JOHNSON STREET BRIDGE

SEISMIC DESIGN CRITERIA

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1.0 SCOPE

This document describes the seismic design criteria for the analysis and design of the new Johnson Street Bridge (JSB). The JSB is considered as a ‘Critical’ bridge within the scope of this document. This designation has been chosen to be consistent with the AASHTO set of design codes which are to be followed for the design of this structure. The ‘Critical’ designation can be considered equivalent to the ‘Lifeline’ designation as described in the Canadian highway Bridge Design Code (CHBDC S6-06).

2.0 CODE/REFERENCE ORDER OF PRECEDENCE

The JSB analysis and design shall be governed by the codes and references in the order of precedence specified below, unless specifically altered later in this document:

1. Seismic design criteria for Johnson Street Bridge (this document).
2. AASHTO LRFD movable highway bridge design specifications, 2nd edition with 2008 and 2010 interim revisions
3. AASHTO LRFD Bridge design specifications, 5th edition with 2010 interim revisions
4. ATC 49
5. ATC 32-1
6. AASHTO guide specifications for LRFD seismic bridge design, 1st edition with 2010 interim revisions
7. Caltrans guide specifications for seismic design of steel bridges
8. MoTI supplement to CHBDC S6-06
9. CHBDC-S6-06

3.0 SEISMIC LOADING

The JSB will be analysed for earthquakes with different return periods, and a different performance level shall be considered for each earthquake. The earthquakes to be considered and the corresponding performance criteria are based on the AASHTO LRFD movable highway bridge design specifications code guidelines which also refer to the AASHTO LRFD bridge design specifications code. The seismic design is to be based on a Design Earthquake with a 7 percent probability of exceedence in 75 years (i.e. a return period of 1000 years), along with an operational level earthquake with a 10 percent probability of exceedence in 50 years (i.e. a return period of 475 years). The AASHTO LRFD movable design code speaks to the need of the Owner and the Designer establishing seismic performance goals consistent with the importance of the bridge. Given that the JSB is a Critical bridge, and the vicinity of the site to the Cascadia Subduction zone, the Cascadia Subduction event with an average return period of 500-600 years shall also be considered. The bridge shall also be analysed for various open positions for earthquake return periods that correspond to the same probability of exceedence for the closed position. Based on the current Johnson Street Bridge operating regime, it is assumed that the new bridge will be in the open position up to 2 hours per 24 hours.

4.0 PERFORMANCE REQUIREMENTS

The approach taken by the AASHTO set of design codes is for the bridges to be designed such that they have a low probability of collapse but may suffer significant damage and service disruptions when subjected to the design level earthquake motion. However, small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage. Based on the AASHTO seismic design philosophy, the JSB shall be designed for multi-level performance criteria as described below.

The following table summarizes the performance requirements and corresponding materials and component design limitations:

Superstructure and Electrical/Mechanical Components:

Design Earthquake	Performance Requirements	Superstructure Steel Members	Superstructure Concrete Members	Deck Joints	Pinion/ Rack	Electrical/ Other Mechanical
10% in 50 years (1 in 475)	Service: Immediate- Full access is available to all traffic within hours. Damage: Minimal to none.	Essentially elastic -buckling and flexural and tensile yielding not permitted in primary members. Minor yielding or local buckling of secondary steel elements may occur.	Essentially elastic- minor inelastic response limited to narrow cracks in concrete and some rebar yielding. This corresponds to $\varepsilon_c \leq 0.004$ and $\varepsilon_s \leq 0.01$.	Functional- some pounding damage may occur.	Functional – no damage and in full service.	Functional – no damage and in full service.

7% in 75 years (1 in 1000)	Possible permanent loss of service.	Ductile response may be relied upon corresponding to this seismicity level. Collapse prevention criteria need to be satisfied. See section 6.1 of the current document.	Ductile response may be relied upon corresponding to this seismicity level. Collapse prevention criteria need to be satisfied. See section 6.1 of the current document.	Possible loss of service / permanent damage	Possible loss of service / permanent damage	Possible loss of service / permanent damage
Cascadia Subduction event	Service: Significantly Limited- Limited access to emergency traffic is possible within days of the earthquake. Full access to public may resume in several weeks to months.	Ductile response may be relied upon corresponding to this seismicity level. Reparability criteria need to be satisfied. See section 6.1 of the current document.	Ductile response may be relied upon corresponding to this seismicity level. Reparability criteria need to be satisfied. See section 6.1 of the current document.	Possible loss of service / permanent damage.	May require extensive repairs.	Possible loss of service / permanent damage.

