MAGNOLIA BRIDGE REPLACEMENT

REHABILITATION ALTERNATIVE

STUDY REPORT - DRAFT

October 2005

Seattle Department of Transportation
Agreement No. T01-34

HNTB

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1. INTRODUCTION

1.1 Purpose

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

1.2 Scope of Work

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

1. Conduct geotechnical fieldwork and conceptual engineering, prepare a technical memorandum for an existing bridge rehabilitation alternative that includes recommendations on ground improvement, pile capacity, shaft capacity, and footing bearing capacity. (See Appendix __, the Geotechnical Technical Memorandum.)
2. Perform an inspection of the existing portion of the bridge that will remain after rehabilitation to determine the condition of the bridge. Collect concrete samples to determine concrete strength, chloride content and depth of carbonation. Test existing reinforcement for corrositivity. Inspect existing timber piles to determine the condition of piles. (See Inspection Report.)
3. Analyze typical bents for current dead load and live loads. Develop a concept for replacement of existing crossbeams that do not have adequate capacity.
4. Analyze the existing bridge from Bent 18 to Bent 82 under current seismic design loads according to AASHTO LRFD Code, Third Edition, dated 2004.
5. Develop a retrofit concept that will bring the existing bridge into conformance with current seismic requirements.
6. Prepare a detailed cost estimate for the seismic retrofits based on unit prices.
7. Investigate the use of cathodic protection for those portions of the existing bridge that will remain after the structure is rehabilitated. (See Corrosion Evaluation Report)
8. Develop a life cycle cost estimate for the rehabilitated bridge over a 75-year life of the structure. This cost will be compared to a life cycle cost for a bridge replacement alternative.
9. Prepare a draft and final report summarizing the conclusions, details, and costs for items included in existing bridge rehabilitation.

The work was completed with the following basic assumptions about the scope of work:

- No additional load analysis required to verify superstructure capacity. Existing Load Ratings will be used to determine capacity.
- The center ramp to Port of Seattle Termina1 91 between Bent 18 and Bent 34 will be demolished and replaced with a new structure at deck level.

- The vertical curve deviation over the BNSF Railway will be eliminated. Assume the ramp structure from surface 15th Avenue West to Bent 18 will be demolished and rebuilt. No analysis required for this structure. Square foot costs will be used for replacement bridge structure.
- The concrete trusses between Bent 61 and Bent 76 will be replaced on the existing alignment. The horizontal curve deviation near Magnolia Bluff will remain in place. The structure at this location will be analyzed for seismic response.
- Crossbeams will be replaced for all remaining bents.
- Soil improvements will be completed as part of the rehabilitation.

1.3 Design Criteria

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

B. LAYOUT
1. The spans and general arrangement of the structure are shown on the existing Magnolia Bridge Plans. The superstructure will be replaced in kind with no additional width added to bridge. The existing bridge width meets existing local agency standards.
2. Design Speed: 35 MPH

C. DESIGN LOADS
1. Dead Load
    a. Structural Dead Loads
        1) Concrete = 160 pcf
        2) Structural Steel = 490 pcf
    b. Superimposed Dead Load
        1) No allowance shall be made for the weight of initial wearing surface.
        2) An allowance shall be made for the weight of a 2" future wearing surface (25 psf).
        3) An allowance of 100 pounds per linear foot shall be provided for utilities.
        4) One 6' wide sidewalk, with 6" depth, on one side of bridge structure.
        5) (1) WSDOT 34" Single Slope Traffic Barrier with a weight of 475 pounds per lineal foot for each barrier. (1) WSDOT 32" Pedestrian Barrier with a weight of 450 pounds per lineal foot, including metal handrail.
2. Live Load - Vehicular live load shall be AASHTO LRFD HL-93.
3. Pedestrian Load – Pedestrian live loads shall be applied in accordance with AASHTO LRFD 3.6.1.6.
4. Seismic Forces

   a. The structure shall be analyzed and designed in accordance with the AASHTO LRFD and WSDOT BDM LRFD Chapter 4.
   b. Acceleration Coefficient = .30g.
   c. Seismic Performance Zone: 4
   d. Importance Category: Essential
   e. Soil Profile Type: Type III for improved ground, see Geotechnical Report.
   f. Use Multimodal Spectral Method for seismic analysis. The elastic seismic response spectrum will be in accordance with AASHTO LRFD section 3.10.6.1.
5. Wind and Thermal Loads
   a. Not reviewed, assumed that controlling lateral loads will be seismic forces.

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

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

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

1.4 Description of Existing Bridge

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

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

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

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

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

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

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

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

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

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

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

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

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

1.5 Discussion of Study Procedure

1.5.1 Dead Load and Live Load Evaluation

The dead loads and live loads were evaluated by creating a longitudinal live load spine model of Bents 18 to 82 using the Risa-2D program. Expansion joints were located similar to existing joints, creating 4-span and 5-span units. The HL-93 live load was applied in different combinations to determine reactions at each bent. The bents were then organized into groups by number of columns, column diameter and column spacing. The controlling bent in each group was then analyzed separately in Risa-2D Version 6 for dead load and live load. In the bent models, the existing superstructure was replaced with prestressed slab spans. The existing crossbeam and column cap were replaced with a new cast-in-place concrete crossbeam. The new crossbeam size was calculated to withstand the moment produced by the new loads. The existing columns were checked using the WinRecol Version 4 program. The timber piles were checked for axial loads.

