Evaluation of transverse impact factors in twin-box girder bridges for high-speed railways

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Abstract

This paper deals with the dynamic behavior of twin-box girder bridges under high-speed railway traffic. Based on several representative examples derived from recently built high-speed bridges, this contribution examines the effects of transverse bending in the upper slab of these structures and evaluates the bending moments in resonance conditions. The analysis is carried out according to one of the reference norms for the assessment of dynamic effects in high-speed bridges (Eurocode). The results demonstrate that the predicted dynamic response for shorter span bridges could be unexpectedly higher than the static effects caused by the design loads, due to transverse resonances induced by the absence of transverse diaphragms between the box girders and the movement of the sliding supports. Moreover, these strong impact coefficients may occur even when the maximum level of vertical vibrations in the deck is not alarming.

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Introduction

In modern high-speed railway lines *twin-box girder bridges* have become one of the most popular solutions for spans between approximately 20 m and 45 m (Figure 1). This success is attributable to their short construction time, which is largely due to the prefabrication of the two main girders.

Fig. 1. Twin-box girder bridge on Madrid-Barcelona high-speed railway line. Characteristic span length L=30 m

Significant dynamic effects may arise when transversely movable supports are deployed in absence of diaphragms between the box girders. This configuration, which can be found in high-speed lines such as the one connecting Spain and France or Madrid and Barcelona (Burón and Peláez, 2002), induces potential resonance responses of the structure that could seriously affect the upper concrete slab (excessive cracking, fatigue) if the dynamic effects are not considered properly.

Some earlier studies on the subject do deal with transverse bending (Hamed and Frostig 2005, Huang and Wang 1993, 1995, Rattigan et al. 2005), but very little has been said about twin-box girder bridges. Cheung and Megnounit (1991) conducted a study specifically devoted to twin-box girder bridges. However it fails to consider the transverse distribution of bending moments.

This work endeavors to launch a comprehensive study where several twin-box girder bridges of increasing span length are analyzed. The numerical models used in this study intentionally follow the prescriptions of Eurocode 1 (EC1) (CEN, EN 1991-2 2002), in an attempt to show the predicted performance at the design stage. The influence of the configuration of the supports on the dynamic response, particularly in the absence of transverse diaphragms between the main girders, is one of the key issues with which this paper is concerned.

Twin-box girder bridges: case studies

This study presents analysis results for four simply-supported decks of spans (20, 25, 30 and 35 m). Their main properties, shown in Figure 2 and Table 1, are derived from existing structures so as to constitute realistic examples leading to meaningful results and conclusions. The bridge deck consists of two prestressed, precast concrete U-shaped girders and a reinforced concrete, cast in-situ upper slab. Each U-girder usually has rigid diaphragms at both ends, where the hollow section is stiffened by a solid infill.

