Interdependence between Structural Engineering and Construction Management
*Edited by* Ozevin, D., Ataei, H., Modares, M., Gurgun, A., Yazdani, S., and Singh, A.

ISBN: 978-0-9960437-6-2

# EFFECT OF SITE AMPLIFICATION ON SEISMIC SAFETY EVALUATION OF FLYOVER PIER

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Bangladesh is a developing country in which a lot of multi-span simply/continuous supported flyovers are being constructed in its major cities. Being situated in a seismically active region, seismic safety evaluation of flyovers is essential for seismic risk reduction. Effects of site amplification on seismic safety evaluation of flyover piers are the main concern of this study. In this regard, failure mode, lateral strength and displacement ductility of piers of a typical multi-span simply supported flyover have been evaluated by Japan Road Association (JRA) recommended guidelines, with and without considering site amplification. Ultimate flexural strengths of piers have been computed using the pushover analysis results. Shear capacity of piers have been calculated using the guidelines of JRA. Lateral strengths have been determined depending on the failure modes of the piers. Displacement ductility of piers has been computed using yield and ultimate displacements of the piers obtained from the pushover analysis results. Selected earthquake time history is used in seismic safety evaluation of the flyover piers. Finally, the ductility design method is used to conduct the seismic safety evaluation of the piers with and without considering site amplification. From the numerical results, it has been revealed that the effects of site amplification on seismic safety evaluation of bridge structures should be carefully taken into account.

*Keywords*: Failure mode, Lateral strength, Shear wave velocity, Ground response, Displacement ductility, Damage state.

## 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. Flyover, one of different bridge forms, is an elevated structure carrying highway over roads, railways and other features. So, in the subsequent discussion of this paper, the terms ‘bridge’ and ‘flyover’ are synonymously used.

It has been well known that Site Amplification (SA) occurs mainly due to local site effects. Local site effects include the characteristics of site soil controlling the shaking of the ground during earthquake (Navidi 2012). If a lightweight flexible structure is built on a very stiff rock foundation, a valid assumption is that the input motion at the base of the structure is the same as the free-field earthquake motion. If the structure is very massive and stiff, and the foundation is relatively soft, the motion at the base of the structure may be significantly different than the free-field surface motion. For designing structures it is important to consider the effect of the site amplification on the response of the structure. Several seismic codes and standards, such as, JRA (2002), have been developed to evaluate seismic safety of bridge structures. The main theme of seismic safety evaluation is 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 (Bhuiyan and Alim 2017).

Based on the above background, the study aims at evaluating the failure mode, lateral strength, displacement ductility demand and capacity to obtain the seismic safety status of a typical pier of flyover, constructed in a major city of Bangladesh, with and without considering site amplification phenomenon. In this regard, the guidelines recommended by JRA (2002) are used. The nonlinear static pushover analysis method has been adopted to obtain lateral strengths and ductility capacity of pier by considering its flexural strength, shear strength and failure mode. Finally, the seismic safety of the pier has been evaluated.

## 2 MODELING OF THE PIER

### 2.1 Physical Model

Kadamtali flyover (22.34°N; 91.82°E) has been constructed to reduce traffic jam in Chittagong city of Bangladesh. Figure 1 shows a 3D view of the flyover. The flyover is approximately 1127 m long and 8.54 m wide. It is spanning around 630 m with 22 spans of variable length from 21.3 m to 42.0 m. There are 21 piers with variable heights ranging from 4.66 m to 8.5 m. Figure 2 shows the transverse section of the typical pier of the flyover. The reinforcement with yield strength of 413 N/mm$^2$ and concrete with compressive strength of 30 N/mm$^2$ are used in the flyover. The deck of the flyover is comprised of four pre-stressed concrete girders with 200 mm reinforced concrete slab. The girders rest on elastomeric neoprene bearing over concrete bearing pad installed on top of each pier. Each elastomeric neoprene bearing with a cross-section of 500 mm × 350 mm consists of 4 numbers of 3 mm steel plates with a total thickness of 70 mm. For this study, a typical pier (Pier 7 as shown in Figure 1) of the flyover has been selected for analysis purpose. Geometric dimensions of the typical pier are presented in Table 1.

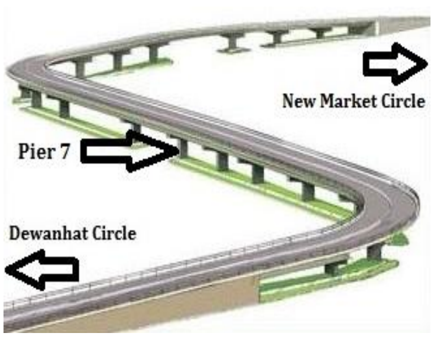


Figure 1. 3-D view of Kadamtali flyover (Mukhlis and Bhuiyan 2017).

