# BOEING ACCESS ROAD BRIDGE SEISMIC RETROFIT

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***Abstract***

The Boeing Access Road Bridge over BNSF and UPRR was built in 1944 and widened in 1966, providing an important connection between I-5 and East Marginal Way with 40,000 ADT. The bridge is comprised of two spans of reinforced concrete girders and three spans of steel girders over seven railroad lines, for a total 337 feet long and 73 feet wide.

Jacobs Engineering performed a seismic analysis and retrofit design of the existing bridge and determined that the bridge did not meet the current collapse prevention criteria for a 1000-year return period earthquake. The following retrofits were implemented to meet the current seismic performance criteria:

1. Ground improvements with compaction grouting to mitigate for liquefaction and lateral spreading.
2. Foundation retrofit including adding micro-piles onto the existing H-pile foundation and bolstering the existing pile caps.
3. Steel jacketing of rectangular concrete columns at intermediate piers to increase ductility and shear capacity.
4. Concrete hinge bearings replacement at Pier 4 by removing deteriorated hinge pedestal bearings and constructing new concrete seat with elastomeric bearings.
5. Girder seat extensions to provide adequate seat length for the steel rocker bearings.
6. Longitudinal seismic restrainers over transverse expansion joint to prevent unseating of girders.

# Extended Abstract

## 1. Bridge Description

The South Boeing Access Road Bridge, originally constructed in 1944 and widened in 1965, is a five span bridge totaling 337-ft in length and spanning over BNSF/UPRR railroads. The total bridge width is 73-ft wide accommodating three lanes of traffic in each direction and a 5-ft sidewalk on the south side of the bridge. The bridge carries approximately 40,000 vehicles per day with truck volumes of approximately 10% of traffic.

The bridge superstructure is comprised of three spans of steel girders with a cast in place concrete deck, and two spans of with cast in place concrete T-beams. The bridge superstructure was widened in kind with a longitudinal joint along the entire length of the deck on the north side.

The bridge substructure at Piers 4, 5 and 6 consists of multi reinforced concrete columns founded on H-piles foundation. Piers 2 and 3 substructure consists of reinforced concrete wall founded on H-Pile foundation.

## 2. Seismic Analysis

### 2.1 Performance Criteria

According to the FHWA Seismic Retrofitting Manual, the minimum performance level required for the South Boeing Access Road Bridge is PL 1 (Life Safety) for the Upper Level Ground Motion (i.e., 1,000-year event) [1]. The bridge is also assigned a Seismic Hazard Level (SHL) IV.

Under a Performance Level 1 (Life Safety), the bridge is expected to experience significant damage and service may be significantly disrupted, but life safety is preserved. The bridge may need to be replaced after a large earthquake.

Bridges can be seismically evaluated using the capacity spectrum method (Method D1) for regular bridges, the structure capacity/demand method (Method D2 or Pushover Method) for regular and irregular bridges, or time history analysis (Method E) for irregular complex bridges[1]. Per the requirements of Table 1-9 of the FHWA Seismic Retrofitting Manual, it is determined that utilizing Method D2 (Pushover Method) is appropriate for the South Boeing Access Road Bridge.

### 2.2 Modeling & Demand Analysis

Seismic forces and displacement demands were determined by performing a linear elastic multimodal response spectral analysis using CSI Bridge Software. A 3D spine model was utilized to obtain the lateral displacement demands. The 3D spine model incorporates the bridge superstructure and substructure elements as shown in Figure 2.

A sufficient number of modes were included in the model such that the total mass participating in the response was at least 90 percent. Modal response contributions were combined using the complete quadratic combination (CQC) method. Response of the structure was analyzed in two orthogonal horizontal directions and the results were combined according to Section 7.4.2 of the FHWA Seismic Retrofitting Manual[1]. Horizontal directions are defined as longitudinal (parallel to the bridge alignment) and transverse (perpendicular to the bridge alignment). Vertical acceleration effects were not included in this analysis.

