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System Performance of Two Prestressed Concrete Bridges

Comportement de deux ponts en béton précontraint

Verhalten von zwei vorgespannten Betonbrücken

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SUMMARY

A non-linear analysis of the behaviour of two alternate bridge designs with the same simple span length of 25 m is performed. The first configuration has 11 parallel prestressed I-beams. The alternate design consists of three spread box beams. The two configurations are designed to satisfy the same Spanish bridge code requirements. Results of the deterministic non-linear analysis are used to calculate member and system reliability indices for both bridges. Redundancy indexes are calculated to compare the system safety performance to the member performance of each bridge configuration.

RÉSUMÉ

On analyse le comportement non linéaire de deux ponts isostatiques de 25 m de portée. Le premier pont est formé de 11 poutres en I en béton précontraint, l'autre par trois poutres-caissons, les deux ayant été calculés pour satisfaire aux conditions des normes espagnoles. Les résultats de l'analyse non linéaire servent à calculer les indices de sécurité des éléments et de l'ensemble des deux ponts. On calcule les indices de "redondance" qui permettent la comparaison entre le comportement et la sécurité de chaque système et celui de ses éléments.

ZUSAMMENFASSUNG

Inhalt des Artikels ist eine nichtlineare Untersuchung des Verhaltens von zwei verschiedenen Brückenkonstruktionen mit der gleichen Spannlänge über 25 m. Die erste Brükkenkonstruktion enthält 11 parallel vorgespannte I-Balken. Die andere Konstellation besteht aus drei gespreizten Hohlkastenträger. Beide Entwürfe sind nach den spanischen Richtlinien für Brücken konstruiert. Die Ergebnisse der deterministischen, nichtlinearen Untersuchung werden verwendet, um Zuverlässigkeitsindizes beider Brücken zu bestimmen. Redundanzindizes vergleichen die Systemsicherheitsleistungen mit den Leistungen einzelner Brückenelemente.

1. INTRODUCTION

Prestressed concrete I-girder bridges have had a long history of excellent performance under regular truck traffic loads as well as overloads. In addition, I-girder bridges are economical and easy to construct and are known to have high levels of system reserve strength and redundancy. On the other hand, the use of spread box girder bridges is becoming more common because, in addition to their good structural performance, they are more aesthetically pleasing than I-girder bridges. For this reason, the spanish bridge authorities have abandoned the use of parallel prestressed concrete I-girder bridges in favor of spread box beam bridges for all overcrossings in new highway constructions.

The object of this paper is to compare the structural performance and the safety levels of these two bridge types. To perform such a comparison, two example bridges with the same span length of 25 m are designed to satisfy the same spanish code requirements. Figures 1 and 2 describe the two alternate bridge configurations.

2. LIMIT STATES AND ANALYSIS PROCEDURE

Although widely used in design codes, it is generally accepted that the linear analysis of a bridge structure is not sufficient to verify the safety and the functionality of a bridge structure: It is also important to verify the adequacy of its ultimate capacity for intact as well as damaged conditions and to verify its system serviceability. For concrete bridges, ultimate capacity is reached when a mechanism forms or when unloading begins due to excessive concrete damage. It has been observed that unloading begins at a load level close to the point when concrete crushing in a main longitudinal member occurs [1,2]. Therefore, this point is used herein to define system failure. It appears that the most suitable serviceability limit state for bridges is a deflection to span length ratio limit [3]. A displacement limit equal to the span length/200 was proposed in reference [2] and is used herein as a system serviceability limit state. In addition, the capacity of the structure to carry load after the complete damage or ductile failure of a main load carrying member should also be performed.

A nonlinear bridge analysis program NONBAN developed in references [2] and [4] is used for the analysis of the two alternate bridge configurations for the four limit states described above. The program uses a grillage (grid) discretization of a bridge structure to model its linear and nonlinear behavior in the longitudinal and transverse directions including the effect of the slab and diaphragms.

Flexural nonlinearity occurs when the loads applied on a beam element are incremented until the internal moments exceed their linear elastic limits. At this point the beam undergoes plastic end rotations. These plastic end rotations are related to the section curvature and the plastic hinge length. To perform the nonlinear analysis using NONBAN, moment versus curvature relationships are obtained using the basic principles of equilibrium. Estimates of the plastic hinge length for are obtained from empirical equations such as the ones proposed by Park and Pauley [5].

