RETROFIT AND REPLACEMENT OF DUMBARTON RAILROAD BRIDGES

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Abstract

The Dumbarton Rail Corridor (DRC) is located in the southern region of the San Francisco Bay Area, connecting east and west bay. The Corridor is approximately 20.5 miles in length and includes a 4.5 mile segment that crosses the southern portion of the San Francisco Bay. The crossing of the Bay includes two bridges: the Dumbarton Bridge and Newark Slough Bridge, approximately 7550 and 422 feet long, respectively. Portions of these Bridges have timber and steel truss bridge sections that are approximately 100 years old, and each has a 100-year old swing span. The existing movable bridges have not been operational since the mid-1980’s and portions of the existing timber bridges were damaged in a fire in the late 1990’s.

The DRC Project proposes to provide commuter rail service by improving the existing rail infrastructure in the DRC Corridor. In order to reopen the Dumbarton and Newark Slough Bridges to accommodate the planned Dumbarton commuter rail service, several activities were performed in order to help determine if the bridges should be retrofitted or replaced. They include: Inspections (Underwater, Above Water), Concrete Coring of 100-year old large diameter caissons, Condition assessment of existing steel and concrete structures, Service Load Rating Analyses, Movable Bridge evaluation, Seismic Analyses, Constructability Review and Cost Estimates.

1. INTRODUCTION

The Dumbarton Rail Corridor (DRC) is located in the southern region of the San Francisco Bay Area, connecting east and west bay. The Corridor is approximately 20.5 miles in length and includes a 4.5 mile segment that crosses the southern portion of the San Francisco Bay. The crossing of the Bay includes two bridges: the Dumbarton Bridge and Newark Slough Bridge, approximately 7550 and 422 feet long, respectively. Portions of these Bridges have timber and steel truss bridge sections that are approximately 100 years old, and each has a 100-year old swing span. The existing movable bridges have not been operational since the mid-1980’s and portions of the existing timber bridges were damaged in a fire in the 1990’s.
The existing Dumbarton and Newark swing steel truss bridges and piers were constructed in 1910. The original construction included: ballasted deck timber trestle at the west approach; steel deck girders and steel through truss with a swing span at the channel crossing; and ballasted deck timber trestle at the east approach. While the entire structure has a single railroad track, the steel spans were built for two parallel tracks. Sections of the approach spans have been modified and rebuilt in the 1960s and 1970s with precast concrete construction.

The DRC Project proposes to provide commuter rail service by improving the existing rail infrastructure in the DRC Corridor. In order to reopen the Dumbarton and Newark Slough Bridges to accommodate the planned Dumbarton commuter rail service, the San Mateo County Transit District (SAMTRANS) engaged HNTB to perform preliminary engineering in support of the environmental document to help determine if the bridges should be retrofitted or replaced.

2. BACKGROUND

The engineering team evaluated the original concrete piers and steel trusses to determine the feasibility of retrofit/rehabilitation of the structures for the proposed Dumbarton Rail service. See Figures 1, 2 and 3 for location and configuration of the existing bridges.
Figure 1. Location Map
Figure 2. Dumbarton Main Span - Swing Span and Steel Trusses (Section 4 to 9)

Figure 3. Newark Slough Swing Span
2.1 Criteria and Constraints

Recent updates to seismic and AREMA railroad criteria pose new design and analyses challenges on the existing bridges. In addition, the structural integrity of the 100 year old caissons is largely unknown to validate the conceptual studies performed thus far. Finally, construction in the San Francisco Bay poses several constraints. These constraints include water quality, biological, protection of endangered species, mammals and considerations of marine, bird and other wildlife breeding and nesting seasons.

2.2 Work Plan

The work plan (Figure 4) below presents the relationships between different activities and how each activity relates to another in order to meet the project objective of identifying a preferred alternative of retrofitting or replacing the Dumbarton Bridges.

3. INSPECTIONS

From November 2007 to January 2008, HNTB engineers and inspectors and Halcrow underwater divers-inspectors conducted inspections of the existing bridges and its supporting piers. CTL Group concrete specialists obtained concrete coring samples of the existing concrete caissons.
3.1 Caisson Inspections

Over several days in November and December 2007, a three-person experienced engineer-diver team conducted detailed above-water and underwater inspections of 22 concrete caissons supporting the spans at the Dumbarton Rail Bridge and 5 concrete caissons of the bridge at the Newark Slough. These existing caissons were built as part of the original steel trusses construction in 1910. All caissons exhibit a zone of corrosion in the steel casing, typically ranging from 1-ft above the Mean Low Water (MLW) elevation to 10-ft below the MLW elevation. Approximately 60% of the steel in that zone has moderate to severe corrosion with areas of exposed concrete. The condition of the exposed concrete varies widely, from minor surface deterioration, to large voids with soft concrete and exposed timber piles (see Figures 5, 6 and 7).

