MAGNOLIA BRIDGE REPLACEMENT TS&L STUDY

EXISTING BRIDGE CONDITION REPORT

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HNTB

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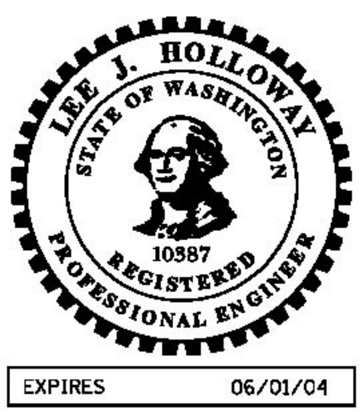

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1. INTRODUCTION

The Magnolia Bridge Replacement TS&L Study scope of work included an assessment of the existing bridge condition, review of the existing bridge seismic retrofit program and to recommend strategies to maintain and extend the service life of the structure. This report summarizes these studies and recommendations.

2. HISTORY OF THE MAGNOLIA BRIDGE

2.1 CONSTRUCTION HISTORY

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 assume that these timber trestles were removed by the Navy when occupied Piers 90 and 91.

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 floorbeams 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 pile bents. The ramps were designed for an AASHTO Seismic Performance Category C with an acceleration coefficient of 0.2G and a Soil Profile Type II. 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. Andersen Bjornstad Kane Jacobs, Inc. (ABKJ) prepared plans addressing this earthquake damage. Repairs completed include the replacement of the concrete transverse bracing of Bents 49 through 75 with steel bracing. Concrete spalls were patched in the longitudinal bracing between Bents 55 and 56, 59 and 60, and 67 and 68. Epoxy injection of concrete cracks was performed in the longitudinal bracing between Bents 50 and 51, 55 and 56, 59 and 60, and 61 and 62. The concrete trusses were also repaired by patching spalls and epoxy injection of the damaged concrete.

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

2.2 RELATED DOCUMENTS

Copies of existing documents related to the Magnolia Bridge were obtained from the City and reviewed in order to complete the studies summarized in 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 Seatttle, 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 prepard 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.

3. CONDITION ASSESSMENT OF EXISTING CONCRETE STRUCTURE

KPFF was retained by HNTB to complete a condition assessment of the concrete portions of the Magnolia Street Bridge in Seattle, Washington. The work was divided into three subtasks.

1. Document Review: This subtask involved reviewing design drawings for the original construction, design drawings for any subsequent structure upgrades and City of Seattle reports addressing inspection and repair. This information was used to plan the field inspections.
2. Visual Condition Survey: This subtask involved performing a visual condition survey of selected portions of the bridge structure. Because the City of Seattle completes regularly scheduled inspections, this subtask did not include a detailed condition survey of the entire bridge. Rather, this inspection was intended to serve as an independent verification of the distress conditions noted by the City in their inspection reports and to provide documentation of any additional distress conditions observed during the visual survey.
3. Condition Assessment Report.

3.1 BRIDGE STRUCTURE

The portion of the Magnolia Bridge that was inspected is approximately 2,842 feet long and consists of three discrete elevated structures. The original structure was constructed primarily of reinforced concrete members, except at the Burlington Northern Santa Fe Railroad crossing which is a steel bridge, and at the east extension, which contains pre-cast concrete bridge girders. The steel structure over the Burlington Northern Railroad was not inspected. The alignment and grade of the original structure creates one long bridge that increases considerably in elevation from east to west and turns toward the southwest at the west end of the bridge. The portion of the original bridge east of the east abutment and the overpass structure over 15th Avenue West were not inspected.

Portions of the bridge were modified to provide additional resistance to traffic and seismic loads. The stiffening of the structure for traffic loads was completed in 1961 and 1974. The modifications to improve the seismic resistance were completed in 1960 for longitudinal seismic forces and in 2001 for transverse seismic forces.

The four distinct structural systems in the 3,005 feet section of the bridge between the east and west abutments are:

- A short section extending from the east abutment at Station 3+81 to the railroad crossing at Station 5+73 (Bent 8). This structure is a reinforced concrete structure and appears to contain precast concrete members. The structural framing for this section does not appear to have been modified, except where some members were removed to allow the overpass over 15th Avenue West to merge along the north edge of the structure.
- The railroad crossing from Station 5+73 (Bent 8) to Station 7+36 (Bent 12). This structure consists of a long span steel trestle with a concrete deck. This portion of the structure was not inspected by KPFF as part of Task 7.0.

- The section from Station 7+36 (Bent 12) to Station 14+61 (Bent 34). This section was strengthened in 1974 by installing steel trusses under the bridge deck to provide additional support to the deck structure. Modifications to the existing expansion joints were also made during this upgrade.
- The section from Station 14+61 (Bent 34) to the west abutment at Station 33+86. This section was strengthened in 1960 by the addition of steel cross-bracing between selected columns. The cross-bracing serves to stiffen the structure in the longitudinal direction. The new cross-bracing was added to the bridge structure at 6 locations between Bents 56 and 78. Additional modifications were completed in 1961 with the installation of steel trusses under the bridge deck to provide additional support to the deck structure. The structure between Bents 61 and 75 was not strengthened because reinforced concrete trusses support the slab structure. The existing railing on both sides of the bridge were removed and replaced and modifications to the existing expansion joints were also made during the 1961 construction. The steel and concrete columns and bracing between Bents 78 and 81 were damaged by a landslide that occurred on January 2, 1997. The slide occurred on the north side at the west end of the bridge. As a result of this landslide, the longitudinal bracing between Bents 76 and 82, and the lower transverse bracing members at Bents 77 through 80 were replaced. Between Bents 76 and 79, two additional column bents supported on drilled shafts were constructed for a total of six new column bents. Transverse bracing of the bents was added in 2001 to repair damage suffered during the February 28, 2001 earthquake. The recent transverse bracing has been added to all bents between Bents 49 to 75.

