EFFECT OF REVISED DESIGN EARTHQUAKE GROUND MOTIONS CONSIDERING ZONES FACTORS ON NONLINEAR RESPONSE OF STEEL BRIDGE PIERS

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ABSTRACT

In 2012, Level 2 Earthquake Ground Motions (Type I) and 'modification factor for zones (zone factor)' in the seismic design specifications for highway bridges in Japan were revised. Design earthquake ground motions should be multiplied by zone factors to evaluate the seismic performance. Regarding one of zones, Type I design earthquake ground motions are remarkably changed from the former earthquake ground motions multiplied by "0.85" to the revised ones multiplied by "1.2". The purpose of this paper is to investigate the effect of design earthquake ground motions considering zone factors on nonlinear response of steel bridge piers. The time-history dynamic analysis on the single-degree-of-freedom system was adopted in order to identify whether the bridges have enough seismic performance. Comparing obtained maximum response displacements of two kinds of earthquake ground motions (Type I and Type II), this paper came to conclusions. The results revealed that, regarding some zones, the Type I design earthquake ground motions have generally caused larger maximum response displacement than that of Type II.

Keywords: zone factors, Type I design earthquake ground motions, nonlinear response analysis, steel bridge piers.

1. INTRODUCTION

The seismic design specifications for highway bridges in Japan (Japan Road Association 2012) have required highway bridges to be checked if the bridges satisfy target seismic performance against Level 1 and Level 2 earthquake ground motions. The Level 2 earthquake ground motions are classified into two types. They are Type I and Type II earthquake ground motions. Type I represents ground motions from large-scale plate boundary earthquakes, while Type II from inland earthquakes. After the 2011 off the Pacific coast of Tohoku Earthquake (the 2011 Tohoku Earthquake), it is recognized that bridges should have enough seismic performance to resist the earthquake which is as large as the 2011 Tohoku Earthquake. In the 2012 seismic design

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specifications, Type I design earthquake ground motions were revised. They were defined based on earthquake ground motions observed in the 2011 Tohoku Earthquake and the 2003 off Tokachi Earthquake.

Design earthquake ground motions for evaluating the seismic performance are set by multiplying zone factors because it is not reasonable to use the same earthquake ground motions for all regions which have different earthquake occurrence frequency respectively. Zone factors for Type I earthquake ground motions were also revised in the 2012 seismic design specifications. After the revision, regarding one of regions, Type I design earthquake ground motions are remarkably changed from the former earthquake ground motions multiplied by "0.85" to the revised earthquake ground motions multiplied by "1.2". There may be possibility that bridges don't satisfy target seismic performance.

A previous study examined the effect of ground motions observed in the 2011 Tohoku Earthquake on the nonlinear response of reinforced concrete bridge piers (Sakayanagi et al. 2011). However, the nonlinear response of steel bridge piers subjected to this revised Type I design earthquake ground motions considering zone factors haven't been even clear. Therefore, this paper's purpose is to investigate the effect of those design earthquake ground motions on nonlinear response of steel bridge piers based on the dynamic analytical results.

2. DESIGN EARTHQUAKE GROUND MOTIONS CONSIDERING ZONE FACTORS

In the previous seismic design specifications (Japan Road Association 2002), there had been three zones, A, B and C, with zone factors 1.0, 0.85 and 0.7, respectively. They had been commonly employed for both Type I and Type II design earthquake ground motions. After the revision of seismic design specifications, zone A is divided into two zones, A1 and A2, as well as zone B into B1 and B2, while zone C is not changed. As a result, there are five zones defined in order to evaluate the seismic performance of bridges. Zone factors for Type I earthquake ground motions, c_{Iz}, is introduced as 1.2 for zones A1 and B1, 1.0 for zones A2 and B2 and 0.8 for zone C. Table 1 represents zone factors of five regions defined in the 2012 seismic design specifications.

