Displacement-Based Seismic Analysis of a Mixed Structural System

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SUMMARY
As energy costs soar, public demand for massive transportation systems has increased. Massive transportation systems often include structures such as station buildings that are linked by elevated bridges. It is essential for these aerial structures to withstand large earthquakes. Structural engineers typically design station buildings according to the IBC which references ASCE 7; while bridge design is based on the AASHTO LRFD bridge design code and the Guide Specification for seismic design. IBC and AASHTO LRFD both use force-based design, but with different spectral loads, analysis parameters, and levels of acceptance. The newly published AASHTO Seismic Guide Specification applies displacement-based seismic design as its framework. However, there is no unified code for the seismic design of this kind of mixed structural systems in the United States. How will these codes be implemented in the design of mixed structural systems without violating either code? Based on displacement-based seismic design concepts, this paper uses a case study to show how the seismic design of mixed structural systems can satisfy the performance objectives of both the IBC and AASHTO codes without producing a design that is overly conservative.

Keywords: building, bridge, displacement-based, seismic

1. INTRODUCTION

Massive transportation has become a popular and sustainable solution for the public need as the cost of energy soars. In a high-seismic region such as Pacific Northwest, seismic demands contribute significantly to the structural design of transportation structures. Massive transportation systems typically include structures such as elevated bridges and buildings such as light rail stations. For this kind of mixed structural system, the building structural engineers design the station according to International Building Code (IBC) and other relevant standards, while the bridge structural engineers design the elevated bridges based on bridge design specifications such as AASHTO LRFD Bridge Design Specifications (LRFD) and AASHTO Guide Specifications for LRFD Seismic Bridge Design (LRFD Seismic Guide). Table 1 summarized the major differences in building and bridge seismic codes.

Due to the inherent differences between building and bridge structures, different seismic design criterion, analysis parameters, and acceptable levels of performance are specified by code. For example, buildings structures are designed for seismic events with 2% probability of exceedance in 50 years and for a service life of 70 years; the return period for the extreme seismic event is about 2500 years. Bridges have a design service life of 75 years, and shall be designed for a seismic event with 7% probability of exceedance in 75 years, which means the return period of about 1000 years.

Seismic events which the structures designed for are anticipated demands during the service life of the structures. On the capacity side, three importance factors (1.0-1.5) are used to characterize the occupancy level of the building structures in IBC; similarly, in AASHTO LRFD three operational categories are classified for bridges according to the importance level of the bridge. These levels of classification quantify the extent of the structural damage allowed; less damage is acceptable for more important structures.

In addition to the requirements for the structural elements, it has been noticed that non-structural element damage plays a more significant role in controlling the hazard level (Priestley 2003) of the building structures. Non-structural elements require the allowable drift ratio to be limited within 0.025. This corresponds to lower possibility of substantial concrete damage for special moment reinforced concrete frames (Lowes and Li). Similar to controlling damage based on allowable drift ratios, controlled material strain limits have also been used recently to quantify damage in bridge columns.

Table 1. Seismic Design Criteria

	Design Earthquake Return Period (years)	Design Response Spectrum	Occupancy Category	Design Philosophy	Deformation Limit
IBC (references ASCE 7)	2475	2/3 of design earthquake	IV	Force-based	yes
AASHTO LRFD	1033	Design earthquake	III	Force-based	No
AASHTO Seismic	1033	Design earthquake	None	Displacement-based	Yes

This paper uses a case study to show that the seismic design can meet both criteria and at the same time not overly conservative. The case study is a light rail station entrance near the University of Washington stadium in Seattle, Washington, which connects a pedestrian bridge over an arterial roadway to an underground station. Figure 1 shows the 3D rendering of the pedestrian bridge and station entrance generated from SAP2000 software.

