Rail-Structure Interaction Analysis for Simple Span Bridges of the Taiwan High Speed Railway

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대만 고속전철 단순교의 레일-구조물 상호작용 해석

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Abstract : The additional stresses and displacements produced by the use of long rail, typical of the high-speed railway, are investigated for the Taiwan high-speed railway bridges. In addition, an important special feature of the Taiwan High Speed Railway Design Specifications specifies that service earthquake has to be considered during the rail-structure interaction analysis before evaluating the stresses and relative displacements of the bridge. As ground motion is taken into account under seismic event, the seismic response of the structure is applied as displacement in the rail-structure interaction analysis. The stresses and relative displacements of the structure are checked according to the consideration of seismic loading.

초 록 : 고속 전철 특징 중의 하나인 장대 레일의 사용으로 인하여 발생하는 추가 응력과 변위에 대하여 대만 고속전철 교량에 대해서 검토한다. 또한 대만 고속 전철 교량 시방 규정의 중요한 특징인 사용지진을 레일-구조물 해석에 고려하도록 하는 규정을 적용 후 단순교의 응력 및 상대변위를 검토한다. 지진 시 지반 운동을 고려하며, 단순교의 지진 응답을 변위로 레일-구조물 상호 작용 해석에 적용시킨다. 지진하중 고려 유무에 따른 단순교의 응력 및 상대 변위를 검토한다.
Key Words : rail-structure interaction, Taiwan high-speed railway, long rail, service earthquake

1. GENERAL

The use of long rail causes stress increase in the Taiwan high-speed railway. Especially, the discontinuity and variations of the stiffness of substructures in the bridge section produce additional stresses and displacements. In other words, binding forces like connection strength and friction force between ties and ballasts act as resistance forces, so that the stress and displacement of the substructure can be distributed. However, these resistance forces no more resist once the displacement reaches a certain bound and these resistance forces behave nonlinearly with bilinear property. And the reduction factor is 1.28-0.4. It means that Taiwanese earthquakes are stronger than Korean ones. The ground effects shall be considered in specific condition sites called particular site (Reference 1).

Besides, capacity for rail-structure interaction must also be satisfied under service earthquake. In particular, not only displacement safety due to acceleration and braking force but also safety under type II earthquake must be checked according to the Taiwan High Speed Railway Design Specification Volume 9. To check the above-mentioned matters, the stress and relative displacement of decks shall be checked with respect to computed deformations of the ground and structure subject to earthquake event.

In this paper, the response properties of long rails

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of simple span bridges for the Taiwan high-speed railway are analyzed under various loading combination cases. Then, the stresses of the rail and the relative displacements are checked according to the substructure properties.

2. SEISMIC ANALYSIS

During earthquake event, seismic accelerations are transmitted from the ground to the structure, that is, the ground motion affects seriously the motion of the structure. Therefore, the transmission of earthquake waves has to be defined considering the region properties and soil properties.

2.1. Ground Analysis

2.1.1. Soil Classification

Soil profile types may be classified into three types according to the period of the ground, T_G:

Type I	$T_G \leq 0.2$	seconds
Type II	0.2 seconds $< T_G \leq 0.6$	seconds
Type III	0.6 seconds $< T_G$.	

The ground period, T_G, can be calculated by the following equation:

$$T_G = 4 \sum_{i=1}^{n} \frac{H_i}{V_{si}} \tag{2.1}$$

where, H_i= thickness (m) of i-th subsoil layer;
V_{si}= shear wave velocity (m/s) of i-th subsoil layer at low strains (around 0.000001);
n= number of layers above the base layer as defined herein.

2.1.2. Modeling of Input Ground Motions

As the stiffness of the ground depends on the shear strain of the ground, the shear strain must be obtained from dynamic analysis under earthquake loading. Artificial earthquake waves are generated for each of the ground and soil conditions. And the obtained spectral density function is corrected by comparing the two response spectra as expressed in Eq.(2.2).

$$S_g(\omega) \rightarrow S_g(\omega) \left[\frac{RSA(\omega)}{RSA_S(\omega)} \right]^2 \tag{2.2}$$

where $RSA(\omega)$ is the design response spectrum and $RSA_S(\omega)$ is the response spectrum generated from the current spectral density function $S_g(\omega)$.

