PERFORMANCE ASSESSMENT OF RC BRIDGE SUBSTRUCTURE USING NONLINEAR TIME HISTORY ANALYSIS

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ABSTRACT

Bangladesh is a riverine country with a large network of rivers. Bridges, therefore, play an important role in the transportation network system. Seismic design of bridges is a concern for Bangladesh as there is no national code or framework for designing bridges. Generally, AASHTO code or Euro codes are followed as design framework. This study incorporates (BNBC, 2020) seismic parameters into the design framework of (AASHTO LRFD, 2020) and evaluates the performance of the structure through nonlinear time history analysis (NLTHA). This study considers nonlinearity of substructure via soil-pile interaction and fiber hinge modeling of pier columns. NLTHA presents insights into damage detection criteria like hysteresis behavior, strain progression, demand ductility and global displacement (drift) etc. at different seismic hazard levels. Based on the performance assessment results of bridge substructure, it is evident that AASHTO based design approach incorporating BNBC seismic parameters can achieve target performance objectives. This study develops a reliable deterministic performance-based design framework for bridges in Bangladesh, a country lacking established regional guideline for seismic design.

Keywords: *AASHTO, BNBC, Soil-Structure interaction, NLTHA, Response Spectrum, fiber hinges.*

1. INTRODUCTION

Bangladesh is a riverine country with a large network of rivers and water bodies. Bridges, therefore, play an important role in the transportation system and infrastructural network. Important and critical bridges make a significant contribution to the economy of Bangladesh. Some of the bridges are located in earthquake-prone regions of the country. Important bridges may be in vulnerable condition during natural disasters such as earthquakes. Therefore, it is crucial to design these structures to withstand seismic excitation while maintaining their structural integrity. During seismic excitation, inertia forces develop in the structural system. These lateral dynamic loads impose major concerns on the vertical members such as piers and foundation piles of the bridge. The overall structural integrity depends on the integrity of the substructures; hence, piers and foundations should be able to sustain the forces generated by ground motion.

Currently, there is no national code for designing and evaluating the performance of bridges in Bangladesh. For bridge design, the American Association of State Highway and Transportation Officials (AASHTO) guidelines are followed. However, the AASHTO code does not include seismic hazard parameters specific to Bangladesh. The Bangladesh National Building Code (BNBC) contains seismic hazard parameters specifically for buildings, which differ slightly from those for bridges. One of the main objectives of this study is to integrate these two codes for the design of bridge substructures. The objectives of the study are: (1) To design bridge substructures using a force-based approach according to (AASHTO LRFD, 2020), and (2) To evaluate the performance of the pier and foundation system under different earthquake hazard levels using nonlinear time history analysis.

2. METHODOLOGY

The seismic design of an RC bridge requires determining several seismic parameters. The zone coefficients and response acceleration parameters are defined according to (BNBC, 2020), while soil classification, site coefficients, and the response modification factor are defined according to (AASHTO LRFD, 2020). These parameters constitute the response spectrum curve, which serves as the basis for seismic design in this case study.

A soil-structure interaction model is prepared to represent the practical situation of the underlying soil and foundation interaction. This is done by following API and AASHTO guidelines regarding the nonlinear behavior of soil stiffness Lateral soil resistance versus displacement curve (p-y) can be constructed following API-Geotechnical and Foundation Design (2011) guidelines for layered clay and sandy soils. The axial shear transfer curve (t-z) and the end-bearing resistance versus displacement curve (Q-z) is constructed using (AASHTO, 2011) guidelines. The case study model of a bridge substructure is designed using response spectrum analysis (RSA), following AASHTO seismic design specifications. The structural design of the bridge is carried out considering the Strength I and Extreme Event I limit-state conditions according to (AASHTO LRFD, 2020). Capacity-protected design of piles is ensured by designing for the hinging force.

