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Autor: López, Angel / Aparicio, Angel C.

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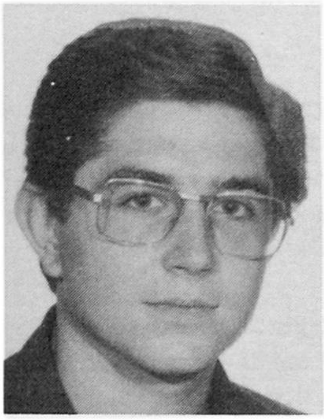
Nonlinear Behaviour of Curved Prestressed Box Girder Bridges

Comportement non-linéaire de ponts courbes en béton précontraint

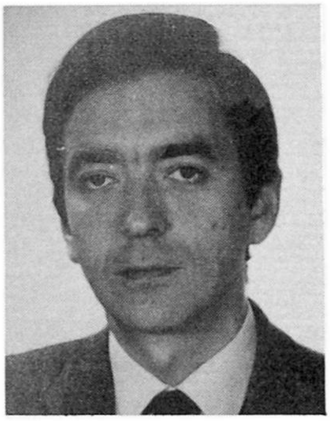
Nichtlineares Verhalten gekrümmter vorgespannter Betonhohlkastenbrücken

Angel LÓPEZ
Dr. Ingeniero de Caminos
Univ. Politécnica de Cataluña
Barcelona, Spain



Angel López, born 1959, received his civil engineering and doctorate degrees in 1982 and 1987, respectively, at Universidad Politécnica de Cataluña. His area of interest is in bridge design and nonlinear analysis of prestressed concrete bridges. He is Associate Professor in Escuela Técnica Superior of Barcelona.

Angel C. APARICIO
Dr. Ingeniero de Caminos
Univ. Politécnica de Cataluña
Barcelona, Spain



Angel C. Aparicio, born 1949, received his civil engineering degree in 1972 at the Universidad Politécnica de Madrid, and his doctorate degree in 1980 at Universidad de Santander. His area of interest is in bridge design and nonlinear analysis of prestressed concrete bridges. He is Professor in Escuela Técnica Superior of Barcelona.

SUMMARY

The goal of this work is to study the response of prestressed concrete curved bridges under increasing external loads. An overview of a mathematical model developed for nonlinear analysis of reinforced and prestressed concrete structures, is presented. This model considers interaction of the internal forces for bending, torsion and shear. A prestressed concrete curved bridge is studied with this model under different load cases. General conclusions on the nonlinear behaviour of the mentioned structures and several criteria for design are also given.

RÉSUMÉ

L'article traite du comportement, sous l'effet de charges croissantes, de ponts courbes en béton précontraint. Un modèle mathématique est développé pour l'analyse non linéaire des structures en béton armé et précontraint, qui permet de tenir compte de l'interaction des efforts de flexion, de torsion et de cisaillement. Un pont courbe en béton précontraint est étudié à l'aide de ce modèle pour différents cas de charge. Des conclusions générales sont tirées sur le comportement non linéaire de ces structures et des critères proposés pour son projet.

ZUSAMMENFASSUNG

Ziel dieser Arbeit ist die Untersuchung des Verhaltens gekrümmter vorgespannter Betonhohlkastenbrücken unter steigender Beanspruchung. Ein mathematisches Modell, welches zur nichtlinearen Analyse vorgespannter Stahlbetontragwerke entwickelt wurde, wird kurz erörtert. Dieses Modell erlaubt die Einbeziehung der Interaktion zwischen Biegung, Torsion und Querkraft in Bezug auf die Verformungen und den Tragwiderstand. Anhand dieses Modells werden verschiedene Belastungsfälle untersucht. Es werden allgemeine Schlüsse über das nichtlineare Verhalten gezogen und Kriterien für den Entwurf gegeben.

1.– INTRODUCTION

In recent years, the traffic requirements have increased the demand for bridges curved and skewed in plan in the highway system. This is even more common in densely populated areas. Very often, the longitudinal scheme of these bridges consists of a continuous beam torsionally clamped at both ends and supported over intermediate piers, with or without torsional restraint at the support sections. Box girder cellular sections and slabs are elegant solutions for the transversal section of the bridge. Both longitudinal schemes can offer a pleasant intrados, accomodate well to the highway layout and junctions, and present, from the resistance viewpoint, high flexural and torsional stiffness.

In curved bridges, coupling between bending and torsional moments differentiates the structural response from straight bridges. This means, from a practical point of view, that the bending moment and torque cannot be obtained separately; their magnitude depends, among other factors, on the ratio of flexural to torsional stiffness.

In curved prestressed concrete bridges over intermediate supports, significant torsional moments are developed even under dead load. Under service loads, very high torsional moments are generated by certain loading cases. Thus, the longitudinal prestressing designed to prevent flexural cracking might not be enough to avoid cracking due to torsion. Nevertheless, transverse prestressing may be designed to prevent cracking under service conditions; but, for increasing loads, cracking is unavoidable.

The results of several tests show that the ratio of flexural to torsional stiffness increases as cracking develops. Thus, the response of a curved prestressed concrete box girder bridge under increasing loads, will be clearly nonlinear when cracking appears. Moreover, the safety of the structure might be affected by the redistribution of internal forces.

