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Seismic Assessment of Existing Bridge Using OPENSEES

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Abstract: *The task of this paper is to determine if the response of existing bridge near village Korti in the vicinity of Pandharpur, designed using the Indian Standard Specifications would meet performance requirements when subjected to the moderate seismic hazard. This paper will provide an analysis of designed bridge behavior in Korti near Pandharpur, and determine if this behavior is acceptable for bridges classified as critical or essential. Analysis of this bridge is carried out using software framework OPENSEES.*

Keywords: *bridge, nonlinear analysis,* ***o****pensees, simulation.*

I. INTRODUCTION

A large number of bridges were designed and constructed at a time when bridge codes were insufficient according to current standards. The deficiencies in highway bridges designed prior period result in excessive seismic displacements and large force demands that were substantially underestimated. The existing bridge inventory designed to previous provisions is thus likely to suffer damage when subjected to seismic scenarios comparable to those observed in severe earthquakes.

This performance-based evaluation approach requires bridges to satisfy different performance criteria for different levels of ground motion. For instance, the bridge may suffer minor damage but should be operational under frequent earthquakes with low intensity. Under infrequent earthquakes with large intensity, the bridge should provide an acceptable level of life-safety. Quantifying the level of risk associated with anticipated earthquake scenarios enables taking rational decisions to retrofit, replace or accept the risk.

II. DESCRIPTION AND MODLLING

2.1 Description of two span bridges at Korti

This bridge carries the two lanes of state road from Pandharpur to Satara having span length of 25 m each. The two spans support the 300mm thick concrete deck with four T-shaped concrete girders having total depth of 1.35m. Each of these girders rests upon 500mm X 300mm X 64mm neoprene bearing pads. The coefficient of friction for these bearing pads is 0.3. Two spans are supported by pier 1.22m diameter with 75mm of concrete cover. The columns are reinforced longitudinally with 24 -16mm bars and transversely with 8mm bars uniformly spaced at 250mm from bottom of the hinge zone to top of foundation, and spaced at 150mm inside the hinge zone. The total column length is 12.27 and length above grade is 6.71m. The size of deck is 11.9m in width with thickness of 0.3m supported on four T-girders (Web size 1.35m X 0.3m). Embankment length is 25m and its depth 0.5m with total weight of about 30000KN.

2.2 Nonlinear Fiber Section [1]

The nonlinear Fiber section for the column- Nonlinear beam-column elements with fiber section (Fig.2.1) are used to simulate the column. Forced-based beamcolumn elements (nonlinearBeamColumn, Mazzoni et al. 2009) are used for the column (1 element, number of integration points = 5) as well as the pile shaft below grade (number of integration points = 3).

The Steel02 material in OpenSees (Mazzoni et al. 2009) is employed to simulate the steel bars and Concrete02 material is used for the concrete (core and cover). Steel02 is a uniaxial Giuffré-Menegotto-Pinto material that allows for isotropic strain hardening. Concrete02 is a uniaxial material with linear tension softening. The default values for the material properties of the Fiber section are listed in Table 2 for Steel02 and Table 3 for Concrete02 (core and cover). The Concrete02 material parameters were obtained from the Mander (1988) constitutive relationships for confined and unconfined concrete. More details on the derivation of the default values and the OpenSees uniaxialMaterial definitions used for each material.

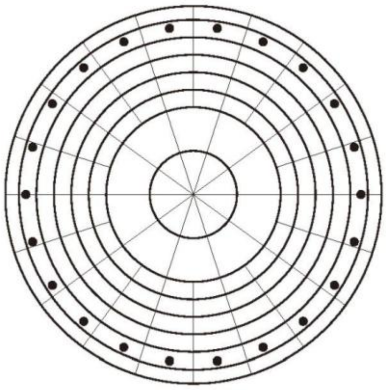

Fig. 2.1 Column fibre section (based on PEER best modelling practices report, Berry and Eberhard, 2007)

Table 4.1 Values for Column Reinforced Concrete (RC) Section Properties
Parameter Value
Longitudinal bar size 16
Longitudinal steel % 2
Transverse bar size 8
Transverse steel % 1.6
Steel unit weight (kN/m3) 77
Steel yield strength (kPa) 460,000
Concrete unit weight (kN/m3) 22.8
Concrete unconfined strength (kPa) 27,600

Table 4.2. Values for Steel02 Material Properties
Parameter Value Typical range
Steel yield strength (kPa) 460,000 345,000-470,000
Young's modulus (MPa) 200,000 -
Strain-hardening ratio* 0.01 0.005-0.025
Controlling parameter R0** 15 10-20
Controlling parameter cR1** 0.925 --
Controlling parameter cR2** 0.15 --

*The strain-hardening ratio is the ratio between the post-yield stiffness and the initial elastic stiffness.
**The constants R0, cR1 and cR2 are parameters to control the transition from elastic to plastic branches.