To capture the true nonlinear dynamic behavior of the bridge, nonlinear time history is performed for scaled ground motions. The ground motions are scaled for the Service-Level Earthquake, Design-Level Earthquake, and Maximum Credible Earthquake. The post-yielding behavior of the structure is modeled using hinges in SAP2000. For three-dimensional dynamic analysis, a fiber hinge (PMM) model is considered. The Takeda hysteresis model is considered for a bridge reinforced concrete column. It is necessary to define the $\sigma-\varepsilon$ relationship for the fiber hinge. Performance objective criteria (Immediate Occupancy, Life Safety, Collapse Prevention) for a bridge with a specific operational category (Critical, Essential, Conventional) are defined according to (FHWA, 2006). Bridge performance criteria are defined by parameters (drift%, ductility) proposed by (Hose et al. 1999). The goal is to assess whether performance objectives are met at different seismic hazard levels.

3. DESIGN AND PERFORMANCE ASSESSMENT

The analysis of the case study model is performed using SAP2000. The objective of this study is to verify the validity of SAP2000 in performing nonlinear time history analysis. The validity of the soil-structure interaction model using SAP2000 is also evaluated.

3.1 Description of Case Study Model

The case study bridge is a conventional bridge. It is 1000 m long with 50 m spans, and the deck system is simply supported on concrete girders. Bridge geometry is considered as shown in Figure 1. The bridge is located in Sylhet. The SPT value of the site soil is within 15<N<50. Therefore, the site class is D according to (AASHTO, 2020). The scour depth is considered when modeling the soil-pile interactions.

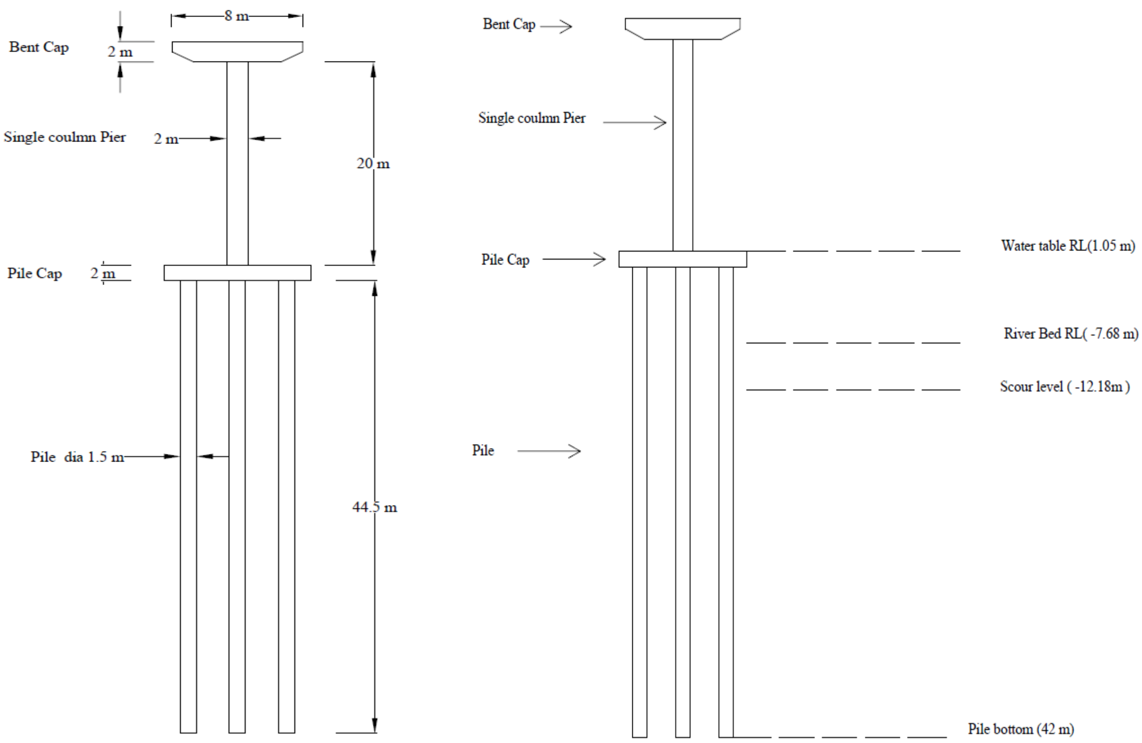


Figure 1: Bridge Substructure Geometric profile with Elevation

3.2 Material Property

Material properties of concrete and steel reinforcement are mentioned in Table 1.