There are three types of the ground category defined in the seismic design specifications. They are Ground type I, Ground type II and Ground type III which correspond to stiff, medium and soft soil condition respectively. In each ground category, there are 3 waves are specified for dynamic analysis.

Table 1: Zone factor (the 2012 seismic design specifications)

Zone	Zone factor	
	c_{Iz} TypeI	c_{IIz} Type II
A1	1.2	1.0
A2	1.0	1.0
B1	1.2	0.85
B2	1.0	0.85
C	0.8	0.7

A lot of steel bridge piers are constructed on the grounds which are generally classified into Ground type II or Ground type III. Focusing on those two ground categories, two kinds of earthquake ground motions are introduced in the dynamic analysis. They are 6 waves of Type I design earthquake ground motions (I-II-1, I-II-2, I-II-3, I-III-1, I-III-2, I-III-3) and 6 waves of Type II design earthquake ground motions (II-II-1, II-II-2, II-II-3, II-III-1, II-III-2, II-III-3) specified in the 2012 seismic design specifications. Regarding the name of earthquake ground motions like 'I-II-1', the first letter expresses to the type of the design earthquake ground motion, and the second expresses to the Ground type. Those 12 waves are modified by zone factors which correspond to five zones respectively.

3. METHOD OF DYNAMIC ANALYSIS

The analytical models are defined by referring to the previous study (Okada et al. 2010). The ratio of each compressive axial force N to yield axial force N_y, N/N_y, is 5% and 15%. The models are hollow steel columns which have rectangular sections. There are some parameters which are dominant to the seismic performance of steel bridge piers defined in the seismic design specifications. They are width-thickness ratio parameters R_F and R_R, slenderness parameter $\bar{\lambda}$ calculated by the following equations.

$$R_F = \frac{b_F}{t_F}\sqrt{\frac{\sigma_y}{E}\cdot\frac{12(1-\mu^2)}{\pi^2 k_F}} \tag{1}$$

$$R_R = \frac{b_F}{t_F}\sqrt{\frac{\sigma_y}{E}\cdot\frac{12(1-\mu^2)}{\pi^2 k_R}} \tag{2}$$

$$\bar{\lambda} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{l}{r} \tag{3}$$

Where, b_F is a width of stiffened plate, t_F is a plate thickness of stiffened plate, σ_y is a nominal yield stress, E is a Young's Modulus, μ is a Poisson's ratio, k_R ($=4n^2$), k_F is a buckling coefficient, n is a number of panels divided by stiffeners, l is a effective buckling length and r is a radius of gyration of area.

The values of R_F and R_R of analytical models exist from 0.3 to 0.5. Those of $\bar{\lambda}$ exist from 0.2 to 0.5. The natural period of analytical models exists from 0.23 to 1.7. The total number of analytical models is 32.

Damping ratio of steel bridge piers is 0.01 according to the seismic design specifications. The mass of models is the sum of the weight of superstructure and 30% of the weight of the bridge pier.

Steel bridge piers are modeled into a single-degree-of-freedom system as shown in Figure 1 (a). The restoring force of steel bridge piers is represented in relationship between horizontal force and horizontal displacement (P-δ relationship). The P-δ relationship is modeled as a bilinear model illustrated in Figure 1 (b). In this figure, P_y is a yield horizontal force, δ_y is a yield horizontal displacement, P_{max} is a maximum horizontal force and δ_m is a horizontal displacement at P_{max}. As for P_{max} and δ_m, the results of elasto-plastic finite displacement analysis in the previous study (Okada et al. 2010) are used. The validity of this FEM analysis has already confirmed by comparing between the experimental results and analytical results in the previous study. As for P_y and δ_y, they are calculated by the following equations respectively.

$$P_y = (\sigma_y - \frac{N}{A})\frac{W}{L} \tag{4}$$

$$\delta_y = \frac{P_y L^3}{3EI} \tag{5}$$

Where, N is a compressive axial force, W is a section modulus, A is a sectional area and L is a height from the bottom of column to the point of generating inertia force.

