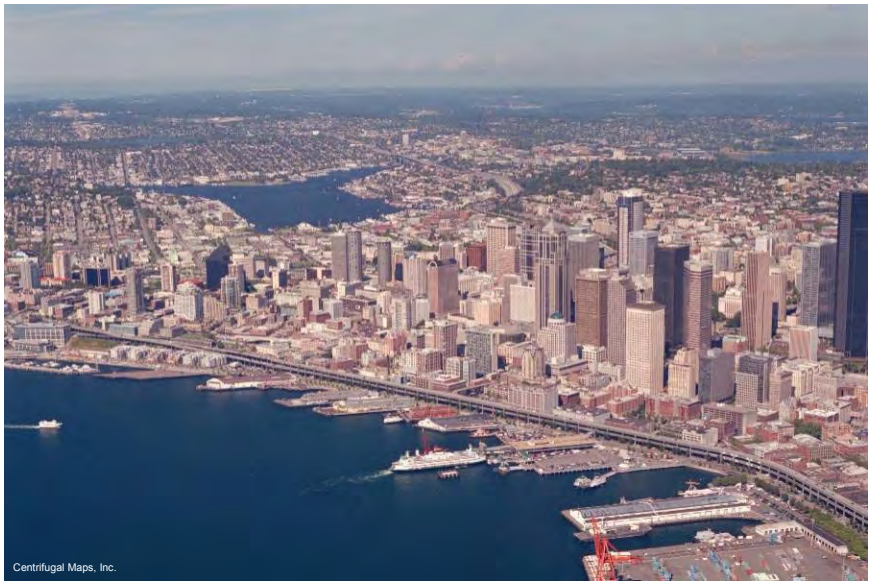
SR 99: ALASKAN WAY VIADUCT PROJECT

Rebuild/Retrofit Alternative Report



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SR 99: Alaskan Way Viaduct Project

Rebuild/Retrofit Alternative Report

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Rebuild/Retrofit Alternative Report

EXECUTIVE SUMMARY

The purpose of this report is to document the rebuild/retrofit conceptual design of the existing Alaskan Way Viaduct, Battery Street Tunnel, and Alaskan Way Seawall. The project plan, including the project definition and a brief description of seismic rebuild/retrofit methodology is presented. The rebuild/retrofit approach includes the development of concepts that are applicable to specific existing seismic vulnerabilities in the Viaduct. The rebuild/retrofit concepts are then applied to the different structures that make up the Viaduct. Finally, construction guidelines and staging concepts are proposed for a rebuild of the complex double-deck structure. Construction costs for the rebuild/retrofit are covered in a separate report, "Draft Conceptual Opinions of Cost" (Parsons Brinckerhoff, August 2002). The Rebuild/Retrofit Alternative is planned to be carried forward in the Environmental Impact Statement as the Rebuild Plan.

Under the Rebuild/Retrofit Alternative, current traffic operations would be maintained upon completion of construction. A seismic rebuild/retrofit design criteria for the existing Viaduct and seawall structures would meet the newest seismic standards anticipated to be adopted by the Washington State Department of Transportation (WSDOT) in 2003. The current load restrictions on the Viaduct, put into place after the Nisqually Earthquake, would be removed. Limited safety improvements would provide an "essential public safety upgrade". The structures would be designed to the same seismic standard as the other build alternatives for the Alaskan Way Viaduct and Seawall Project. Providing improved operational benefits is not the objective of the Rebuild/Retrofit Alternative.

In the development of seismic rebuild/retrofit concepts, the vulnerabilities of the structures that make up the Alaskan Way Viaduct facility were identified. The concepts developed would overcome the structural vulnerabilities while also making some improvements to the physical and operational constraints of the facility.

The Alaskan Way Viaduct is a complex facility composed of different types of structures. Due to the age of the structure, the soil conditions, the number of structure types, and the complexity of the structural configurations, a large number of different seismic vulnerabilities are present. The different seismic vulnerabilities have to be addressed through different rebuild or retrofit concepts to provide consistent and cost effective solutions. Each structure type is evaluated to determine if a rebuild or a retrofit is the preferred concept to best

meet the project design criteria. The complete rebuild/retrofit solution would be constructed while maintaining a minimum of two lanes of traffic in each direction on SR 99 through most of the construction duration.

In many Viaduct locations, the seismic design leads to the complete replacement of the major structural elements. There also is degradation of the concrete and reinforcement throughout the structure. In addition, many superstructure members are substandard for highway loading and need to be strengthened or replaced. Further, the bridge railings are deficient and should be replaced throughout for safety. This requires a large amount of deck to be replaced to provide the strength necessary to properly anchor the new railings for the current traffic impact loading. A rebuild would solve the seismic vulnerabilities, correct the load rating and degradation deficiencies, address the railing/deck deficiencies, and improve some known operational deficiencies.

There are sections of the Viaduct where a seismic retrofit in combination with strengthening of the superstructure for highway loading and bridge railing/deck replacement is sufficient to meet the project design criteria. In these locations, a retrofit approach is proposed. This approach is applicable to about 20% of the structure at the northern end of the Viaduct.

Retrofit of the Battery Street Tunnel would be a fire/life safety upgrade only, not an operational or seismic upgrade. A seismic vulnerability analysis has not been undertaken for the Battery Street Tunnel, and is recommended in order to determine if any seismic deficiencies exist in the tunnel. Life safety improvements are planned for the tunnel to upgrade the following: ventilation system, fire detection and suppression system, electrical system, general and emergency lighting systems, tunnel communication and traffic systems, and emergency egress. It is recommended that WSDOT adopt rules and regulations that apply to the transportation of hazardous materials in the Battery Street Tunnel. The mechanical and electrical systems as they currently exist are substandard for the operation of unrestricted traffic including the transportation of hazardous materials in the tunnel.

The Alaskan Way Seawall is approximately 8,000 feet long from Washington Street to Eagle Street. Studies and site investigations have verified that the structural condition of the seawall is poor and the existing structures do not meet current earthquake design standards. In the Rebuild Plan, the seawall would be rebuilt with a concrete secant pile wall. Jet grouting would stabilize the liquefiable soil behind the existing wall.

Additional information on the structural concept design of the Viaduct is provided in the appendices, where the following subjects are addressed: analysis methods, base isolation, double-deck beam-column connection analysis, and typical double-deck frame analysis.

DESCRIPTION OF REBUILD/RETROFIT PROCESS

The Viaduct rebuild/retrofit alternative would include all 183 bents of the mainline aerial structure as well as the associated ramps at Western, Elliott, Seneca, Columbia, and 1st Avenue South. Both the retrofit of the Battery Street Tunnel for life safety purposes and rebuild of the Alaskan Way Seawall are also included. The methodology used for developing the rebuild/retrofit approach will be described below, including the definitions of rebuild/retrofit, listing of seismic vulnerabilities, development of rebuild and retrofit concepts including base isolation, double-deck construction requirements and analysis methods.

Rebuild/Retrofit Definition

The definition used to distinguish the rebuild/retrofit from other alternatives is that current traffic operations are to be maintained. Under this definition, the rebuild/retrofit of the Viaduct would not be designed to provide improved operational benefits. Some spot safety improvements would occur as a result of the rebuild/retrofit to form an "essential public safety upgrade". The Alaskan Way Viaduct and Seawall structures would be designed to the same seismic standard as the aerial and tunnel plans. These standards would be the new seismic standards anticipated to be adopted by WSDOT in 2003.

Retrofit of the Battery Street Tunnel would be a fire/life safety upgrade only, not an operational or seismic upgrade. A seismic vulnerability analysis has not been done on the Battery Street Tunnel. Although tunnels historically perform well in seismic events, it is recommended that a detailed seismic analysis be preformed.

Seismic Rebuild/Retrofit Methodology

The purpose of the seismic rebuild/retrofit would be to bring the existing structure to current structural design standards. The project seismic design criteria uses the proposed AASHTO Guidelines based on the National Cooperative Highway Research Program (NCHRP) Project 12-49, FY98, "Comprehensive Specification for the Seismic Design of Bridges". The State of Washington Department of Transportation (WSDOT) has approved these criteria for the project.

This design standard recognizes two levels of earthquakes. The low level earthquake is termed the "Expected Earthquake". It has a probability of exceedence of 50% in 75 years and is represented by an acceleration response spectrum (ARS) with a peak ground acceleration (PGA) of 0.13 g and a peak spectral acceleration of 0.33 g. Peak ground acceleration or Zero Period Acceleration (ZPA) is the maximum acceleration anticipated at the ground surface during a seismic event. Peak spectral acceleration is the maximum acceleration anticipated to be experienced in a structure. Peak acceleration is

amplified by the response of a structure and is often 40% to 60% greater than the PGA.

The major seismic event is termed the "rare earthquake" or "maximum considered earthquake" (MCE). It has a probability of exceedence of 3% in 75 years and has an ARS with a PGA of 0.65 g and a peak spectral acceleration of 1.61 g. This seismic event considers a major movement of the Seattle Fault. Since the Seattle Fault is very near to the project site, the ARS has been adjusted to represent specific near fault ground motions.

There are different Life Safety Performance Levels for each of the two design earthquakes. In the event that a major earthquake strikes the Viaduct, there would be no collapse of the structure but partial or complete replacement may be required. According to these design criteria, significant damage to the structure is acceptable and is likely to occur during a major seismic event.

The expected earthquake event is the more probable seismic event and would be much lower in magnitude. According to the criteria, the structure would not sustain any major damage from this event and would be open for immediate service following an inspection of the structure.

The Viaduct has been divided into similar structural configuration type sub-segments. The design methodology considers each sub-segment of the facility individually. The seismic vulnerabilities as well as the inherent strengths and stiffness of the structure are evaluated. Also considered is the relative stiffness of adjacent columns, bents, and frames, the local soil conditions and the right of way restrictions. Concepts developed meet the design criteria required for the "Life Safety Performance Level".

Included in the structure types are some tall but weak and brittle columns. Adjacent segments have short, stiff, and brittle columns. In many cases, the pier caps are weaker than the columns. Many different concepts are needed for the different situations and a single solution for the rebuild/retrofit would not achieve the goals of the project.

The objective for seismic design is to develop concepts that would allow adequate deformations to occur in the structure that would meet the actual ground motions. While it is common to use a force notation, usually a percentage of gravity loads, to compare lateral load capacity, the displacements tolerated by the structure are the keys to modern seismic design. For example, although values are used, such as 0.13 g capacity or a demand of 1.61 g, they are used because this was the only method of reference understood for many years. The more important values would be the few inches of elastic displacement, the additional inches of post yield deformations, and the corresponding displacements caused by a specific set of ground motions acting in a given direction.