2.1.3. Computation of Ground Properties

As the modulus of elasticity in shear, G, is a strain-dependent function, G can be determined by an iterative procedure. The strain included in the given input range is calculated to determine the stiffness and damping of the ground. This procedure is performed in the frequency domain to use the spectral analysis method.

After computation of the stiffness and damping of the ground, the stiffness and mass matrices ($\underset{\sim}{G}^j$, $\underset{\sim}{M}^j$: ymmetric matrices), can be expressed by the following equations.

$$\underset{\sim}{G}^j = \frac{1}{h_j} \begin{bmatrix} G_j & 0 & -G_j & 0 \\ 0 & \lambda_j + 2G_j & 0 & -(\lambda_j + 2G_j) \\ -G_j & 0 & G_j & 0 \\ 0 & -(\lambda_j + 2G_j) & 0 & \lambda_j + 2G_j \end{bmatrix} ; \underset{\sim}{M}^j = \rho_j h_j \begin{bmatrix} 1/3 & 0 & 1/6 & 0 \\ 0 & 1/3 & 0 & 1/6 \\ 1/6 & 0 & 1/3 & 0 \\ 0 & 1/6 & 0 & 1/6 \end{bmatrix} \tag{2.3}$$

where ρ_i and h_i are specific mass and thickness of layer, respectively.

2.1.4. Computation of Ground Period and Shape

Assembling the matrices of Eq.(2.3) for the region defined in $0 \leq z \leq h$, we obtain:

$$\left[\underset{\sim}{G} - \omega^2 \underset{\sim}{M} \right] \underset{\sim}{\Delta} = \underset{\sim}{F} \tag{2.4}$$

where $\underset{\sim}{G}$ and $\underset{\sim}{M}$ are $(2N+2) \times (2N+2)$ matrices assembled from $\underset{\sim}{G}^j$ and $\underset{\sim}{M}^j$, respectively, and, $\underset{\sim}{\Delta}$ and $\underset{\sim}{F}$ are - vectors:

$$\Delta_{2j-1} = U_j \qquad ; 1 \leq j \leq N+1$$
$$\Delta_{2j} = W_j \qquad ; 1 \leq j \leq N+1$$

$$F_1 = -\tau_1 = -\tau_{xz}\big|_{\substack{x=0 \\ z=0}} \qquad F_2 = -\sigma_1 = -\sigma_z\big|_{\substack{x=0 \\ z=0}}$$

$$F_{2N+1} = \tau_{N+1} = \tau_{xz}\big|_{\substack{x=0 \\ z=h}} \qquad F_{2N+2} = \sigma_{N+1} = \sigma_z\big|_{\substack{x=0 \\ z=h}}$$

$$F_{2j-1} = F_{2j} = 0 \quad ; \; 2 \leq j \leq N$$

Consequently, F_1 and F_{2N+1} are the amplitudes of the shear tractions at the surface and base of the stratum at $\chi=0$, respectively. Similarly, F_2 and F_{2N+2} are the amplitudes of the normal tractions at the surface and base of the stratum at $\chi=0$, respectively.

Finally, the eigenvalue problem is defined by Eqs.(2.4) and (2.5)

$$\underset{\sim}{F}=\underset{\sim}{0} \qquad (2.5)$$

From Eqs.(2.4) and (2.5), we obtain the frequency, $\omega_1=(=1/T_1)$, at the surface of the ground, which gives us the period and, following, the mode shape of ground motion.

2.2. Structural Analysis

The relative displacement between decks is computed by combining the displacement of the bridge with the earthquake motion.

The spectral analysis method, particularly the Multi-mode Spectral Analysis Method, described in the Design Specification Volume 9, is adopted for the analysis of the Taiwan High Speed Railway bridges.

