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Lateral strength and ductility of piers of Bahaddarhat overpass in Chittagong, Bangladesh

A. K. M. T. Alam Khan[1], Hafizul Alim[2] and M. A. Rahman Bhuiyan[3]

[1]*Institute of Earthquake Engineering Research, Chittagong University of Engineering and Technology Chittagong 4349, Bangladesh*
[2]*Department of Civil Engineering, Lamar University, USA*
[3]*Department of Civil Engineering, Chittagong University of Engineering and Technology Chittagong 4349, Bangladesh*

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Abstract

This study is mainly concentrated towards the assessment of lateral strength and ductility of Bahaddarhat overpass located in Chittagong, Bangladesh. In this regard, the ductility method of analysis suggested by Japan Road Association (JRA) is employed to analytically evaluate the lateral strength and ductility of piers of the overpass considering the different modes of failure. The lateral strengths in bending are obtained using the results of nonlinear sectional analyses of the pier sections, while the shear strength of the piers are estimated using JRA defined equation taking into account the effect of depth, volumetric ratio of lateral steel, crushing strength of concrete, yield strength of steel. The fibre model with conventional constitutive models for concrete and steel for the pier sections at critical locations is developed to obtain the moment-curvature relationships. The nonlinear pushover analyses of the piers are carried out to obtain force-displacement relationships. The material nonlinearity is considered in the sectional analysis whereas; both material and geometric nonlinearity are considered in the pushover analysis. The lateral seismic force, allowable lateral force, yield displacement, ultimate displacement and displacement ductility are obtained from force-displacement relationships of the piers. Lateral strength and allowable displacement ductility are presented in tabular form. Finally, the seismic safety of piers of the overpass has been evaluated using the ductility method for design earthquake ground motion records.

Keywords: Overpass, Lateral Strength; Ductility; Pushover Analysis; Moment-Curvature Relationship; Force-Displacement Relationship

1. Introduction

Bridge, in general, is a structure that crosses over a body of water, traffic or other obstructions permitting smooth and safe passage of vehicles. There are several forms of bridges which are widely used in transportation systems. Overpass is an elevated structure carrying highway over roads, railways and other features. A number of overpasses are being constructed in Dhaka and Chittagong metropolitan cities with a view to reducing the traffic congestions. One of these is the Bahaddarhat overpass located in Chittagong and has been constructed in 2013. This kind of structure plays very important role for evacuation and emergency routes for rescues, first aid, medical services, fire-fighting and transporting urgent disaster commodities. In view of importance of life lines in transportation network, it is the key issue to minimize as much as possible loss of the bridge functions during earthquakes. In the last few earthquakes, for instance, the Kobe earthquake in 1995, the Northridge earthquake in 1994, the Chi-Chi earthquake in 1999, and the Chile and Haiti earthquakes in 2010 have demonstrated that a number of highway bridges have collapsed or have been severely damaged, even though they were subjected to earthquake ground shaking of an intensity that has been frequently less than the current code intensities (Alim, 2014).

Bangladesh lies within a seismically active zone. Due to the country's position adjacent to the very active Himalayan front in the north and Burma deformation front in the east expose it to strong shaking from a variety of earthquake sources that can produce tremors of magnitude 8 or greater. It is reported that the potential for magnitude 8 or greater earthquakes on the nearby Himalayan and Burmese fronts is very high (Akhter 2010). Hence, it is necessary to predict the probable losses due to future earthquakes, to assess the seismic safety, plan for seismic retrofitting, pre-earthquake and disaster mitigating plan. One of the ways to assess probable losses under an earthquake required is to investigate the seismic vulnerability of structures. Seismic vulnerability can be assessed in two ways: empirically and analytically. Empirical vulnerability analyses are virtually impossible for Bangladesh, since structural damage data due to earthquakes are not available. Hence, analytical vulnerability analysis is an effective way to have been employed for evaluating vulnerability of bridge structures. Several seismic codes and standards, such as JRA, 2002; CalTrans, 1999; Euro Code, 1998; ASHTO, 1998, have been developed to evaluate seismic safety of bridge structures. The main philosophy lied in seismic safety evaluation that the structures shall resist earthquakes of small to moderate magnitudes without damage while for the large magnitude earthquake excitations the reparability and no collapse condition of the structures shall be ensured. In this case, the structures are allowed to undergo large deformations showing nonlinear behavior and energy dissipation for minimizing the losses.

On the basis of the background, the study aims at obtaining the lateral strength and allowable ductility of piers of the Bahaddarhat overpass. In this regard, the nonlinear static analysis method has been used to evaluate the lateral load and deformation characteristics of piers. The lateral strengths and ductility of piers are obtained by considering their failure modes, bending and shear strengths. The bending strengths are obtained from sectional analysis results, while the shear strengths are estimated by using code defined equations. Finally, the seismic safety of piers of the overpass has been evaluated using the ductility method as recommended in JRA (2002) for design earthquake ground motion records.