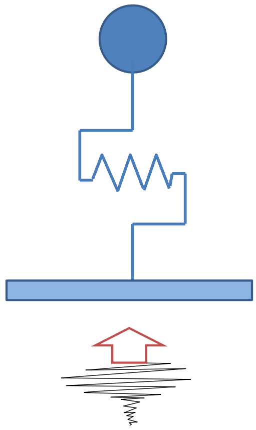

Figure 1 (a): Single-degree-of-freedom system

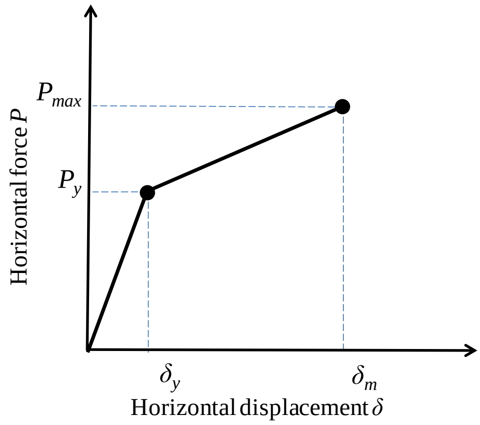

Figure 1 (b): Bilinear model

4. ANALYTICAL RESULTS

The maximum response displacements among the analytical results are focused on to estimate the seismic performance in this paper.

As mentioned above, for one of ground categories, there are 3 waves of design earthquake ground motions of Type I or Type II respectively. If the mean value of the analytical results of 3 input earthquake ground motions is kept below its allowable values, it is commonly evaluated that the seismic performance is enough. For the reason, the mean value of maximum response displacement is adopted when estimating seismic performance. In this paper, as the allowable value, δ_m which corresponds to the horizontal displacement at P_{max} is adopted according to the seismic design specifications. A mean value of the maximum response displacement with 3 waves of Type I design earthquake ground motions and that of 3 waves of Type II design earthquake ground motions are defined as ’Type I δ_{max}’ and ’Type II δ_{max}’ respectively.

Though the seismic design specifications were revised in 2012, Type II design earthquake ground motions have not been changed. If ‘Type II δ_{max}’ exceeds δ_m, it is evaluated that the model doesn’t satisfy the target seismic performance. Therefore, the models whose response is larger than δ_m by Type II design earthquake ground motions are excluded in the following investigation of the seismic performance.

In this paper, the two zones out of five zones are focused on to examine the results. One of them is A2 zone including the city area such a part of Tokyo and Osaka where a lot of steel bridges have been constructed more than those in any other area. The other is B1 zone including a part of Miyazaki and Kochi prefectures where the design earthquake ground motions have increased remarkably according to the revision of the 2012 seismic design specifications.

4.1. Feature of Maximum Response Displacement

Regarding all models on each ground, the comparisons between ‘Type I δ_{max}/δ_m’ and ‘Type II δ_{max}/δ_m’ in A2 zone and B1 zone are shown in Figure 2 and Figure 3 respectively. As mentioned above, δ_{max} is the mean value of the maximum response displacement with 3 waves which are classified into Type I or Type II design earthquake ground motions in each ground category, and δ_m is the horizontal displacement at P_{max} gained from the previous analysis study (Okada et al. 2010).

‘Type I δ_{max}/δ_m’ represents the value of ‘Type I δ_{max}’ divided by δ_m. In Figure 2 and Figure 3, the bold line indicates that the value of ‘Type I δ_{max}/δ_m’ and that of ‘Type II δ_{max}/δ_m’ are the same.

According to Figure 2, regarding A2 zone, Type II δ_{max} is generally larger than Type I δ_{max} in the case that either Type I δ_{max} or Type II δ_{max} is close to δ_m.