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

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

1.5.2 Seismic Evaluation

Determining the seismic performance of the existing structure and developing a structural scheme that meets current code requirements involved a four-step process: creating a model of the existing structure to analyze the bridge; identifying structural members that have insufficient capacity; developing structural systems to accommodate seismic forces; and analyzing the rehabilitated structure to verify that the proposed structural systems have capacity.

1.5.3 Model Existing Structure

A wire-frame model of the existing structure from Bent 18 to the Abutment at Bent 82 was created using the LUSAS finite element modeling program, Version 13.6 (see Figure 1). The superstructure was modeled as a single spine element with appropriate stiffness and mass properties, based on a new prestressed slab structure. The superstructure was typically separated into 4-span or 5-span units, similar to the configuration used for the dead load and live load evaluation. The units were free to move longitudinally at the expansion bents, but were pinned in the transverse direction. The concrete columns and steel bracing were modeled with representative section and material properties. Any existing longitudinal concrete bracing was assumed to be replaced with steel bracing similar to the 1961 longitudinal bracing retrofits. Cracked section properties were used for the columns with a 50% reduction in the gross moment of inertia for the sections. The structure was modeled down to the top of the pile caps with representative section properties for the footing pedestals. The footings were modeled with translational springs that were developed based on analysis of the pile stiffness using the LPILE program and the stiffness of the pedestal bases with passive pressure of the surrounding soils.

The structure was analyzed using a linear elastic multi-modal spectral analysis as defined in both the LRFD Code guidelines and the WSDOT LRFD Bridge Design Manual. A seismic response spectrum curve based on LRFD Code guidelines was applied to the modeled structure. The acceleration was 0.30g with a Type III soil profile. Forces and moments were combined using a complete quadratic combination (CQC) method. Two load combinations were considered for the analysis. The longitudinal design load combination included 100% of the longitudinal load case plus 30% of the transverse load case. The transverse design load combination included 30% of the longitudinal case plus 100% of the transverse load case. The Magnolia Bridge is considered to be an Essential Structure as defined in the LRFD Code.

There are two primary structural systems on the existing Magnolia Bridge. Bent 18 to Bent 46 is an unbraced system where seismic forces are resisted by shear and flexure in the columns. Bent 47 to Bent 81 is a braced system where seismic forces are resisted by axial forces in column and bracing members. The response of the structure varied depending on the type of structural system. In general, the unbraced structural units between Bent 18 and Bent 46 had longitudinal periods in excess of two seconds, while the braced structure between Bent 62 to Bent 76 had periods below one second. The transverse periods for both the unbraced frames and the braced frames were approximately one second or less, with the braced frames tending to have shorter periods. The braced frame structure results in a very stiff structure with a relatively short structural period. There is no ductility in the system as would normally be designed in new bridge foundations. The structure response is in the plateau part at the top of the response spectrum so the force in these areas of the structure is maximum.

1.5.4 Check Existing Structure Capacity

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

Columns

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

Transverse and Longitudinal Bracing

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

Foundations

The timber piles in all footings did not have sufficient lateral or axial capacity. Most of the foundations between Bent 49 and Bent 81 had uplift due to the seismic loads.

1.5.5 Develop Structural Systems to Accommodate Seismic Forces

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

Bent 18 to Bent 46

Since the columns do not have adequate capacity to resist seismic forces in shear and flexure a structural system needs to be provided to resist these forces. There are a couple of options available, including: column jacketing, providing longitudinal and transverse bracing for each bent, or providing shear walls for multiple span units. CalTrans recommends using column jacketing for structures where the D/C ratios do not exceed 6. Since the D/C ratios are high, column jacketing was disregarded as a viable option for this structure. Shear walls were not used because of the height of the structure and because bracing is much less expensive. For this rehabilitation study and cost estimate, the braced system was used. (See Figure 3) Transverse cross-bracing was provided at every bent. Longitudinal bracing was provided at every other span along the exterior line of columns on the north and south sides of the structure, so that all bents were braced in the longitudinal direction. The drawback of this system is that access under the bridge is limited due to the bracing systems. A shear wall system may require only half as many spans to be obstructed. The braced system was used for the Bents 18, 19 and 35 to 46. The connections to the columns will be similar to the collars used for the 2001 seismic retrofit.

A different system was used for Bent 20 to Bent 34 since the interior columns at these bents will be demolished as part of the Access Ramp Replacement. At these locations the interior columns will be replaced with 4’-0” diameter columns that are designed to resist all the seismic forces in bents. (See Figure 2)

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

Bent 47 to Bent 81

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

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

1.5.6 Proposed Rehabilitation

The Rehabilitation Alternative would bring the existing bridge up to current design standards and extend the life of the structure for about 75 years. This would be done by rehabilitating elements of the bridge, such as columns and foundations, to meet current standards, or replacing elements, such as the bridge deck, that can not be rehabilitated. Table 1 presents the results of the capacity analyses as a listing of deficiencies and proposed approaches to eliminating the deficiencies.

Table 2 describes the proposed rehabilitation elements and is keyed to Figure 1.