3. RESULTS OF DETERMINISTIC ANALYSIS

To perform the nonlinear analysis, the two bridge systems described above are discretized as grillages as outlined by Hambly [6]. Reference [4] demonstrated that this scheme was sufficiently accurate to study the global linear and nonlinear behavior of typical I-beam as well as box beam bridge configurations. The nonlinear incremental analysis is performed under the effect of two side-by-side trucks with a minimum lateral distance of 1.2 m between the axles of the adjacent trucks. Each of the applied trucks is assumed to have the configuration of a typical 5 axle semi-trailer observed over the spanish road network with an average total weight equal to 407 kN. Both bridges are assumed to have elastomeric supports at the ends of every longitudinal beam such that vertical displacements are restrained but all end rotations are free. For the box beams this assumes a single support at the center of each end section under the existing diaphragm.

In a first step, a linear elastic analysis of the two structures is performed. First member failure is assumed to occur when the most heavily loaded longitudinal member reaches its maximum capacity. The concrete stress-strain relationships used for the derivation of the nonlinear material properties assume that concrete crushes when the compression strain is equal to 0.0035. It is found that first longitudinal member failure would occur in the 11-girder bridge when the weights of the two side-by-side trucks are incremented by a load factor of 8.98. For the three-box girder bridge, this value is 8.53. For both bridges, the most critical loading position was obtained when the two trucks are transversely placed on the extreme edge of the bridge and the rear tridem axle placed at the middle of the span. In both cases, failure occurred at the midpoint of the external longitudinal girder.

In a second stage, a complete nonlinear analysis is performed for the same trucks and loading positions described above. For the 11-girder bridge, system failure occurred at a load factor of 10.40. In this case, failure occurred when the exterior girder under the load reached its maximum plastic rotation at the middle of the span. For the three-box girder bridge, system failure occurred at a load factor of 9.18. At this load factor, failure occurred transversely in the slab section joining the web of the exterior box girder and the middle box girder. Because of the known limitations of the grid model to simulate the actual behavior of the deck slab, another analysis is performed assuming that the slab will continue to carry loads beyond the level at which the strain in the transverse slab elements reaches the value of 0.0035. In this case, system failure is assumed to occur at the point at which concrete crushes in a main longitudinal member. This level was reached at a load factor equal to 10.92 for the three-cell bridge.

System serviceability limit, i.e. when girder deflection equals span length/200, is reached at a load factor equal to 9.03 for the 11 girder bridge and at a load factor of 9.17 for the box girder bridge. Figure 3 shows a comparison between the load factor versus maximum displacement curve for the two bridge configurations analyzed.

To study the effect of the boundary conditions of the box-girder bridge, two elastomeric supports are placed at the beams' end sections under each web to restrain its torsional rotation. First member failure assuming linear elastic response occurs in the external member at a load factor equal to 9.08. Accounting for the nonlinear behavior, slab crushing in the transverse direction occurs at a load factor equal to 10.41. If the nonlinear analysis is continued beyond this load level then the load factor at system failure is equal to 12.67 when the exterior longitudinal member crushes. The serviceability displacement limit is reached at a load factor equal to 10.43.

Damaged conditions assume that the external member of the 11-girder bridge is completely damaged. The nonlinear analysis shows that failure of the slab occurs at a load factor of 1.91. If the slab is assumed to be able to sustain this load, crushing in a main longitudinal member would then occur at a load factor of 6.92. Damage to the three-box girder bridge is simulated by assuming complete damage of the external web of the external girder. For the box girder with only one support at each section end, the slab would fail under the effect of the dead load alone. If loading is continued assuming that slab failure is only a local failure, then system failure would occur at a load factor of 5.67. For the box girder bridge with supports under each web, the load factor at which transverse slab failure occurs is 1.3 and the load factor at which longitudinal member failure occurs is 6.21. The load factors calculated for the damaged box girder bridge are lower than those observed for the 11-girder bridge because damage to one web will not only reduce the external box's moment capacity by one half but will also reduce the torsional rigidity of the member to a practically negligible level. The results of all the analyses performed herein are summarized in table 1.

4. RELIABILITY AND SYSTEM SAFETY

To account for bridge strength and load uncertainties, a reliability analysis is performed. The safety margin Z for a bridge system is defined as:

