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SAFETY EVALUATION OF EXISTING PARTIALLY PRESTRESSED CONCRETE GIRDER BRIDGES

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Abstract—An efficient method for evaluating the safety of existing girder bridges as a function of the load and resistance parameters is presented. The bridge capacity is determined using a nonlinear finite element program in terms of the truck load which is increased until structure collapse occurs. It is determined that the reliability of fully prestressed bridges is higher than that of partially prestressed bridges, with the bridge system reliability is one-and-half to two times the girder reliability index. It is also found that the system reliability is more senstive to accidental or local damage of the exterior girders than the interior ones.

INTRODUCTION

Nearly 45% of the 600,000 bridges in the U.S.A. have been classified by the FHWA as severely damaged or deteriorated because of heavy vehicles, natural environmental hazards or lack of maintenance. Most rating and evaluation techniques for highway bridges are aimed at individual components (girders). However, bridge systems have a considerable reserve capacity and thus have higher reliability levels than those of the individual components. This is due to the fact that bridges are *multi-load-path* structures. Therefore, the failure of one component is not sufficient to trigger the failure of the bridge system.

It is generally recognized the the bridge resistance is random due to the variability of material properties, uncertainties in predicting mechanical deterioration of the structural materials, simplified methods of analysis and uncertainties in predicting the loads applied to the structure. Therefore, this study is intended to develop models for evaluating the system reliability and redundancy of partially prestressed concrete girder bridges when subjected to random traffic loadings. To carry out this objective, an analytical investigation into the reliability evaluation of slab-and-girder bridges using three-dimensional nonlinear geometric, material and time-dependent analysis is conducted. Also, a measure which distinguishes between single-load-path and multi-load-path structures is investigated. The effect of bridge deficiency on the reliability and overall redundancy is evaluated.

SYSTEM LIVE LOAD MODEL

The system live load depends on the truck configuration, truck transverse position and simultaneous number of trucks on the bridge. There are several cases that could be considered when evaluating the system reliability. The number of these cases depends on the bridge span length and width (i.e. number of lanes). For long-span bridges there is a possibility of multiple truck presence along the same lane. Further, each lane could be occupied by a truck with a possible correlation between the heavy trucks. Multiple presence cases are slightly complicated and will not be addressed here.

The statistical properties of the system live load, including the dynamic effect, for a single truck and two correlated trucks are presented in Table 1 [1].

SYSTEM DEAD LOAD MODEL

Dead loads are made of the weights of the structure itself and the nonstructural elements. The variation of the weight of the structure depends on the uncertainties in the cross-sectional area and unit weight of the material. Many researchers have investigated the statistical uncertainties in dead load [2–4]. Since bridges are constructed of different materials, it is convenient to consider the following components of dead load: the weight of the precast girders; the weight of the cast-in-place slab; and the weight of the wearing surface (asphalt).

The bridge dead loads are assumed to be normally distributed and their statistical properties are presented in Table 2 [5].

SLAB-AND-GIRDER BRIDGE

The slab-and-girder bridge consists of two major types of structural members, namely: (a) a reinforced concrete slab and (b) a number of flexible girders which span in the direction of the traffic and carry all the loads to the abutments.

The slab-and-girder bridge system is a favored structural choice both on economic and aesthetic

Table 1. The mean/nominal ratio of the system live load moments for a single and two correlated trucks

Truck type	Span (ft)			
	40	60	80	100
Single	1.96	2.11	2.15	2.32
Two correlated	3.53	3.76	3.83	4.11

Coefficient of variation for all cases = 0.18.
The nominal values are those of AASHTO truck moments.

grounds. The use of a shear mechanism which ensures composite action between the girders and the slab makes it possible to use smaller supporting girders. If steel I-beam or precast prestressed concrete girders are used, expensive shoring which supports the weight of the wet cast-in-place slab concrete can be avoided. This makes construction relatively rapid, easy and minimizes traffic interruption when it is a problem.

The basic design problem of determining the distribution of wheel loads among the girders has been solved in previous research [6, 7].