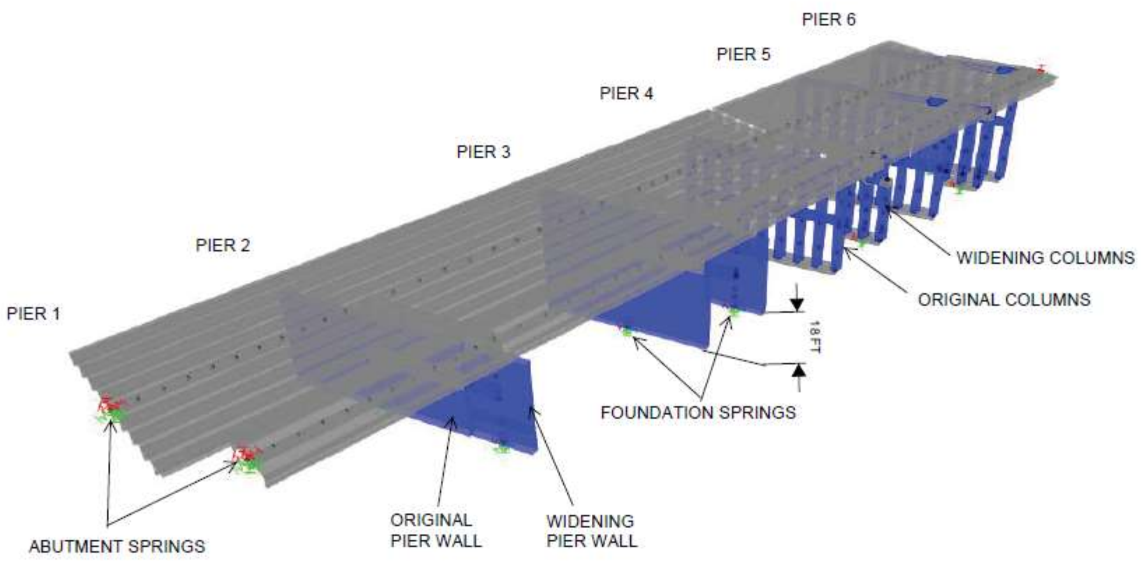


**Figure 2: Bridge 3D Model**

The following assumptions were made for all response spectrum models:

1. Members were modeled with frame (beam) elements with 6 degrees of freedom at each joint and were located along the center of gravity of each section being modeled.
2. The superstructure was modeled as a spine with a minimum of 10 equal segments per span. Superstructure frame elements were modeled at the composite section neutral axis.
3. Body constraints were assigned to the crossbeams joints to ensure forces were properly distributed to the substructure components when subjected to the seismic loads.
4. Columns and walls were modeled with equivalent cracked stiffness in accordance with Table 7-1 of the FHWA Seismic Retrofitting Manual or with a calculated equivalent cracked stiffness.

The bridge intermediate piers are founded on concrete footings with two rows of steel H-piles. Linear foundation springs were established for the pile groups in accordance with FHWA Seismic Retrofitting Manual[1] and the WSDOT BDM[2].

At Piers 3, 4 and 5 the 1965 widening foundations were constructed at much shallower depth than the original bridge foundations (approximately 18-Feet). Therefore, a significant length of the original pier wall at Pier 3 and columns at Piers 4 and 5 are buried underground. To envelope the bridge response, two sets of models are developed. The first model ignores the soils surrounding the wall and columns at Piers 3, 4 and 5, and the foundation stiffness is only based on the piles group linear springs. This model will represent a softer response of the bridge. The second model accounts for the stiffness provided by the soils surrounding the wall and columns at Piers 3, 4 and 5, in addition to the piles group linear springs. This model represents a stiffer response of the bridge.

The abutment at Pier 1 is founded on a narrow concrete spread footing with concrete struts and dead-man anchors providing lateral stability to the abutment. The abutment spread footing translational and rotational springs were developed in accordance with the FHWA Seismic Retrofitting Manual[1] and the WSDOT BDM[2].

