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# Elasto-plastic Seismic Responce Behavior of 2-dof Mass-Varying Structure Assuming Railway Elevated Stations

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## Abstract

Elevated station is a kind of railway station in Japan. Elevated station consists of viaduct, which supports railway track, and shed (a structure for the roof of the platform) at the top of the viaduct (hereinafter, a shed at the top of the viaduct is referred to as a shed on viaduct). While the viaduct is a large structure of reinforced concrete and the shed on viaduct is a small structure of steel, the mass of shed on viaduct is significantly smaller than that of the viaduct. Thus, the elevated station is a mass-varying structure, so its seismic response behavior is very different from general buildings. In particular, there is a case where response of the shed on viaduct is amplified, depending on the ratio of mass of the viaduct and the shed on viaduct and the natural period of them.

In previous studies, the seismic response behavior of 2-dof mass-varying structure assuming railway elevated station is examined.[1] According to that, by experiments and analysis, it is proved that the response of the shed on viaduct is greatly amplified when the mass of the shed on viaduct is small and the natural period of the shed on viaduct and that of the viaduct is close. In addition, it is also proved that the reinforcement method applying inertial mass damper to the shed on viaduct is effective to reduce its seismic response.

The above study is on condition that the viaduct and the shed on viaduct is elastic state. On the other hand, in Japan in recent years, the occurrence of so-called huge earthquakes is predicted, the viaduct and the shed on viaduct should significantly yield by that. However, on condition of their yielding, it is not enough that theoretical studies of the seismic response behavior of mass-varying structure assuming railway elevated station. Because of this, the way of evaluating it is only by other than to by modeling each structures and running time-history response analysis. Therefore, from the viewpoint of reasonable design and the safety of the shed on viaduct, it should be important to clarify the seismic response of the shed on viaduct in consideration of the nonlinear properties associated with their yielding.

Above this, in this paper, with 2-dof mass-varying structure assuming railway elevated station, it is examined that the seismic response of the shed on viaduct considering nonlinear characteristics. In this study, since there are so many parameters (the mass of the viaduct and of the shed on viaduct, the stiffness of them and nonlinear characteristics of them), it'll be complex to examine the relation between parameters and their seismic response. Therefore, by organizing the equation of motion, the influence factor to the seismic response is theoretically derived, and the tendency is examined. Next, based on this theoretical study, by time-history response analysis on huge earthquakes, the tendency of seismic response is considered from the result of numerical calculation too.

Keywords: Elevated station, Mass-Varying Structure, Seismic response, Response analysis, Elasto-plastic



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## 1. Introduction

As shown in Fig. 1, since the shed on the viaduct (hereinafter referred to as the "elevated shed") is lighter than the viaduct, there is a concern that the earthquake response of the elevated shed may increase due to resonance, if the natural periods of the viaduct and the elevated shed are close to each other. Although elevated sheds are not counted as buildings under the Building Standards Law, they are classified as buildings under the Ministerial Ordinance that establishes technical standards for railways (hereinafter referred to as the Technical Standards Ministerial Ordinance). The Technical Standard Ministerial Ordinance sets a safety requirement against foreseeable loads, and it is stated in the explanation that the Building Standards Law is applied mutatis mutandis. Hitherto, the authors have clarified the difference in the response characteristics in the translation direction according to the own natural period ratio between the elevated shed and the viaduct in previous studies on the Building Standards Law considering the distribution coefficient in the height direction of the force coefficient stipulated in Ministry of Construction Notification No. 1793 [2] (Ai coefficient). However, since an eigenvalue analysis of the entire model including the viaduct is necessary depending on conditions, although it allows precise design, it is a complicated method.

On the other hand, the Design Standards for Railway Structures and Commentary (Seismic Design) edited in 2012 (hereinafter referred to as the 'seismic standards') [3], describe the seismic design of facilities associated with railway structures. The appendix indicates a method for calculating the response amount of a pole considering the influence of rocking and resonance in the translational direction as an interaction with the viaduct. According to the seismic standards, the concept of this calculation method can be applied to sheds: however, because the structural forms (frame type, foundation structure, etc.) differ between poles and sheds, it is important to establish a method to calculate the degree of the response considering the elevated shed characteristics.



