VERIFICATION OF PHILIPPINE BRIDGE DESIGN SEISMIC PERFORMANCE BASED ON JAPANESE SPECIFICATIONS FOR HIGHWAY BRIDGES

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ABSTRACT

The author gives importance to a lifeline structure like bridges. Verification of Philippine bridge design seismic performance based on the Japanese Specifications for Highway Bridges is the key factor. Serviceability of a bridge is very important especially after a disaster like earthquake. Serviceability of a bridge must be maintained to give way for an effective rescue operation and bringing of relief goods to the affected area. Maintaining the effectiveness of the bridge after a big earthquake is not only economical and advantageous to the government but also reduces the possibility of human casualties.

Using the January 1995 Kobe Earthquake Ground Motion (Type II-near-fault type earthquake) and March 2011 Tohoku Earthquake Ground Motion (Type I-subduction type earthquake) standard values of the design horizontal seismic coefficient for the level 2 Earthquake Ground Motion in the verifications of the Model bridge in accordance with the Japanese Specifications for Highway Bridges is more appropriate to the model bridge which is located near to an active fault, just like Kobe earthquake.

In this study, the importance of residual displacement is deemed necessary and very important in the serviceability of the bridge. Philippine bridge design is so ductile that even after a very large displacement of the column after input of Kobe and Tohoku earthquake ground motion the bridge is still on safe condition. On the other side bridges with high ductility performance, usually endure residual displacement.

Keywords: Residual Displacement.

1. INTRODUCTION

As a country located in the Pacific Ring of Fire Philippines obviously is vulnerable to disasters like earthquakes. With a booming construction industry which includes the construction of bridges, Bridge design seismic specification is very important. Bridges play a very important role in the development of the country. Although the Philippines are not totally like Japan where an earthquake is frequent, the Philippines also have a history of big earthquakes that resulted in some bridges collapsed.

Since the unpredictability of an earthquake is a key factor the author is motivated to introduce the importance of verification of the seismic performance of existing bridges in our region. One great example of this unpredictability of a big earthquake is the March 2011 Great East Japan Earthquake, which shocked not only Japan but the whole world. It was concluded that we cannot totally rely on predictions of earthquake occurrences at all times. The 2011 disaster was beyond the prediction of Japanese seismologist even in the modern era. The earthquake with a magnitude 9.0 is the strongest earthquake in the history of Japan.

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Philippines are a country of many active faults throughout the Regions. So there is a big possibility that a Kobe like earthquake will hit the Philippines someday. And because of this it just advantageous and helpful not only to the government but as well to the commuting public that a lifeline structure like a bridge is serviceable and useful even a big earthquake like the Kobe earthquake happened in the future.

2. DATA

A circular reinforced concrete column, designed in accordance with the AASHTO Standard Specifications for Highway Bridges 16th Edition 1996 and DPWH Design Criteria and Standards for Highway Bridges and Airports 2004 Edition, is considered in this study see Figure 1.

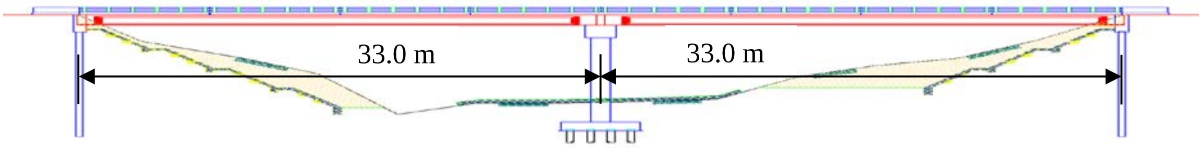

Figure 1. Maglambing Bridge Plan (Model Bridge)

The bridge is a two-(2)-span pre-stressed Girder Bridge with a total length of 66.0 linear meters, with its support fixed at both ends. It is a two-(2) way bridge with a total width of 9.34 meters including the two concrete shoulders. The bridge is supported by two (2) circular columns, which are confined columns, on the footing on piles foundation. The column diameter, D, is 1.60 m., and its height from the bottom of the column to the gravity center of the superstructure, h, is 14.68 m. The total weight, *W*, supported by the columns is 9258 kN. The conventionally designed column is reinforced longitudinally with 56 pcs. - 36 mm diameter deformed bars with 16-mm diameter spirals reinforcement used to confine the concrete core, spaced at a 100 mm at 1800mm from the top of the footing and at the top with 1800 mm from the bottom of the coping with75 mm at the center portion of the pier respectively.