$$Z = (R - D) - (L + I) = LF - LL \tag{1}$$

where the incremental load factor, LF, is equal to the resistance R minus the dead load D (LF=R-D). LF determines the capacity of a bridge system to support the applied live load. LL is the maximum applied live load effect expected in a given return period. LL depends on the static truck load effects L and the dynamic effect I. Since in this study the calculation of the bridge system capacity is presented as a function of the typical average truck configuration, both LF and LL are herein normalized with respect to the effect of that truck. Both LF and LL are random variables. LF is random due to the uncertainties in determining the system resistance R and the dead load D. A deterministic estimate of LF for each of the four limit states previously identified is obtained using the nonlinear analysis program as seen in section 3. LL is random due to the uncertainties associated with predicting the maximum expected load in a given return period. It is a function of the number of trucks that cross the bridge during the return period, the number of trucks that are simultaneously on the bridge when the maximum load effect is measured, the positions of the trucks on the bridge deck, the weights of the trucks, the distribution of the weights to the individual axles and the axle configurations. In addition, the load effect is a function of the dynamic impact. All these factors are random and produce high levels of uncertainty. Reference [7] provides a simple truck live load model which has been proven to be valid for the spanish truck traffic [8]. The model gives the maximum expected lifetime load effect as a function of a typical truck configuration with a characteristic weight. The total truck load effect L+I is given as:

$$L + I = amW_{95}Hi \tag{2}$$

where "a" is the effect of a representative truck configuration with a one unit total load. "m" is factor representing the variation of the random trucks from the configuration of the representative truck. W_{95} is the chracteristic 95 percentile value of the truck weight histogram for a given jurisdiction. H is the headway factor representing the number of representative trucks of weight W_{95} that will produce the same effect as the maximum expected lifetime load effect. "i" is the impact factor. When normalized with respect to the effect of the typical spanish semi-trailer truck equation 2 becomes:

$$LL = \frac{mW_{95}Hi}{407kN} \tag{3}$$

Since the analysis performed in the previous section uses an average truck configuration "m" is herein assumed to be equal to 1.0 and its coefficient of variation (COV) is assumed to be equal to 8 % [7]. W_{95} for typical spanish truck gross weights is given as 559 kN [9]. A COV of 10 % is used to account for the uncertainty and site to site variability of W_{95} [7]. The dynamic impact factor for two trucks is shown to be about 1.10 with a COV of 8 % [10].

H is a function of the return period. Two different loading conditions are identified: Extreme loading condition and regular truck traffic condition. Extreme loading condition is defined as the maximum expected lifetime load. The expected bridge lifespan is usually around 50 to 75 years. The extreme loading condition is normally used in the evaluation of member safety and for the safety for the ultimate limit state. Reference [7] shows that for 25 m spans and for a 50-year return period H is on the order of 2.78 times the effect of one representative truck (or 1.39 times the effect of two side-by-side trucks). Regular traffic condition is defined as the recurrent load expected to be regularly applied on the bridge. A two-year exposure period is used herein to define the maximum load expected under regular traffic conditions. This condition is used for the analysis of the serviceability limit state and for the analysis of damaged bridges. Reference [10] shows that the expected 2 year load is about 93% of the maximum lifetime load. Thus, H becomes equal to 1.30 times the effect of two side-by-side trucks. Reference [7] recommends a COV equal to 7 % for H. In addition, a COV of 10 % is added to LL to account for possible future changes in truck traffic patterns and growth in truck traffic rates. The final COV obtained for the normalized live load effect LL is then equal to 20 %.

According to reference [10] prestressed concrete member capacities are on the average higher than the nominal or code specified resistances by a factor of 1.05 with a COV of 7.5 %. Actual dead load values are on the average 1.04 times higher than the estimated values obtained from bridge plans with a COV of 9 %. For the bridges studied in this paper, this would produce a bias of 1.05 on LF (LF=R-D) and a COV of 10 %. In addition, a COV of 8 is added herein to account for the variablity in the lateral and longitudinal positions of the trucks. The final bias on LF is then equal to 1.05 and the final COV is equal to 13 %. Cornell [11] explains that such a COV on member capacity would result in a lower COV (on the order of about 10 %) for the complete system. However, in this study it is proposed to use the same COV for member capacity as well as system capacity to account for the uncertainties associated with modeling the nonlinear behavior and predicting the system capacity. Thus, it is herein assumed that a bias of 1.05 and a COV of 10 % are valid for all four limit states studied.

Knowing the mean values and the COV of LF and LL and assuming that LF follows a lognormal distribution while LL follows an extreme type I distribution, the probability of failure P_f and the safety index β can be calculated using a first order reliability program [12].

