SOUTH PARK BRIDGE REPLACEMENT PROJECT

INTERMEDIATE DESIGN MEMORANDUM
(DRAFT)

July 2nd, 2009

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SOUTH PARK BRIDGE

Final Design

INTERMEDIATE DESIGN MEMORANDUM

(DRAFT)

July 2, 2009

This report presents documentation of design consideration related the Intermediate Design (Phase I) of the final design of the South Park Bridge replacement. The report is intended to be a complimentary document of the Phase I design submittal and should read in that context. Specifically, this document is intended to be used with a fundamental understanding of the preliminary design and with an understanding of the South Park Bridge Intermediate Design Plans and Design Requirements.

I hereby certify that this report was prepared by me or
under my direct supervision and that I am a duly licensed
Professional Engineer under the laws of the state of Washington.

Signed: ______________________________ Lic. No.: 35681

Printed Name: Salima Nagji Date: __________

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HNTB

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1 EXECUTIVE SUMMARY

The purpose of this document is to describe the assumptions, project background, design considerations, and objectives of the final design for the new South Park Bascule Bridge as they exist at the conclusion of the Intermediate Design (Phase I) stage of design development. This document supplements and updates the information provided in the South Park Bridge Replacement Preliminary Design Report prepared for King County by PB Americas Inc., August 2007. This document also supplements the South Park Bridge Design Requirements documents and Intermediate Design Plans.

During the Preliminary Engineering and Environmental Documentation (PEED) phase, King County selected a double-leaf bascule bridge to replace the existing structure from design alternatives presented in the *Draft Environmental Impact Statement (DEIS)*. The main considerations that govern the design for the project are safety, environmental impact, function, aesthetics and impact to the South Park businesses and neighborhood. The bridge type selection process is documented in the DEIS.

Intermediate design for the project began in May of 2008; the design team reviewed the preliminary design and provided recommendations that would enhance the design of the bascule bridge. Based on advancement of the design and a value engineering process, some elements of the project and construction activities have changed or have been further refined from the preliminary engineering phase. Major changes include:

- Revisions to the roadway geometry.
- Revisions of the construction techniques for the two in-water bascule piers now involve the sinking of caissons within a cofferdam instead of using a pier supported by drilled shafts. The change was recommended due to better seismic performance, shorter construction timeline, and lower cost.
- Revisions to the amount of impervious surface area and stormwater Best Management Practices (BMPs).
- Relocation and reduced area of the seismic ground improvement methods (earthquake drains) away from the shores of the Duwamish Waterway.

A Value Engineering Study was performed in July 2008. Recommendations from this study were addressed and are documented in this technical memorandum.

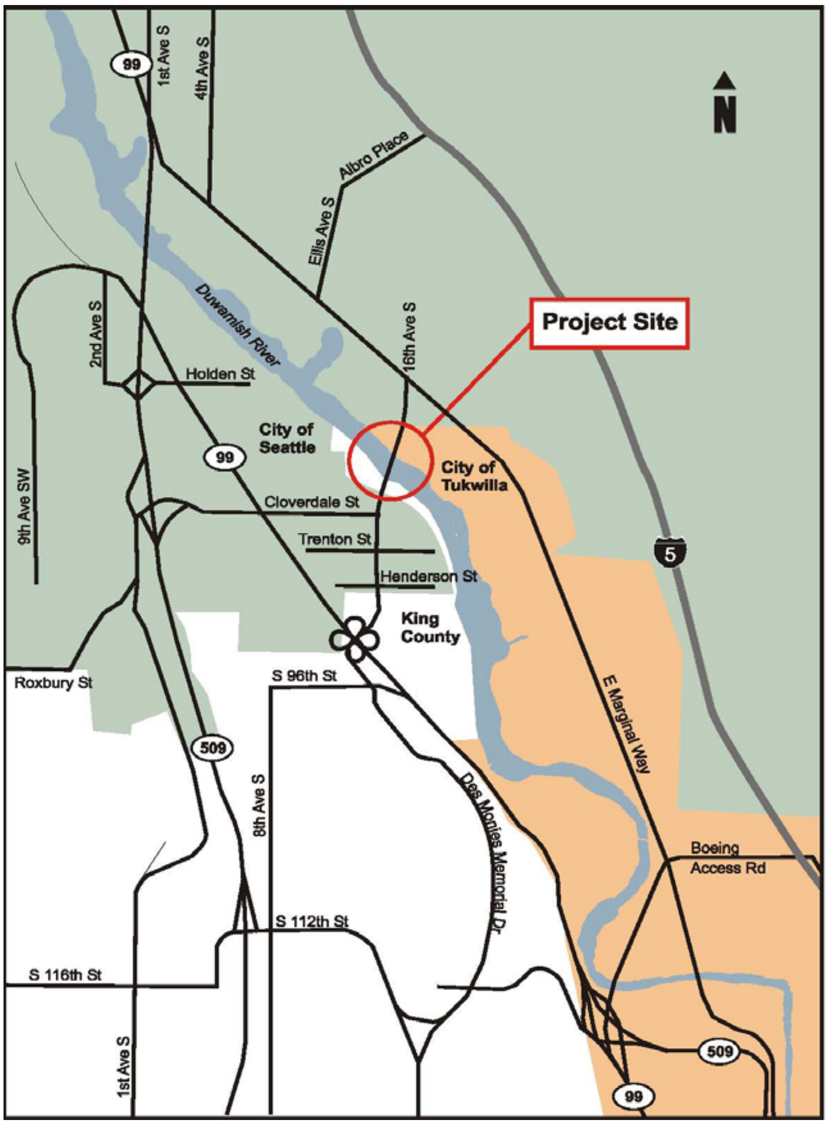
A Final Environmental Impact Statement (FEIS) and Revised Biological Assessment were completed in February of 2009. A Biological Opinion (BO) and Record of Decision (ROD) are expected in the summer of 2009.

This technical memorandum documents the design decisions that have been made as the project advances from a preliminary design level to an intermediate design level. The decisions made as part of the Intermediate Design phase will be carried forward into Final Design.

2 INTRODUCTION

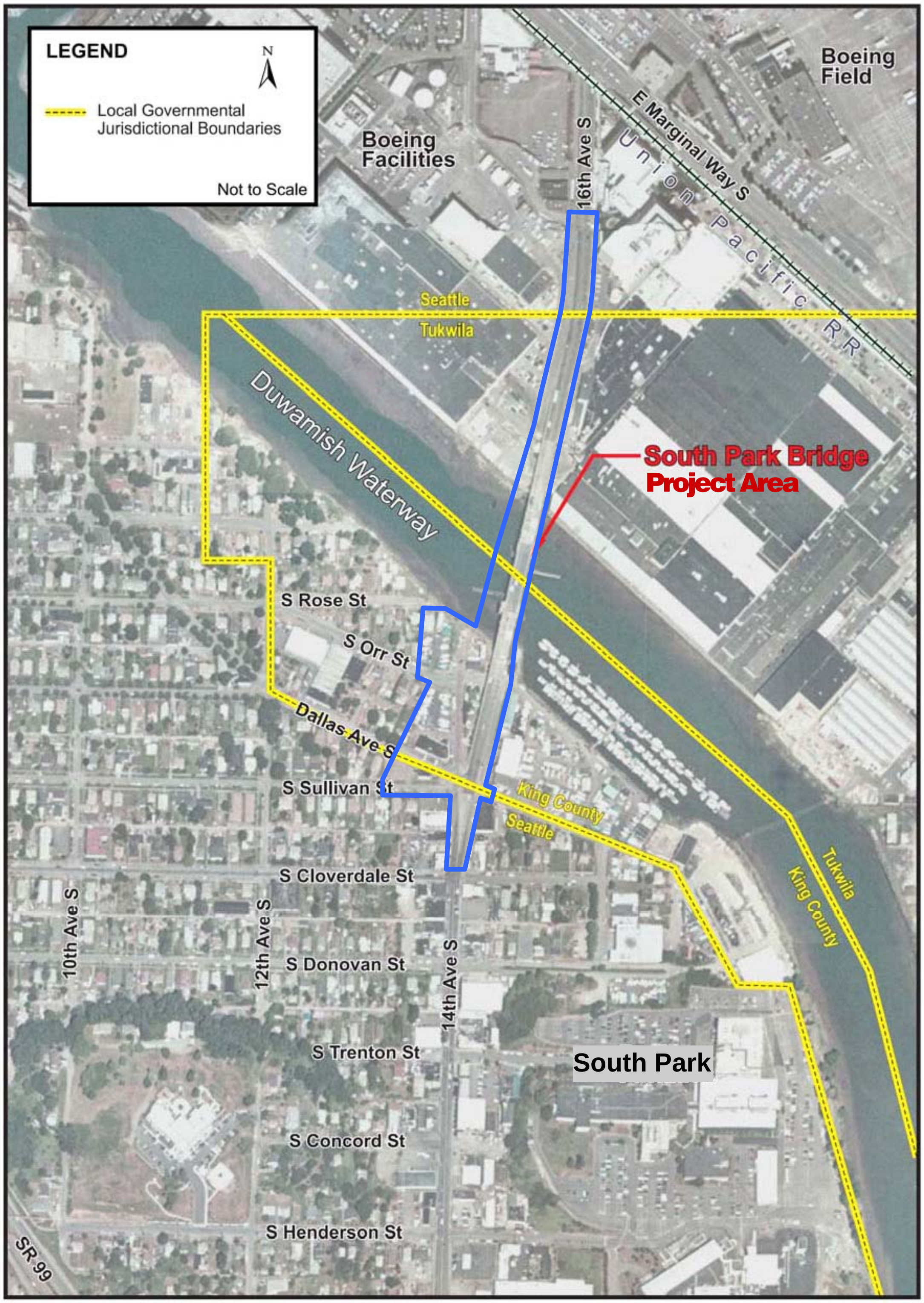
2.1 Project Location

The existing South Park Bridge No. 3179 is an 80-year-old bridge that is located in an industrial area on the Duwamish Waterway. The bridge is approximately 5 miles south of downtown Seattle as shown in Figure 1. The project encompasses the roadway corridor defined by 16th Avenue South (between East Marginal Way South and the South Park Bridge) and 14th Avenue South (between the South Park Bridge and South Cloverdale Street). The existing bridge is approximately 1,045 feet long between abutments and the overall length of the project area is approximately 2,400 feet. The existing roadway consists of four 9.5-foot lanes and a total pavement width of approximately 38 feet not including sidewalks.



Source: South Park Bridge Replacement Preliminary Design Report, PB Americas Inc., August 2007

Figure 1
Project Location



Source: South Park Bridge Replacement Preliminary Design Report, PB Americas Inc., August 2007

Figure 2
Project Area and Street Map

The project area is governed by three local government jurisdictions; King County, City of Seattle, and City of Tukwila (See Figure 2). The area between South Cloverdale Street and Dallas Avenue South is in the City of Seattle and the area between Dallas Ave South and the center of the Duwamish Waterway channel is in unincorporated King County. The project is in the City of Tukwila from the center of the Duwamish Waterway to the Section Line (32/29) and in the City of Seattle from the section line north toward East Marginal Way.

2.2 Project Limits

The project is primarily located within Township 24N, Range 4E, Section 32 with some encroachment into Sections 29, 28, 33. Land uses in the project area are mixed residential, retail commercial, and industrial. The Boeing Company Plant 2 dominates the north side of the Duwamish Waterway. Retail, commercial, and light industrial land fronts 14th Avenue South and the waterway banks on the south side. Single family residences generally characterize the area south of the bridge.

2.3 Purpose of the Project

This project proposes to construct a new bridge adjacent to the existing bridge, realign the roadway, and remove the existing bridge. The purpose of this bridge replacement project is to provide a safe facility for the public, to maintain and improve the existing waterway while minimizing environmental impacts, and to correct structural and functional deficiencies associated with the existing bridge. During the preliminary engineering phase, the bascule bridge alternative was selected for the new South Park Bridge. The bridge will be constructed downstream of the existing bridge to allow for continued traffic operations while the new bridge is under construction.

The new roadway cross-section will have four 11-foot-wide lanes, two 5-foot-wide bike lanes, and 6-foot-wide sidewalks on each side. The sidewalks will be separated from the bike lanes by a traffic barrier and will have pedestrian railings on the outside. The proposed bridge will have a solid concrete surface unlike the grated surface currently used in the existing facility.

As part of this project, the existing storm drainage system will be reconfigured to provide water quality treatment facilities, low impact development treatment methods, separate public and private conveyance systems, and two new outfalls into the Duwamish Waterway.

The project will incorporate a contemporary architectural theme and will preserve some of the historical components of the existing bridge.

2.4 Previous Studies

A number of previous studies have been conducted on the rehabilitation, retrofit, and/or replacement of the existing South Park Bridge. A partial list of previous studies can be found in the *South Park Bridge Replacement Preliminary Design Report*[24].

Studies and reports conducted subsequent to the *Preliminary Design Report* include:

- *Bascule Bridge Review, Report of Observations*, HNTB Corporation[11]
- *Evaluation of Value Engineering Proposals, Report of Findings*, HNTB[14]
- *Technical Memorandum, Bascule Pier Foundation Alternative Evaluation*, HNTB[12]

- *Addendum to South Park Bridge Project Biological Assessment*, King County[18]
- *Green Building Report of Finding*, HNTB[15]
- *Supplemental Geotechnical Data Report, South Park Bridge*, Shannon & Wilson[29]
- *Hydraulics/Stormwater Pollution Prevention Plan Technical Information Report*, HNTB[17]
- *Supplemental Geotechnical Report, South Park Bridge*, Shannon & Wilson[28]

2.5 Proposed Studies

Proposed studies and reports yet to be completed include:

- Sediment Characterization Report, South Park Bridge, Windward Environmental, LLC
- Soil Characterization Report, South Park Bridge, Southern End, Herrera, Inc.
- Hazardous Building Material Report
- Test Pile Report of Findings, Shannon & Wilson, Inc.
- Hydro-Acoustic Monitoring Report of Findings, Environ International Corporation

3 EXISTING CONDITIONS

3.1 Existing Roadway

3.1.1 Functional Classification

The section is repeated from the Preliminary Design Report and is restated to provide basic roadway information.

This section of 14th/16th Avenue South is part of a regional major arterial linking key travel routes in the South Seattle and southwest King County areas, through the South Park community, to the Duwamish manufacturing and industrial center and downtown Seattle. The bridge and 14th/16th Avenue South connect East Marginal Way South to the north and SR 99 to the south of the South Park community. This roadway is designated as a truck route by King County and carries an average of 13% truck traffic. This roadway is currently classified as a Major Arterial. The posted speed on the existing bridge is 30 mph.

3.1.2 Existing Roadway

The section is repeated from the Preliminary Design Report and is restated to provide basic roadway information.

The existing roadway section has two northbound lanes, two southbound lanes, and sidewalks on both sides. The existing roadway does not meet current design standards because the width of the lanes is too narrow at 9½ feet resulting in an overall bridge width of 52 feet including sidewalks.

3.2 Existing Bridge

3.2.1 Span Layout and Structural Types

This section remains unchanged. The reader's attention is directed to the Preliminary Design Report.

3.2.2 Existing Condition and Deficiencies

This section remains unchanged. The reader's attention is directed to the Preliminary Design Report.

3.2.3 Rehabilitation History

This section remains unchanged. The reader's attention is directed to the Preliminary Design Report.

3.3 Soil Conditions

Geotechnical investigations conducted by Shannon & Wilson, Inc. and communicated in the 2009 *Draft Supplemental Geotechnical Report*[28] and the 2009 *Supplemental Geotechnical Data Report*[29] indicate that subsurface conditions under the existing and proposed bridge alignment generally consist of man-made fill, marsh, and estuarine deposits within the upper portion of the soil profile. These deposits are underlain primarily by loose to medium dense sandy alluvial deposits and very soft to medium stiff clayey and silty estuarine deposits. Another layer of medium dense to dense sand and gravel with shells and organics may represent beach deposits locally underlying the alluvium and estuarine deposits. These normally consolidated, non-glacial soils are underlain by glacially consolidated deposits (glaciomarine drift and glaciolacustrine deposits) consisting of stiff to hard, clayey silt to silty clay with some sand and gravel.

The elevation (NAVD88) of the top of glacial soils along the proposed bridge alignment varies between approximately -45 and -105 feet. In the vicinity of proposed bridge abutment 1, the elevation of the top of glacially overridden deposits is approximately -45 feet. The elevation drops northward to approximately -105 feet in the vicinity of proposed pier 4, and then rises to approximately -80 feet below proposed bridge abutment 6.

The geotechnical report indicates that the upper, non-glacial soils have insufficient load-bearing capacity to support the bridge and have the potential to liquefy in a seismic event. Liquefaction of the upper, non-glacial soil would result in ground settlement and instability of the approach embankments. Ground improvements are required beneath the approach embankments to mitigate liquefaction and its hazards.

Sampling and analysis of the sediments within the Duwamish Waterway have been undertaken to characterize contamination. Additional sampling and analysis is planned. At the time this document was developed, the comprehensive findings of the sampling and analysis were not available.

3.4 Existing Utilities

A number of existing utilities run through the project area, including: water, sanitary and combined sewer, stormwater, steam, telephone, natural gas, fuel oil, fiber optic cables, and electrical power lines. The interested parties include King County, the City of Seattle, the Boeing Company, Qwest, Puget Sound Energy, Comcast, and Seattle Public Utilities. The only utilities that run along the current bridge alignment on 16th Avenue South are the water and sanitary sewer lines which serve the tender house. A 2-inch waterline connects to the Seattle Public Utilities 12-inch waterline on the north side of the bridge. The sanitary sewer line connection could not be tracked and was assumed to connect to the sanitary sewer system within the Boeing property that ultimately connects to the King County Metro Sewer system along East Marginal Way. A large number of utilities lie under the north bridge approach and serve Boeing Plant 2. A smaller number of utilities serve the South Park neighborhood on the south shore that conflict with the planned bridge construction and related soil improvement.

The approximate locations of existing utilities have been incorporated into the project base map. Where conflicts occur with existing utilities, potholing will be performed in July, 2009 to confirm the location of these utilities. The design team is coordinating with the Boeing Company to determine which utilities can be abandoned and which will need to be relocated.

The following existing utilities at the Boeing property will be abandoned or removed:

- 6-inch Air – Abandoned and capped at both ends of the work area
- Gas Line #1 (wharf) - Abandoned and capped at both ends of the work area
- 6-inch Chemical Line – To be removed and disposed by Boeing Company
- Industrial Waste Line – Abandoned
- Fire Protection that runs under existing and proposed bridge – Removed and capped adjacent to the Building 2-41 and 2-10

The following utilities servicing Boeing facilities will be relocated or are unaffected:

- Waterlines – Two waterlines from an existing water vault in the public right-of-way will be relocated along with the relocation of the water vault servicing Boeing Building 2-15.
- Waterline – The waterline servicing building 2-31 will be reconnected to a relocated water main.
- Steam lines – Boeing lines crossing public right of way will be unaffected but will have storm drain crossings within close proximity. Potholing will determine if there will be impacts to the stream lines.
- Underground power lines - Boeing lines crossing the public right-of-way will be unaffected but will have storm drain, sewer, and water crossings within close proximity.
- Sump pump – A pump services the pedestrian undercrossing in the public right-of-way and discharges into the storm sewer. The discharge will be connected to a relocated storm sewer line.

- Gas Line #2 (existing abutment) – This line will be abandoned in place and a new gas line will be installed on an alignment yet to be determined. The new location is being coordinated with Boeing and is likely to be located toward the north end of the project limits.

