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A PROPOSAL FOR THE RATIONAL SEISMIC DESIGN OF RC 2-STORY RIGID-FRAME VIADUCTS IN CONSIDERATION OF THE YIELD PROCESS

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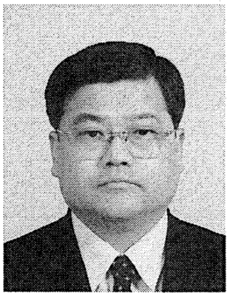
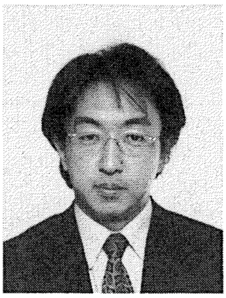
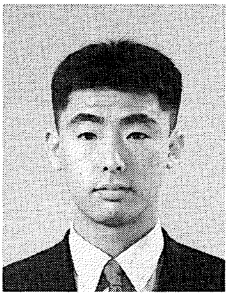
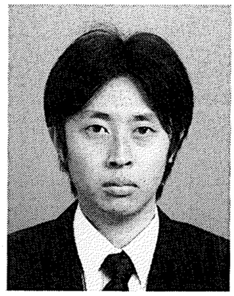

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Time-history earthquake response analysis is carried out to study the impact of the member yield process on the seismic performance of RC 2-story rigid-frame viaducts with the stiffness of the columns and cross-beams as parameters. Next, the equal energy assumption is applied to the relationship between lateral force and top-end displacement, as obtained through static non-linear analysis of a viaduct, as a way to study the conformity of the non-linear maximum response displacement that was approximated with the maximum response displacement obtained from dynamic analysis. Finally, a simple and practical seismic design method for such 2-story rigid-frame viaducts is proposed on the basis of this static non-linear analysis taking into account the member yield process.

Key Words : *RC 2-story viaduct, yield process, static analysis, equal energy assumption, seismic design*

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1. INTRODUCTION

The 1996 revision to the Design Specifications for Highway Bridges incorporated a seismic design method for single-story rigid-frame bridge piers based on push-over analysis[1]. Where rigid-frame bridges incorporate 2-story rigid-frame piers or piers with widely differing natural periods properties, or where bridges are marked by complex behavior during earthquakes, however, such static methods are inappropriate and dynamic analysis should generally be used to verify seismic performance. However, in a study carried out since this revision, the maximum non-linear response displacement of multi-span continuous rigid-frame bridges and similar designs was estimated by introducing the equal energy assumption or equal displacement assumption into push-over analysis, and the results compared with dynamic analysis. The findings suggest that, aside from some cases where higher vibration modes are dominant, it is possible to carry out seismic design using static analysis just as with single-story rigid-frame bridge piers[2]. Consequently, the assumption is made that the primary vibration mode of an RC 2-story rigid-frame viaduct of typical size is more dominant than other vibration modes. Further, it is therefore possible to also make use of a seismic design method based on the equal energy assumption and static non-linear analysis[3].

Surveys of damage to rigid-frame viaducts in the Hyogoken Nanbu Earthquake have demonstrated that the effects of differences in response mechanisms among viaducts and the effects of member damage on overall structural behavior are not clear[4]. For this reason, there is a need to study the effects of the sequence by which members yield (the yield process) in 2-story rigid-frame viaducts and, for double deck structures, the location of plastic hinges. In particular, locations where excess plasticization of cross-beams does not occur need to be located.

In this research, the effects of the yield process on top end displacement in RC 2-story rigid frame viaducts is investigated by carrying out dynamic analysis for viaducts with various simulated yield processes achieved by varying the stiffness of the columns and cross-beams. The equal energy assumption is also applied to the relationship between lateral force and top-end displacement as obtained from static non-linear analysis of RC 2-story rigid frame viaducts. The approximate non-linear maximum response displacement obtained in this analysis is compared with the maximum response displacement obtained in the dynamic analysis so as to verify the precision of the static method. The aim of this methodology is to establish a method for the seismic design of RC 2-story rigid-frame viaducts based on static analysis, a goal that has been long-sought by designers.

2. EFFECT OF THE MEMBER YIELD PROCESS ON VIADUCT RESPONSE AS DETERMINED BY DYNAMIC ANALYSIS

2.1 Viaduct Model

The research focused on the Standard Design of RC 2-story rigid-frame viaducts used for the Tohoku Shinkansen (Bullet Train line), and heights of 10 m, 12 m, and 14 m were studied. Figure 1 shows the analytical viaduct model for the 12 m case. The solid line in the figure represents the rigid zone, while the spring under the footings simulates piles and the ground. Two ground models were selected from seismic design ground categories I to III. Table 1 shows the ground natural period T_g(s), the weighted average of N value, and the layer thickness (m) of the ground assumed in this study.

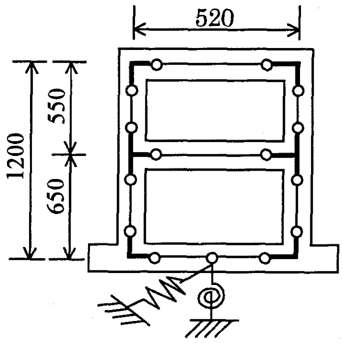

Figure 1. Viaduct Model unit: cm

Table 1. Ground Data Used

Ground model	No.I-1	No.I-2	No.II-1	No.II-2	No.III-1	No.III-2
Natural Period (Sec)	0.12	0.12	0.28	0.34	0.61	0.61
Weighted Average of N Value	25.3	19.3	25.6	16.7	9.51	10.3
Layer Thickness (m)	4.0	7.1	20.6	16.6	22.8	24.9

