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Full Length Research Paper

Comparison of high speed railway bridge substructures

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ABSTRACT

This paper studies an analysis and design of substructural bridge by using AASHTO code, in comparison with another engineering program, SAP 2000 software program, and the results obtained from the determination of the moment and shear using both methods. The difference in calculating moment in the two methods showed SAP program 30% more than AASHTO code. While in calculating of shear using both methods, it was observed that the difference was lesser, about 0.5%, so the design in the SAP program has no assurance of the outcome of the moment compared with the determination of the shear. The reason for the difference in these results is that SAP program relies on finite element theory which depends on dividing model into mini elements and this gives more accurate and greater results than AASHTO code; because AASHTO code calculates the data for whole member without dividing it into mini parts.

Keywords: Bridge, substructures, AASHTO code, SAP 2000 software program.

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INTRODUCTION

Bridge structures crossing navigable waterways must not only be designed to resist gravity, wind, and earthquake loads, but must also be capable of resisting ship and barge collision loads. Design specifications used both in the US and internationally employ empirical models of vessel crush behavior to produce codified procedures for computing equivalent static design loads due to vessel impact.

Ben C. Gerwick, Inc. developed the design and design requirements for the pier protection system for the main piers of the New Sidney Lanier Bridge. The pier protection system consists of two rubble-mound islands placed around the central bridge piers. The final design maintains a 60 ft clearance from the rectangular pier footings to the top of slope. The side slopes of the islands are protected with armor stone, slope at 2:1 to the river.

AASHTO-LRFD (2007) code design criteria addressing consideration shall be given to safe passage of vehicles on or under a bridge. The hazard to errant vehicles within the clear zone should be minimized by locating obstacles at a safe distance from the travel lanes.

Kim (2013) investigates the design comparison of totally prefabricated bridge substructure system. A joint element is used in order to predict the inelastic behaviors of segmental joints. This study documents the design comparison of totally prefabricated bridge substructure and presents conclusions and design recommendations based on the analytical findings.

Yashavant and Sangita (2013) discusses the comparative analysis of two standards namely AASHTO and IRC followed in construction of bridge superstructures subjected to load of heavy vehicles. The results of bending moment and stress for self-weight and superimposed weight are the same, but are different for the moving load consideration.

METHODOLOGY

AASHTO Code was used in this paper to design and analyse the substructure of the bridge and the parameters used in the analysis and design as shown subsequently.

Seismic

Seismic performance zones

The solution to the seismic response of bridge substructure was obtained by numerical integration of the nonlinear equations of motion using the Hilber–Hughes–Taylor (HHT) algorithm (Hilber et al., 1977; Hughes, 1987).

Each bridge shall be assigned to one of the four seismic zones in as shown in Table 1.

Seismic Zone 1: For bridges on sites in Zone 1 where the acceleration coefficient is less than 0.025 and the soil profile is either Type I or Type II, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake. For all other sites in Zone I, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.2 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake. For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed beatings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment. If each bearing supporting an uninterrupted segment or simply-supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing. Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the beating. For all bridges in Seismic Zone 1 and all single span bridges, these seismic shear forces shall not be less than the connection force specified here in.

Seismic Zone 2: Structures in Seismic Zone 2 shall be analyzed according to the minimum requirements specified in AASHTO code, except for foundations, seismic design forces for all components, including pile bents and retaining walls, shall be determined by dividing the elastic seismic forces, obtained from Article 3.10.8, by the appropriate response modification factor, R, (AASHTO, 2007). Seismic design forces for foundations, other than pile bents and retaining walls, shall be determined by dividing elastic seismic forces, obtained from AASHTO (2007), by half of the response modification factor, R, for the substructure component to which it is attached. The value of RI2 shall not be taken as less than 1.0. Where a group load other than extreme event I, governs the design of columns, the possibility that seismic forces transferred to the foundations may be larger than those calculated using the procedure specified above, due to possible over strength of the columns, shall be considered.

