RESPONSE OF A HIGH-SPEED TRAIN BRIDGE UNDER THE EXTREME EVENT OF A PASSING TRAIN AND AN EARTHQUAKE

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Summary

In recent years, high-speed rail links have become a competitive alternative to air travel for distances up to 800 km. As a consequence, intensive construction programmes for high-speed railway links started in Europe and Asia. Due to the demanding requirements of the railway alignments long stretches of bridges and viaducts had to be built (Taiwan) or are in the planning stage (Portugal). It is not uncommon that in some countries a third of the length of such lines consists of bridges and tunnels.

The possibility of the occurrence of an earthquake has been considered in seismic areas, leading to the installation of automatic systems that stop all trains in the wake of an earthquake. These systems are clearly effective to prevent accidents after the seismic event, caused by damage to the rail line. However, given that a seismic event typically lasts less than 30 seconds and comes without warning, the occurrence of an earthquake while a train is travelling at high-speed over a bridge cannot be prevented.

It is the objective of this paper to address this issue and present the results of the following investigations into such an extreme event:

- Moving load analysis of a three span steel bridge with spans of 120m/140m/120m.
- Time History Analysis of a real earthquake.
- Combination and superposition of the results.
- Discussion of the results and possible design criteria.

Keywords: High-speed Rail, steel truss bridge, dynamic analysis, Earthquake, Time History, three span bridge

1. Introduction

Currently many new high-speed Railway lines are designed and are under construction all over the world. Additional design procedures are essential in cases where these railways lines are planned in seismic areas in order to minimise potential future damage due to earthquake.

This paper reports on a three-span high-speed railway bridge in Taiwan for which a dynamic analysis was performed. Usually the design against earthquake forces and the dynamic analysis of moving vehicles are carried out in two separate design steps.

For the earthquake design often the response spectrum method is used while for the dynamic analyses of passing trains time-history approaches are necessary – either in the modal or the time domain [12]. In order to simulate the effects of a train passing over the bridge during the occurrence of an earthquake a time history analysis for both these events was performed and the results superimposed. This paper reports on the results of this investigation.

2. Investigated Bridge

The Taiwan High-Speed Rail (THSR) will connect Taipeh, T´aichung, T´ainan and Kao-hsiung with a total length of over 350 km with 242 km on bridges (Fig. 2).

The bridge analysed for this investigation is situated at T´aichung in great proximity to a known earthquake fault line (Fig. 2). It has been designed as a large steel-truss with a steel-concrete composite deck with three spans. The structural systems consists of a continuous steel truss with a maximum span length of 150m and a truss height of nearly 18m, as can be seen in Figs. 3 to 5. **Fehler! Verweisquelle konnte nicht gefunden werden.** lists some properties of this bridge. The bridge is situated close to the T´aichung railway station (Fig. 1) and is one of three steel bridges crossing the river and the highway in close vicinity to each other. Fig. 3 shows the steel truss as it was launched during construction.

The last severe earthquake in the T´aichung area occurred on September 21, 1999. The people of Taiwan refer to it as 921. The epicentre was located in the central mountain region of the island. Overall there were over 2000 people killed, over 8000 injured and over 400,000 people left homeless.

Table 1. Selected properties of the investigated bridge.

#	Material	Spans [m]	Type	Lanes	Self weight [t]	Dead load [t]	Maximum Speed [km/h]	Damping [%]
1	Steel and composite	150/120/140	Steel truss with composite deck	2	10.000	6.000	350	1.27

Fig.1 T´aichung station and adjacent steel truss bridges.

Fig. 2 Earthquake Faults & HSR in Taiwan.

