MAGNOLIA BRIDGE REPLACEMENT

TYPE, SIZE AND LOCATON STUDY

July 2007

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HNTB

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1. EXECUTIVE SUMMARY

The project will replace the existing 4,400 foot long Magnolia Bridge that was built in 1929 and is showing signs of deterioration and is susceptible to collapse during a seismic event.

The Type, Size and Location effort included extensive investigation and evaluations of multiple alignments, bridge layouts and structure types. The alternatives were developed through an Alignment Study, a Rehabilitation Study, a Bridge Concept Study and a Bridge Alternative Study. The Mayor's office, the Seattle Department of Transportation (SDOT), the Design Advisory Group (DAG) and the public was consulted at each phase of the project.

Alternative A is the preferred alignment alternative to replace the Magnolia Bridge. Alternative A will replace the existing bridge with a new structure immediately south of the existing bridge. Ramps will provide access from the bridge's mid-span to the waterfront and the Port of Seattle's uplands property. Connections at the east and west ends of the bridge will be similar to the existing bridge.

Alignment Alternative A was selected as the preferred alternative because it:

- Responds to local transportation needs
- Was a strong alternative based on environmental and technical analysis
- Received significant neighborhood, business, and governmental agency support, including that of the Port of Seattle
- Provides the least disruption to residents on Magnolia's eastern edge and businesses located under and next to the bridge
- Allows Interbay business owners greater certainty in planning for future expansion or development
- Costs less than other proposed alternatives
- Would result in a Finding of No Significant Impact (FONSI)

The preferred bridge type for replacement of the existing Magnolia Bridge is a cast-in-place concrete box superstructure on drilled shafts supporting curved flared columns. Precast concrete boxes were preferred for the crossing over BNSF railway. A haunched concrete box (arched underside) was selected for the Magnolia Bluff structure and the bridge over 15th Avenue West.

The cast-in-place concrete box was selected as the preferred alternative because it:

- Provides for the optimum pier location at the bluff by eliminating the pier on the side slope.
- Provides a clear span at the Park to optimize future use of the property.
- Provides the opportunity to use aesthetically haunched boxes at the bluff and over 15th Avenue.
- Includes longer spans with fewer piers for less foundation cost.

- Accommodates the transitions for the 15th Avenue Overpass ramp and 23rd on and off ramps with a clean smooth appearance.
- Allows use of longer spans around curved alignments.
- Eliminates pier caps for a clean appearance.

The bridge base cost was estimated at $60.4 million, with an estimated project base cost at $157.9 million in 2006 dollars. Without any risk, construction and right of way cost inflation, estimated at 6.5 and 10.0 percent per year, respectively, would result in year of expenditure costs of $193.6 million through project completion in February 2012. When risks such as market conditions at the time of bid, changing design criteria, and changes in project scope are considered, year of expenditure costs increase. At the 90th percentile probability—where there is a ten percent chance that the cost will be exceeded—the year of expenditure project cost is $261.9 million and the completion date is April 2013.

2. INTRODUCTION

2.1 PROJECT DESCRIPTION

This project will replace the existing bridge with a new bridge with the same street connections to 15th Avenue West, 23rd Avenue West, and West Galer Street on Magnolia Bluff, but not to Terminal 91. The new bridge will have the same number of lanes as the existing bridge, but the lanes will be wider and the outside lanes will accommodate bicycles. The new sidewalk will be wider than the existing bridge sidewalk.

The Magnolia Bridge is the primary structure in one of three corridors linking the Magnolia neighborhood to the rest of the City of Seattle. The other two corridors, West Dravus Street and West Emerson Street, are located north of the Magnolia Bridge.

Magnolia Bridge crosses over Port of Seattle and BNSF facilities situated between Magnolia on the west and Queen Anne on the east. The bridge connects to West Galer Street at the top of the Magnolia Bluff and to 15th Avenue West and West Garfield Street at the foot of Queen Anne Hill.

The Magnolia Bridge is old and requires constant maintenance to keep it open to high traffic loads. The design and construction cost to bring the bridge up to current standards through rehabilitation would be similar to the cost for complete replacement. The bridge was closed to repair landslide damage in 1997 and earthquake damage in early 2001. During the bridge closures, the West Dravus Street and West Emerson Street bridges could not handle the additional traffic without lengthy delays.

The Magnolia Bridge is located in northwest Seattle, in King County, Sections 23 and 26, Township 25N, Range 3E (see Figures 1 and 2). The bridge's latitude is 47 degrees, 38 minutes, 1.3 seconds north; its longitude is 122 degrees, 22 minutes, 46.2 seconds west. Site elevation is 15 feet above sea level at Port of Seattle's Terminal 91, rising to 140 feet at Magnolia Bluff.

The Magnolia Bridge crosses over the Port of Seattle Terminal 91 and BNSF railroad track facilities situated between Magnolia on the west and Queen Anne on the east. The bridge connects to West Galer Street at the top of the Magnolia Bluff and to 15th Avenue West and West

Garfield Street at the foot of Queen Anne Hill. The bridge is one of three corridors connecting the Magnolia neighborhood to the rest of Seattle. The other two corridors are West Dravus Street and West Emerson Street. The Magnolia Bridge carried an annual average weekday traffic volume of 18,900 vehicles in 2005. The West Dravus Street and West Emerson Street 2005 daily volumes at 15th Avenue West were 21,500 and 20,600 vehicles, respectively.

The Magnolia Bridge, with a total project length of about 4,400 feet, spans about 550 feet of Smith Cove, an inlet on the Elliott Bay shoreline. According to mapping by the Washington Department of Ecology (Ecology), the bridge lies within Water Resource Inventory Area (WRIA) 8, the Cedar/Sammamish watershed. The majority of the Elliott Bay shoreline lies within WRIA 9, the Green/Duwamish watershed. Because of Smith Cove's location on the Elliott Bay shoreline, it interacts with water and fish from both watersheds. Smith Cove lies within the Duwamish Hydrologic Unit Code (HUC) 17110013, which includes all of the Magnolia neighborhood and Discovery Park.

2.2 PROCESS

2.2.1 Alignment Study

The process of generating initial design concepts and selecting and evaluating alignments commenced in 2002. Initial alignment alternatives were developed from previous team discussions, the first open house, stakeholder interviews, and previous studies. The evaluation of alignments involved extensive analysis, refinement, and elimination of problematic alignments. The primary means of selecting and evaluating the alignments was through the screening process described below.

The first level screening was performed by the Magnolia Bridge Replacement Study team. The team worked through 25 alignments and eliminated 12 alignments from further consideration. The remaining 13 alignments were consolidated into nine alternatives that were carried forward.

The second level screening recommended three alternatives to carry forward: Alternatives A, D, and H. These three alternatives were developed to a greater level of detail in the environmental impact statement process. Preliminary design was prepared in sufficient detail to evaluate each of the three alternatives.

The three alternatives were evaluated through a screening process and a preferred alternative was recommended. In March 2006, Mayor Nickels directed the Seattle Department of Transportation (SDOT) to choose Alternative A as the preferred alternative to replace the Magnolia Bridge. Alternative A will replace the existing bridge with a new structure immediately south of the existing bridge. Ramps will provide access from the bridge's mid-span to the waterfront and the Port of Seattle's uplands property. Connections at the east and west ends of the bridge will be similar to the existing bridge.

2.2.2 Existing Bridge Inspection and Rehabilitation Study

Magnolia Bridge Rehabilitation Alternative was added to the project to determine if bridge rehabilitation is a viable alternative to bridge replacement. In order to evaluate and develop the rehabilitation alternative, a feasibility study was authorized to identify structural elements that do not meet current design requirements, identify a rehabilitation concept for those structural elements, and estimate the cost to modify the structural elements to conform to current design requirements.

The intent of the Rehabilitation Alternative feasibility study was to develop a rehabilitation concept to bring the existing bridge structure up to current design standards. Existing geometric and structural deficiencies were identified and solutions developed to rehabilitate or replace the structural elements to bring the bridge into conformance with current AASHTO Load and Resistance Factor Design (LRFD) code requirements.

2.2.3 Bridge Concept Study

During concept development, Bridge Engineers, Architects, and Urban Planners worked together to conduct architectural/structural studies on bridge aesthetics, span length, pier configuration, and superstructure type. Structure types and configurations were investigated during concept development to determine the most feasible structure type and layouts for the project. Black-line sketches were created for the span length study, pier configuration study, and the superstructure type study to assist in selecting bridge type and layout. At the end of concept development, a qualitative matrix was used to screen the concepts to select three alternatives that best meet the project goals and objectives. The screening was completed as a coordinated effort with the City and the Design Advisory Group.

2.2.4 Bridge Alternative Study

The three alternatives were advanced through the Bridge Alternative Study which included preliminary design and analysis to determine span arrangement, structure depth, foundations, aesthetics, public feedback, quantities and costs. A complete set of plan and elevation drawings was prepared for each of the selected bridge types. The preferred structure type was selected from the three bridge types at the completion of the Bridge Alternative Study through a coordinated effort with the City and the Design Advisory Group.

For the three final alternatives, plan and elevation sheets, typical section sheets, and 3-D renderings were prepared for each alternative. Superstructure, substructure, and foundation systems were developed in sufficient detail to determine estimated quantities for cost estimating. Seismic models were generated for typical units to determine the load capacity requirements for the substructure and foundations.

Following selection of recommended structure type, the layout was finalized to optimize span depths and pier locations. A final set of Type, Size and Location plans were prepared to identify layout of the preferred bridge concept.

2.3 PURPOSE

The purpose of the Magnolia Bridge Replacement Project is to replace the existing Magnolia Bridge structure, approaches, and related arterial connections with facilities that maintain convenient and reliable vehicular and non-motorized access between the Magnolia community and the rest of the City of Seattle. The bridge provides an important link to the Magnolia community in Seattle (see Figure 1 and Figure 2). Because the existing bridge provides the only public vehicular access to the land between North Bay, also referred to as Terminal 91, Smith Cove Park, Elliott Bay Marina, and U.S. Navy property, the project purpose also includes maintenance of access to these areas.

2.4 NEED

2.4.1 Structural Deficiencies

The City of Seattle has identified the Magnolia Bridge as an important bridge that should remain standing following a “design” seismic event (an earthquake with a peak ground acceleration of 0.3G that is anticipated to happen every 475 years and may measure 7.5 on the Richter scale). Even with the repairs completed following the February 2001 earthquake, the existing bridge is susceptible to severe damage and collapse from an earthquake that is less severe than the “design” seismic event.

The original bridge was constructed in 1929 and has been modified, strengthened, and repaired several times. The west end of the bridge was damaged by a landslide in 1997, requiring repair and replacement of bridge columns and bracing, the construction of six additional supports, and a retaining wall north of the bridge to stabilize the bluff from further landslides. Repairs after the 2001 earthquake included replacement of column bracing at 27 of the 81 bridge supports. A partial seismic retrofit of the single-span bridge structure over 15th Avenue West was completed in 2001. The other spans were not upgraded.

Inspections of the bridge conclude that the concrete structure is showing signs of deterioration. The concrete is cracking and spalling at many locations, apparently related to corrosion of the reinforcing steel. The bridge requires constant maintenance in order to maintain its load capacity, but there does not appear to be any immediate load capacity problem. The existing foundations have insufficient capacity to handle the lateral load and uplift forces that would be generated by a “design” seismic event. The existing foundations do not extend below the soils that could liquefy during a “design” seismic event. If the soils were to liquefy, the foundations would lose their vertical-load-carrying ability and the structure would collapse.

2.4.2 System Linkage

There are three roadway connections from the Magnolia community, with more than 20,000 residents, to the rest of Seattle. As the southernmost of the three connections, the Magnolia Bridge is the most direct route for much of south and west Magnolia to downtown Seattle and the regional freeway system.

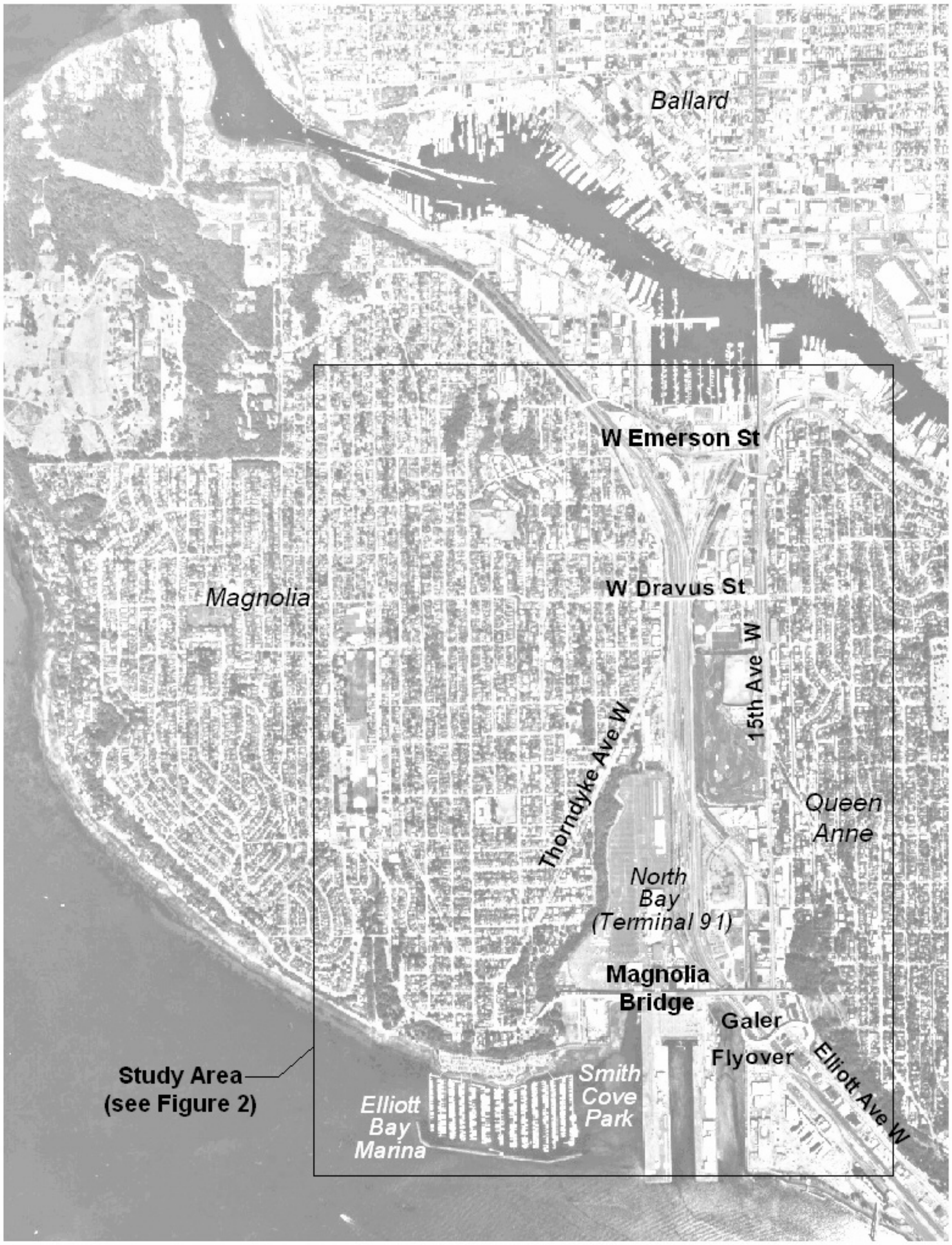


Figure 1
Vicinity Map

In meetings with the public and the Seattle Fire Department, the importance of this route for emergency services has been emphasized. The loss of use of this bridge in 1997 and again in 2001 demonstrated to the City that the remaining two bridges do not provide acceptable operation. During the bridge closure following the February 2001 earthquake, the City addressed community concerns about reduced emergency response time to medical facilities outside of Magnolia by stationing paramedics at Fire Station 41 (2416 34th Avenue West) 24 hours a day.

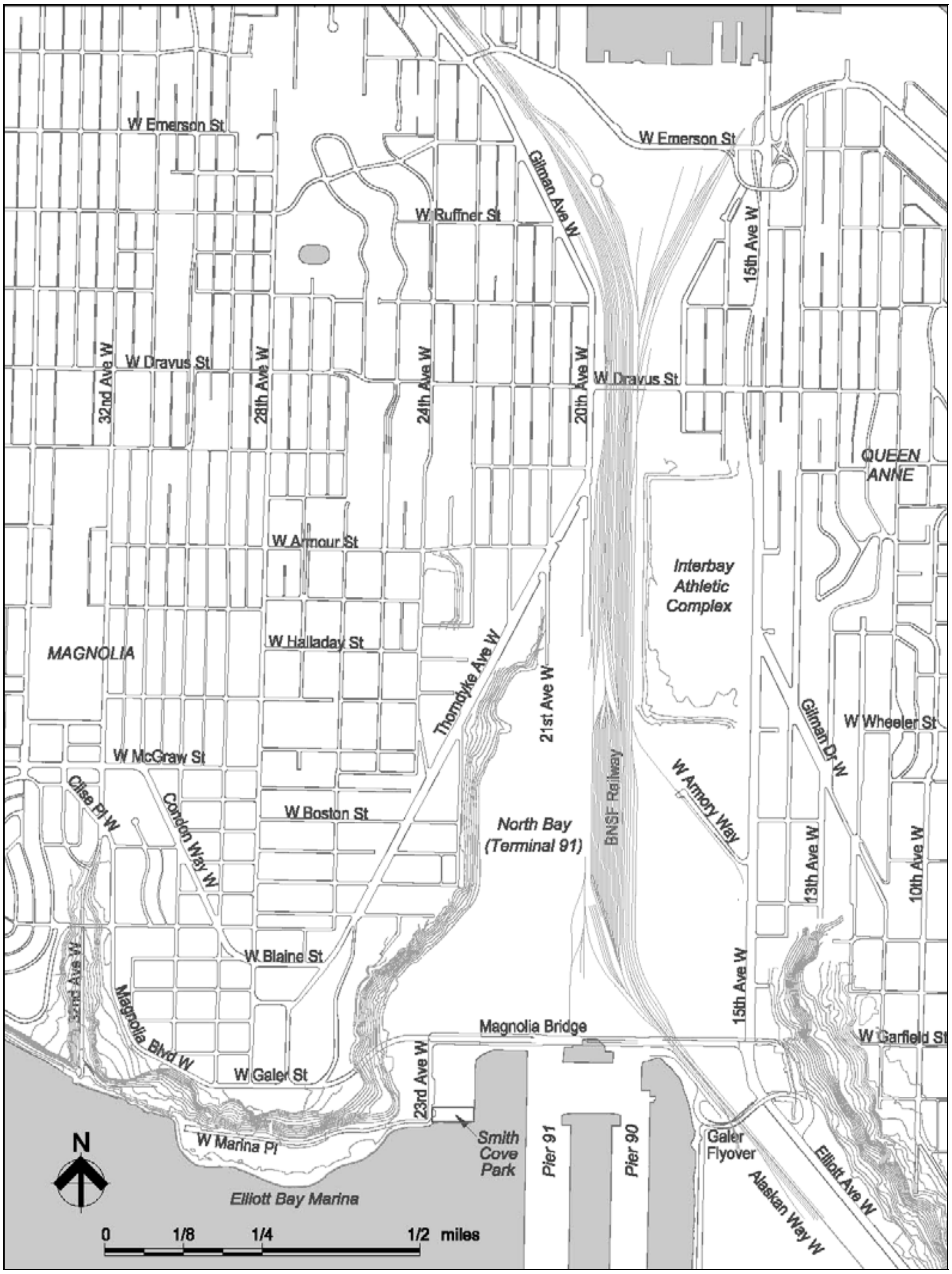


Figure 2
Study Area

2.4.3 Traffic Capacity

The three Magnolia community connections to the 15th Avenue West corridor are adequate for the present volume of traffic. Each of the three connections carries 30 to 35 percent of the 60,100 daily vehicle trips (2001 counts) in and out of the Magnolia community. Loss of the use of the Magnolia Bridge for several months after the February 2001 earthquake, and in 1997 following the landslide at the west end of the bridge, resulted in lengthy 15- to 30-minute delays and increased trip lengths for many of the users of the Magnolia Bridge. These users were required to use one of the two remaining bridges at West Dravus Street and West Emerson Street. Travel patterns in the Magnolia community changed substantially resulting in negative impacts on local neighborhood streets. The increase of traffic through the West Dravus Street and West Emerson Street connections also resulted in congestion and delay for the regular users of these routes. Losing the use of any one of these three bridges would result in redirected traffic volumes that would overwhelm the capacity of the remaining two bridges.

2.4.4 Modal Interrelationships

The Magnolia Bridge carries three of the four local transit routes serving Magnolia and downtown Seattle destinations. The topography of the east side of Magnolia, East Hill, would make access to the 15th Avenue West corridor via the West Dravus Street Bridge a circuitous route for transit. Use of the West Emerson Street connection to 15th Avenue West would add significant distance and travel time for most trips between Magnolia and downtown Seattle.

The Magnolia Bridge has pedestrian facilities connecting the Magnolia neighborhood to Smith Cove Park and Elliott Bay Marina as well as to 15th Avenue West/Elliott Avenue West. These facilities need to be maintained. The Elliott Bay multi-use trail connects Magnolia with downtown Seattle through Myrtle Edwards Park. The trail passes under the Magnolia Bridge along the west side of the BNSF rail yard, but there are no direct connections to the bridge.

Bicycle facilities on Magnolia Bridge need to be maintained or improved. Even with the steep (about 6.3 percent) grade, bicyclists use the Magnolia Bridge in both directions. There are no bike lanes on the bridge, so cyclists use the traffic lanes and sidewalks. Once cyclists cross the bridge, they must either travel with motor vehicles on Elliott Avenue West or find a way back to the Elliott Bay Trail using local east-west streets such as the Galer Flyover.

2.4.5 Transportation Demand

The existing Magnolia Bridge provides automobile access for Port of Seattle North Bay (Terminal 91) to and from Elliott Avenue West/15th Avenue West. Truck access between Terminal 91 and Elliott Avenue West/15th Avenue West is accommodated via the Galer Flyover. Future planned expansion of the Amgen facility on Alaskan Way West and redevelopment of underutilized portions of North Bay and other areas of Interbay will increase demand for traffic access to the Elliott Avenue West/15th Avenue West corridor. The Port of Seattle has a master planning process under way (July 2003) for its North Bay (Terminal 91) property and the Washington National Guard property east of the BNSF railroad between West Garfield Street and West Armory Way. This area contains 82 acres available for redevelopment. There are also 20 or more acres of private property available for redevelopment east of the BNSF Railway between West Wheeler Street and West Armory Way. Redevelopment of the North Bay property will include public surface streets with connections to the replacement for the Magnolia Bridge. Forecasts of future (year 2030) traffic demand indicate that the access provided by the Galer Flyover and West Dravus Street would be inadequate. The capacity provided by the existing Magnolia Bridge or its replacement would also be needed.

2.4.6 Legislation

Seattle Ordinance 120957, passed in October 2002, requires that the Magnolia Bridge Replacement Study: (1) identify possible additional surface roads from Magnolia to the waterfront (avoiding 15th Avenue West and the railroad tracks); (2) obtain community input on the proposed roads; and (3) identify the cost for such roads and include it in the total cost developed in the Magnolia Bridge Replacement Study.

2.5 PROJECT GOALS AND PRINCIPLES

2.5.1 Project Goals

Through a public involvement program including interviews with individual stakeholders, an open house and the Design Advisory Group, several common ideas or desires for the replacement facility have been expressed. These common ideas and desires are shown below and form the goals for the project:

- Provide a safer and more reliable route(s) to Magnolia.
- Maintain Magnolia's aesthetic qualities and community feel.
- Provide additional access points into Magnolia.
- Provide a route that will support Magnolia Village businesses.
- Maintain or improve traffic flow on the 15th Avenue W. corridor.
- Support redevelopment of vacant or underutilized Interbay properties.
- Improve access to the waterfront to and from Magnolia.
- Minimize impact to existing traffic patterns during construction.
- Minimize impact to the existing houses, businesses and right-of way.
- Improve public access to the waterfront.
- Maintain or improve the level of bicycle and pedestrian connections within and beyond the project area.

2.5.2 City of Seattle Principles

In addition to these goals, the City of Seattle will develop the replacement facility using the following principles:

- Provide fair access to information regarding project progress to community, mass media and interested individuals.
- Create a transparent process of alternatives development, evaluation and selection.
- Avoid cost overruns by identifying all major contributing factors.
- Consider aesthetics when developing the new structure.
- Keep estimated probable costs of construction in line with industry standards for similar projects.

2.6 EXISTING BRIDGE CONDITIONS

An Existing Bridge Condition Report was prepared; a copy of the report is included in Appendix A.

Figure 3
Existing Bridge

2.6.1 Description of Existing Bridge

The construction of a bridge at the Magnolia Bridge site was started in 1913. The structure constructed at that time consisted of a timber trestle carrying 23rd Avenue West over the Great Northern Railroad.

In 1929, this original structure was replaced with the West Garfield Street Viaduct, now known as the Magnolia Bridge, which remains in use today. The structure laid out in 1929 extended from 15th Avenue West to Dartmouth Avenue crossing a number of streets and rail tracks. The structure itself was made up of reinforced concrete slab and girder spans, steel girder spans (over the railroad), and reinforced concrete trusses. Timber trestles connected to 23rd Avenue West to and from the north. It is assumed that these timber trestles were removed by the Navy when they occupied Piers 90 and 91beginning in 1942.

In 1953, the slabs were strengthened between Bents 22 and 28 by adding steel bracing underneath.

In 1957, the structure was lengthened to the east approximately 760 feet. This extended structure, carrying a westbound lane of West Garfield Street over 15th Avenue West, consists of concrete girder, steel box girder span over 15th Avenue West and steel plate girder spans over the railroad tracks.

In 1960, much of the existing concrete longitudinal bracing was replaced with steel bracing between Bent 56 and Bent 78.

In 1962, steel trusses to strengthen the deck slabs were added to each span between Bent 34 and Bent 61 and between Bent 76 and the West Abutment. New transverse floor beams and steel columns were added between Bent 61 and Bent 76. This rehabilitation also included the replacement of expansion and/or fixed joints in fourteen suspended spans located between Bent 38 and Bent 80, the full replacement of one of the suspended spans, and the replacement of the bridge railing between Bent 46 and the West Abutment. The north sidewalk was removed between Bent 46 and the West Abutment.

The expansion joints were rehabilitated on the eastern half of the structure in 1969, followed by further rehabilitation of the expansion joints on the western half of the structure in 1975. Additional stiffening trusses were added to the spans between Bent 12 and Bent 35 in 1974.

In 1982, the bridge railing was again replaced in the western half of the bridge (between Bent 40 and the West Abutment) with Jersey type barrier.

New off and on ramps to the Elliott Bay Marina were constructed in 1991. The ramps consist of a prestressed concrete slab supported on steel pile bents. Also included in this bid package were repairs of concrete spalls and cracks at existing Bents 43, 44, 45, and 46 and the strengthening of the existing portions of the ramps to an HS20 live load capacity.

In 1985, the bridge deck was repaired and covered with a Latex Modified Concrete wearing surface between Bent 43 and the West Abutment.

Emergency repairs were necessitated by a landslide that occurred on January 2, 1997 on the north side of the west end of the bridge. This slide damaged the steel and concrete columns and bracing between Bents 78 and 79, 79 and 80, and 80 and 81 of the Magnolia Bridge. The City of Seattle prepared plans addressing the damage caused by this landslide. Repairs completed included the replacement of the longitudinal bracing between Bents 76 and 77, 77 and 78, 78 and 79, 79 and 80, 80 and 81, and 81 and 82. The lower transverse bracing members were replaced at Bents 77, 78, 79, and 80. Additional four-column towers supported on drilled shafts were constructed between Bents 76 and 77, 77 and 78, and 78 and 79. Cleaning, patching, and epoxy injection of damaged bridge columns and cross members were done as directed by the engineer during this repair.

On February 28, 2001, the Nisqually Earthquake damaged the structure. This damage was mostly localized in the lateral bracing members of the column bents between Bents 49 and 75. Additional damage occurred in the concrete truss spans of the superstructure. Repairs included the replacement of the concrete transverse bracing of Bents 49 through 75 with steel bracing. Concrete spalls were patched in the longitudinal bracing between Bents 55 and 56, 59 and 60, and 67 and 68. Epoxy injection of concrete cracks was performed in the longitudinal bracing between Bents 50 and 51, 55 and 56, 59 and 60, and 61 and 62. The concrete trusses were also repaired by patching spalls and epoxy injection of the damaged concrete.

As part of the West Galer Street Flyover construction in 2001, a partial seismic retrofit was constructed on the portion of the Magnolia Bridge over 15th Avenue West. The columns and foundations at Piers 7 and 8 (piers adjacent to 15th Avenue West) were retrofit, transverse shear blocks were added to the connection of the superstructure at Piers 7 and 8, and longitudinal restrainers were added between spans at Piers 6, 7, 8 and 9.

2.6.2 Inspection Conclusions

Based on field observations of the Magnolia Street Bridge, the opinion is that the concrete structure is showing signs of aging. The concrete cover is cracking and spalling at many locations along the length of the bridge. The observed distress of the concrete appears to be primarily related to corrosion of the underlying reinforcing steel. Based only on a visual inspection, there does not appear to be any indication that the structure has a serious load capacity problem. However, since the bridge is more than 70 years old and has deteriorated, major rehabilitation or replacement should be planned. Numerous local repairs have been made over the years.

2.7 EXISTING SITE CONDITIONS

2.7.1 Roadway Classification

Functional classifications of existing arterial facilities located in the project area are tabulated below:

2.7.1.1 Principal Arterials:

- Elliott Avenue West
- 15th Avenue West
- W Dravus Street (between 15th Avenue W and 20th Avenue West)

2.7.1.2 Minor Arterials:

- West Garfield Street
- West Galer Street
- Magnolia Boulevard West
- West Clise Way
- Condon Way West
- Thorndyke Avenue West
- 20th Avenue West
- West Dravus Street (west of 20th Avenue West)
- Gilman Drive West

2.7.2 Geotechnical

2.7.2.1 Interbay Golf Course/ Landfill Area

The Interbay golf course/landfill area is located between Thorndyke Avenue West and 15th Avenue West (West to East) and West Dravus Street and West Wheeler Street (North to South). Generally, borings in this area consisted of very loose to dense, silty, gravelly sand and medium stiff to hard, clayey silt and silty clay. Underlying the fill and land refuse were intertidal deposits consisting of loose to medium dense, silty sand and sandy silt an stiff, clayey silt, which were further underlain by glacial deposits consisting of dense to very dense sand and gravel, and hard clay and silt. Within the golf course, glacial deposits were encountered at depths ranging from 30 to 90 feet below ground surface. To the south of the golf course, glacial deposits were encountered at depths ranging from 12 to 55 feet; to the east of the golf course, up the hill of east 15th Avenue West, glacial deposits were encountered as shallow as 2 feet below ground surface. North of the golf course, in the vicinity of West Dravas Street Bridge, glacial deposits were encountered about 70 feet below ground surface. Groundwater depths near the Interbay Golf Course area ranged from 15 to 28 feet below ground surface.

2.7.2.2 Magnolia Bridge and 15th Avenue West Area

This area includes the land south of the Magnolia Bridge (in the project area) between Pier 91 and West Galer Street ramp to the Magnolia Bridge. It is a former marine, mudflat, which is part of Smith Cove. In the early 1900s, the area was filled in to the present existing grade. Subsurface conditions consist of suficial fill and/or colluvial soils overlying mudflat and glacially overridden sediments. Fill encountered consisted of very loose to medium dens sand and soft clay mixed in with colluvium. The colluvium encountered in this area is likely a result of numerous historic landslides that occurred on the western slope of Queen Anne Hill. The underlying mudflat deposits consisted of very loose to loose silt with some thin layers of soft clay. Approximately 70 feet east of 15th Avenue West, these mudflat deposits generally were not encountered.

Underlying the mudflat deposits were glacial deposits consisting of very stiff to hard clay and silt, which were encountered at depths ranging between 32 and 105 feet below ground surface (increasing in depth westward from 15th Avenue West). Groundwater encountered near the eastern side of the Magnolia Bridge ranged in depth from 5 to 18 feet below ground surface.

2.7.2.3 West of Elliot Bay Tidal Land

This area includes the portion in the project area west of the Elliot Bay Tidal land. Generally in this area, subsurface explorations revealed dense to very dense silty sand and gravel overlying very stiff to hard clayey or sandy silt. Further north, along Thorndyke Avenue West, some very loose to medium glacial deposits were encountered below ground surface. Groundwater was encountered at depths ranging from 5 to 14 feet, west of the bridge, and between 20 to 39 feet further north along Thorndyke Avenue West.

2.7.3 Utilities

2.7.3.1 Utility Services

Public utility services within the study area are numerous and fall under both city and county jurisdictions. They include water, sanitary sewer and stormwater drainage, wastewater treatment, natural gas, electricity, telecommunications, and garbage and recycling services. Table 1 lists the local service providers for the identified utilities within the study area and is followed by a brief discussion of each provider. Existing utility service mains are generally located within the public right-of-way. Service is extended to customers through overhead, side/lateral, and branch connections. Many utility mains span the North Bay/Terminal 91 property in multiple locations.

Table 1
Local Utility Service Providers

Utility Service	Service Provider
Water Service	Seattle Public Utilities
Sanitary Sewer and Drainage Service	Seattle Public Utilities
Wastewater Treatment	King County
Natural Gas	Puget Sound Energy
Electricity	Seattle City Light
Telecommunications	Qwest
Garbage and Recycling	Seattle Public Utilities

Source: HNTB Corporation and Mirai Associates 2003.

2.7.3.2 Water Service

Water service within the study area is provided and maintained by Seattle Public Utilities (SPU). The municipal water utility was established in 1890, when the City of Seattle purchased the Spring Hill Water Company and the Union Water Company. Potable water is supplied to Seattle customers through the Cedar River Pipeline, South Fork Tolt River Pipeline, and from three wells in the Highline Well Field. These pipelines distribute water to mains that are generally located within the public right-of-way.

2.7.3.3 Storm and Sanitary Sewer Services

SPU is responsible for managing and maintaining drainage services, including stormwater drains and sanitary (wastewater) sewers and pump stations. Stormwater runoff and wastewater flows are transported within conveyance infrastructure such as storm drains, sewer mains, combined storm and sanitary sewer mains, and overflow systems. Conveyance systems may also use ditches, culverts, and creeks. SPU drainage services include operation, maintenance, and repair of storm and sanitary sewer infrastructure, construction of trunk lines and detention ponds for alleviation of flood and erosion problems, preservation and enhancement of creek habitat, and protection of surface water quality through regulated installation of water quality controls and by positive prevention efforts.

2.7.3.4 Wastewater Treatment

King County provides wastewater treatment service within the City of Seattle. King County currently operates and maintains three treatment plants: West Point Treatment Plant, South Treatment Plant, and Vashon Treatment Plant. A fourth treatment plant, Brightwater, is planned for construction. The County system includes 42 pump stations and 19 regulator stations. Combined sewer overflows (CSOs), or wastewater discharged during high volume periods, is also a component of the King County system. The South Magnolia CSO storage tank is among the recommended improvements included in the Regional Wastewater Services Plan 2000-2030; this project is roughly scheduled for 2010.

2.7.3.5 Natural Gas

Puget Sound Energy (PSE) supplies natural gas to the study area. Natural gas is purchased in the summer and stored in underground reservoirs until it is distributed during the winter. PSE is owned by investors and regulated by the State of Washington Utilities and Transportation Commission. PSE is responsible for extension of natural gas lines and connections of new permanent service lines. Construction and engineering services for natural gas improvements are provided under a contractual agreement with Potelco, Inc. and Pilchuck Contractors, Inc.

2.7.3.6 Electricity

Seattle City Light (SCL) has been providing electricity to local residences and business, and to public streets since 1910. SCL is a non-profit public utility that is owned by Seattle citizens and is governed by the City. SCL services include installation and/or relocation of electrical infrastructure, temporary connections or disconnections, and electrical equipment repair. SCL also provides technical information regarding electrical services and establishes programs for the conservation of electricity.

2.7.3.7 Telecommunications

Telecommunications services encompasses both voice and data networks, such as telephone, DSL (digital subscriber line), internet, wireless, long distance, and directory services. Qwest provides these services to 14 western states including Washington. Qwest is responsible for installation, repair, and improvement of telecommunications infrastructure.

2.7.3.8 Garbage and Recycling

SPU operates and maintains garbage and recycling services for residential customers. Since 2001, SPU has operated under contractual arrangements with Rabanco Companies and Waste Management for the provision of commercial garbage collection. Rabanco Companies is known as Emerald City Disposal and Recycling. Rabanco Companies serves businesses within the study area. Private companies, hired at the expense of the business owner, provide commercial recycling services.

2.7.3.9 Major Utility Infrastructure

Figure 5 depict the existing utilities (public and private) within the study area based on information made available by the utility purveyors. The figures highlight the locations of major utility infrastructure within the study area, which includes the following:

- Twin 48-inch and 96-inch King County sanitary sewer force mains, which run both north-south and east-west across the Terminal 91 property (Figure 4 and Figure 5);
- King County lift station located on Alaskan Way West;
- City of Seattle CSO line situated on the east and west sides of Terminal 91; and
- The gas line corridor that runs through Terminal 91.

For security reasons, power facilities are not depicted in the figures. Power facilities (both overhead and underground) are prevalent in the study area, primarily along the 15th Avenue West corridor (overhead transmission) and within the existing Magnolia Bridge corridor and Port property (overhead and underground facilities).

Port of Seattle utility lines are interspersed throughout the North Bay/Terminal 91 complex and are known to vary in terms of their age and functioning condition. For example, existing Terminal 91 sanitary sewer mains are reported to be in “severe distress” and would require evaluation prior to undertaking potential relocation (Birr, pers. comm., 2003). Public utility main lines are generally housed within existing right-of-way. Service is extended to individual property owners and to Port tenants through smaller side/lateral and branch connections.

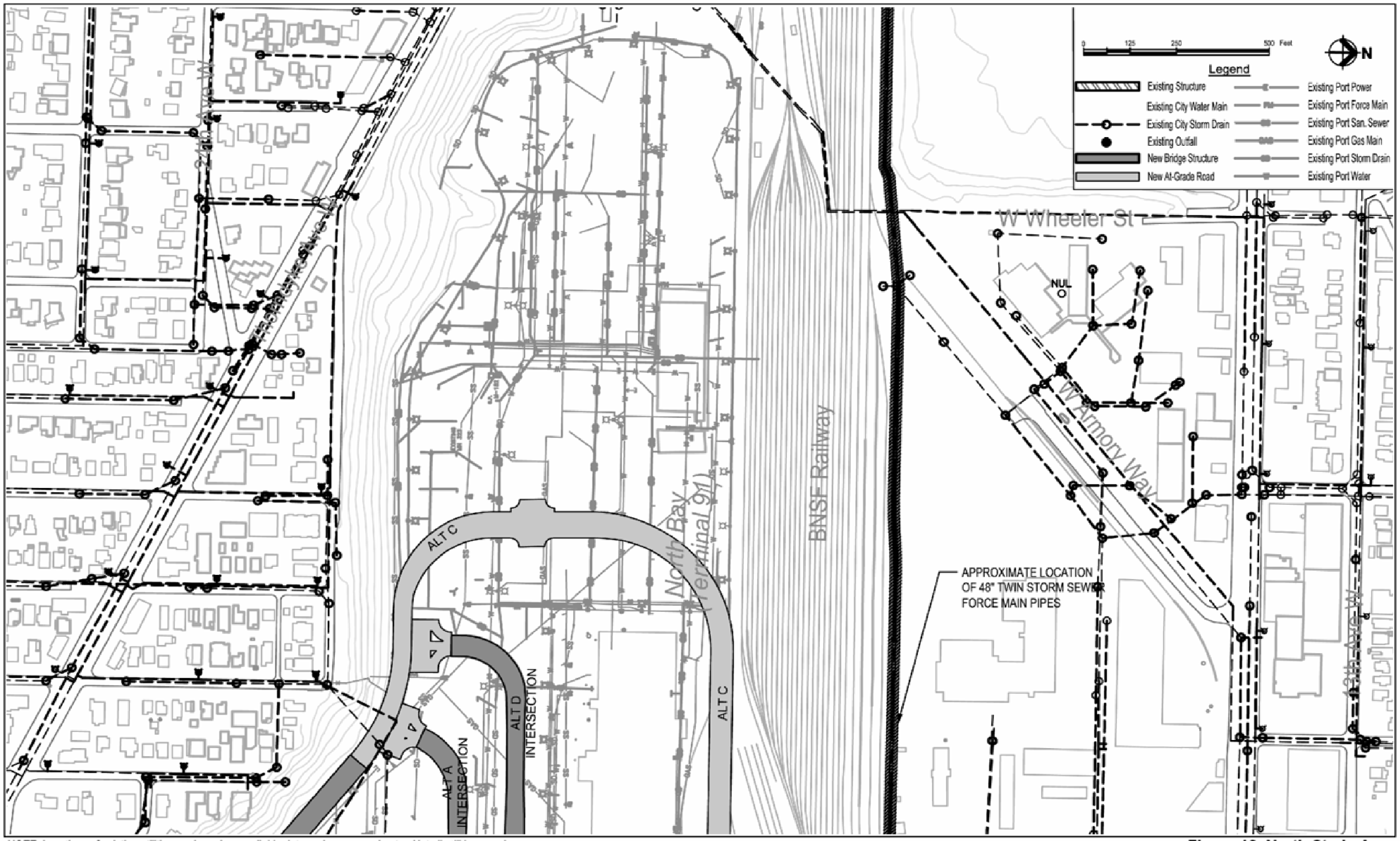


Figure 4
Existing Utilities – North Study Area

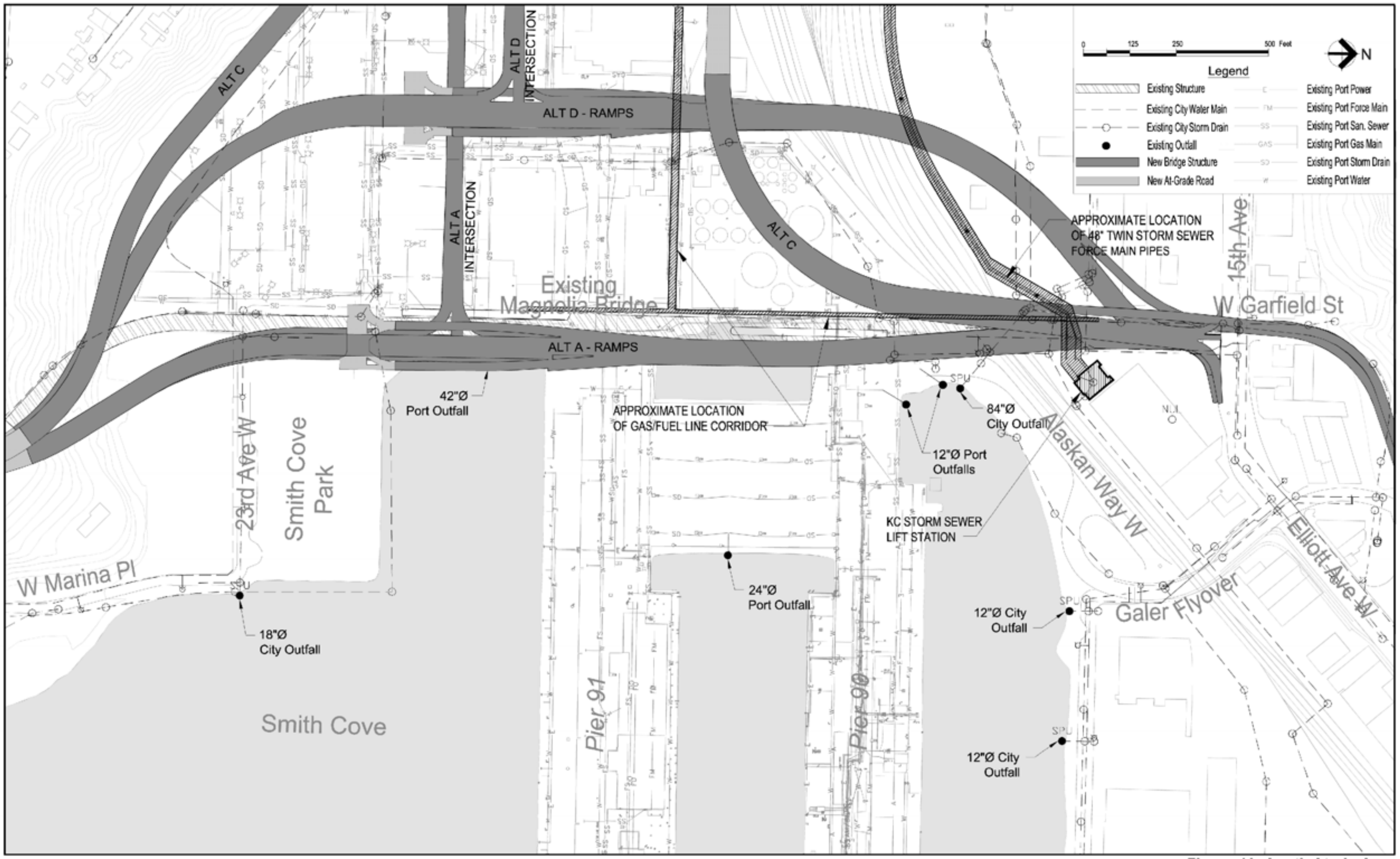


Figure 5
Existing Utilities – South Study Area

2.7.4 Land Use and Zoning

2.7.4.1 Existing Land Use

Figure 6 shows the general locations of existing land uses in the study area. The alternatives would primarily be located over land used for industrial and commercial purposes, with western connections to residential areas in the Magnolia neighborhood.

Single-family residential neighborhoods are located to the east and west of the project site, on the upper portions of the Magnolia Bluff and Queen Anne Hill. Multifamily residential buildings are generally located on the lower portions of both hills closer to the project site.

Interbay, which is the lowland area between Magnolia and Queen Anne, is used for a mix of industrial and commercial businesses. A variety of retail commercial, service, small office, and light industrial uses are located along the Elliott Avenue West/15th Avenue West corridor. The National Guard Armory is located to the west of this corridor, and BNSF railroad tracks run up the middle of the industrial area in Interbay. The Amgen offices are located along Elliott Bay to the southeast of the existing bridge.

The Port's North Bay/Terminal 91 property is located to the west of the railroad tracks and east of the Magnolia Bluff. The Port is a major landholder in the study area. Major current uses on Port property include cold storage, fish processing, fuel distribution, and vehicle storage for the Seattle School District.

Land uses to the north include a mix of light industrial and multifamily residential uses on the west side of the railroad tracks, the Interbay Golf Course and P-Patch on the east side of the tracks, and commercial/retail uses along Thorndyke Avenue West, 20th Avenue West, and 15th Avenue West.

Figure 6
Existing Land Use

2.7.4.2 Existing Zoning

Figure 7 shows the current zoning designations in the project vicinity. Generally, existing land uses described above are consistent with the zoning designations.

The uphill portions of the Magnolia and Queen Anne neighborhoods are zoned Residential Single Family 5000, with lower areas on both hills zoned Lowrise 1, 2, or 3. Lowrise zoning designations allow multifamily residential development 25 to 30 feet in height, with densities of one dwelling unit per 800 to 1,600 square feet of lot area.

The Port's North Bay/Terminal 91, including properties south of the bridge along Elliott Avenue West, and BNSF Railway property are zoned General Industrial 1/45 (IG1), which allows industrial development in areas characterized as having access to waterways and rail. This zoning designation indicates a height limit of 45 feet. The National Guard Armory and properties located along 15th Avenue West, south of West Armory Way, are zoned General Industrial 2/45 (IG2), which is intended to allow a broad mix of activities.

Some property fronting the eastern side of 15th Avenue West (south of West Armory Way) and fronting both sides of Elliott Way West (south of the existing bridge) is zoned Industrial Commercial. This zone is intended to promote development of businesses that incorporate a mix of industrial and commercial activities. Some areas to the east of 15th Avenue West are zoned Industrial Buffer (IB), which provides additional development regulations to limit impacts on neighboring non-industrial areas.

Parcels fronting 15th Avenue West north of West Armory Way are zoned Commercial 1 and Commercial 2, which indicate an auto-oriented, primarily retail/service commercial area that serves surrounding neighborhoods and the larger community or citywide clientele. A Neighborhood Commercial zone (NC-3), which allows less intensive commercial uses, is located along 15th Avenue West north of Gilman Drive West.

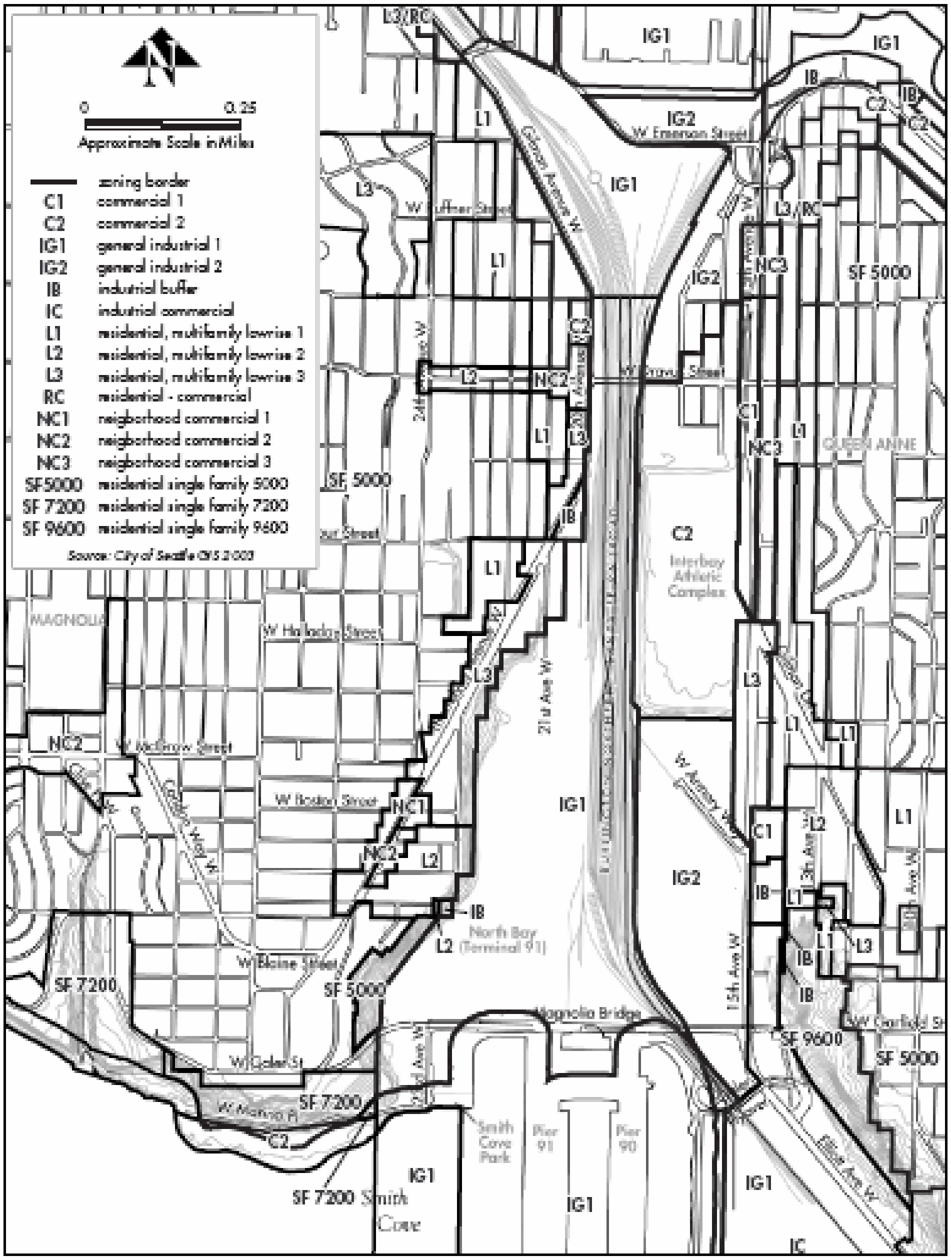


Figure 7
Existing Zoning

2.7.5 Environmental

For a discussion of the Project environmental elements, refer to the Magnolia Bridge Replacement Project Environmental Assessment and the NEPA Magnolia Bridge Replacement Project Biological Assessment.

2.7.6 Hazardous Materials

Historic records for the project area were reviewed along with local, state, and federal environmental databases to identify former and current land uses that could result in contamination of soil and/or groundwater along the New Magnolia Bridge alignment. These sites included metal manufacturers, junk and wrecking yards, auto repair shops, gasoline stations/bulk fuel distributors, print shops, laundries, bulk fuel terminals, railroads, and other industrial sites. Properties adjacent to the proposed project that store or have stored heating oil were also included. Eighteen of the 35 sites are on or adjacent to the New Magnolia Bridge alignment. For information regarding site locations, refer to the Magnolia Bridge Replacement Project Environmental Assessment

3. REHABILITATION ALTERNATIVE

Magnolia Bridge Rehabilitation Alternative was added to the project to determine if bridge rehabilitation is a viable alternative to bridge replacement. In order to evaluate and develop the rehabilitation alternative, a feasibility study was authorized to identify structural elements that do not meet current design requirements, identify a rehabilitation concept for those structural elements, and estimate the cost to modify the structural elements to conform to current design requirements. The complete Rehabilitation Alternative Study Report is included in Appendix B.

The intent of the Rehabilitation Alternative feasibility study was to develop a rehabilitation concept to bring the existing bridge structure up to current design standards. Existing geometric and structural deficiencies were identified and solutions developed to rehabilitate or replace the structural elements to bring the bridge into conformance with current AASHTO Load and Resistance Factor Design (LRFD) code requirements.

3.1 REHABILITATION DESIGN CRITERIA

A. GOVERNING CRITERIA
   1. AASHTO LRFD “Bridge Design Specifications,” Customary U.S. Units, Third Edition, 2004, with 2005 interim.
   2. WSDOT LRFD Bridge Design Manual, July 2005.
   3. WSDOT Geotechnical Design Manual, latest version.
   4. WSDOT Highway Design Manual, latest version.
   5. MCEER/ATC-49, Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, 2003.

B. LAYOUT
   1. The spans and general arrangement of the structure are shown on the existing Magnolia Bridge Plans. The superstructure will be replaced in kind with no additional width added to bridge. The existing bridge width meets existing local agency standards.

2. Design Speed: 35 MPH

C. DESIGN LOADS
1. Dead Load
    a. Structural Dead Loads
        1) Concrete = 160 pcf
        2) Structural Steel = 490 pcf
    b. Superimposed Dead Load
        1) No allowance shall be made for the weight of initial wearing surface.
        2) An allowance shall be made for the weight of a 2" future wearing surface (25 psf).
        3) An allowance of 100 pounds per linear foot shall be provided for utilities.
        4) One 6' wide sidewalk, with 6" depth, on one side of bridge structure.
        5) (1) WSDOT 34" Single Slope Traffic Barrier with a weight of 475 pounds per lineal foot for each barrier. (1) WSDOT 32" Pedestrian Barrier with a weight of 450 pounds per lineal foot, including metal handrail.
2. Live Load - Vehicular live load shall be AASHTO LRFD HL-93.
3. Pedestrian Load – Pedestrian live loads shall be applied in accordance with AASHTO LRFD 3.6.1.6.
4. Seismic Forces
    a. The structure shall be analyzed and designed in accordance with the AASHTO LRFD and WSDOT BDM LRFD Chapter 4.
    b. Acceleration Coefficient = .30G.
    c. Seismic Performance Zone: 4
    d. Importance Category: Essential
    e. Soil Profile Type: Type III for improved ground, see Geotechnical Report.
    f. Use Multimodal Spectral Method for seismic analysis. The elastic seismic response spectrum will be in accordance with AASHTO LRFD section 3.10.6.1.
5. Wind and Thermal Loads
    a. Not reviewed, assumed that controlling lateral loads will be seismic forces.

D. LOADING COMBINATIONS
1. Load combinations shall be in accordance with AASHTO LRFD Table 3.4.1-1. Strength I.

E. MATERIALS – NEW BRIDGE
1. Concrete – f'c = 4000 psi.
2. Reinforcing steel shall be AASHTO M31 Grade 60.
3. Pretensioning steel for precast members shall be 0.5-inch or 0.6-inch diameter low-relaxation strand AASHTO M203, Grade 270.
4. Structural Steel
    a. Structural steel shall conform to the following AASHTO requirements:
       AASHTO M270 Gr. 36 for thickness to 2 inches.
    b. Structural steel tubing (Hollow Structural Sections, HSS) shall conform to the following ASTM requirements:
       ASTM A500 Grade B with minimum CVN requirements.

F. MATERIALS - EXISTING BRIDGE
1. Concrete – f'c = 4000 psi.
2. Reinforcing steel Grade 40.
3. Structural steel 36 ksi.

3.2 REHABILITATION STUDY FINDINGS

3.2.1 Dead Load and Live Load Evaluation

The results of the dead load and live load evaluation indicated that the existing columns are adequate for the rehabilitated dead loads and current code required live loads. The existing timber piles were not adequate according to the AASHTO LRFD code requirements. A resistance factor of 0.45 is recommended by Shannon & Wilson for piles when soil properties are determined using standard penetration test methods. When checking the capacity of the piles for Strength Limit State I with the new dead load and code-required HL-93 live load, the piles are loaded about two times over capacity. The seismic rehabilitation with drilled shafts would provide additional vertical capacity so there would be no change to the timber piles required to accommodate the additional load.

An investigation of the service loads on the foundations was performed to compare the foundation loads of the rehabilitated structure to the foundation loads of the existing structure. The total dead load for the proposed superstructure is more than the total dead load for the existing superstructure. The proposed superstructure dead load is approximately 300 psf and the existing is approximately 165 psf. There is also an increase in live loads because the LRFD HL-93 is greater than the HS-20 live used for the existing superstructure. The total service dead loads and live loads for the proposed structure result in an approximately 80% increase in axial loads at the footing level compared to existing loads. A service load combination check of the existing timber piles for HS20 live load and existing slab dead load indicates that the existing piles have sufficient capacity. A service load check of the existing timber piles for HL-93 live load and proposed slab dead load indicates that the existing piles are not sufficient. As stated above, the seismic rehabilitation with drilled shafts would provide additional vertical capacity so there would be no change to the timber piles required to accommodate the additional load.

