Simplified Evaluation Method for Running Safety of Railway Structures in Consideration of Nonlinear Behavior

Munemasa TOKUNAGA       Kenji NARITA       Keiichi GOTO
Structural Mechanics Laboratory, Railway Dynamics Division

In order to efficiently determine the weak points of the running safety in railway sections during large-scale earthquakes or existing structures with low yield seismic intensity, this paper proposed a simple evaluation method for running safety of railway vehicle during large-scale earthquakes based on numerical simulations which quantified the degree of influence of nonlinear behavior of structures on the wheel derailment limit. The method can evaluate the occurrence of derailment only from the structural response such as the acceleration of the structure top and the angular rotation at the structural boundary.

Keywords: seismic running safety, derailment, dynamic interaction analysis, large-scale earthquakes, structural linear response

1. Introduction

In Japan, railway derailments occurred in the 2004 Mid Niigata Prefecture Earthquake, the 2011 off the Pacific coast of Tohoku Earthquake, and the 2015 Kumamoto Earthquake. In response to the frequent occurrence of large-scale earthquakes in recent years, multifaceted countermeasures are being implemented for the entire railway system to prevent derailment during earthquakes, such as Earthquake Early Warnings and countermeasures for vehicles, tracks and civil structures. When implementing seismic measures for existing structures, in addition to improving the yield strength and deformation performance of structures themselves, it is also important to improve seismic running safety, i.e., to accurately find derailment weak points and address these with appropriate countermeasures.

The purpose of this study is to establish a simple evaluation method for seismic running safety that simultaneously considers the non-linear behavior of structures and the differential displacement (Fig. 1) at structure boundaries [1], which can be applied up to large-scale earthquakes. In this study, a series of dynamic interaction analyses between vehicles and structures were carried out.

2. Method

For the analyses, we used DIASTARS III [2, 3], a dynamic interaction analysis program for vehicles and structures, which can analyze vehicle behavior before and after derailment.

2.1 Vehicle modelling

The railway vehicle was modelled as a multibody system consisting of a body, two bogie frames and four wheelsets with springs and dampers. In this model, the vehicle had 31 degrees of freedom. Actual vehicles are equipped with stoppers to prevent excessive relative displacements at each connection. To take these stoppers into account, bi-linear springs were used in the model. The adequacy of these vehicle models was verified through vibration experiments with a vibration table and a full-scale vehicle model [4]. The vehicle specifications were set to those of a recently designed high-speed Shinkansen railway vehicle.

The dynamic interaction forces were calculated based on the vertical and horizontal relative displacement between the wheels and rails. The dynamic interaction force in the vertical direction was modelled as a Hertz contact force and that in the horizontal direction was modelled as creep force and flange pressure. The Hertz contact force is a function of the relative vertical displacement $\delta_z$. Creep force is a horizontal force generated by the creep of the wheel moving forward as it rolls along the rail. This creep force saturates at the upper limit of the friction force when the slipping ratio $S$ in the horizontal direction is large.

Flange pressure is caused by the contact between the wheel flange and the rail. It is calculated from the rail-tilting spring and the relative horizontal displacement between the wheel flange and the rail, $\delta_y$. The contact point and the contact angle were calculated based on contact functions derived from the geometric configuration of the system. The derailment threshold was set to $\pm 70$ mm based on the relative horizontal movement of the wheels and rails [5].

2.2 Bridge modelling

Figure 2 shows a generalized model of bridges used to evaluate the derailment limits for vibrational displacement and differential displacement. As shown in the figure, the
effect of vibration displacement was evaluated by inputting acceleration such as seismic motion into a bridge with one degree of freedom in the transverse direction. Tracks were modelled to be straight and rigid. The effect of differential displacement was evaluated using a model in which the vehicle runs on a rigid track with rail irregularities in the transverse direction. The track shape was assumed to be horizontal translation and folding, considering general structural configuration in Japanese high speed railway line. In order to ignore the interaction between the vehicle and the structure, the amount of structural material is sufficiently large, about 50 times the mass of one train (unit mass of structure 100 t/m, unit mass of vehicle 1.8 t/m). The structure was modeled with a trilinear skeleton curve and a SDOF system with standard hysteresis characteristics. The skeleton curve was set based on the yield frequency $f_{eq}$ (the reciprocal of the equivalent natural period $T_{eq}$), the yield seismic intensity $k_{y}$, and the maximum seismic intensity $k_{hmax}$ as parameters. The modal damping ratio of the structure was 5%. A seismic wave or a sine wave was applied as an external force to the acceleration input mass point at the lower end of the structure.

