SEISMIC PERFORMANCE EVALUATION FOR CONTINUOUS USE OF AN EXISTING BRIDGE IN NEPAL

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ABSTRACT

Road network is still the primary transportation system in Nepal, and bridges are the most sensitive part of that system. Most of the bridges were constructed about 40-50 years ago based on the design life of 50 years and traffic forecast for 30 years. The design was based on an allowable stress method where a small value of seismic design force was considered. Therefore, it is necessary to evaluate the seismic performance for the continuous use of an existing bridge. The structure was evaluated separately for the allowable stress method and ultimate capacity method as per the revised code. The seismic performance of the bridge was then estimated with the Capacity Spectrum Method and the Response History Analysis. These analyses imply that the structure has enough capacity to meet the seismic demand as per the revised code or when the double lane standard deck replaces the existing one. However, the failure mode is a shear failure, so retrofitting should be done for the continuous use of an existing structure.

Keywords: Allowable stress, Ultimate capacity design, Steel jacketing, Non-linear earthquake responses.

1. INTRODUCTION

Nepal is situated upon the Alpine-Himalayan or Alpine belt (the second most active seismic zone following the Pacific ring of fire), where 17 percent of the world's most massive earthquake occurs. Nepal is a landlocked developing country with an 83% hilly area and 17% plain area. Road network is still the predominant public transportation system in Nepal, with the limited railways and air services to some region. Road and rail networks can be realized only by constructing bridges over the obstacles. Several bridges were constructed not only in Nepal but also around the world when the seismic design method was in the development phase or was insufficient in comparison with the new standards. Before 2010, Bridge designers referred to the Indian Road Congress (IRC) guidelines and their own experience to design the bridges in Nepal based on the design life of 50 years and traffic forecast for 30 years. At several locations, those bridges are found incapable of addressing the current traffic flow. In those IRC codes, a small value of seismic design force was considered, and also there was no provision for shear design of a compression member. Therefore, their seismic capacity was difficult to estimate.

In Nepal, till now, bridges are designed using traditional methods, i.e., allowable stress method. In this method, the structural components are designed for a linear elastic case, and adequate safety is achieved by assuming significantly less value of allowable stresses, which results in relatively large sections. Due to these reasons, most of the structures designed under allowable stress have been performing satisfactorily for many years (Pillai and Menon, 2005). However, the response level of a structure due to an earthquake is poorly estimated by allowable stress method, and thus lesser value of displacement was considered in design (Teresa R. A., 2012).

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The concepts of capacity design and ductility design are needed for the analysis of structural response under the severe seismic case (Priestley et al., 1996). Due to the high importance of bridge structures, even functional damage of that structure is not acceptable in case of major earthquakes. Therefore, the structures designed according to allowable stress should be evaluated by non-linear analysis such as capacity spectrum method or response history analysis to check its seismic performance in severe earthquake conditions and propose retrofitting technique if needed. The retrofitting technique should be acceptable for developing countries like Nepal in terms of economy and technology. Therefore, steel jacketing is used here in this study as retrofitting methods of the existing structure.

2. TARGET BRIDGE STRUCTURE

The target bridge structure is a reinforced concrete cast in situ bridge of three spans, and each span is 20 m. This bridge was designed by using the working load/allowable stress method under the Indian Road Congress (IRC) guidelines (IRC 6:2000) with a pile foundation over the Dhanewa river in the Nawalparasi district of Nepal.