3.2 EXISTING DOCUMENTATION

The following documents were reviewed by KPFF in preparation for selecting the inspection locations:

- Original design drawings, 1929: Sheets 1, 1A, 1B, 2, 2A, 2B, 3, and 5.
- Longitudinal bracing, 1960: Sheets 1 through 8.
- Bridge rehabilitation, 1961: Sheets 1 through 63.
- Earthquake damage repair, 2001: Sheets S1 through S29.
- List of Open Work Slips, dating from April 1, 1987 through July 10, 2002.
- Bridge Inspection Report, dated June 6, 2002
- List of Closed Work Slips, dating from June 1, 1979 through July 24, 2001.
  (This list does not include descriptions of the completed work.)
- List of 15 major projects on the bridge, including the original construction of the bridge.

3.3 SELECTION OF FIELD INSPECTION LOCATIONS

The first step in selecting locations for visual inspection was to walk the length of the bridge to determine if any serious structural issues were apparent that would require further detailed observation. No serious structural conditions were observed during this initial under bridge walk through.

The next step in the selection process was to review the list of open work slips to determine where there appeared to be a higher level of reported damage to the structure. With these locations identified, nine inspection locations were selected as being representative of the conditions along the bridge. This selection also included several locations that had only one of two items on the list of open work slips. The section of roadway between Bents 62 and 76 is supported with concrete trusses that have not been strengthened during any previous rehabilitation work. Therefore, upon review of our selections, HNTB added a request to inspect 4 of the seven trusses. HNTB selected the sections of roadway with concrete trusses to be inspected.

Based on our initial walk through, review of the work slips and consultation with HNTB; the following twelve locations were selected for the under deck field inspection.

3.3.1 Station 3+81 to 5+73 (192 feet)

- East Abutment to Bent 1
- Bent 5 to Bent 6

3.3.2 Station 7+36 to 14+61 (725 feet)

- Bent 16 to Bent 17
- Bent 23 to Bent 24
- Bent 29 to Bent 30 (Field change to Bent 30 to 31)

3.3.3 Station 14+61 to 33+86 (1925 feet)

- Bent 41 to Bent 42
- Bent 53 to Bent 54
- Bent 64 to Bent 65
- Bent 70 to Bent 71
- Bent 72 to Bent 73
- Bent 74 to Bent 75
- Bent 80 to Bent 81

Inspection of Bent 4 was added by KPFF in the field to include an expansion joint in this section of the bridge. The inspection between Bent 29 to Bent 30 was shifted to Bent 30 to Bent 31 by KPFF, as access was restricted at the intended inspection location.

In addition, the entire length of the bridge was walked to check the upper deck surface for deterioration or damage.

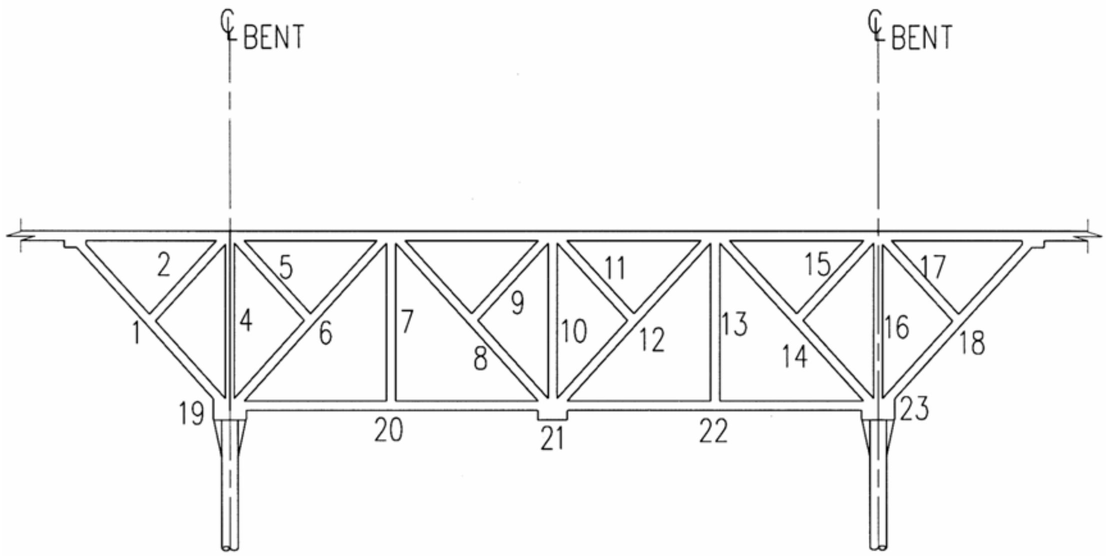
3.4 FIELD OBSERVATIONS

The underside of the bridge from Station 3+81 to 14+61 was inspected from the ground using a man-lift. This portion of the field inspection work was completed on September 17th and 18th, 2002. The underside of the bridge from Station 14+61 to 33+86 was inspected using an under bridge inspection truck (UBIT). As the inspection work with the UBIT was done from the westbound lane (north side), it was difficult to inspect the south side of the bridge from 14+61 to 33+86. This portion of the field inspection work was completed on October 8th and 10th, 2002.