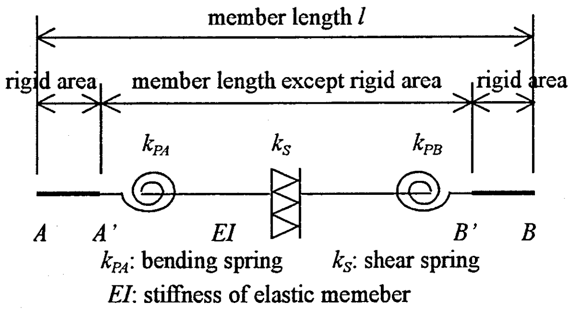

Figure 2. Member Model

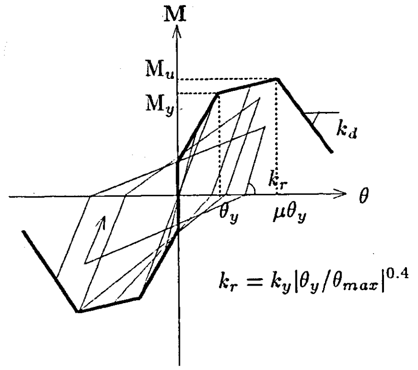

Figure 3. Bending Moment-Rotation Angle Relationship

2.2 Dynamic Analysis Method and Input Earthquake Motion

a) Dynamic Analysis Method

The member model used in dynamic analysis of the model shown in Figure 1 was Giberson's[5] as shown in Figure 2. A 3-mass point, 4 degree-of-freedom system was employed by contracting the matrix for the number of horizontal layer by ignoring the rotational inertia of each viaduct node and accounting for the sway and rocking of the foundations caused by ground deformation. In preparing the mass matrix, the mass of a single span was taken to be the sum of the top-layer longitudinal girder, middle longitudinal girders, slab, track slab, and soundproof walling. The damping constants for each element were 0.02 for RC members and 0.10 for the pile–ground spring, and the damping force was provided by Rayleigh damping. Numerical integration was implemented using Newmark's β method with the assumption $\beta = 1/4$, and the calculation time increment was 1/500 second.

b) Skeleton Curve and Hysteresis Characteristics

The bending moment–rotational angle relationship of the RC member used to study the behavior of the viaduct was given by a stiffness degradation model consisting of a tetra-linear skeleton curve, such as that shown in Figure 3. The stiffness k_d after the member reached the ultimate state was given by the following equation proposed by Machida et al.[6]:

$$\left(-\frac{k_d}{k_y}\right) = \frac{1.299}{\mu - 1} - 0.0539 \tag{1}$$

Here, μ is the member ductility factor. Machida et al. calculate k_d using a ductility factor μ obtained from cyclic loading tests on reinforced concrete specimens[7]. However, in this study, μ was taken to be as shown below. This is the ductility factor μ proposed by a working group of the Japan Society of Civil Engineers' Special Committee for Hanshin Earthquake Disaster Surveying and Research[8]. It was adopted here for its simplicity and for its good general applicability. The equation was originally proposed on the basis of work to collect and analyze a wide selection of previous results related to the deformation performance of RC columns.

$$\mu = \mu_0 + (1 - \mu_0)(\sigma_0 / \sigma_b) \tag{2}$$

$$\mu_0 = 12\left(\frac{0.5V_c + V_s}{V_{mu}}\right) - 3 \tag{3}$$

Where, σ_0: compressive stress (tf/m^2), σ_b: compressive stress (tf/m^2) at which equilibrium breaks down (more strictly defined as 'the axial force resulting in the ultimate compressive strain at the concrete extremities while also causing reinforcing bars to reach their yield strength at the point where the resultant tensile force acts'), V_c: shear capacity (tf) without hoop ties, V_s: shear capacity (tf) contributed by hoop ties, V_{mu}: shear force (tf) at the bending yield strength

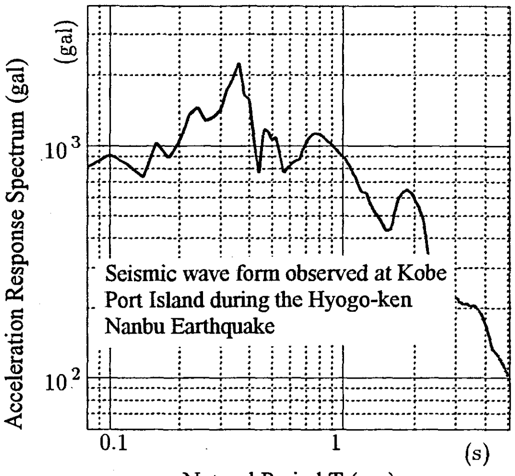

Figure 4. Acceleration Response Spectrum of Input Earthquake Motion

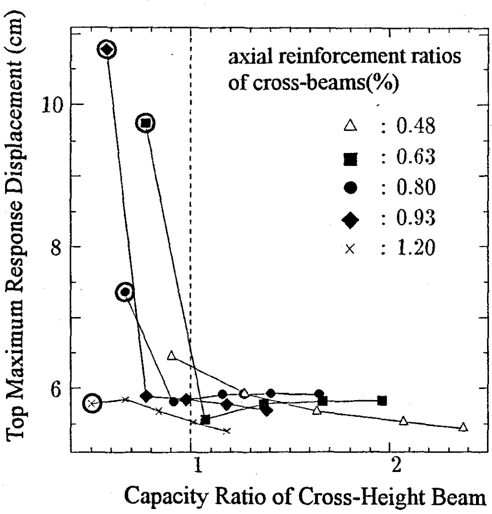

Figure 5. Top Maximum Response Displacement-Capacity Ratio Relationship of Cross-Beam

An origin point directed model with a tri-linear skeleton curve was used for the shear force–shear angle relationship of the shear spring in the center of the member (Figure 2). The coordinates of these lines were set using the method given in reference[9].

For the purpose of this study, the skeleton curve in Figure 3 was set using only the bending moment–curvature relationship as obtained from the axial force under normal loading, taking no account of the fluctuations in axial force acting on the column during an earthquake. The hysteresis properties of the RC member were based on Takeda's model. The pile–ground spring was a linear spring based on the Design Specifications for Highway Bridges[1].

c) Input Earthquake Motion

The input earthquake motion was the waveform observed at Kobe Port Island during the Hyogoken Nanbu Earthquake; this was an NS component with an −86 m acceleration waveform and was considered to be the bedrock waveform. Figure 4 shows the acceleration response spectrum. This seismic motion was input into the bedrock of the analyzed ground as specified in Table 1, and multiple reflection theory was used to estimate the seismic response at the bottom of the foundation. The dynamic analysis was based on this seismic waveform.