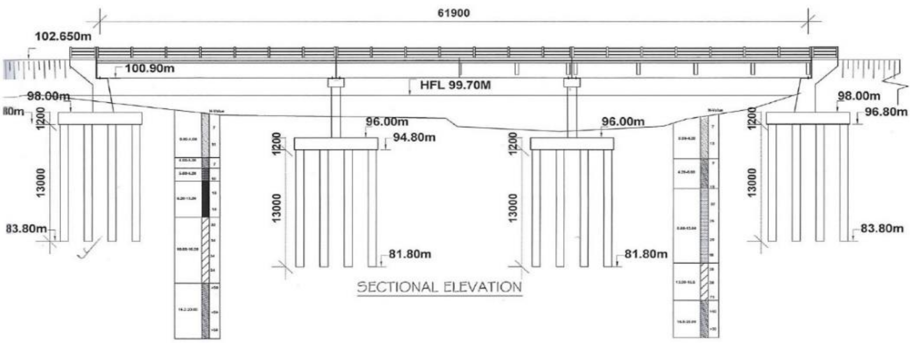

Figure 1. Bridge cross-section (longitudinal sectional elevation).

The existing bridge deck is of 6.0 m width, and the new double lane deck proposed to meet the social traffic demand is 7.5 m carriageway width with 1.5 m footpath on both sides, as shown below in Figure 2. The total height from the pier base to the acting point of the inertia force of the superstructure is 7.332 m. The identical elastomeric bearing is used on both sides of the span, which performs as free (longitudinally free, transversely restrained) and restrained (longitudinally and transversely restrained). The elastomeric rubber bearing of size 400 x 300 x 52 mm (having a damping ratio of less than 0.06, designed according to IRC 83 Part II) was used for this target structure. The bearing has lateral stiffness of 2700 kN/m, vertical stiffness of 851269 kN/m, and rotational stiffness of 8300 kNm/rad.

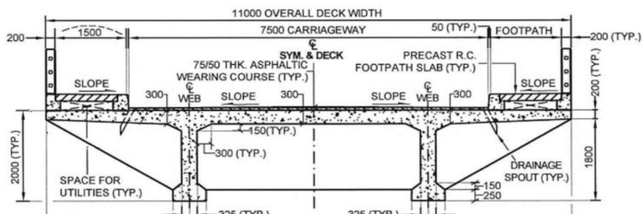

Figure 2. New superstructure section detail.

Table 1. Pier details.

Diameter of circular Pier	1600 mm
Height of Pier	4900 mm
Height of Pier Cap	1200 mm
Total number of main rebar	47 nos.
Diameter of main rebar	32 mm
Spacing of lateral ties	150 mm
Diameter of lateral rebar	12 mm
Compressive strength of concrete	25 MPa
Yield strength of rebar	500 MPa

The seismic design code was recently updated as IRC: SP: 114-2018 based on the principle of capacity design. The fundamental difference in seismic design described in IRC 6:2000 and IRC: SP: 114-2018 are listed below in Table 2.

Table 2. Seismic design criteria between IRC 6:2000 and IRC: SP: 114:2018.

Description	**IRC 6:2000**	**IRC: SP: 114-2018**
Design Method	Allowable Stress Design Method.	Ultimate Capacity Design Method
Horizontal Seismic Coefficient	αβλ α= coefficient based on location β= coefficient based on soil λ= Importance factor (1 or 1.5)	(Z/2) I (Sa/g)/R Z= Zone factor I= Importance factor (1, 1.2 or1.5) Sa/g= Average acceleration coefficient R= Response reduction factor
Vertical seismic coefficient	1/2 x Horizontal seismic coefficient	Time period of the superstructure in the vertical direction is determined. Zone factor as 2/3 of zone factor considered in the horizontal case.
Seismic force due to the live load	Considered only in the transverse direction.	Considered in the transverse direction and vertical direction.
Design Seismic Forces (F) / Moments (M)	Same as the seismic forces/moments obtained for longitudinal and transverse direction. ($F_{longitudinal} = F_x$ and $M_{longitudinal} = M_x$).	Determined by combining the seismic forces/moments obtained for longitudinal and transverse direction. ($F_{longitudinal}=F_x+0.3F_y+0.3F_z$ and $M_{longitudinal}= M_x+0.3M_y$).

In the allowable stress design method, a high factor of safety (3 for concrete and 2.5 for reinforcement) is considered. It is based on the concept that the allowable stress should be higher than the working stress. In the ultimate capacity design method, the load factor is considered a factor of safety. The ratio of the ultimate load to the capacity should be less than 1 to verify the ultimate design method. According to the revised IRC design codes, the seismic force and the total forces were evaluated for the existing superstructure and the new proposed bridge deck.