Fig. 3 Three span steel truss under construction

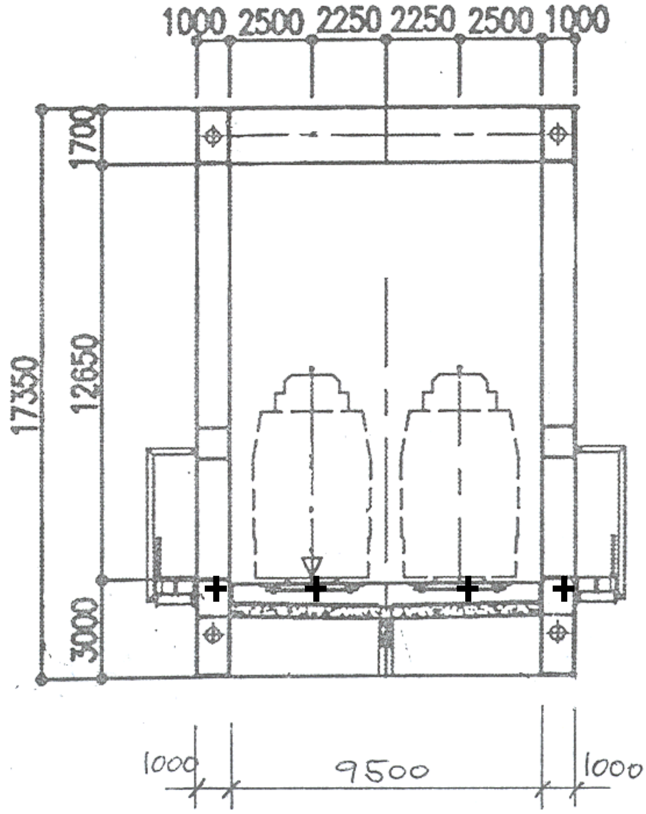


Fig.4 Cross section steel truss bridge #1

Fig.5 Portal frame under construction

3. Dynamic Analyses

All dynamic computer analyses for this project were carried out with the software system *RM2004* [9] including the features described in [6,7]. Functions for modal analysis as well as a time integration scheme in the time domain (modified Newmark, [12] are implemented in this software system and have been used for this project. The investigation focused on four governing pieces of information for the given bridge: (1) Maximum vertical acceleration of any point on the bridge; (2) Maximum variation of cant along the rail tracks over the bridge; (3) Maximum end rotation at the supports. (4) Horizontal displacements of the bridge.

The three-span steel truss bridge was modelled with 3D beam elements. The deck consisted of steel-concrete composite elements. In the transverse direction the deck was discretisised with four elements in order to enable the model to account for vibrations of the deck slab. The axes of the bottom chords and the exact position of the track axes were modelled to a high degree of detail in order to simulate the load introduction of the moving system into the structure properly. Reference [11] gives some guidance on how to take the effect of the track system into account by using 3D train and bridge models.

The loading model used in the present dynamic analyses is based on the data in appendix F of the THSR design specification Volume 9 [8]. These specifications give axle-loads and axle-lay-outs for one specific Taiwanese norm-train. The effects of the moving mass does not have to be considered in the numerical analysis since allowances for this effect are already made in the loading specifications. The bridge girder has twin tracks but only one track has to be loaded since the design specifications state that only one train at a time needs to be considered.

An eigenmode analysis was carried out to gain an understanding of the natural modes and their frequencies of the three span truss bridge system using the RM2004 program system [9]. As can be seen from Figs. 6 and 7 the first vertical eigenmode occurs at 1.05Hz and the second vertical eigenmode at 1.29Hz.

Subsequently, a detailed time-history analysis was performed in the time domain taking into account a wide range of possible train velocities in order to evaluated the effects of passing trains on this bridge. The computed values for the maximum acceleration, the variation of cant and end rotation together with the allowable values for these results are shown in Table 2. The calculation was started with the train located at the beginning of the southern (left in Figs.6 and 7) span (node 200). The train model was then moved along the bridge at a constant velocity (v) along the track towards and past the northern (right) end of the structure (node 337). The response of the bridge was monitored during this procedure and data was recorded as time series for the governing result points.

Figs. 8 to 10 show the results of the moving load analyses at mid span of each span respectively (nodes 226, 271 and 320, Fig. 11). Due to the much higher stiffness of the bottom chords compared to the more flexible composite deck the computed accelerations of the bottom chord remained moderate while the acceleration values in the composite deck immediately below the tracks were found to be much higher. However, none of the computed acceleration results exceeded the given limit of 3.5m/s².

The well-known time history of the ground-motions induced by the Fernando earthquake (Fig 12) was used to investigate the vertical accelerations of the given bridge due to an earthquake. These displacements were applied in a time-delayed sequence at the foundations of the piers of the bridge. The response of the bridge was monitored and the results were recorded at the critical points already identified during the previous analyses (Fig 13).