Fig. 1 Example of an elevated shed

## 2. Examination of the influence of viaduct rocking

The deformation of an elevated shed in an integrated model with a viaduct and an elevated shed during an earthquake includes deformation of the viaduct associated with rocking. The appendix of seismic standards contains a method for calculating the horizontal response seismic intensity of a pole as an example considering the effect of rocking on structures on the viaduct: however, considering the influence of rocking on an elevated shed, the shed differs from the pole in the two following ways: firstly, the multiple column spans perpendicular to the rail tracks on the station viaduct. The second is that the structure of a shed differs from cantilever-type poles and fits with a ramen structure such as a whole covering or a partially covered type. On this occasion, internal forces act on the shed to resist the rocking deformation of the viaduct (bending moment and shearing force). For these reasons, there is a possibility that the impact on the inter-story deformation angle of the shed will be different from that in the case of the pole when the viaduct is rocked. Therefore, as well as confirming the degree of rocking of the station viaduct with a large number of spans, the effect of viaduct rocking on the response of a ramen-structure shed was examined using static incremental analysis of an elevated shed-viaduct model.

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## 2.1 Analysis model

The analysis model used in the study was a two-dimensional frame model for the elevated shed and viaduct, as shown in Fig. 2. There were two types of elevated shed: whole covering and partially covered. Table 1 shows a list of column base conditions and member cross-sections of the elevated shed in the analysis model, and Table 2 shows the member cross-sections of the viaduct. In the separation model of an elevated shed with pin support as the column base fixing condition, the member section of the elevated shed had a member



Fig. 2 Analysis model (Unit: mm)



Fig. 3 Effect of rotational deformation of multi-span viaduct

Analysis case	Elevated shed Type	Column base fixing Condition	Column	Beam	Interlayer deformation angle at 0.25 of layer Shear force coefficient
C1-P	Partially	Pin	H-300×300	$H-340 \times 250$	1/223
C1-F	covered type	Rigid connection	imes 10  imes 15	imes 9  imes 14	1/950
C2-P	Whole covering type	Pin	$\Box$ -450×450	H-600×300	1/235
C2-F		Rigid connection	imes 22	$\times 14 \times 23$	1/794
C2-F(×2)	6.51		Double the rigidity against C2-F		

Table 1List of Analysis Cases

Table 2 Member cross-sections of viaducts (Common to each case, Unit: mm)



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stress that was less than the allowable stress level and the interlayer deformation angle was set to be approximately 1/200 when a horizontal force was applied on a layer shear force coefficient of 0.25. The station viaduct was made of 3 spans of reinforced concrete perpendicular to the rail track in accordance with seismic standards. We set the foundation support conditions as pin support at the tip of the pile, and horizontal ground springs at 1m pitch width. The same viaduct model was used in all case analyses.

## 2.2 Effects of viaduct with multiple spans

In order to examine the effect of rocking on the multi-span station viaduct, a static incremental analysis was applied to the two-dimensional frame model with only the viaduct among the models shown in Fig. 2 to apply a horizontal force to the beam core of the viaduct. From the relationship between the horizontal deformation  $\theta_v$  and the rotational deformation angle  $\theta_v$  at the core position of the beam of the viaduct at the yield point (Fig. 3), the correction factor  $k_{\theta} = \theta_v / \delta_v$  resulted in 0.0281 (1/m), given the rocking shown in the seismic standards. Since the result was within the range of  $k_{\theta} = 0.0166$  to 0.0719 (1/m) of the general section viaduct (1 span) listed in the annex to the seismic standards, and the overdesign factor of horizontal response seismic intensity (=  $1 + k_{\theta} \times H_s$ ) ( $H_s$ : height of the elevated shed) was about 1.14, the effect of rocking on the station viaduct was considered to be too large to be ignored.

## 2.3 Influence of the structure of the elevated shed

This study examined the effect of rocking the viaduct on an elevated shed with a ramen structure. The study examined the behavior of the elevated shed when the viaduct was rocked. Figure 4 shows the schematically disassembled structures. First, the behavior of the entire viaduct and elevated shed was decomposed into horizontal force acting on the viaduct and elevated shed (Fig. 4 (1)). When horizontal force ( $_{s}Q$ ) was applied only to the elevated shed (Fig. 4 (2)), the elevated shed deformed. However, since the mass ratio and rigidity ratio of the viaduct and elevated shed are quite different, the impact on the viaduct was small. On the other hand, when horizontal force ( $_{v}Q$ ) acted only on the viaduct (Fig. 4 (3)), the deformation of the viaduct affected the elevated shed response, and the interlayer deformation angle of the elevated shed  $_{v}\theta_{s} = _{v}\delta_{s}/H_{s}$  became smaller relative to the viaduct rocking  $_{v}\theta_{v}$ . This was because the beam was resisted in the direction that reduces the deformation of the column of the elevated shed differs depending on whether it is half-covered or not, and on the fixing condition of the elevated shed column base. Therefore, with regard to the relationship between rocking and elevated shed response due to differences in the shape and rigidity of the elevated shed and the column base fixing conditions, we compared with a shed-viaduct coupled model in which horizontal force was applied only to the viaduct (Fig. 4 (3)). It was examined by static incremental analysis (elasticity).