A mathematical model of the structure is generated and tested for capacity. In this analysis, the structure is analytically pushed to test and verify the lateral capacities. This type of analysis is called a non-linear ultimate displacement (pushover) analysis. In the pushover analysis, the goal is to determine the initial structural failure modes due to lateral movements similar to what would occur in an earthquake. This analysis is described in more detail in Appendix A.

Vulnerabilities

One of the initial tasks in designing a seismic retrofit for a complex bridge is to define the seismic vulnerabilities present in the specific structure. Generally, the vulnerabilities are determined from the actual construction details of the structure. These details are best documented in the as-built plans. There are additional important sources of information about vulnerabilities such as site inspection, reviews of maintenance records and conversations with the owner.

The seismic vulnerabilities were presented as part of the assessment of the existing conditions in the No Action Alternative Report (August 2002). It is likely that the existing Viaduct can experience the following modes of failure from seismic events:

- Lateral spreading, where the soil surrounding the foundations would move towards Elliott Bay, could cause the structure to collapse.
- Liquefaction of the soils adjacent to the Viaduct foundations could cause collapse of the structure.
- Loss of the waterfront seawall could cause massive slope instability leading to collapse of the Viaduct, other waterfront structures, and disruption to major waterfront utilities.
- Brittle cracking in the Viaduct columns and beams could cause catastrophic damage in the structure and lead to collapse.
- Lateral racking of the Viaduct frames could lead to progressive failure and ultimate collapse.

Development of Rebuild/Retrofit Concepts

After the seismic vulnerabilities of each of the different types of structures are determined, seismic rebuild/retrofit concepts are developed to overcome the vulnerabilities. There are generally quite a number of seismic vulnerabilities found in every structure configuration that was designed and built during the period when the Alaskan Way Viaduct was constructed. It normally takes many individual rebuild/retrofit concepts working together to effectively design a complete seismic upgrade for a complex structure like the Viaduct. Further, all the rebuild/retrofit concepts to be included must be able to be constructed within the local physical and operational constraints.

A pushover analysis provides information to design a retrofit concept to overcome the local failures. It identifies areas where both additional ductility and strength are needed.

For double-deck type structures such as those that make up the Viaduct, there are a limited number of rebuild/retrofit concepts that are effectively used. Following the collapse of the Cypress Street Viaduct on Interstate Route 880 in Oakland, hundreds of bridge and structural engineers were involved in a program to develop a sound and constructible retrofit for the remaining double-deck Viaducts. This program included extensive peer reviews by notable and experienced engineers and professors, followed by large-scale laboratory testing of the approved solutions before starting construction.

During the double-deck research process, it became apparent that the key seismic vulnerability in the 1950 – 1960 era double-deck structure is the lower level pier cap to column joint. Failure of this joint allows the upper level structure to fail suddenly in a very brittle and catastrophic manner. Many multistory concrete buildings have experienced this failure. Unfortunately, there is no method to properly reinforce the joint except to replace the joint with a properly detailed connection. In order to construct this joint the entire structure including both decks must be supported with temporary supports before demolition can begin. Once the structure is placed on the temporary supports, it is possible to replace the substandard columns.

In order to maintain the integrity of the traffic supporting decks during a seismic event, weak column/strong beam behavior is preferred. The selection of weak column/strong beam behavior for bridges is due to the typical detailing found in most normal single deck bridges combined with the number of available plastic hinges that can form while the structure remains stable. In the weak column/strong beam method, the girders and pier caps are designed and detailed to be stronger than the maximum force that the weaker columns can possibly develop. Large scale testing on bridges has been performed to prove the weak column/strong beam solution works, so the concepts presented here are based on weak column/strong beam behavior. To force the columns to fail before the beams or deck, the pier caps have to be strengthened substantially and new longitudinal framing members added to the structural system at the lower level.

Constructing a complete seismic rebuild/retrofit of a double-deck Viaduct with the full substructure replacement is considered difficult, especially when traffic on the structure is maintained. With this in mind, construction guidelines are recommended to limit the risk to the public, based upon similar previous experience. See the report section "Rebuild/Retrofit Construction Guidelines for Double-Deck Structures".

Base isolation was rejected as a rebuild/retrofit concept that could practically be applied to all structure types of the Viaduct and be designed to comply with the project's seismic design criteria for the existing complex structure. It requires large movement capability in the structure. The structure is made up of many types of three-span units that are only two inches apart and does not allow for freedom of movement in the magnitude and directions required in a design earthquake. Different units would collide against and damage each other. The units would also pound against the on- and off-ramps. The base isolation approach is to be considered for application in specific, limited locations. A description of the base isolation approach to seismic retrofitting is included in Appendix B.

A panel of the American Society of Civil Engineers (ASCE) independently reviewed the base isolation retrofit concept and concluded that it is not suitable for the Alaskan Way Viaduct. Refer to the ASCE Expert Team, "Alaskan Way Viaduct Phase 1 – Retrofit Option" report dated April 24, 2002.

There is lateral spreading potential during a seismic event in the area of the Alaskan Way Viaduct south of the rebuilt seawall. In this location, soil improvement is recommended at each of the Viaduct foundations. For additional information, refer to the memorandum "Evaluation of the Top of Glacially Overridden Soils and Thickness of Liquefiable Soils, Alaskan Way Viaduct Corridor, Seattle, Washington," (Shannon & Wilson, June 21, 2002).

Refinements and Level of Effort

The engineering performed at this stage of the process is conceptual and serves to support the Environmental Impact Statement. Refinement in engineering design is not included in the scope of this study. The selected concepts are developed to provide a basis for cost estimating and determining impacts on the surrounding environment. With this in mind, the concepts have to perform seismically; have to be constructible while maintaining traffic; and have to provide enough quantities to derive a reasonable cost estimate for this level of design. A refined design would be developed for the selected preferred alternative.

Previous Studies

A number of reports have been published on the seismic vulnerability and capacity of the Alaskan Way Viaduct. Those reports completed before July 2001 are referenced and highlighted in the "Report of the Structural Sufficiency Review Committee" (T.Y. Lin International, June 2001). The Committee's report summarized previous studies of the Viaduct, evaluated typical structural units for seismic vulnerability, and provided cost data for retrofitting as well as replacing the double-deck Viaduct.

The Structural Sufficiency Review Committee Report also has provided information about the methods for retrofitting the Viaduct.

Following the Structural Sufficiency Review Committee's report, there are published reports concerning the following:

- Alaskan Way Viaduct Rebar Weld Joint Analysis, Dwight Company Inc., April 25, 2002.
- Load Rating of the Alaskan Way Viaduct (completed in 2001 by CES, Inc.) was modified based on the actual material properties and reported by the WSDOT Bridge Preservation Office, April 2002.
- Alaskan Way Viaduct In-Depth Inspection Report, Washington State Department of Transportation, Bridge Preservation Office, July 23, 2002.
- Battery Street Tunnel Structural Condition Survey, Parsons Brinckerhoff Quade & Douglas, Inc, March 2002.
- Alaskan Way Viaduct No Action Alternative Report, Parsons Brinckerhoff, August 2002.
- In-Depth Mechanical and Electrical Inspection and Recommendations, Parsons Brinckerhoff, March 2002.
- Alaskan Way Seawall Design Report, Berger/ABAM, July 2002.
- Alaskan Way Seawall Wave Study, Berger/ABAM in association with Pacific International Engineering, September 2002.

Another important reference is the report of the peer review of the San Francisco Double-Deck Seismic Retrofit Project, December 1993. The Peer Review Committee was established as a result of the Governor of California 's Inquest of the Loma Prieta Earthquake. This report documents many of the attempted solutions to the design flaws of the double-deck viaducts constructed in San Francisco and Oakland during the 1960s. The San Francisco double-deck viaducts were modeled after the Alaskan Way Viaduct with some technological modifications so the research in the early 1990s has many direct applications to the Alaskan Way Viaduct.

DESCRIPTION OF REBUILD/RETROFIT PLAN

The entire Alaskan Way Viaduct, Seawall, and Battery Street Tunnel require seismic or other life safety rebuilds, retrofits, and upgrades. The proposed concepts are described below.

As mentioned in the No Action Alternative report, the Viaduct bridge railing does not meet safety standards and would have to be upgraded in the rebuild/retrofit plan. The upgrade would also require major deck and girder modifications to meet current bridge design specifications. This upgrade for both the northbound and southbound decks, as well as the ramps, will have to be considered with the appropriate traffic staging during the rebuild/retrofit construction.

Spokane Street to South Abutments

The retrofit plan does not include the Spokane Street Interchange, or the at-grade segment between the Spokane Street Interchange and the mainline Viaduct south abutments.

South Transition Structure

The south transition structure is the facility that is used to move traffic from the side-by-side at-grade configuration to the over-under configuration of the double-deck Viaduct. This sub-segment is located between two active railroad yards. There are a number of seismic vulnerabilities inherent in this structure type as well as some specific to the location. The ground below the structure as well as the structural geometry and reinforcement detailing are the primary vulnerabilities.

Seismic Vulnerabilities

The structure site is underlain by a thick layer of very soft and mostly granular saturated soils. These soils are highly susceptible to liquefaction. There is a dense layer at a depth of about 100 feet with more soft material down to the glacial till about 200 feet down. For any foundation to provide support through and after the design earthquake, the liquefaction problem would have to be mitigated. The choices are either a large, deep foundation into glacial till, or ground improvements. Lateral spreading also is a concern in this area, as described in the "Geotechnical and Environmental Memoranda" report (Shannon & Wilson, July 2002). Ground improvement is planned to mitigate the poor geotechnical site conditions.

Due to the traffic movements for the double-deck configuration, there must be outrigger pier caps to span over the transitioning lower deck. The pier caps are

normally cast monolithic with the reinforced concrete columns. Outrigger pier caps constructed in the 1950s are vulnerable to longitudinal ground movements. The longitudinal response causes severe torsion in the pier cap in addition to both vertical and horizontal shear and vertical and horizontal flexure stresses. Sufficient reinforcing is not provided to address these seismic loads and some of the mentioned defects promote brittle and catastrophic failure.

The columns have substandard confinement and shear reinforcing. The main column reinforcing splices also are substandard and the splices are poorly located. Pile caps have no top mat reinforcing to transfer overturning forces to the pile foundations. There is no tension connection between piles and pile cap. The piles have inadequate capacity for lateral loads. A rebuild of the structure in this area is proposed, because of these numerous deficiencies.

Rebuild/Retrofit Concepts

Retrofit concepts for the south transition structure are separated into ground improvements and structural upgrades. Both of these approaches are required in the conceptual design.

Ground improvements

Construction of 200 feet deep drilled shafts in soft saturated soil is difficult. For the depth and area of improvement required, the most promising ground improvement methods would be either vibroflotation with stone columns or jet grouting. There are no recent geologic borings taken at the sites of the existing foundations. The original borings were taken before the original piles were driven. The pile driving likely had an impact on the existing soil and that impact will need to be determined.

