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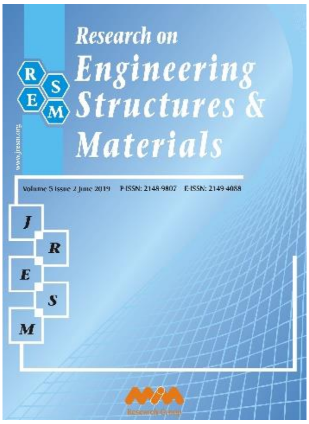

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Research Article

Computational study of the vertical impact coefficient on girders of pier access bridges

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Abstract

Bridges in Brazil are designed according to design code NBR 7188:2013 [1] and NBR 7187:2003 [2], in which the moving load model is composed of a three-axle vehicle. The configuration of the moving load model follows the pattern of an older version of the code, the NB-6 (1960). Despite the updating of load values, the present moving load model is not appropriate to represent the current traffic effects in Brazilian bridges. The dynamic effects induced by the moving load are taken into account by the impact coefficient, applied in the load model. The static values of the load model are obtained by multiplying its load by this coefficient. The objective of this work is to perform a dynamic analysis of bridge girders, to determine the dynamic effects, to compare with the static effects and to measure the accuracy of the impact coefficient. The results obtained for the beams, showed that for some cases, the impact coefficients had a good approximation to transform the static efforts into dynamic ones. However, in other cases, these coefficients did not show the same result. Through the study it was possible to identify that the impact coefficients provided in the code can be enhanced from new studies taking into account the dynamic analysis of loadings of Brazilian bridges.

1. Introduction

It is called a bridge a construction with the purpose of crossing obstacles to the normal continuity of a way, such as rivers, sea arms, deep valleys, other routes, among other cases. And it is called a viaduct the bridge that has as its objective the transposition of valleys, other ways or obstacles in general not constituted by water [3].

As for the port structures, the construction built on the sea is considered to be a bridge, which provides the connection between the coast and the offshore dock, in order to allow the mooring of ships to loading or unloading and the passage of people and vehicles [4]. An example of such a structure is the port of Açu access bridge (Fig. 1).

Bridges in Brazil are designed according to NBR 7188:2013 [1] and NBR 7187:2003 [2], called "Road and pedestrian live load on bridges, viaducts, footbridges, and other structures" and "Design of reinforced and prestressed concrete bridges - Procedure", respectively. The moving load model is composed of a 3-axle vehicle plus a uniformly distributed load, to be applied in the region outside the vehicle boundaries and multiplied by a dynamic amplification factor, called the vertical load weighting coefficient, which is a function of the bridge span length, number of spans and material used in the structure.

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Fig. 1 View of the access bridge port complex Açu [5]

In the study of the structural performance of bridges, the dynamic effects are not included, such as load mobility, vehicle oscillation upon reaching the bridge, impact on the bridge deck due to track irregularities, speed variation among others [6].

In studies of load capacity and dynamic analysis, computational mathematical modeling has become essential, representing the structure as faithfully as possible, to calculate precisely the stress and deformations present in the structural elements [7].

This study intends to compare the dynamic effects and the static effects caused by the moving load model, on a typical pier access bridge. For this, the study seeks to develop numerical and analytical models to simulate the passage of vehicles on some types of bridges.

After the studies, the representative values obtained from the dynamic effects due to the moving load will be compared with the dynamic effects recommended by NBR 7188:2013 [1], when applied to the same bridge systems, in order to evaluate if the code is properly considering the effects of dynamic loading.

2. Methodology

The bridge used was designed with spans of 10 m, 20 m, 30 m, and 40 m. The bridges are 11.50 m wide, have 20 cm slab thickness and four longitudinal girder beams, spaced equally every 310 cm (Fig. 2).