Table 1: Concrete and steel reinforcement property

Material	Strength, MPa	Modulus of Elasticity, GPa
Concrete	27.6	24.8
Reinforcement	413.7	200

3.3 Loads and Loads Combination

Load combinations and factors at different limit states are derived from (AASHTO LRFD, 2020). Design basis earthquake is considered based on seismic hazard level of 7% probability of exceedance in 75 years as shown in Figure 2. According to AASHTO, combination for elastic seismic forces on each axis is 100% of the absolute value of the force effect on that axis and 30% of the absolute value of force effect on the perpendicular axis. This is considered in the load case of extreme events.

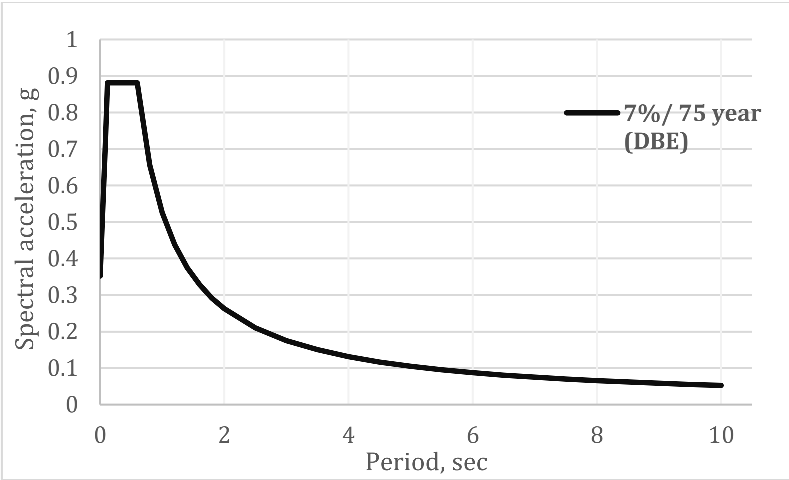


Figure 2: Design Response Spectrum Curve for Sylhet

3.4 Modelling of Soil Layers on SAP2000 as Links

SAP2000 does not provide an option to model soil as a finite element. However, the nonlinearity of the soil can still be modeled using link elements. Multilinear plastic links are used to model the nonlinear behavior of soil stiffness. These links are modeled in the three directions of each pile element. p-y curves are assigned in the global X and global Y directions. t-z curves are assigned in the global z direction of each pile segment. The Q-z curve is assigned in the global Z direction at the bottom of each pile. For the nonlinear time history analysis, the hysteretic behavior of the links is modeled using the Takeda hysteresis type.

3.5 Design of Pier and Piles

Design of pier and pile, as shown in Figure 3, comply with AASHTO, 2020 provisions for seismic design category D.