3.2.2 Seismic Evaluation

Moments and axial forces were determined for the seismic forces from the model. The columns, bracing and footings were checked for the applied seismic forces.

3.2.2.1 Columns

The unbraced columns in Bents 18 to 46 do not have sufficient bending capacity for loads in either the longitudinal or transverse direction. The Demand to Capacity (D/C) ratios for flexure in the columns between Bent 18 and Bent 46 were approximately 10. Even with a reduced demand on the columns by applying a Response Modification Factor (R-Factor) to the substructure the columns will be over capacity. Generally, the braced columns between Bents 49 to 81 had sufficient axial capacity for compression, but there were a couple of columns, especially between Bent 69 to Bent 74 that had insufficient capacity for axial compression. Some of the braced columns between Bents 49 to 81 also do not have sufficient axial capacity for tension in the member due to seismic loads.

3.2.2.2 Transverse and Longitudinal Bracing

The steel transverse bracing has insufficient axial capacity primarily between Bent 62 and Bent 76, although there are a couple of other locations that were also over capacity. Most of the existing longitudinal steel bracing members, and those areas where concrete members were replaced with steel bracing, do not have sufficient axial capacity for forces due to seismic loads.

3.2.2.3 Foundations

The timber piles in all footings did not have sufficient lateral or axial capacity. Most of the foundations between Bent 49 and Bent 81 had uplift due to the seismic loads, which can not be accommodated with the current timber pile connection to the footing.

3.2.3 Seismic Structural Systems

The existing Magnolia Bridge structure has two primary structural systems. Bent 18 to Bent 46 is an unbraced system where the seismic forces are resisted by shear and flexure in the columns. The seismic forces in Bent 47 to Bent 81 are resisted through axial forces in the braced frame action of the columns. In both structural systems, the existing foundations do not have sufficient capacity to resist the seismic forces. The intent of the proposed seismic rehabilitation is to strengthen the existing bracing, columns and footings and connect them with the superstructure so they act together as a unit.

3.2.3.1 Bent 18 to Bent 46

Since the columns do not have adequate capacity to resist seismic forces in shear and flexure a structural system needs to be provided to resist these forces. There are a couple of options available, including: column jacketing, providing longitudinal and transverse bracing for each bent, or providing shear walls for multiple span units. CalTrans recommends using column jacketing for structures where the D/C ratios do not exceed 6. Since the D/C ratios are high, column jacketing was disregarded as a viable option for this structure. Shear walls were not used because of the height of the structure and because bracing is much less expensive. For this rehabilitation study and cost estimate, the braced system was used. Transverse cross-bracing was provided at every bent. (See Figure 10) Longitudinal bracing was provided at every other span along the exterior line of columns on the north and south sides of the structure, so that all bents were braced in the longitudinal direction. The drawback of this system is that access under the bridge is limited due to the bracing systems. A shear wall system may require only half as many spans to be obstructed. The braced system was used for the Bents 18, 19 and 35 to 46. The connections to the columns will be similar to the collars used for the 2001 seismic retrofit.

A different system was used for Bent 20 to Bent 34 since the interior columns at these bents will be demolished as part of the Access Ramp Replacement. At these locations the interior columns will be replaced with 4’-0” diameter columns that are designed to resist all the seismic forces in bents. (See Figure 9.)

The timber pile foundations will be supplemented with grade beams and drilled shafts to provide sufficient lateral and vertical capacity for seismic loads.

3.2.3.2 Bent 47 to Bent 81

Many of the elements in the braced frame system for Bent 47 to Bent 81 are inadequate for current seismic loads. As a result these elements either need to be retrofitted or replaced to provide adequate capacity. Columns that do not meet requirements for axial compression and tension would be cased in steel jackets to provide the needed capacity. Longitudinal and Transverse Bracing will be added to any bents that are not currently braced. Any bracing that does not conform to code detailing requirements and/or strength requirements would be replaced with new bracing. Timber pile foundations that do not have sufficient lateral or vertical capacity would be supplemented with grade beams to transfer the load to drilled shaft foundations.

The rehabilitated structure was analyzed using the same model with new members added. The results of the rehabilitated seismic forces were checked against the capacity of the new members and found that the rehabilitated structure would perform in accordance with the current code requirements.

3.3 REHABILITATION ALTERNATIVE DEFICIENCIES AND APPROACH

The Rehabilitation Alternative would bring the existing bridge up to current design standards and extend the life of the structure for about 75 years. This would be done by rehabilitating elements of the bridge, such as columns and foundations, to meet current standards, or replacing elements, such as the bridge deck, that can not be rehabilitated. Table 2 presents the results of the capacity analyses as a listing of deficiencies and proposed approaches to eliminating the deficiencies. Table 3 describes the proposed rehabilitation elements and is keyed to Figure 8.

Table 2
Summary of Results

Deficiency	Proposed Approach
Ramp Structure over 15th Avenue West does not have sufficient seismic capacity.	Retrofit 15th Avenue West overpass structure to include longitudinal restrainers, transverse shear blocks, column jacketing, and additional pile foundation.
Vertical curve on ramps from 15th Avenue West to railroad crossing does not meet stopping sight distance requirements.	Replace approach fill, walls and ramps from Bent 1 to Bent 18 and build in a new profile that meets requirements. Cost based on cost per square foot cost estimate for new structure.
Roadway slab superstructure of bridge west of 15th Avenue does not meet current live load capacity requirements.	Replace superstructure with prestressed slab bridge. Spans 18 to 61 and Spans 78 to 82 uses 1'-6" prestressed slab with 5" deck. Spans 62 to 77 use 2' – 2" prestressed slab with 5" deck
Crossbeam will not support current live loads.	Replace crossbeam and column cap pedestals.
Timber piles do not meet current dead and live load capacity requirements.	Grade beams and drilled shafts are proposed for seismic performance. They will provide sufficient additional dead and live load capacity.
Concrete truss spans have reached end of service life and contain non-redundant structural elements.	Replace truss spans with prestressed slab bridge.
Horizontal curve from Bents 68 to 76 does not meet sight distance requirements.	Request a deviation for this section since no accidents have been recorded in this area.
Center ramp from Bents 20 to 34 does not meet current live load requirements and does not have sufficient seismic capacity.	Remove and replace interior deck and columns. Replace with prestressed slabs and 4' diameter circular columns that will take all the lateral seismic forces.
Insufficient lateral seismic capacity for unbraced Bents 18, 19 and 35 to 46. Moments exceeded capacity of columns.	Provide lateral cross bracing between columns.
Insufficient uplift capacity of columns to resist lateral seismic overturning forces for Bents 47 to 81.	Case columns in steel jackets that will carry the uplift force to the foundation.

Deficiency	Proposed Approach
Insufficient longitudinal seismic capacity for Bents 18, 19, and 35 to 82.	Provide longitudinal cross bracing for each bent.
Timber pile foundations are not sufficient for seismic lateral forces and uplift forces for Bents 18, 19, 35 to 46 and 59 to 81.	Provide grade beam between columns with two 6' diameter drilled shafts.
Timber pile foundations are not sufficient for seismic lateral forces and uplift forces for Bents 20 to 34 at the center ramp.	Lateral forces will be resisted by new interior columns, therefore provide grade beam between exterior columns with two 6' diameter drilled shafts between outside two columns.
Timber pile foundations are not sufficient for seismic lateral forces and uplift forces for Bents 47 to 58.	On and off ramps preclude placement of drilled shafts adjacent to bridge, therefore provide grade beam between columns with two 4' diameter drilled shafts placed longitudinally to columns each side of bridge.
The lateral cross brace system for Bent 62 to Bent 76 does not have sufficient strength to resist lateral seismic forces.	The proposal is to replace the existing bracing with new bracing that meets current code requirements.
Bracing systems in which all braces are oriented in the same direction are not allowed by current design guidelines MCEER/ATC 49. This requirement is so there is redundancy in the bracing system.	Replace existing Z bracing with X bracing.
The width to thickness ratio, b/t, for the braces does not meet current design guidelines MCEER/ATC 49 requirements. This requirement is to prevent local buckling of the bracing members.	Replace existing Z bracing with X bracing that meets thickness requirements.
Most of the existing longitudinal bracing system does not have sufficient strength to resist longitudinal seismic forces.	Replace the existing bracing with new longitudinal bracing.
Potential test measurements on reinforcement indicates a high probability of active corrosion at specific locations.	Provide a galvanic type corrosion protection utilizing a flame-spray zinc at specific locations.
Soils are potentially liquefiable.	Provide injection grouting of soils to prevent liquefaction and loss of capacity of foundations.

Deficiency	Proposed Approach
On and off ramps to 23rd Avenue were designed to and HS20 live load and a seismic acceleration coefficient of 0.2G.	Current code requirements are 0.3G and HL-93 live load which are both larger than original design. Therefore the ramp may need strengthening but was not included in current study effort.

Table 3
Rehabilitation Elements

Rehabilitation Element	Description
1. 15th Avenue W overpass	Retrofit the eastern 880 feet of this structure, which has 16 spans, for increased seismic capacity. Only the span over 15th Avenue W has been previously retrofitted.
2. Ramp and structure west to 15th Avenue W to west side of railroad	The ramp and spans in this 843-foot long section would be removed and replaced with new structure. This would eliminate a design deficiency (inadequate stopping sight distance, where the railroad structure connects to the ramp to 15th Avenue W.
3. Roadway deck and supporting crossbeams, west side of railroad to Magnolia Bluff	The existing superstructure in this 2,454-foot section would be replaced with pre-stressed slab spans. The seven concrete truss spans in this section would also be replaced. All crossbeams and column caps would be replaced.
4. Center ramp to Terminal 91	The 529-foot center ramp located west of the railroad would be replaced with new foundations, columns, and deck.
5. Railroad to center ramp and center ramp section to 23rd Avenue W ramps	Cross bracing would be provided between columns in the north-south (lateral) and east-west (longitudinal) directions.
6. From marina ramps to west end of bridge	Columns would be encased in steel jacket to provide seismic capacity. Existing column bracing in the east-west (longitudinal) direction would be replaced.
7. Timber pile foundations from railroad to west end of bridge	Grade beams and drilled shafts would be connected to existing column foundations to increase seismic capacity.
8. Connection to Anthony's Seafood Distributing	This connection would be removed and not replaced when the bridge deck is replaced (item 3).
9. Throughout	Ground around foundations would be treated by compaction grouting to resist liquefaction during an earthquake.

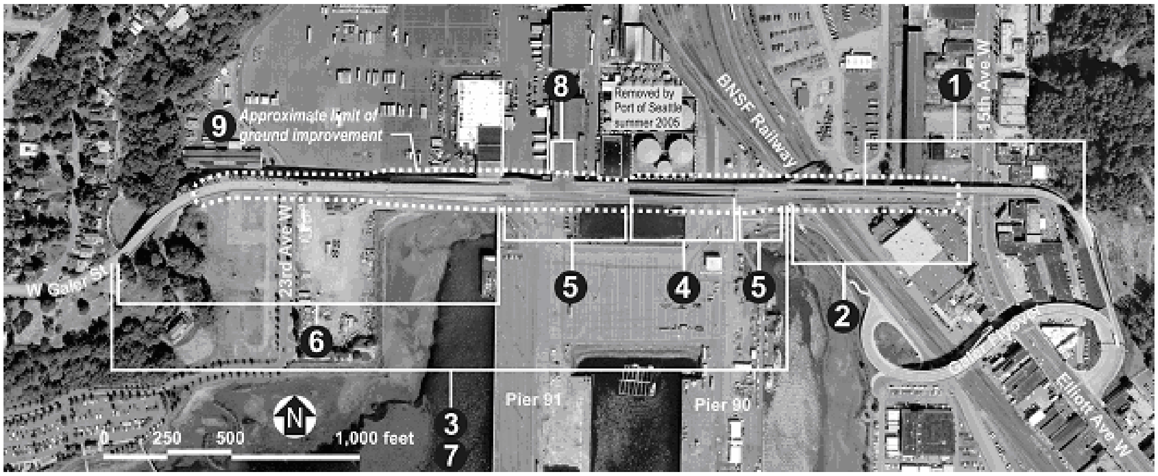


Figure 8
Rehabilitation Alternative Key Map

3.3.1 East Ramp Structure Over 15th Avenue West

The ramp structure over 15th Avenue West, design in 1957, does not have sufficient seismic capacity to meet current design code standards. Previous seismic studies determined that the columns do not have sufficient confinement reinforcement, foundations do not have sufficient seismic capacity, there is potential for girders to fall off pier caps and there is insufficient transverse girder restraint. The proposed retrofit of the ramp structure will include longitudinal restrainers, transverse shear blocks, column jacketing, and additional pile foundation.

The structure was partially retrofitted in 2001 as part of the West Galer Flyover construction. This included: retrofitting the columns and foundations at Pier 7 and 8, adding transverse shear blocks at the superstructure connection to Pier 7 and 8, and adding longitudinal restrainers between spans at Piers 6, 7, 8, and 9. The proposed retrofit would provide the same retrofits for all piers that were performed for Piers 7 and 8.

3.3.2 Vertical Stopping Sight Distance

The vertical crest curve on the ramp from 15th Avenue West to railroad crossing does not meet stopping sight distance requirements. As part of the Magnolia Bridge Rehabilitation, the vertical curve deviation over the BNSF railroad would be eliminated. This revision would require raising the profile grade several feet in the section of bridge between 15th Avenue West and Bent 18. As a result of this fix, it is assumed that the existing bridge structure from 15th Avenue West to Bent 18 would be removed and replaced with new structure. The replacement structure would likely be an MSE wall transitioning into a steel structure over the railroad. The span lengths would be much longer for this new structure and a single span would cross the BNSF Railway.

3.3.3 Slab Capacity

It was determined from the bridge load ratings that the existing cast-in-place concrete superstructure from Bent 18 to Bent 62 and Bent 75 to Bent 82 was inadequate for an HS-20 load.

As a result, it is assumed that all superstructure on the existing bridge west of Bent 18 would be removed and replaced. The steel trusses and braces and any associated collars and connections that were installed in 1961 and later to support the existing cast-in-place concrete deck would also be removed. The replacement structure will be either 1’-6” or 2’-2” deep precast concrete flat slabs with a 5 inch wearing surface, similar to the superstructure used on the existing ramps accessing 23rd Avenue West, constructed in 1991. (See Figure 9 through Figure 13.)

3.3.4 Crossbeam Capacity

The existing drop-down crossbeams for Bent 18 to Bent 62 and Bent 75 to Bent 82 were also found to be inadequate for an HS-20 load rating. Therefore, it is proposed that all drop-down crossbeams and the ornamental column caps would be removed and replaced with a 4-foot by 4-foot cast-in-place concrete crossbeam similar to the type used on the existing ramps accessing 23rd Avenue West. A 4-foot by 5-foot deep concrete crossbeam would also be utilized in the replacement of the trusses for Bent 62 to Bent 75. (See Figure 9 through Figure 13.)

3.3.5 Dead and Live Load

Timber piles do not meet current dead and live load capacity requirements. Grade beams and drilled shafts are proposed for seismic performance. The drilled shafts would also provide sufficient additional capacity to support the new dead load and live load requirements.

3.3.6 Concrete Trusses

Concrete truss spans have reached end of their service life and contain non-redundant structural elements. The City of Seattle directed HNTB to assume that the trusses will be replaced as part of the bridge rehabilitation project. The trusses would be replaced with 2’-2” deep concrete flat slab and a 5-inch wearing surface. The new superstructure will be supported on a new 4-foot by 5-foot deep crossbeam and column extension.

3.3.7 Horizontal Sight Distance

The horizontal curve from Bents 68 to 76 does not meet sight distance requirements. The structure would need to be replaced to improve the sight distance at this location. It was decided to pursue a deviation request for this section since no accidents attributable to sight distance restrictions have been recorded in this area.

3.3.8 Center Ramp

The center ramp from Bents 20 to 34 does not meet current live load requirements and does not have sufficient seismic capacity. It also results in a substandard edge of lane taper rate where the eastbound through lane transitions from the center ramp area to the two-lane ramp to 15th Avenue West. The columns supporting the center ramp also show the most significant signs of corrosion. The proposed solution is to remove and replace interior deck and columns. The replacement structure is a prestressed slab supers tructure with 4’ diameter circular columns. The new interior columns will take all lateral seismic forces. Drilled shaft foundations would be required to support the applied seismic forces. (See Figure 9.)

3.3.9 Unbraced Bents

The unbraced Bents 18, 19 and 35 to 46 do not have sufficient moment capacity to resist the lateral seismic forces. All lateral load is currently resisted by shear and flexure in the columns. The proposed solution is to provide lateral bracing between the columns so lateral forces are resisted by braced frames. (See Figure 10.)

3.3.10 Column Seismic Overturning Capacity

As the braced frames get taller in the bents leading up to Magnolia, the capacity of the columns is exceeded by the seismic forces. The lateral force is resisted by a moment couple of upward and downward forces in the columns. The uplift capacity of column is insufficient to resist lateral seismic overturning forces for Bents 47 to 81. In order to increase the tension capacity of the columns, steel jackets around the columns are proposed that would carry the uplift force to the foundation. The steel jacket would need to be detailed with studs so that it acts compositely with concrete column. (See Figure 13.)

3.3.11 Longitudinal Seismic Capacity

The existing longitudinal bracing is insufficient for calculated longitudinal seismic forces for Bents 18, 19, and 35 to 82. Longitudinal cross bracing is proposed for each bent. Bracing would be along exterior column lines. Cross bracing would be added to exterior column lines between Bents 18 and 19, 35 and 36, 37 and 38, 39 and 40, 41 and 42, 43 and 44, and 45 and 46. Existing concrete bracing would be replaced with steel cross bracing between Bents 43 and 44. Cross bracing would be added to both column lines for Bents 47 and 48, 48 and 49, 52 and 53, 53 and 54, and 57 and 58. Existing concrete bracing would be replaced with steel cross bracing on Bents 50 and 51, 55 and 56, 59 and 60, 61 and 62, 63 and 64, 67 and 68, 71 and 72, and 75 and 76. (See Figure 12.)

3.3.12 Timber Pile Foundations

The timber pile foundations are not sufficient to resist lateral seismic forces and for the increased dead and live loads to meet current design requirements. Timber piles have a low lateral resistance and are not connected to footings so they have no uplift capacity. For Bents 18, 19, 35 to 45 and 59 to 81, a grade beam would be provided between columns. The grade beam would be supported by two, 6-foot diameter drilled shafts. (See Figure 10.)

For Bents 20 to 34 at the center ramp, lateral forces would be resisted by new interior columns. A grade beam would be provided between exterior columns with two 6-foot diameter drilled shafts between the outside two columns. (See Figure 9.)

For Bents 47 to 58, the on and off ramps would preclude placement of drilled shafts adjacent to bridge. The grade beam would be provided between columns with two 4-foot diameter drilled shafts placed longitudinally to columns on each side of bridge. (See Figure 11.)

The shaft length required was dependent on the location of the bent within the limits of the project. The shafts east of Bent 48 have a layer of Estuarine Deposits at a depth of approximately 65 to 100 feet so the shaft needs to be founded below this layer. The shafts west of Bent 48 have a competant layer of Glacial Till at a depth of approximately 40 to 50 feet so the shaft depth is significantly shallower. The axial compression load on the shafts was up to about 2000 tons. This required a shaft length of 100 feet for Bents 18 to 46, and a shaft length of 40 to 50 feet for Bents 47 to 81. The actual embedment length into the Glacial Till material was controlled by the uplift capacity requirements.

3.3.13 Transverse Bracing Strength

The existing steel transverse cross brace system for Bents 62 to Bent 76 needs to be strengthened to accommodate the large seismic loads specified under current design standards. The proposal would replace the existing bracing with new bracing that has adequate capacity for seismic loads and meets current code requirements. (See Figure 13.)

3.3.14 Lateral Bracing Orientation

Bracing systems in which all braces are oriented in the same direction are not allowed by current design guidelines MCEER/ATC 49. This requirement is so there is redundancy in the bracing system. The proposed solution would replace existing "Z" bracing with "X" bracing. (See Figure 13.)

3.3.15 Lateral Bracing Section Properties

The width to thickness ratio, b/t, for the braces does not meet current design guidelines MCEER/ATC 49 requirements. This requirement is to prevent local buckling of the bracing members. Existing "Z" bracing would be replaced with "X" bracing that meets thickness requirements. (See Figure 13.)

3.3.16 Reinforcement Corrosion

Potential test measurements on reinforcement indicates a high probability of active corrosion at specific locations. A galvanic type corrosion protection would be provided by utilizing a flame-spray zinc at specific locations. (See Magnolia Bridge Rehabilitation Alternative – Existing Bridge Condition Report, October, 2005.)

3.3.17 Liquefiable Soils

Soils are potentially liquefiable. Provide injection grouting of soils to prevent liquefaction and loss of capacity of foundations. (See Magnolia Bridge Rehabilitation Geotechnical Technical Memorandum, October, 2005)

3.3.18 23rd Avenue West On and Off Ramps

On and off ramps to 23rd Avenue were designed to HS20 live load and a seismic acceleration coefficient of 0.2G. Current code requirements are 0.3G and HL-93 live load which are both larger than original design. Therefore the ramp may need strengthening, but it was not included in current study effort. No costs were determined for the seismic upgrade of the ramps. The upgrade will be added as a potential risk item during the cost evaluation process.

3.3.19 Access Under the Bridge

The proposed additional cross bracing would obstruct access under certain parts of the bridge. The transverse bracing will have a clean look from the side, but will obstruct any traffic flow between columns, parallel to the bridge alignment, on the Port of Seattle property. The longitudinal bracing would obstruct certain bays for traffic across the structure.

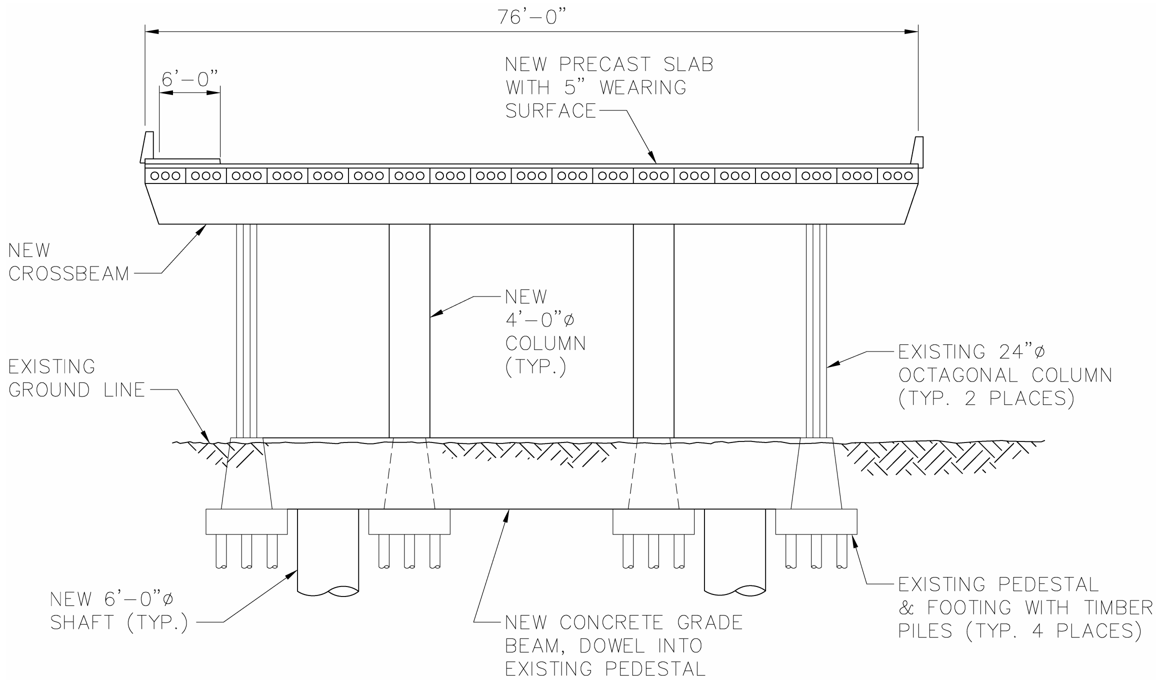


Figure 9
Center Ramp Removed, Bents 20 through 34

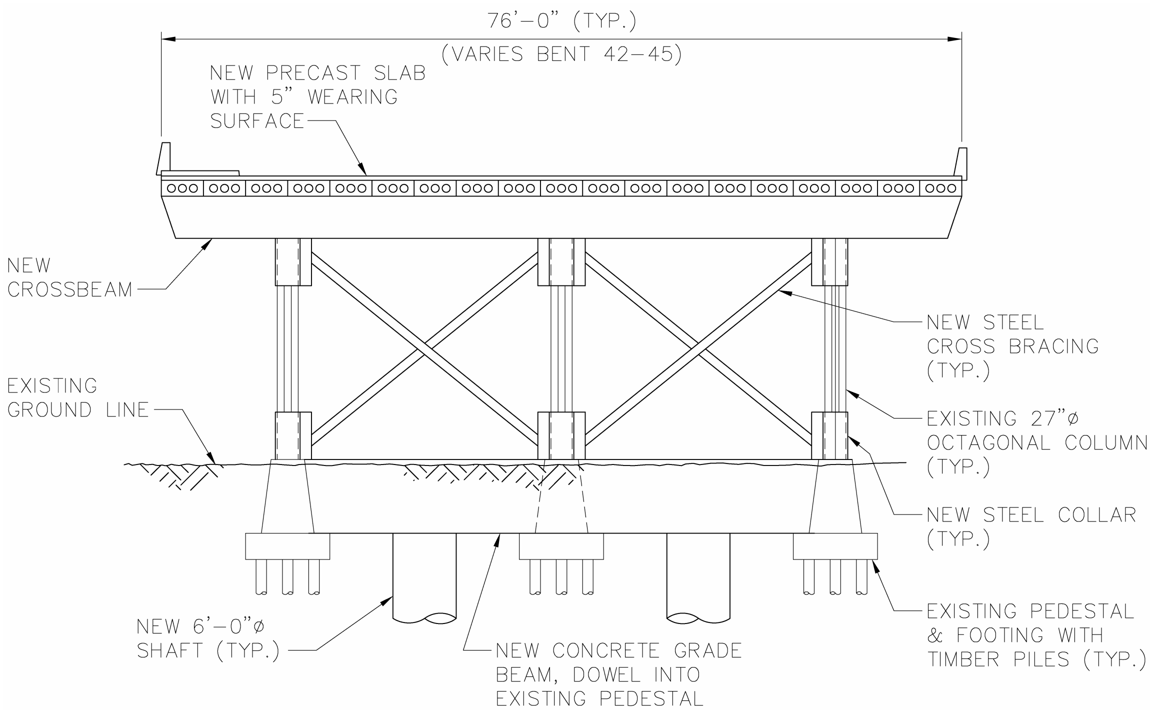


Figure 10
Column Lateral Bracing, Bents 18, 19, and 35 through 46

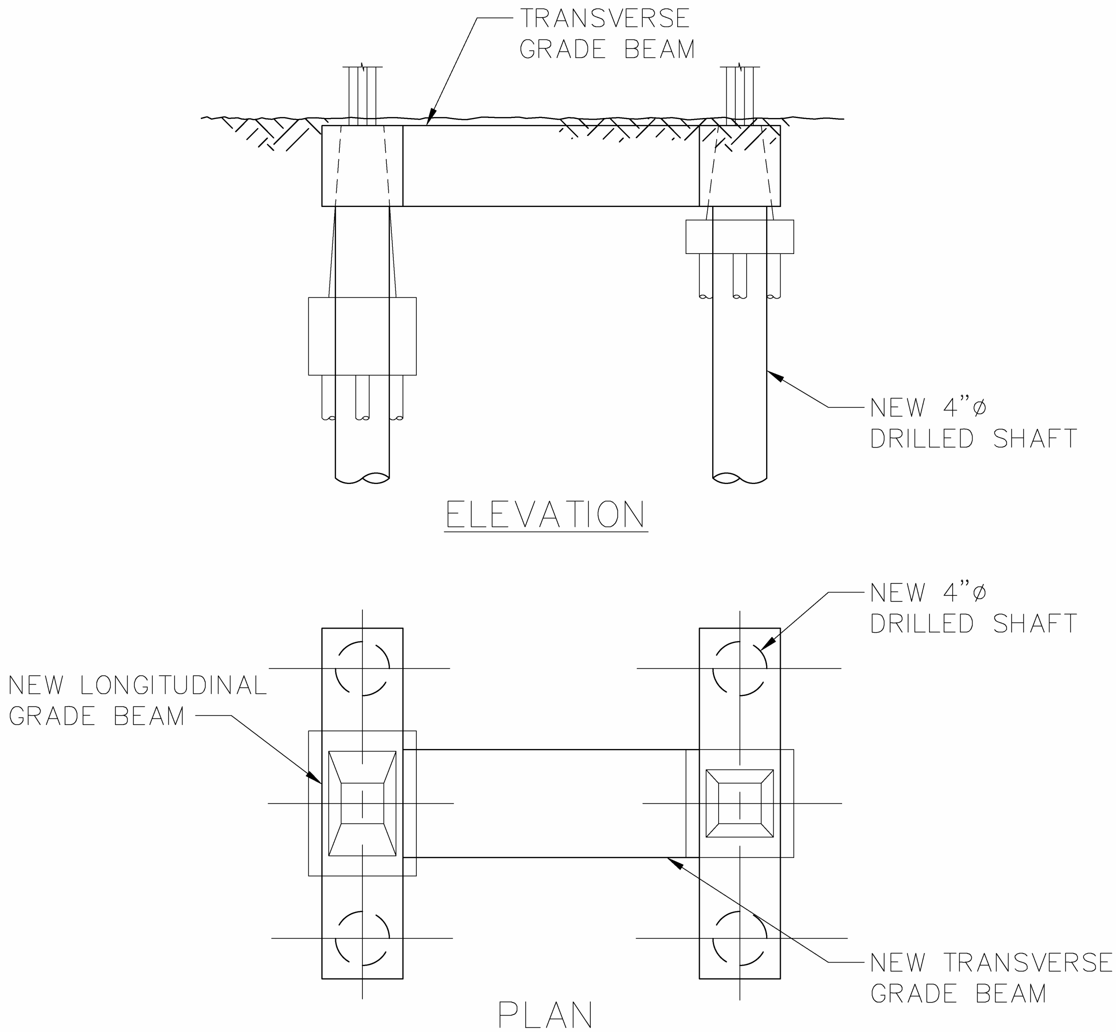


Figure 11
Grade Beam and Drilled Shaft Arrangement, Bents 47 through 58

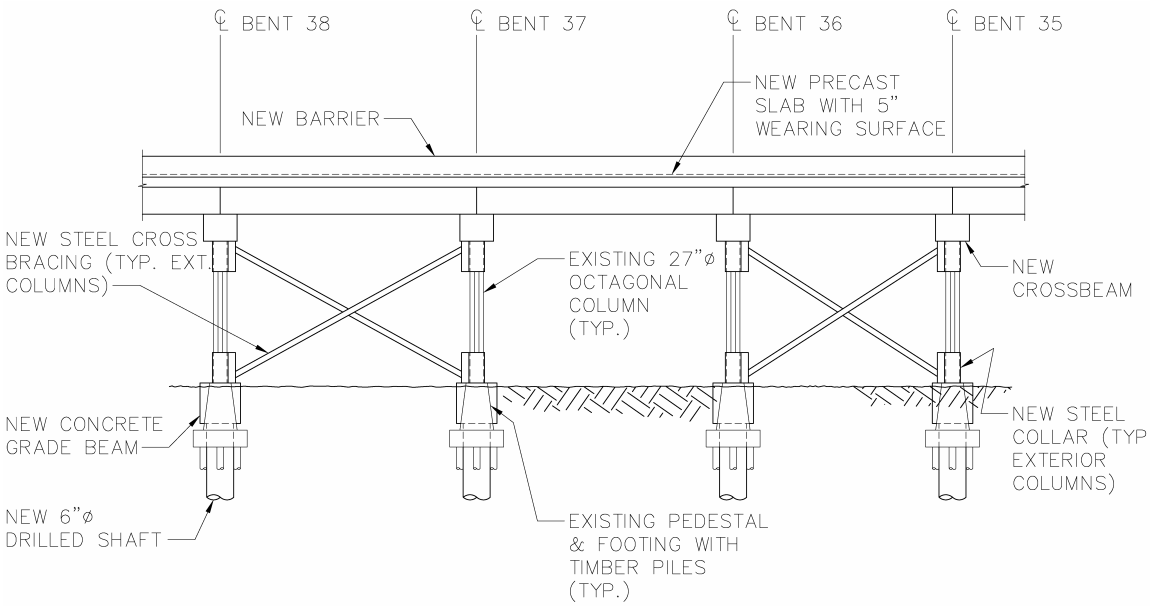


Figure 12
Column Longitudinal Bracing (see text for other locations)

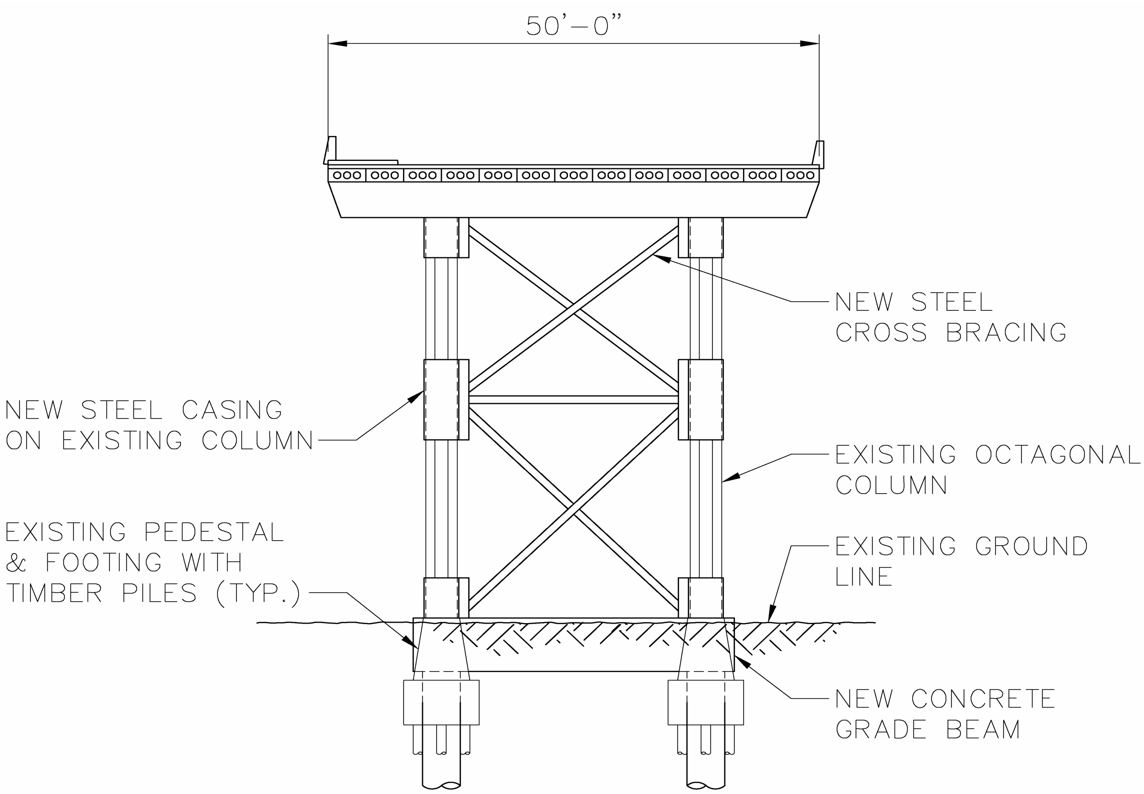


Figure 13
Transverse Cross Bracing, "X" Orientation, Bents 62 through 76

3.4 REHABILITATION COST ESTIMATE

A cost estimate of the rehabilitation was prepared based on square foot costs for new structure where indicated and unit price for rehabilitating specific structural elements. Unit prices were determined based on bid tabulations for similar projects, summarized August 2005.

Neither HNTB nor City of Seattle has control over the cost of labor, materials, or the Contractor's methods of determining bid prices or market conditions. HNTB cannot and does not warrant, represent, make any commitments, or assume any duty to assure, that bids or negotiated prices will not vary from any estimate of construction cost or evaluation prepared by or agreed to by HNTB.

Table 4
Summary of Rehabilitation Cost

			Demolition	
Item	**Location**	**Cost (2005 $)**	**Area**	**Cost**
Structure Rebuild (including demolition)	15th Ave West to Bent 18	$ 11,900,000	52,490	$ 2,100,000
15th Avenue West Overpass Retrofit	15th Ave West Overpass	$ 2,200,000	2,388	$ 60,000
Superstructure (including demolition)	Bent 18 to 81	$ 13,810,000	149,260	$ 3,700,000
Crossbeams	Bent 18 to 81	$ 1,600,000		
New Columns - Bent 20 to 34	Bent 20 to 34	$ 240,000		
Column Extensions	Bent 62 to 75	$ 74,000		
Column Steel Casing	Bent 47 to 81	$ 4,710,000	40,121	$ 1,000,000
Lateral Steel Bracing	Bent 18 , 19, 35 to 46	$ 790,000	3,039	$ 100,000
Lateral Steel Bracing	Bent 47 to 81	$ 2,800,000		
Longitudinal Bracing	Bent 18 to 19, 35 to 46	$ 600,000		
Longitudinal Bracing	Bent 47 to 81	$ 3,900,000		
Column to Foundation Connection	Bent 47 to 81	$ 200,000		
Foundation Improvements	Bent 18 to 81	$ 16,100,000		
Cathodic Protection	Bent 18 to 46	$ 533,000		
Total		$ 59,460,000	247,000	$ 6,960,000
Maintenance Costs				
15th Ave West Overpass - Future Wearing Surface	15th Ave West Overpass	$ 749,000	One Time	
Expansion Joint Maintenance - Joint Seal Replacement	Bent 18 to 81	$ 51,000	One Time	

3.5 REHABILITATION CONCLUSIONS

The conclusion of the Rehabilitation Study is that it is not economical or desirable to rehabilitate the existing bridge. The cost of rehabilitation is similar to replacement costs and would result in a longer traffic construction detour and increased obstructions to port operations under the existing bridge. Rehabilitating the existing bridge would require replacement of existing deck slabs and trusses, strengthening foundations for live loads, providing cross bracing for lateral and longitudinal seismic resistance, and corrosion protection for existing reinforcement. Rehabilitation was eliminated from future consideration.

4. BRIDGE REPLACEMENT DESIGN CRITERIA

Design Criteria listed below provide general guidelines, and will be evaluated and modified as needed for specific applications, with approval of SDOT. In general, City and County Design Standards (C/C DS) as specified in Chapter 42 of Local Agency Guidelines (September 2002) will be used, with support of the current Washington State Department of Transportation Design Manual (WSDOT DM) and City of Seattle “Seattle Street Improvement Manual,” December 1991. In case there is a conflict between any of the criteria and/or standards listed, the standards with the latest date shall take precedence.

4.1 ROADWAY DESIGN CRITERIA

4.1.1 Minimum Level of Service (LOS)

The City has established level of service standards in Goal G14 of Seattle’s Comprehensive Plan. These standards provide a gauge to judge the performance of the arterial and transit systems in serving future growth and infrastructure development. For arterial level of service, the City has adopted a methodology using volume-to-capacity analysis at several screenlines within the City. These screenlines measure p.m. peak hour directional traffic volumes on one or more arterials crossing the screenline. Two screenlines are located in the vicinity of the Magnolia Bridge project:

Screenline 2 covers all arterials connecting Magnolia to 15th Avenue West, including the Magnolia Bridge

Screenline 5.11 covers the Ballard Bridge at the crossing of the Lake Washington Ship Canal

At the Magnolia screenline, the Level of Service (LOS) standard specifies that volume-to-capacity ratio remain below 1.00, and the Ballard Bridge screenline specifies a standard of 1.20. These ratios correspond broadly to LOS E and F, respectively.

While the Seattle Comprehensive Plan set minimum LOS standards for the purpose of land use planning and regulation, SDOT has goals to improve LOS above E or F where practical.

For the Magnolia Bridge Replacement Project, LOS estimates will be prepared for the arterial intersections of interest for 2010 and 2030 AM and PM peak hour conditions. For those locations, projected to operate at LOS E or F, the intersection analyses will be reviewed by Seattle Department of Transportation staff to determine opportunities for mitigation on a case-by-case basis.

4.1.2 Roadway Classification

The Magnolia Bridge Replacement will be designed to meet the requirements for a Principal Arterial between 15th Avenue West/Elliott Avenue West and the connection to the street system on Magnolia.

4.1.3 Design Speed (DS)

Design Speed (DS) will be 5 mph over speeds posted 35 mph or below. For streets with posted speed of 40 mph and above, a DS of 10 mph over posted speed will be used.

4.1.4 Maximum Allowable Grade

Maximum allowable grade will be 9% for Principal Arterials and 10% for Minor Arterials (see Table 5). On Bridge sections desirable grade is 6%.

4.1.5 Minimum Sight Distance

Stopping Sight distance (SSD) will be evaluated for each case separately. For Principal Arterial with DS of 40 mph minimum 305 ft and desirable 330 ft SSD will be used. For Minor Arterial with DS of 35 mph, minimum 250 feet, and desirable 260 feet SSD will be used. For other DS values, minimum SSD will be used according to City and County Design Manual (C/C DM), with desirable SSD from WSDOT Design Manual.

4.1.6 Lane Widths

Lane widths will be designed per City and County Design Standards and per the Seattle Street Improvement Manual.

4.1.7 Pedestrian Requirements

Concrete sidewalks will be designed according to City of Seattle Standard Plan 420.1. A 10.5-foot width will be used for arterial streets. A minimum six-foot width will be used for sidewalks adjacent to non-arterial streets in residential, commercial, and industrial zones, or may conform to the existing sidewalk and planting strip location as directed and approved by SDOT.

4.1.8 Bicycle Requirements

Bicycle facilities will be designed per WSDOT DM (chapter 1020).

4.1.9 Drainage Requirements for Water Quality and Detention

Outline of storm drains layout, detention and water quality facilities for final selected alignment will be provided in accordance with City publication number 280, Design Guidelines for Public Storm Drain Facilities.

4.1.10 Structural Live Loads

See Table 7 - Preliminary Bridge Design Criteria.

4.1.11 Structural Seismic Criteria

See Table 7 - Preliminary Bridge Design Criteria.

4.1.12 Illumination Standards

Illumination design will be in conformance with the Seattle Municipal Code and National Electric Code, the National Electric Safety Code, Washington State Electrical Code Chapter 296-44 WAC, and Illuminating Engineering Society (IES) recommendations.

For collector streets, 1999 IES standards are 2.5 maintained foot candle, 3:1 uniformity.

For industrial/commercial streets, standards are 2.0 maintained foot candle, 3:1 uniformity, and for residential non-arterial streets, standards are 1.5 maintained foot candle, 4:1 uniformity.

Intersections require higher illumination levels.

4.1.13 Signal Criteria, Including CCTV and Interconnect System

All traffic control devices, such as traffic signals, traffic signs, or channelization will be designed according to Manual of Uniform Traffic Control Devices (MUTCD) – Millennium Edition, subject to approval by SDOT.

4.1.14 Landscape Requirements

Landscape Plans will be according to the City Street Improvement Manual pages 2-21 through 2-23, and 3-14 through 3-16. The species and location of street trees will be subject to approval by the City Landscape Architect. Planting strips of 5-1/2 feet minimum width will be proposed between curb and sidewalk.

4.1.15 Railroad Clearance Requirements

Lateral distance from centerline of railroad track to obstruction six inches high or more shall be minimum 8.5 feet, desirable 10.0 feet. This distance will be increased by 1.5 inch for every degree of track curvature.

Vertical clearance above tracks shall be minimum 23.5 feet.

4.1.16 Clearance Requirements for Oversized Vehicle Routes

Clearance for oversized vehicle routes (15th Avenue W/Elliott Avenue W) will be evaluated for all geometric elements, including but not limited to travel lane width and turning movements. Vehicle WB-67 and directives included in WSDOT DM (pages 430-3 and 910-7) will be used. Shy distance from the edge of travel lane to barrier curb will be increased when required, by 1 foot on the left side and by 2 feet or more on the right side. Vertical clearance above 15th Avenue W/Elliott Avenue W shall be a minimum 20.0 feet, as per direction from City of Seattle engineering staff.

Table 5
Roadway Design Criteria

Criteria	Principal Arterials	Minor & Collector Arterials	Access Streets & Alleys	Reference
DESIGN SPEED				
	Designated: 40 mph Others: 10 mph over posted if posted is 40 mph or more	5 mph over posted if posted speed is 35 mph or less	5 mph over posted if posted speed is 35 mph or less	C/C DS pg.6 WSDOT DM pg. 440 – 3
HORIZONTAL ALIGNMENT				
MIN. RADIUS WITH SUPER-ELEVATION WITH NORMAL CROWN (-2%)	410' 675' (40 mph)	300' 465' (35 mph) 305' (30 mph)	115' 180' (25 mph)	COS STREET IMPROVEMENT MANUAL pg. 3 - 6 C/C DS pg.6
HORIZONTAL STOPPING SIGHT DISTANCE (SSD)	For DS 40 mph 305' – min. 330' – desirable	For DS 35 mph 250' – min. 305' – desirable For DS 30 mph 200' – min. and desirable	For DS 25 mph 155' – min. 165' – desirable	Min. per C/C DS pg.6 Desirable per WSDOT DM pg. 650 – 3
TAPER LENGTH	L = WS^2/60	L=WS^2/60	L=WS^2/60	COS STREET IMPROVEMENT MANUAL pg. 3 -10
LANE WIDTH	Through Lane: 11' Curb Lane: 12' Curb Lane Veh/Bic: 14' Turn Only Lane: 12' Bus Only Lane: 12' Parking Lane: 8' Parking Lane o/Bus Rte: 10'	Through Lane 11' Curb Lane 12' Turn Only Lane 12'	A/Streets: 10' min. Alleys: 8' min.	COS STREET IMPROVEMENT MANUAL pg. 3 – 10 C/C DS pg.6

Criteria	Principal Arterials	Minor & Collector Arterials	Access Streets & Alleys	Reference
VERTICAL ALIGNMENT				
GRADE	MAX. 9% MIN. 1%	MAX. 10% MIN. 1%	MAX. 17% MIN. 1%	COS STREET IMPROVEMENT MANUAL pg. 3 – 5
VERTICAL CURVE LENGTH	3V or 120' Desirable MIN. 75'	3V or 120' Desirable MIN. 75'	MIN. 75'	COS STREET IMPROVEMENT MANUAL pg. 3 – 6 C/C DS pg.6
SSD FOR VERT. CURVE	For 40 mph 305' – min. 330' – desirable	For 35 mph 250' – min. 265' – desirable For 30 mph 200' – desirable	For 25 mph 155' – min. 165' – desirable	Min. per C/C DS pg.6 Desirable per WSDOT DM
CROSS-SECTION				
CROSS SLOPES	STD. 2% MAX. 4% MIN. 1%	STD. 2% MAX. 4% MIN. 1%	*Access Street:* STD. 2% MAX. 4% MIN. 1% *Alleys:* STD 4.7% MAX. 6% MIN. 2%	COS STREET IMPROVEMENT MANUAL pg. 3 - 6
CUT SLOPE	MAX. 2H:1V	MAX. 2H:1V	MAX. 2H:1V	COS STREET IMPROVEMENT MANUAL pg. 2-17
FILL SLOPE	MAX. 2H:1V	MAX. 2H:1V	MAX. 2H:1V	COS STD. PLAN NO. 400
SIDEWALK WIDTH	Min. 10'-6" (incl. planter) or match existing if wider	Min. 6'-0" STD. 10'-6" or match existing if wider	Min. 6'-0" STD. 10'-6" or match existing if wider	COS STREET IMPROVEMENT MANUAL pg. 3 - 13 COS STD PLAN NO. 400

Criteria	Principal Arterials	Minor & Collector Arterials	Access Streets & Alleys	Reference
SIDEWALK SLOPE	STD 2.0% Special case: Min. 1% Max. 5% Must comply with ADA	STD 2.5% Special case: Min. 1% Max. 5% Must comply with ADA	STD 2.5% Special case: Min. 1% Max. 5% Must comply with ADA	COS STREET IMPROVEMENT MANUAL pg. 3 - 13 COS STD. PLAN NO. 400 and 420
INTERSECTIONS				
INTERSECTION SIGHT DISTANCE - NO CONTROL	195' (40 mph)	140' (30 mph) 165' (35 mph)	115' (25 mph)	C/C DS pg.6
CROSS STREET ANGLE	75 - 105 Degrees	75 - 105 Degrees	75 - 105 Degrees	WSDOT DM pg. 910-2
CURB RADIUS	Arterial to Arterial: 25' Arterial to Residential: 20' Arterial to Commercial: 25' High Vol Trucks and/or Bus: 30'	Residential to Residential: 20'	Residential to Residential: 20'	COS STREET IMPROVEMENT MANUAL pg. 3 - 11
CLEARANCES				
LATERAL CLEARANCE	Curb face to fixed object: 3' Edge of s/walk to f/object: 3' Pole, FH to f/object: 3'	As for Arterials	As for Arterials	COS STREET IMPROVEMENT MANUAL pg. 3 – 21 C/C DS pg.6 WSDOT IL 4053.00
VERTICAL CLEARANCE	Over roadway surface: 20.00' Over s/walk, bike path: 8.00' minimum 10.00' desirable New roadway under existing bridge min. 15.50'	As for Arterials	As for Arterials	COS Engineering Staff directions WSDOT DM pg. 1120-2

Criteria	Principal Arterials	Minor & Collector Arterials	Access Streets & Alleys	Reference
RAILROAD TRACK CLEARANCE	Lateral Distance from CL of RR track to obstruction 6” or more Minimum 8.5' Desirable 10.0' Increase clearance by 1.5” for every degree of track curvature. Vertical Distance above tracks: min. 23.00' USE 23.50'	As for Arterials	As for Arterials	COS STREET IMPROVEMENT MANUAL pg. 3 - 22 BNSF RR Standard WSDOT p.1120-2

Abbreviations used in this table:

ADA	Americans with Disabilities Act
BNSF RR	Burlington Northern Santa Fe Railroad
C/C DS	City and County Design Standards
COS	City of Seattle
DM	Design Manual
SSD	stopping sight distance
WSDOT	Washington State Department of Transportation

4.2 ROADWAY/BRIDGE RAMP GEOMETRICS DESIGN CRITERIA

The Roadway Ramp Geometrics Design Criteria (see Table 6) have been established using WSDOT DM and City and County Design Standards pg. 6, as specified in Chapter 42 of Local Agency Guidelines (September 2002). Any portion of the project that may be used to convey military traffic will be designed to WSDOT DM standards, with FHWA coordination and approval.

Table 6
Roadway Ramp Geometrics Design Criteria

Criteria	Guideline	References	
RAMP DESIGN SPEED			
Mainline DS For 40 mph	25 mph (min), 35 mph (desirable)	WSDOT DM Fig. 940-1	City and County Design Standards pg. 6 will be used for all design.
HORIZONTAL CURVES			If the Project is designated as a part of National Highway System then WSDOT DM will apply.
MIN. RADIUS FOR NORMAL CROWN SECTION DS = 25 mph DS = 30 mph DS = 35 mph	 2,640' 3,340' 4,380'	WSDOT DM pg. 640-1	
MIN. RADIUS W/ SUPER-ELEVATION DS = 15 mph DS = 20 mph DS = 25 mph DS = 30 mph DS = 35 mph	 from 50' to 430' from 90' to 1,000' from 150' to 1,000' from 230' to 1,500' from 310' to 2,000'	WSDOT DM pg. 640-12	
TRAVELED WAY WIDTH FOR TWO-LANE RAMP (Radius to CL) R = 3,000' to tangent R = 1,000' to 3,000' R = 500' R = 300' R = 200' R = 150' R = 100'	 24' 25' 27' 28' 29' 31' 34'	WSDOT DM pg. 640-8a, see also 430-5	

Criteria	Guideline	References	
TRAVELED WAY WIDTH FOR ONE-LANE RAMP (Radius to outside edge) R = 300' to tangent R = 200' R = 130' R = 100' R = 75'	15' 17' 18' 19' 21'	WSDOT DM pg. 640-9a	City and County Design Standards pg. 6 will be used for all design. If the Project is designated as a part of National Highway System then WSDOT DM will apply.
SHY DISTANCE On right On left	1' to 2' 2'	WSDOT DM pg. 910-3	
SHOULDER WIDTH On right On left	4' to 6' 2'	WSDOT DM pg. 430-3, 430-4	
SUPERELEVATION RATES FOR DS = 15 mph TO DS = 40 mph	from 2% to 10%	WSDOT DM pg. 640-12	
VERTICAL ALIGMENT			
RAMP GRADES DS = 25 mph to 30 mph DS = 35 mph to 40 mph DS => 40 mph	7% max., 5% desirable 6% max., 4% desirable 5% max., 3% desirable	WSDOT DM fig. 940-2	
VERTICAL CURVES Minimum length of vertical curve for design speed (VCLm) for crest and sag DS = 25 mph DS = 30 mph DS = 35 mph DS = 40 mph	75' 90' 105' 120'	WSDOT DM fig. 650-2, see fig. 650-7 and 650-8	City and County Design Standards pg. 6 will be used for all design. If the Project is designated as a part of National Highway System then WSDOT DM will apply.
SIGHT DISTANCE			

Criteria	Guideline	References	
DESIGN STOPPING SIGHT DISTANCE DS = 25 mph DS = 30 mph DS = 35 mph DS = 40 mph	165' 200' 260' 330'	WSDOT DM fig. 650-2	
SIGHT DISTANCE FOR TURNING VEHICLES (LEFT - P VEHICLE) DS = 25 mph DS = 30 mph DS = 35 mph DS = 40 mph	300' 380' 480' 590'	WSDOT DM fig. 650-2	
DESIGN SIGHT DISTANCE FOR MANOUVERS (urban stop) DS = 30 mph DS = 40 mph	510' 725'	WSDOT DM fig. 650-5	

4.3 PRELIMINARY BRIDGE DESIGN CRITERIA

The Preliminary Bridge Design Criteria (see Table 7) have been established using WSDOT Bridge Design Manual and AASHTO LRFD Bridge Design Specifications, 3rd Edition with Interim Revisions through 2006.

Table 7
Preliminary Bridge Design Criteria

Criteria	Guideline	References
DESIGN SPECIFICATIONS	AASHTO LRFD Bridge Design Specification, 3rd Edition with Interim Specifications and WSDOT Bridge Design Manual	
DESIGN METHOD	Load and Resistance Factor Design	
DESIGN LIVE LOAD	HL-93 per AASHTO LRFD	
DESIGN SEISMIC LOAD	Acceleration Coefficient = 0.3g Seismic Performance Zone: 4 Importance Category: Essential Soil Profile Type: IV	
FOUNDATIONS	See Shannon & Wilson report "Draft Geotechnical Report, Magnolia Bridge Replacement Project, Concept Design", dated November, 2006	

5. ALIGNMENT STUDIES

The process of generating initial design concepts and selecting and evaluating alignments commenced in 2002. The entire Alignment Study Report is included in Appendix C. Initial alignment alternatives were developed from previous team discussions, the first open house, stakeholder interviews, and previous studies. The evaluation of alignments involved extensive analysis, refinement, and elimination of problematic alignments. The primary means of selecting and evaluating the alignments was through the screening process described below.

5.1 FIRST LEVEL SCREENING

Members of the Magnolia Bridge Replacement Study team met to screen the initial candidate alignments. The team worked through 25 alignments and eliminated 12 alignments from further consideration. The remaining 13 alignments were consolidated into nine alternatives that were carried forward.

5.1.1 Alignments Considered

Twenty-two alignments were prepared and considered for evaluation. In addition, three new alignments and one variation were presented by team members for evaluation during the meeting. Table 8 identifies the candidate alignments that were considered, with a brief description of each.

The objective of the meeting was to reduce the number of candidate alignments to 6 or 8 for further study and to confirm that all reasonably possible alignments had been identified and considered. The assumptions, design constraints, and criteria used to evaluate the alignments are presented in Sections 5.1.2 and 5.1.3.