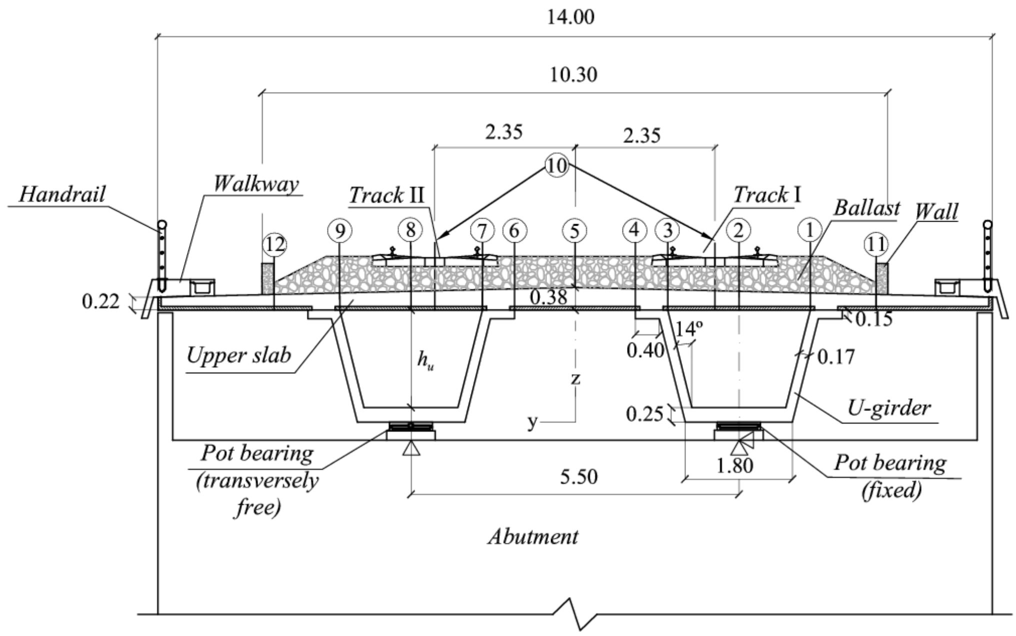

Fig. 2. Representative cross-section of a twin-box girder bridge and post-process points

L (m)		20	25	30	35
Upper slab	ρ (kg/m^3)	2500			
	f_{ck} (MPa)	35			
U-girders	h_u (m)	1.44	1.89	2.35	2.8
	ρ (kg/m^3)	2500			
	f_{ck} (MPa)	45			
Dead loads	*Ballast+tracks* (kg/m)	11000			
	Walls (kg/m)	480			
	Walkways (kg/m)	2450			
	Handrails (kg/m)	900			

Table 1. Main properties of the bridges

As regards the longitudinal constraints, both pots at one end are fixed and those at the opposite end are free. In a generic manner, the end of the deck where the longitudinal constraints are placed is referred to as *fixed abutment*.

Numerical model

General aspects and assumptions

Two different linear elastic analyses were performed: static and transient dynamic analysis solved by mode superposition under the action of railway traffic. With this purpose a suitable finite element model (FEM) was devised. The meshing process, the static analyses and the extraction of frequencies and mode shapes were performed using the commercial code ANSYS, while the intensive computations associated with the passing of trains across the bridges at different speeds were implemented with a suitable FORTRAN routine. This routine carries out the time-integration by the Newmark-β linear acceleration algorithm, using a time step equal to 1/25 times the smallest period among the modes considered.

A point load model is adopted for the railway excitation, following the European standards. Therefore, train-bridge interaction is neglected in the analysis, which is also supported by previous works (Doménech et al. 2014). The numerical model also disregards track irregularities, since the regulations merely treat them by means of a multiplying factor. The effects of soil-structure interaction are also neglected; this is usual in bridges supported on short piles lying on a stiff foundation (Antolín et al. 2013; Liu et al. 2014).

Deck geometry

Figure 3 shows the mesh in the area near the abutments. The structure is discretized using four-node shell elements with six degrees of freedom (dofs) per node and out-of-plane shear deformation capabilities. For the rigid diaphragms at both ends of the girders (shaded elements in Figure 3), eight-node hexahedral solid elements with three dofs per node were used.

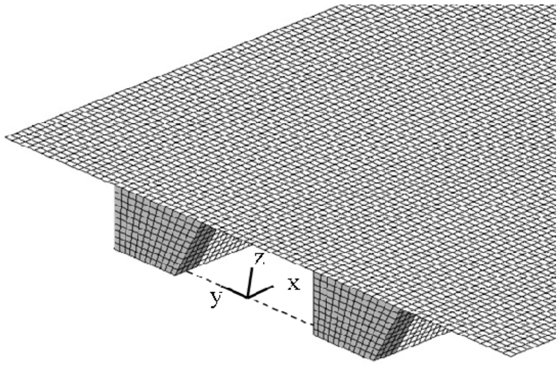

Fig. 3. FE mesh at the fixed abutment

All the elements have a length of 0.25 m in direction X. The size along direction Y (slabs) and direction Z (webs) does not remain constant for all the span lengths, but is rather similar. The average length in direction Y is 0.22 m for the upper slab and 0.14 m for the lower slab. Along the webs the average size is 0.18 m.

Permanent loads, e.g., ballast, track, walkways, etc., are distributed as additional masses of the elements of the upper slab. As regards the boundary conditions, the model considers pot bearings as ideal supports, a common assumption that previous research works also adopted (Majka and Hartnett 2009; Antolín et al. 2013). In the fixed abutment the bottom center node of the solid meshes at the diaphragm positions in each of the girders is constrained in the longitudinal and vertical directions (X and Z), whereas only one of them is fixed in transverse direction Y. At the opposite abutment the boundary conditions are identical except for the constraints in X, which are not present. Additionally, kinematic constraints are used in order to tie this restrained central node to a number of adjacent rows/columns of nodes, covering an area similar to the real pot dimensions.

