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Seismic analysis of 2nd Meghna bridge with unified foundation and steel superstructure

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ABSTRACT: Government of Bangladesh is implementing three new bridges construction along Dhaka-Chittagong Highway. A comprehensive study on 2nd Meghna Bridge is introduced herein as an example. The 2nd Meghna is designed with 930m in length and 4-lane in width having continuous narrow box steel girder type superstructure and SPSP foundation integral with that of existing. The existing Meghna is PC box girder type bridge which necessitates seismic-retrofitting according to the updated BNBC (2006). Moreover, the mode of vibration and the natural frequency of 2nd bridge and those of existing bridge are different under seismic loading due to different material application and their different structural system. All these necessitate on carrying out an accurate seismic analysis based on 3D integral model in which the stiffness of unified foundations is expressed by that of 3D equivalent spring and their superstructure and substructure are presented by elastic beam elements. The optimized response, obtained from spectrum and dynamic analyses, is used to design the structure members.

1 INTRODUCTION

Dhaka-Chittagong National Highway No.1 (N1) is the lifeline of economy of Bangladesh, which are under widening into 4-lane to cope with higher traffic demand. But, the existing 2-lane Meghna, constructed in 1991, is becoming bottlenecks against the widening of N1. Therefore, to accommodate the higher traffic volume, the 2nd bridge with 4-lane is planned to construct next and parallel to the existing bridge.

The existing bridge was designed in accordance with the outdated seismic design standard with the seismic acceleration coefficient of 0.05; however, the value has been increased to 0.15 in BNBC (2006). Moreover, the Meghna riverbed is subjected to severe scouring since after construction due to the turbulence in river flow. The superstructure has been deteriorating; particularly the hinge bearings and expansion joints are losing their proper function due to insufficient maintenance. All these facts above make an urgency to retrofit the entire bridge components so that they can sustain against strong earthquake shaking and severe riverbed scouring.

The 2nd bridge is continuous narrow box steel girder type monolithic with Steel Concrete Composite (SCC) deck slab. The foundation of 2nd bridge is designed with the form of Steel Pipe Sheet Pile (SPSP) and their foundations are appraised to integrate with those of existing so that they can form a closed well shape and make geometric conformity with existing one.

The superstructure of 2nd bridge is steel type whereas that of existing bridge is rigidly connected PC box girder type having hinge and expansion joint at center of each span. Due to their two different material application and different structural configurations, their mode of vibration and natural frequency are different under seismic loading. Taking these facts into consideration, their response is analyzed using 3D integrated model in which the stiffness of unified foundations is expressed by that of 3D equivalent spring and their superstructure and substructure are modeled by elastic beam elements. Three types of analyses such as BNBC (2006) based response spectrum analysis and dynamic analysis without/with HBR bearing damping are carried out to predict structural response. For latter two cases, a design acceleration waveform, which is fitted with the shape of 1968 Hyuga-nada Earthquake wave (Lvel-1 Type-II soil profile) recorded by JRA (2012), is applied at each pier base. The magnitude of waveform with respect to periodic time is confirmed with that from BNBC (2006). The design of bridge substructure and foundation is carried out based on optimum level of performance obtained from those three methods.

2 DESIGN EARTHQUAKE LEVEL

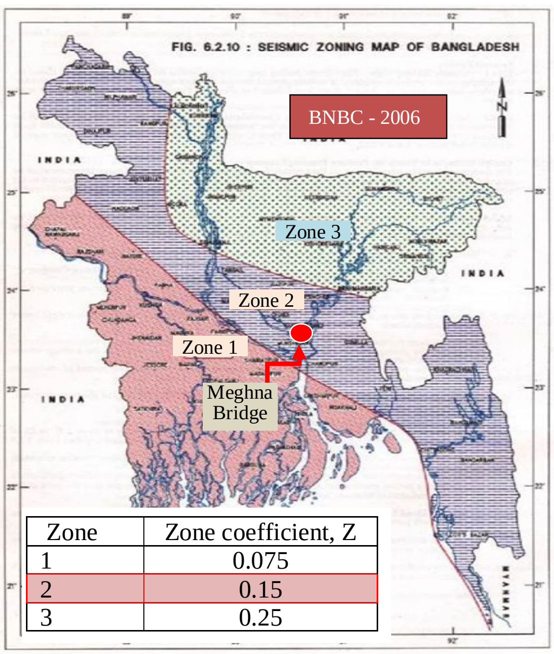
2.1 *BNBC*

(i) *Seismic zone*

The seismic zones are defined in the Bangladesh seismic zoning map (Fig.1). Based on the severity of the probable intensity of seismic ground motion and damages, Bangladesh is divided into three seismic zones (BNBC-2006), i.e. Zone 1, Zone 2 and Zone 3. These three zones along with their zone coefficients are tabulated in Fig. 1. The Meghna Bridge is located under Zone 2, which represents that the bridge should be designed under moderate severe earthquake load with a coefficient of 0.15.

