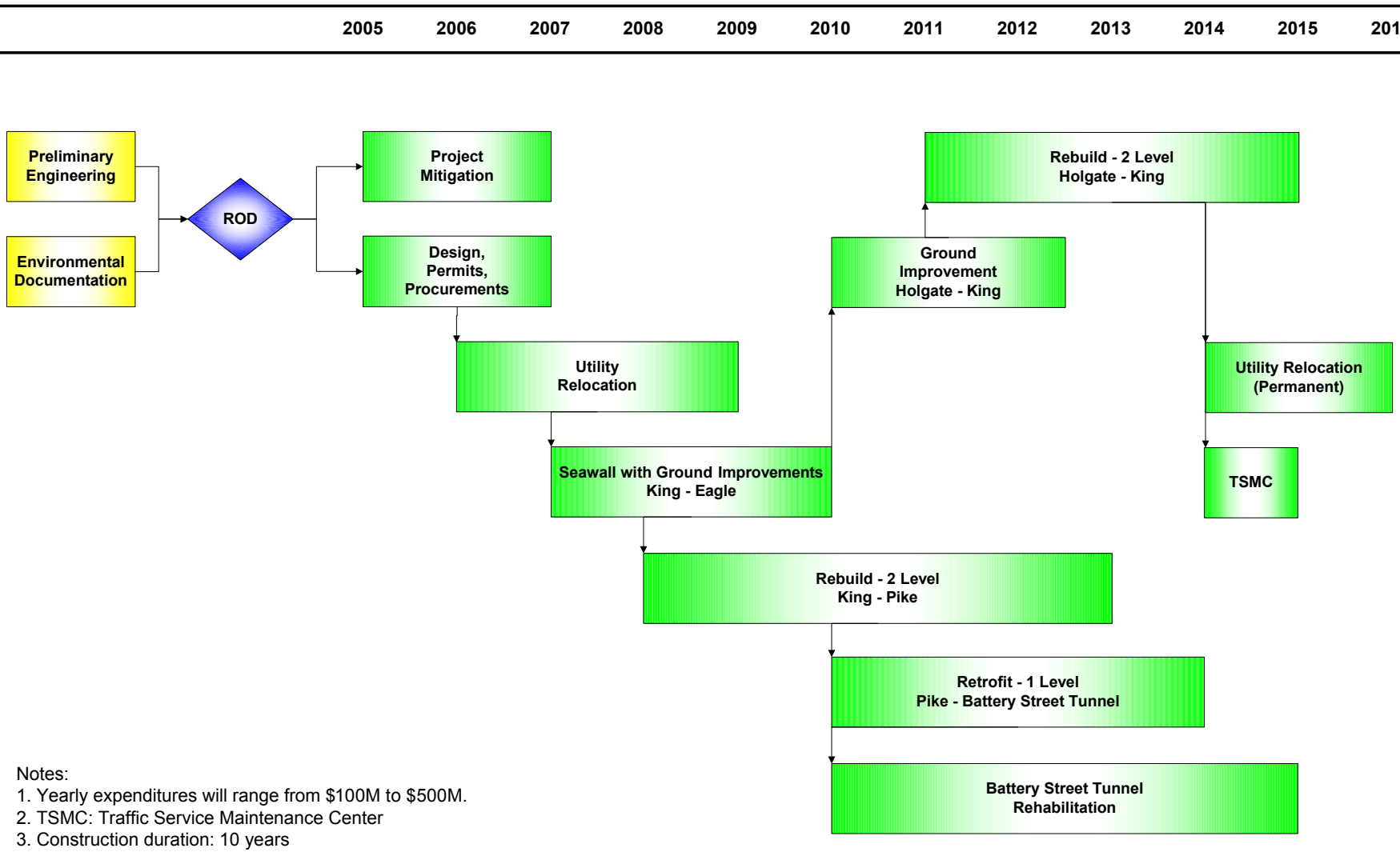


500-Year Earthquake
Type B Seawall Rebuild Concepts

Figure 21
Rebuild/Retrofit 500

SR 99 Alaskan Way Viaduct & Seawall Replacement Project
Construction Sequence Flowchart

