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SEISMIC ANALYSIS OF HIGH PIER VIADUCT WITH HIGHWAY AND RAILWAY

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Bannosu Viaduct one of the Honshu Shikoku Bridges, neighboring to the Bisanseto Suspension Bridge, is panned to carry 4 lane roadways and 4 railway lines. This viaduct has the high piers, about 40~70m, and the pile foundations deeply penetrated into the comparative soft ground.

The behavior of these structures during an earthquake, therefore, is affected by the dynamic characteristics of foundation-pile systems.

The design of the structures considering earthquake-proof and the running stability of railway cars during the earthquake is required.

In particular, to secure the running stability of railway cars, it is important to design the viaduct having a definite range of dynamic characteristic. The viaduct must be designed to avoid resonating with the ground vibraion.

1. INTRODUCTION

Bannosu Viaduct belonged to the Kojima-Sakaide Route of the Honshu-Shikoku Bridges is about 3 km long duble deck type bridge with the upper for roadway and the lower for railway, and has such high piers as 40 to 70 m height due to connect with South Bisan-Seto Bridge which is the suspension bridge over the Bisan-Seto. The pile system is adopted to these foundations because the ground is comparatively soft and the supporting layer is 30 to 70 m deep from the ground surface. This viaduct is therefore essentially liable to oscillate, and the earthquake-resistant design is important in structural design. The design of this bridge is required to bear the earthquake force but also to certify the runnability of train.

The railway cars on the vibrated rails, if their amplitudes are beyond the ultimate values determined by the frequency, would be induced to such the dangerous situation that they are finally derailed and turn over. The design for this viaduct must be done to the specially ranged dynamic characteristics.

This report describes mainly the seismic analysis of the high pier viaduct for bannosu Viaduct on the soft ground to secure the running stability of train.

2. OUTLINE OF THE VIADUCT

This viaduct is a highway-railway bridge whose structural standard is shown in Table 1, the general view in Fig. 1.

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Table 1 Structural standard.

	Highway	Railway	
Class	1 - 2nd	Conventional Line	Shinkansen
Design Speed	100 km/h	120 km/h	160 km/h
Minimum Redius of Curve	750 m	1,000 m	1,300 m
Maximum Longitudinal Grade	4.8%	1.5%	1.5%
Cross Grade	2%		
Number of Lanes	4		
Gauge		1.067 mm	1.435 mm
Distance between Track-center		3.8 m	4.3 m
Number of Tracks		2	2

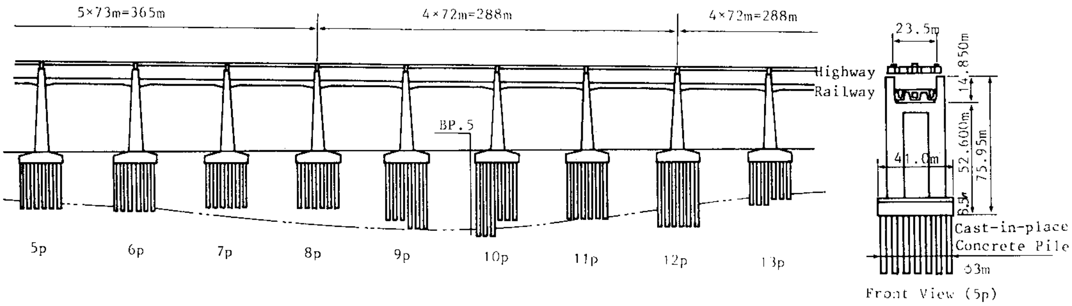

Fig.1 A side view of Bannosu Viaduct.

3. FUNDAMENTAL CONDITION FOR SEISMIC ANALYSIS

This chapter describes first such fundamental conditions for seismic analysis as the ground properties, the limit conditions of vibration for bridges to secure the running stability of train, the earthquake motion to be input in analysis, and so on.

(1) Characteristics of Ground Motion

It is necessary to clear the shear modulus and the damping constant of ground for the dynamic analysis at the earthquake. These ground constants are generally affected by such many elements as the sort of soil, the void ratio, the confining pressure, and particularly the magnitude of strain.

The equivalent linear model is adopted with the equivalent shear modulus and the hysteretic damping constant to reflect these gound properties to the dynamic analysis in this report.

As for the effects of strain on shear modulus G and damping constant ß, the investigation results are used which were conducted by Public Works Research Institute, Ministry of Construction[1].