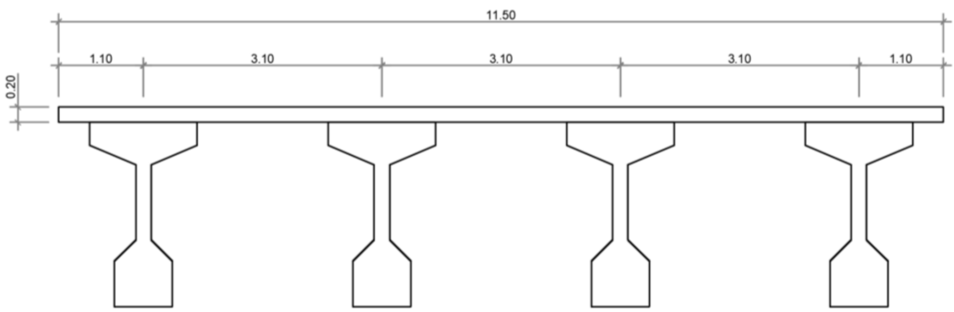

Fig. 2 Cross section of the bridge with 40 m of span. Dimensions in m.

The bridge was considered with the slab simply supported on the beams, non-structural elements were disregarded and for each span of the structure, a different section of the girders was adopted (Fig 3).

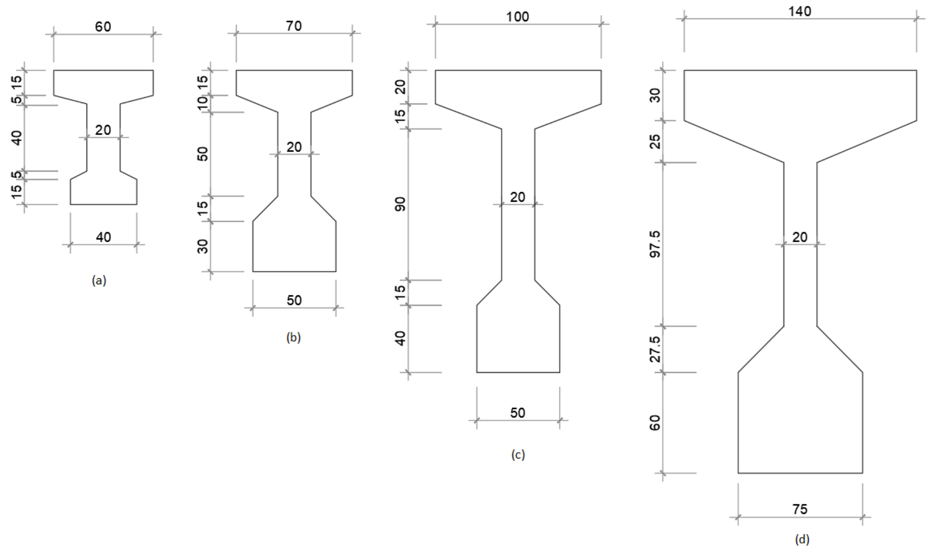

Fig. 3 Cross section of bridge girder beams. Dimensions in cm.

(a) span of 10m, (b) span of 20m, (c) span of 30m and (d) span of 40m.

The numerical-computational model was implemented using the computational tool ANSYS 19.1 Academic version, which analyzed the passage of the vehicle on the structure. Through the software, the usual techniques of discretization using the finite element method were used. The bridge model was made using the grid analogy, in which for both slab and girder were used beam elements, BEAM188 type, which have two nodes and six degrees of freedom per node, being these the translations in the x, y, and z directions and rotations about the x, y, and z directions. Each finite element that constitutes the mesh of the structure has a length of 0.25 m (Fig. 4). Table 1 presents the values of the main properties of the numerical model of the bridge.

Table 1 Properties of the material of the bridges.

Properties	Value Adopted	Unity
Concrete specific mass	2500	kg/m^3
Compressive strength of concrete	30	MPa
Concrete modulus of elasticity	26838	MPa
Concrete coefficient Poisson	0.2	