It is obvious that a complete understanding of such complex structural behaviour requires an important effort from the experimental and theoretical points of view. In this paper, a mathematical model developed in [1] for the nonlinear analysis of reinforced and prestressed concrete curved box girder bridges is briefly described. This paper will focus mainly on the structural response of a curved prestressed box girder bridge, trying to draw general conclusions regarding the behaviour of such structures.

2.– DESCRIPTION OF ANALYTICAL MODEL

A brief description of the developed analytical model is presented herein. Further details, validity and discusions may be found in [1], [2] and [3].

The behaviour of box girder bridges can be reasonably simulated by means of the beam theory when the span/cell width ratio is greater than approximately five. In such range of span/width ratio, the influence of transverse distortion and warping of the cross-section on the longitudinal stresses may be, for usual prestressed concrete box girders, very small. Therefore, undeformability of sections in their plane, as well as Navier's hypothesis, can be assumed.

The analytical model developed can be described attending essentially to four aspects: structural idealization, material models used, sectional analysis, and structural analysis.

2.1.– Structural Idealization

The structure is divided longitudinally into a number of straight short elements interconnected by nodes. Inside each element, a short number of integration sections is considered. Loads must be applied only at the structural nodes. With this longitudinal scheme, a large number of joints is needed to idealize medium size structures. However, the longitudinal integration along the elements is extremely simple, making the global process of structural analysis probably faster than other methods.

Each cross-section is idealized by panels (Fig. 1.), which can resist only normal and shear stresses in their plane. The position and thickness of the panel, as well as the concrete cover allowed to spall, are known. Also, several reinforcing steel layers, with any orientation and a layer of transverse prestressing, can be defined at each panel.

Longitudinal prestressing tendons are individually defined, for which their steel area, position, and slopes at the integration sections with respect to the longitudinal axis of the bridge are given. Prestressing active effect is introduced by means of an initial strain in the prestressing steel.

2.2.– Material Models

Concrete in panels is assumed to be under a biaxial stress state. Before cracking, a biaxial isotropic elastic constitutive model is used, with variable modulus of elasticity. Given the panel strains (ε_x, ε_y and γ), the principal strains ε_1, ε_2, and their orientation at the middle plane are obtained by means of bi-dimensional elasticity equations.

Once the principal strains of the middle plane, as well as the curvatures induced by bending and torsion, are known, the principal strains throughout the panel thickness can be obtained. As a result, the stress ratio α is:

$$\alpha = \frac{\sigma_1}{\sigma_2} = \frac{\varepsilon_1 + \nu\varepsilon_2}{\varepsilon_2 + \nu\varepsilon_1} \quad (1)$$

where ν is the Poisson's ratio, assumed constant.

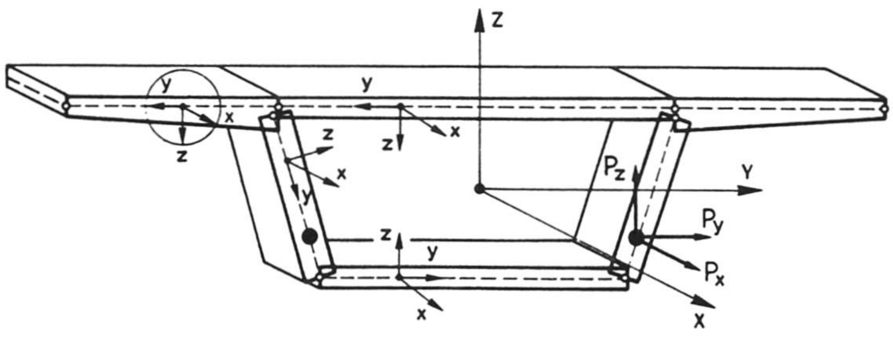


Fig. 1. Discretization of the cross-section in panels which only resist normal and shear stresses.

With this value, the ultimate stress and strain for the maximum compressive direction are determined, according to the Kupfer and Gerstle failure envelope [4]. An apparent elasticity modulus for concrete is obtained from Saenz's equation [5] which represents the concrete behaviour , namely,

$$E_a = \frac{\sigma_2}{\varepsilon_2} = \frac{E_{co}}{1 + \left(\frac{E_{co}\varepsilon_{2p}}{\sigma_{2p}} - 2\right)\left(\frac{\varepsilon_2}{\varepsilon_{2p}}\right) + \left(\frac{\varepsilon_2}{\varepsilon_{2p}}\right)^2} \tag{2}$$

hence the principal stresses are obtained as

$$\sigma_1 = \frac{E_a}{1-\nu^2}(\varepsilon_1 + \nu\varepsilon_2) \qquad \sigma_2 = \frac{E_a}{1-\nu^2}(\varepsilon_2 + \nu\varepsilon_1) \tag{3}$$

For cracked concrete, the evolutive truss analogy with peak stress reduction, according to Vecchio and Collins [6], is adopted to represent the panel behaviour. In this case, the ultimate stress and strain for the maximum compressive direction are determined by the following equations :

$$\sigma_{2p} = \frac{f_c}{\lambda} \qquad \varepsilon_{2p} = \frac{\varepsilon_{co}}{\lambda} \tag{4}$$

where,

$$\lambda = \sqrt{\frac{\gamma_m}{\varepsilon_2} - 0.3} \qquad \gamma_m = \varepsilon_1 - \varepsilon_2$$

The stress in the direction orthogonal to the cracks is assumed to be zero.

Reinforcing and prestressing steel are assumed to be subjected to a uniaxial longitudinal stress state, with a multilinear stress-strain relationship.