Table 8
Candidate Alignments

No.	Alignment Location	Alignment Description	Bridge/ Structure Length	Total Route Length	Notes
1	Existing bridge footprint	Replace existing bridge at the same location with new structure. Construct drop ramps similar to the existing configuration.	3,800	3,800	Total 4 ramps to connect with T91
1 A	Existing bridge footprint	Replace existing bridge at the same location with new structure. Move west end connection at W Galer St to the south. Construct diamond I/C in the mid span to provide access to waterfront and Uplands.	3,800	3,800	Diamond I/C at mid span. Revised west end connection to W Galer St
2	Existing bridge footprint, W Marina Dr, 32nd Ave W	Replace bridge at east end, drop to surface west of RR tracks, continue surface road to Smith Cove Park, connect W Marina Dr with 32nd St with surface road or low bridge in tidelands. Improve 32nd Street	1,500 east 1,000 west	8,600	Two separate structures
3	Existing bridge footprint, revise west end at connection to W Galer St	Replace bridge at east end, drop to surface west of RR tracks, continue surface road, replace west end structure with fill and/or new structure, add new surface road connecting to 21st Ave W.	1,500 east 1,400 west	5,000	Two separate structures
4	Existing bridge footprint, revise west end at connection to W Galer St	Replace bridge at east end; construct new bridge south of existing in close proximity. Replace west end structure with new coming straight from W Galer St and swing to the north over Smith Cove Park. Provide drop ramps for Uplands/waterfront connection.	3,800	3,800	Drop ramps or diamond I/C at T91
5	South of existing bridge footprint, turn to the North over 15th Ave W	Replace bridge at east end north of existing, turn to the south to cross RR tracks, continue parallel to the existing approx. 400' to the	4,000	4,000	Drop ramps at Smith Cove Park, full I/C at 15th Ave W

No.	Alignment Location	Alignment Description	Bridge/ Structure Length	Total Route Length	Notes
		south over the water, connect straight with W Galer St. Construct ramps to connect with Uplands/waterfront.			
6	Long arc north of existing bridge (500' to 700') connecting to 15th Ave W, and W Galer St at the existing locations	Construct new bridge in the form of long arc north of existing bridge. Construct new ramps to connect with 15th Ave W at existing connection point. Construct ramps to connect with Uplands/waterfront.	4,300	4,300	Diamond I/C at mid span, full I/C at 15th Ave W
7	W Galer St flyover, along west side of RR tracks, W Galer St	Surface road from W Galer St flyover, cross under existing bridge, run along west side of RR tracks for approx. 1700', turn west connect with new structure at W Galer St.	1,700	4,800	
8	W Galer St flyover, along west side of RR tracks, W Galer St, and Thorndyke Ave W/ 23rd Ave W	Surface road from W Galer St flyover, cross under existing bridge, run along west side of RR tracks for approx. 2200', turn west connect with new structure at W Galer St, and Thorndyke Ave W/ 23rd Ave W.	1,700	5,300	
9	W Galer St Flyover, along west side of RR tracks, Thorndyke Ave W/W Halladay St, 20th Ave W, 21st Ave W	Surface road from W Galer St Flyover, cross under existing bridge, run along west side of RR tracks, connect with 20th Ave W, 21st Ave W and to Thorndyke Ave W at W Halladay St with a fill ramp or bridge.	1,000	5,400	
10	North of existing bridge, cross RR tracks, cross Port uplands, Thorndyke Ave W/23rd Ave W	Begin 500' +/- north of existing bridge with at grade I/S, northwest (at angle) cross over RR tracks, drop and continue as a surface road, turn north and construct new fill and/or structure to connect with W Galer St.	1,500	4,000	Bridge at skew angle to RR tracks Fill ramp or bridge west end

No.	Alignment Location	Alignment Description	Bridge/ Structure Length	Total Route Length	Notes
11	W Wheeler St, cross RR tracks, cross Port uplands, Thorndyke Ave W/23rd Ave W	Begin at Wheeler St and 15th Ave W with at grade I/S, continue straight west and cross RR tracks, cross Port uplands with elevated structure, connect to Thorndyke Ave W/23rd Ave W. Construct half diamond I/C to provide connection with Port uplands from east side only, and surface road connection with 21st Ave W.	2,000	2,500	Half diamond I/C at T91
12	W Armory Way, cross RR tracks, cross Port uplands, Thorndyke Ave W/W Halladay St	Begin at W Armory Way and 15th Ave W with at grade I/S, continue on W Armory Way and cross over RR tracks at angle, turn west, and continue elevated structure, connect to Thorndyke Ave W at W Halladay St.	1,700	3,000	Bridge at skew angle to RR tracks
13	W Wheeler St, cross RR tracks, cross Port uplands, Thorndyke Ave W/ W Halladay St	Begin at Wheeler St and 15th Ave W with at grade I/S, continue straight west and cross over RR tracks at angle, continue elevated structure, connect to Thorndyke Ave W/23rd Ave W.	1,700	2,500	Bridge at skew angle to RR tracks
14	North of existing bridge, cross RR tracks, south side of existing bridge, W Galer St	Begin 900' +/- north of existing bridge with at grade I/S, cross over RR tracks, drop and continue as a surface road, turn south along toe of bluff, construct new fill and/or structure to connect with W Galer St.	1,400 east 1,800 west	4,000	
15	W Armory Way, cross RR tracks, cross Port uplands, W Galer St and Thorndyke Ave W/23rd Ave W	Begin at W Armory Way and 15th Ave W with at grade I/S, continue on W Armory Way approx. 500', turn west and cross over RR tracks, drop down and continue surface road, split to two connections: one south with new fill and/or structure to connect with W	1,300 east 1,800 west 1,000 ramp	4,900	

No.	Alignment Location	Alignment Description	Bridge/ Structure Length	Total Route Length	Notes
		Galer St, second as a ramp connecting to Thorndyke Ave W/23rd Ave W.			
16	W Armory Way, cross RR tracks, cross Port uplands, W Galer St and 21st Ave W	Begin at W Armory Way and 15th Ave W with at grade I/S, continue on W Armory Way approx. 500', turn west and cross over RR tracks, drop and continue as a surface road, split to two connections: one south with the new structure to connect with W Galer St, second connecting to 21st Ave W.	1,400 east 1,800 west	5,100	
17	Existing bridge footprint, add direct connection to the 23rd Ave W	Replace existing bridge at the same location with new structure. Construct direct connection via bridge to the 23rd Ave W.	3,800 south 1,500 north	5,300	
18	Existing bridge footprint, W Marina Dr, 32nd Ave W, add direct connection to the 23rd Ave W	Replace bridge at east end, drop to surface west of RR tracks, continue surface road to Smith Cove Park, connect W Marina Dr with 32nd St. Add direct connection to the 23rd Ave W via ramp or bridge.	1,500 east 1,000 west 1,200 north	8,600	Three structures
19	W Armory Way, cross RR tracks, cross Port uplands, cross Thorndyke Ave W, W Smith St/26th Ave W, 23rd Ave W, W Marina Dr	Begin at W Armory Way and 15th Ave W with at grade I/S, continue on W Armory Way and cross over RR tracks at angle, turn west, and continue elevated structure, meet Thorndyke Ave W at grade, run along W Smith St, and terminate at 26th Ave W. Improve 23rd Ave W with connection to the South with W Marina Dr.	1,500 north 1,000 south	8,000	Two structures
20	Existing bridge footprint, across bluff connect with W Blaine St, W Marina Dr, 32nd Ave W, add direct	Replace bridge at east, continue elevated structure across bluff north of existing bridge, connect with W Blaine St and Condon Way W. Construct drop ramps with connection	4,400	6,900	

No.	Alignment Location	Alignment Description	Bridge/ Structure Length	Total Route Length	Notes
	connection to the 23rd Ave W	to road to Smith Cove Park, connect W Marina Dr.			
21	W Dravus St, Emerson Viaduct	Remove existing Magnolia Bridge without replacement. Improve connections through W Dravus St and Emerson Viaduct.	0	0	Scope of improvements to two other crossing need to be specified
22	W Armory Way, cross over RR tracks, cross Port uplands, W McGraw St	Begin at W Armory Way and 15th Ave W with at grade I/S, continue on W Armory Way approx. 500', turn west and cross over RR tracks, drop and continue surface road, construct tunnel under bluff along W McGraw St with west portal at 32nd Ave W/ McGraw St.	2,800 tunnel 1,600 east	5,500	
23	First Connection: W Galer St flyover, along west side of RR tracks, W Galer St, Second Connection: W Armory Way, cross RR tracks, cross Port uplands, Thorndyke Ave W/23rd Ave W	Surface road from W Galer St flyover, cross under existing bridge, run along west side of RR tracks for approx. 1700', turn west connect with new structure at W Galer St. Begin at W Armory Way and 15th Ave W with at grade I/S, continue on W Armory Way and cross RR tracks at angle, turn west, and continue elevated structure, connect to Thorndyke Ave W at 23rd Ave W.	1,500 south 2,200 north	8,200	Proposed two crossings, one at the South end, another at the North end of T91
24	W Armory Way, north of existing bridge, cross RR tracks, W Galer St, 21st Ave W, existing bridge footprint	Begin at W Armory Way and 15th Ave W with at grade I/S, straight west to cross RR tracks, drop to surface and turn South at bottom of the bluff, connect with new structure at W Galer St. Add at grade street from W Galer St Fly over existing bridge footprint with access to the W Marina Dr. Construct	1,400 north 1,500 south	6,100	

No.	Alignment Location	Alignment Description	Bridge/ Structure Length	Total Route Length	Notes
		new surface street from 21st Ave W to the South, crossing northern route at grade and terminating at street serving Smith Cove Park.			
25	W Armory Way, North of existing bridge, cross RR tracks, Thorndyke Ave W/W Crockett St	Begin at W Armory Way and 15th Ave W with at grade I/S, straight west to cross over RR tracks, continue elevated structure over Port uplands, and cross 23rd Ave W at grade, continue along W Crockett St to Thorndyke Ave W.	2,200	3,200	Mid span ramp connection to T91

5.1.2 Assumptions and Design Constraints

Development and review of each of the candidate alignments were based on the following assumptions and design constraints:

- Some alignments would rely on future roads through Port of Seattle property to provide access to the waterfront and marina. If the alignment would not provide that access directly, it was assumed that the Port roads would provide the necessary public access routes. This assumption will be confirmed with Port staff.
- Alignments that would connect to 15th Avenue West must be able to make a direct free-flowing connection for both southbound traffic from Magnolia and northbound traffic on 15th Avenue West bound for Magnolia. This assumption will be confirmed after further traffic analysis.
- For the purpose of preliminary development, 6.5% was used as the maximum allowable roadway grade. In addition, a bridge crossing rail lines was assumed to have 5 feet of structure depth and provide 23.5 feet minimum vertical clearance over the rail.
- Alignments were not considered if they would impact the petroleum tank farm immediately north of the existing bridge. The cost of remediation necessary to cleanup contamination was considered to be prohibitive.

5.1.3 Analysis Criteria

During the meeting, the team developed a list of baseline fatal flaw elements to be used to eliminate an alignment from further consideration. If an alignment failed to provide the required element, significantly impacted or degraded the element, or was critically and negatively impacted by the element, it would have a fatal flaw. The baseline fatal flaws are as follows:

1. *Vehicular Access to Magnolia* – The alignment should provide equal or better access to Magnolia
2. *Vehicular Access to Interbay* – The alignment should not prohibit or interfere with access to and from the Interbay area.
3. *Vehicular Access to Marina/Waterfront from Magnolia* – The alignment should provide a workable access route to the marina/waterfront area from Magnolia.
4. *Public Access to Waterfront* – The alignment should not interfere with or limit public access to the waterfront.
5. *Olmsted Legacy or Critical Waterfront Parcels* – The alignment should not have a significant negative impact to the Olmsted plan or to important waterfront lands.
6. *Traffic flow on 15^th^ Avenue* – The alignment should not degrade traffic flow on 15th Avenue.
7. *Construction Impacts* – The construction impacts of the alignment should be acceptable to the community.
8. *Cost* – The cost of the alignment should be reasonable.
9. *Hazardous Material* – The alignment should not be critically impacted by identified hazardous materials or contaminated areas.
10. *Major Displacement/Relocation* – The alignment should not cause excessive displacement or relocations of businesses or residents.
11. *Neighborhood Impacts* – The alignment should not have a significant negative impact on the adjacent neighborhoods.
12. *Bicycle and Pedestrian Connections* – The alignment should provide adequate bicycle and pedestrian access by maintaining existing facilities, and not preclude future facilities.

5.1.4 Conclusions

5.1.4.1 Alignments Eliminated from Further Consideration

Alignments 5, 7, 8, 9, 10, 14, 15, 17, 19, 20, 21, and 22 were eliminated from further consideration due to fatal flaws. Specific elimination elements are listed in Table 2. The alignments that were not eliminated have been renamed and will be refined for further evaluation.

- Alignments 7, 8, and 9 rely on the Galer flyover as the connection to 15th Avenue West. They were eliminated because the flyover would have limited capacity to carry traffic from both the waterfront/Port areas and traffic to and from Magnolia.
- Alignment 14 was eliminated after the meeting. Upon further study, construction of a flyover connection at 15th Avenue West at the eastern terminus would have a significant impact on the properties on 15th Avenue West.
- Alignment 21 would eliminate the Magnolia bridge and improve the capacity of West Dravus Street and the Emerson Viaduct. This alignment was eliminated because it would fail to provide equal or improved access to Magnolia. Stakeholder and open house input has indicated a strong desire to maintain a direct connection to the south end of Magnolia and, in addition, to consider a new fourth connection. In view of this, reducing the number of connections to Magnolia would not be acceptable to the community.
- Alignment 22 would construct a tunnel to the interior of Magnolia. This alignment was eliminated due to the high cost of tunnel construction relative to the other bridge options. In addition, the western tunnel portal in Magnolia would displace many residents and its construction would have a large impact on the neighborhood.

5.1.4.2 Alternatives Carried Forward

The first level screening produced nine alternatives to carry forward. To avoid confusion, these alternatives have been renamed with letters in lieu of numbers. The new names for the alternatives carried forward are shown in Table 9.

- Alignments 1A and 4 were considered as variations of Alignment 1. These alignments will be considered together and refined as Alternative A.
- Alignment 18 was considered to be a variation of Alignment 2. These alignments will be considered together and refined as Alternative B.
- Alignment 3 will be further considered and refined as Alternative C.
- Alignment 6 will be further considered as Alternative D.
- Alignment 11 will be further considered as Alternative E.
- Alignment 13 was a variation of Alignment 12 with a different eastern connection point. These alignments will be considered together and refined as Alternatives F2 and F1 respectively.
- Alignment 24 was considered to be a variation of Alignment 16. These alignments will be considered together and refined as Alternative G.
- Alignment 23 will be further considered as Alternative H.
- Alignment 25 will be further considered as Alternative I.

Table 9
First Level Screening Alignment Evaluation

No.	Comments	Fatal Flaw	Disposition	New Name	
1	This alignment will need access to waterfront from Magnolia	none	Consider further	Rename as	A
1A	This is a variation of Alignment 1, consider when refining the alternative	none	Consider as part of Alternative A	Consider with	A
2		none	Consider further	Rename as	B
3	1. Questionable improvement to Magnolia access 2. Indirect access route	none	Refine and further consider	Rename as	C
4	Consider as a variation of Alignment 1	none	Consider as part of Alternative A	Consider with	A
5	1. Significant in-water construction 2. Interferes with waterfront access 3. Impacts waterfront property	4, 5, 8	Eliminate		
6	1. 15th Ave connection is questionable 2. Requires refinement to reduce impact to Port facilities.	none	Consider further	Rename as	D
7	1. Inadequate traffic capacity at Galer flyover 2. Impact to Port facilities	1, 6, 10	Eliminate		
8	Inadequate traffic capacity at Galer flyover	1, 6	Eliminate		
9	1. Inadequate traffic capacity at Galer flyover 2. Indirect route to Magnolia from 15th Ave	1, 6	Eliminate		
10	1. Poor single-point connection to Magnolia 2. Traffic distribution at Magnolia connection problematic 3. Other alignments provide a better connection to Magnolia 4. Connection at 15th has major impact	1,10	Eliminate		
11	Contingent on Port providing access to waterfront	none	Consider further	Rename as	E
12		none	Consider further	Rename as	F1
13		none	Variation of Alternative F	Rename as	F2
14	Eliminated after further evaluation of the impacts at the 15th Ave connection	6,10	Eliminate		

No.	Comments	Fatal Flaw	Disposition	New Name	
15	1. Traffic distribution at Magnolia connection problematic 2. Alignment 16 provides a better connection to Magnolia	1	Eliminate		
16		none	Consider further	Rename as	G
17	1. Requires elevated intersection 2. Access to the waterfront from Magnolia is difficult	2, 3, 8	Eliminate		
18		none	Variation of Alternative B	Rename as	B
19	1. Access to south not needed 2. Requires 2 bridges on magnolia side	8	Eliminate		
20	1. Impact to parklands on Magnolia 2. Does not improve access to Magnolia 3. Elevated intersection costly	1, 8, 11	Eliminate		
21	Does not improve access to Magnolia	1	Eliminate		
22	1. Tunnel portal at Magnolia has significant construction and traffic impact. 2. Significant cost impact	7, 8, 10, 11	Eliminate		
23		none	Consider further	Rename as	H
24	This is a variation of Alignment 16, consider when refining the alternative	none	Variation of Alternative G	Consider with	G
25		none	Consider further	Rename as	I

5.2 SECOND LEVEL SCREENING

5.2.1 Alternatives Considered

Nine alternatives were carried forward from the first level screening. Brief descriptions of these alternatives are given below. In each of the alternatives, it is assumed there would be a north-south surface road connecting to 21st Avenue West at the north end and West Marina Place at the south end.

5.2.1.1 Alternative A

Alternative A would replace the existing bridge with a new structure immediately south of the existing bridge. The alternative would construct a diamond interchange in the bridge's mid-span to provide access to the waterfront and the Port uplands property. Connections at the east and west ends of the bridge would be similar to existing.

5.2.1.2 Alternative B

Alternative B would replace the eastern end of the bridge to cross the BNSF Railway tracks and drop to ground level west of the railroad tracks. The surface road would provide access to Port uplands property and continue along the waterfront. Past Smith Cove Park and the marina, the alternative would connect West Marina Drive to 32nd Avenue West with a surface road or low bridge over the tidelands. The section of 32nd Avenue West between the waterfront and Clise Place West would be reconstructed.

5.2.1.3 Alternative C

Alternative C would replace the eastern end of bridge to cross the BNSF tracks and drop to ground level. West of the railroad tracks, the surface road would turn to the north through the Port property. This alternative would replace the west end of the existing bridge with fill and/or a new structure that would wrap from north to south along the contours of the Magnolia hillside before connecting to West Galer Street. The alternative would also add a new surface road with a connection to 21st Avenue West.

5.2.1.4 Alternative D

Alternative D would construct a new bridge in the form of a long arc north of the existing bridge. New ramps would be constructed to connect with 15th Avenue West (at the existing connection point). This alternative would construct a diamond interchange in the bridge mid-span to provide access to waterfront and the Port uplands property.

5.2.1.5 Alternative E

Alternative E would construct a flyover ramp from 15th Avenue West northbound to West Wheeler Street and continue straight west across the railroad tracks and Port uplands with an elevated structure. The west end of this alternative would connect to the intersection of Thorndyke Avenue West and 23rd Avenue West. The alternative would construct half of a diamond interchange to provide a connection with the Port uplands from the east side only (the grade is too steep to connect from the west). A new surface road connection would be created with 21st Avenue West to the north and the waterfront to the south.

5.2.1.6 Alternative F

Alternative F consists of two options:

5.2.1.6.1 Option F1

Option F1 would be a flyover ramp from 15th Avenue West continuing on West Armory Way and crossing over railroad tracks at an angle. This option would then turn west and continue on an elevated structure to connect with Thorndyke Avenue West at West Halladay Street.

5.2.1.6.2 Option F2

Option F2 would be a flyover ramp from 15th Avenue West continuing straight west to West Wheeler Street, and connecting with Thorndyke Avenue West at West Halladay Street. Access to the marina area in both options would be provided via an extension of 21st Avenue West southerly across Port uplands.

5.2.1.7 Alternative G

Alternative G would construct a flyover ramp from 15th Avenue West northbound to West Armory Way. The alternative would continue on West Armory Way approximately 500 feet, turn west and cross over the railroad tracks, drop down to ground level and continue westerly along a surface road. The main route would then continue southward with new fill and/or a structure to connect with West Galer Street. The secondary surface connection northward would connect to 21st Avenue West and southward to West Marina Drive.

5.2.1.8 Alternative H

Alternative H would include a north and a south crossing (the south crossing would not provide the necessary capacity alone):

- South Crossing: A surface road from the west end of the West Galer Street flyover would cross under the existing bridge, run along the west side of railroad tracks for approximately 1,700 feet, and turn west to connect with a new structure ascending to Magnolia at West Galer Street. Access to Port uplands and the waterfront would be provided by a surface connection north to 21st Avenue West and south and west to West Marina Drive.
- North Crossing: Beginning with a flyover at West Armory Way and 15th Avenue West, the alternative would continue on West Armory Way and cross the railroad tracks at a skewed angle. The alternative would continue the elevated structure, turn west, and connect to Thorndyke Avenue West at 23rd Avenue West.

5.2.1.9 Alternative I

Alternative I would begin with a flyover at West Armory Way and 15th Avenue West, move straight west across the railroad tracks, and continue on an elevated structure over the Port uplands. The alternative would cross over 23rd Avenue West and continue along West Boston Street to Thorndyke Avenue West. Ramps to and from the east would provide surface access to the Port uplands and the marina.

5.2.2 Project Team – First Evaluation

The study team developed detailed criteria to evaluate the nine alternatives carried forward. Evaluation criteria were split into four general categories:

- Environmental
- Transportation
- Urban Design
- Cost

Each alternative was evaluated based on equal weighting of all four categories. The results of each category were totaled to help prioritize the surviving alternatives in terms of functionality and impacts.

5.2.2.1 Recommendations

The Project team met with the City of Seattle on November 25, 2002 to discuss the alternative recommendations that had been developed from the evaluation criteria. A "First Evaluation," dated November 29, 2002, was prepared by the Project Team to document the results of this discussion. A summary of the evaluation is contained in Table 10.

Table 10
Second Level Screening First Evaluation Summary
Project Team – November 29, 2002

Alternative	Comments: Advantages	Comments: Disadvantages	Evaluations: Environmental	Evaluations: Transportation	Evaluations: Urban Design	Evaluations: Cost	Recommended for Further Development
A	• No business or residential displacements identified. • Good access to Magnolia. • Retains dramatic views and entry into Magnolia. • Lowest right-of-way costs.	• Requires construction adjacent to or over shoreline. • Existing bridge shut down for extended periods. • Interbay property separated from water. • High construction costs.	**	*	X	*	
B	• No business displacements identified. • Improved access to waterfront and Magnolia Village area. • Could create a beautiful route into Magnolia. • Medium construction, right-of-way & relocation costs.	• Potential direct impacts to aquatic shoreline and relatively high geological hazard impacts. • Less direct route to Galer and Thorndyke areas. • Much more compatible with a second access route. • Highest mitigation costs.	X	**	**	**	√
C	• No residential displacements identified. • Improved access to waterfront from Magnolia. • Low relocation and right-of-way costs.	• Requires construction adjacent to or over shoreline. • Less direct and slower route to Magnolia. • All Magnolia traffic comes through center of Port property. • High construction and mitigation costs	*	*	X	*	
D	• No residential displacements identified. • Improved access to waterfront, Magnolia, and Port property. • Allows land to be connected to water. • Low mitigation and right-of-way costs	• Potential displacement of businesses on Port of Seattle properties. • Some bridge closures during construction. • Some view blockage of water from Port uplands. • Highest construction costs.	**	**	**	X	√
E	• No shoreline impacts. • Possible traffic benefits along 15th Ave. • Include Thorndyke improvement per Olmsted plan. • Medium construction costs.	• Business and residential displacements. • No direct access from Magnolia to waterfront. • Ramps impact land use along 15th Avenue corridor. • Highest relocation and right-of-way costs.	X	X	X	X	

Alternative	Comments: Advantages	Comments: Disadvantages	Evaluations: Environmental	Evaluations: Transportation	Evaluations: Urban Design	Evaluations: Cost	Recommended for Further Development
F	• No shoreline impacts. • Possible traffic benefits along 15th Ave. • Original Olmsted route: include Thorndyke improvement per Olmsted plan. • Lowest construction costs.	• Business and residential displacements. • No direct access from Magnolia to waterfront. • Does not adequately support development on Port property. • Highest relocation costs.	X	X	X	**	
G	• No shoreline impacts. • Improved access to waterfront and Port property. • Central access for Port property. • Medium construction costs.	• Requires significant construction in steep slope areas. • Less direct route to Magnolia. • Ramps impact land use along 15th Avenue corridor. • High mitigation and right-of-way costs.	*	X	X	**	
H	• No shoreline impacts. • Two access points to Magnolia. • Choices will reduce unnecessary traffic on bluff and Thorndyke. • Lowest mitigation costs.	• Business displacements on Port of Seattle properties. • Worse access to waterfront and port property from 15th Ave. • Ramps impact land use along 15th Avenue corridor. • High construction costs.	**	**	**	X	√
I	• No shoreline impacts. • Good access to Magnolia. • Parcelization of Port property is workable. • Medium construction costs.	• Business and residential displacements. • No direct access from Magnolia to waterfront. • Neighborhood has heavy localized impacts along Boston. • High relocation costs.	X	X	X	X	

Notes: ** = Best Alternatives, * = Good Alternatives, X = Not Recommended, √ = Recommended for Development

5.2.3 Design Advisory Group – First Evaluation

The Design Advisory Group met on December 4, 2002 to review the Project Team's evaluation of the nine alternatives carried forward from the first screening. The comments from this meeting are summarized below. (Each bullet represents a comment made by a Design Advisory Group member.)

- Time is needed to digest the information that has been presented on the nine alternatives before any meaningful recommendation can be made.
- The analyses need to be reviewed before the alternatives can be rated.

- Alternative A is good because it has worked for so long.
- Favor Alternatives A, B, and D, but Alternative B is the best. Alternative B would be a beautiful ride. How would a bike path be built onto the bridge?
- Positive comments for A, B, and H. Alternative B would be a nice entry into Magnolia.
- Alternative B provides opportunity to use Port property and doesn't compromise land development. It presents a tremendous opportunity to create an interesting shoreline.
- Don't route into Thorndyke and don't relocate any businesses. Concern about waterfront and park access makes Alternative B look good.
- If B works with the flow of traffic on 15th Avenue West and existing businesses in Interbay, then maybe it would work, but it introduces intersections.
- Alternative B may have fatal flaws: one intersection to the Village, seismic issues.
- Some combination of Alternatives B and H would be good. Make Alternative B a smaller alignment along the bluff with a second connection.
- Can some of the alternatives be combined in a different way? Alternative B is good because of the waterfront usage. H is good because of two access points. Could the Alternative B alignment be part of Alternative H?
- There is a lot to like in Alternative D, but Alternative H is better.
- Alternative D is good because it goes straight to Magnolia with no intersections.
- Alternative D would be good from a monorail perspective.
- Alternatives E and F work well because of the vessels that use fisherman's terminal and Pier 91.
- Alternative F doesn't rate well, but it goes up and over the railroad at a good spot.

5.2.4 Public and Community Group Comments

The nine alternatives were presented to the public at an Open House on December 5, 2002. Written comments were gathered from tablets posted next to each alternative, from mail-in comment forms, and from email. Table 11 quantifies the number of written comments that were for or against specific alternatives. The four alternatives with the greatest number of positive comments were Alternatives A, B, D and H.

Table 11
Open House Comment Distribution

Alternative	Positive Comments	Negative Comments
A	56	6
B	36	38
C	0	27
D	34	9
E	6	38
F	4	35
G	4	20
H	16	16
I	6	38

Source: EnviroIssues, 2002

5.2.5 Project Team – Second Evaluation

On December 12, 2002, members of the Project Team and Seattle City staff met to determine which three alternatives should be recommended to be carried forward for further evaluation. Alternative B received little consideration at this time due to the Elliott Bay Marina to 32nd Avenue West Access Agreement restrictions (see paragraph 5.2.5). The team believed that this alternative should be eliminated from further consideration. During the discussion, good western and eastern connection points were identified to help eliminate alternatives that would provide poor connections.

5.2.5.1 Western Connections

Two good western connections were identified:

- The western connection of the existing Magnolia Bridge
- The intersection of 23rd Avenue West and Thorndyke, which would provide enough space and a functional "T-shaped" intersection.

Western connections that would connect to Thorndyke at intersections other than 23rd Avenue West were eliminated because they could create significant neighborhood impacts (cut-through traffic from those attempting to get to the Village or those trying to leave southern and western Magnolia). It was also noted that the northern Thorndyke connection might not truly serve the purpose of "getting people to Magnolia." Although cars could physically get to the neighborhood, drivers' ability to get to the Village or access southern or western points of Magnolia would not be well served by a northern connection. The northern Thorndyke connection may only work in partnership with a southern route.

5.2.5.2 Eastern Connections

Four viable eastern connections were identified. These connections could be modified in terms of elevation–whether surface intersections or grade separations are provided.

- West Wheeler Street
- The existing West Garfield Street connection to the Magnolia Bridge
- West Armory Way
- West Galer Street

Other eastern connections resulted in significant residential and/or business displacements and/or made poor transportation connections, and were eliminated.

5.2.5.3 Recommendations

The Project Team recommended that Alternatives A, D, and H be carried forward. The Team suggested that Alternative H either connect to the existing Garfield overpass that currently provides linkage to the bridge over the railroad, or that a southern exit ramp be provided from the West Galer Street Flyover to 15th Avenue West.

The Team recommended that Alternatives B, C, D, F, G, and I be eliminated from further consideration.

5.2.6 Design Advisory Group – Second Evaluation

The Design Advisory Group met on January 8, 2003 to review the Project Team's second evaluation of the nine alternatives carried forward from the first screening. The Group determined that Alternative B has merit, but there is not enough information at this time to directly compare it with Alternatives A, D, and H. The Group does not want to drop this alternative and supports carrying Alternative B forward along with Alternatives A, D, and H for further evaluation and development of direct quantitative information for comparison.

5.2.7 City of Seattle Determination

The City of Seattle determined that Alternative B was not a viable option because it would violate the City's shoreline policies. The Seattle Municipal Code states: "Except for bridges necessary to cross a water body, new streets shall be permitted in the Shoreline District only if necessary to serve lots in the Shoreline District or to connect to public access facilities." Seattle's Comprehensive Plan states: "Streets, highways, freeways and railroads should be located away from the shoreline in order to maximize the area of waterfront lots and minimize the area of upland lots. Streets, highways, freeways and railroads not needed for access to shoreline lots shall be discouraged in the Shoreline District."

On April 15, 2003, the Mayor of Seattle said in a letter to the Magnolia neighborhood, "I have decided not to pursue a Magnolia bridge replacement plan that includes the shoreline alternative, known as Alignment B, and have directed SDOT to no longer consider it."

5.2.8 Alternatives Eliminated from Further Consideration

All of the evaluation criteria and comments were used along with the identification of the best western and eastern connections to eliminate alternatives from further consideration. Alternatives B, C, E, F, G, and I were eliminated from further consideration for the following reasons:

5.2.8.1 Alternative B

- Would violate City of Seattle shoreline policies.

5.2.8.2 Alternative C

- Low public support (traffic flow is poor given the 90-degree turn on the Port property and poor direct access to Magnolia).
- Would take drivers out of the desired direction of travel and add stop lights.
- Low preliminary evaluation rankings.

5.2.8.3 Alternative E

- Although Alternative E connects at desirable locations, it would result in an adverse change in traffic patterns. Connecting to Thorndyke only works when in combination with a southern route.
- People in south and west Magnolia not happy with indirect route and traffic cutting through neighborhoods.

- Low preliminary evaluation rankings.
- Low public support.

5.2.8.4 Alternative F

- Poor connection point to Thorndyke.
- Doesn't provide good connection for future development of the Port property.
- Low preliminary evaluation rankings.
- Low public support.

5.2.8.5 Alternative G

- Does not include a southern connection and would create a very long route compared to existing.
- Low public support (impression that it's catering exclusively to Port property access).
- Low preliminary evaluation rankings.

5.2.8.6 Alternative I

- Poor connection to Thorndyke and poor eastern connection point.
- Would create severe neighborhood impacts.
- Low public support (especially given residential dislocation on the west along West Boston Street).
- Low preliminary evaluation rankings.

5.2.9 Alternatives Carried Forward

The second level screening recommended three alternatives to carry forward: Alternatives A, D, and H. These three alternatives will be developed to a greater level of detail in the environmental impact statement process which is the next phase of this study.

5.2.9.1 Alternative A

This alternative received good public support because it would not be much of a change from current conditions. There would be some environmental issues dealing with construction near and over water. Provisions for ramps to and from the west, and access to the marina need further study.

5.2.9.2 Alternative D

This alternative received good public support because it would swing to the north and open up the waterfront. The impact on existing businesses needs further study.

5.2.9.3 Alternative H

This alternative also received good public support and would rely on two alignments working in combination to effectively support traffic. The alignment needs a ramp from West Galer Street onto southbound Elliott Avenue West. Connections to the Port property from the north alignment need to be investigated.

Further evaluation under NEPA/SEPA will likely induce some modifications to the three alternatives as currently presented. The connection points and the general routing will remain the same, but specific ramp locations and alignments will be modified as necessary to provide design enhancements and reduce impacts.

5.3 SCREENING OF ALTERNATIVE A, D, AND H

The three alternatives analyzed in the initial environmental discipline reports were Alternatives A, D and H. In March 2004, the City removed Alternative H from consideration because review of traffic operations found that the option would be unable to handle the future forecast traffic volumes. Alternative H was replaced with Alternative C, the next best of the nine evaluated alignments.

In Spring 2005, the City received feedback through the public involvement process that rehabilitating the existing bridge structure to current load and design standards should be evaluated. The Rehabilitation Alternative was developed and analyzed for its environmental effects.

5.3.1 Alternative A

Alternative A (Figure 14) builds a new structure immediately south of the existing bridge. Construction can be staged to allow the existing bridge to be used as long as possible. Two ramps provide access to the waterfront and the Port of Seattle North Bay/Terminal 91 complex to and from the east. Connections at the east and west ends of the bridge are similar to the existing bridge.

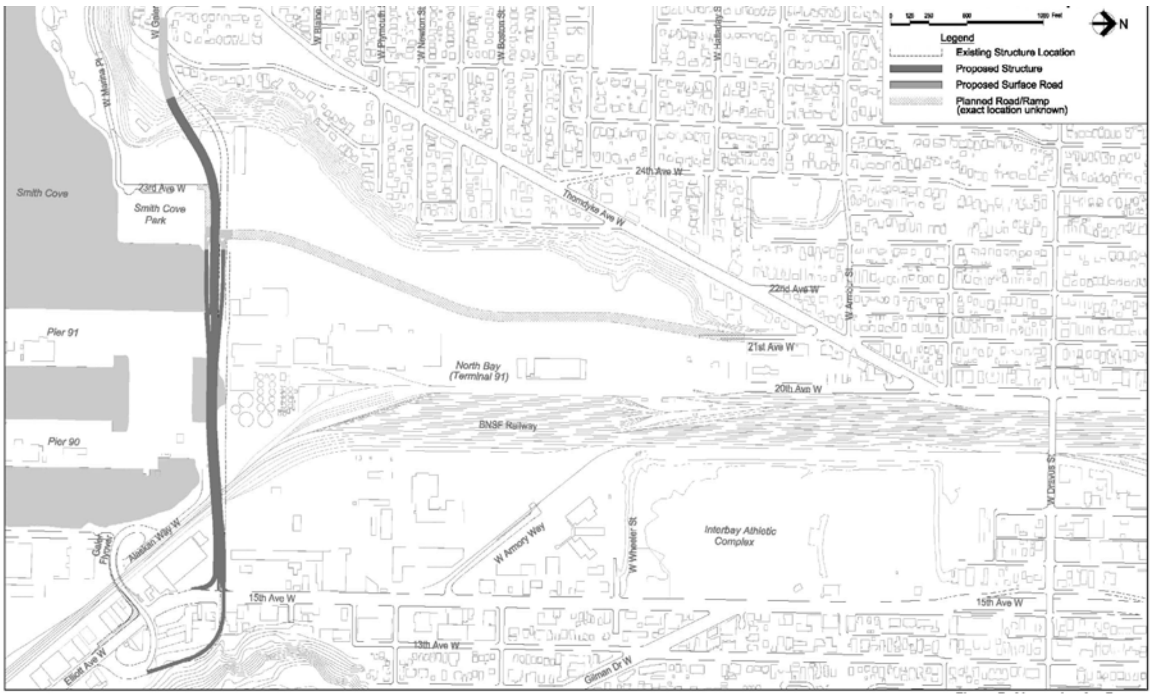


Figure 14
Alternative A

5.3.2 Alternative D

Alternative D (Figure 15) builds a new bridge in the shape of a long arc north of the existing bridge. Construction can be staged to allow the existing bridge to be used longer than Alternative A. Connections at the east and west ends of the bridge are similar to the existing bridge and ramps provide access to and from the waterfront and the North Bay/Terminal 91 complex.

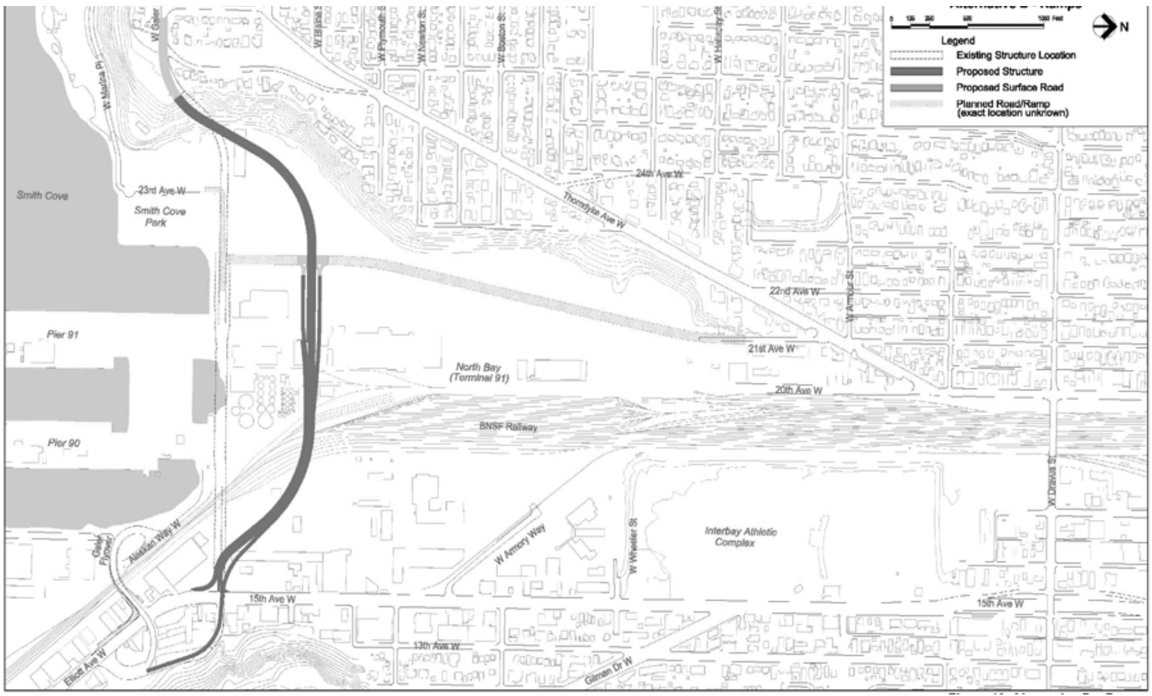


Figure 15
Alternative D

5.3.3 Alternative H – Removed from consideration

Alternative H (Figure 16) was removed from consideration because there would be excess traffic congestion and delay at the Alaskan Way West and Galer Flyover intersection.

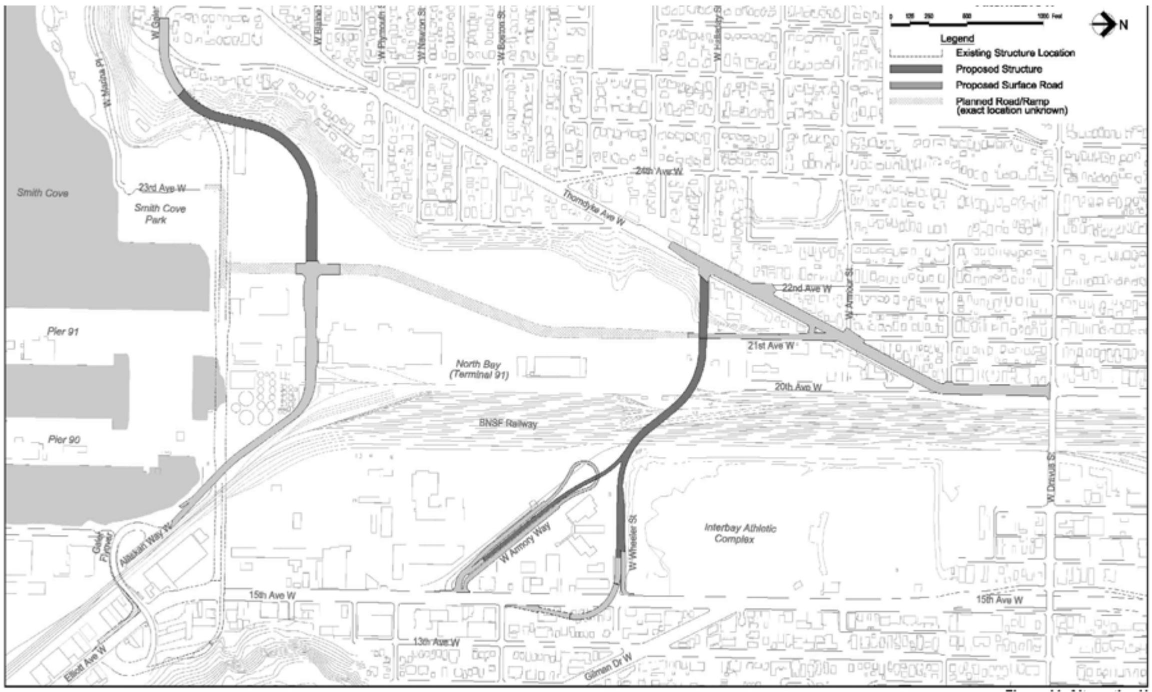


Figure 16
Alternative H — Removed from consideration

5.3.4 Alternative C – Added to replace Alternative H

Alternative C (Figure 17) constructs 2,200 feet of surface roadway within the Port of Seattle North Bay/Terminal 91 property between two structures. A bridge descends from Magnolia Bluff along the toe of the slope and reaches the surface while still next to the bluff. After turning south along the east side of Terminal 91, the road rises to cross the railroad tracks and connects to 15th Avenue West. Alternative C provides a unique surface/structure combination that is distinctly different from Alternatives A and D.

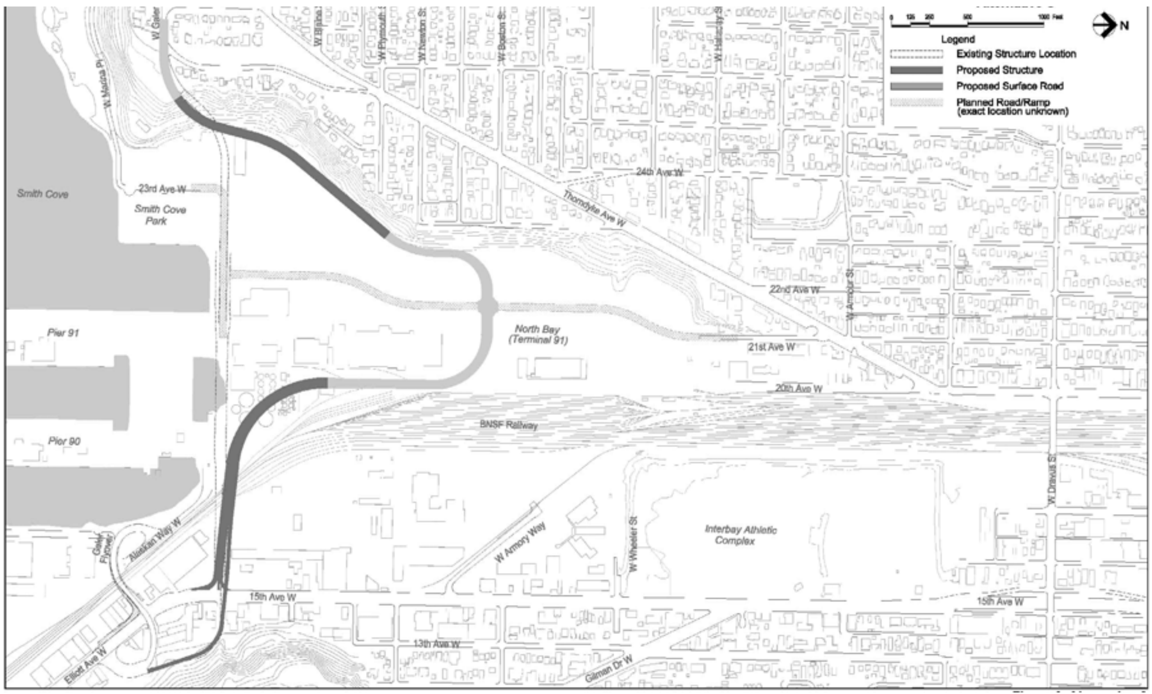


Figure 17
Alternative C

5.4 ALIGNMENT STUDY CONCLUSIONS

In March 2006, Mayor Nickels directed SDOT to choose Alternative A as the preferred alternative to replace the Magnolia Bridge. Alternative A will replace the existing bridge with a new structure immediately south of the existing bridge. Ramps will provide access from the bridge's mid-span to the waterfront and the Port of Seattle's uplands property. Connections at the east and west ends of the bridge will be similar to the existing bridge. Alternatives A and D were similar technically in design and transportation operation. Alternative A, however, was favored by the project's Design Advisory Group, the Port of Seattle, Terminal 91 industrial tenants and the general public.

Alternative A was selected as the preferred alternative because it:

- Responds to local transportation needs
- Was a strong alternative based on environmental and technical analysis
- Received significant neighborhood, business, and governmental agency support, including that of the Port of Seattle
- Provides the least disruption to residents on Magnolia's eastern edge and businesses located under and next to the bridge
- Allows Interbay business owners greater certainty in planning for future expansion or development
- Costs less than other proposed alternatives
- Would result in a Finding of No Significant Impact (FONSI)

6. ENVIRONMENTAL STUDIES

Table 12 summarizes impacts of the build alternatives following analyses in fourteen discipline reports prepared from late 2003 through 2005. The initial drafts of the discipline reports were prepared in fall and winter 2003. In spring 2004, Alternative H was dropped from further consideration and Alternative C was added. Revised discipline reports incorporating Alternative C were prepared in summer 2004. All fourteen discipline reports have been through City of Seattle review and have completed review by the Washington State Department of Transportation.

The Rehabilitation Alternative (see Section 3 of this report) was developed as a refinement of previous (1992 through 1997) recommendations for rehabilitation and retrofit measures, and to bring the project up to current design standards that would allow the project to receive funds through federal and state bridge programs. Preliminary impacts were estimated in fall 2005 on the basis of conceptual development and design analysis through September 2005. The Rehabilitation Alternative involves bringing the bridge up to current load and design standards using the existing bridge structure, to the extent possible. This alternative would include replacing the bridge deck (roadway), stabilizing the foundation and concrete columns supporting the bridge (improving the soil by injecting grout to prevent liquefaction during an earthquake), replacing existing concrete bracings currently supporting the bridge with steel bracings, and adding a corrosion control system. This alternative retains the horizontal and vertical alignment of the existing bridge, except where it is feasible to correct geometric deficiencies such as inadequate stopping sight distance.

The environmental study conclusions table was prepared to support a review of the project's environmental classification. When the project included Alternative H, there were traffic pattern revisions and business and residential displacements that resulted in the project classification as a NEPA Class I project requiring an Environmental Impact Statement (EIS). The Notice of Intent (NOI) to prepare an EIS was published in the April 25, 2003 Federal Register. A SEPA Declaration of Significance was published in the SEPA Register.

By dropping Alternative H, project alternative impacts were substantially reduced. All project build alternatives would use the same eastern and western arterial system connections as the existing structure that the alternatives replace. The project would retain substantially the same lane configuration and vehicle carrying capacity as the existing facility. The discipline report analyses indicated that impacts are not significant or could be successfully mitigated. On this basis, the Federal Highway Administration and Washington State Department of Transportation have classified the project as NEPA Class III, requiring preparation of an Environmental Assessment.

Results of the 14 discipline results are being summarized in an Environmental Assessment (EA). The draft EA will be circulated in late 2007 or early 2008.

Table 12 Environmental Study Conclusions

NEPA Element of the Environment	Alternative A	Alternative C	Alternative D	Rehabilitation Alternative
Geology, Soils, and Topography	Slope instability at cuts mitigated by retaining walls. Liquefaction and lateral spreading mitigated by ground improvement measures.	Slope instability at cuts mitigated by retaining walls. Liquefaction and lateral spreading mitigated by ground improvement measures.	Slope instability at cuts mitigated by retaining walls Liquefaction and lateral spreading mitigated by ground improvement measures	Liquefaction and lateral spreading mitigated by ground improvement measures. Mitigate groundwater impacts caused by ground improvement measures.
Waterways, Hydrological Systems, and Floodplains	Project would add up to 1.2 acres of impervious surface to study area. About 3.2 acres would be in 200-foot shoreline area.	Project would add up to 0.2 acres of impervious surface to study area. About 0.2 acres would be in 200-foot shoreline area.	Project would remove 0.3 acres of impervious surface from study area.	Project would be similar to No Build. About 2.7 acres would be in 200-foot shoreline area.
Water Quality	Storm water would be treated before discharge.			
Wetlands	No areas with potential wetland characteristics were identified during the field reconnaissance.			
Vegetation	Minor impacts to upper intertidal vegetation that is not habitat for endangered species. One-half acre of forest removed.	About 0.3 acre of forest and disturbed vegetation displaced.	About 0.3 acre of forest and disturbed vegetation displaced.	Minor impacts to upper intertidal vegetation for ground improvement and foundation rehabilitation that is not habitat for endangered species; 0.3 acre or less of vegetation disturbance for foundation rehabilitation.
Fish, Wildlife, and Habitat	About 0.1 acre of intertidal habitat removed for four bridge piers. About 0.5 acre of forest habitat removed.	About 0.3 acre of forest and disturbed habitat at the west end of the bridge would be removed.	About 0.3 acre of forest and disturbed habitat at the west end of the bridge would be removed.	0.3 acre or less of habitat disturbance for foundation rehabilitation.
Air Quality	Air quality standards for CO met in the analysis years.			
Noise	Under all alternatives, future noise levels would exceed WSDOT impact thresholds at first row sensitive receptors along West Galer Street. However, no substantial noise impacts, defined by WSDOT as 10 dbA over existing conditions, or severe noise impacts, defined by WSDOT as 75 dbA or 15 dbA Leq over existing conditions, were predicted under any of the analyzed alternatives.			

NEPA Element of the Environment	Alternative A	Alternative C	Alternative D	Rehabilitation Alternative
Energy	Operational energy about the same as No Build	Daily operational energy use higher than No Build due to 0.5 mile travel distance increase.	Daily operational energy use higher than No Build due to 0.1 mile travel distance increase.	Operational energy about the same as No Build
Prime and Unique Farmlands	No farmlands present.			
Hazardous Materials	Potential for contamination in foundation excavations. Lead-based paint on steel portions of existing bridge to be demolished.	Potential for contamination in foundation excavations. Potential for asbestos and lead-based paint in buildings to be demolished. Lead-based paint on steel portions of existing bridge to be demolished.	Potential for contamination in foundation excavations. Potential for asbestos and lead-based paint in buildings to be demolished. Lead-based paint on steel portions of existing bridge to be demolished.	Potential for contamination in foundation excavations. Lead-based paint on steel portions of existing bridge to be demolished.
Traffic and Transportation	Operates the same as the No Build alternative. Requires 14- to 20-month bridge closure with 8-minute detour.	Operates similar to the No Build alternative. Adds half-mile (about one minute vehicle travel time) to route. Less than 20-second average delay at surface street intersection. Requires 8- to 14-month bridge closure with 8-minute detour.	Operates similar to the No Build alternative. Requires 6- to 12-month bridge closure with 8-minute detour.	Operates the same as the No Build alternative. Requires 21- t o 27-month bridge closure with 8-minute detour.
Visual Quality	Some impact due to increase structure width compared to No Build. Cleaner appearance under the bridge compared to No Build with removal of existing structure and its steel retrofit framing.	Somewhat reduced impact due to increased distance from park land compared to No Build. Cleaner appearance under the bridge compared to No Build with removal of existing structure and its steel retrofit framing.	Somewhat reduced impact due to increased distance from park land compared to No Build. Cleaner appearance under the bridge compared to No Build with removal of existing structure and its steel retrofit framing.	Similar to No Build, but removal of much of the under-bridge steel retrofit framing.

NEPA Element of the Environment	Alternative A	Alternative C	Alternative D	Rehabilitation Alternative
Land Use	Consistent with Seattle, Port, and BINMIC policies. Would be constructed in Shoreline District (similar to existing bridge).	Consistent with Seattle, Port, and BINMIC policies.	Consistent with Seattle, Port, and BINMIC policies.	Consistent with Seattle, Port, and BINMIC policies. Some construction would be in Shoreline District.
Socioeconomics	No residential displacement. Potential relocation of or alternative access to one business prior to construction.	No residential displacement Requires mitigation of impacts to Trident Seafoods and alternative access to one business. Displaces one business.	No residential displacement. Potential relocation of three businesses and one vacant business property, and potential relocation or alternative access to one business.	No residential or displacement. Potential relocation or alternative access to one business prior to construction.
Recreation	Bridge would be built over about 0.9 acre of park land and three bridge piers would be constructed on park land. This use would be mitigated through a joint development agreement.	Bridge would be built over about 0.3 acre of park land. This use would be mitigated through a joint development agreement.	Bridge would be built over about 0.3 acre of park land. This use would be mitigated through a joint development agreement.	Construction would be in existing rights of way and easements adjacent to park land.
Displacements/ Environmental Justice	Potential displacement of Anthony's Seafood – building access revision may avoid this displacement or business would be relocated.	Potential displacement of Anthony's Seafood – building access revision may avoid displacement or business would be relocated. Trident Seafood building access would be reconfigured. Snider Petroleum business would be relocated.	Potential displacement of Anthony's Seafood – building access revision may avoid this displacement or business would be relocated. Snider Petroleum business would be relocated. The building housing part of City Ice operations would be removed.	Potential displacement of Anthony's Seafood – building access revision may avoid this displacement or business would be relocated.
Services and Utilities	No change in demand for public services. No increase for emergency response vehicles between 15th Ave. W and Magnolia.	0.5 mile increase for emergency response vehicles between 15th Ave. W and Magnolia.	0.1 mile increase for emergency response vehicles between 15th Ave. W and Magnolia.	No change in demand for public services. No increase in distance for emergency response vehicles between 15th Ave. W and Magnolia.

NEPA Element of the Environment	Alternative A	Alternative C	Alternative D	Rehabilitation Alternative
Cultural, Historic, and Archaeological Resources	Potential for below ground archaeological resources. An archaeologist may monitor some areas during construction. No historic resources affected.	Potential for below ground archaeological resources. An archaeologist may monitor some areas during construction. No historic resources affected.	Potential for below ground archaeological resources. An archaeologist may monitor some areas during construction. No historic resources affected.	Potential for below ground archaeological resources. An archaeologist may monitor some areas during construction. No historic resources affected.

7. ALIGNMENT A ALTERNATIVE LOCATION STUDY

This study investigated the optimum location for the new bridge at Alignment A from the standpoint of maintenance of traffic, ease of construction, construction duration, and environmental impacts.

Alternative A, as selected in the Alignment Study, is parallel to the existing bridge and located far enough to the south to construct most of the mainline structure west of the BNSF Railway crossing without having to build the structure in stages. This alignment was developed in the Alignment Study because large sections of the new bridge could be constructed while maintaining traffic on the existing structure, thus reducing the required closure time for the Magnolia Bridge and the use of a detour route or routes for the bridge's users.

Several alignments are evaluated in this study, including: Alignment A-South, located south of the existing bridge; Alignment A-North, located on top of the existing bridge alignment; and Alignment A-Hybrid, which is shifted far enough south to allow staged construction of the section of bridge between existing Bent 46 and Bent 66. This is the section between the west side of Pier 91 to about 23rd Avenue West

For the three alignment alternatives investigated as part of this study, there are a couple of similarities in the alignment which are common to all. These include:

- The structures east of the BNSF Railway are the same for all three alternatives and are located directly on top of the existing bridge alignment. This includes the approach fill ramp and structure from 15th Avenue West and the 15th Avenue Overpass.
- The new structure west of existing Bent 66, the bluff structure, will be located to the south of the existing alignment and can be constructed without impacting the existing bridge structure. The horizontal alignment of the section is revised to accommodate larger radii curves required to meet the design standard horizontal sight distance.
- The west abutment location on Magnolia Bluff is the same for all three alternatives.

Prior to this study, only Alignment A-South was evaluated during the Alignment Study phase.

7.1 STAGED DEMOLITION OPTIONS EXISTING BRIDGE

Before investigating the alignment alternatives, the existing bridge structure was evaluated to determine the staged construction options available based on the existing bridge layout. The existing bridge was analyzed to determine feasible alternatives for demolishing portions of the structure while maintaining a desirable minimum roadway width of 22 feet, which will accommodate two 11-foot lanes of traffic during construction. The existing bridge staging options were evaluated in five sections: the 15th Avenue West overpass structure; the 15th Avenue West ramp and thru-girder bridge over the BNSF Railway, the center two-way ramp structure to the Terminal 91 Main Gate; the bridge transition structure to the off- and on-ramps to 23rd Avenue West; and the structure west of Bent 46 to Magnolia Bluff.

The following provides descriptions of each section of the existing bridge along with possible staging options in those areas. The discussions focus on removing portions of the existing structure on the south side of the existing bridge because the alignments will either be in the current location or to the south of the existing bridge. For a plan view of a feasible demolition plan, refer to Figure 18. Removing portions of the existing bridge by partial demolition should be reviewed in detail during final design because the existing structure has reduced live load capacity, has significant deterioration, and is comprised of multiple support systems that were installed over life of the bridge. Partial demolition may result in the remaining portions of the structure requiring temporary support systems to provide sufficient live load capacity.

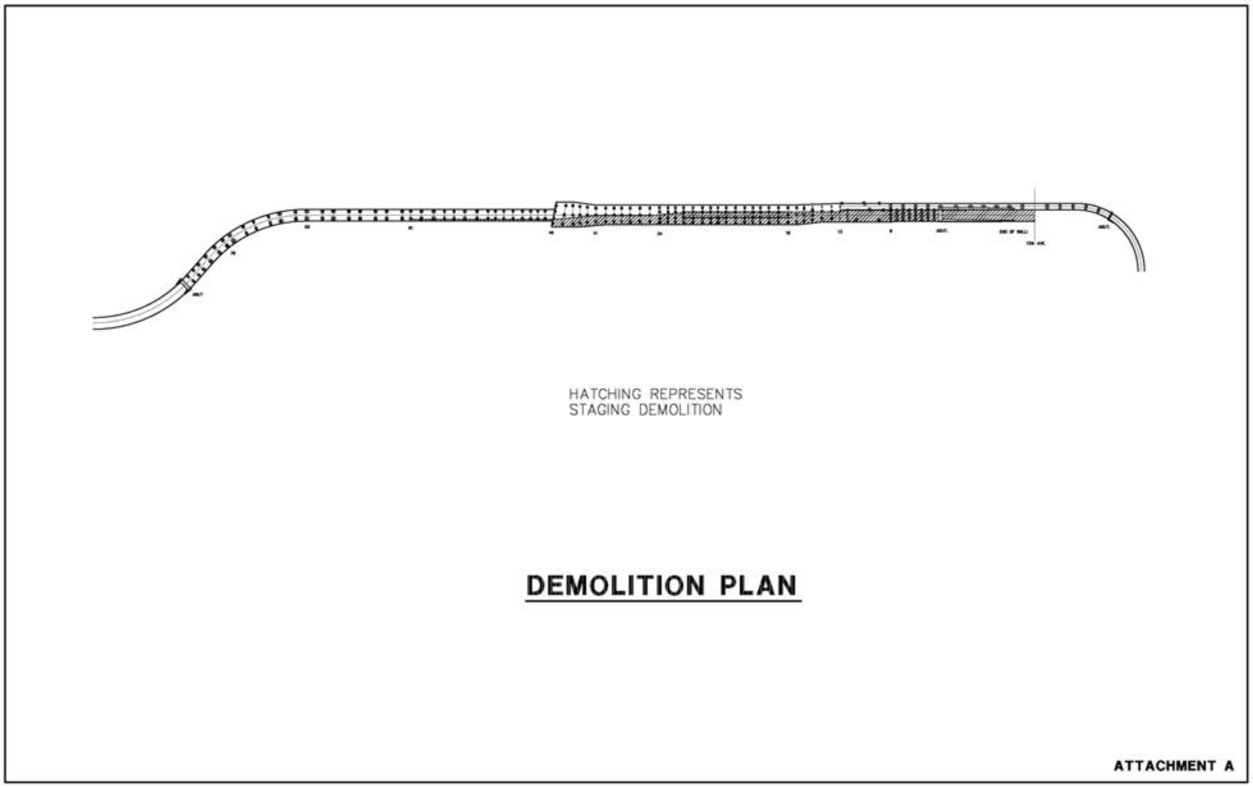


Figure 18
Demolition Plan

7.1.1 15th Avenue West Overpass Structure

The 15th Avenue West overpass currently carries westbound traffic over 15th Avenue West. The bridge has an overall width of 25 feet and a roadway width of 20 feet. The superstructure is precast concrete girders supported on single column bents. Due to the limited structure width and single column support system there are few options for demolishing portions of the bridge while maintaining traffic.