Substructure:

Design Earthquake	Performance Requirements	Bents	Piers	Piles, Footings, Pile Caps	Restrainers, Lock-up Devices	Column-Cap beam, Column-Footing, Pile-Pilecap Joints
10% in 50 years (1 in 475)	Service: Immediate-full access to normal traffic is available to all traffic within hours. Damage: Minimal to none.	Essentially elastic- minor inelastic response limited to narrow cracks in concrete and some rebar yielding. This corresponds to $\varepsilon_c \leq 0.004$ and $\varepsilon_s \leq 0.01$.	Essentially elastic- minor inelastic response limited to narrow cracks in concrete and some rebar yielding. This corresponds to $\varepsilon_c \leq 0.004$ and $\varepsilon_s \leq 0.01$.	Essentially elastic- minor inelastic response limited to narrow cracks in concrete and some rebar yielding. This corresponds to $\varepsilon_c \leq 0.004$ and $\varepsilon_s \leq 0.01$.	Functional (limits to be determined in consultation with the supplier).	Essentially elastic joint response- principal stresses to be limited to values per ATC 49.
7% in 75 years (1 in 1000)	Service: Possible permanent loss of service.	Significant inelastic response may occur. $\varepsilon_c \leq 0.75\varepsilon_{cu}$ $\varepsilon_s \leq 0.75\varepsilon_{su}$.	Significant inelastic response may occur. $\varepsilon_c \leq 0.75\varepsilon_{cu}$ $\varepsilon_s \leq 0.75\varepsilon_{su}$.	Footings, pile caps and piles to be capacity protected corresponding to overstrength column demands in bents. Piles under wall piers may yield if unavoidable, but these must	Failure may occur but this should not lead to global structural collapse.	Inelastic joint response may occur- principal stresses to be limited to values per ATC 49. Secondary force transfer mechanisms must be provided if significant inelastic

				possess enough ductility to withstand imposed displacement demands.		response is predicted.
Cascadia Subduction event	Service: Significantly Limited- limited access to emergency traffic is possible within days of the earthquake. Full access to public may resume in several weeks to months.	Inelastic response permitted- cracks may require epoxy injection, extensive spalling may occur, rebar may yield significantly. $\varepsilon_c \leq 0.5\varepsilon_{cu}$ $\varepsilon_s \leq 0.5\varepsilon_{su}$.	Inelastic response permitted- cracks may require epoxy injection, extensive spalling may occur, rebar may yield significantly. $\varepsilon_c \leq 0.5\ \varepsilon_{cu}$ $\varepsilon_s \leq 0.5\ \varepsilon_{su}$..	Footings, pile caps and piles to be capacity protected corresponding to overstrength column demands in bents. Piles under wall piers may yield, if unavoidable, but must possess enough ductility to withstand imposed displacement demands.	Functional (limits to be determined in consultation with the supplier).	Inelastic joint response may occur- principal stresses to be limited to values per ATC 49. Secondary force transfer mechanisms must be provided if significant inelastic response is predicted.

5.0 SEISMIC ANALYSIS

5.1 General

A global 3D model of the bridge shall be produced for analysis and design of the JSB. Multimodal response spectral analysis shall be performed as a minimum requirement for designing the bridge for the operational level earthquake (1 in 475). Although the AASHTO LRFD movable design code refers to the multimodal response spectral analysis as the minimum analysis requirement, AASHTO LRFD bridge design specifications document makes it mandatory to carry out time history analysis for multi-span Critical bridges that are irregular. Pushover and nonlinear time history analyses shall therefore be used to confirm the performance requirements for the levels of seismicity other than the functional earthquake (i.e. 1 in 1000 and the Cascadia Subduction event).

5.2 Design Ground Motions:

Design response spectra acceleration parameters shall be obtained using established site-specific procedures. Vertical ground motions must be considered for the design and performance assessment and confirmation of the JSB. At least three acceleration time histories shall be used for each of the three (longitudinal, transverse and vertical) motion components for non-linear dynamic analysis for confirming performance requirements of the bridge. The design actions shall be taken as the maximum response calculated by combining the three ground motions in each of the three directions. If a minimum of seven time histories is used for each component, the design actions may be taken as the mean response calculated for each principal direction.