During our field observations, an attempt was made to correlate field observations to the list of outstanding work slips provided by the City of Seattle. In general, the City of Seattle "work slips" and the KPFF inspection results agree closely with regard to the distress types and locations that exist on the bridge. However, the KPFF inspection results indicated that considerably more distress exists than is currently identified by the work slips.

The concrete truss members referenced in this report are located between Bents 62 and 75 and are identified by the numbers 1 through 23 (See the truss elevation sketch below).

The photographs on the pages that follow illustrate some of the distress and deterioration that exist on the bridge. Photograph references are made to identify specific photographs in the inspection database.



ELEVATION OF NORTH & SOUTH TRUSS

TRUSSES LOCATED BETWEEN BENTS 62 TO 75
LOOKING SOUTH

Figure 1 - East Abutment: Station 3+81 (Reference Photograph 6)

The east abutment has extensive cracks on the face of the abutment wall. Sounding the wall with a hammer indicates that the concrete surface is solid, suggesting the cracks are not the result of surface delaminations.

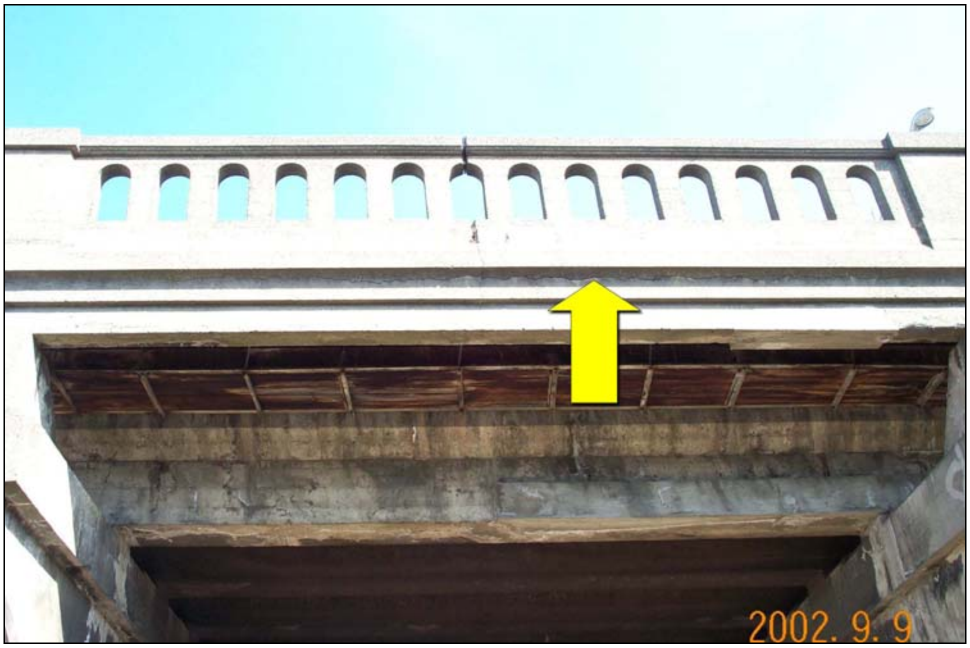


Figure 2 - Bents 2 to 3: Station 4+29 to 4+53 (Reference Photograph 8)

The above photo shows a horizontal crack in the slab below the guardrail.

Figure 3 - Bent 4: Station 4+77 (Reference Photograph 9)

A close up of the same crack as shown above.

Figure 4 - Bent 4: Station 4+77 (Reference Photograph 11)

Cracks in the vertical end of the abutment beam. White deposits (efflorescence) indicate water migrating through the cracks.

Figure 5 - Bent 4: Station 4+77 (Reference Photograph 17)

Cracking in the top of the arch between columns. This type of cracking appears to be relatively isolated in this section of the bridge.

Figure 6 - Bent 5: Station 5+01 (Reference Photograph 22)

Vertical crack running the full height of the column.

Figure 7 - Bents 16 to 17: Station 8+70.45 to 9+03.15 (Reference Photograph 27)

Cracked grout pad above the transverse steel beams. The crack does not extend all the way through. The black pad between the grout and the deck is deteriorated.

