Discussion Issues Regarding Three-Dimensional Modeling for Analysis of Bridges

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Abstract Computational modelling for bridge analysis has evolved toward greater sophistication with complex three-dimensional FEM models. Recent software products allow modelling to be completed relatively quickly and easily. However, there are a number of issues that are not precisely clarified, including at the normative level in the Eurocodes. Such an issue is taking into consideration the soil-structure interaction in the horizontal and vertical directions under various types of actions, including the influence on the acceleration response spectra; determining and controlling the stiffness of railway bridges in the transverse direction; the interaction between the railway track and the bridge structure and the non-linear behaviour of the connection in the horizontal and vertical directions; horizontal reactions in railway bridge bearings from seismic actions, etc.
This paper focuses on some aspects of the raised issues, regarding FEM modelling of bridges. Furthermore, recommendations for improving of accuracy of computational models are given.

1 Introduction

The implementation of three-dimensional models for analysis is a common practice in bridge design. Sometimes models are not required to significantly higher accuracy but to easily obtain the relevant stresses required for the design. Usually, models are based on beam (frame) finite elements in order to apply the resulting forces directly, without the additional integration necessary for plane (shell) finite elements. However, using beam-based FEM models of plane structural elements of the bridge presents a major disadvantage. That is why designers prefer to apply FEM models that correspond to the nature of the respective structural elements of a bridge.
The use of three-dimensional models applying both plane (shell) and beam (frame) finite elements leads to two groups of features that should be considered in bridge design:

- Contradictions between the results and the design codes;
- Specific effects that cannot be predicted compared to two-dimensional models.

Usually, the calculation procedures provided in the design codes /including the Eurocodes/ are adapted for "by hand" calculation. For example, in EN 1992 the bending moment capacity design procedure for ultimate limit states (uses as a basis) assumes a bi-linear stress-strain diagram in the compressive zone. This is appropriate for a manual approach, however, it is complicated for implementation in a programming algorithm. Further examples can be pointed out. This paper focuses on some of these examples regarding analysis models. There are also cases where the theoretical model underlying the design codes themselves is based on assumptions that do not fully correspond to current understandings and come into conflict with three-dimensional modelling seeking to more accurately represent a structure and/or an action more accurately.
In the past, as a rule bridges were modelled through linear elements and the effects of the spatial behaviours of bridges were obtained with various additional procedures. For example, in girder bridges, the unbalanced impact of traffic in the transverse direction was estimated through solving additional transverse models. This approach is now irrelevant, since the distribution of forces in the transverse direction is directly estimated through three-dimensional models. Moreover, the three-dimensional models obtain additional effects whose interpretation presents some difficulties a number of which are discussed in this paper.

2 Modelling of the soil-structure interaction

Computer modelling aims to describe structures as accurately as possible and to include all structural elements in order to obtain the corresponding internal stresses required for the design of elements. Although in the earlier stages of the development of computational techniques simplification of the models used was sought due to technical limitations, nowadays structures are modelled in their entirety using the latest software, including foundations as well as their interaction with the ground. Both in the Eurocode system and in other documents, guidance for modelling of this interaction through the implementation of springs or suitable impedances is specified. Hence in EN 1998-2 [1] there is a special clause, 4.1.4, which points out some guidelines with additional references to EN 1998-5 [2] and is applicable to shallow and deep /pile or well/ foundations. (Figure 1).