2. Modeling of the Bridge

2.1 Physical model

Chittagong city is surrounded by many primary and secondary road networks. In Chittagong Metropolitan Master Plan, there is a guideline for improvement of traffic network to reduce the traffic congestion within the city. A 1331.60 m long overpass connecting CDA

(Chittagong Development Authority) Avenue road and Shah Amanat Bridge approach road has been constructed to reduce traffic congestion at the Bahaddarhat junction. There are 25 spans of variable lengths excluding the two approach roads at both ends of the overpass. The span length of the overpass varies from 35 m to 42 m. The length of each approach road located at the ends of the overpass is 165.3 m. The deck of the overpass comprises six to seven pre-stressed concrete girders with 200 mm reinforced concrete slab including asphalt wearing course. The girders rest on elastomeric rubber pad installed on top of each pier and abutment. There are 24 piers having variable heights ranging from 3.65 m to 7.29 m and two abutments at its ends. The 3-D view of the Bahaddarhat overpass and the geometric dimensions of deck, piers and re-bar details of piers are presented in Fig.1 and Fig.2 respectively with Table 1. Relevant material properties of the target overpass are presented in Table 2. The sectional elevation of the overpass is presented in Fig. 2.

Fig. 1. 3-D view of Bahaddarhat overpass (Photo Courtesy: Chittagong development Authority)

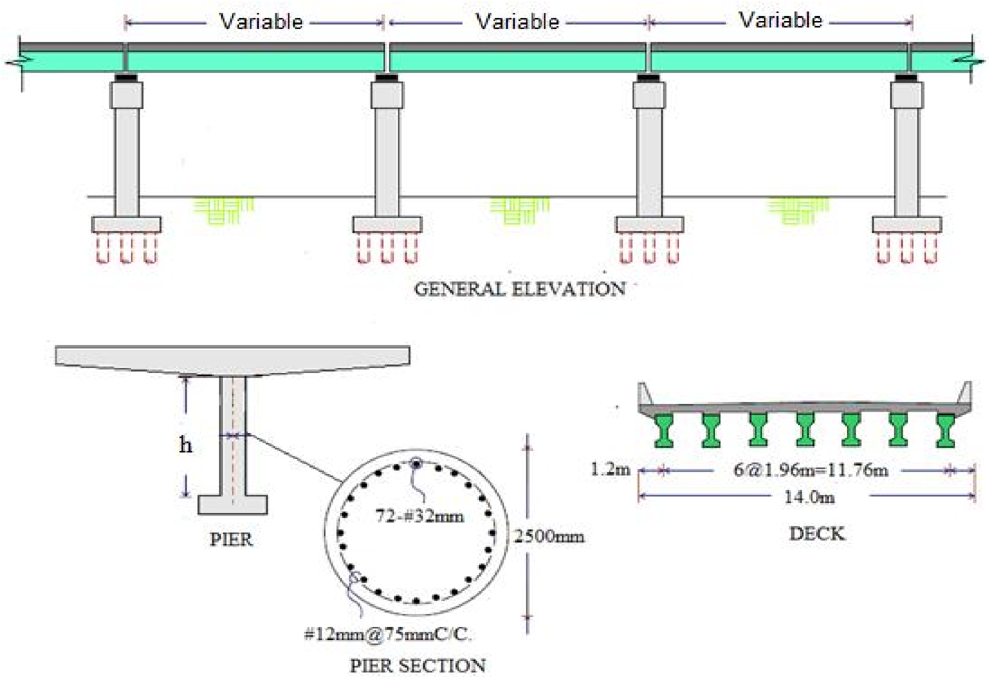


Fig. 2. Geometrical model of Bahaddarhat overpass

2.2 Analytical model of the bridge

The analytical model of a tributary deck along with a pier (pier-girder system) is shown in Fig.3. This simplification holds true only when the bridge superstructure is assumed to be rigid in its own plane which shows no significant structural effects on the seismic performance of the bridge system when subjected to earthquake ground acceleration in longitudinal direction (Ghobarah *et at.*, 1988). The pier-girder system is approximated as a continuous 2-D finite element frame using the numerically solved nonlinear analysis program. Finite element model with frame elements is used to estimate the pier-girder system with a finite number of degrees of freedom. The superstructure & substructure of the system are modelled as a lumped mass system divided into a number of small discrete segments. The mass of each segment is assumed to be distributed between two adjacent nodes (Alim, 2014).