2.3 Analysis Results

The longitudinal reinforcement ratio and the shear reinforcement ratio of the columns and cross-beams were varied about the values used in the standard viaduct design. The standard was an axial reinforcement ratio of 2.44% and a shear reinforcement ratio of 0.44% for the columns, and an axial reinforcement ratio of 1.60% and shear reinforcement ratio of 0.25% for the cross-beams. Response analysis was carried out by combining the viaduct and ground models (Table 1) so as to study the effects of differences among yield processes on the earthquake response of the viaducts.

a) Yield Type and Member Capacity Ratio

First, the axial reinforcement ratio of the columns was fixed and the axial reinforcement ratio of the cross-beams was varied from 0.48% to 1.20%. Viaduct models with cross-beams of varying capacity ratio (defined as the shear capacity/shear force at the moment when the flexural capacity is reached) were established and analyzed by varying only the shear reinforcement ratio in the cross-beams. Further, to study the effect on viaduct response of plastic deformation after the cross-beams yield, the axial reinforcement ratio of the cross-beams was made lower than that in the standard viaduct. Figure 5 shows the results. Symbols enclosed in circles indicate that shear failure of the cross-beams occurred during dynamic analysis.

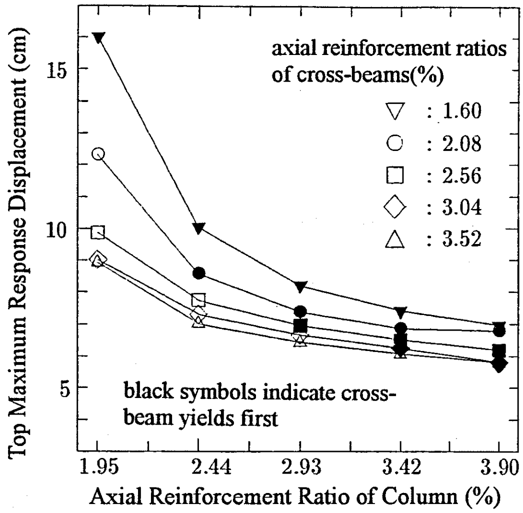

Figure 6. Reinforcement Ratio-Top Maximum Response Displacement Relationship of Cross-beam

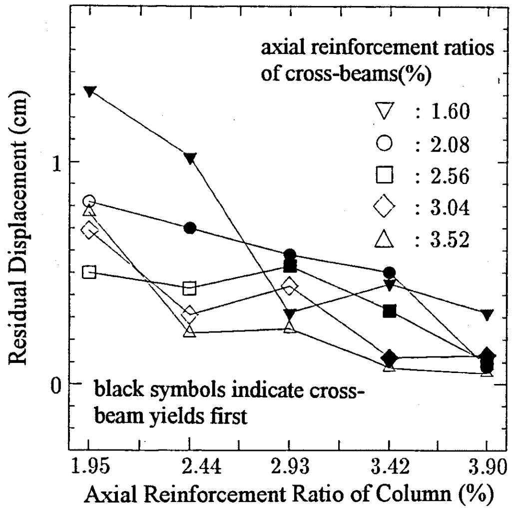

Figure 7. Reinforcement Ratio-Residual Displacement Relationship of Member

In the case of members that suffered bending failure, given that the bending stiffness of the members is constant for identical axial reinforcement ratios, the response deformation of the bending springs obtained from the active inertia force and bending moment–rotation angle relationship is almost unvarying as long as it is over the deformation performance shown in Figure 3. Figure 5 indicates that, for capacity ratios of 1 or more (where shear failure of the cross-beams does not occur), there is almost no difference in top maximum response displacement. Since this study takes into account the rise in ductility factor of the cross-beams according to the increase of capacity ratio based on equation (2), it can be concluded that for the seismic waveform used in this study and for the viaduct models analyzed, the deformation performance of cross-beams has no effect on the overall response of the rigid-frame viaduct.

It has been reported by Kamiyama et al.[10] that in scale model experiments on RC 2-story rigid-frame viaducts as used along the already completed Sanyo Shinkansen, and in cases where shear failure of the columns does not occur when earthquake motion of the Hyogoken Nanbu Earthquake class is was applied, the capacity and ductility factor of each member has very little effect on the response properties of the viaduct. Consequently, to enable a study of the effects on viaduct response of member yield process, it was hypothesized that the failure mode of members was bending failure only. A model accounts for the shear reinforcement so that the capacity ratio of the members would be greater than 1 was used for the analysis.

b) Relationship between Amount of Member Reinforcement and Maximum Response Displacement of Viaduct

Figures 6 and 7 show the results for a model of a 12-m viaduct and Category III ground model. Figure 6 gives the maximum response displacement at the viaduct top when the axial reinforcement ratio of the cross-beams was fixed and the axial reinforcement ratio of the column was varied. Following this, cross-beams with five different axial reinforcement ratios were prepared, and Figure 7 shows the results of residual displacement of a viaduct under identical conditions. In order to account for the free vibration of the viaduct after an earthquake, zero acceleration was added to the end of the input earthquake motion to compute the response displacement, and residual displacement of the viaduct was assumed to be the displacement when this settled to a constant value.