Rebuild 2500 Plan with Seawall
Full-Funding Scenario



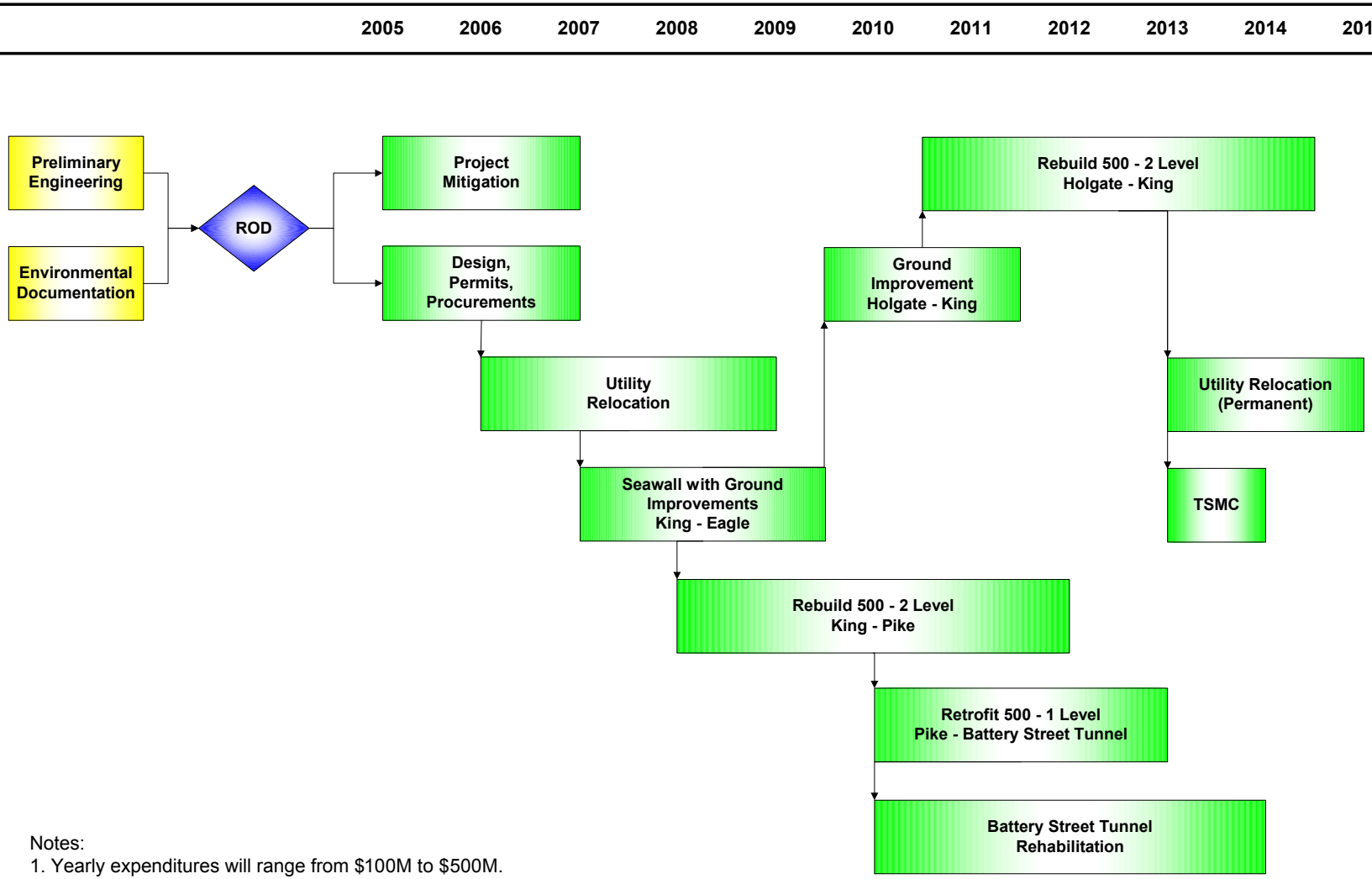
Notes:
1. Yearly expenditures will range from $100M to $500M.
2. TSMC: Traffic Service Maintenance Center
3. Construction duration: 10 years

Construction Sequence Flowchart: Rebuild 2500 Plan with Seawall, Full Funding Scenario

Figure 22

SR 99 Alaskan Way Viaduct & Seawall Replacement Project
Construction Sequence Flowchart