The station entrance building is a two-story, 12 meter tall reinforce concrete structure. The lower story supports the bridge and a low roof. The upper story supports a high roof over the end of the bridge and stairs. The lateral system consists of three bays of special reinforced concrete moment frames transverse to the bridge direction and special reinforced concrete shear walls (in the form of deep columns) in the longitudinal direction. Main transverse members are concrete beams that support the end of the bridge, the low roof framing, and the high roof framing. Secondary steel framing spans between the concrete frames to support the glazing system, ceiling loads and metal roof deck. The station roof at grade supports the station entrance building. The pedestrian bridge is a curved post-tension prestressed concrete box-girder bridge, with one bicycle ramp and one pedestrian extension into the station entrance building, as shown in Figure 1. The seismic demand of the bridge on the station entrance frame includes lateral displacement demand and the connection force demand passing from the bridge bearings to the frame beam.

Ideally it would be necessary to perform a linear or nonlinear time-history analysis to determine the behavior for this kind of irregular structure. However, response spectrum analysis is more conventional and provides adequate results. Linear modal response spectrum analysis was used in the design of this mixed structure to satisfy the force-based requirements of the IBC and AASHTO design codes. Additionally, a non-linear static (pushover) analysis was performed on the station entry structure to verify the compatibility of the deformation and ductility of the mixed structural systems.

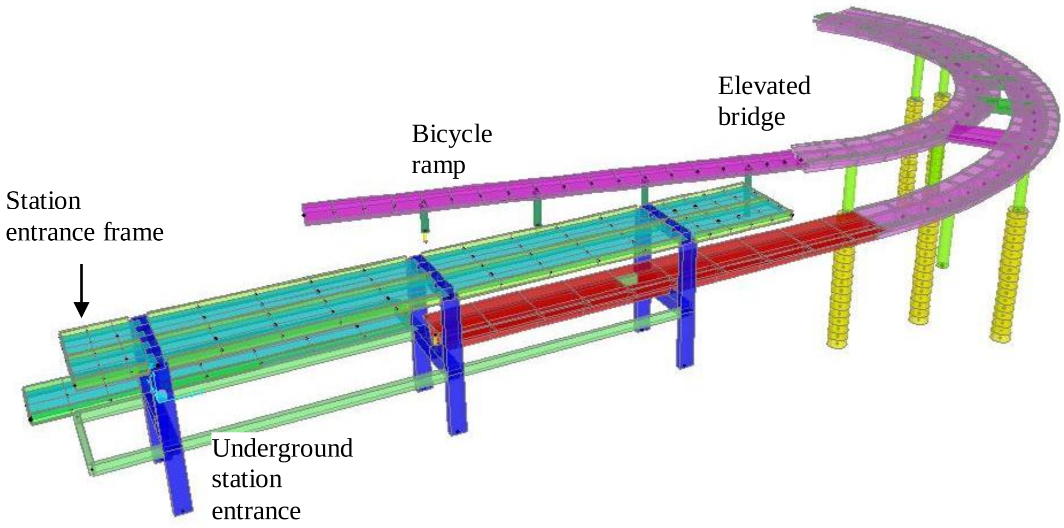


Figure 1. Bridge and Station Entry systems

2. CASE STUDY: IBC AND AASHTO SEISMIC DESIGN PARAMETERS

2.1 Structural Occupancy Level

Heavy pedestrian use is expected considering the station is near a major public university and its stadium. The station entry was designed for occupancy category III, which represents a substantial hazard to human life in the event of failure. A seismic importance factor of 1.25 was assigned to the station entry structure according to ASCE 7-05. The bridge was identified as an essential bridge with an importance factor of 1.0 according to AASHTO LRFD (2010).

2.2 Soil Properties

The soil under the structures is dense soil classified as Class C by both AASHTO LRFD and ASCE 7-05.

2.3 Design Response Spectrum

According to ASCE 7-05, the design spectrum for the station entrance is 2/3 of the maximum considered earthquake (MCE), which is an event with a 2 percent probability of exceedance in 50 years (approximately 2500 years return period). The design spectrum parameters are***:***
Sds =0.87, Sd1 =0.4

According to AASHTO LRFD (2010), the bridge was designed for seismic event with 7% possibility of exceedance in 75 years of bridge design life, the design spectrum parameters are***:***
Sds =1.0g, Sd1 =0.48g

The response spectrum parameters mean that the seismic design category is D based on either IBC (2006) or AASHTO LRFD (2010) criteria. The seismic design categories define different levels of criteria for component detailing and system performance. Figure 2 shows the design response spectrum under different codes.