#### 2.4 Rocking correction factor for elevated shed

From the above study, we found that the effects of a viaduct rocking on the interlayer deformation angle of an elevated shed can be more appropriately evaluated by considering the shed shape and column base fixing conditions: however, the examination presented in this report needed to model the entire structure including the viaduct. When these operations were omitted, the maximum value of 0.83 in Fig. 5 was set to  $k_{\theta}$  for the purpose of evaluating the response characteristics of a ramen-structured shed safely. In addition, the rocking correction coefficient with a reduction factor of 1.0 was used for single-column sheds such as Y-type sheds unlike the ramen structure type.

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Fig. 5 Influence of the structure of the shed

## 3. Study on interaction in translational direction

The effect of the translational interaction between an elevated shed and a viaduct was examined by a parametric study using a two-mass system model. When inputting an L2sp II ground motion, it is highly likely that both the elevated shed and the viaduct are in the plastic zone. Therefore, when examining parametric studies, it is desirable to verify the response characteristics using models that take into account the non-linear characteristics of elevated sheds and viaducts. However, the response characteristics varied greatly depending on the analysis conditions (for example, yield seismic intensity and natural period etc.), and it was difficult to organize the conditions given to the response. Therefore, in this chapter, we conducted a parametric study on the analysis model of elevated shed (elastic)-viaduct (elastic-plastic) after organizing the influencing factors on the response characteristics by solving the equation of motion of the two-mass models. With correction of the effects of plasticization of the elevated sheds by the method described in Section 3.3, we obtained results considering plasticization of both elevated sheds and viaducts.

#### 3.1 Discussions based on theoretical formulas under harmonic external forces

Before conducting the parametric study, we derived a theoretical formula for obtaining the maximum response displacement at the time of harmonic external force input, with reference to the literature [3], and discussed the physical quantity contributing to the maximum response displacement.

From the equation of motion of the two-mass models in which only the viaduct yields (elevated upper house: elastic, viaduct:

complete elastoplastic), we were able to express the ratio of the maximum response displacement at the time of harmonic external force input as (1). From (1), the ratio of maximum response displacements (shed/viaduct) was expressed as a function of four valuables: 1) viaduct response plasticity ratio, 2) mass ratio (shed/viaduct), 3) natural period ratio (shed/viaduct), and 4) ratio of the equivalent natural period of the viaduct to the harmonic external force period. In the parametric study described in the next section, the maximum response displacement results for various combinations of viaduct parameters and elevated sheds parameters are organized using the above four variables.



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$$\frac{U_{s}}{U_{v}} = \sqrt{\frac{S(U_{v})^{2} + C(U_{v})^{2}}{\left(\frac{T_{eq}}{T_{g}}\right)^{4} - 2\left(\frac{T_{eq}}{T_{g}}\right)^{2}\left(\frac{T_{eq}}{T_{s}}\right)^{2}\left(\frac{m_{s}}{m_{v}} + 1\right) + \left(\frac{T_{eq}}{T_{s}}\right)^{4}\left(\frac{m_{s}}{m_{v}} + 1\right)^{2}}}$$
(1)

Where,

 $U_v$ ,  $U_s$ : Maximum response displacement of viaduct and elevated shed, respectively

 $T_{eq}$ ,  $T_s$ : Equivalent natural period of viaduct alone, and elastic natural period of elevated shed alone

- $T_{g}$ : Harmonic external force period
- $m_v$ ,  $m_s$ : Mass of viaduct and elevated shed, respectively
- $C(U_v)$ ,  $S(U_v)$ : Fourier series coefficients approximating a function representing the restoring force characteristics of the viaduct, as shown in the following formula

$$C(U_{\nu}) = \frac{1}{\pi} \left( \theta_{\nu}^{*} - \frac{1}{2} \sin 2\theta_{\nu}^{*} \right), \quad S(U_{\nu}) = -\frac{1}{\pi} \sin^{2} 2\theta_{\nu}^{*} \qquad \theta_{\nu}^{*} = \cos^{-1} \left( 1 - \frac{2}{\mu_{\nu}} \right)$$

 $\mu_{v}$ : Rate of viaduct response plasticity

#### 3.2 Influence of the structure of the elevated shed

Figure 6 and Table 3 show the analysis model and analysis parameters used for the parametric study, respectively.