Overall structural integrity of the original caissons is affected by large areas of spalling with deep voids at the caissons, particularly the voids with exposed timber piles. Since a majority of the steel shells at the caissons are still intact, it is impossible to determine the overall extent of the deterioration of the concrete.

Figure 5. Dumbarton Pier 6, Northwest Caisson, SE Quadrant, 9.5-ft below MLW Typical Steel in the 1-ft above to 10-ft below MLW, Typical Condition of Steel Shell.
Figure 6. Dumbarton Pier 6, Northwest Caisson, SW Quadrant, 6.5-ft below MLW with Large Void (28’ W X 2.3’ H X 5’ D) in Caisson

Figure 7. Dumbarton Pier 6, Northwest Caisson, SE Quadrant, Large Spall with Exposed Timber Pile
3.2 Steel Trusses and Swing Bridges Inspections

The Dumbarton and Newark Swing Bridges are open deck swing spans which opened to traffic in 1910. The span lengths for Dumbarton and Newark swing spans are 310-ft and 182-ft, respectively. Six 180-ft span steel trusses approach to the Dumbarton swing span from east and west. The structures have not seen any rail traffic since the mid 1980s.

Light to moderate blanket rust is present on almost all truss members as the protective paint system has failed. Pack rust is found under most connection plates, top surfaces of top and bottom chords and end posts. The floor system (floorbeams and stringers) and cross girders have extensive corrosion and severe section loss in several locations. The severity of the section loss and pack rust found in the floor system and connections warrants their replacement. All of the secondary members (laterals) and their connection gusset plates have substantial section loss and severe corrosion which warrants replacement of these members in their entirety. (See Figures 8, 9, 10 and 11 for bridge conditions).
Figure 8. Dumbarton Bridge - Typical Failed Paint on Truss Member

Figure 9. Dumbarton Bridge - Typical Corrosion and Pack Rust of Steel Truss Member
Figure 10. Dumbarton Swing Span – View of Floor Beam

Figure 11. Newark Swing Span - View of Floor Beam and Pier Top

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3.3 Concrete Coring

In January 2008, structural concrete specialists took concrete core samples from the concrete caissons that support the original steel trusses. A total of nine deep concrete core samples were taken with depths ranging from 16 to 72 feet from the top of caissons.

Concrete conditions were remarkably consistent for all cores drilled. Concrete in upper portions of the caissons (upper 12 to 14 feet), extending to approximately the tidal fluctuation zone consisted of good quality concrete (See Figure 12).

Concrete samples extracted from the tidal fluctuation zone (region at water line and below) were less sound and of lower quality. Cement paste was very soft and often washed away from the core surface, leaving exposed aggregate (See Figure 13). This condition is consistent with a concrete mix of very high water to cement ratio. In several locations, sample lengths consisting of only cement paste were extracted (See Figure 14). This condition is consistent with segregation of the mix during original concrete/cement placement.

Below the tidal fluctuation zone, concrete quality was poor, typically disintegrating to rubble during coring. At the main span center pier core hole, concrete quality was poor from the tidal fluctuation zone and below (See Figure 15 and 16).

Based on preliminary observations, it is believed that the poor quality of the concrete in the lower portions of the caissons is due to washout of the cement paste during original construction. Historical ASCE papers (Schneider 1913) describing the bridge construction indicate that concrete was simply dumped into the caissons in still water conditions, as was customary at the time.
Figure 12. Typical Concrete Core Samples, Upper Portions of Piers

Figure 13. Typical Concrete Sample From Tidal Fluctuation Zone
Figure 14. Concrete Sample Showing High Level of Cement Paste

Figure 15. Poor Quality Concrete, Typical of Samples Extracted Below the Tidal Fluctuation Zone.
4. LOAD RATING ANALYSES

A two dimensional computer model was created to complete a load rating evaluation for the six Dumbarton steel through truss spans and swing span. The analysis and modeling was performed using a proprietary program called T187, a structural analysis computer program developed by HNTB. The analysis focused on the vertical load resisting members within the truss and floor system. The result of the examination contains load ratings for the top and bottom chords, hangers, diagonals, floorbeams and stringers.