Table 2. Results for the investigated bridges.

#	maximum acceleration [m/s²]	allowable acceleration [m/s²]	maximum end rotation [rad]	allowable end rotation [rad]	maximum variation in cant [mm/m]	allowable variation in cant [mm/m]
1	**2.24**	**3.5**	**0,121**	**0,285**	**0,1115**	**0,4**

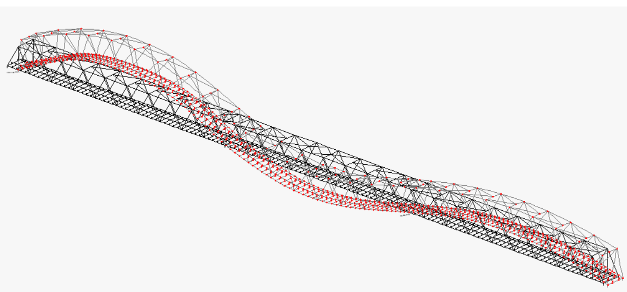
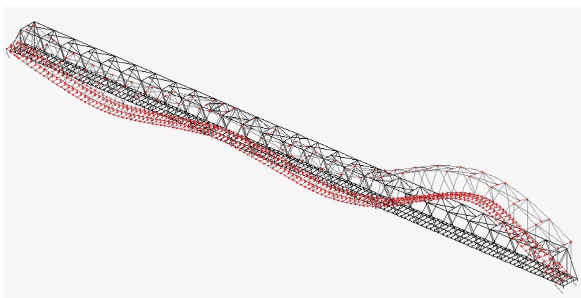


*Fig. 6 Eigenmode 7 – predominantly vertical: **1.05** [Hz].*



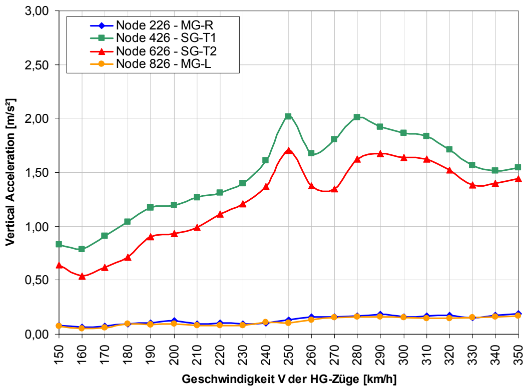


Fig. 8 Accelerations (moving load) - node 226.

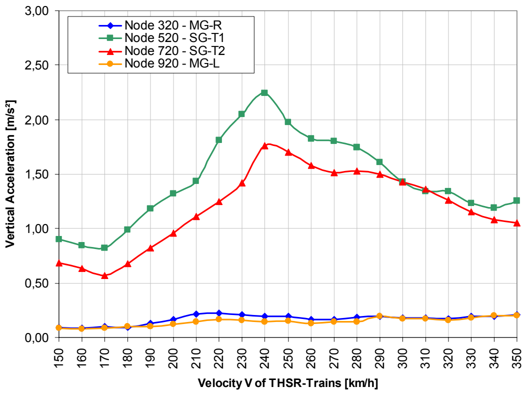


Fig. 9 Accelerations (moving load)- node 271.

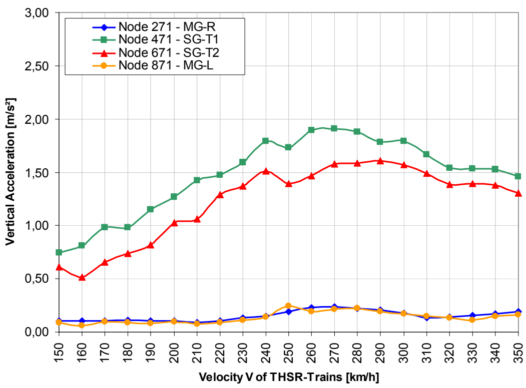


Fig. 10 Accelerations (moving load)- node 320.

Fig. 7 Eigenmode 10 – 1.29 Hz.