Static and dynamic loads

From a practical point of view it is customary to refer the maximum dynamic effects to some particular static load scenario by means of the so-called *impact coefficients*, i.e. the ratio between maximum dynamic and static values of the internal forces. As a common practice in

Europe, the reference static forces to be applied are the UIC-71 train defined in EC1, which represents the static effect of vertical loading due to normal rail traffic. In this study the variables of real interest are the dynamic internal forces; therefore the UIC-71 loads are located in a convenient, straightforward position, acting symmetrically with respect to the mid-span section.

The most unfavorable dynamic load usually occurs when the trains circulate at speeds such that a given vibration mode experiences resonance. According to EC1 only one loaded track is considered during the dynamic analyses, and the dynamic loads to be applied are the 10 trains prescribed in the High Speed Load Model A (HSLM-A model). They constitute an envelope of the dynamic effects of the existing conventional high-speed trains.

Description of the analyses and post-processing points

The response of the four subject bridges is computed first in terms of transverse bending moments under the static action of the UIC-71 loads placed at mid-span. These response variables are then evaluated under the circulation of HSLM-A trains along each of the tracks on the bridge (track I and track II, according to Figure 2) in two different ranges of velocities of interest, which are [72, 420] km/h and [72, 540] km/h in steps of 3.6 km/h. The impact coefficients are evaluated separately in each range of circulation speeds.

The static and dynamic results are computed at five sections {A, B, C, D, E} corresponding to x/L = {0.25, 0.375, 0.5, 0.625, 0.75}, where L is the span length. In each section several points for obtaining bending moments and also vertical accelerations are considered. Figure 2 shows the locations of the points: transverse bending moments are computed at points from 1 to 9, and accelerations are obtained at points 11, 10, 5 and 12. Notice that when the loaded track is I, point 10 is located between points 2 and 3; conversely, if the loaded track is II, point 10 is placed between 7 and 8.

Results

Natural frequencies and mode shapes

All the cases of study have a similar pattern in their mode shapes: the first three eigenforms are global ones and they essentially govern the dynamic response; the modes above the third one may be local or global, and their main effect on the internal forces is a pseudo-static contribution. Table 2 gathers the natural frequencies of the first four eigenforms.

L (m)	1st mode	2nd mode	3rd mode	4th mode
20	4.141	5.750	6.230	9.288
25	3.671	4.991	5.741	8.803
30	3.232	4.335	5.512	8.191
35	2.862	3.822	5.329	7.428

Table 2. First four natural frequencies (Hz) of the bridges

Figure 4 shows the first four modes and their frequencies for the 25 m bridge. The first mode is a transverse bending of the upper slab. In this eigenform the girders rotate as rigid bodies and have little torsion, with also a limited longitudinal bending. In longitudinal bending the U-girders do not behave as a single beam, but their main bending vibrations correspond to modes 2 and 3 with similar frequencies and shapes: in both modes there is a predominant longitudinal bending of one of the U-girders, complemented by a kind of rigid-body rotation and a limited bending of the other. The bridges of span 20 m, 30 m and 35 m feature similar mode shapes.