(ii) *Site coefficient*

The parameter site coefficient is determined based on site soil characteristics. According to BNBC, there are four types of soil such as S1, S2, S3 and S4 that are classified based on the depth of the soil, shear wave velocity and soil type. The coefficients are specified in Table 1. In accordance with geological data surveyed for this project, the soil profile under the Meghna Bridge site is classified with soil type S3. Accordingly, the value of S3 is determined at 1.5.



Zone	Zone coefficient, Z
1	0.075
2	0.15
3	0.25

Figure 1. Seismic zoning map

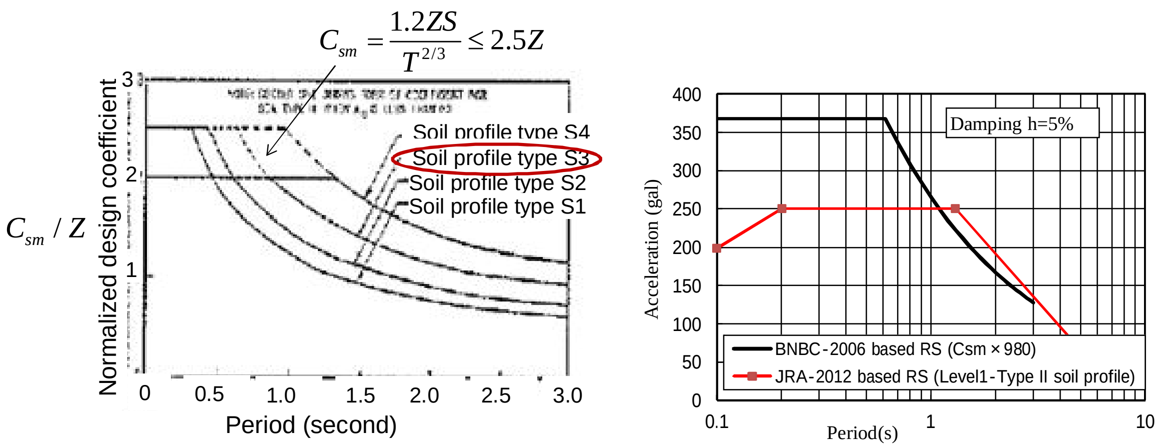
Table 1. Site coefficients according to soil profile

Type	Site soil characteristics	Coefficient, S
S1	A soil profile with either: A rock like material characterized by shear wave velocity greater than 762 m/s or by other suitable means of classification or, stiff or dense soil condition where the soil depth is less than	1.0
S2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 61m.	1.2
S3	A soil profile 21 m or more in depth and containing more than 60 m of soft to medium stiff clay but not more than 12 m of soft clay.	1.5
S4	A soil profile containing more than 12 m of soft clay characterized by shear wave velocity less than 152 m/s.	2.0

(iii) *Design Response Spectrum (RS)*

BNBC has a provision on design Response Spectrum whose magnitude is almost equal to the magnitude of the response spectra proposed by AASHTO LRFD (2007). The design response spectra are formulated in Fig. 2 (a), in which T_m = Periodic time of m^{th} mode vibration, C_{sm} = Design seismic coefficient.

The Response Spectra shown in Fig. 2(a) were derived based on the return period of 475 years as the specification of AASHTO. The RS corresponding to Soil profile S3 is used to predict the earthquake level which is expressed by design seismic coefficient C_{sm}. A comparison between the level of earthquake corresponding to S3 by BNBC (2006) and that represented by JRA (2012) is compared in Fig. 2(b). It makes a clear sense on the level of earthquake intensity and identifies that the seismic acceleration by BNBC (2006) is found to be higher by about 50% in short-periodic region, compared to Level-1 Type- II soil profile recorded by JRA (2012).