In the computations of Superstructure and Sub-structure weight we consider the concrete weight ratio (γc) equal to 24 kN/m³. 34.5 MPa is used for Girder Concrete Compressive Strength and 21 MPa for the Compressive Strength of the Concrete ($f'c$) of Cast in Place Concrete, $Ec=4730\sqrt{(f'c)}$ for the Modulus of Elasticity of the Concrete, 414 MPa for Steel Tensile Strength (fy) and Es=200000 for Modulus of Elasticity of Steel.

Model bridge properties are used as a reference for the assessment of its seismic performance using the Japan Road Association Specifications for Highway Bridges, Part V, Seismic design, Japan, 2002. Please be noted that the model structure considered in this study is only one type of bridge structure in the Philippines.

It should be noted that the deck end condition of the model in this research is not considered in the analysis. Although this condition affects the behavior of the bridge significantly, the bridge is simplified to the model.

3. THEORY AND METHODOLOGY

3.1. Computation of Moment of Inertia

Comparison of bridge design seismic performances between Japan and Philippine bridge specification

Table 1. Moment of Inertia

Members	Concrete Strength (fc)Mpa	Modulus of Elasticity (Ec)Mpa	Modulus of Elasticity (Ec)kN/m^2	Area (A) (m^2)	Inertia (I) (m^4)
GIRDER	21	21675.58	2.17E+07	4.16	1.67
COPPING	21	21675.58	2.17E+07	15.58	4.69
PIER 1	21	21675.58	2.17E+07	2.01	0.32
PIER 2	21	21675.58	2.17E+07	2.01	0.32
FOOTING	21	21675.58	2.17E+07	60	125

gives us an idea how Japanese specifications give importance to the effect of the residual displacement compared to the Philippine specification. Computations of the Bridge Moment of Inertia are needed in the computation and estimating the Flexural Stiffness of Columns. The stiffness of the Columns dominates the computed performance of the bridge subjected to earthquake ground motions. In linear analysis, the stiffness controls the predictions of the period of the structure, distribution of loads within the structure and deformation demands. It also controls the yield displacement which affects the displacement ductility demand of the structure in the nonlinear analysis.

Computation of Moment of Inertia differs by its shape. For Coping and Footing which are rectangular in shape, the computation of *I* is expressed in Eq. (1). Where a is the width of the surface and b is the depth of the surface.

$$I = \frac{ab^3}{12} \qquad (1)$$

Computation of *I* for Circular column is expressed by the following Eq. (2).Where D corresponds to the diameter of the column.

$$I = \frac{\pi D^4}{64} \qquad (2)$$

3.2 Creation of Analysis Model.

Appropriate model is needed and it should be established according to the purpose of the analysis and the level of design earthquake ground motion. Modeling is very important in any analysis, of Static Analysis or Dynamic Analysis. A model should provide us the correct number of nodes and the number of elements for the subject structure that can be used in the analysis. Results from this model bridge totaled nineteen (19) nodes and the total number of elements is eighteen (18).

Table 2. Eigenvalue results

mode	frequency hz	period sec	Longitude effective mass ratio-X %	vertical effective mass ratio-Y %	% of each vibration modecontributes to the response: Longitude stimulating factor-X	% of each vibration modecontributes to the response: vertical stimulating factor-Y	strain damping %
1	0.5223	1.91	82	0	33.04	0	5.734
2	2.294	0.44	82	0	0.375	0	3.004
3	3.53	0.28	81	57	0	26.26	3.118
4	9.07	0.11	82	57	0.68	0	3.021
5	10.74	0.09	82	63	0	8.5	3.632
6	11.57	0.09	86	63	7.74	0	6.351
7	18.13	0.06	86	88	0	17.64	5.379
8	18.9	0.05	86	88	0	0	3
9	19.27	0.05	86	88	0.39	0	3.01
10	23.89	0.04	86	96	0	9.67	4.681

3.3 Computation of Eigenvalue.

Eigenvalue computation needs parameters as material properties and section properties. Eigenvalues computation is used to obtain frequency value in every mode to be used in the computation of the natural period of the bridge structure. With the given parameters of the structure, Eigenvalue computation can be calculated see Table 2 for tabulated results.