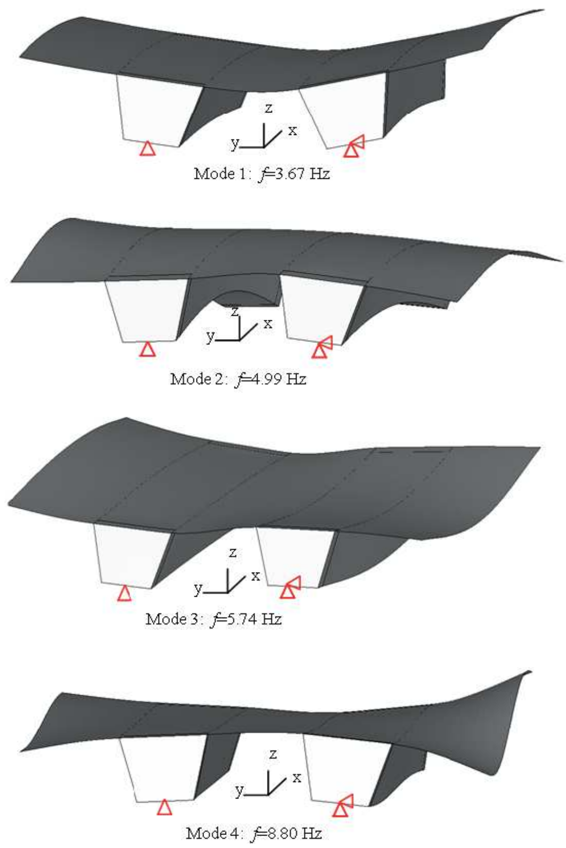

Fig. 4. First four vibration modes for the 25 m bridge

Envelopes of internal forces versus speed

Figure 5 shows the maximum absolute values of transverse bending moment (M_x) due to the circulation of the HSLM-A trains at the most unfavorable post-process points. The values are plotted against the circulating speed for all bridges and for an increasing number of mode contributions (up to 200 modes, showing a satisfactory convergence). These results correspond to the circulation of the trains along track I, and a uniform damping ratio of 1% is assigned to all mode contributions following the prescriptions of EC1. For the sake of comparison, Figure 5 also shows the maximum absolute static value among all the post-process points under the action of the UIC-71 train. Particularly for the shortest structures, the maximum dynamic values largely exceed the static ones created by the UIC-71 design loads.

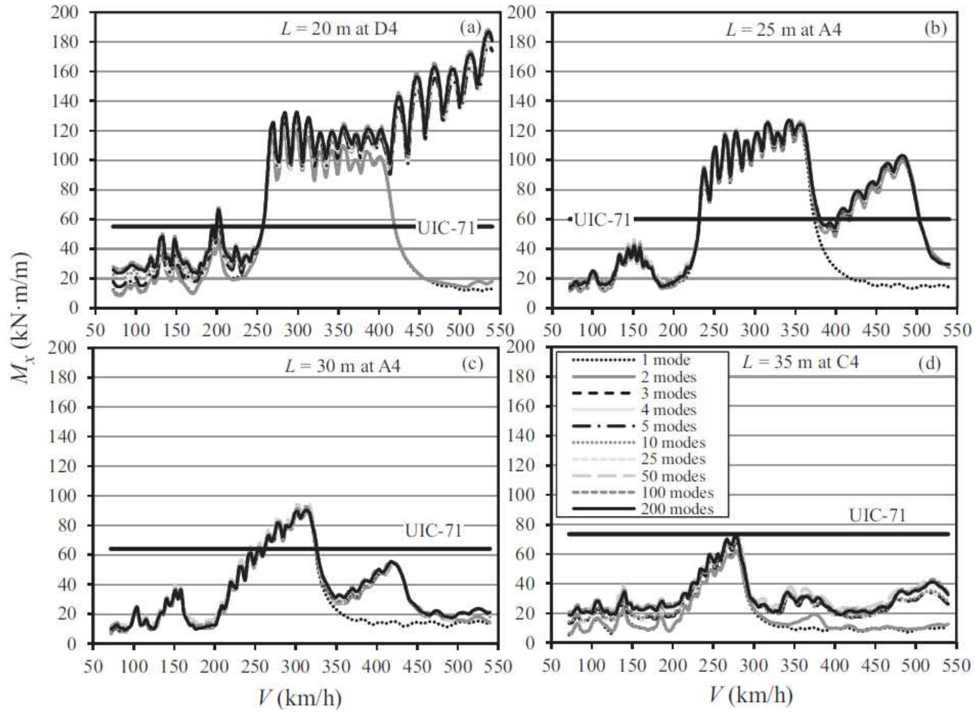

Fig. 5. Envelopes of maximum absolute transverse bending moments due to live loads. Trains circulating along track I. Legend in (d) applies to all subplots.

As can be seen in Figure 5, the maximum resonance peaks of the transverse bending moments are mainly governed by the contribution of the first eigenform at speeds below 300–350 km/h, which is a frequent velocity limit in many high-speed railway lines. The contribution of the longitudinal bending modes is also noticeable at speeds higher than 350 km/h, especially for the shortest spans (*L*= 20 m, 25 m); but as the span length increases, the first mode prevails.