Figure 6 shows that the maximum response displacement at the top of the viaduct can be reduced by increasing the amount of axial reinforcement in the columns and cross-beams, thus increasing their stiffness. The residual displacement as determined by dynamic analysis, as shown in Figure 7, similarly reveals that increasing the amount of axial reinforcement reduces the residual displacement. However, the contribution to reducing the response displacement of the viaduct by increasing the axial reinforcement in the cross-beams is less than that made by increasing the reinforcement in the columns. If the quantity of axial reinforcement in the cross-beams is increased until the columns yield first, it is possible to reduce the maximum response displacement of the viaduct top by just as much. But in this case, the amount of shear reinforcement required in the cross-beams to prevent shear failure means an extremely dense arrangement of steel. And it has been pointed out that if the stiffness of the cross-beams is raised, the failure mode of the column member may change from bending failure to shear failure[3]. Taking into consideration the distribution of damage to viaducts during earthquakes, it is easily understood that the cross-beams should ideally yield first so that their plastic deformation absorbs the seismic energy[9]. Similar results are seen when other viaduct and ground models are combined for analysis.

In conclusion, efficient reduction of the displacement at the top of a 2-story rigid-frame viaduct can be achieved by increasing the flexural capacity of each member yet remaining within the range in which the cross-beams yield first.

c) Bending Yield Strength Ratio of Members; the Yield Process Relationship
Next, the relationship of the ratio η with the yield process of the members was studied for all dynamic analysis models that were analyzed. η means that the ratio of bending yield strength of cross-beam to bending yield strength of column. The bending yield strengths of cross-beam and column are obtained by dividing the ultimate bending moment capacity by the respective shear span.

The first step in this analysis was to vary the axial reinforcement ratios of the columns and cross-beams of viaducts with varying heights from 1.95% to 3.90% and from 1.60% to 3.52%, respectively. Dynamic analysis is then carried out on structural systems obtained by combining these viaducts with the ground models shown in Table 1. Then, the maximum value in a case where the cross-beam yields first from among all η that were set was studied. The results are shown in Table 2. The yield process of members under the input earthquake motion varied according to the amount of axial reinforcement in the columns and cross-beams, but the various processes can be categorized as indicated in Table 2.

Table 2. Effect of Ratio of Cross-Beam Yield Strength to Column Bending Yield Strength on Member Yield Process

η	Member yield process
~0.9	m.h. yielding ⇒ c.l. yielding ⇒ c.u. yielding ⇒ m.h. ultimate ⇒ c.u. ultimate ⇒ c.l. ultimate
0.9	m.h. and c.l. yielding at same time ⇒ c.u. yielding ⇒ m.h. ultimate ⇒ c.u. ultimate ⇒ c.l. ultimate
0.9~	c.l. yielding ⇒ m.h. yielding ⇒ c.u. yielding ⇒ m.h. ultimate ⇒ c.u. ultimate ⇒ u.l. ultimate

m.h.: cross-beam, c.l.: lower column, c.u.: upper column

In this study, dynamic analysis was carried out by causing the seismic waveform to act at the bottom of the footings for each ground model, as explained above, but the yield process was determined by the sectional specifications of each member rather than by the characteristics of the input waveform. These results were obtained based on an analysis of only a limited range of structural models, but Table 2 does demonstrate that, if the bending yield strength of the cross-beams is less than 0.9 times the bending yield strength of the columns, the cross-beams yield first. On the other hand, if it is greater than 0.9 times, the columns yield first.

On the basis of the above results, it can be stated that where the cross-beams of a 2-story rigid frame bridge do not support the superstructure or in other cases where plasticization of these beams is acceptable, a rational seismic design method is to design each member so that η (the ratio of cross-beam bending yield strength to column bending yield strength) is not greater than 0.9 and the stipulated safety margin for the design seismic force are satisfied. Further, for a double-deck structure, it is necessary to set η higher than 0.9 so that excessive plasticization does not occur in the cross-beams. In this way, it is possible to implement a seismic design taking into account the acceptable state of damage to an RC 2-story rigid-frame viaduct after an earthquake by considering the yield process of the viaduct members.

3. ELSTO-PLASTIC RESPONSE DISPLACEMENT ACCORDING TO EQUAL ENERGY ASSUMPTION

3.1 Static Non-linear Analysis

Using the skeleton model in Figure 1, the static horizontal displacement was gradually increased in order to study the yield strength and deformation performance of an RC 2-story rigid-frame viaduct. The relationship between bending moment and rotation angle for individual members was assumed to be the tetra-linear curve described above in Section 2. The viaduct model chosen for analysis was missing the pile–ground spring, which was removed in order to focus on the lateral force–displacement relationship of the viaduct itself.

The results of an eigenvalue analysis of viaducts with various yield processes, as analyzed in Section 2, confirmed that in the viaduct models used here, the effective mass of the primary mode ranges from 85% to 90%, and that it dominates other vibration modes. Thus the deformation mode vector $\{\phi\}$ for the first layer and the second layer provided in static non-linear analysis is in proportion to the primary natural vibration.

3.2 Ultimate Displacement of Rigid-frame Viaduct

This analysis focused on viaducts with heights of 10m, 12m, and 14 m. As in the analysis explained in Section 2, viaduct models with differing yield processes and natural periods were chosen, and suitable modifications were made by varying η (the ratio of cross-beam bending yield strength to column bending yield strength).

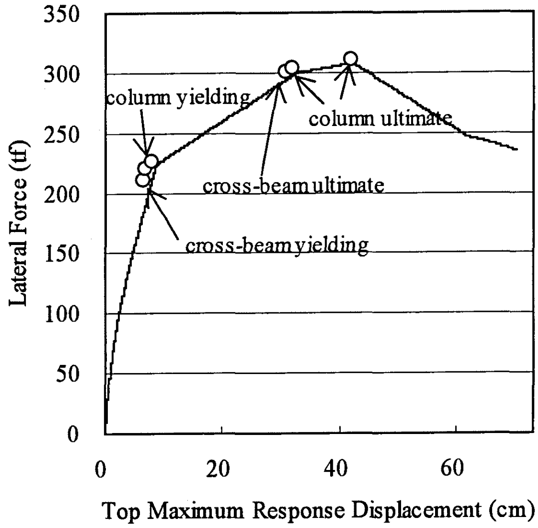

Figure 8. Example of Lateral Force of Viaduct-top Lateral Displacement Relationship