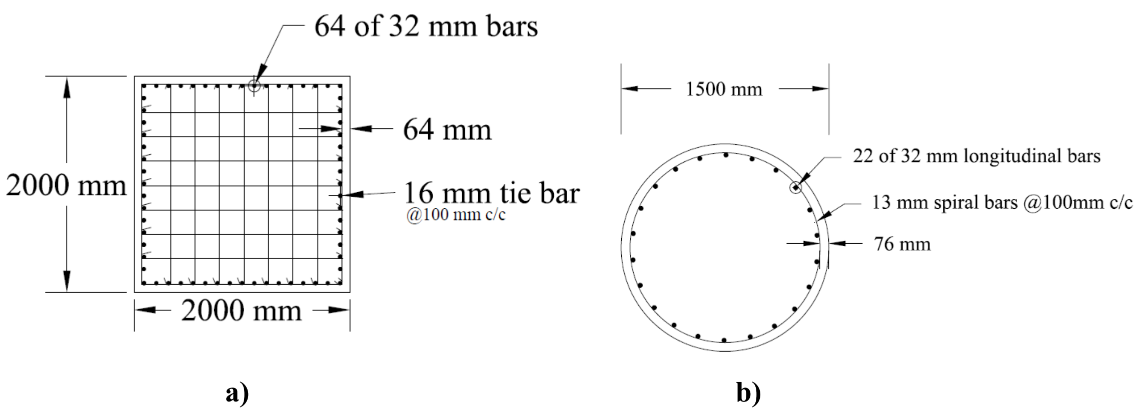
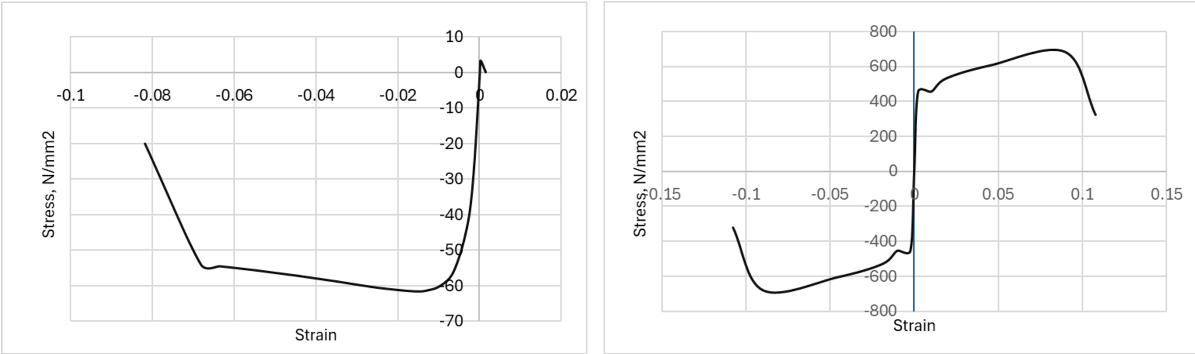


Figure 3: Cross Section Detailing of a) Pier and b) Pile

3.6 Hinge Property Modelling

Plastic hinge length at pier bottom is determined according to article 4.11.7 of AASHTO guide specification for seismic design. The stress-strain σ−ε relation are defined, as shown in Figure 4, separately for confined concrete (core—Mander et al. 1988), unconfined concrete (cover—Mander et al. 1988), and reinforcing steel (ASTM A706).



a) b)

Figure 4: Material stress-strain relation for a) confined concrete (Mander et al. 1988) and b) Reinforcement steel (ASTM A706)

3.7 Validation of Concentrated Plasticity Models of RC Bridge Pier

A reinforced concrete bridge column test was selected from the PEER Structural Performance Database (2003). The selected test is part of the experimental program conducted at PEER (Calderone et al. 2000). In this section, the cyclic force-displacement response of the analytical model is compared with the experimental results. The purpose of the validation is to assess how accurately the performance of the RC columns can be predicted using SAP2000 fiber hinge modelling, especially when testing large column samples is not feasible. The experimental column specimen no. 328 from this program is selected for modelling. Table 2 and Table 3 list the properties of tested column specimen. Figure 5 shows the lateral load time-history in terms of target displacement ductility values.