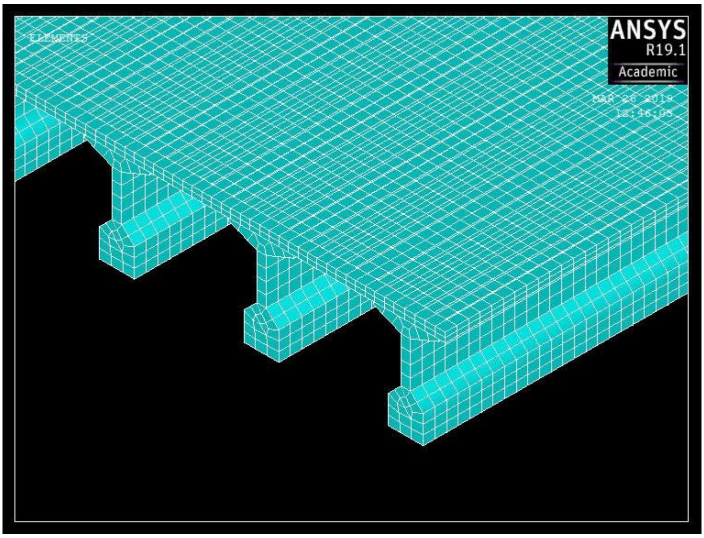

Fig. 4 Finite element mesh, complete perspective

For the moving load, the TB-450 load train (Fig. 5) was considered, which is defined by a three-axle vehicle with a total load of 450 kN, distributed equally on each wheel. It was also considered that the vehicle transits in the most unfavorable position for structure, in order to generate the greatest internal forces, and with a speed of 80 km/h.

For the application of the load from the load train type in the structure, each node of the numerical-computational model belonging to the passage of the vehicle was identified, and, at every 0.5 m, the loads were applied as a function of time.

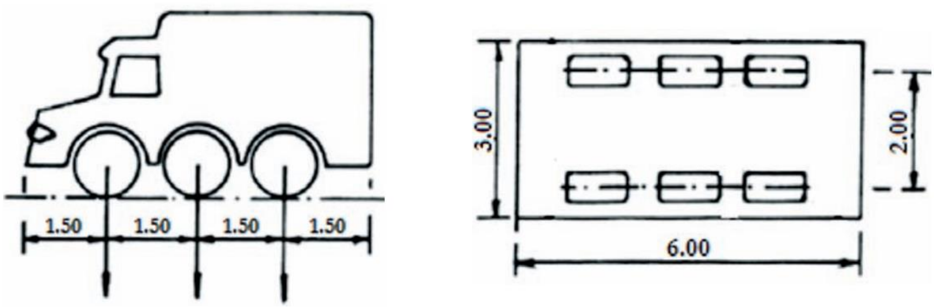

Fig. 5 Truck used in load train type TB-450, according to NBR 7188:2013 [1]. Dimensions in m.

To represent the dynamic interaction between the vehicle and the structure, Eq (1), provided by Fryba [8], was used to represent the harmonic load crossing the bridge.

$$P(t) = P + Q\ sen(\Omega t) \tag{1}$$

Where P is the moving load value in kN, Q is the amplitude in kN, Ω is the circular frequency of the harmonic force in rad/s and t is the time in seconds. According to Melo [9], the value of the oscillation frequency of the vehicle can be between 2.0 Hz and 4.0 Hz, being the last value adopted for the study. A value of 0.10P was also chosen for the amplitude.

For the modeling of damping in the structure, it was used the Rayleigh damping matrix [10], which considers two main plots, α being the contribution rate of the mass matrix, Eq (2), and β rate of contribution of the stiffness matrix, Eq (3). From the most important natural frequencies, ω_1 and ω_2, it is possible to calculate such values.

$$\alpha = 2\xi \frac{\omega_1 \omega_2}{\omega_1 + \omega_2} \tag{2}$$

$$\beta = 2\xi \frac{1}{\omega_1 + \omega_2} \tag{3}$$

Where ξ is damping ratio, where the value of 2% is adopted, and ω_1 and ω_2 are the first and second natural frequencies of the structure in rad/s, respectively.

In order to compare the dynamic effects due to moving load with the dynamic effects recommended by NBR 7188:2013 [1], the dynamic amplification factor (DAF) according to Eq (4) was calculated, so as to check with the correction factor of code.

$$DAF = \frac{Dynamic\ Effort}{Static\ Effort} \tag{4}$$

The correction factor of NBR 7188:2013 [1], due to vertical moving loads, is given by the vertical impact coefficient (CIV), as shown in Eq (5).

$$CIV = \begin{cases} 1.35; L < 10m \\ 1 + \left(\frac{21.2}{L+50}\right); 10m \leq L \leq 200m \end{cases} \tag{5}$$

Where L is the span length for isostatic spans and the average span for continuous bridges.