Table 1: Geometric Dimensions

Pier No.	Pier Height (m)	Dia. (m)	Longitudinal Reinforcement
1	3.653	2.5	72 @ D32
2	4.917	2.5	72 @ D32
3	5.857	2.5	72 @ D32
4	6.557	2.5	72 @ D32
5 to 21	7.29	2.5	72 @ D32
22	7.057	2.5	72 @ D32
23	5.417	2.5	72 @ D32
24	4.153	2.5	72 @ D32

Table 2: Material Properties

Material Name	Description of Material Properties
Confined Concrete	Compressive strength = 35 MPa Tensile strength = 3 MPa Strain at Peak Stress = 0.0025 (mm/mm) Confinement factor = 1.2 Specific weight = 2.4 E-05 N/mm3
Steel	Modulus of elasticity = 200000 MPa Yield strength = 485 MPa Strain hardening parameter = 0.0075 Fracture/ buckling strain = 0.06 Specific weight = 7.8 E-05 N/mm3
Unconfined Concrete	Compressive strength = 35 MPa Tensile strength = 3 MPa Strain at Peak Stress = 0.0025 (mm/mm) Confinement factor = 1.0 Specific weight = 2.4 E-05 N/mm3

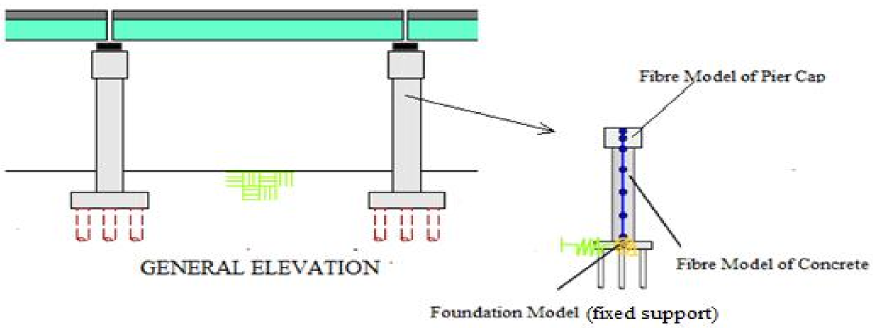


Fig. 3. Analytical model of the overpass pier

The superstructure consisting of RC decks and post-tensioned prestressed concrete girders is modelled using linear beam-column elements so that the superstructure remains elastic under the seismic loads applied in the longitudinal direction (Ghobarah *et at.*, 1988). The body of the bridge pier is modelled using the fibre elements. Each fibre has a stress–strain relationship, which can be specified to represent unconfined concrete, confined concrete, and longitudinal steel reinforcement. The confinement effect of the concrete section is considered on the basis of reinforcement detailing (Alim, 2014). The distribution of inelastic deformation and forces is sampled by specifying cross-section slices along the length of the element. The nonlinear force-displacement behaviour of the bridge pier should be considered in seismic analysis of a bridge system, especially in a seismically active zone. In such a region, the bridge piers are expected to incur large displacements during earthquakes, which lead to the fact that the linear force-displacement behaviour of a bridge pier will result in a very uneconomic design. The foundation movement effect is neglected in the analysis.

3. Lateral Strength and Ductility of the Pier

The lateral strength, ductility and mode of failure of bridge piers are computed using the method of nonlinear static analysis (i.e., pushover method) and the analytical method suggested by Japan Road Association (JRA, 2002). The sectional analysis has been conducted by professional software (Response, 2000). In addition, numerically solved nonlinear analysis program is used to conduct the pushover analysis in order to derive the force-displacement relationship of a pier. Evaluation of the adequacy of existing bridge bent to withstand imposed seismic loads requires assessment and comparison of anticipated demand and available capacities. With a view to achieve the goal, inelastic pushover analyses are carried out for obtaining the force-displacement relations (Alim, 2014). The procedure illustrates in Fig.4 to evaluated both yield and ultimate displacement and capacity of the bent.

3.1 Development of force-displacement relationship

The force-displacement relationship of piers can be derived from the results of moment-curvature relation at each section from top to bottom of a pier as obtained from sectional analysis of piers (Alim *et al.*, 2014). Sectional properties of the piers are related to the characteristics of the materials i.e., stress-strain relationship and strength of materials. Different model of concrete are developed for seismic model (Park *et al.*, 1985; Madas and Elnashai, 1992; Spoelstra *et al.*, 1999).