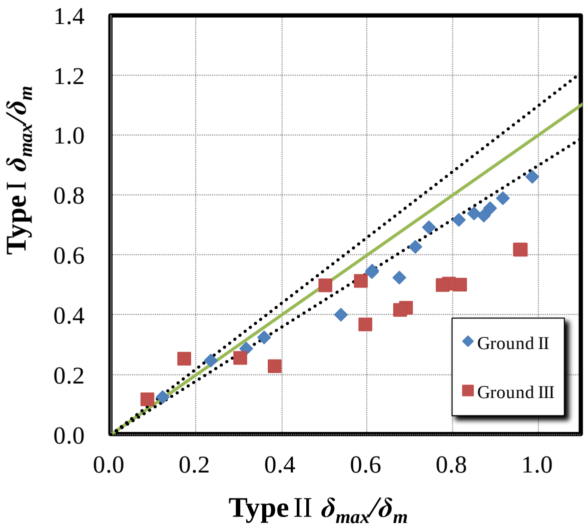

Figure 2: Comparison between Type I δ_{max} and Type II δ_{max} (A2 zone)

On the other hands, according to Figure 3, regarding B1 zone, Type I δ_{max} tends to be generally larger than Type II δ_{max}. The number of models which the Type I δ_{max} exceeded δ_m is 4 as for Ground type II, 4 as for Ground type III.

Besides, the effect of the natural period T and slenderness parameter $\bar{\lambda}$ of analytical models on Type I δ_{max} is shown in Figure 4. It indicates that Type I δ_{max} exceeds δ_m extremely in the both cases natural period T is 1.6 seconds approximately and $\bar{\lambda}$ =0.5. The maximum value of ‘Type I ${}_{max}/\delta_m$’ is generated by the earthquake ground motions classified into Ground type II. Therefore, the results of Ground type II are focused on to investigate. Figure 5 shows the name of the Type I design earthquake ground motion which give maximum value of ‘δ_{max}’ out of 3 waves in Ground type II. For example, “I-II-1” stands for one of the Type I design earthquake ground motions in Ground

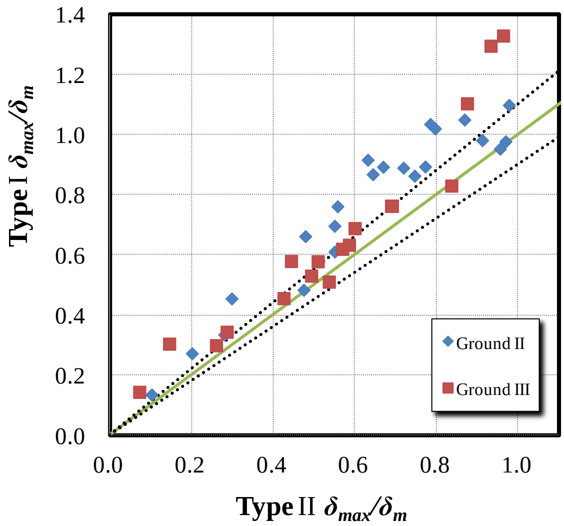

Figure 3: Comparison between Type I δ_{max} and Type II δ_{max} (B1 zone)

type II specified in the seismic design specifications. Figure 5 indicates the largest value of 'Type I δ_{max}' is generated by I-III-3.

Figure 6 shows the response accelerations of I-III-3 and 3 waves of Type II design earthquake ground motion with damping ratio 0.01. The figure shows the value of the response acceleration of I-III-3 become larger than those of Type II in the case the natural period T is 1.6 seconds approximately. The results mentioned above mean that it may be possibility that the steel bridge piers which have been already constructed in B1 zone have not enough seismic performance. Therefore, in some cases, some kinds of countermeasures may be needed.

5. CONCLUSIONS

In this paper, a time-history nonlinear dynamic analysis on the single-degree-of-freedom system for steel bridge piers was conducted to investigate the effect of revised Type I design earthquake ground motions considering zone factors on nonlinear response of steel bridge piers. The conclusions obtained from this study are shown as follows.