A 2-story rigid-frame viaduct is a statically indeterminate structure, so the ultimate state of the entire viaduct must be defined. Conventionally, the seismic design of rigid frame structures was based on push-over analysis, for which the ultimate state is stipulated by the Design Specifications for Highway Bridges. The ultimate state of the structural system was assumed to have been reached when one of the hypothesized multiple plastic hinges reached the ultimate state, except in the case of 1-story rigid-frame viaducts [11]. In this study, however, it is possible to account for the effects on the structural system of the loss of bending capacity beyond the maximum bending moment; this is achieved using the relationship between member bending moment and rotation angle, as shown in Figure 3. Thus the ultimate displacement of a 2-story rigid-frame viaduct was studied on the basis of the lateral force–displacement (P-δ) relationship obtained by gradually increasing the lateral displacement under the conditions described above. Figure 8 shows an example of the analytical results. Lateral force P represents the sum of the lateral forces acting on the first and second layers while the lateral displacement δ is the displacement at the top of the viaduct.

The member yield process as given by static analysis is different for each model, but in all models, when the six bending springs reach the yield point, the overall viaduct stiffness as represented by the P-δ relationship changes. Because a 2-story rigid-frame structure is a sixth-grade statically indeterminate structure, when the sixth plastic hinge forms, the structural form changes from statically indeterminate to statically determinate, and there is a corresponding decline in stiffness. If the displacement continues to increase after the six bending springs have reached the ultimate state, the lateral strength of the viaduct passes a peak, with the gradient of the P-δ curve turning downward. It is possible to clearly define the ultimate point of a viaduct by setting the capacity decline zone following the maximum bending moment with a member model. In light of this discussion, the yield point of a 2-story rigid-frame viaduct in this study is taken to be the moment when the six bending springs reach their yield points. The ultimate point is the moment at which the six bending springs all reach the ultimate point, regardless of the yield process.

The yield processes as obtained from static analysis of all the viaduct models generally coincided with the results of dynamic analysis for various input seismic motions. Therefore, by determining η (ratio of bending yield strength of cross-beam to yield bending strength of column) as shown in Table 2, the designer of a 2-story rigid-frame viaduct who uses the static non-linear seismic design method proposed in Section 4 can choose the intended yield process without the need for dynamic analysis.

3.3 Elasto-plastic Response Displacement According to Equal Energy Assumption

The yield point, the ratio of post-yield stiffness to yield stiffness, and the elastic response lateral force for a modeled viaduct are obtained from its P-δ relationship, as shown in Figure 9. The elasto-plastic response displacement δ_{st} at the top of the viaduct is then estimated using Equation (4), which is derived from the equal energy assumption; this assumption hypothesizes that the areas of the triangle OAB and the quadrilateral OCDE shown in Figure 9 are equal.

$$\delta_{st} = \frac{1}{r}\left\{ r - 1 + \sqrt{1 - r + r\left(\frac{k_{hc} \cdot W}{P_y}\right)^2} \right\}\delta_y \tag{4}$$

Where, r: ratio of post-yield stiffness to yield stiffness, k_{hc}: seismic coefficient used in the ductility design method stipulated in the Design Specifications for Highway Bridges [1], P_y, δ_y: yield strength (tf) and yield displacement (m) obtained from the P-δ relationship of the viaduct ,W: equivalent weight of superstructure and pier (tf). This is calculated using equation (5)[12].

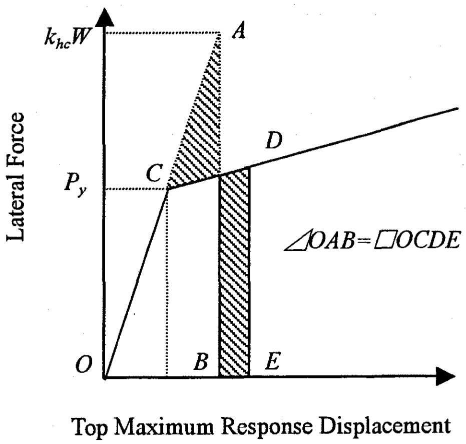

Figure 9. Estimation of Elasto-plastic Response by the Equal Energy Assumpiton

$$W = \frac{gKT^2}{4\pi^2} \tag{5}$$

Where, T: natural period (s) of the primary mode of the rigid-frame viaduct, K: initial stiffness of the rigid-frame viaduct (tf/m), g: acceleration due to gravity (m/s^2).

The seismic design carried out in this study was based on the strong earthquake motion of the Hyogoken Nanbu Earthquake. There are two conventional methods of setting the scale of such earthquake motion for the purpose of seismic design. One is based on a seismic hazard estimate considering earthquake activity at the location of the structure[13] and the other accounts for the non-exceeding probability based on the frequency distribution of an acceleration response spectrum obtained for each ground category [14]. In this study, the seismic motion stipulated by the Design Specifications for Highway Bridges is adopted. This is the acceleration record of the seismic motion that caused particularly severe damage during the Hyogoken Nanbu Earthquake, and it reflects the properties of each ground category observed. Because the Design Specifications for Highway Bridges stipulate the seismic load for both the seismic coefficient used for static analysis and seismic wave that the amplitude is adjusted in the frequency zone that corresponds to it, it is easy to study the correlation between static analysis results and dynamic analysis results.

3.4 Analytical Results

For the skeleton model shown in Figure 1 without the ground spring, the maximum response displacement δ_{dy} at the top of the viaduct was calculated using dynamic analysis. The result was compared with the elasto-plastic response displacement δ_{st} obtained from static non-linear analysis based on the equal energy assumption, as represented by Equation (4). The input earthquake motion used for this dynamic analysis was the type-II earthquake motion (three waves for each ground type) stipulated in the Design Specifications for Highway Bridges [1]. This has the spectral properties indicated in Figure 10, as explained above. δ_{dy} was calculated using the three waves for each ground category, and the average value of δ_{st}/δ_{dy} was calculated.