Using information from existing drawings and field measurements, member section properties and dead loads were calculated with Excel spreadsheets. Member section properties were adjusted to account for effects of corrosion in the bridge members. These modified section properties were incorporated in the computer model for the dead and live load analysis. Results of the computer analysis were used to carry out the load rating evaluation, which was done in conformance with the 2007 American Railway Engineering and Maintenance-of-way Association (AREMA) Manual.

The new rail corridor is expected to be a single track designed for Cooper E-80 loading. Results from the load rating evaluation demonstrated that all vertical load resisting truss and floor...
system elements can safely support a Cooper E-80 loading, with the exception of the stringers and end floorbeams. Their reduced load rating is primarily due to section loss of the members from corrosion. However, the stringers and all floorbeams exceed their fatigue category stress range. For this reason, it is recommended that the stringers and all floorbeams be replaced if the existing steel through trusses were reused for the new rail corridor. The main truss members can be reused.

The load rating of the existing concrete spans indicate that they are capable of supporting a Cooper E-72 train loading, which they were originally designed for.

5. SEISMIC ANALYSES

5.1 Seismic Design Criteria

Project specific performance-based design criteria (Criteria) are under development for the Dumbarton Bridge. The principle design code reference is the 2007 edition of the AREMA Manual for Railway Engineering [AREMA 2007]. Criteria stipulated that the structure must comply with specified performance levels for two distinct seismic levels, an upper level seismic event with a mean return period of 1000 years that corresponds to the Survivability Limit State in AREMA and a lower level event with a mean return period of 91 years that corresponds to the AREMA Serviceability Limit State. At the time of writing this report Criteria was incomplete, but it is anticipated that Criteria will eventually specify the following:

- Allowable damage levels for each seismic demand level
- Material strain or force limits for each damage levels
- Drift and/or residual displacement limits
- Required analysis assessment procedures
- Methodology to consider seismic hazards

5.2 Seismic Analyses of Dumbarton Main Channel Structures

The existing Dumbarton Rail Corridor Bridge substructure is comprised of four unique construction types, designated as Segments 1 through 4 (See Table 1 below). Segment 1 consists of timber trestles; Segment 2 consists of 4-pile concrete bents; Segment 3 consists of 2-pile concrete bents; and Segment 4, the navigational channel, is constructed with large diameter concrete caissons.
Seismic finite element models were prepared incrementally using FE ADINA computer program recognizing that the Dumbarton Bridge is comprised of four unique segments as described in Table 1. For each segment, stand-alone models of individual bents were first prepared. These stand-alone models were useful for a number of reasons including checking modeling techniques used for global models, performing pushover analyses, and performing parametric studies of alternate retrofit strategies. Additionally, since superstructure flexural continuity was not provided across bents, the transverse responses of the stand-alone models are representative of the transverse response of the global bridge.

Global models only considered the portion of the structure extending from the West Abutment to the easterly end of the navigational channel, or the end of Segment 4 shown in Table 1. The total length of this portion of the structure is 6597 ft.

### Table 1 – Division of Dumbarton Bridge Into Four Segments by Bent Types

<table>
<thead>
<tr>
<th>Segment</th>
<th>Bents</th>
<th>Length (ft)</th>
<th>Description</th>
<th>Model Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No. of Nodes</td>
</tr>
<tr>
<td>1</td>
<td>1-60</td>
<td>1770</td>
<td>3(^2) New 30 ft concrete box girder spans with 42 in. precast/prestressed concrete 3-pile bents</td>
<td>3709</td>
</tr>
<tr>
<td>2</td>
<td>61-89</td>
<td>900</td>
<td>Existing 30 ft concrete box girder spans with 24 in. precast/prestressed concrete 4-pile bents</td>
<td>2967</td>
</tr>
<tr>
<td>3</td>
<td>90-142</td>
<td>2340</td>
<td>Existing 45 ft concrete box girder spans with 48 in. precast/prestressed concrete 2-pile bents</td>
<td>3327</td>
</tr>
<tr>
<td>4</td>
<td>143-159</td>
<td>1587</td>
<td>Existing navigational channel with superstructure replaced and new intermediate bents added</td>
<td>4439(^1)</td>
</tr>
<tr>
<td>1, 2, 3 &amp; 4</td>
<td>1-159</td>
<td>6597</td>
<td>Segments 1, 2, 3, and 4 combined into global model</td>
<td>14,459</td>
</tr>
</tbody>
</table>

\(^1\)Properties of Retrofit Option 1  
\(^2\)Subsequent to configuring retrofit models, spans for Segment 1 were increased to 60 ft. Four-pile super bents were added so that the longitudinal stiffness of the 30 ft and 60 ft spans remained essentially identical, thus not affecting the retrofit assessment.