Rebuild 500 Plan with Seawall
Full-Funding Scenario



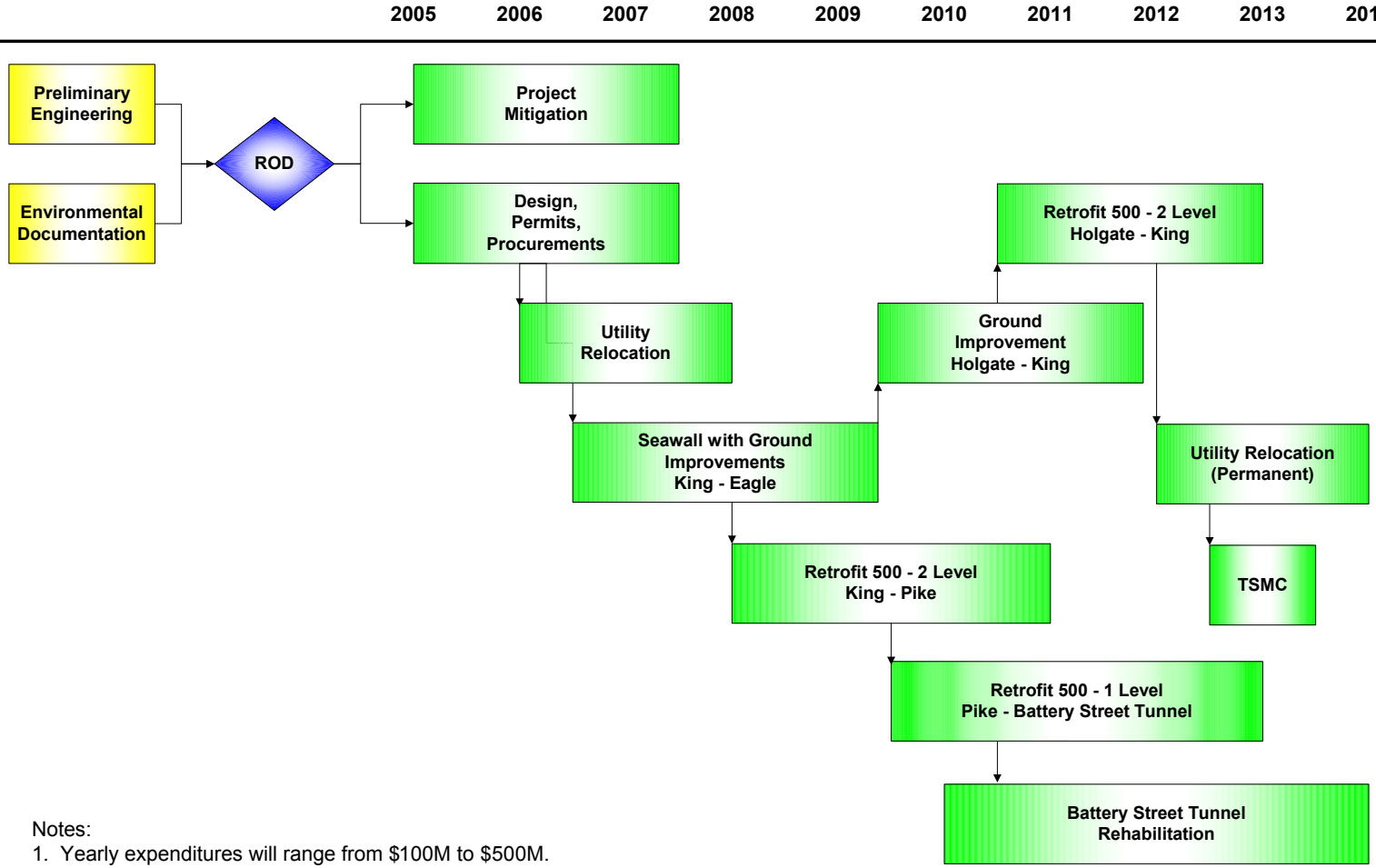
Notes:
1. Yearly expenditures will range from $100M to $500M.
2. TSMC: Traffic Service Maintenance Center
3. Construction duration: 9 years

Construction Sequence Flowchart: Rebuild 500 Plan with Seawall, Full Funding Scenario

Figure 23

SR 99 Alaskan Way Viaduct & Seawall Replacement Project
Construction Sequence Flowchart