An option is to convert this overpass into a two way structure to allow access to and from Magnolia during construction of the new bridge. This would result in opposing traffic with 10-foot lanes. This is below the desirable minimum lane width. Traffic operations and safety analyses determined that this roadway configuration is not viable for construction period traffic.

The option of converting the overpass into a two way structure will be discussed in further detail in the following section; Two-Way Traffic on 15th Avenue West Overpass Structure.

7.1.2 15th Avenue West Ramp and Thru-Girder Bridge Over BNSF Railway

This section of the Magnolia Bridge includes the mainline approach fill ramp and bridge structure from 15th Avenue West, the thru-girder structure over the railroad, and the structure from the railroad to the center two-way ramp. The approach fill ramp is retained by conventional concrete cantilever retaining walls and a concrete cantilever abutment wall. The approach structure is concrete T-beams supported on five column bents. The approach fill ramp and structure have an overall width of approximately 47 feet with a roadway width of 36 feet providing two eastbound and one westbound traffic lanes. At Bent 8, the 15th Avenue West overpass and the approach ramp structure from 15th Avenue West combine to form a single structure over the railroad. The structure is comprised of three separate steel thru-girders and has an overall width of approximately 72 feet. The distances between the centerlines of the south side and north side thru-girders to the center girder are 38'-10" and 23'-10", respectively. Each thru-girder has three spans and the interior supports are steel columns. The columns are skewed approximately 45 degrees to the bridge. The structure between the railroad crossing (Bent 12) and Bent 20 is supported on three- or four-column bents. The concrete slab structures are strengthened by steel truss retrofits that are supported directly on the columns.

At the thru-girder bridge over BNSF Railway, there are only three main longitudinal load carrying members on the structure over the railroad, so there are limited options for demolishing portions of the bridge for staged construction. One feasible option for demolishing the structure in stages while maintaining traffic is to remove either of the two exterior thru-girders. This would leave either a 47-foot wide structure on the south or a 25-foot wide structure to the north. The structure to the south is adequate for current loading. The structure to the north may need to be retrofitted to accommodate two lanes of traffic, since it was probably designed for a single lane of traffic.

Removing more than one thru-girder would be very complicated and would require temporary longitudinal support, retrofitting existing transverse members, and providing temporary vertical support. This work would need to be completed with minimal disruption to railroad traffic. Since it would be complicated and expensive to remove more than one thru-girder while maintaining traffic, it is not a recommended option for a staged construction option.

At the 15th Avenue West approach fill ramp, a temporary sheet pile wall could be installed to support the fill ramp so that a portion of the existing concrete cantilever wall could be removed for staged construction. A portion of the existing T-beam structure and five-column bent could also be removed if needed.

The structure between Bent 13 and Bent 20 could be removed along the center column line of the three-column bents depending on the preferred demolition for the railroad structure. If additional demolition is required, additional structure width could be removed and temporary supports could be provided at the bents, where necessary. A temporary barrier would be necessary between Bent 13 and Bent 20, but there is adequate room to accommodate a temporary barrier and allow for two lanes of traffic.

The options for staging the structure over the BNSF Railway is discussed in further detail in the following section; Staged Construction Options – Structure over BNSF Railway.

For a typical section of the thru-girder structure over the railroad showing the limits of demolition, refer to Figure 19.

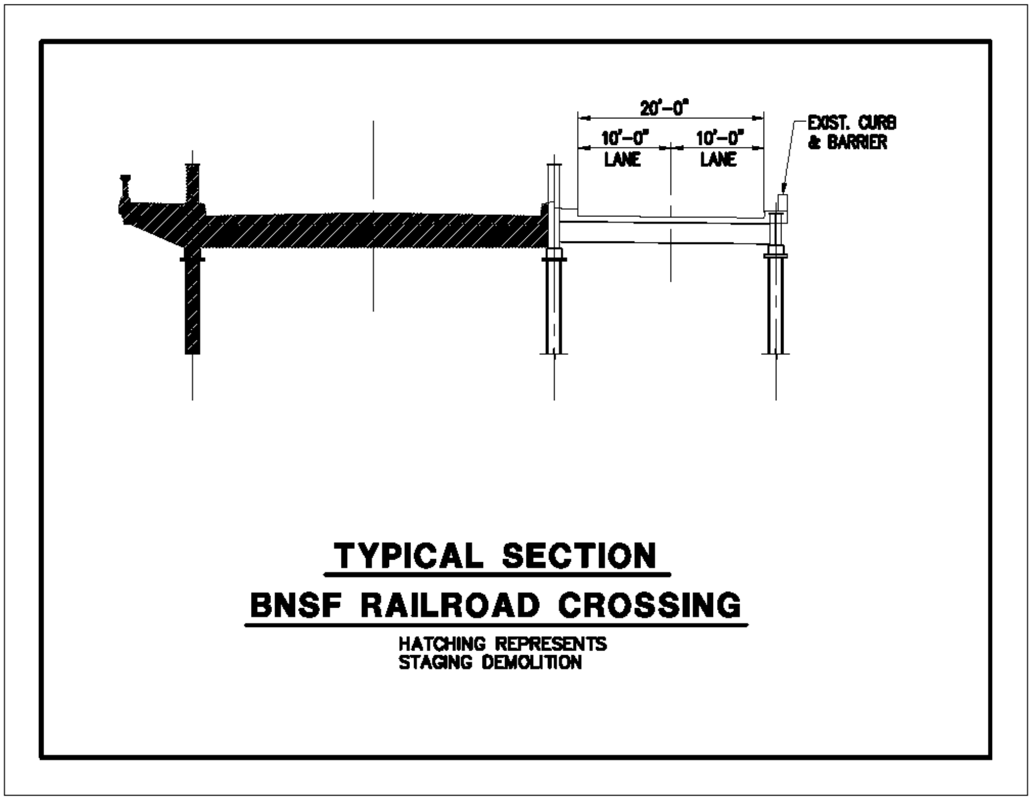


Figure 19
Typical Section – BNSF Railroad Crossing Hatching Represents Staging Demolition

7.1.3 Center Two-Way Ramp Structure

The center two-way ramp structure runs from existing Bent 20 to Bent 34. There are three separate structures supported on a four-column bent. The overall width of the three structures is approximately 75 feet. The two outside structures carry westbound and eastbound through traffic and the center ramp allows Port of Seattle Terminal 91 access for traffic to and from the east. The outside concrete slab structures are strengthened by steel truss retrofits that are supported directly on the columns. Truck traffic is prohibited from using this ramp and must exit through the Terminal East Gate and over West Galer Flyover to Alaskan Way West.

The westbound structure on the north side of the center ramp currently has an overall width of 26.5 feet with a roadway width of approximately 23 feet. Since there is adequate width on this structure to accommodate two lanes of traffic, it would be feasible to demolish the existing thru traffic ramp on the south side of the bridge along with the center ramp. Two-way traffic could be routed to the north side structure.

The structure between Bent 31 and Bent 34 could be removed along a line from the interior column on the westbound structure at Bent 31 to the center column at Bent 35. A temporary barrier would be necessary between Bent 31 and Bent 34, but there is adequate room to accommodate a temporary barrier and allow for two lanes of traffic.

For a typical section of the structure in the center ramp area showing the limits of demolition, refer to Figure 20.

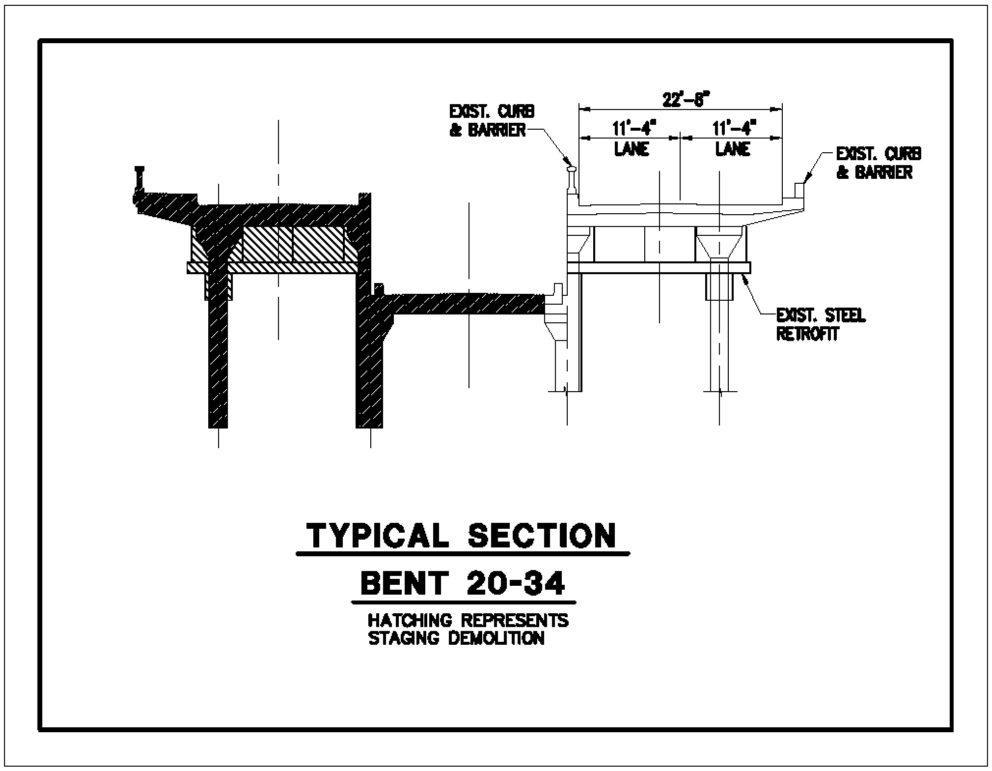


Figure 20
Typical Section – Bent 20-34 Hatching Represents Staging Demolition

7.1.4 23rd Avenue West Ramp Transition Structure

The 23rd Avenue West ramp transition runs from Bent 34 to Bent 46. The bridge has an overall width of 75 feet at Bent 34 and transitions to 95 feet between Bent 40 and Bent 44 to accommodate the on- and off-ramps to 23rd Avenue West. The structure is supported on three-column bents with the exception of Bent 46, which is a four-column bent. Bents 45 and 46 also lie at approximately an 11 degree skew to the bridge. As with the center ramp structure, the concrete slab spans are strengthened with steel truss and frame retrofits that are supported directly on the columns.

The structure between Bent 35 and Bent 41 could be demolished along the south edge of the center column. This would provide a temporary structure width of 37.5 feet. This would be adequate room for a temporary traffic barrier and two lanes of traffic.

The structure between Bent 41 and Bent 46 can be removed along a line from the center column at Bent 41 and the south interior column at Bent 46. This would provide enough width for two lanes of traffic and a temporary barrier while transitioning the temporary traffic lanes from the north side of the bridge to the structure west of Bent 46. The existing westbound off-ramp to 23rd Avenue West can also remain open during construction of the new bridge structure. Traffic currently using the eastbound on-ramp would be direct to a surface street detour along the west side of Terminal 91 to Thorndyke Avenue West via 21st Avenue West.

Refer to Figure 21 for a typical section of the structure in the 23rd Avenue West ramp transition area showing the limits of demolition.

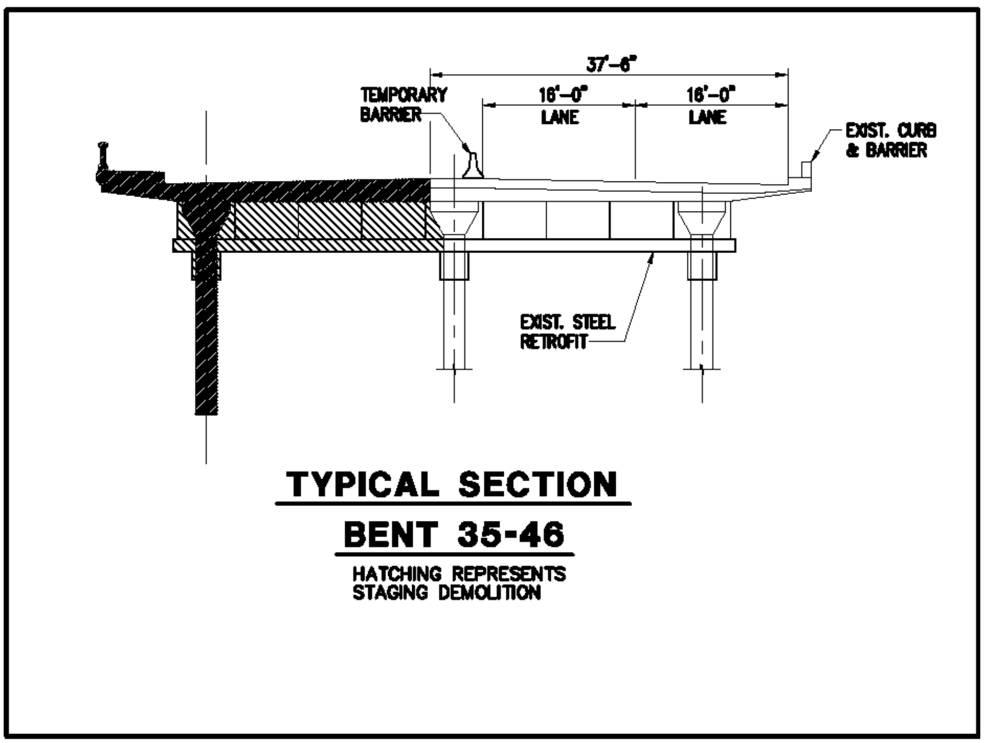


Figure 21
Typical Section – Bent 35-46 Hatching Represents Staging Demolition

7.1.5 Magnolia Bluff Structure West of the Ramp Transition

The superstructure in this section of the existing bridge is less than 50 feet wide and the bridge is supported on two-column bents with both longitudinal and transverse bracing. Between existing Bent 46 and Bent 61, the existing concrete slab is supported by steel truss retrofits which are tied to the columns. West of Bent 61, the superstructure is comprised of two concrete trusses supported directly on top of the columns. Since the bridge is only supported on two-column bents with transverse bracing, the structure demolition should be limited to only the south overhang extending beyond the face of column. Removing additional structure would be very complicated and would require temporary crossbeams, columns, bracing, and foundations. Since the profile grade is rising in this area and providing temporary substructure to support the superstructure will be expensive and time consuming, it is not recommended to remove additional structure beyond the overhang.

For a typical section of the structure west of Bent 46 showing the limits of demolition, refer to Figure 22.

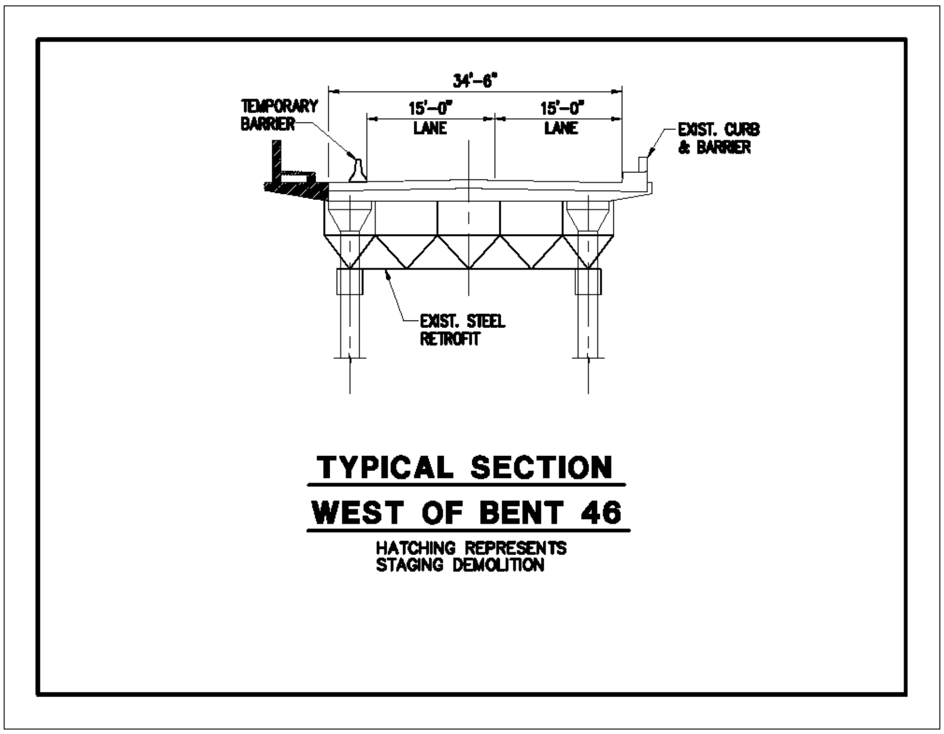


Figure 22
Typical Section – West of Bent 46 Hatching Represents Staging Demolition

7.2 STAGED CONSTRUCTION OPTIONS OVER BNSF RAILWAY

A critical area for staging the construction of the new Magnolia Bridge is over the BNSF Railway. All three alternatives lie on approximately the same alignment on top of the existing bridge alignment. At this location there is little flexibility in shifting the alignments due to existing building structures and major utilities to the north and south of the alignment. The other issue that complicates the construction of the new bridge in this section is that there is limited structure depth. Structure depth is limited by the maximum grade of the ramp between 15th Avenue West and by clearances required over the railroad.

A couple options are available for providing access over the railroad during construction of the new bridge. The options available for demolishing portions of the existing thru-girder bridge over the BNSF Railway have been presented above. In addition to the option of converting the existing 15th Avenue West overpass structure for temporary two-way traffic operation, there are a couple of other options available including: demolishing the existing 15th Avenue West overpass structure and rebuilding the ramp with adequate width to accommodate two lanes of traffic, providing access over the railroad via the existing West Galer Flyover and building a temporary ramp structure to access the remaining existing bridge structure to the west of the BNSF Railway, or using a surface street detour that would connect to 21st Avenue West and Thorndyke Avenue West after crossing the railroad using the West Galer Flyover.

7.2.1 Two-Way Traffic on 15th Avenue West Overpass Structure

This option provides access over the BNSF Railway via the 25-foot wide structure on the north side of the existing bridge. The southern half of the structure would be demolished to allow construction of the new bridge. The 15th Avenue West overpass structure would carry two-way traffic to the intersection with 15th Avenue West south of West Galer Street. If two-way traffic is provided on this structure, modifications would be necessary at the intersection with 15th Avenue West to allow two-way traffic to access West Galer Flyover and two-way traffic onto the 15th Avenue West. This would require additional phases at the traffic signal to separate the 15th Avenue West overpass and West Galer Flyover traffic flows creating lengthy delays.

This option would require staged construction of the new bridge over the railroad as the new alignment overlaps the existing bridge.

As was stated before, the primary disadvantage of this option is that the lane widths would be 10 feet wide and this is not desirable from an operational standpoint. It may be possible to demolish some of the existing curb to provide some additional lane width. Vehicle length and weight restrictions may be necessary.

7.2.2 15th Avenue West Overpass Rebuild

This option includes demolishing the existing 15th Avenue West overpass to allow construction of the new bridge. Access over the BNSF Railway would be via a 47-foot wide structure on the south side of the existing bridge. The northern section of the structure over the railroad would be demolished to provide for construction of the new bridge. Traffic would be staged to use the south side ramps to access 15th Avenue West while the 15th Avenue West overpass is constructed, and switched to the 15th Avenue West overpass while the south side ramps are constructed. Both traffic detours would require modifications to the intersections on 15th Avenue West to allow two-way traffic onto the Magnolia Bridge.

This option would also require staged construction of the new bridge over the railroad because the new alignment overlaps the existing bridge.

The current Alternative A design concept for the 15th Avenue West overpass structure has a width of 23.6 feet with a roadway width of 21 feet. As with the existing ramp structure, the lane widths with two-way operation would only be 10.5 feet and this is not desirable from an operational standpoint. Therefore, it may be necessary to provide additional width for the ramp structure to allow its use for two-way traffic during bridge construction.

If two-way traffic is provided on this structure, modifications would be necessary at the intersection with 15th Avenue West to allow two-way traffic to access West Galer Flyover and two-way traffic onto the 15th Avenue West. This would require additional phases at the traffic signal to separate the 15th Avenue West overpass and West Galer Flyover traffic flows creating lengthy delays.

7.2.3 Temporary Access Ramp

For this option, access over the railroad would be provided by the existing West Galer Flyover. Traffic would then be routed along Alaskan Way West which parallels the BNSF Railway tracks. A temporary access ramp would either connect to the existing bridge on the north side or to the new bridge on the south side. For the north side connection, a temporary roadway would cross under the existing bridge between existing Bent 14 and Bent 16. Access to the existing bridge west of the railroad would be provided by a temporary ramp built parallel to the bridge in the area formerly occupied by the tank farm (demolished in summer 2005). The access ramp would tie into the existing bridge structure near Bent 30. For the south side connection, a temporary roadway would be constructed across port property and Jacobs Lake and connect to the new bridge at Pier 11.

This option would not require staged construction of the new bridge over the railroad. The existing bridge east of Bent 30 would be closed to traffic and the existing bridge would be demolished to allow for construction of the new bridge.

The primary advantage with this option is that the structure over the railroad could be built in a single stage. This would be less complicated and would likely be more economical than two-stage construction.

The primary disadvantage with this option is the detour route would be circuitous and the West Galer Flyover structure, which is the primary access to the expanding Amgen campus and Port of Seattle Pier 90 and 91, would see an increase in traffic congestion.

7.2.4 Surface Street Detour

Another detour option is a surface detour route that would provide access to the Magnolia Bluff via an access road through the Port of Seattle property. The route would follow a similar path as the temporary access ramp option, but will continue through the Port of Seattle property along the BNSF right-of-way, and access Magnolia Bluff via 21st Avenue West and Thorndyke Avenue West.

7.3 ALIGNMENT A ALTERNATIVES

7.3.1 A-South Alignment

This is the original Alternative A alignment developed in the Alignment Study. This alignment has the bridge structure located far enough to the south of the existing bridge to construct most of the mainline structure west of the BNSF Railway crossing without having to build the structure in stages. For a plan view of the A-South Alignment refer to Figure 23.

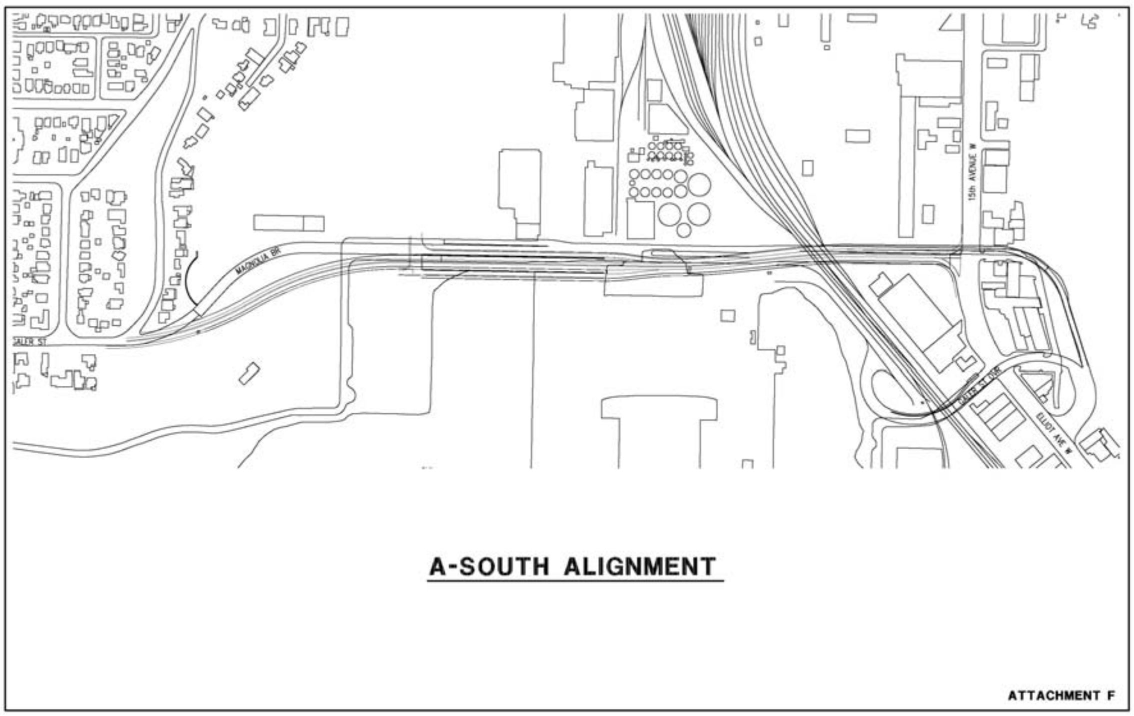


Figure 23
A-South Alignment

As was stated previously, the approach ramp from 15th Avenue West and the 15th Avenue West overpass both lie directly on top of the existing alignment. As the bridge crosses the BNSF Railway, the alignment would begin to move south away from the existing bridge alignment. At approximately existing Bent 26, the alignment would straighten out and continue parallel to the existing bridge alignment. The new on- and off-ramps accessing 23rd Avenue West would have a flatter profile grade than the existing ramps. Since the ramp length would be longer, portions of the existing bridge would need to be removed to accommodate full width construction of the mainline bridge structure. The eastbound on-ramp from 23rd Avenue West would be located within the tidelands in Smith Cove. Also, more than half the width of the mainline structure would be located over the existing pier and tidelands at the northern end of Smith Cove. The off-ramp to 23rd Avenue West would lie on the existing bridge alignment and would be constructed after the existing bridge is demolished. The mainline structure west of existing Bent 46 would be located entirely south of the existing mainline bridge, with the exception of the west abutment which would be located near the existing bridge abutment.

The advantages of A-South alignment include:

- Shortest bridge detour time because a portion of the new parallel bridge would be constructed before closing the existing bridge.
- Large portion of the mainline bridge could be constructed parallel to the existing bridge while maintaining traffic during construction.
- More flexibility in staging the demolition of the existing bridge to facilitate construction of the new bridge.
- The possibility exists that the bridge construction could be staged in a way that will allow access to Magnolia via either the existing bridge or new bridge with a minimal closure time.
- Port east-west traffic circulation could be provided for on the north side of the bridge in front of the existing buildings.

The disadvantages of this alignment include:

- Right-of-way and easements would need to be purchased and negotiated since the new alignment would be located south of the existing bridge alignment.
- This alignment would have impact on the tidelands since the on-ramp from 23rd Avenue West and the mainline structure would be constructed over tidelands west of Pier 91.
- Ground improvements may need to be completed in two stages because the new bridge would be located adjacent to the existing bridge.

7.3.2 A-North Alignment

This alignment has the bridge located directly as close as possible to the existing bridge alignment. For a plan view of the A-North Alignment refer to Figure 24. As with the A-South alignment, the structures east of the BNSF Railway crossing would be on the same alignment as the existing bridge. The structure over the BNSF Railway and west to the 23rd Avenue West on- and off-ramp connection would follow the existing bridge alignment, although the alignment would move to the south slightly at the ramp connection. The alignment would need to be shifted to the south to accommodate the off-ramp, which cannot be moved north of the current off-ramp structure because of the proximity to Building 50 (CityIce cold storage). As a result of the increased ramp widths and increased mainline structure width, the south edge of the new mainline structure would be located approximately 18 feet south of the existing bridge. The on-ramp from 23rd Avenue West would be located over the existing pier at the northern end of Smith Cove. The off-ramp to 23rd Avenue lies on the existing bridge alignment and would be constructed after the existing bridge is demolished. The mainline structure west of existing Bent 46 would be located approximately 18 feet south of the existing mainline bridge until existing Bent 66, where the alignment would be south of the existing bridge. The west abutment would be located near the existing bridge abutment.

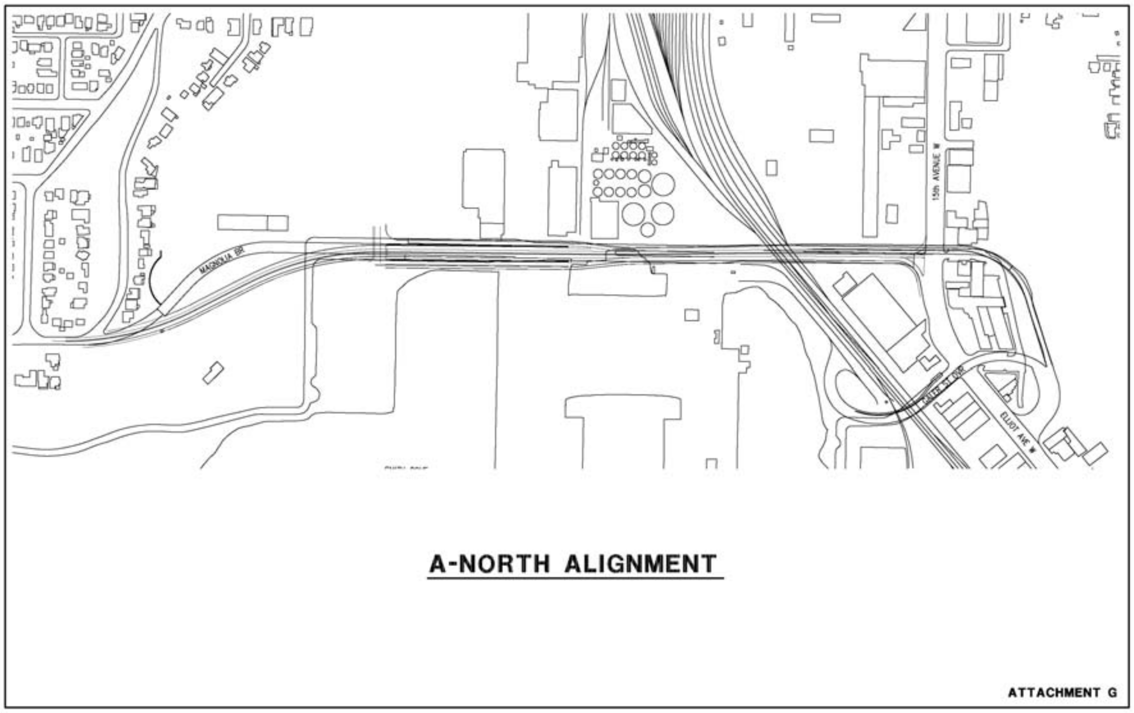


Figure 24
A-North Alignment

The advantages of A-North alignment include:

- Limited right-of-way/easement requirements since new alignment would be located on the existing bridge alignment.
- Limited contractor complications/disruptions during construction and reduced cost of the bridge structure since traffic would be detoured around construction site on a surface route.
- Complete all ground improvements during initial construction of the foundations because the existing bridge would be closed and demolished.

The disadvantages of this alignment include:

- Longer detour period to construct bridge because most of the alignment lies directly on the top of existing structure and the existing bridge would need to be demolished and traffic detoured before construction on the new bridge can begin.
- The alignment would have to be shifted 18 feet to the south to accommodate the off-ramp to 23rd Avenue. This is due to proximity of Building 50 (CityIce cold storage) to the new ramp.
- Difficult staged construction of the new bridge to get around portions of the closure. This would increase the cost of construction.

- This alternative would have impact on tidelands since the eastbound on-ramp from 23rd Avenue West and the south half of the mainline structure would be over tidelands west of Pier 91.

7.3.3 A–Hybrid Alignment

For this alignment, the A-North alignment would be shifted to the south far enough to accommodate a two-stage construction sequence for the mainline bridge west of existing Bent 46. This would maintain traffic flow on the existing bridge structure during the construction of a portion of the new structure. After completion of half the new bridge the traffic could be switched to the new structure and the existing bridge could be demolished and the remaining half of the structure built in its place.

The A-North alignment would need to be shifted at least 10 feet to the south to accommodate two, 11-foot traffic lanes, a temporary barrier, and a shy distance behind the barrier. If the project requires maintaining pedestrian traffic on the structure during construction, then the alignment would need to be shifted further south to accommodate a sidewalk.

The advantages of A-Hybrid alignment include:

- Less right-of-way/easement required than A South because the new alignment would be located just south of the existing bridge alignment.
- Reduced tideland impact over A-South since more of the structure would be located outside of tidelands.
- The bridge could remain open during the construction of most of the bridge structure since enough structure width would be built in each stage to accommodate two-way traffic.

The disadvantages of this alignment include:

- Expensive to demolish and construct the bridge in multiple stages requiring re-mobilization of equipment for staged demolition of existing bridge and staged construction of the new bridge.
- Right-of-way/easement would need to be negotiated because new alignment would be located south of the existing bridge alignment.
- This alignment would impact the tidelands since the on-ramp from 23rd Avenue and a portion of the mainline structure would be located over the tidelands west of Pier 91.
- Ground improvements would need to be completed in two stages because the new bridge would be located adjacent to the existing bridge.
- Construction duration would be much longer because the bridge would be built in multiple stages.
- Alternative would only be about 5-foot shift from A-South alignment due to geometric constraints.

7.4 EXISTING BULKHEAD WALLS

A study on the existing bulkhead walls was performed to determine if the bulkhead walls affect the selection of the alignment.

There are currently three separate bulkhead walls running along the length of the existing Magnolia Bridge. These include: the Smith Cove Bulkhead Wall, which bounds the north and east side of Smith Cove, Jacob's Lake Bulkhead Wall, which bounds the north, east and west side of the lake, and the Pier 90 North Bulkhead Wall, which bounds the north edge of the inlet east of Pier 90. Of these three bulkhead walls, only the Smith Cove Wall will remain after completion of the new Magnolia Bridge. The Jacob's Lake Wall will be abandoned after the lake is drained and backfilled with soil. A large portion of this bulkhead wall was already abandoned when the area south of the lake was backfilled with soil to create a staging area and thru way between Piers 90 and 91. The Pier 90 North Wall is already abandoned and is likely buried below the surface of the ground.

Even though two of the three walls will be abandoned prior to completion of the new bridge, consideration needs to be given to all of the walls since the new bridge alignment will lie directly above the walls and thus construction of the foundations may be impacted as a result of the existing walls.

For a plan view of the location of the bulkhead walls relative to the existing bridge, refer to bridge plan drawings in Appendix H. The location of the bulkhead wall was based on the original 1929 bridge plans.

7.4.1 Smith Cove Bulkhead Wall

This wall bounds the northern edge of Smith Cove and the west side of Pier 91. The section of wall bounding the west side of Pier 91 runs perpendicular to the existing bridge at Bent 44. Approximately 20 feet from the south edge of the existing bridge the bulkhead angles 45 degrees to the northwest. The wall continues at this bearing until it hits the existing south column line approximately 10 feet west of Bent 45. At this point the wall continues west paralleling the south column line to Bent 55 at which point the wall angles to the north and then northeast for a distance of approximately 70 feet.

According to the note on sheet 86 of the original 1929 bridge plans, the section of wall between Bent 45 and 55 is integral with the existing bulkhead wall: "Where the bulkhead has to be cut for the construction of the footings the contractor must provide anchor rods and anchors of sufficient section, as decided by the City Engineer, to hold this bulkhead in place. The bulkhead shall be neatly butted against, or carried by the new piers." Since the bridge footings are integral with the bulkhead wall, it may not be feasible to remove any of the existing foundations during construction of the new bridge.

The original bridge plans from 1929 indicate this wall is timber construction.

7.4.2 Jacob's Lake Bulkhead Wall

This wall originally bounded the eastern edge of Pier 91, the western edge of Pier 90, and the northern edge of the inlet between the piers. A section of the inlet between the piers was filled with soil at a later date creating Jacob's Lake. This lake is bounded on the north, east and west side by the original bulkhead wall.

The section of wall bounding the east side of the lake runs perpendicular to the existing bridge near Bent 22. Approximately 25 feet from the south edge of the existing bridge the bulkhead wall turns 90 degrees and heads west. This section of wall parallels the existing bridge until it is 10 feet west of Bent 26 where it heads north until it hits the south interior column line. At this point the wall continues west paralleling the south interior column line until 10 feet east of Bent 36 at which point the wall angles 45 degrees to the southwest until Bent 37. Approximately 25 feet from the south edge of the existing bridge the bulkhead wall runs perpendicular to the existing bridge.

As with the Smith Cove Bulkhead Wall, the section of wall between Bent 27 and 35 is integral with the existing bulkhead wall. Since this wall will be abandoned before completion of the project there are no issues with removing existing foundations or sections of the bulkhead wall to allow for construction of the new bridge foundations other than the bulkhead wall stability during construction and demolition.

The original bridge plans from 1929 indicate this wall is timber construction, with the exception of the section of wall south of the bridge running from Bent 22 to Bent 26 which is a sheet piling wall.

7.4.3 Pier 90 North Bulkhead Wall

This wall originally bounded the eastern edge of Pier 90 and the northern edge of the inlet east of Pier 90. The northern section of the inlet was backfilled with soil to provide a thruway between the south edge of the existing bridge and the inlet to allow access to the Port of Seattle property.

The section of wall bounding the east side of Pier 90 runs perpendicular to the existing bridge between Bent 17 and 18. Approximately 30 feet from the south edge of the existing bridge the bulkhead wall turns 90 degrees and parallels the existing bridge alignment until it hits the east side of the BNSF tracks where it angles 45 degrees to the southeast.

The original bridge plans from 1929 indicate this wall is a sheet piling wall.

For the most part this wall lies outside of the new Magnolia Bridge Alignments so this wall will not be impacted during the construction of the foundations for the new bridge.

7.4.4 Bulkhead Versus A-South Alignment

For this alignment, the existing Smith Cove bulkhead wall lies approximately 40 to 45 feet north of the mainline bridge alignment. This is near the centerline of the westbound off-ramp bridge. Since this structure will be supported on a single column bent located at the centerline of the structure, the existing bulkhead wall will be in conflict with the foundation. There is flexibility to shift the westbound off-ramp alignment to the north to avoid conflicts with the existing bulkhead.

If the westbound off-ramp alignment is shifted north to avoid the existing bulkhead wall, then the foundations for the mainline bridge and the eastbound on-ramp will be located within the limits of Smith Cove, as it is defined by the existing bulkhead wall. The foundations for the westbound off-ramp will be located outside the limits of Smith Cove.

For a typical section of the A-South Alignment with the westbound off-ramp shifted and showing the location of the existing bulkhead wall, refer to Figure 25.

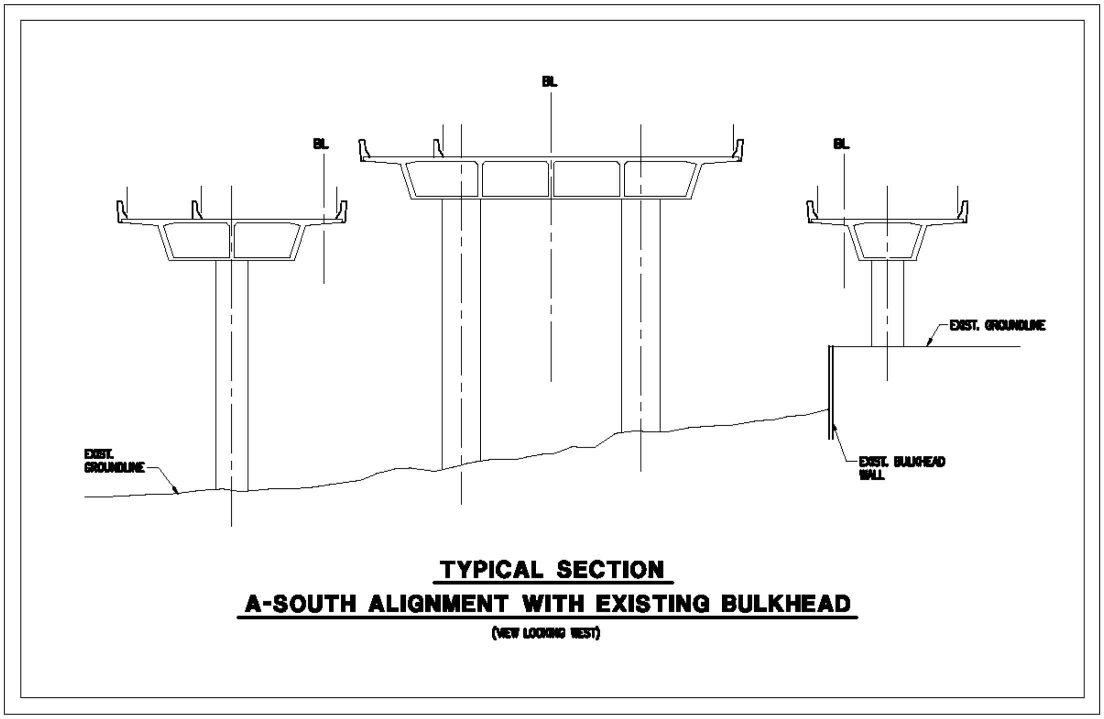


Figure 25
Typical Section – A-South Alignment with Existing Bulkhead

7.4.5 Bulkhead Versus A-North Alignment

For this alignment, the existing Smith Cove bulkhead wall follows the mainline bridge alignment plus or minus a few feet in either direction. If the mainline bridge structure is supported on a two column bent, then the eastbound on-ramp and half the mainline bridge foundations will be located within the limits of Smith Cove, as it is defined by the existing bulkhead wall. The westbound off-ramp and half the mainline bridge foundations will be located outside the limits of Smith Cove. If a single column bent is preferred for the mainline bridge structure, then there will be conflicts between the existing bulkhead wall and new bridge foundations.

For a typical section of the A-North Alignment showing the location of the existing bulkhead wall, refer to Figure 26.

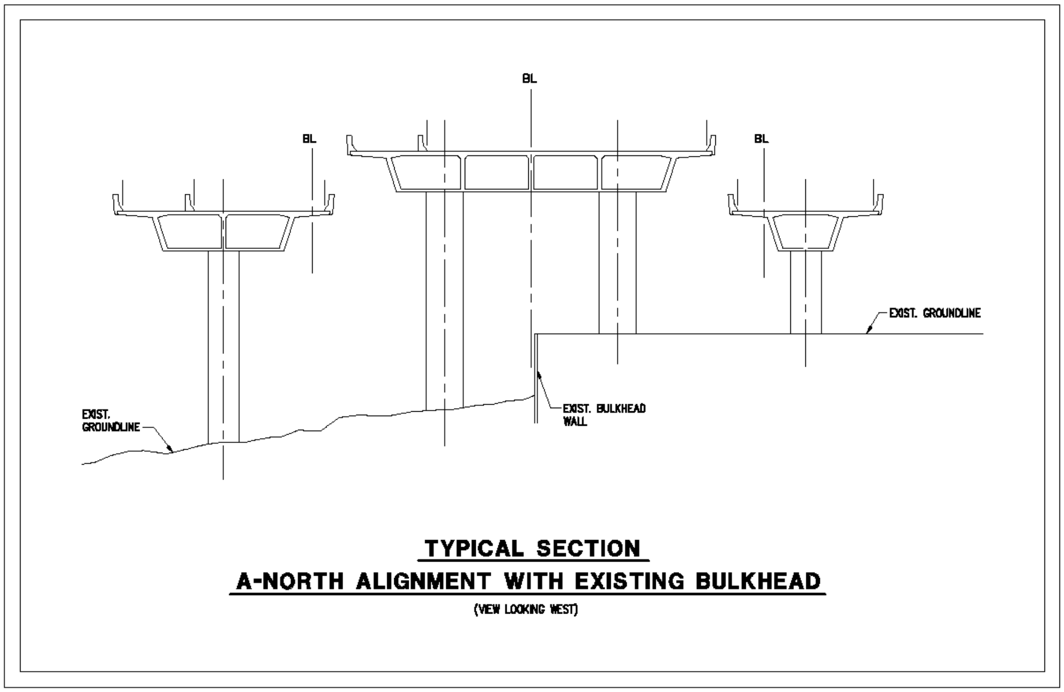


Figure 26
Typical Section – A-North Alignment with Existing Bulkhead

7.5 ALIGNMENT A STUDY CONCLUSIONS

Based on the findings in this study, the A-South Alignment is recommended for the location of the new Magnolia Bridge. This alignment will have the shortest construction detour time because the new bridge will be located far enough to the south of the existing bridge to construct most of the mainline structure west of the BNSF Railway without having to close the existing bridge. By locating the alignment to the south of the existing bridge there are more options available for demolishing portions of the existing bridge while still maintaining traffic on the structure. Staged construction of the new bridge will be limited, since most of the full width of the mainline structure can be built parallel to the existing bridge. As a result, the construction cost for this alignment should be the least expensive of the three alternatives.

8. BRIDGE CONCEPT STUDY

This study investigates conceptual structure types on Alignment A and evaluates them in regards to cost, aesthetics, ease of construction, total construction duration, maintenance/life cycle costs, design performance, environmental permitting, impact on Port of Seattle and BNSF Railway operations, and user impacts. The superstructure type, pier type, wall finish, railing style, and luminaire style are evaluated in this study. The superstructure type and pier type are evaluated at multiple locations, because there is variability in structure type requirements along the length of the bridge. The wall finish, railing style, and luminaire style are considered common components to all sections of the bridge and are evaluated globally for the entire project. The decision on type of wall finish, railing style, and luminaire style is independent from the evaluation of the superstructure and pier type. The structure can be designed to accommodate any railing and luminaire style evaluated for the project.

During concept development, Bridge Engineers, Architects, and Urban Planners worked together to conduct architectural/structural studies on bridge aesthetics, span length, pier configuration, and superstructure type. Structure types and configurations were investigated during concept development to determine the most feasible structure type and layouts for the project. Black-line sketches were created for the span length study, pier configuration study, and the superstructure type study to assist in selecting bridge type and layout. A qualitative matrix was used to screen the concepts to select up to three alternatives that best meet the project goals and objectives.

For ease of evaluating structure alternatives during the bridge concept study, the bridge has been divided into three distinct sections; the Low Level Bridge, the BNSF Railroad Overpass Bridge, and the Magnolia Bluff Bridge, see Figure 27. The low level bridge structure includes the bridge structures east of the BNSF Railroad and the bridge structure between the BNSF Railroad crossing and 23rd Avenue Ramp Transition, including the on and off-ramp structures to 23rd Avenue. The BNSF Railroad Overpass Bridge includes the structure associated with the BNSF railroad crossing. The Magnolia Bluff Bridge includes the section of Magnolia Bridge from the 23rd Avenue Ramp connection to the abutment at the Magnolia bluff.

For each section of the bridge, the feasible superstructure types were evaluated based on the criteria presented above. The initial evaluation process investigated multiple structure types for the project. The feasible structure types were included in an evaluation matrix that provides the advantages and disadvantages of those structure types based on the evaluation criteria. The selected superstructure types for each section of the bridge are noted in the evaluation matrix. This selection is based on the advantages and disadvantages of each structure type and in some cases the determination of fatal flaws in the alternatives.

In selecting the preferred superstructure type and pier type, consideration was given to the site conditions and the type of foundations recommended for the project. Most of Alignment A is underlain by fill and soft and loose Holocene deposits. These normally consolidated, non-glacial deposits range from about 30 to 115 feet thick along Alignment A. The deepest layer of Holocene deposits is located between 15th Avenue West and 23rd Avenue. Because of the loose/soft soils that are present, shallow foundations are not suitable for support of the proposed bridge replacement. Deep foundations that extend into the underlying dense and hard glacial deposits are recommended for supporting the proposed bridge. The foundations are expected to be either drilled shaft or large diameter, open-ended, steel shell piles.

The soft and loose Holocene deposits are susceptible to liquefaction and lateral spreading. A non-liquefied crust lying above the Holocene deposits would likely subject large lateral forces to the foundations during a seismic event. As a result, ground improvements would be necessary in the Holocene deposits to mitigate liquefaction and lateral spreading concerns. The ground improvements would primarily be located between 15th Avenue West and the end of the ramp structures at 23rd Avenue. Ground improvement depth would likely vary between 20 feet, near 23rd Avenue, to 50 feet, near the west end of the BNSF Railroad Crossing, based on the current borings. The geotechnical consultant, Shannon & Wilson, has recommended either stone columns or compaction grouting as feasible options for improving the soil properties. Ground improvements would be required at each pier location, with the improvements extending approximately 40 feet outside of the footing plan for stone columns and 30 feet outside of the footing plan for compaction grouting.

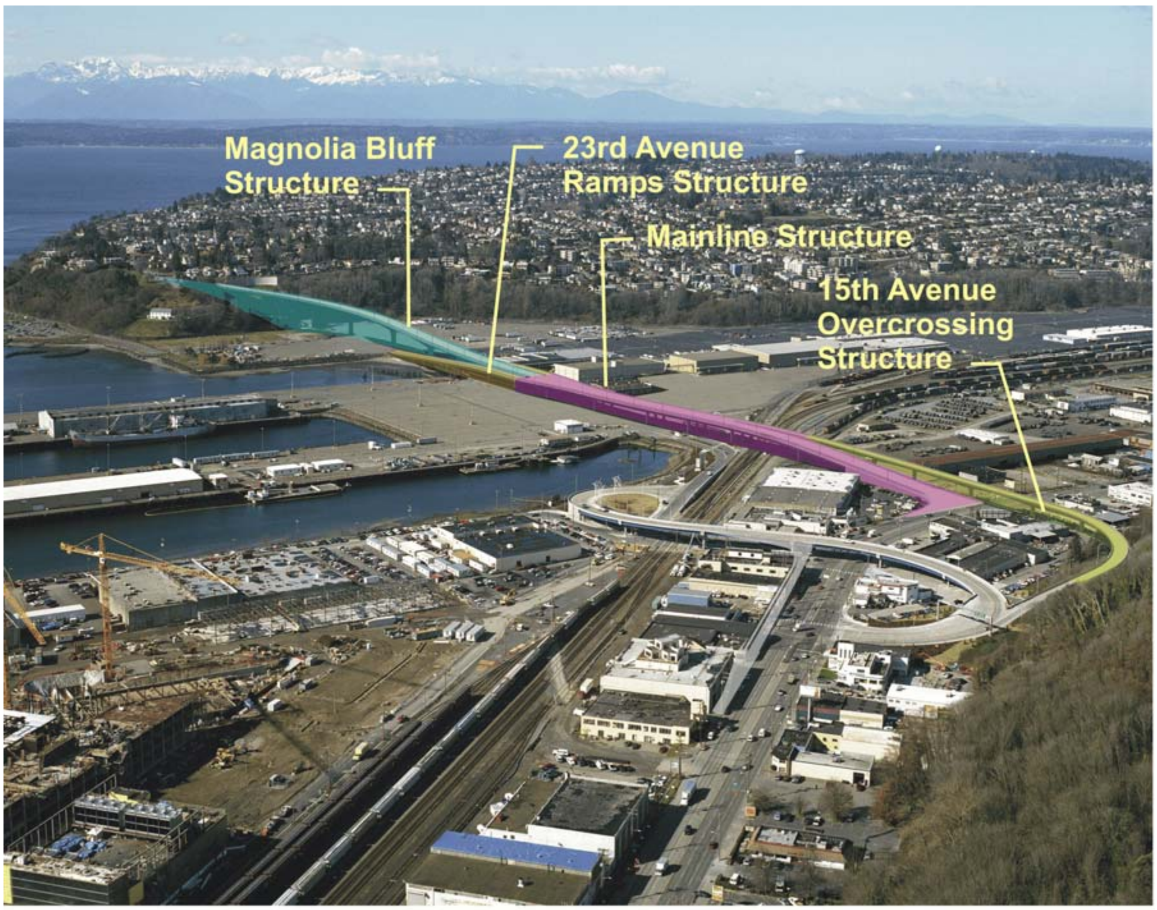


Figure 27
Bridge Structure Sections

8.1 LOW-LEVEL STRUCTURE

The Low Level Bridge includes the 15th Avenue Overpass Ramp, the 15th Avenue Approach Ramp, the Mainline Bridge between the BNSF Railroad and the 23rd Avenue Ramp Transition over the Port of Seattle facilities, the Westbound Off-Ramp to 23rd Avenue and the Eastbound On-Ramp from 23rd Avenue.

The structure type alternatives for the Low Level Bridge Structure are evaluated under the assumption that the same structure type will be used for all these structures. A common structure type is recommended to provide a consistent aesthetic appearance throughout the length of the Low Level Bridge. A common structure type will also reduce the cost of construction because of reduced mobilization costs, standardized equipment, falsework, and formwork and standardized labor requirements for all the segments of the Low Level Bridge.

Several feasible structure types were evaluated for the Low Level Bridge. These included: pre-stressed concrete girders, concrete tub girders, cast-in-place concrete box girders, and steel plate girders, steel box girders.

Above-deck structure alternatives, such as an arch, cable stayed or truss, were not investigated for the Low Level Bridge Structure. The site conditions in the low level bridge do not justify a signature long span structure with reduced structure depth and increased cost. Three of the five structures are one lane ramp structures that are sloping into the ground. These structures are also located on curved alignments and included large width transitions which are difficult to accommodate with an overhead structure.

8.1.1 WSDOT Standard Precast Prestressed Concrete Girders

Precast prestressed concrete girder bridges have been used extensively in Washington State for modern highway bridge structures. Prestressed girders offer simple to construct and cost effective options for new bridge construction. These bridge structures require minimal maintenance over the life of the structure. Prestressed girders can also be erected without having to install temporary shoring or falsework. This is a major advantage in areas where traffic flow needs to be maintained during construction or in environmentally sensitive areas where minimal disruption to surrounding area is necessary. Since the girders can be erected without temporary shoring, the construction time for superstructure is less than cast-in-place structures.

The primary disadvantage with precast girders is the span length limitation for straight girders on curved structures. Because the girders are chorded members the span length has to be limited on structures with tight radius alignments. Large width transitions can also be complicated and more expensive since additional girders would be required to accommodate the girder flare.

Aesthetically, there are several different prestressed girder sections available that will modify the appearance of the bridge. A straight prestressed girder bridge on a curve does not have a clean appearance from below, because the supporting girders will be straight while the deck is curving. Width transitions will look cluttered from below, because additional girders will need to be added to accommodate the increased structure width.

Typically, prestressed girders will have an integral cast-in-place concrete pier crossbeam that will be deep enough to support the dead load of the superstructure. As a result, the crossbeam cap will sit lower than the superstructure. It is possible to have the bottom of the crossbeam flush with the girders, but this will increase the cost as temporary supports will be required to support the girders until the crossbeam and deck are place. Expansion joints would have to be located over piers for prestressed girder bridges. This will require a drop down pier cap at the expansion joints. This cap will likely have a different appearance than the integral caps used on the intermediate piers.

There are several different types of precast prestressed concrete girder options available for new bridge construction. For this project, Washington State Department of Transportation (WSDOT) standard precast prestressed girders were evaluated, because these are readily available from local fabricators. The WSDOT standard precast girders can span up to 180 feet, with the span length controlled by the maximum shipping weight of 200 kips. The WSDOT standard precast prestressed concrete girder options available include:

- Concrete I-Girders with cast-in-place deck slab
- Concrete Tub Girders with cast-in-place deck slab
- Concrete Bulb Tee Girders with cast-in-place slab
- Concrete Decked Bulb Tee Girders with asphalt wearing surface
- Concrete Flat Slabs with cast-in-place concrete wearing surface
- Concrete Ribbed or Double-Tee Girders with asphalt wearing surface

The prestressed concrete I-Girders and prestressed concrete Tub Girders with cast-in-place concrete decks are the preferred type of superstructure for WSDOT prestressed concrete bridges.

WSDOT has limited the use of Decked Bulb Tee, Ribbed, and Double-Tee to state routes with Average Daily Traffic (ADT) of 30,000 vehicles or less. This limitation is to reduce the potential for longitudinal cracking in the overlay and to improve the durability of the joints between girders. Because there are maintenance concerns with these types of superstructure for bridges with high ADT volumes, the decision was made to remove these from the list of potential precast superstructure options. Removing these superstructure types from the evaluation is not a major issue, because the precast I-Girders and Tub Girders have similar span capability.

The precast Flat Slab option was determined to be an undesirable structure type since the span capability of this superstructure is limited to 80 feet. The foundation cost is expected to be high at the Magnolia Bridge site because competent bearing soil is located over 100 feet below existing ground surface and ground improvement will be required at each pier location. As a result, the flat slab option will cost significantly more than other precast alternatives since more foundations and more ground improvement will be required.

The preferred options for a precast prestressed girder superstructure will include WSDOT standard prestressed concrete I-Girders and Tub Girders with a cast-in-place concrete deck. Both of these superstructure options can also be post-tensioned to increase the span capability of the girders.

8.1.1.1 Prestressed Concrete I-Girders

WSDOT standard precast concrete I-Girders are available in several shapes and sizes, including: W95G, W83G, WF74G, WF58G, WF50G, WF42G, W74G, W58G, W50G and W42G. The number specifies the girder depth in inches and the WF denotes that the girder has a wide top and bottom flange. The girders with wide flanges have increased span capability with the same girder depth. The following table provides the approximate span range for each available I-Girder section, based on the tables in the WSDOT Bridge Design Manual (BDM).