Table 1 Summary of Results

Deficiency	**Proposed Approach**
Ramp Structure over 15th Avenue West does not have sufficient seismic capacity.	Retrofit 15th Avenue West overpass structure to include longitudinal restrainers, transverse shear blocks, column jacketing, and additional pile foundation.
Vertical curve on ramps from 15th Avenue West to railroad crossing does not meet stopping sight distance requirements.	Replace approach fill, walls and ramps from Bent 1 to Bent 18 and build in a new profile that meets requirements. Cost based on cost per square foot cost estimate for new structure.
Roadway slab superstructure of bridge west of 15th Avenue does not meet current live load capacity requirements.	Replace superstructure with prestressed slab bridge. Spans 18 to 61 and Spans 78 to 82 uses 1'-6" prestressed slab with 5" deck. Spans 62 to 77 use 2' – 2" prestressed slab with 5" deck
Crossbeam will not support current live loads.	Replace crossbeam and column cap pedestals.
Timber piles do not meet current dead and live load capacity requirements.	Grade beams and drilled shafts are proposed for seismic performance. They will provide sufficient additional dead and live load capacity.
Concrete truss spans have reached end of service life and contain non-redundant structural elements.	Replace truss spans with prestressed slab bridge.
Horizontal curve from Bents 68 to 76 does not meet sight distance requirements.	Request a deviation for this section since no accidents have been recorded in this area.
Center ramp from Bents 20 to 34 does not meet current live load requirements and does not have sufficient seismic capacity.	Remove and replace interior deck and columns. Replace with prestressed slabs and 4' diameter circular columns that will take all the lateral seismic forces.
Insufficient lateral seismic capacity for unbraced Bents 18, 19 and 35 to 46. Moments exceeded capacity of columns.	Provide lateral cross bracing between columns.
Insufficient uplift capacity of columns to resist lateral seismic overturning forces for Bents 47 to 81.	Case columns in steel jackets that will carry the uplift force to the foundation.
Insufficient longitudinal seismic capacity for Bents 18, 19, and 35 to 82.	Provide longitudinal cross bracing for each bent.
Timber pile foundations are not sufficient for seismic lateral forces and uplift forces for Bents 18, 19, 35 to 46 and 59 to 81.	Provide grade beam between columns with two 6' diameter drilled shafts.
Timber pile foundations are not sufficient for seismic lateral forces and uplift forces for Bents 20 to 34 at the center ramp.	Lateral forces will be resisted by new interior columns, therefore provide grade beam between exterior columns with two 6' diameter drilled shafts between outside two columns.
Timber pile foundations are not sufficient for seismic lateral forces and uplift forces for Bents 47 to 58.	On and off ramps preclude placement of drilled shafts adjacent to bridge, therefore provide grade beam between columns with two 4' diameter drilled shafts placed longitudinally to columns each side of bridge.
The lateral cross brace system for Bent 62 to Bent 76 does not have sufficient strength to resist lateral seismic forces.	The proposal is to replace the existing bracing with new bracing that meets current code requirements.
Bracing systems in which all braces are oriented in the same direction are not allowed by current design guidelines MCEER/ATC 49. This requirement is so there is redundancy in the bracing system.	Replace existing Z bracing with X bracing.

Deficiency	**Proposed Approach**
The width to thickness ratio, b/t, for the braces does not meet current design guidelines MCEER/ATC 49 requirements. This requirement is to prevent local buckling of the bracing members.	Replace existing Z bracing with X bracing that meets thickness requirements.
Most of the existing longitudinal bracing system does not have sufficient strength to resist longitudinal seismic forces.	Replace the existing bracing with new longitudinal bracing.
Potential test measurements on reinforcement indicates a high probability of active corrosion at specific locations.	Provide a galvanic type corrosion protection utilizing a flame-spray zinc at specific locations.
Soils are potentially liquefiable.	Provide injection grouting of soils to prevent liquefaction and loss of capacity of foundations.
On and off ramps to 23rd Avenue were designed to and HS20 live load and a seismic acceleration coefficient of 0.2G.	Current code requirements are 0.3G and HL-93 live load which are both larger than original design. Therefore the ramp may need strengthening but was not included in current study effort.