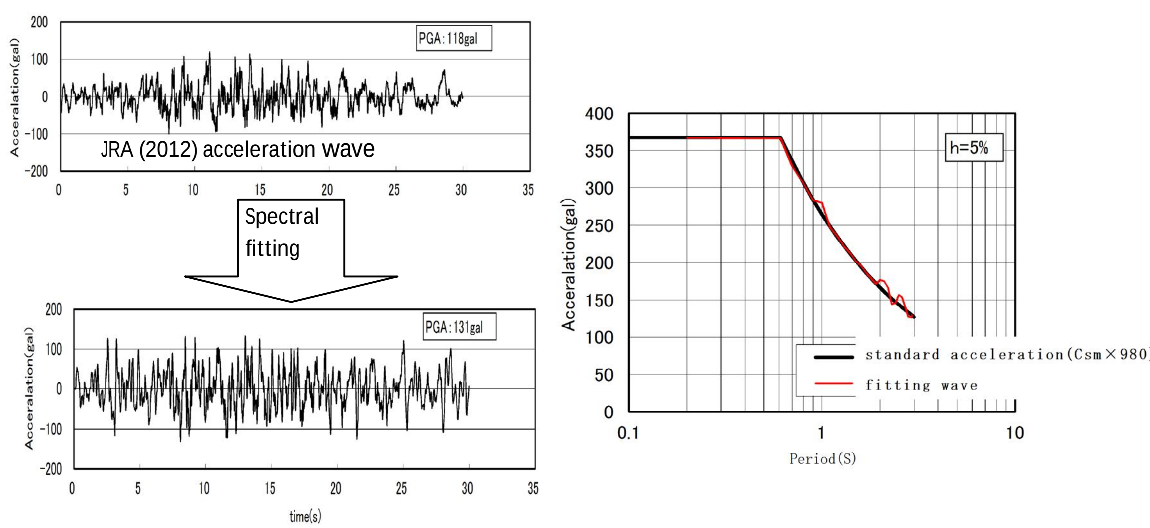


(a) BNBC based RS (b) BNBC and JRA based acceleration magnitude

Figure 2. Design Response Spectra

2.2 *Derivation of Design Earthquake Waveform*

Due to non-availability of earthquake waveform in BNBC, the design wave is derived from Japanese record following the curve fitting method. To this regard, the 1968 Hyuga-nada Earthquake wave (referred to as Lvel-1 Type-II soil profile) having Peak Ground Acceleration (PGA) of 118gal is taken as reference. The PGA of design wave is determined at 131gal (Fig. 3(a)) with adjustment of targeted frequency domain. Their accuracy is also confirmed by the comparison between the acceleration of fitting wave and the standard acceleration by BNBC. The details are shown in Fig. 3(b).



(a) Derived design acceleration waveform (b) Accuracy of design waveform

Figure 3. Derivation of design acceleration waveform

3 BRIDGE STRUCTURE AND SEISMIC MODEL

3.1 *Type of Existing Bridge*

The existing 2-lane Meghna is 13-span bridge with a total length of 930m and maximum span of 87m for 9 spans. Its 11 spans from Dhaka side are PC box type having hinge and expansion joint at the mid of each span. The end 2 spans at Chittagong side are PC-T girder type. As a part of rehabilitation work under this Project, all damaged expansion joints are planned to replace by new one, and also in consideration of thermal stress, the damaged hinges except that at the center of P5-P6 span will be fixed by filling concrete into the embedded space and placing PC cables. The remaining hinge (pot bearing) at the center of P5-P6 is planned to replace by new one with complete set.

Perspective of existing Meghna Bridge

3.2 *Type of New 2nd Bridge*

The 2nd Meghna is a 12-span continuous bridge with total 930m in length having 87m maximum span. The superstructure is consisting of three continuous narrow box steel girders which are monolithically joined with Steel Concrete Composite (SCC) deck slab. The steel girders are designed with 1.5m in width, 3.3m in height and their bottom end is supported by High Dumping Rubber (HDR) bearing placed at pier cap. They are connected transversely by I-shaped cross beams. The expansion joint is provided at both ends near abutment. All piers are designed with form of wall shape in order to ensure least disturbance to river water flow.

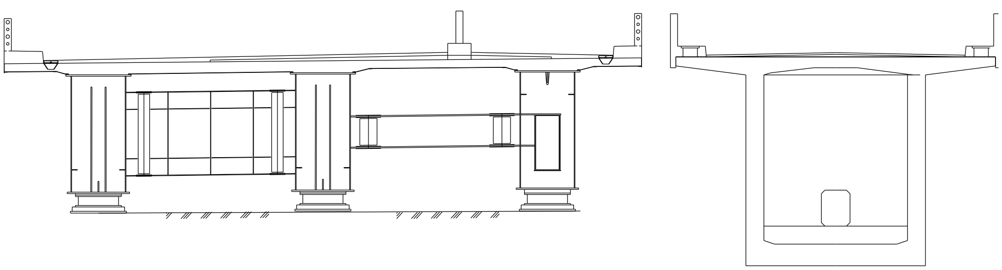


Figure 4. Cross section: Meghna Bridge

3.3 *Type of Foundation*

Two types of foundations such as Steel Pipe Sheet Pile (SPSP) type and Cast-in-place (CIP) RC pile type are taken into consideration for 2nd bridge design. The severe riverbed scouring and the design earthquake force are considered as governing factors for the selection of SPSP foundation. In consideration of above two factors along with the necessitating of seismic-retrofitting for existing bridge foundation, the foundations of P2~P10 are designed with unified SPSP closed to well shape. Beside these, the remaining pier foundations (P1, P11) and foundation of abutments (A1, A2) are designed with the form of CIP RC pile. The RC pile foundation of latter functions independently due to their non-integrity with existing one.