- Fiber Optic communications line – To be relocated around new bridge on Boeing property. Boeing has concerns about the downtime of the communication line.

During the construction of the bridge, power poles will be affected. Seattle City Light has agreed to provide temporary power for construction and to serve the bridge.

During the final design phase of the project, the design team will continue to coordinate with the Boeing Company, Seattle Public Utilities, Seattle City Light, Comcast, Qwest, and Puget Sound Energy.

4 PROPOSED BRIDGE REPLACEMENT

4.1 Proposed Alignment and Profile

4.1.1 Alignment

4.1.1.1 14th Avenue South /16th Avenue South

The alignment from the preliminary engineering phase of the project was designed to meet the King County's Urban Principle Arterial standards. This preliminary design alignment was challenged during the Intermediate Design phase as part of the Value Engineering Study. A proposal was made to design to the AASHTO Low-Speed Urban Street Standards in lieu of King County's Urban Principal Arterial standards and to modify the alignment to cross the navigation channel at a reduced skew that would minimize the length of the bascule bridge.

The reduced skew alignment might be feasible, but it would reduce the efficiency of the river crossing and presented the following problems:

- The combined horizontal sight distance going north out of the South Park neighborhood would be radically reduced due to the vertical curve and horizontal sweep of the alignment.

- The length of the approach spans would have to be increased.

- The combination of vertical and horizontal curves is not conducive to a minor arterial setting.

- Light pollution from headlights of southbound traffic would extend into the residential areas of South Park.

Although the reduced skew alignment was not considered, the South Park Bridge intermediate design alignment did revise the preliminary design to meet the AASHTO Low Speed Urban Street Design standards. The revision allows a minimum horizontal curve of 510 feet with a normal crown that simplifies the design at the bascule bridge.

The intermediate design alignment provides the minimum required separation between Boeing Building 2-15 and the back of sidewalk. Due to a revision in the roadway section, See Section 4.5 Proposed Typical Section, the intermediate design alignment provides adequate clearance from the existing bridge.

The intermediate design alignment accommodates the 40 feet of horizontal clearance between the back of sidewalk and Boeing Building 2-15. The 40-foot separation allows 30 feet for fire access clearance required by Boeing and a 10-foot separation between the Boeing security fence and the back of sidewalk.

4.1.1.2 South Orr Street

The intermediate design horizontal alignment smoothes out the sharp horizontal curves in the preliminary design. This will allow easier navigation along the reverse curves for vehicles with trailers while meeting the criteria for Urban Local Access Roadways - Subaccess.

4.1.1.3 South Sullivan Street/Dallas Avenue South

The location of the intersection was revised from the preliminary design to meet the minimum spacing requirements from a major intersection. The intersection of 14th Avenue South/Dallas Avenue South Sullivan Place was moved as far west as possible to provide maximum separation.

4.1.1.4 Boeing Access under the North Approach

The preliminary engineering design did not address the need for maintaining the Boeing access under the north approach. In the intermediate design, the roadway will meet the following criteria:

- Accommodate WB-40 vehicles
- Provide a roadway width of 35 feet around the horizontal curves.

4.1.2 Profile

4.1.2.1 14th Avenue South/16th Avenue South

The proposed vertical profile for 14th Avenue South/16th Avenue South has been developed to meet the following design criteria and remains consistent with the preliminary design with the exception of the minimum stopping sight distance requirement: The minimum stopping sight distance requirement for AASHTO Low Speed Urban Street Design is less than King County's Urban Principle Arterial standards.

Design Speed	35 mph
Posted Speed	30 mph
Maximum Profile Grade	5.0%
Minimum Stopping Sight Distance	250 feet

A 6.0% profile grade was considered during the Value Engineering Study and was dismissed due to the following:

- The proposal does not conform with American Disabilities Act (ADA) requiring a maximum 5.0% grade. The ADA states that within public right-of-way the sidewalk grade can follow the roadway grade as long as appropriate landings are provided, but there is high vehicular traffic in the area and the Dallas Avenue South/14th Avenue South intersection is at the terminus of the profile grade

- The high percentage of trucks using corridor (13%) will have difficulty going up the steep grade for such a short distance.

- The reduced construction costs do not yield significant savings by matching the existing grade in advance of the connection in the preliminary design. However, the intermediate design achieved cost savings at the touch down points by reducing the deck height of the bridge and lowering the roadway profile. Lowering the roadway profile allowed the intersection of 14th Avenue South/Dallas Avenue South to be raised only 1.5 feet instead of the 5.0 feet required by the preliminary design.

4.1.2.2 South Orr Street

The ORR street profile under the south approach was revised to meet the minimum 14.5-foot vertical clearance requirement

4.1.2.3 Boeing Access under the North Approach

The Boeing access under the north approach was designed to maintain a minimum 14-foot vertical clearance as required by the Boeing Company.

4.2 Street Parking

The intermediate design alignment will eliminate parking between Dallas Avenue South and South Cloverdale Street. To mitigate for the loss of parking, four parking spaces have been incorporated along the east side of this section of the roadway and parking has been added at the proposed terminus of South Sullivan Street. The proposed design will increase the total number of parking spaces by 8 spaces.

4.3 Project Datum

This section remains unchanged. The reader's attention is directed to the Preliminary Design Report.

4.4 Pavement Structure

The pavement design was completed by King County and was provided via an email dated November 10, 2008. The following pavement sections were recommended:

- 14th Avenue South/16th Avenue South

  1.0 ft minimum thickness PCC,
  0.5 ft minimum compacted thickness CSBC

- Dallas Avenue South and South Sullivan Place

  0.6 ft minimum thickness PCC
  0.5 ft minimum compacted thickness CSBC

- South Orr Street and Boat Access

  0.3 ft minimum HMA Class ½” PG 64-22
  0.7 ft minimum compacted thickness CSBC

4.5 Proposed Typical Section

The proposed typical roadway sections are shown in the Intermediate Design Plans.

The typical roadway section was revised from the Preliminary Engineering design to include sidewalks and bike lanes on both sides. The proposed bridge will have four 11-foot-wide traffic lanes, two 5-foot-wide bike lanes, and two 6-foot-wide sidewalks, all symmetrical about the centerline of the roadway. A traffic barrier will separate the bike lane and the sidewalk. The Minnesota Combination Traffic Barrier was selected to increase visibility. An artist inspired pedestrian railing is proposed for the outside edge of the sidewalk. The total width of the bridge will be 70 feet 8 inches, which is a significant improvement from the existing bridge width of 52 feet. The 4-foot-wide median included in the preliminary design was eliminated because it would not add substantial value and would require a wider bridge.

Providing a sidewalk on both sides of the roadway will accommodate bridge operation and maintenance needs as well as the natural flow of foot traffic. Providing less width by eliminating the sidewalk on one side would be counterintuitive to the long-term needs of the community and region.

Providing separate bike lanes on both sides of the roadway will allow for commuter bike traffic to travel in the same direction as vehicles and avoid conflicts with pedestrian traffic. Most commuters prefer to ride on bike lanes adjacent to the travel way. In addition, the City of Seattle *Bicycle Master Plan, June 2007* designates 14th Avenue South and 16th Avenue South as bicycle routes. According to this plan, “*bicycle facilities should be provided as part of the 16th Avenue South bridge crossing. This is a critical connection in the bicycle network*.”

4.6 ADA Compliance

The 5% maximum profile grade and the 6-foot-wide sidewalk comply with the requirements of the Americans with Disabilities Act (ADA) accessibility guidelines.

4.7 Bridge Span Layout

This section remains unchanged. The reader’s attention is directed to the Preliminary Design Report.

4.8 Navigation Channel Clearance

This section remains unchanged. The reader’s attention is directed to the Preliminary Design Report.

4.9 Traffic Signal Revision at South Cloverdale Street and 14th Avenue South

In the *Technical Transportation Report* by PB Americas Inc.[26], northbound and southbound left-turn lanes at the intersection of 14th Avenue South and South Cloverdale Street were recommended. However, the *Traffic and Transportation Analysis Memorandum* by The Transpo Group[30], indicates that this intersection would operate at acceptable levels of service through year 2030 (LOS C in the AM peak, and LOS B in the PM peak). Since the Transpo Group report contains analysis with more recent data, the recommendations of that report are warranted. Based on this same report, the City of Seattle 14th Avenue South Project improvements did not include left-turn lanes at the intersection of South Cloverdale Street and 14th Avenue South. Therefore, no left-turn lanes will be installed as part of this project as well.

4.10 Stormwater

4.10.1 Drainage Approach and Threshold Discharge Areas

The drainage approach to the project identified the threshold discharge areas, critical areas, impervious surfaces, and pervious surfaces within the existing and proposed threshold drainage areas. The areas were evaluated for both existing and proposed drainage conditions as part of the design process. A *Hydraulics/Stormwater Pollution Prevention Plan Technical Information Report* (2009) was developed for the project and documents the background data, targeted treatment areas, water quality treatment methods, and other stormwater design criteria for this project. The proposed pervious and impervious surfaces were identified, quantified, and used to determine the targeted treatment areas. Table 1 summarizes the impervious and targeted treatment areas for the project.

4.10.2 Drainage Design Treatment Goals

Tributary area and threshold discharge area storm event modeling was performed according to the 2005 *King County Surface Water Design Manual* (KCSWDM) using the King County Runoff Time Series Version 5 Program (KCRTS). The drainage design approach would provide treatment for runoff from the new impervious surface generated by the proposed project. The new impervious surface is less than 50% of the existing impervious surface, so the existing (replaced) impervious surface is not part of the targeted treatment area. The project lies within a basic water quality treatment criteria area and the Duwamish Waterway is a flow control exempt water body. According to the KCSWDM, any of the basic and/or enhanced treatment options are allowed as viable alternatives for water quality treatment facilities.

4.10.3 Drainage Design Water Quality Treatment

The preliminary design phase proposed vaults for water quality treatment. At the Eco-Charrette held during the intermediate design phase (August 6th, 2008), participants recommended that the project consider Low Impact Development (LID) or natural treatment methods. The primary goal of the Eco-Charrett was to explore natural drainage solutions and opportunities, evaluate ways to recycle materials, and utilize existing facility features in the design of a new bridge and roadway. Four LID methods were evaluated in the *Low Impact Development/Natural Drainage System Evaluation Technical Memo* (February, 2009) documenting an alternative selection and evaluation process. Where possible, the intermediate design was revised to incorporate the natural drainage treatment methods. The preferred alternative includes a rain garden and biofiltration swale as natural solutions for drainage water quality treatment.

Table 1
Impervious and Targeted Treatment Areas

	Southern Portion of the Threshold Discharge Area (Acres)	Northern Portion of the Threshold Discharge Area (Acres)	Project Total (Acres)
Existing Impervious Surface	4.25	5.33	9.58
Proposed Impervious Surface (New and Replaced)	3.55	5.52	9.07
New Impervious Surface	1.15	0.73	1.88
Replaced Impervious Surface*	2.40	4.79	7.19
Net Added Impervious Surface	-0.70	0.19	-0.51
Targeted Treatment Surface	1.15	0.73	1.88
Existing Impervious Surface Treated at West Point	1.70	0.00	1.70
Proposed Impervious Surface Treated at West Point	1.38	0.00	1.38
Total Treatment Area	**2.53**	**0.73**	**3.26**

Notes: Existing impervious area that is removed to sub-grade and then replaced
Source: HNTB Corporation, May 2009

The project will utilize a combination of water quality facilities to accommodate site constraints and achieve water quality treatment goals. The treatment methods for the project include an underground water quality wet vault, a biofiltration swale, a rain garden, and treatment via discharge to the West Point Water Quality Treatment Facility.

4.10.3.1 Wet Vault

The water quality wet vault was designed according to Section 6.4.2 of the KCSWDM as well as methods and criteria in the wet vault sizing worksheets.

4.10.3.2 Biofiltration Swale

The biofiltration swale was designed according to Section 6.3.2 and biofiltration swale worksheet manual sizing method of the KCSWDM. The biofiltration swale was identified as a potential treatment method and preferred natural drainage solution during the Eco-Charrett performed to evaluate opportunities to provide “green” design principals for this project.

4.10.3.3 Rain Garden

The rain garden is similar to a large sand filter and is classified as a basic treatment method. The design methods and criteria are based on the guidelines established in the KCSWDM and *Department of Ecology Stormwater Management Manual for Western Washington*. The project has been designed to achieve or exceed the minimum compliance criteria, core requirements, and special requirements of the KCSWDM. As designed, the project effectively achieves higher treatment goals than the minimum required treatment criteria. In addition, the rain garden has been designed to treat a larger storm event than specified in the minimum requirements (100% opposed to the minimum 60% of the 2-year storm event).

4.10.3.4 West Point Treatment Facility

The intent of the design approach is to treat runoff from the new impervious surface and not change the existing conditions for the West Point Water Quality Treatment Facility. This project improves areas tributary to the West Point Water Quality Treatment Facility, but does not increase the impervious surface area draining into the combined storm/sewer system. The project reduces the impervious surface, increases the pervious surface, and slightly reduces the total area draining into the combined storm/sewer system. This approach improves the current capacity issues experienced by the City of Seattle combined storm/sewer system.

4.10.4 Conveyance Design

The conveyance system has also been designed as a pipe network separating King County property stormwater runoff from Boeing facilities and the City of Seattle property runoff. Separating the runoff enables each entity to monitor, maintain, and treat stormwater runoff with autonomy and individual agency responsibility. A backwater analysis has been performed to evaluate system performance during high water conditions on the river. The conveyance system has been designed to accommodate conveyance for onsite and offsite runoff adhering to the methods, materials, and other criteria established in Chapter 4 of the KCSWDM.

4.10.5 Outfall Design

Two new outfalls are included as part of this project. Several studies and reports have been used for the design of the outfall systems. The following is a list of studies and reports that have been developed to support the river channel hydrologic characteristics for this design:

- *Analysis of Hydraulic Effects and Riverbed Scour*, Parametrix, Inc.[22]
- *South Park Bridge Project Geology and Soils Technical Report*, PB Americas/Shannon & Wilson[27]
- *South Park Bridge Project Water Resources Technical Report*, Parametrix Inc.[23]
- *South Park Bridge Project Fish, Wildlife, and Vegetation Technical Report*, Parametrix Inc.[20]

Several design elements were reviewed in order for the outfalls to meet the design criteria. These design elements include river velocities, flows, soils, shore stability, and flooding characteristics. The outfalls meet or exceed the minimum requirements outlined in Section 4.2.2 of the KCSWDM. The outfalls have been designed to be one foot above the mean high water surface elevation and are fitted with tide flex valves preventing water from entering the conveyance system. Rock shoring has been sized to account for slope protection and discharge scour. Anchor restraints have been proposed as additional protection.

4.11 Structural Type Section

4.11.1 The Main Span

A structure type selection was conducted during the preliminary engineering phase of the project. A double-leaf bascule movable bridge with a distance of 227 feet from centerline-of-trunnion to centerline-of-trunnion was deemed to be the preferred structure type for reasons cited in the Draft Environmental Impact Statement (DEIS).

The selection of this structure type was challenged during the Intermediate Design phase as part of a Value Engineering study. A skewed set of double-leaf bascule bridges was considered along with a single leaf bascule bridge.

The double-leaf bascule movable bridge as proposed during the preliminary engineering phase of the project was deemed to be the preferred main span structure type when evaluated with respect to environmental considerations, initial cost, operations and maintenance, seismic considerations, and technical merit.

See Section 8 BASCULE SPAN, for additional discussion related to the details of the main span structure.

4.11.2 The Approach Spans

The preliminary design of the approach structures to the moveable span was reviewed and refined to meet the project's updated design criteria, final adjustments of the roadway alignments and cross section, and architectural considerations. The review of the preliminary design confirmed and concurred with the approximate length of each approach structure, the span arrangement, and the structural types. The south approach superstructure, 303 feet in length, comprises two continuous spans, each 151'-6" long. The spans utilize precast/prestressed concrete girders (WF74G) with a cast-in-place concrete slab for the superstructure and drilled-shaft foundations for the substructure. The north approach structure comprises the same structural types and span arrangement except that the spans are 152'-10" and 152'-4" long, for a total structural length of 305'-2".

Review and refinement of the preliminary design incorporated a change in the seismic design criteria and updated geotechnical information. The updated seismic criteria required a more rigorous analysis for the 975-year earthquake event. Coupled with the architectural considerations, revisions were made to the configuration of the intermediate piers, including the shape and size of the pier cap, the size and number of columns and the size and number of drilled shafts

4.11.3 Structural Earth Walls

The preliminary structural system design for the roadway embankment to the north and the south approach structures was reviewed and refined. Updated geotechnical information indicates that the Structural Earth Walls (SEW) will be located in an area where large soil settlement is anticipated. According to the geotechnical investigation, embankment settlements of up to 7 inches are estimated for the north approach, and 5 inches for the south. Another consideration is the architectural design of the wall surface treatment. The architectural fascia panels are designed with rustication to accentuate the horizontal joints. Consequently, minimizing the horizontal differential settlement of the wall is a primary concern.

Including these new design constraints in the review of the preliminary design concept resulted in a confirmation that SEW is still the appropriate wall type. However, some additional design considerations and provisions will need to be addressed. A two-stage wall construction is recommended to mitigate the settlement issue. In stage one, the approach earth embankments would be constructed with a SEW system. The settlements of the embankments would be monitored until approximately 90% of the total anticipated settlement is reached. It is estimated that this would occur within one month at the north embankment and within two months at the south embankment. The remaining settlement would occur over the course of six-months. With the installation of earthquake drains, the rate of the ground settlement may occur faster than anticipated.

Upon reaching 90% settlement, the precast fascia panels would be installed as part of stage two. We expect the remaining settlement, if any, would be sufficiently small and would not create noticeable or undesirable miss-alignment of the horizontal rustication between fascia panels.

The updated geotechnical engineering investigation also concludes that both proposed south and north approach embankments with SEW systems will be susceptible to damage under the considered 975-year earthquake event due to liquefaction of the soil layers underneath the wall foundation mats. Consequently, ground improvement by installing earthquake drains is recommended by the project geotechnical engineer, Shannon & Wilson, Inc. Drains are specified to mitigate the liquefaction phenomenon of the soil layers for the wall where the height will be 10 feet or more.

An alternative to the earthquake drains is to extend the proposed two-span approach structures to span the problematic areas that require ground improvement. This could be achieved by moving the current abutments to locations where the wall height would be less than 10 feet; and adding an intermediate pier at each approach. The intermediate piers would have three 8-foot diameter shafts that would penetrate through the liquefiable layers into the competent soil layers below. About a 100-foot embedment of the shafts would be required.