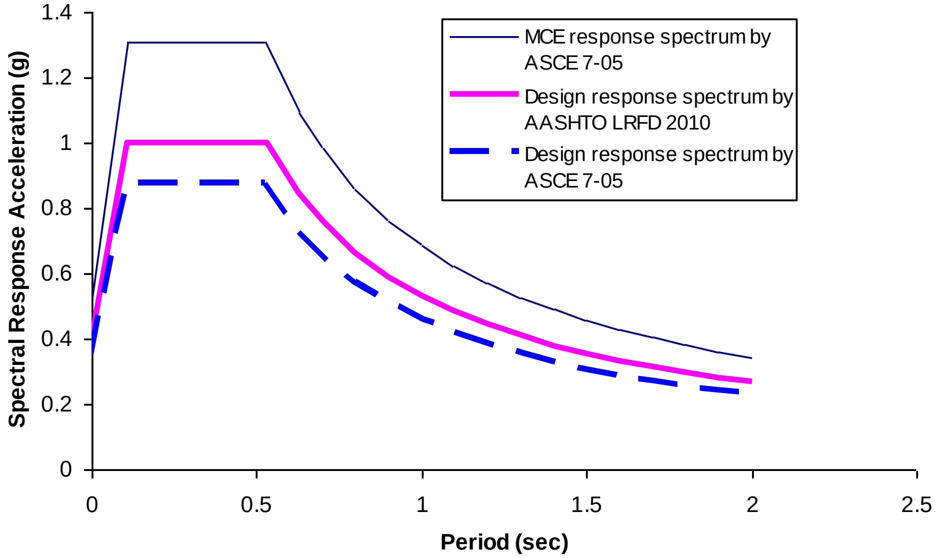


Figure 2. Design Response Spectra

2.4 Response Modification factors

For the station entrance, ASCE 7-05 defined the response modification factor R=8 for special moment frame (SMF), and R=5 for special reinforced shear walls.
For the bridge, AASHTO LRFD (2010) defined column moment modification factor R=3.5 for essential bridges with multiple bent columns. Bridges generally have less redundancy than buildings. Buildings have more secondary structural element which do not exist in bridges.

2.5 Overstrength factor

For the station entrance, ASCE 7-05 defined overstrength factor Ω_o=3 for SMF, and Ω_o=2.5 for special reinforced concrete shear wall. For the bridge, Q=1.3 for reinforced concrete components.

2.6 Deformation and Ductility Limit

For the station entrance, ASCE 7-05 defined the deflection amplification factor Cd=5.5 for SMF, and Cd=5 for special reinforced concrete shear wall. The allowable story drift ratio limit ranges from 0.011-0.015 for the Occupancy category three for special reinforced concrete moment frames and special reinforced concrete shear walls. Note that the 0.011 and 0.015 limits result from the 0.015 and 0.02 drift limits divided by the redundancy factor (rho) per ASCE 7-05 Section 12.12.1.1. For the bridge, AASHTO Seismic (2009) specified ductility demand less than 6 for bridges with multiple columns bents. Table 2 summarized the major seismic parameters for the case study.

Table 2. Case study: seismic Design Parameters

	Design Peak Acceleration Sds	Importance factor	Force modify-cation factors	Redun-dancy factor	Deform-ation Limit	Over strength factor	Ductility factor
IBC	0.87g	1.25	5 (long.) 8 (trans.)	1.3	0.011-0.015	2.5 (long.) 3 (trans.)	5 (long.) 5.5 (trans.)
AASHTO LRFD	1.0g	1.0	3.5	N/A	N/A	1.3	N/A
AASHTO Seismic	1.0g	1.0	N/A	N/A	N/A	1.4	6

3. EARTHQUAKE RESPONSE MECHANISM OF THE STRUCTURAL SYSTEM

Seismic design for building structures is based on a “strong column and weak beam” principle, allowing beam damage to dissipate the earthquake energy and accommodate the building frame deformation demand, thus preventing collapse by limiting column damage. On the other hand, seismic design for bridges is based on a “strong beam (girders) and weak column” principle. Girders and diaphragms are not permitted to have large plastic deformations, and the columns are the elements intended to deform plastically to dissipate the earthquake energy.