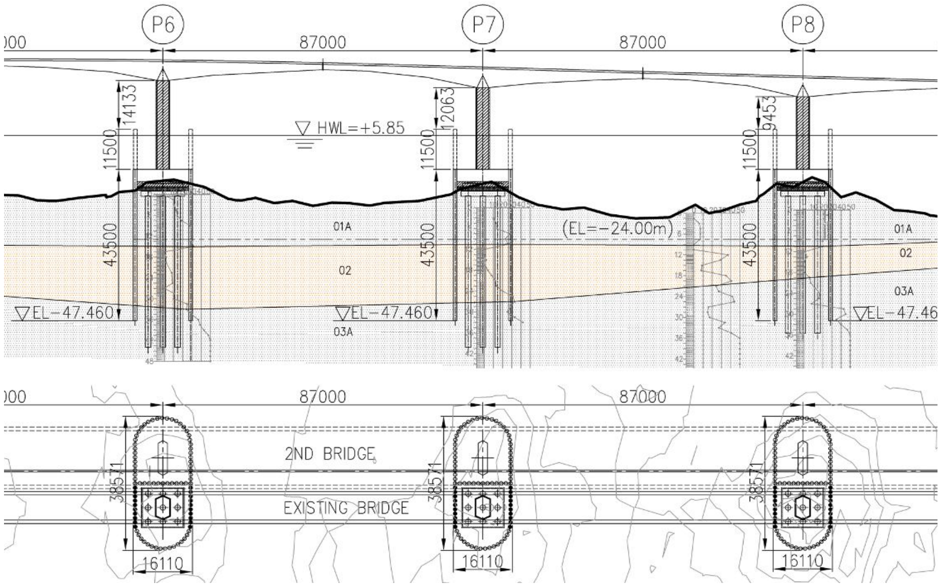


Figure 5. Meghna Bridge foundation design

3.4 *Finite Element Model for Seismic Analysis*

The nonlinear seismic analysis of 2nd bridges and the retrofitting of existing bridges are carried out based on a 3D integrated model which is shown Fig. 6. In the modeling, the superstructure of 2nd bridge and existing bridge is represented by elastic beam element. Particularly for 2nd bridge, three narrow box steel girders are simplified into single elastic beam, the cross beam on the supports are represented by elastic beam and pier top by rigid beam. Three elastic rubber bearings are modeled by spring elements having horizontal and rotational degrees of freedom (DOF), however, they are considered with High Damping Rubber (HDR) bearing represented by vertical spring with one or two order higher stiffness.

The substructure is modeled by elastic beam element, whereas the unified SPSP foundation is expressed by 3D springs having stiffness equivalent to that of SPSP. The new footing over the existing pile cap is modeled by rigid element.

In case of existing bridge, all center hinges except at the center of P5-P6 span will be fixed as part of rehabilitation works. Therefore, PC box girder is segmented into two which will be connected by center hinge remaining. The hinge section remaining is modeled by the roller support having translation in X-longitudinal direction, rotation about Y and Z axes but that about X axis is fixed. Concentrated masses are applied as equivalent masses of superstructure and substructure respectively.

The dynamic analysis is carried out for the entire bridge structure. The commercial FEM package software TDAP III (3-dimensional dynamic analysis program) developed in Japan is used. The design seismic acceleration waveform is applied at each of the pier base. Moreover, the results obtained from seismic analysis under following three analyses cases are examined;

(a) Response spectrum with elastic element in the modeling,
(b) Dynamic analysis with acceleration waveform applying at the base of nonlinear pier and obtaining *M*- f relation,
(c) In addition to (b), the hysteretic behavior of HDR bearing used in the spring modeling and obtaining *M*- f relation.

The hysteretic behavior of nonlinear spring used in the analysis is shown in Fig.7, in which the initial and secant stiffness are determined from rubber properties. The *M*-φ relation is examined as a reference at Pier base P6 of existing bridge, which is shown in Fig. 8. It exhibits nonlinearity in their relation and includes the same trend assumed in spring behavior.