Table 2 Rehabilitation Elements

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

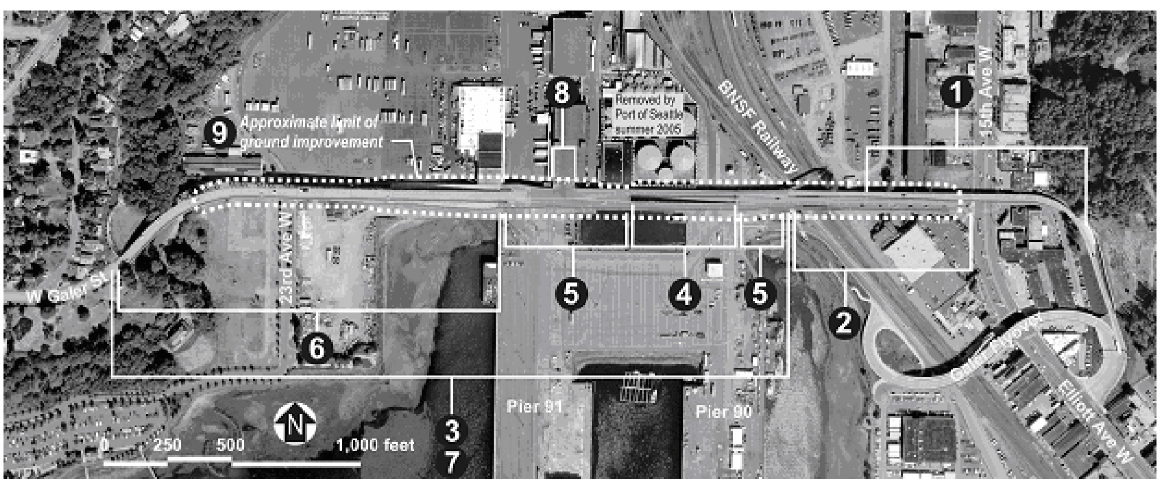

Figure 1 Rehabilitation Alternative Key Map

1.6 Discussion of Deficiencies and Proposed Approach

1.6.1 East Ramp Structure Over 15th Avenue West

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

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

1.6.2 Vertical Stopping Sight Distance

The vertical crest curve on the ramp from 15th Avenue West to railroad crossing does not meet stopping sight distance requirements. As part of the Magnolia Bridge Rehabilitation, the vertical curve deviation over the BNSF Railway would be eliminated. This revision would require raising the profile grade several feet in the section of bridge between 15th Avenue West and Bent 18. As a result of this fix, it is assumed that the existing bridge structure from 15th Avenue West to Bent 18 would be removed and replaced with new structure. The replacement structure would likely be an MSE wall transitioning into a steel structure over the railroad. The span lengths would be much longer for this new structure and a single span would cross the BNSF Railway. Cost of replacement structure is based on cost per square foot for new structure, as determined in the SCORE evaluation for the replacement structure cost of Alternative A-6 or A-7.

1.6.3 Slab Capacity

It was determined from the bridge load ratings that the existing cast-in-place concrete superstructure from Bent 18 to Bent 62 and Bent 75 to Bent 82 was inadequate for an HS-20 load. As a result, it is assumed that all superstructure on the existing bridge west of Bent 18 would be removed and replaced. The steel trusses and braces and any associated collars and connections that were installed in 1961 and later to support the existing cast-in-place concrete deck would also be removed. The replacement structure will be either 1'-6" or 2'-2" deep precast concrete flat slabs with a 5" wearing surface, similar to the superstructure used on the existing ramps accessing 23rd Avenue West, constructed in 1991. (See Figures 2 through 6 on pages 18 through 20.)

1.6.4 Crossbeam Capacity

The existing drop-down crossbeams for Bent 18 to Bent 62 and Bent 75 to Bent 82 were also found to be inadequate for an HS-20 load rating. Therefore, it is proposed that all drop-down crossbeams and the ornamental column caps would be removed and replaced with a 4-foot by 4-foot cast-in-place concrete crossbeam similar to the type used on the existing ramps accessing 23rd Avenue West. A 4-foot by 5-foot deep concrete crossbeam would also be utilized in the replacement of the trusses for Bent 62 to Bent 75. (See Figures 2 through 6 on pages 18 through 20.)

1.6.5 Dead and Live Load

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

1.6.6 Concrete Trusses

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

1.6.7 Horizontal Sight Distance

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

1.6.8 Center Ramp

The center ramp from Bents 20 to 34 does not meet current live load requirements and does not have sufficient seismic capacity. It also results in a substandard edge of lane taper rate where the eastbound through lane transitions from the center ramp area to the two-lane ramp to 15th Avenue West. The columns supporting the center ramp also show the most significant signs of corrosion. The proposed solution is to remove and replace interior deck and columns. The replacement structure is a prestressed slab supers tructure with 4' diameter circular columns. The new interior columns will take all lateral seismic forces. Drilled shaft foundations would be required to support the applied seismic forces. (See Figure 2 on page 18.)

1.6.9 Unbraced Bents

The unbraced Bents 18, 19 and 35 to 46 do not have sufficient moment capacity to resist the lateral seismic forces. All lateral load is currently resisted by shear and flexure in the columns. The proposed solution is to provide lateral bracing between the columns so lateral forces are resisted by braced frames. (See Figure 3 on page 18.)

1.6.10 Column Seismic Overturning Capacity

As the braced frames get taller in the bents leading up to Magnolia, the capacity of the columns is exceeded by the seismic forces. The lateral force is resisted by a moment couple of upward and downward forces in the columns. The uplift capacity of column is insufficient to resist lateral seismic overturning forces for Bents 47 to 81. In order to increase the tension capacity of the columns, steel jackets around the columns are proposed that would carry the uplift force to the foundation. The steel jacket would need to be detailed with studs so that it acts compositely with concrete column. (See Figure 6 on page 20.)

1.6.11 Longitudinal Seismic Capacity

The existing longitudinal bracing is insufficient for calculated longitudinal seismic forces for Bents 18, 19, and 35 to 82. Longitudinal cross bracing is proposed for each bent. Bracing would be along exterior column lines. Cross bracing would be added to exterior column lines between Bents 18 and 19, 35 and 36, 37 and 38, 39 and 40, 41 and 42, 43 and 44, and 45 and 46. Existing concrete bracing would be replaced with steel cross bracing between Bents 43 and 44. Cross bracing would be added to both column lines for Bents 47 and 48, 48 and 49, 52 and 53, 53 and 54, and 57 and 58. Existing concrete bracing would be replaced with steel cross bracing on Bents 50 and 51, 55 and 56, 59 and 60, 61 and 62, 63 and 64, 67 and 68, 71 and 72, and 75 and 76. (See Figure 5 on page 20.)