5.3 **Recommended Seismic Strategy**

The recommended seismic retrofit strategy for each of the four segments is as follows:

Segment 1, Timber Trestles – The existing timber trestles will be replaced with new construction.

Segment 2, 4-Pile Concrete Bents – These existing bents have numerous deficiencies including insufficient displacement capacity, inadequate bent cap strength, shear deficiencies in concrete piles, plastic hinges requiring retrofit below the mudline, and deficient shear keys. In
consideration of these numerous deficiencies, replacement of the existing 30’ spans with longer spans is recommended.

Segment 3, 2-Pile Concrete Bents – Retrofit is recommended at Segment 3. The existing 48 in. hollow PC/PS concrete piles have sufficient displacement capacity to avoid retrofit to the piles below the mudline, with the exception of inserting isolation casings over the piles at selected locations. To increase ductility of piles at the connection to the pilecap in the transverse direction, a composite wrap is recommended. However, the composite wrap can be economically placed above the water level. Bent caps have sufficient strength to respond as capacity-protected members. The recommended retrofit at the bent cap is the addition of shear keys to ensure adequate transfer of forces between the superstructure and substructure.

Segment 4, Navigational Channel – Two retrofit options are recommended for further study. Option 1 requires the replacement of the superstructure, but retrofits the existing large diameter caissons with new piling and a concrete shell around the full circumference of the caissons. Additionally, to avoid the need to match the existing 180 ft superstructure spans, new intermediate bents will be added between caissons to reduce spans other than the moveable span to a maximum length of approximately 92-ft. Option 2 calls for the full replacement of all substructure and superstructure elements.

Of the two retrofit options, Option 1 provided the advantage of stiffening the entire structure and permitting the retrofit and reuse of Segment 3. This was due to the much greater stiffness of the retrofitted caissons. Since there are a number of considerations other than seismic performance that will influence the selection of replacement or rehabilitation of the existing substructure at Segment 4, it is recommended to continue studying both options and defer the selection of the preferred option until the regulatory agencies have commented on the two options.

6. PROPOSED SCOPE OF FUTURE RETROFIT OR REPLACEMENT

6.1 General

The summary of the proposed scope of the work in order to bring the existing bridge to current AREMA standards is based on the engineering studies completed to date. The studies included inspections, condition assessment and load ratings, seismic analyses, constructability review, and cost estimates. The work has been subjected to extensive peer reviews by SAMTRANS engineering staff and consultants. This scope of work could change in the future, based on continuing refinements, engineering studies and meeting regulatory agency requirements. At the
time of completing this paper, the current proposed structure retrofit or replacement options are summarized in the Table 2 and highlighted below.

Section 1 & 11 - Replacement of a 1,770 foot long section of timber trestles with concrete box girders.
Section 2 - Replacement of a 900 foot long section of concrete spans with concrete box girders.
Section 3 and 10 - Seismic retrofitting of sections of existing concrete box girders.
Sections 4 to 9 - Replacement of existing truss spans with new concrete bridge sections. The work proposed for Section 7 includes replacing the existing swing span with a new movable bridge (Bascule or Swing).

6.2 Substructures

The superstructures in Sections 4 to 9 are supported on 100 year old pairs of caissons, as follows: One 13’ Diameter, Twenty 18’ Diameter (ten pairs) and One 40’ Diameter.

There are two options for providing substructure support for the new superstructures in Sections 4 to 9 as depicted on Figures 17, 18 and 19. Option 1 includes retrofitting the caissons and installing 21 large diameter piles. Option 2 includes removing the existing caissons and installing new large diameter Cast-in-Steel-Shell (CISS) piles.