Table 2
Construction Cost Comparison for the Approach Embankments options

<table>
<tr><th colspan="2" rowspan="3">Item Descriptions</th><th colspan="2">Option 1</th><th colspan="2">Option 2</th></tr>
<tr><th colspan="2">MSE Wall with
EQ Drains</th><th colspan="2">MSE Wall with Bridge
Extension</th></tr>
<tr><th>Quantities</th><th>Cost Estimate</th><th>Quantities</th><th>Cost Estimate</th></tr>
<tr><td rowspan="4">MSE Walls</td><td>SE Wall</td><td>154 ft</td><td rowspan="4">$1,162,000</td><td>14 ft</td><td rowspan="4">$376,000</td></tr>
<tr><td>SW Wall</td><td>159 ft</td><td>19 ft</td></tr>
<tr><td>NE Wall</td><td>292 ft</td><td>152 ft</td></tr>
<tr><td>NW Wall</td><td>308 ft</td><td>168 ft</td></tr>
<tr><td rowspan="2">Earthquake Drains</td><td>South Approach</td><td>90 ft x 138 ft,
55 ft deep</td><td rowspan="2">$871,000</td><td>0</td><td rowspan="2">$0</td></tr>
<tr><td>North Approach</td><td>90 ft x 138 ft,
60 ft deep</td><td>0</td></tr>
<tr><td rowspan="2">Bridge Span</td><td>South Approach</td><td>0</td><td rowspan="2">$0</td><td>140 ft</td><td rowspan="2">$4,224,000</td></tr>
<tr><td>North Approach</td><td>0</td><td>140 ft</td></tr>
<tr><td colspan="2">Total Construction Cost</td><td colspan="2">$2,033,000</td><td colspan="2">$4,600,000</td></tr>
</table>

Source: ABKJ, June 2009

For comparison, construction cost estimates of these two options are summarized in Table 2. The construction cost for the extended spans is estimated to be $4.6 million compared to $2.0 million for the walls with ground improvement. The proposed two-span approach structure connected with the SEW walls with earthquake drain ground improvement is the most economic option.

A third approach option of a single-span structure with extended roadway embankment encroaching on the river bank was also considered, but was dismissed for environmental considerations.

5 DESIGN CRITERIA AND SPECIFICATIONS

5.1 Design Specifications and Guidelines

The design specifications and guidelines are addressed in the project specific *Design Requirements for the South Park Bridge – Final Design*[13]. The reader's attention is directed to that document for additional detailed information and to Section 5.2.1 for structural design specifications.

5.2 Seismic Design Criteria

Seismic design of the bascule span of the South Park Bridge generally follows AASHTO's *LRFD Movable Highway Bridge Design Specifications* which is a force-based design approach.

5.2.1 Major Design Guidance

The following design guidance documents were used in the seismic design of the bascule span:

- *LRFD Movable Highway Bridge Design Specifications* 2nd Edition, AASHTO[2].
- *AASHTO LRFD Bridge Design Specifications*[1]
- *Bridge Design Manual LRFD*, WSDOT[31]

5.2.2 Seismicity

The South Park Bridge is located in a moderately active tectonic province that has been subjected to numerous earthquakes of low to moderate strength and occasionally to strong shocks during the brief 165-year record history in the Pacific Northwest. Seismicity in the region is attributed primarily to the interaction between the Pacific, Juan de Fuci, and North American plates.

5.2.3 Design Earthquakes

The bridge is designed using two levels of earthquakes: design and operational.

Design Earthquake: An upper level seismic event that has ground motions corresponding to a 7.5% probability of being exceeded in 75 years, or an approximate return period of 975 years.

Operational Earthquake: A lower level seismic event that has ground motions corresponding to 50% probability of being exceeded in 75 years, or an approximate return period of 108 years.

One-half of the seismic loads due to the design level earthquake are used for positions other than the closed position in accordance with AASHTO specifications. Seismic loads resulting from operational level earthquake is not reduced. During bridge operation, seismic effects on operating components are ignored.

5.2.4 Site-Specific Response Spectra Curves

Shannon & Wilson Inc. performed seismic hazard studies and recommended that the reference ground motion for the "no collapse" seismic design of the bridge be based on a 975-year return period. A functional level reference motion was also recommended based on a 108-year return period.

The horizontal and vertical reference ground motions were developed by Shannon & Wilson Inc. and are presented in the *Supplemental Geotechnical Report*[28]. Based on the subsurface conditions at the project site, the reference ground motion for Site Class E is interpreted to represent the shaking level near the ground surface.

5.2.5 Site-Specific Time Histories

The details of the development of the time histories are presented in *Technical Memorandum – Caisson Analysis of South Park Bridge*[6]. The reference ground motion for the project site, developed by Shannon & Wilson Inc., was based on probabilistic seismic hazard analysis. To provide insight into what events (magnitude, distance, epsilon) are the most important for the hazard at a given ground motion level, the hazard curve can be broken down into its contributions from different earthquake scenarios. This process is called deaggregation. Earth Mechanics Inc. (EMI) conducted the deaggregation analysis to assist selection of appropriate acceleration time histories for spectrum matching. The deaggregation results for the PGA and the 1-second period spectral acceleration at are provided in the technical memorandum[6]. Based on the deaggregation analysis, the seismic hazard is dominated by magnitude 6.0-7.5 earthquakes at distances of 6-12 miles from the bridge site. From these scenarios, the following natural earthquake records were selected as seed motions for the 975-year spectrum matching:

- Set 1: 1979 Imperial Valley Earthquake at Array 6 in Plaster City
- Set 2: 1989 Loma Prieta Earthquake at Fremont - Mission
- Set 3: 1976 Gazli Earthquake at Karakyr
- Set 4: 1994 Northridge Earthquake at LA - Centinela St

In addition to meeting the magnitude and distance criteria, considerations were given such that the selected seed motions should have a spectral shape that closely matches the reference spectrum. For this, EMI employed a computer program to search the seed motions; the results are provided in the technical memorandum[6]. The acceleration spectra of the horizontal components of these four seed motions had been adjusted to match the design acceleration spectra as shown in Fig. 6 to Fig.17[6].

The fault normal (FN) component was used for the bridge longitudinal direction and the fault parallel (FP) component was for the bridge transverse direction. Only horizontal FN and FP components were used in wave scattering (site response) analysis; the vertical components were used directly in the bridge structure analysis.

Additional one set of seed motion was selected for the 108-year spectrum matching as shown in Fig. 18 to Fig. 20[6].

No lateral spreading is expected for the bascule pier foundations as a result of liquefaction mitigation measures.

Since the caissons supporting the main bridge will be embedded deep below the ground surface, the effective shaking to the bridge would come from ground shaking at some depth, probably closer to the caisson bottom. A rigorous approach was employed considering depth-varying motions along the caisson depth adhering to a wave propagation theory.

5.2.6 Seismic Performance Criteria

Seismic design of the bascule spans of the South Park Bridge considered both the Design Earthquake (upper level) and Operational Earthquake (lower level).

5.2.6.1 Design Earthquake

After a design level earthquake, the damage to the bridge may be described as "minor to moderate damage with some loss of operation." Moderate damage implies visible and significant signs of damage with repairs or stabilization likely to be completed under emergency contracts and is characterized by:

- Minimal damage to the superstructure.
- Limited damage to piers including yielding of reinforcement and spalling of concrete cover.
- Minimal damage to caissons and caisson caps.
- Small permanent deformations, not interfering with serviceability of the bridge.
- Damage to expansion joints that can be temporarily bridged with steel plates.

Design earthquake characteristics for mechanical machinery are as follows:

- No damage is permitted to motor, trunnion, rack, and rack pinion shaft.
- Minimal damages are permitted to rack pinion bearing support.
- Moderate damages are permitted to rack pinion bearing shaft coupling, center locks, and live load shoes.

All the damaged components of the machinery shall be replaceable.

Limitations for the design level earthquake expressed in terms of permanent lateral displacement or drift resulting from soil deformations at the pier from the non-linear time history analysis are as follows:

- 3" between top and bottom of caisson in horizontal directions
- 3" at bottom of caisson in horizontal and vertical directions

Current condition of the river floor and one half (1/2) the full depth of scour from the scour analysis shall be considered with the design level earthquake.

5.2.6.2 Operational Earthquake

After an operational level earthquake the bridge may suffer "no loss of operations," with minor damage to the structure. Minor damage implies essentially elastic performance, and is characterized by:

- Minor inelastic response

- Narrow cracking in concrete
- No apparent permanent deformations
- Damage to expansion joints that can be bridged with steel plate
- Caisson settlement under the operational level earthquake should not exceed 1.0 inch.

To protect the operating components, restraint devices will be provided at the counterweight and live load shoes. The restraint system will provide fixity in the transverse and downward vertical directions at the tail of the counterweight and the live load shoes in the fully closed position. Transverse restraint will also be provided at the tail of the counterweight in the fully open position. These devices have been designed and detailed to resist the forces resulting from the design level earthquake (with appropriate response factors) discussed in the previous section.

5.2.7 Response Modification Factors

The response modification factors (R-factors) are designed to achieve the performance goals outlined in the previous sections. R-factors are used to scale down the elastic forces from the linear response spectrum analysis to account for the energy dissipation capacity of the structure. The structural components will experience nonlinear behaviors such as cracking of concrete and some yielding of steel during a seismic event.

R-factors for steel superstructure components should be:

- 1.0 for main structural members and connections for mechanical and hydraulic components;
- 0.8 for critical connections, including trunnion joints, shafts, counterweight sheaves, and counterweight supports; and
- 1.2 for secondary members for the design level earthquake.

The R-factor for caissons and pier walls should be 1.5 for the design level earthquake.

R-factors greater than 1.0 should not be used for the operational level earthquake.

5.3 Seismic Analysis

Demands on structural components of the bridge are determined by the analysis of global three dimensional computer models of the bridge that represent its dominant linear and nonlinear behavior and the effects of soil-structure interaction. Demands are evaluated as load-type quantities (forces and moments) or as displacement-type quantities (displacements, relative displacements, and rotations) as required by the evaluation rules for various components.

Seismic demands are generally determined by nonlinear, multi-support, dynamic, time-history analysis. Four sets of ground motions are used for the design level earthquakes; one set of ground motions is used for operational level earthquakes. The design is based on the maximum response obtained from these analyses in conjunction with the performance goals for the two levels of events. The model was developed with due consideration to the load path described in *LRFD Movable Highway Bridge Design Specification*[2] Section 3.5.2.2 for trunnion bascule spans, where the lateral load is resisted by the lateral bracing system; transferred to the girders; then through the trunnion, trunnion bearing, and trunnion support; and finally into the pier. Damping and stiffness from any additional span restraints was also be included in the model. Participation of secondary machinery such as span locks and centering devices was considered.

The nonlinear structural model explicitly considers the geometric nonlinearity, nonlinear boundary conditions, other inelastic elements, and inelastic structural components (if any).

Rayleigh damping is incorporated into the model with values for each element group representing the expected extent of inelastic energy dissipation in that group.

5.4 Soil-Structure Interaction

Soil-structure interaction is considered using nonlinear springs in the global model. The properties of the springs are determined from local models. To generate the soil spring representing the soil-structure interaction in the global model, behavior of the caisson was modeled using an elastic theory approach as well as a nonlinear pushover approach. The details of the development of the non-linear springs are presented in *Technical Memorandum – Caisson Analysis of South Bridge Park Bridge*[13].

5.5 Project Specific Design Criteria

The project specific design criteria have been documented in the *Design Requirements for the South Park Bridge – Final Design*[13] document. This is a living document that will be updated as necessary through out design development phase of the project.

6 FOUNDATIONS AND GROUND IMPROVEMENTS

6.1 Ground Improvements

Geotechnical investigation at the project site conducted by Shannon & Wilson, Inc. indicated the potential risk of soil liquefaction and associated settlement and lateral spreading that would impact the approach embankments. Ground improvements using earthquake drains were recommended beneath and adjacent to the approach embankments in the 2009 Draft Supplemental Geotechnical Report[28] to mitigate these effects.

Lateral spreading could also induce undesirable lateral loads on the foundations supporting new bridge piers and bascules. The preliminary engineering report proposed a block of improved soil be installed around the foundation elements to resist lateral spreading loads. Under the current design, caissons are proposed to support the bascule piers and the drilled shafts are proposed to support the approach piers. The caissons and the drilled shafts have been designed to resist anticipated lateral spreading forces and ground improvement around the bridge foundation elements will not be required.

The recommended ground improvement areas are shown in the Intermediate Design Plans. The recommendation for drain layout can be found in Section 13.4.2 of this document.

6.2 Bascule Pier Foundations

6.2.1 Deep Foundation Selection

Caissons have historically been used as economical foundations in poor soil conditions for numerous bridge projects. In recent bridge construction history, open caissons were proven to be economical when compared to drilled shafts for bridge structures such as the new Tacoma Narrows Bridge, Bill E. Emerson Bridge, and several other bridges over the Mississippi River.

According to the geotechnical report[25], at the locations of the bascule piers, the competent soil layer lies very deep below the ground surface and the soil layers above it are primarily silty sands which do not provide adequate vertical or lateral resistance.

In order to reach the competent layer and obtain capacity, drilled shafts would have to be very long with considerable lengths of the shafts unsupported in the water and loose soil. Lateral loads such as seismic forces will subject the shafts to significant bending and shear, consequently requiring a very large shaft cross section with expensive permanent steel casings as proposed in the Preliminary Design[24]. As an alternative, caissons primarily rely on soil bearing at the bottom and sides of the caisson to resist the applied loads. Caissons are especially efficient for resisting large seismic overturning forces applied to them by controlled foundation rocking.

A more detailed comparision was conducted by HNTB and the study and conclusions were presented in *South Park Bridge Replacement - Bascule Pier Foundation Alternatives Evaluation*[12].

6.2.2 Caisson Construction Sequence

Caissons are built by controlled sinking through water and soil until the desired tip elevation is reached. For the South Park Bridge caisson, located in relatively shallow water, a sand-island construction method is envisioned. The sand island method assumes the construction of a temporary island through which the caisson will be sunk to the desired tip elevation. The sand-island serves as a working platform, enabling forming and concreting operations to be performed at the ground level with conventional materials and equipment. The sand island will be confined by a steel sheet-pile cofferdam which is then filled with sand. The steel "cutting edge" caisson tip can be fabricated off-site, barged to the site and assembled on top of the sand island.

On top of the cutting edge, reinforced concrete caissons will be cast in sections enabling controlled sinking of the caisson. The cross section is such that dredge wells are created in a grid or other configuration that will allow access from the top all the way to the riverbed. As the cutting edge sinks into the riverbed, clamshell buckets, which are mounted from a crane on a barge or on temporary trestles, excavate soil from the riverbed via the dredge wells, allowing the cutting edge to sink down into the soil. This process will proceed until the cutting edge reaches a required depth (tip elevation). The caisson will then be sealed by a blocks of concrete placed in the bottom of the dredge wells. The seal concrete will be placed under water by the tremie method.

Since the new caissons are located very close to the existing piers, there is a potential for settlement of the existing bridge due to construction of the new caissons. Use of a sheet-pile cofferdam surrounding the perimeter of the caisson will mitigate such movements. Potential lateral movement of soil from the dredging operation can be reduced by increasing the flexural stiffness of the sheet piles. Without the sheet-pile cofferdam, the adjacent existing piers could experience lateral displacement due to their proximity to the sinking caisson.

The cofferdam will also help confine the settlement due to construction of the sand island within the footprint of the cofferdam. This will mitigate propagation of ground settlement laterally away from the cofferdam, hence reducing the settlement of the existing bridge. The settlement at the bottom of the caisson due to construction of the sand island was estimated to be about 0.3 inch[7].

The footprint of the cofferdams is anticipated to be about 6 feet wider than that of the caisson on each side. This will locate the cofferdams entirely outside of the navigation channel and behind the pier protection, thereby, minimizing the impact on the hydraulic opening at the bridge site.

6.2.3 Caisson Cross Section and Layout

The caisson cross sections were preliminarily sized for expected seismic loading by checking the bearing at the bottom of the caisson and skin friction between caisson skin and surrounding soil. The cross section has to be large enough to provide a resulting bearing pressure less than the allowable; the dredging wells have to be large enough for dredging operation to take place; and the caisson has to be heavy enough to sink under its own weight. The cross sections have been sized to be 58’ x 58’ for both caissons. The bottoms of the caissons are expected to be at elevation -105 feet for the north caisson and -70 feet for the south caisson. The size of the caisson cap for the caisson will be 60’ x 90’ to accommodate the bascule pier and control tower.

6.3 Approach Span Foundations

The geotechnical limitation of the upper 50 feet of liquefiable soil layers accompanied by a potential of lateral spreading hazards under the considered earthquakes called for the use of deep foundations at the intermediate piers and abutments of the approach spans. The intermediate piers and abutments of the approach spans will be constructed without pile caps to limit the amount of potentially contaminated soil that will need to be removed from the site. Meanwhile to avoid the risk of causing damage to the existing bascule bridge during construction, the use of driven piles have been eliminated. Drilled-shaft foundations were deemed the most appropriate among the available foundation systems. Ground improvement is not anticipated or assumed in the design of the approach span foundations.

7 BASCULE PIERS

7.1 Pier Layout

The layout of the bascule piers is generally in accordance with the Preliminary Design. The layout provides a channel clearance of 125 feet compared to the existing channel clearance of 118 feet. The distance between the centerlines of the trunnions is 230 feet and the distance between the centerline of the bascule piers is 252 feet. Unlimited vertical clearance is provided for a minimum width of 165 feet measured along the bridge when the bridge is in the full open position.

7.1.1 Pier Elevation

The bascule piers built on caisson caps that are supported by sunken caisson foundations. The pier floor (top of the caisson cap) was set at elevation -3 feet. This elevation is low enough to accommodate the counter weight inside the piers but as high as possible to be close to the Mean High Water and minimize water head and the potential for water leaking into the piers.

The bascule piers mainly consist of the following floors from low to high elevations:

- Footing level (top of caisson cap)
- Machine level
- Equipment level

- Roadway level
- Operator level

7.1.2 Maintenance Access

Maintenance access will be provided to all levels of the bascule piers and the trunnion bearings by stairways and walkway. The stairways were designed with consideration of the International Building Code (IBC) and Occupational Safety and Health Administration (OSHA) requirements.. Access to the roadway level from within the bascule pier on the side opposite the control tower will be provided by a stairway and a access hatch (floor door) to avoid the need for maintenance staff crossing the roadway. Maintenance access will be provided between bascule spans, when the bridge is in the closed position, by maintenance walkways and openings within the floor beams.

A minimum 6'-8" height clearance will be provided for personnel where possible. A vertical clearance of 8 to 10 feet will be provided for machinery maintenance activities, such as equipment removal and replacement, where practical. Overhead, stationary, grated-galvanized-steel walkways may have to be removed for some of the equipment removal and replacement operations. A 3-foot horizontal clearance is desired to access machinery and provided were practical. Lifting equipment and pick points will be provided, where possible, to aid in machinery maintenance.

7.1.3 Trunnion Support Layout

The reader's attention is directed to section 8.1.5.

7.2 Fixed Deck on Bascule Pier

The underslung counterweight configuration (Section 8.1.2.5) requires a separate riding surface supported on the pier to carry traffic between the approach spans and the bascule span. The following guidelines have been used to design and detail the counterweight span:

- Match the cross-section of the approach and bascule spans.

- Transfer only horizontal and vertical reactions to the pier (i.e., provide a simply supported structure).

- Minimize the depth of the counterweight deck to maximize available space within the pier for the counterweight.

- Minimize the distance from the counterweight deck to the bascule deck in the open position to reduce the potential for accidental vehicular access to the interior of the pier. This scenario would afford little chance of survivability for the occupants of the vehicle and significant potential for damage to the bridge with extended downtime.

- Minimize live load deflection at midspan of the counterweight deck (longitudinal centerline of the bridge) for compatibility at the interfaces with the bascule and approach spans.

- Simplify layout of joints around the counterweight span.

The counterweight deck will span transversely across the width of the pier and be supported on longitudinal interior pier walls that will also serve as the limits of the counterweight pit. The bearing-to-bearing span length is 68’-0”. This distance will provide adequate space for the counterweight, bascule girders, and roughly six feet of clearance on both sides.