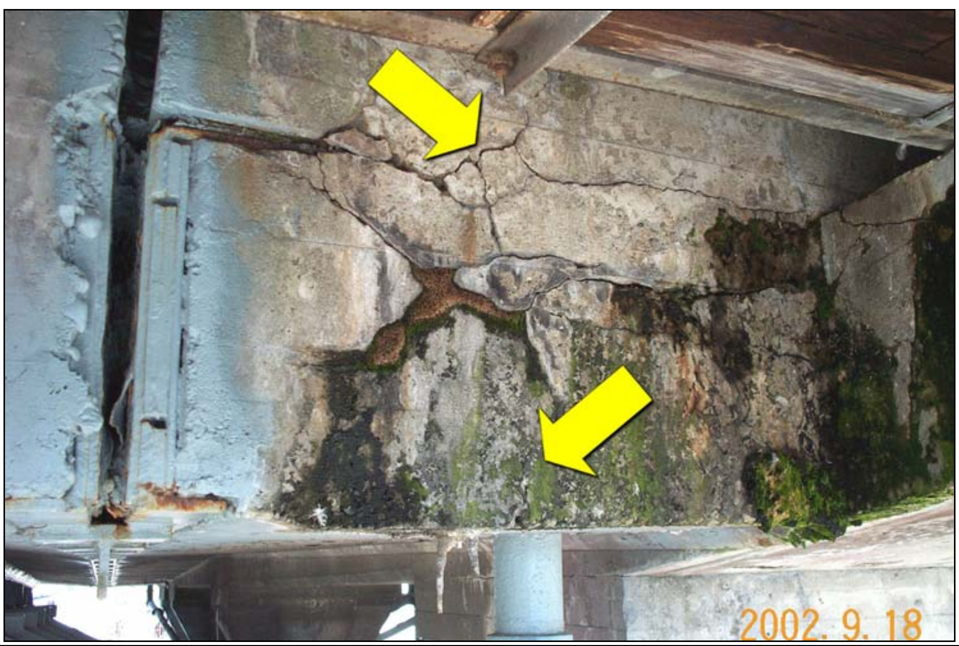


Figure 8 - Bent 17: Station 9+03.15 (Reference Photograph 33)

The edge beam is heavily cracked at the expansion joint west of Bent 17.

Figure 9 - Bent 24: Station 11+32 (Reference Photograph 41)

Cracks in the column and the edge of the beam on the south side of the center ramp at Bent 24.

Figure 10 - Bent 23: Station 11+00 (Reference Photograph 44)

The underside of the slab with longitudinal crack.

Figure 11 - Bent 24: Station 11+32 (Reference Photograph 46)

South side of edge beam. Bottom portion of the beam has delaminated from the upper portion. Water is leaking through the cracks.

Figure 12 - Bents 30 to 31: Station 13+32 to 13+68 (Reference Photograph 51)

South side of the bridge showing previous repairs (epoxy injection).

Figure 13 - Bents 30 to 31: Station 13+32 to 13+68 (Reference Photograph 52)

The grout pad between the steel beam and the concrete slab at the north end of the steel support beam has separated from the concrete slab.

Figure 14 - Bents 41 to 42: Station 17+07 to 17+39.50 (Reference Photograph 61)

Cracking in the south side of the deck edge beam.

Figure 15 - Bents 41 to 42 Station 17+07 to 17+39.50 (Reference Photograph 66)

Spalled deck soffit with exposed reinforcing.

Figure 16 - Bents 41 to 42 Station 17+07 to 17+39.50 (Reference Photograph 67)

Exposed deck slab reinforcing in the same crack further east of Photograph 66. Bars are coated with primer.

Figure 17 - Bent 53: Station 20+95 (Reference Photograph 73)

Cracks in the column capital on the south side of the bent.

Figure 18 - Bents 64 to 65: Station 25+58 to 26+18 (Reference Photograph 76)

Cracks in a transverse beam along the north side of the bridge between Bents 64 and 65. This cracking is typical at the transverse beams along the north side of the bridge.

Figure 19 - Bents 64 to 65: Station 25+58 to 26+18 (Reference Photograph 80)

Exposed reinforcing steel on the topside of south truss Member 11.
This condition is typical on many of the sloped truss members.

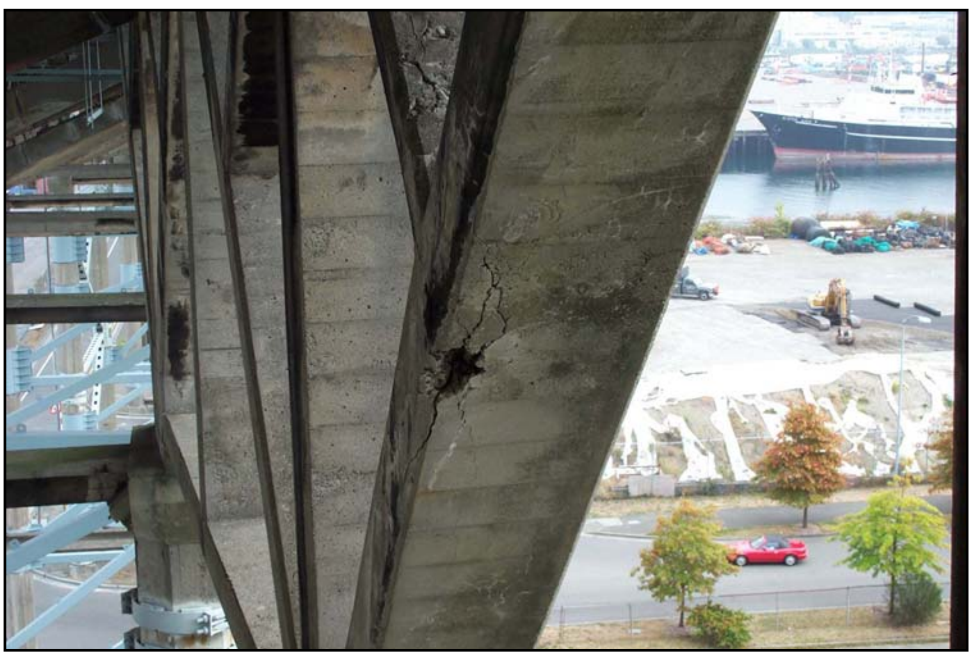


Figure 20 - Bents 70 to 71: Station 28+67 to 29+27 (Reference Photograph 88)

Shows cracking and spalling on Member 12 of the south truss. It appears that the reinforcing is heavily corroded at this location. Additional spalling should be expected to occur.