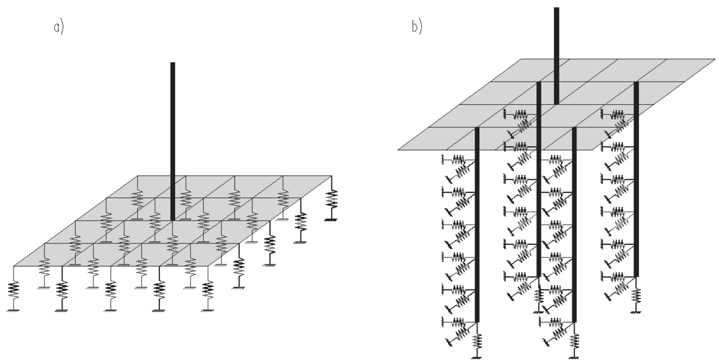


Figure 1: Spring supports for shallow (a) and pile (b) foundation

However, the main problem designers face is how to determine the values of these springs. The cited guidance is very general. Accordingly, if the parameters of the soil are not precisely determined, the design is required to contain upper and lower probabilistic values of the deformation characteristics of soil. An exact determination is nearly impossible and even if there are "more precisely" determined values for a given foundation or soil layer, these can vary widely in terms of bridge length and depth. Hence, at least two property values of the soil should always be implemented. Moreover, with regard to the design of different types of actions (long-term actions and short-term actions), the number of values that should be considered increases. Seismic action and braking forces are impulse actions and soil is an inert medium that responds relatively slowly. Accordingly, the deformation characteristics of soil that are determined for long-term actions cannot be accepted as reliable for impulse actions.

Taking into consideration the soil-structure interaction is vital for horizontal actions such as braking forces, wind /for reinforced concrete bridges this is negligible / and seismic actions. However, the interaction for vertical effects is not insignificant for continuous superstructures, arched bridges and indeterminate structures. Under long-term actions, the determination of the interaction of the abutment and the embankment is important, especially regarding the temperature actions in integral bridges. This statement is confirmed by research carried out on the basis of the results of in-situ tests for a road overpass before it has been put into operation [3]. Regarding the impact of moving live loads, it has been established that if the interaction with the soil is not taken into account in the model and a rigid support is adopted, there is almost no difference between model results and those measured in situ.

Therefore, in soil-structure interaction modelling, if there is no available data on the deformab ility parameters of the soil for impulse actions such as earthquake and braking force, overestimated values of the determined characteristics in comparisons with those for static actions should be implemented. In the scientific literature, an incensement within four to eight times is proposed. According to EN 1998-2, the calculations should be carried out with values in a suitable range in order to obtain conservative results. Normally, the spectral multimodal method is applied for seismic design. This is a pseudo-dynamic approach based on so-called "response spectra" mainly in terms of acceleration which describes a relationship between the natural periods of the different modal shapes of the structure with the accelerations of the masses. (Figure 2).

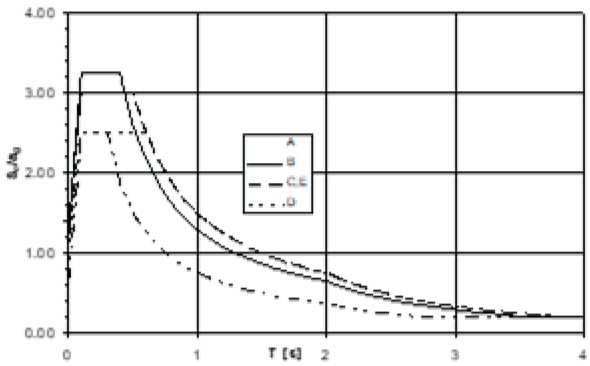


Figure 2: Elastic response spectrums according to EN 1998

However, a contradiction arises when modelling the soil-structure interaction, as the basic premise of the period-acceleration relationship is initially based on single-mass cantilevers fixed at their base. The modelling of the interaction with the soil /allowed by the EUROCODE system/ generally leads to a reduction in the stiffness of the structure, an increase in the natural period and hence a reduction in the seismic forces when taking into consideration the response spectra. On other hand, the reduction in the stiffness influences the displacements in the structure and increases the forces due to second-order moments. Therefore, lower values of the deformation characteristics of the soil for impulse loads are also acceptable. That is why for the seismic design of relatively tall bridges and high importance factor bridges it is advisable to implement linear or non-linear time-history analysis, especially in areas with high seismic hazard. In cases where time-history analysis has not been preformed, it should be approached carefully. For relatively rigid bridges with short natural periods, the adoption of higher values of soil deformation characteristics should be sought.

In summary, the implementation of a soil-structure interaction with characteristics obtained from those for long-term action can be recommended, however, multiplied by 1.5 to 2.5 in case of short-term live loads and up to 8 times for impulse actions. In addition, for raft foundations it is advisable to apply non-linear springs allowing the exclusion of tension. Thus, whether there is a probability of the occurrence of the "rocking" phenomenon will be calculated directly.

The modelling of the interaction between the abutments and the embankment behind them should be completed in accordance with the regulations in which these issues are addressed (for example [4]). According to this document, under a seismic action, horizontal springs can be modelled as linear springs with stiffness value equal to half of the maximum stiffness allowed according to the relevant documents (Figure 3). However, differential stiffness of the springs for temperature and impulse actions (earthquake and braking forces) should be considered in the analysis.