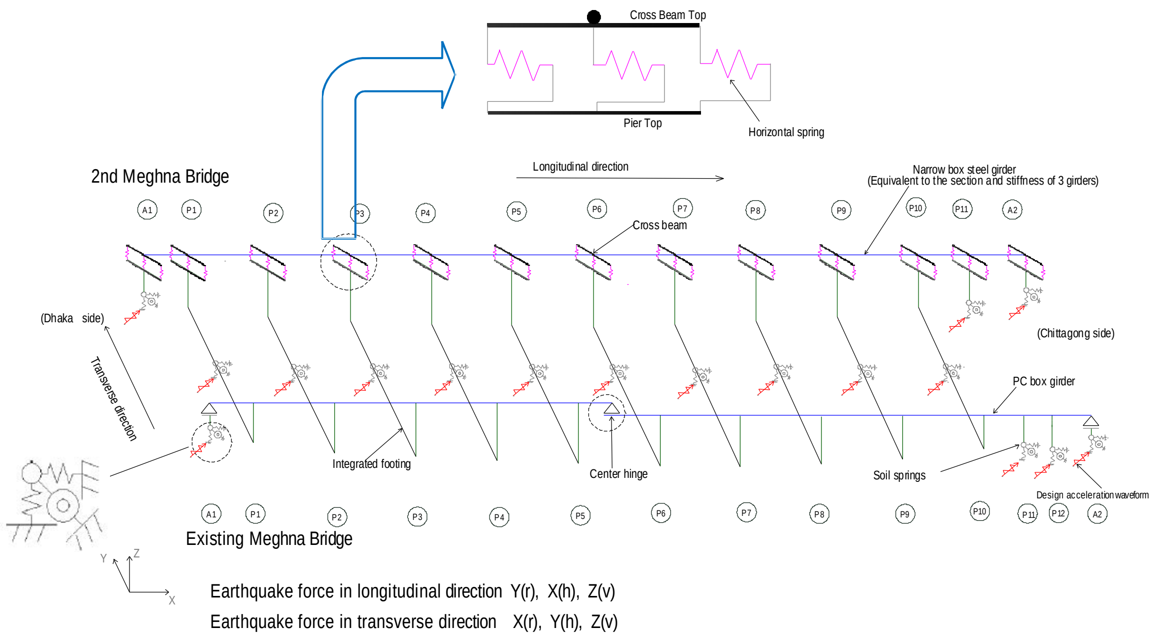


Figure 6. FEM based seismic response analysis model

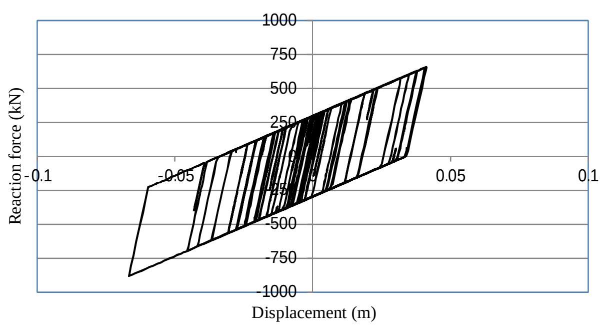


Figure 7. Hysteretic behavior of nonlinear spring

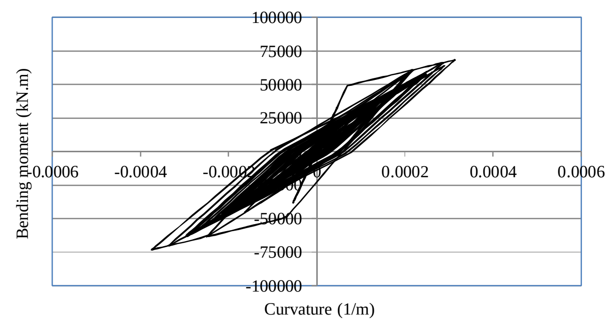


Figure 8. M-f relation obtained at Pier base P6 (existing)

4 VIBRATION CHARACTERISTICS

Based on the assumption of entire bridge structure as elastic body and their modeling by linear spring, the eigen value analysis is carried out. The natural vibration mode from low-ordering, having higher effective mass ratio, is obtained for 2nd bridge and existing bridge along longitudinal direction and transverse direction. The characteristics of only distinct vibration are shown in Table 2. It represents that 1st mode of vibration occurs along longitudinal direction of 2nd bridge, whereas 2nd mode and 4th mode of vibration occur along its transverse direction. Moreover, their vibration mode exceeds the natural period of rubber bearing with an order of 2 sec. But, due to the existence of rigidly connected PC box girder, the natural period of existing bridge along longitudinal and transverse directions shortens with an order of 0.7~0.6 sec respectively. Of which, 18th mode and 19th mode of vibration occur along the transverse direction and 23rd mode of vibration repeats along longitudinal direction. In addition, the 2nd bridge repeats its vibration at this mode along longitudinal direction.

Table 2. Vibration characteristics of new 2nd bridge and existing bridge

Mode	Natural period (s)	Effective mass ratio	New 2nd bridge		Existing bridge	
			Longitudin-	Transverse	Longitudin-	Transverse
1	2.2	47%	○			
2	2.1	35%		○		
4	1.9	6%		○		
18	0.7	2%				○
19	0.7	10%				○
23	0.6	22%	○		○	
25	0.6	4%	○		○	

5 RESPONSE SPECTRAUM ANALYSIS AND DESIGN

5.1 *Damping Properties*

In accordance with JRA (2012) guidelines, the modal damping coefficients of bridge structure members are determined, which are summarized in Table 3. Based on JRA recommendation necessary for response spectral analysis, 1st to 4th mode of vibration occur with a modal damping coefficient of around 0.04, 18th mode of vibration with that of 0.06, and 22nd~24th mode of vibration occurs with that coefficient in the range of 0.06-0.07. In the dynamic analysis with acceleration waveform whether Rayleigh damping is included in the hysteresis of bearing spring, the above defined vibration modes along longitudinal and transverse directions are used. The Rayleigh damping properties used for the analysis are shown in Fig. 9(a) and (b).

Table 3. Damping coefficient of structural members

Structure member	When structural members modeled as elastic element	
	Existing bridge	New 2nd bridge
Superstructure	0.03 (concrete structure)	0.02 (steel structure)
Bearing	-	0.03 (In case of dynamic analysis with nonlinear hysteretic behavior, damp-
Pier	0.05 (In case of dynamic analysis with nonlinear hysteretic behavior, damping=0.02)	
Foundation	0.2	

Table 4. Structural response by analysis case (M-P6)

Analysis Case	Displacement of superstructure (2nd bridge) (cm)		Bending moment at SPSP pile cap (kN.m)	
	Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction
(a) Response spectrum analysis	48	55.9	212,350	418,850
(b) Dynamic analysis	17.9	27.7	119,694	403,677
(c) Dynamic analysis with damping	9.8	6.9	97,205	332,138