Stone columns by the vibroflotation method can improve the soil by increasing the density and minimizing liquefaction risk. However during the densification process large settlements would occur (as much as 18 inches) affecting an area approximately 40 feet in diameter. The existing Viaduct and Seattle City Light (SCL) duct banks could not tolerate that much settlement so the vibroflotation would have to be kept away from the existing foundations and the duct bank. Additionally, the transition structure is located within a working railroad yard. The tracks would have to be realigned during construction before any trains could use the adjacent tracks. New piles could be either driven or drilled into the densified soil.

Jet grouting is a method to improve the soil by injecting a cement grout under pressure and mixing it into the soil to form soilcrete columns. The soilcrete can develop compressive strengths as high as 1000 pounds per square inch. The jet grout columns would be placed side-by-side to form a volume of improved ground that would not liquefy. New piles may be drilled into the soilcrete mass.

Before undertaking a ground improvement method, a test program should be undertaken to determine the parameters that would best serve the project, including spacing of ground improvements, rate of installation, depths, mixes, settlements, heave, leakage of materials, protections to the structures and railroad tracks and the environment.

Structural Upgrades

The top of the outrigger column bent will be designed as a "pinned" connection. This concept is important so that the torsion demand on the reinforced concrete beam is reduced. However, when the top of column is released, the column capacity for bending at the top of footing is inadequate. To achieve stability, the column and lower cap beam must be replaced. When replacing the column with a stronger and stiffer column, a stronger foundation is also required. Although the torsion would be reduced by the top of column flexural release, there is still torsion from the eccentric reaction at the top of column combined with the longitudinal shear. The outrigger beam would still need major strengthening with torsion and shear reinforcing cored through the existing concrete and prestressing to resist the flexure loads. With the change in structural configuration, the eastside columns and foundations would also need strengthening. Girders and cap beams that are inadequate to carry the design traffic live loading would be strengthened or replaced. Temporary supports would be required for this construction. Staging of the work over the southbound lanes would have to be done so that work would not take place over live traffic lanes.

South Viaduct Double-Deck Structure

This structure includes the Viaduct from the end of the transition structure at approximately S. Massachusetts Street (Bent 175), to the 1st Avenue S. ramps (Bent 121). This segment was designed by WSDOT and constructed in about 1956. The key features around the Viaduct that affect potential construction are the railroads on both sides and below the Viaduct. The rail line below the Viaduct may be relocated before construction is started. There is also a traffic storage pocket located under the Viaduct from Bent 143 to Bent 125 (near the Seahawks Stadium). This ferry traffic storage would also have to be relocated. There are a number of seismic vulnerabilities inherent in this structure type as well as some specific to the location, such as lateral spreading. The condition of the ground below the structure is also vulnerable to liquefaction and lateral spreading. The vulnerability and the rebuild concepts described for the South Transition structure are applicable to this structural configuration. Other vulnerabilities and retrofit concepts are discussed below.

Seismic Vulnerabilities

The non-linear ultimate displacement (pushover) analysis indicates that the Viaduct may sustain damage at a spectral acceleration as low as 0.13 g. This level of damage may require a closure of the Viaduct for limited repair. The structure still remains stable at this point, but with additional seismic loading, the structure will experience successive and increasing damage leading to collapse. Based upon the equal energy principle applied to the calculated force deflection curve, it was found that the structural limit would be reached between approximately 0.25 g to 0.30 g. This is consistent with the findings by the Alaskan Way Viaduct Structural Sufficiency Review Committee that estimated the structural limit of the double deck frame at 0.26 g.

Rebuild/Retrofit Concepts

The rebuild/retrofit concept would eliminate the major structural drawbacks in the existing structure and bring the structure into compliance with the current structural design codes. The proposed columns are designed to meet the ductility requirements. The rebuilt structure is expected to possess high enough seismic resistance and energy dissipation capacities to sustain a life-safety performance level earthquake. The structural work planned in this concept consists of the installation of three new bents in a typical three-span frame system, or replacing the existing double-decks and the replacement of the damaged existing concrete railing. The proposed construction includes the following:

- New foundations
- New columns with a pin at the bottom of the top pier cap
- Strengthened bent cap beams at the upper deck level.
- New bent cap beams at the lower deck levels
- New longitudinal edge girders at both deck levels
- New transverse floor beams/diaphragms underneath the decks to support the vertical loads from the decks
- Replace the existing decks and bridge railings.

The relative performance of the existing and rebuilt/retrofitted structures is presented in Figure 1, below.

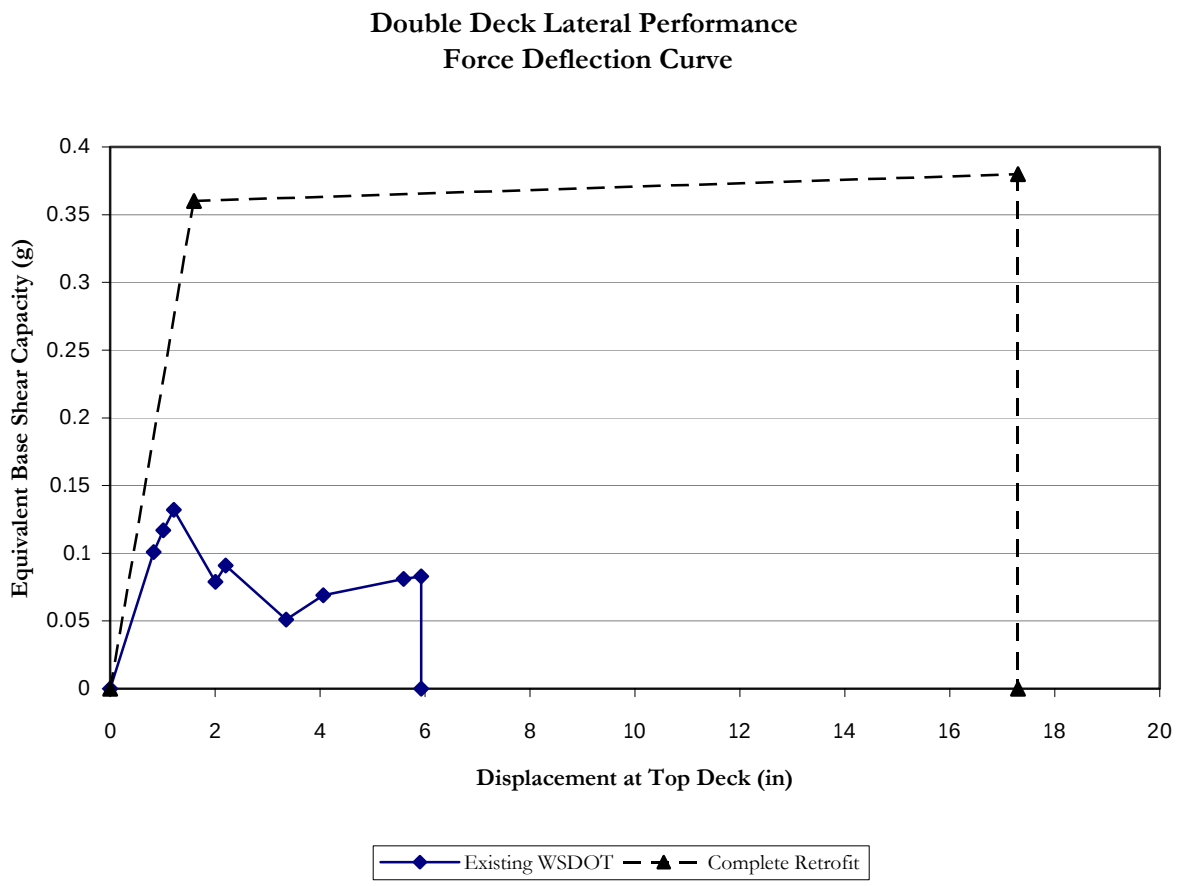

Figure 1: Performance Comparison Curves - Existing versus Retrofit/Rebuild

In Figure 1, the difference in energy dissipation of the existing versus the complete retrofit is substantial. The added deformation capacity provided by modern detailing provides toughness and durability.

Middle Viaduct Double-Deck Structure

The middle Viaduct from approximately Railroad Way S. to Union Street (Bent 121 to Bent 60) was designed by the City of Seattle Engineering Department (SED). The design was completed in about 1952, followed by construction. The structural details used for the middle Viaduct double-deck are similar to the southern Viaduct with a few exceptions that are described below. The foundation conditions are quite different from the subsurface of the southern Viaduct. The middle Viaduct is founded on strong glacial deposits. However there is a 30 to 60 foot thick, liquefiable, hydraulically placed fill layer overlaying the glacial deposits. Portions of the middle Viaduct are located very close to the historic buildings in Pioneer Square.

Seismic Vulnerabilities

One of the main differences in the older SED Viaduct is the type of reinforcing steel used. The larger bar sizes are all square bars with minimal deformations. The lack of mechanical stress transfer to the concrete causes a much lower bond stress and greatly increases the vulnerability of bond slip. Bond slip would occur when the tension or compression strains in a bar cause the bar to slip with respect to the concrete. When the bar slips, no additional load can be transferred between the bar and the concrete. Further, there is no method of "healing" the slippage. In a load reversal situation such as a long duration earthquake, this vulnerability poses serious problems. Initiation of damage to the SED designed double-deck Viaduct occurs with bond slip of the top reinforcing bars in the lower level pier cap. A non-linear ultimate displacement (pushover) analysis indicates that the Viaduct may sustain damage at a spectral acceleration as low as 0.10 g. This level of damage may require a closure of the Viaduct for limited repair. The structure still remains stable at this point, but with additional seismic loading, the structure will experience successive and increasing damage leading to collapse. Based upon the equal energy principle applied to the calculated force deflection curve, it was found that the structural limit would be reached between approximately 0.25 g to 0.30 g. This is consistent with the findings by the Alaskan Way Viaduct Structural Sufficiency Review Committee that estimated the structural limit of the double-deck frame at 0.26 g.

The soil conditions in the middle Viaduct are greatly improved over the south Viaduct. For the middle Viaduct, the piles are driven into the glacial till. The depth to till increases from north to south along the Viaduct. One added vulnerability is the seawall. The retrofit of the seawall is reported separately; see the "Alaskan Way Seawall Design Report" (Berger/ABAM, July 2002). The soft and liquefiable fill was placed on the original sloping shoreline after the existing seawall was constructed. The liquefaction of the fill could cause major problems in two ways. If the soil pressures from liquefaction cause a failure of the seawall, then the foundation material would flow toward Elliott Bay taking the Viaduct, some buildings, and utilities with it. If the seawall could restrain the liquefied soil then local liquefaction puts the existing piles at considerable risk due to loss of lateral support. The resulting local differential settlements would cause major damage to the Viaduct. The existing frame in this area suffered up to five inches of differential settlement during the Nisqually Earthquake, causing major damage.