Figure 3 shows the shape of the differential displacement (angular rotation) and the buffer section model at the structural boundary. In this study, the shape of the folding, parallel movement and the single angular rotation, which is the general shape of differential displacement in railway viaducts, were considered. The buffer section was provided before and after the structural boundary to smooth the discontinuity of curvature.

Figure 4 shows the setting of the input acceleration and the phase shift of the angular rotation. As shown in the figure, sinusoidal waves with front and rear buffer waves were used as the input acceleration. The angular rotation was set in the almost end of the sinusoidal wave where the response of the vehicle was amplified by acceleration inputs.

Figure 5 shows the dynamic model of the line section structures used for the verification of the proposed method. The pier was modeled using a single non-linear spring, and the rigid frame viaduct was modeled using two non-linear springs placed at both ends in the transverse direction to the line. The unit length mass of the rigid beam element representing the superstructure was set to 36 t/m assuming the actual structural parameter, and the weight ratio of the structure / vehicle was set to be about 20. Damping was given by the mode damping ratio of 5% for each mode of the structure.

2.3 Analysis cases

Table 1 shows a list of the cases used for analysis.

There were three types of rolling stock: A had the specifications used in the displacement limit standard [3], and B and C had the specifications of recently designed Shinkansen rolling stock. The length of these vehicles was 25 m, the bogie center spacing was 15 m, and the axle spacing was 2.5 m. The mass per vehicle including the bogie frame and wheel set was 46 t for A and B, and 49 t for C.

As seismic waves, 6 design seismic wave [6] and 8 observed seismic waves [7] were used. The observed seismic motions are waves with characteristic frequency components while the seismic waves were the following waves which have actually caused derailments in the past: the
Table 1 Analysis cases

| Vibration displacement | Differential displacement | Vibration displacement + Differential |
|------------------------|---------------------------|--------------------------------------|
| 3.1                    | 3.2                       | 3.3                                  |
| Type                   |                           |                                      |
| A, B, C                | A                         |                                      |
| f (km/h)               | 260                       | 20~540                               |
| number of car          | 1                         | 1 car                                |
| d (m)                  | linear, 0.3, 0.7          | 0.3, 0.5, 0.7                        |
| f_{eq} (Hz)            | 0.1~3.0 (30 cases)        | 0.5, 1.0, 1.5, 2.0                   |
| mass ratio α           | about 50                   | about 50                             |
| Shape                  | Straight                   | Single angle                         |
| angle amount           | -                         | -                                    |
| Waveform               | Sinusoidal wave           | Sinusoidal wave                       |
| Amplitude              | Sinusoidal wave            | Sinusoidal wave                       |
| Predominate frequency  | Predominate frequency      | Predominate frequency                 |
| of structure           | of structure               | of structure                          |
| fs                      | 0.5~6400 gal (60 cases)   | 0~2000 gal (21 cases)                |
| num. of case           | 75600 cases                | 119070 cases                         |

3. Derailment limit by generalized model

3.1 Derailment limit of vibration displacement

Figure 6 shows the relationship between derailment limits of the structural response in the case of type-A vehicle. As the structural response the acceleration $PSA$, velocity $PSV$, displacement $PSD$, and $SI$ value of the structure was focused when vehicle derailment occurs. For $PSA$ and $PSD$, the relative values against the input acceleration points were evaluated. The X-axis of the figure is the predominant frequency $f_{s}$ of the structure, which was calculated as the predominant frequency in the frequency domain calculated by Fourier transform of the structural response acceleration.

In Fig. 6 (a), the lower limit of the 90% confidence range of PSA decreases in the region where $f_{s}$ is greater than 2.5, indicating that the variation is large. In the region where $f_{s}$ is less than 0.5, the PSA limit value converges to about 7 to 8 m/s². In addition, in Fig. 6 (c), there are cases where the $PSD$ value is significantly small in the range of $0.5 < f_{s} < 1.0$ Hz. This is the case where the frequency band around the predominant frequency of seismic motion is directly transmitted into the top of the structure because the yield frequency $f_{y}$ of the structure is sufficiently high. Focusing on the limit value of the displacement limit standard shown in Fig. 6 (d), when the yield seismic coefficient $k_{y}$ is 0.3, lots of analysis results where the limit value is less than the displacement limit standard can be seen, which indicates that the assessment using $SI$ value has a possibility to provide dangerous evaluations when applied to structures with remarkable non-linear responses.