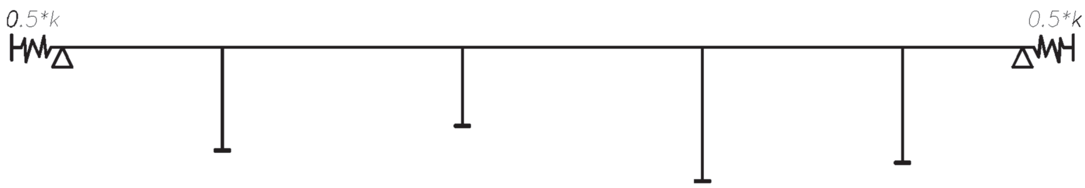


Figure 3: Distribution of embankment stiffness at the two abutments [4]

3 Track-structure interaction for railway bridges

For railway bridges, the study of the interaction between the bridge structure and a track with continuous welded rails, whether it is ballasted or ballastless, is of great importance. Generally, this issue is related to three groups of requirements as follows:

- to ensure traffic safety;
- to ensure bridge structure safety;
- to ensure the travel comfort of the passengers.

For the most designed bridge structures in EN 1991-2 [4] EN 1990:2003/A1:2006 [5], the requirements and the procedures related to these three groups are definite. Generally, the verifications are obtained indirectly through observing specific limitations. This paper focuses on the occurrence of additional stresses in a track with continuous rails regarding the safety of the railway track due to the interaction with the bridge structure. Furthermore, a critical analysis of the relevant regulation in EN 1991-2 [4] is presented and particular recommendations are proposed.

In EN 1991-2 [4], three possible methods are provided for checking the additional stresses that occur in ballasted tracks with continuous rails. Two of these methods, which have different scopes and accuracy, are indirect. The third procedure is more accurate and is based on a computational model for determining rail stresses. In some countries, there are additional guidelines for determining the stresses, but in Bulgaria such guidelines have not been created. According EN 1991-2 [4], the additional stresses in continuous tracks on the bridge can be considered as a result of the influence of the following effects:

- the rotation of the bridge structure at the abutments and/or above the piers (mutual rotation);
- the thermal expansion of the structure connected to the railway and the fact that outside of the bridge the road is practically motionless due to temperature;
- a similar effect but from traction and braking forces.

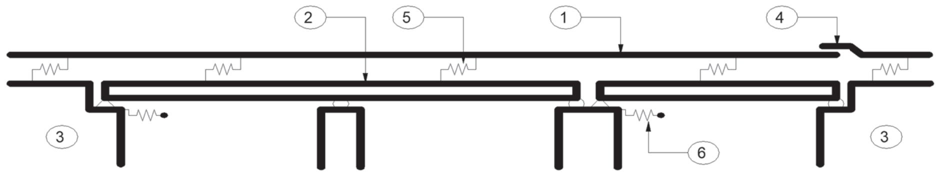
The combined response of the structure and track is mainly ensured by shearing the ballast bed, and in the case of bridges with ballastless tracks, by shearing the support pads of rail fastenings. The permissible additional design values of rail stresses as a result of the combined response of the structure and track should be limited as follows:

- for tension of 92 MPa;
- for compression of 72 MPa.

The model that is recommended has the form represented in Fig. 4, which can be classified as outdated. To model the influence of the substructure as springs probably aimed to simplify the modelling process. This approximation literally serves only to complicate the model since with today's software capabilities, additional modelling of the foundation, piers and bearings is not a complicated task and will yield more precise results. Therefore, instead of such an approximation of the influence of the substructure, it is advisable to implement a three-dimensional model, including the consideration of the soil-structure interaction. Moreover, the proposed model in EN 1991-2 does not consider the vertical displacement of the piers - due to temperature difference and/or soil settlements.

The other task to be solved by designers is modelling the interaction between the rails and the bridge structure. According to EN 1991-2, the connection through springs in the model is only in the horizontal direction and the interaction in the vertical direction is neglected, although vertical displacement, which is mainly due to rail fasteners should also be taken in to account.