The deck will be normal-weight reinforced concrete supported on closely spaced floorbeams with uniformly spaced diaphragms for significant force distribution between the floorbeams. Diaphragms will also be provided at the traffic barrier and bearing locations. Tapered overhang brackets will support the joint at the bascule span deck to provide clearance as the bascule deck rotates under the counterweight deck during operation of the span. To keep deflections to a minimum, the optional live load deflection and span-to-depth ratio criteria from *LRFD Bridge Design Specifications*[1] Section 2.5.2.6 have been used as guidance in proportioning the span. The floorbeams will be designed and detailed as composite to utilize the bending stiffness of the concrete deck slab in compression, and they will be cambered to provide a level bottom flange under dead load.

7.3 Control Towers

Two control towers will be provided on the west and east sides of the north and south bascule piers respectively. The control tower on the west side of the north pier will be use to house the bridge tender for operation of the bridge. The requested minimum size of the operator’s room was 16’ square. The design development lead to a control tower footprint of 15’ x 30’ with the corners of the structure beveled 45 degree. Amenities will include a kitchen, bathroom, and storage. The control tower height will allow a tender to view traffic and bridge gates from about 17 feet above the roadway level. A walkway will be provided around the control towers and each tower will be appropriately glazed to allow the operator to view pedestrians, vehicular and marine traffic. Stairways will be provided for vertical transportation. Elevators will not be provided. The design of the control towers will recognize the technical aspects of the International Building Code (IBC) as a matter of best practice.

7.4 Structural Layout Effects from the Span Drive System

The layout of the span drive system is presented on a drawing in Appendix A. The drive system will consist of two main drive motors connected to a central reducer. The central reducer will have a differential gear set to assure that the same torque is supplied to the floating shafts that connect the central reducer to the secondary reducers. The differential gear set will require no additional maintenance. It will be enclosed in the reducer and lubricated in the same manner as the other gear sets.

Since each secondary reducer will have the same speed and input torque, the torque to the pinion and racks will be equal. The reducers will transfer a significant amount of torque into the machinery floor, and a support column will be added under the floor to react this torque.

At the start of the intermediate design phase, King County performed a value engineering study. One proposal suggested that hydraulic cylinders be used to drive the span in lieu of a gear driven system. A discussion of the advantages and disadvantages of a gear driven system and a hydraulic system is presented in the following section.

7.4.1 Span Drive Layout Options

7.4.1.1 Option 1 – Gear Driven

Option 1 would raise and lower the bascule span using a gear driven system. This option would use girder mounted racks and drive pinions with mechanical load sharing via cross shafts and a differential gear set in the central reducer.

- The entire drive system would be mounted above the MHW level
- The channel side pier wall would be located to provide the space for the system
- The gears would be housed in enclosed reducers. An oil bath would provide the necessary lubrication for gears and bearings. The rack and pinion gear would need to be lubricated with open gear grease.
- The initial cost for the gear driven system is estimated to be $4,670,000.

7.4.1.2 Option 2 –Hydraulic Cylinders

Option 2 would use pressurized hydraulic cylinders to raise and lower the span. The two cylinders would be pressure balances. Compared with Option 1:

- The machinery for a hydraulic system could be located remotely with rigid piping that runs to each cylinder.
- The initial cost of a hydraulic system would be less than a gear driven system. This system is estimated at $3,040,000 which includes the additional structural cost for the hydraulic torque tube.
- A remote possibility exists that the hydraulic system could inadvertently discharge hydraulic fluid into the environment.
- Hydraulic machinery would be inherently maintenance-intensive.
- The hydraulic system would be superior to the electro-mechanical system with respect to seismic considerations primarily because the hydraulic cylinders would articulate at their connections thus providing flexibility

7.4.2 Span Drive Selection

The gear driven system (Option 1) was chosen by King County and the City of Seattle as the preferred system. The primary reason was maintenance. The hydraulic systems would require more frequent maintenance and would require component replacement long before the gear driven system. Therefore, the life-cycle cost of the hydraulic drive system would be significantly higher. The susceptibility to seismic events was considered; however, due to the good performance of other gear driven bascule bridges in the immediate area, seismic events were not a factor.

8 BASCULE SPAN

8.1 Bascule Geometry and Configuration

8.1.1 Structure Type and Layout Study

During the preliminary engineering phase, a study was conducted by PB Americas, Inc. to compare the feasibility of both a Scherzer rolling lift bridge and a fixed-trunnion bascule bridge. The results of this study indicate that a double-leaf fixed-trunnion bascule is the preferred type for the replacement structure[24]. These results were confirmed as part of the value engineering evaluation performed by HNTB[14]. The span arrangement of the intermediate design deviates only slightly from the design concept presented in the referenced preliminary design report, and these minor differences are the result of implementing value engineering concepts that serve to provide a more economical and efficient structure.

8.1.2 Bascule Leaves

8.1.2.1 Deck

The deck system used for this bridge will be a lightweight overfilled grid deck. Structural lightweight concrete will be used to reduce the weight of the superstructure, thereby reducing the size of the counterweight and improve the seismic performance of the bridge. The relatively close floorbeam spacing (subsequent discussion) will permit the use of a commercial-off-the-shelf product for the deck without the use of stringers. This approach will increase the biddability of this item and avoid the proprietary difficulties of an exodermic deck system.

8.1.2.2 Deck Drainage

Stormwater runoff will not be collected on the bascule span. It will accumulate in the roadway gutter for conveyance off the ends of the span in the closed position to be collected at inlets in the approach spans. In the open position, water will simply run off the span into the counterweight pit where it will collect in the bascule pit and be pumped to the sanitary sewer system. The stormwater runoff quantity of a 2-year event (approximately 0.1cfs) will be assumed for design.

8.1.2.3 Floorbeams

The deck will be directly supported on floorbeams that will be spaced closely enough to eliminate the need for stringers. A maintenance walkway will be provided along the centerline of the bridge. To avoid increasing the bascule girder depth and to provide adequate clearance for the walkway beneath the floorbeams, the selected floorbeam depth will match the girder depth at the toe and access holes will be provided through the web of each floorbeam. The floorbeam bottom flanges will be level. The top flanges will be sloped to match the roadway cross-slope and facilitate installation of the deck. The floorbeam webs will be oriented perpendicular to grade to simplify the end connections at the bascule girders.

Shallower floorbeams will be used in the vicinity of the trunnion where passage through the web will not be required and clearance to the machinery platform in the open position will be critical.

8.1.2.4 Girders

The design of the bascule girder has been conceived with the following principles in mind:

- Utilize the benefits of plate girder construction.
- Maintain the historical nature and visual character of the existing truss.

Fabrication of a plate girder superstructure will be significantly less expensive and more efficient than a truss for this span. A conventional plate girder, modified with triangular perforations in the web to emulate a truss, will adhere to both of these principles. The resulting structural system will give the appearance of a truss without the complexities of numerous bolted connections and complex framing at every panel point.

The design of the “trussed” portion of the girder will be similar to that of a conventional truss. The flanges and web material at the top and bottom of the girder will act as the chords of the truss while the remaining web material will serve as the diagonals and verticals. The diagonals and verticals will be stiffened perpendicularly to the plane of the web for additional cross-sectional area and out-of-plane stiffness. The stiffener on the vertical will also serve as the floorbeam connection plate. The end product will effectively be a truss composed of tee-section members (the diagonals and verticals will be rotated 90 degrees relative to the chords) with integral gusset plates at the joints that will not require intricate framing and bolted connections.

The primary difference between the trussed plate girder and a conventional truss is that the secondary bending response in the truss elements will be significant due to full fixity at the joints. The bending stiffness of each element will contribute to the overall stiffness of the structure. As a result, all elements of the girder must be evaluated under the effects of combined axial and flexural behavior instead of axial behavior alone.

8.1.2.5 Counterweight

Two options were considered for configuration of the counterweight: 1) drive-on and 2) underslung. The drive-on option would accommodate traffic and provide the benefit of reducing the size of the counterweight, reducing the size of the pier, and eliminating the need for a counterweight deck structure. For medium- to long-span bascule bridges that inherently require a long counterweight, however, the safety concerns associated with a lengthy longitudinal cavity when the bridge is open typically outweigh these benefits when other protective measures are not in place. As a result, the underslung configuration was identified as the preferred option for this bridge.

The counterweight will be comprised of conventional steel framing filled with heavyweight concrete to balance the span. Two levels of pockets will be provided on the trunnion side of the counterweight to provide for weight and center of gravity adjustment. The width of the counterweight will be reduced at the bottom to pass between the trunnion columns when the span operates, and seismic restraints will be provided to engage the counterweight in both the fully open and closed positions.

8.1.3 Bascule Layout

The typical roadway section for the project will be carried across the bridge. The roadway section is four 11-foot-wide vehicular lanes and two 5-foot-wide bicycle lanes (at the same level of the roadway), flanked by twin 6-foot-wide sidewalks. Combination traffic barriers will separate the sidewalk from the roadway/bikeway and an artistic pedestrian railing will provide protection for pedestrians at the fascia. The section is symmetric about the centerline of the bridge. This symmetric section simplifies geometry, shop drawings, fabrication, erection, and balancing of the entire bascule span since detailing for all of the major structural steel elements is similar with respect to a group of elements (e.g., bascule girders, floorbeams, overhang brackets, bracing, etc.).

Each leaf will be primarily supported by two bascule girders. The following dimensions define the balance and equilibrium conditions of a single leaf:

- Centerline of trunnion to toe of leaf: 115’-0”
- Centerline of trunnion to live load bearing: 16’-3”
- Centerline of trunnion to back of counterweight: 35’-0”
- Total length from back of counterweight to toe of leaf: 150’-0”

The main girders will be spaced at 51’-0”, symmetric with respect to the centerline of the bridge. Typical floorbeam spacing from the live load shoe to the toe of the leaf (midspan) will be 8’-2”, and floorbeam spacing in the vicinity of the trunnion will be 6’-0”. The overfilled grid deck wll be supported directly on the floorbeams.

The concrete-filled grid deck will be the primary lateral force-resisting component of the bridge. A supplemental bracing system will be provided between the bottom flanges of the bascule girders that will also serve as compression flange bracing for vertical loads.

Two span locks—one near each girder—will be provided at the end floorbeam of one leaf, designed to engage openings in the end floorbeam of the opposite leaf. Maintenance access for the span locks will be provided via a walkway along the centerline of each leaf, a platform between the last two floorbeams on each leaf, and openings in the end floorbeams near each device.

8.1.4 Tail Locks

A tail lock will be provided at the rear face of the counterweight at the centerline of the bridge. When the bridge is in the closed position, the tail lock will be engaged to resist vertical and transverse motions of the counterweight during a seismic event.

8.1.5 Trunnion Supports

Trunnion-type bascule spans can utilize either a single-bearing or a double-bearing support system at the trunnion. The benefit of a single-bearing support would be a reduction in the number of bearings requiring maintenance and minimal structure interference with the counterweight in the open position. On the other hand, the bearings would be very large and a stiff trunnion girder would be required. The trunnion girder would be non-redundant and sensitive to distortion during a seismic event. This sensitivity renders it less than desirable for use in high-seismic regions because repair or replacement of the trunnion girder would pose a major hindrance to normal operation of the span.

A double-bearing system is, therefore, the preferred alternative. The individual bearings will be supported on individual columns on each side of each bascule girder. With two bearings at each girder, the bearings will be smaller than the single-bearing alternate and the trunnion girder will be eliminated. This support system will be less sensitive to distortion during an earthquake and, in the event unacceptable damage were sustained, the individual bearings could be repaired or replaced without a significant disruption to operation of the span. A disadvantage of this arrangement is the addition of trunnion columns inboard of each bascule girder that presents a conflict with the counterweight. This interference will be addressed by notching the counterweight, effectively reducing its width so it passes between the trunnion columns as the leaves rotate to the open position.

8.1.6 Constructability

The bascule leaf design will incorporate a girder field splice on the channel side of the live load shoe in order to permit phased erection of the bascule leaves. With the splice in this position, the pier segments of the leaves can be fully erected and supported at the trunnion and live load shoes. Span segment assembly can take place concurrent with pier segment erection and be ready for installation upon completion. Extending the pier segments far enough beyond the live load shoe (minimum 20'-0" from centerline of trunnion) will provide the option of temporarily securing both leaves together at their tips, floating the entire assembly between the piers, and lifting the assembly vertically into position to splice with the tail segments (without having to rotate it). Once the splices are secured, the temporary tie at midspan will be released.

8.2 Rack and Pinion Gears

The existing rolling bascule span has the rack teeth pointed up. This configuration allows debris to collect in the grease and teeth of the rack. King County maintenance has constructed a guard along the length of the rack to keep the debris out. The new trunnion bascule span will have a rack attached to the bottom of the girder with the teeth pointed in the down position. This configuration is common on a majority of trunnion bascules. A rack gear with outward and downward facing teeth prevents hard particles and debris from collecting in the rack teeth, which can cause excessive wear and damage to the gears.

9 APPROACH SPANS

9.1 Superstructure

The superstructure design for the South Park Bridge approach spans of is based on the WSDOT *Bridge Design Manual LRFD*[31] and the *AASHTO LRFD Bridge Design Specification*[1]. The superstructure calls for the use of nine, WF74G precast/prestressed concrete, WSDOT standard girders. This will allow a lower roadway profile and limit the maximum slope to within 5%, while still maintaining the required vertical clearances and not increasing the number of approach span piers.

Therefore, each approach superstructure comprises two spans. Each span will consist of nine WF74G girder lines, spaced at 7'-10" maximum, with an 8-inch cast-in-place reinforced-concrete deck. The girders and the deck slab will be designed as composite members. The composite girders will be designed as continuous spans over the center pier for live load. Expansion joints will be provided at the bascule pier and at the abutment.

WSDOT standard details will be used for standard components, such as the girders and interior diaphragms. Each of the north and south approach spans are being designed independently and then compared and adjusted to produce a more uniform design. The straight girders will be laid out to optimize a best fit to the curved alignment of the road.

The WSDOT program PGSuper[31] will be used to design as much of the approach spans as possible, including the girders and the negative moment regions of the deck slab.

All loads on the bridge will be estimated based on known values and include; vehicle live load, pedestrian live load, utilities, warning gate, architectural (fascia) truss, barrier, sidewalk, and future overlay. The architectural truss will be supported along the edge of the deck with the load being distributed to the three exterior girders. As a redundant system, the truss will also be designed to have anchor bolts mounting to the end diaphragms of the superstructures at Piers 2 and 5. The warning gate load will also be distributed to the three exterior girders. Based on further investigation, an additional intermediate diaphragm will be required to effectively support and uniformly distribute the warning gate load onto the exterior three girders as specified by WSDOT design practice.

9.2 Substructure and Foundations

The seismic design requirements govern the design of the substructure of the approach spans. The clear height of the columns at Pier 2 of the south approach will be 11.68 feet. The structural dynamic response analyses reveal that, with short stubby columns at this intermediate pier, there will not be a contra-flexural point formed in the middle segment of those columns under the horizontal earthquake loads. Therefore, a column hinge core has been designed to take place at the top of the columns under seismic loads. The column will be designed to resist all bending demands for strength and service limit states. The inner core will then be idealized and designed as a hinge only for seismic loads. The design of the column hinge follows the procedures and requirements as outlined in the *Bridge Design Manual LRFD*[31] Section 7.4.7.

In contrast to the columns at Pier 2 of the south approach structure, each column at Pier 5 of the north approach structure will have a contra-flexural point near a distance of one-third of the height from the top of the column. Under seismic loads, a column hinge core has been designed to take place at the bottom of the columns at Pier 5.

These two approach structures need to meet the seismic design requirements of both AASHTO LRFD Bridge Design[1] and AASHTO Guide Specification for LRFD Seismic Bridge Design[3] under the 975-year earthquakes. Consequently, nonlinear analyses including a pushover have been performed to evaluate the performance of the structures in response to the design earthquakes.

Four shafts, 8 feet and 7 feet in diameter, will be required at the south and the north bridge abutments, respectively. Both will be spaced on 20.5-foot centers along the centerline of each abutment. This will provide adequate flexural rigidity in the abutment cap in order to effectively distribute the earthquake loads to the shafts. For the intermediate piers, three 9-foot-diameter drilled shafts, spaced at 32.5 feet, were used. Each shaft will support a square column with architectural rustications. Penetrating through the liquefiable layers into the competent soil layers below, the 7-foot and 8-foot shafts will reach about 100 feet of embedment. An embedment depth of 120 feet will be required for the 9-foot-diameter shafts.

10 MECHANICAL AND ELECTRICAL SYSTEM

10.1 Mechanical System

Since the size of the machinery depends upon the dimensions of a specific manufacturer's model, the machinery manufacturer and model listed in the plans were used as the basis for geometric design. Although the plans present an item of a particular manufacturer, the Contractor will be allowed to substitute machinery from other manufacturers with some changes to the dimensions. The use of substitude machinery will be specifically stated in the specification.

One exception to substitution will be the span locks located at the girder tips. The Earle Cushionloks were chosen based on their ability to automatically adjust to wear on the lockbar. The guides and receivers are spring mounted and automatically eliminate the gaps as the bar wears. An adjusting screw is also provided for large amounts of bar wear.

10.1.1 Bascule Span Drive Machinery

As mentioned previously, a gear driven system was selected over a hydraulic system for the span drive machinery. Several factors were examined, and the primary reason this type of drive was chosen was maintenance.

The layout of the span drive system is presented on a drawing sheet in Appendix A. The drive machinery wil be located on a machinery platform in the bascule piers. The drive system will consist of two main 75 horsepower electric drive motors connected to a central reducer located at the span centerline. The DC main motors will run in an over-speed condition to decrease the operating time. The main motors have been sized to open the span in 90 seconds under the influence of AASHTO specified loads. A motor brake will be provided for each main motor. The motor brakes will be located on the reducer input shaft so that motor failure and removal of the motor for repair will not reduce braking capacity.

The central reducer will be a parallel shaft double reduction gear reducer. The central reducer will be a custom configuration since it will have a differential gear set and a spiral bevel gear to attach the auxiliary drive

The auxiliary drive will consist of a two speed 50 HP AC motor, connected to an auxiliary reducer, and a spring released clutch. While the main drive motors are operating the clutch will be disengaged to protect the auxiliary motor. The auxiliary motor have been sized to open the span in 5 minutes under the influence of AASHTO specified loads.

A floating shaft using single-engagement gear couplings on each side of the central reducer will connect the output shafts to an outboard reducer. The connection at the outboard reducer will be made with a single-engagement gear coupling with an integral brake drum for the machinery brake. The machinery brakes will be located downstream of the differential. This will assure braking capacity in the unlikely event of a failure in the differential. The output shaft of the outboard reducers will be connected to the rack pinions. The rack and rack pinions will be the final gear set in the drive system. The rack pinion will be straddle-mounted between two self-aligning pillow blocks with spherical roller bearings to provide maximum stability and limit deflection during operation. The rack will be made of a forging that is made in segments. In the event of a rack tooth failure, the rack can be replaced in sections. The rack will be bolted to a support that will be mounted to the bottom flange of the bascule girder.

Plain cylindrical bearings will be used to support the trunnion shaft. A journal (trunnion shaft) will rotate relative to a fixed bronze bushing and the motion between the two contact surfaces will be sliding. The plain sliding bearing was chosen over the self-aligning roller bearing for the following reasons.

- The cost for large spherical roller bearings is higher than for plain bearings. The delivery time for large pillow block roller bearings is a concern. The delivery time originally quoted was over 60 weeks. Recently, with additional capacity improvements, this was shortened to 40 weeks, but the delivery time is still significantly more than for a plain bearing.