Table 3. Load Summary.

Description		**Vertical load (kN)**	**Horizontal Force (kN)**		**Design Moment (kNm)**	
			Transverse	Longitudinal	Transverse	Longitudinal
Existing deck with the old design code	Seismic load	-	312.72	312.72	1803.14	1,803.14
	Total load	3,550.98	355.48	670.31	2,076.56	4,805.31
Existing deck with the revised design code	Seismic load	594.29	714.73	575.81	4,124.60	2,904.15
	Total load	4,245.85	750.43	941.85	4,188.95	5,924.05
	Ultimate load	4,799.55	1,107.80	1,052.48	6,251.25	5,613.77
Existing deck with the revised design code	Seismic load	858.89	1,038.93	762.53	6,344.82	3,934.30
	Total load	6,084.68	1,074.63	1,103.37	8,188.77	6,759.54
	Ultimate load	7,123.87	1,594.09	1,324.63	9,937.51	7,111.48

From this table, we can see that what should be the capacity of the existing substructure so that it can meet the current seismic demand as per the revised code and if the new deck replaces the existing one to fulfill the social need.

3. SEISMIC EVALUATION

To evaluate the non-linear earthquake responses of the target bridge, numerical analyses were conducted. At first, both the fixed-base model and soil-structure interaction model was created, and the time period of the target structure was determined. The target structure is modeled as:

Pier: Beam element with its axial stiffness, bending stiffness, initial yield moment capacity and ultimate moment capacity (Takeda model)
Bearing: Spring element (K_h, K_v, K_θ) (Linear model)
Superstructure: Beam element (Takeda Model)
Expansion Joint: GAP Element
Pile: Beam element with its axial and bending stiffness (Takeda model)
Soil: Equivalent linear soil model with its lateral stiffness

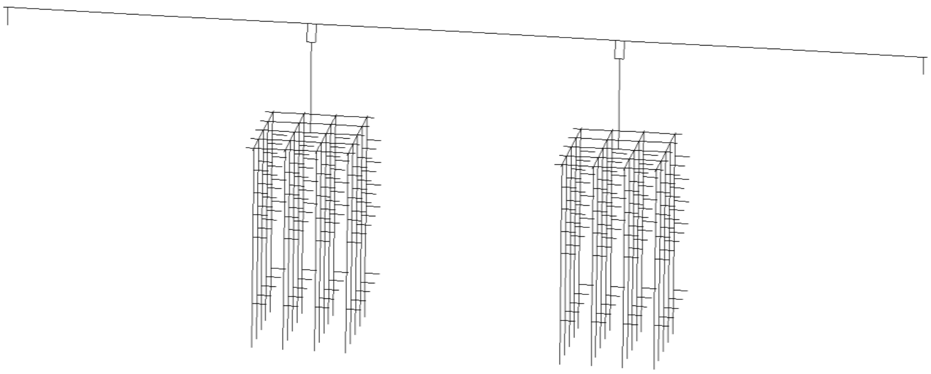

Figure 3. Numerical model.

Table 4. Fundamental natural period of the target structure.

Description (based on complete structure modeling)	Existing Superstructure		New Superstructure	
	Longitudinal	Transverse	Longitudinal	Transverse
Fixed Base Model	1.099 sec	0.398 sec	1.403 sec	0.471 sec
Soil Structure Interaction Model	1.114 sec	0.440 sec	1.423 sec	0.523 sec

The seismic performance of any structure will be greatly affected if soil-pile structure interaction is considered for analysis as it affects the natural period and damping of that structure. However, here we find that the fundamental natural period does not change significantly in the case of the SSI model, as the pile group with a cap gives a very rigid support condition. The rotational stiffness of the group piles is very high. That is why the study mainly focuses on the fixed base model.