Retrofit Concepts

The initial approach to developing a retrofit concept for this part of the structure was to start with the pushover analysis of the existing structure and provide a rebuild/retrofit for the first major failure observed. This analysis assumes that the seawall has been retrofit or replaced so that the potential for lateral spreading has been eliminated. Local liquefaction would be considered in the design of the

rebuild/retrofit foundations by selection of foundation types. The selected foundations would bear against the glacial till.

The solution to the Viaduct seismic vulnerabilities must include a complete new substructure. The requirements for the rebuild/retrofit were presented previously for the Southern Viaduct.

In the Pioneer Square area from Bent 116 to Bent 103, there is insufficient space to construct a new substructure frame outside of the existing frame on the east side of the Viaduct, thus a modified construction method would be required. Temporary supports may be used to support the structure while the substructure is demolished and replaced in the same location. During construction, two lanes of northbound traffic would be maintained on the upper deck while two lanes of traffic are on the lower deck. New pier cap bolsters can be constructed alongside the existing pier caps by coring through the concrete pier caps and installing sections of reinforcing steel that are spliced together with mechanical connecters to form closed stirrups. This construction, including the temporary supports, new foundations, new columns, bolstered pier caps, new pinned existing beam to new column upper joints, new beam to column lower joints, and new edge girders, was successfully completed on the I-280 double-deck retrofit in San Francisco with traffic on the top deck.

North Transition Structure

The north transition structure is the facility in the vicinity of Pike Street (Bent 60 to 53) that is used to move traffic from the over-under double-deck configuration to the side-by-side configuration on separate structures. This sub-segment is located in a congested area that has a building located under the Viaduct at one location. There are a number of seismic vulnerabilities inherent in this structure type as well as some specific to the location. This structure performs the same function as the south transition structure except this north transition supports both directions of traffic on a structure that is tall.

Seismic Vulnerabilities

Seismic vulnerabilities for the double-deck framed structure are applicable to this structure and have been discussed in the previous subsections of this report. The primary additional vulnerability is the outrigger pier cap element. Bent 55 (at approximately Pike Street) has visible classic shear cracks caused by vertical loading over the life of the structure. The vulnerability in shear strength, demonstrated by the cracks, adds to the torsional vulnerability caused by the longitudinal seismic response.

Rebuild/Retrofit Concepts

The rebuild/retrofit of this transition would require the same substructure replacement as the middle Viaduct. In addition, the outriggers would need to be

replaced or retrofitted as described for the south transition. In this case, the new bents would allow strengthening and pinning the existing upper outrigger pier caps, and full replacement of the lower pier caps.

North Concrete Separation Structure

The North Concrete Separation Structure is composed of the nearly parallel Viaducts north of the North Transition, near Pine Street. The northbound separation includes Bents E52 to E45 and the southbound separation includes Bents W52 to W41. These structures are all reinforced concrete T-girder bridge structures with two-column reinforced concrete columns supported on pile foundations. Bridge spans are 40 feet to 54 feet. Expansion joints are located about 135 feet apart. Expansion piers were constructed by splitting the normal columns and pier cap to form half thickness members.

Seismic Vulnerabilities

Columns for the northbound structure vary from the tall and slender columns at the transition to short columns at the railroad tunnel portal. There is a significant side slope under the structure so the rows of columns in each bent include the much shorter and stiffer eastside or uphill columns with the much taller westside columns. Along each frame, there is an eightfold difference in stiffness between the shortest and tallest columns. The vertical elements would resist the deformations imposed by ground motions in proportion to the stiffness of the members so the stiffest members would attract the largest forces.

Columns supporting the southbound structure are nearly uniform in length across and along the structure. These columns cannot withstand the design seismic deformations elastically, and the reinforcement detailing would not permit full development of plastic or post yield deformations before brittle collapse occurs.

The foundations are pile supported with questionable uplift capacity. Typically, there is no pile cap top reinforcing for any lateral overturning.

The pier caps and edge girders are not reinforced for cyclic lateral loads and potential bar slip decreases the member capacity. The beam to column joints are not properly reinforced and deck girders are deficient in reinforcing, especially in shear. In short, the same vulnerabilities exist in the single deck Viaduct as those in the double-deck Viaduct. There also is the relative column stiffness issue.

Rebuild/Retrofit Concepts

The rebuild/retrofit concept selected for this section is to construct a new lateral force resisting frame under the Viaduct. This new frame would be supported on either single large diameter drilled shafts installed through the deck or multiple

small diameter drilled piles that can be constructed by low headroom equipment. The foundation system would be joined to a fully reinforced pile cap. New columns would be cast monolithic with the pile cap. New lateral framing, similar to pier caps, would join the tops of the columns.

The new columns can be sized to provide the stiffness and ductility to meet displacement requirements. Connections are made to the existing decks by cradling the girders and pier caps into the new framing elements. Existing columns would be enclosed within steel jackets to provide the confinement as well as shear capacity to prevent a catastrophic collapse. The footings would be strengthened.

Work above the deck would be determined by traffic handling requirements and bridge railing/deck replacement. The actual seismic retrofit construction would be below the deck.

North Railroad Crossings

The north railroad crossings include frames from Bent E44 to Bent E40 and from Bent W41 to Bent W36 (west of the Pike Place Market). These steel superstructures are unique in the Viaduct so they could span over the mainline railroad tracks emerging from a portal below. These frames are constructed with steel girders, steel pier caps and cross beams, some steel and some concrete columns incorporating steel cross bracing, which would support the 43 feet wide concrete decks. Column heights vary from bearings placed directly on the railroad tunnel abutment to tall and slender steel columns behind the new Marriott Hotel located to the west of the Viaduct.

Seismic Vulnerabilities

These frames have a number of unique seismic vulnerabilities in addition to many of the typical Viaduct vulnerabilities. The relative difference in stiffness of the substructure supports dictates that resistance to ground movement would be concentrated at the short supports adjacent to the portals. This concentration would likely break the support connection in the initial response. Restraint would then shift to the next stiffest support with the same result. These failures would continue until all restraint is gone. After the short columns or supports are damaged, the tall columns would be exposed to larger deflections. The larger deflections in the taller slender columns would cause the slender columns to buckle due to P-delta effects. If any of the main structural elements, like the columns, are strengthened without consideration of the overall strategy, all the lateral connections to braces and girders are jeopardized.

The foundations of the tall slender columns and the column anchorage have limited lateral capacity and excess vertical capacity. This becomes a

consideration with any rebuild/retrofit concept that either increases the weight or lateral demands on the foundation.

The girder seats at the end piers allow very little displacement. This is quite typical for a 1960s girder design when lateral deformations were not considered a major problem. These types of short girder seat lengths have resulted in a number of very spectacular bridge failures in seismic events around the world.

Rebuild/Retrofit Concepts

Looking at the weaknesses in the entire structure a successful retrofit would require improved foundations for both vertical and lateral loads. More important is a need for more uniform lateral support conditions such that more of the supports would share the lateral restraint. The lateral load path has to be developed from the improved foundations to the concrete deck. The existing abutment wall that supports a number of bents also is suspect. New foundations must miss the railroad tunnel and also not overload the railroad portal wing wall. One method of providing these improvements would be as follows:

- Construct new foundations near the ends of each frame to support new columns.
- Construct new concrete columns continuous with the foundations that are very stiff.
- Construct new pier caps that extend outside of the existing substructure.
- Erect new plate girders the length of the existing frame on to the new pier caps.
- Slide new transverse beams under the existing girders to be supported on the new plate girders.
- Provide lateral connections between the existing girders and the new transverse beams and between the new beams and the new plate girders.
- Lift the plate girders onto new bearings. The new bearings are to be a base isolation unit such as a lead core bearing or a friction pendulum bearing.
- Construct new drilled shafts adjacent to the portal abutment wing wall. These stiff foundations in combination with the base isolation would decrease the seismic loading imposed by the bridge foundations onto the wall and eliminate the Viaduct's dependence on the abutment wall for support during a seismic event.

North Viaduct

The north Viaduct extends from the railroad crossing to approximately 300 feet south of the Battery Street Tunnel. This structure runs from Bent E40 and Bent W36 (Virginia Street) to the north mainline abutment (north of Bell Street). There are a number of reinforced concrete T-girder frames of different lengths, widths, numbers of lanes and heights. The separate northbound and southbound structures are joined from Bent 35 (approximately Virginia Street) northwards. There is an important street crossing of Elliott Avenue and access ramps are provided at Elliott Avenue and Western Avenue.

Seismic Vulnerabilities

The seismic vulnerabilities are very similar to the other reinforced concrete Viaducts. The footings are weak and inadequately reinforced; piles have little uplift capacity; columns are not well confined nor do they have adequate shear reinforcing; beam to column joints are not adequately reinforced; pier caps are not detailed for load reversal in seismic events; lap splices are inadequate; bond stress is exceeded before bar yield; torsion resistance is inadequate throughout.

Rebuild/Retrofit Concepts

The north Viaduct structure from the north transition structure to Bent 16 would be rebuilt similarly to the north transition structure due to similar structure type configurations. The retrofit concept for the Viaduct from Blanchard north (Bents 15 to 1) is to replace a single column in each bent with a much stiffer element. This new force collector would need an improved foundation constructed with either low headroom equipment or through the deck. The new column would be fixed to the new properly detailed pile cap. The top of column would have a flexural pin to limit the force transfer to the existing superstructure. The pin, something like a steel pipe grouted in place would be installed in a hole cored through the existing deck. The hole would then be filled with concrete. Temporary supports are required before removing the existing column. The existing columns that remain would be encased in grouted steel jackets to provide adequate confinement reinforcement in the plastic hinge areas of the columns. Pins, as previously described, would also be added to the top of all existing columns to limit the force transferred to the existing bents. The existing footings also would be strengthened. The superstructure would be retrofitted to handle the design traffic live load based upon the load rating analysis.

Viaduct Ramps

There are a number of ramps connecting both levels of the Viaduct to various city streets. The ramps include many shapes, sizes, and structure types. There are reinforced concrete T-girder ramps at Elliott Avenue and Western Avenue. There are prestressed AASHTO girder ramps at Seneca and Columbia Streets.

The First Avenue South Ramps include a half dozen different types of structures within the ramps. All have different seismic vulnerabilities.

The rebuild/retrofit construction of the ramps would cause the closure of all the entrance or exit lanes to and from the ramps for many months. During the ramp closure period, the ramps could each be retrofitted or removed and replaced with new seismic resistant structures. The ramp traffic movements would be replicated by the erection of temporary ramps in the stadium area using the existing ramp stub outs. The 1st Avenue South ramps would be removed and permanently replaced with new ramps in the stadium area to provide the same traffic movements with improved traffic performance characteristics.