4. RESULTS AND DISCUSSION

4.1. First Verification Method

The author used two types of Earthquake Ground Motion in the verification of the model bridge design seismic performance, the 2011 Tohoku earthquake (Type I) and the 1995 Kobe earthquake (Type II). Demand force ($k_{hc}W$) must be equal to or less than the lateral strength *(Pa)* of reinforced concrete column as shown in Eq. (3).

$$k_{hc}W \leq Pa \quad (3)$$

Design horizontal seismic coefficient for level 2 earthquake ground motion(k_{hc}),equivalent weight(*W*), lateral strength(*Pa*) can be obtained by Eq. (4), (5) and (6) respectively where (*Cs*) is the force reduction factor, (*Cz*) zone modification factor equal to 1.0 for Kobe area, (*Wu*) weight of super structure, (*Cp*) equivalent weight coefficient, (*Wp*) weight of pier, (*h*) height of super structural inertia force and (Mu) ultimate bending moment.

$$k_{hc} = Cs\ Cz\ k_{hc0} \quad (4)$$

$$W = Wu + CpWp \quad (5)$$

$$Pa = Mu/h \quad (6)$$

Force reduction factor (*Cs*), standard value of the design horizontal seismic coefficient for level 2 earthquake ground motion (k_{hc0}) and ductility capacity of the reinforced concrete column (μ_a) can be obtained as shown in Eq. (7), (8) and (9) respectively where (*T*) is the natural period of the bridge, δ_u is ultimate displacement of the reinforced concrete column, δ_y is the yield displacement of pier and α is a safety factor for reinforced concrete columns resulting to flexural failure.

$$C_s = \frac{1}{\sqrt{\mu_a - 1}} \quad (7)$$

$$k_{hc0} = 1.24\ T\text{^}(-4/3) \quad (8)$$

$$\mu_a = 1 + \frac{\delta_u - \delta_y}{\alpha\delta_y} \quad (9)$$

Where the yield displacement of the pier (δ_y) and δ_u is ultimate displacement of the reinforced concrete column can be obtained in Eq. (8) and (9) respectively.is the product of the horizontal displacement at the time of yielding of axial tensile reinforcing bars at the outermost edge of the columns bottom section. (δ_{y0}) and the result from dividing the ultimate bending moment at column bottom section (Mu), with the bending moment at the time of yielding of axial tensile reinforcing bars at the outermost edge of the column's bottom section (*My0*) as shown in Eq. (10). For the ultimate displacement of reinforce concrete column (δ_u) computation can be as shown in Eq. (11), where φ_u is the ultimate curvature at the column's bottom section, φ_y is the yield curvature at the column's bottom section and L_p is the plastic hinge length and h is the height of the super structural inertia force from the bottom of the column.

$$\delta_y = \frac{M_u}{M_{y0}}\delta_{y0} \quad (10)$$

$$\delta_u = \delta_y + (\emptyset u - \emptyset y)\ Lp\ (h-(Lp/2)) \quad (11)$$

Table 3-a.First Verification Method Results (using Kobe Earthquake)

Pa	KhcW	Khc	Khc0	Cs	μa	δu	δy	W
2164kN	3890kN	0.38021	0.52143	0.72916	1.44043	0.39011	0.23492	10230kN

Table 3-b.First Verification Method Results (using Tohoku Earthquake)

Pa	KhcW	Khc	Khc0	Cs	μa	δu	δy	W
2164kN	4818kN	0.47094	0.64587	0.72916	1.44043	0.39011	0.23492	10230kN

Results for the first verification method using the two types of Earthquake Ground Motion (Type I and Type II) are shown in Table 4-a and Table 4-b respectively. For the Kobe earthquake results of 3,890.0 kN for (k_{hc} W) which is greater than the value of lateral strength of the reinforced concrete column (Pa) of 2,164.0 kN. So much with the Tohoku earthquake results which value of ($k_{hc}W$) is much bigger of 4,818 kN.