To verify the transient analysis by means of the computational model, it is used the expression that determines the dynamic coefficient for a simply supported damped beam crossed by a harmonic load, Eq (6), described by Fryba [8], and the result obtained by the symbolic algebraic software wxMaxima is compared to the value of the model in the software ANSYS.

$$\delta = 1 + \frac{Q}{P}\frac{{\omega_{(1)}}^2}{\Omega^2}\frac{1}{\left(\frac{{\omega_{(1)}}^2}{\Omega^2}-1\right)^2+4\left(\frac{\omega^2}{\Omega^2}+\frac{{\omega_b}^2}{\Omega^2}\right)}\left\{\left[\left(\frac{{\omega_{(1)}}^2}{\Omega^2}-1\right)^2+4\frac{{\omega_b}^2}{\Omega^2}\right]^{1/2}+2\frac{\omega}{\Omega}e^{-\frac{\omega_b l}{(2c)}}\right\} \tag{6}$$

Where $\omega_{(1)}$, Eq (7), is the circular frequency for the first mode of vibration of the beam simply supported in rad/s; ω, Eq (8), is the load circular frequency in rad/s; ω_b, Eq (9), is the damped circular frequency of the beam simply supported on rad/s; l is the beam span length in m and c is the moving load speed in m/s.

$$\omega_{(1)} = \frac{\pi^2}{l^2}\sqrt{\frac{EJ}{\mu}} \tag{7}$$

Where E is the modulus of elasticity of the material adopted in Pa, J is the moment of inertia of the geometry used in m^4 and μ is the mass of the beam per unit length in kg/m.

$$\omega = \frac{\pi c}{l} \tag{8}$$

$$\omega_b = \frac{\omega_{(1)}\xi}{\sqrt{1-\xi^2}} \tag{9}$$

3. Results

3.1. Model Verification

A transient analysis was performed for a simply supported beam, with 1 m of height, 40 cm large and 10 m of length, with a modulus of elasticity of 25 GPa and a specific weight of 2500 kg/m^3, subject to a moving load of 100 kN, with amplitude of 10 kN, circular frequency of 30 rad/s and with speed of 60 km/h. For this case, a damping ratio of 5% is considered.

Table 2 shows a good approximation between the numerical-computational model for transient analysis performed in the software ANSYS and the expression that determines the dynamic coefficient for a simply supported damped beam being crossed by a harmonic load, described by Fryba [8].

Table 2 Dynamic coefficient for a damped beam subjected to a harmonic moving load.

Dynamic Coefficient δ		Error (%)
ANSYS	Fryba	
1.121	1.113	0.68

The numerical model analyzed the bridges using the grid analogy. The same bridges were solved, considering them as isolated beams, through software FTOOL, in order to make a comparison between the resolution methods. The value of the maximum bending moment for all beams was calculated for the load applied on beam 1 along with the total bending moment.

The loading is applied in the middle of the girder and it is formed by a concentrated load of 100 kN. Fig. 6 shows an example of loading and the boundary conditions that have been applied to the extremities of the girders of the models.

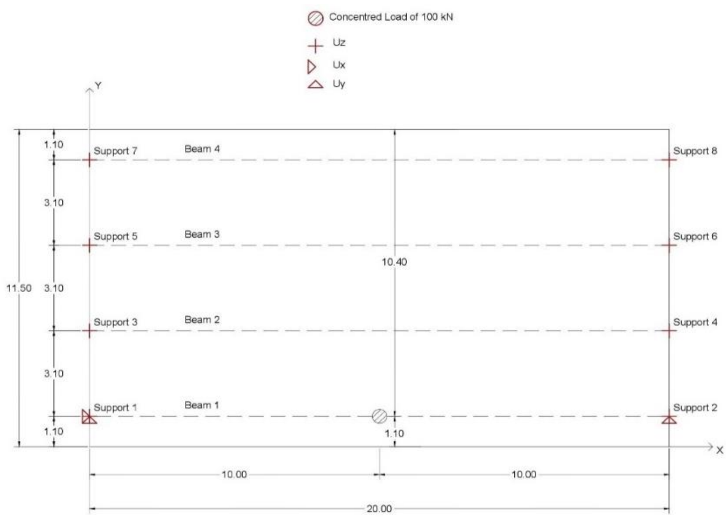

Fig. 6 Example of load model and boundary conditions for the situation of 20 m span, the dimensions in meters

