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Seismic safety assessment of flyover piers in Chattogram city

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ABSTRACT: Lifeline systems, such as bridges, are prone to natural hazards. Bridges are essential components of an overall transportation system as they play important roles in evacuation and emergency routes for rescues, first-aid, firefighting, medical services and transporting disaster commodities. The performance of highway bridge systems observed in past and recent earthquakes—including the 1971 San Fernando earthquake, the 1994 Northridge earthquake, the 1995 Great Hanshin and 2011 Tohoku earthquakes in Japan, the 1999 Chi-Chi earthquake in Taiwan, the 2010 Chile earthquake, and the 2010 Haiti earthquake—have demonstrated that bridges are highly susceptible to damages during earthquakes. In order to take necessary steps to improve the seismic performance and subsequently reduce the seismic vulnerability of both new and existing bridges, seismic safety assessment is the prerequisite work. The study presents the seismic safety assessment of piers of a flyover in Chattogram city in terms of failure mode, lateral strength, shear capacity and residual displacements of piers as per the guidelines of Japan Road Association (JRA). Seismic safety assessment of piers have been also done for two performance objectives to achieve desired performance level corresponding to selected hazard levels at pier site following the major performance levels of FEMA-356. In this case, acceleration response spectra as suitable for site condition of the flyover are used in safety assessment. The assessment results have shown that the piers are found safe when subjected to earthquake ground motion records corresponding to the design acceleration spectra of BNBC.

1 INTRODUCTION

Flyover, one of the mostly used bridge structures in built-in area, is an elevated passage constructed over the road for the purpose of road transportation. Flyover has become the simple substitute to reduce traffic congestion at the intersections of densely populated built-in cities. Recently, a huge number of flyovers are being constructed in order to reduce the traffic blockage in two major cities of Bangladesh, Chittagong and Dhaka (Bhuiyan & Alim 2017). Bangladesh is highly vulnerable to earthquake hazards due to its geographical position. Chittagong is at high risk zone for earthquake, situated over the Chittagong-Tripura Fold Belt (Mukhlis & Bhuiyan 2017).Due to existence of active faults, there is a high probability of occurrence of a large magnitude earthquake in Bangladesh (Ali & Chowdhury 1992, 1994).

Several seismic codes and standards, such as ATC (1996), Eurocode (1998), JRA (2002), Caltrans (1999), AASHTO (2012), have been developed to evaluate seismic safety of bridge structures. The basic concept of seismic design of bridge structures in small-to- moderate earthquakes bridges should be within the elastic range without significant damage and in moderate to large earthquakes bridges should not cause collapse (AASHTO 2012)

On the basis of the background, the study aims at assessing seismic safety of a multi-span simply supported flyover. In this regard, the failure mode, seismic lateral strength and residual displacement of bridge piers have been considered in the safety assessment nonlinear static (pushover analysis)procedure has been employed in evaluating the associated parameters and checked with the analytical method suggested by Japan Road Association (JRA 2002). Finally, the seismic safety assessment of piers has been done for two performance objectives for achieving desirable performance objectives as defined in FEMA-356 (2000).

2 DESCRIPTION OF THE FLYOVER

The selected multi-span simply supported flyover is the largest highway overpass in Chattogram city to date. This flyover is 5.2km long and 16.5 m wide. Different cross-sections of a typical pier and pier cap are also shown in Figure 1. The layout of the flyover is presented in Figure 2. There are 92 spans whose length varies from 29m to 46mwith the approach roads at both ends of the flyover. The deck of the flyover comprises 8 pre-stressed concrete girders with 200mm reinforced concrete slab with 75 mm wearing course over it. The depths of the girders vary from 1.7m to 2.2m.The girders rest on elastomeric bearing located on top of each pier. The flyover is comprised of 93 y-shaped piers which have variable heights ranging from 0.15m to 7.34m. There are also variations in lengths and widths of pier caps of the flyover.

3 MODELING OF THE PIERS

3.1 *Physical Model of the Piers*

There are some notable guidelines for traffic and transportation provided by the Chattogram Development Authority in its Chittagong Metropolitan Master plan which was prepared by UNDP and UNCHS in the year 1995 and was approved by the government in 1999 and replaced the 1961 master plan regarding the control of the traffic congestion. A height-wise distribution of total 93 piers is shown in Figure 2. Among the 93 piers of the flyover, 10 representative piers have been considered in the analysis. A typical transverse section of pier is shown in Figure 3. The geometric dimensions and rebar details of representative piers are shown in Table 1. Material properties of the pier are represented in Table 2.