When the trains circulate along the opposite track (track II) the predominant mode contributions for each span length do not differ significantly from the results shown in Figure 5. However, the influence of the loaded track on the dynamic response amplitude is in general quite noticeable. This is shown in Figure 6(a), where the transverse bending moment at the critical post-process points for the bridge of 25 m span is plotted, considering the contribution of the first 200 modes and the circulation of the trains alternatively along track I and track II, in opposite directions. These results highlight that the dynamic behavior of twin-box girder bridges under moving loads is clearly three-dimensional.

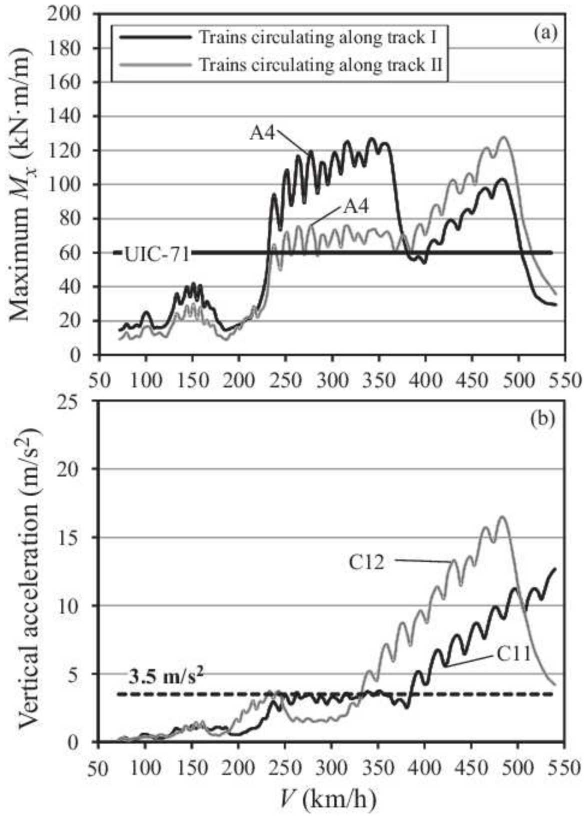

Fig. 6. Envelopes of maximum dynamic results for the 25 m bridge. (a) Transverse bending moments; (b) vertical accelerations.

Impact coefficients

On a standard basis, the impact coefficients for transverse bending moments are used for the design of the transverse reinforcement in the upper slab. In the initial design stages of twin-box girder bridges, the coefficients presented in this section may thus provide a helpful first estimate of what may be expected from transverse resonance phenomena.

The impact coefficient is evaluated as the quotient between the maximum dynamic value in the upper slab and the maximum static one, both of them having the same sign. The maximum static values used for the evaluation of the impact factors are obtained after placing

UIC-71 loads symmetrically along track II. The maximum dynamic transverse bending moments in the upper slab are positive, and are caused by the circulation of the trains along track I. They have been collected in Figure 7.

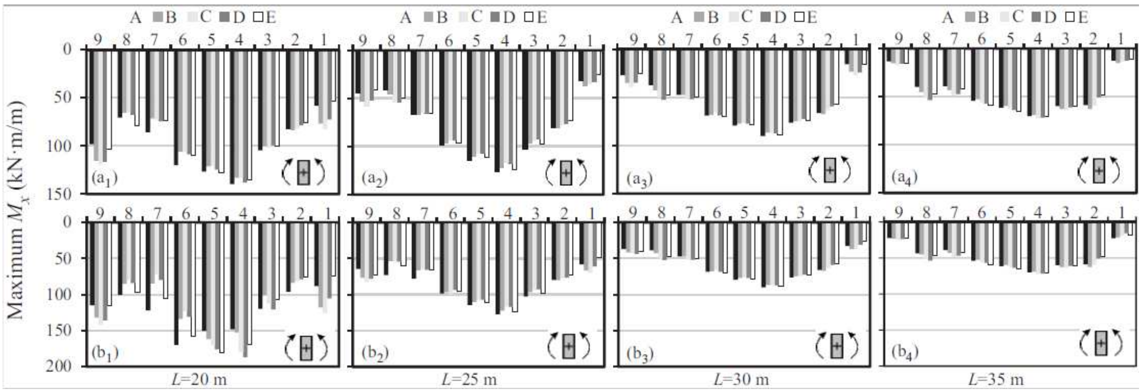

Fig. 7. Envelopes of maximum positive transverse bending moments under the circulation of HSLM-A trains along tracks I and II. (a_i) V_{max}=350 x1.2=420 km/h; (b_i) V_{max}=450 x 1.2=540 km/h.