Seismic Vulnerabilities

All of the ramp structures have insufficient column ties to adequately confine the vertical compression members and allow for ductile behavior during a design level seismic event. Transverse reinforcement typically consists of #3 hoops at 12 inch spacing. The column vertical steel has lap splices in the plastic hinge zone that compounds the problem. The footings have no top reinforcement and during an earthquake, where tensile axial loads may be imposed on the structure due to overturning, they would perform poorly. The Seneca and Columbia ramps are made of pre-cast pre-stressed girders that have bearing seats of insufficient length to meet present day standards. During the design earthquake, the girders could slide off of the bearing seats.

Rebuild/Retrofit Concepts

New traffic barriers and deck would be provided for all ramps.

Elliott and Western Ramp Retrofits

All existing columns would be retrofit with steel jackets, the footings would be enlarged and strengthened, and pin connections would be placed on the top of all existing columns to minimize lateral load transfer to the frames. A new stiff single column/foundation system would be added at the mid-span of each existing bent. A new longitudinal girder would connect these new columns. The new system would take a significant lateral load and control deflections. This would decrease the seismic loading to the existing bent frames.

Columbia and Seneca Ramp Retrofits

All existing columns would be retrofit with steel jackets. The foundation system would be strengthened by adding depth and width to the footing and by the addition of piles. Restrainers, seat extenders, and shear keys would be added to each bent cap beam. The end span would be rebuilt to provide an adequate expansion joint at the mainline structure.

Battery Street Tunnel

The Battery Street Tunnel is a four-lane cut-and-cover structure connecting the northern end of the Alaskan Way Viaduct to Aurora Avenue North. The tunnel is within the Battery Street right-of-way. The tunnel walls are constructed along the property lines and the tunnel roof serves as the base of Battery Street.

It has been found that the tunnel life safety features, do not meet current tunnel standards. Similar to the Alaskan Way Viaduct, which does not meet current standards for seismic safety, the following life safety improvements are needed in the tunnel:

- Ventilation system
- Fire detection and suppression system
- Electrical system
- General and emergency lighting systems
- Tunnel communication and traffic systems
- Egress System

For additional information, refer to the "In-Depth Mechanical and Electrical Inspection and Recommendations" (Parsons Brinckerhoff, March 2002). 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.

Design Basis and Assumptions

The existing tunnel structure and utilities information is based on a set of drawings from the City of Seattle, Alaskan Way Viaduct, State of Washington, Department of Highways, Contract No. 4314, dated May 1952 and March 2002 inspections. The mechanical and electrical information is based upon the March 2002 inspections.

The final design assumes no change to the existing traffic pattern or civil standards through the Battery Street Tunnel. Traffic would be maintained during construction of the tunnel retrofit. At this time, the tunnel retrofit would be limited to a fire/life safety upgrade.

Description of Proposed Mechanical and Electrical Systems

Mechanical and electrical inspections for the Battery Street Tunnel were performed on March 23 and 24, 2002. The inspection included tunnel ventilation, air quality, fire detection and suppression, electrical power, lighting, traffic, and emergency communication systems. The "In-Depth Mechanical and Electrical Inspection and Recommendations" (March 2002) report provides all specific findings and provides recommendations under three categories: emergency, maintenance, and rehabilitation.

Ventilation System

Ventilation systems in tunnels provide two main functions: maintaining air quality during normal operations, and removal of smoke and heated gases in the event of a fire emergency within the tunnel. The existing Battery Street Tunnel ventilation system only provides for maintaining air quality during normal operations. Tunnel fans are activated when carbon monoxide (CO) levels reach a pre-determined threshold. This threshold level of 250 ppm exceeds the standard of 125 ppm. There is no fan assisted emergency exhaust system. Both of these shortcomings present a safety concern to the traveling motorists. Motorists may be exposed to high CO levels during congested conditions and a tunnel fire scenario would expose motorists and the tunnel structure to smoke and hot gases, which could lead to loss of life and property damage. A new ventilation system to address these issues is proposed.

Fire Detection and Suppression System

Fire detection and suppression systems in tunnels provide for the rapid detection and extinguishment of tunnel fires. The Battery Street Tunnel fire detection systems uses heat activated devices (HADs), which are only partially operable, and manual pull alarm stations, which are not functioning. The fire suppression system is not designed to extinguish large tanker truck fires. This is a potentially unsafe condition that could lead to significant loss of life and property in the event of a significant tunnel fire event. The fire detection and suppression system is proposed to be upgraded.

Electrical System

Electrical systems in tunnels provide for a reliable supply to support both normal and life/safety operations. The existing Battery Street Tunnel electrical system is in poor condition and needs to be replaced. Much of the system is original to the 1952 installation. Wiring has reached the end of its useful life, conduits are asbestos type, effective grounding is missing, and lighting feeders are not routed in conduits thus subject to failure during a fire event. The tunnel also lacks an emergency power supply system. In the event of a power system failure, all life

safety systems would be inoperable. These shortcomings provide for an aging and unreliable electrical power system that is proposed to be replaced.

Lighting System

Tunnel lighting provides motorists adequate visibility to maintain traffic flow and to identify objects within the tunnel. Emergency lighting systems are provided to maintain a minimum light level and identify exits in the event of a power outage or tunnel fire. The Battery Street Tunnel lighting system is owned and maintained by SCL. Lighting levels at the tunnel portal area are not adequate and the system lacks any kind of emergency lighting function. Drivers entering the tunnel during the daytime are subjected to a "black hole" effect as a result of the perceived difference in the external and internal light levels. This can result in accidents at the tunnel portal areas. The lack of any kind of emergency lighting system in the Battery Street Tunnel is a safety concern, as power outages will result in a completely dark tunnel making it difficult to see and to identify emergency exit routes. Adequate general and emergency lighting is proposed to be provided.

Tunnel Communication and Traffic Systems

Tunnel communication systems allow motorists to communicate with tunnel operators in the event of a tunnel incident or emergency. The Battery Street tunnel has an emergency telephone system that is operational. Two of the emergency telephone locations require telephone replacement or repair. Traffic systems in modern tunnels consist of vehicle detectors that automatically detect vehicle incidents, closed circuit television cameras (CCTV) to allow operators to monitor tunnel operations, and traffic signals/signs that stop vehicles and provide announcements in the event of a tunnel incident. The Battery Street Tunnel does not have any provisions for vehicle detectors, CCTV cameras, and traffic signals. There are signs and flashing beacons ahead of each tunnel portal to indicate when the tunnel is closed. The lack of existing traffic systems in the Battery Street tunnel is a safety concern as these systems allow operators to quickly detect, verify, and take action during a tunnel incident. The tunnel communication and traffic systems are recommended to be upgraded or replaced.

Egress System

The Battery Street Tunnel presently has emergency egress doors located along the tunnel route in the center wall to provide egress from one side of the tunnel to the other. These doors do not properly function and may not be acceptable for current use. The walkway width inside the tunnel is too narrow to comply with NFPA 502 requirements. The plan is to repair the existing doors as well as provide egress stairs at a spacing to meet the 1200-foot exiting requirements. These would provide a safe means of egress to the surface either through adjacent buildings or through public right-of-way.

Control of Hazardous Materials

It is recommended that WSDOT adopt rules and regulations that apply to the transportation of hazardous materials in the Battery Street Tunnel. The mechanical and electrical systems as they currently exist are substandard for the operation of unrestricted traffic including the transportation of hazardous material in the tunnel. It should be noted that hazardous cargo restriction signs are posted on the north approach. There is no corresponding signage on the south approach.

Emergency Response Plan

It is recommended that WSDOT provide procedures to carry out a complete and coordinated program of fire protection for the Battery Street Tunnel that includes written preplanned emergency response procedures and standard operating procedures. This information was not available for review upon inspection.

Description of the Proposed Structures

The tunnel ventilation system and emergency egress are key elements of the fire life safety system. The ventilation dampers would be placed at 700 or 800-foot maximum intervals. The exhaust tunnel would be about 14-foot inside diameter. A 16-foot diameter tunnel boring machine (TBM), commonly used for boring sewer systems tunnels, would be used for the tunnel excavation. The other life safety systems and utilities may be located inside the new exhaust tunnel. The plan area of the underground ventilation structure is to be 40 foot x 60 foot.

The proposed retrofit structures include a 2000 foot long, 14 foot inside diameter exhaust tunnel; six 10 foot by 15 foot exhaust shaft and damper system and an emergency exit located at 4th Avenue; a 40 foot by 60 foot ventilation structure located in the parking lot at the southeast side of 4th Avenue and Battery Street; a TBM launching shaft, approximately 25 foot by 60 foot, would be located at the south end of the exhaust tunnel adjoining Western Avenue. A bucket elevator would be provided at the shaft for tunnel muck removal by dump trucks. A smaller TBM retrieval shaft would be constructed at the north portal.

The emergency exits and exhaust shafts should be located to avoid any major existing utilities. They may be constructed from the roadway with only one lane in addition to the parking lane required for a construction easement. Shafts would be located at 2nd Avenue, 4th Avenue, and 6th Avenue. Construction of the exhaust damper system would result in traffic impacts at the surface streets since this would be top down construction. During construction, traffic would have to be detoured away from these intersections. Traffic control could be used to allow some traffic along the cross streets through the impacted intersections. Battery Street would be affected as well.

The ductwork would consist of dampers installed into the exterior tunnel walls between the walkway and the deck girders. The opening would be approximately 150 square feet. The rectangular shafts would be constructed down to the depth of the main ventilation tunnel. Mined adits of 14-foot diameter would connect the vertical shafts and the bored ventilation tunnel. The exhaust air would be drawn from either side of the Battery Street Tunnel through the dampers. The ventilation tunnel would be connected by a 14-foot diameter mined adit to the ventilation structure located at 4th Avenue and Battery Street.

Alaskan Way Seawall

The Alaskan Way Seawall is a key element in the Alaskan Way Viaduct and Seawall Project. The seawall is approximately 8,000 feet long from Washington Street to Eagle Street. Studies and site investigations have verified that the structural condition of the wall is poor and the existing structures do not have adequate capacity to meet the demand of current earthquake design loadings. The Seawall Rebuild Option would be constructed in the Rebuild/Retrofit Alternative.

The Seawall Rebuild Option would construct a new concrete secant pile wall behind the existing seawall. Jet grouting techniques would be used to stabilize a block of liquefiable soil behind the new secant pile wall. A new pre-cast fascia panel system would be placed at the exposed areas of the seawall. For additional information about the Seawall Rebuild Option, see the "Alaskan Way Seawall Design Report" (Berger/ABAM Engineers, Inc., July 2002).

REBUILD/RETROFIT CONSTRUCTION GUIDELINES FOR DOUBLE-DECK STRUCTURES

In order to provide for reasonable public safety during construction of the seismic rebuild/retrofit, the following guidelines are recommended. WSDOT and the City of Seattle will need to agree with these recommendations. Other similar retrofit bridge projects use this approach.