We modeled a viaduct with a trilinear restoring force characteristic with reference to an actual viaduct model designed in accordance with the seismic standards used in the previous chapter. The equivalent stiffness  $K_{eq}$  at the second break point of the viaduct was set in such a way that the equivalent natural periods  $T_{eq}$  were 0.6 and 0.8 seconds. In addition, the initial stiffness and the first break point strength  $Q_{yl}$  were constant values, and the second break point strength (yield strength)  $Q_{y2}$  were set in such a way that the response plasticity factors  $\mu_{v}$  were approximately 1.0, 2.0, 3.0 and 4.0 at the time of L2spII ground motion input to the viaduct alone by each  $T_{eq}$ . Viscous damping was set to 10% of the mass of the viaduct combined with the damping of the structural member and dissipation decay. The elevated shed was elastic in its restoring force characteristics and the mass ratios (shed/ viaduct) were 0.03, 0.05 and 0.10, and the natural period ratio (shed/ viaduct, where the natural period of the viaduct is  $T_{eq}$ ) was set to 0.1 to 1.2 (0.1 intervals) to include 0.17 to 0.64 seconds, which were shown in the previous study [2] by the authors to be the natural period of the elevated shed. The viscous damping was set at 2% with respect to the elevated shed point mass. The input ground motion was L2spII ground motion and ground type was G2.

Figure 7 shows the relationship between the natural period ratio (shed/viaduct) and the maximum response displacement in all cases of the parametric study. Although we confirmed that the response value tended to increase as the natural period ratio increased as a whole, the results varied greatly depending on the characteristics of the viaduct, and the results also varied greatly even if the natural period ratio was the same. Therefore, when estimating the maximum response displacement of an elevated shed using this graph (for example, estimation using an envelope with an excess rate of 5%), some conditions may produce an unnecessarily large estimate. Figure 8 shows the analysis results arranged according to the elevated shed and viaduct characteristics shown in (1). Overall, response displacement increased as the natural period ratio increased, and reached a peak when the natural period ratio exceeded 1.0. In addition, when compared with the same viaduct equivalent natural period, the greater the plasticity rate of the viaduct (lower yield strength  $Q_{y2}$ ) or the greater the mass ratio, the smaller the response displacement. When compared with the same viaduct plasticity rate, the greater the equivalent natural period of the viaduct became, the greater the response displacement.

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As described above, using the information obtained from the viaduct design documents (equivalent natural period, plasticity, maximum response displacement, etc.), we can organize the response displacement spectrum by the natural period and mass of the elevated shed, and derive a reasonable response displacement.



1	able 3 Analysis	parameters	
	Restoring Force characteristic	Linear	
Elevated elevated	Mass ratio (Shed/ viaduct)	0.03, 0.05, 0.10	
shed	Natural period ratio (Shed/ Viaduct)	$0.1 \sim 1.2$ 0.1 Intervals	
	Viscous damping	2%	
	Restoring force characteristic	Takeda model	
	Mass $m_v$	691[t]	
Viaduct	${ m Equivalent} \ { m Natural period} \ { m $T_{eq}$}$	0.6, 0.8 Sec.	
	Viscous damping	10%	
	Yield strength $Q_{\!\scriptscriptstyle Y\!2}$	To set as $\mu_v \approx 1.0, 2.0$ 3.0, 4.0 when inputting L2sp II	



Fig. 7 Response displacement analysis results when the elevated shed is elastic

(Plotted all analysis results)



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Fig. 8 Response displacement spectra organized by characteristic of viaducts and elevated sheds

#### 3.3 Response displacement spectra considering plasticization of elevated shed

Until the previous section, we examined the elevated shed as elastic for convenience. In this section, we estimated the effect of plasticization of the elevated shed and the elasto-plastic response from the above elastic response results using the constant energy law, and created a response displacement spectrum that considers the plasticization of the elevated shed.