The original bridge abutment at Pier 6 consists of a buried multi column bent founded on a concrete footing with H-Piles. Linear foundation springs were established for the pile groups in accordance with FHWA Seismic Retrofitting Manual[1] and the WSDOT BDM[2]. Similar to the intermediate piers foundations, two models are developed to investigate the effect of soils surrounding the buried columns. The widened portion of the abutment is supported on a single row of H-Piles without concrete columns. The lateral stiffness of the H-Piles was modeled based on soil-structure interaction using L-Pile software program.

Seismic loads and dead load effects are included in accordance with the FHWA Seismic Retrofitting Manual[1]. Live loading effects were not considered in the analysis because the presence of live load would not significantly influence the structure's capacity to meet life safety requirements.

The horizontal acceleration response spectrum curve was generated using the provisions established in the AASHTO Seismic Guide Specifications assuming 7% probability of exceedance in 75 years (approximately 1000-year return period) and 5 percent damping[3]. Based on the geotechnical investigation, the soil conditions are within Site Class D. The resulting design accelerations are $A_s$ = 0.48g, $S_{DS}$ = 1.11g and $S_{D1}$ = 0.58g [4].

The geotechnical analysis of the post seismic slope stability indicate that Piers 5 and 6 are susceptible to flow failure as a result of liquefaction of the silt soil layer below fill. The post seismic lateral flow failures loads are applied at Piers 5 and 6 only at depths equal to 15-Feet and 56-Feet, respectively. Values of the lateral flow loads are provided in Table 2 of the Geotechnical Report[4].

### 2.3 Seismic Demand Analysis Results

The assumption that displacements of an elastic system will be the same as those of an elasto-plastic system is not valid for the inelastic performance of short-period structures. Therefore, AASHTO Seismic Guide Specifications, Section 4.3.3 provisions are used to determine the displacement magnification factor $R_d$ for short-period structures[3].

The displacement magnification factor was considered for both orthogonal directions. It was determined that $R_d$ is equal to 1.0 and 1.43 in the bridge longitudinal and transverse directions, respectively. Utilizing the response spectral multimodal analysis, the bridge displacement demands are reported in both the longitudinal and transverse directions for each intermediate pier.

The force demands generated from the application of the lateral soil forces in the bridge longitudinal direction are reported. L-Pile software and CSI Bridge are used to determine the demands on the steel H-Piles and concrete columns.

The seismic demands on capacity protected members such as the superstructure and foundations are determined based on the over strength plastic hinging forces of the existing columns. An over strength factor of 1.4 is used as required per section 8.5 of AASHTO Guide Specifications[3]. In the bridge transverse direction at Piers 2 and 3, the seismic demands on the foundation elements are determined based on the anticipated elastic forces for the walls since the walls will not form plastic hinges when loaded in their strong direction.

The required bridge seat width at Pier 4 is calculated in accordance with the AASHTO Seismic Guide Specifications requirements[3].

### 2.4 Seismic Capacity Analysis & Results

Method D2 (Pushover) analysis of the FHWA Seismic Retrofitting Manual is used to determine the inelastic displacement capacity of the bridge[1]. The inelastic displacement capacity of a pier is directly related to its columns' ability to withstand plastic hinging and to accommodate the plastic rotational demands.

The evaluation of the plastic rotational capacity of columns is determined in accordance with Section 7.8.2 of the FHWA Seismic Retrofitting Manual. Potential plastic hinge locations and local deformation limit states such as compression failure of concrete, buckling of longitudinal reinforcement, low-cycle fatigue, lap splice failure, and shear failure are identified for each column within a pier. The limit state resulting in the least plastic rotation of a member is determined to be the controlling limit state[1].

The plastic rotational capacity of a plastic hinge is directly proportional to the column curvature capacity which is sensitive to the axial load magnitude. Therefore, the column plastic rotational capacity is determined at several axial load values that envelope the anticipated axial loads of the different columns within a pier. CSI Bridge analysis program is used to perform the pushover analysis. The displacement capacity of individual piers is evaluated in the longitudinal and transverse directions independently.