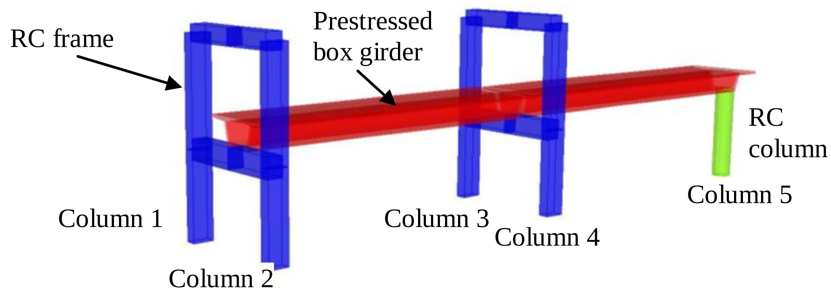
For a mixed structural system, a clear lateral load resistance path has to be defined based on the basic principles for building and bridge designs. Figure 3(a) shows the earthquake resistance system (ERS) for the station entrance and portion of the bridge that participates with the station entrance ERS. The bridge box girder is assumed pinned on the two station entrance transverse beams. The station entrance frame column size is 0.6m*1.2m, the beam size is 0.9m*1.1m. The longitudinal span is 20 meters, and the transverse span of the frame is 6 meters. The prestressed box bridge girder is 1.5 meters deep.

Figure 3(b) shows the transverse plastic mechanism for the system, the plastic hinges at beam ends and column bottoms are the main earthquake fusing elements transversely. The RC frame is designed as special moment frame in transverse direction.

Figure 3(c) shows the longitudinal plastic mechanism. The bridge girder sits on the elastomeric bearings on the top of station beams. The deep columns are detailed as special cantilevered shear walls in the longitudinal direction and the plastic hinges are at bottoms.

The pedestrian bridge girder and connections to the station entrance frame are designed as capacity protected elastic members. The seismic demands of the bridge on the station entrance frame includes lateral displacement demand and the connection force demand passing from the bridge bearings to the frame beams.

Table 3 shows the force demand on the plastic hinges based on the elastic analysis and the response modification factors described in section 2. It is noted here that for elastic analysis, IBC and AASHTO also have different recommendation for estimation of the member cracked section properties. ACI 318-08 recommended using cracked section property of 0.35EI for beam, and 0.7EI for column. AASHTO



(a): 3D Model of the Station Frames and the Bridge

(b): Transverse Section (c): Longitudinal Section

Figure 3. Seismic Resistance Systems

Seismic (2009) recommended using section property of 0.3EI to 0.5EI for concrete columns based on reinforcement ratio and axial loads. In Table 3, the forces are calculated based on frame beam sections of 0.35EI, frame column sections of 0.7EI, and the bridge end column section of 0.5EI. It can be seen that the column forces demand are significantly different for building and bridge design. Forces based on AASHTO LRFD (2010) code are 20 percent larger in the bridge longitudinal (x) direction, and 90% larger in the transverse (y) direction, than those calculated from ASCE 7-05.

Table 3. Force-based seismic demand

		Column 1	Column 2	Column 3	Column 4	Column 5
IBC (2009)	Myy (KN-m)	897	861	884	866	0
	Vx (KN)	120	102	117	108	0
	Mxx (KN-m)	374	379	333	334	288
	Vy (KN)	137	140	122	122	66
AASHTO LRFD (2010)	Myy (KN-m)	1108	1058	1092	1065	0
	Vx (KN)	145	122	140	128	0
	Mxx (KN-m)	709	719	639	640	538
	Vy (KN)	261	266	233	233	123

4. DISPLACEMENT BASED SEISMIC DESIGN INTEGRATING IBC AND AASHTO

The modal response spectrum analysis was conducted for the mixed structural model in Figure 3. The design response spectrum, structural dimensions, and section properties were introduced in previous sections. The displacement demand at the bridge girder bearing location was calculated from this linear response spectrum analysis. The station entrance frame has to provide enough displacement capacity to meet the displacement demand. The displacement capacity was based on the nonlinear push-over analysis of the structural model. The nonlinearity is limited to the predefined plastic hinges shown in Figure 3, and the Mander-confined concrete model is used to determine the component strength. The displacement capacity of the structure was determined by ASCE 41 acceptance criteria for the maximum beam and column plastic rotation limits at collapse prevention level. Figure 4 shows the push-over curve in the longitudinal direction.