The horizontal connection has the same characteristics in both directions and can be modelled with non-linear links, as many software products feature such capabilities. The vertical link also



1.Track 2. Superstructure (a single deck comprising two spans and a single deck with one span shown) 3. Embankment 4. Rail expansion device (if present) 5. Longitudinal non-linear springs reproducing the longitudinal load/ displacement behaviour of track 6. Longitudinal springs reproducing the longitudinal stiffness K of a fixed support to the deck taking into account the stiffness of the foundation, piers and bearings, etc.

Figure 4: A model of a track/structure system according to EN 1991-2 [3]

has non-linear behaviour, however, a specific feature of this link is that it has different characteristics in the two directions. Uplift occurs due to deformation of the steel fasteners and has an elastic behaviour. The downward displacement /against the slippers/ is due to deformation of the rubber pads and this connection has other parameters. In summary, the proper modelling of the interaction between the rails and the bridge structure is significant for the precision of the computer model.

When the vertical deformation of the fasteners is not considered, this leads to larger bending moments in the rails especially at the joint between the substructure and the abutments. This effect is even more noticeable in the case of a ballastless track.

Furthermore, for the model in Fig.4, the length of the rails that should be assumed outside the bridge is not specified. There are additional guidelines for this issue. According to these guidelines, a value of ½ of the length of the bridge behind each of the two abutments should be considered. Based on our research, a length at least equal to the length of the bridge should be provided for to achieve more sustainable results.

The other issue that needs to be determined is the distance of the springs between the superstructure and the rails. In terms of horizontal effects (temperature and braking force), this parameter can be taken as an n-fold of the distance between sleepers (or between fasteners for a ballastless track). However, this is not appropriate regarding the effects of the rotation of the structure in the joints. There, the distance should necessarily correspond to the distance between each of the fastenings. Concerning the links outside the bridge, it is not essential to take soil settlements into account. Therefore, rigid supports can be modelled and connected to the rails with non-linear links.

Additional stresses in tracks with continuous rails obtain three groups of actions: temperature variation, braking/traction force and rotation of the structure at the joints because of a vertical moving load (for example load model LM71). Considering the non-linear behaviour of the connections and the fact that the stresses due to moving loads and temperature effects are obtained from different models, their summation cannot be linear by superposition.

The stresses from the temperature variation occur over a long period for an unloaded track due to a moving load. The parameters of the non-linear links for this load case are different from those for a loaded track. Moreover, for the load case of a temperature variation, it is permissible to model a reduced / up to 50% / stiffness of the reinforced concrete elements of the bridge structure.

Taking into account the above consideration and based on our research, the following procedure of summing the stresses in the rails is proposed (Eq.1).

$$\sigma = \sigma_1 + \sigma_2 + \sigma_3 \tag{1}$$

where:

σ – the additional stresses in rails

σ_1 – the additional stresses in rails due to LM71, braking force and temperature variation in a model with characteristics for short-term actions and nonlinear links with parameters for a loaded track;

σ_2 – the additional stresses in rails due to temperature variation in a model with characteristics for short-term actions and nonlinear links with parameters for a loaded track;

σ_3 – the additional stresses in rails only from the temperature difference in a model with characteristics for long-term actions and non-linear links with parameters for an unloaded track.

Even though clause 6.5.4.4 in EN 1991-2 [5] states that (6) a linear "superposition" of the stresses is permitted, our current research for specific cases found stresses up to 15-20% greater than the non-linear superposition proposed in the codes. Although it is not directly related to modelling issues, a question about the definition of "additional stresses" arises, since the obtained stresses from the model include the main stresses as well.
For example, from a temperature difference in the track outside of bridge, the stresses, assuming an infinite length, are obtained under the assumption that there is no displacement at rail fastenings. In this case, the stresses are as follows (Eq.2):

$$\sigma = \alpha_t \cdot \Delta_t \cdot E_s \tag{2}$$

Therefore, one way to define these stresses as additional is to obtain from the model what the maximum stresses are and from these to subtract the stresses for an "infinite" railway track. In addition to the longitudinal and vertical displacements that should be limited, the transverse displacements and vibrations should also be checked. Although longitudinal and vertical vibrations can be obtained based on an appropriate two-dimensional model, it is advisable to implement a three-dimensional model for calculating transverse vibrations.
Regarding the insurance of traffic safety, it is required to limit the lateral displacements from characteristic actions. In EN 1990:2003/A1:2006 [6] item A.2.4.4.2, requirements are listed for controlling the following:

- vertical acceleration of the deck
- deck twist
- vertical deformation of the deck and angular rotations relative to the vertical at each end of the superstructure (at the abutment joints or between the two superstructures)

- transverse deformation and vibration of the deck based on the change of the radius of the curvature along the length of the track and the relative displacement of the superstructure relative to the abutment or between the two superstructures (see also table A2.8 [6].)