The flexural and shear capacity of the superstructure, crossbeams and foundations are calculated in accordance with the AASHTO LRFD Bridge Specifications[5]. Column joint shear capacities are calculated in accordance with the FHWA Seismic Retrofitting Manual[1].

The existing bridge seat width capacity at Pier 4 is determined based on the geometry provided in the bridge as-built plans.

### 2.4 Seismic Evaluation Results

Based on the displacement capacity and demands, a displacement Capacity/Demand (C/D) ratio is calculated for each Pier. A displacement C/D ratio less than 1.0 indicates a structural deficiency and a need for seismic retrofit. Although, a column displacement C/D ratio larger than 1.0 may be interpreted as an indication of the column's satisfactory performance under seismic loading, some column damage can be expected at these inelastic displacements, particularly when the displacement demand exceeds four times the yield displacement.

A force C/D ratio less than 1.0 for a crossbeam or another capacity protected element indicates that this particular element does not have adequate capacity to ensure that plastic hinging only develops in the columns. Therefore, seismic retrofit would be required to ensure that capacity protected elements behave elastically as assumed in the analysis.

Tables 2 and 3 below provide a summary of the C/D ratios for the critical elements in the South Boeing Access Road over BNRR Bridge for the seismic load case and post-seismic lateral spreading load case, respectively:

| BRIDGE ELEMENT | Pier 1 | | | Pier 2 | | | Pier 3 | | | Pier 4 | | | Pier 5 | | | Pier 6 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | C | D | C/D | C | D | C/D | C | D | C/D | C | D | C/D | C | D | C/D | C | D | C/D |
| **Foundation** | | | | | | | | | | | | | | | | | | |
| Pile Axial Comp (kips) | N/A | N/A | N/A | 180 | 222 | 0.81 | 180 | 222 | 0.81 | 180 | 521 | 0.35 | 180 | 449 | 0.40 | 180 | 339 | 0.53 |
| Pile Shear (kips) | N/A | N/A | N/A | 133 | 22 | 6.1 | 146 | 37 | 3.9 | 146 | 29 | 5.1 | 146 | 15 | 9.4 | N/A | N/A | N/A |
| Pile Moment (kip.ft.) | N/A | N/A | N/A | 60 | 25 | 2.4 | 63 | 55 | 1.2 | 63 | 34 | 1.9 | 63 | 15 | 4.3 | N/A | N/A | N/A |
| **Column / Wall** | | | | | | | | | | | | | | | | | | |
| $\Delta_{Longitudinal}$ (in.) | N/A | N/A | N/A | 8.7 | 8.5 | 1.0 | 14.7 | 8.3 | 1.8 | 7.77 | 6.28 | 1.2 | 5.5 | 5.8 | 0.95 | 13.3 | 6.6 | 2.0 |
| $\Delta_{Transverse}$ (in.) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 5.73 | 2.85 | 2.0 | 4.4 | 3.1 | 1.4 | 4.2 | 2.6 | 1.6 |
| Shear (kips) | N/A | N/A | N/A | 1668 | 1312 | 1.3 | 1684 | 1206 | 1.4 | 82 | 93 | 0.88 | 128 | 154 | 0.83 | 56 | 85 | 0.66 |
| **Superstructure Frame** | | | | | | | | | | | | | | | | | | |
| Shear (kips) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 99 | 74 | **1.3** | 59 | 40 | 1.5 |
| Moment (kip.ft.) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 1769 | 1736 | **1.02** | N/A | N/A | N/A |
| **Seat Width** | | | | | | | | | | | | | | | | | | |
| Seat Width (in.) | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 22 | 25.6 | 0.86 | N/A | N/A | N/A | N/A | N/A | N/A |