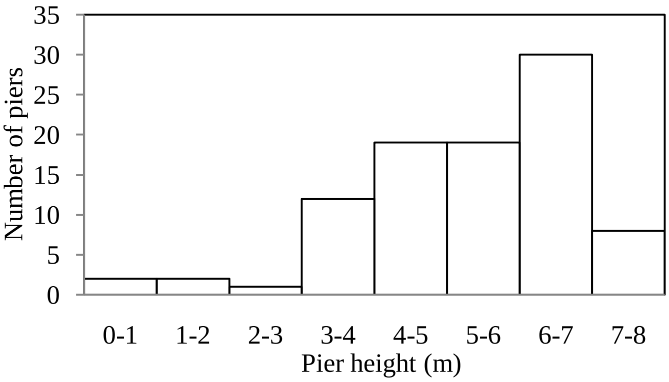


Figure 1. Height-wise distribution of flyover piers.

Table 1. Geometric dimensions and rebar details of flyover piers.

Sl. no.	Pier no.	Pier height (m)	Pier dimension (mm)	Longitudinal reinforcement
1	2	1.593	2500 × 3600	128 @ D32
2	19	4.883	2500 × 3600	128 @ D32
3	30	5.948	2500 × 3600	128 @ D32
4	37	6.995	2500 × 3600	128 @ D32
5	41	6.510	2500 × 3600	128 @ D32
6	51	7.335	2500 × 3600	128 @ D32
7	64	5.497	2500 × 3600	128 @ D32
8	72	4.521	2500 × 3600	128 @ D32
9	88	3.994	2500 × 3600	128 @ D32
10	91	2.620	2500 × 3600	128 @ D32

3.2 *Analytical Model of the Piers*

The analytical model of a tributary deck along with a pier (pier-girder system) is shown in Figure 4. 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 & Ali 1988). The pier-girder system is approximated as a continuous 2-D finite frame element with a finite number of degrees of freedom. The superstructure & substructure of the system are modeled 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. A professional software (SeismoStruct 2016) has been used for modeling purpose of each pier of the flyover. A 2-D finite element model of a typical pier is shown in Figure 4.