The specified values in table A2.8 [6] are obtained under the assumption that the deformations of the superstructure, the substructure and foundations/piles are considered. Thus, a three-dimensional model needs to be applied, including the modelling of the soil-structure interaction. In addition, according to item A.2.4.4.2.4 (3) [6] "The first natural frequency of lateral vibration of a span should not be less than 1.2Hz", which is a rather inexplicit definition. It can be interpreted as a vibration of a span from the superstructure without considering the substructure or a mode with hardly any response of the piers. That is why a three-dimensional model is necessary.

4 Reactions in the bearings of railway bridges under seismic action

Usually for a system of simply supported superstructures of railway bridges, to reduce additional stresses in the rails and to avoid implementation of devices for longitudinal deformation of continuous rails, longitudinally fixed bearings are placed successively at one of the supports, and longitudinally movable ones at the other end. This is repeated alternately in each span. In such superstructures with more than one fixed bearing in the transverse direction as a result of transverse horizontal action / for example transverse seismic action /, a relatively large longitudinal reaction occurs, sometimes even impossible to be absorbed. The design of a single fixed bearing/device at one support in order to absorb the longitudinal reaction will eliminate such an additional longitudinal reaction from a transverse action, however, this is not always possible because the longitudinal force due to seismic and moving action in this bearing/device will be very large, especially in a double-track bridge in areas with high seismic hazard. Such an effect is present only in a three-dimensional model. If there is only one bearing/device to absorb the longitudinal reaction at one support (Figure 5a), no longitudinal reaction from a transverse action occurs. However, this is not always possible because of the reaction, especially in a double-track bridge, as the longitudinal force in this bearing/device will become very large. If more than one fixed bearing is placed (Figure 5b), the longitudinal force from the seismic action in the transversal direction becomes very large and sometimes almost impossible to absorb.

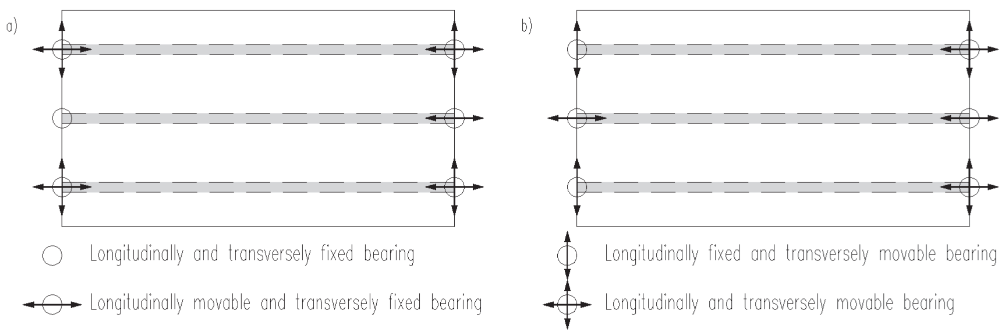


Figure 5: Bridge superstructure with one (a) and two (b) longitudinally fixed bearings

Since every bearing/device has some allowable slag which is taken into account, the longitudinal reaction from a transverse action will be reduce depending on the value of the slag itself. However, this will induce longitudinal displacements, which are also limited regarding the additional stresses in the rails. A device/bearing can be designed with a correspondingly controllable slag to minimize the longitudinal reaction and at the same time to restrict the longitudinal displacement due to longitudinal actions within the controllable limit avoiding the necessity of a device for longitudinal deformation of the continuous rails. In this case, in the three-dimensional model, the devices must be modelled with nonlinear links taking into account the slag in order to obtain realistic values for both reactions and displacements.

5 Summary

On the basis of the studied issues, it can be concluded that the computer modelling of bridge structures can significantly affect the precision of the results. Furthermore, the implementation of three-dimensional computer models allow to cover more effects. Spring elements are often applied in order to model the connection to nonstructural elements. Even though certain regulations for spring characteristics exist, there are many issues that need to be solved additionally by designers.

6 References

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