Both Options 1 and 2 are recommended to be further analyzed during the next phase of design. Both options are being carried through the environmental clearance phase.
### Table 2 – Dumbarton Bridge Scope as of June 15, 2009

<table>
<thead>
<tr>
<th>SECTIONS</th>
<th>1 and 11</th>
<th>2</th>
<th>3 and 10</th>
<th>4 through 9</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Option Name</strong></td>
<td><strong>Option 1 (Retrofit)</strong></td>
<td><strong>Option 2 (Replace)</strong></td>
<td><strong>Option 1 (Retrofit)</strong></td>
<td><strong>Option 2 (Replace)</strong></td>
</tr>
<tr>
<td><strong>Existing Structure</strong></td>
<td>Timber Trestle (portions burnt)</td>
<td>PC/PS Box Girder Spans on 4-Pile-Bents</td>
<td>PC/PS Box Girder Spans on 2-Pile-Bents</td>
<td>Main Channel Movable, Steel Thru Truss, and Transition Spans on Large Diameter Caissons</td>
</tr>
<tr>
<td><strong>Proposed Structure</strong></td>
<td>New Concrete Spans</td>
<td>Replace with Concrete Spans</td>
<td>Seismic Retrofit Existing Structures</td>
<td>Replace Superstructure with New Movable Bridge and 92' Concrete Spans</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>In-Water Construction Duration</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Approx. 30~36 months in-water construction</strong></td>
<td><strong>Approx. 1~2 months in-water construction</strong></td>
<td><strong>Approx. 24~28 months in-water construction</strong></td>
<td><strong>Approx. 30~36 months in-water construction</strong></td>
</tr>
<tr>
<td><strong>Pile Driving</strong></td>
<td>4' PC/PS pile</td>
<td>4' PC/PS pile</td>
<td>N/A</td>
<td>4.5' PC/PS, 3 steel, 4' steel piles</td>
</tr>
<tr>
<td></td>
<td>114 piles</td>
<td>36 piles</td>
<td>110'/pile</td>
<td>21, 120, 12 piles</td>
</tr>
<tr>
<td></td>
<td>110'/pile</td>
<td>120', 150', 150'/pile</td>
<td></td>
<td>150'/pile</td>
</tr>
<tr>
<td><strong>Total Piles / Length</strong></td>
<td>114 piles/12,540 ft</td>
<td>36 piles/3,960 ft</td>
<td>110'/pile</td>
<td>54 piles/7,020 ft</td>
</tr>
<tr>
<td><strong>Concrete in the Bay</strong></td>
<td>30 cubic yards</td>
<td>168 cubic yards</td>
<td>1322 cubic yards</td>
<td>14,959 cubic yards</td>
</tr>
<tr>
<td></td>
<td>+ 513 cubic yards</td>
<td>+ 43 cubic yards</td>
<td>+10,373 cubic yards</td>
<td>+14,959 cubic yards</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>543 cubic yards</td>
<td>231 cubic yards</td>
<td>1322 cubic yards</td>
<td>25,332 cubic yards</td>
</tr>
<tr>
<td><strong>Fill in the Bay</strong></td>
<td>4,200 cubic yards</td>
<td>1,305 cubic yards</td>
<td>1322 cubic yards</td>
<td>4,271 cubic yards</td>
</tr>
<tr>
<td></td>
<td>1,305 cubic yards</td>
<td>427 cubic yards</td>
<td>1,365 cubic yards</td>
<td>1365 cubic yards</td>
</tr>
</tbody>
</table>

**Note:**
- PC/PS pile - Precast / Prestressed Concrete Pile
- CISS pile - (Concrete) Cast-In-Steel-Shell Pile
- "Concrete in the Bay" quantities include substructure from top of pile to mudline only (no superstructure included)
- "Fill in the Bay" quantities include net change of substructure volume from mudline to bottom of pile only

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Figure 17 – Proposed Scope at Sections 4 to 11.
Figure 18 – Option 1 (Retrofit) at Dumbarton Main Channel Substructure

Figure 19 – Option 2 (Replacement) at Dumbarton Main Channel Substructure
7. CONCLUDING REMARKS

The engineering work performed on the Dumbarton Railroad Bridges has helped SAMTRANS determine the scope, constructability, cost and challenges of retrofitting or replacing the Bridges. It has also helped the environmental scientists determine the environmental impacts of the proposed retrofit or replacement of the Bridges. SAMTRANS will be circulating a draft environmental impact statement and environmental impact report (DEIS/R) on the DRC project in summer 2009. The decision on retrofit or replacement of the Dumbarton Railroad Bridges will depend on several factors including project cost, environmental impacts, constraints and public and agency reviews and comments. SAMTRANS will identify a Locally Preferred Alternative (LPA) including whether the Bridges will be retrofitted or replaced by the end of 2009.

REFERENCES

Schneider, E. J. 1913. Construction Problems, Dumbarton Bridge, Central California Railway, Transactions of the American Society of Civil Engineers, March 19, 1913. Paper No. 1271, 1573-1623