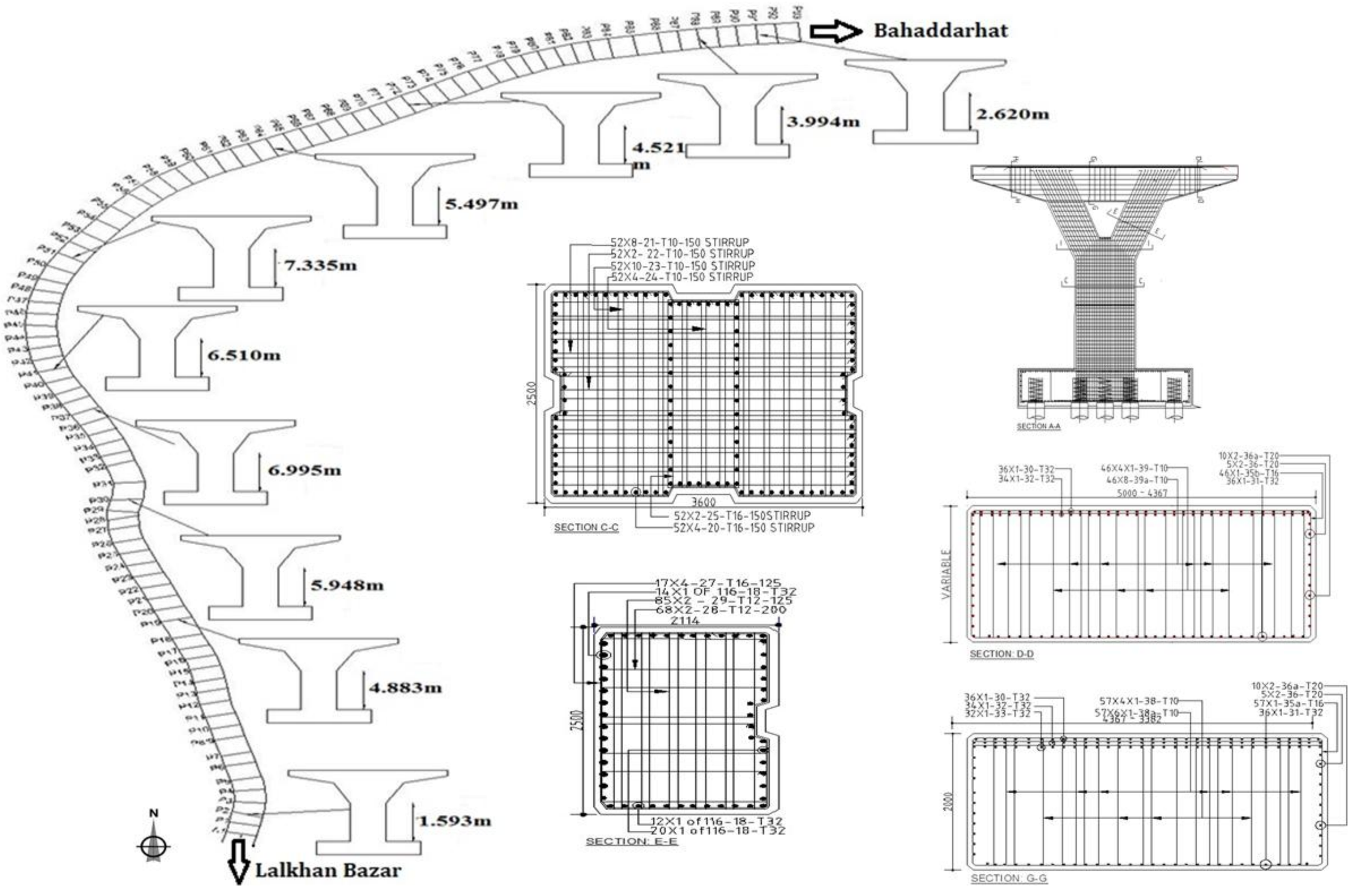


Figure 2. Layout of the flyover and typical cross-sections of components of pier and pier caps.

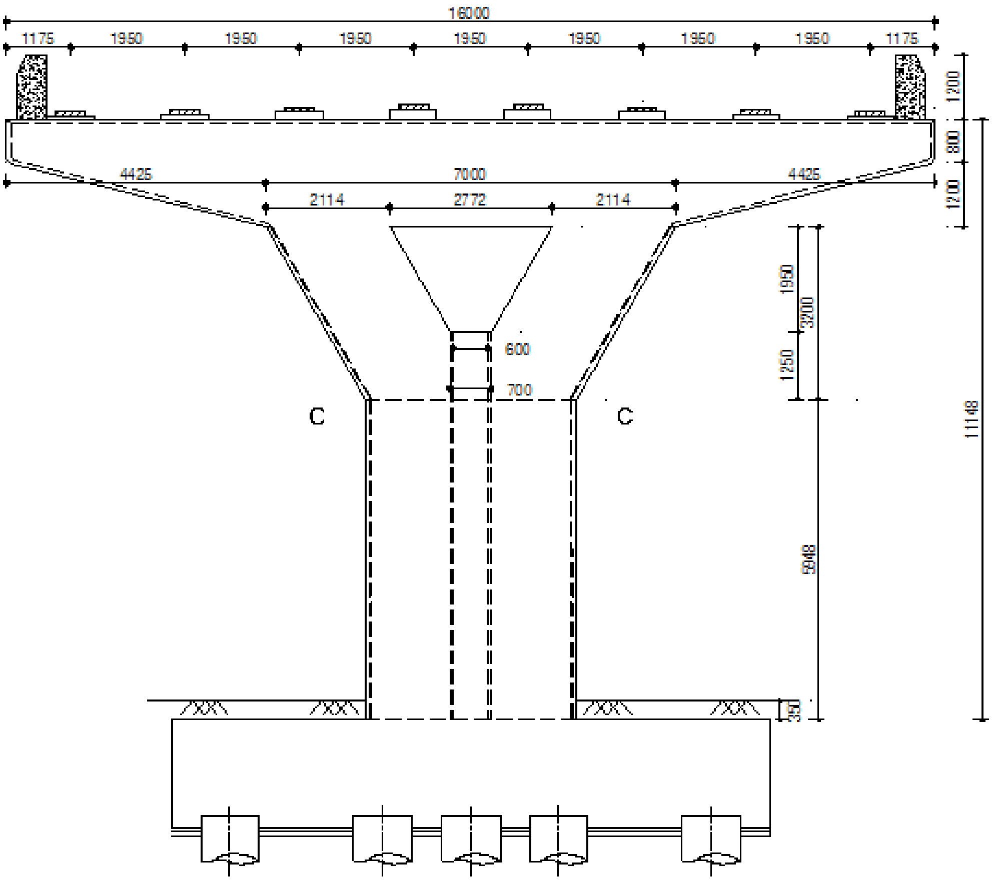


Figure 3. Typical transverse sectional view of a pier with dimensions.