5.2 *Results Obtained From Response Spectrum Analysis and Dynamic Analysis*

5.2.1 *Comparison of analysis results*

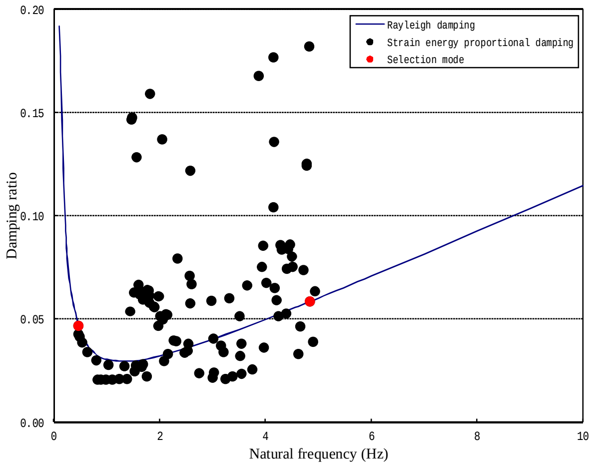
Referring to the Subsection 3.4, the analyses with three cases (a) response spectrum, (b) dynamic and (c) dynamic with damping are carrying out for comparative study. The obtained results are expressed by displacement response of superstructure as well as bending moment at SPSP pile cap top (M-P6), which are summarized in Table 4. If the results from Case (b) are compared to those from Case (c), it should be observed that the displacement response of superstructure of 2nd bridge along longitudinal direction obtained from Case (c) is reduced to 55% of that from Case (b), whereas that along transverse direction is reduced to 25%. Moreover, the bending moment at the SPSP pile cap top obtained about longitudinal axis from Case (c) is reduced to 80% of that from Case (b). This advantage effectuates on expansion joint design that can easily absorb displacement response whether HDR bearing is considered in the design under Case (c).

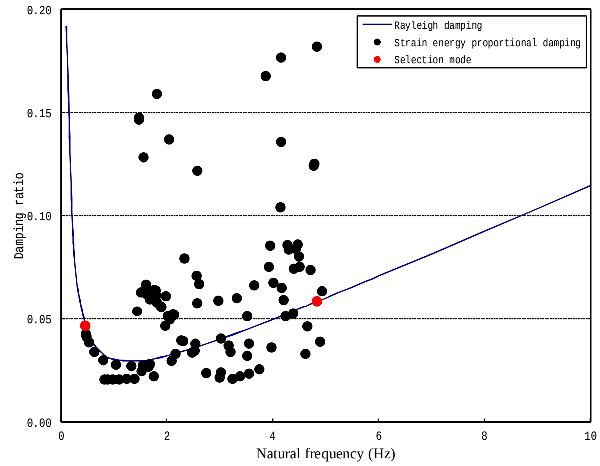
Longitudinal direction

Mode	Frequency (Hz)	Damping ratio	a	b
1	0.461033	0.0465457	0.23961422	0.003581
98	4.83657	0.058354		

Transverse direction

Mode	Frequency (Hz)	Damping ratio	a	b
2	0.467608	0.0424666	0.21842945	0.00360394
98	4.83657	0.058354		





Longitudinal direction

Transverse direction

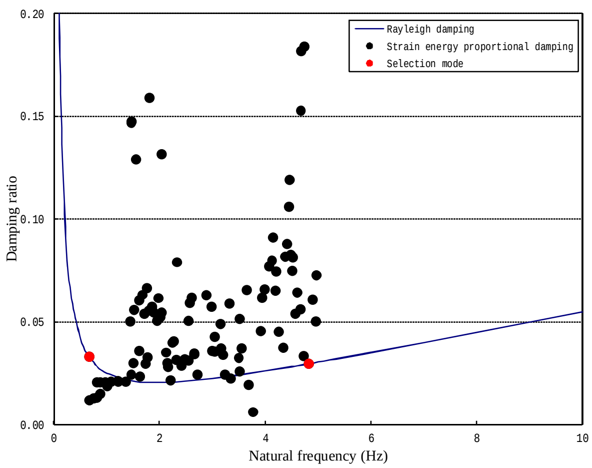
(a) Analysis Case (b)

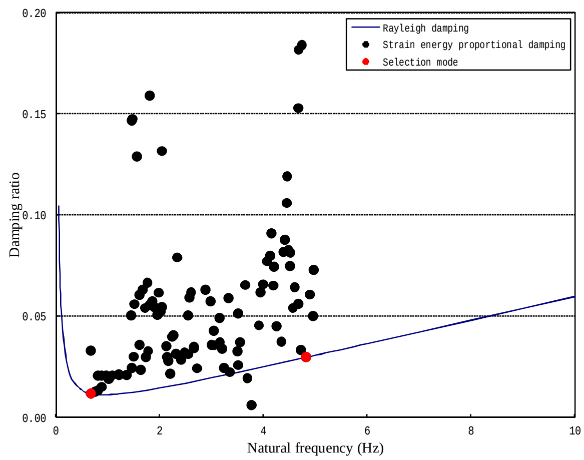
Longitudinal direction

Mode	Frequency (Hz)	Damoing ratio	a	b
1	6.730E-1	3.285E-2	0.24776	0.00168
97	4.817E+0	2.955E-2		