Figure 21 - Bents 70 to 71: Station 28+67 to 29+27 (Reference Photograph 85)

The corner of the slab is cracking along the south side of the bridge between Bents 70 to 71.

Figure 22 - Bent 72: Station 29+70 (Reference Photograph 90)

Exposed reinforcing bars at a column where the diagonal tees were removed for installation of the lateral seismic upgrades.

Figure 23 - Bents 72 to 73: Station 29+70 to 30+30 (Reference Photograph 93)

Delaminations and spalls on the soffit of Member 20 of the south truss.

Figure 24 - Bents 72 to 73: Station 29+70 to 30+30 (Reference Photograph 94)

Spalling and exposed reinforcing on top of Member 9 in the south truss.

Figure 25 - Bent 73: Station 30+30 (Reference Photograph 91)

Severely cracked grout pad above the steel framing at the expansion joint west of Bent 73. View is looking west.

Figure 26 - Bents 74 to 75: Station 30+73 to 31+33 (Reference Photograph 97)

Crack pattern in the bottom of the slab between Bents 74 and 75. This pattern is typical in this area.

Figure 27 - Bents 74 to 75: Station 30+73 to 31+33 (Reference Photograph 99)

Spalling and exposed reinforcing at Member 21 on the north truss.

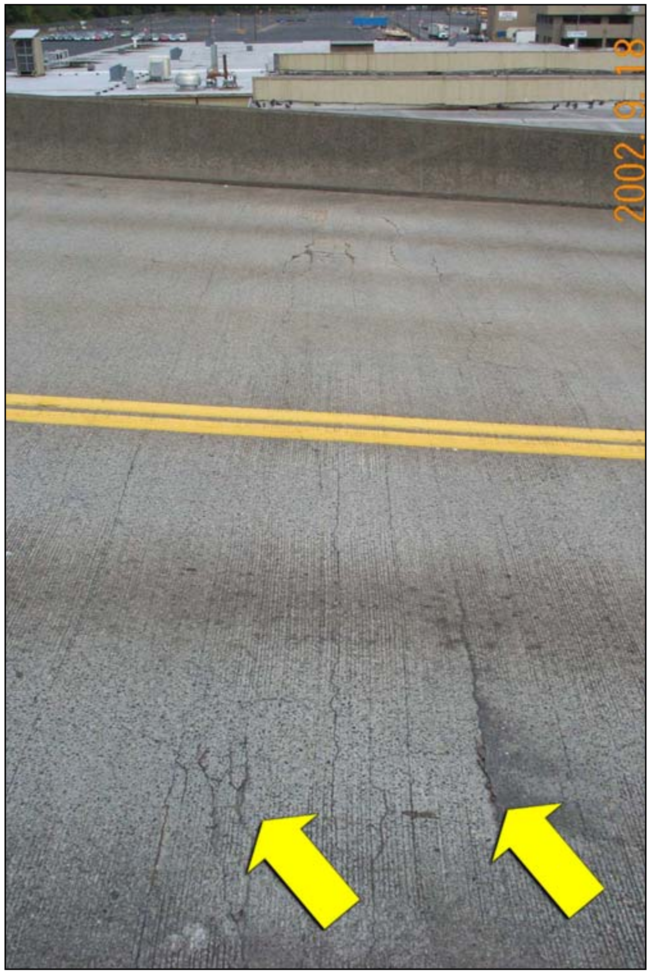


Figure 28 - Bent 48 Station 19+31 (Reference Photograph 108)

Cracks in the top of the deck near Bent 48.

Figure 29 - Bent 48 Station 19+31 (Reference Photograph 109)

Cracks in the deck soffit near Bent 48 below the section of slab shown in Photograph 108 above.

3.5 RECOMMENDATIONS

The KPFF inspection was limited to only 13 areas along the length of the bridge (see Sections 3 and 3.1 for further detail). Therefore, our recommendations are not based on a comprehensive assessment of the entire structure.

During our field inspection, we noticed numerous areas where the existing concrete cover is about to spall. If the bridge is not replaced or retrofitted soon, the loose and delaminated concrete should be removed and the resulting cavities filled with polymer concrete patching mortar during regularly scheduled maintenance performed by the City.

The steel stiffening trusses that were installed in 1961 and 1974 are grouted between the top of the transverse girder and the bottom of the slab. For these trusses to function properly, the grout needs to be in good condition and in solid contact with the bottom of the slab. In several truss support locations the grout is cracked or not in contact with the slab. See Photographs 27 and 52; Figures 7 and 13, respectively. It is recommended that the structure be evaluated to determine the significance on load capacity of the deck slab, if one or more of these transverse girders do not provide adequate support to the slab. If it is determined that the existing conditions do not provide adequate support, the grout above all the transverse steel girders should be inspected and repaired as necessary.

Several locations inspected by the consulting team identified typical defects that commonly occur on reinforced concrete bridge structures. The list provided below is not intended to be a detailed summary of all of the defects that exist. Instead, they are examples of the types of defects observed during the inspection.

It is recommended that these defects be evaluated in detail during the next scheduled inspection by the City.