Redundancy is defined as the capability of a bridge system to continue to carry load after the failure of its most critical member. Hence, to study the redundancy of a system, it is useful to examine the difference between the safety indices of the system and the safety index of the most critical member. If the safety index of the system for the ultimate limit state is defined as $\beta_{ult.}$, the safety index of the system for the serviceability limit state is $\beta_{serv.}$, the safety index for the damaged condition is $\beta_{damaged}$ and the safety index for member failure assuming linear elastic response is β_{member}, then the redundancy indices for the ultimate limit state, $\Delta\beta_u$, the serviceability limit state $\Delta\beta_s$ and the damaged condition $\Delta\beta_d$ are defined as:

$$\begin{aligned} \Delta\beta_u &= \beta_{ult.} - \beta_{member} \\ \Delta\beta_s &= \beta_{serv.} - \beta_{member} \\ \Delta\beta_d &= \beta_{damaged} - \beta_{member} \end{aligned} \tag{4}$$

Reference [2] proposed a set of reliability conditions that adequately redundant bridge systems should satisfy. For example, to be clasified as adequately redundant, a bridge system must produce $\Delta\ \beta_u$, $\Delta\ \beta_s$ and $\Delta\ \beta_d$ values respectively equal to or higher than +1.0, -1.0 and -0.5.

The results in table 1 show that in all the cases analyzed, the safety index for member is above 5.38. Knowing that a target member safety index of 3.5 was used in North America for the development of bridge design and evaluation codes, the member safety index values obtained herein indicate that these bridges provide highly conservative levels of safety. This however does not necessarily mean that these bridges are adequately redundant. In fact it is observed that only the bridge with end torsional rotations restrained provide an adequate level of redundancy for the ultimate limit state. None of the bridge configurations studied provide adequate redundancy for the damaged condition. However, it is observed that the 11-girder bridge shows the best performance for this case. On the other hand, all the bridges provide adequate levels of system serviceability performance.

The same calculations are repeated assuming that the COV of LF is equal to 10 %. The effect of the change in this COV is relatively small because this produces a change in the system safety indices as well as a change in the member safety indices such that the net effect on the redundancy indices is small. Similar observations are also made in reference [2] indicating that the measures of redundancy proposed in equation (4) are robust and not very sensitive to variability in the data base.