Table 2: Specimen property of test column (Calderone et al. 2000)

Specimen No.	**Height**	**Diameter**	**Long bar**	**Transverse Steel inside P.H. zone**	**Transverse Steel outside P.H. zone**
328	6'-0"	2'-0"	28 – No. 6	No. 2 @1" pitch	No. 2 @1" pitch

Table 3: Specimen property of test column (Calderone et al. 2000)

Specimen No.	**f'_c (MPa)**	**f_{ym} (MPa)**	**f_{um} (MPa)**	**ε_{sh}**	**ε_u**	**f_{yhm} (MPa)**
328	34.5	441.3	634	0.02	0.14	602

Δ_y=14.88 mm

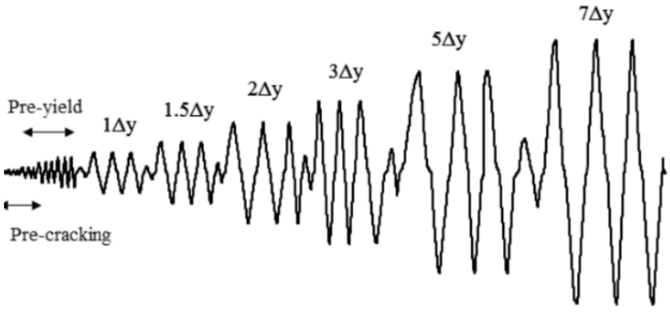
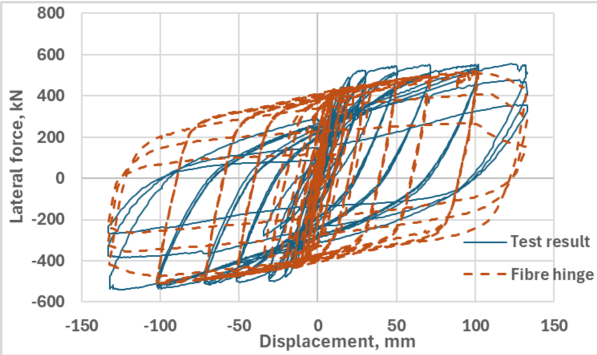
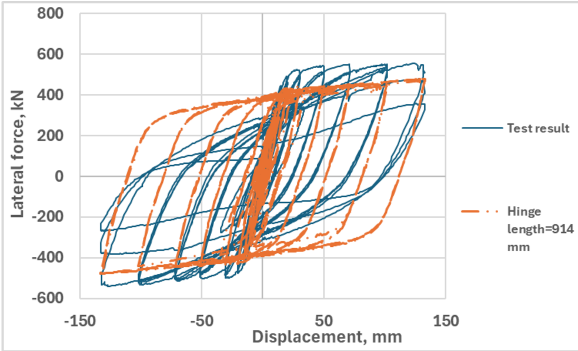


Figure 5: Imposed lateral displacement time history (Calderone et al. 2000)



a)



b)

Figure 6: Lateral Load vs Displacement from load test and SAP2000 fiber hinge model a) hinge length =376 mm; b) hinge length= 914 mm

It is evident from Figure 6 that fiber hinge can approximately simulate the actual nonlinear behavior and hysteresis characteristic. It is also seen that hinge length effects the degradation of hysteresis loop. If hinge length is assigned such as the case in Figure 6. (a), then almost realistic nonlinear behavior can be achieved.

3.8 Time history analysis using selected ground motion

For Sylhet district, recorded maximum earthquake magnitude varies within 6-7. Due to lack of locally recorded seismic motion data, well recorded ground motion data is acquired from 'PEER Ground Motion Database'. These ground motion data comply with the site-specific criteria of Sylhet.
The ground motions are scaled using spectral matching methods using SAP2000. For this case study, target spectra at service level, design level and maximum credible earthquake level are considered following AASHTO and BNBC. Spectral matching is performed on time domain which preserves the original ground motion phase characteristics. A total of 21 analysis were performed on the model (7 records x 3 hazards level). Key characteristics of the selected ground motions are shown in Table 4. The two horizontal components of each ground motion were applied simultaneously to longitudinal and lateral directions of the model. A 5 % damping ratio was considered. P-delta effects were included in the analysis.