2.3.– Sectional analysis

Assuming Navier's hypothesis and undeformability of cross-section in its plane, the strain field is completely determined by six sectional deformations (i.e. axial deformation, shear distorsions, twist and curvatures). On the other hand, the stress field is also determined by six internal forces (i.e. axial force, shears, torsion and bendings).

The section behaviour is governed by three sets of fundamental relationships: *compatibility* between panel strains and sectional deformations; *constitutive relationships* of the materials, and *equilibrium* between internal forces and stresses over the panels and tendons.

It is not possible to obtain an explicit constitutive relationship between sectional deformations and internal forces; thus, to solve the sectional analysis, the procedure to be followed is:

- The strains at each point of the section are obtained by means of geometrical relationships from the six general sectional deformations, assuming Navier's hypothesis, and undeformability of cross-sections in their plane;

- stresses are obtained, once the strains are known, by means of the constitutive relationships for the materials (2.2);

- internal forces are deduced by integration of stresses over the panels and from the stresses of the prestressing tendons.

It should be noticed that this model does not separate the torsional, flexural, axial and shear behaviour, making it possible to predict the sectional response under combined actions.

2.4.– Structural analysis

The developed method of nonlinear structural analysis is based on the classical matrix analysis of structures formed of bars, extended to take into account material nonlinearities on reinforced and prestressed concrete structures.

The derivation of the member stiffness matrix is made by considering the three basic equations at the element level, expressed in global coordinates:

- Incremental equilibrium equations:

$$\vec{\triangle}\sigma(s) = \underset{\sim}{N}(s) \cdot \vec{\triangle}F_2 \tag{5}$$

where $\vec{\triangle}\sigma(s)$ is the vector of internal forces at any section of the member, $\underset{\sim}{N}(s)$ is the equilibrium matrix, depending on the beam geometry, and $\vec{\triangle}F_2$ is the vector of nodal forces at one of the member ends.

- Incremental kinematics equations:

$$\vec{\triangle}(\delta_2 - \delta_1) = \int_1^2 \underset{\sim}{N}^T(s) \cdot \vec{\triangle}\varepsilon(s)\; ds \tag{6}$$

where $\vec{\triangle}(\delta_2 - \delta_1)$ is the vector of relative displacements between both member ends, and $\vec{\triangle}\varepsilon(s)$ is the vector of sectional deformations.

- Constitutive sectional relationship following the procedure described in Section 2.3.

Combining the above equations, the general member force-displacement incremental relationship is obtained:

$$\vec{\triangle}F = \underset{\sim}{K}\vec{\triangle}\delta \tag{7}$$

where $\vec{\triangle}F$ is the vector of nodal forces at both member ends, $\underset{\sim}{K}$ is the member tangent stiffness matrix. $\vec{\triangle}\delta$ is the vector of member end displacements.

Assembling the elementary equations for the complete structure, a similar global expression is obtained, which relates nodal applied loads and nodal displacements.

The proposed analysis model is numerically solved by using an incremental iterative method. It follows a modified Newton-Raphson scheme, and allows, at each iteration, to apply only a fraction of the residual forces; see for instance Nayak and Zienkiewicz [7].