The earthquake motion used in this analysis is obtained from the record on the free field ground surface near Kaihoku Bridge by the Miyagi-Ken-Off Earthquake (Jun. 2, 1978). In this case, the wave is led with the method of normalizing the maximum acceleration perpendicular to the bridge axis as 100 gal, and the following will call this wave record as Kaihoku Bridge Tr, Miyagi-Ken-Off Earthquake.

(2) Allowable Condition of Vibration for Bridges

It is necessary to restrict the horizontal vibration of railway to secure the running stability of train.

The safety limit curve against derailment for two axle freight car and Shinkansen car[2], shown in Fig. 5 is used in this report.

(3) Earthquake Motion to be Considered in Analysis

Because the problems in the seismic analysis of this viaduct are the amplitude and the frequency, the earthquake motion considered in analysis is necessary to be input as the wave shape with the time history.

a) Expectation for Amplitude of Acceleration

The scale of the earthquake which is a basic condition for the seismic analysis should be determined with the synthetical consideration of the degree of the damage by earthquake, the probability of occurrence, the type or the degree of importance of the structure, and the like.

Considering the above mentioned factors synthetically, it can be adopted for the earthquake on this analysis that the magnitude is around 8 and distance is 150 km, and it could be broken out at one to two times in 100 years[3].

There have been proposed many equations about the relation between the maximum acceleration of ground, the earthquake scale and the epicentral distance. As a recent proposition, the New Method of Seismic Design was publicized in 1977[4]. According to this method, the maximum horizontal acceleration of the ground type 1 (bedrock area) can be estimated to be 65 gal.

Next, there must be considered the superposition of the earthquake strengths worked for each direction and the probability for outbreak on this case[5].

When one direction of earthquake strength is 1 and another direction perpendicular to the former is a (a $=2/3$ in this case), the earthquake strength to free direction can be $\sqrt{1+a^2}$ and therefore this value is 1.2 so that the acceleration at the bedrock layer is 78 gal.

And, moreover, considering the deviation between the estimated value and the measured value on which the above method is based, the expectation of the acceleration on this seismic analysis is 100 gal when the probability of the measured one being smaller than the estimated one is 70 %.

b) Selection of Wave Shape

The wave shape of earthquake motion should be selected with the synthetical consideration of the degree of acceleration, the scale of earthquake, and the characteristics of acceleration response of spectrum from the past records. It can be concluded that as the result, the Kaihoku Bridge T_r, Miyagi-Ken-Off Earthquake, is most appropriate for this case.

This analysis as the above results adopts the wave shape led with the method of converting the maximum acceleration to 100 gal in the record of the Kaihoku Bridge T_r, Miyagi-Ken-Off Earthquake.

4. CONSIDERATION OF OSCILLATION ANALYSIS METHOD

The consideration of the dynamic interaction between the ground and the structure is of significance to analyze the oscillation behavior at earthquake for the structure penetrated into the deep soft-ground.

There have been so many research reports for the dynamic interaction between ground and structure.

It may be necessary to evaluate the application for these methods which have each characteristics respectively, and otherwise the finite element method is adopted in this case.

As for the analytical model of the ground and pile foundation system with the finite element method, there are several problems like extent of area, boundary condition, dividing of elements, and model of group of piles.

The thickness of ground element is assumed to be equal to the width of group of piles perpendicular to the earthquake motion so that it can be the plane strain problem.

The ground of this vaiduct is the complicated multilayer structure. This stratum is therefore divided into about ten layers as shown in Fig. 2 which is based on the classification by the veocity of shear wave in division and the results of response analysis of a few models.

According to the results of a few trials, it should be necessary that the width of ground is at least four times of the depth of ground.

The group of piles is represented as a one-dimensional beam put at the foundary between the ground elements, which is converting two lines of piles to one virtual pile with the equal bending rigidity.