We know that the response of a structure decreases in accordance with hysteresis damping due to plasticization. Therefore, we used the calculation formula (2) for the response reduction rate  $F_h$  based on the plastic ratio  $\mu_s$  of the elevated shed shown in the limit strength calculation notification (Ministry of Construction Notification No. 1457 in 2000). We also estimated the effect of elongating the natural period when the elevated shed was plastic from the previous study [5] using (3).

$$F_h = \frac{1.5}{1+10h}$$
(2)

$$T_{s}^{'} = \sqrt{\mu_{s}} \times T_{s} \tag{3}$$

Where,

$$h = 0.25 \times \left(1 - \frac{1}{\sqrt{\mu_s}}\right) + 0.05$$



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For example, when the response plasticity of the elevated shed  $\mu_s$  was 3.0, in the event of L2sp II ground motion, h,  $F_h$  and  $T_s'/T_s$  became 0.16, 0.59 and 1.73, respectively. Therefore, by applying these numerical values to the response displacement spectra shown in Fig. 8, it is possible to consider the effect of plasticization of elevated sheds. Figure 9 shows a comparison between an example of response displacement spectra considering the effect of plasticization by this method (0.05 as mass ratio, 0.8 s as viaduct equivalent natural period, 3.0 as viaduct response plasticity ratio) and the results of elasto-plastic time history response analysis with a bilinear skeleton curve of the elevated shed in the analysis model in Fig. 6. Here, the natural period of the elevated shed on the horizontal axis was the elastic natural period  $T_s$ . It is possible to see that the model was able to accurately evaluate the effect of plasticization.

## 4. Creation and verification of response displacement spectra

We created a response displacement spectrum for the duration of the L2sp II ground motion by considering the effect of rocking, obtained in Chapter 2, on the response displacement considering only the translation direction obtained in Chapter 3. Figure 10 shows the flow chart for how this was created. The response displacement spectrum was then produced on the basis of the response displacement spectra (① in Fig. 10) arranged in section 3.2 according to the characteristics of the viaduct and elevated shed when the elevated shed was elastic, and updated this with corrections made by extending the natural period (② in Fig. 10) and reducing the response due to hysteresis damping (③ in Fig. 10) as the effects of plasticization of elevated shed in section 3.3, and multiplying by the rocking correction coefficient obtained in Chapter 2 (④ in Fig. 10). The response displacement spectra were shown in multiple graphs in the same manner as in Fig. 8 for each characteristic of the viaduct and elevated shed, but here the mass ratio was 0.05, the viaduct equivalent natural period 0.8 s, and the elevated shed plasticity ratio 1.5. This case is shown in Fig. 11 as a representative example. Figure 11 also shows the results of nonlinear time history response analysis using the full-cover type shed-viaduct integrated frame model shown in Figure 2. Comparing the created response displacement spectra with the response analysis results using the nonlinear frame model, the model enabled us to predict the response of the elevated shed more accurately.

Fig. 12 shows the response displacement spectrum when the elevated shed plasticity ratio is 3.0, which was created with the method shown in Fig. 10. Compared with the response displacement spectrum when the elevated shed is elastic state (Fig. 8), the natural period ratio at which the response displacement reaches peak was about 1.0 when the elevated house is elastic state, but has moved to about 0.7. As a result, in the region where the natural period ratio is equal to or less than 1.0, the response displacement is often large as compared with the case that the elevated shed is elastic state. This is because the natural period ratio of the elevated shed that resonates with the viaduct is shorter than 1.0 in consideration of the elongation of the natural period due to the plasticization of the elevated shed. In addition, the response displacement that peaks out is smaller due to the damping effect due to the plasticization of the elevated shed.

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Fig. 10 Flow chart for creating response displacement spectra







Fig. 12 Response displacement spectra organized by characteristic of viaducts and elevated sheds

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# 5. Conclusions

We proposed a method to obtain through simple means the response displacement of the elevated shed when applying L2sp II ground motion by organizing the effects of rocking and the translational direction in order to consider the coupled behavior of the viaduct and the elevated shed, and by conducting analyses. The findings were as follows:

- Although the influence on the elevated shed response due to the rocking of the station viaduct could not be ignored, we showed that the rocking correction factor was lower in the ramen-structure-typed shed compared to the single column type.
- We theoretically examined the influence of the interaction in the translation direction and arranged for the influence factors on the response displacement of the elevated shed at the time of earthquake input. We also showed that the response displacement can be estimated by classifying the characteristic values and targets of viaduct and elevated shed.
- Using the interaction in the translational direction and the rocking correction factor, we created a response displacement spectrum that considers the coupled behaviors of the viaduct and the elevated shed confirmed its validity.

Although not included in this report, we also proposed a method to calculate the deformation capacity of the elevated shed from the components of the elevated shed. Therefore, by estimating the response displacement using the proposed response displacement spectra and comparing it with the deformation capacity, it is possible to check the safety of an elevated shed during L2sp II ground motion.

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