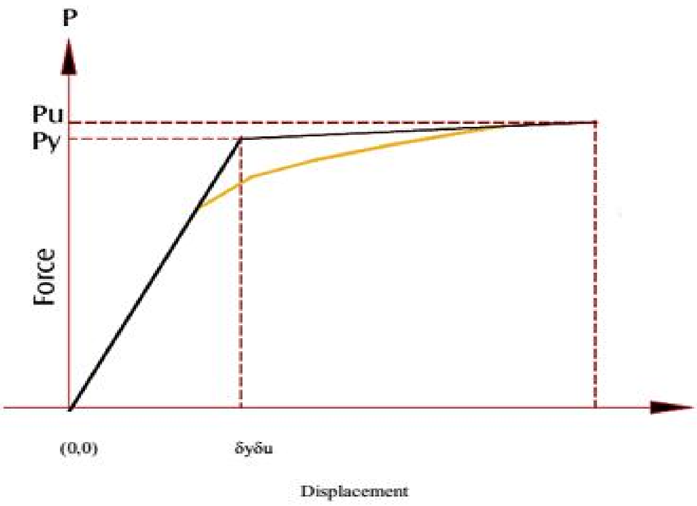
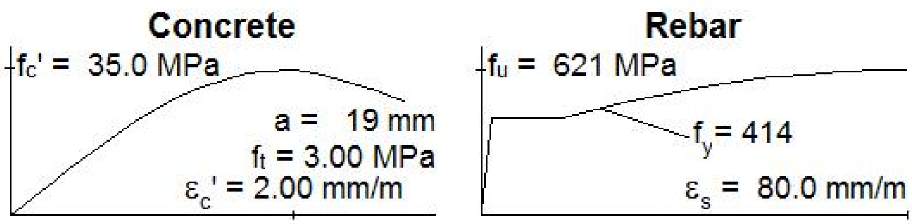


Fig. 4. Pushover analysis of the bent



(a) (b)

Fig. 5. Constitutive model of materials (a) Concrete and (b) Steel

A concrete model, which has been used extensively in recent years, was developed by Hoshikuma *et al.*, (1997). The descending branch of the material law as well as the increase of strength and corresponding strain because of confined reinforcing steel is taken into account which is shown in Fig. 5. The nonlinear model for reinforcing steel is used in the study and the constitutive model as shown in Fig. 5.

The model stress-strain curve consists of three parts i.e., an ascending branch, falling branch, and sustaining branch. The stress-strain curve can be expressed as below.

$$f_c = \begin{cases} E_c\varepsilon_c\left\{1 - \frac{1}{n\left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)}\right\} & (0 \le \varepsilon_c \le \varepsilon_{cc}) \\ fcc\left\{1 - E_{des}\left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)\right\} & (\varepsilon_{cc} \le \varepsilon_c \le \varepsilon_{cu}) \end{cases}$$

(1)

Where, n is coefficient and E_{des} is deterioration rate and are given as,

$$n = E_c\varepsilon_{cc} \big/ \left(E_c\varepsilon_{cc} - \sigma_{cc}\right) \tag{2}$$

$$\sigma_{cc} = \sigma_{c0} + 3.8\alpha\rho_s\sigma_{sy};\ \varepsilon_{cc} = 0.002 + 0.033\ \frac{\rho_s\sigma_{sy}}{\sigma_{c0}} \tag{3}$$

$$E_{des} = 11.2\frac{\sigma_{c0}^2}{\rho_s\sigma_{sy}} \tag{4}$$

where, σ_{c0} is design strength of concrete, σ_{sy} is the yield strength of reinforcement, α and β are shape factors and ρ_s is the volumetric ratio of tie reinforcements. The ultimate displacement d_u is defined as displacement at the gravity centre of superstructure when the concrete compression strain at out-most reinforcements reaches the following ultimate strain, ε_{cu}

$$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & type - I \quad earthquake \\ \varepsilon_{cc} + 0.2\,\sigma_{cc}\big/E_{des} & type - II \quad earthquake \end{cases} \tag{5}$$