Table 4.3. Values for Concrete02 Material Properties
Parameter Core Cover
Elastic modulus (MPa) 25,312 25,312
Compressive strength (kPa) -46,457 -27,600
Strain at maximum strength -0.00367 -0.002
Crushing strength (kPa) -44,9790
Strain at crushing strength -0.036 -0.006
Ratio between unloading slope 0.1 0.1
Tensile strength (kPa) 6504 3864
Tensile softening stiffness (kPa) 1,771,820 1,932,000

Table 4. 4. Values for Bridge Deck
Bridge Deck Parameters
Parameter Value
Deck length (m) 50.0
Deck width (m) 11.9
Deck depth (m) 1.35

Table 4.5. Values for Deck Material Properties
Parameter Value
Elastic modulus (MPa) 28,000
Shear modulus (MPa) 11,500
Cross-section area (m2) 5.72
Moment of inertia @ transverse axis (m4) 2.81
Moment of inertia @ vertical axis (m4) 53.9
Weight per unit length (kN/m) 130.3

2.3 Abutment Model-Simplified Model (SDC 2004) [1]

The simplified model of the embankment-abutment system provides several nonlinear springs to better represent abutment-bridge interaction that is neglected with the elastic or roller abutment models. The general scheme of the simplified model is presented in Fig. . It consists of a rigid element of length *dw* (superstructure width), connected through a rigid joint to the superstructure centerline, with defined longitudinal, transverse and vertical nonlinear response at each end.

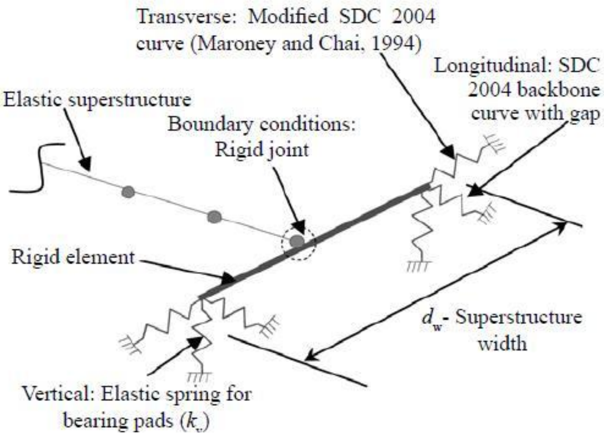

Fig. 2.2 General scheme of the Simplified abutment model [2]

2.4 Specifications of Performance Based Earthquake Engineering Input Motions:

To conduct a PBEE analysis, input motions must be defined. Following ground motions are used for this project.

Table 2.1

Input Motions (10 Records in Total; 1 Records Selected) Display Intensity Measures

Record#	Bin	Motion	#Points	Timestep (Sec)	Duration (Sec)
☑ 1	LMLR	BORREGO/A-ELC	2000	0.0200	40.0000
☑ 2	LMLR	LOMAP/A2E	1998	0.0200	39.9600