Tokachi-oki earthquake (Shinkancho observation, NS direction), the Tokachi-oki earthquake (Hiroo observation, NS direction), and the Kushiro-oki earthquake, (Kushiro City observation, NS direction), Kushiro-oki earthquake (Kushiro city observation, EW direction), Hyogoken Nambu earthquake, Niigata Chuetsu earthquake, Tohoku region Pacific Ocean offshore earthquake (Aoba-ku observation, NS direction), Kumamoto earthquake (Kasuga-cho, Nishi-ku), NS direction).

Figure 7 shows the coefficient of variation of the limit value for each structural response when the derailment occurs. The coefficient of variation was calculated based on the mean and standard deviation of all analysis cases. From the figure, the coefficient of variation of $PSD$ and $PSV$ is about 50% in the entire frequency domain, and the variation is larger than $PSA$ or $SI$. $SI$ has the smallest coefficient of variation of about 20% among all four indicators in the region of $f_{s} > 1.6$ Hz. $PSA$ provides a coefficient of variation of about 5 to 10% in the region of $f_{s} < 0.5$ Hz, and a coefficient of variation of about 10 to 20%, which is equal to or less than the $SI$ in the region of $0.5 < f_{s} < 1.6$ Hz, although it provides the coefficient is about 30 to 50% when $f_{s} > 1.6$ Hz. Considering these results of non-linear behavior of the structure, $PSA$ seems to have the smallest variation in the general structure range of about $0.5 < f_{s} < 1.6$ Hz. The same tendency as that of vehicle type-A was obtained for vehicle types-B and C, which is therefore not described here.
3.2 Derailment limit of differential displacement

Figure 8 shows the derailment limit for differential displacement (angular rotation). From the figure, it can be seen that $\theta_{\text{lim}}$ decreases with the increase of the train speed with an inversely proportional relationship, and that $\theta_{\text{lim}}$ varies about 10 mrad due to the influence of vehicle type.

3.3 Derailment limit when vibration displacement and differential displacement simultaneously applied

During actual seismic motion, within a few seconds of the seismic amplitude increasing, a train may pass through different structures which have different vibration characteristics. As the relationship between the excitation period and input phase which trains experience, is not fully understood, it cannot be easily expressed with a general model. In this study, as shown in Table 1, the derailment limit was evaluated when the vibration displacement and the differential displacement were simultaneously applied by introducing sinusoidal input and angular rotations with increasing amplitudes for each. The excitation frequency of the sine sinusoidal input, the phase shift of the input acceleration and angular rotation and the train speed were used as parameters as shown in Fig. 4.

Figure 9 shows the derailment limit when the vibration displacement and the differential displacement (in the case of single angle rotation) are simultaneously applied. The Y value $\Lambda = a_{\text{lim}}/a_{\text{eq}}$ is the limit dimensionless value of $a_{\text{lim}}$ by $a_{\text{eq}}$ when only the vibration displacement was considered. The X value ($\Theta = \theta_{\text{lim}}/\theta_{\text{eq}}$) is also the limit dimensionless value of $\theta_{\text{lim}}$ by $\theta_{\text{eq}}$ when only the unequal displacement is applied. The figure indicates that the derailment limit tends to be constant against the train speed, and the influence of the train speed can be taken into account by making the $\theta_{\text{lim}}$ dimensionless by $\theta_{\text{eq}}$. The correlation equation of $\Lambda^{0.7} + \Theta^{1.8} = 1$ was determined by trial and error to ensure it remained generally below the lower limit, using the mode value as reference. As shown in the figure, many derailments occur when $(\Lambda, \Theta)$ exceeds the values obtained by the correlation equation, and the excess probability is 89.6% for all analysis cases in the case of a single angular rotation.