Retrofit 500 Plan with Seawall
Full-Funding Scenario



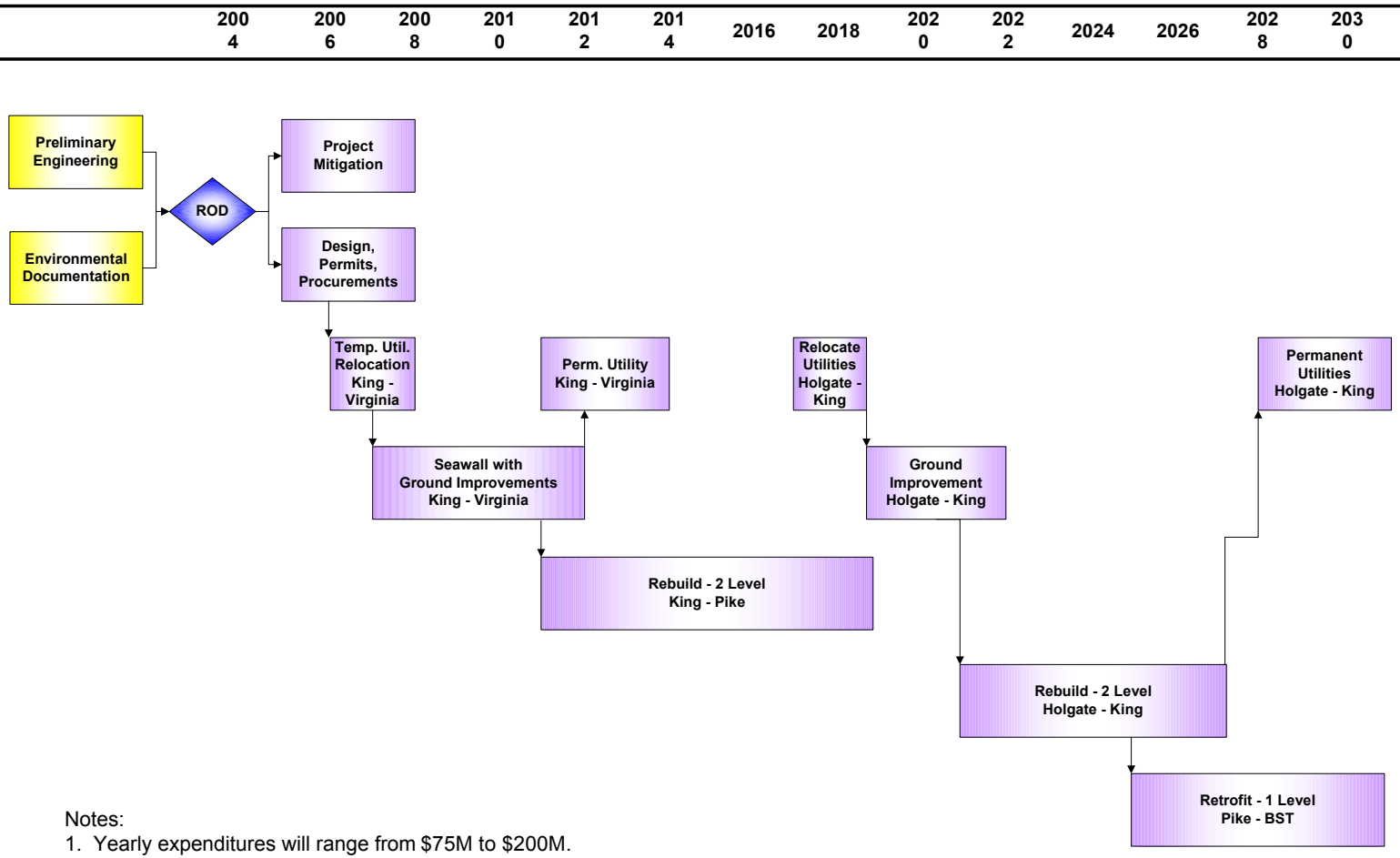
Notes:
1. Yearly expenditures will range from $100M to $500M.
2. TSMC: Traffic Service Maintenance Center
3. Construction duration: 8 years

Construction Sequence Flowchart: Retrofit 500 Plan with Seawall, Full Funding Scenario

Figure 24

SR 99 Alaskan Way Viaduct & Seawall Replacement Project
Construction Sequence Flowchart