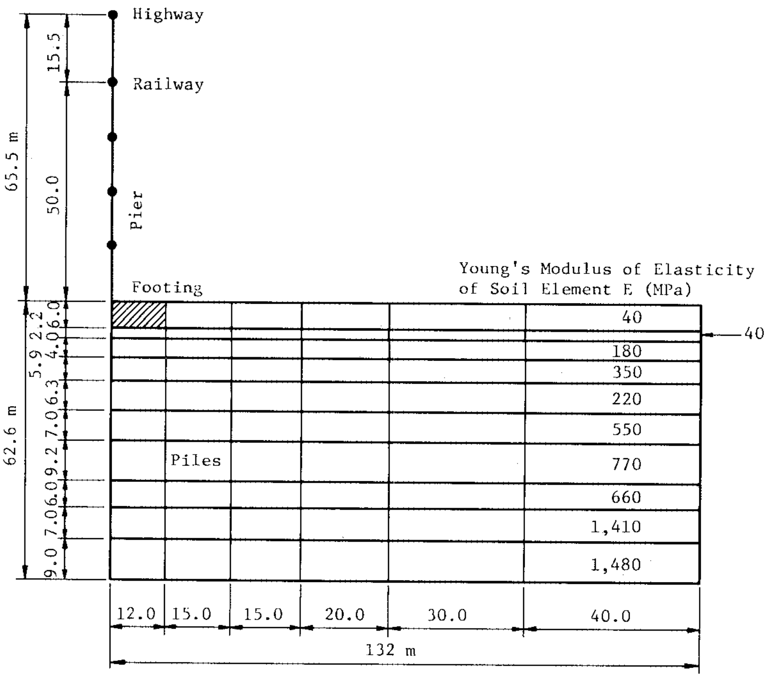

Fig.2 The total system of high frequency bridge.

The coupling oscillation analysis of the pier-superstructure system and the ground-foundation system is for convenience sake divided for the actual analysis because of the capacity of computer.

The following shows the fundamental equation in this case. The absolute joint displacement of the pier-superstructure system is expressed as $\{U\}$ and the original coordinates as $\{q\}$.

$$\{U\}=\{u\}+\{1\}\cdot u_f+\{Y\}\cdot \varphi+\{1\}\cdot z \qquad (1)$$

$$\{u\}=[\phi]\cdot\{q\} \qquad (2)$$

$$\{\ddot{q}_n\}+2h_n\omega_n\{\dot{q}_n\}+\omega_n^2\{q_n\}=-F_{un}(\ddot{u}_f+\ddot{z})-F_{\varphi n}\ddot{\varphi} \qquad (3)$$

where

u : relative displacement of joint to footing

u_f, φ : relative displacement and rotational displacement of top of footing against bedrock

Y : horizontal displacement of joint caused by a unit of rotation at the top of footing

Z : horizontal displacement of bedrock caused by earthquake motion

ϕ : modal matrix

h_n : damping constant of nth-mode vibration

ω_n : nth-mode natural circular frequency

F_{un}, $F_{\varphi n}$: stimulant coefficient of nth-mode vibration

[ ], { } : matrix and vector

$\dot{}$, $\ddot{}$: marks for velocity and acceleration respectively

The earthquake motion for pier-superstructure system should be comprised of response horizontal displacement, horizontal acceleration, rotational displacement, and rotational acceleration at the top of footing in the ground-foundation system. However, the influence of rotational acceleration is relatively small so that it can be closely resembled with such method of obtaining first the relative displacement of pier-superstructure system caused by response horizontal acceleration at the top of footing and adding the influence of horizontal displacement and rotation at the top of footing.

Although, generally, the influence of mass and rotational inertia in the pier-superstructure system against the vibration of the ground-foundation system should be also considered, in this case, this problem can be solved in a resemble manner by the method of converting the pier-superstructure system to the lumped mass model and adding it to the model of ground-foundation system[6),7)].

All the displacement and cycles on the rail level which are necessary for evaluation of running stability of railway cars are obtained as follows.

a) to determine the typical analysis model comprised of total system such as the ground-foundation-pier-superstructure at each site point and to conduct the direct response analysis of time hitory.

b) to conduct the response analysis inputting response horizontal acceleration of time history at the top of footing obtained from the result of a) at the bottom of pier model of pier-superstructure system considered.

c) to obtain all the response displacement with the consideration of (1) maximum response horizontal displacement at the rail level which is obtained by b), (2) maximum response horizontal displacement at the top of footing obtained by a), and (3) horizontal displacement at the rail level caused by maximum response rotational angle at the top of footing in case of regarding the pier-superstructure system as rigid body. The assumed value for the total response value is supposed to be the sum of each absolute value for (1), (2), and (3), taking consideration of safety.

d) to check the running stability of railway cars comparing the total response displacement which is computed in c) with allowable condiction of vibration, regarding the cycle of the mode which contributes most to the displacement of superstructure as the cycle at the rail level.