Table 3 gathers the impact coefficients for the bending moment considering maximum train speeds of 420 km/h and 540 km/h. It is seen that they are more affected by the increase in speed for the shortest span, while they remain almost constant when the velocity rises to 540 km/h for the longest spans. Values higher than 2.0 are obtained in several cases. If not taken properly into account, this effect may have an influence on the transverse cracking of the concrete slab, which in turn may result in reductions in both the stiffness and the first natural frequency, thus leaving the bridge even more exposed to resonance phenomena (at lower speeds).

V_{max}	L=20 m	L=25 m	L=30 m	L=35 m
420 km/h	2.54	2.11	1.41	0.98
540 km/h	3.39	2.11	1.41	0.98

Table 3. Impact coefficients for transverse bending moment

Vertical accelerations

The maximum level of vertical vibrations usually constitutes a critical Serviceability Limit State (SLS) for other types of simply-supported high-speed bridges (ERRI D214/RP9 2001; Frýba 2001; EN 1991-2 2002; Museros and Alarcón 2005). The vertical accelerations under the circulation of HSLM-A trains have been computed considering a maximum number of mode contributions up to 30 Hz, which is a limit usually prescribed by structural codes (ERRI D214/RP9 2001). The maximum peak values of the vertical acceleration of the bridge deck calculated along each track shall not exceed 3.5 m/s^2 for ballasted tracks, according to Eurocode (CEN, EN 1990-A2, 2005).

The analyses have shown that the 35 m bridge satisfies the 3.5 m/s^2 criterion in the whole range of speeds. The 30 m bridge presents a good behavior up to 400 km/h approximately. The 20 and 25 m bridges also behave well up to 350 km/h (approx.), where resonances of the second and third modes start to increase the response significantly. Consequently, the potential use of twin-box girder bridges for *very high-speed* lines (V>350 km/h) should be examined with particular care.

Finally, Figure 6(b) shows the influence of the loaded track on the envelopes of maximum acceleration versus speed, for the 25 m bridge. The most unfavorable circulating track is not the same over the whole range of speeds, a fact that was also observed for transverse bending moments, and underlines the importance of using three-dimensional models in the dynamic analysis of this type of bridge.

Conclusions

In this work the dynamic response of several representative twin-box girder bridges under high-speed railway traffic has been analyzed. The aim of this study was to investigate the unusual performance predicted at the design stage when the transversally sliding bearings beneath one of the U-girders are modelled as ideal rollers and without transverse diaphragms between the box girders. The main conclusions are the following:

- The impact coefficients for transverse bending moments are higher than 2.0 and tend to decrease with the span length. Such extreme values highlight the need for future research work to support or contradict whether they are excessively conservative due to other effects that should be considered in the calculations, such as a performance of the pot bearings far from the ideal behavior implemented in most numerical models.
- At speeds below 350 km/h the transverse bending moments are mainly governed by resonances of the first eigenform. The introduction of diaphragms or cross-bracings between the girders could significantly reduce those transverse bending moments in spite of a certain amount of complexity being added to the construction process. This stiffening measure would be in line with the California codal recommendation of the first torsional frequency being at least 1.2 times greater than the first vertical bending frequency. Such interpretation of this code would be reasonable from an engineering point of view, given that the first eigenform is not a torsional mode but a transverse bending one that is not contemplated in (California High-Speed rail Authority 2014).
- The potential use of twin-box girder bridges for very high-speed lines (V>350 km/h approx.) should be examined with particular care due to excessively high vertical accelerations appearing in the ballast. Structures that are stiffer and more massive than the ones analyzed in this paper could be required to satisfy the acceleration SLS (3.5 m/s^2) at such very fast speeds.
- The dynamic behavior of twin-box girder bridges under moving loads is clearly three-dimensional: the contribution of the first transverse bending mode to the corresponding bending moments and the influence of the loaded track are significant.

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