Table 13
WSDOT Standard Precast Concrete I-Girder Span Capability

Girder Type	Span Range (Feet)
W95G	155 - 165
W83G	145 - 180
WF74G	130 - 175
WF58G	105 - 145
WF50G	95 - 130
WF42G	80 - 115
W74G	110 - 150
W58G	90 - 125
W50G	65 - 110
W42G	45 - 85

The span capability varies depending on girder spacing. The girder spacing provided in the WSDOT BDM varies between 5 feet and 12 feet. The shorter span length will correspond to 12-ft. girder spacing and the longer span length will correspond to 5-ft. girder spacing. The span range is limited for the W83G and W95G bcause the maximum span length is controlled by the maximum shipping weight of 200 kips.

As was mentioned previously, the foundation costs are expected to be high for the new bridge structure since competent bearing soil is over 100 feet below existing ground surface and ground improvements are required at each pier location. As a result, it is recommended, where feasible, to maximize the span length for the prestressed girder superstructure. The additional cost of the superstructure will be offset by the reduced cost in the foundations and ground improvements at each foundation. The optimum girder type and span arrangement will be determined during the Bridge Alternative Development phase of the project.

8.1.1.2 Post-Tensioned Concrete I-Girders

To further maximize the span capability of the precast concrete I-Girders, the WF74G, W83G and W95G girder sections can be precast in multiple segments and post-tensioned together in the field. Because the span capability of these girders was controlled by the maximum shipping weight, casting and shipping the girders in sections and post-tensioning the individual sections in the field allows for a longer span length. The span capability of the post-tensioned I-Girders is dependent on whether the post-tensioning is applied before or after concrete deck placement. The span length is maximized if the post-tensioning is applied after the concrete deck placement. The following table provides the approximate span range for each available post-tensioned I-Girder section, based on the tables in the WSDOT Bridge Design Manual (BDM) for post-tensioning after deck placement.

Table 14
WSDOT Standard Post-Tensioned Concrete I-Girder Span Capability

Girder Type	Span Range (Feet)
WF74PTG	155 - 195
W83PTG	175 - 205
W95PTG	190 - 235

The span capability varies depending on girder spacing. The girder spacing provided in the WSDOT BDM varies between 6 feet and 14 feet. The shorter span length will correspond to 14-foot girder spacing and the longer span length will correspond to 6-foot girder spacing.

This option will be more expensive than traditional precast concrete I-Girders, because the girder segments will need to be joined with a cast-in-place closure joint and post-tensioned in the field. Temporary falsework will also be required to support the girders in the final position until the precast segments have been post-tensioned.

8.1.1.3 Concrete Tub Girders

WSDOT standard precast concrete Tub Girders are available in several different depths and widths. According to the WSDOT BDM, these girders are currently available in sizes ranging from 54 inches to 78 inches in depth and having bottom flange widths ranging from 4 feet to 6 feet. The span capability varies depending on the size and spacing of the Tub Girders. The maximum span capability of these girders is 145 feet for a 66 inch deep girder section and is controlled by the maximum shipping weight of 200 kips.

Concrete Tub Girders are going to cost approximately twice as much as an equivalent I-Girder section, but fewer girders are needed because of the girder width. Tub Girders also offer a completely different aesthetic look for the bridge as compared to standard I-Girders.

Concrete Tub Girders are not recommended for a variable width structure, because the width of the tub girders makes it difficult to add additional girders to accommodate the structure flare.

8.1.1.4 Post-Tensioned Concrete Tub Girders

Because the span capability is limited due to the maximum shipping weight, these girders are more efficient if used as post-tensioned girders. The Tub Girders can be cast in multiple segments and post-tensioned in the field to allow for longer span length. By post-tensioning the girders in the field, the maximum span length can be increased up to approximately 195 feet for a 78 inch deep girder.

As with the post-tensioned concrete I-Girders, the cost for post-tensioned concrete Tub Girders will more expensive than traditional concrete Tub girders. Temporary falsework will be required to support the girders in the final position until the segments are joined with a closure pour and post-tensioned.

8.1.1.5 Recommendation WSDOT Standard Precast Prestressed Concrete Girders

Because the foundation costs are expected to be high, the span lengths should be maximized wherever possible, to reduce the total number of foundations. The added cost of building longer spans will be offset by the reduction in the number of foundations and the associated ground improvements. The optimum girder type and span arrangement will be determined during the Structure Alternative Development phase of the project.

It is recommended to advance several prestressed concrete girders to determine the optimum girder for this project. For this study, the WSDOT WF74G and W83G concrete I-Girders were evaluated because the current profile limits the structure depth to 8 feet for the structure in between the BNSF Railroad and the ramp transition to 23rd Avenue and 7 feet for the 15th Avenue Overpass structure. The WF74G is feasible at both locations since the structure depth does not exceed 7 feet. The typical layout is WF74G girders at 7-ft spacing with a maximum span length of 160 feet.

It is also recommended to advance the WF74PTG concrete post-tensioned I-Girder for the same reasons the WF74G concrete I-Girder was evaluated. The typical WF74PTG girder evaluation was based on an 8-ft girder spacing with a maximum span length of 185 feet.

The standard precast concrete Tub Girder is not recommended because the span length is limited to only 145 feet. Because this superstructure type is more expensive than concrete I-Girders and the foundation costs are high, it is more feasible to evaluate this girder with post-tensioning, where the span length will be similar to the concrete I-Girder. A 78 inch deep post-tensioned Tub Girder section with a 5-ft bottom flange width is recommended for this study. The U78PTG5 tub girder evaluation was based on 10-ft girder spacing with a maximum span length of 180 feet.

For a typical section of a prestressed girder bridge in the low-level section of the bridge, refer to Figure 28.

8.1.2 Cast-in-Place Concrete Box Girder

Cast-in-Place Concrete Box Girders have been used throughout Washington on various bridge projects and interchange structures. This structure type is used extensively in California because of its long-term durability, reliable seismic performance, and low maintenance. Concrete Box Girder structures are desirable for curved bridge alignments because it can be cast on a curve and the box structure has high torsional rigidity. Variable width structures can be easily accommodated with this alternative by varying the girder web spacing and adding internal webs as required.

The disadvantage of this bridge type is the need for temporary shoring to support the structure during construction. The temporary shoring will need to remain in place until the entire box structure is completed and post-tensioned, if required. This will increase the construction time over the other structure alternatives where temporary shoring is not necessary. The temporary shoring can be located to accommodate traffic, but movement under the bridge structure will be restricted. Temporary shoring will also be problematic in environmentally sensitive areas where disruption needs to be kept to a minimum. There is also a practical limitation on the height of temporary falsework, where it is no longer feasible and cost effective to cast the structure in place.

Aesthetically, a Concrete Box Girder structure has a clean graceful appearance. The pier crossbeams can be made integral with the box structure so that the bottom of the structure will have a smooth, unbroken appearance and the columns will look as if they disappear into the box. Concrete Box Girder structures handle curved alignments and large width transitions well, because the exterior overhang can remain constant while the width of the box varies. Expansion joints can be located at internal hinges within the span near points of inflection, so that a drop down pier crossbeam is not necessary and a smooth look can be maintained on the underside of the structure throughout the length of the bridge.

Subtle changes can be made to the box girder structure to change the appearance of the bridge. The bottom of the box can be haunched to give the structure an arched appearance. The sides of the box can be positioned vertically to create a box shape or sloped at varying angles to create a trapezoidal shape. The corners can also be rounded to provide a smooth appearance without any angle breaks on the underside of the bridge.

WSDOT uses traditionally reinforced concrete box girder structures for continuous spans up to 130 feet. Since the foundation costs are expected to be high it is recommended that a post-tensioned concrete box girder system is evaluated for the Magnolia Bridge project, so that the span length can be maximized. Cast-in-place post-tensioned box girder structures have a span-to-depth ration of 25 for continuous girders. Haunched cast-in-place box girder structures have a span-to-depth ratio of 36 at the center of the span and 18 at the intermediate pier for continuous girders. A post-tensioned concrete box girder structure can be used for spans up to 300 feet in length. Longer spans are possible, but for this evaluation the maximum span length will be limited to 200 feet or less for the low-level structure, so the structure depth can be kept to 8 feet deep or less. The span length can be optimized to determine the most feasible alternative for the bridge from both a cost and aesthetic standpoint.

Segmental construction is not recommended for the low-level structures since the structure is located close to the ground and there are few limitations on providing temporary shoring for construction of a cast-in-place structure. A variable width structure is also more complicated to construct with segmental construction. Segmental construction is beneficial and cost effective in locations where long sections of bridge with constant section are used and temporary shoring is very difficult. For this section of bridge structure, segmental bridge construction is not a cost effective solution.

A haunched girder structure is not recommended for the low level structures on the bridge. In general, these sections of the bridge will be located less than 25 feet above the ground and in the case of the ramp structures they will be sloping toward the ground line. Since the structures are low to the ground, the aesthetic appeal of the haunched girder will not be visible since surrounding buildings will block the view of the bridge. The structure located between the BNSF Railroad and the ramp transition for 23rd Avenue is a 75-foot wide structure that is going thru a width transition to accommodate the ramps. The haunched structure will not be as aesthetically pleasing for a wider structure that is going through a width transition because of the expanse of the structure and the odd shape of the edge of the transition.

For a typical section of a cast-in-place concrete box girder bridge in the low-level section of the bridge, refer to Figure 29.

8.1.3 Steel Plate Girder and Steel Box Girder

Steel Girder structures have been used throughout Washington on various bridge projects and interchange structures. Steel girder structures are feasible for curved bridge alignments since the girders can be fabricated to follow the curvature of the bridge. A variable width superstructure can also be achieved with this alternative, by a combination of varying the girder spacing and adding or subtracting girders as required. Seismic performance of steel girders may be better than concrete structures because the weight of steel structures is less and results in a reduced seismic force. The limitation with steel structures is that the frame action provided by concrete structures is more difficult to achieve since steel structures typically are supported on bearings. The reduced weight and seismic loads may also reduce the foundation size and cost.

Steel girders will be erected without temporary shoring and only a few temporary supports are necessary to support the girders until the individual girder sections are spliced together, but these supports can be located away from thoroughfares. This is an advantage in areas where traffic flow needs to be maintained during construction or in environmentally sensitive areas where minimal disruption to surrounding area is necessary. The construction time is less than cast-in-place structures, because the girders can be erected with minimal temporary shoring using smaller cranes since the sections are lighter.

The primary disadvantages of a steel girder superstructure are the variable cost of steel and the long term maintenance costs after the bridge is built. Changing steel market conditions can drive up the cost of a project over a very short amount of time. The cost of maintenance will also be higher for a steel structure. The steel girders will require periodic inspection, maintenance, cleaning, and painting. Steel girder bridges are available in either Plate Girder or Box Girder sections. Both girder sections can be haunched to provide a variable depth appearance for the superstructure. The steel girders will be painted because weathering steel does not perform well in Western Washington because of the moisture.

The appearance of steel box girders can be modified by adjusting the slope on the girder webs. WSDOT BDM states that sloped webs shall not be used for haunched steel box girders. Steel box girders will have a much smoother and less cluttered appearance than steel plate girders, since most of the lateral bracing can be enclosed in the box structure. This will also reduce the amount of exposed steel that will require maintenance and painting. The steel plate girders typically use bird screens to keep the birds from nesting on the girder flanges.

Most steel girder bridges are supported on a cast-in-place concrete drop down pier crossbeams. An integral concrete crossbeam is possible, similar to the West Galer Flyover structure, but this detail is much more complicated and requires temporary support for the girders until the crossbeam is completed and post-tensioned.

It is recommended to advance a steel girder superstructure for the structure on the Magnolia Bridge because of the reduced weight of the structure. Haunched girders are not recommended for low level sections of the bridge, since the structures are low to the ground. The aesthetic appeal of the haunched girder will not be visible since surrounding buildings will block the view of the bridge. The haunched girders will also be complicated by the large width transition to accommodate the ramp structures to 23rd Avenue.

A steel box girder is not recommended for a variable width structure. Variable width steel box girders are difficult to lay out because of the difficulty in adding boxes and complications with varying box sections.

For a typical section of a steel plate girder bridge in the low-level section of the bridge, refer to Figure 30.

8.2 LOW-LEVEL STRUCTURE SEGMENTS

8.2.1 15th Avenue Overpass Structure

The 15th Avenue Overpass is a one lane bridge that carries westbound traffic over 15th Avenue West. The approach ramp and bridge structure runs from the 15th Avenue West/West Galer intersection and continues to the eastside of the BNSF Railroad tracks where it merges with the 15th Avenue Ramp structure before crossing the railroad. The bridge structure is approximately 800 feet long and has an overall width of 23.6 feet. A MSE wall approach ramp structure will be utilized at the beginning of the bridge. The MSE wall approach ramp and approximately 200 feet of the bridge structure will be located on a horizontal curve with a 315-foot radius. Since the new bridge will be located in approximately the same position as the existing ramp, there is flexibility in span lengths and pier locations. A 100-foot minimum span length will be provided over 15th Avenue West. The current bridge profile allows for a 7-foot deep superstructure, while providing the minimum vertical clearance of 20 feet over 15th Avenue West. The superstructure will be supported on reinforced concrete single column bents. The bridge will likely be separated into two separate units to accommodate the 800 foot length.

The following section provides advantages and disadvantages for the three structure types presented in the previous section.

8.2.1.1 Prestressed Girders

Refer to Figure 31 for an elevation view of prestressed girder option for low level structures. The span length for a straight girder will need to be limited to approximately 60 feet in the area of the bridge located on the horizontal curve to limit the amount of overhang slab in the curve. Beyond this section the span length can be increased to a more optimum span length. For aesthetic reasons, the same structure depth is desirable over the entire bridge length.

The smallest WSDOT I-Girder section that can accommodate the 135-foot span over 15th Avenue would be the WF58G girder. This smaller section will also work well for the shorter spans along the horizontal curve. The U66G4 precast Tub Girder is also an applicable section for this structure. The span capability of this girder far exceeds the 60-foot span length in the area of the horizontal curve.

Since the superstructure will be constructed over traffic, and construction time and low cost is important, prestressed girders are a recommended alternative. The disadvantage of prestressed girders for this structure is the limited span lengths that can be used along the horizontal curve.

8.2.1.2 Cast-in-Place Concrete Box Girder

See Figure 32 Straight and Figure 33 haunched.

A cast-in-place concrete box girder meets the requirements for this structure. The span lengths can be maximized to take full advantage of the 7-foot structure depth available over 15th Avenue West. A 175-foot span could be utilized for this structure while maintaining the minimum vertical clearances. The same span length can be used for the entire length of the bridge, since there are no issues with cast-in-place concrete box girders on a horizontal alignment.

Since the design criteria require a 20-foot minimum final vertical clearance over 15th Avenue West, there is sufficient depth to accommodate a temporary shoring system over 15th Avenue West while maintaining minimum construction vertical clearance openings. The structure on either side of 15th Avenue West can be cast on a scaffolding system if access is not required under the structure.

8.2.1.3 Steel Girders

See Figure 32 Straight and Figure 33 haunched.

A steel plate girder or steel box girder structure is feasible for this structure. The steel plate girder bridge would be supported on three girders. The box girder would be a single box structure. For both structures, the span lengths can be maximized to take full advantage of the 7-ft structure depth available over 15th Avenue West. The maximum span length would be 175 feet for either structure type, and this span length could be used for the entire length of the bridge, since there are no issues with steel girder bridges on a horizontal alignment.

The girders could be easily erected over 15th Avenue West without disrupting traffic. Temporary shoring at the splices will be required in some locations, but these can be located in those areas where they will not impact traffic flow.

8.2.2 15th Avenue Approach Ramp

The 15th Avenue Approach Ramp is a three lane bridge that provides access to 15th Avenue from the Magnolia Bridge. The approach ramp runs from the intersection with 15th Avenue West until the structure over the BNSF Railroad. This section of the Magnolia Bridge is approximately 500 feet long and has a total width of 57.6 feet. The bridge accommodates two eastbound lanes, one westbound lane, and a sidewalk on the south side of the structure. A mechanically stabilized earth (MSE) wall structure will be utilized at the beginning of the structure from 15th Avenue West. The bridge structure length will be dependent on the height of the MSE Wall structure. The superstructure will be supported on reinforced concrete multi-column bents.

The following provides advantages and disadvantages for the three structure types presented in the previous section.

8.2.2.1 Prestressed Concrete Girders

Either prestressed I-Girders or post-tensioned Tub Girders are a feasible superstructure alternative for the 15th Avenue Approach Ramp. Since the bridge is on a tangent there are no limitations on the span capability as a result of horizontal curvature. The span length should be maximized where poor soil conditions exist and foundation cost is high.

Recommend using either a WSDOT WF74G I-Girder or a post-tensioned U78PTG5 tub girder in this location.

8.2.2.2 Cast-in-Place Concrete Box Girder

A cast-in-place concrete box girder is a feasible superstructure alternative for the 15th Avenue Approach Ramp. There are no access requirements under the bridge in this location, so the bridge superstructure can be supported on a scaffolding system.

8.2.2.3 Steel Girders

A steel plate girder or steel box girder is a feasible superstructure alternative for the 15th Avenue Approach Ramp. The steel I girder bridge would be supported on multiple girders and the box girder would be either a two or three box structure.

8.2.3 Mainline Bridge Structure

The section of bridge between the BNSF Railroad and the 23rd Avenue ramp transition is a four lane structure that carries Magnolia Bridge traffic over the Port of Seattle property and transitions in width to allow for access ramps to 23rd Avenue. The Port of Seattle structure is approximately 800 feet long and runs from the Westside of the BNSF Railroad Crossing to the ramp transition for the 23rd Avenue access ramps. The structure in this section of the bridge will be located on a slight horizontal curve that moves the alignment south of the existing bridge. The bridge width varies between 69.6 feet up to approximately 125 feet at the ramp transition. The bridge accommodates two eastbound lanes, two westbound lanes, and a sidewalk on the south side of structure. The current bridge profile allows for an 8-ft deep superstructure, while providing the minimum vertical clearance of 20 feet over the Port of Seattle property. The superstructure will be supported on reinforced concrete multi-column bents. The bridge will likely be a single unit for this section.

The following section provides advantages and disadvantages for the three structure types p presented in the previous section.

8.2.3.1 Prestressed Girders

See Figure 31.

Prestressed concrete I-Girders are a feasible option for the bridge structure in this section of the bridge. The span lengths may need to be reduced to accommodate slight horizontal curvature and the large width transitions near the ramp structures. The prestressed girders would be flared and additional girders added to the superstructure to handle the width transition. Both of these factors would likely increase the square-foot cost of the bridge structure in the area near the ramp transition.

Recommend using either a WSDOT WF74G or W83G girder for this location to maximize the span lengths to reduce the number of foundations. Assumed 160-foot span lengths for this evaluation, but span length will be optimized in the alternative development phase.

Post-tensioned concrete tub girders are not recommended for this section of the bridge because of the large width transition. Variable width structures are difficult with tub girders since the girders are so wide and adding additional girders is difficult.

8.2.3.2 Cast-in-Place Concrete Box Girder

See Figure 32 and Figure 33.

A cast-in-place concrete box girder structure would be feasible in this section of the bridge. The cast-in-place structure is ideal for the curved alignment and the large width transition that occurs in this section of the bridge. The span lengths can be maximized to take full advantage of the 8-ft. structure depth available over the Port of Seattle property. A 200-foot span could be utilized for this structure and still maintain the minimum vertical clearances. The same span length can be used for the entire length of this segment, since a reduction in span length is not required for cast-in-place concrete girders due to curvature or large width transitions.

This section of the bridge is located primarily over flat, open paved areas, with the exception of Jacob's Lake. The areas outside of Jacob's Lake could be cast on a scaffolding system since the structure is only 20 feet above existing grade and access does not need to be provided continuously under the structure. The existing bridge currently has a couple of thoroughfares under the existing bridge. If access is required during construction of the new bridge, there is sufficient depth to accommodate a temporary shoring system over a thoroughfare while maintaining minimum construction vertical clearance opening.

Assuming Jacob's Lake is filled prior to construction of the new bridge; the structure could be cast on either scaffolding or a more complex temporary shoring system if required. The type and size of the scaffolding system will be largely dependent on whether Jacob's Lake can be backfilled prior to the construction of the new bridge. If portions of the existing bridge need to remain open during construction of the new bridge, the fill would need to be placed after completion of the new bridge structure.

8.2.3.3 Steel Girder

See Figure 32 and Figure 33

A steel plate girder structure would be feasible for this structure. The steel plate girder bridge would be supported on multiple girders. The span lengths can be maximized to take full advantage of the 8-foot structure depth available over the Port of Seattle. The maximum span length would be 200 feet for either structure type, and the span length could be used for the entire length of the bridge, since there are no issues with steel girder bridges on either a horizontal alignment or a width transition.

The girders could be easily erected over the Port of Seattle property without disrupting traffic movements. Temporary shoring would be required in some locations, but these can be located in those areas where they will not impact traffic movement.

A steel box girder structure is not recommended for the low level structure on the bridge, since the bridge width is transitioning to accommodate the 23rd Avenue ramps. Variable width superstructures are difficult to achieve, since the box girders are so wide and adding additional girders will be difficult.

8.2.4 Westbound Off-Ramp to 23rd Avenue Bridge

The Westbound Off-Ramp to 23rd Avenue is a one lane bridge that carries westbound traffic down to 23rd Avenue West to provide access to the Marina. The ramp structure is located on a tangent running parallel to the mainline bridge alignment. The Westbound Off-Ramp structure is approximately 750 feet long and has an overall width of 23.6 feet. A MSE wall approach ramp structure will be utilized at the end of the bridge as the ramp transitions toward the existing ground line. The bridge structure length will be dependent on the height and length of the MSE Wall structure. The new ramp will be located on top of the existing mainline bridge structure. Ideally, the westbound off-ramp piers would be in the same location as the piers for the Magnolia Bluff Structure so that all piers are aligned when looking at the bridge from the side. This will be dependent on the structure type and span lengths used on the Magnolia Bluff structure. The superstructure will be supported on reinforced concrete single column bents. The bridge will be a single unit.

The current ramp alignment appears to straddle the existing bulkhead wall at the north end of Smith Cove. This bulkhead wall follows the south column line of the existing bridge and is integral with the existing foundations. If the alignment stays in its current position there will likely be impacts between the existing bulkhead wall and the new ramp foundations. The exact location and the condition of the existing bulkhead wall needs to be investigated in further detail. The ramp alignment may need to be shifted to the north to avoid impacts with the existing bulkhead wall.

The following provides advantages and disadvantages for the three structure types presented in the previous section.

8.2.4.1 Prestressed Concrete Girders

Either prestressed I-Girders or post-tensioned Tub Girders are a feasible superstructure alternative for the Westbound Off-Ramp to 23rd Avenue. Since the bridge is on a tangent there are no limitations on the span capability as a result of horizontal curvature. The span length should be maximized in this area since the foundation cost is high.

Recommend using either a WSDOT WF74G or W83G I-Girder or a post-tensioned U78PTG5 tub girder in this location.

A Prestressed concrete I-Girder superstructure would be advantageous for this ramp if part of the structure lies in the tidelands, since no temporary falsework will be required to erect the superstructure.

The post-tensioned concrete tub girders may require temporary supports to post-tension the girders in the final position. This impact may require additional permitting during the construction of the Westbound Off-Ramp to 23rd Avenue.

8.2.4.2 Cast-in-Place Concrete Box Girder

A cast-in-place concrete box girder would be feasible for the Westbound Off-Ramp to 23rd Avenue. Since this is a ramp structure there should be very few access requirements under the bridge, so the bridge superstructure can be supported on a scaffolding system without disrupting traffic flow under the bridge. If access is required during construction of the new bridge, there is sufficient depth at the beginning of the off-ramp to accommodate a temporary shoring system over a thoroughfare while maintaining minimum construction vertical clearance opening.

With a cast-in-place concrete box structure there may be issues with temporary falsework since the existing bulkhead wall lies under the new ramp structure alignment.

8.2.4.3 Steel Girders

A steel plate girder or steel box girder is a feasible superstructure alternative for the Eastbound On-Ramp from 23rd Avenue. The steel I girder bridge would be supported on two or three girders and the box girder would likely be a two box structure.

Steel girder superstructure would be advantageous for this ramp if part of the structure lies in the tidelands, since no temporary falsework will be required to erect the superstructure.

8.2.5 Eastbound On-Ramp from 23rd Avenue Bridge

The Eastbound On-Ramp from 23rd Avenue is a one lane bridge that provides eastbound access to the Magnolia Bridge from 23rd Avenue. The ramp structure is located on a tangent running parallel to the mainline bridge alignment. The Eastbound Off-Ramp structure is approximately 750 feet long and has an overall width of 34.6 feet. A MSE wall approach ramp structure will be utilized at the beginning of the bridge. The length of the MSE wall will be controlled by the structure depth and the proximity of the end of the ramp to the Westside of Smith Cove. The bridge structure length will be dependent on the height and length of the MSE Wall structure. A large portion of the new ramp will be located within the tidelands of Smith Cove. Since there is no existing structure in this location there is flexibility in span lengths and pier locations, but the number of piers should be minimized to reduce environmental impact within Smith Cove. Ideally, the westbound off-ramp piers would be in the same location as the piers for the Magnolia Bluff Structure so that all piers are aligned when looking at the bridge from the side. This will be dependent on the structure type and span lengths used on the Magnolia Bluff structure. The superstructure will be supported on reinforced concrete single column bents. The bridge will be a single unit.

The following section provides advantages and disadvantages for the three structure types presented in the previous section.

8.2.5.1 Prestressed Concrete Girders

Either prestressed I-Girders or post-tensioned Tub Girders are a feasible superstructure alternative for the Westbound Off-Ramp to 23rd Avenue. Since the bridge is on a tangent there are no limitations on the span capability as a result of horizontal curvature. The span length should be maximized in this area since the foundation cost is high.

Recommend using either a WSDOT WF74G or W83G I-Girder or a post-tensioned U78PTG5 tub girder in this location.

A Prestressed concrete I-Girder superstructure would be advantageous for this ramp since the bridge structure lies in the tidelands and no temporary falsework will be required to erect the superstructure.

The post-tensioned concrete tub girders may require temporary supports to post-tension the girders in the final position. This impact may require additional permitting during the construction of the Eastbound On-Ramp from 23rd Avenue.

8.2.5.2 Cast-in-Place Concrete Box Girder

A cast-in-place concrete box girder would be very feasible for the Westbound Off-Ramp to 23rd Avenue. Since most of this structure is located over Smith Cover there should be very few access requirements under the bridge, so the bridge superstructure can be supported on a scaffolding system. If access is required during construction of the eastern end of the ramp, there is sufficient depth to accommodate a temporary shoring system over a thoroughfare while maintaining minimum construction vertical clearance opening.

With a cast-in-place concrete box structure there will be the temporary falsework required, because this will be located within the tidelands in Smith Cove. The temporary falsework will increase the amount of environmental impact for the construction of the bridge. This impact may require additional permitting during the construction of the Eastbound On-Ramp from 23rd Avenue.

8.2.5.3 Steel Girders

A steel plate girder or steel box girder is a feasible superstructure alternative for the Westbound Off-Ramp to 23rd Avenue. The steel I girder bridge would be supported on three girders and the box girder would be a single box structure.

Steel girder superstructure would be advantageous for this ramp since the bridge structure lies in the tidelands and no temporary falsework will be required to erect the superstructure.

8.2.6 Low-level structure Evaluation Matrix

The five structure types evaluated for this section of bridge will include: Prestressed Concrete Girders, Post-tensioned Concrete Tub Girders, Cast-in-Place Concrete Box Girder, Steel Plate Girders and Steel Box Girders. The post-tensioned concrete Tub Girder and steel Box Girder are not recommended for the low level structure because of the difficulty of handling the large transition width near the ramp structures. These structures will be included in the evaluation matrix. The cost estimates were determined using base structure costs based on square foot costs of similar structures from August, 2004 unit prices.

For an elevation view of the Low-Level Structure options including: prestressed concrete girders, straight concrete box or steel girder/box and haunched concrete box or steel girder/box, refer to Figure 31, Figure 32, and Figure 33, respectively.

Evaluation meetings were held at SDOT and with the Design Advisory Group to decide which alternatives to advance to Bridge Alternative Study phase. The selected alternatives are identified in the evaluation matrix Table 14.

Table 15
Low-Level Bridge Evaluation Matrix

Structure Type	Bridge Base Cost (2004 $)	Advantages	Disadvantages
Prestressed Concrete Girders (Advanced to Bridge Alternative Development Phase)	$21,000,000	• Lower cost • Straightforward to construct, no temporary shoring • Shorter construction duration • Low maintenance • Reduced environmental permitting required for construction	• Span Limitations on curved alignments • Does not accommodate width transitions very well • Less aesthetically pleasing
Post-Tensioned Concrete Tub Girders	$22,000,000	• Average cost • Low Maintenance • Shorter construction duration • Aesthetically pleasing	• Span limitations on curved alignments, tight radius curves are very difficult • Does not accommodate width transitions very well • Requires temporary shoring
Cast-in-Place Concrete Box Girder (Advanced to Bridge Alternative Development Phase)	$24,000,000	• Average cost • Accommodates curved alignments well • Accommodates width transitions well • Low maintenance • Aesthetically pleasing • Architectural options available • Long span capability	• Increased construction duration • Requires temporary shoring • Increased environmental permitting required for construction • Increased impacts below bridge during construction
Steel Plate Girder (Advanced to Bridge Alternative Development Phase)	$26,000,000	• Accommodates curved alignments well • Accommodates width transitions well • Shorter construction duration • Minimal temporary shoring • Reduced environmental permitting required for construction • Reduced foundations due to less weight • Long span capability	• Higher superstructure cost • Higher maintenance/Life cycle costs

Structure Type	Bridge Base Cost (2004 $)	Advantages	Disadvantages
Steel Box Girder	$26,000,000	• Accommodates curved alignments well • Shorter Construction Duration • Minimal temporary shoring • Reduced environmental permitting required for construction • Reduced foundations due to less weight • Long span capability	• Higher superstructure Cost • Does not accommodate width transitions very well • Higher maintenance/Life cycle costs

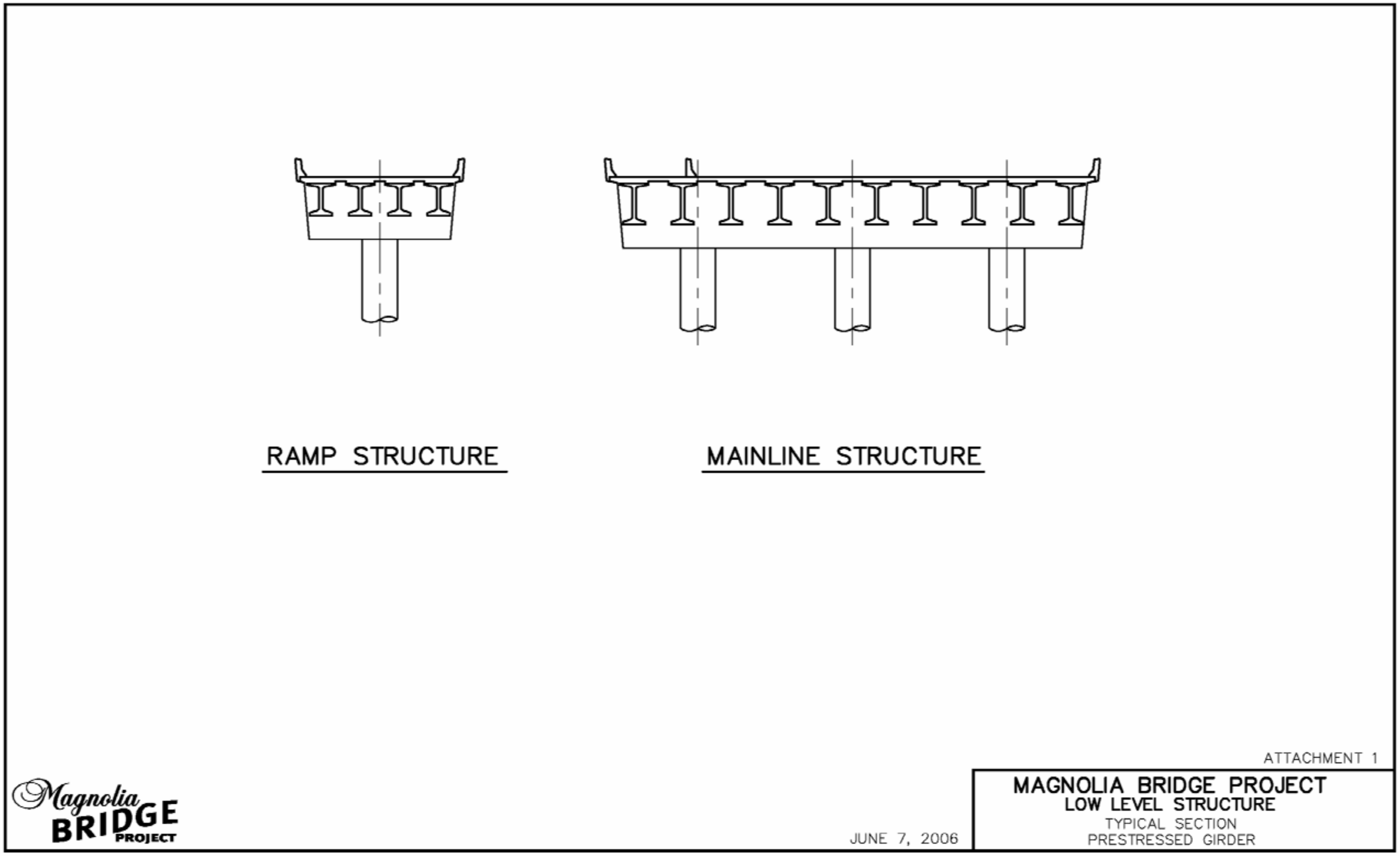


Figure 28
Low Level Structure
Typical Section – Prestressed Girder

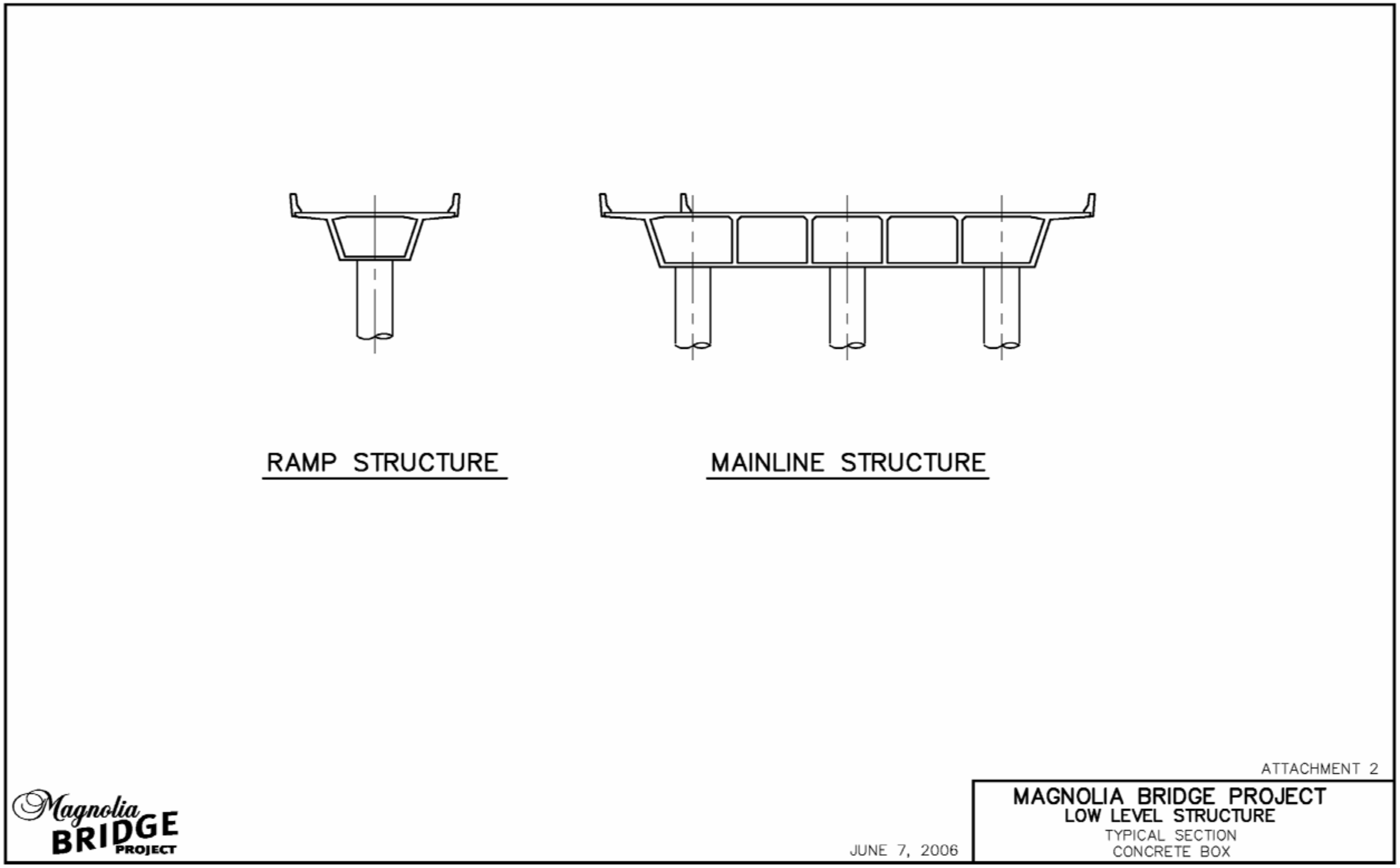


Figure 29
Low Level Structure
Typical Section – Concrete Box

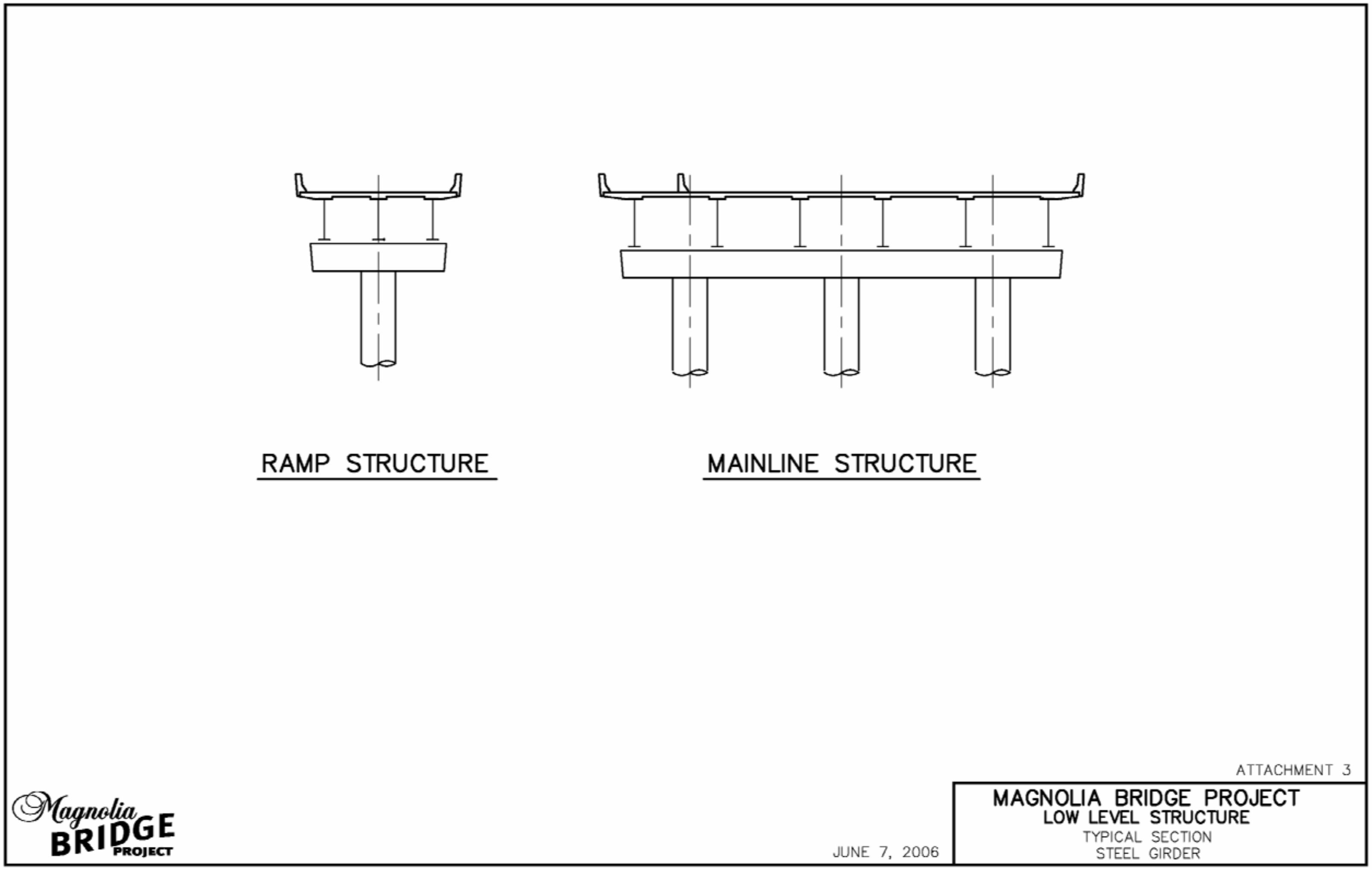


Figure 30
Low Level Structure
Typical Section – Steel Girder

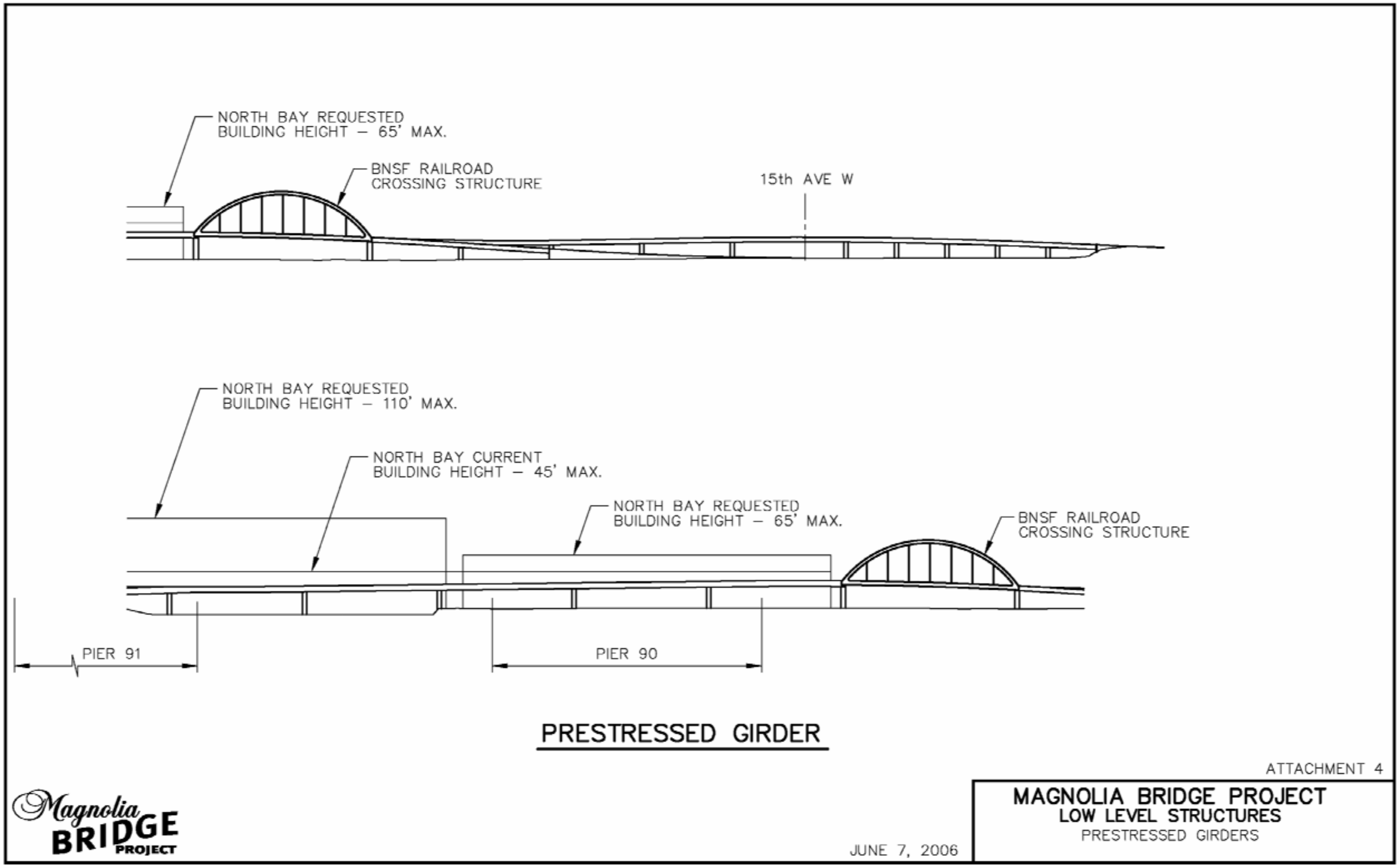


Figure 31
Low Level Structures
Prestressed Girders

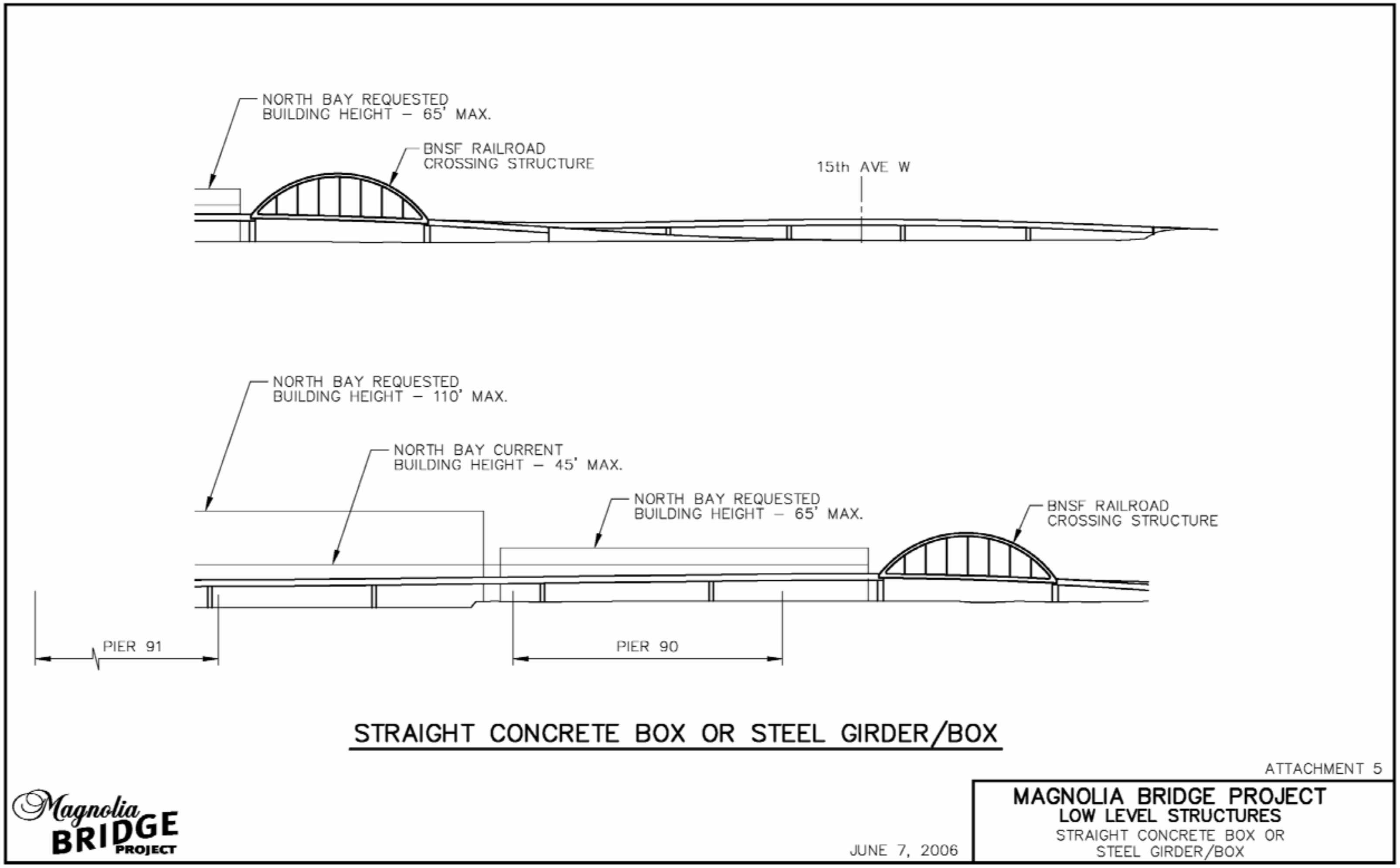


Figure 32
Low Level Structures
Straight Concrete Box or Steel Girder/Box

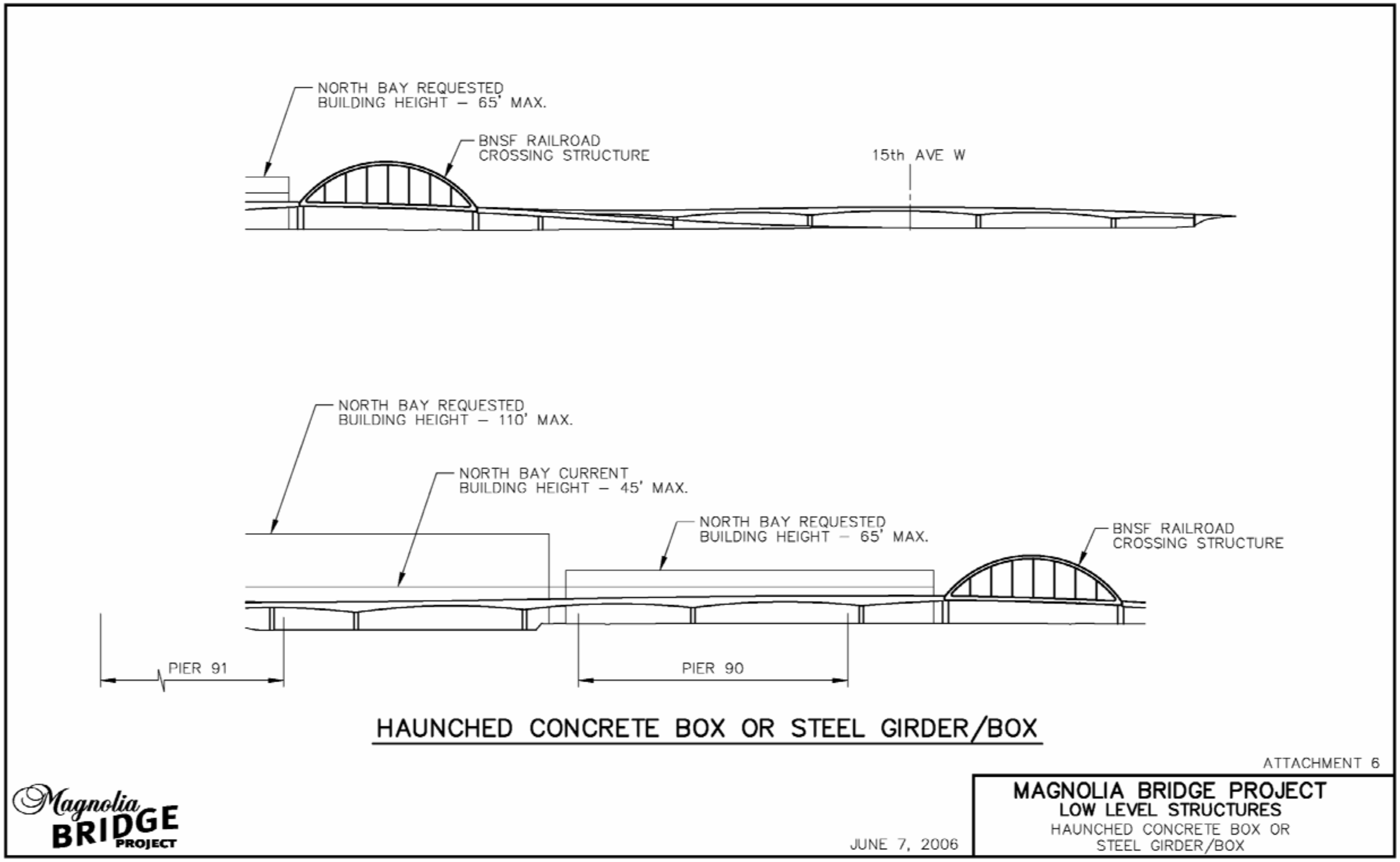


Figure 33
Low Level Structures
Haunched Concrete Box or Steel Girder/Box

8.3 BNSF RAILROAD CROSSING STRUCTURE

The BNSF Railroad Crossing Structure is a four lane structure that carries Magnolia Bridge traffic over the BNSF railroad. The bridge accommodates two eastbound lanes, two westbound lanes, and a sidewalk on the south side of structure.

BNSF has restrictions on work around the track to maintain safe operations during train movements. This bridge crosses the mainline BNSF tracks that carry about 80 trains per day. In discussions with BNSF Railway, they indicated that they could only provide work windows in the range of 2 to 8 hours on an irregular basis. When trains are passing, all work must stop and cranes are required to be turned away from the tracks in a locked position. All work within 12 feet of the tracks must be stopped when trains are passing. Any work that may cause settlement will be closely monitored, BNSF will adjust grade by reballasting if the deflection of the track exceeds ¼ inch.

There is an existing pier in the middle of the BNSF tracks that is about 12 feet clear to centerline of tracks. A replacement to the existing pier would include a crash wall that will encroach on the 12-ft. clearance. The center pier foundation would likely be a drilled shaft or driven pile foundation with ground improvements. Given the size of the equipment, the limitations on operations adjacent to the tracks, the limited construction windows, and potential settlement of tracks during pile driving or ground improvements, it is recommended that the center pier not be replaced and a single span structure be selected.

A structure that spans the railroad will allow construction of the piers to occur outside the railroad envelope and remove any obstructions that may reduce visibility for the railroad engineers. The main span over the railroad will be approximately 210 feet with 25-foot clearance provided between the centerline of track and the face of column. The 25-foot clear distance will eliminate the need for crashwalls around the columns. The required vertical clearance over the railroad is 23.5 feet.

Detour options are being considered that require staged construction of the BNSF Railroad Crossing Structure. Staged construction would include maintaining traffic on half the bridge while the other half is demolished and rebuilt, then traffic would be switched to the new bridge and the process repeated for the other half of the bridge. The ability of the structures to accommodate staging is discussed in the following sections.

Above deck structure alternatives were investigated to provide long spans with reduced depth. A two-span steel girder bridge was considered in this location even though a center pier is not recommended as it will disrupt railroad operations. The above deck structure alternatives considered include; Steel Arch Bridge, Cable Stayed Bridge, Cable Stayed Extradosed Bridge, Steel thru-Truss, or Steel Thru-Girder.

8.3.1 Railroad Crossing Structure Depth Study

The previous structure types evaluated all provided 5 foot structure depth. Alternative profiles were developed that would allow greater structure depth over the railroad; 5 foot, 7.5 foot, 10 foot, and their resulting slopes on the 15th Avenue Ramp.

The 5 foot structure depth over the railroad provides a 6.49% slope. This 6.49% slope and a match point just east of the railroad tracks gives a profile for the 15th Avenue crossover that meets minimum standard but could be improved significantly if the match point could be moved to the west. In that case the sag curve can be removed from the profile resulting in a much better design.

The 7.5 foot structure (skew) depth shows the slope on the 15th Avenue ramp increasing to 7.10%. The steeper slope is a disadvantage. It does however provide an appropriate profile for the 15th Avenue crossover.

The slope of the 15th Avenue Ramp is even steeper for the 10 foot structure depth option. Profile is raised even higher above existing profile as when 7.5 foot structure depth is used. No advantage compared to the 7.5 foot structure.

The following discussion outlines the advantages and disadvantages of each of the three profile options from a structures perspective, focusing on feasible structure types, constructability, and cost. For cost comparison of alternatives, see BNSF Railroad Crossing Structure Evaluation Matrix, Table 15.

8.3.1.1 5 foot structure depth with a 6.49% grade

This is the same structure depth that has been proposed for this structure to date and was the basis for the recommended alternatives presented in the Bridge Concept Study Report completed in early June 2006. The two-span steel structure, steel tied arch, and two-span cable stayed bridge were the preferred structures for this option.

For a detailed explanation of the advantages and disadvantages of each of these structure types reference the Bridge Concept Study section. These are preliminary estimates based on an initial analysis of the structure. Staged construction of the two-span steel plate girder and the steel tied arch will increase the cost of the structure and are reflected later in this memo. Staged construction of the cable stayed bridge is not recommended.

8.3.1.2 7.5 foot structure depth with a 7.10% grade

A 7.5 foot structure depth will accommodate either a prestressed girder (Figure 38) or steel plate girder (Figure 37) structure over the BNSF railroad. To accommodate this depth the structure will likely be supported on skewed piers that are parallel to the existing track alignments. A 160 foot span length will span the tracks and be located 25 feet clear of the existing tracks so no crash walls will be required. Forty-five degree skewed piers are not desirable for structures, but they can be accommodated if necessary.

The split structures (independent profiles for the 15th Avenue Overcrossing and 15th Avenue Approach Ramp roadways) proposed can be easily accommodated with either structure alternative since the superstructure will be supported on girders. Both of these structure alternatives can be erected over the BNSF Railroad tracks with minimal disruption to train operations.

If the 7.5 foot structure depth is considered a feasible alternative, the structure depth could be optimized and reduced. One option would be to reduce the span length over the railroad by locating the piers within 25 feet of the exterior tracks and provide crashwalls. This may not be preferred by the BNSF as they may want to have the 25 feet clear to accommodate an access road or future track, but is worth investigating if the slope is critical. Another option is to make the structure over the railroad a three-span continuous steel plate girder structure. A multiple span structure will be more efficient and will require less structure depth for the same span length over the railroad.

8.3.1.3 10.0 foot structure depth with a 7.70% grade

A 10.0 foot structure depth will accommodate a single span steel plate girder structure over the BNSF Railroad. The superstructure will be supported on square (perpendicular to the roadway alignment) piers located 25 foot clear of the existing tracks so no crash walls will be required. The span length will be approximately 210 foot, which is the same span length for the steel tied arch proposed for the 5 foot structure depth.