Seismic Zones 3 and 4: Structures in Seismic Zones 3 and 4 shall be analyzed according to the minimum requirements specified in AASHTO code for all components of a column, column bent and its foundation and connections (Xiao et al., 2012; Yamashita and Sanders, 2009).

Table 1. Acceleration coefficient for seismic zones.

Acceleration coefficient	Seismic zone
$A \leq 0.09$	1
$0.09 < A \leq 0.19$	2
$0.09 < A \leq 0.29$	3
$0.29 < A$	4

Table 2. Site coefficient (AASHTO, 2007).

Site coefficient	Soil profile type			
	I	II	III	IV
S	1.0	1.2	1.5	2.0

Site effects

Site effects shall be included in the determination of seismic loads for bridges. The site coefficient, S, is specified in Table 2.

In locations where the soil properties are not known in sufficient detail to determine the soil profile type, or where the profile does not fit any of the four types, the site coefficient for Soil Profile Type II shall be used.

Soil profile type I

A profile shall be taken as Type I is composed of:

i) Rock of any description, either shale-like or crystalline in nature, or
ii) Stiff soils where the soil depth is less than 60,000 mm, and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.

Soil profile type II

A profile with stiff cohesive or deep cohesion-less soils where the soil depth exceeds 60,000 mm and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays shall be taken as Type II.

Soil profile type III

A profile with soft to medium-stiff clays and sands characterized by 9000 mm or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesion-less soils shall be taken as Type III.

Soil profile type IV

A profile with soft clays or silts greater than 12000 mm in depth shall be taken as Type IV.

Bending moment

Bending moment's equations according to AASHTO code is as follows:

$$Mr = \phi Mn \quad (1)$$

Where
Mr = The factored flexural resistance
ϕ = flexural resistance factor
Mn = nominal resistance (kN.m)

$$Mn = Asfy(ds - a/2) \quad (2)$$

Where
As = steel area cm^2
fy = yield strength of steel, kN/m^2
ds = the corresponding effective depth from the extreme fiber to the centroid of the tensile force in the tensile reinforcement. In (cm)
a = depth of equivalent rectangular stress block (cm)

Shear

Equation of shear according to AASHTO code is as follows:

$$Vr = \varphi Vn \quad (3)$$

Where
Vn = the nominal shear resistance (kN)
Vr = the factored shear resistance (kN)
ϕ = 0.9, shear resistance factor

$$Vn = 0.25f'cbvdv + Vp \quad (4)$$

Vc = shear resistance due to concrete (kN)
= $0.083\beta\sqrt{f'_c}\, bv\, dv$ (AASHTO, 2007)

Where:
bv = effective web width taken as the minimum web width within the depth dv (cm.)
dv = effective shear depth (cm). It is the distance measured perpendicular to the neutral axis between the resultants of the tensile and compressive force due to flexure.

β = factor indicating ability of diagonally cracked concrete to transmit tension for non pre stressed sections, β may be taken as 2.0

SAP 2000 14 program

The SAP name has been synonymous with state-of-the-art analytical methods since its introduction over 30 years ago. SAP 2000 follows the same tradition featuring a very sophisticated, intuitive and versatile user interface powered by an unmatched analysis engine and design tools for engineers working on transportation, industrial, public works, sports, and other facilities. From its 3D object based graphical modeling environment to the wide variety of analysis and design options completely integrated across one powerful user interface, SAP 2000 has proven to be the most integrated, productive and practical general purpose structural program on the market today. This intuitive interface allows to one create structural models rapidly and intuitively without long learning curve delays. Now one can harness the power of SAP 2000 for all analysis and design tasks, including small day-to-day problems. Complex models can be generated and meshed with powerful built in templates. Integrated design code features can automatically generate wind, wave, bridge, and seismic loads with comprehensive automatic steel and concrete design code checks per US, Canadian and international design standard.

RESULTS

Bending moment and shear

Seismic loading

Bending moment involves the moment in pier-cap and column by using AASHTO code and the calculations are shown in Table 3.

Dead and live loads by AASHTO code and SAP program

The results of AASHTO code of moment and shear are shown in Table 4.