Transverse direction

Mode	Frequency (Hz)	Damoing ratio	a	b
2	6.766E-1	1.170E-2	0.06550	0.00188
97	4.817E+0	2.955E-2		



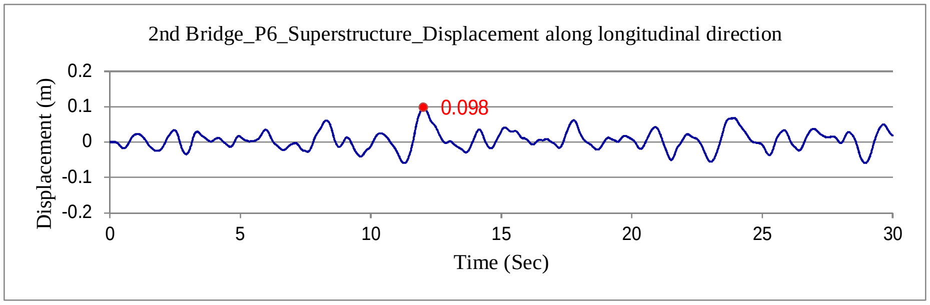


Longitudinal direction

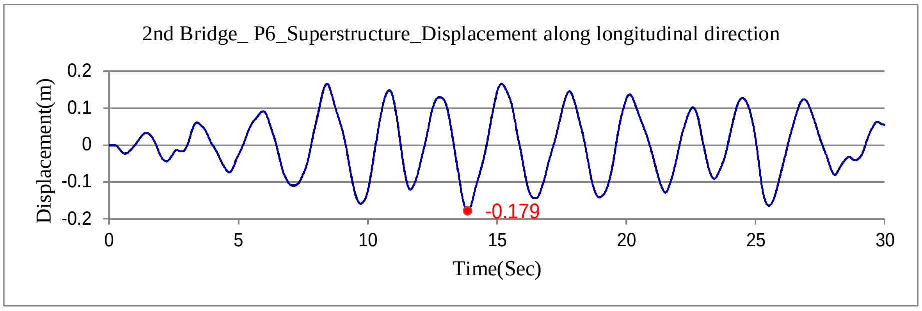
Transverse direction

(b) Analysis Case (c)

Figure 9. Relation between damping and natural frequency for HDR bearing



(a) Analysis Case (b)



(b) Analysis Case (c)

Figure 10. Superstructure displacement response (2nd bridge)

5.2.2 *Section force at the pier base*

The section force at the pier base of M-P6 along bridge longitudinal direction is summarized in Table 5. It is to be noted that the existing bridge piers is retrofitted with 25cm thick RC lining based on the section force determined hereunder. The retrofitting design with 25 cm thickness provides adequate margin against the Cases of (b) and (c).

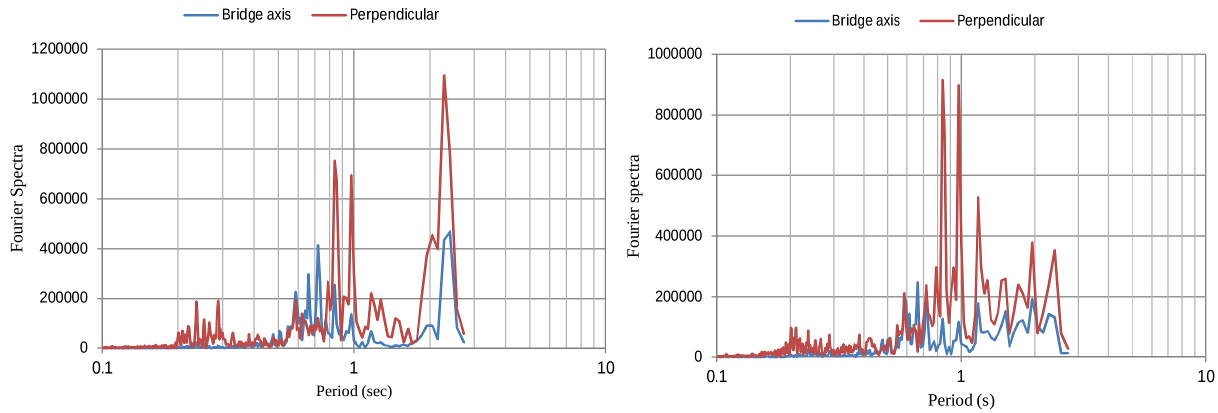
Table 5. Section force of M-P6 according to analysis case

Analysis case	2nd Bridge		Existing Bridge	
	Horizontal force (kN)	Bending moment (kN. m)	Horizontal force (kN)	Bending moment (kN.m)
(a) Response spectrum analysis	6,666	148,657	10,623	137,089
(b) Dynamic analysis	3,204	60,564	5,462	72,880
(c) Dynamic analysis with damping	2,414	51,089	5,386	71,351

5.2.3 *Reaction force at the SPSP pile cap top*

The reaction force along the longitudinal and transverse directions together with the bending moment about said axes are calculated at the top of SPSP pile cap under the three Cases (a), (b) and (c). The obtained results are expressed numerically along with by bar chart for P6-P8 in Table 6, in which the ratio of the results obtained from response spectrum (Case (a)) to that from dynamic analysis (Case (b)) is shown outside of the parenthesis. However, the detailed design of SPSP foundation is conducted based on the maximum reactions obtained.