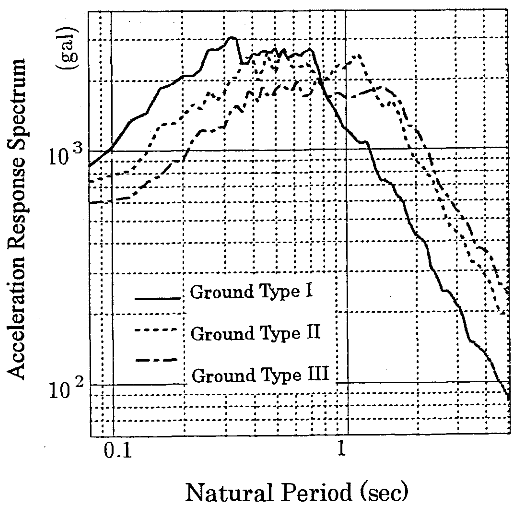

Figure 10. Acceleration Response Spectrum of Type-II Earthquake Motion

Figures 11 to 13 show the analytical results for hypothetical ground types I, II, and III, respectively. References in Figures 11 to 13 to earlier yielding of the cross-beam and of the column are the results for viaduct models with the η values (ratio of cross-beam bending yield strength to column bending yield strength) shown in Table 2, representing values less than 0.9 and 0.9 or higher, respectively. The one-degree-of-freedom model in each figure is the result of applying the equal energy assumption and performing dynamic analysis on an RC bridge pier provided the hysteresis properties with the same bending moment – rotation angle relationship as in Figure 3. The amount of axial reinforcement in the bridge pier was varied here to achieve the natural periods indicated in Figures 11 to 13 while maintaining a yield seismic coefficient of greater than 0.25. The damping constant was 0.05.

The value of δ_{st}/δ_{dy} obtained from the analysis of 2-story rigid-frame viaducts is 0.97 to 1.86 for ground type I, 0.75 to 1.25 for ground type II, and 0.73 to 1.19 for ground type III. While the results are on the safe side for almost all period ranges in the case of type-I ground, they fall on the dangerous side for ground types II and III. However, there was no observed variation in the precision of the estimates with different yield processes, whatever the ground type. However, for all three types of ground, the value of δ_{st}/δ_{dy} tended to fall as the natural period of the viaduct became shorter.

Even when the elasto-plastic response displacement of an RC bridge pier that can be modeled as a one-degree-of-freedom system is estimated using the equal energy assumption, as shown in Figures 11 to 13, very similar results are obtained. It is known from previous studies that, if the equal energy assumption is applied to a structure that can be modeled as a one-degree-of-freedom system then, as the hypothetical ground moves from type I to type III, the response displacement obtained from equal energy assumption tends to be underestimated for the result of dynamic analysis if the structure has a short natural period[15]. It has been reported that this effect is particularly prominent when the motion is near-field, such as in the case of the Hyogoken Nanbu Earthquake [16]. So, although the equal energy assumption can be used to calculate elasto-plastic response displacement during an earthquake extremely easily without the need for dynamic analysis, its precision varies widely according to the hypothesized earthquake motion, the natural period of the structure, and other factors. It can be concluded that ultimately the scattering of the precision of δ_{st}/δ_{dy} when the object of analysis is the RC 2-story rigid frame viaduct shown in Figure 11 to Figure 13 is caused by applying the equal energy assumption to the estimation of the elasto-plastic response displacement during an earthquake, and that replacing a 2-story rigid frame viaduct with a one-degree-of-freedom system does not cause any problems.

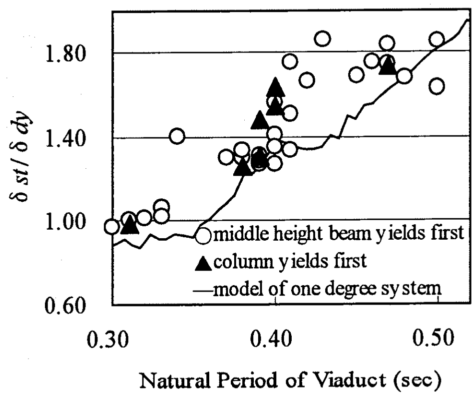

Figure 11. Comparison of Static Analysis and Dynamic Analysis (Ground Type I)

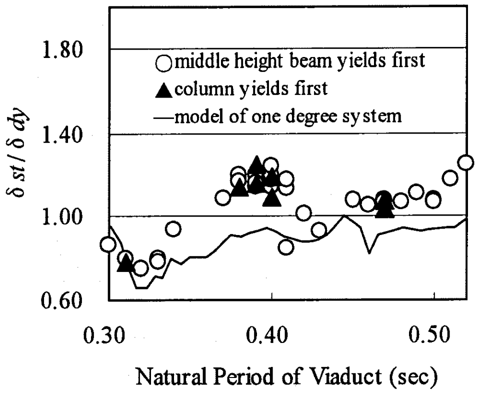

Figure 12. Comparison of Static Analysis and Dynamic Analysis (Ground Type II)

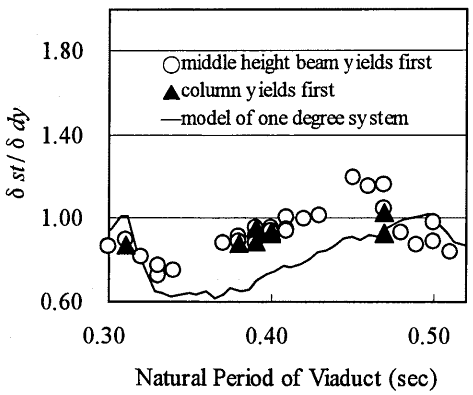

Figure 13. Comparison of Static Analysis and Dynamic Analysis (Ground Type III)

But, in any case, it is essential to account for the fact that when the maximum response displacement is estimated based on the model proposed in this study, the evaluation may give results on dangerous side for certain natural periods and ground types.