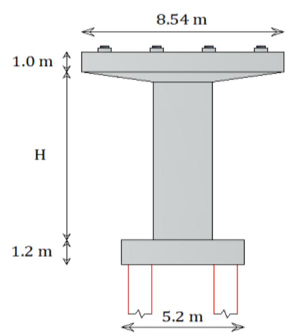


Figure 2. Transverse section of typical Pier.

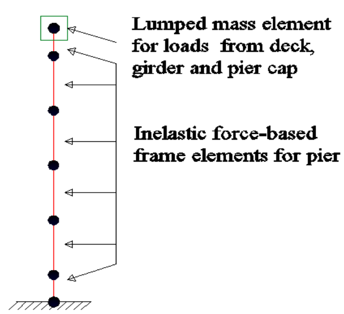


Figure 3. Analytical model of typical pier.

Table 1. Geometric dimensions of a typical pier of Kadamtali flyover.

| Pier No. | Pier height, H (m) | Pier Dimension (m x m) | Longitudinal Reinforcement |
|---|---|---|---|
| 7 | 8.27 | 1.2 x 2.5 | 66-Y25 bar |

## 2.2 Analytical Model

The superstructure and substructure of the system are modeled as a lumped mass system divided into a number of small discrete segments using nonlinear seismic analysis software SeismoStruct 2016. The mass of each segment is assumed to be distributed between two adjacent nodes. The body of the flyover pier is modeled using fiber elements. The section of bridge pier has been modelled with original geometric dimension as force based inelastic frame element of beam-column element types where, 198 section fibres with 5 integration sections have been used for discretization. The typical piers are then subdivided into 6 elements along its height. The loads from deck, prestressed concrete girders and pier caps are calculated and modeled as lumped mass element on the pier top. The base of the pier is assumed to be fixed neglecting the foundation movement effect. Analytical model of the typical pier is shown in Figure 3.

# 3 LATERAL STRENGTH AND DUCTILITY CAPACITY OF THE PIER

## 3.1 Pushover Analysis of the Pier

The sectional analysis has been conducted by Response 2000 to obtain the moment-curvature (M-Φ) relationship of pier cross section which is used to derive the force-displacement relationship of the pier. In addition, SeismoStruct 2016 is used to conduct the pushover analysis in order to derive the force-displacement relationships of the pier along with the bilinear idealization, from which the yield displacement ($\delta_y$), ultimate displacement ($\delta_u$) and ultimate strength ($P_u$) are found to be 37 mm, 175 mm and 2454 kN respectively.

## 3.2 Evaluation of Failure Mode, Lateral Strength and Ductility Capacity of the Pier

Failure mode, lateral strength and ductility capacity of the typical pier is analyzed according to the procedure suggested by JRA (2002), depending on the flexural strength ($P_u$), shear strength ($P_s$) and shear strength under static loading ($P_{s0}$) In this study, safety factor depending on importance of bridges and the type of ground motion, α is taken as 3.0 assuming important bridge in near field region. Modification factor on the effects of alternating cyclic loading, $c_c$ is taken as 0.6 for Type I earthquake and 1.0 for calculating $P_{s0}$ in this study. The value of average Shear Stress of Concrete, $\tau_c$ is found to be 0.37 N/mm$^2$ depending on $\mathbf{f_c}'$ value, modification factor $c_e$ is found to be 0.79 depending on effective height, d of pier section and modification factor $c_{pt}$ is found to be 1.5 depending on tensile reinforcement ratio of the pier. Using these values, failure mode, lateral strength and ductility capacity of the typical pier are calculated as shown in Table 2.