5. STRUCTURAL TYPE OF VIADUCT

It can be expected that the motion of the structure which has the deep penetration in the soft ground at earthquake is affected so much by the motion of ground. It is desirable that the natural period of ground-foundation system is different from the one of pier-superstructure system with a view of reducing the influence of ground motion.

It is supposed to be such that the dynamic characteristics of ground-foundation system is dominated by the gound motion. Therefore, it is desireable that the nature period of pier-superstructure system is less than the half or more than two times of the excelled period of ground motion to avoid resonating with ground motion. The excelled period of ground motion is around 1 sec, accordingly, the structural type of viaduct can be classified as follows.

a) High Frequency Bridge

High frequency bridge is the type of which the first-mode natural period of pier-superstructure system is less than around 0.5 sec.

The bridges which are supposed to be the high frequency bridge from our experience are such as steel girder bridge or PC girder bridge with the span of 40~80 m long. Because the most econmic type of bridge is such as the continuous steel box girder bridge with the RC slab for roadway and the continuous PC box girder bridge for railway with the span of 60~80 m long, this case adopted the above mentioned type of bridge with the span of 80 m long.

b) Low Frequency Bridge

Low frequency bridge is the type of which the first-mode natural period of pier-superstructure system is more than around 2.0 sec.

The typical low frequency bridge is a long span bridge such as suspension bridge, cable stayed bridge, and the like. However, these types of bridges are excluded because of the site condition and the economic condition in this case so that it is supposed to be as three spans continuous truss for superstructure and rigid pier for the end/non rigid pier for the middle to give the long period property needed.

The basic view of truss is as follows. The height of main truss is 18 m high. The width of main truss is 26.6 m wide. And, the sway bracing is framed truss. The floor is RC floor slab for roadway and open grating for railway.

Although there are several comparative ideas for the structure of the middle pier, the rocker pier with poor resistance to horizontal displacement is adopted and the simple truss for perpendicular to bridge axis with the length of three spans is formed for the purpose of lengthening the vibration period.

From the appropriate span length of 90~120 m for superstructure, in this case, 120 m long truss is considered.

c) Middle Frequency Bridge (to compare with a) and b))

Middle frequency bridge is the type of which the first-mode natural period of pier-superstructure system is equal to around 1 sec.

The middle frequency bridge is given by converting the flexible middle pier of low frequency bridge to the high rigid pier of RC structure.

For any cases of the above mentioned structural types, the ground condition is BP. 5 and the foundation is the cast in place concrete pile with 3 m in diameter.

6. OSCILLATION BEHAVIOR OF VIADUCT AND RESPONSE ANALYSIS

The seismic response analysis of viaduct is conducted with dividing into the ground-foundation system (total system) and the pier-superstructure system as mentioned above.

(1) High Frequency Bridge

On the model of superstructure, only the mass is taken out because the influence of vibration of girder is small in case of high frequency bridge for the transverse rigid stiffness of girder. The analytical model of total system is shown in Fig. 2.

The main modes are shown in Table 2 from the result of the natural oscillation analysis only for the pier-superstructure system.

Table 3 shows the oscillation characteristics of main modes for the total system model including the ground in high frequency bridge. And Fig. 3 shows the mode diagram.

Table 4 shows the maximum response value at the top of footing in case of inputing the design earthquake wave to the bearing layer. The rotational displacement in the table is the value obtained from the height difference of center and end of footing.

Table 5 shows the response displacement of the main mode at the point correspond to the rail level and the square roote for sum of square of such value as combined one in case of inputting the response acceleration wave at the top of footing to the analysis model for the pier-

Table 2 Natural vibration characteristics of pier-superstructure system (High Frequency Bridge).

Vibrational Mode	Natural Period (sec.)	Participation Factor	Effective Mass (%)
1	0.561	28.0	38.9
2	0.294	28.1	39.3
3	0.071	17.6	15.3

Table 3 Natural vibration characteristics of total system (High Frequency Bridge).

Vibrational Mode	Natural Period (sec.)	Participation Factor	Effective Mass (%)
1	0.828	154.1	50.4
2	0.656	88.5	16.6
3	0.478	23.6	1.2

Table 4 The maximum response at the top of the footing.

(High Frequency Bridge)

	Lateral Displacement	Rotation
Response	1.35cm	0.287×10^{-3}rad

Table 5 Response displacement at the rail level.