4.2 Second Verification Method

Residual displacement is the amount of retained displacement from a large ground motion shaking after an earthquake. Large residual displacement can cause difficulty in the repair process. To address this problem, residual displacement must be verified so as to be safe. Residual displacement of the column (δ_R) must be equal to or less than the allowable residual displacement (δ_{Ra}) of the column as shown in Eq. (12).

$$\delta_R \leq \delta_{Ra} \tag{12}$$

Residual displacement can be obtained as shown in Eq. (13), where CR is the modification factor on residual displacement valued 0.60 for a reinforced concrete column, μ_r is the maximum response ductility ratio of piers, r is the ratio of the secondary post-yielding stiffness to the yielding stiffness of a pier and the ratio value of 0 shall be taken for reinforced concrete column. Residual displacement can be obtained as shown in Eq. (14).

$$\delta_R = c_R(\mu_r - 1)(1 - r)\delta_y \tag{13}$$

$$\mu_r = \frac{1}{2}\left(\frac{C_z k_{hc0}}{P_a}\right)^2 + 1 \tag{14}$$

For the computation of the allowable residual displacement, (δ_{Ra}) is provided as shown in Eq. (15). Where h is the height of the super structural inertia force from the bottom of the column.

$$\delta_{Ra} = \frac{h}{100} \tag{15}$$

Table 4-a. Second Verification Method Results (using Kobe Earthquake)

δR	δRa	μr	khc0	Pa	δy
0.4282m.	0.1468m.	4.0381	0.5214	2164kN	0.2349m.

Table 4-b. Second Verification Method Results (using Tohoku Earthquake)

δR	δRa	μr	khc0	Pa	δy
0.657m.	0.1468m.	5.06612	0.5214	2164kN	0.2349m.

Zone modification factor (Cz), standard value of the design horizontal seismic coefficient for level 2 earthquake ground motion (k_{hc0}) and the lateral strength of the reinforced concrete column (Pa) values are the same value for the first modification method. See Table 5-a and Table 5-b for tabulated results.

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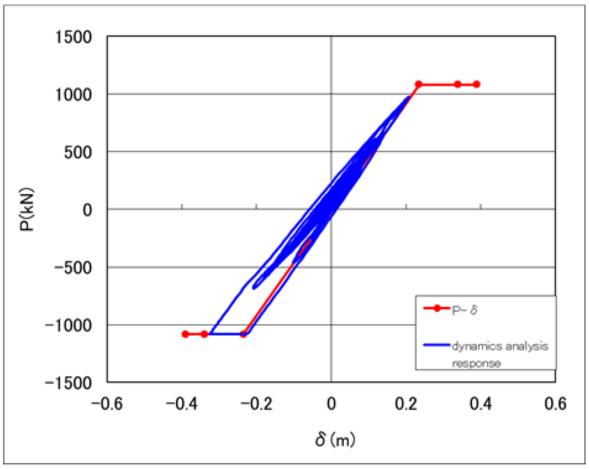
4.7 Analysis Computation Results

Dynamic response analysis results described the behavior of the column after application of the Kobe

Table 5. Dynamic Analysis Response Results

δ (Dynamic Analysis Response)	(m)	0.324
δy (Yield Displacement)	(m)	0.235
δa (Allowable Displacement)	(m)	0.338
δu (Ultimate Displacement)	(m)	0.39



δ_u=0.390 δ_a=0.338 δ=0.324 δ_y=0.235

Figure 2. Dynamic Analysis Responses

earthquake ground motion. Tabulated results are as shown in Table 5. The graph in Figure 2 explains that the bridge column displacement behaves inside the allowable displacement of the column which tells us that the bridge column is safe

5. CONCLUSIONS

Bridges are a lifeline structures, quick response to disaster and quick recovery of the area depends on the serviceability of concerned bridges. Verification of bridge design seismic performance is very important especially for existing and old bridges. For a third world country it is very advantageous to the government to preserved old bridges serviceability rather than constructing a new one. After the comparison of specifications and verification using two different earthquakes conclusions are concluded as follows:

i. Based on the dynamic response analysis the model bridge can withstand the Kobe like earthquake input ground motion. Actual displacement less than allowable displacement.

ii. Model Bridge is so ductile that even after a very large displacement at column after input of Kobe and Tohoku earthquake ground motion the Model Bridge will not collapsed.

iii. Bridges with high ductility performance, usually endure residual displacement.

iv. Based on Japanese verifications on bridge seismic performance, Model Bridge failed in both demand force verification and displacement verification. One interesting factor is the verification of residual displacement which is not included in our bridge seismic performance specification.

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