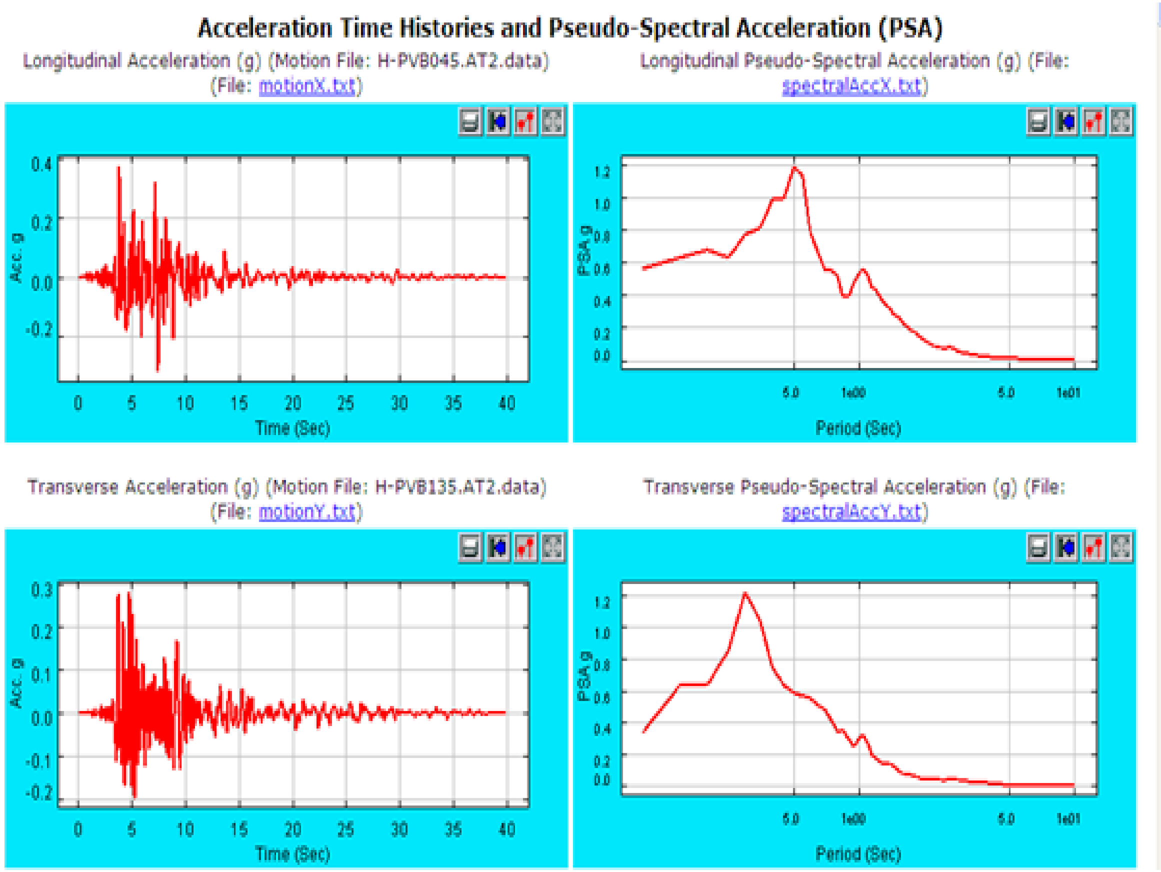

Fig. 2.3 Time histories and response spectra of individual record

III. Results

3.1 SRSS Responses for Each Performance Group

PG1: Max tangential drift ratio SRSS (col)
PG2: Residual tangential drift ratio SRSS (col)
PG3: Max long relative deck-end/abut disp (left)
PG4: Max long relative deck-end/abut disp (right)
PG5: Max absolute bearing disp (left abut)
PG6: Max absolute bearing disp (right abut)
PG7: Residual vertical disp (left abut)
PG8: Residual vertical disp (right abut)
PG9: Residual pile cap disp SRSS (left abut)
PG10: Residual pile cap disp SRSS (right abut)
PG11: Residual pile cap disp SRSS (col)

Table 3.1

Rec.	PG1 (%)	PG2 (%)	PG3 (m)	PG4 (m)	PG5 (m)	PG6 (m)	PG7 (m)	PG8 (m)
1	0.1127	0.00032	0.01018	0.0092	0.0104	0.0104	0.00794	0.00794
2	0.4584	0.00066	0.03344	0.0396	0.0339	0.03397	0.06808	0.00709

PG9 (m)	PG10 (m)	PG11 (m)
0.00025	0.00026	9.4e-6
1.6048	2.1e-04	2.7e-05

3.2 Intensity Measures (Free-field Response)

The intensity measures include:

- PGA (Peak Ground Acceleration)
- PGV (Peak Ground Velocity)
- PGD (Peak Ground Displacement)
- D5-95 (Strong Motion Duration)
- CAV (Cumulative Absolute Velocity)
- Arias Intensity
- SA (Spectral Acceleration; assuming 1 second period)
- SV (Spectral Velocity), SD (Spectral Displacement)
- PSA (Pseudo-spectral Acceleration)
- PSV (Pseudo-spectral Velocity)

The strong motion duration (D5-95) is defined according to the time domain bounded by the 5% and 95% cumulative Arias intensity of the record. All of the spectral intensity measures are defined at an effective viscous damping of 5%.

Table 3.2 Longitudinal direction

Rec.	PGA (g)	PGV (cm/sec)	PGD (cm)	D(5-95) (sec)	CAV (cm/sec)	Arias Bracketed (cm/sec)	SA (g)	SV (cm/sec)	SD (cm)	PSA (g)	PSV (cm/sec)
1	0.139	26.491	12.94	37.7	488.64	20.68	0.182	22.31	4.515	0.1817	28.36
2	0.199	13.756	3.875	37.1	665.82	47.17	0.256	45.45	6.349	0.2556	39.89

Table 3.3 Transverse direction

Rec.	PGA (g)	PGV (cm/sec)	PGD (cm)	D(5-95) (sec)	CAV (cm/sec)	Arias Bracketed (cm/sec)	SA (g)	SV (cm/sec)	SD (cm)	PSA (g)	PSV (cm/sec)
1	0.057	13.194	10.16	37.5	414.87	12.28	0.133	20.46	3.291	0.1325	20.68
2	0.154	11.864	5.605	36.5	555.25	32.70	0.177	26.01	4.402	0.1772	27.65

Table 3.4 Horizontal SRSS

Rec.	PGA (g)	PGV (cm/sec)	PGD (cm)	D(5-95) (sec)	CAV (cm/sec)	Arias Bracketed (cm/sec)	SA (g)	SV (cm/sec)	SD (cm)	PSA (g)	PSV (cm/sec)
1	0.139	26.491	12.94	37.7	716.99	33.02	0.0988	10.49	2.447	0.0985	15.38
2	0.210	17.251	5.713	36.8	961.41	79.89	0.1434	12.01	3.550	0.1429	22.30

IV. Conclusion

This project presents highlights to assess the seismic response of a two span bridge. The focus is on describing the methodology adopted to idealize the bridge and its foundation system, while only summary of results from the extensive elastic and inelastic analyses under the effect of input ground motions are presented. The demands corresponding to the ground motions are well within the collapse limit state and the capacity of bridge components. Under the ground motions, the response of the bridge was acceptable.

The presented assessment study confirmed there is no need to retrofit different bridge components to mitigate potential seismic risk.

Acknowledgements

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