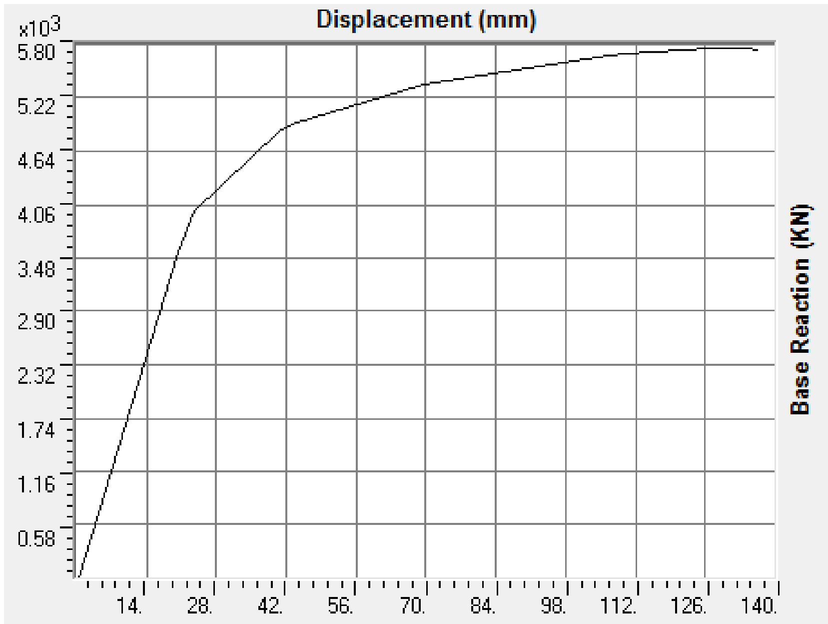


Figure 4. Example of displacement capacity from push-over analysis

Table 4 summarized the displacement demands based on response spectrum analysis, and the displacement capacities, for column 1 to 5 shown in Figure 3. The ASCE 7-05 drift ratio controlled displacement capacity is also shown in the table. The displacement demands in Table 4 are based on combination of 100% principle direction +30% orthogonal directions, and are not adjusted for structural period or any other parameters. The displacement capacity based on push-over analysis used nonlinear analysis based on small displacement assumption, considering P-delta effect, and using one direction acceleration as loading. It should be noted that the push-over capacity is limited by the model, the section properties, and the effect of non-structural elements.

Table 4. Displacement-based seismic demand and capacity

		Column 1	Column 2	Column 3	Column 4	Column 5
IBC (2009)	Demand dx (mm)	25.45	25.09	25.23	24.97	0
	Demand dy (mm)	34.18	34.16	30.80	30.80	14.09
AASHTO LRFD (2010)	Demand dx (mm)	27.64	27.05	27.38	26.95	0
	Demand dy (mm)	35.50	35.47	32.33	32.33	14.38
Capacity Dx (mm) (Push-over)		118.44	116.20	118.24	115.81	139.01
Capacity Dy (mm) (Push-over)		115.55	115.51	116.64	116.62	83.11
Drift limitation controlled capacity: 74.30 (0.015 drift)						

5. CONCLUSION AND FUTURE WORK

This paper introduced how to incorporate both building and bridge seismic design codes in designing a mixed structure. By comparing the codes, it can be seen that force-based seismic design resulted in very different demands. The designer may choose more conservative numbers (larger force demands) to design the structure. This decision appears to be conservative. However, it will significantly increase the demand for other capacity-protected linear members. If not appropriately detailed, the earthquake resistance mechanism might not be achieved as originally intended, resulting in poor structural performance.

The displacement-based approach circumvents the forces and the modification factors, but focused on the nonlinearity of the components that are designed to act as fuses. This will reduce the demands for other elastic members. It is worth to note that the conclusions are based on ignoring the different detailing requirements from the building and bridge design codes, which may contribute to the different parameters in the building and bridge design codes.

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