SYSTEM RESISTANCE MODEL

The bridge resistance, R, is considered as a function of three variables, A_1, A_2, and A_3

$$R = R_n A_1 A_2 A_3 \tag{1}$$

in which R_n is the nominal bridge resistance, A_1 is a factor representing uncertainties in the basic resistance, A_2 is a factor representing uncertainties in the analysis and model, and A_3 is a factor representing uncertainties in dead load.

The coefficient of variation of the resistance is expressed as

$$V_R = \sqrt{V_{A_1}^2 + V_{A_2}^2 + V_{A_3}^2}, \tag{2}$$

where V_{A_1}, V_{A_2}, and V_{A_3} are the coefficients of variation of the basic resistance, analysis, and dead load, respectively. The distribution of the resistance is assumed to be log-normal [5].

Finite element model

It is well known that the behavior of reinforced and prestressed concrete structures deviates, even for relatively low loading levels, from the linear elastic behavior that classically has been assumed to be valid. For ultimate loading levels the behavior of prestressed concrete structures is highly nonlinear,

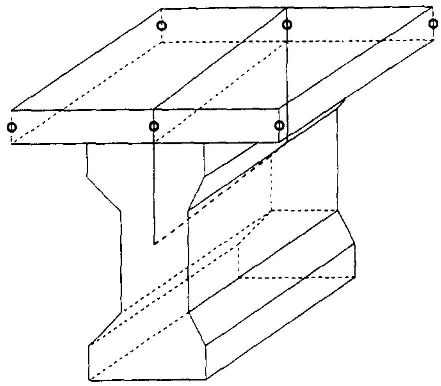

Fig. 1. Stiffened plate element.

not only because of the material properties but also because sometimes the magnitude of the displacements and strains no longer can be considered *small*.

To assess the safety of the bridge system against failure, an accurate estimation of the ultimate load has to be made. Therefore, the prediction of the behavior of the structure in the inelastic and ultimate ranges is needed.

There is an extensive literature about the application of finite elements to the analysis of concrete structures. Comprehensive reviews have been made by Scordelis [8, 9], Schnobrich [10], Bazant *et al.* [11], ASCE Committee on Finite Element Analysis of Reinforced Concrete Structures [12], Kang [13], and Chan [14]. Buckle and Jackson [15] developed a filamented beam element with a rectangular cross-section for the analysis of beam–slab systems, and Chan [14] developed a similar element with a trilinear torque–twist relationship to model concentric and eccentric edge beams in shell structures, taking into account geometric and material nonlinearities and time-dependent behavior.

Mari and Scordelis [16] developed a filamented reinforced concrete beam element. The element has six degrees of freedom at each end and one internal degree of freedom at midlength that is eliminated by static condensation. The element can have an arbitrary cross-section defined by a special shape matrix. A trilinear torque–twist relationship is used to model the torsional behavior of the element. The prestressing effect is introduced by means of an equivalent load vector and the participation of the prestressing steel to the element stiffness is added directly. Each concrete and steel filament is considered to be subjected to a unixial stress-state. Parabolic–linear, bilinear, and multilinear approximations of the

Table 2. Statistical properties of bridge dead load

Dead load component	Mean-to-nominal ratio	Coefficient of variation
Precase girder	1.03	0.08
Cast-in-place slab	1.05	0.10
Wearing surface	Mean = 3.5 in	0.15

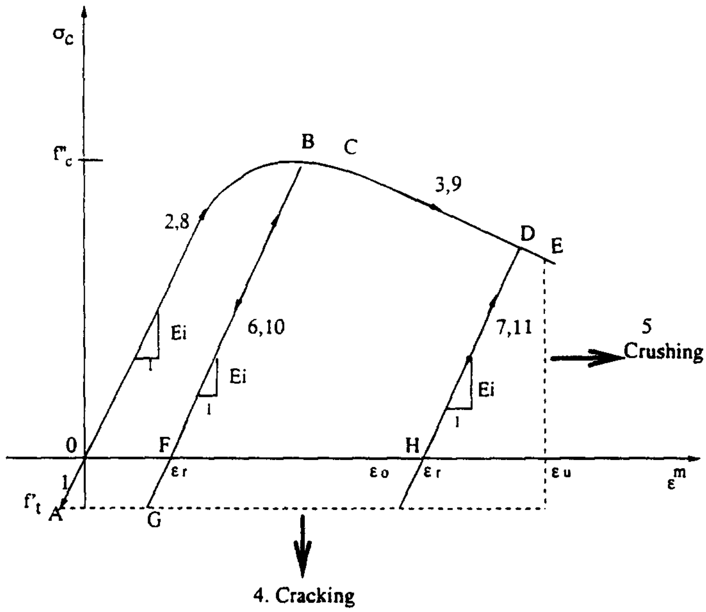

Fig. 2. Stress–strain diagram of concrete.