After the definition of loading and boundary conditions, the model was solved using software ANSYS, resulting in the values of maximum normal force (N_i), and maximum bending moment (M_i) for each girder, as shown in Table 3 to Table 6. The tables also show the calculation used to find the total moment, taking into account the distance from the centroid of the slab to the centroid of each girder (e). Then, they were compared with the results obtained by the software FTOOL, as shown in Table 7. The calculation to obtain the total moment for each beam is constituted by the moment M_i of the precast beam in the central section plus the force N_i multiplied by the distance from the center of the slab to the center of the beam, as shown in Fig. 7, where the point O and the point G are, respectively, the centroids of the slab and the girder [11].

Table 3 Results obtained with loading in the beam 1 for the span of 10 m

Beam	N_i (kN)	M_i (kN.m)	e (m)	Mtotal (kN.m) = Mi + Ni.e
1	3.64	216.37	0.15	216.93
2	-3.91	30.43	0.14	29.89
3	-0.67	-2.41	0.14	-2.50
4	0.94	-0.71	0.15	-0.56
Total Moment				243.75

Table 4 Results obtained with loading in the beam 1 for the span of 20 m

Beam	N_i (kN)	M_i (kN.m)	e (m)	Mtotal (kN.m) = Mi + Ni.e
1	11.66	393.12	0.32	396.83
2	-11.96	103.17	0.29	99.68
3	-4.88	6.82	0.29	5.39
4	5.19	-9.81	0.32	-8.15
Total Moment				493.75

Table 5 Results obtained with loading in the beam 1 for the span of 30 m

Beam	N_i (kN)	M_i (kN.m)	e (m)	Mtotal (kN.m) = Mi + Ni.e
1	15.86	558.52	0.52	566.75
2	-16.11	177.84	0.48	170.05
3	-7.74	26.30	0.48	22.56
4	7.98	-19.39	0.52	-15.25
Total Moment			744.11	

Table 6 Results obtained with loading in the beam 1 for the span of 40 m

Beam	N_i (kN)	M_i (kN.m)	e (m)	Mtotal (kN.m) = Mi + Ni.e
1	17.88	712.98	0.86	728.30
2	-17.76	243.81	0.82	229.28
3	-9.14	51.20	0.82	43.72
4	9.01	-15.27	0.86	-7.55
	Total Moment			993.75

Table 7 Results obtained with loading on beam 1

Model	Total Bending Moment		Error (%)
	ANSYS	FTOOL	
10 m	243.75	250.00	2.50
20 m	493.75	500.00	1.25
30 m	744.11	750.00	0.78
40 m	993.75	1000.00	0.62

3.2. Application of the Model

After the verifications, modal and transient structural analysis were performed in order to obtain the values of displacements, shear forces and bending moments in the bridge.

With the modal analysis, the values of the natural frequencies for the first two modes of vibration were obtained, as shown in Table 8.

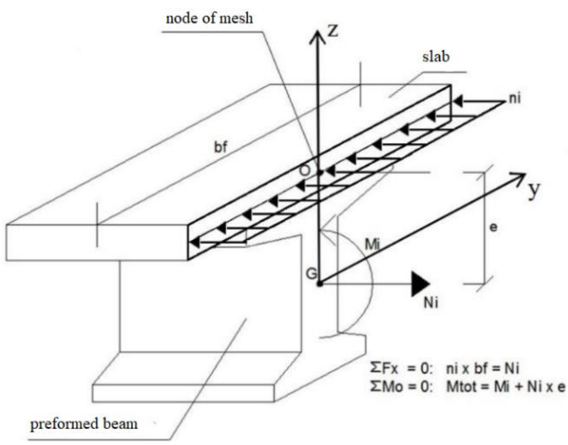

Fig. 7 Loading applied in section [11]

Table 8 Natural frequencies of structures

10 m span bridge		20 m span bridge		30 m span bridge		40 m span bridge	
Mode	Frequency (Hz)	Mode	Frequency (Hz)	Mode	Frequency (Hz)	Mode	Frequency (Hz)
1	9.178	1	5.455	1	3.602	1	2.516
2	12.842	2	5.720	2	3.678	2	2.800

Table 9 shows the parameters α and β used in the transient analysis of the structure, with a rate of damping 2%, for the modeling of the concrete bridges studied.