(High Frequency Bridge) Unit: cm

Vibrational Mode	1	2	3	Total
Displacement	0.56	0.68	-	0.88

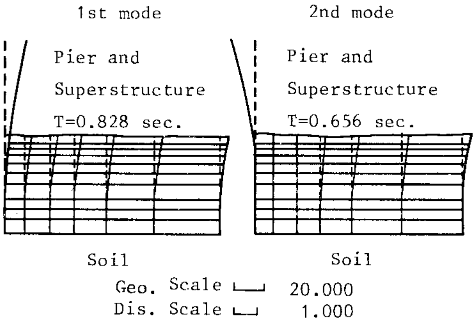

Fig. 3 Vebrational mode of total system (High Frequency Bridge).

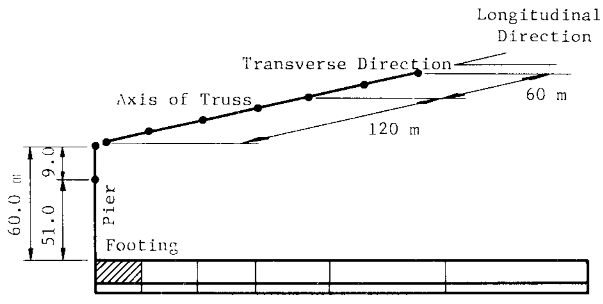

Fig. 4 The total system of low frequency bridge.

superstructure system.

When the height from the top of footing to the rail level is 50 m high, from the combined method of displacement mentioned in 4, the absolute maximum horizontal displacement at the rail level is about 3.7 cm $(=1.35+0.287\times10^{-3}\times50\times10^{2}+0.83)$.

The maximum horizontal displacement at the rail level in another direct response analysis for the total system is approximately 2.6 cm so that the calculation method for total displacement mentioned in 4 is understood to be the safer one.

(2) Low Frequnecy Bridge

Fig. 4 shows the model for the total analysis of the low frequency bridge.

The analysis for the pier-superstructure system of low frequency bridge is conducted with the model for framed structure analysis formed with the gravity center line of pier and truss because the rigidity of the middle pier is small and the influence of the motion of girder in the direction of perpendicular to bridge axis is not ignored.

Table 6 shwos the main modes from the result of natural vibration analysis only for the pier-superstructure system.

Considering the displacement of low frequency bridge, the first mode is excelled at the middle of span and the third and ninth mode is excelled at the end support.

Table 7 shows the oscillation characteristics of main mode from the result of natural vibration analysis for the total system model. The first mode is the vibration mainly of girder and the vibration of ground or end pier is expressed in the second and third mode. Table 8 shows the maximum response value at the top of footing in case of inputing the design earthquake wave to the bearing layer. And Table 9 shows the response value of main mode at the point correspond to the rail level and the combined values with them in case of inputing the response acceleration wave at the top of footing to the analysis

Table 6 Natural vibration characteristics of pier-superstructure system (Low Frequency Bridge).

Vibrational Mode	Natural Period (sec.)	Participation Factor	Effective Mass (%)
1	4.045	37.8	45.0
3	0.558	19.3	11.7
9	0.292	23.0	16.7

Table 7 Natural vibration characteristics of total system (Low Frequency Bridge).

Vibrational Mode	Natural Period (sec.)	Participation Factor	Effective Mass (%)
1	4.126	28.9	1.4
2	0.888	-106.0	18.9
3	0.730	-167.6	47.1

Table 8 The maximum response at the top of the footing (Low Frequency Bridge).

	Lateral Displacement	Rotation
Response	1.32 cm	0.512×10^{-3} rad

Table 9 Response displacement at the rail lelvel (Low Frequency Bridge).

Unit: cm

Vibrational Mode	At the Middle Point of the Center Span	At the End Pier
1	11.77	0
3	-1.65	1.24
9	0.85	1.59
Total	11.92	2.02

model for the pier-superstructure system.

The absolute horizontal displacement value is 15.8 cm at the center of center span and 5.9 cm at the end support with the same method as the high frequency bridge.

(3) Middle Frequency Bridge

The oscillation analysis of middle frequency bridge is conducted with the model for framed structure in the same way as the low frequency bridge.

The absolute maximum horizontal displacement at the position correspond to the rail level is obtained such as 9.8 cm at the center of center span and 6.0 cm at the end support.

