Seismic Analysis of Truss Bridges with Tall Piers

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Abstract—*This research work is concerned with study of seismic analysis of truss bridges with tall piers. A case study has been performed on an existing railway bridge over river Makru in Northeast region of India. The total length of this four span bridge is 495m and piers heights are 35m, 75m, 100m, 100m, 75m and 12m from left to right bank of the river. For analysis three spans with pier height of 75m and 100m are considered. It is general practice to use hollow section for tall bridges. In this bridge also, piers with hollow sections are used. Performance of the existing bridge having four columns in a bent is evaluated under seismic conditions. Tall bridges are very flexible, however, as there are no limits on lateral displacement of bridges, the performance is within acceptance criteria. A comparison between performances of bridges under time history and push over is done.*

1. INTRODUCTION

Bridges are also called as lifeline structures because they serve in case of emergency. During earthquake, bridges play a very important role of evacuation and rescue. Damage to bridges also causes huge economic loses. It is therefore necessary that bridges should be designed in such a way that they continue to serve even during natural calamities like earthquakes. The two modes of transportation on land are roadways and railways. Railway and highway bridges have been utilized most for transportation of passenger and freight segment. All the countries in the world are focusing on expansion and growth in railway because railways can carry large number of passengers, heavy loads to longer distances which lead to substantial reduction in energy consumption and pollution. It is comparatively safer and more comfortable to passengers. Road transport is more flexible because its route and timing can be adjusted to suit individual’s requirement. Also capital investment in roadways is less than railways. Development of land based transportation is also important as they are instrumental in providing internal security and making efficient arrangements of defense of country against any external threat.

During Earthquakes, large amount of energy is transferred from ground to the structure. Bridge can sustain greater earthquakes if it can dissipate more energy. In case of bridges, piers are designed to bear the damage unlike buildings where strong column weak beam philosophy is adopted. Energy dissipation can take place elastically, through formation of plastic hinges in abutment and pier or only piers but it is required that inelastic actions should be at accessible locations and should not cause failure under gravity loads.

However for energy dissipation failure should take place in ductile mode so using capacity design method, brittle mode of failure is avoided. Behavior of pier during earthquakes depends upon type of bridge which determines type of forces coming on pier, height of pier, arrangement of piers in a bent and pier section. If the pier height varies significantly along longitudinal direction, stiffness of each span changes drastically and bridge becomes irregular even when pier height in a span is greater than 30m, bridge is termed as irregular and simple analysis methods given in codes are applicable for regular bridges only.

The need of the study is as follows:

- Bridge may require tall piers to cross deep valleys and high mountainous regions.
- Tall piers lead to very high bending moment and seismic displacement demands.
- Tall piers are more vulnerable to seismic damage so proper knowledge about its behavior under seismic conditions is necessary.
- In India, scope of seismic design methodology give in codes is limited as it does not give detailed analysis provisions for bridges with pier height greater than 30 m. Performance of tall pier bridge is governed by higher modes.
- P-Δ effect becomes significant in case of tall piers. Hollow sections are generally used in tall piers.
- Knowledge about behavior of hollow sections under seismic conditions is limited.

2. PARAMETRIC STUDY

An existing Railway bridge is considered for study. It has simply supported truss superstructure. It crosses river and soil profile is such that it has got shorter pier at both the ends and taller piers in the middle. At middle span, bridge has piers with height 100m. According to IITK-RDSO Guidelines, 2010 [1], as pier height is greater than 30m, this bridge is a case requiring special studies and analysis. In the present study, existing bridge has been modeled and its performance is evaluated. In order to study the effect of response reduction factors, bridge piers are designed and analyzed for different values of response reduction factors. Along with pier height 100m study is carried out with pier height of 75m also. In order to compare performance of a bridge under linear and non linear analysis is performed on SAP. Thus this section includes a parametric study of performance of bridges with different pier heights and cross sections, designed with different values of response reduction factors.

Description of the Bridge Considered for Study:

This bridge is an existing Northeast frontier Railway Bridge over river Makru in Northeast region of India. The total length of this four span bridge is 495m with four spans of 106m each and one span of 75m. Piers height is 35m, 75m, 100m, 100m, 48 and 12m from left to right bank of the river. This study has been focused on middle span where pier height is 100m. Fig. 1 shows elevation of bridge. The Superstructure is steel truss simply supported on piers. M40 grade concrete is used for pier and foundation and Fe250 structural steel for superstructure. Fe500 grade steel is used for reinforcing bars. Subsurface conditions below middle span consist of shale up to 40 m according to the geotechnical investigation report available for the site.