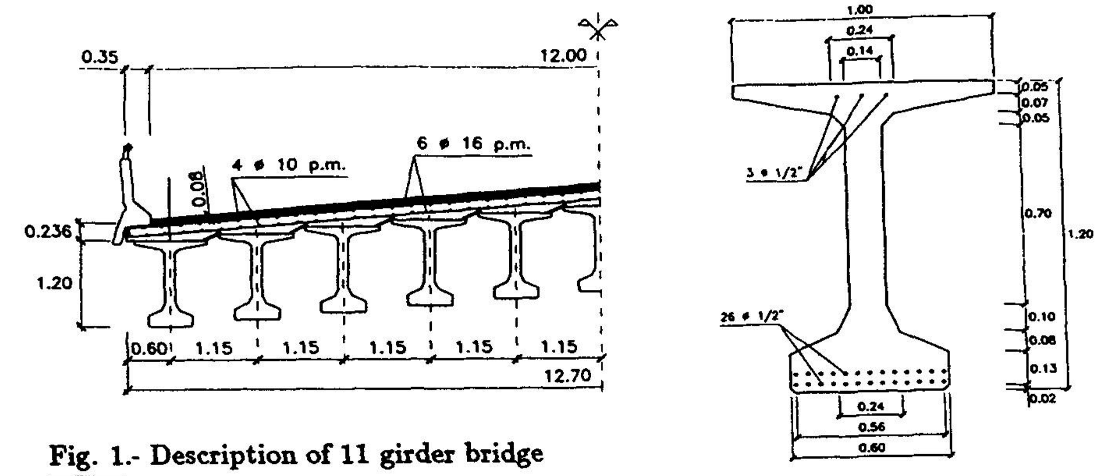

Fig. 1.- Description of 11 girder bridge

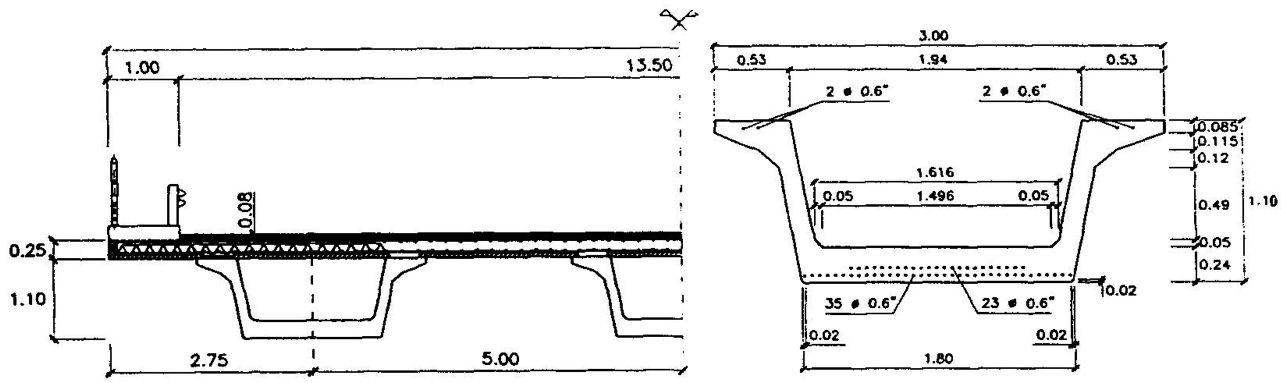

Fig. 2.- Description of 3 box-girder bridge

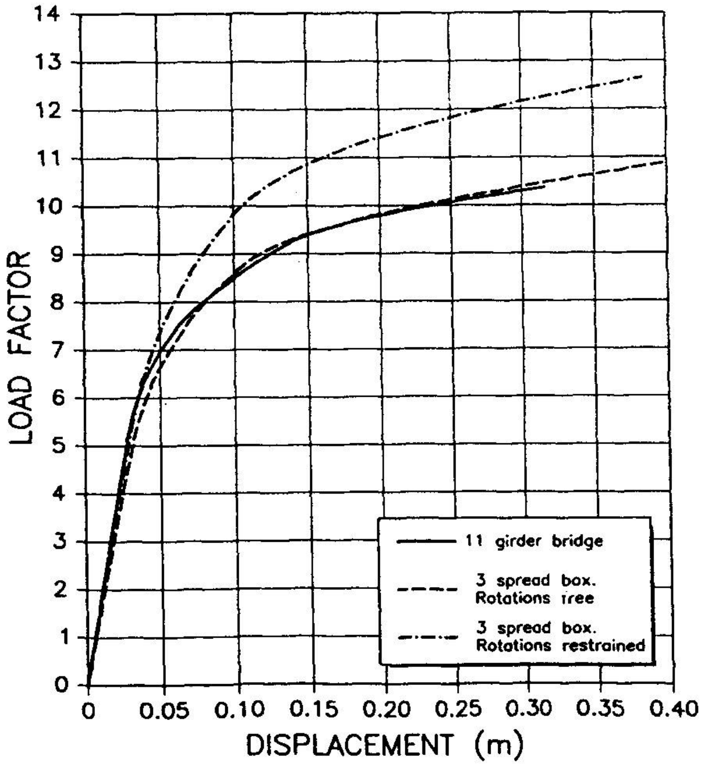

Fig. 3.- Load factor versus maximum deflection for the 3 cases analysed

Table 1. Summary of results.

			LF COV = 13%			LF COV = 10%		
CONFIGURATION	LIMIT STATE	LF	Beta	Delta beta	target	Beta	Delta beta	target
11-GIRDER BRIDGE	1 st member - Linear elastic	8,98	5,56	x	x	5,84	x	x
	Transverse slab failure	x	x	x	x	x	x	x
	Longitudinal member failure	10,40	6,09	0,53	1,00	6,41	0,57	1,00
	System serviceability	9,03	5,84	0,28	-1,00	6,14	0,30	-1,00
	Damaged condition	6,92	4,90	-0,66	-0,50	5,13	-0,71	-0,50
3-SPREAD BOX BEAM BRIDGE	1 st member - Linear elastic	8,53	5,38	x	x	5,64	x	x
one support at center	Transverse slab failure	9,18	5,64	0,26	x	5,92	0,28	x
of end section	Longitudinal member failure	10,92	6,27	0,89	1,00	6,60	0,96	1,00
	System serviceability	9,17	5,90	0,52	-1,00	6,20	0,56	-1,00
	Damaged condition	5,67	4,22	-1,16	-0,50	4,41	-1,23	-0,50
3-SPREAD BOX BEAM BRIDGE	1 st member - Linear elastic	9,08	5,60	x	x	5,88	x	x
one support under each	Transverse slab failure	10,41	6,09	0,49	x	6,41	0,53	x
web of end section	Longitudinal member failure	12,67	6,82	1,22	1,00	7,20	1,32	1,00
	System serviceability	10,43	6,36	0,76	-1,00	6,70	0,82	-1,00
	Damaged condition	6,21	4,53	-1,07	-0,50	4,74	-1,14	-0,50

5. CONCLUSIONS

The system performance of two alternate bridge configurations is analyzed. The results indicate that the three-cell bridge configuration provides the best system performance for the ultimate limit state. However, it is the 11-girder bridge that provides the best system performance in case of damage to one critical member. Although, the redundancy indices obtained are relatively low, the values of the system safety indices obtained are high (greater than 4.2 even for the damaged condition), indicating that these bridges are conservatively designed and provide overall high levels of system safety.

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