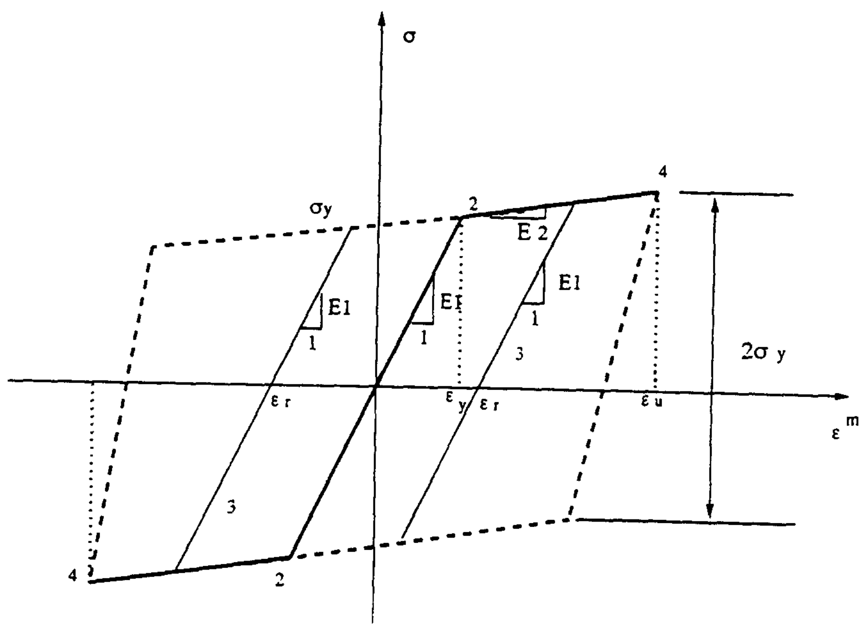

Fig. 3. Stress–strain diagram of reinforcing steel.

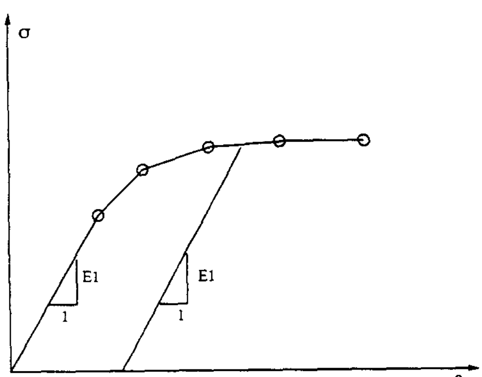

Fig. 4. Stress–strain diagram of prestressed steel.

stress–strain curves are utilized for concrete, reinforcing steel, and prestressing steel, respectively. A simple model for inelastic load reversal is incorporated. A perfect bond between the concrete and steel is assumed. A tangent stiffness formulation, coupled with a time integration solution is used. In order to solve the nonlinear equations either load control or displacement control can be used to allow the complete load–displacement curve to be traced. A similar formulation of nonlinear geometric, material, and time-dependent reinforced shell element was used by Chan [14].

Mari and Scordelis [16] and Chan [14] developed, verified, and compared the results of the computer programs PCF3D and NASHL with other analytical or experimental results. The existing programs are modified and combined in the PCB3D (three-dimensional prestressed concrete bridge). More details about PCF3D and NASHL are presented in [14, 16]. The finite element method in this study utilizes a stiffened plate which consists of two thin shell elements and one beam element as shown in Fig. 1. The two thin shell elements are connected to the beam element by rigid links. The plate element accounts for inplane and out-of-plane behavior of the reinforced concrete deck. The eccentric beam element developed by Mari and Scordelis [16] is used to model the prestressed concrete girders.