- The friction of a plain bearing is higher than a roller bearing, however, friction in this case will stabilize the span. If a drive failure occurred for mechanical or electrical reasons, the span will more likely stay put due to higher friction. From a stability standpoint, the plain bearing is better.

- Plain bearings are more susceptible to misalignment. In the event of an extreme event, seismic or vessel collision, the spherical bearings would remain aligned. The plain cylindrical bearings will be mounted on supports that are adjustable, so that after such an event, they can be manually realigned.

10.1.2 Bascule Span Center Locks

Jaw-type systems and lock-bar-type systems were investigated as part of the intermediate design. Both designs have been used successfully on bascule spans. The span locks purpose is to keep the tips of the two cantilevered spans together by transferring vertical shear from one leaf to another. Forces from live load and seismic loads cause deflections where this shear must be transferred. Lateral and longitudinal loads will not be resisted by the span lock. Lateral stability will be provided by a centering device at the tip, the trunnion columns, and by a tail lock at the counterweight. The trunnion columns resist longitudinal excitation, and the center joints will be sacrificed if substantial movement from a seismic event occures.

Jaw-type systems transfer the shear effectively from one leaf to another without transferring moment. As gaps develop from wear they can easily be adjusted to drive farther to eliminate the gaps by simply adjusting a limit switch. Since they would be a linkage, jaw-type systems would be more complex, with more parts.

Lock-bar-type systems use a bar, a guide, and a receiver on the other span. It is a simpler mechanism with only a moving bar. Some clearance would be necessary to allow for the movement of the lock bar in the guides and receiver. This clearance would result in relative movement between the bascule leaves under traffic loading. Over time, the clearance would typically increase due to wear, and adjustment with shims will be required.

A proprietary lock-bar-type system was chosen over the jaw-type system because of simplicity. In order to eliminate gaps, the system will have spring loaded wear plates in both the guide and receiver. As the bar or guides wear over the capacity of the automatic spring system, the spring loaded wear plates can be adjusted by screws.

10.1.3 Bascule Span Tail Locks

To stabilize the span in the event of a seismic event, a tail lock will be added to the rear of the span at the counterweight and rear pier wall. Seismic forces exist in all three axes, but the counterweight needs to move down as the span opens. Therefore, a movable tail lock will be required.

Although not required, a ball screw actuator for the tail lock could be used that is the same type as that used in span center locks. The proprietary spring-loaded receiver and guides will not be necessary, however, a common actuator would aid maintenance.

10.2 Electrical Control System

The bridge control system will consist of a central control station located in the north control tower and will provide the operator control of all components of the bridge operating system. This system will include the traffic signals, four traffic gates, four resistance barriers, four span locks, tail locks, and controls to raise and lower the spans using either the main drive motors or auxiliary drive motors. In addition to the control features, indicating lights giving the status of the motor and machinery brakes, system voltage, main drive motor amperage, span position, and bypass functions will also be provided.

Electrical equipment will be located in both the north control tower and the south control tower. Splitting the equipment will reduce the number of required electrical cables between the north bascule pier and the south bascule pier. These cables will be routed between the bascule piers using a micro-tunnel system that will provide a series of fiber reinforced glass ducts within a steel casing. This duct system will allow the cross channel cables to be replaced in the future, if required, without the need to obtain environmental permits or disturb the soils in order to install a new cable. In addition, the ducts will provide additional protection to the cables which will result in a longer service life.

Electrical and control system designs and commissioning specifications will comply with 2008 editions of the following codes and standards:

- *LRFD Movable Highway Bridge Design Specifications*, AASHTO[2]
- NFPA 70 *National Electric Code* (NEC), National Fire Protection Agency
- MUTCD *Manual on Uniform Traffic Control Devices*, Federal Highway Administration
- 33 CFR Part 118 *Navigation Lights/Signals*, U.S. Coast Guard
- NETA ATS1 *International Electrical Testing Association Acceptance Testing*
- NFPA – 79 *Electrical Standard for Industrial Machinery*
- ANSI/NECA-1 *Standard for Good Workmanship in Electrical Contracting*
- ANSI/ISA – 84 *Standard for Safety Instrumented Systems*
- IEC – 61508 *Standard for Functional Safety*
- IEEE Standard C2 *National Electric Safety Code*

10.2.1 Electrical Power System

10.2.1.1 Electrical Power Service

Electrical power to the bridge will be provided from the north side for both bascule piers. The service will be 480 volt, three-phase, four-wire, grounded wye. A main service disconnect switch will be provided near the north abutment to feed the north bascule pier motor control center. A three-phase sub-feeder will be used to deliver power to the south bascule pier motor control center. This sub-feeder will be routed through submarine conduits located within the micro-tunnel system between the bascule piers. A diesel-powered standby generator will be located near the north abutment for emergency operations of both bascule leaves.

10.2.1.2 Motor Control Centers

Motor control centers will be located on each bascule pier. They will serve as the primary electrical power distribution source for the bridge. The motor control center will have all motor starters and will feed the main motor drives, control tower lighting, and receptacle distribution panels. Step-down distribution transformers will be provided in each bascule pier to provide 120/240 volt, single-phase power for lighting and receptacles within the control towers and bascule piers.

10.2.2 Control System

The bridge control system will consist if a Programmable Logic Controller (PLC) to provide primary bridge control and monitoring of the bridge operation. The electrical service to the PLC will be protected with an Uninterrupted Power Source to eliminate the need to re-boot when the grid power is restored after a power outage. Additionly, hard wired relays will provide a backup means of operation in the event of a problem occurring within the PLC system or with the main drive motors. The PLC system will use a primary processor with a “cold-backup” processor configuration. A remote I/O (input/output) network will connect distributed I/O throughout the bridge and will be routed through the micro-tunnel system. There will be additional hard-wired interlocks and controls for safety circuits, including the emergency stop circuits. The control will be designed to be fail-safe with a level of redundancy to provide a level of fault tolerance.

In addition to the PLC and hard-wired safety circuits, there will be a manual emergency operation mode. Should the PLC primary and backup processors fail, the bridge traffic controls and bascule leaves can be operated from the control console station in the north control tower in a manner similar to the primary control system using many of the same control switches and without the need of the operator to leave his work station to activate the manual operating system.

Interlock bypasses will be provided for the operator in the unlikely event of a failure of an interlock permissive not functioning properly.

10.2.3 Main Motor Drives

The main motor drives will be four quadrant regenerative, SCR (silicone rectifiers) type. The drives will convert 480 volts AC to 500 volts DC and control motor direction, torque, and speed. The drives will be fed individually from branch circuit breakers in the motor control center. There will be four drive motors and four drive controllers. Two motors and drive controllers will operate each leaf. Under normal operating conditions, both drive motors will operate at the same time for each leave in an overspeed operation in order to provide a faster operating time for the spans. Individual single motor operation will be able to be selected from the control console which will provide a level of redundancy, but when operating with a single drive motor, operating speeds will be slower with motor speeds limited to the base speed of the drive motor.

11 PIER PROTECTION

11.1 Geometric Constraints

Bridge fender systems serve primarily as navigation aids to vessel traffic by delineating the shipping channel beneath the moveable span. The fender system will be designed to be robust enough to survive a multitude of bumps and scrapes from barge traffic, and to absorb kinetic energy while redirecting an errant barge or other vessel.

The navigation channel of the Duwamish Waterway is 125 feet wide and centered between the two bascule piers. The fenders will be flared a minimum of 10 feet from the drip line of the superstructure, measured perpendicular from the centerline of the superstructure coping to the beginning of the fender flare.

The foundations of the existing bascule piers have a 21-foot-thick footing with piles on a 3-foot grid. The existing foundation will be removed to only about 3-feet below the mud line. Therefore, the geometry of the new fender system will need to avoid the foot print of the existing bascule piers. The fenders will also be extended to cover the concrete seal pads of the existing bascule footings.

A system of vertical 2-foot-diameter, open-ended, steel-pipe piles with 10” x 10” fiberglass reinforced structural plastic lumber wales has been designed for the new fender system. The minimum pile embedment depths are 40 feet below the assumed scour elevation at -33.00 feet. The pile length will be equal to the embedment length plus the distance to the pile head which is set at 7’-8” above Mean High Water. The fender system has been designed to provide adequate stiffness for a collision force of 168 kips. The corresponding deflection under the designed impact energy is 42.4 inches.

The location and angle of the bascule piers in relation to the channel will create a section of the fender that has approximately 6’-9” of clearance from the edge of the channel to the face of the new bascule pier. The minimum distance between the piles and the pier wall is 4 feet. There will be a submarine cable located 8’-3” west of the center line of the bascule span. A 5-foot-clear corridor for the cable will be required. Therefore, for this segment of the fender, vertical piles have been relocated to clear the corridor.

A vessel pilot house to deck collision with the bascule span will not be considered because the bascule span is assumed to be open while vessels to pass.

11.2 Fender Design

11.2.1 Vessels Information

Information on vessels plying the Duwamish Waterway is shown in Table 3.

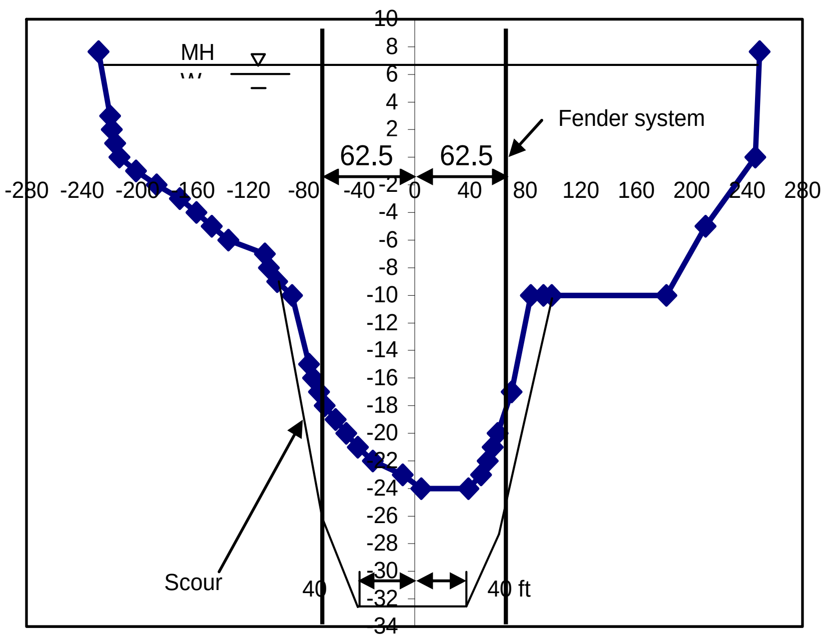
Table 3
Vessel Information

Design Vessel	DWT (ton)	Annual Passage Number	
		Wing wall	Parallel wall
Sailboat/Yacht	150	300	600
Tugboat	598	250	400
Tugboat & BMC 30	2,598	100	200
Tug & Crowley 400	12,633	1	2
Empty Hopper Barge	200	N.A.	N.A.

Source: South Park Bridge Replacement Preliminary Design Report

11.2.2 Channel Profile

The profile of water depths at the South Park Bridge Station 34+00.09 are shown in Figure 3.



Source: South Park Bridge Replacement Preliminary Design Report, PB Americas Inc. August 2007

Figure 3
Profile of Water Depths at South Park Bridge Station 34+00.09

11.2.3 Vessel Collision Force and Energy for a Rigid Pier

The collision forces and kinetic energies of different vessels are summarized in following Table 4 and Table 5.

Table 4
Collision Forces on a Fender System

Vessel Type	Design impact speed (Knots)	Impact angle 10° Force (kips)	Impact angle 10° Energy (kip-ft)	Impact angle 90° Force (kips)	Impact angle 90° Energy (kip-ft)
Sailboat/Yacht	8	158	18	783	332
Tugboat	8	408	119	2271	2819
Tugboat & BMC 30	4	454	148	2593	3640
Tug & Crowley 400	2	507	234	2914	6417
Empty Hopper Barge	1	32	1	185	19

Note: 1 Impact angle of 10 degrees occurs on the parallel walls
2 Impact angle of 90 degrees occurs on the flared walls.
Source: ABKJ, June 2009

Table 5
Collision Forces on a Bascule Pier

Vessel Type	Design impact speed (Knots)	Impact angle 45° at each direction Force (kips)	Impact angle 45° at each direction Energy (kip-ft)
Sailboat/Yacht	8	563	172
Tugboat	8	1611	1405
Tugboat & BMC 30	4	1835	1823
Tug & Crowley 400	2	2061	2915
Empty Hopper Barge	1	131	9

Source: ABKJ, June 2009

11.2.4 Annual Frequency of Pier Collapse

The annual frequency (AF) of a bridge component collapse is summarized in Table 6.

Table 6
AF - Annual Frequency of a Bridge Component Collapse

Heavy duty fender system, Design Force H=35 kips

		N	PA	PG	PC	AF
South parallel fender wall	Sailboat/Yacht	600	0.000086	0.020	0.0865	0.00009
	Tugboat	250	0.000086	0.077	0.2271	0.00037
	Tugboat & BMC 30	100	0.000172	0.049	0.3058	0.00026
	Tug & Crowley 400	2	0.000172	0.042	0.3792	0.00001
South west fender wingwall	Sailboat/Yacht	300	0.000086	0.020	0.2917	0.00015
	Tugboat	250	0.000086	0.077	0.8479	0.00140
	Tugboat & BMC 30	100	0.000172	0.049	0.8760	0.00074
	Tug & Crowley 400	1	0.000172	0.042	0.8911	0.00001
South east fender wingwall	Sailboat/Yacht	300	0.000086	0.020	0.5979	0.00031
	Tugboat	250	0.000086	0.077	0.8613	0.00142
	Tugboat & BMC 30	100	0.000172	0.049	0.8785	0.00074
	Tug & Crowley 400	1	0.000172	0.042	0.8919	0.00001
			Subtotal AF			0.00550
North parallel fender wall	Sailboat/Yacht	600	0.000086	0.020	0.0865	0.00009
	Tugboat	250	0.000086	0.069	0.2271	0.00034
	Tugboat & BMC 30	100	0.000172	0.043	0.3058	0.00023
	Tug & Crowley 400	2	0.000172	0.038	0.3792	0.00000
North west fender wingwall	Sailboat/Yacht	300	0.000086	0.020	0.4390	0.00022
	Tugboat	250	0.000086	0.069	0.8528	0.00126
	Tugboat & BMC 30	100	0.000172	0.043	0.8769	0.00065
	Tug & Crowley 400	1	0.000172	0.038	0.8914	0.00001
North east fender wingwall	Sailboat/Yacht	300	0.000086	0.020	0.0985	0.00005
	Tugboat	250	0.000086	0.069	0.8418	0.00125
	Tugboat & BMC 30	100	0.000172	0.043	0.8749	0.00065
	Tug & Crowley 400	1	0.000172	0.038	0.8908	0.00001
			Subtotal AF			0.00476
			Total AF			0.01025

Note: the total PG for the either side system in the north bank is evenly divided for three components for simplicity.

South or north bascule pier, Design force H=EQ/2=2000 kips

		N	PA	PG	PC	AF
South or north	Sailboat/Yacht	900	0.000086	0.040	0.0000	0.00000
	Tugboat	500	0.000086	0.267	0.0000	0.00000
	Tugboat & BMC 30	200	0.000172	0.180	0.0000	0.00000
	Tug & Crowley 400	3	0.000172	0.145	0.0033	0.00000
			Subtotal AF			0.00000
			Total AF			4.9E-07

Note: AF = N*PA*PG*PC
Source: ABKJ, June 2009

11.2.5 Design Philosophy of South Park Bridge Fender System

- A bridge fender system has been deemed to be necessary for the South Park Bridge by King County in consultation with other agencies including the U.S. Coast Guard.
- Primary function of the fender system is to delineate the channel and redirect errant vessels.
- The fender system will be considered sacrificial.

- The fender system will be a flexible, energy absorbing structure. The design will mitigate the potential for damage to vessels and the fender during a minor collision, while redirecting some vessel impacts that would damage a more rigid fender system.

- The design of the bascule piers are controlled by seismic and/or service load forces which are anticipated to exceed full ship impact forces.

11.2.6 Design Methodology

- Work done by fender system is equal to the absorbed energy.

- The absorbed energy will redirect or possibly bring an errant vessel to rest.

- For an assumed vessel size and approach angle, the critical speed at which a vessel would cause the fender system to fail will be determined.

11.2.7 Energy Capacity and Maximum Deflection of Fender System

A 32-foot, double-panel, fender system will be used for structural analysis. Iteratively increased static forces were applied on the critical position of the panel until either any member of its wales or piles yielded. The maximum static force that the fender system can resist is 168 kips with a corresponding 42.4 inches of deflection. The total energy capacity is equal to 297 kip-feet. By comparison, the fender system is equivalent to the Department of Transportation of Florida State's Heavy Duty Fender System, which has 295 kip-feet of energy capacity.

Corresponding to this energy capacity of 297 kip-feet, the maximum impact speeds under impact angles of 10 and 90 degrees for different vessels are summarized in following table.

Table 7
Maximum Impact Speeds (knots)

Vessels	10 degrees	90 degrees
Sailboat/Yacht	23.1	4.6
Tugboat	11.5	2.3
Tugboat & BMC 30	5.5	1.1
Tug & Crowley 400	2.3	0.5
Empty Hopper Barge	20.0	4.0

Source: ABKJ 2009

12 DESIGN VARIANCES

12.1 Roadway Design

The following design deviations were incorporated in the Intermediate Design plans:

- The northeast corner of 14th Avenue South and Dallas Avenue South will have a 20-foot radius. The King County standard is a 25-foot radius. King County confirmed acceptance in an email dated October 31, 2008,

- The intersection angle between Dallas Avenue South and 14th Avenue South will be 69º55'56" and 66º33'19". King County standards require 85 to 95 degree intersection angles. WSDOT and AASHTO allow 60 to 120 degree intersection angles.

- The intersection angle between Dallas Avenue South and Boat Access will be 81º10'00". King County standards require 85 to 95 degree intersection angles. The two roadways are existing roadways requiring minor improvements.

- The intersection angle between South Orr Street and Boat Access will be 121º38’56”. King County standards require 85 to 95 degrees intersection angles. The subaccess roadways are designed to match existing conditions.

- The intersection angle between South Sullivan Street and Sullivan Place will be 66º09’06”. This is the angle that results when Sullivan Place meets Dallas Avenue South at 90 degrees. Since Dallas Avenue South is a higher classification roadway consisting of greater truck traffic, it seemed appropriate to design the intersection with Sullivan Place at 90 degrees. In addition, the majority of traffic between South Sullivan Street and Sullivan Place will be moving between the west and north legs of the intersection.

- The horizontal sight distance on South Orr Street will be 68 feet. AASHTO standards require a sight distance of 115 feet for a 20 mph design speed. South Orr Street provides local circulation between the east and west side of the bridge and has fairly low traffic movement. To meet the designed stopping sight distance, we recommend that the speed be posted at 10 mph. This design is an improvement over the preliminary concept by providing two 111-foot horizontal curves. The preliminary alignment proposed extremely tight horizontal radii of 25 and 30 feet.

- The vertical stopping sight distance on the Dallas Avenue South (east leg) will be 174 feet. For a 30 mph design speed, King County standards require 200 feet. The proposed roadway reconstruction will be shorter than the required sight distance. To mitigate for the sub-standard sight distance, the roadway will be illuminated at the sag vertical curve.