Moreover, the numerical value shown inside the parenthesis represents the ratio of the results obtained from dynamic analysis with damping (Case (c)) to that from dynamic analysis without damping (Case (b)). In particular, the maximum bending moment about longitudinal axis obtained from Case (c) is reduced to 79~95% of that obtained from Case (b). Therefore, the detailed design of pier and SPSP foundation based on elastic rub-



(a) Analysis Case (b) (b) Analysis Case (c)

Figure 11. Fourier spectral applied on bending moment obtained at pile cap top (M-P6)

ber bearing (Case (b)) provides adequate margin against the consideration of HDR bearing in the design.

Table 6. Reaction force and bending moment at SPSP pile cap top

Direction	Reaction	Piers				
		P2-P5	P6	P7	P8	P9-P10
Longitudinal	H (kN)	0.84-1.81 (0.79-0.96)	10,000kN 1.10(0.79)	0.99(0.78)	0.96(0.74)	0.93-1.00 (0.88-0.93)
	M (kN· m)	1.46-3.57 (0.81-0.95)	200,000kN· m 1.78(0.81)	1.61(0.82)	1.48(0.79)	1.29-1.48 (0.88-0.90)
Transverse	H (kN)	0.8-1.24 (0.79-1.02)	10,000kN 1.12(0.86)	1.04(0.89)	0.77(0.85)	0.55-0.70 (0.98-0.98)
	M (kN· m)	1.01-1.31 (0.80-0.96)	400,000kN· m 1.27(0.82)	1.15(0.82)	0.92(0.76)	0.86-0.97 (0.94-1.08)

Legend: Dynamic Analysis Response Spectrum Analysis

5.2.4 *Fourier spectral analysis*

The Fourier Spectra is applied on bending moment at pile cap top (M-P6) under Case (b), and the obtained results are expressed in Fig. 11(a). The spectrum having the natural period more than 2 sec has significant effect on the vibration of 2nd bridge along longitudinal and transverse directions, whereas that having natural period less than 1 sec effectuates mainly on the vibration of existing bridge.

The peak vibration at 0.85 sec and 1.0 sec along transverse direction equals to the vibration having natural period of 0.7 sec (Table 2). Specifically, the induced period is increased due to the stiffness reduction as a result of crack occurrence in the pier.

The Fourier Spectra is also applied on the bending moment obtained at pile cap top (M-P6) under Case (c), and the obtained results are expressed in Fig. 11(b). The spectrum having the natural period more than 2 sec has significant effect on the vibration of 2nd bridge due to application of HDR bearing. The reaction at foundation is approximately reduced by 20%, which in turn, effects significantly on the section force at pile cap top of existing bridge along longitudinal and transverse directions.

6 CONCLUSIONS

A comprehensive and detailed hydraulic model for Meghna Bridge is developed in order to determine general souring at the riverbed as well as local scouring along the bridge centerline. Based on this analysis, the design scouring level is determined for P2 at -5 m.MSL, P3 ~ P5 and P10 at -15 m.MSL and P6 ~ P9 at -24 m.MSL (DD Report, 2014). The seismic analysis of SPSP foundation is carried out taking into account of these scouring depths as an input parameter. Three methods, BNBC based Response Spectrum analysis and dynamic analysis without/with damping in rubber bearing, are applied to obtain the seismic response and the foundation is designed according to the results from response spectrum.

As design summary for instance, the SPSP foundation of M-P6 is designed with a size of 16.1m along longitudinal direction and 38.57m along transverse direction, in which new pile cap with an adequate thickness covering the top of existing footing is likely to be integral to the SPSP well by shear stud connectors. The pile diameter is designed at 1.0m with 14.0mm thickness and its material is specified with SKY490. The SPSP foundation well (M-P6~M-P8) is embedded up to the level of -47.46 m.MSL, whereas M-P9 is embedded to relatively shallow depth of -39.46 m.MSL. The bottom of SPSP well is terminated at the horizon containing very dense brown and grey clayey/silty sand layer. The pier is designed with the form of wall shape in order to ensure least disturbance to river water flow.

It is observed that when the earthquake is applied along the bridge longitudinal axis, it effectuates greatly on SPSP foundation design. The reaction force at pier base obtained from response spectrum analysis (Case (a) is mostly greater than that obtained from dynamic analysis without (Case (b)) or with HDR bearing (Case (c)). Moreover, consideration of Case (a) provides some margin and safety in the design, compared to that obtained from Case (b) or Case (c).

The commencement of civil works is expected to schedule at the end of this year 2015.

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