Bridge Modeling

Only middle three spans of bridge having four piers of 75m and 100m are modeled and the mass of adjacent span is lumped onto piers to simulate the behavior of existing bridge. Full dead load of superstructure in longitudinal direction is lumped on pier with hinge and half of dead load is lumped on both the piers in transverse and vertical direction. Seismic mass is contributed by full dead load and half live load in transverse and vertical direction. Seismic mass contribution of live load in longitudinal direction is taken as zero according to IITK RDSO guidelines (2010).

Superstructure has been modeled using finite elements truss members with moment restraints. Elements are capable of transmitting only axial loads. Superstructure is assumed to remain elastic during earthquake. Superstructure has been connected to substructure using two types of links. Hinge link on one end and roller link on other end to simulate simply supported conditions. Hinge link allows rotation but no translation and roller link allows rotation as well as translation in longitudinal direction so as to allow free movement of superstructure in longitudinal direction. Both the hinges restrain the translation in transverse direction. Substructure consists of pier cap and pier. Pier cap is modeled using rigid element. Pier is modeled using finite element column beam element (Fig. 2). Each pier is divided into elements of 4-5m length each.

It is general practice to use hollow sections for tall piers considering economy also it has been seen in literature review that in case of tall pier it is necessary to subdivide pier into smaller elements to simulate its behavior more realistically (Zhong. L et al, 2008) [5]. In bridge model, Piers have been modeled using hollow sections in section designer (SD) tool of SAP 2000. In this chapter, the design, results and discussion are provided assuming bridge pier to be fixed at base.

Non linear behavior of column is modeled using lumped plastic P-M2 and P-M3 hinges to represent non linearity in Y and X axis respectively. In case of simply supported bridge with single column, horizontal component of earthquake does not cause any axial load in column. When vertical component of earthquake has not been taken into account, the only axial load in column is due to gravity which is known to us during design. So corresponding to gravity axial load on a section which remains constant during earthquake, plastic moment capacity can be found. These hinges represent non linear behavior of column and they are provided at the end of each element because it is difficult to predict location of formation of hinge in case of tall pier (Zhongguo Guan et al., 2011).

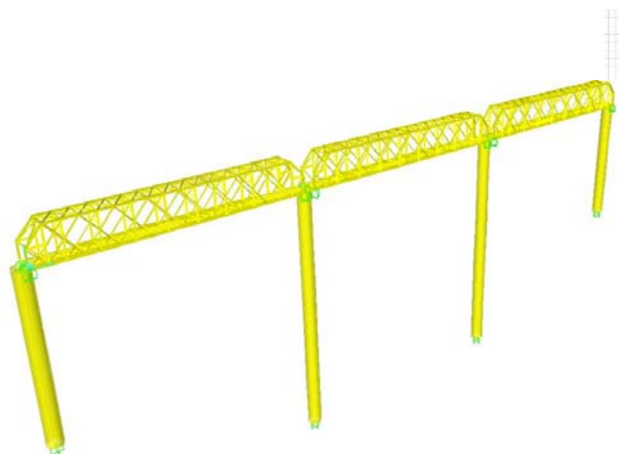

Fig. 1: Snapshot of the Bridge Model from SAP2000

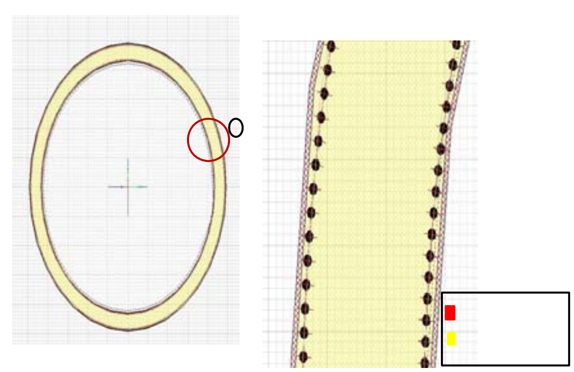

Fig. 2: Model of Pier in Sectional Designer
(a) Pier Section and (b) Zoomed Section

Bridge lies in zone V having seismic zone factor 0.36. The bridge is considered to be important so according to IITK-RDSO Guidelines (2010) importance factor of 1.5 is used. It was observed that for 100m high pier minimum reinforcement is required even for response reduction value of 1.5 that means strength does not govern the design. Response reduction factor given in IITKRDSO Guidelines is 2.5 so to carry out further study a higher value of seismic zone factor was considered.