- The rail posts at Bent 24 have a hollow ring when sounded with a hammer. See work slip 6400. The railing may not be in satisfactory condition to withstand a vehicular collision.
- The horizontal crack on the south side of the bridge between Bents 2 and 5 should be checked to determine whether it adversely affects the capacity of the bridge guardrail. (See Photographs 8 and 9; Figures 2 and 3, respectively.)
- The extent of damage to the slab on the west side of the expansion joint at Bent 17 needs to be checked further (See Photograph 33; Figure 8). Corrosion may adversely affect the structural capacity of the walkway and traffic railing.
- The extent of damage to the slab on the south side of Bent 24 needs to be checked further (See Photograph 46; Figure 11). Corrosion may adversely affect the structural capacity of the walkway and traffic railing.
- The extent of damage to the slab on the south side between Bents 41 and 42 needs to be checked further by chipping into the existing concrete to determine the extent of damage to the reinforcing (See Photograph 61; Figure 14). Corrosion may adversely affect the structural capacity of the walkway and traffic railing.
- The north ends of the transverse beam along the north side of the bridge between Bents 46 to 82 need to be checked to determine whether the condition adversely affects the bridge guardrail along the north side of the bridge (See Photograph 76; Figure 18).
- The spalled concrete on Member 12 of the south truss between Bents 70 and 71 needs to be removed and the extent of reinforcing corrosion assessed (See Photograph 88; Figure 20). Corrosion may adversely affect the structural capacity of the concrete truss.

- The extent of cracking in the slab near Bent 48 needs to be assessed. (See Photographs 108 and 109; Figures 28 and 29, respectively). The cracking illustrated in these photographs may suggest that the slab may be delaminated and may soon spall which could adversely affect the slab's structural capacity.

3.6 CONCLUSION

Based on our field observation of the 72-year old Magnolia Street Bridge, it is our opinion that the concrete structure is showing signs of aging. The concrete cover is cracking and spalling at many locations along the length of the bridge. The observed distress of the concrete appears to be primarily related to corrosion of the underlying reinforcing steel. Based only on a visual inspection, there does not appear to be any indication that the structure has a serious load capacity problem. However, since the bridge is 72 years old and has deteriorated, major rehabilitation or replacement should be planned. Numerous local repairs have been made over the years.

4. BRIDGE SEISMIC RETROFIT PROGRAM

HNTB reviewed the conclusions of the 90% PS&E Submittal Plans for the "Bridge Seismic Retrofit Project – Phase II" prepared by CH2M Hill dated February 24, 1995 and the Draft "Magnolia Viaduct Bridge Seismic Retrofit Program Report" prepared by Parsons Brinckerhoff dated June 16, 1997, both provided by the City of Seattle. The recommendations of these retrofit studies were reviewed based upon the findings of the field condition assessment of the Magnolia Bridge by KPFF performed in the fall of 2002 and the review of repairs and retrofits completed since the release of the retrofit program report.

4.1 SUMMARY OF THE RECOMMENDATIONS OF THE "BRIDGE SEISMIC RETROFIT PROJECT – PHASE II"

In 1995, CH2M HILL, in association with the Tudor Engineering Company, developed a recommended retrofit strategy for the Magnolia Bridge for the City of Seattle Engineering Department. The goal of this study was to retrofit all spans of the existing bridge for Level A (Superstructure Retrofit) to prevent unseating. To accomplish this, the installation of cable restraints was recommended across the superstructure expansion joints to prevent the spans from falling from their supports during a significant earthquake. Bridge seat extension brackets were also recommended at some of the expansion joints located at supports to increase the bearing support length needed to prevent unseating of the superstructure spans during the design seismic event.

The retrofit techniques used for this study were designed for the maximum allowable seismic response coefficient. Using the maximum design coefficient for the superstructure retrofit would allow the substructure to be retrofitted in the future without requiring additional strengthening of this superstructure retrofit. These retrofits were designed for an AASHTO Seismic Performance Category D (based on an acceleration coefficient of 0.3G and an Importance Classification of I, indicating the bridge to be an essential structure).

During the development of the "Bridge Seismic Retrofit Project – Phase II", CH2M Hill noted a concern with the connection between the reinforced concrete truss spans and their supporting bents. This connection consists of an embedded wide flange section wrapped with reinforcing bars and steel rods connecting the truss to the bent. CH2M Hill estimated this connection is likely to be adequate longitudinally, but would possibly blow out the side of the truss during the design earthquake. As this was determined to be a substructure concern and thus out of scope of the "Bridge Seismic Retrofit Project – Phase II" design, this connection was not studied by CH2M Hill in further detail.

4.2 SUMMARY OF THE "MAGNOLIA VIADUCT BRIDGE SEISMIC RETROFIT PROGRAM REPORT"

In 1997 the City of Seattle requested that Parsons Brinkerhoff prepare a report for the Magnolia Bridge as part of the Bridge Seismic Retrofit Program. This report discussed the seismic vulnerability of the bridge and summarized retrofit and replacement options for the viaduct. Options considered included a partial seismic retrofit of the existing structure, a full seismic retrofit of the existing structure, full replacement of the viaduct, and a "Do Nothing" option.

