Method for Estimating Maximum Response Displacement of Sheds on Viaducts Considering Coupled Vibration Behavior Between the Shed and the Viaduct

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It is considered that the best way to calculate the earthquake response of sheds on viaducts, is to use an integral model of the shed and the viaduct. Calculations based on integral models require a vast amount of work because of the need to understand the structural design of both the shed and the viaduct. This report describes the factors affecting the earthquake response of sheds. A method is proposed for estimating the maximum response displacement of sheds considering the coupled vibration behavior between the shed and the viaduct based on structural design information obtained for the shed and the viaduct. The applicability of this method was verified under the earthquake conditions that appear in the seismic design standards for railway structures.

Keywords: shed, viaduct, interaction, rocking

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 earthquake response of the elevated shed. As a result, a seismic design method was proposed based 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 [1] (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’ [2]), 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.

Therefore, this report organizes the different response characteristics of elevated sheds considering the coupled behavior of the viaducts and the elevated sheds by considering the influence of rocking and the effect on the translational direction. A simple method is proposed to obtain the response displacement of an elevated shed in the case of an L2 ground motion spectrum II (hereinafter referred to as L2sp II ground motion), as defined in the seismic standards, from the design value of the viaduct and the design target value of the elevated shed without designing an integrated model with the viaduct.

![Fig. 1 Example of an elevated shed](image)

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.

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 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 \( k_\theta = \theta_v / \delta \), 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 \( = 1 + k_\theta \times H_r \) (\( H_r \): 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.

| 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 covered type | Pin                         | H-300 × 300 \( \times 10 \times 15 \) | H-340 × 250 \( \times 9 \times 14 \) | 1/223                                                         |
| C1-F          | Rigid connection    |                             |        |      |                                                              |
| C2-P          | Whole covering type | Pin                         | □ -450 × 450 \( \times 22 \) | H-600 × 300 \( \times 14 \times 23 \) | 1/235                                                         |
| C2-F          | Rigid connection    |                             |        |      |                                                              |
| C2-F(×2)      |                    |                             |        |      | Double the rigidity against C2-F                              |

Table 1 List of Analysis Cases

| Pile | Underground beam | Upper beam | Column |
|------|------------------|------------|--------|
| 1200 | 900              | 900        | 1000   |
| (50345) | 85 710 85 | (50345)   | 830   |
| Main reinforcement : 20-D32 | Shear reinforcement : D19-2 bond @150 | Shear reinforcement : D19-2 bond @125 | Shear reinforcement : D19-1.5 bond @100 D19-1.5 bond @125 |

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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 \( Q \) 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 \( Q \) 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 \( \theta = \delta / H \) became smaller relative to the viaduct rocking \( \theta_s \). This was because the beam was resisted in the direction that reduces the deformation of the column of the elevated shed because the elevated shed had a ramen structure. In addition, we assumed that the size 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).

Figure 5 shows the ratio \( \theta_s / \theta \) of the interlayer deformation angle of the elevated shed to the viaduct rocking in each analysis case. Here, we set \( \theta_s \), as the average value of the rotation angles of all nodes on the elevated shed column base. In all analysis results, \( \theta_s / \theta \) was never more than 1, and the interlayer deformation angle of the elevated shed was smaller with respect to rocking of the viaduct. In addition, when compared with the column base fixing conditions, the column base rigid connection was larger than the column base pin, and the effect of viaduct rocking on the deformation of the elevated shed was small in the column base pin. Since the column base of an actual elevated shed had a certain degree of rigidity, including the exposure type and the root winding type, we considered the results would lie between those of the pin and the rigid connection analyzed this time.

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 \( k_0 \), 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.

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 variables: 1) viaduct response plasticity factor, 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.

\[
\frac{U_v}{U_s} = \frac{S(U_s)^2 + C(U_s)^2}{\left(\frac{T_e}{T_s}\right)^4 - \frac{T_e}{T_s} \left(\frac{T_e}{T_s}\right)^2 \left(\frac{m_s}{m_v}\right) + 1} \left(\frac{T_e}{T_s}\right)^4 \left(\frac{m_s}{m_v}\right)^2
\]

(1)

Where,

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

\[
C(U_s) = \frac{1}{\pi} \left(\frac{\mu_s}{2} - \frac{\sin 2\mu_s}{2}\right) \quad S(U_s) = -\frac{\sin^2 2\mu_s}{2} \quad \mu_s = \cos^{-1}\left(1 - \frac{2}{\mu_v}\right)
\]

\(\mu_v\): 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 \(Q_1\) were constant values, and the second break point strength (yield strength) \(Q_2\) 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 and natural period of the elevated shed were determined relative to the viaduct. In particular, 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 \(T_p\) 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 [1] 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 analy-
The viscous damping was set at 2% with 0.8 μ.

In this section, we estimated the effect of plasticization of the elevated shed using the constant energy law, and created a response displacement spectrum considering the plasticization of the elevated shed. We know that the response of a structure decreases in accordance with hysteresis damping due to plasticization.

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.

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 3.0. 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 Fig. 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.

1. Response displacement spectra of shed (elastic) - viaduct (elastic-plastic)
2. Elongation of natural period by plasticization
3. Response reduction by hysteresis damping
4. Effect of viaduct rocking

Interactive in translational direction (Chapter 3)

Effect of plasticization of the shed (Section 3.3)

Response displacement spectra that consider rocking of the shed (elastic) - viaduct (elastic-plastic)

Fig. 10 Flow chart for creating response displacement spectra

Fig. 11 Accuracy checking with frame model

5. Conclusions

We 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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