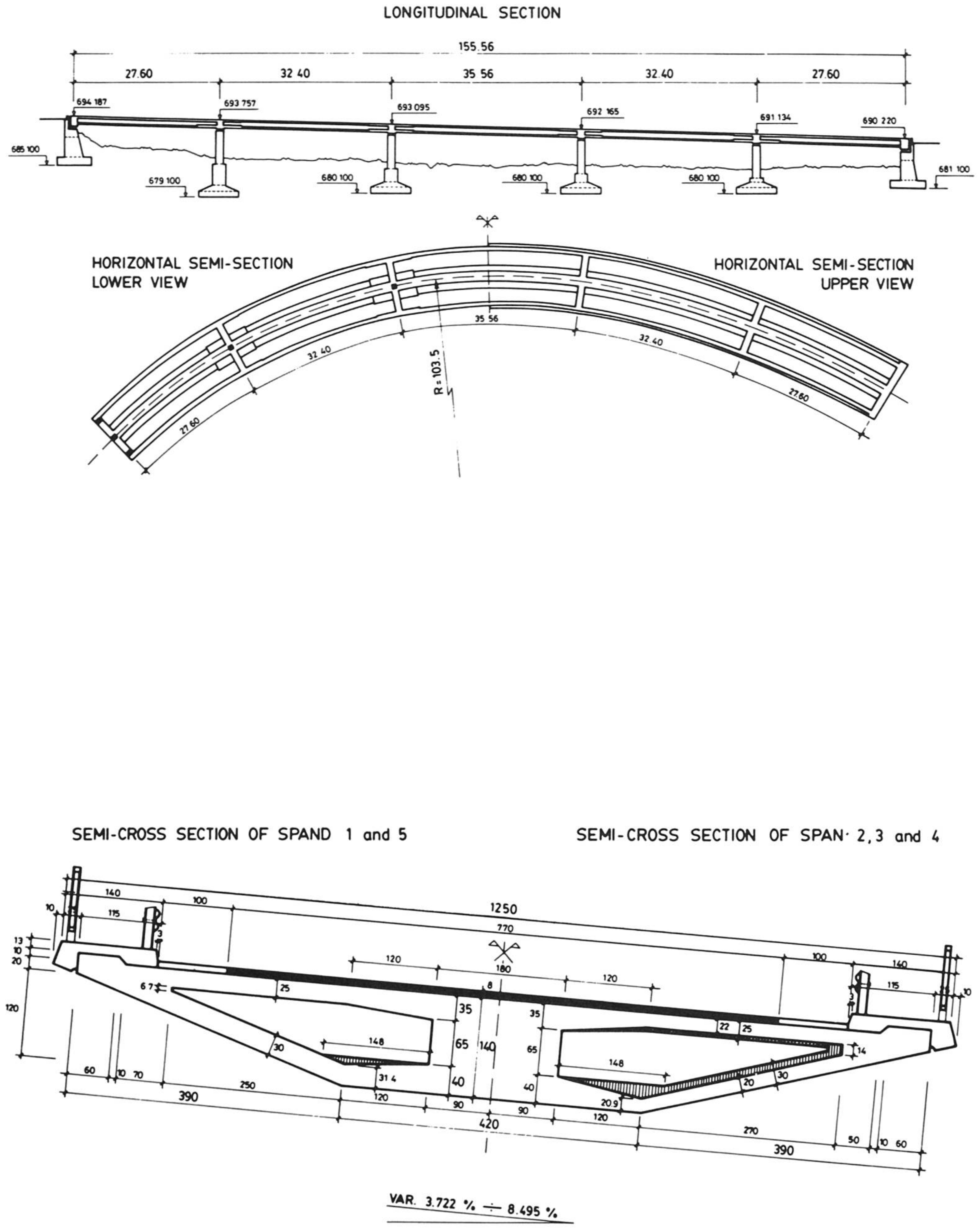


Fig. 2. Plan view and generic cross-section of the "Puente I".

3.– STRUCTURAL BEHAVIOUR OF A CURVED PRESTRESSED BOX GIRDER BRIDGE

3.1.– Description of the Bridge Analyzed

The structure chosen for the present study is the curved prestressed concrete bridge called "Puente I", at the Santamarca junction in La Paz Highway, in Madrid (Spain), and designed by Arenas and Aparicio [8]. The geometry is curved in plan, with a radius of 103.5 m (Fig. 2a.), and the bridge superstructure consists of a five span continuous beam (27.60 + 32.40 + 35.56 + 32.40 + 27.60 m) supported by single intermediate piers.

The cross-section is a double trapezoidal box (Fig. 2b.) with constant depth of 1.40 m, and 12.50 m wide. The longitudinal prestressing tendons are located in the 1.80 m wide central web designed to resist shear loads. Finally, two inclined slabs ("struts") close the box and give it torsional stiffness.

The longitudinal prestressing is composed of three sets of tendons. The first one, anchored at both ends of the superstructure, consists of six Freyssinet 37T15 tendons; the second set covers the central span and contiguous piers and is composed of four Freyssinet 12T15; while ten Freyssinet 12ϕ7 tendons are located in the upper slab over the supporting piers.

Stronghold 14ϕ7 tendons are transversally distributed, 100 cm apart, in the "struts" in the end spans. There is also transversal prestressing in the upper slab, composed of Stronghold 14ϕ7 tendons 120 cm apart along the whole bridge length.

3.2.– Results of the Analysis

Under dead load, the torsional moment T at the abutments, the bending moments M_1 and M_2 at the pier sections, and the bending moment M_c in the middle of the central span, are shown in the following table, in kNm units:

	T	M_1	M_2	M_c
HP	4210	7990	5770	5840
SW	1410	-16190	-18500	9940
DL	440	-3680	-4260	2240
HP+SW+DL	6060	-11880	-16990	18020

where
- HP: Hyperstatic due to Prestressing.
- SW: Self–Weight.
- DL: Superimposed Dead Load.

In order to simulate the response of the present curved bridge up to failure, successive increments of a 400 kg/m^2 Live Load (LL) are applied. Three loading cases have been considered, and they correspond to the following:

LL400e: live load extended over the exterior half width along the whole bridge length,

LL400c: live load extended over the whole bridge length and width,

LL400i: live load extended over the interior half width along the whole bridge length.