4. Evaluation method for seismic running safety

4.1 Proposal of 4. Evaluation method

In this study, $RSI$ (Running Safety Index) that simultaneously considers the vibration displacement and differential displacement was proposed for evaluating the seismic running safety as shown in (1). When $RSI$ is greater than 1, there is a higher possibility that the structure will cause a derailment due to its seismic response. On the other hand, when $RSI$ is less than 1, it is judged to be safe.

$$RSI = \left(\frac{a}{a_{\text{lim}}}\right)^{0.7} + \left(\frac{\theta}{\theta_{\text{lim}}}\right)^{1.8}$$

(1)

Where, $a$ is the response value of the acceleration at the top of the structure, $a_{\text{lim}}$ is the acceleration limit which is defined by $a_{\text{lim}}^{\text{eq}}$ (m/s²) shown in (2) (Fig. 6 (a)). $\theta$ is the response value of the angular rotation at the structure boundary, and $\theta_{\text{lim}}$ is the limit angular rotation which is defined by $\theta_{\text{lim}}^{\text{eq}}$ (mm) described in the following equation.

$$a_{\text{lim}}^{\text{eq}} = \begin{cases} 7.22 & \left(0.1 < T_{eq} \leq 0.4 \right) \\ 8.56 - 3.33T_{eq} & \left(0.4 < T_{eq} \leq 1.5 \right) \\ 3.56 & \left(1.5 < T_{eq} \leq 2.0 \right) \\ 1.48 - 1.04T_{eq} & \left(2.0 < T_{eq} \leq 5.0 \right) \end{cases}$$

(2)

Where, $T_{eq}$ is the predominant period (s) during the non-linear response of the structure, which can be also calculated as $T_{eq} = 2\pi \sqrt{PSD/PS4}$ (s) as a simple method. This simple method is established assuming the sinusoidal response at the moment when the maximum displacement and acceleration occurs.

Figure 10 shows a comparison of the assessment value between the conventional SI and proposed $\alpha$.
confirmed that the assessment values of both are almost the same, which indicates that the proposed equation can provide equivalent assessment results to the conventional $SI$. In addition, the proposed $\alpha_{\text{lim}}$ has the advantage of enabling direct assessment of the vibration displacement from the elastic acceleration response spectrum of the seismic motion without complicated numerical integration as required for the $SI$ value, even within the structural linear response.

Figure 8 shows the proposed limit angular rotation $\theta_{\text{lim}}^{\text{pro}}$ and $\theta_{\text{lim}}^{\text{prec}}$. $\theta_{\text{lim}}^{\text{pro}}$ is the limit value for a single angular rotation, and is an equation that is inversely proportional to the train speed $V$ so it is lower than the analysis results. In this figure, the applicable range of $V$ is from 120 km/h to 500 km/h. $\theta_{\text{lim}}^{\text{prec}}$ is the limit value for multiple angular rotations. The derailment limit for multiple angular rotations $\theta_{\text{lim}}^{\text{prec}}$ is divided by $\theta_{\text{lim}}^{\text{pro}}$ with the maximum value of 1.6 obtained by theoretical solution assuming parallel movements assumed in rigid frame viaducts.

4.2 Validity of proposed evaluation method

Figure 11 shows the structure specifications of about 5 km of the actual Shinkansen section targeted in this study: the structure height, yield seismic intensity, and equivalent natural period the structure. The target line section is composed of rigid frame viaducts, girder type viaducts, and overhead bridges with rigid frame piers. The nonlinearity of the structure was expressed as a standard trilinear nonlinear horizontal spring based on the skeleton curve calculated by pushover analysis in advance. From the figure, their height varies between about 5 to 15 m and the ground type is G2-5. The type-C vehicle was used. Seismic motion was applied and the train speed was 320 km/h.

The lowest figure describes the comparison of the seismic input amplitude between the derailment limit of analysis results and the proposed method shown in equation (3). The figure indicates that the derailment locations and its amplitudes by the proposed method and the running analysis are almost consistent. The structure contribution in the figure is the sum of the contribution of vibration displacement $(\alpha/\alpha_{\text{lim}})^{0.7}$ and the contribution of differential displacement $(\theta/\theta_{\text{lim}})^{1.8}$ in (3). Derailment occurred at around 1.21 km due to the predominance of differential displacement, where differential displacement accounts for about 60 to 70% of the total contribution. This tendency is remarkable in each overpass located about 2.7 to 4.0 km, and the ratio of differential displacement reaches about 70 to 80% of the total contribution.