Table 2. Material properties of flyover piers.

Materials	Material properties (MPa)
Reinforcement	Yield Strength, f_y = 500
Concrete	28 days cylinder crushing strength, $f'c$ = 30
	Modulus of Elasticity, E_c = 25.7×10^3

The body of piers is modeled by using fiber element. The section of the flyover pier has been modeled with original geometric dimension as force based inelastic frame element where 5 integration sections with 198 section fibres have been used for discretization. The piers are then subdivided into 6 inelastic frame elements along its height and pier caps are subdivided into 30 elastic frame elements along its length (Figure 4). The loads from deck, pre-stressed girders are calculated and modeled as lumped mass element on top of the pier cap. The base of the pier is assumed to be fully restraint to neglect the foundation movement effect.

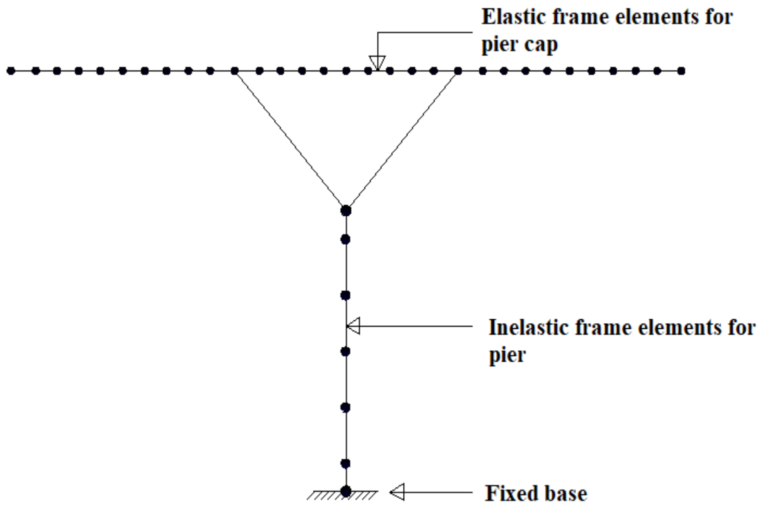


Figure 4. Analytical model of a typical flyover pier.

4 SEISMIC SAFETY ASSESSMENT OF FLYOVER PIERS

4.1 *Pushover Analysis*

The pushover analysis results for the piers are presented in the form of force-displacement relationships in Figure 5. Nonlinear structural analysis software (SeismoStruct 2016) is used to conduct pushover analysis. For a fiber-based modeling approach, implemented in SeismoStruct software, material strains constitute the best parameter for identification of the performance state of a given structure. In this study five performance criterion (cracking, spalling, crushing, yielding and fracture) are used.

4.2 *Assessment of Lateral Strength and Failure Mode of Flyover Piers*

Failure mode of the piers is analyzed according to the procedure suggested by Japan Road Association (JRA 2002). Strength and displacement ductility factor (μ_a) are determined depending on the failure mode of the piers. Based on the flexural strength (P_u), shear strength (P_s) and shear strength under static loading (P_{so}), failure mode of piers are obtained. According to the Japan Road Association (JRA 2002) guideline, the mode of failures in Eq. 1 and lateral strength in Eq. 2 are given below, respectively:

$$\text{Failure Mode} = \begin{cases} Flexural failure \ldots\ P_u \le P_s \\ Shear\ failure\ after\ flexural\ damage \ldots P_s < P_u \le P_{so} \\ Shear\ failure \ldots P_{so} < P_u \end{cases} \quad (1)$$

$$\text{Lateral Strength, } P_a = \begin{cases} P_u \ldots Flexural failure \\ P_u \ldots Shear failure after flexural damage \\ P_{so} \ldots Shear failure \end{cases} \quad (2)$$