2.2.1. Modeling

The bridge is modeled as a three-dimensional space frame with joints and nodes selected to model realistically the stiffness and inertia effects of the structure. Each joint or node has six degrees-of-freedom, three translational and three rotational. The structural mass is lumped with a minimum of three translational inertia terms.

The discontinuities are introduced in the superstructure at the expansion joints. The earthquake restrainers' effect at the expansion joints is approximated by superimposing one or more linearly elastic members owing the stiffness properties of the engaged restrainer unit.

The infrastructure of the bridge (pier + piles) is modeled as a whole by means of spring elements and the mass is lumped. Foundation conditions at the base of piers are modeled using equivalent linear spring coefficients. The connection between the rails and the superstructure is modeled by nonlinear springs.

2.2.2. Mode Shapes and Periods

The required periods and mode shapes of the bridge in the direction under consideration shall be calculated by established methods for fixed base condition using the mass and elastic stiffness of the whole seismic resisting system.

Mode shapes and frequencies are obtained by solving:

$$\lfloor \underline{k}-\omega^2\underline{m}\rfloor\,\hat{\underline{v}}=\underline{0} \qquad (2.6)$$

where $\underline{k}$ and $\underline{m}$ are the known stiffness and mass matrices of the mathematical model of bridge system, respectively, $\hat{\underline{v}}$ is the displacement amplitude vector, and ω is the frequency. This analysis yields to the dimensionless mode shapes $\underline{\varphi}_1, \underline{\varphi}_2, \Lambda, \underline{\varphi}_n$ and their corresponding circular frequencies $\underline{\omega}_1, \underline{\omega}_2, \Lambda, \underline{\omega}_n$. The mode period can then be obtained using

$$T_i=\frac{2\pi}{\omega_1} \quad (i=1,2,K,n) \qquad (2.7)$$

2.2.3. Multimode Spectral Analysis

The response should, as a minimum, include the effects of a number of modes equivalent to three times the number of spans up to a maximum of 25 modes.

The uncoupled normal mode equations of motion are of the form

$$\ddot{Y}_i(t)+2\omega_i\zeta_i\dot{Y}_i(t)+\omega_i^2Y_i(t)=P_i\frac{(t)}{M_i} \qquad (2.8)$$

where subscriptrefers to the mode number, Y_i, ω_i and ζ_i are the mode amplitude, frequency, and damping ratio, respectively, and the effective modal load $P_i(t)$ and generalized mass M_i are given by

$$\begin{aligned} P_i(t) &= \underline{\phi}_i^T\underline{m}\underline{B}\ddot{v}_g(t) \\ M_i &= \underline{\phi}_i^T\underline{m}\underline{\phi}_i \end{aligned} \qquad (2.9)$$

where $\underline{B}$ is a vector containing ones and zeros corresponding to those components in the direction of excitation and those components in the other orthogonal directions, respectively.

The maximum absolute value of $Y_i(t)$ during the whole seismic excitation time-history is given by

$$Y_i(t)_{\max} \frac{T_i^2 S_a(\zeta_i, T_i)}{4\pi^2} \frac{\underline{\phi}_i^T \underline{m}\,\underline{B}}{\phi_i^T \underline{m}\,\underline{\phi}_1} \tag{2.10}$$

where $S_a(\zeta_i, T_i)$ is the acceleration response spectral value for the prescribed earthquake excitation. According to the Standards, $S_a(\zeta_i, T_i)$ is obtained from:

$$S_a(\zeta_i, T_i) = gC_{sm} \tag{2.11}$$

where C_{sm} is the elastic seismic response coefficient for the mth mode.

To determine the maximum value of any particular response quantity $Z(t)$ e.g., shear, moment, displacement or relative displacement), use is made of the fact that it is linearly related to the normal model amplitude, i.e.,

$$Z(t) = \sum_{i=1}^{n} A_i Y_i(t) \tag{2.12}$$

where coefficient A_i is known reduction function.