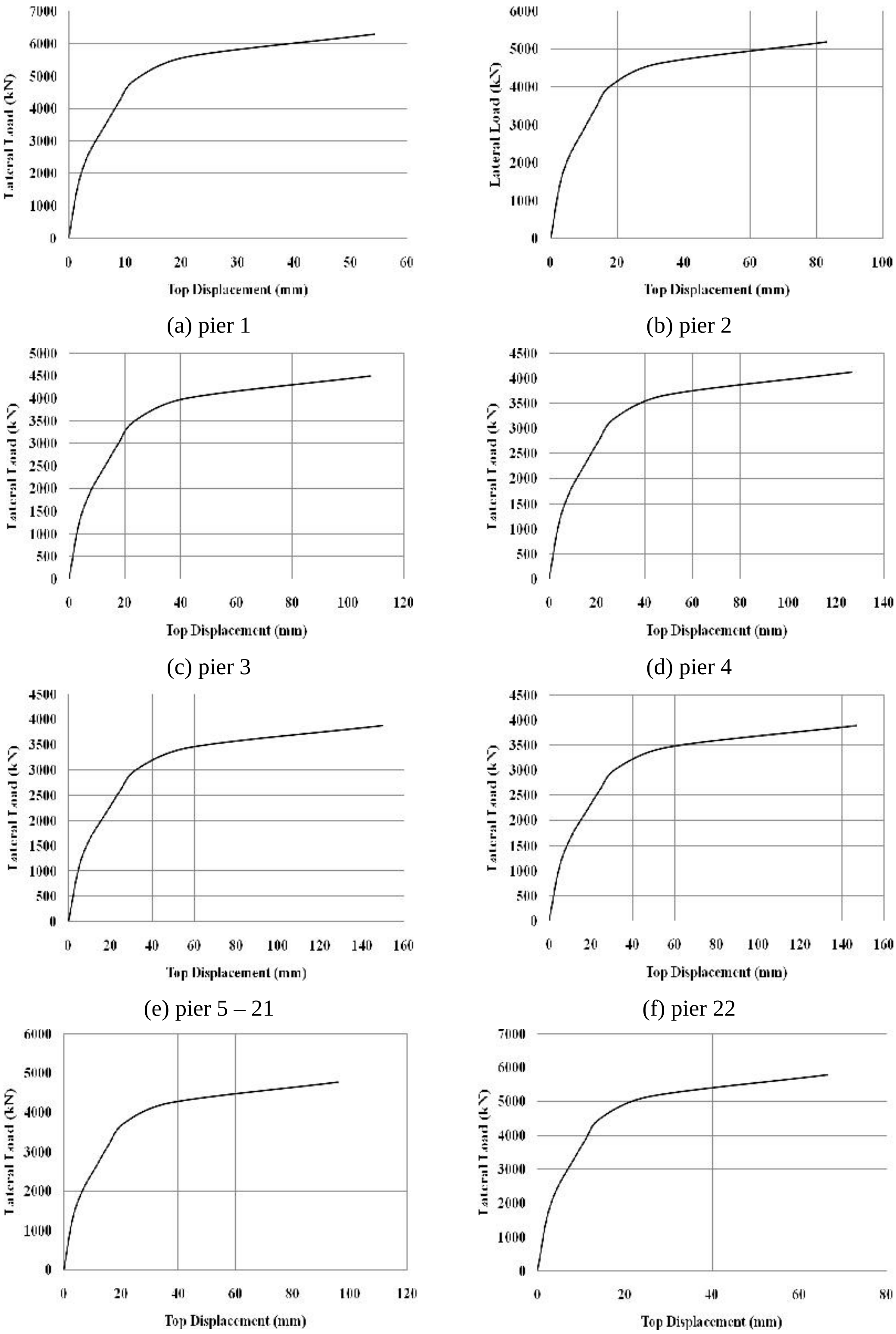
where, α and β are modification factors depending on confined sectional shape: for circular $\beta = 1.0$ and $\alpha = 1.0$; for square $\beta = 0.2$ and $\alpha = 0.4$. To obtain the force-displacement relationship at top of the bridge pier, the pier is divided into N slices (50 slices are recommended in the code) along its height. For sectional analysis, it is mainly focused on three sections: (a) section at the top level, (b) section at one-third level from the bottom of the pier, and (c) section at the base level. This is because the configuration of the reinforcement at this level is different. Finally, the force displacement relationship at the top of the bridge pier is obtained using the moment-curvature diagrams and shear stress-strain diagram (Alim, 2014). Fig.6 shows numerical evaluation of moment curvature of piers. Steps for obtaining the force-displacement relationships are as follows:

- The pier is divided into N slices along its height.
- The moment-curvature diagrams for each cross-section are obtained through sectional analysis.
- The horizontal force P is applied at the top of the pier.
- The bending moment diagrams of the pier for the applied force P are drawn.
- The curvature from bending moment and moment-curvature diagram is obtained.
- The displacement, δ at the top of the pier is estimated using the following Equation

$$\delta = \sum_{i=1}^{N} \emptyset d_y\, d_i \tag{6}$$

where, is the curvature of the pier section i, d_y is the width of the pier cross section i and d_i is the distance from the top of the pier to centre of gravity of section i.

- In a similar way, several forces P are applied and the corresponding displacement obtain.