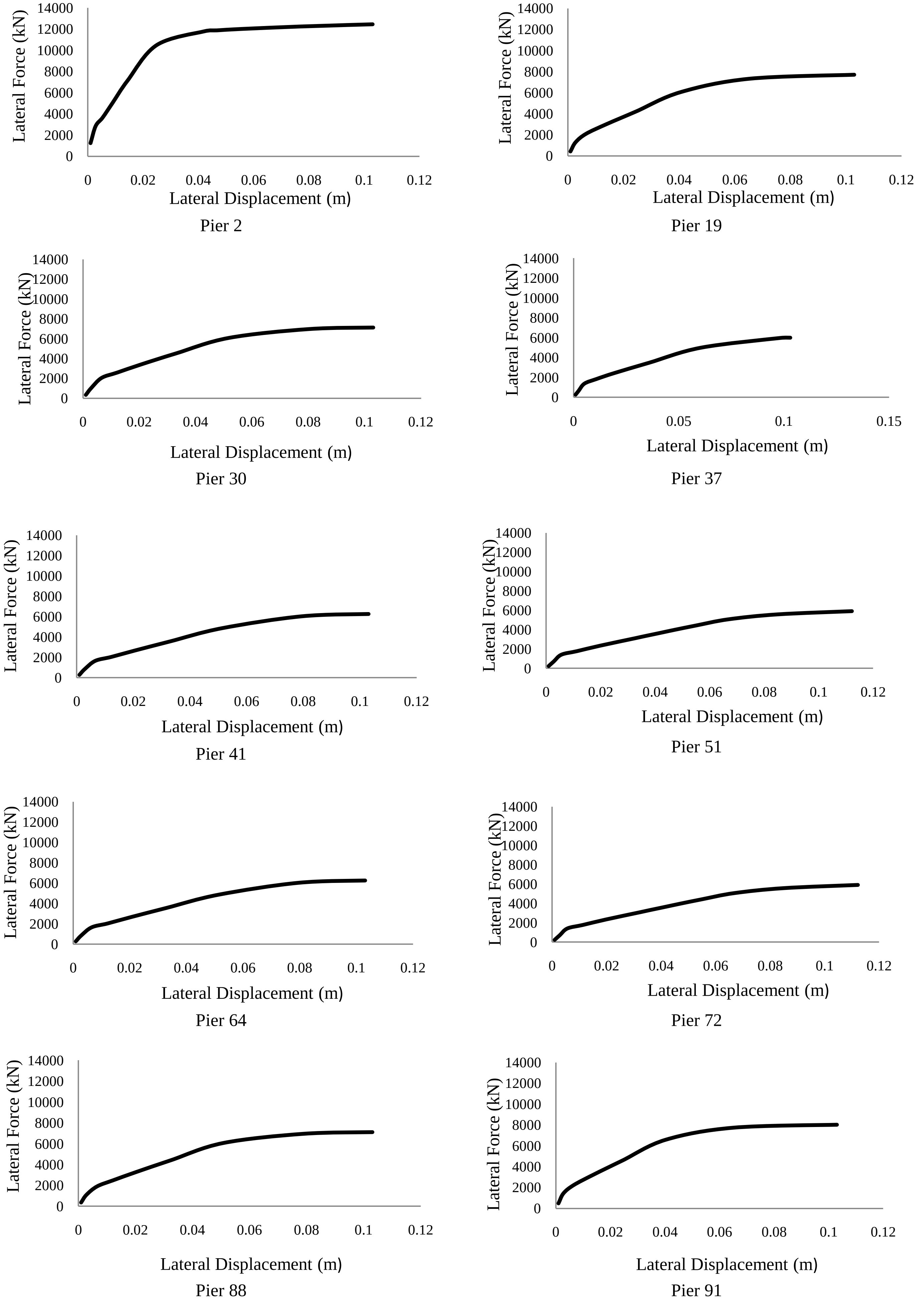


Figure 5. Force-displacement relationship of the piers.

Shear strength,

$$P_s = S_c + S_s \quad (3)$$

$$S_c = c_c c_e c_{pt} \tau_c bd \quad (4)$$

$$S_s = \frac{A_w f_{sy} (\sin\theta + \cos\theta) d}{1.15a} \quad (5)$$

where, P_s= shear strength (N); S_c= shear strength resisted by concrete (N); S_s=shear strength borne by hoop tie (N); b = width of pier section (mm); d = effective depth of pier section (mm); A_w= sectional area of hoop ties arranged with an interval of $\alpha$and an angle of θ (mm); a= spacing of the stirrup (mm); f_{sy}=yield point of hoop ties (N/mm^2); c_c = modification factor on the effects of alternating cyclic loading and taken as 0.6 for Type I, 0.8 for Type II earthquake and 1.0 for calculating P_{so}). The values of τ_c, c_e and c_{pt} are given in Table 3, Table 4 and Table 5.

Table 3. Average shear stress of concrete, τ_c (N/mm^2).