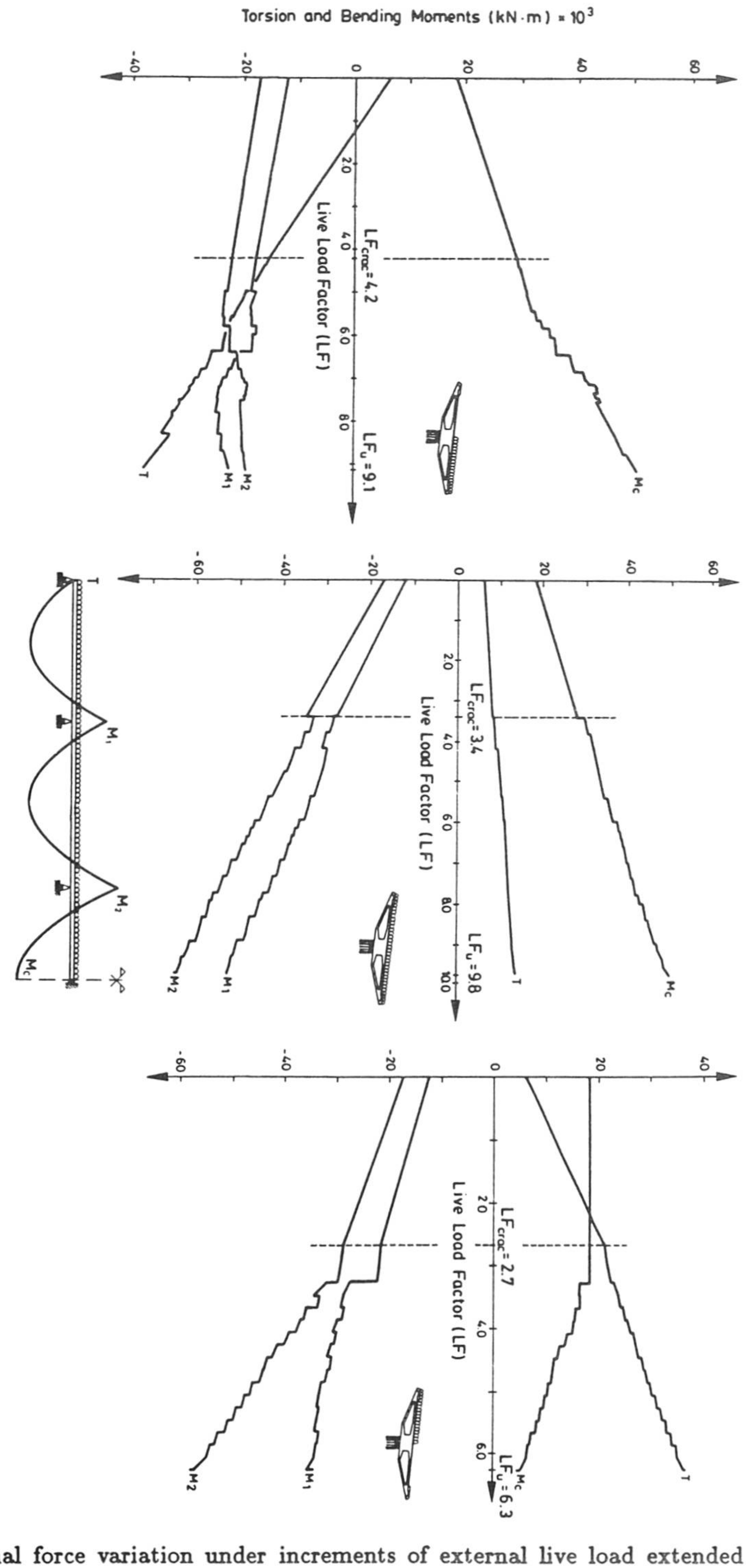


Fig. 3. Internal force variation under increments of external live load extended over the exterior/centered/interior half width along the whole bridge length.

The total live load is scaled by the increment previously mentioned, i.e. 400 kg/m^2, and the **Live Load Factor** (LF) stands for the number of times this increment is applied.

3.2.1.– Eccentric exterior live load (LL400e) (Fig. 3a.)

The characteristic value (LF=1) for the uniform live load over the whole bridge length and exterior half width, induces the following increments in the internal forces:

	T	M_1	M_2	M_c
LL400e	5660	-3180	-4000	170

The live load must be increased up to LF=3.3 in order to produce initial cracking. Near the abutments (6 m) and due to the torque value, cracks are fully developed. Cracking also evolves, in the upper slabs, on the support cross-section of pier #2. This is due to the increasing negative bending moments generated by a decrease in the torsional rigidity of the extreme span.

At LF=4.1, the combined bending and torsional forces acting on the support cross-sections of pier #1, produce cracking of the upper slab and the struts. At this load level, the structure response is clearly nonlinear both in displacements and internal forces; for instance, the bending moment over pier #1 is -27700 kNm, 11% larger than the linear predicted value, namely -24920 kNm.

Cracking spreads progressively under increasing live loads. For instance, at LF=5.1, cracking extends over half of the extreme span. Obviously, the affected areas over supports are also enlarged. At this load level, cracking due to torsional moments, appears in the second span at the cross-sections without transversal prestressing in the lower slab and struts.

Increasing live loads cause the fast propagation of cracks over almost the entire deck. However, at LF=6.3, failure is imminent at the cross-sections near abutments: concrete at the struts reaches ultimate strain under crushing, and the lateral reinforcement of the central web suffers excessive plastic elongation.

The calculated internal forces at ultimate loading are:

	T	M_1	M_2	M_c
HP+SW+DL+6.3LL_e	36210	-33350	-56370	4910
Linear analysis	41720	-31910	-42190	19090
Nonlinear/linear	0.87	1.05	1.34	0.26

Note that the structural response is highly nonlinear; for example, the central span bending moment increases by 14180 kNm with respect to the linear analysis. The redistribution of internal forces is mainly due to the decrease in torsional rigidity of the structure.

3.2.2.– Centered live load (LL400c) (Fig. 3b.)

The internal force increments computed for the characteristic value (LF=1) when the uniform live load extends over the whole bridge length and width, are

	T	M_1	M_2	M_c
LL400c	530	-4530	-5150	2770

It is important to notice that the obtained torque is small compared to the bending moments. Therefore, for this particular load case, the behaviour of the structure will be similar to that of a straight bridge. In any case, the response is linear until cracking appears; this occurs at LF=3.4 and it is due to the negative bending moments generated at the support cross-section of pier #2.