King County will prepare the design deviations and process them for approval.

12.2 Mechanical/Electrical Design

The National Electric Code (NEC) requires a lateral clearance of up to 42 inches adjacent to some electrical devices. The configuration of the primary drive motors, auxiliary drive motor, and the bridge locks do not provide the specified lateral clearance. Rearranging the configuration to provide the required lateral clearance is not practical from a construction cost and operation standpoint.

The intent of the lateral clearance requirement is to provide refuge for persons working on the electrical devices in the event of an errant spark or unintentional contact with conductors.

To mitigate the need for the lateral clearance and provide for the safety of maintenance personnel, it is common practice to de-energize and lock-out the above mentioned electrical devices prior to accessing the equipment.

A variance from the NEC lateral clearance requirements will be required to construct and operate the machinery in the configuration shown in the Intermediate Design plans.

Efforts to obtain the NEC variance will be started subsequent to the Intermediate Design submittal.

13 CONSTRUCTION

13.1 Construction Overview

On-site construction and demolition activities are expected to occur over a 32-month period from May 2011 through December 2013, based on the proposed Intermediate Design construction schedule. The proposed sequence and durations are deemed to be a feasible method for completing the project within the applied constraints.

The proposed sequence and durations are based on the following assumptions:

- “In-water” work is allowed between August 1 and February 15.
- The construction contractor is provided notice-to-proceed far enough in advance to initiate “in-water” work on August 1 of the first year of construction.
- The construction contractor works 6-day weeks and multiple shifts to complete the construction of selected elements of the bridge.
- No labor or material issues will conflict with the natural progress of construction.

The proposed bascule bridge will to be built directly adjacent to the existing bridge for the purpose of keeping the existing bridge in operation while the new bridge is being constructed. Therefore, no major bridge closures are expected. However, routine disruptions to traffic and parking will occur due to the presence of construction on the project site and the changeover period necessary to divert traffic from the old bridge to the new bridge.

Due to the limited staging and lay-down areas for the project, it has been assumed that the portion of bascule superstructure between the bascule piers will be assembled on floating platforms, floated to the site, hoisted vertically into position, and connected to the previously erected heels of the bascule span.

The construction schedule also assumes the existing bridge will withstand construction of the proposed bridge adjacent to it and will remain functional throughout the duration of construction. This may require consolidation grouting beneath the existing bridge foundations and/or strengthening or modifying the existing bridge to withstand differential settlement and translation of the piers. The ability of the existing bridge to withstand construction of the proposed bridge will be better understood once the results of the posed test piles installation is completed and the monitoring results are available.

13.2 Construction Constraints

13.2.1 Environmental Constraints

13.2.1.1 Hazardous Materials

This section is updated to include soils and/or materials. The construction contractor will be required to develop and implement a Hazardous Materials Handling Plan. At a minimum, the plan will address the following:

- Excavation controls to prevent spillage of hazardous materials into the Duwamish Waterway.
- Plan for removal and proper disposal of contaminated or hazardous materials expected to be encountered based on soil/material characterization profiles. Not all soils will require special handling.
- Contaminated sediment disturbance controls in the Duwamish Waterway such as silt fences, sand blankets etc.
- Plan for removal and proper disposal of unanticipated contamination pockets (e.g., buried containers).
- Control of worker health and safety risk during demolition operations and excavation and transportation of contaminated or hazardous materials.
- Plan for construction schedule impacts due to the presence of unknown contamination pockets.
- Testing, treatment, and control of contaminated dewatering effluent.

13.2.1.2 In-Water Work Window

The project area is within a designated critical habitat area for a few types of fish identified as endangered species. Work within the water column of the Duwamish Waterway is referred to as “in-water” work. In-water work will be limited by the potential presence of the protected species as well as other species.

The window allotted for in-water construction is assumed to be from August 1 to February 15 of each year.

In-water work includes construction of the cofferdam, and pile driving outside the cofferdam. Excavation and other work within the cofferdam are not considered to be subject to the general restrictions of the in-water work window.

13.2.1.3 Noise and Vibration

The existing bridge may be adversely affected by vibration from construction activities. Vibration and other monitoring equipment will be installed on the existing structure and monitored. The construction of noise barriers is not feasible in most locations.

Noise and vibration within the water column of the Duwamish Waterway due to the impact driving of piles will be mitigated by the use of bubble curtains.

13.2.2 Site Constraints

The U.S. Coast Guard will require precautions be taken to accommodate marine traffic during construction and demolition of the bridges. The details of the precautions will be established during final design.

13.3 Environmental Controls

13.3.1 Temporary Erosion and Sedimentation Controls

The goal of the Temporary Erosion and Sediment Control (TESC) design is to reduce sediment transport due to construction activities. Best Management Practices in accordance with the King County *Surface Water Management Manual*, 2005 Edition, as supplemented by the Department of Ecology (DOE) *Stormwater Management Manual for Western Washington*, 2005 Edition will be implemented for this project.

13.3.2 Spill Prevention, Control, and Countermeasures (SPCC) Plan

This section is supplemented with:

The contractor's SPCC Plan will address material and equipment staging areas.

13.3.3 Erosion and Sedimentation Control Features

The Contractor will be required to follow and revise, as necessary, the erosion control plan which, at a minimum, will include the following 12 components:

- Mark clearing limits with High Visibility Fencing
- Establish construction access
- Flow rate control plan
- Sediment control plan
- Soil stabilization plan
- Slope protection plan
- Drain inlet protection plan
- Channel and outlet stabilization plan
- Pollutant control plan
- Dewatering control plan
- Maintain best management practices
- Material and equipment staging areas
- Manage the project

As part of the Erosion and Sedimentation Control Plan, the Contractor will install erosion and sedimentation control features consisting of the most appropriate Best Management Practices (BMPs) including, but not limited to, the following:

- Sediment Source Controls:

Stabilized construction entrance
Temporary mulching and seeding
Erosion control mats and blankets as required
Dust control
Materials on hand for erosion and sediment control
Concrete handling
Saw cutting and surfacing pollution prevention
Material delivery, storage and containment
Project scheduling
Sand drainage blankets

- Runoff Conveyance and Treatment of Eroded Soils:

Rock Check dams
Interceptor Ditches/Swales
Outlet protection
Storm drain inlet protection (inserts and excerpts)
Straw wattles or coir logs
Silt fence
Compost filter berm or gravel berm
Sediment trap/water quality vault
Additional water quality measures (Chitosan) may be necessary to meet discharge requirements (contractors option).

The contractor will be required to maintain river system BMPs for the duration of the in-water work. Landside BMPs will be implemented prior to the start of construction and be phased with the construction sequence for the duration of the project. Contaminated soils will be contained, transported, and disposed of according to State Law at approved disposal sites. TESC BMPs will be applied to materials transported, stockpiled, and/or disposed of off-site.

The Contractor will be required to have a DOE-certified Construction Erosion and Sediment Control Lead present to supervise the project whenever work is being performed that could result in the release of turbid water. If conditions warrant, additional erosion and sediment controls may be required.

13.3.4 Stormwater Treatment Facilities

The construction contractor will be required to install a water quality vault near the north bridge approach, and a sediment trap/sump at the south bridge approach early in the construction sequence. The sediment traps will be used as sedimentation ponds to control erosion and runoff into the Duwamish Waterway during construction. Runoff from the new bridge will be routed into these sediment treatment facilities prior to discharge. All permanent stormwater facilities used as temporary facilities will be cleaned prior to project acceptance. Water quality facilities will be designed in accordance with the KCSWDM.

13.4 Construction Sequence and Details

13.4.1 General Project Timing, Sequence, and Traffic Impact

The construction schedule has been revised and is attached in Appendix B

The intermediate design has incorporated a proposed construction staging/traffic control plan. The contractor is not bound by this staging plan and he/she can propose a new plan subject to approval by King County.

The project will be built using three stages. The first stage will include construction of the bascule bridge, approach spans, earthquake drains, stormwater water vault along the north side, and the intersection of South Sullivan Street/Dallas Avenue South. Traffic will remain on the existing lane alignment. The west sidewalk will be closed to allow for bridge monitoring and construction contractor access to proposed bridge from the existing bridge as well as the close proximity of construction of the north approach embankment. All pedestrian traffic will be routed along the east sidewalk.

The second stage will include completion of the approaches including the structural earth walls. Traffic lanes will need to be reduced from four lanes to two lanes. Each lane can carry approximately 900 vehicles per hour (vph) per lane during the peak period. The traffic data provided by King County shows a peak hour traffic of 930 vph. It is assumed that during construction, some of the traffic will gravitate toward alternate routes to avoid the construction area.

The third stage will include completion of the connection to the existing roadway, construction of the east side of the roadway, the water quality treatment facility along the south side, the South Orr Street roadway and shoreline mitigation. At this stage the proposed bridge will be made operational and traffic will be shifted to the proposed bridge. For a portion of this stage, traffic will be allowed on the two westerly traffic lanes. Pedestrian traffic will use the new west sidewalk. The existing bridge will also be removed during this stage.

Complete closure of 14^{th} and 16^{th} Avenue South will only occur when making final connections to the existing roadway. A detour route has been established for this event and the duration of the closure will be minimized.

13.4.2 Ground Improvements

Ground improvements using earthquake drains will be performed beneath and adjacent to the approach embankments. These drains will mitigate the potential for liquefaction in the area by allowing excess pore-water pressure to be relieved during a seismic event. This is achieved by providing a pathway for water to flow up the drains and into a gravel layer. Relieving excess pore pressure will help to maintain the shear strength of the potentially liquefiable soil during a seismic event. Ground improvements will occur in the first stage of construction in concurrence with the utility relocation activities.

Earthquake drains are large-flow capacity vertical drains (typically 3 to 8 inches in diameter) wrapped with a geotextile filter fabric. They are typically installed by inserting a tubular steel mandrel containing the drain into the ground, using static force and relatively small vibrations. Once the design depth is reached, the mandrel is withdrawn, leaving the earthquake drain in place.

The earthquake drains will be placed at 3-foot to 5foot spacings beneath and adjacent to each of the approach embankments. For the North Approach Embankment, the earthquake drains will be be installed to a minimum elevation of -40 feet NAVD88. For the South Approach Embankment the drains will be installed to the top of the glacially overridden deposits (average elevation of -35 feet NAVD88). A free draining gravel layer will be required to overlay the drain pipe heads which will be capped with a geotextile to prevent fine soil from infiltrating and clogging the gravel layer. A minimum 2 feet of fill will be placed above the geotextile before allowing construction traffic to operate over the improved area.

A schematic drawing of the earthquake drain system is shown in Figure 4.

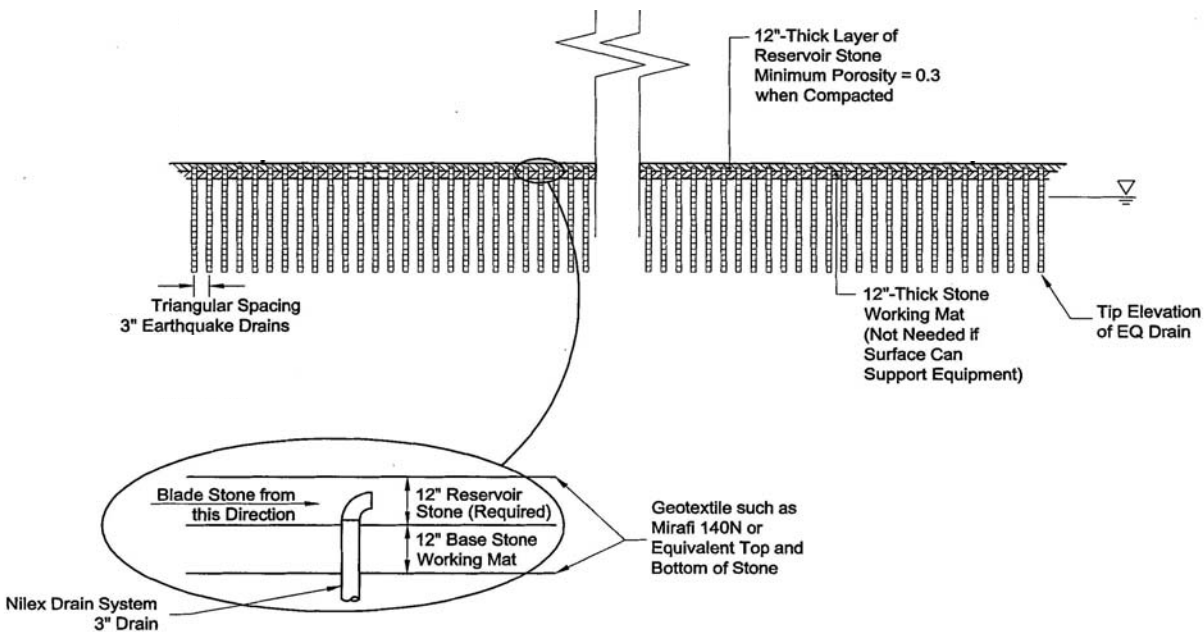


Figure 4
Earthquake Drain Schematic

13.4.3 Utility Relocation

The proposed underground utility relocation work will be coordinated with the proposed soil-improvement efforts to avoid conflicts. The earthquake drain pattern will be curtailed to avoid utilities that cannot be conveniently relocated.

The Intermediate Design submittal has been coordinated with the Boeing Company, Seattle City Light, Seattle Public Utilities, Comcast, Puget Sound Energy and Qwest on utility removals, relocations, and added service.

13.4.4 Construction of Working Trestles

Two temporary construction trestles will likely be constructed on the west side of the existing bridge with one extending from the south shore and the other extending from the north shore. Several open-ended steel piles will support the work trestles. Additionally, open-ended steel piles will be used to guide the installation of sheet pile for the cofferdams. Most of these piles will be in areas below the mean higher high water of the Duwamish Waterway. Trestle steel piles will be impact driven and/or vibrated to the necessary depth to reach capacity. An impact hammer may be used to proof test the bearing capacity of a portion of the piles. To minimize potential effects to fish, a confined bubble curtain will be installed around piles during impact driving to minimize acoustic impacts. The construction of each of the two T-shaped temporary working trestles is estimated to take approximately 20 days, with both trestles being constructed concurrently for scheduling reasons.

13.4.5 Construction of New Main Piers

A series of caisson segments will be constructed and then sunk into final position by excavating soils beneath the caisson from hollow voids (dredge wells) within the caissons. In order to minimize the risk of potentially contaminated sediments becoming mobilized in the Duwamish Waterway, excavation activities will occur within the confines of sealed cofferdams installed before excavation activities begin. A sand island will be constructed within the cofferdams by excavating the floor of the waterway within the confines of the cofferdams to an elevation of approximately -30 feet, then backfilling the cofferdams with clean granular material. Then each segmental caisson will be sunk into final position by excavating soils beneath the caisson from within the dredge wells confined within the caisson itself. Once the sand island is installed, excavation will be contained within the walls of the caisson and the material removed will consist of the clean sand or river sediments encountered at an elevation below -30 feet. The sediments below elevation -30 feet are free of contaminants that have the potential to negatively impact aquatic species.

13.4.6 Approach Span Piers and Abutments

The approach span piers and abutments will be supported on drilled shafts. No footings will be used in order to minimize the amount of potentially hazardous material that is excavated. The piers and abutments will be constructed over dry land and are to be constructed concurrently with the bascule piers. Material that is excavated from the shafts, some of which may be contaminated or hazardous, will be hauled offsite and disposed of in an appropriate manner. These shafts can be installed by any manner that the contractor chooses provided vibration transmitted to adjacent buildings are within acceptable limits.

13.4.7 Bridge Girders and Decks

The bascule girders have been designed with a field splice adjacent to the face of the bascule piers. This will allow the majority of the bascule superstructure (including decking) to be erected on floating work platforms away from the construction site. Once the bascule superstructure is assembled and the bascule piers are ready to receive the superstructure, the bascule leafs can be floated into place and hoisted vertically into position.

The precast, prestressed concrete girders for the approach spans will be erected from the construction trestles and the shore. The forming of the new concrete bridge deck on the approach spans will not require any special construction methods. Construction activities for this phase of work will be either staged from the newly placed approach span girders or from cranes staged on the work trestle. The temporary work trestle may be used to transport supplies such as formwork, rebar, and concrete. The contractor will be required to install debris containment below the approach spans to prevent material from falling into the water during construction.

13.4.8 Removal of Cofferdam and Temporary Construction Trestle

Once the caissons have been sunk to approximately 22 feet above their final tip elevations, the cofferdams will need to be removed to allow construction of the caisson caps and first level of bascule pier walls.

The construction trestles, including the steel pipe piles, will be removed when they are no longer necessary and likely stored for reuse for the temporary demolition trestle used for removal of the existing bridge.

13.4.9 Partial Pier Protection

A partial pier protection system for the new bridge will be constructed prior to shifting traffic to the new bridge. The fenders protect the new piers from errant vessels in the navigational channel. The remainder of the pier protection will be completed once the existing bridge is demolished and removed. A barge-mounted crane will be used to extract the existing fender piles and to install the new piles. Floating booms will be used to contain any debris that could enter the water.

13.4.10 Shoreline Improvements

Existing riprap, rocks, and debris along both shorelines will be removed within the project vicinity. Cleanup will be required before stabilizing materials will be installed on the embankment. Along the north shore, a wharf consisting of a concrete deck on timber piling will be partially removed to accommodate the construction of Pier 5 and the 18-inch stormwater outfall. During final design, the area will be designated to be graded back to a 3:1 slope with vegetation for slope stabilization.

Along the south side, it is assumed the area will be graded back and vegetated to mitigate for the 0.02-foot rise in the 100-year elevation of the waterway and to provide fish habitat. The proposed mitigation has not been characterized and will need to be addressed during the final design phase.

13.4.11 Submarine Cable Connection

The design incorporates a 16-inch-diameter bored micro-tunnel between the caissons approximately 15 feet below the lowest existing channel elevation to accommodate the submarine electrical cables for power and communications between the north and south caissons. The micro-tunnel will carry 6 ducts, four 4-inch ducts for bridge power and communication and two 4-inch ducts for future use.

13.4.12 Removal of Existing Bridge

After traffic has been routed to the new bridge, the existing bridge will be closed for demolition. The existing movable spans will be blocked in the full-open position.

The contractor will likely build temporary demolition trestles on the east side of the existing bridge adjacent to both the north and south portions of the bridge. Materials salvaged from the previous construction trestles will likely be used.

Before removal of the existing bascule piers begins, the remaining upriver portion of the existing fender system will be removed to allow construction access from a barge. The remaining portion of the new fenders will be completed as one of the last activities in the construction process after the existing bridge is removed.

Barge-mounted equipment and equipment on the temporary construction trestles will be used to remove sections of the existing bridge. Containment will be installed below the bridge spans to prevent water contamination. One or two temporary bents may need to be installed under each of the four approach trusses to facilitate removal of the steel sections. The bents, consisting of driven steel-pipe piles would be installed under the half or third points of each truss. If possible, the south approach pier could be removed during low tide while the area is dry.

The contractor will be required to install cofferdams around each of the existing main piers and intermediate piers (five in total). Cofferdam installation from a barge can begin concurrent with trestle construction, but the approach spans will most likely have to be removed before the cofferdam can be completed. The contractor will then remove the main piers and their foundations to approximately 3 feet below the mudline.

Following this work, the construction trestle and cofferdams will be removed, including temporary steel-pipe piles.