Design compressive strength of concrete, f_c' (N/mm^2)	21	24	27	30	40
Average shear stress of concrete τ_c(N/mm^2)	0.33	0.35	0.36	0.37	0.41

Table 4. Modification factor, c_e in relation to effective height, d of a pier section.

Effective height, d (mm)	Below 1000	3000	5000	Above 10000
c_e	1.0	0.7	0.6	0.5

Table 5. Modification factor, c_{pt} in relation to axial tensile reinforcement ratio, P_t.

Tensile reinforcement ratio (%)	0.2	0.3	0.5	Above 1.0
c_{pt}	0.9	1.0	1.2	1.5

The lateral strength, shear strength and failure modes are tabulated in Table 6, where shear strength is greater than lateral strength. According to Japan Road Association (JRA 2002), it falls in the category of flexural failure mode.

Table 6. Lateral strength, shear strength and failure mode of piers.

Sl. No.	Pier no.	Pier height (m)	Flexural strength P_u (kN)	Shear Strength P_s (kN)	Failure criteria	Failure mode	Lateral strength P_a (kN)
1	2	1.593	12864	23102	$P_u < P_s$	Flexural failure	12864
2	19	4.883	8007	23102	$P_u < P_s$	Flexural failure	8007
3	30	5.948	7407	23102	$P_u < P_s$	Flexural failure	7407
4	37	6.995	6376	23102	$P_u < P_s$	Flexural failure	6376
5	41	6.510	6647	23102	$P_u < P_s$	Flexural failure	6647
6	51	7.335	6187	23102	$P_u < P_s$	Flexural failure	6187
7	64	5.497	7449	23102	$P_u < P_s$	Flexural failure	7449
8	72	4.521	8310	23102	$P_u < P_s$	Flexural failure	8310
9	88	3.994	9102	23102	$P_u < P_s$	Flexural failure	9102
10	91	2.620	10799	23102	$P_u < P_s$	Flexural failure	10799

4.3 *Seismic Safety Assessment of Flyover Piers*

Two performance levels namely III-C and V-E have been selected from the guidelines of FEMA-356 (2000) corresponding to two different hazard levels as per Bangladesh National Building Code (BNBC 2006, 2020). The hazard levels consist of 20% and 2% probability of exceedance in 50 years having 225 years and 2475 years of return periods, respectively. Major performance levels of FEMA-356 (2000) for choosing performance levels are shown in Figure 6. Seismic safety assessment of piers has been done to achieve two desired performance objectives as shown in Table 7.

The normalized response spectra for 5% damping corresponding to hazard level of 20% and 2% probability of exceedance in 50 years per Bangladesh National Building Code(BNBC 2006, 2020) are shown in Figure 7.The normalized acceleration response spectrum coefficient, C_s is a function of the structure period and soil type. The variations of C_s with time periods for soil type SB according to BNBC(2006)and for soil type SC according to BNBC (2020) are considered in the study as shown in Figure 7(a) and 7(b) respectively.

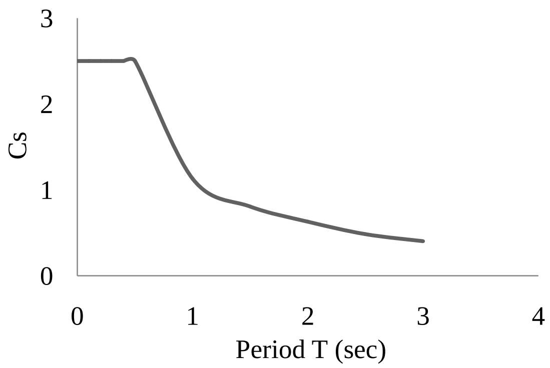
Nonstructural performance levels	Structural performance levels: I: immediate occupancy	III: life safety	V: structural stability (collapse prevention)
A: operational	I-A: operational	NR	NR
B: immediate occupancy	I-B: immediate occupancy	III-B	NR
C: life safety	I-C	III-C: life safety	V-C
E: not considered	NR	NR	V-E: structural stability

Note: NR, not recommended.

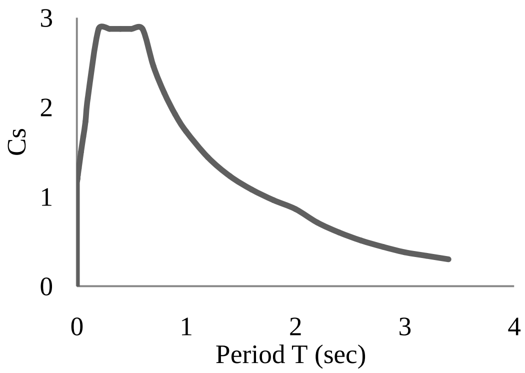
Figure 6. FEMA-356 (2000) recommended major performance levels (Chen & Lui 2006).