Bridge Demolition

1. Limit the number of bents to be removed at one time.
2. Temporary supports shall be provided before any bridge column is removed.
3. For partial bridge demolition such as column removal, only low impact equipment is allowed.
4. Construction contractor is required to submit a complete demolition plan and have the plans approved prior to starting any bridge demolition.
5. Traffic and any public access adjacent to the bridge demolition shall be protected by a solid protective cover. The cover shall be capable of supporting a minimum uniform live load of 50 psf and an additional falling object of 300 pounds having fallen 50 feet.
6. Traffic on bridge frames under partial demolition (e.g. upper deck railing demolition and replacement) shall be limited to a maximum of two lanes each direction prior to the demolition.
7. Overload or Permit Trucks would not be allowed on the temporary supported structure, unless the design specifically allows for them.

Temporary Supports

1. Working Drawings for temporary supports shall be submitted for approval before any materials are ordered or fabricated.
2. Working Drawings for temporary supports shall be sealed by a Structural Engineer registered in the State of Washington.
3. Temporary supports shall be designed to provide support of the dead and live loads anticipated throughout construction.
4. Temporary supports shall provide a minimum lateral capacity equal to 15% of the vertical load capacity being replaced.

5. Temporary supports shall provide a minimum lateral stiffness of 120% of those elements to be replaced.

6. Temporary supports shall have a continuous load path from the top of the superstructure to the foundation, including all connections and braces.

7. Temporary shoring and falsework shall comply with the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction.

Bridge Reconstruction

1. Any construction falsework or temporary supports shall be protected from highway traffic by an appropriate concrete barrier.

2. Details shall be provided to minimize the traffic vibrations and impact on freshly placed concrete.

3. Drilling and coring of existing concrete required for additional reinforcing, such as prestressing, shall be located to cause no damage to the existing reinforcing.

4. Approved or standard bridge railing shall be provided for the entire Viaduct.

5. A new deck shall be provided over the entire Viaduct. It shall be designed to accommodate a 2" thick future overlay.

Temporary Structures

Temporary structures are those facilities constructed and removed during the course of the main construction of the project. These structures may include roadway bridges, pedestrian bridges, box culverts, earth retaining structures, and some underground structures. These facilities will be defined as having a lifespan of less than five years so many of the fatigue factors would be reduced and would increase the available capacity. The probability factors on the load side are reduced, especially seismic requirements. Guidelines for temporary structures are presented below:

1. All temporary structures shall be designed for AASHTO HS-20 live load.

2. Permit loads may be restricted from temporary structures.

3. Load combinations are by the WSDOT Bridge Design Manual.

4. Seismic design requirements shall comply with the WSDOT Bridge Design Manual, Section 4.1.5.e, but the acceleration coefficient shall not be less than 0.20 g.

CONSTRUCTION STAGING

Staging of the rebuild/retrofit to meet traffic requirements determines many of the various concepts that may be used. There are a number of solutions to the construction staging issue. In all cases, both the substructure and superstructure of the viaduct must be strengthened or replaced. The double-deck structure reconstruction is the largest, most challenging area. A brief description of some of the different construction methods is described below.

It would be possible to complete this work with two lanes of traffic on each deck during peak hours but there are challenges for traffic to be maintained on the lower deck during work on the upper deck elements. Traffic below construction work was allowed at I-280, however the average daily traffic was very low because much of the viaduct was already closed from Loma Prieta Earthquake damage. The traffic on I-280 was substantially less than is experienced on the Alaskan Way Viaduct. For this method to be completed safely, the lower deck or southbound traffic would need to be moved from one side or the other or limited during off-peak hours.

Double-Deck Retrofit Construction Methods

There are a number of methods for completing the rebuild/retrofit construction of the double-deck portion of the Viaduct. Some of these methods are described briefly below. Method A has been selected as the basis for the design methodology. This method can be used throughout and it should provide the more conservative bounds for cost estimation. Method B should be reviewed during Preliminary Engineering for potential cost and time savings.

Method A - Replacement by Supporting the Structure on Temporary Supports

This allows removing the columns and pile caps, then rebuilding the foundation, columns, pier caps, edge girders, and other deck supporting elements. The construction guidelines presented in the previous section were developed for this type of rebuild/retrofit construction. This method was used successfully on the I-280/Route 101 Interchange double-deck retrofit in San Francisco from 1990 through 1995. The resulting Alaskan Way Viaduct structure would have the same spans and slightly larger roadway widths as the existing structure because the existing pier caps are strengthened but maintained. The existing decks would be replaced throughout due to the poor existing condition of the deck and traffic railing replacement.

The installation of temporary supports would have a major impact on the construction. The supports require new foundations that can support the gravity loads of the Viaduct, the tributary live load, and required lateral support.

Jacking up the Viaduct onto the temporary supports would be a very complex process to ensure the Viaduct is not damaged in the load transfer. The supports severely limit access by the contractor for bridge removal and reconstruction, causing inefficiencies in crew time. The schedule is driven by the limitations imposed for the use of the temporary supports. Completing the work on a single bent may require many months, so completing a single frame with four bents will need to be carefully planned and coordinated.

Method B - Replacement by Constructing New Supporting Elements Outside of the Existing Viaduct

This reconstruction method takes advantage of the space available around the Viaduct in some locations. The first step would be to locate new foundations and columns outside of the column line of the existing Viaduct. The spans could be modified for greater efficiency. After the new foundations are constructed and the bottom of the new columns poured, new pier caps and new edge girders would be constructed around a properly detailed beam to column joint. The pier caps and edge girders would be prestressed. Intermediate transverse beams could provide support to the existing deck girders, and the railing could be replaced with the traffic impact loads now resisted by the edge girders. The upper level columns also would be constructed followed by upper level pier caps and deck modifications. After the new substructure is completed, the old columns would be removed. Great care in detailing would be necessary to prevent damage during the final load transfer to the new substructure.

In a slight modification of this method, the entire lower deck could be replaced when the new pier caps are constructed. Portions of the upper deck may not be replaced but strengthened due to traffic constraints.

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Appendix A: Analysis Methods

Overview

The non-linear ultimate displacement (pushover) analysis is an effective type of analysis to be performed in the initial design stages of seismic retrofits. The pushover analyses performed on the Alaskan Way Viaduct have produced important results to date. Other types of analysis are used in various situations and stages in the retrofit process. Response Spectra Analysis was performed and the results indicated that the structure would fail under project design criteria loading, but the results do not predict the sequence of failure. A section analysis was performed to develop the member properties used in the other analyses. Each type of analysis will be described briefly below. At this level of design actual construction details are not to be developed, but enough information should be provided to support the construction cost estimating necessary in the EIS process as well as to identify impacts caused by the construction.

Non-Linear Ultimate Displacement (Pushover) Analysis

The pushover analysis is a capacity driven approach that allows an engineer to determine the location and likely sequence of failures or post yield performance up to and including the final collapse of the structure. A key use of the pushover analysis is to identify the load path of the structure.

Structure Modeling

A computer model of the structure is generated using either two-dimensional or three-dimensional geometry and member properties. For elements that are assumed to remain elastic throughout the analysis conventional member and material properties may be used and verified through the analysis. For members and joints expected to yield or fail, the relationships representing the force/deflection or moment/curvature characteristics are required. These non-linear properties are included in the model and located in places expected to experience the high loads relative to member size. Development of the non-linear properties is described in a following subsection.

Foundation Modeling

Depending on the type of foundation, loading, and site conditions, the foundation may be modeled discretely; or as non-linear springs; or as linear springs, depending upon the situation or configuration. When a structure is detailed to provide an adequate foundation to remain elastic with a reasonable factor of safety then using elastic springs to model the foundation conditions may be adequate. Elastic springs provide very little damping and are applicable only when relative deformations would be very small. This type of modeling is

fairly simple to determine and evaluate. It is most appropriate for a conceptual study such as the engineering in support of an environmental document.

Non-linear springs take into account the post yield performance of the foundation system under larger lateral movements as well as the soil properties under higher strains. This soil structure modeling is much more realistic but much more complex and requires considerable geotechnical study.

Foundations may be modeled using soil elements and actual piles within the soil matrix. This modeling provides the most realistic results with much higher damping than other foundation models. This type of modeling is typically only used on large foundations for the final design.

Results of Analysis

Taking the structure displacement and level of lateral force at the initial failure, a point can be plotted on a force versus displacement curve. The structure model is pushed further until the next failure and this process is repeated until a mechanism forms and the structure theoretically collapses. The force and displacement values at each local failure are plotted to provide a characteristic curve of the structure that tells most of the key information about the seismic resistance of the structure. This force versus displacement curve is a key tool used to evaluate seismic resistance in a structural system. A good performing structure has a very long flat plateau in the post yield stage.

Section Analysis

Section analyses are used to determine the post yield non-linear properties of the vulnerable structural members. These analyses give designers the information about designs that can provide ductility and toughness to a structural system. The results of these analyses are used as data for any of the static or dynamic non-linear analyses used in the design process.

Modeling

Modeling reinforced concrete elements includes the cover concrete, core concrete, and the mild and/or high strength reinforcing. The model may also include embedded steel shapes or steel shell elements that contain concrete; reinforcing, prestressing or other rolled steel shapes. Within the analysis, the cover concrete would spall at predetermined strain values, while confined concrete would perform to a completely different and variable material property depending on confinement and axial loads. Steel elements in the cross sections allow strain hardening for the higher strains.

Results of Analysis

The results of section analyses are the moment curvature curves used in other non-linear analyses. The curvature values provide relative ductility

characteristics of a cross section or reinforcing pattern. Curvature values of greater than 15 are indicative of good ductility, while maximum curvature of less than five indicates a very brittle section. The moment values provide the maximum overstrength moment capacity a section can provide for a defined axial load. The overstrength moment is normally less than the 130% of nominal capacity specified by most design codes.

Response Spectrum Analysis

Response spectrum analysis (RSA) is a method of linear elastic dynamic structural analysis. In RSA, the structure model is evaluated for modes of vibration. The frequencies of the modes are used to determine the forcing function based on an acceleration response spectrum (ARS). The frequency (or period) of the individual mode of vibration is entered in ARS and the return is the acceleration for that mode. Typically, the ARS assumes a constant 5% damping applies to all modes of vibration. Modal displacements are calculated and combined by various methods to determine structure displacements. Forces are back calculated from the displacements. RSA is the most common method of analysis used in structural design to determine the elastic demands on structures. Developing models is very quick and results are generally envelope type values. One weakness is that for structures located near major faults there is a near fault effect that is characterized by a nearly constant velocity movement. By definition, constant velocity means no acceleration so this generally governing effect is not handled by this analysis. RSA was performed on a couple of representative frames from the Viaduct.