The proposed method enables us to quickly evaluate the seismic running safety of a long line section during the earthquake with the same accuracy as precise analysis, quantitatively evaluate each component due to the vibration displacement and the differential displacement and select effective countermeasures.

5. Conclusions

In this study, the conclusions obtained based on the nonlinear analysis considering the dynamic interaction between the vehicle / structure are summarized below:

1) With respect to the vibration displacement of the structure, the maximum acceleration $PSA$ at the top of the structure during an earthquake has a small variation in the limit value at which derailment occurs even when the structure is nonlinearized. A derailment limit acceleration $PSA$ which is almost the lower limit of a comprehensive analysis of vehicle types, structural
2) With respect to the differential displacement that occurs on the track surface during earthquakes, a limit value that depends on the train speed taking into account the effect of multiple angular rotations was defined.

3) The coupling effects of the vibration displacement and the differential displacement were evaluated based on parametric analyses that vary using the phase of the sinusoidal waveform and angular rotations, to propose a derailment limit curve considering the couplings.

4) A simple method, which evaluate vehicle running safety during large-scale earthquakes only by the nonlinear structural response: the track acceleration in the transverse direction and the differential displacement at the structure boundary, was proposed. The proposed method was verified by comparing results obtained for derailment locations and amplitudes for long sections of Shinkansen line, through detailed simulation. The proposed method enables us to extract seismically vulnerable locations and assess the contributions to derailment of vibration displacement and differential displacement. This in turn makes it possible to select and prioritize derailment countermeasures.

References

[1] Railway Technical Research Institute, (2007), Design Standards for Railway Structures and Commentary (Displacement Limits), Maruzen co.,Ltd, Tokyo (in Japanese).

[2] Wakui, H., Matsumoto, N., and Tanabe, M., “A Study on Dynamic Interaction Analysis for Railway Vehicles and Structures - Mechanical model and practical analysis method -,” Quarterly Report of RTRI, Vol.35, No.2, pp. 96-104, 1994.

[3] Tanabe, M., Goto, K., Watanabe, T., Sogabe, M., Wakui, H., & Tanabe, Y.: “A simple and efficient numerical model for dynamic interaction of high speed train and railway structure including derailment during an earthquake,” Procedia engineering, pp. 199, 2729-2734, 2017.

[4] Nobuyuki MATSUMOTO, Makoto TANABE, Hajime WAKUI, Masamichi SOGABE, A DYNAMIC INTERACTION ANALYSIS MODEL FOR RAILWAY VEHICLES AND STRUCTURES WHICH TAKES INTO ACCOUNT NON-LINEAR RESPONSE, Doboku Gakkai Ronbunshuu A, 2007, Volume 63, Issue 3, Pages 533-551, 2007 (in Japanese).

[5] Miyamoto, T., Matsumoto, N., Sogabe, M., Shimomura, T., Nishiyama, Y. and Matsuo, M.: “Full-scale Experiment on Dynamic Behavior of Railway Vehicle against Heavy Track Vibration,” Journal of Environment and Engineering, Vol.2, No.2, pp. 419-428, 2007.

[6] Railway Technical Research Institute, (2012), Design Standards for Railway Structures and Commentary (Seismic Design), Maruzen co.,Ltd, Tokyo (in Japanese).

[7] Earthquake and Seismic Intensity Information, https://www.data.jma.go.jp/svd/eqev/data/kyoshin/jishin/index.html, 2018.09.12 access.

Authors

Munemasa TOKUNAGA, Dr. Eng.
Assistant Senior Researcher, Structural Mechanics Laboratory, Railway Dynamics Division
Research Areas: Bridge dynamics, Seismic design, Dynamic Interaction between Vehicle and Structure

Kenji NARITA
Researcher, Structural Mechanics Laboratory, Railway Dynamics Division
Research Areas: Bridge Dynamics, Seismic Design, Dynamic Interaction between Vehicle and Structure

Keiichi GOTO, Dr. Eng.
Senior Researcher, Structural Mechanics Laboratory, Railway Dynamics Division
Research Areas: Bridge Dynamics, Seismic Design, Dynamic Interaction between Vehicle and Structure