Rebuild 2500 Plan with Partial Seawall
Limited Funding Scenario



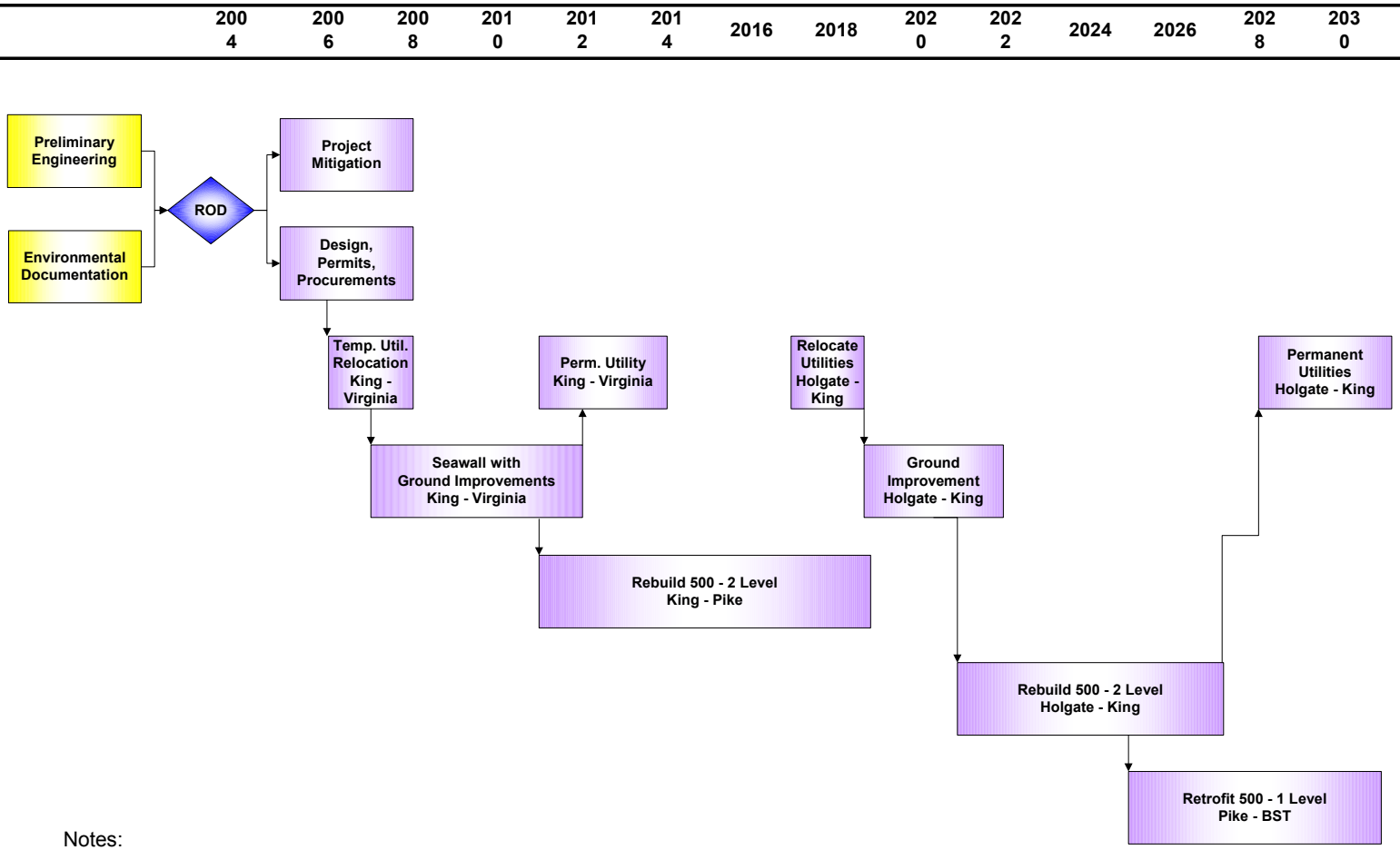
Notes:
1. Yearly expenditures will range from $75M to $200M.
2. Not scheduled: Seawall, Virginia to Eagle; Battery Street Tunnel rehabilitation; Traffic Service Maintenance Center
3. Construction duration: 25 years

Construction Sequence Flowchart: Rebuild 2500 Plan with Partial Seawall, Limited Funding Scenario

Figure 25

SR 99 Alaskan Way Viaduct & Seawall Replacement Project
Construction Sequence Flowchart