At LF=3.8, cracking appears on the support cross-section of pier #1 and on the halfway section of the central span. The cracking progression, similar to the classical response observed in a straight bridge, stabilizes at LF=5.4. Under this external load, the bending moment over pier #1 is -32980 kNm, 9% smaller than that obtained using a linear analysis, i.e. -36340 kNm.

Increasing live loads cause the propagation of cracks in the areas where large bending moments are developed, namely, at the center of the span and on the sections over intermediate piers. From a theoretical point of view, failure occurs at LF=9.8; it is due to large elongation of the longitudinal prestressing tendons at the cross-section over pier #2. It should be noted that at this particular cross-section, the transverse reinforcement and the concrete of the central web, show failure signs as well.

The computed forces under ultimate load are:

	T	M_1	M_2	M_c
HP+SW+DL+9.8LL_c	13710	-52100	-64110	48520
Linear analysis	11250	-56270	-67460	45170
Nonlinear/linear	1.22	0.93	0.95	1.07

As shown in this table, the response of the structure is clearly nonlinear. However, the nonlinearity is less pronounced than in the case of LL400 exterior. Furthermore, the redistribution of internal forces is of opposite sign to the one obtained in the preceeding load case (i.e. LL400e); following the usual response pattern of straight bridges, the bending moments of the central span decrease by a constant value, 3350 kNm, with respect to the linear analysis. On the other hand, the large increase experienced by the torque at the cross-section over the abutment, is an unexpected result.

3.2.3.– Eccentric interior live load (LL400i) (Fig. 3c.)

The characteristic value (LF=1) for the uniform live load over the whole bridge length and interior half width, induces the following increments in the internal forces:

	T	M_1	M_2	M_c
LL400i	-5000	-1320	-1090	2680

Until cracking is reached, at LF=3.2, on the central web of the cross-section over pier #1, the structural response is linear. At LF=5.0, the first important cracks appear on the lower slab of the cross-section halfway in the central span and along the struts over pier #1. Upon increasing the load, cracking due to torsion develops in the cross-sections near the abutments, as well as in the first sections of the second span where transversal prestressing is missing.

Cracking of the deck is completed under successive increments of the external load, and failure conditions are attained at LF=9.1. Failure is due to concrete crushing at the struts and plastification of the transversal reinforcement of the central web, at the cross-sections in the vicinity of the abutment.

The computed internal forces under ultimate load are:

	T	M_1	M_2	M_c
HP+SW+DL+9.1LL_i	-37180	-22240	-19010	50370
Linear analysis	-39440	-23890	-26910	42410
Nonlinear/linear	0.94	0.94	0.71	1.19

The previous table shows that the structural response is highly nonlinear. The bending moment at the central span, for instance, decreases by a constant value of 7960 kNm with respect to the linear analysis. However, the force redistribution is less important than for the live load on the exterior half width of the bridge, as shown in section 3.2.1..

3.2.4.– Summary of results at failure

As shown in the preceding sections, one can conclude that the deck response after cracking is nonlinear under any loading case; nevertheless this nonlinear behaviour depends strongly on each particular load case. This is clearly observed in Figure 4. and the following table which summarizes, for each loading case, the ultimate internal forces at failure loading.

	T	M_1	M_2	M_c	NL/L (M_2)	LF (failure)	Failure Mode
$LL400_e$	36210	-33350	-56370	4910	1.34	6.3	Torsional
$LL400_c$	13710	-52100	-64110	48520	0.95	9.8	Flexural
$LL400_i$	-37180	-22240	-19010	50370	0.71	9.1	Torsional