Modeling

Most RSA bridge models feature a stick model composed of linear elastic beam elements connecting nodes or joints. Foundations can be either fixed, linear springs or releases. Masses are determined within the program based on section properties, densities, and lengths but additional mass may be specified for large joints or deck surfaces and railings.

Results of Analysis

The key values from the analysis results are the forces and displacements. The forces are used directly for design under the AASHTO Specification with the load combinations and appropriate response modification factors. Displacements are used under the AASHTO Guidelines. The displacements are used to compare with the capacities calculated by pushover analysis.

Non-Linear Time History Analysis

While it requires more detailed modeling than other analysis methods, non-linear time history analysis (NLTH) should be used to check the final design of the retrofit or any other build alternatives being considered for the Viaduct retrofit or replacement. Fully developed material and section properties are used that allow post yield behavior. Expansion joints are modeled and contact between adjacent structures is captured in the analysis. The sequence and the degree of large deformations are automatically considered. Actual damping is determined as a result of the hysteretic behavior of plastic hinges or other force control devices.

The most important feature of NLTH is the ability to reasonably model the near fault condition. The near fault ground motion has only been recorded within ten miles of major seismic events. The motion is generally characterized by a very large ground displacement that moves with a nearly constant velocity. By definition, the nearly constant velocity means that there is almost no acceleration so this behavior is often neglected in the RSA method described previously because the period of the movement is quite large. However, the entire mass of the structure responds to this movement and the change in direction of the ground movement causes the "fling effect" often mentioned in the descriptions of this phenomenon. This effect governs the design requirements of the structural system so the impacts have to be evaluated in the analysis.

Continuous structures like long Viaducts or long tunnels in soft ground can experience the full motion of waves traveling through the soil. This wave motion can cause the structure to move longitudinally, vertically, or horizontally and the structure has to maintain its integrity through the full range of movements. This means that fairly large and detailed models are necessary to capture the governing behavior of the structure. The model has to allow post yield or non-linear behavior that the structure would experience in order to develop the actual displacements the structure might physically encounter. In this method of design, the forces generated within the structural model are a result of the capacity limiting elements used to permit greater displacement capacity.

Modeling

The structure model is generally a "stick model" composed of beam type elements very similar to the models used for RSA analysis. The process of developing a NLTH model starts with static analysis followed by RSA. Then non-linear properties are introduced into the model individually and tested through static non-linear analyses. After the model successfully completes testing the first displacement time history is introduced. Normally three separate ground motions, each with independent X, Y, Z direction time histories

are used. However, experience indicates that the near fault effect would govern the response regardless of the additional motions due to long-range movements.

Results of Analysis

The main result of this analysis is the displacements and deformations the structure experiences over the duration of the event. Maximum and minimum forces, contact and pounding between elements and their duration throughout the event are also available results. Final structural damping is a result rather than an assumed input as with RSA. One other result from NLTH is the evaluation of global structural stability since if a mechanism forms during the input displacement time history the structure model would collapse or become mathematically unstable.

Appendix B: Base Isolation Retrofit Approach

Base Isolation

Seismic base isolation is a fairly common and popular concept for the retrofit or new construction of many types of structures including many bridges. The basic principal of seismic isolation is to provide a single element that when excited with seismic ground motions can deform and dissipate the energy while not damaging the basic structure. The two most popular types of base isolation are lead core bearings and friction pendulum bearings.

Lead core bearings are made with a typical rubber bearing filled with a cylindrical lead core. The lead is sized to provide the necessary stiffness to resist normal service loads. In a major earthquake, the lead melts so the stiffness is provided by the rubber portion of the bearing. This type of bearing has become cost competitive since the original patent expired and there are many manufacturers.

Friction pendulum bearings use a shoe that slides on the inside surface of a finely machined steel bowl. The initial stiffness necessary to resist lateral service loads is provided by the friction between the shoe and the bowl. In a major earthquake, the friction between the shoe and the bowl is overcome and the bowl moves with the ground and the structure remains in place. The material between the bowl and the shoe is very specific and patented for each manufacturer.

For seismic isolation to work properly all the other elements of the structure have to be both stronger and stiffer than the isolation device. For example, if a beam or column is weaker in lateral capacity than the isolation bearing, then the beam or column would fail before the bearing has the opportunity to act.

Base isolation devices are very good at dissipating energy. Typical damping for these devices is between 25% and 30% of critical damping. These devices also protect the structures from damage much like a structural fuse. Mathematically seismic isolation is very similar to formation of ductile plastic hinges in structures, except that during the formation of plastic hinges significant damage occurs to the structure. The performance curves described in the movement of the isolation bearings and plastic hinges in reinforced concrete are quite similar. A well detailed plastic hinge can provide similar damping and similar displacements to a base isolation device but the spalling and cracking of the concrete and the high strains in the reinforcing steel may render the facility standing but damaged beyond repair after a major earthquake. A base isolated structure may need to be forced back into position after an earthquake, but the damage is generally confined to the bearing.

Examples of Base Isolation Use

For a seismic retrofit of the Alaskan Way Viaduct, base isolation was considered. This method has been used in many facilities, including the following applications, around the world:

1. Very specific situations as a fuse element in the I280 project (CA - lead core bearings)
2. Benecia Martinez Bridge (CA - friction pendulum)
3. Aurora Bridge (WA - friction pendulum)
4. Rte 242/680 overcrossing (CA - lead core)
5. Sunset Interchange SP-EN Flyover, Issaquah Wash. (WA - friction pendulum, in construction)
6. Rte 680/24 connector (CA - lead core)
7. Bayshore Boulevard Overcrossing (CA - lead core)
8. Yahagi River Bridge (Japan – lead core)
9. Tempisque River Bridge, (Costa Rica -lead core, designed, not yet constructed)
10. Ebina Junction, (Japan – lead core)
11. Monzen Expressay, (Japan – lead core)
12. Legacy Parkway Bridge 19 (UT- elastomeric pads, in construction)
13. Crystal Springs Dam Bridge Replacement. (CA – lead core)

In the design of the double-deck retrofit program in San Francisco, base isolation with lead core bearings placed on top of the foundations was investigated. The elements modeled were lead core bearings and the element properties such as stiffness and damping were provided by the only vender available at that time.

Feasibility of Base Isolation for the Alaskan Way Viaduct Rebuild/Retrofit

A full 3-dimensional dynamic analyses of the San Francisco double-deck structure was executed using the Caltrans response spectra for the site. The AASHTO ARS for the life safety event for the Alaskan Way Viaduct Project is comparable. The analysis results indicated a great reduction in internal forces.

However, the structure displaced nearly 24 inches and the internal forces still exceeded the capacity of the existing structure. With the agreement of Caltrans' Independent Peer Review, further base isolation of the double-deck structures was not pursued. The lead core bearings were still used in a couple of specific applications as controlled force control elements placed between the reinforced concrete box girder superstructure and the tops of the stiffened piers. The analyzed structure from the I-280 project was a little shorter and substantially stiffer than the Alaskan Way Viaduct and it is located in a site with very dense silty sand foundation material. Thus, soft soil conditions and liquefaction were not a consideration.

For the retrofit of the Alaskan Way Viaduct, the base isolation concept is not recommended for the following reasons:

1. The existing structure is made up of many segments with various structural configurations and different seismic performance characteristics. A non-linear ultimate displacement (pushover) analysis of two typical subsegments indicates that damage is expected at a spectral acceleration as low as 0.13 g. The structure still remains stable at this point, but with additional seismic loading, the structure will experience successive and increasing damage leading to collapse. These subsegments do not have the ultimate capacity to force the isolation bearing to slip or yield in a design level earthquake. The result is that the structure fails before the bearing fuse forms.

2. All other elements in the structural system including the soil have to be stiffer than the bearing. In many cases, the ground does not meet this requirement and in most cases, the structure is too flexible.

3. If the soil and structure could be made stiff enough and strong enough for the bearings to function then each frame unit has to be separated from adjacent obstructions for at least 12 inches to avoid pounding. Note this requires an expansion joint of about 24 inches between adjacent frame units.

4. To meet the approximate 24-inch clearance at expansion piers the split expansion piers would have to be replaced.

5. Meeting the approximate 24-inch expansion requirement requires a large and complex roadway expansion joint.

6. The complex roadway joint detail is very expensive.

7. Restraining or limiting the movement to a lower value causes a major increase in internal forces that the structure cannot safely take. Attempting to stop the structure from moving once the bearing has

started to work would cause an abnormally high reaction. For example, when placing girders on bridge piers gently and under control, a dynamic impact factor of 30% or higher is considered normal in design. For out of control situations like dropping the girder a few inches, an impact factor of 100% is common practice. Allowing the base isolated structures to "pound" each other defeats the principal of base isolation and could be quite dangerous.

Appendix C: Joint Shear Analysis of Double-Deck Beam Column Connections

Overview

The retrofitted capacity of a typical double-deck Viaduct constructed before 1990 is largely governed by the internal stresses in the beam to column joints. Retrofitting this connection requires the replacement of the joint with properly detailed reinforced concrete. Replacing the joint entails temporarily supporting both decks of the structure and demolishing the column in the joint area. Since the column is then disconnected from the structure, it is appropriate to remove the column to the foundation and improve the foundation. Then a new column with a properly detailed joint can be constructed. The actual capacity of the beam column joints in the Viaduct is described below.

Analysis

The 1989 Loma Prieta Earthquake caused serious damage to reinforced concrete column-beam joints in San Francisco area bridges. This prompted systematic research that led to improved provisions for seismic-resistant design of reinforced concrete bridges. Based on the latest research and design provisions, the design of typical beam-column joints in Alaskan Way Viaduct double-deck structures is evaluated. The specific objective is to evaluate the joint shear capacity in the existing structure and to develop possible retrofit solutions.

The investigation of the joint failure mechanism can be approached by considering the shear forces concentrated in the joint area. This shear force comes from the member end moments represented by the pulling force from tension bars and the compression force from concrete compression zones in the connected members. In an ultimate event such as a major earthquake, the member end moments can be extremely high.

Theoretically, the stresses in a joint can be modeled as a biaxial stress condition consisting of the above-mentioned shear stress and the normal tensile and/or compression stresses provided by member axial forces in vertical and horizontal directions. The corresponding principal tensile and compression stresses in diagonal directions can be derived through a Mohr circle analysis. Then the concrete cracking status will be determined by the principal tensile stress level. It should be noted that reinforced concrete is not a true homogeneous or an isotropic material. Therefore, the use of Mohr circle implies a certain degree of approximation. In reality, the complexity of the joint shear problem and the development of cracks in reinforced concrete depends greatly on the reinforcement ratio and the detailing of rebar in the joint region, which will not

be addressed in a concept level design. In order to proceed with the study of the structure, relevant experimental findings on reinforced concrete cracking criteria are adopted here. The diagonal cracking is initiated in the joint when the principal diagonal tension stress is approximately $3.5\sqrt{f'c}$ psi. Once the principal tensile stress exceeds $5\sqrt{f'c}$ psi, joint shear failure in a poorly reinforced section is indicated (Priestley, Seible and Calvi, "Seismic Design And Retrofit of Bridges", 1996).