According to study of Sitharam and Sil (2014) PGA in some locations in Tripura which is located at 100km from our bridge site is as high as 0.36 as obtained by deterministic seismic hazard analysis. Zone factor of 0.36 is assumed and analysis is carried out for higher values of response reduction factors. As the seismic design becomes more critical with higher value of zone factor, results for analysis with zone factor 0.26 are provided here [2-4].

For 100m tall pier bridge, even after considering higher value of zone factor, minimum reinforcement governed design at response reduction factor value of 2.5. Bridge becomes very flexible at higher values of response reduction factor. Size of pier cannot be reduced because period of vibration becomes very high.

To carry out further study, hypothetical section has been assumed in which stiffness is kept constant so that period of vibration does not increase but strength is modified to achieve the required reduction factor.

3. RESULTS

Some of the results are presented here after performing the analysis of the model in the software (table 1-4, Fig. 3- 13).

Table 1: Modal analysis results

MODES	TIME PERIOD
1	2.350 sec
2	2.168 sec
3	1.157 sec
4	0.590 sec
5	0.556 sec
6	0.549 sec
7	0.544 sec
8	0.526 sec

Response Spectrum Analysis Results:

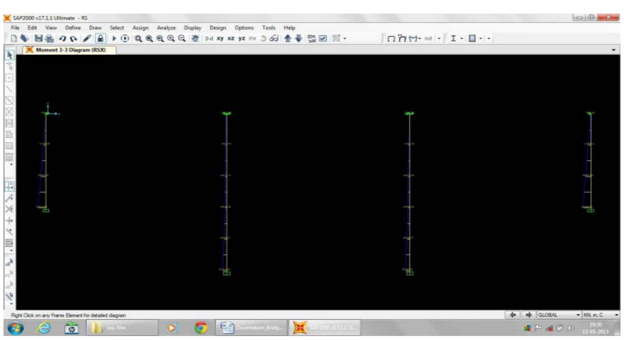

Fig. 3: Bending Moment Envelope Diagram in X-Direction

Table 2: Response Spectrum in X-direction

Pier height (m)	Bending moment (kN-m)
100	60521.3
75	107618.8

Table 3: Response Spectrum in Y-direction

Pier height (m)	Bending moment (kN-m)
100	59768
75	105254.871

Push Over Analysis

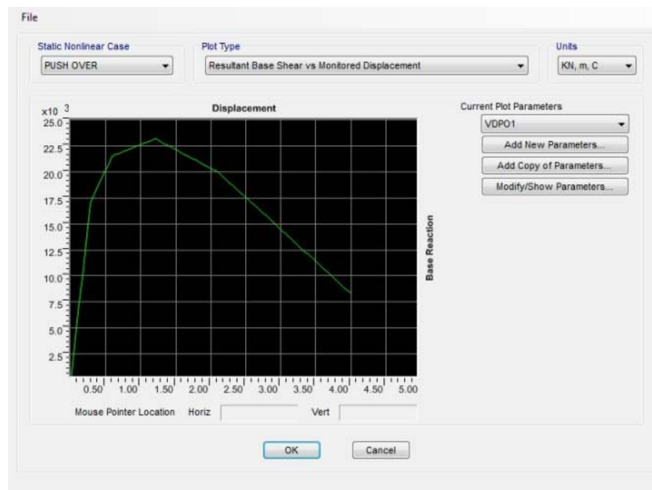

Fig. 4: Push Over Curve in X-direction

Ultimate Displacement = 1.2m

Yield Displacement = 0.38m

Ductility = Ultimate Displacement / yield displacement

Ductility = 1.2/0.38

= 3.16

Base Shear = 23160.535 KN

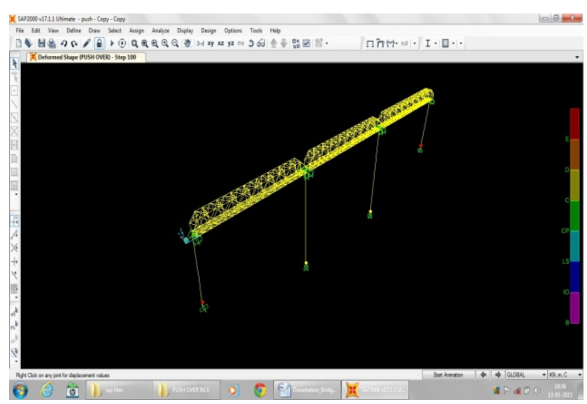

Fig. 5: Hinge Formation

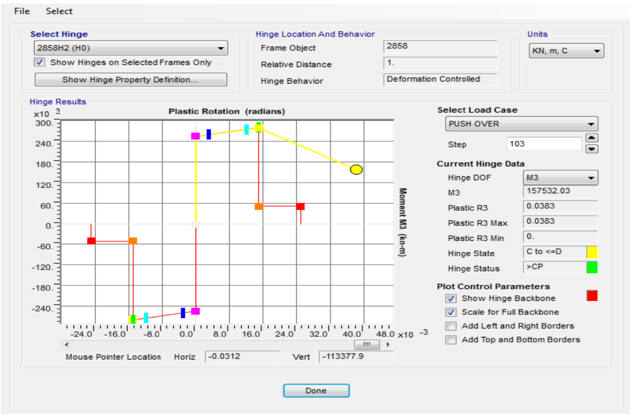

Fig. 6: Hinge Results

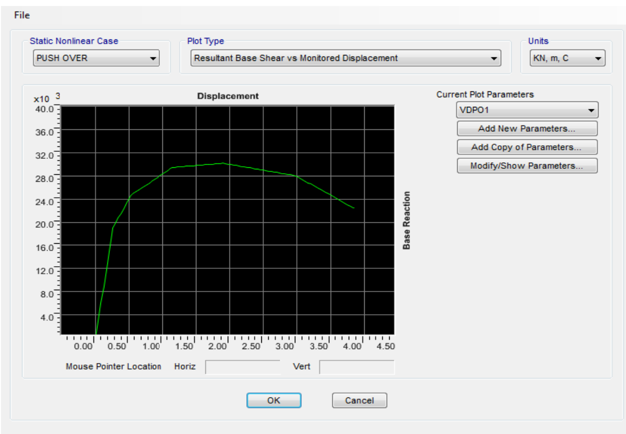

Fig. 7: Push Over Curve in Y-direction

Ultimate Displacement = 1.900m

Yield Displacement = 0.345m

Ductility = Ultimate Displacement / yield displacement

Ductility = 1.9/0.345

= 5.51

Base Shear = 6796 KN

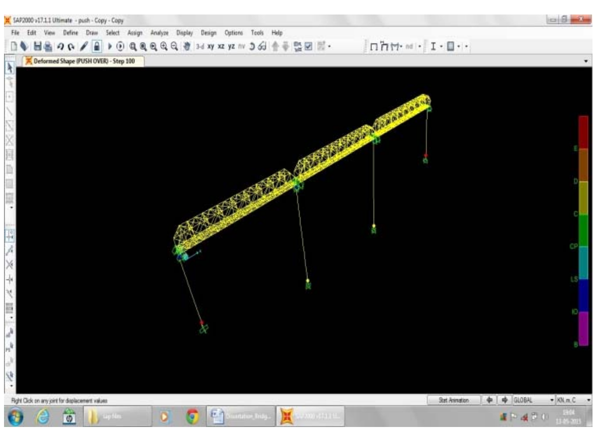

Fig. 8: Last Step Hinges