Resistance of structural members

The results of factored moment and shear are shown in Table 5.

Steel reinforcement

Steel reinforcement for pier cap column is shown in Table 6.

Table 3. Moment and shear due to seismic loading.

Seismic zone	Load case	Moment			Force	
		M_x (*kN.m*)	M_y (*kN.m*)	M (*kN.m*)	V_x (*kN*)	V_y (*kN*)
Zone I	Case 1	242.4	6452.88	6457.43	60.6	537.87
	Case 2	808	1935.864	2097.72	202	161.322
Zone II	Case 1	393.36	10487.4	10494.77	98.34	873.95

Table 4. Moment and shear due to applied dead and live loads.

AASHTO code	
Mu (kN.m)	Vu (kN)
10494.77	14347.936

Table 5. Factored moment and shear.

φMn (kN.m)	φVn (kN)
13819.781	28363.5

Table 6. Steel reinforcement for pier cap column.

Structural member	As, long direction	As, short direction	Av
Pier cap	#25@130 mm #25@200 mm	13 #25	#13@170 mm
Column	#36@100 mm	1400 mm^2	859.5 cm^2

Moment and shear from SAP program

The results of moment and shear calculated by SAP program of the model shown in Figure 1 are as shown in Table 7.

The moment diagram is shown in Figure 2. The deflection is shown in Figure 3. The shear, moment diagram and deflection curve are shown in Figure 4.

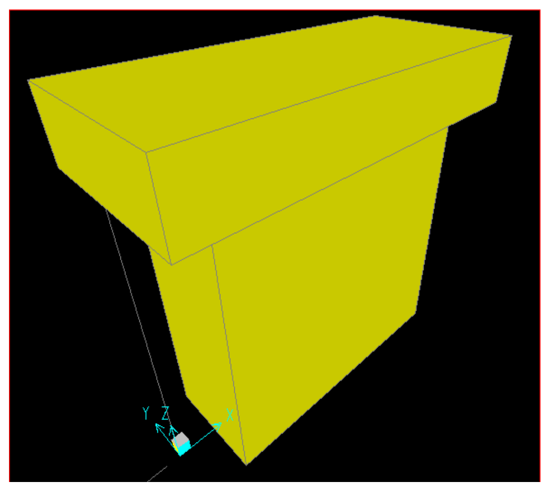

Figure 1. Pier – column model.

Table 7. Moment and shear from SAP program.

SAP	
Mu (kN.m)	Vu (kN)
15751	14425

DISCUSSION

The bridge has unchanging cross section with similar supports and a uniform mass and stiffness and it is high speed railway bridge.

The beam weight is equal to 364.215 KN/m. The AASHTO code take the live load = 84.158 KN/m.

It was found that the SAP analysis provides a more conservative analysis than its counterpart in terms of the bending moment, but the differences were no significant when comparing the shear forces. This difference may be because that SAP program built upon the finite element theory while AASHTO code is depending on the conditions and environmental effects in the US states and special aspects.

CONCLUSIONS

The paper presents the results of a comparison of two different ways to estimate both shear forces and bending moments in a bridge substructure. The first applies the