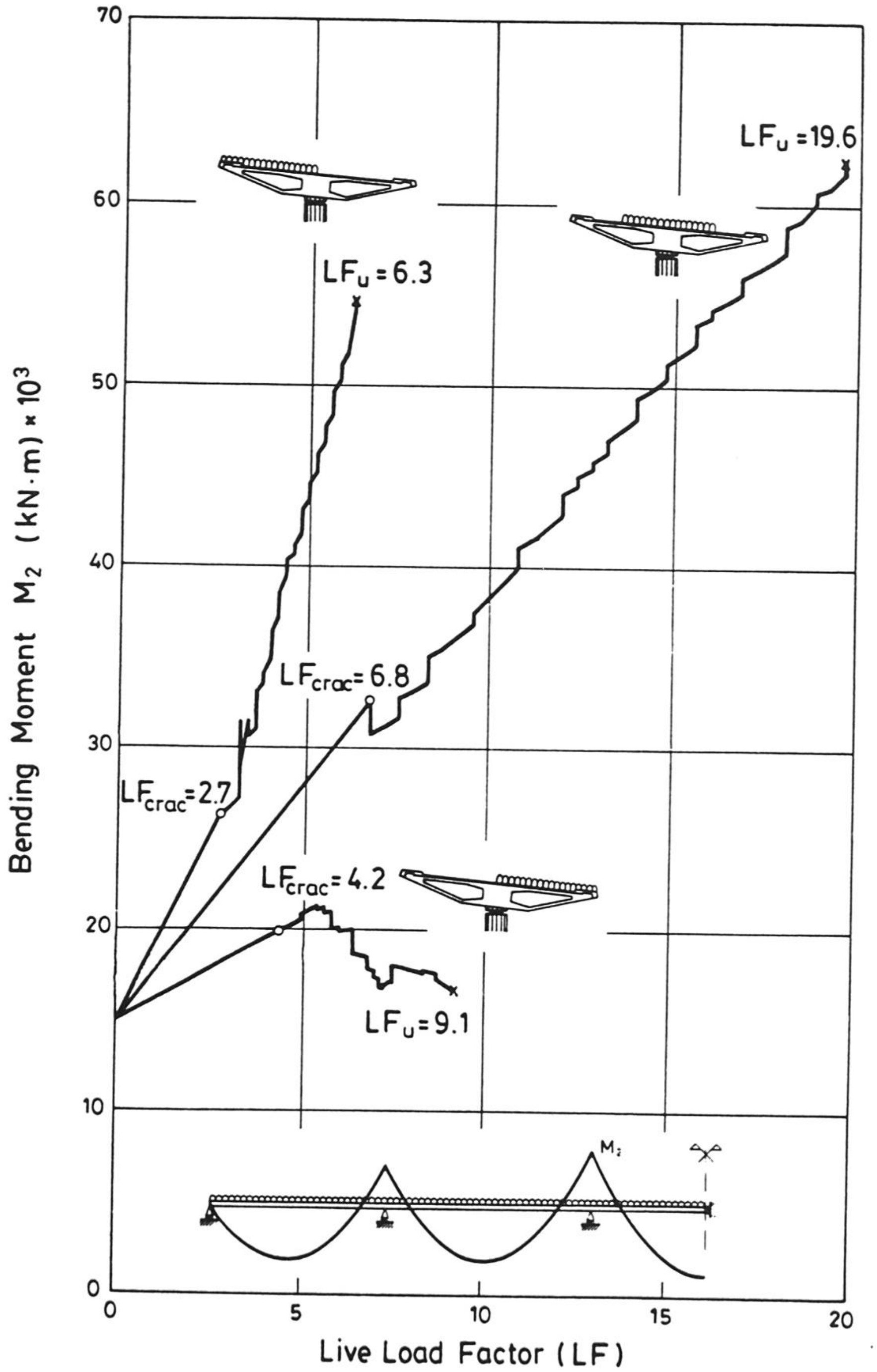


Fig. 4. Variation of the bending moment at pier #2 for each loading case.

3.3.– Substitution of Transversal Prestressing by Reinforcing Steel: Influence on the Structural Response

Transversal prestressing is usually an expensive and inconvenient solution, especially when applied to bridges with low width/span ratio. This technique tends to be avoided because of the large proportion of anchors with respect to kg of prestressed steel, and the large number of jacking and grouting operations. However, if no cracking is desired on the deck, this technique may become mandatory.

In this section, "Puente I" is studied under the hypothesis of transversal passive reinforcement instead of transversal prestressing, and the consequences of this decision are analyzed.

The analysis presented in section 3.2.1. is repeated, i.e. uniform increments of live load (400 kg/m^2) over the whole bridge length and exterior half width; but the transversal prestressing is substituted by a mechanical equivalent amount of passive reinforcement.

As a marginal result of this new study, the hyperstatic forces due to transversal prestressing, in the analysis performed in section 3.2.1., can be deduced after comparisons of both results, namely

	T	M_1	M_2	M_c
HP_t	360	690	490	500
HP_t/HP	8.6	8.6	8.5	8.6
$HP_t/(HP+SW+DL)$	5.9%	-5.8%	-2.9%	2.8%

where
HP_t: Hyperstatic due to Transversal Prestressing.

The preceding table proves that these forces must, at least, be considered in order to verify the service limit state. The importance of this phenomenon, due to the longitudinal deformations caused by the Poisson effect, will increase as the uniformity of the transversal prestressing at the section decreases; for instance, when only the top of the deck is transversally prestressed for resisting transversal bending.

Other conclusions deduced from the results (Fig. 5.) are:

- As expected, cracking appears under smaller loads, at LF=1.8. It is important to notice that this analysis assumes tensile strength of concrete. Therefore, the actual decompression load is lower than computed.
- The torsional rigidity of the cracked sections is lower than that computed for "Puente I" with transversal prestressing; and when live load increases the internal force redistribution evolves as a result.
- Failure is attained at LF=6.4 and it is due to concrete crushing at the struts on the cross-sections in the vicinity of the abutments.

– The computed moments at failure are:

	T	M_1	M_2	M_c	NL/L (M_2)	LF
Prestressing	36210	-33350	-56370	4910	1.34	6.3
Reinforcing	35500	-31480	-57650	3210	1.34	6.4

In summary, one can conclude from the preceding analysis that the substitution of transversal prestressing by a mechanical equivalent passive reinforcement, does not diminish the ultimate strength of the structure. However, due to the fact that cracking occurs under a lower live load and redistribution is more pronounced, it may be possible to obtain an ultimate failure load larger than with the one obtained with transversal prestressing.