3.5 Comparison with Estimation Using Skeleton Curve

The hysteresis characteristic used in this study was a model that takes into account the decline in bending capacity. It is based on a skeleton curve linking the crack point, yield point, and point of maximum bending moment, as shown in Figure 3. (This is referred to as "member model 1".) However, Design Specifications for Highway Bridges ignores the initial crack point and uses a bilinear model of yield stiffness and post-yield stiffness (r = 0). The applicability of the equal energy assumption when this bilinear skeleton curve is used ("member model 2") was verified. The definitions of the yield point and ultimate point of the viaduct are the same as with member model 1. Further, the hysteresis characteristics during dynamic analysis are based on the Takeda model, as in the case of analysis using member model 1.

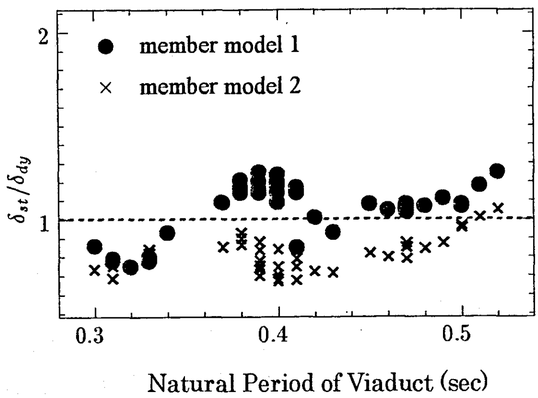

Figure 14. Comparison of Estimation Precision of Response Displacement with Different Member Models

Figure 14 shows the ratio δ_{st}/δ_{dy} (where δ_{st}, and δ_{dy} are the elasto-plastic response displacements according to the equal energy assumption and dynamic analysis, respectively) for a viaduct modeled with each skeleton curve. Type-II ground is hypothesized for each member model. Table 3 presents the statistical results.

Table 3. Comparison of Estimate Precision ($\delta st/\delta dy$) by Member Model

	Member Model 1		
Ground Type	Type-I	Type-II	Type-III
Mean Value	1.43	1.04	0.93
Coefficient of Variation (%)	18.5	13.4	10.2
	Member Model 2		
Ground Type	Type-I	Type-II	Type-III
Mean Value	1.25	0.81	0.79
Coefficient of variation (%)	29.6	11.5	12.9

The use of member model 2 for the static non-linear analysis yields evaluations on the dangerous side for all ground types, in contrast with the results for member model 1, but the evaluations were generally very close to the response displacement obtained in dynamic analysis. The tendency for response displacement to be underestimated for a structure with a short period, or as the ground changes from type I to type III was identical to that observed for member model 1. It can be concluded that, ultimately, the discrepancy between results obtained with the two member models does not affect the applicability of the equal energy assumption to a 2-story rigid-frame viaduct. In cases where member model 2 yields results more toward the dangerous side than member model 1, the difference can be resolved by changing the setting of allowable displacement, as discussed below. Experiments are now needed to verify the selection of an appropriate member model.

4. PROPOSED SEISMIC DESIGN METHOD FOR RC 2-STORY RIGID-FRAME VIADUCTS BASED ON STATIC NON-LINEAR ANALYSIS

The above study of response displacement using static non-linear analysis and the equal energy assumption indicates that evaluations fall on the dangerous side for certain structure periods and ground types. For such situations, an allowable displacement for which the safety of a viaduct is guaranteed was studied by dynamic

Table 4. Axial Reinforcement Ratios of Column and Cross-Beam for Various Allowed Displacements (η =0.8)

	Definition (1)	Definition (2)	Definition (3)
Displacement estimated by Equation (4)	27.8	27.8	26.4
Displacement obtained by dynamic analysis	28.6	28.6	31.1
Ultimate displacement of the viaduct	37.1	37.1	38.7
Allowable displacement of the viaduct	30.1	30.3	28.7
Axial reinforcement ratio of the cross-beam	2.10	2.10	2.10
Axial reinforcement ratio of column	2.20	2.20	2.44

analysis. This results in three possible definitions of the allowable displacement value. These are (1) the displacement at which one bending spring reaches the ultimate state according to static non-linear analysis, (2) the displacement at which four bending springs reach the ultimate state, and (3) the displacement calculated by Equation (6) as stipulated in the Design Specifications for Highway Bridges.

$$\delta_a = \delta_y + \frac{\delta_u - \delta_y}{\alpha} \tag{6}$$

Where, δ_a: allowable displacement of the viaduct, δ_y: yield displacement of the viaduct , δ_u: ultimate displacement of the viaduct, α : safety factor (1.5 for Type-II earthquake motion).

The results of trial calculations based on these three definitions of allowable displacement reveal that, regardless of the definition selected, there is little variation in the sectional specifications. And even if the elasto-plastic response displacement estimated by the equal energy assumption underestimates the maximum response displacement δ_{dy} as compared with the results of dynamic analysis, δ_{-dy} guarantees adequate safety for the ultimate displacement of the viaduct as defined in Section 3. (2). Table 4 gives examples of trial designs for each definition of allowable displacement.

The allowable displacement as calculated by Equation (6) always yields the smallest of the values obtained using the three definitions. When applied to the actual design process, the allowable displacement δ_a calculated by Equation (6) should be used to obtain the safest evaluation and thus prevent excessive plasticization of some members.

Methods of estimating the elasto-plastic response displacement based on the equal energy assumption in the seismic design of structures that can be modeled as one-degree-of-freedom systems (such as single-column RC bridge piers) have been stipulated in many design guidelines. This work confirms that it is also possible to estimate the elasto-plastic response displacement of RC 2-story rigid-frame viaducts to the same precision as is possible in the case of such single-column piers. However, the precision of the elasto-plastic response displacements calculated based on the equal energy assumption differs for each hypothetical earthquake motion and for each natural period of the structure, as shown in Figures 11 to 13. Hence, if the allowable displacement is set according to Equation (6), the safety of designed structures will not be uniform.