The split structures proposed can be easily accommodated since the superstructure will be supported on girders. This structure can be erected over the BNSF railroad tracks with minimal disruption to train operations.

If the 10.0 foot structure depth is considered a feasible alternative, the structure depth could be optimized and reduced. One option would be to reduce the span length over the railroad by locating the north end of pier on the west side of the railroad tracks and the south end of the pier located on the east side of the railroad tracks within 25 feet of the exterior tracks and provide crashwalls. This may not be preferred by BNSF as they may want to have the 25 feet clear to accommodate an access road or future track, but is worth investigating if the slope is critical. Another option is to make the structure over the railroad a three-span continuous steel plate girder structure. A multiple span structure will be more efficient and will require less structure depth for the same span length over the railroad.

8.3.2 Two Span Steel Girder Bridge

If the existing center pier is replaced, a two span steel girder bridge with 45 degree skewed piers is an option. The depth to competent bearing material is approximately 100 feet at the railroad overpass. This will require deep driven piles or deep drilled shaft foundations. As noted above, the BNSF Railway will place restrictions on work adjacent to the track. The restriction make it so a center pier will be difficult and costly to construct, there is not enough clearance or work window to construct a deep drilled shaft and the pile driving operations will interfere with adjacent track usage. The restrictions are severe enough that a two span steel girder bridge is not recommended.

8.3.3 Steel Thru-Girder Bridge

The steel thru-girder option was evaluated and not recommended because it is a fracture critical structure and the depth required for the edge girders will make it a very obtrusive structure. Edge girders may be as much a 14 feet and will complicate fabrication and construction of the bridge.

Refer to Figure 34.

8.3.4 Steel Thru-Truss

The steel thru-truss option was evaluated and not included in the final structure alternatives for the BNSF Railroad crossing. Single span truss bridges are typically not used for new highway bridge construction because of the high cost of labor involved with constructing individual truss members and the long term performance of the multiple connections.

Refer to Figure 34.

8.3.5 Steel Arch Bridge

Refer to Figure 35.

A steel arch bridge is an aesthetically pleasing option for an above deck structure over the BNSF railroad. A tied arch and the half-through arch are both feasible alternatives. The tied arch is the preferred at this location because the arch forces can be resisted by the bottom tie without transferring them to the substructure. This will reduce the foundation loads. A half-through arch has large lateral forces that need to be resisted by the footings. This is not practical at this location because of the depth of the depth to competent soil material. The columns and substructure could be detailed to make the structure look like a half-through arch if this is a preferred aesthetic look for the span over the BNSF railroad.

A concrete arch was not selected for this structure because of the complications associated with constructing the concrete arch over the active railroad.

For a steel arch, the main arch and deck level tension tie members will likely be built-up steel box sections to provide redundancy and provide an aesthetic appearance. Vertical hangers are typically wire ropes or rolled sections. To reduce the amount of obstructions for the motorist or pedestrian on the bridge, it is recommended to use wire rope for the hangers. Typically tied arch structures have lateral bracing between the two arch planes to resist the lateral forces acting on the bridge. It may be feasible to remove the lateral bracing and design the main arch members to resist the lateral loads.

Erection of the arch structure over the railroad will require temporary supports placed within the limits of BNSF right-of-way in order to erect the individual sections of the arch and tie girder. Since there will be 25 feet clearance to the outside tracks and the existing bridge currently has piers located within the tracks, it may be feasible to use temporary shoring to erect the bridge in its final location.

As an alternative, the entire arch could be built adjacent to the bridge and moved into place. This would require a temporary track closure, but similar operations have been completed in closure windows less than 24 hours. Once the arch is in place the deck could be cast and any remaining work completed on the structure in its final position.

Some of the construction staging options being considered includes the BNSF structure built in stages, one half at a time. It is feasible to construct the structure over the railroad in stages, but it will increase the cost and complicate construction since the arch will need to be erected in separate stages adjacent to operational structures. The structure would likely be supported on a three arch system with an arch at each side of the roadway and one in the middle between the 15th Avenue overpass and the 15th Avenue approach ramp.

8.3.6 Cable Stayed Bridge

A Cable-Stayed Bridge is another feasible alternative for the superstructure over the BNSF Railroad. The cable-stayed bridge is a very graceful and aesthetically pleasing signature span structure that would make a gateway structure for the Magnolia Bridge. Refer to Figure 35 and Figure 36.

Cable-stayed bridges are primarily used for high level, long span structures where it is beneficial to construct the bridge without providing temporary falsework from the ground. Cable-stayed bridges also have the advantage of a shallow deck. Since temporary falsework over the BNSF Railroad is limited and structure depth available is minimal, the cable-stayed bridge is a feasible superstructure alternative for the span over the BNSF Railroad.

There are several span, tower and cable layout options available for the cable-stayed structure over the BNSF railroad; a two span, symmetrical or asymmetrical, or a three span structure.

For a two-span alternative the main tower can be located on either the east or west side of the bridge. If a symmetrical span layout is preferred, a tower located on the west side of the BNSF Railroad is recommended in order to avoid the two separate structures at varying elevations on the east side of the railroad. A tower on either the east or west side of the tracks would be feasible for an asymmetrical structure, although the east side structure may be more complicated as a result of the grade separated structures. A symmetrical balanced three span alternative with towers on both sides of the railroad would also be feasible. For this alternative the back span lengths would be approximately 40 to 50 percent of the main span length.

There are several options for the number of towers at each pier including one each side of the roadway, one in the middle of the roadway, or one on each side and in the middle. The type of tower chosen will have an affect on aesthetics, cost and cable arrangements. For this location, one cantilever tower each side of the roadway is preferred over a single tower pier. Since the bridge is relatively wide in comparison to the main span length, a single tower alternative is not recommended because the bridge width may have to be increased to provide vertical clearance under the cables. A single tower alternative would be further complicated by the fact that two separate structures, with unbalanced widths are merging near the main span over the railroad.

Some of the construction staging options being considered includes the BNSF structure built in stages, one half at a time. It is feasible to construct the structure over the railroad in stages, but it will increase the cost and complicate construction since the cable stays will need to be erected in separate stages adjacent to operational structures. The structure would likely be supported on a three tower system with a tower at each side of the roadway and one in the middle between the 15th Avenue overpass and the 15th Avenue approach ramp.

The height of the tower is dependent on several factors, including: the span layout, span lengths and cable arrangement. For a two span cable-stayed bridge, the tower height above the deck will be approximately 20 percent of the length of the two spans, about 80 feet. For a three span cable-stayed bridge, the tower height above the deck will be approximately 20 percent of the main span length, about 40 feet. The cable arrangement and tower height will be refined in the structure alternative development phase of the project.

There are also many different cable arrangements that can be used to support the superstructure. The cable arrangement can have a significant impact on the aesthetics of the bridge structure. As was stated above the cable arrangement may also affect the tower height for the structure.

The superstructure can be either concrete, steel or a steel composite concrete deck. The preferred superstructure type is dependent on cable arrangement, structure depth, cost, aesthetics and constructability. A precast segmental concrete superstructure is not recommended for this structure, since concrete is much heavier and the cost of precast elements for this short of a bridges structure is not cost effective. An orthotropic steel deck is also not recommended because of the high cost of these types of structures. The preferred superstructure type is either a steel plate girder or steel box girder system with a precast or cast-in-place concrete deck.

A cable-stayed bridge alternative could be constructed over the railroad with minimal disruption to train traffic. Minimal closures may be required to pick sections into place, but no temporary shoring should be necessary to build the structure. The structure would be built by advancing the superstructure erection from the tower.

8.3.7 Cable-Stayed Extradosed Bridge

Refer to Figure 36.

The Cable-Stayed Extradosed Bridge is another feasible alternative for the superstructure over the BNSF Railroad. The extradosed bridge offers a sleek and aesthetically pleasing alternative to a traditional cable-stayed bridge. Extradosed bridges are typically used where structure depth and tower height is limited. The towers on an extradosed bridge are about one-half the height of those on a conventional cable-stayed bridge structure. This results in a less obtrusive structure.

Extradosed bridges have been designed and constructed all over the world. There are currently no extradosed bridges in operation in the United States. The Pearl Harbor Memorial Bridge in New Haven, Connecticut, scheduled to begin construction in 2006, will be the first extradosed structure to be built in the United States. The Stillwater Bridge over the St. Croix River on the border between Minnesota and Wisconsin is also planned as an extradosed structure.

The typical extradosed bridge is a hybrid design between a cable-stayed bridge structure and a segmental concrete box girder structure. The addition of cable supports for the superstructure increases the span capacity of the box girder structure. This will allow a shallower girder structure to span much further. This is required at this location since a traditional steel girder structure is not feasible because the structure depth is limited to only 5 feet over the railroad.

An extradosed bridge is designed to have a self supporting bridge deck. The superstructure on an extradosed bridge would be heavier and more rigid than a traditional cable-stayed bridge so it can carry a portion of its own dead load.

A symmetrical three-span layout or two-span layout is feasible at this site. The preferred configuration for an extradosed bridge over the railroad is a symmetrical two span bridge. A tower on either the east or west side of the tracks would be feasible for two span structure, although the east side structure may be more complicated as a result of the grade separated structures. The tower would be located at 25 feet clear of BNSF railroad so it could be built without interfering with track operations. A tower would be located each side of the roadway. The towers could be in the vertical position or flared out away from the deck structure. The height of the tower above the deck will be approximately 20 percent of the span length, or about 40 feet. The three span option would have towers about 20 feet tall but will have complications because of the split approach structure on the east side. The cable arrangement and tower height will be refined in the structure alternative development phase of the project.

Staged construction of the extradosed structure over the BNSF railroad will be complicated and expensive. A third tower would likely need to be added to the structure to accommodate staged construction, so it can be constructed one half at a time.

As with a traditional cable-stayed bridge there are many different cable arrangements available to support the structure. The cables could extend out to midspan as for a cable-stayed bridge or they can be located locally near the support piers. The cable arrangement may also be driven by the construction sequence.

The superstructure can be either concrete or a steel composite concrete deck. The preferred superstructure type is dependent on cable arrangement, structure depth, cost, aesthetics and constructability. The superstructure for extradosed bridges has traditionally been concrete. Either cast-in-place or segmental construction can be used for construction. Segmental construction would be preferred, since this is located over the railroad and temporary supports will not be allowed as they will disrupt railroad operations. Minimal closures may be required to pick sections into place, but no temporary shoring should be necessary to build the structure using segmental construction.

A steel plate girder or box girder composite concrete deck superstructure is also a feasible alternative. A steel superstructure would be a good alternative to concrete since the composite steel deck is much lighter and the cost of precast concrete elements for this short of a bridge structure is not cost effective. The preferred deck structure will be determined in the structure alternative development phase of the project.

8.3.8 BNSF Railroad Crossing Structure Evaluation Matrix

The eight structure types evaluated for this section of bridge will include: Two Span Steel Girder, Steel Plate Thru-Girder, Steel Truss, Steel Arch, Cable-Stayed Bridge, Cable-Stayed Extradosed Bridge, prestressed girder and steel plate girder. The cost estimates were determined using base structure costs based on square foot costs of similar structures from August, 2004 unit prices.

Evaluation meetings were held at SDOT and with the Design Advisory Group to decide which alternatives to advance to Bridge Alternative Study phase. The selected alternatives are identified in the evaluation matrix Table 15.

For an elevation view of the BNSF Railroad Crossing Structure options, refer to Figure 34 to Figure 38.

Table 16
BNSF Railroad Crossing Structure Evaluation Matrix

Structure Type	Bridge Base Cost (2004 $)	Advantages	Disadvantages
Two Span Steel Girder - 5 feet deep	$3,600,000	▪ Lower cost ▪ No temporary shoring	▪ Center pier very difficult to construct ▪ High impacts to BNSF Railroad operations during construction ▪ Skewed piers ▪ Less aesthetically pleasing ▪ Higher maintenance/Life cycle cost
Steel Truss - 5 feet deep	$7,000,000	▪ Lightweight	▪ Higher Cost ▪ Higher maintenance/Life cycle ▪ Not aesthetically pleasing, more obtrusive ▪ Difficult to erect and place in final position ▪ Increased disruption to railroad operations
Steel Plate Thru-Girder - 5 feet deep	$5,000,000	▪ Lower cost ▪ Easy to construct, no temporary shoring	▪ Not aesthetically pleasing, very obtrusive ▪ Fracture Critical ▪ Higher maintenance/Life Cycle Cost
Steel Arch - 5 feet deep	$7,000,000	▪ Aesthetically pleasing, signature span bridge, gateway structure	▪ Higher cost ▪ High maintenance/Life cycle cost ▪ Difficult to erect and place in final position ▪ Increased impacts to BNSF Railroad operations during construction
Cable-Stayed Bridge - 5 feet deep	$22,000,000 Reduce Low-Level Structure Cost by: $3,200,000	▪ Aesthetically pleasing, signature span bridge, gateway structure ▪ Minimal impacts to BNSF Railroad operations during construction	▪ Higher cost ▪ Higher maintenance/Life cycle costs for steel superstructure
Cable-Stayed Extradosed	$22,000,000	▪ Aesthetically pleasing, signature span bridge, gateway structure	▪ High Cost ▪ Higher

Structure Type	Bridge Base Cost (2004 $)	Advantages	Disadvantages
Bridge - 5 feet deep	Reduce Low-Level Structure Cost by: $3,200,000	▪ One of only a few bridges of this type in the United States ▪ Low Maintenance ▪ Minimal impacts to BNSF Railroad operations during construction	maintenance/Life cycle costs for steel superstructure
Steel Girder - 7.5' Deep (Advanced to Bridge Alternative Development Phase)	$3,100,000	▪ Lower Cost ▪ No temporary shoring ▪ Minimal impacts to BNSF Railroad operations ▪ Square piers	▪ Higher maintenance/Life cycle costs for steel superstructure
Prestressed Girder – 7.5' Deep (Advanced to Bridge Alternative Development Phase)	$2,7000,000	▪ Lower cost ▪ No temporary shoring ▪ Low maintenance ▪ Minimal impacts to BNSF Railroad operations	▪ Skewed piers ▪ Less aesthetically pleasing

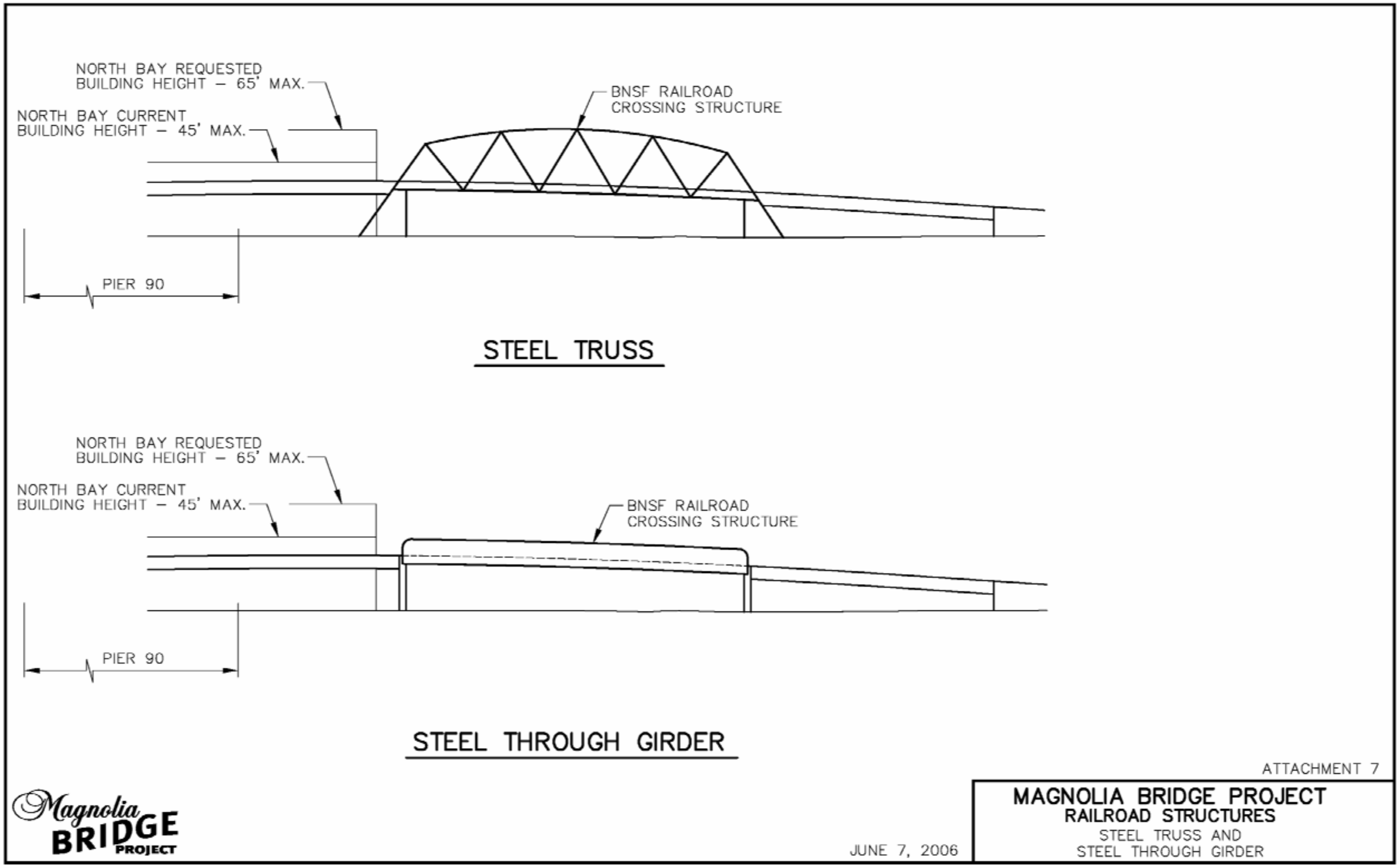


Figure 34
Railroad Structures
Steel Truss and Steel Through Girder

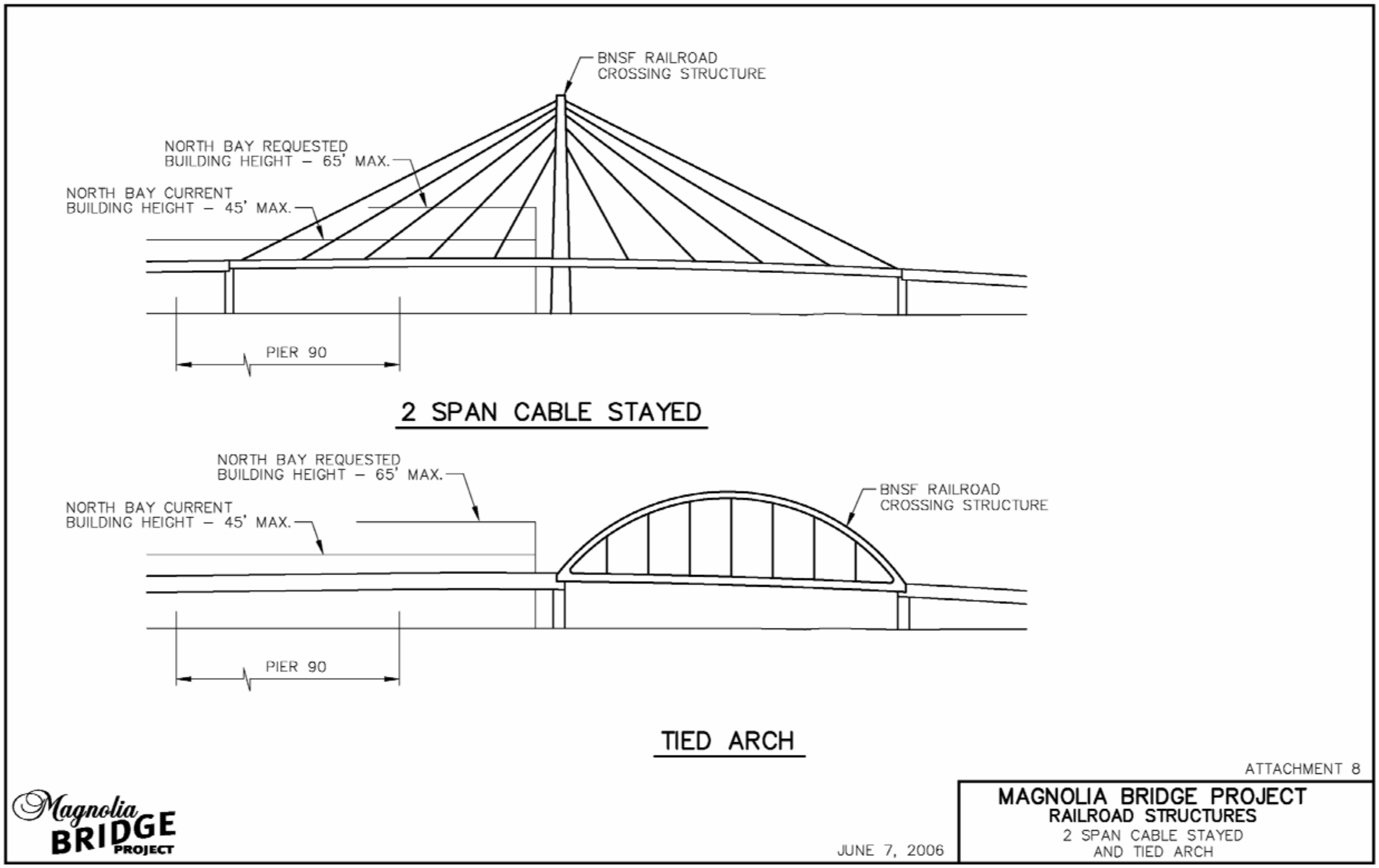


Figure 35
Railroad Structures
2 Span Cable Stayed and Tied Arch

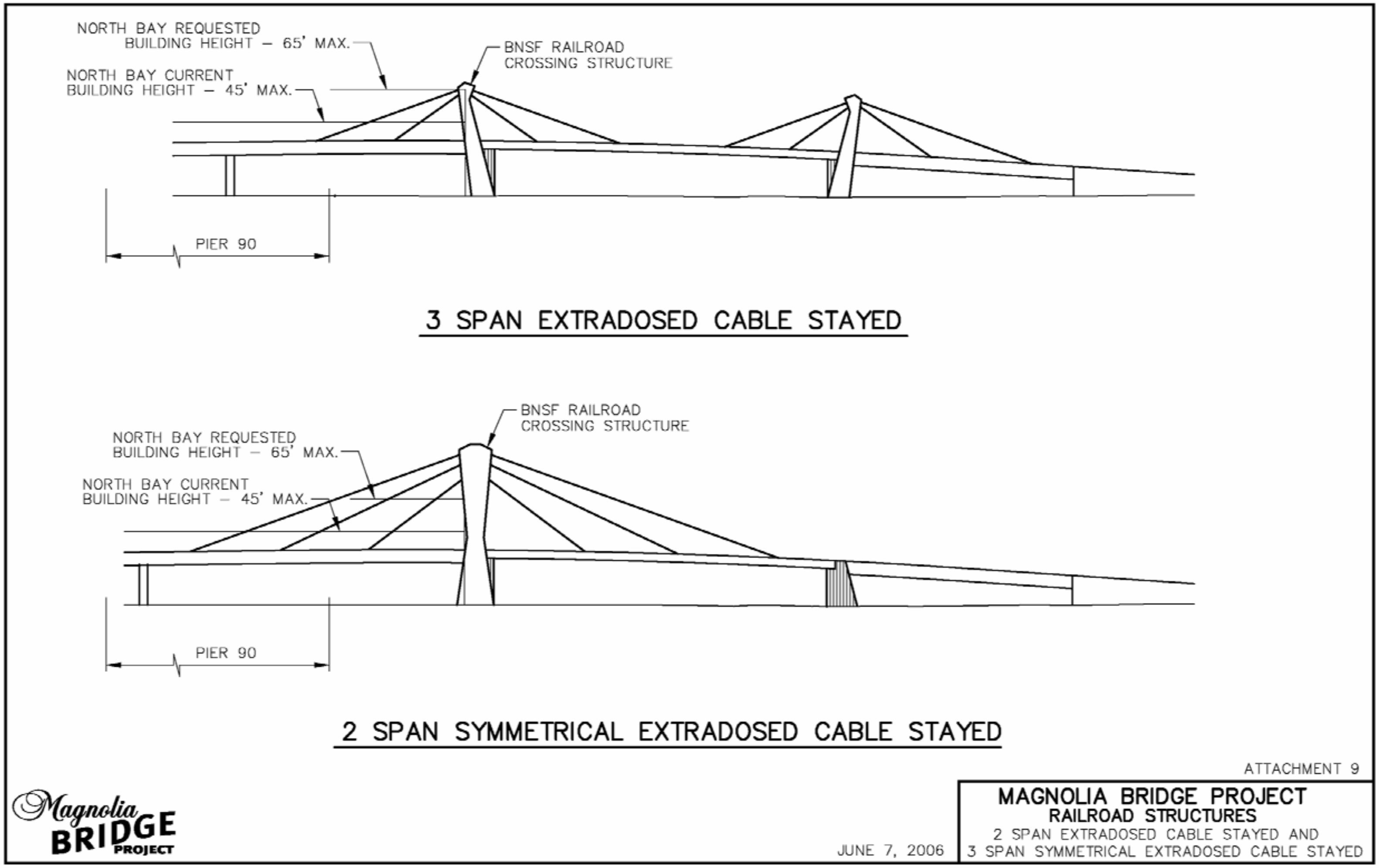


Figure 36
Railroad Structures
2 Span Extradosed Cable Stayed and 3 Span Symmetrical Extradosed Cable Stayed

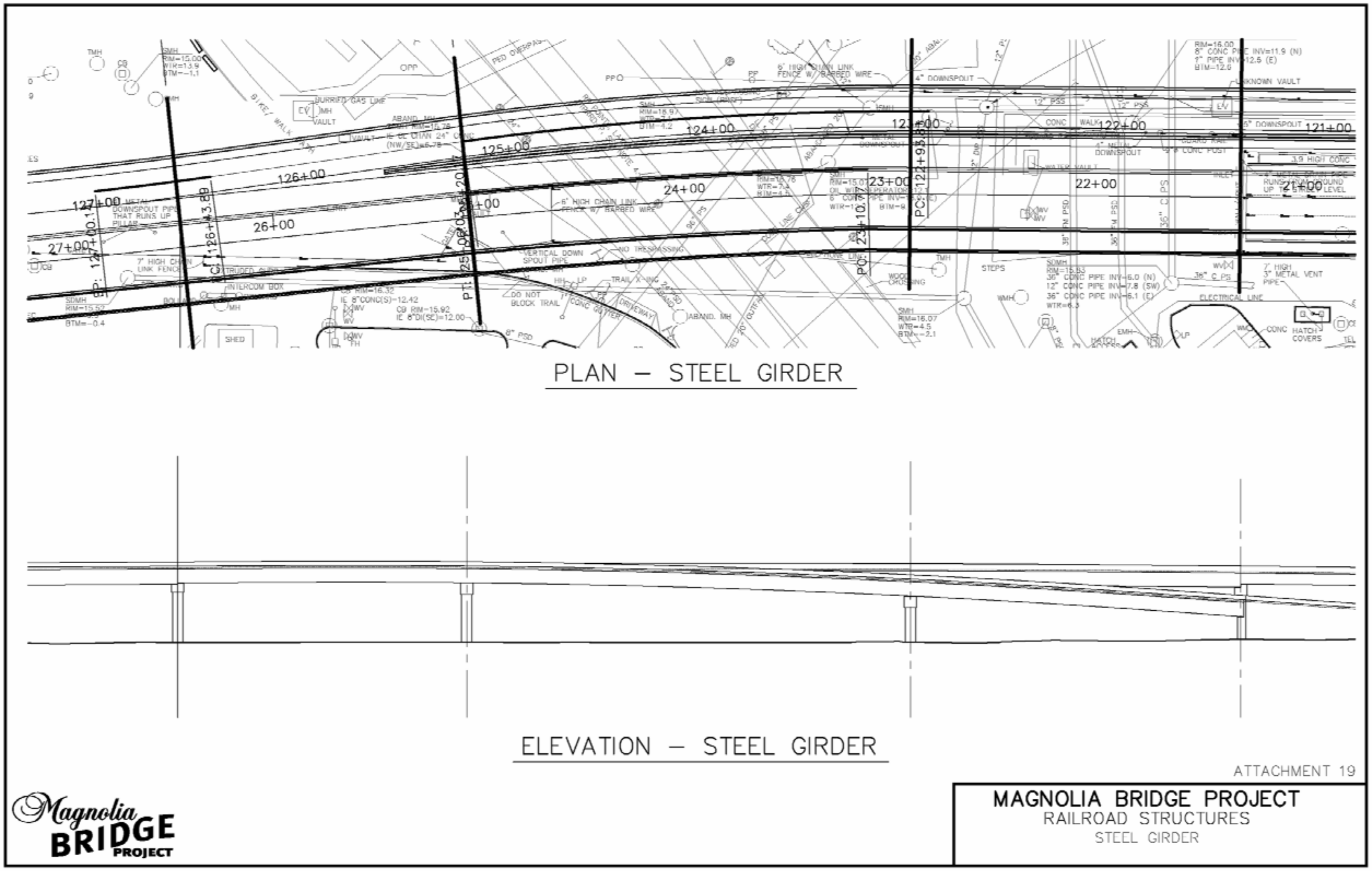


Figure 37
Railroad Structures
Steel Girder

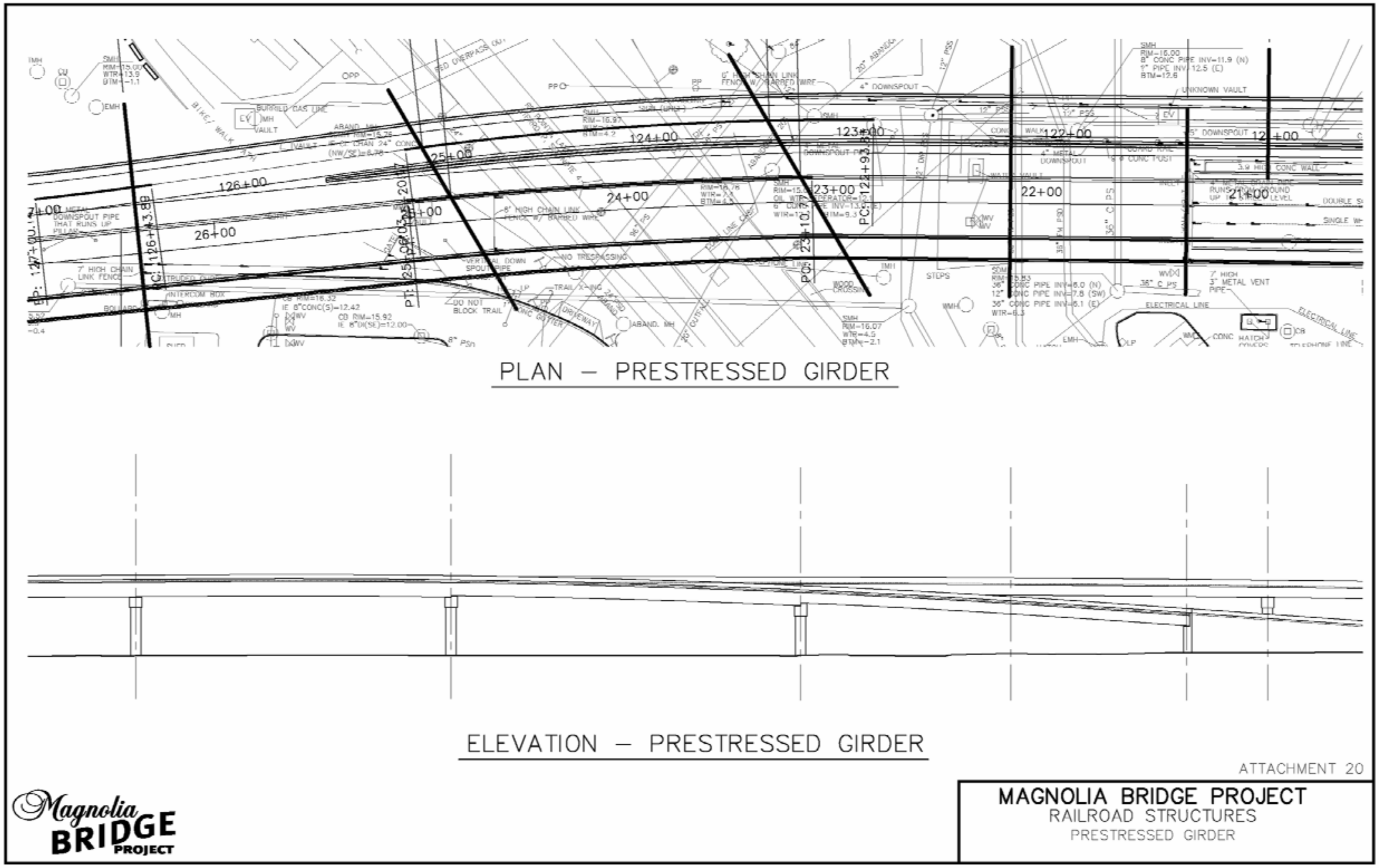


Figure 38
Railroad Structures
Prestressed Girder

8.4 BLUFF STRUCTURE TYPE

The Magnolia Bridge Bluff Structure is the section of mainline bridge located west of the 23rd Avenue ramp transition. This is a three lane structure that provides access to the Magnolia bluff neighborhood. The Bluff Structure is approximately 1800 feet long and has a total width of 58.6 feet. The bridge accommodates one eastbound lane, two westbound lanes, and a sidewalk on the south side of the structure. About half the length of the Bluff structure will be located on a series of horizontal curves with a 1200 foot radius. The curves are separated by a 75 foot long tangent section. An MSE wall structure will be utilized at the end of the structure where the bridge ties into the bluff. The new bridge will be located south of the existing bridge over vacant land, so there is flexibility in span lengths and pier locations. The structure in this area is located up to 100 feet above the existing ground, so there is no limitation on the depth of the superstructure. The superstructure will be supported on reinforced concrete single or two column bents. The bridge will likely be broken into two or three separate units depending on the preferred structure type. The foundation material improves along the length of the Bluff Structure. The depth to good material varies from 110 feet depth at the east end of the structure to just below ground surface about half way along the structure up to the Magnolia bluff.

The height of the bridge and the lack of site constraints provide the opportunity to consider multiple span arrangements and structure types. The Bluff Structure is highly visible since it will be located up to 100 feet above the ground. As a result, multiple structure type alternatives were evaluated for this section of the bridge, including both above-deck and below-deck structural systems.

The following is a list of the structure types investigated for the Magnolia Bluff Structure. The list of structures is separated based on whether the structure is an above-deck or below-deck alternative. A brief description of each alternative and its applicability for the Bluff Structure will be discussed in further detail in the following section.

Above-Deck Structure Alternatives

- Tied Arch Option
- Half-Through Arch Option
- Cable-Stayed Bridge – Two-Span Bridge Option
- Cable-Stayed Extradosed Bridge – Three-Span Bridge Option

Below-Deck Structure Alternatives

- Prestressed Concrete Girder
- Concrete Box Girder – Straight and Haunched Options
- Steel Plate and Steel Box Girder – Straight and Haunched Options
- Deck Arch – Full Height Option – With or Without Spandrel Columns
- Deck Arch – Partial Height Option – With or Without Spandrel Columns

Above-deck structure alternatives were evaluated for the Bluff Structure since this section of the bridge is the most visible on the Magnolia Bridge. An above-deck signature span structure at this location will be very prominent since the bridge structure is so high relative to the existing ground line.

The primary concern with building an above-deck structure is that the view from the bluff will be obstructed by the bridge structure. The Magnolia residents may want a less obtrusive structure that will not impact the views of downtown Seattle. The cost for an above-deck structure will also be more than the below-deck structure alternatives.

Several below-deck structure alternatives were evaluated for the Magnolia Bluff Structure. Although, these structure types are not as prominent as an above-deck structure, they have distinguishing aesthetic traits that would make a very prominent structure on the bluff.

The primary advantage of the below-deck alternatives is the decreased cost for the structures relative to the above-deck alternatives. The construction duration and complexity will also be reduced for most of these alternatives.

8.4.1 Tied Arch Option

Refer to Figure 39.

The tied arch bridge option is similar to the one described for the crossing over the BNSF Railroad. For this option, the tied arch superstructure will sit atop tall vertical columns at the deck level. Two different options were investigated including: a long single-span alternative at the highest point of the structure and a multi-span alternative with a series of tied-arch spans in succession near the highest portion of the bridge. A steel alternative is preferred for this structure type to facilitate ease of construction and also to reduce the weight of the superstructure for better seismic performance.

The tied arch is not recommended for this section of the bridge since a significant portion of the bluff structure is located on a curved alignment. Arch bridge structures are typically located on tangent alignments. It may be possible to construct straight arches with a curved roadway but it will increase the cost of the structure because of the additional width between arch ribs to curve the roadway. The height of the bridge structure in this section will likely complicate the erection of the arch spans and increase the cost as well. This option is also very obtrusive as much of the view from the Magnolia Bluff will be obstructed.

8.4.2 The Half-Through Arch Option

Refer to Figure 40.

The half-through arch option is similar to the described for the crossing over the BNSF Railroad. For this option, the half-through arch structure will be carried all the way to the ground. The foundations will be designed to take all the lateral loads from the arch. At this location, the foundation material is competent so the foundations can likely be designed to carry the lateral loads from the arch without increasing the cost significantly. A long signature span structure located at the highest point of the bridge was investigated for this option. The structure below the deck would likely be concrete and the arch structure above deck would be steel.

The half-through arch alternative is not recommended for this section of the bridge, for the same reasons that the tied arch option was not recommended.

The costs are going to be more than the tied arch bridge since the foundations will need to be designed for the lateral loads and there will be temporary shoring required to construct the arch below the deck.

8.4.3 Cable-Stayed Bridge Option

Refer to Figure 41.

The cable-stayed bridge option is similar to the one described for the crossing over the BNSF railroad. This alternative would provide a gateway, signature span bridge at the entrance to the Magnolia neighborhood. The cable-stayed bridge would be located at the end of the structure near the bluff, where the structure is most prominent. The remaining portion of the Bluff structure would be a different approach structure. A two span, symmetrical or asymmetrical, or a three span structure were investigated for this option. For this alternative, two cantilever towers at each pier are preferred over a single tower pier, since the bridge width is not very wide and the roadway has unbalanced lanes in each direction. The cable stayed bridge is shown with a 400-foot span and 80-foot towers. Actual span length and tower height would be optimized during alternative development.

The cable-stayed bridge option is not recommended for this section of the bridge because the tall towers and splayed cables will obstruct the view from Magnolia Bluff. This structure would be further complicated since a significant portion of the structure is located on a curved alignment. Cable-stayed structures are typically located on tangent alignments, and although it is feasible to construct a cable-stayed bridge on a curve, it will increase the cost of the structure.

8.4.4 Cable-Stayed Extradosed Bridge Option

Refer to Figure 42.

The cable-stayed extradosed bridge option is similar to the one described for the crossing over the BNSF railroad. This alternative would provide a gateway, signature span bridge at the entrance to the Magnolia neighborhood. The cable-stayed extradosed bridge would be located at the end of the structure near the bluff, where the structure is most prominent. The remaining portion of the Magnolia Bluff bridge structure would be a different approach structure. A symmetrical three-span layout would be the preferred configuration for an extradosed bridge at this location. For this alternative, the piers would have two cantilever towers supporting the superstructure. The extradosed cable stayed bridge is shown with a 400 foot span and 40 foot towers. Actual span length and tower height would be optimized during alternative development.

The cable-stayed extradosed bridge is not recommended for the Bluff Structure because of the tower heights and the possibility of blocking views from Magnolia Bluff. The curved alignment will complicate the design, details and construction of the bluff structure. Cable-stayed structures are typically located on tangent alignments, and although it is feasible to construct a cable-stayed bridge on a curve, it will increase the cost of the structure.

8.4.5 Prestressed Concrete Girder

Refer to Figure 43.

Prestressed concrete girders are a recommended alternative for the Magnolia Bluff Structure. In this location the span lengths should be maximized to reduce the total number of foundations. Longer span lengths are also going to be more aesthetic, since the structure is so high above the ground.

The span length is limited to approximately 160 feet in the area of the bridge located on the 1,200 foot horizontal curve to accommodate a straight girder with a curved deck. Beyond this section of the structure, the span length could be increased to a more optimum span length. For aesthetic reasons, it is recommended that the span length should be limited to 160 feet for the entire structure, since the tallest portion of the bridge is located on the curves. Typically longer span lengths are used in the taller sections of a bridge structure. Longer spans in the lower sections of the bridge will not be aesthetic.

Recommend using either a WSDOT WF74G or W83G I-Girder or a post-tensioned U78PTG5 tub girder in this location.

The post-tensioned concrete tub girders are expected to cost more than the I-girders since temporary shoring would be required to support the individual sections until post-tensioning is applied and the deck is poured.

8.4.6 Concrete Box Girder

Refer to Figure 44 and Figure 45.

A cast-in-place post-tensioned concrete box girder is a recommended alternative for the Bluff Structure. The concrete box girder structure is ideal for the curved alignment that occurs in this section of the bridge. With a concrete box girder there is a lot of flexibility in the span arrangements for the structure in this location. The span lengths can be maximized, up to 300 plus feet, to take full advantage of the unlimited structure depth.

Both a straight box and a haunched box are feasible alternatives. The haunched box would add a very smooth, curved shape to the underside of the structure. The piers could be detailed to merge with the curve and shape of the haunched box girder to produce a continuous, blended look with the superstructure. A haunched girder alternative would likely cost approximately 10 percent more than the straight girder option

The primary disadvantage of a cast-in-place concrete box girder is the need for temporary falsework to support the structure during construction. This will be more complicated for this section of bridge since the structure height is over 100 feet. The temporary falsework will be expensive and will require temporary substructure to support the superstructure during construction.

Because this section of the bridge is over 1,800 feet long and the structure width is constant throughout, segmental construction should be investigated to determine if this is a cost effective construction option.

8.4.7 Steel Girders

Refer to Figure 44 and Figure 45.

A steel plate girder or steel box girder structure is a recommended alternative for this structure. The steel girder bridge would utilize multiple girders and the box girder would be either a two or three box structure. Both the plate girder and box girder structures are ideal for the curved alignment that occurs in this section of the bridge. With these structures there is a lot of flexibility in the span arrangements for the structure in this location. The span lengths can be maximized to take full advantage of the unlimited structure depth.

Straight and haunched sections are feasible alternatives for either the plate girder or box girder alternative. A haunched I-girder or box girder would add a very smooth, curved shape to the underside of the structure. A haunched girder alternative would likely cost approximately 10 percent more than the straight girder option

The girders could be erected from the ground in this location. Temporary shoring would be required in some locations, but these can be located in those areas where they will not impact traffic movement.

The primary disadvantages of this structure alternative are the cost of steel and the longterm maintenance required after the structure is built. Some of the steel cost may be offset by the reduced foundation size and cost, as a result of better seismic performance of steel girders due to reduced weight of steel structures.

8.4.8 Deck Arch – Full Height Alternative

Refer to Figure 46.

The Full Height Deck Arch Alternative is a series of arch spans running from the 23rd Avenue Ramp Transition to the Magnolia Bluff. This alternative is a true arch alternative that runs from the deck level to the ground line. Because of the variable height profile grade, each arch will have its own distinct shape. The span lengths and arrangement can be varied to alter the appearance of the arch structures. Two separate cast-in-place concrete arches will support the bridge superstructure.

The superstructure can be designed to span between the piers and the midpoint of the arch or spandrel columns can be added to reduce the superstructure depth. In either case full height columns will be required at each pier location. The decision on whether to include spandrel columns is largely dependent on the preferred aesthetic appearance of the structure.

The primary disadvantage of this structure will be the high cost for construction. The arch structures would likely have to be built on temporary shoring structures and each span will be different, so duplication of forms would be difficult. The construction will also be complicated since the bridge is located on a horizontal curve which will increase the construction cost.

From an aesthetic standpoint, true arch structures are typically used to span large gaps or obstacles, in this case the Bluff Structure does not span any particular obstacle.

Because of cost, constructability and applicability, the Full Height Deck Arch alternative is not recommended for this section of the bridge.

8.4.9 Deck Arch – Partial Height Alternative

Refer to Figure 47.

The Partial Height Deck Arch Alternative is a hybrid of the haunched box girder alternative and the Full Height Deck Arch alternative where a series of shallow arch spans will run from the 23rd Avenue Ramp Transition to the Magnolia Bluff. This alternative is also a true arch alternative, but the arch is shallower and only goes part of the way to the ground. The arch depth will be the same for all spans and will follow the profile grade. The arches will be supported on vertical columns at the pier locations. The span lengths and arrangement can be varied to alter the appearance of the arch structures. Two separate cast-in-place concrete arches will support the bridge superstructure.

The superstructure can be designed to span between the piers and the midpoint of the arch or spandrel columns can be added to reduce the superstructure depth. The superstructure can be either cast in place concrete box or precast girders. Full height columns will be required at each pier location. The decision on whether to include spandrel columns is dependent on the preferred aesthetic appearance of the structure, constructability and cost implications.

The primary disadvantage of this structure will be the high cost for construction. The arch structures would likely have to be built on temporary shoring structures. The construction will be complicated by the horizontal curves which will increase the construction cost.

The Partial Height Deck Arch is a recommended option for a below-deck bridge for the Magnolia Bluff Structure. This option is preferred over the full height deck arch since the construction should be less complicated and the cost decreased. This will also be a very aesthetic option for the structure.

8.4.10 Bluff structure Evaluation Matrix

The nine structure types evaluated for this section of bridge will include: Tied Arch, Half-Through Arch, Cable-Stayed Bridge, Cable-Stayed Extradosed Bridge, Prestressed Concrete Girder, Straight or Haunched Concrete Box Girder, Straight or Haunched Steel Plate Girder, Straight or Haunched Steel Box Girder, Full Height Deck Arch and Partial Height Deck Arch. The cost estimates were determined using base structure costs based on square foot costs of similar structures from August, 2004 unit prices.

Evaluation meetings were held at SDOT and with the Design Advisory Group to decide which alternatives to advance to Bridge Alternative Study phase. The selected alternatives are identified in the evaluation matrix Table 16.

For an elevation view of the Bluff Structure options, refer to Figure 39 thru Figure 47.

Table 17
Bluff Structure Evaluation Matrix

Structure Type	Bridge Base Cost (2004 $)	Advantages	Disadvantages
Steel Tied Arch with Straight Concrete Box Girder Attachment 10	$28,000,000	• Aesthetically pleasing, signature span bridge, gateway structure	▪ Higher cost ▪ Long construction duration ▪ Higher maintenance/Life cycle cost ▪ Difficult to erect and place in final position ▪ Very difficult on curved alignments • Tall structure, obstructed view of Seattle from Magnolia Bluff
Half-Through Arch with Straight Concrete Box Girder Attachment 11	$28,000,000	• Aesthetically pleasing, signature span bridge, gateway structure	▪ Higher cost ▪ Long construction duration ▪ High maintenance/Life cycle cost for steel superstructure ▪ Difficult to erect and place in final position ▪ Temporary shoring required to construct arch below deck ▪ Very difficult on curved alignments • Tall structure, obstructed view of Seattle from Magnolia Bluff
Two-Span Cable Stayed Bridge with Straight Concrete Box Girder Attachment 12	$40,000,000	• Aesthetically pleasing, signature span bridge, gateway structure	▪ Higher cost ▪ Long construction duration ▪ Typically located on straight alignments ▪ High maintenance/Life cycle costs for steel superstructure • Tall towers, obstructed view of Seattle from Magnolia Bluff
Three-Span Cable Stayed Extradosed Bridge with Straight Concrete	$40,000,000	▪ Aesthetically pleasing, signature span bridge, gateway structure ▪ One of only a few bridges of this type in	▪ Higher cost ▪ Long construction duration ▪ Typically located on straight alignments ▪ High

Structure Type	Bridge Base Cost (2004 $)	Advantages	Disadvantages
Box Girder Attachment 13		the United States • Low Maintenance	maintenance/Life cycle costs for steel superstructure • Tall towers, obstructed view of Seattle from Magnolia Bluff
Prestressed Concrete I-Girders Attachment 14 (Advanced to Bridge Alternative Development Phase)	$21,000,000	▪ Low cost ▪ Straightforward to construct, no temporary shoring ▪ Short construction duration ▪ Low maintenance • Minimal environmental permitting required for construction	▪ Span limitations on curved alignments ▪ Less aesthetically pleasing ▪ Short spans on a high level structure • Higher foundation costs
Straight or Haunched Concrete Box Girder Attachment 15 – Straight Attachment 16 - Haunched (Advanced to Bridge Alternative Development Phase)	$21,000,000 Straight $23,000,000 Haunched	▪ Average cost ▪ Accommodates curved alignments well ▪ Low maintenance ▪ Aesthetically pleasing ▪ Architectural options available • Long span capability	▪ Long construction duration ▪ Requires temporary shoring or segmental construction ▪ Increased environmental permitting required for construction • Increased Port of Seattle impacts during construction
Straight or Haunched Steel Plate Girder or Steel Box Girder Attachment 15 – Straight Attachment 16 – Haunched (Advanced to Bridge	$21,000,000 Straight $23,000,000 Haunched	▪ Average cost ▪ Handles curved alignments well ▪ Aesthetically pleasing ▪ Short construction duration ▪ Minimal temporary shoring ▪ Minimal environmental permitting required for construction ▪ Long span capability	• High maintenance/Life cycle costs

Structure Type	Bridge Base Cost (2004 $)	Advantages	Disadvantages
Alternative Development Phase)		▪ Lower seismic loads due to lower mass; Reduced foundations	
Full-Height Concrete Deck Arch with Straight Concrete Box Girder Attachment 17	$58,000,000	▪ Aesthetically pleasing • Low maintenance	▪ Higher cost ▪ Difficult to construct on a curve ▪ Requires temporary shoring ▪ Not typically used for a viaduct structure ▪ Long construction duration • Variable arch dimensions
Partial-Height Concrete Deck Arch with Straight Concrete Box Girder Attachment 18	$42,000,000	▪ Aesthetically pleasing • Low maintenance	▪ Higher cost ▪ Difficult to construct on a curve ▪ Requires temporary shoring ▪ Long construction duration •

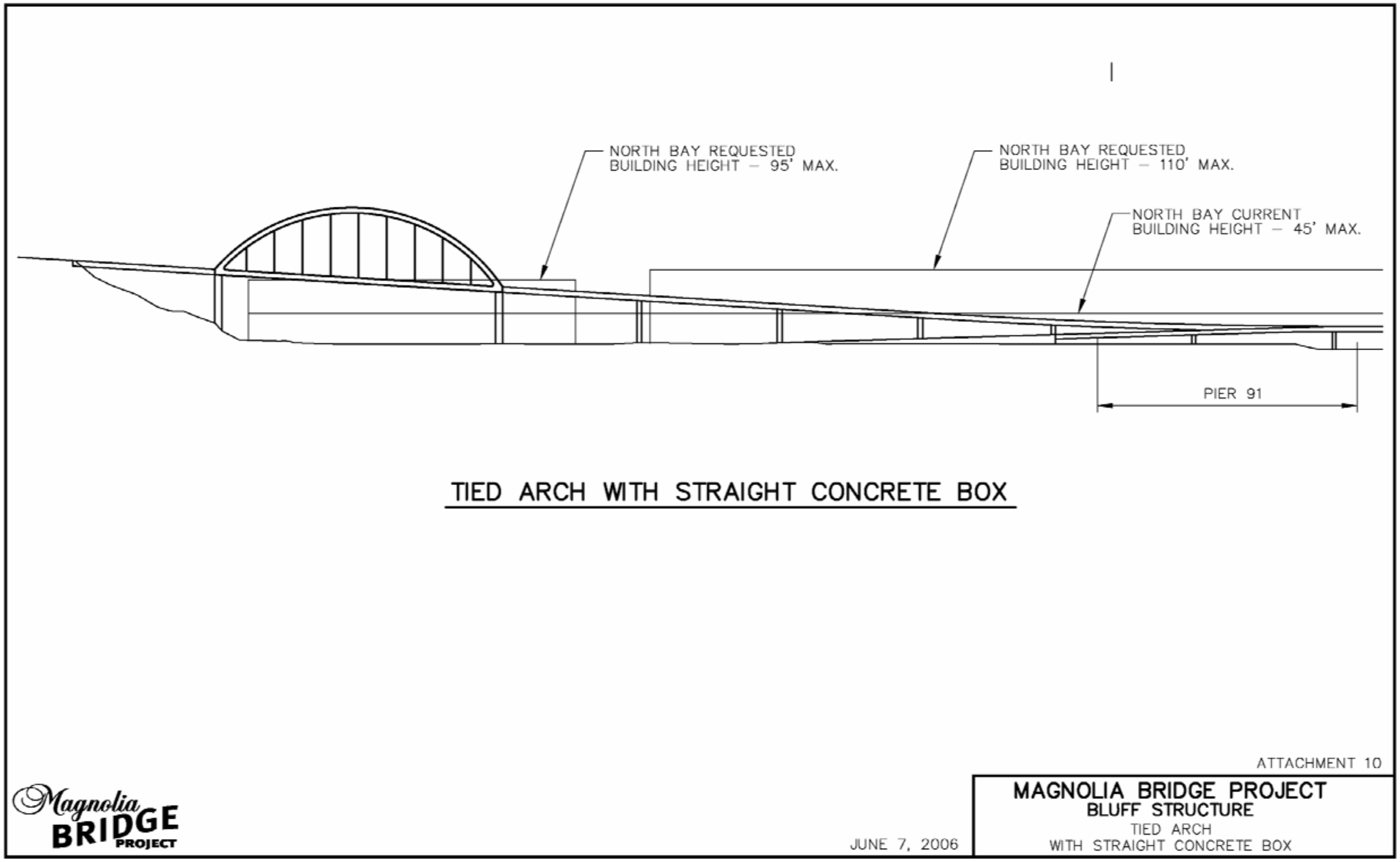


Figure 39
Bluff Structure
With Straight Concrete Box

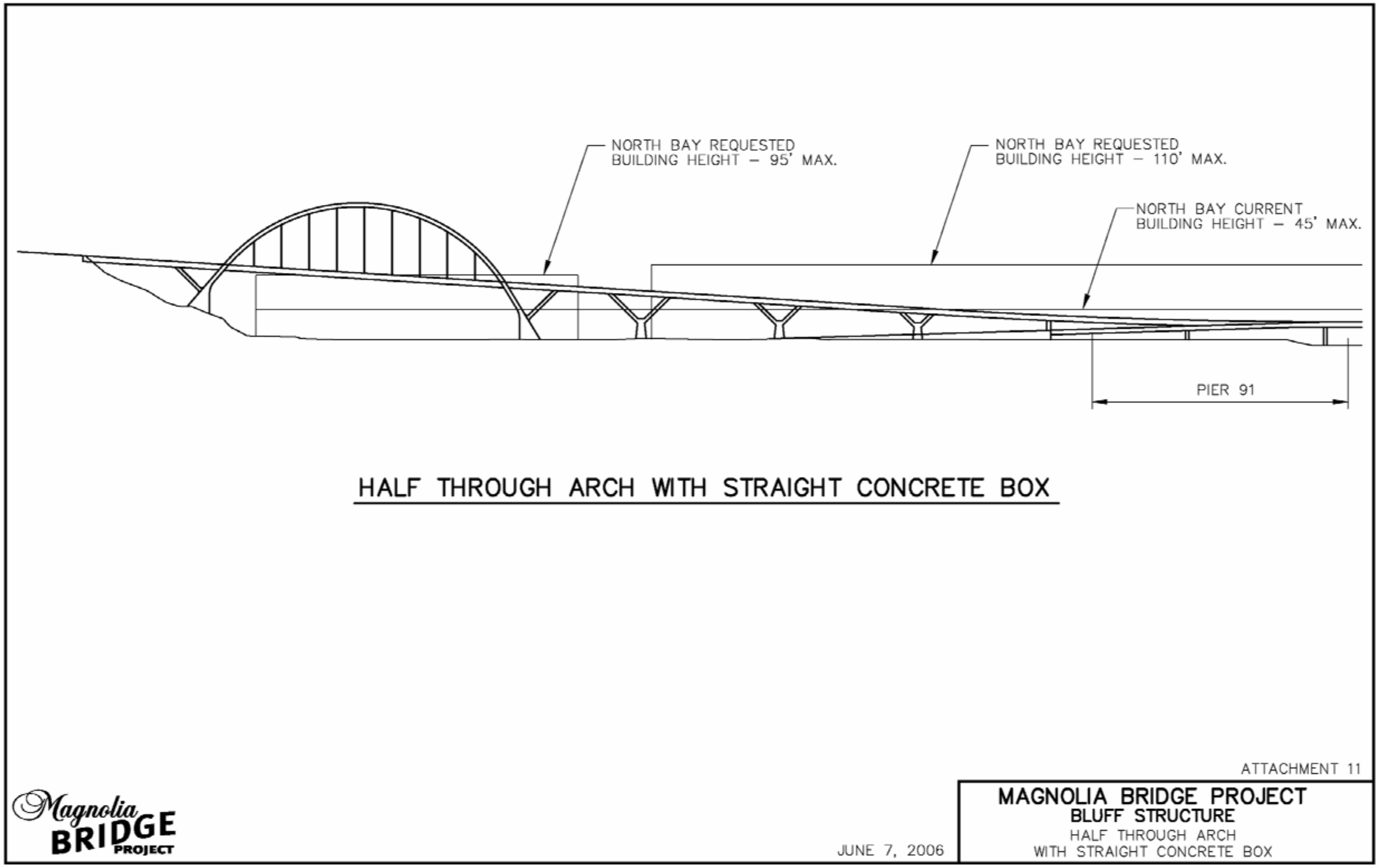


Figure 40
Bluff Structure
Half Through Arch with Straight Concrete Box

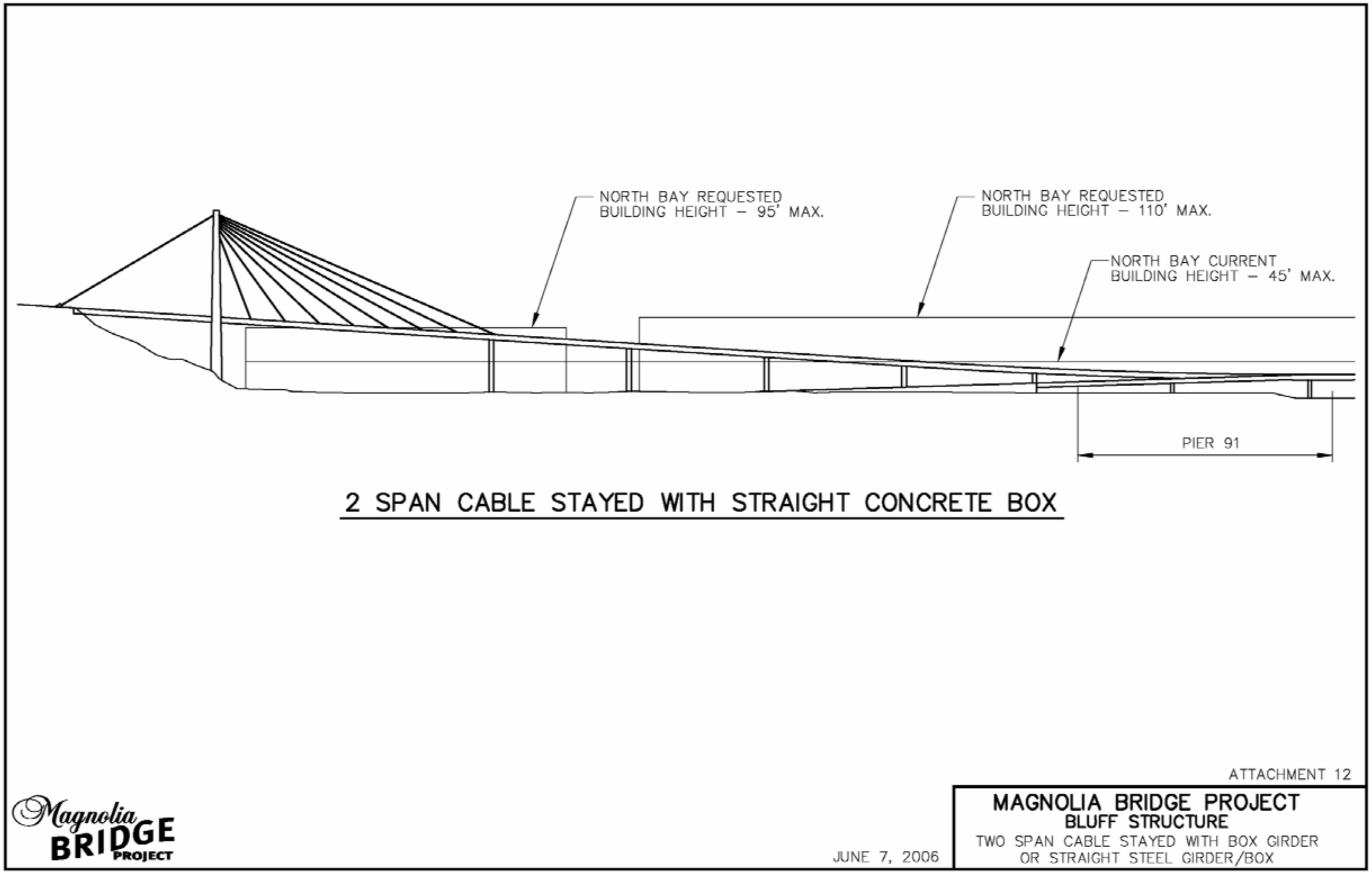


Figure 41
Bluff Structure
Two Span Cable Stayed with Box Girder or Straight Steel Girder/Box