Table 2. Failure mode, lateral strength and ductility capacity of the typical pier.

| **Pier No.** | **Ultimate Strength, $P_u$ (kN)** | **Shear Strength, $P_s$ (kN)** | **Shear Strength, [for $c_c$=1] $P_{s0}$ (kN)** | **Failure Criteria** | **Failure Mode** | **Lateral Strength, Pa (kN)** | **Ductility Capacity, μa** |
|---|---|---|---|---|---|---|---|
| 7 | 2454 | 3002 | 3509 | $P_u \leq P_s$ | Flexural Failure | 2454 | 2.26 |

# 4 SEISMIC SAFETY EVALUATION OF THE PIER

Seismic safety of bridge components are generally expressed in terms of damage conditions of those components subjected to seismic ground motions. Piers are generally the most critical components of bridge. Different forms of Engineering Demand Parameters (EDP) such as displacement ductility demand ($\mu_d$) is generally used to measure the Damage State (DS) of the bridge piers (Bhuiyan and Alam 2012, Bhuiyan and Alim 2017). Table 3 summarizes the definitions of various damage states and their corresponding damage criteria available in the literature. Displacement ductility demand ($\mu_d$) has been estimated from the results of time history analysis (THA) for specific ground motion history with/without considering site amplification (SA) using Eq. (1), where, $d_u$ = maximum pier displacement from THA, $d_y$ = yield displacement of pier.

$$\mu_d = \frac{d_u}{d_y} \tag{1}$$

Table 3. Damage states of bridge pier.

| **Bridge component** | **EDP (Displacement Ductility, $\mu_d$)** | **Damage states (DS)** | **Physical Phenomenon** |
|---|---|---|---|
| Pier | $\mu_d < 1.0$ | No Damage | No Physical phenomenon |
| | $1.0 < \mu_d < 1.2$ | Slight (DS=1) | Cracking and spalling |
| | $1.2 < \mu_d < 1.76$ | Moderate (DS=2) | Moderate cracking and spalling |
| | $1.76 < \mu_d < 4.76$ | Extensive (DS=3) | Degradation without collapse |
| | $\mu_d > 4.76$ | Collapse (DS=4) | Failure leading to collapse |
| Reference | Hwang *et al*. (2001) | FEMA (2003) | FEMA (2003) |

## 4.1 THA without Considering Site Amplification (SA)

For simplicity in the analysis and unavailability of sufficient seismic data, site specific ground motion data was not developed. Time history analysis is conducted by applying 6.7 magnitude Northridge 1994 earthquake ground motion records with a recorded PGA of 0.45g at 4.06 s as shown in Figure 4 at the fixed base of pier. Rayleigh damping has been chosen for the pier under consideration with 5% damping ratio using two fundamental modes in transverse and longitudinal direction with a period of 0.525 s and 0.252 s respectively found from the eigenvalue analysis. The displacement response at the top of pier is represented with respect to time as shown in Figure 5 with a maximum displacement of 154 mm at 7.28 s without considering SA.

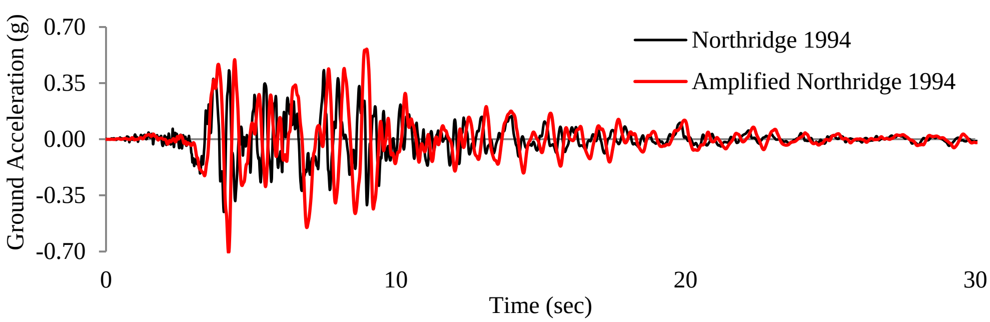


Figure 4. Ground motion accelerations for THA.

## 4.2 THA Considering Site Amplification (SA)

In this study, one dimensional site response analysis software program "Strata" (SHAKE-type) has been used for the modeling of soil profile. Shear wave velocity ($V_s$) is estimated using empirical formula proposed by Hossain and Ansary (2015) as shown in Eq. (2) for different soil layer up to 15.5 m with a maximum value of 399 m/s at that depth. The shear wave velocity of half-space bedrock is taken as 800 m/s according to BNBC Final Draft (2015).

$$V_s = 169\, N^{0.2638} D^{0.2396} \tag{2}$$

Where, $V_s$ = shear wave velocity (ft/s); D = depth (ft); N = corrected SPT value. Ground response of the borehole site are evaluated by reproducing the ground motion time history at the ground surface level and computing the acceleration transfer function, which has been found to be 2.78, by applying original Northridge 1994 earthquake ground motions at the bedrock layer with equivalent linear analysis. The amplified Northridge 1994 earthquake at the ground surface level of pier is shown in Figure 4 indicating PGA of 0.70g at 4.22 s. Time history analysis is then conducted by applying amplified Northridge 1994 earthquake ground motion records at the fixed base of pier. The displacement response at the top of pier is represented with respect to time as shown in Figure 5 with a maximum displacement of 268 mm at 4.14 s considering SA.