In this study, a particular standard design of railway viaduct was the subject of analysis. The proposed method of estimating elasto-plastic displacement based on static non-linear analysis is not limited to railway viaducts. When it is applied to a railway viaduct, however, the choice of definition for allowable displacement may also be affected by the characteristics of train loading. Therefore, a future work will be to study ways of equalizing the safety of structures designed by this method and ways of setting the allowable displacement according to design conditions. This might entail, for example, adjusting the setting of the safety factor α proposed in Equation (6).

Based on the discussions above, a seismic design method based on static non-linear analysis and the equal energy assumption is proposed. Figure 15 is a flow chart of this proposed design method. The method includes a examination of the yield process and the damage distribution of viaduct members considering the appropriate strength hierarchy between columns and cross-beam with reference to Table 2. When the elasto-plastic response displacement calculated using static non-linear analysis and equal energy assumption satisfies the allowable displacement defined by Equation (6), the design is complete.

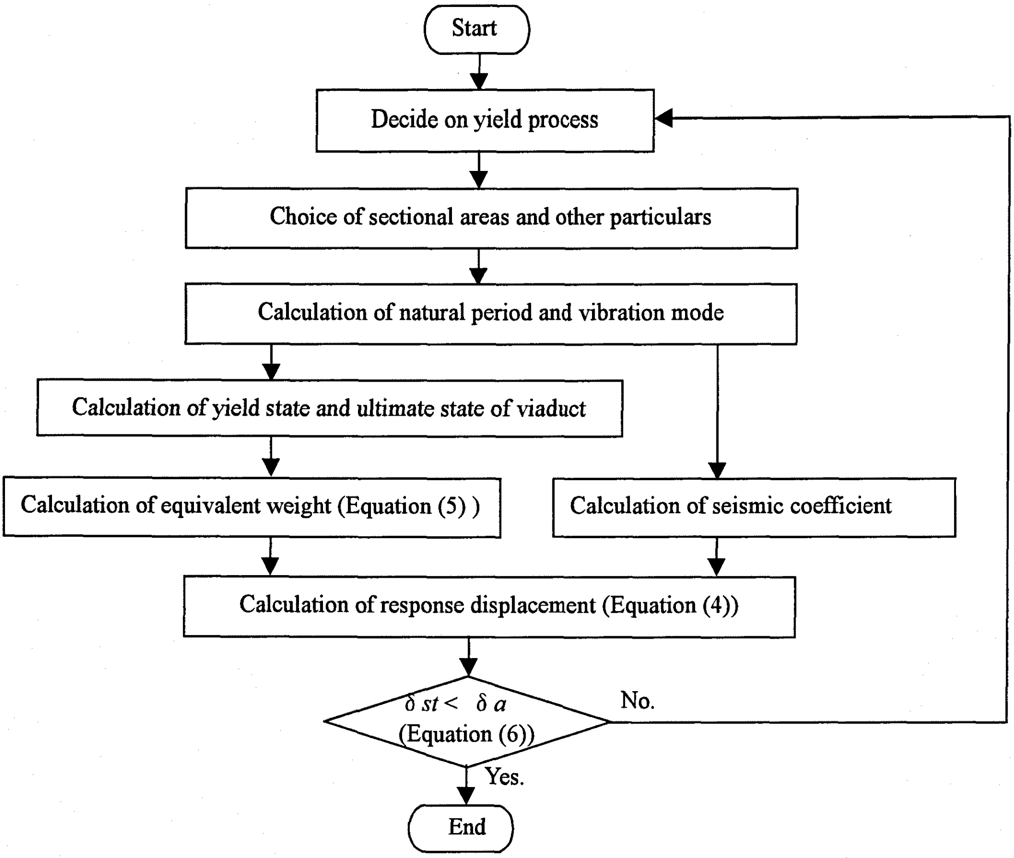

Figure 15. RC 2-story Rigid-Frame Viaduct Seismic Design Flow Chart

As noted at the beginning, the object of this study is the transverse direction of an RC 2-story rigid-frame bridge pier of height of 10 m, 12 m, or 14 m. Member failure is limited to bending failure. It is, therefore, necessary to conduct a separate study of the applicability of the equal energy assumption in cases where the natural period of the structural system deviates from the range indicated in Figure 11; this would be the case, for example, if the pier height is different or member stiffness is different, or in cases where the earthquake motion differs from that in Figure 10.

5. CONCLUSIONS

This research project began with a dynamic analysis to determine the effect of the member yield process on the seismic performance of RC 2-story rigid-frame viaducts. The elasto-plastic response displacement was then estimated the basis of the equal energy assumption to look into the conformity of this method with dynamic analysis. Finally, a simple and practical seismic design method for RC 2-story rigid-frame viaducts based on static non-linear analysis and taking into account the member yield process was proposed. The following results were obtained:

(1) The effects of differences in yield process on the response displacement at the viaduct top were studied. The results revealed that, while the cross-beam yields first, the response displacement of the viaduct can be effectively reduced by increasing the amount of longitudinal reinforcing bars in all members.

(2) The results of an analysis in which the amount of reinforcement was varied revealed that the yield process is specific to each viaduct regardless of the ground type and other factors. A suitable ratio of cross-beam bending yield strength to column bending yield strength for use in designing RC 2-story rigid-frame viaducts with a chosen yield process was proposed.

(3) The yield point and ultimate point of an RC 2-story rigid-frame viaduct was defined using a member model that takes account of the decline in bending capacity.

(4) The elasto-plastic response displacement obtained from static non-linear analysis based on the equal energy assumption, taking into account the secondary stiffness, was compared with the maximum response displacement obtained in dynamic analysis. The results show that the precision achieved is equal to that of dynamic analysis for an RC bridge pier that can be modeled using a one-degree-of-freedom system.

(5) It was confirmed that differences in the setting of post-yield stiffness and the choice of skeleton curve for the maximum bending moment of a member have no effect on the applicability of the equal energy assumption to an RC 2-story rigid-frame viaduct.

(6) A simple seismic design method for the transverse direction of an RC 2-story rigid-frame viaduct was proposed on the basis of the static non-linear analysis method described.

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