(a) pier 1 (b) pier 2

(c) pier 3 (d) pier 4

(e) pier 5 – 21 (f) pier 22

(g) pier 23 (h) pier 24

Fig. 7. Force-displacement relationships for piers of the overpass

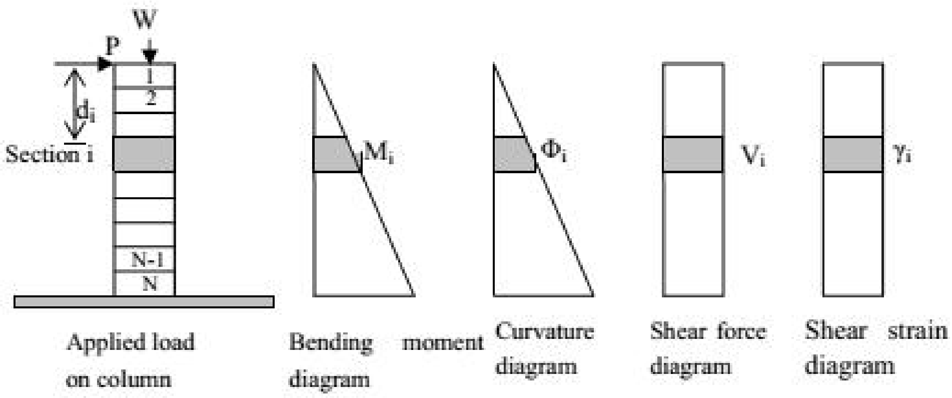


Fig. 6. Numerical evaluation of moment curvature of piers (JRA, 2002)

Following the procedural steps as discussed above the lateral load–displacement relationship at top of each pier is obtained. The lateral load –displacement characteristics of the piers of as obtained from the pushover analysis are presented in Fig.7 (a) to (h). The force-displacement relationship of each pier shows a similar fashion with different yield, ultimate lateral loads and deformations. The yields, ultimate deformations of the piers along with their failure mode have been used to compute the allowable ductility of the piers. The failure modes of the piers are evaluated based on their bending and shear capacities, which subsequently helps compute the allowable ductility of the piers.

3.3 Evaluation of Seismic Load, Lateral Strength, Ductility and Failure Mode of Piers

Ductility and lateral strength of each pier is evaluated considering its shear and flexural capacities and failure mode. Equations 7, 8 and 9 illustrate the ways of evaluating the failure mode, lateral strength and allowable ductility of each pier. Lateral strength of each pier tabulated form in Table 5. The lateral seismic loads of the piers under design seismic excitations are estimated using Equation (10) and are presented in Table 5.

$$\textbf{Failure Mode} = \begin{cases} \textbf{flexural failure} \ldots\ldots\ldots\ldots \mathbf{p_u \le p_s} \\ \textbf{shear failure after flexural damage} \ldots\ldots \mathbf{p_u \le p_{so}} \\ \textbf{shear failure} \ldots\ldots\ldots\ldots \mathbf{p_u \ge p_{so}} \end{cases} \tag{7}$$

The lateral capacity P_a and the allowable displacement ductility factor μ_a are given as

$$\mathbf{p_a} = \begin{cases} \mathbf{p_u} \ldots \textbf{flexural failure} + \textbf{shear failure after flexural damage} \\ \mathbf{p_{so}} \ldots\ldots\ldots\ldots \textbf{shear failure} \end{cases} \tag{8}$$

$$\mu_a = \begin{cases} 1 + \dfrac{\delta u - \delta y}{\alpha \delta y} \ldots\ldots\ldots\ldots \textbf{Flexural failure} \\ 1 \ldots\ldots\ldots\ldots \textbf{Shear failure} \end{cases} \tag{9}$$

where which α = safety factor depending on importance of bridges and the type of ground motion (α = 3.0 and 2.4 for important and ordinary bridges, respectively, under the far field ground motions, and α = 1.5 and 1.2 for important and ordinary bridges, respectively, under the near fault ground motions), δy and δu = yielding and ultimate displacement of the pier.

The seismic safety of the pier can be estimated such that the lateral load capacity P_a of the pier must be greater than or equal to the lateral load demand during seismic excitations,

$$P_a \geq \frac{S_s \cdot W}{g \cdot R} \tag{10}$$

where S_s is the elastic response acceleration (in the current study the elastic response accelerations for two peak ground accelerations, such as PGA = 0.15g and 0.25g, are considered) , W is the tributary weight and R is the response modification factor which can be assumed as

$$R = \sqrt{2\mu_a - 1} \tag{11}$$

Shear strength of concrete can be calculated by following equation (JRA, 2002),

$$P_s = S_c + S_s \tag{12}$$

$$S_c = C_c\, C_e\, C_{pt}\, f_c'\, bd \tag{13}$$

$$S_s = \{A_w f_{sy}(\sin\theta + \cos\theta)d\}/1.15a \tag{14}$$

where

- P_s = Shear Strength (N)
- S_c = Shear Strength resisted by concrete (N)
- S_s = Shear Strength borne by hoop tie (N)
- a = Spacing of the stirrup (mm)
- d = the effective depth of the pier section (mm)

The value of C_e and C_{pt} given in Table 3 and Table 4,

Table 3
Value of C_e in relation to effective height d of a pier section (JRA, 2002)

Effective Height (mm)	Below 1000	3000	5000	Above 10000
C_e	1.0	0.7	0.6	0.5

Table 4
Value of C_{pt} in relation to effective height d of a pier section (JRA, 2002)