Rebuild 500 Plan with Partial Seawall
Limited Funding Scenario



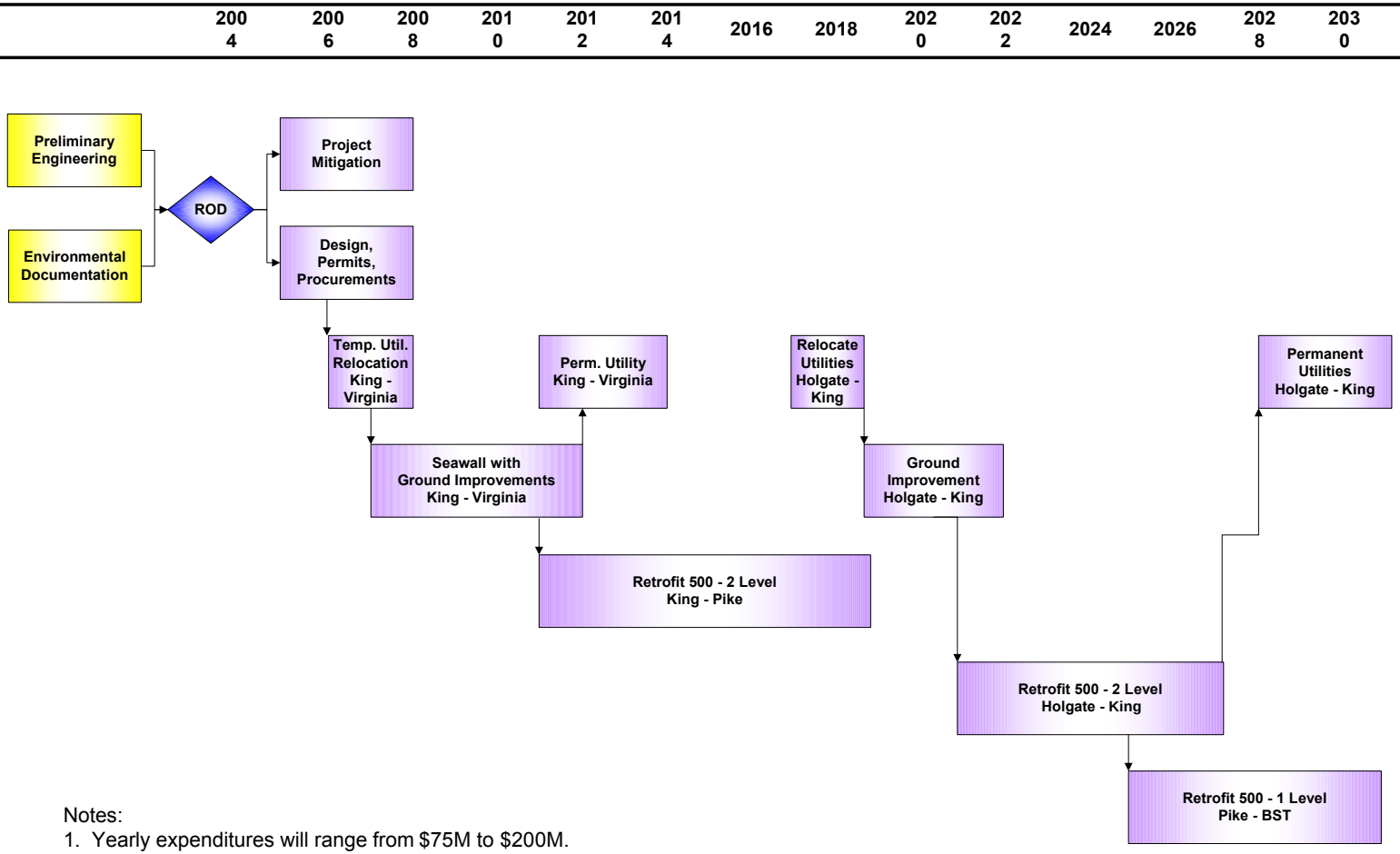
Notes:
1. Yearly expenditures will range from $75M to $200M.
2. Not scheduled: Seawall, Virginia to Eagle; Battery Street Tunnel rehabilitation; Traffic Service Maintenance Center
3. Construction duration: 25 years

Construction Sequence Flowchart: Rebuild 500 Plan with Partial Seawall, Limited Funding Scenario

Figure 26

SR 99 Alaskan Way Viaduct & Seawall Replacement Project
Construction Sequence Flowchart

Retrofit 500 Plan with Partial Seawall
Limited Funding Scenario



Notes:
1. Yearly expenditures will range from $75M to $200M.
2. Not scheduled: Seawall, Virginia to Eagle; Battery Street Tunnel rehabilitation; Traffic Service Maintenance Center
3. Construction duration: 25 years

Construction Sequence Flowchart: Retrofit 500 Plan with Partial Seawall, Limited Funding Scenario

Figure 27