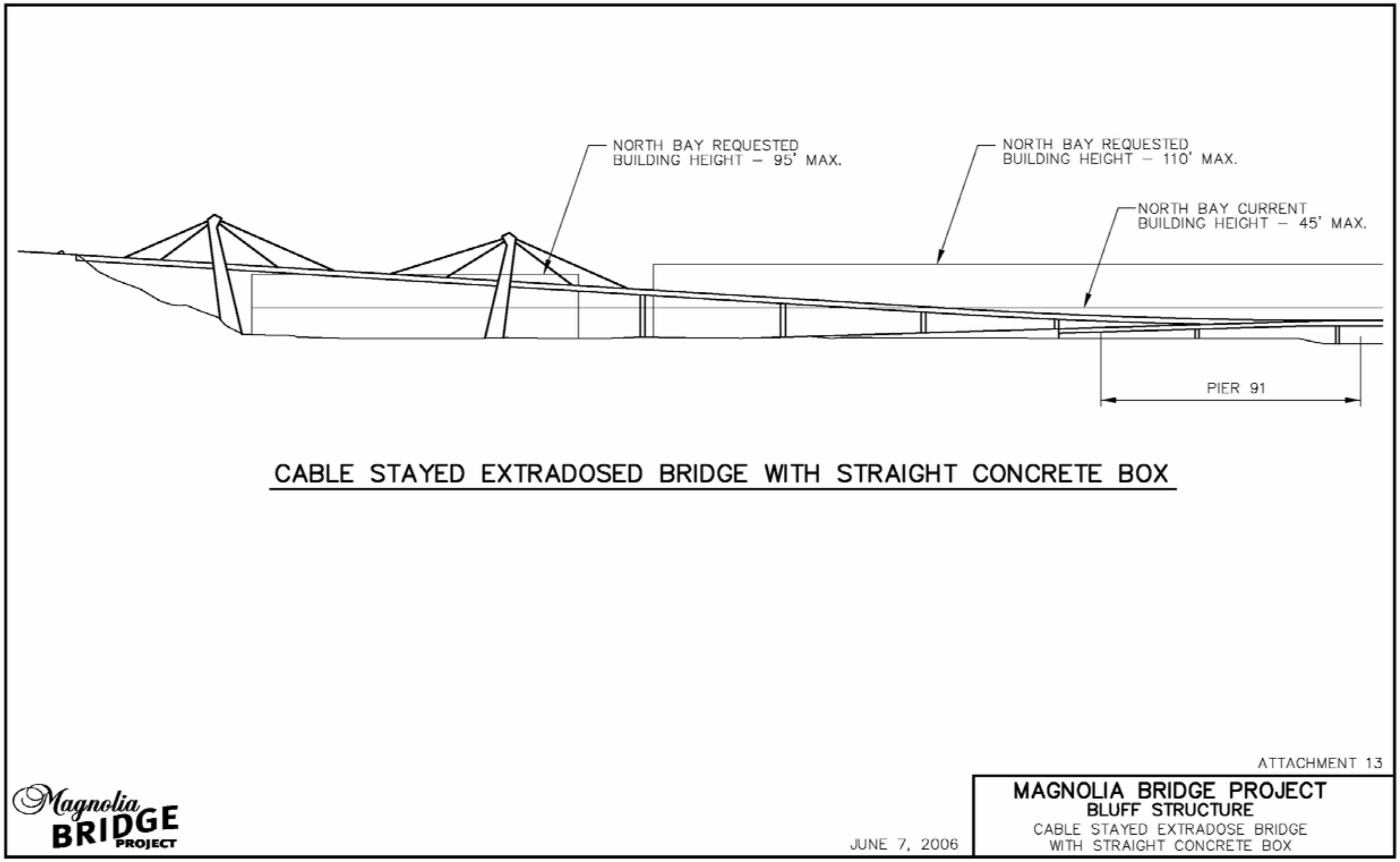


Figure 42
Bluff Structure
Cable Stayed Extradose Bridge with Straight Concrete Box

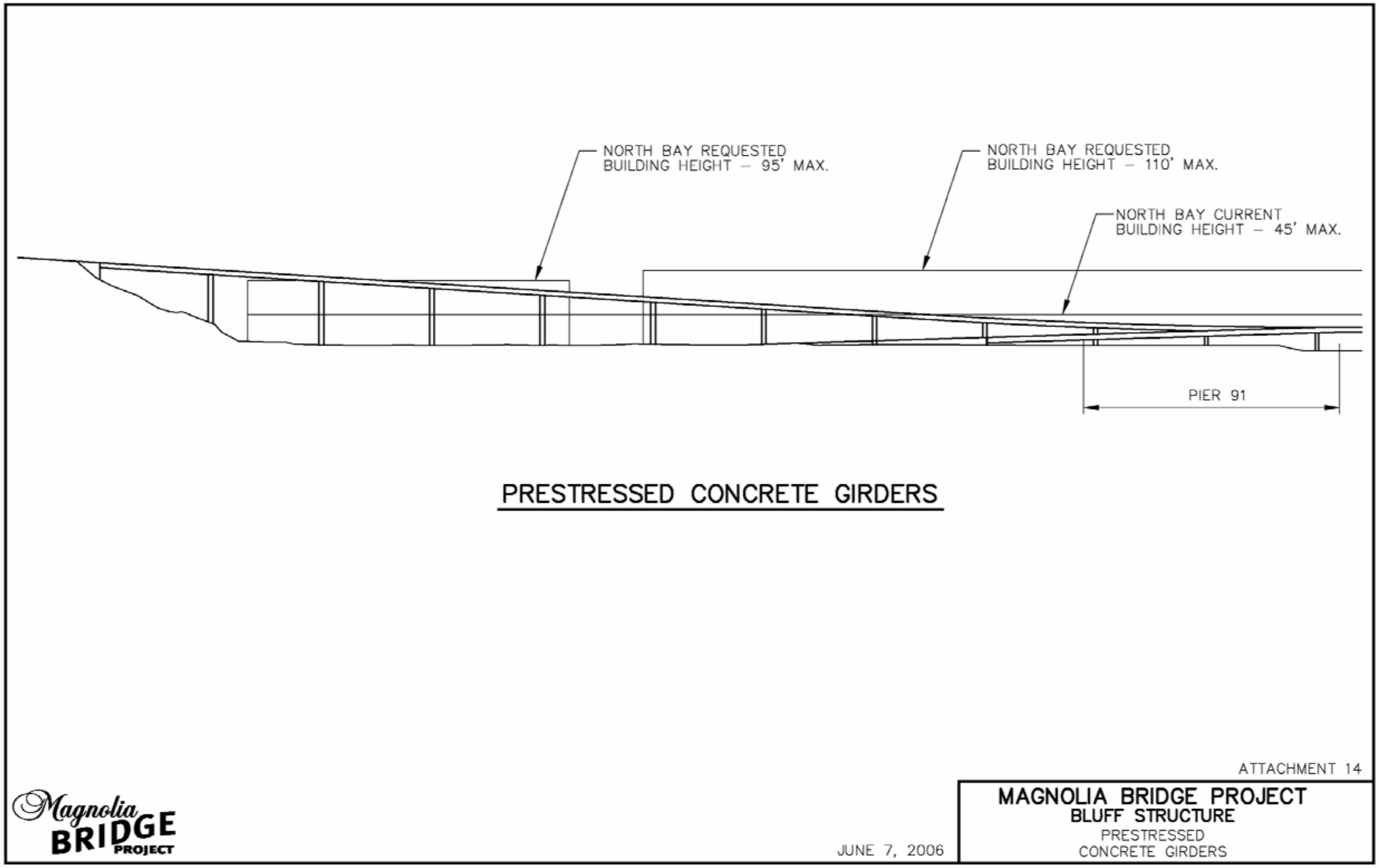


Figure 43
Bluff Structure
Prestressed Concrete Girders

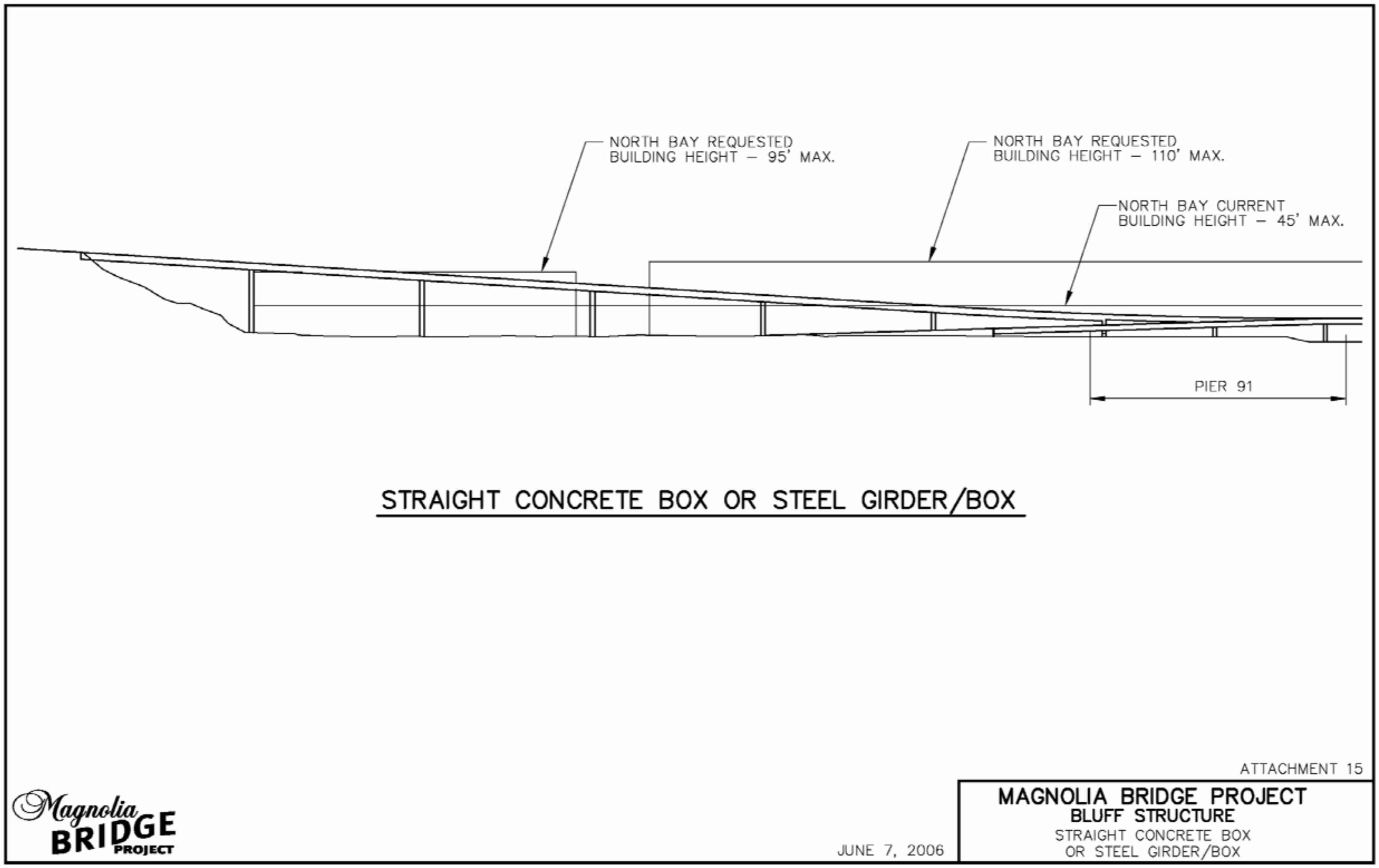


Figure 44
Bluff Structure
Straight Concrete Box or Steel Girder/Box

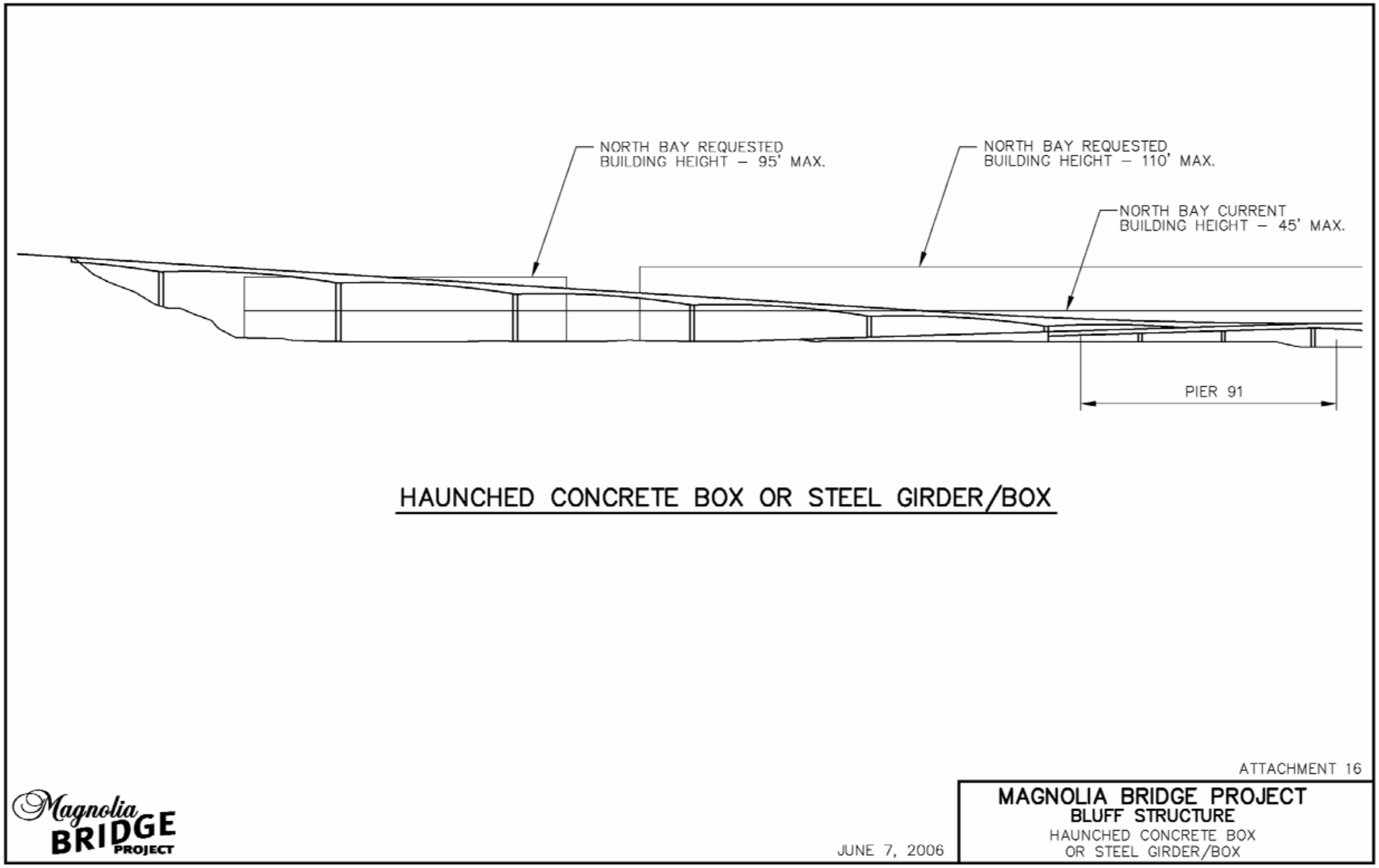


Figure 45
Bluff Structure
Haunched Concrete Box or Steel Girder/Box

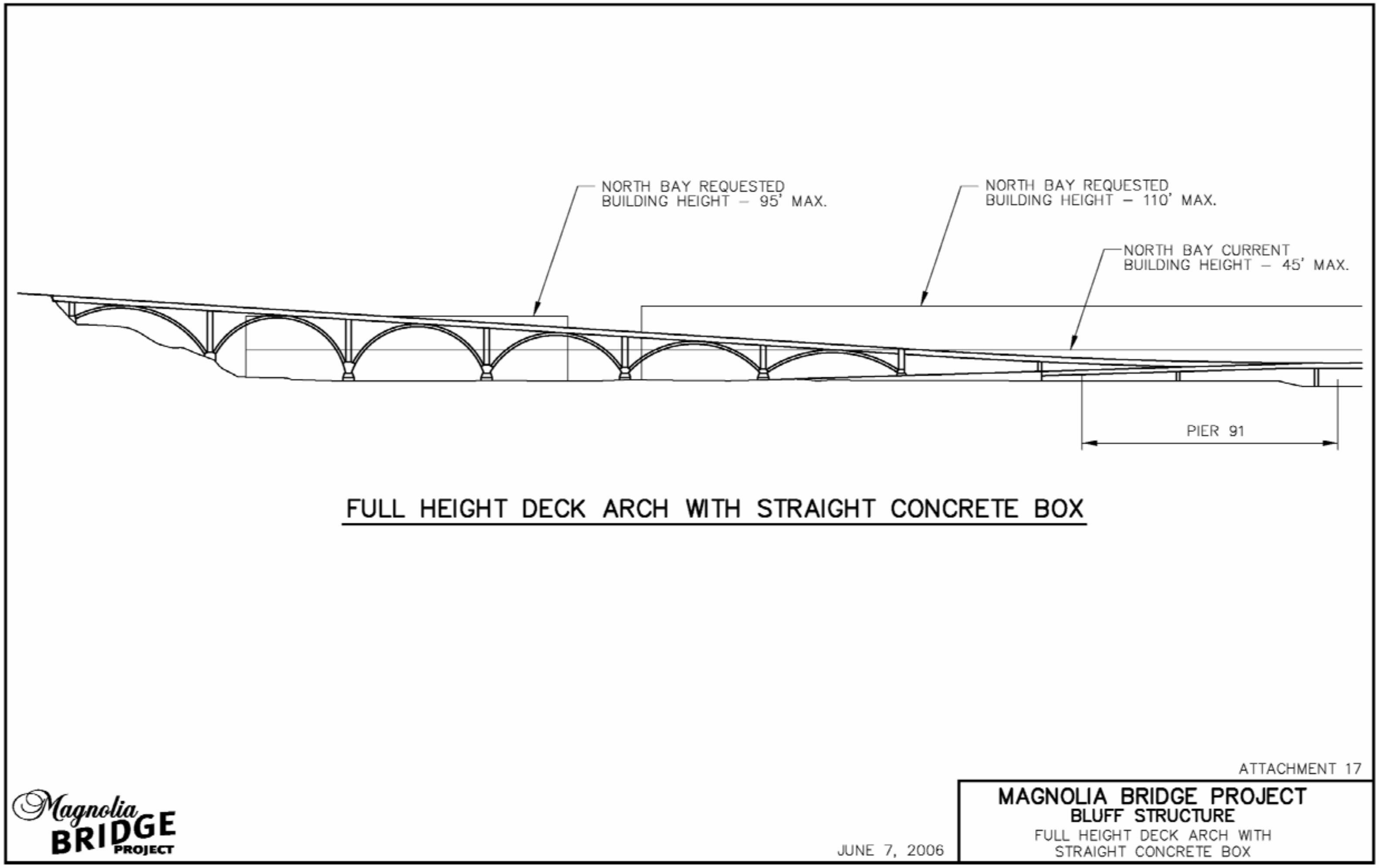


Figure 46
Bluff Structure
Full Height Deck Arch with Straight Concrete Box

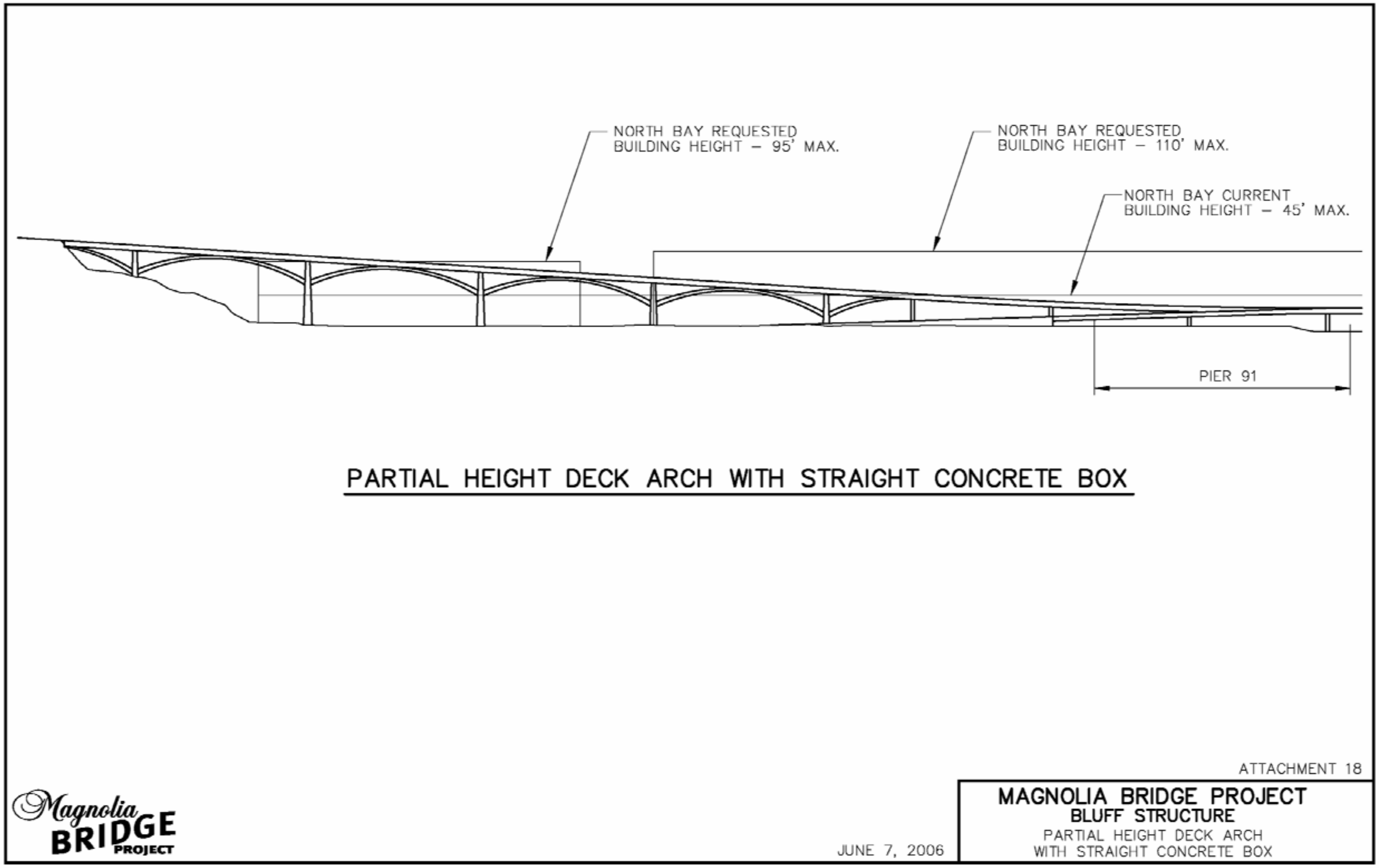


Figure 47
Bluff Structure
Partial Height Deck Arch with Straight Concrete Box

8.5 BRIDGE CONCEPT STUDY CONCLUSIONS

The three alternatives for selected for advancement to the Bridge Alternative Study were prestressed girder, steel girder and cast-in-place concrete box. They were selected because they were the least cost options while meeting project requirements. It was decided based on public and community input that a signature structure such as cable stayed or arch was not appropriate at this site. The three structure types are applicable for all sections of the bridge, including 15th Avenue Overpass, BNSF railway crossing, Low Level Mainline and Bluff Structure. The three alternatives were advanced through the Bridge Alternative Study which includes preliminary design and analysis to determine span arrangement, structure depth, foundations, aesthetics, public feedback, quantities and costs. A complete set of plan and elevation drawings was prepared for each of the selected bridge types, see Appendix D. The preferred structure type was selected from the three bridge types at the completion of the Bridge Alternative Study.

9. FOUNDATION STUDY

9.1 GROUND IMPROVEMENTS

As part of the construction of the new Magnolia Bridge, ground improvements would be necessary to mitigate liquefaction and lateral spreading. The ground improvements would primarily be located between 15th Avenue West and existing Bent 60. Ground improvement depth would likely vary between 20 feet, near Bent 60, to 50 feet, near the center ramp, based on the current borings. The geotechnical consultant, Shannon & Wilson, has recommended either stone columns or compaction grouting as feasible options for improving the soil properties. Ground improvements would be required at each pier location, with the improvements extending approximately 40 feet outside of the footing plan for stone columns and 30 feet outside of the footing plan for compaction grouting.

The ground improvements could be done before or after foundation construction. If necessary, the ground improvements could be completed in stages and the foundations would be designed for a temporary condition until all ground improvements were completed.

There are advantages and disadvantages for both stone columns and compaction grouting. The decision to use either of the ground improvement methods is dependent on local soil conditions and project requirements. A brief description of each method is provided in the following sections.

9.1.1 Stone Columns

Stone columns can be installed to a depth in excess of 100 feet. The cost of installing stone columns is approximately $15 per cubic yard of improved material, based on August, 2006 estimated unit prices.

Some of the disadvantages using stone columns for ground improvement include: large vibrations during installation; limited to vertical installation only; difficult to install in dense beach deposits which may necessitate pre-drilling the hole; and in-water construction is complicated due to beach deposits and rip-rap. Shannon and Wilson recommends that no stone columns be installed within 30 feet of the existing bridge structure in order to prevent vibrations that may cause settlement of the existing bridge. This also applies for any foundations which are located near any utility lines, especially high pressure water lines. At these locations the vibrations may cause permanent damage or failure of the utility line.

Stone columns are preferred in those locations where vibrations are not a major concern during construction. The construction staging would affect how much of the project can use stone columns because the maintenance of traffic on the existing structure will limit where stone columns are applicable.

9.1.2 Compaction Grouting

Compaction grouting can also be used for depths in excess of 100 feet. The cost of compaction grouting is approximately $60 per cubic yard of improved material, based on August, 2006 estimated unit prices.

Some of the advantages of using compaction grouting for ground improvement include: minimal vibrations during installation; ability to be installed at an angle and low overhead applications; installation around existing foundations; and installation adjacent to existing utilities.

Compaction grouting is preferred in those locations where vibrations are a major concern during construction, such as near utilities or the existing bridge.

9.1.3 Ground Improvement Conclusion

Compaction grouting was assumed for cost estimating purposes at this stage of the project because of proximity to the existing bridge and existing utilities. The possibility of using stone columns will be investigated during final design.

9.2 SHAFTS / PILES

9.2.1 Drilled Shafts

An oscillator or rotator method is recommended to construct the drilled shafts because the rotator method will cut through obstructions and boulders. An oscillator will cut through some obstructions but boulders would have to be removed through the shaft. It is expected drilled shafts will be embedded about 25’ into the competent glacial material. Drilled shafts can be installed as close to existing structure as desired without damage.

9.2.2 Driven Piles

Typical driven piles are open or closed end steel pipe piles. The closed end driven pipe pile will not likely be able to penetrate the beach sand layer. Therefore the pipe should be driven open ended. Driving the pipe piles open ended will require additional embedment into the glacial deposits, about 40 feet to 50 feet into glacial deposits to get the 200 ton allowable.

There will be considerable vibrations and noise as a result of pile driving. Driving piles may need a noise variance. The typical pile driving is only allowed from 7:00 am to 10:00 pm in a populated area. Because of the vibrations, pile driving will likely not be allowed within 25 feet of existing bridge or utilities.

9.2.3 Shafts / Piles Conclusion

Drilled shafts were selected as the preferred foundation because they can be installed adjacent to existing structure and utilities, do not generate significant noise during construction, will accommodate underground obstructions, have a minimum footprint requiring less ground improvement, do not require cofferdams or dewatering, and will generate less contaminated material during construction.

10. AESTHETIC STUDIES

The bridge architecture and site element alternatives were studied and developed using a context sensitive design approach and assuring the project "fits" with the surrounding environment and is compatible with adjacent neighborhoods/communities. With this approach the architectural themes were developed into four categories, including "Baseline", "Historic", "Maritime" and "Progressive" as indicated below:

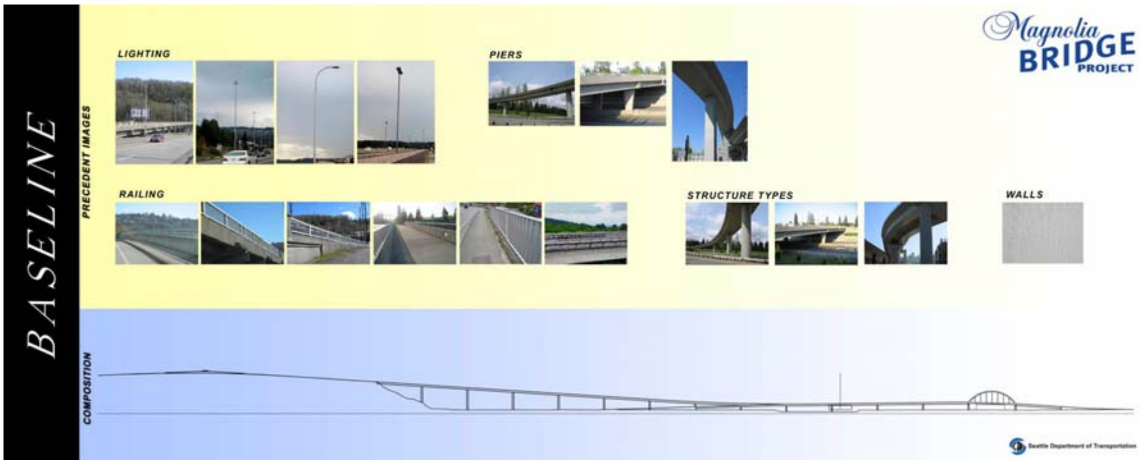


Figure 48
Architectural Theme - Baseline

The "Baseline" theme, Figure 48 reflects existing architectural styles and elements that are commonly utilized on structures within the Puget Sound area.

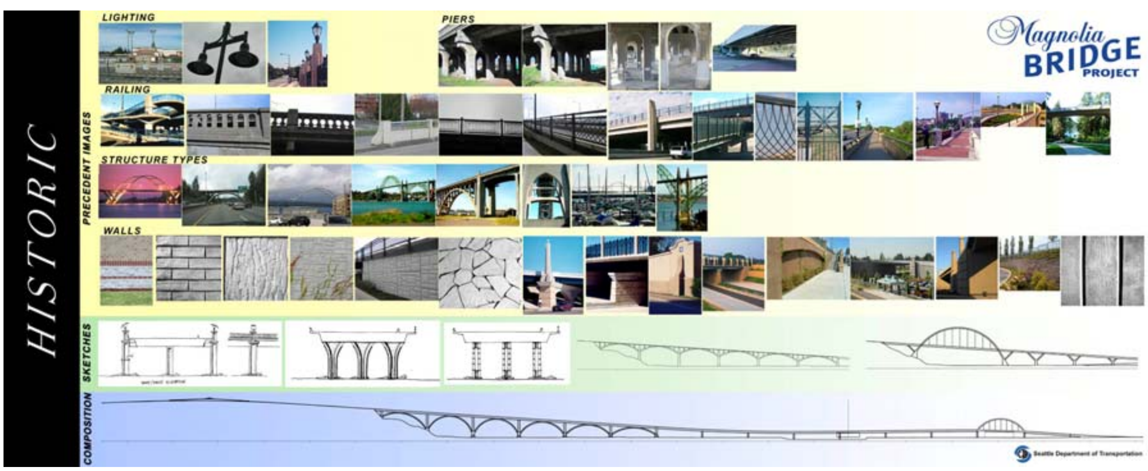


Figure 49
Architectural Theme - Historic

The "Historic" theme, Figure 49 represented the historic appearing elements on the existing bridge, such as the arched piers and the balustrade railings on the upper deck.

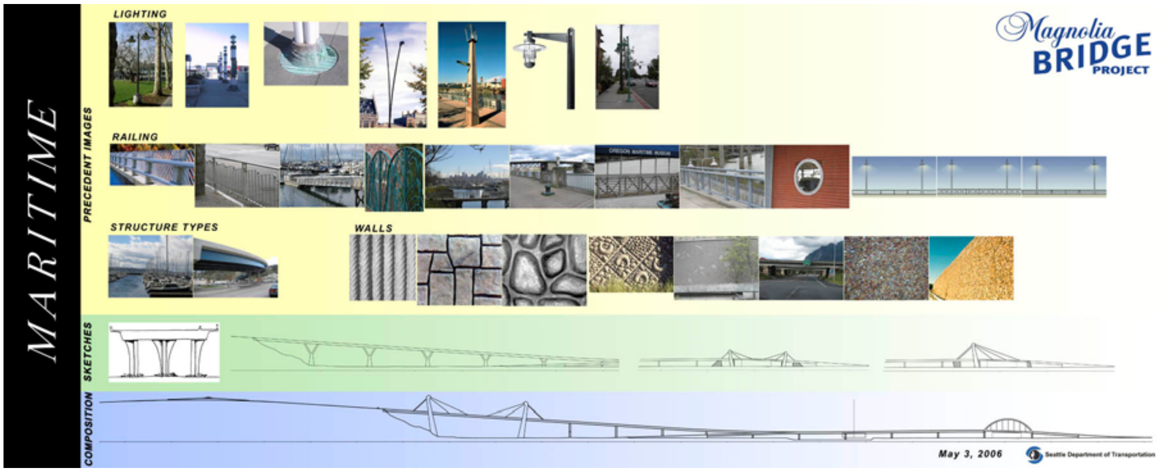


Figure 50
Architectural Theme - Maritime

The "Maritime" theme, Figure 50 reflects the nature of the adjacent marina and the historic nature of the shipping industry adjacent to the existing structure.

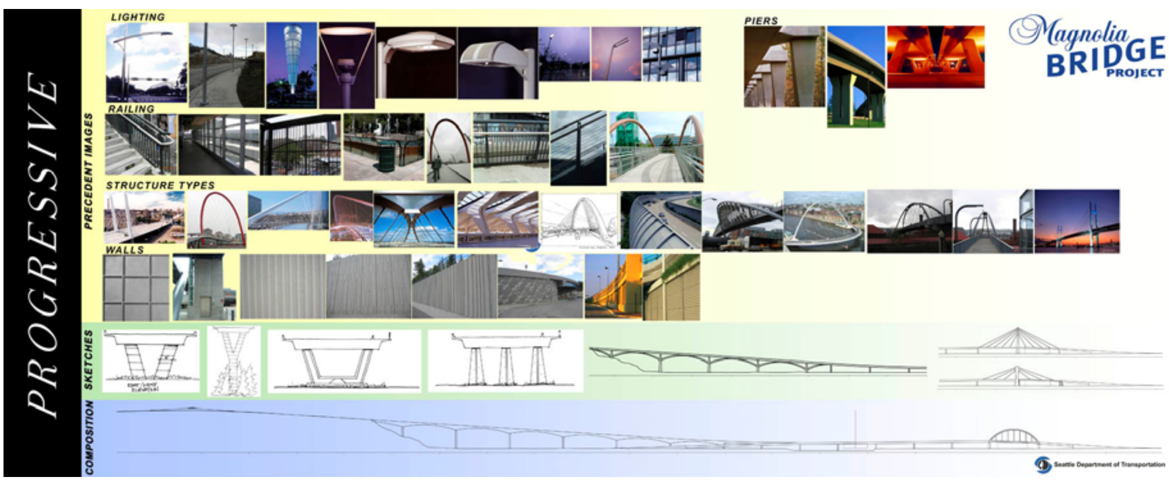


Figure 51
Architectural Theme - Progressive

The "Progressive" theme, Figure 51 indicates a timeless architectural style and anticipated future development of the North Bay area.

Each of these architectural styles were presented to the Design Advisory Group and determined that the "Maritime" or "Progressive" style of architectural style was preferred.

Each component of the bridge and site was studied separately and evaluated against a set of architectural criteria for compatibility with the architectural themes and other components of the structure. The components were broken into the following groups, including: pier studies, structure types, railings, lighting (both roadway and pedestrian), paving, barriers, overlooks, and pedestrian access ramps. The architectural criteria, was developed in a matrix format for purposes of evaluation and included the following:

Architectural Element Criteria

- Cost
- Aesthetics
    1. Structure compatibility/material relationships
    2. Context with existing and future development
    3. Level of Excitement
    4. Color, Texture, Materials
    5. Theme compatibility (Marriage of Maritime and Progressive)
- Maintenance-
    1. Life Cycle Cost
    2. Ease of maintenance/repair
- Constructability
- Form
    1. Timelessness
    2. Hierarchy
    3. Scale
- Design Performance

10.1 PIER STUDIES

Approximately 50 pier options were developed initially, and screened against the criteria. Several options were discarded due to structural inefficiency or incompatibility with the structure type. However, the studies below indicate the options that were taken further in the evaluation process and fell into five categories, see Figure 52 and Figure 53 including: “Flared Top”, “Angular Flare”, “Tapered Top”, “Tapered Double Columns” and “Straight Double Columns”. Each of these options, were evaluated and screened by identifying the advantages, disadvantages and order of magnitude costs of each short list to 3, see Figure 54.

Each of these options, were presented to the Design Advisory Group, Public Open House and the Seattle Design Commission. Based upon comprehensive comments from all stakeholders, it was determined that the preferred alternative is the single curved flared-top pier. This decision was based upon the compatibility with the structure types, efficiency in cost, flexibility with the various levels/heights of the structure, and opportunities to add various textures, colors, and enhancements.

Architectural form-liner alternatives were studied for each pier type. The preferred alternative is of a tight rope pattern, which will give architectural relief to the piers, provides continuity of the architectural style of the structure and serves as a deterrent to vandalism or graffiti. The pattern was determined to be cost effective, compatible with the preferred themes and easily maintainable, based upon feedback from the Design Advisory, Public Open House and Seattle Design Commission.

Several colors were evaluated for both the piers and the structure. The color application is a pigmented sealer that is applied to the concrete finish when cured. The pigmented sealer is cost effective, easily maintainable, allows for easy re-painting if graffiti is applied and adds visual quality to the structure, while reduces the effect of bulk of the structure when seen from below. A particular color was not chosen during the initial phase of design. However, a warm taupe color was preferred.

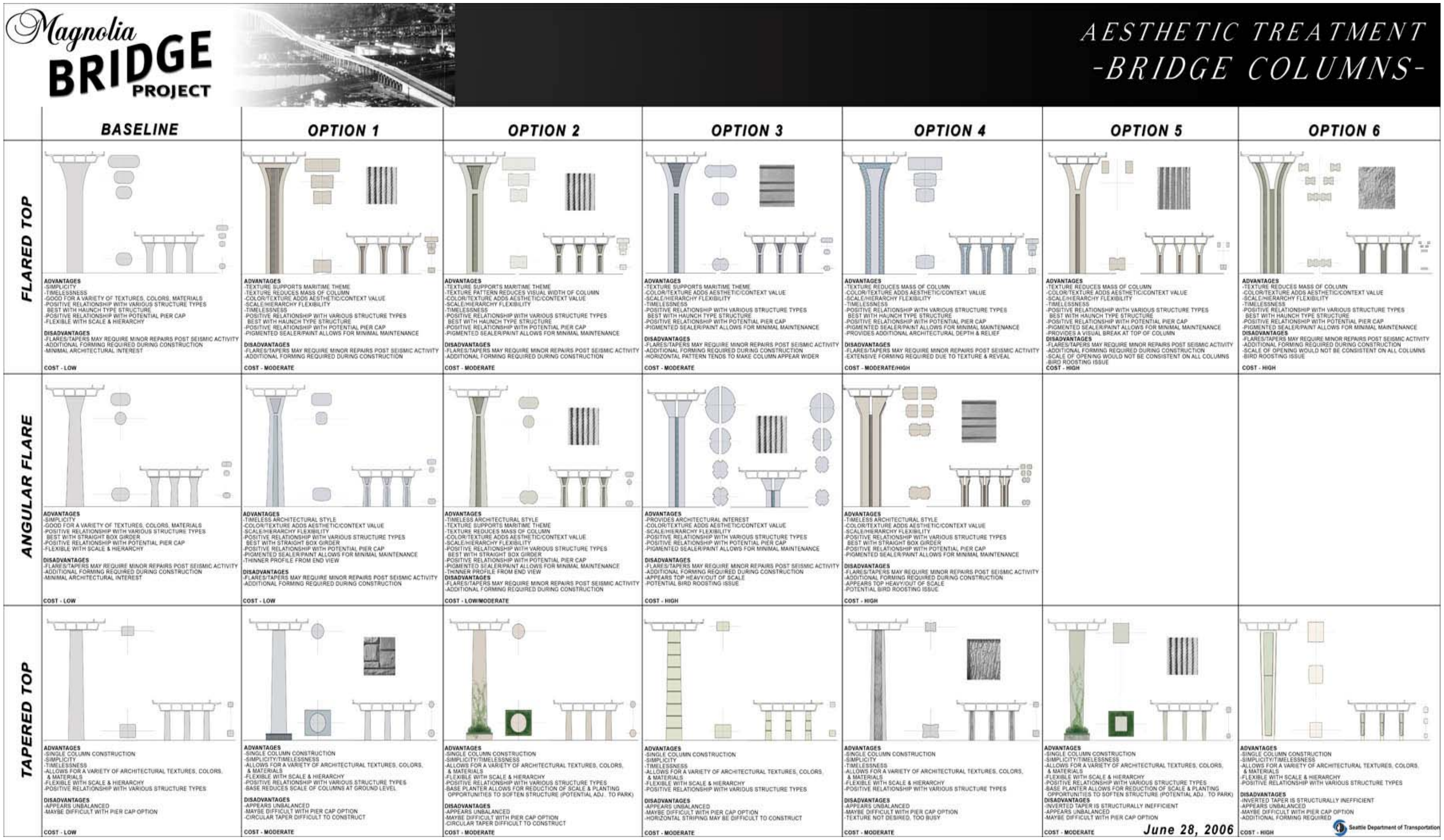


Figure 52
Aesthetic Treatment – Bridge Columns

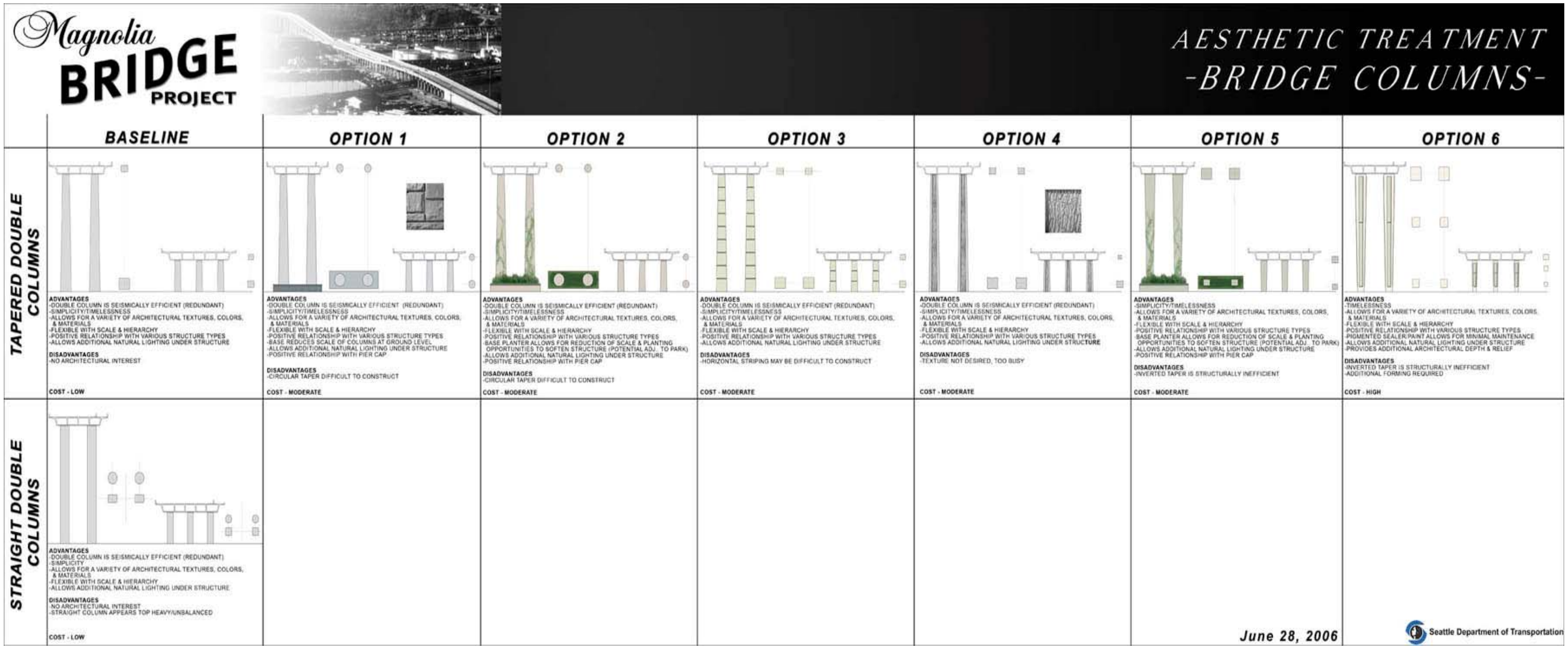


Figure 53
Aesthetic Treatment – Bridge Columns

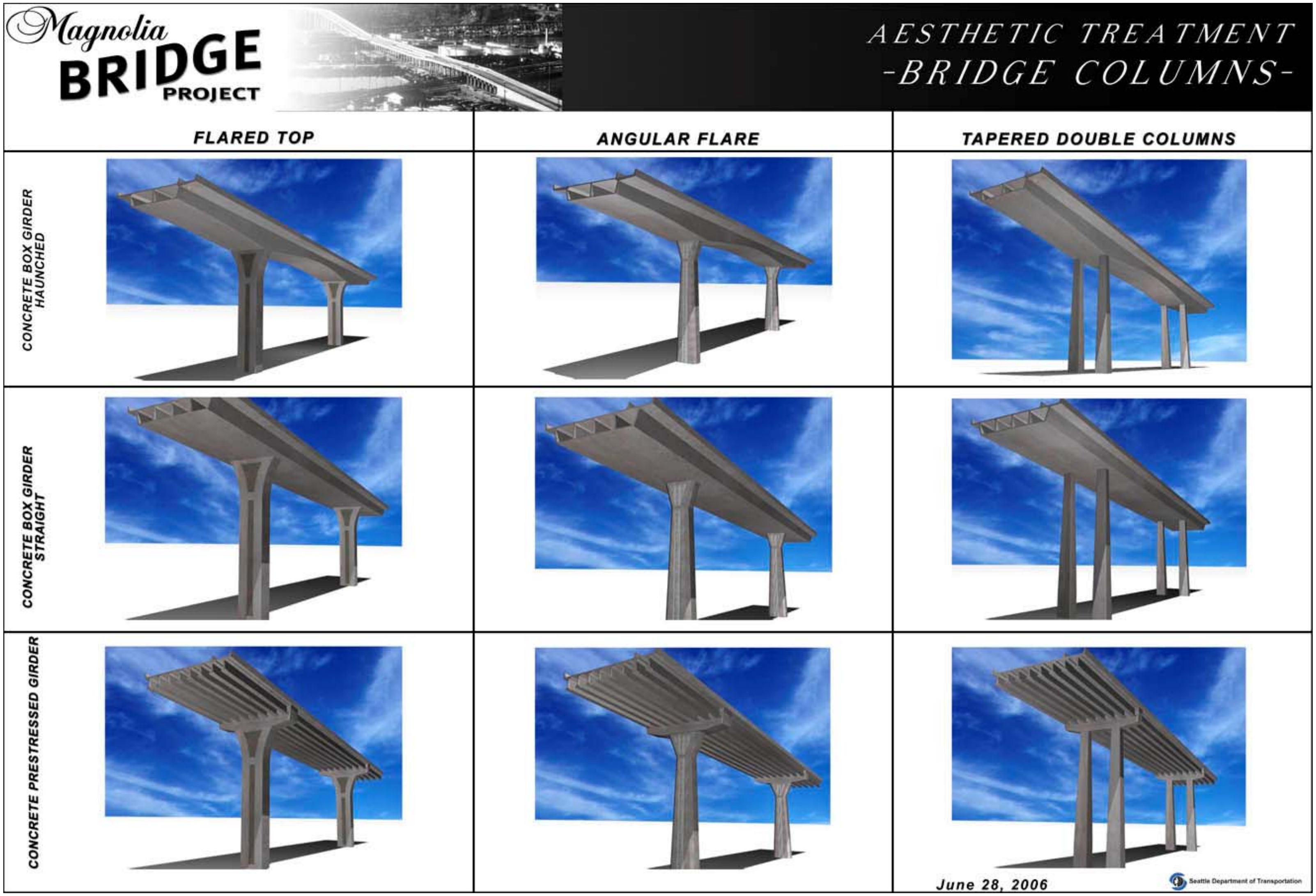


Figure 54
Aesthetic Treatment – Bridge Columns

10.2 RAILINGS AND LIGHTING

Architectural railings and lighting were evaluated in this stage of design, Figure 55. Several alternatives were developed that reflected the "Maritime" and "Progressive" architectural themes that were initially developed and preferred. Each option for railings, roadway lighting and pedestrian lighting were screened against the architectural criteria and the advantages, disadvantages and order of magnitude costs were determined for each. These elements will aid in providing the urban design elements to the bridge and provide continuity throughout. Preferred alternatives have not been selected during the initial design phase. However, further screening will occur during final design.

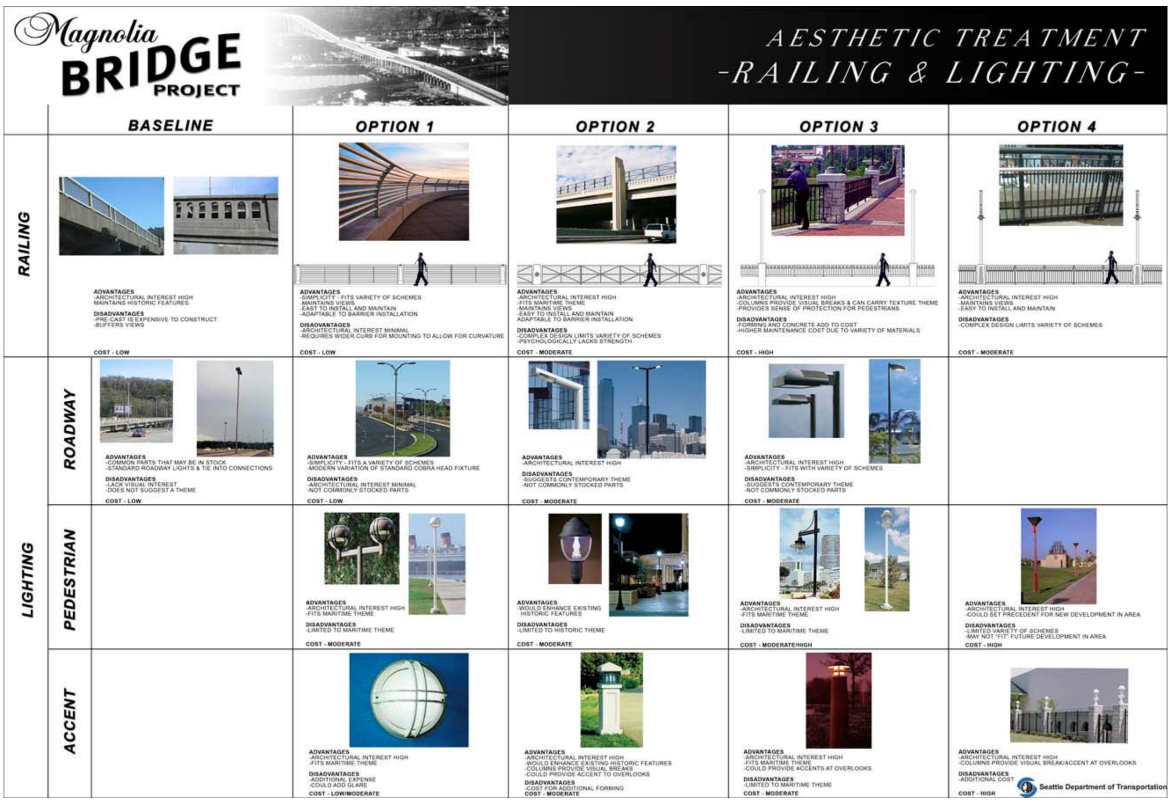


Figure 55
Aesthetic Treatment – Railing and Lighting

10.3 OVERLOOKS

In the Visual Quality Discipline Report dated January, 2005, it was identified that several viewpoints along the bridge offer tremendous opportunities for pedestrian viewing of downtown Seattle, Elliott Bay and other adjacent viewpoints. Several options for pedestrian overlooks were considered and evaluated against the criteria and compatibility with the preferred architectural style of the structure, Figure 56. It is anticipated the overlook areas would accommodate pedestrians outside of the primary sidewalk zone and would accommodate ADA requirements. These overlook areas were identified as an opportunity to include artwork as well. Although a preferred alternative has not been selected as part of this design phase, each overlook alternative will be screened further and evaluated against the preferred architectural style of the bridge. This will occur during the final design effort.

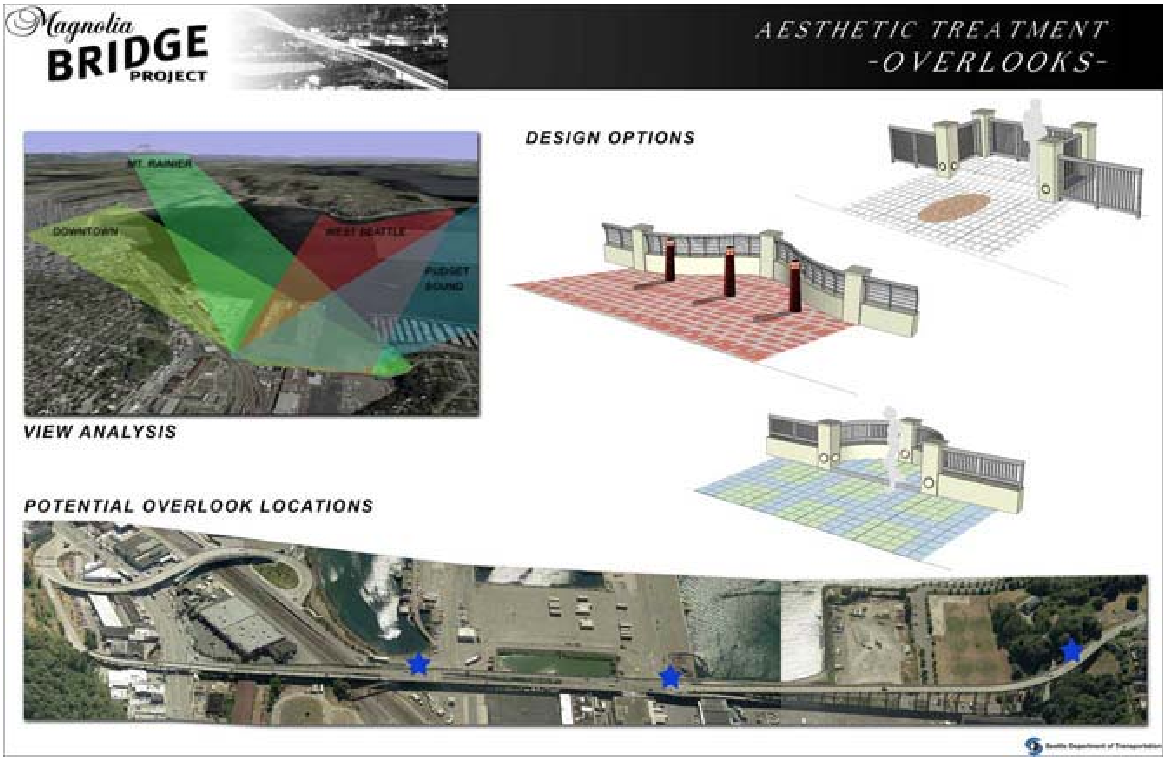


Figure 56
Aesthetic Treatment – Overlooks

10.4 PEDESTRIAN ACCESS/BIKE PATH

During the initial design phase, the need to gain access from the bridge to the existing trail on the west side of the BNSF tracks was identified. Several initial concepts for pedestrian access have been developed and are identified below, Figure 57. A preferred alternative has not been selected during this phase of design and it was identified that additional coordination with the Port of Seattle, Seattle Parks and Seattle Transportation will need to occur during the final design phase.