**Table 2: Capacity/Demand Ratios Summary (Seismic Case)**

| BRIDGE ELEMENT | Pier 4 | | | Pier 5 | | | Pier 6 | | |
|---|---|---|---|---|---|---|---|---|---|
| | C | D | C/D | C | D | C/D | C | D | C/D |
| **Foundation** | | | | | | | | | |
| Pile Axial Comp (kips) | N/A | N/A | N/A | 180 | 4832 | 0.04 | 180 | 9391 | 0.02 |
| Pile Shear (kips) | N/A | N/A | N/A | 133 | 78 | 1.7 | 133 | 225 | 0.59 |
| Pile Moment (kip.ft.) | N/A | N/A | N/A | 183 | 101 | 1.8 | 183 | 279 | 0.66 |
| **Column** | | | | | | | | | |
| Shear (kips) | 35 | 105 | 0.33 | 66.0 | 661.0 | 0.10 | 77 | 1443 | 0.05 |
| Moment (kip.ft.) | 1445 | 2796 | 0.52 | 821 | 14170 | 0.06 | 675 | 23470 | 0.03 |

**Table 3: Capacity/Demand Ratios Summary (Post-Seismic Case)**

## 3. Seismic Retrofit Design

### 3.1 Ground Improvements

Ground improvement options for mitigation of the post-seismic stability issues at Piers 5 and 6 of the South Boeing Access Road Bridge were considered, including vibrocompaction, vibroreplacement (stone columns), deep soil mixing/cutter soil mixing, jet grouting, compaction grouting, and deep foundation shear elements. Given the low overhead and relatively small area, compaction grouting was used for mitigation of the post-seismic stability issues at the bridge.

The improvement area extended over the width of the bridge plus about 5 feet on each side and from the base of Bent 5 to approximately 30 feet east of Bent 5. The primary grout pipe spacing was about 7 feet on center. The grout pipes would extended to a depth of about 25 feet.

### 3.2 Concrete and Steel Rocker Bearings Replacement

The concrete rocker bearings at Pier 4 were in poor condition and required removal and replacement. Although the steel rocker bearings at Piers 1, 2, 3 and 4 are generally in fair to good condition, steel rocker bearings are prone to failure during a seismic event; therefore, they are typically replaced.

The initial plan was to replace all the rocker bearings at Piers 1, 2, 3 and 4 with elastomeric reinforced bearing pads or fabric pad sliding bearings on reinforced concrete pedestals. However, at Piers 2 and 3, the proximity of the railroad tracks did not allow for the installation of the temporary shoring system for bearing and pedestal construction.

Since the steel rocker bearings are generally in fair condition and their failure will not result in the bridge collapse, the design team determined that not replacing the steel rocker bearings at Piers 1, 2, 3 and 4 is consistent with the seismic performance criteria for the project of PL1 (Life safety). Seat extension were provided at Pier 1 to prevent span unseating in case the fixed steel rocker bearings fail. Seat extensions and longitudinal seismic restrainers were also provided at Pier 4 to prevent span unseating.

### 3.3 Longitudinal Seismic Restrainers at Pier 4

Longitudinal seismic restrainers are required at intermediate piers with an expansion joint to prevent spans unseating during a seismic event[1]. Longitudinal seismic restrainers were attached to the soffit of the concrete slab on both sides of Pier 4 with a sufficient gap at one end of the longitudinal restrainers to allow for thermal movement.

### 3.4 Piers 4 and 5 Column and Foundation Retrofit

### 3.5 Steel Column Jackets Retrofit

The steel jackets provide adequate confinement to preserve the column core as it is going through extreme displacements during a seismic event. The steel jackets also improve the lap splice performance near the base of the column by providing a clamping pressure, which results in a frictional restraint against slippage of longitudinal bars. Furthermore, the steel column jacket will enhance the columns shear strength. Improving the columns ductility, lap splice performance and shear strength will result in enhancing the columns displacement capacity to accommodate the anticipated displacement demands.

### 3.6 Foundation Retrofit

The existing H-Piles at Piers 4 and 5 do not have the adequate strength to handle the demands associated with plastic hinging forces of the columns. The existing pile-footing connection is assumed to have no tension capacity since the piles are only embedded 6-inches into the footing.

The existing footings were widened and a new row of micropiles was added on both sides. Furthermore, the existing footing was be thickened and resin bonded drilled dowels were added. The new micropiles acting both in tension and compression will resist the anticipated column plastic hinging forces.