Table 7. Defining performance objectives.

Performance objectives	Performance level	Hazard level
1	life safety (III-C)	20%/50 years
2	collapse prevention (V-E)	2%/50 years



(a)



(b)

Figure 7. Normalized response spectrum: (a) 20% probability of exceedance in 50 years (BNBC2006) (b) 2% probability of exceedance in 50 years (BNBC2020).

The seismic safety of the piers is evaluated by comparing the lateral force demand (P_d) to lateral strength (P_a). Equation 6 is used to determine lateral force demand for a particular spectral acceleration,

$$P_{demand} = \frac{WS_a}{gR} \qquad (6)$$

where, W =seismic dead load; S_a=spectral acceleration; g =acceleration due to gravity; R =response modification factor. Response modification factor, R can be determined by using Equation 7.

$$R = \sqrt{2\mu_a - 1} \qquad (7)$$

where, μ_a is allowable displacement ductility obtained from Table 8.

Table 8. Defining ductility capacity based on structural performance.

	Performance level	Physical phenomenon	Ductility capacity (μ_a)
	life safety (III-C)	Extensive damage to structural components	1.76
	collapse prevention (V-E)	failure leading to near collapse	4.76
Reference	FEMA-356 (2000)	FEMA-356 (2000)	Hwang et al.(2001)

The safety level of the piers in achieving performance objective 1 and 2are tabulated in Table 9 and Table 10 respectively. It is seen from the results that all the piers are in "Safe" stage. Spectral acceleration (S_a) varies-depending on the values of C_s corresponding to fundamental time periods of the piers and zone factor (Z) for Chattogram city as per Bangladesh National Building Code (BNBC 2006, 2020).The zone factor (Z) for Chattogram city is 0.15 and 0.28 corresponding to the hazard levels 1 and 2 respectively.

The ultimate limit states denote the state when concrete strain at the location of axial compressive reinforcement reaches the ultimate strain. The ultimate displacement (δ_u) shall be calculated by the Eq. 8 with consideration of plastic hinges occurring at the damaged sections as δ_u values are hardly identified by the force-displacement relationships of piers shown in Figure 5.

Table 9. Safety assessment of the piers in terms of lateral strength for performance objective 1.

Sl. no.	Pier no.	Time Period (sec.)	Spectral acceleration S_a (m/s^2)	Lateral force demand, P_{demand} (kN)	Lateral strength P_a, (kN)	Safety status
1	2	0.16	7.49	4819	12864	Safe
2	19	0.30	4.93	3762	8007	Safe
3	30	0.41	3.99	4278	7407	Safe
4	37	0.41	4.00	3169	6376	Safe
5	41	0.37	4.28	3209	6647	Safe
6	51	0.42	3.94	3216	6187	Safe
7	64	0.33	4.62	3569	7449	Safe
8	72	0.29	5.04	3823	8310	Safe
9	88	0.28	5.16	4492	9102	Safe
10	91	0.21	6.25	4579	10799	Safe

Table 10. Safety assessment of the piers in terms of lateral strength for performance objective 2.

Sl. no.	Pier no.	Time Period (sec.)	Spectral acceleration S_a (m/s^2)	Lateral force demand, P_{demand} (kN)	Lateral strength P_a, (kN)	Safety status
1	2	0.16	6.95	2432	12864	Safe
2	19	0.30	7.91	3286	8007	Safe
3	30	0.41	7.91	4602	7407	Safe
4	37	0.41	7.91	3409	6376	Safe
5	41	0.37	7.91	3224	6647	Safe
6	51	0.42	7.91	3516	6187	Safe
7	64	0.33	7.91	3322	7449	Safe
8	72	0.29	7.91	3265	8310	Safe
9	88	0.28	7.91	3748	9102	Safe
10	91	0.21	7.91	3154	10799	Safe

$$\delta_u = \delta_y + (\phi_u - \phi_y) L_p (h - L_p / 2) \quad (8)$$

$$L_p = 0.2h - 0.1\,D \quad (9)$$

where, L_p= plastic hinge length (mm) calculated fromEq.9, in which $0.1D \le L_p \le 0.5D$; D = sectional depth (mm) (D shall be the diameter of a circular section, or the length of the rectangular section in the analytical direction); δ_u= ultimate displacement (mm) of the piers; δ_y = yield displacement (mm) of the piers; ϕ_y = yield curvature at the pier bottom section(1/mm); ϕ_u = ultimate curvature at the pier bottom section (1/mm); h = height from the pier bottom to the height of the super structural inertial force (mm).