Four areas of seismic vulnerability were identified in the retrofit report:

- Superstructure – The bearing support length of the simply supported and suspended spans is insufficient. These spans would likely fall off their supports during the design earthquake.
- Substructure – The reinforced concrete columns and bracing making up the entire substructure are neither strong enough nor ductile enough to accommodate the forces and movements generated by the design earthquake.
- Foundations – The lateral load and uplift capacity of the unreinforced concrete plinths and the timber piles making up the structure foundation are insufficient for the forces generated by the design earthquake.
- Soil Liquefaction –There is a high potential for liquefaction of the clays and sands located along a majority of the viaduct during the design earthquake. The piling supporting the viaduct does not appear to extend beyond these liquefiable soils, leading to a loss of load carrying capacity during this seismic event.

The options considered to mitigate a portion or all of the seismic vulnerability concerns listed above included the following (Note that all estimates were in 1998 dollars and need to be revised and updated for a valid current comparison:

- Partial Seismic Retrofit (Level A – Superstructure only) – This option consists of installing cable restrainers below the deck and across the joints of the superstructure as needed to prevent it's falling off the supports. The cost for this Level A Retrofit was estimated by Parsons Brinkerhoff to be $976,000 including design, construction and construction management in 1998 dollars.

  Concerns not addressed by this option included failure of the substructure and the liquefaction of the soil supporting the structure during a moderate earthquake. This could lead to a partial or full failure of the bridge during a seismic event.

- Full Seismic Retrofit (Level B – Superstructure and substructure serviceable for emergency vehicle use) – This option includes the superstructure retrofit discussed above as well as retrofitting the substructure and addressing the soil liquefaction concerns. The cost for this Level B Retrofit was estimated by Parsons Brinkerhoff to be $28,800,000 including design, construction and construction management in 1998 dollars.

  Concerns with the strength and ductility of the substructure would be addressed by replacing the concrete bracing with steel bracing, adding shear walls in both the longitudinal and transverse directions, adding additional steel longitudinal bracing, and adding horizontal struts.

The foundation would be retrofitted by the installation of drilled shafts to protect against soil liquefaction and to increase both the uplift and downward force capacity necessary for support during the design earthquake. Shelling of the unreinforced concrete plinth is also necessary.

Various methods of retrofitting needed to prevent soil liquefaction and lateral spreading were considered, including displacement grouting, permeation grouting, deep soil mixing, and jet grouting. For this site, deep soil mixing (mixing of cementitous materials into the ground) and jet grouting (high pressure injection of cement slurry to form wall panels or cylindrical shapes in the ground) were recommended as being the most useful and cost effective.

Even with the full seismic retrofit discussed above, Parsons Brinkerhoff was still concerned that the bridge would not be operational to emergency vehicles in the event of the design earthquake.

- Full Replacement of the Viaduct – This option would provide full replacement of the existing structure to current design standards, including seismic requirements. The soil for the new foundations would still require strengthening (deep soil mixing, jet grouting, etc.) to prevent soil liquefaction and lateral spreading. This option was estimated to cost $59,000,000 including design, construction and construction management in 1998 dollars.

4.3 SUMMARY OF GEOTECHNICAL STUDIES OF THE MAGNOLIA VIADUCT BRIDGE SITE

Two recent geotechnical studies were released for the subject site by Shannon and Wilson, Inc. The first of these was released on October 10, 1997 to support the Magnolia Bridge Slide Repair project. The second report, released on September 17, 2002, was the “Existing Subsurface Information and Conceptual Foundation Recommendations, Geotechnical Type, Size, and Location Study for the Magnolia Bridge Replacement”, which was completed to provide necessary information for the possible replacement of the existing bridge.

In the Magnolia Bridge Slide Repair geotechnical report, the cause of the landslide, which occurred immediately north of the west end of the bridge, was believed to have resulted from a combination of geologic conditions, steep topography, heavy runoff from rainfall and snowmelt, rising groundwater level and seepage pressure, and subsequent saturation and loss of strength of the slope soils on the hillside. This hillside is overlain by a six to seven foot thick layer of topsoil, fill, and outwash consisting of loose to medium dense, silty sand underlain by glacial till consisting of very dense, silty, gravelly sand. A soldier pile retaining wall was constructed in the area of the slope failure as part of the 1997 repair work for this slide.

The geotechnical report for the bridge replacement further studied the site. The design criteria used to estimate the potential liquefaction depth was a 475-year, Magnitude 7.5 earthquake with a peak ground acceleration of 0.3g. The subsurface investigation found that the bridge is located in a former marine mudflat (consisting of very loose to loose silt with some thin layers of soft clay) which was filled in the early 1900s to the present existing grade. The fill encountered consists of very loose to medium dense sand and soft clay mixed in with colluvial soil that likely resulted from numerous landslides that occurred on the west slope of Queen Anne Hill. Potential liquefaction was found to be a possibility from about thirteen to sixty feet deep in the vicinity of the bridge under the design earthquake.

4.4 VALIDITY OF THE EXISTING RETROFIT PROGRAM REPORT CONCLUSIONS

Recommendations for restraints across the superstructure's expansion joints to prevent the spans from falling from their supports during a significant earthquake are still valid.

Concerns with the strength and ductility of the concrete substructure bracing proved to be well-founded. Inspection of the structure following the Nisqually Earthquake indicated significant damage to much of this bracing. The construction done per the plans prepared by ABKJ for repairing damage from this earthquake replaced much of the concrete bracing with steel bracing. However, reinforced concrete longitudinal and transverse bracing remains at some locations. These remaining concrete braces would likely be damaged during a significant seismic event.