1.6.12 Timber Pile Foundations

The timber pile foundations are not sufficient to resist lateral seismic forces and for the increased dead and live loads to meet current design requirements. Timber piles have a low lateral resistance and are not connected to footings so they have no uplift capacity. For Bents 18, 19, 35 to 45 and 59 to 81, a grade beam would be provided between columns. The grade beam would be supported by two, 6-foot diameter drilled shafts. (See Figure 3 on page 18.)

For Bents 20 to 34 at the center ramp, lateral forces would be resisted by new interior columns. A grade beam would be provided between exterior columns with two 6-foot diameter drilled shafts between the outside two columns. (See Figure 2 on page 18.)

For Bents 47 to 58, the on and off ramps would preclude placement of drilled shafts adjacent to bridge. The grade beam would be provided between columns with two 4-foot diameter drilled shafts placed longitudinally to columns on each side of bridge. (See Figure 4 on page 19.)

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

1.6.13 Transverse Bracing Strength

The existing steel transverse cross brace system for Bents 62 to Bent 76 needs to be strengthened to accommodate the large seismic loads specified under current design standards. The proposal would replace the existing bracing with new bracing that has adequate capacity for seismic loads and meets current code requirements. (See Figure 6 on page 20.)

1.6.14 Lateral Bracing Orientation

Bracing systems in which all braces are oriented in the same direction are not allowed by current design guidelines MCEER/ATC 49. This requirement is so there is redundancy in the bracing system. The proposed solution would replace existing “Z” bracing with “X” bracing. (See Figure 6 on page 20.)

1.6.15 Lateral Bracing Section Properties

The width to thickness ratio, b/t, for the braces does not meet current design guidelines MCEER/ATC 49 requirements. This requirement is to prevent local buckling of the bracing members. Existing “Z” bracing would be replaced with “X” bracing that meets thickness requirements. (See Figure 6 on page 20.)

1.6.16 Reinforcement Corrosion

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

1.6.17 Liquefiable Soils

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

1.6.18 23rd Avenue West On and Off Ramps

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

1.6.19 Access Under the Bridge

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

1.7 Cost Estimate

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

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

2. LIFE CYCLE COST ANALYSIS

[To be completed during schedule and cost risk analysis.]

3. RELATED DOCUMENTS

Copies of existing documents related to the Magnolia Bridge were obtained from the City of Seattle in order to complete the studies summarized in this report and specific text from these reports was used to prepare this report. These documents include the following:

- West Garfield Street Viaduct construction plans prepared by the City of Seattle, dated 1929.
- West Garfield Bridge Repairs, Etc. construction plans prepared by the City of Seattle, dated 1953.
- W. Garfield Street-15th Ave. West Grade Separation construction plans prepared by the City of Seattle, dated 1957.
- West Garfield Street Bridge Rehabilitation construction plans prepared by Arnold, Arnold & Associates, dated 1959.
- Magnolia Bridge (Garfield Street Bridge) Rehabilitation construction plans prepared by Arnold, Arnold & Associates, dated 1959.
- Magnolia Bridge (East Half) Rehabilitation construction plans dated prepared by the City of Seattle, 1967.
- Magnolia Bridge (East Half) Rehabilitation construction plans prepared by Arnold, Arnold & Associates, dated 1974.
- Magnolia Bridge (West Half) Expansion Joint Rehabilitation construction plans prepared by the City of Seattle, dated 1975.
- Magnolia Bridge (West Half) Rail Replacement construction plans prepared by the City of Seattle, dated 1981.
- Magnolia Bridge (West Half) Resurfacing construction plans dated prepared by the City of Seattle, 1985.
- Magnolia Bridge Emergency Slide Repair Emergency Contract #3 construction plans prepared by the City of Seattle, dated 1997.
- Magnolia Bridge Earthquake Damage Repair construction plans prepared by Andersen Bjornstad Kane Jacobs, Inc., dated 2001.
- Magnolia Extension Bridge Seismic Retrofit Program Phase 1-Contract 5 (part of West Galer Street Flyover contract) construction plans prepared by CH2M Hill, dated 2000.
- Magnolia Bridge Load Rating calculations prepared by the City of Seattle, Parsons Brinckerhoff, and Lin & Associates, Inc., dated 1997 through 1999.
- Magnolia Bridge Extension – Bridge Seismic Retrofit Strategy Report prepared by CH2M Hill, dated 1993.