Tensile Reinforcement (%)	0.2	0.3	0.5	Above 1%
C_{pt}	0.9	1.0	1.2	1.5

The yield and ultimate displacements, mode of failure allowable displacement ductility of piers of the overpass have been obtained using their lateral force-displacement relationships shown in Figs. 7 (a) to (h) and Eqns. (8), (9) and (12)–(14) as presented in Table 5. Equations (8) to (11) are used to evaluate the seismic safety of the overpass piers. In this case, two peak ground accelerations (i.e., PGA of 0.15g and 0.25g) complying seismic performance requirements of highway bridge structures and the like in the region surrounding the overpass location. Table 6 shows the lateral strength, failure mode and seismic safety status of the overpass for a ground motion records having PGA of 0.15g while Table 7 presents those for the overpass subjected to a ground motion records having PGA of 0.25g. Generally, tall piers seem to be vulnerable to flexural failure whereas the relatively short piers are susceptible to shear failure rather than flexural failure.

Table 5
Ductility of the overpass piers

Pier No	Pier Height (m)	Yield Displacement, δ_y (mm)	Ultimate Displacement, δ_u (mm)	Safety Factor, α	Mode of Failure	Allowable Displacement Ductility, μ_a
1	3.653	12.00	54.26	3.0	Shear	1.00
2	4.917	15.00	83.10	3.0	Bending	2.51
3	5.857	25.00	107.98	3.0	Bending	2.11
4	6.557	28.00	126.41	3.0	Bending	2.17
5 to 21	7.290	25.00	149.55	3.0	Bending	2.66
22	7.057	28.00	146.82	3.0	Bending	2.41
23	5.417	20.00	95.93	3.0	Bending	2.27
24	4.153	12.00	66.43	3.0	Shear	1.00

Moreover, all the overpass piers do not comply the seismic load requirements for a PGA of 0.25g whereas for a PGA of 0.15g, most of the piers can be considered to be safe except piers 1 and 24. Fig. 8 and 9 show the seismic lateral load and allowable lateral load for each of the overpass piers subjected to ground motion records having PGA of 0.15g and 0.25g.

Table 6
Lateral seismic load, allowable lateral load and seismic safety of the overpass pier for a PGA of 0.15g

Pier No.	Pier Height (m)	Lateral Seismic Load, P_{us} (KN) (when Z = 0.15)	Allowable Lateral Load, P_a (KN)	Safety Status
1	3.653	6469	5421	Not Safe
2	4.917	3224	4574	Safe
3	5.857	3609	4164	Safe
4	6.557	3538	3839	Safe
5 to 21	7.290	3112	3424	Safe
22	7.057	3306	3475	Safe
23	5.417	3443	4389	Safe
24	4.153	6469	5421	Not Safe

Table 7
Lateral seismic load, allowable lateral load and seismic safety of the overpass pier for a PGA of 0.25g

Pier No.	Pier Height (m)	Lateral Seismic Load, P_{us} (KN) (when Z = 0.25)	Allowable Lateral Load, P_a (KN)	Safety Status
1	3.653	10781	5421	Not Safe
2	4.917	5373	4574	Not Safe
3	5.857	6015	4164	Not Safe
4	6.557	5897	3839	Not Safe
5 to 21	7.290	5186	3423	Not Safe
22	7.057	5510	3479	Not Safe
23	5.417	5738	4388	Not Safe
24	4.153	10781	5421	Not Safe