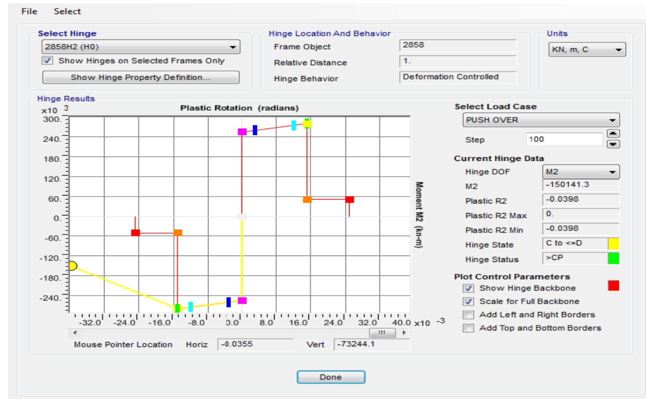

Fig. 9: Hinge Results

Time History Analysis Results

IN X-DIRECTION

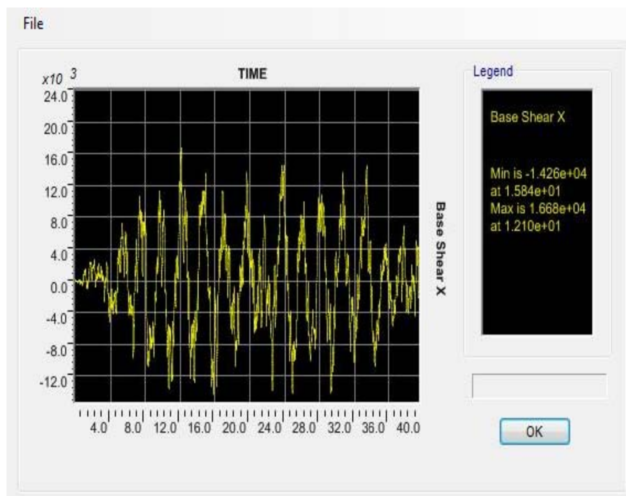

Fig. 10: Base Shear

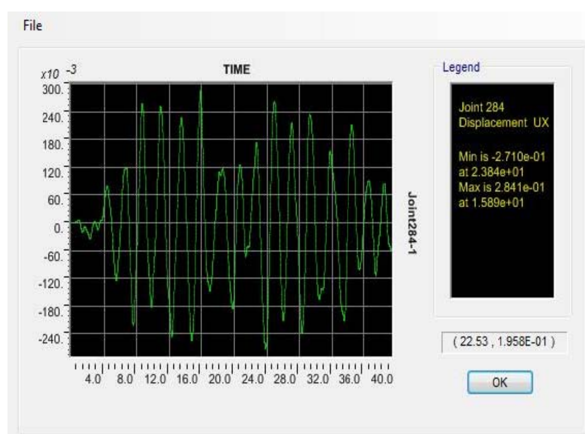

Fig. 11: Top Joint Displacement

4. IN Y DIRECTION

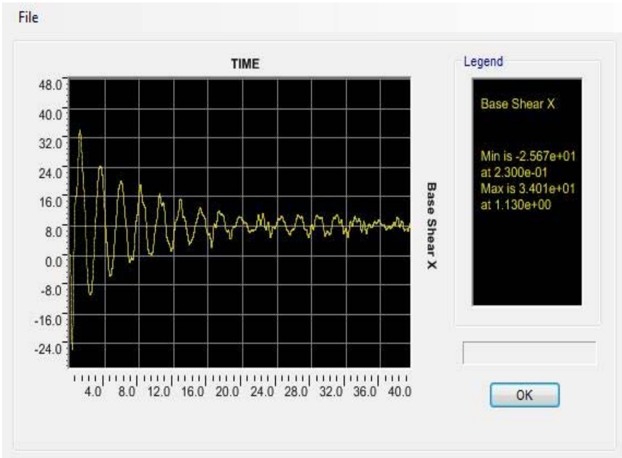

Fig. 12: Base Shear

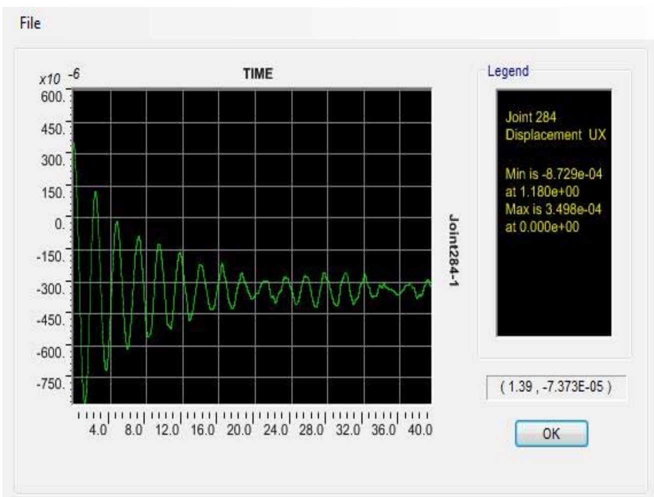

Fig. 13: Top Joint Displacement

Table 4: Comparison of Results

	PUSH OVER	TIME HISTORY
BASE SHEAR		
X	23160.54 kN	16,680 kN
Y	6796 kN	2645 kN
TOP DISPLACEMENT		
X	0.380 m	0.284 m
Y	0.345 m	0.184 m

5. CONCLUSIONS

- Response Spectrum analysis was performed and the bending moment values in x and y directions are found to be similar.
- Push over analysis was performed in both X and Y direction and in both the cases formation of hinges took place at the bottom of the pier which is the required condition.
- Time history analysis was also performed for all the bridge models.
- Displacements obtained for all bridge models were very close to that obtained from pushover analysis.
- No hinges are formed in time history analysis.

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