Additional substructure retrofits recommended in the "Magnolia Viaduct Bridge Seismic Retrofit Program Report" that have not been done include the construction of reinforced concrete longitudinal shear walls between Bents 7 and 8 and the construction of both reinforced concrete longitudinal and transverse shear walls between Bents 12 and 13, 13 and 14, 14 and 15, 15 and 16, 16 and 17, 17 and 18, and 18 and 19. These transverse shear walls include the construction of a footing supported by auger-cast piles outside each of the existing columns at these bents.

The "Magnolia Viaduct Bridge Seismic Retrofit Program Report" also recommends retrofitting the foundation by installing drilled shafts to protect against soil liquefaction and to increase the uplift and downward force capacity necessary to resist the design earthquake. Shelling the unreinforced concrete plinth is also needed to adequately strengthen the existing foundation. Also recommended is retrofitting the foundation to protect against soil liquefaction and lateral spreading through the use of deep soil mixing and/or jet grouting.

The Load and Resistance Factor Design (LRFD) specifications for design of highway bridges was introduced by AASHTO in 1994. Use of this specification to analyze and design a seismic retrofit for the Magnolia Bridge may result in some differences in retro member sizes but the vulnerabilities described in these reports exist and the conclusion reach is valid.

5. STRATEGIES TO MAINTAIN AND EXTEND THE SERVICE LIFE OF THE STRUCTURE

5.1 SHORT TERM STRATEGIES

Annual condition inspections should be continued to ensure the safe continued usage of the structure. Particular attention should be made to those areas and members that have been identified as having a low rating factor (below 1.00) from the latest bridge load rating analysis. We also recommend additional inspections be performed immediately following any seismic event.

Repairs recommended as a result of these inspections should be addressed as soon as maintenance monies will allow, especially for deficiencies in members with low load rating factors. For example, full grout contact between the steel slab stiffening trusses and the concrete slab is necessary to avoid over-loading the slab and causing a premature failure of the slab. All of these repairs will help maintain the current condition of the bridge.

To help ensure the safe continued use of the concrete truss spans of the structure, monitoring instrumentation has been proposed to track significant structural movement of these truss spans. The recommended approach for monitoring is to measure the rotations and relative movements of all the trusses by installing crack gages at the ends of each truss and to monitor for structural degradation of the members by placing crack gages along a portion of the high tension members.

Placing crack gages over the expansion joints at the end of each truss span will provide a measurement of rotation at the ends of the span as well as the displacement of the truss span in relation to adjacent spans. To monitor for structural degradation, crack gages will be placed along the inside face of the bottom chord. Typical operating movements of these expansion joints will be determined during the first few months of monitoring. Upon determination of typical movement limits, a threshold level will be set for the crack gages discussed above. If the designated threshold level of any of these gages is exceeded, an alarm will be triggered which will notify the City of Seattle. A total of thirty crack gages will be required (sixteen at the expansion joints and fourteen along the bottom chords).

5.2 LONG TERM STRATEGIES

If the existing bridge is expected to be in service for over 10 years, additional maintenance and repairs should be considered. As a first step, an in-depth inspection including sampling and testing the concrete for strength, chloride ion penetration depth and delamination of the concrete should be considered as part of a long-term maintenance program. Based on the results of this inspection, repair needs can be determined. Possible repairs include injection of cracks with epoxy grout, repair of spalled concrete, and replacement of deficient concrete and grout. These repairs could help maintain the integrity of the existing structure.

The value of preservation measures to slow corrosion of the reinforcement (such as a cathodic protection system) could also be determined based on the results of the in-depth inspection. The Oregon Department of Transportation has had success using cathodic protection on some concrete bridges along the coast. These bridges were coated with zinc to act as an anode when connected to the reinforcement.

Strengthening those structural elements lacking the moment and/or shear capacity to carry an HS-20 live load could also be evaluated. Deficient members could be identified through a 3-dimensional model that includes all of the substructure and superstructure elements.

The completion of the full Level B seismic retrofit recommended by Parsons Brinkerhoff as discussed in Section 4 (restraining the spans over the expansion joints, replacing the remaining concrete longitudinal and transverse bracing, adding longitudinal and transverse bracing at the locations discussed above, retrofitting the foundation material to resist soil liquefaction and lateral spreading, and strengthening the foundation to resist earthquake forces) should be considered to help ensure the structure retains its structural integrity following a significant seismic event. Strengthening the connection of the reinforced concrete truss spans to their supporting bents should also be considered as part of a seismic retrofit program.

Estimating the costs to identify, evaluate, and design repair, strengthening and preservation measures is beyond the scope of this study. The seismic retrofit was estimated to cost $28,800,000 in 1998 costs. Therefore, we anticipate that the costs for repair, strengthening, preservation and seismic retrofit would approach the cost of replacing the existing bridge. We also anticipate the service life of the rehabilitated bridge would be less than a new bridge and the continuing maintenance costs on the existing will be higher than for a new bridge. In our opinion, the prudent course of action is to pursue replacement of the existing bridge.

APPENDIX A

EXISTING BRIDGE PLAN AND ELEVATION

(Copied from *Bridge Seismic Retrofit Project – Phase II, Magnolia Viaduct 90 Percent PS&E Submittal Plans* prepared by CH2M HIL in association with Tudor Engineering Company dated February 24, 1995)