3.1. Capacity spectrum method

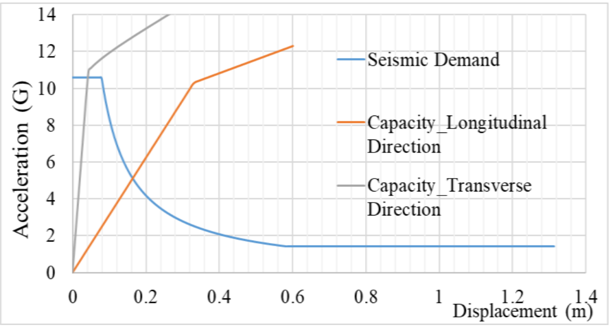

Figure 4. Capacity curve versus demand spectrum (existing superstructure).

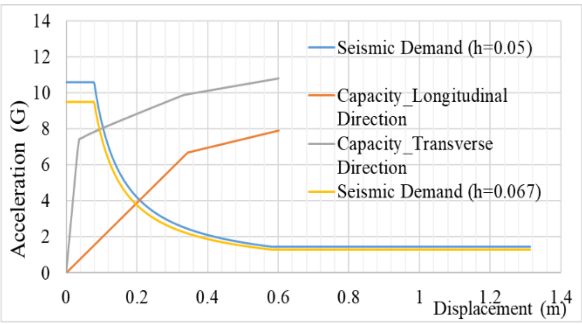

Figure 5. Capacity curve versus demand spectrum (new superstructure).

For the existing superstructure, the bridge pier can resist up-to 1.0G, while the bridge was designed under a seismic coefficient of 0.12G only. The maximum displacement is 0.33 m in the longitudinal direction, while 0.05 m in the transverse direction. Thus, the pier behaves stiffer in the transverse direction comparatively. For the new proposed structure, the pier section passed into a plastic state before reaching the maximum demand though the capacity is 0.67G.

3.2. Response history analysis

The following earthquake ground motions based on ground type (Type II soil) are considered for response history analysis as per JRA Specification. The STERA_WAVE (Dr. Taiki Saito, http://www.rc.ace.tut.ac.jp/saito/index.html) software is used to generate artificial ground motion from these earthquakes.

(a) 2011 Great East Japan (Tohoku) Earthquake
(b) 1995 Hyogo-ken Nanbu (Kobe) Earthquake
(c) 2015 Nepal (Gorkha) Earthquake
(d) Random phase earthquake

3.2.1. Analysis of longitudinal direction

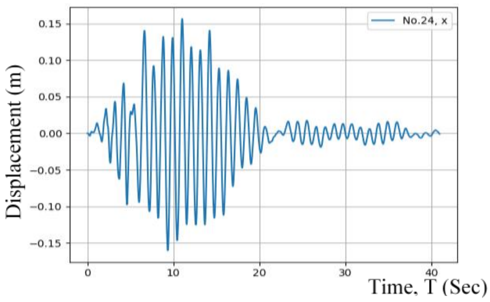

Figure 6. Displacement at bearing top.

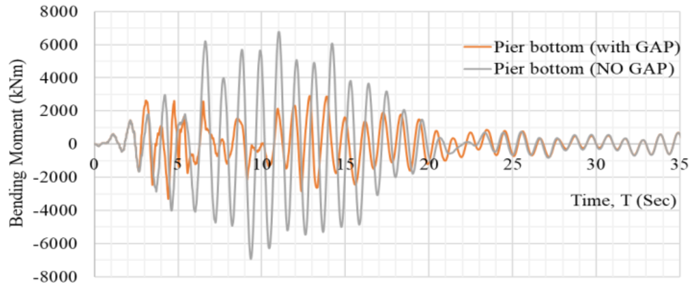

Figure 7. Time history of bending response of pier.