- Magnolia Viaduct – Bridge Seismic Retrofit Strategy Report prepared by CH2M Hill, dated 1994.
- Magnolia Bridge Extension – Bridge Seismic Retrofit Project Phase II 100% PS&E Submittal prepared by CH2M Hill, dated 1995.
- Magnolia Viaduct – Bridge Seismic Retrofit Project Phase II 100% PS&E Submittal prepared by CH2M Hill, dated 1995.
- Bridge Seismic Retrofit Program – Draft Magnolia Viaduct Summary Report prepared by Parson Brinckerhoff, dated 1997.
- Magnolia Bridge Slide Repair Geotechnical Report prepared by Shannon & Wilson, Inc., dated 1997.
- Magnolia Bridge Viaduct Post-Earthquake Structural Condition Report prepared by Andersen Bjornstad Kane Jacobs, Inc., dated 2001
- Magnolia Bridge Inspection Report prepared by the City of Seattle, dated 2002.
- Magnolia Bridge List of Work Slips prepared by the City of Seattle, last dated July 2002.
- Magnolia Bridge (Garfield Street Bridge) Chronology of Modification 1929 to 2001 prepared by the City of Seattle, no date.
- Magnolia Bridge Existing Bridge Condition Report prepared by HNTB Corporation, dated 2003.
- Magnolia Bridge Rehabilitation Alternative Existing Bridge Condition Report prepared by HNTB Corporation, dated 2005.