2.2.4. Displacements

The displacements can be estimated by combining the respective response quantities (e.g., force, displacement, or relative displacement) from the individual modes by the Square Root of the Sum of the Squares (SRSS) method. Displacements obtained using the SRSS method of combined modes are generally adequate for most bridge systems because they have well-separated modes of vibration characterized by significant differences in the natural periods for each of the modes. For bridges with closely spaced modes (within 10%), other more appropriate methods of combining or weighting individual contributions should be considered to obtain the total final response.

3. Rail-Structure Interaction

3.1. General & Provisions

In the case of long rail, the stress of the rail itself

Table 1. Provision for Rail-Structure Interaction for Taiwan High Speed Railway

	Rail stress	Displacement
Braking, traction and 20℃ temperature variation between rails and deck	-7/+92 N/㎟	7 mm: between decks or deck and abutment, 4 mm: between bridge deck and rail.
Braking, type II earthquake* and 20℃ temperature variation between rails and deck	-147/+167 N/㎟	25 mm: between decks or deck and abutment.

*Service earthquake that is 1/3 of reparable damage earthquake.

is increased in a large amount by the continuity of rail. Additional stress and deformation are generated by variations of the substructure and cause the rail-structure interaction. The relative displacement between decks must be considered for the evaluation of stress in the rails.

As shown in the Design Specification Volume 9, Subsection 3.C.5.0, the provisions to check the rail-structure interaction considering seismic effects are:

3.2. Modeling for rail-structure interaction analysis

Bilinear models are used for the ballasts and fastenings, which connect the rail to the structure as shown in Fig. 1. Loading conditions are described in reference 7.

For rails and decks, frame elements are used. The

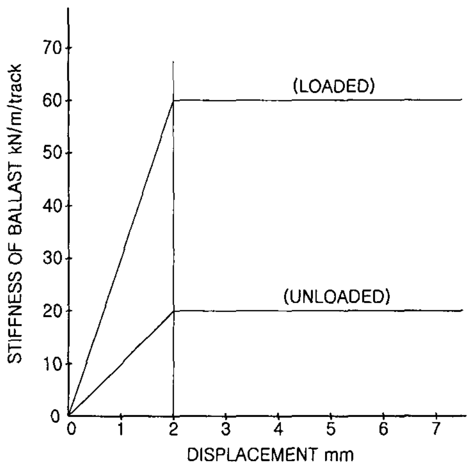


Fig. 1. Element properties (Ballast + Fastenings)

springs connected to decks are nonlinear and spaced to each other by 5.0 *m*.

3.3. Results of Rail-Structure Interaction Analysis

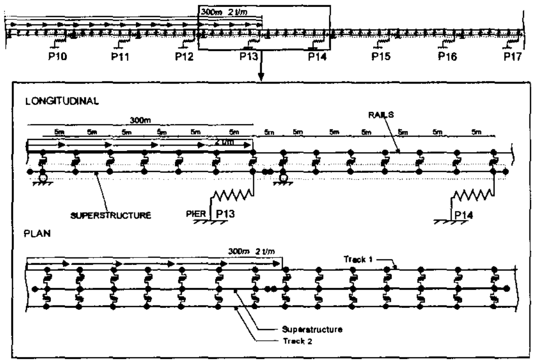
Service earthquake constitutes one of the most important features in loading factors. Therefore, three load combination cases are considered: braking force case, service earthquake load case and simultaneous braking force & service earthquake loaded case.

3.3.1. Braking Force Load Case

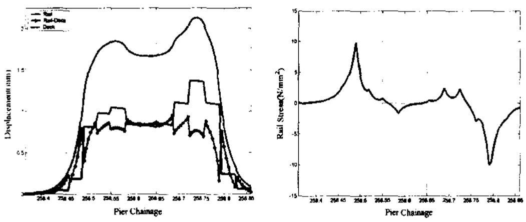
The braking force on a rail applies over a length of 300 m with an intensity of 2 t/m. The maximum relative displacement between rail and deck occurs at the mid-length of the loaded rail. And the stresses present maximum and minimum values at the extremities of the loaded rail.