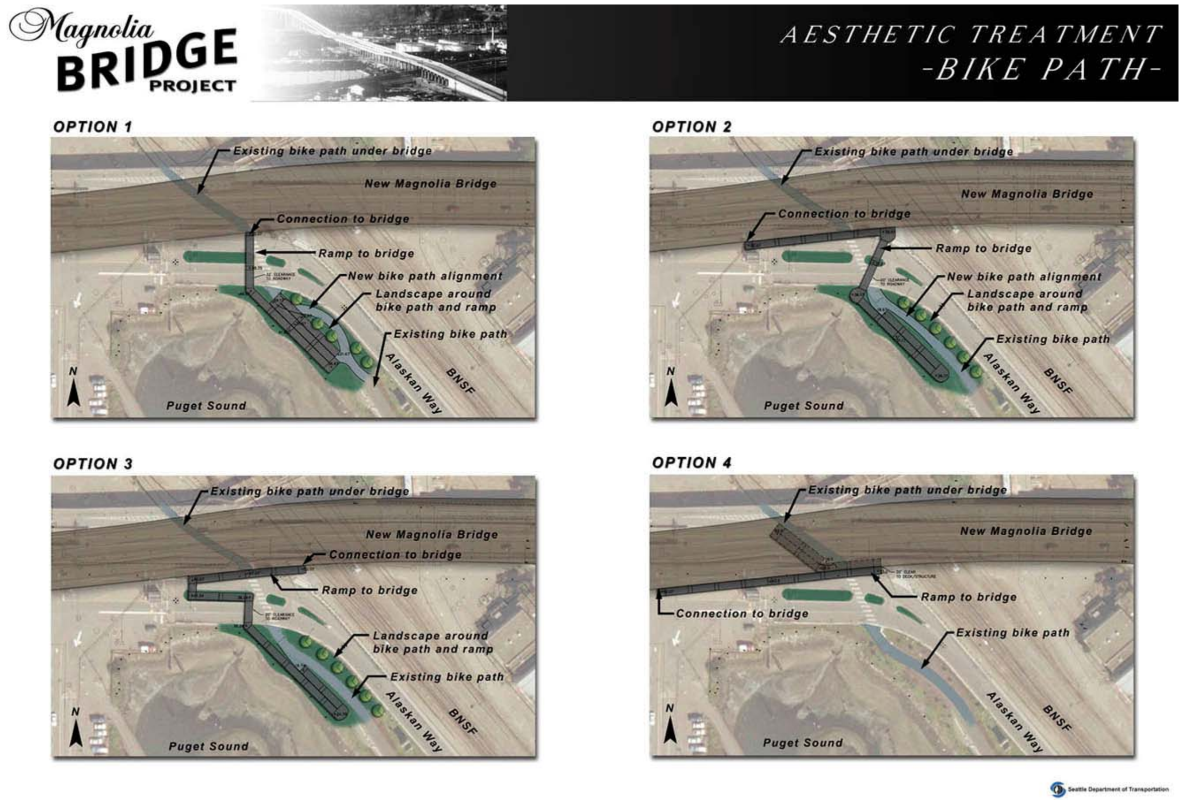
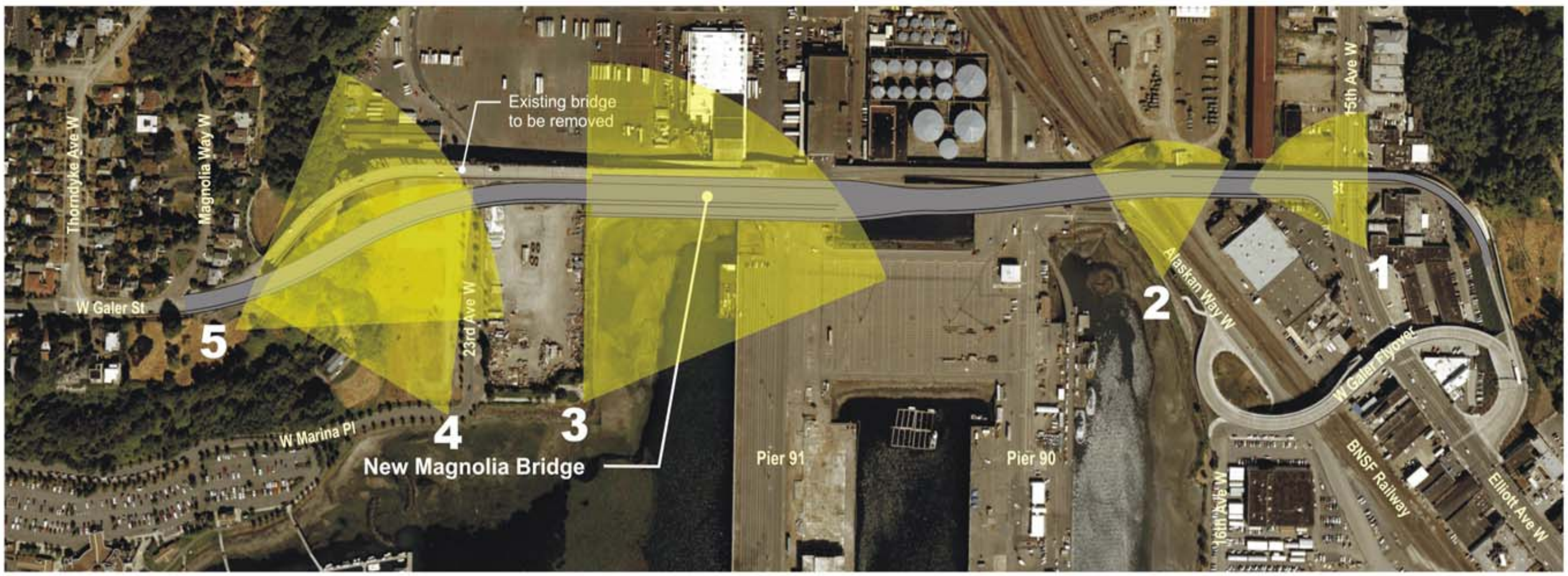


Figure 57
Aesthetic Treatment – Bike Path

10.5 RENDERINGS OF ALTERNATIVES



1. From 15th Ave W toward 15 Ave W crossing and ramp
2. Elliot Bay Trail toward railroad crossing
3. From Smith Cove Park toward on-ramp and viaduct
4. From W Marina Pl and 23rd Ave W toward bluff structure
5. From Ursula Judkins Viewpoint toward new bridge

City of Seattle
Magnolia Bridge Replacement Project

3D RENDERING VIEWPOINT LOCATIONS

HNTB 8/25/06

Figure 58
3D Rendering Viewpoint Locations

Figure 59
EXISTING BRIDGE
(looking north on 15th Avenue West)

Figure 60
PRESTRESSED CONCRETE GIRDERS
(looking north on 15th Avenue West)

Figure 61
HAUNCHED CAST-IN-PLACE CONCRETE BOX GIRDER
(looking north on 15th Avenue West)

Figure 62
STRAIGHT CAST-IN-PLACE CONCRETE BOX GIRDER
(looking north on 15th Avenue West)

Figure 63
EXISTING BRIDGE
(looking north from Alaskan Way West)

Figure 64
PRESTRESSED CONCRETE GIRDERS
(looking north from Alaskan Way West)

Figure 65
STRAIGHT CAST-IN-PLACE CONCRETE BOX GIRDER
(looking north from Alaskan Way West)

Figure 66
EXISTING BRIDGE
(looking northwest from Smith Cove Park)

Figure 67
PRESTRESSED CONCRETE GIRDERS
(looking northeast from Smith Cove Park)

Figure 68
HAUNCHED CAST-IN-PLACE CONCRETE BOX GIRDER (MAINLINE STRUCTURE)
STRAIGHT CAST-IN-PLACE CONCRETE BOX GIRDER (23rd AVENUE RAMPS STRUCTURE)

Figure 69
EXISTING BRIDGE
(looking north in Smith Cove Acquisition park site)

Figure 70
PRESTRESSED CONCRETE GIRDERS
(looking north in Smith Cove Acquisition park site)

Figure 71
HAUNCHED CAST-IN-PLACE CONCRETE BOX GIRDER
(looking north in Smith Cove Acquisition park site)

Figure 72
CAST-IN-PLACE CONCRETE BOX GIRDER

Figure 73
PRESTRESSED CONCRETE GIRDERS

10.6 AESTHETICS STUDIES CONCLUSION

During this initial TS&L stage, the primary focus was on the architectural and aesthetic treatments that were associated with the structure types. As determined and outlined above the basic structure type that was preferred included the haunched cast in place concrete box structure over 15th Avenue NW, the constant depth cast in place concrete box structure at the Low Level Mainline bridge and the haunched cast in place concrete box structure at the Bluff Bridge.

The secondary focus during this TS&L stage was on the urban design elements. These elements included lighting, railings/barriers, piers, architectural finishes, colors and patterns. A good response and preferred alternatives were received during the process from the various stakeholders. However, there was no consensus reached in the process on preferred architectural finishes. The urban design elements will be developed in more detail and a preferred alternative selected during the preliminary design phase of the project.

11. OPEN HOUSE COMMENTS

11.1 OVERVIEW

The sixth Magnolia Bridge Project Open House was held on September 13, 2006, from 5:30 to 8:30 p.m. at the Blaine School in Magnolia. Stations were set up in the Blaine School lunchroom to present the bridge structure type alternatives being evaluated, along with the history of the project, images of the Preferred Alternative (Alternative A), and potential bridge amenities and detour routes. The open house was held to share the possible structure types that might be used for the new Magnolia Bridge and to gather public feedback that will guide the upcoming selection of a bridge structure type for each section of the new bridge.

Approximately 60 people signed in at the meeting. Information on the benefits and costs of the proposed structure types, additional views of proposed bridge columns, and a guide to the sections of the bridge being used for design work was provided in a packet with a comment form. Project team members were on hand to answer questions and explain each of the alternatives under consideration.

Public input was gathered at the meeting in several ways: (1) through discussions with project team members, (2) on large flip charts located near different information stations where the public was invited to write comments or questions, (3) on comment forms (meeting attendees were invited to complete the comment form and leave it at the meeting or mail it in at a later date), and (4) through oral comments heard after the presentation.

11.2 GENERAL SUMMARY

The following are common issues and concerns raised during the open house, either on flipcharts, during the question and comment period after the presentation, or on comment forms. This list is not all-inclusive, but attempts to capture the key points heard repeatedly from the public.

- Many attendees preferred the cast-in-place concrete box girder over the prestressed concrete girders, particularly for the Magnolia bluff section of the bridge. The prestressed concrete girders option was supported by those who specified a design for the 15th Avenue W Overcrossing.

- Curved flare columns received the most positive written comments from attendees.

- Functional bridge design was the most popular project priority chosen by members of the public, while project cost was not chosen by any commenter. Table 18 provides a summary of the number of members of the public that chose a given project priority on their comment form.

Table 18
Project Priorities Provided on Comment Forms

Potential Priority	Comments selecting given aspect as a project priority
Functional bridge design	7
Attractive bridge design	6
Bike & pedestrian facilities	5
Other: Minimal future maintenance	1
Other: Minimum impact on area businesses	1
Other: Road surfacing	1
Other: No local taxing district	1
Other: More space at bus stops	1
Project Cost	0

- Transit, bicycle and pedestrian access—Several citizens raised questions about access for mass transit, bicyclists, and pedestrians on the new bridge. There was interest in overlooks from a few citizens, though others thought they were an inefficient use of project funds.

- Traffic calming—A few citizens on the Magnolia bluff are concerned about the high speed of cars entering Magnolia and asked for traffic calming measures.

- Impacts—Noise impacts and impacts to the Ursula Judkins viewpoint were a concern of some citizens.

11.3 PUBLIC INPUT

Comment forms were collected at the public open house on September 13, 2006. Verbatim comments are provided below and are grouped by question. Blank spaces indicate sections that were left blank by respondents.

Which bridge structure concepts to you prefer, and why?

- Prestressed concrete and tapered columns – lower cost, clean appearance, apparently will work well
- Cast in place, curved flare columns – like the look
- 15th Ave: Prestressed concrete – Angular Flare column
- Mainline: Prestressed concrete – Angular Flare column
- 23rd Ave Ramps: Straight Cast-in-place box girder – Curved Flare columns
- Mag. Bluff: Haunched Cast-in-place box girder – Curved Flare columns
- Aesthetic treatment: Option 1 for all segments
- 23rd - Haunched Cast-in-Place Box Girder
- Magnolia Bluff - Haunched Cast-in-Place Concrete Box Girder
- Curved Flared columns
- Pre-cast whenever possible
- Overlook points for pedestrians
- Bike access
- Solar powered lighting with back up power grid lighting
- I like the curved flare on columns. Yes, please include overlooks. Yes, bike access from bridge to Myrtle Edwards trail is important. Ramp, not elevator, please.
- Curved flare columns and boxed in (girder concealed) looks cleaner. Haunched “B” for all segments (except 15th Ave Overcrossing where style “A” pre-stressed girders would be more expedient, less disruptive to traffic flow, faster to construct) because less columns, most graceful design.
- Aesthetic treatment options #1 appears most fluid, elegant, and timeless and most unobtrusive.

Which bridge structure concepts do you dislike, and why?

- The one that takes the most time to build
- I don’t like the elevator idea. Unsafe, unclean, and a maintenance problem. Ramp is better.
- Overlook areas seem frivolous, extravagant, especially considering there still isn’t any improved access from Magnolia to the waterfront. No traffic-speed management provisions apparent. Open girders look messy, unfinished, providing nesting areas.

Which project priorities are most important?

- Functional bridge design; bike & pedestrian facilities
- Future maintenance – the less needed the better
- Attractive bridge design; Functional bridge design
- Attractive bridge design; bike & pedestrian facilities
- Attractive bridge design; Functional bridge design; Bike & Pedestrian facilities

- Functional bridge design & Economic impact; Minimum impact on area businesses, T 90 – 91, Magnolia Village
- Attractive bridge design; Functional bridge design; Road surfacing. We are living on the west end of the bridge. Road noise is a major problem for our quality of life. We urge that the new bridge road be provided with a low noise paving and that other methods be explored for noise abatement.
- 1: Attractive bridge design
  2: Functional bridge design

- Don't do local taxing district for bridge – very bad idea! Will we do the same for 520? Viaduct?
- Attractive bridge design; Bike & Pedestrian paths need to be wider than existing to be safe, allow for bypass. Bus stops need to have more space, out of pedestrian & bike paths.

Additional comments?

- Bridge should provide views for vehicle occupants and pedestrians. Lighting should be designed to discourage birds from sitting on them.
- Who do I contact about the traffic lights at 15th & Dravus. Traffic gets backed up to the bridge over the RR tracks during evening rush hours when lights are on blink.
- During Alernate selection a perpendicular North Ramp and traffic light were abandoned due to lack of need and expense. If need were to be assessed for overlooks they would be abandoned – there is no need for the expense of overlooks, they should be removed from Plan!
- Impressed by site location new bridge and minimal traffic delay and minimal cost of my preferred concepts. Prefer railings option 4.
- Please work to keep existing trees
- Make the structure to include Artistic features including architectural detailing in columns
- Concerns – minimum disruption during construction. Maintaining traffic flow on 15th
- Great presentation – thanks! Don't hear only the few people who want to slow traffic. Make it so we can go faster <u>with safety</u>. This impacts more people. Figure out a way to deal with safety issues without slowing speed.
- Initial project goal to improve access from/to Magnolia and waterfront was sacrificed – Why?? "Preferred" alignment A fails to respond to neighborhood/local Magnolia need for access to Interbay, Smith Cove Park, Mid-Span on-demand-signal would almost traffic and allow legal auto access from bluff.
- Stop light at top of bridge to slow traffic and also allow pedestrian crossing

11.4 FLIPCHART COMMENTS

Structure Types

- The haunched cast-in-place designs have the cleanest lines and look most modern
- Magnolia Bluff structure A – with Angular Flare columns – my vote

Bridge Amenities

- Railing – Option 1; Roadway – Option 2; Lighting – Option 1; Accent – Option 3; Overlook – Middle option
- Higher wall creates feeling of protection – like center schematic
- If access to Port from eastbound is not a priority, overlooks should be discarded!

12. BRIDGE ALTERNATIVE STUDY

The three alternatives selected during the Bridge Concept Study were advanced through the Bridge Alternative Study that included preliminary design and analysis to determine span arrangement, structure depth, foundations, aesthetics, public feedback, quantities and costs. A complete set of plan and elevation drawings was prepared for each of the selected bridge types, see Appendix D. The preferred structure type was selected from the three bridge types at the completion of the Bridge Alternative Study through a coordinated effort with the City and the Design Advisory Group.

For the three final alternatives, plan and elevation sheets, typical section sheets, and 3-D renderings were prepared for each alternative; see Figure 58 thru Figure 73. Superstructure, substructure, and foundation systems were developed in sufficient detail to determine estimated quantities for cost estimating. Seismic models were generated for typical units to determine the load capacity requirements for the substructure and foundations.

12.1 CAST-IN-PLACE CONCRETE BOX

The discussion on the advantages of the cast-in-place concrete box included the desire to provide a haunched structure at the bluff and at 15th Avenue Overcrossing. The haunched bluff structure was a clear winner at the public open house. By providing a haunch at both the bluff structure and at 15th Overpass will tie the ends of the bridge together. A consistant bridge type along the length of the project is preferred. At the center mainline section of the bridge, the concrete box provides a clean graceful appearance because the pier crossbeams can be made integral with the box structure. The bridge will have a cleaner appearance from under the bridge for the future users of the Cruise Terminal and Northbay development.

Longer spans were provided to reduce the number of costly foundations and ground improvements and to limit number of piers in the water. The longer spans allowed the westernmost pier to be located off the bluff hillside. Expansion joints were located at internal hinges within the span near points of inflection, so that a drop down pier crossbeam is not necessary and a smooth look was maintained on the underside of the structure throughout the length of the bridge.

Concrete Box Girder structures are desirable for the curved bridge alignments because it can provide relatively long spans around the curve. The width transitions can be easily accommodated with the cast-in-place concrete box by varying the girder web spacing and adding internal webs as required.

A precast concrete tub with skewed piers was proposed at the railroad crossing so it would provide a similar box appearance but allow construction to proceed over the railroad without falsework. The skewed piers were used to shorten the span to meet structure depth requirements.

The disadvantage of the concrete box bridge type is the need for temporary shoring to support the structure during construction. The temporary shoring will need to remain in place until the box structure is completed and post-tensioned. This will increase the superstructure construction time over the other structure alternatives where temporary shoring is not necessary. Segmental concrete construction will be investigated at the bluff structure when the bridge is at a high point off the ground. The cast-in-place concrete alternative was estimated at higher cost than prestressed girders and lower cost than steel girders. The increase in cost of the superstructure is offset by the reduced number of foundations and columns required due to the increased span lengths.

12.2 PRESTRESSED GIRDER

Prestressed girders offer simple to construct and cost effective options for new bridge construction. Prestressed girders can be erected without having to install temporary shoring or falsework. Since the girders can be erected without temporary shoring, the construction time for superstructure is less than cast-in-place structures but the foundation has a longer construction time because more are required. Prestressed girders used for evaluation were W58G for 15th Avenue Ramp because length was limited by the curvature, WF74G at the railroad crossing because of depth limitations and W83G for the remaining mainline and bluff structures.

The primary disadvantage with precast girders is the span length limitation for straight girders on curved structures. Since the girders are chorded members the span length has to be limited on structures with tight radius alignments, this occurs on the bluff structure and at 15th Avenue Overpass Ramp. A straight prestressed girder bridge on a curve does not have a clean appearance from below, because the supporting girders will be straight while the deck is curving.

The prestressed girder alternative requires more piers. This increases the foundation construction costs and ground improvement costs.

Large width transitions can also be more complicated and expensive because additional girders would be required to accommodate the girder flare. Width transitions will look cluttered from below, since additional girders will need to be added to accommodate the increased structure width.

The crossbeam cap will sit lower than the superstructure to allow girders to be erected onto the crossbeams. Expansion joints would have to be located over piers for prestressed girder bridges resulting in a drop down pier cap at the expansion joints.

The estimated cost of prestressed girder superstructure is less than either steel or cast-in-place concrete box but the savings is offset by the cost of additional piers, foundations and ground improvement.

12.3 STEEL GIRDER

Steel girder structures are feasible for curved bridge alignments because the girders can be fabricated to follow the curvature of the bridge and can be haunched to provide the desirable appearance at the bluff and 15th Avenue Overpass. Variable width superstructure can also be achieved by a combination of varying the girder spacing and adding or subtracting girders as required. Seismic performance of steel girders may be better than concrete structures because the weight of steel structures is less and results in a reduced seismic force. This is offset by the loss of frame action because the steel girders are supported on bearings.

Steel girders will be erected without temporary shoring and only a few temporary supports are necessary to support the girders until the individual girder sections are spliced together. The construction time is less than cast-in-place structures, because the girders can be erected with minimal temporary shoring using smaller cranes because the sections are lighter. There will be crossbeams required at the piers to support the steel girders.

The primary disadvantages of a steel girder superstructure is the high cost of steel and the long term maintenance costs after the bridge is built. The steel girders will require periodic inspection, maintenance, cleaning, and painting. The steel girders can be haunched to provide a variable depth appearance for the superstructure. The steel girders will be painted because weathering steel does not perform well in Western Washington because of the moisture.

The estimated cost of steel girder superstructure is more than either prestressed girders or cast-in-place concrete box and it was found that the increased cost was not offset by reduction in cost of piers, foundations or ground improvement.

12.4 BRIDGE ALTERNATIVE STUDY CONCLUSIONS

Bridge alternatives were reviewed with the Design Advisory Group on September 4, 2006 and discussed at a public open house on September 13, 2006. On September 27, 2006, members of the Project Team and Seattle City staff met to determine which one of the three alternatives; cast-in-place concrete box, prestressed girder or steel girder, should be recommended to be carried forward for further evaluation. After discussion of advantages, disadvantages and costs, identified in Table 18, The Project Team recommended that the cast-in-place concrete box alternative with precast concrete tubs at the railroad crossing be advanced for final design.

The cast-in-place concrete box was selected as the preferred alternative because it:

- Provides for the optimum pier location at the bluff by eliminating the pier on the side slope.
- Provides a clear span at the Park to optimize future use of the property.
- Provides the opportunity to use aesthetically haunched boxes at the bluff and over 15th Avenue.
- Includes longer spans with fewer piers for less foundation cost.
- Accommodates the transitions for the 15th Avenue Overpass ramp and 23rd on and off ramps with a clean smooth appearance.
- Allows use of longer spans around curved alignments.
- Eliminates pier caps for a clean appearance.

Table 19
Structure Type Selection Matrix

Structure Type	Bridge Base Cost (2006 $)	Advantages	Disadvantages
15th Avenue Overpass			
Prestressed Concrete Girders	$ 3,000,000	• Lower cost • Straightforward to construct, no temporary shoring • Shorter construction duration • Low maintenance	• Span limitations on curved alignments • Girder depth step changes • Less aesthetically pleasing
Haunched Cast-in-Place Concrete Box Girder (Selected)	$ 3,100,000	• Average cost • Accommodates curved alignments well • Low maintenance • Aesthetic haunched shape over 15th Ave • Long span capability	• Longer construction duration • Requires temporary shoring • Increased impacts below bridge during construction
Straight Cast-in-Place Concrete Box Girder	$ 3,100,000	• Average cost • Accommodates curved alignments well • Low maintenance • Aesthetically pleasing • Long span capability	• Longer construction duration • Requires temporary shoring • Increased impacts below bridge during construction
Steel Plate I-Girders (Not shown, appearance similar to Prestressed Concrete Girders)	$ 3,400,000	• Average cost • Accommodates curved alignments well • Shorter construction duration • Minimal temporary shoring • Long span capability • Lower seismic loads due to less weight; Reduced foundations	• Higher superstructure cost • High maintenance/Life cycle costs

Structure Type	Bridge Base Cost (2006 $)	Advantages	Disadvantages
Mainline Structure Types			
Prestressed Concrete Girders	$ 19,500,000	• Lower cost • Straightforward to construct, no temporary shoring • Shorter construction duration • Low maintenance	• Does not accommodate width transitions well • Less aesthetically pleasing
Straight Cast-in-Place Concrete Box Girder (Selected)	$ 20,800,000	• Average cost • Accommodates width transitions well • Low maintenance • Aesthetically pleasing	• Longer construction duration • Requires temporary shoring • Increased impacts below bridge during construction • Steel Box Girder over railroad
Steel Plate I-Girders (Not shown, appearance similar to Prestressed Concrete Girders)	$ 20,000,000	• Average cost • Accommodates width transitions well • Shorter construction duration • Minimal temporary shoring • Lower seismic loads due to less weight; Reduced foundations	• Higher superstructure cost • High maintenance/Life cycle costs •
23rd Avenue Ramps Structure Types			
Prestressed Concrete Girders	$ 6,500,000	• Lower cost • Straightforward to construct, no temporary shoring • Reduced environmental permitting required for construction • Shorter construction duration • Low maintenance	• Less aesthetically pleasing •
Straight Cast-in-Place Concrete	$ 6,700,000	• Average cost • Low maintenance • Aesthetically pleasing	• Longer construction duration • Requires temporary shoring

Structure Type	Bridge Base Cost (2006 $)	Advantages	Disadvantages
Box Girder (Selected)		• Long span capability	• Increased environmental permitting required for construction • Increased impacts below bridge during construction
Steel Plate I-Girders (Not shown, appearance similar to Prestressed Concrete Girders)	$ 7,100,000	• Average cost • Shorter construction duration • Minimal temporary shoring • Reduced environmental permitting required for construction • Long span capability • Lower seismic loads due to less weight; Reduced foundations	• Higher superstructure cost • High maintenance/Life cycle costs
Magnolia Bluff Structure Types			
Prestressed Concrete Girders	$ 18,900,000	• Lower cost • Straightforward to construct, no temporary shoring • Shorter construction duration • Low maintenance	• Span limitations on curved alignments • Less aesthetically pleasing
Haunched Cast-in-Place Concrete Box Girder (Selected)	$ 19,800,000	• Average cost • Accommodates curved alignments well • Low maintenance • Aesthetically pleasing • Long span capability	• Longer construction duration • Requires temporary shoring • Increased impacts below bridge during construction
Haunched Steel Box Girder (Not shown, appearance similar to Cast-in-Place Concrete Box Girder)	$ 27,200,000	• Higher cost • Accommodates curved alignments well • Shorter construction duration • Minimal temporary shoring	• Higher superstructure cost • High maintenance/Life cycle costs

Structure Type	Bridge Base Cost (2006 $)	Advantages	Disadvantages
		• Long span capability • Lower seismic loads due to less weight; Reduced foundations	
Column Type (cost based on Bluff Structures Columns)			
Curved Flare Column (Selected)	$6,000,000	• Texture opportunity on surfaces • Positive relationship to haunched bridge type • Classic appearance	• Custom forms • May require minor repairs post seismic activity
Angular Flare Column	$5,800,000	• Texture opportunity on surfaces • Positive relationship to haunched bridge type • Timeless architectural style	• Custom forms • May require minor repairs post seismic activity
Tapered Column	$5,600,000	• Reduced forming costs • Clean appearance	• Limited opportunity for textures and highlights

13. CONSTRUCTION

13.1 ANTICIPATED CONSTRUCTION SEQUENCE

Generally, construction of the new bridge will be completed as much as possible before the existing bridge is demolished in order to maintain bridge use for 15th Avenue West to Magnolia traffic for as long as possible. It is anticipated that bridge access will be closed for about one year for construction of the eastern section where the replacement structures are on the existing alignment.

The assumed sequence of work is as follows, actual construction method will be according to contractor method and means:

- Construct Magnolia Bluff structure using cast-in-place on falsework or balanced cantilever construction. Section can be built from Pier 16 to Pier 11 without impacting existing structure.
- Demolish existing wharf at Smith Cove and existing 23rd on ramp.

- Build work bridges for access to foundations and construction of bridge spans over the water at Smith Cove and Jacob's lake.
- Construct 23rd on ramp and mainline bridge until the construction conflicts with existing bridge.
- Construct a temporary detour, see Section 13.3 below. Switch traffic to temporary detour.
- Demolish the existing Mainline Bridge and 23rd Street Off-Ramp
- Construct new mainline bridge over railroad and 15th Avenue Ramp.
- Switch traffic to new bridge.
- Construct 23rd off ramp and complete construction of 23rd on ramp.

13.2 CONSTRUCTION STAGING AREAS

A construction staging area could be created north of Smith Cove Park on existing asphalt pavement. Other construction staging areas may be identified as final design plans and construction contracts are developed. The project site has convenient street, rail, and marine access which will allow prefabricated bridge components and other construction materials to be brought in as needed.

13.3 MAINTENANCE OF TRAFFIC DURING CONSTRUCTION

Three types of detours may be used to handle traffic during bridge construction: existing city streets; new surface streets through Terminal 91; and staged construction and temporary ramps to keep traffic in the existing corridor. All of these types of detours are expected to be used. Actual maintenance of traffic will depend on Port facilities in place during construction; such as Cruise Terminal and Northbay development. Traffic detours will also depend on Contractor's proposed method and means of construction, therefore final detour options will not be determined until construction. The following is discussion of potential traffic detours and advantages and disadvantages of each alternative.

13.3.1 Use existing streets

With the existing bridge closed to traffic, traffic to and from 15th Avenue West can use the remaining two connections to Magnolia: West Dravus Street and West Emerson Place see Figure 74. The West Dravus Street route will add approximately 1.7 miles to the commute between the Magnolia Bluff and the intersection of Elliott Avenue West and the Galer Flyover. Vehicles traveling this route will encounter eight signalized intersections where the route across the existing bridge has only one. The additional travel time imposed by this detour will be about eight minutes per commuting vehicle, but is expected to be greater in periods of heaviest traffic. Traffic will be managed at congested intersections through modifications to traffic signal timing and using traffic control personnel.

13.3.2 Use new surface street detours

SDOT and the Port of Seattle are investigating providing a surface road connection that will use the Galer Flyover and a detour road along the east side of Terminal 91 next to the BNSF railroad, see Figure 74. This detour will connect Elliott Avenue West and Alaskan Way West with 21st Avenue West and Thorndyke Avenue West. SDOT and the Port of Seattle are also discussing a surface detour on the west side of Terminal 91 at the base of Magnolia Bluff to connect 21st Avenue West with 23rd Avenue West and West Marina Place. A temporary traffic signal may be needed at the 21st Avenue West intersection with Thorndyke Avenue West.

13.3.3 Maintain traffic in the existing bridge corridor

The west section of the New Magnolia Bridge will be south of the existing bridge and will be built while the existing bridge remains open to traffic. Only the existing eastbound on-ramp from 23rd Avenue West will be closed and removed during this phase and 23rd Avenue Traffic will use a surface detour route to 21st Avenue West and West Thorndyke Street. Two options will be investigated for maintaining traffic in the existing corridor when the existing bridge between 15th Avenue West and Pier 90 is demolished.

- Use the Galer Flyover, Alaskan Way West, and a new temporary ramp to the remaining bridge west of Pier 90 see Figure 75, or a new temporary ramp to the New Magnolia Bridge at Smith Cove.
- Use the Galer Flyover, Alaskan Way West, and a new temporary ramp to the New Magnolia Bridge at Smith Cove.

Demolishing and replaceing the 15th Avenue West overpass and railroad crossing with a wider structure that will allow temporary two-way traffic while the ramp and railroad crossing to the south is replaced was investigated. This alternative is not preferred because it does not provide sufficient traffic capacity because of the split intersection at the connection of the 15th Avenue Ramp and Galer Flyover connection.

Figure 74
Surface Street Detours

Figure 75
Existing Corridor Detour

13.4 PROJECT CONSTRUCTION SCHEDULE

The goal is to accomplish construction in less than three construction seasons or 2 ½ years and to minimize the temporary detour time to limit the traffic impact to the Magnolia community. There are in-water work windows that will also need to be included in the schedule. The initial indication is that in-water work will be allowed from July 17th to February 14th.

The construction schedule was developed assuming approximately 4 days per drilled shaft, 5 days per 20' column pour, and a superstructure span complete an average of one every 30 days. In order to complete the project in two and a half years, there will need to be multiple work zones with multiple drilling rigs for drilled shafts, multiple column forms, and multiple spans of superstructure on falsework. The preliminary construction schedule is shown Figure 76 and Figure 77.

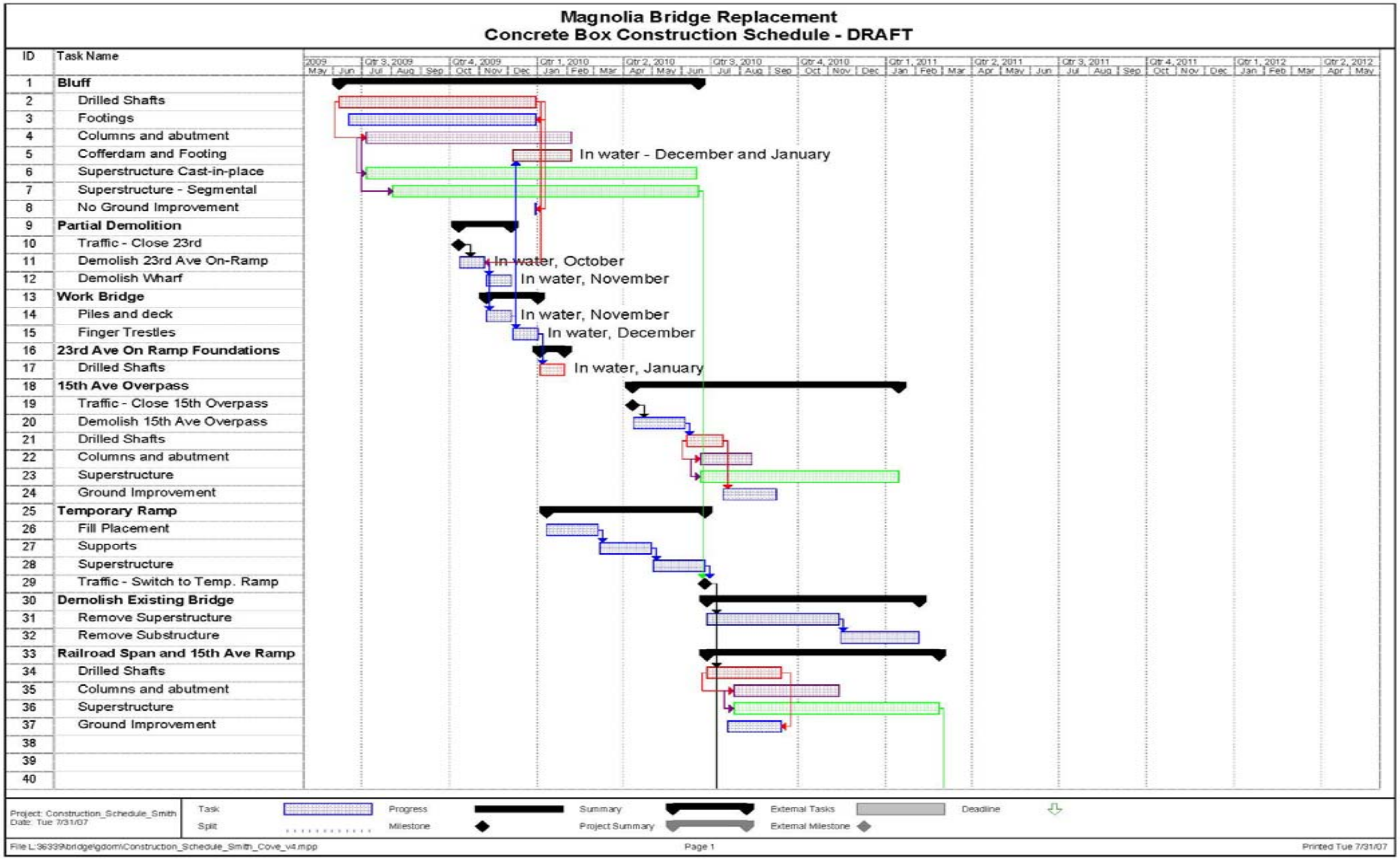


Figure 76
Concrete Box Construction Schedule

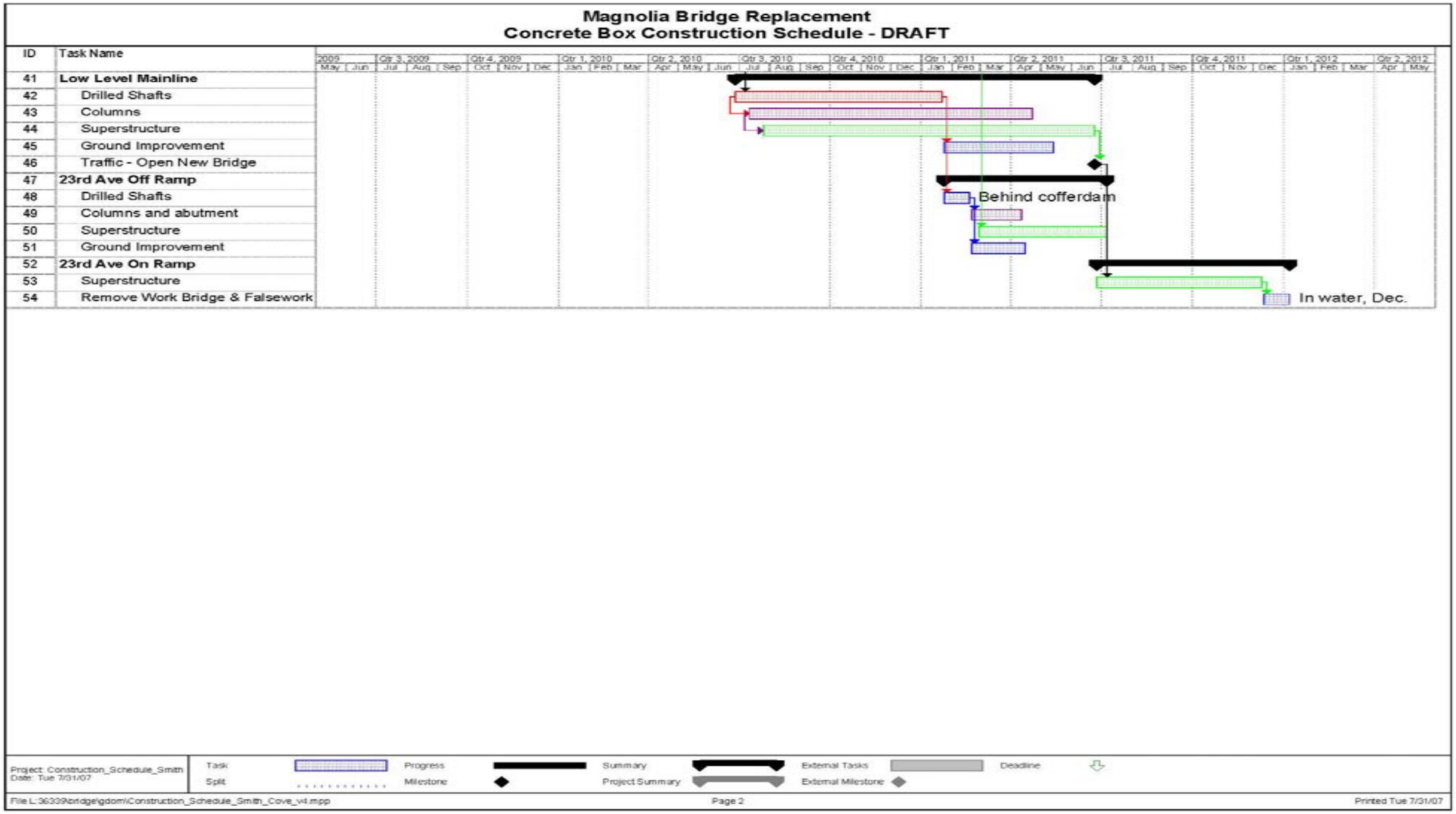


Figure 77
Concrete Box Construction Schedule

HNTB

A basic project schedule is shown in the Table 20 below.

Table 20
Basic Project Schedule

Construction Stage	Duration (months)
Mobilization of material and equipment	1
1 – Initial construction with traffic maintained on existing bridge	12
2 – Traffic Detoured	12
3 – Traffic on new structure during demolition and cleanup	6
Total Construction Time	31

14. COST ESTIMATE - SELECTED ALTERNATIVE

14.1 CONSTRUCTION BASE COST (2006 DOLLARS)

A cost estimate of the Magnolia Bridge Replacement Project was prepared based on unit prices for estimated quantities. Unit prices were determined based on bid tabulations for similar projects, summarized August 2006.

Neither HNTB nor City of Seattle has control over the cost of labor, materials, or the Contractor's methods of determining bid prices or market conditions. HNTB cannot and does not warrant, represent, make any commitments, or assume any duty to assure, that bids or negotiated prices will not vary from any estimate of construction cost or evaluation prepared by or agreed to by HNTB.

Table 21
Project Base Cost in 2006 Dollars

ITEM NO	ITEM	UNIT	QUANTITY	UNIT COST	COST (2006 Dollars)
	Mitigation				
1	Erosion and Sedimentation Control	Mo	30	$60,000	$1,800,000
	Sub total - Mitigation				**$ 1,800,000**
	Roadway Demolition				
2	Hazardous Material Abatement	CY	5,000	$300	$1,500,000
3	Misc. Demolish Existing Roadway	LS	1	$50,000	$50,000
	Sub total - Roadway Demolition				**$ 1,550,000**
	Temporary Detour				
4	Temporary Ramp Roadway	LS	1	$150,000	$150,000
	Sub total - Road Re-routing				**$ 150,000**
	Relocations				
5	Power Pole Relocation (15th Ave, Bluff)	LS	1	$500,000	$500,000
6	FAA Approach Radar & Assoc. Utilities Relocation	LS	1	$100,000	$100,000
7	Port Access Facility (to accommodate temp roadway)	LS	1	$100,000	$100,000
	Sub total - Utilities Relocations				**$ 700,000**
	Roadway				
8	MSE Wall (15th Overcrossing Ramp)	SF	6,500	$45	$292,500
9	Select Fill (15th Overcrossing Ramp)	CY	3,500	$15	$52,500
10	Gravel Base (15th OX ramp, 15th Ramp, 23rd ramps,bluff)	CY	1,500	$25	$37,500
11	Asphalt Pavement (Bluff, Misc, 23rd, 15th, Galer)	Ton	100	$110	$11,000
12	Concrete Pavement (23rd, 15th, Garfield, sidewalks, islands)	CY	2,300	$250	$575,000
13	Concrete Curb	LF	3,600	$30	$108,000
14	Pavement markings	LF	40,000	$1	$40,000

ITEM NO	ITEM	UNIT	QUANTITY	UNIT COST	COST (2006 Dollars)
15	Site Restoration (between 23rd ramps)	LS	1	$50,000	$50,000
	Subtotal				**$ 1,166,500**
	Traffic				
16	Signals (15th Ave intersection)	LS	1	$550,000	$550,000
17	Signage	LS	1	$250,000	$250,000
18	Impact Attenuator	LS	1	$40,000	$40,000
	Subtotal - Traffic				**$ 840,000**
	Storm Drainage				
19	Catch Basin	EA	13	$3,000	$39,000
20	Drain Inlet	EA	51	$1,000	$51,000
21	Scupper Drain	EA	13	$1,000	$13,000
22	Manhole	EA	14	$4,500	$63,000
23	Water Quality Unit 1	EA	4	$10,000	$40,000
24	Water Quality Unit 2	EA	3	$50,000	$150,000
25	6" Std. Galv. Steel Pipe SD (incl.hangers/fittings)	LF	1,445	$150	$216,800
26	12" Conc. SD Pipe	LF	2,661	$50	$133,000
27	15" Conc. SD Pipe	LF	618	$75	$46,300
28	18" Conc. SD Pipe	LF	17	$75	$1,200
	Subtotal - Storm Drainage				**$ 753,300**
	Sewer Relocations				
29	10" PVC SS (Port Force Main)	LF	831	$50	$41,600
30	12" PVC SS (COS CS from Elliot Ave to Metro Pump)	LF	455	$50	$22,700
31	27" RCP SS (King County Metro Trunk Sewer)	EA	1	$100,000	$100,000
	Subtotal - Sewer Relocations				**$ 164,300**
	Fire Protection				
32	8" Steel Pipe (in Ground)	LF	2,500	$70	$175,000
33	6" Steel Pipe (dry risers)	LF	500	$150	$75,000
34	Fire Hydrant Assembly	EA	10	$5,000	$50,000
35	Deluge Systems	EA	5	$150,000	$750,000
	Subtotal - Fire Protection				**$ 1,050,000**
	Sub total - Roadway				**$ 3,974,100**
	Sub total - Utilities, Drainage and Roadway				**$ 8,174,100**
	Allowances				
36	Landscaping @	**3.0%**			$246,000
37	Allowance for Unidentified @	**5.0%**			$409,000
	Sub total - Allowances				**$ 655,000**
	SUBSTRUCTURE				
38	STRUCTURE EXCAVATION CLASS A INCL. HAUL	CY	3,089	$33	$102,000
39	STRUCTURE BACKFILL	CY	1,149	$33	$38,000
40	COFFERDAM	SF	7,634	$40	$305,000
41	SHORING EXTRA EXCAVATION CLASS A	LS	1	$50,000	$50,000
42	CONC. CLASS 4000 FOR BRIDGE	CY	4,464	$600	$2,678,000
43	STEEL REINF. BAR FOR BRIDGE	LB	1,058,000	$1.25	$1,323,000
44	SOIL EXCAVATION FOR SHAFT INC. HAUL	CY	9,373	$400	$3,749,000
45	FURN. & PLACING TEMP. CASING FOR SHAFT	LF	6,186	$350	$2,165,000

ITEM NO	ITEM	UNIT	QUANTITY	UNIT COST	COST (2006 Dollars)
46	CASING SHORING (@ 10 LF/SHAFT)	LF	800	$350	$280,000
47	ST. REINF. BAR FOR SHAFT	LB.	1,641,980	$1.25	$2,052,000
48	CONC. CLASS 4000P FOR SHAFT	CY	9,373	$200	$1,875,000
49	CSL TESTING	EA.	80	$3,000	$240,000
50	CSL ACCESS TUBE	L.F.	92,948	$3	$279,000
	SUBTOTAL SUBSTRUCTURE				**$ 15,136,000**
	SUPERSTRUCTURE				
51	CONC. CLASS 5000 FOR BRIDGE	CY	20,920	$750	$15,690,000
52	EPOXY-COATED ST. REINF. BAR	LB	1,213,800	$1.50	$1,821,000
53	ST. REINF. BAR FOR BRIDGE	LB	4,047,700	$1.25	$5,060,000
54	POST-TENSIONING PRESTRESSING STEEL	LB	1,267,800	$6.00	$7,607,000
55	STRUCTURAL LOW ALLOY STEEL (MISC. @ 10LB/LF STRUCT.)	LB	53,409	$6.00	$320,000
56	PIER PROTECTON/IMPACT ATTENUATORS	EA	8	$40,000	$320,000
57	TRAFFIC BARRIER	LF	9,771	$100	$977,000
58	TRAFFIC BARRIER RAIL	LF	9,771	$300	$2,931,000
59	PEDESTRIAN BARRIER	LF	3,780	$100	$378,000
60	PEDESTRIAN RAIL	LF	3,780	$300	$1,134,000
61	EXPANSION JOINT SYSTEM	LF	646	$400	$258,000
62	SLIDING DISC BEARING	EA	50	$3,750	$188,000
63	BRIDGE PEDESTRIAN LIGHTING	LF	3,780	$60	$227,000
64	BRIDGE ROADWAY LIGHTING	LF	8,444	$30	$253,000
	SUBTOTAL SUPERSTRUCTURE				**$37,164,000**
	SUBTOTAL BRIDGE				**$52,300,000**
	Allowance for Unidentified	**5%**			**$2,615,000**
	BRIDGE DECK AREA	SF	262,928		
	BRIDGE SQUARE FOOT COST (with 10% Mobilization)		**$219**		
	Ground Improvement				
65	Compaction Grouting - Mainline	CY	158,800	$60	$9,528,000
66	Compaction Grouting - 15th Overcrossing	CY	10,700	$60	$642,000
	SUB-TOTAL		**169,500**		**$ 10,170,000**
67	**Pedestrian Connection (14'x700')**	SF	10,000	$300	**$3,000,000**
	SE Walls				
68	Mainline at Low Level	SF	2160	$45	$97,000
69	23rd Ave Off Ramp	SF	5150	$45	$232,000
	SUB-TOTAL		**7310**		**$ 329,000**
	Moment Slab and Barrier				
70	Mainline at Low Level	LF	254	$300	$76,000
71	23rd Ave Off Ramp	LF	490	$250	$123,000
	SUB-TOTAL		**744**		**$ 199,000**
	Approaches - Select Fill				
72	Mainline at Low Level	CY	1,800	$15	$27,000
73	23rd Ave Off Ramp	CY	880	$15	$13,000

ITEM NO	ITEM	UNIT	QUANTITY	UNIT COST	COST (2006 Dollars)
	SUB-TOTAL		**2,680**		**$ 40,000**
	Bridge Approach Slabs				
74	Mainline at Low Level	SY	160	$225	$36,000
75	Mainline at Bluff	SY	162	$225	$36,000
76	23rd Ave On Ramp	SY	95	$225	$21,000
77	23rd Ave Off Ramp	SY	64	$225	$14,000
78	15th Ave	SY	64	$225	$14,000
	SUB-TOTAL		**545**		**$ 121,000**
	Demolition				
79	Smith Cove Wharf Phase 1	SF	26,400	$30	$792,000
80	Magnolia Bridge Phase 1	SF	34,100	$30	$1,023,000
81	23rd On Ramp Phase 1	SF	9,400	$30	$282,000
82	Magnolia Low Level Bridge Phase 2	SF	39,200	$30	$1,176,000
83	15th Ave Ramp Overcrossing Phase 2	SF	26,900	$30	$807,000
84	15th Ave Ramp Approach Fill Ramp Phase 2	SF	15,100	$30	$453,000
85	23rd Off Ramp Phase 3	SF	8,000	$30	$240,000
86	Magnolia Bluff Phase 3	SF	68,100	$30	$2,043,000
87	Magnolia Low Level Bridge Phase 3	SF	30,200	$30	$906,000
	SUB-TOTAL		**257,400**		**$ 7,722,000**
	Work Bridge				
88	Smith Cove	SF	17,000	$75	$1,275,000
89	Jacob's Lake	SF	8,000	$75	$600,000
	SUB-TOTAL		**25,000**		**$ 1,875,000**
	Temporary Detour Ramp				
90	SE Walls	SF	16000	$30	$480,000
91	Select Fill	CY	5410	$15	$81,000
92	Beam Guardrails on Fill and SE Walls	LF	1120	$20	$22,000
93	Temporary Bridge (including Beam Guardrails)	SF	4210	$150	$632,000
	SUB-TOTAL		**26740**		**$ 1,215,000**
94	**Traffic Maintenance**				**$2,215,000**
	Additional Items and Allowances				
95	Hazardous materials (not included in Item No. 2)	CY	2,000	$300	$600,000
96	Archeologist	LS	1	$100,000	$100,000
97	Railroad flagging	mo	5	$40,000	$200,000
98	Temporary measures	mo	30	$40,000	$1,200,000
					$ 2,100,000
	PROJECT SUB-TOTAL				**$ 92,730,100**
	Mobilization	10%			**$9,273,000**
	SUB-TOTAL				**$102,003,100**
	Engineering during construction	3%			**$3,060,100**
	Construction Management	15%			**$15,300,500**

ITEM NO	ITEM	UNIT	QUANTITY	UNIT COST	COST (2006 Dollars)
	TOTAL (2006 $)				**$120,363,700**
	Right of Way				**$ 24,272,000**
	Environmental and Design				13,276,000
	GRAND TOTAL				**$157,911,700**

14.2 COST AND SCHEDULE UNCERTAINTY EVALUATION

Project budgets were prepared for the three alternative alignments resulting from the second screening of alternatives. The budgets included construction costs, contingencies, right-of-way acquisition design costs, and inflation to the estimated start of construction. Two of the alignments each had two mid-bridge access options, ramps or an elevated intersection, resulting in five build alternatives. A cost and schedule uncertainty workshop, similar to the Cost Risk Assessment® process of WSDOT, identified the risks/uncertainties using input from the team members and CITY to quantify the uncertainty in project costs (both in current dollars and in inflated dollars) and in project completion dates, consistent with available information and design levels. This uncertainty information was presented in the form of “probability distributions,” which approximately express the relative likelihood of all possible outcomes.

Two additional cost and schedule uncertainty evaluations were conducted during the alternatives development and evaluation phase. The first evaluation resulted in decreasing the number of deep foundations and the amount of ground improvement for Alignments A and D by increasing span lengths over areas of liquefiable soils. The second evaluation was conducted for the Rehabilitation Alternative.

The TS&L project phase concluded with a cost and schedule evaluation for the recommended alignment and bridge types. The analysis assumes the project is fully funded with no delays (i.e., there are no funding constraints or other funding uncertainties).

Table 22 presents the results of this process. The project base cost, as shown in Table 21, is $157.9 million in 2006 dollars. Without any risk, construction and right of way cost inflation, estimated at 6.5 and 10.0 percent per year, respectively, would result in year of expenditure costs of $193.6 million through project completion in February 2012. When risks such as market conditions at the time of bid, changing design criteria, and changes in project scope are considered, year of expenditure costs increase. At the 90th percentile probability—where there is a ten percent chance that the cost will be exceeded—the year of expenditure project cost is $261.9 million and the completion date is April 2013.

The complete Cost and Schedule Uncertainty Evaluation for the Selected Alternative summary report is in Appendix E.

Table 22 Probability Distributions for Total Project Cost and Completion

	Total Project Cost (2006 $M)	Total Project Cost (YOE $M)	Project Completion Date	Award Date
Base (no risk)	157.91	193.55	Feb 2012	Aug 2009
Mean	187.10	237.10	Sep 2012	Jan 2010
Std Dev	13.24	18.69		
Percentiles				
1%	155.78	194.76	Jan 2012	Aug 2009
5%	166.01	207.72	Feb 2012	Aug 2009
10%	170.60	214.09	Apr 2012	Oct 2009
20%	175.94	221.48	May 2012	Nov 2009
25%	178.12	224.42	Jun 2012	Nov 2009
30%	179.93	227.02	Jun 2012	Nov 2009
40%	183.53	231.88	Aug 2012	Nov 2009
50%	186.86	236.45	Sep 2012	Nov 2009
60%	190.03	241.00	Oct 2012	Jan 2010
70%	193.58	246.20	Dec 2012	Feb 2010
75%	195.62	249.27	Dec 2012	Apr 2010
80%	198.15	252.66	Jan 2013	Jun 2010
90%	**204.54**	**261.91**	**Apr 2013**	**Jun 2010**
95%	209.63	269.22	Jun 2013	Aug 2010
99%	218.34	281.82	Sep 2013	Sep 2010

15. TYPE, SIZE AND LOCATION RECOMMENDATION

Following selection of recommended cast-in-place concrete box structure type, the layout was finalized to optimize span depths and pier locations. The selected structure option was modified to provide a more balanced span arrangement and to meet the aesthetic desires indicated at the public open house meeting. The layout of the cast-in-place concrete box was revised so a pier would not need to be built on the Magnolia bluff side slope. Spans were increased and a pier eliminated to accomplish this task. The revision increased the span length from 260 feet to 360 feet at the bluff. The revision also allowed the bridge to span across the park at the base of the bluff without a pier in the middle of the future play area. A final set of Type, Size and Location plans were prepared to identify layout of the preferred bridge concept. The plans are included in Appendix G and Appendix H. The selected alternative from the Type, Size and Location Study is Alignment Alternative A with a cast-in-place concrete box with haunched spans at 15th Overpass and Magnolia Bluff , constant depth cast-in-place concrete box for the Low Level Mainline structures and ramps and a precast concrete box tub section over the railroad with a curved flare pier.

The alternatives were reviewed with the Mayor's office in October, 2006 and the following notice was released:

<u>Magnolia Bridge Replacement Project</u>; The Magnolia Bridge Design Team has completed a series of public outreach events presenting three different bridge structural types for replacing the bridge. SDOT is recommending moving ahead with the design of a concrete box structure supported by columns that flare out at the top. The two most publicly visible segments of the bridge, 15th Avenue overcrossing and the Magnolia Bluff, will be a haunched box (arched underside) while the central segment will be a straight box. This design has the support of the project Design Advisory Group(DAG), received the most positive comments at the public open house last month, and received such accolades as "elegant,… light,… transparent" from the Design Commission.

16. APPENDIX A - EXISTING BRIDGE CONDITION REPORT

17. APPENDIX B – REHABILITATION ALTERNATIVE REPORT

18. APPENDIX C – ALIGNMENT STUDY REPORT

19. APPENDIX D –BRIDGE ALTERNATIVE DRAWINGS, CONCRETE BOX, PRESTRESSED GIRDER, AND STEEL GIRDER

20. APPENDIX E – COST AND SCHEDULE UNCERTAINTY EVALUATION, SUMMARY RESULTS

21. APPENDIX F - GEOTECHNICAL MEMORANDA

22. APPENDIX G – TS&L PLANS - ROADWAY, UTILITY AND RIGHT OF WAY

23. APPENDIX H– TS&L PLANS – BRIDGE