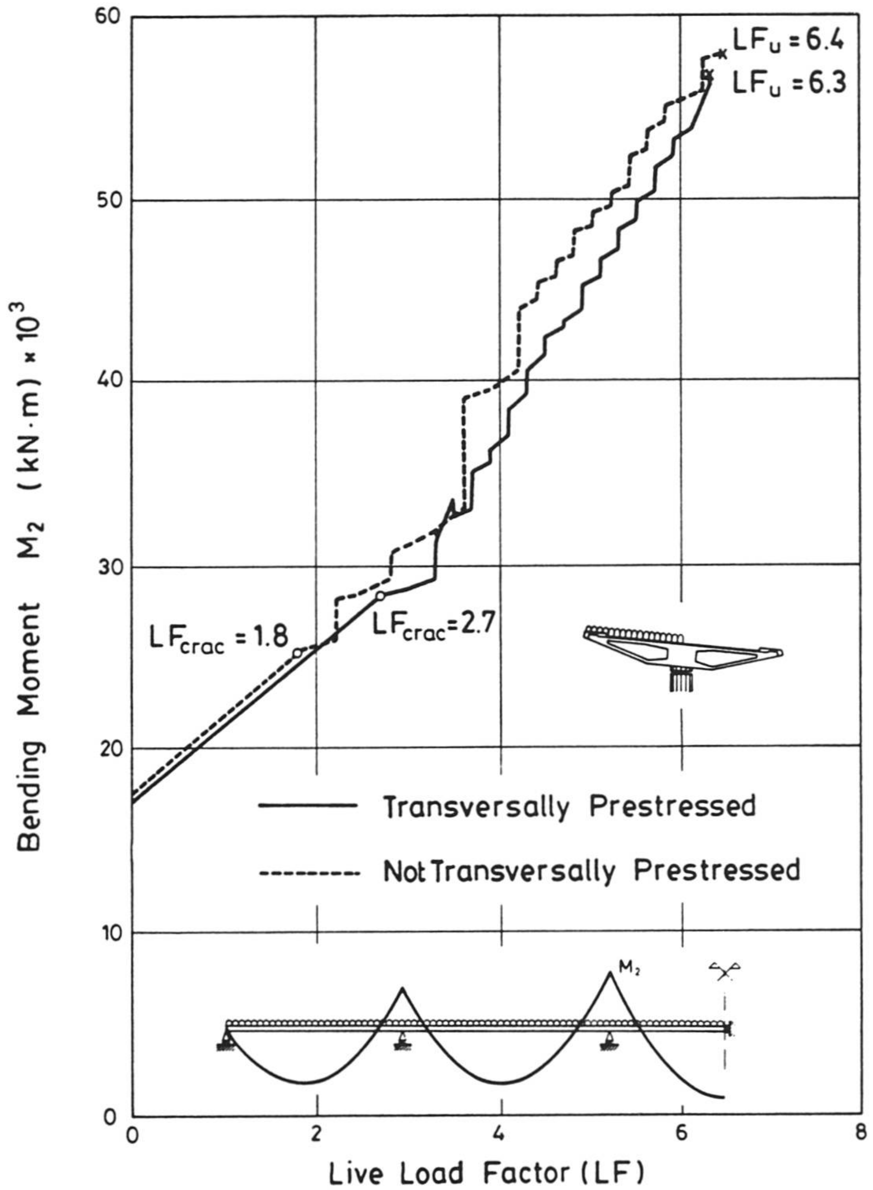


Fig. 5. Variation of the bending moment at pier #2 when transversal prestressing is substituted by a mechanical equivalent amount of passive reinforcement.

4.– CONCLUSIONS

Among the 51 conclusions discussed in [1], seven of them should be emphasized here:

1.– The structural response under increasing load of curved prestressed box girder bridges, supported over intermediate piers without torsional restrains, is highly nonlinear after cracking appears.

2.– The form and degree of redistribution of internal forces depends on the loading case considered. Thus, variations of the bending moment over supports of -29% (interior live load case) and +34% (exterior live load case), with respect to the linear analysis, are obtained for eccentric live load cases at ultimate failure conditions. However, for centered load, the redistribution is only -5%, similar to the redistribution obtained for a straight bridge.

3.– Due to progressive cracking and structural coupling between bending and torsion, internal forces are redistributed. Moreover, their interaction from a strength point of view, modifies the ultimate internal forces response.

4.– Hence, the type of failure depends on the loading case considered: for instance, a centered live load produces a bending failure on the cross-sections located over the support piers; while exterior and interior load cases induce failure essentially due to torsion at the bridge ends.

5.– Ultimate internal forces response can be evaluated accurately with plastic sectional analysis methods [9], if the interaction between bending and torsion is taken into account. Nevertheless, the use of the plastic structural analysis methods [10] in order to obtain the internal forces assumes that total redistribution can occur, which is not always the case.

6.– Significant variations in the post-cracking response are produced by transversal prestressing of the top and bottom slabs. However, if the transversal steel mechanical ratio remains constant, there is no noticeable influence on the ultimate load capacity.

7.– The following criteria are proposed for the design of curved prestressed box girder bridges:

7.1.– In order to guarantee a linear behaviour of the structure, the probability of cracking due to bending and torsion, under service loads, must be reduced. In this case, the response of the structure can be simulated accurately with linear models. Consequently, a total longitudinal prestressing and, may be, transversal prestressing should be designed.

7.2.– On the other hand, when extensive cracking is allowed, if internal forces are computed using a linear elastic analysis and if the ultimate internal forces response is evaluated without any interaction between bending, torsion and shear, the ultimate limit state condition is poorly determined and sometimes unsafe. A realistic analysis can only be achieved by a nonlinear structural analysis where interaction between internal forces is taken into account.

7.3.– A simplified method to obtain an approximate security level is: (a) Given a load case, internal forces (S_d) are computed using a linear elastic analysis where flexural and torsional rigidities are evaluated from the cracked sections [11]. (b) To obtain the ultimate internal forces response (R_d) from a plastic sectional analysis which takes into account interaction at failure. (c) The ultimate state condition is verified when $S_d \leq R_d$.

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