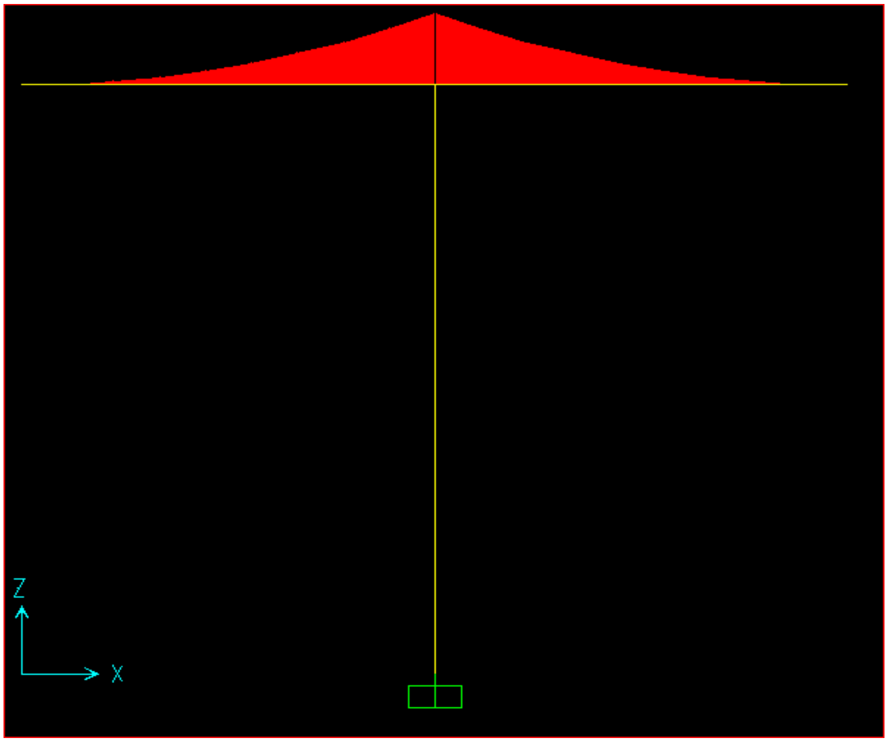

Figure 2. Moment diagram.

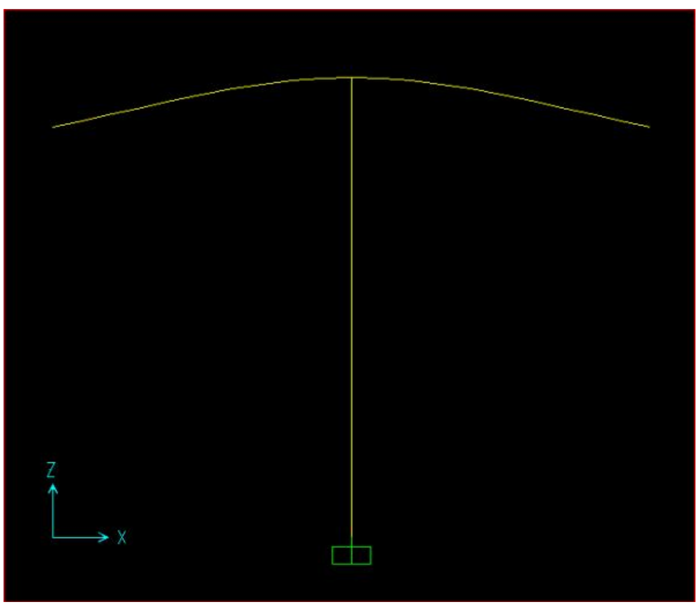

Figure 3. Deformation shape (deflection).

well known AASHTO recommendations, and the second makes use of the SAP 14 program. It was found that the latter provides a more conservative analysis than its counterpart in terms of the bending moment, but the differences were no significant when comparing the shear forces.

Pier cap

Seismic effect

In the AASHTO code we can see that it takes the regard for effect of seismic load y including all of the dead,