4. FIGURES

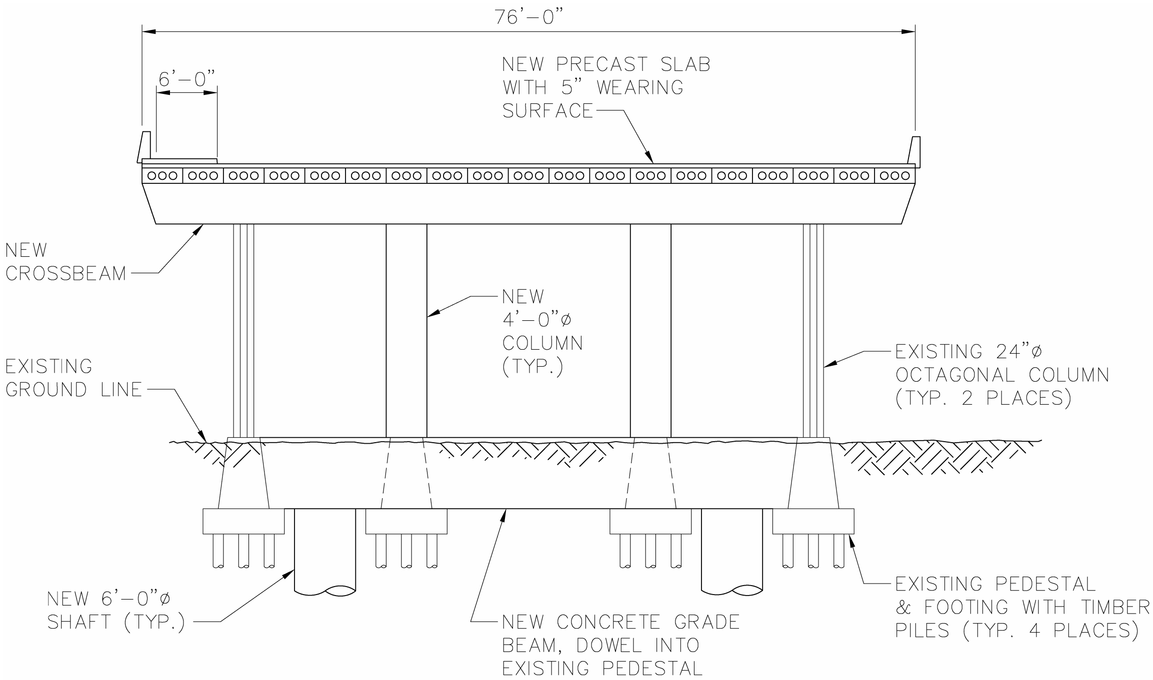

Figure 2 Center Ramp Removed, Bents 20 through 34

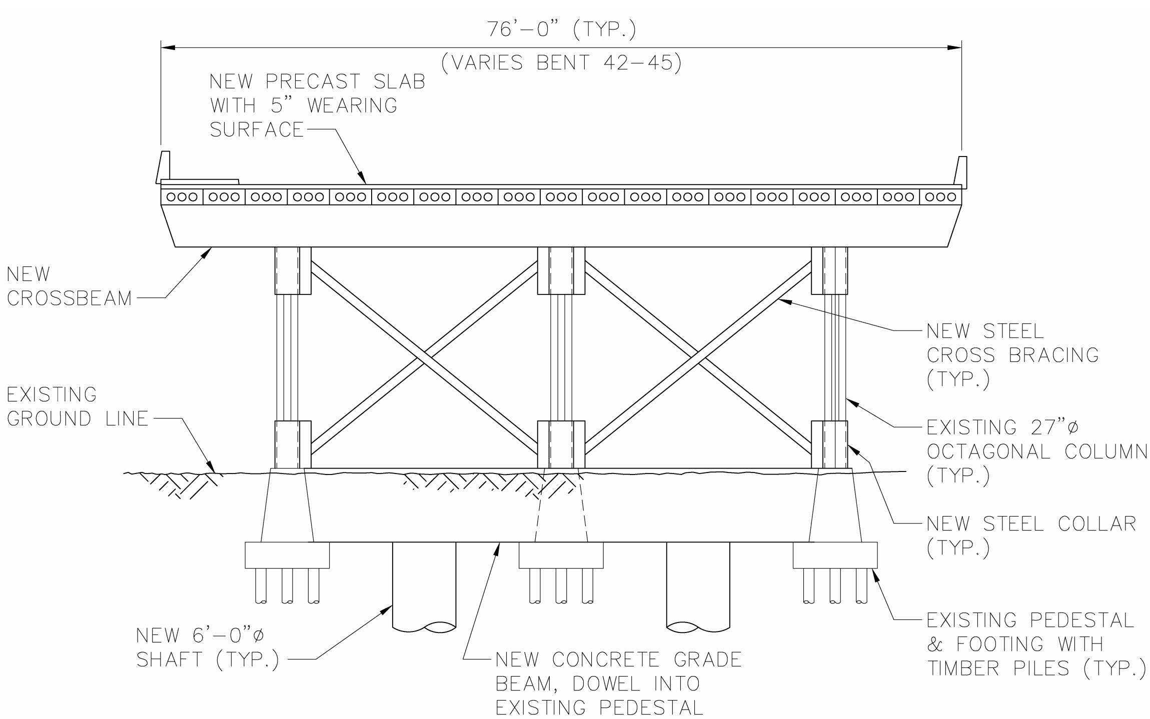

Figure 3 Column Lateral Bracing, Bents 18, 19, and 35 through 46

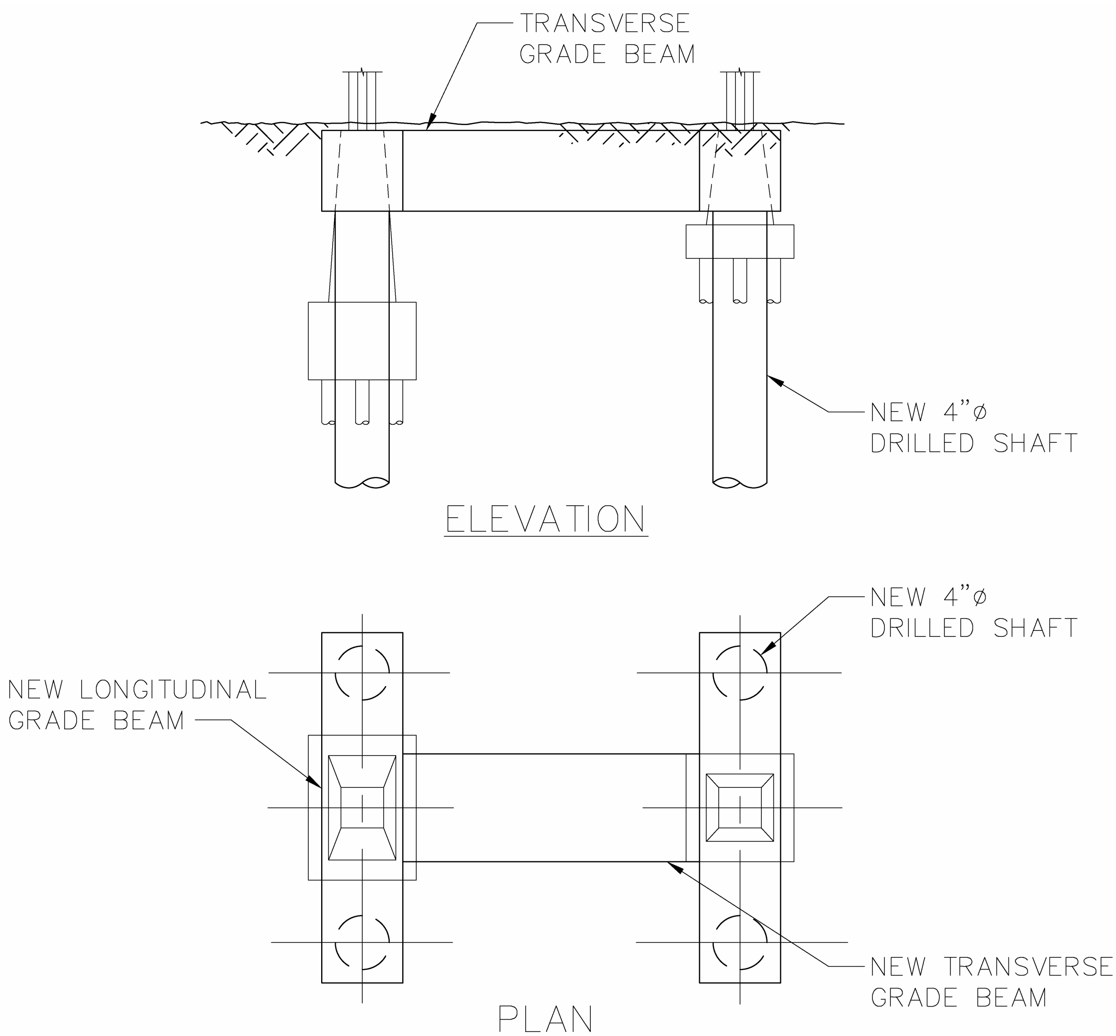