7. EVALUATION ON RUNNING STABILITY OF TRAIN

The following is mentioned about the evaluation on the running stability of train.

The safety limit curve against derailment of train shows the limit amplitude and the limit frequency at the rail point for being derailed when the stationary sine wave motion works on the rail of car model.

The definition of frequency and the number of waves necessary to come to be derailed are another problem to apply the safety limit curve to the earthquake respouse wave as an irregular wave.

Because of the modal analysis in this study, the frequency at the rail level is represented by the natural frequency of the mode which dominates with the displacement in the vibration of the pier-superstructure system and the ground-foundation system.

The number of waves is not taken into consideration in this case for safety side because the result of the simulation to define the safety limit curve against derailment indicates that the railway car reaches to the limit state with vibration less than five waves[8].

Accordingly, Table 10 shows totally the relationship between the absolute maximum horizontal displacement and the frequency at the rail level for each type of viaduct. The item of the excellent period in the table shows as follows that (1) is the natural period of the mode with the maximum effective mass in the analysis for the total system, (2) is the natural period of the mode excelling in the displacement of the superstructure in the pier-superstructure system, and (3) is the period at the rail level which is the value of (1) in case that the ground-foundation system is excellent in the response displacement, and the value of (2) in case that the pier-superstructure system is excellent.

Table 10 Excellent period and displacement of railway.

		High Frequency Bridge	Low Frequency Bridge		Middle Frequency Bridge	
			At the Middle Point of the Center Span	At the End Pier	At the Middle Point of the Center Span	At the End Pier
Excellent Period (sec.)	(1) Total System	0.828	0.730	0.730	0.730	0.730
	(2) Pier-Superstructure System	0.561 ∿ 0.294	4.045	0.558 ∿ 0.292	0.816	0.816 ∿ 0.344
	(3) Superstructure	0.828 (1.21 Hz)	4.045 (0.25 Hz)	0.730 (1.37 Hz)	0.816 (1.23 Hz)	0.730 (1.37 Hz)
Absolute Lateral Displacement at the Rail Level		3.7 cm	15.8 cm	5.9 cm	9.8 cm	6.0 cm

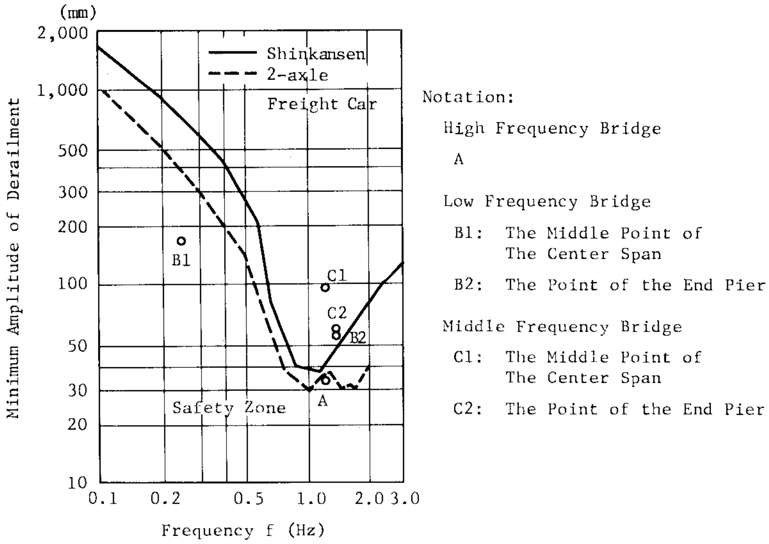

Fig. 5 Inspection for the running stabilits of railway cars.

Fig. 5 comes from entering the result, of Table 10 to the safety limit curve.

8. CONCLUSION

(1) The method of response analysis at the earthquake time is simplified by the method of dividing to the pier-superstructure system and the ground-foundation system.

(2) The structural type of viaduct classified to three types.

The middle frequency bridge is the most disadvantageous for the running stability of train because the period at the rail level is in the severe area of the safety limit curve and the response displacement value is large due to the resonance with the ground motion. The low frequency bridge is in safe on the running stability of train for characteristics itself at the middle of span but is still in question partly at the point near the end support. On the otherhand, the high frequency bridge is the most appropriate structual type in these three types because the response displacement value is small for almost no resonance with the ground motion.

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