The bending response is in an elastic state for the longitudinal direction. The bearing top has a maximum displacement of 150 mm, while the shear displacement limit of that bearing is only 36.4 mm. It means the bearing will fail, and then the pounding will occur. The pounding force is calculated by inserting the gap element at abutment and pier location. The displacement of the pier top and the bending response of the pier are decreased when the gap element is used. It means there is no damage to the pier. As the abutment restricted the movement of the deck, the pounding force shall be borne by the superstructure only and thus damages. Therefore, restrainers or dampers should be used to limit that displacement.

3.2.2. Analysis of transverse direction

Both free and fix bearings are restrained in the transverse direction. Shear blocks are used near to bearing to restrain it. It means the complete structure behaves rigidly in the transverse direction. Due to this large stiffness, the time period of the structure becomes small, and hence the seismic load increases in that direction. The pier passed into a plastic state but still safe as the maximum curvature is in the limit.

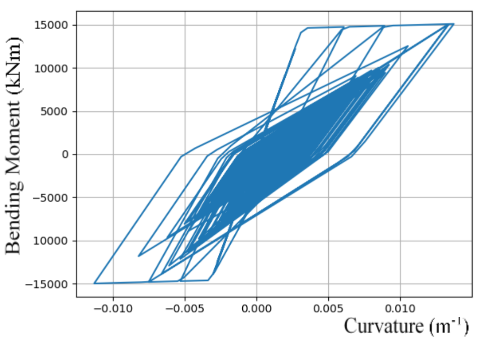

Figure 8. Maximum bending moment response under simulated the Great East Japan (Tohoku) Earthquake.

Table 5. Summary results for the Tohoku Earthquake.

Description	**Existing deck**	**New deck**
Permissible Shear strength of pier (P_s)	1807.89 kN	1807.89 kN
Permissible Ultimate moment curvature	0.015 m^{-1}	0.015 m^{-1}
Pier top displacement	0.069 m	0.121 m
Ultimate horizontal Strength (P_u)	2297.18 kN	2472.19 kN
Maximum moment curvature	0.0083 m^{-1}	0.0138 m^{-1}
Failure mode ($P_u > P_s$)	Shear Failure	Shear Failure

From the above analysis, it can be summarized that the pier has a shear failure, even the maximum curvature response is less than the ultimate curvature of the pier. The pier top displacement is also less than the limit displacement (137.9 mm).

Therefore, the structure is safe for both cases, i.e., for the load according to the new seismic code and when the new one replaces the existing superstructure. However, the failure mode is a shear failure in a transverse direction, which is not acceptable for safety purposes, and thus steel jacketing is used to increase its shear capacity. The grouting material (non-shrinkage mortar or epoxy) is used to fill the gap between the steel plate and the existing concrete pier. The shear strength (determined as per Seismic Rehabilitation of Concrete Structure, JCI, and JRA, Part V Seismic Design) increased to 3122.51 kN by using a steel plate of 2mm thickness having tensile yield strength as 345 MPa. Thus, the failure mode changed to flexure failure.

4. CONCLUSION

The seismic capacity of the target structure was evaluated according to the revised IRC code (force-based analysis) and the JRA specification (displacement-based analysis). From these analyses, the following results could be summarized as:

1. The existing bridges of Nepal, designed with allowable stress method, seem to satisfy the seismic demand as per the revised code. It is because the factor of safety for concrete and steel was considered as 3 and 2.5, respectively.
2. The existing intermediate lane deck can be replaced with the standard two-lane deck having sufficient footpath to meet the present traffic demand as the pier has relatively higher bending capacity. However, the failure mode is a shear failure, so a steel plate of 2mm thickness and tensile yield strength of 345 MPa is proposed for the continuous use of the existing substructure.
3. The maximum pier top displacement in the transverse direction was found to be 87.74% (121 mm) of its ultimate limit displacement (137.9 mm) in the case of the Great East Japan (Tohoku) earthquake for the new superstructure.
4. There will be a pounding effect due to earthquakes in a longitudinal direction, so some restrainer or damper should be used to decrease the movement. If an expansion joint of 25cm will be provided with the new standard superstructure, the pounding force can be neglected. The bearings also should be replaced with the bearings having large shear deformation capacity.

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