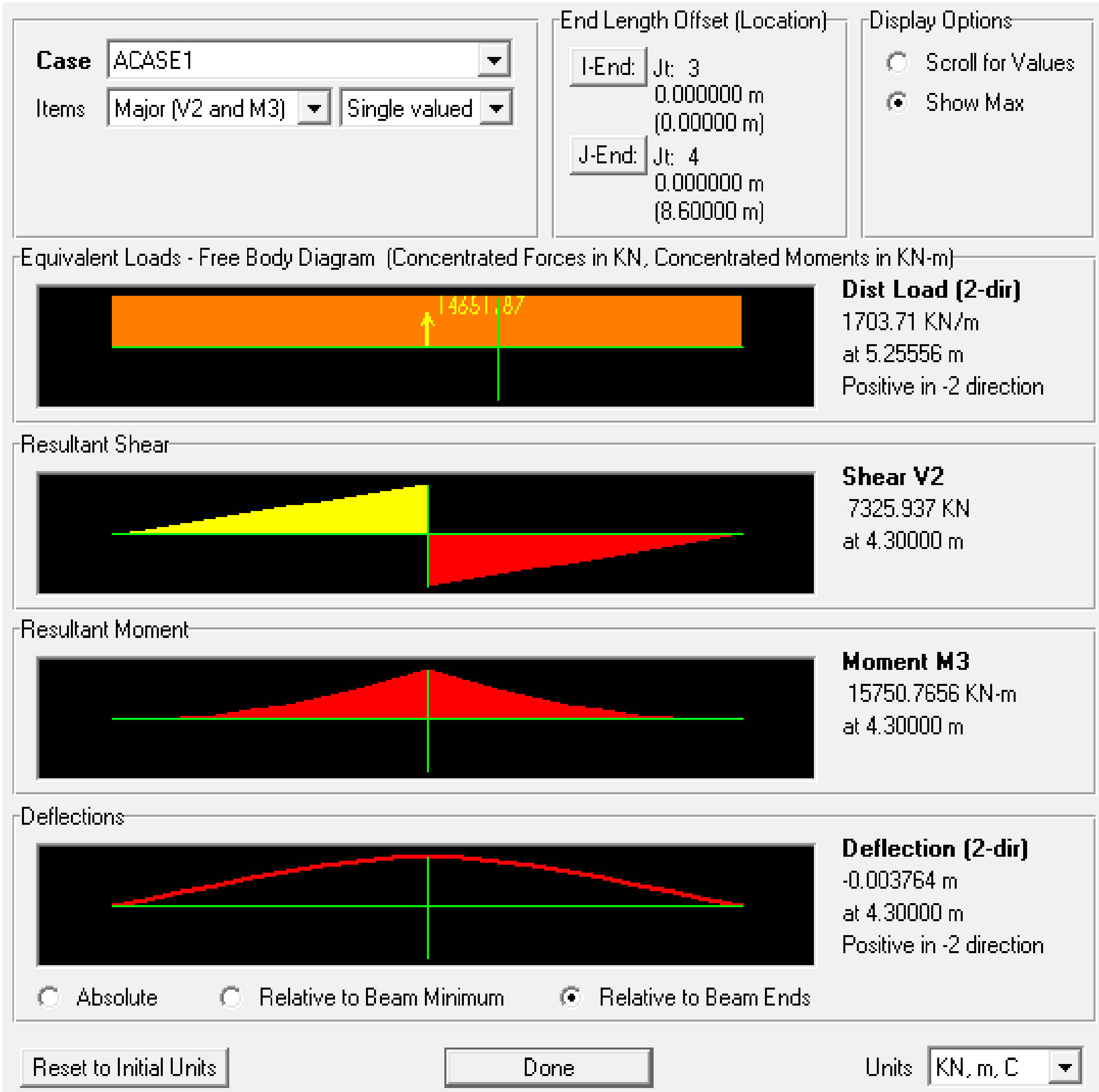

Figure 4. Shear, moment diagram and deflection curve.

buoyancy load, stream flow force, earth pressure, and earth quake effect in its calculation. This code takes 30% as longitudinal motion and 100% as transversal motion effect for this purpose. BS code does not have any calculation for seismic load.

Temperature effect

The AASHTO code has the calculation of the minimum temperature and shrinkage steel reinforcement. It is found to be 10#7 near the surfaces of the concrete.

Pier column

Slenderness effect

The AASHTO code can study the slenderness effect in pier column. This column is un-braced and affected by the motion of the pier cap so this column is slender and the moment that is produced from the slenderness effect must be regarded.

Seismic effect

AASHTO code has the calculation for this effect for both of longitudinal and transversal motion due to earthquake. The pier column need was found to be 60#18@10 cm.

Temperature effect

AASHTO studies the temperature effect on pier column but due to symmetry of bridge superstructure no force is developed at intermediate bent due to temperature expansion/shrinkage of the superstructure.

Skin reinforcement

The AASHTO code has the consideration that the design for the skin reinforcement needed in the pier column must to be distributed at d/2 from the flexural tension reinforcement.

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