3.3.2. Service Earthquake Load Case

The rail shall be designed to be safe under service earthquake in strong earthquake region such as Taiwan. Relative stresses of rails are checked once the earthquake direction and phase difference for the maximum displacement between decks are decided.

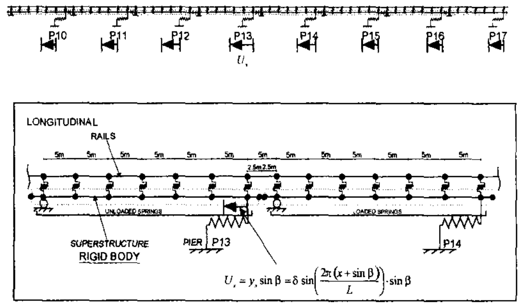


(a) Load status by braking force

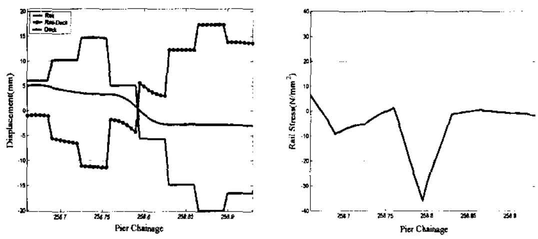


(b) Displacement between rail and deck & Rail stress

Fig. 2. Results of braking force

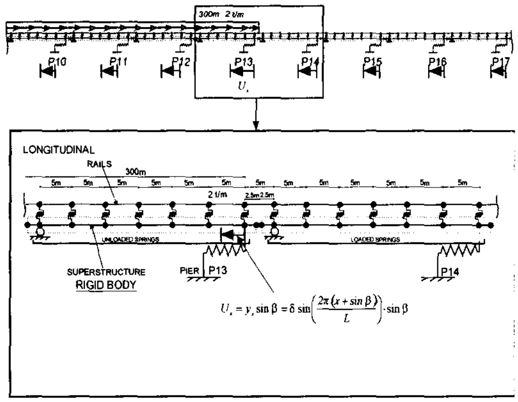


(a) Load status by service earthquake

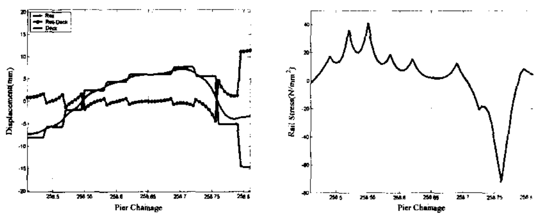


(b) Displacement & Rail stress

Fig. 3. Result of service earthquake



(a) Load Status



(b) Displacement & Rail stress

Fig. 4. Result of braking & service earthquake

3.3.3. Braking Force & Service Earthquake Load Case

The load combination of braking and service earthquake is checked so that the train can be stopped safely even under service earthquake. Due to the nonlinear connecting spring, the stress of rail is larger than the sum of braking and service earthquake case.

4. CONCLUSIONS

Considering the earthquake effects represents a very important factor in high-speed railway bridge design, especially in highly seismic regions like Taiwan. Therefore, the analysis method described here has been developed and applied to design Taiwan high-speed railway bridges.

The capacity for rail-structure interaction including service earthquake has been verified to satisfy the provision of Taiwan High Speed Railway Design Specification. Analysis of the results obtained all over the length of the bridge, that is about 1.0 km, proved that the capacity depends essentially on soil conditions rather than structural conditions.

REFERENCES

1) Taiwan High Speed Rail Corporation, Taiwan High Speed Railway Design Volume 9, 2000.
2) Matlab User's Manual, The Mathworks, 1999년
3) American Association of State Highway and Transportation Officials, AASHTO Standard specifications for highway bridges, 1998.
4) 건설교통부, 도로교 표준 시방서, 1996.
5) Weaver & Johnston, Structural dynamics by finite elements, Prentice Hall, 1987.
6) 권기준 외, 대만 고속전철 교량의 레일-구조물 상호작용 평가, 한국산업안전학회 2000년 추계학술 발표회 논문집, 2001.