- Regarding A2 zone where a lot of steel bridges piers are constructed, the maximum response displacement obtained by Type I design earthquake ground motions tends to be smaller than that of Type II. The results mean that there may be possibility that the steel bridge piers in A2 zone have enough seismic performance.
- Regarding B1 zone where the Type I design earthquake ground motions have increased remarkably according to the revision of the seismic design specifications, the maximum response displacement obtained by Type I design earthquake ground motions tends to be larger than that of Type II. Besides, there are some models whose response exceeds its allowable value. It may be possibility that the steel

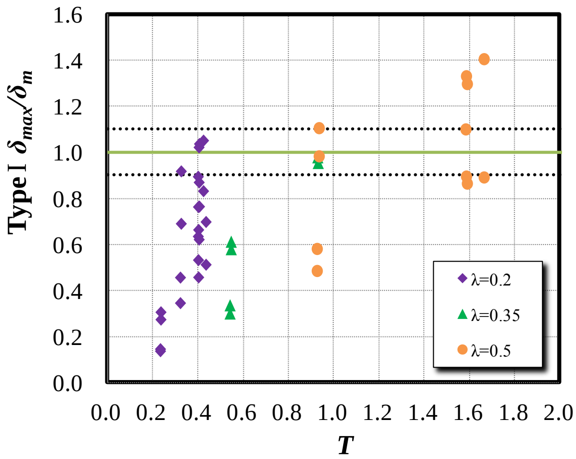

Figure 4: Effect of the T and $\bar{\lambda}$ on Type I δ_{max}/δ_m (B1 zone)

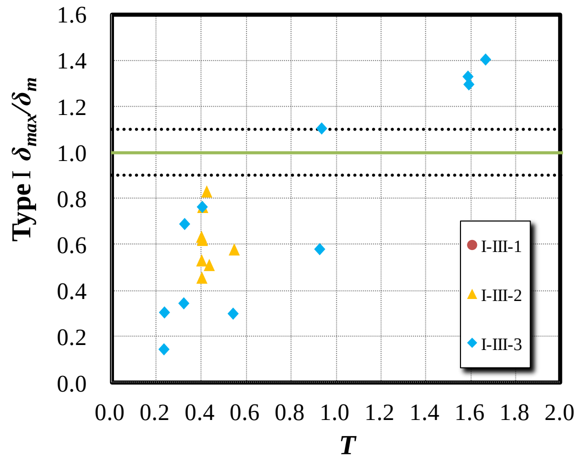

Figure 5: Effect of individual wave (Ground Type III) on Type I δ_{max}/δ_m (B1 zone)

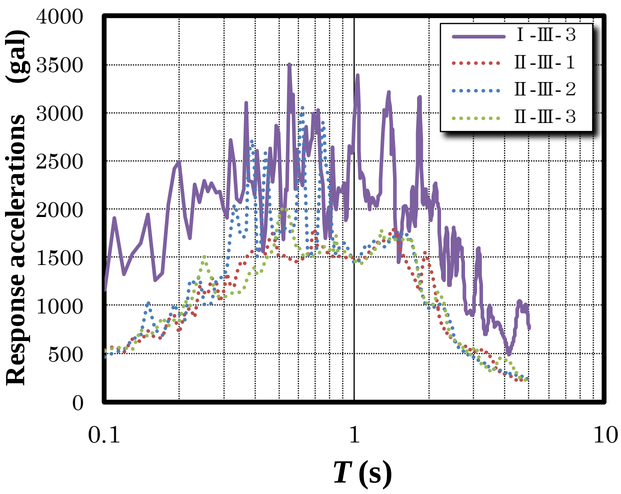

Figure 6: Acceleration response spectra (I-III-3 and 3 wave of Type II)

bridge piers in B1 zone doesn’t satisfied target seismic performance against the revised Type I design earthquake ground motions considering zone factors.

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