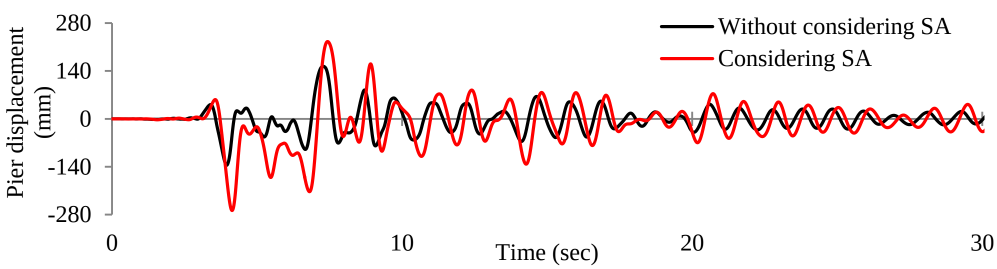


Figure 5. Displacement response at pier top.

### 4.3 Damage State Evaluation of Typical Pier

Damage state of bridge pier subjected to the selected ground motion intensity is shown in Table 4. Maximum pier displacement and displacement ductility demand of pier for different modeling considerations are shown in Figure 6 and Figure 7 respectively.

Table 4. Damage state evaluation of pier.

| Modeling Considerations of Bridge Pier | Earthquake Name | PGA (g) | $d_u$ (mm) | $d_y$ (mm) | $\mu_d = \frac{d_u}{d_y}$ | Damage State | Physical Damage Condition |
|---|---|---|---|---|---|---|---|
| Without Considering SA | Northridge-1994 | 0.45 | 154 | 37 | 4.16 | DS 3 | Extensive |
| Considering SA | Northridge-1994 | 0.70 | 268 | 37 | 7.24 | DS 4 | Collapse |

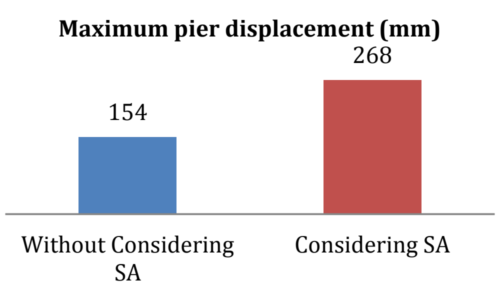


Figure 6. Maximum displacement of pier (mm).

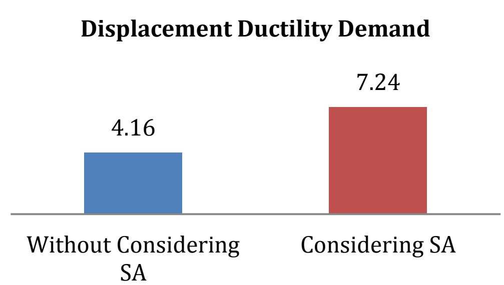


Figure 7. Displacement ductility demand of pier.

It has been observed that maximum pier displacement ($d_u$) and displacement ductility demand ($\mu_d$) increases 74% if site amplification is considered in the model. Consequently, the physical damage state is changed from extensive damage of the pier to collapse of pier leading to failure in response to the applied ground motion of Northridge 1994 earthquake for considering SA in the analysis procedure.

## 5 CONCLUSIONS

Effects of site amplification (SA) on seismic safety evaluation of flyover piers have been analytically evaluated. A typical pier has been selected for the detailed evaluation of displacement ductility capacity and displacement ductility demand corresponding to considerations of SA. In both cases, the pier displacement ductility demand of 4.16 and 7.24 corresponding to a ground motion intensity of 0.45g exceeds the pier displacement ductility capacity of 2.26, leading to the unsafe condition. Seismic safety of pier is also evaluated based on the damage state using the displacement ductility demand of pier with and without considering SA. Displacement ductility demand of pier is found to be extreme with a 74% increased value when SA is included in the pier model. Moreover, the physical damage state is drastically changed from extensive damage to collapse of pier leading to ultimate failure. Hence, from the numerical results, it has been revealed that site amplification has great impact on seismic response analysis and safety evaluation of bridge piers and should be carefully considered in the modeling and analysis procedure.

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