Land under the existing south approach will be regraded and landscaped and the waterfront access road constructed. Land under the existing north approach will be made level with the existing grade at the Boeing facilities. Selected areas of the north approach will be repaved to restore Boeing's circulation. The remaining areas will be seeded to establish turf.

14 AESTHETIC CONSIDERATIONS

The architectural direction has been modified to reflect the community's desire to recognize historic context of the existing bridge while also incorporating art. At the request of King County and community stakeholders the bridge railing was selected as a way to incorporate certain historic items to be salvaged from the existing bridge and meet project requirements to include art. The design team developed an architectural approach for the project that blends the historic character of the existing bridge with the contemporary design requirements of a new century.

The architectural treatment of the bridge is centered on a visual appearance that is reminiscent of the existing bridge but stops short of attempting to replicate the historical fabric for purposes of imitation. Features of the architectural approach to replacement of the South Park Bridge are discussed below.

The mass, scale, and proportions of the existing bridge have been emulated by choosing appropriate shapes, materials, and surface treatments of the components of the bridge. This can be seen in the selection of a truss element for the bascule girders and haunched approach girders visually treated to suggest a truss by use of a truss fascia panel. Scale and surface treatment of the bascule piers are also reminiscent of the existing bridge horizontal rustications and pedestal base.

Two eight-sided control towers constructed with brick facing for the street and main levels will be topped by hipped standing seam metal roofs. The control towers will be outfitted with four-over-one transom windows on all sides, break metal sills and precast red brick panels between corner windows. The operational control tower will be located on the west side of the north bascule pier. Due to physical constraints of the new and old structures, the control tower on the east side of the south bascule pier will be constructed once the existing structure is demolished.

The control towers will be elevated to improve the bridge tender's sight lines of the navigation channel as well as the roadway and pedestrian/bicycle surfaces. The sight lines of bridge tenders will be further augmented with articulating closed circuit television cameras strategically located to provide a full range of visual feedback for safe operation of the bridge. Audio and loud speaker systems will provide additional bridge tender sensory features. The control towers will be outfitted with a full circumference catwalk to further enhance the tender's ability to view the surrounding conditions.

The brick portions of the two control towers will mix two reddish shades that vary from reddish-brown to burgundy. The main and approach span fascia girder color will be TNEMEC 28RD "Monterrey Tile", a burgundy shade. The standing seam metal roof color will closely match TNEMEC 14RD "Red Clay".

A conceptual cross section of the bridge and its approaches has been developed to provide a 6-foot-wide sidewalk on both sides of the bridge without substantially increasing the width of the bridge. The sidewalk on the east side of the bridge will offer several functional and community advantages. The east sidewalk will provide a means of egress for stranded motorists, safe access to the north control tower, and full community circulation. The east sidewalk will also provide safe access for maintenance and operations of the traffic gates and lights without closure of a traffic lane. To minimize the width of the bridge, a barrier curb and combination railing (barrier) will be designed rather than a barrier between the vehicular traffic lanes and pedestrians. Consistent with *AASHTO LRFD Bridge Design Specifications*[1], the Minnesota combination railing has a TL-4 crash-test rating.

The roadway lighting will consist of decorative fixtures that are a blend of old and new architectural styles and will be provided as a visual clue to the historical origins of the bridge while meeting the modern day contextual requirements. The lighting will be full cut-off and will be directionally controlled to minimize the wash of light into surrounding communities. Fixtures will be mounted to a variable length mast arm on a decorative "16 flat flute" pole with powder coat painted a standard silver color.

The bridge abutments will consist of retained earth panels that repeat the horizontal rustication treatment used on the bascule piers and approach piers. Finally, all vertical concrete surfaces will receive a concrete coating, with sealer, that will provide a uniform medium light grey finish color to improve the final appearance and eliminate, or greatly reduce, the possibility of mold or mildew forming on the surfaces.

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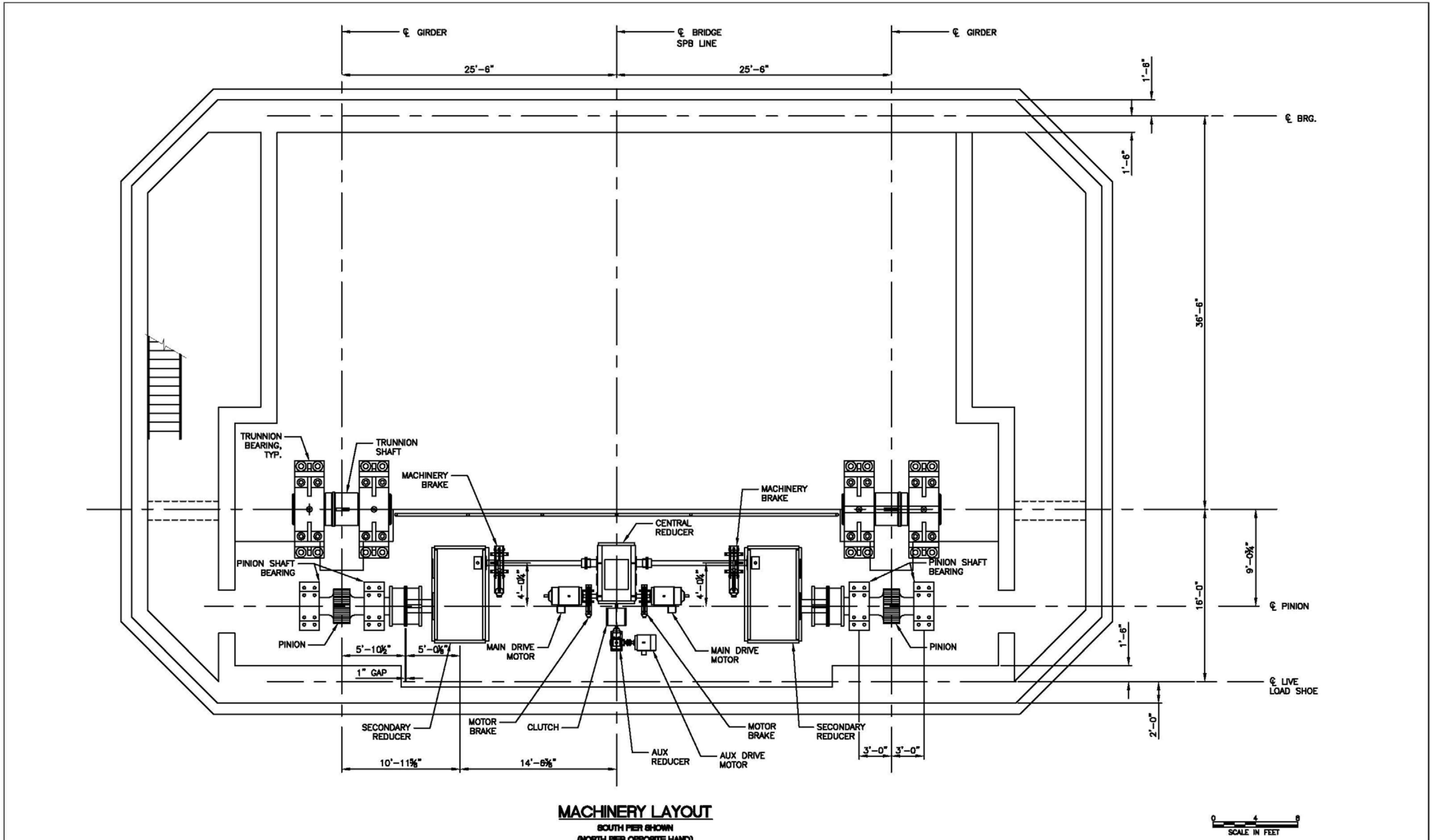
Appendix A – Machinery Layout



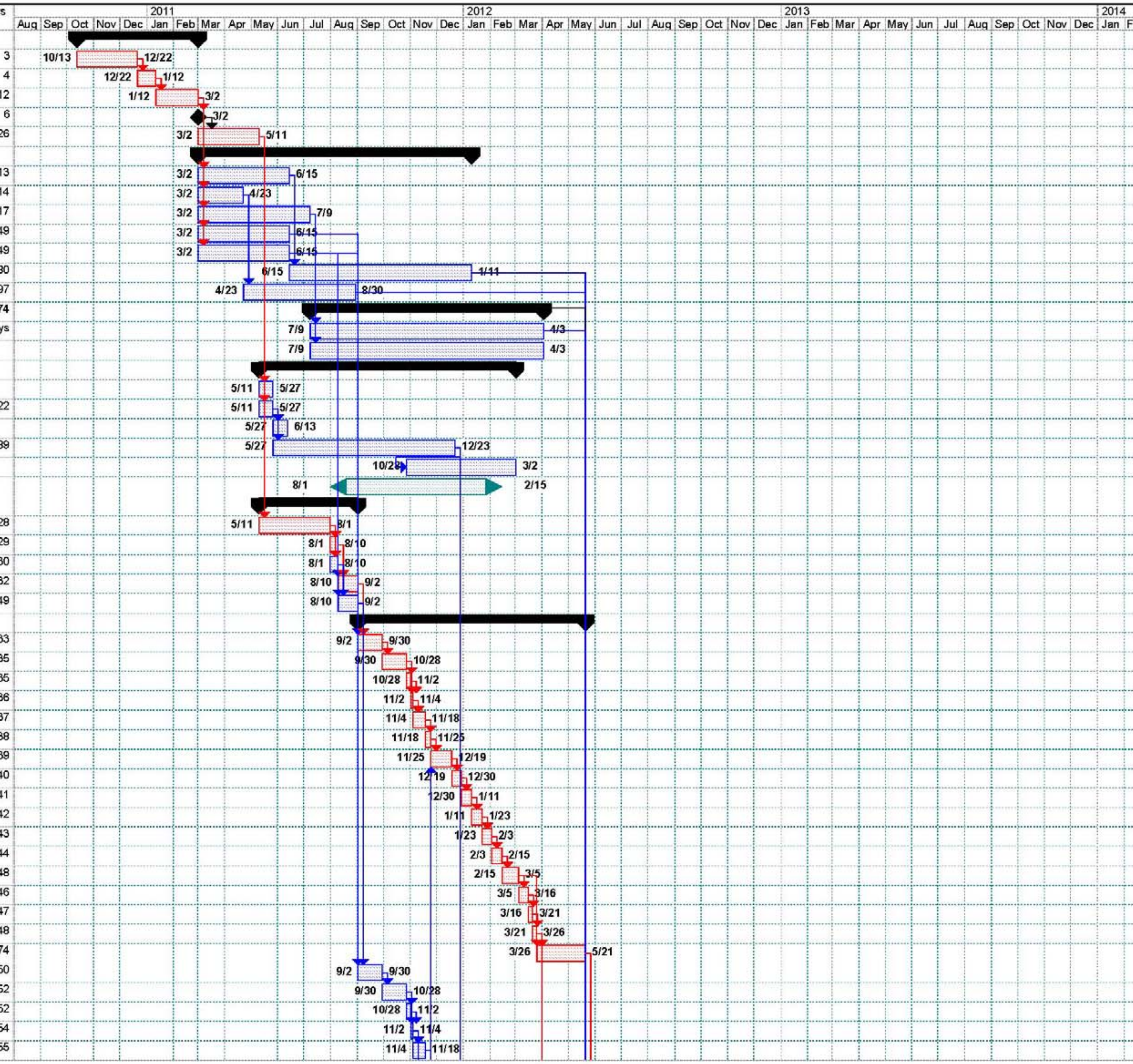
Appendix B – Construction Schedule

HNTB Job No. 45647

SOUTH PARK BRIDGE - INTERMEDIATE DESIGN CONSTRUCTION SCHEDULE (DRAFT A)

SUBJECT TO CHANGE PENDING DESIGN DEVELOPMENT

ID	Notes	Task Name	Duration	Start	Finish	Predecessors	Successors
1		**Procurement**	**120 days**	**Wed 10/13/10**	**Wed 3/2/11**		
2		Advertisement	70 edays	Wed 10/13/10	Wed 12/22/10		3
3		Bid Evaluation	21 edays	Wed 12/22/10	Wed 1/12/11	2	4
4		Award	42 days	Wed 1/12/11	Wed 3/2/11	3	,8,9,10,11,12
5		NTP	0 days	Wed 3/2/11	Wed 3/2/11	4	6
6		Contractor Mobilization	10 wks	Wed 3/2/11	Wed 5/11/11	5	19,20,26
7		**Shop Drawing Review, Approval & Procurem**	**270 days**	**Wed 3/2/11**	**Wed 1/11/12**		
8		Steel Shop Drawing Rev & Appr	90 days	Wed 3/2/11	Wed 6/15/11	4	13
9		PC Girder Shop Drawing Rev & Approval	45 days	Wed 3/2/11	Sat 4/23/11	4	14
10		Mech-a-Elect Shop Drawing Review & Appr	111 days	Wed 3/2/11	Sat 7/9/11	4	16,17
11		Design & Fabricate Temporary Cofferdams	15 wks	Wed 3/2/11	Wed 6/15/11	4	32,49
12		Design & Fabricate Temp Trestles N & S	15 wks	Wed 3/2/11	Wed 6/15/11	4	29,30,32,49
13		Bascule Leaves Fabrication	180 days	Wed 6/15/11	Wed 1/11/12	8	71,72,80
14		Approach PC Girder Fabrication	110 days	Sat 4/23/11	Tue 8/30/11	9	96,97
15		**M & E System Procurment**	**230 days**	**Sat 7/9/11**	**Tue 4/3/12**		**73,74**
16		Mechanical System Procurement	230 days	Sat 7/9/11	Tue 4/3/12	10	'2FS-40 days
17		Electrical System Procurement	230 days	Sat 7/9/11	Tue 4/3/12	10	
18		**Site Preparation**	**254 days**	**Wed 5/11/11**	**Fri 3/2/12**		
19		Hazard Material Remediation	14 days	Wed 5/11/11	Fri 5/27/11	6	
20		TESC BMP Installation	14 days	Wed 5/11/11	Fri 5/27/11	6	21,22
21		Building Demo & UST Removal	14 days	Fri 5/27/11	Mon 6/13/11	20	
22		Relocate Existing Utilities	30 wks	Fri 5/27/11	Fri 12/23/11	20	-8 wks,88,89
23		Earthqauke Drain Installation	18 wks	Fri 10/28/11	Fri 3/2/12	22FS-8 w	
24		**Allowable 1st Year In-Water Construction**	169.95 days	Mon 8/1/11	Wed 2/15/12		
25		**Construct Temporary Work Trestle**	**97.98 days**	**Wed 5/11/11**	**Fri 9/2/11**		
26		In-Water Standby Time	70 days	Wed 5/11/11	Mon 8/1/11	6	27,28
27	WTR WRK	**Partially Remove Existing Fender for Bar**	1.33 wks	**Mon 8/1/11**	**Wed 8/10/11**	26	29
28	WTR WRK	**Patially Remove Existing Fender for Bar**	1.33 wks	**Mon 8/1/11**	**Wed 8/10/11**	26	30
29	WTR WRK	**Construct Work Trestle South**	20 days	**Wed 8/10/11**	**Fri 9/2/11**	27,12	32
30	WTR WRK	**Construct Work Trestle North**	20 days	**Wed 8/10/11**	**Fri 9/2/11**	28,12	49
31		**Bascule Span Foundation Foundation & Pier**	**224.4 days**	**Fri 9/2/11**	**Mon 5/21/12**		
32	WTR WRK	**Construct Cofferdam South Bascule**	4 wks	**Fri 9/2/11**	**Fri 9/30/11**	29,11,12	33
33		Excavate within Cofferdam -South Bascule	4 wks	Fri 9/30/11	Fri 10/28/11	32	34,35
34		Install Exterior Waler Truss - South	4 days	Fri 10/28/11	Wed 11/2/11	33	35
35		Backfill Cofferdam with Sand- South	0.4 wks	Wed 11/2/11	Fri 11/4/11	33,34	36
36		Install cutting Shoe- South	2 wks	Fri 11/4/11	Fri 11/18/11	35	37
37		Set Dredge Equipment South	0.93 wks	Fri 11/18/11	Fri 11/25/11	36	38
38		Cast & Dredge Seg 1&2 South	20 days	Fri 11/25/11	Mon 12/19/11	37,54	39
39		Cast & Dredge Seg 3 South	10 days	Mon 12/19/11	Fri 12/30/11	38	40
40		Cast & Dredge Seg 4-South	10 days	Fri 12/30/11	Wed 1/11/12	39	41
41		Cast & Dredge Seg 5-South	10 days	Wed 1/11/12	Mon 1/23/12	40	42
42		Cast & Dredge Seg 6-South	10 days	Mon 1/23/12	Fri 2/3/12	41	43
43	WTR WRK	**Remove Cofferdam- South**	10 days	**Fri 2/3/12**	**Wed 2/15/12**	42	44
44		Cast Pier Cap-South Bascule	16 days	Wed 2/15/12	Mon 3/5/12	43	45,48
45		Complete Excavation South Bascule	10 days	Mon 3/5/12	Fri 3/16/12	44	46
46		Clean & Install Seal-South	4 days	Fri 3/16/12	Wed 3/21/12	45	47
47		Set Submarine Tube Const. Platform	4 days	Wed 3/21/12	Mon 3/26/12	46	68,48
48		Constuct Bascule Pier -South	8 wks	Mon 3/26/12	Mon 5/21/12	44,47	71,96,74
49	WTR WRK	**Construct Cofferdam North Bascule**	4 wks	**Fri 9/2/11**	**Fri 9/30/11**	11,30,12	50
50		Excavate within Cofferdam- North Bascule	4 wks	Fri 9/30/11	Fri 10/28/11	49	51,52
51		Install Exterior Waler Truss -North	4 days	Fri 10/28/11	Wed 11/2/11	50	52
52		Backfill Cofferdam with Sand- North	0.4 wks	Wed 11/2/11	Fri 11/4/11	50,51	53,54
53		Install cutting Shoe- North	2 wks	Fri 11/4/11	Fri 11/18/11	52	55