5.3 Geotechnical Considerations and Soil Structure Interaction

5.3.1 General

The effects of foundation and abutment flexibility and capacity, based on the best estimate of site conditions and soil parameters, shall be taken into account in analysing the overall bridge response and the relative distribution of earthquake effects to various bridge components. A site-specific geotechnical study shall be carried out to ascertain the site conditions and soil parameters for the JSB.

5.3.2 Soil Springs for Soil Structure Interaction

Winkler springs for modeling soil-foundation-structure interaction shall be incorporated into the global 3D model of the bridge. As a minimum requirement, springs accounting for bearing, side and base friction, and passive resistance shall be used as and when applicable. The springs may be non-linear in nature with different properties in different directions, as appropriate.

5.3.3 Liquefaction, Lateral Spreading and Downdrag Analysis

The liquefaction and lateral spreading potential for the JSB site shall be investigated and appropriately incorporated into the global analysis, if warranted. In addition, any lateral spreading and pile downdrag effects shall also be included as and when required.

5.4 Global Modeling and Closed Versus Open Positions

Global 3D models shall be used to analyze the Johnson Street Bridge. Analyses shall be carried out to ascertain bridge behavior and performance in several open positions as well as the closed position. For the closed position, the global model shall include both the bascule and approach spans. A sufficient number of open position scenarios shall be investigated to have confidence in the analyses, as well as to understand the bounds of bridge performance for the applicable levels of seismicity. For the open positions, the bascule and the approach spans shall be analysed separately accounting for the absence of connectivity of the structures to each other.

5.5 Combination of Seismic Force Effects

For response spectrum analysis, the maximum seismic force due to seismic load in any one direction shall be based on the CQC combination of modal responses due to the ground motion in that direction. The maximum force due to the three orthogonal ground motions shall be obtained by the SRSS combination rule. For time-history analysis, the three orthogonal acceleration records shall be applied simultaneously and therefore no further combination of forces or displacements is necessary.

5.6 Equivalent Stiffness for Analysis

For concrete members expected to undergo plastic hinging or significant cracking, cracked stiffness properties shall be used for the elastic and pre-hinging part of the inelastic analysis.

6.0 COMPONENT DESIGN AND CAPACITIES

Components shall be designed and detailed to fulfill the design intent as spelled out in the AASHTO set of design codes. Component capacities shall be determined such that they are in line with the philosophies described below:

6.1 Steel Components

The primary superstructure steel elements shall be designed as essentially elastic for the 1 in 475 year earthquake. Local and global buckling shall be prohibited, and connections and members shall be designed to behave essentially elastically for this level of seismicity. For higher levels of seismicity, the superstructure shall either be strong enough to withstand force demands corresponding to the substructure over strength capacity development, or elastic demands, whichever is lower. Superstructure members may rely on explicitly accounted for ductilities for the other seismicity levels (i.e. 1 in 1000 earthquake and the Cascadia Subduction event), provided that repair and collapse prevention criteria are met for the corresponding levels of seismicity.

6.2 Concrete Components

The primary superstructure concrete elements shall be designed as essentially elastic for the 1 in 475 year earthquake. For higher levels of seismicity, the superstructure shall either be strong enough to withstand force demands corresponding to the substructure overstrength capacity development, or elastic demands, whichever is lower. Superstructure members may rely on explicitly accounted for ductilities for the higher seismicity levels, provided that repair and collapse prevention criteria are met for the corresponding levels of seismicity.

Substructure concrete elements shall be designed to be essentially elastic for the 1 in 475 year earthquake. For other seismicity levels (i.e. 1 in 1000 and the Cascadia Subduction event), the substructure elements shall have carefully designed and detailed plastic hinge locations at column bases and tops.

Capacity design principles shall be employed for the design of regions outside the plastic hinge zones as well as for brittle failure mechanisms (such as shear failure) within the plastic hinge zones. No rebar splicing shall be allowed with in the plastic hinge zones. For rebar terminations outside of plastic hinge zones, due consideration shall be given to steel overstrength and tension shift effects.