Safety of piers can also be assessed by allowable residual displacement as per Japan Road Association (JRA 2002) using Eqs.10 to 13.

$$\delta_R \le \delta_{Ra} \quad (10)$$

$$\delta_R = c_R\ (\mu_r - 1)\ (1 - r)\delta_y \quad (11)$$

$$\mu_r = ½ \left\{\left(\frac{P_{demand}}{P_a}\right)^2 + 1\right\} \quad (12)$$

$$W = W_u + c_p W_p \quad (13)$$

where, δ_R=residual displacement (mm) of a pier; c_R=modification factor on residual displacement, a factor of 0.6 shall be taken for reinforced concrete columns;μ_r =maximum response ductility of piers; r = ratio of the secondary post-yielding stiffness to the yielding stiffness of a pier, a ratio of 0 shall be taken for reinforced concrete columns; δ_y = yield displacement (mm) of the piers; P_a= lateral strength of an reinforced concrete columns; W = equivalent weight (N) in the ductility design method; W_u= weight of the super structural part supported by the pier concerned (N); W_p= weight of the pier (N); c_p= equivalent weight (N) coefficient; δ_{Ra} =allowable residual displacement (mm) of piers. δ_{Ra} shall be 1/100 times the height from the bottom of the pier to the height of inertia force of the superstructure. Table 11 shows the equivalent weight calculation coefficient, c_p which is required to calculate the equivalent weight, W.

Table 11. Equivalent weight calculation coefficient, c_p.

Bending failure or shear failure after flexural yielding	0.5
Shear failure	1.0

Table 12. Safety assessment of the piers in terms of residual displacement for performance objective 1.

Sl. No.	Pier no.	Yield Displacement δ_y (mm)	Ultimate Displacement δ_u (mm)	Residual displacement δ_R (mm)	Allowable residual displacement δ_{Ra} (mm)	Safety status
1	2	32	362	0	72	Safe
2	19	58	673	0	100	Safe
3	30	59	828	0	115	Safe
4	37	74	857	0	117	Safe
5	41	68	810	0	113	Safe
6	51	79	1020	0	119	Safe
7	64	61	788	0	105	Safe
8	72	50	681	0	101	Safe
9	88	46	699	0	97	Safe
10	91	38	489	0	82	Safe

Table 13. Safety assessment of the piers in terms of residual displacement for performance objective 2.

Sl. No.	Pier no.	Yield Displacement δ_y (mm)	Ultimate Displacement δ_u (mm)	Residual displacement δ_R (mm)	Allowable residual displacement δ_{Ra} (mm)	Safety status
1	2	32	362	0	72	Safe
2	19	58	673	4	100	Safe
3	30	59	828	35	115	Safe
4	37	74	857	19	117	Safe
5	41	68	810	14	113	Safe
6	51	79	1020	30	119	Safe
7	64	61	788	8	105	Safe
8	72	50	681	2	101	Safe
9	88	46	699	4	97	Safe
10	91	38	489	0	82	Safe

The safety level of the piers in achieving performance objective 1 and 2 in terms of residual displacements are tabulated in Table 12 and Table 13 respectively. It is seen that residual displacements of piers are less than allowable residual displacements showing all the piers are in the “Safe” stage.

5 CONCLUSIONS

Lateral load resistance of statistically selected piers of a multi-span simply supported flyover was assessed based on the force-displacement relations as obtained from the pushover analysis results. Later, the failure mode of the flyover piers has been obtained following the guidelines of Japan Road Association. On the basis of lateral load resistance, failure modes, seismic demand and seismic safety of the piers have been evaluated for two performance objectives as defined by FEMA-356. In this case, two acceleration spectra corresponding to earthquakes of 20% and 2% probability of exceedance in 50 years were used to achieve desired performance level of III-C and V-E respectively for the piers.The allowable residual displacement has also been checked by following the Japan Road Association guideline. From the results it has been found that the lateral capacities of the piers are quite capable of withstanding the lateral forcesagainst both of the performance objectives. In the current analysis only a single pier has been considered for simplicity; however, for getting the detailed seismic responses of different components of the flyover a nonlinear time history analysis, considering multi-spans of the flyover, pounding effects, site conditions and soil-structure interactions, might have been be carried in future.

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