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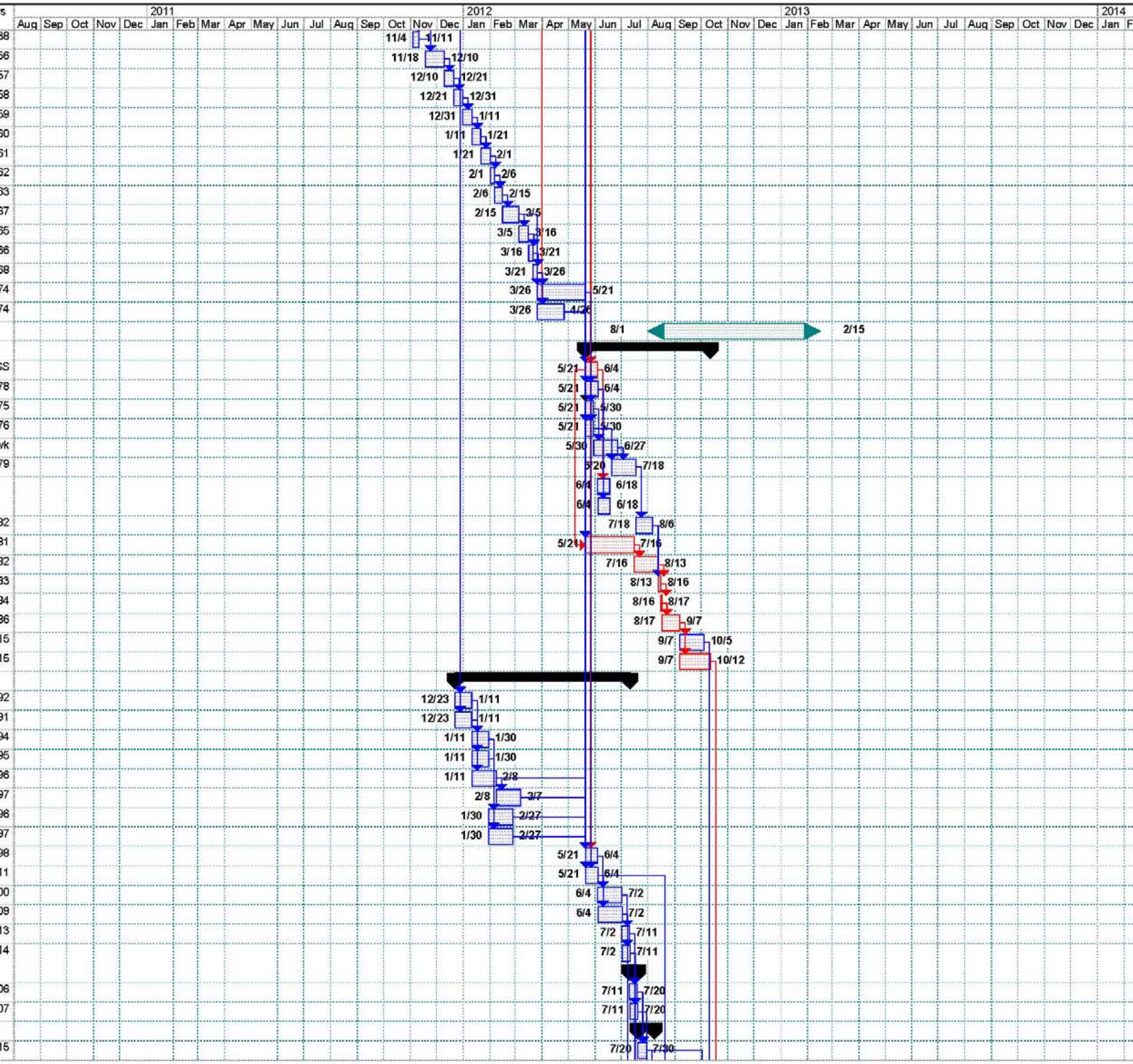
HNTB Job No. 45647

SOUTH PARK BRIDGE - INTERMEDIATE DESIGN CONSTRUCTION SCHEDULE (DRAFT A)

SUBJECT TO CHANGE PENDING DESIGN DEVELOPMENT

ID	Notes	Task Name	Duration	Start	Finish	Predeces	Successors
54		Set Dredge Equipment North	0.93 wks	Fri 11/4/11	Fri 11/11/11	52	38
55	EXT. HRS.	Cast & Dredge Seg 1&2 North	19 days	Fri 11/18/11	Sat 12/10/11	53	56
56	EXT. HRS.	Cast & Dredge Seg 3- North	9 days	Sat 12/10/11	Wed 12/21/11	55	57
57	EXT. HRS.	Cast & Dredge Seg 4- North	9 days	Wed 12/21/11	Sat 12/31/11	56	58
58	EXT. HRS.	Cast & Dredge Seg 5- North	9 days	Sat 12/31/11	Wed 1/11/12	57	59
59	EXT. HRS.	Cast & Dredge Seg 6- North	9 days	Wed 1/11/12	Sat 1/21/12	58	60
60	EXT. HRS.	Cast & Dredge Seg 7- North	9 days	Sat 1/21/12	Wed 2/1/12	59	61
61	EXT. HRS.	Cast & Dredge Seg 8- North	4 days	Wed 2/1/12	Mon 2/6/12	60	62
62	WTR WRK	**Remove Cofferdam- North**	8 days	**Mon 2/6/12**	**Wed 2/15/12**	61	63
63		Cast Pier Cap-North Bascule	16 days	Wed 2/15/12	Mon 3/5/12	62	64,67
64		Complete Excavation North Bascule	10 days	Mon 3/5/12	Fri 3/16/12	63	65
65		Clean & Install Seal-North	4 days	Fri 3/16/12	Wed 3/21/12	64	66
66		Set Submarine Tube Const. Platform	4 days	Wed 3/21/12	Mon 3/26/12	65	67,68
67		Constuct Bascule Pier - North	8 wks	Mon 3/26/12	Mon 5/21/12	63,66	72,97,73,74
68		Bored Submarine Cable Pilot Tubes	4.5 wks	Mon 3/26/12	Thu 4/26/12	47,66	74
69		**Allowable 2nd Year In-Water Work**	28.48 wks	Wed 8/1/12	Fri 2/15/13		
70		**Bascule Span Superstructure Construction**	**124 days**	**Mon 5/21/12**	**Fri 10/12/12**		
71		Bascule Leaf Erection-Heel Portion South	2 wks	Mon 5/21/12	Mon 6/4/12	48,13,16I	77,80SS
72		Bascule Leaf Erection-Heel Portion North	2 wks	Mon 5/21/12	Mon 6/4/12	67,13,16I	78
73		Install Electrical Feed to North Bascule Pier	1.33 wks	Mon 5/21/12	Wed 5/30/12	67,15	75
74		Install Electrical Feed to South Bascule Pier	1.33 wks	Mon 5/21/12	Wed 5/30/12	68,48,67,	75,76
75		Install Mech & Electric-South	4 wks	Wed 5/30/12	Wed 6/27/12	73,74	76FS-1 wk
76		Install Mech & Electric-North	4 wks	Wed 6/20/12	Wed 7/18/12	74,75FS-	79
77		Form & Cast South Counterweight Partial	2 wks	Mon 6/4/12	Mon 6/18/12	71	
78		Form & Cast North Counterweightt. Partial	2 wks	Mon 6/4/12	Mon 6/18/12	72	
79		Mech & electrcal System Testing & Adjustm	2.67 wks	Wed 7/18/12	Mon 8/6/12	76	82
80		Erect Center Portion Bascule Leaves on Ba	8 wks	Mon 5/21/12	Mon 7/16/12	13,71SS	81
81		Install Deck, Pavement & Barriers Bascule	4 wks	Mon 7/16/12	Mon 8/13/12	80	82
82		Float-In, Center Bascule Span	3 days	Mon 8/13/12	Thu 8/16/12	81,79	83
83		Open New Bascule Span for River Traffic	1 day	Thu 8/16/12	Fri 8/17/12	82	84
84		Complete CNWT and Balancing	3 wks	Fri 8/17/12	Fri 9/7/12	83	85,86
85		Mech & Elect System Testing & Adjusting	4 wks	Fri 9/7/12	Fri 10/5/12	84	115
86		Construct Bascule Fixed Span Deck	30 days	Fri 9/7/12	Fri 10/12/12	84	115
87		**Approach Structure Construction**	**172.36 days**	**Fri 12/23/11**	**Wed 7/11/12**		
88		Construct Approach Pier Foundations- Sout	2.67 wks	Fri 12/23/11	Wed 1/11/12	22	90,92
89		Construct Approach Pier Foundations- Nortl	2.67 wks	Fri 12/23/11	Wed 1/11/12	22	91
90		Construct Approach Abut Foundations-Sout	2.67 wks	Wed 1/11/12	Mon 1/30/12	88	94
91		Construct Approach Abut Foundations-Nortl	2.67 wks	Wed 1/11/12	Mon 1/30/12	89	95
92		Construct Approach Pier-South	4 wks	Wed 1/11/12	Wed 2/8/12	88	93,96
93		Construct Approach Pier-North	4 wks	Wed 2/8/12	Wed 3/7/12	92	97
94		Construct Approach Abut-South	4 wks	Mon 1/30/12	Mon 2/27/12	90	96
95		Construct Approach Abut-North	4 wks	Mon 1/30/12	Mon 2/27/12	91	97
96		Approach Span Girder Erection-South	2 wks	Mon 5/21/12	Mon 6/4/12	94,14,48,	98
97		Approach Span Girder Erection-North	2 wks	Mon 5/21/12	Mon 6/4/12	95,93,67,	99,111
98		Construct Approach Deck -South	4 wks	Mon 6/4/12	Mon 7/2/12	96	100
99		Construct Approach Deck -North	4 wks	Mon 6/4/12	Mon 7/2/12	97	101,109
100		Construct Approach Sidewalk & Barricades-	1.33 wks	Mon 7/2/12	Wed 7/11/12	98	103,113
101		Construct Approach Sidewalk & Barricades-	1.33 wks	Mon 7/2/12	Wed 7/11/12	99	104,114
102		**Utility Lines**	**8.4 days**	**Wed 7/11/12**	**Fri 7/20/12**		
103		Install Under Deck Utility Lines - South	1.33 wks	Wed 7/11/12	Fri 7/20/12	100	106
104		Install Under Deck Utility Lines -North	1.33 wks	Wed 7/11/12	Fri 7/20/12	101	106,107
105		**Luminaries, Traffic Signals and Gates**	**15.96 days**	**Fri 7/20/12**	**Wed 8/8/12**		
106		Install Luminaries, Traffic Signals and Gate-	1.33 wks	Fri 7/20/12	Mon 7/30/12	103,104	107,115



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HNTB Job No. 45647

SOUTH PARK BRIDGE - INTERMEDIATE DESIGN CONSTRUCTION SCHEDULE (DRAFT A)

SUBJECT TO CHANGE PENDING DESIGN DEVELOPMENT

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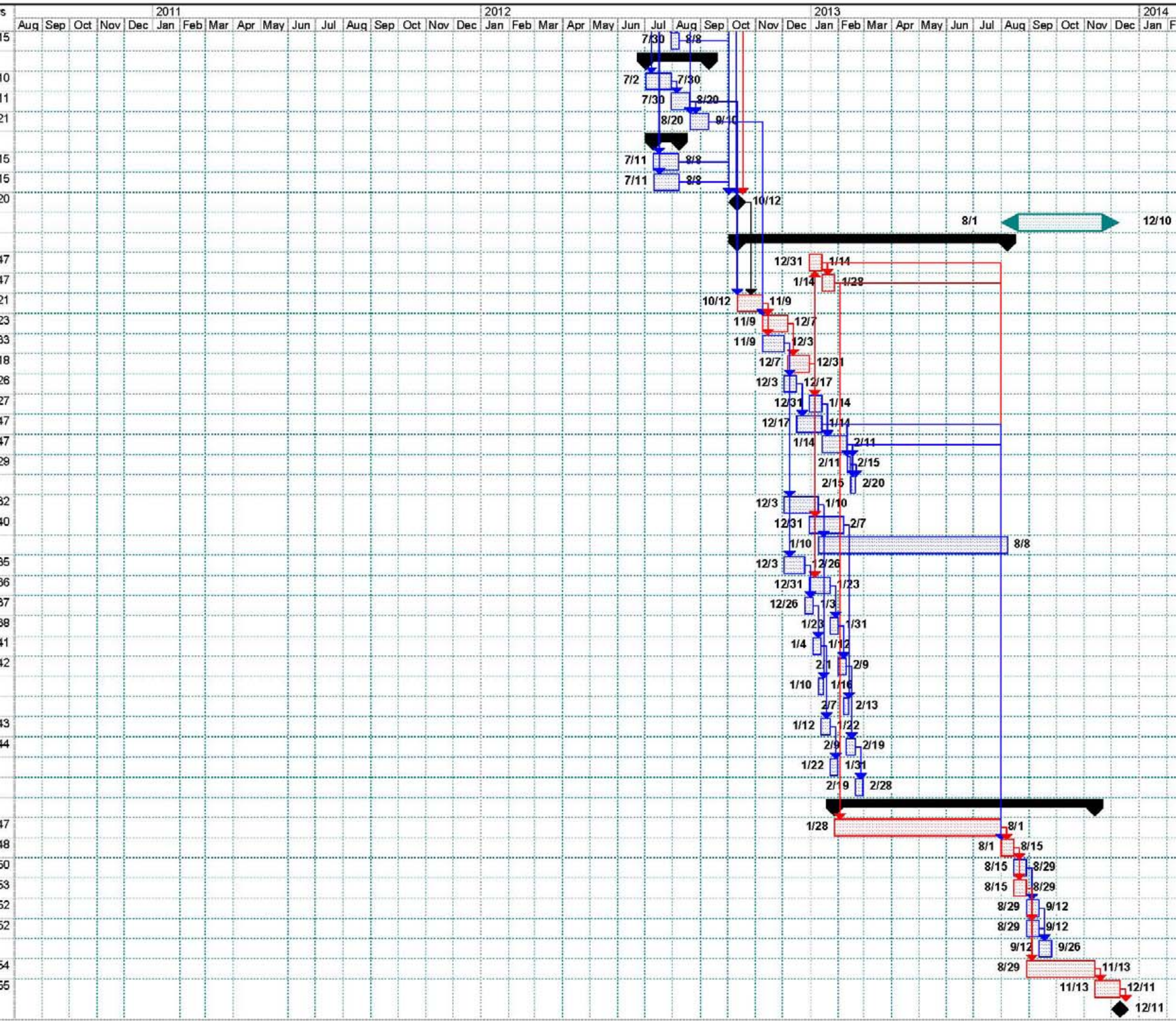
107		Install Luminaries, Traffic Signals and Gate-	1.33 wks	Mon 7/30/12	Wed 8/8/12	104,106	115
108		**Remove Temporary Work Trestls for New Bri**	**60 days**	**Mon 7/2/12**	**Mon 9/10/12**		
109		In-Water Standby Time	24 days	Mon 7/2/12	Mon 7/30/12	99	110
110	WTR WRK	**Remove Construction Trestle-South**	3 wks	**Mon 7/30/12**	**Mon 8/20/12**	109	120,111
111	WTR WRK	**Remove Construction Trestle-North**	3 wks	**Mon 8/20/12**	**Mon 9/10/12**	97,110	121
112		**Construct Approach Roadways**	**24.42 days**	**Wed 7/11/12**	**Wed 8/8/12**		
113		Construct Approach Roadways-South	4 wks	Wed 7/11/12	Wed 8/8/12	100	115
114		Construct Approach Roadways-North	4 wks	Wed 7/11/12	Wed 8/8/12	101	115
115		Divert Traffic to New Bridge	0 days	Fri 10/12/12	Fri 10/12/12	113,114,	120
116		**Allowable 3nd Year In-Water Work**	112 days	Thu 8/1/13	Tue 12/10/13		
117		**Remove Existing Bridge**	**256.98 days**	**Fri 10/12/12**	**Thu 8/8/13**		
118	WTR WRK	**Remove Existing Fender-South**	2 wks	**Mon 12/31/12**	**Mon 1/14/13**	123	119,147
119	WTR WRK	**Remove Existing Fender-North**	2 wks	**Mon 1/14/13**	**Mon 1/28/13**	118	146,147
120	WTR WRK	**Drive Steel Piles for Construction Trestle**	4 wks	**Fri 10/12/12**	**Fri 11/9/12**	110,115	122,121
121	WTR WRK	**Drive Steel Piles for Construction Trestle**	4 wks	**Fri 11/9/12**	**Fri 12/7/12**	111,120	123
122	WTR WRK	**Construct Trestle-South**	3.33 wks	**Fri 11/9/12**	**Mon 12/3/12**	120	124,130,133
123	WTR WRK	**Construct Trestle-North**	3.33 wks	**Fri 12/7/12**	**Mon 12/31/12**	121	,134,131,118
124		Remove existing Bascule Deck-South	2 wks	Mon 12/3/12	Mon 12/17/12	122	126
125		Remove existing Bascule Deck-North	2 wks	Mon 12/31/12	Mon 1/14/13	123	127
126		Remove existing Bascule Leaf-South	4 wks	Mon 12/17/12	Mon 1/14/13	124	128,147
127		Remove existing Bascule Leaf-North	4 wks	Mon 1/14/13	Mon 2/11/13	125	128,129,147
128		Remove existing Mech & Elect-South	0.67 wks	Mon 2/11/13	Fri 2/15/13	126,127	129
129		Remove existing Mech & Elect-North	0.67 wks	Fri 2/15/13	Wed 2/20/13	127,128	
130		Remove Approach Spans/Piers - South	5.5 wks	Mon 12/3/12	Thu 1/10/13	122	139,132
131		Remove Approach Spans/Piers - North	5.5 wks	Mon 12/31/12	Thu 2/7/13	123	140
132		Const. South Landscaping & Roads	180 days	Thu 1/10/13	Thu 8/8/13	130	
133	WTR WRK	**Construct Existing Basc Pier Cofferdam-**	3.33 wks	**Mon 12/3/12**	**Wed 12/26/12**	122	135
134	WTR WRK	**Construct Existing Basc Pier Cofferdam-**	3.33 wks	**Mon 12/31/12**	**Wed 1/23/13**	123	136
135		Remove Existing Lower Bascule Pier-South	1.33 wks	Wed 12/26/12	Thu 1/3/13	133	137
136		Remove Existing Lower Bascule Pier-North	1.33 wks	Wed 1/23/13	Thu 1/31/13	134	138
137		Remove Existing Bascule Pier Foundation-S	1.33 wks	Fri 1/4/13	Sat 1/12/13	135	141
138		Remove Existing Bascule Pier Foundation-N	1.33 wks	Fri 2/1/13	Sat 2/9/13	136	142
139	WTR WRK	**Install Cofferdam Approach Piers South**	5 days	**Thu 1/10/13**	**Wed 1/16/13**	130	
140	WTR WRK	**Install Cofferdam Approach Piers North**	5 days	**Thu 2/7/13**	**Wed 2/13/13**	131	
141		Remove Existing Approach Piers-South	1.33 wks	Sat 1/12/13	Tue 1/22/13	137	143
142		Remove Existing Approach Piers-North	1.33 wks	Sat 2/9/13	Tue 2/19/13	138	144
143		Remove Existing Approach Piers Foundatio	1.33 wks	Tue 1/22/13	Thu 1/31/13	141	
144		Remove Existing Approach Piers Foundatio	1.33 wks	Tue 2/19/13	Thu 2/28/13	142	
145		**Construct New Fender Sys & Remove Trestle**	**248 days**	**Mon 1/28/13**	**Wed 11/13/13**		
146		In-Water Standby Time	185 edays	Mon 1/28/13	Thu 8/1/13	119	147
147	WTR WRK	**Constr Part of New Fender System-South**	2 wks	**Thu 8/1/13**	**Thu 8/15/13**	146,118,	149,148
148	WTR WRK	**Constr Part of New Fender System-North**	2 wks	**Thu 8/15/13**	**Thu 8/29/13**	147	150
149	WTR WRK	**Remove So Demo. Trestle & Cofferdams**	2 wks	**Thu 8/15/13**	**Thu 8/29/13**	147	151,153
150	WTR WRK	**Remove No. Demo. Trestle & Cofferdams**	2 wks	**Thu 8/29/13**	**Thu 9/12/13**	148	152
151	WTR WRK	**Complete So. Prop. Fender Sys.**	2 wks	**Thu 8/29/13**	**Thu 9/12/13**	149	152
152	WTR WRK	**Complete No. Prop. Fender Sys.**	2 wks	**Thu 9/12/13**	**Thu 9/26/13**	150,151	
153	WTR WRK	**Shoreline Restoration**	65 days	**Thu 8/29/13**	**Wed 11/13/13**	149	154
154		Cleaning / Site Preparation / Demob.	4 wks	Wed 11/13/13	Wed 12/11/13	153	155
155		Contractor Off Site	0 wks	Wed 12/11/13	Wed 12/11/13	154	



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