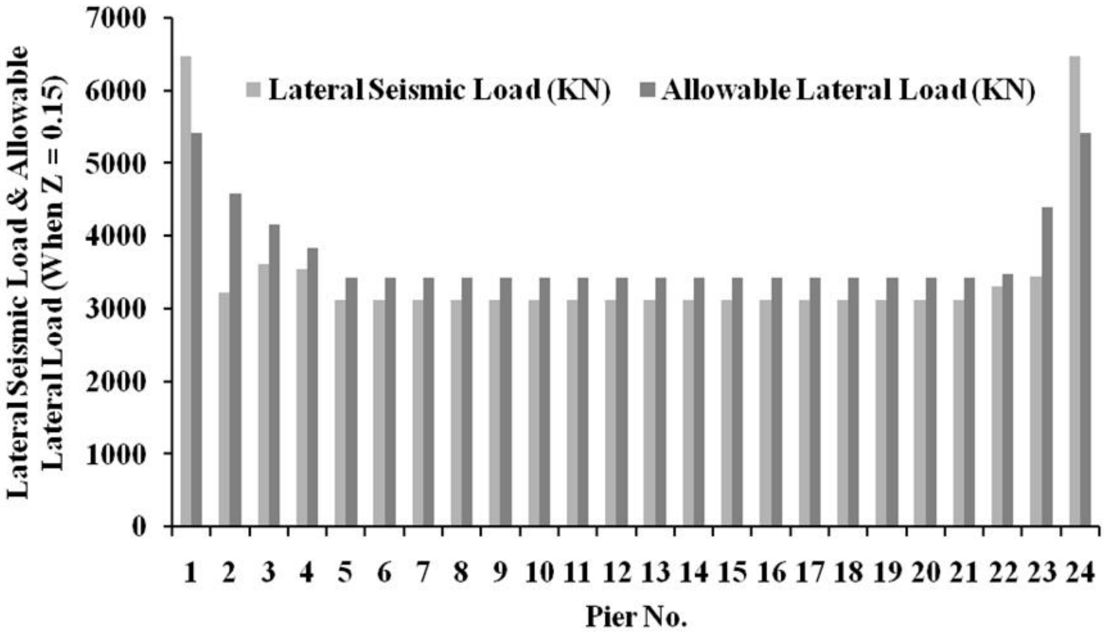


Fig. 8. Seismic lateral load and allowable lateral load of the overpass piers subjected to a ground motion having PGA of 0.15g

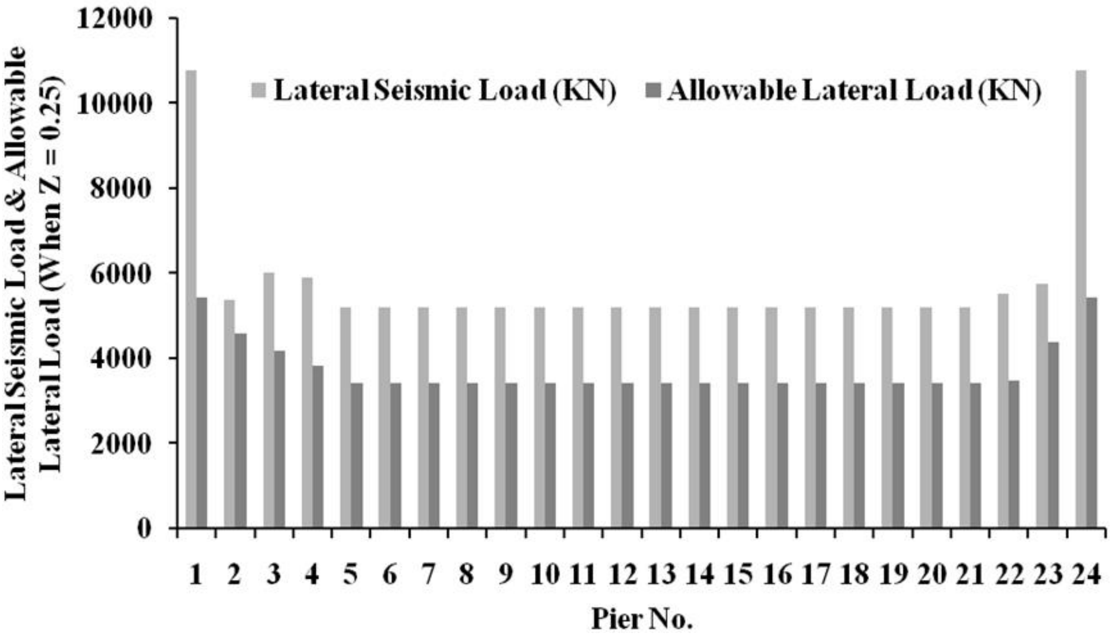


Fig. 9. Seismic lateral load and allowable lateral load of the overpass piers subjected to a ground motion having PGA of 0.25g

4. Conclusions

Lateral strength and ductility of the piers of Bahaddarhat overpass have been analytically evaluated using the equivalent static method as suggested by Japan Road Association (JRA) considering the different modes of failure. The current study has concentrated towards the analytical assessment of lateral strength and ductility of piers considering medium soil condition around the foundation of piers.The lateral strength in bending has been obtained using the results of nonlinear sectional analysis of each pier section of the overpass, while the shear strength of the pier is estimated using the JRA recommended analytical expressions, taking into account the effect of depth, volumetric ratio of lateral steel, crushing strength of concrete, yield strength of steel. The moment-curvature relationship at the critical section of pier has been developed using the fiber model with conventional constitutive models for concrete and steel. The force-displacement relationship of each pier is derived by conducting pushover analyses of pier considering material and geometrical nonlinearities. The lateral seismic force, allowable lateral force, yield displacement, ultimate displacement and displacement ductility are obtained from force-displacement relationships of the piers. Finally, the seismic safety of piers of the overpass has been evaluated using the ductility method for two far field earthquake ground motion records. From the numerical results it has been found that the most of the piers demonstrate bending mode of failure except two piers (pier 1 and 24) in which the shear mode of failure is dominated. All the piers do not comply the seismic performance requirements during the earthquake for the ground motion records having PGA of 0.25g; however, for the ground motion record having PGA of 0.15g, most of the piers presents compliance seismic performance requirements during earthquake.

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