Table 4: Selected ground motion records from PEER NGA-West2 database

Event	Year	Station	Magnitude
Cape Mendocino	1992	Eureka - Myrtle & West	7.01
Chi-Chi_ Taiwan	1999	TCU015	7.62
Iwate_ Japan	2008	Matsuyama City	6.9
Coalinga-01	1983	Parkfield - Cholame 1E	6.36
Loma Prieta	1989	APEEL 10 - Skyline	6.93
Niigata_ Japan	2004	FKS030	6.63
Northridge-01	1994	Antelope Buttes	6.69

The target response spectra are defined at three different levels of earthquake, service level earthquake (SLE), design basis earthquake (DBE), maximum credible earthquake (MCE) as shown in Figure 7. Target spectra are defined following seismic design guidelines of AASHTO.

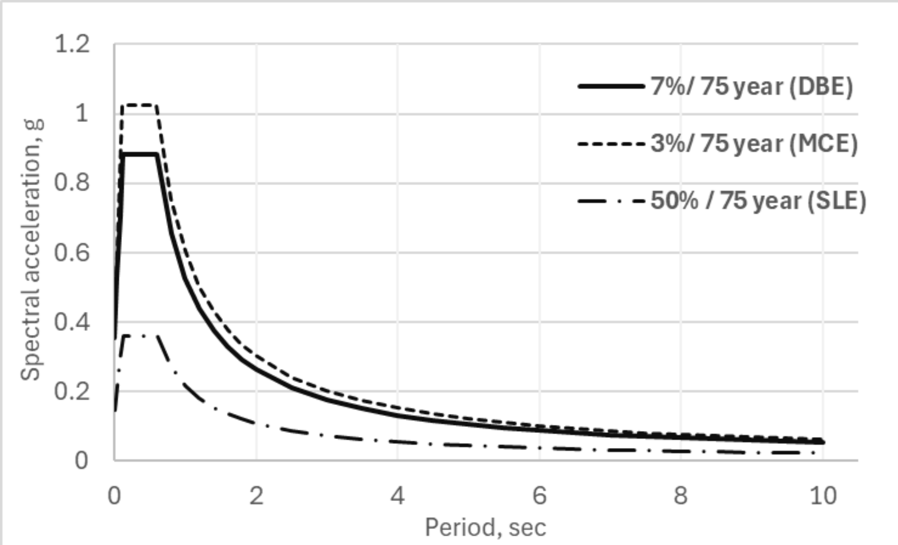


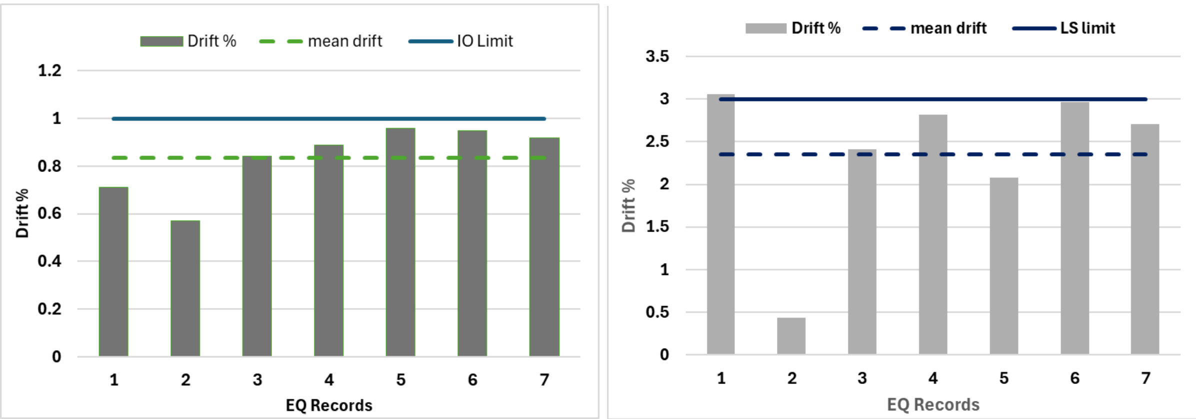
Figure 7: Design Spectra for site class D at 3% PE/75year, 7% PE/ 75 year, 50% PE/ 75-year hazard level