Figure 4 Grade Beam and Drilled Shaft Arrangement, Bents 47 through 58

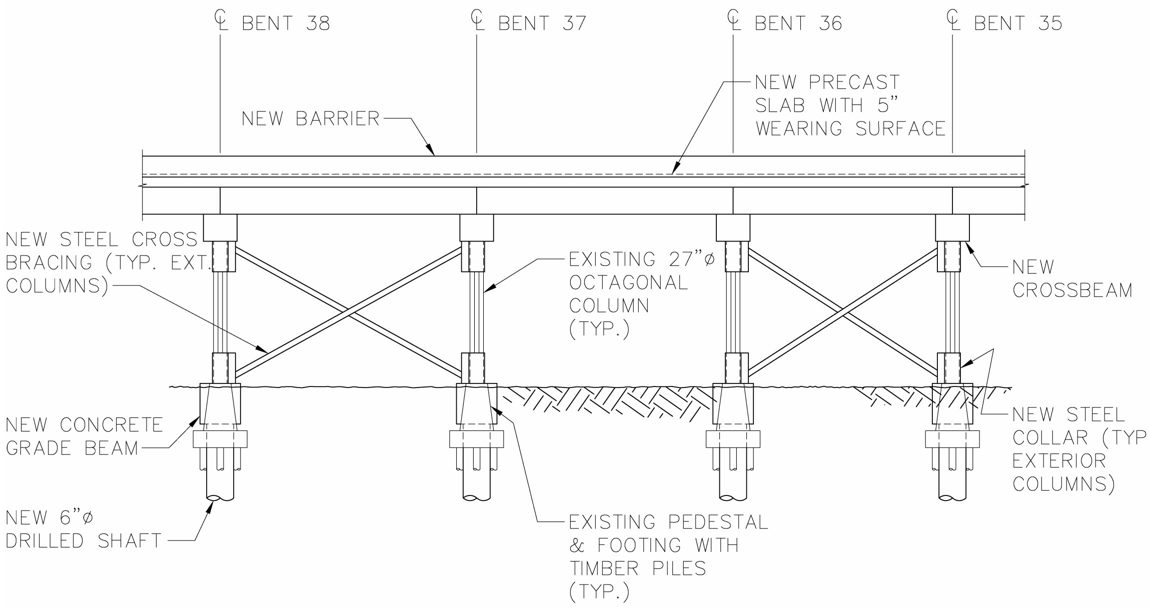

Figure 5 Column Longitudinal Bracing (see text for other locations)

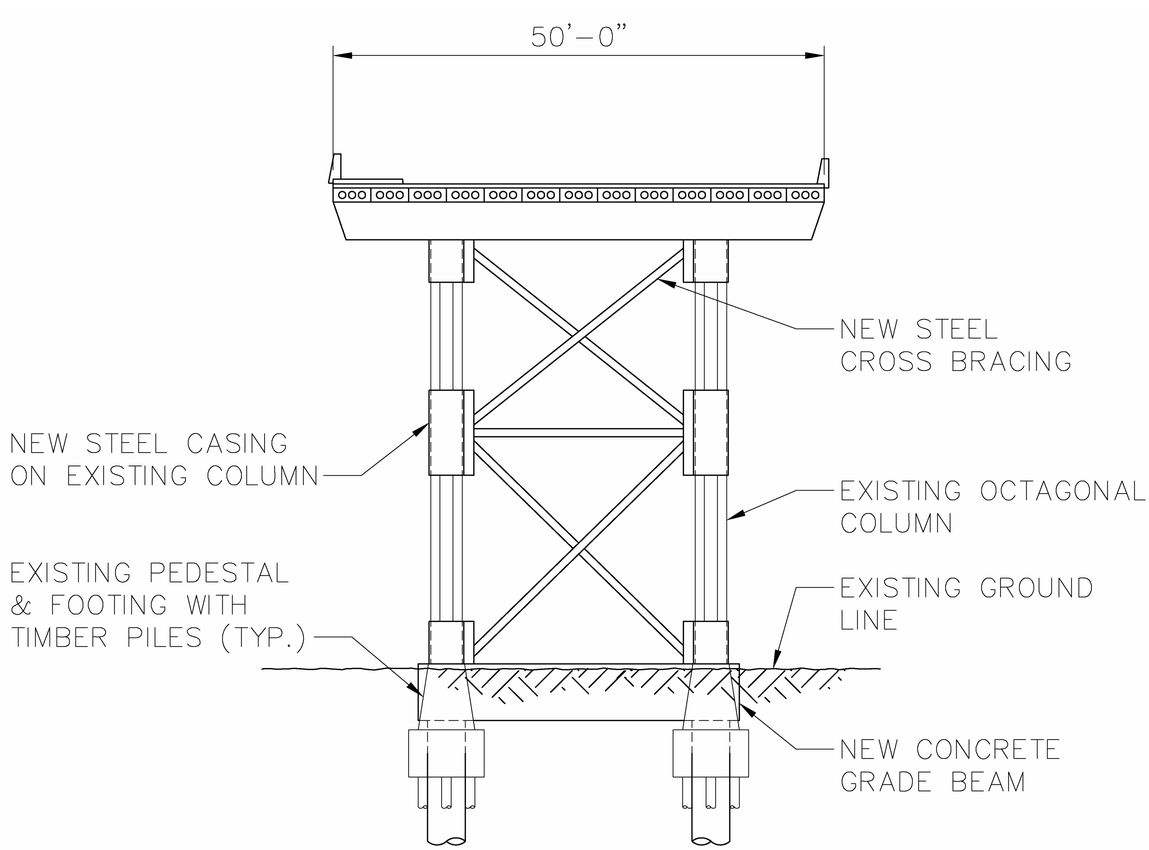

Figure 6 Transverse Cross Bracing, "X" Orientation, Bents 62 through 76