For a bent in a lateral loading condition, the reversed moment distribution pattern will make one side joint an opening moment and the other a closing moment. The opening moment tends to crack the joint in diagonal direction from the inner corner to the outer one while the closing moment produces the parallel cracks in the orthogonal direction. Because the stress state in a joint depends highly on the internal forces in the connected members, the current joint shear study is therefore associated with the pushover analysis on the same structure.

The first joint in this study is an upper level knee joint in a typical City of Seattle Engineering Department designed bent where a 2′-5″x6′-10$^{1/2}$″ beam joins into a 4′-0″x4′-6″ column. The nominal compressive strength of concrete (f′c) is assumed 5,000 psi. It is important to mention that based on the as-built drawings, the typical embedment length of column reinforcement into the joint is only 4′-6″ instead of the full length of 6′- 10$^{1/2}$″. This reduces the shear plane height significantly, resulting in much higher shear stress than if the rebar was developed to the full height. Also, neither vertical nor horizontal joint shear reinforcement is found in the design. This is substandard to the latest provision in practice, e.g., at least 20% and 10% of total column reinforcement required for vertical and horizontal shear reinforcement respectively in Caltrans Seismic Design Criteria, Version 1.2. By trial pushing of the bent to about 0.28g, rebar slippage starts at column moment equal to 5,000 kip-ft. The principal tensile stress in the joint is computed as 347 psi, i.e., $5.0\sqrt{f'c}$ which implies that joint shear failure would occur at the same time as rebar bond fails. It is also found that the initial crack (the principal tensile strength of $3.5\sqrt{f'c}$) begins when lateral force equivalent to 0.22 g. At the other joint of the bent, the highest combination of the opening moment with dead load moment is as low as 2,200 kip-ft when the lateral load is at the highest level of 0.34 g, resulting in a relatively low tensile stress ($0.6\sqrt{f'c}$) in the joint in this particular case.

In a similar manner, the transposed T-shaped beam-to-column joint at the lower level does not have adequate shear reinforcement either. The main shear stress is produced by the combination of the tension force from the lower column tension bars and the compression from the upper column compression zone. The calculation indicates that the principal tensile stress for the closing moment

model would be $8.1\sqrt{f'c}$ and $5.8\sqrt{f'c}$ for the opening moment model at 0.34 g. That means joint shear failure would occur before the structure fails. Based on $3.5\sqrt{f'c}$ and $5\sqrt{f'c}$ criteria, the lower-level joint cracks at a lateral force of 0.18 g and fails at about 0.24 g.

From the above analysis, the closing moment tends to have more serious impact on a joint, although reversed open, close and reopen moment cycles would be experienced in a real earthquake event. The development of the cracks should be more concentrated and rapid in the direction perpendicular to the inner-to-outer corner diagonal because of the lack of shear reinforcement to sustain the built up stresses in the joint region. This finding is consistent with the observed concrete chipped off on the top outer corner of a bent as happened on Bent 100, near S. Washington Street.

At many locations, the residual joint shear capacity for the post-Nisqually Earthquake condition of existing joints is less than the original substandard design capacity. Due to the catastrophic nature of joint shear failure, the adequacy of the beam-column joints should be addressed. The method of strengthening the shear reinforcement and repairing the concrete can be very costly. One way of fixing the problem is to apply prestressing to the joints. Based on the calculation for a typical City-designed bent, about 1100 kips and 2800 kips jacking forces are needed at upper and lower level, respectively, to keep the joints uncracked until the structure fails by other modes.

Appendix D: Typical Double-Deck Frame Analysis

Overview

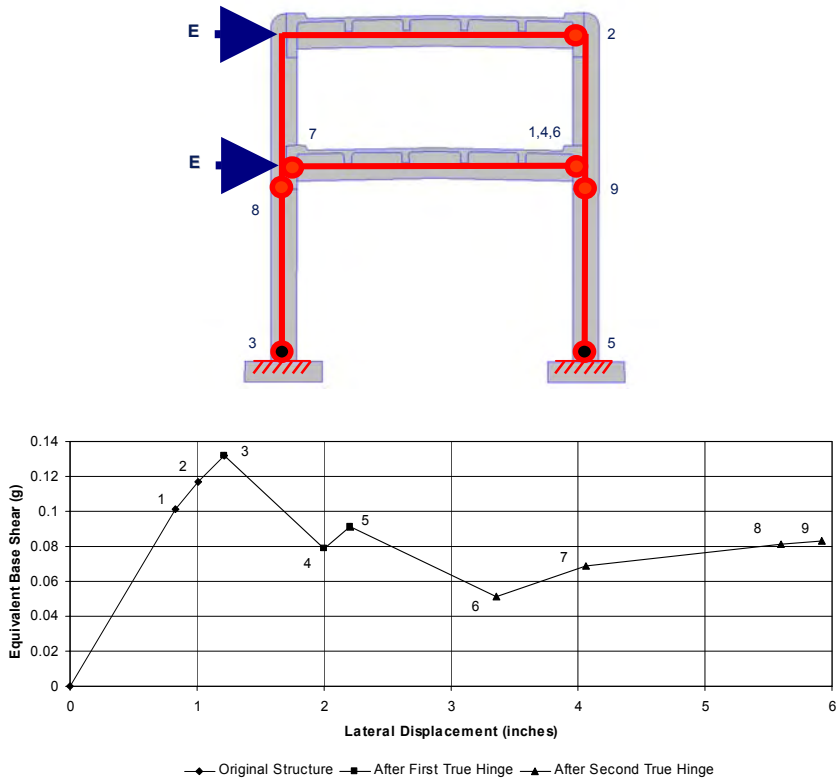
A typical double-deck frame was selected for further refinement of the retrofit design. This was done to demonstrate the retrofit development process in details and to further study the strength and ductility achieved from different levels of retrofit. In addition, the design was refined sufficiently such that quantity for the retrofit could be approximated. The frame selected is comprised of Bents 151 through 154. The frame was designed by WSDOT and constructed in 1956.

The first step of the retrofit process is to determine vulnerabilities inherent in the existing structure. Next, constraints that would limit possible retrofit schemes are identified. Structural analysis and design tools are then utilized iteratively to develop the rebuild/retrofit concepts. The most suitable rebuild/retrofit scheme is retained and refined so that member sizes can be determined.

Vulnerabilities

A non-linear ultimate displacement (pushover) analysis was performed to identify the vulnerabilities in the existing structure. A pushover analysis was selected, as it is an efficient structural capacity assessment tool that also gives valuable information on the lateral deformation capacity, which indicates the ability of the structure to dissipate seismic energy. A more detailed discussion of the pushover analysis method is found in Appendix A.

The pushover analysis indicates that the Viaduct may sustain damage at a spectral acceleration as low as 0.13g. This level of damage may require a closure of the Viaduct for limited repair. The structure still remains stable at this point, but with additional seismic loading, the structure will experience successive and increasing damage leading to collapse. Based upon the equal energy principle applied to the calculated force deflection curve, it was found that the structural limit would be reached between approximately 0.25 g to 0.30 g. This is consistent with the findings by the Alaskan Way Viaduct Structural Sufficiency Review Committee that estimated the structural limit of the double deck frame at 0.26 g. The capacity analysis indicates that the typical double-deck Viaduct frame would become unstable and fail in a design level seismic event. A summary of the failure sequence and the predicted lateral load-displacement curve is illustrated in Figure D-1.



Note: Predicted sequence of failure and the load versus lateral drift characteristic of Bent #152.

Figure D-1: Bent #152 Predicted Sequence of Failure

Development of Retrofit Scheme

The pushover analysis result suggested that a series of damages would occur prior to structural collapse. The next step is to develop a series of retrofit schemes that address the deficiencies. This would permit a better understanding of the structural behavior, a demonstration of design for different levels of performance objectives, and identification of the risks associated with different levels of retrofit.

First Level of Retrofit: Bent-Cap Strengthening

The initial failure mode of the Viaduct is by bar slippage in the pier caps. The pier caps can be strengthened be adding either more reinforcing or prestressing or a combination of both. The pier cap is strengthened by the addition of concrete bolsters to the existing pier caps. The concrete bolsters require coring through the sides of the pier cap to install shear and torsion closed hoop reinforcing. This reinforcing is spliced with mechanical connectors. The pier cap longitudinal reinforcing is placed through holes cored through the deck girders. The beam to column joint also has to be cored to provide the connection to the

prestress anchorage as an anchor block is placed on the outside of the column. Through this type of strengthening the pier cap deficiency can be overcome. However, this construction has added substantial weight to the columns.

The existing columns do not conform to modern tied column standards. The increased column loads make the column less strong and more brittle. In a pushover analysis, this first stage improvement has reduced the capacity of the structure to 0.12 g.

Second Level of Retrofit: Adding Column Jackets

The second stage of retrofit is driven by the actions of the very weak columns. The columns need some confinement and shear reinforcing. The standard retrofit for this deficiency is the steel jacket. However, some strength needs to be added. In this case, a concrete encasement should be constructed. The new main reinforcing should be drilled into the foundation and some bars should be placed in holes cored vertically through the lower deck pier cap. The hoop reinforcing requires two mechanical couplers for each hoop. Additional confinement is added to the beam to column joint, but the core of the joint is still weak.

Refinement of Second Level of Retrofit

As previously mentioned the foundation also needs to be strengthened. The additional weight added to the pier cap bolsters and the column jacket would cause overloads to the piles and pile caps. The solution here is to add small diameter drilled shafts in each pile group corner. The bottom of the existing pile cap is exposed and the concrete chipped away to reveal the ends of the bottom reinforcing. New bars are mechanically connected to the bottom mat. Dowels are drilled and grouted to the sides and top of the pile cap. The column jacket bars are drilled and grouted into the existing pile cap. New pile cap reinforcing is added and concrete is placed.

All new gravity loads are shared with the new piles. If a load starts to cause the existing piles to settle, the additional load would be distributed to the new piles. The new piles at the corners are much stronger than the existing piles so the new piles would resist most of the seismic overturning loads.

As a result of the improvements of this second level retrofit, the capacity of the structure is improved dramatically. An elastic lateral capacity of 0.32 g was calculated in a pushover analysis. The joint strength has been improved somewhat by the additional confinement of the cored reinforcement, bolstered pier cap and new edge girders so full member capacities can most likely be developed. Due to the weak beam to column joint core, there is still limited ductility available. The beams and columns can be developed but plastic hinges would not form due to the under-reinforced joint core. For this reason the

structure is still susceptible to brittle failure at a relatively low displacement and a joint replacement must be done.