Table 9 Parameters α and β used in the transient analysis of the structure

Span (m)	Circular natural frequency of the mode 1 (Hz)	Circular natural frequency of the mode 2 (Hz)	Circular natural frequency of the mode 1 (rad/s)	Circular natural frequency of the mode 2 (rad/s)	α	β
10	9.178	12.842	57.667	80.689	1.34525082	0.00028911
20	5.455	5.720	34.274	35.941	0.70174992	0.00056968
30	3.602	3.678	22.634	23.108	0.45737298	0.00087447
40	2.516	2.800	15.807	17.595	0.33306377	0.00119753

Then, by means of static analysis and transient analysis, the maximum internal forces for each situation were determined, as shown in Table 10 to Table 13.

Table 10 Comparison between the dynamic effects due to moving load with the dynamic effects recommended by NBR 7188: 2013 [1], for 10m span bridge

Efforts	Static loading	Dynamic loading	DAF	Correction Factor - NBR 7188: 2013 [1]
Displacement (m)	3.99	4.39	1.10	
Shear effort (kN)	268.92	295.43	1.10	1.35
Bending moment (kN.m)	595.08	643.97	1.08	

Table 11 Comparison between the dynamic effects due to moving load with the dynamic effects recommended by NBR 7188: 2013 [1], for 20m span bridge

Efforts	Static loading	Dynamic loading	DAF	Correction Factor - NBR 7188: 2013 [1]
Displacement (m)	9.20	11.35	1.23	
Shear effort (kN)	292.47	307.32	1.05	1.30
Bending moment (kN.m)	1273.51	1541.98	1.21	

Table 12 Comparison between the dynamic effects due to moving load with the dynamic effects recommended by NBR 7188: 2013 [1], for 30m span bridge

Efforts	Static loading	Dynamic loading	DAF	Correction Factor - NBR 7188: 2013 [1]
Displacement (m)	10.83	14.44	1.33	
Shear effort (kN)	300.48	303.50	1.01	1.27
Bending moment (kN.m)	1920.13	2209.85	1.15	

Table 13 Comparison between the dynamic effects due to moving load with the dynamic effects recommended by NBR 7188: 2013 [1], for 40m span bridge

Efforts	Static loading	Dynamic loading	DAF	Correction Factor - NBR 7188: 2013 [1]
Displacement (m)	8.55	9.76	1.14	
Shear effort (kN)	304.76	304.54	1.00	1.24
Bending moment (kN.m)	2523.14	2822.48	1.12	

In Tables 10 to 13, it is possible to observe that only the passage of a transient load on the structure does not cause increase of the DAF in the middle of the span of the structure, but rather the passage of a transient harmonic load does, where it is able to better represent the vehicle-pavement-structure interaction.

The maximum displacement in the middle of the span calculated on the basis of the Brazilian code presented close results with the values obtained by the transient loads, demonstrating a good conversion of the static to the dynamic forces, except for the 30 m span case.

The maximum shear forces in the supports generated by the transient loads were shown to be lower than those calculated using the Brazilian code, presenting an exaggerated design for this region.

The maximum bending moments in the middle of the span calculated on the basis of the Brazilian code presented close results with the values obtained by the transient loads, demonstrating a good conversion from the static to the dynamic efforts.

4. Conclusion

The vehicle-pavement-structure interaction is directly related to the value of the load amplitude (Q), which takes into account the weight of the vehicle, the effect of the irregularity of the lane or even the overlap of both actions. For future studies, there is a need for a better understanding of the load amplitude value, in order to develop computational models with greater precision for the studied situations.

The results obtained for the beams showed that, for some cases, the impact coefficients had a good approximation to transform the static efforts into dynamic ones. However, in other cases, these coefficients did not show the same result.

In his studies, Rossigali [12] obtained similar results when comparing the internal forces caused by a real load of a Brazilian road structure to the forces obtained by the increase of the loads by the impact coefficient.

Through this study, it was possible to identify that the coefficients of impact recommended by the code can be improved from new studies taking into account the current loads of the Brazilian bridges.

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