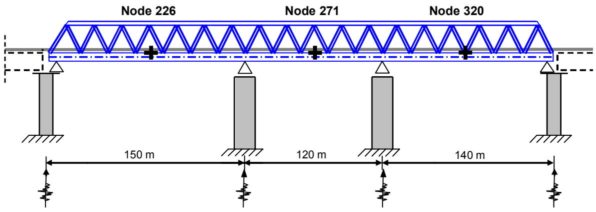


Fig.11 Investigated Nodes on the structure.

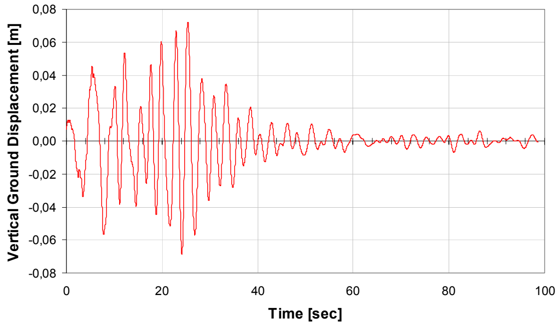


Fig.12 Vertical ground displacement of Fernando earthquake

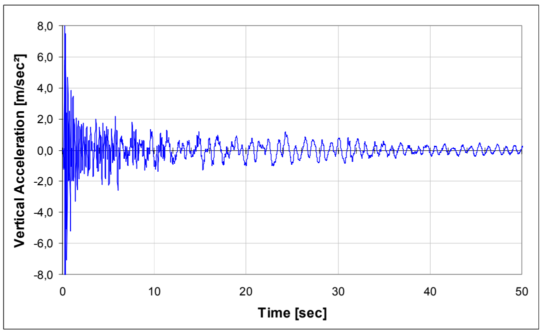


Fig.13Accelerations (ground motion) – node 226.

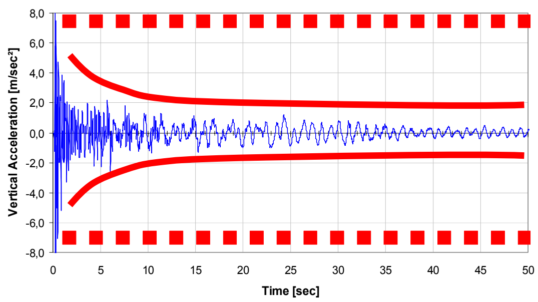


Fig.14 Superposition of Moving Train and Fernando Earthquake – Vertical Accelerations.

A large proportion of the given railway line is situated on bridges. The probability of a train passing over a railway bridge in the event of an earthquake is therefore relatively high. In [1] an automatic monitoring system is presented to reduce the risk of derailments of moving trains due to seismic events. The main purpose of this monitoring is to limit or even stop the moving train when a severe earthquake occurs. Further it is mentioned in [1] that under vertical earthquake, the maximum vertical acceleration shall not be higher than 7m/s². Detailed factors influencing the permitted deck accelerations are described in [2]. The check of the vertical acceleration according [1 & 8] contains a safety factor of 2. In case of emergency this safety factor declines to one.

In the case of a linear analysis superposition of different computation results is allowable. For the above described three-span steel truss bridge an attempt was undertaken to check the possible order of magnitude. Therefore the results from the linear earthquake time history analysis were superposed with the maximum results of the moving load analysis. The maximum vertical acceleration of 2.24m/s² was observed at a speed of 240km/h and occurred at node 520, mid span of the third span below the track. Discounting the initial peak in acceleration which is due to numerical reasons, the combined results stay below the limit of 7m/s² (Fig.14). This results indicates, that a moving train remains safe on the bridge when the ground motions of this Fernando earthquake are considered.

4. Conclusions

The possibility of a simultaneous occurrence of an earthquake with a train moving over a bridge is small for most train lines. However, the risk increases with land becoming rarer and train lines including an increasing percenteage of bridge structures. One method of analysis was shown to check in a simple way the risk of derailment. As en example one three span steel truss bridge was shown. A moving load analysis as well as an earthquake time history analysis were performed and the results were superposed. For the presented example it can be assumed, that an earthquake during a moving load passage of a HSR train is not a major hazard.

5. Acknowledgements

The authors gratefully acknowledge that the work for this paper was funded by the F.F.F. on behalf of the Austrian Government as part of the research project FEMBRIDGE.

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