3.9 Performance Evaluation

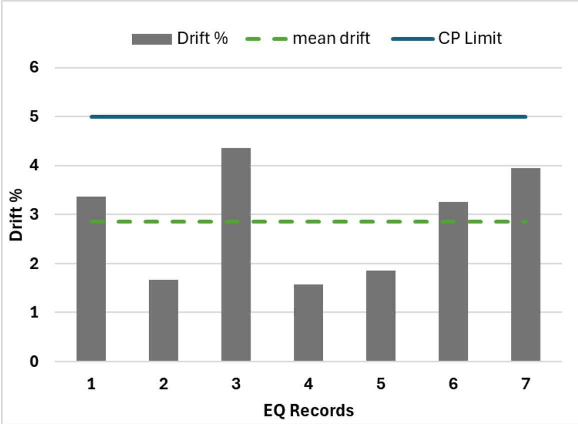
The bridge substructure is designed to act like cantilever in both longitudinal and lateral direction. The fundamental period of the structure in both longitudinal and lateral direction is 2.35 seconds. So, the difference in response between two directions is not significant. For a conventional bridge, it is required for drift% to be within immediate occupancy criteria at service level earthquake. Also, drift should be within life safety, collapse prevention at DBE and MCE level respectively. As per the results, shown in Table 5, the structure satisfies the criteria.

Table 5: Performance check in terms of global displacement (drift%)

	Hazard level	Drift %	Damage level (Hose et al. 1999)
NTHA	3% in 75 years	2.86	Life safety
	7% in 75 years	2.35	Life safety
	50% in 75 years	0.83	Immediate Occupancy



a) 50% in 75 years (SLE) b) 7% in 75 years (DBE)



c) 3% in 75 years (MCE)

Figure 8: Drift ratio% of the bridge pier from NTHA at different ground motion records for different hazard levels

It is noticeable from Figure 8 that the mean drift% at SLE is under immediate occupancy whereas mean drift% is under life safety condition at both DBE and MCE level. To evaluate ductility of the structure, pushover analysis is done. FEMA-440 Equivalent Linearization method was adopted to determine ductility demand which are shown in Table 6.

Table 6: Performance check in terms of local strain

NTHA	Hazard level	Reinforcement tensile strain	Ductility Demand, μ_D	Damage level (Hose et al. 1999)
	3% in 75 years	0.00088	1.475	Life safety
	7% in 75 years	0.014	1.31	Life Safety
	50% in 75 years	0.018	1	Immediate occupancy

The ductility demand is $\mu D = 1.475$ at MCE level which is within allowable limit of collapse prevention ($\mu CP = 6$). Similarly, SLE and DBE level ductility demands are also within allowable limits of performance objective criteria.

It can be concluded that designing the bridge according to (AASHTO, 2020) design provision incorporating seismic hazard coefficient from (BNBC, 2020) is sufficient to meet the prescribed safety and performance standards. This approach is a reliable framework for deterministic performance assessment of bridges.

4. CONCLUSIONS

The bridge substructure designed for Bangladesh's seismic conditions following (AASHTO LRFD, 2020) exhibits satisfactory performance and meets the required performance objectives. From nonlinear time history analysis, it is evident that performance criteria can be achieved if piles are designed following damage protection design (at hinging force) rather than conventional elastic design. The damage protected design is not only more effective but also more economical. This study shows that

(AASHTO LRFD, 2020) can be successfully adapted for bridge design in Bangladesh as there is no national code dedicated to bridge design.

There are several areas where further research and development can be done. A probabilistic performance assessment framework can be adapted which accounts for uncertainties in modelling and structural response. Soil-structure interaction can be done using more advanced finite element software like OpenSees, PLAXIS, FLAC3D, Abaqus etc.

5. DECLARATION OF USE OF AI

The authors confirm that no artificial intelligence (AI) tools were used at any stage of this work. The research design, data analysis, interpretation of results, and preparation of the manuscript were conducted entirely by the authors without assistance from AI-based technologies.

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