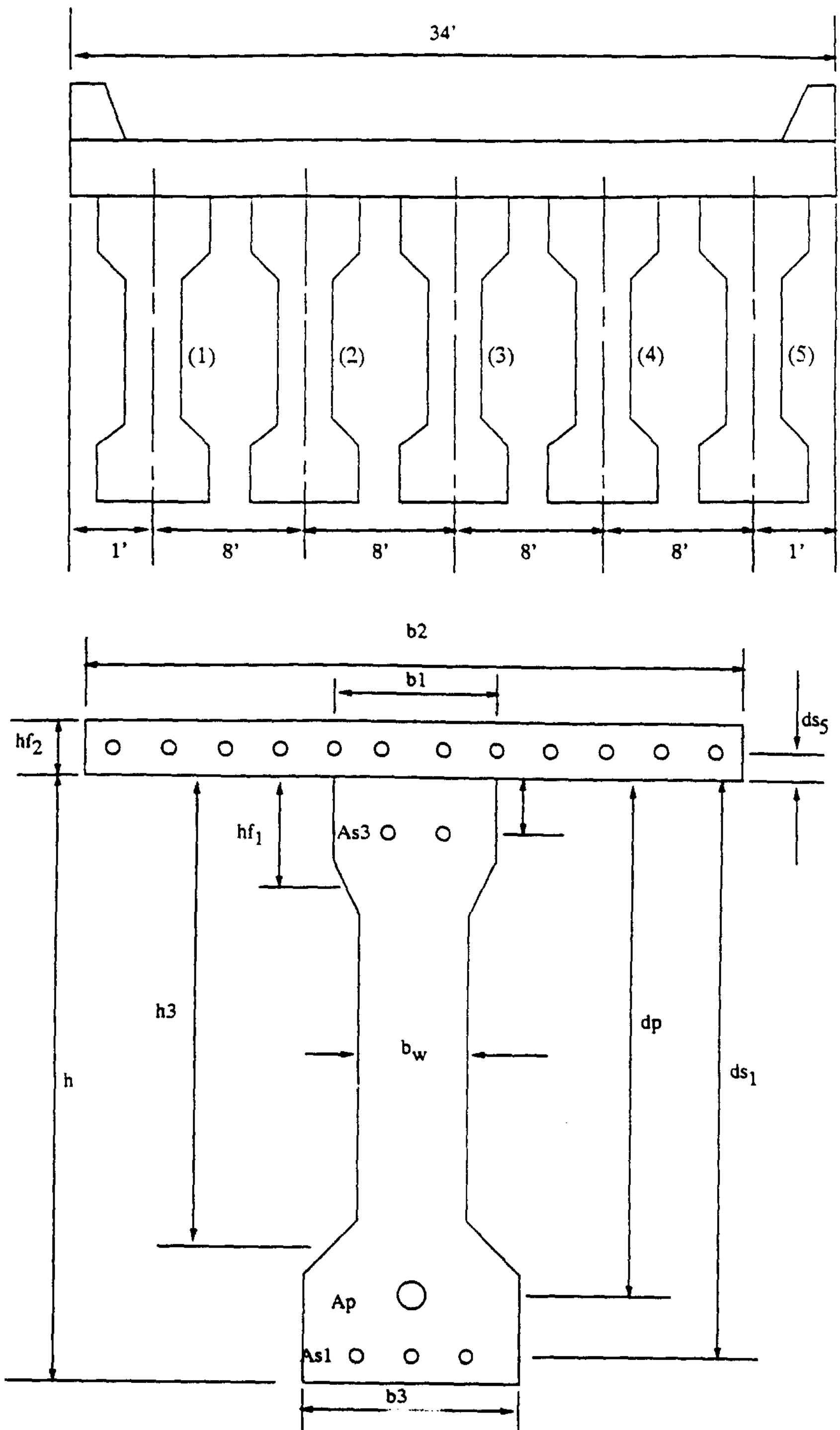

Fig. 5. Cross-section of partially prestressed concrete girder used in the parametric studies.

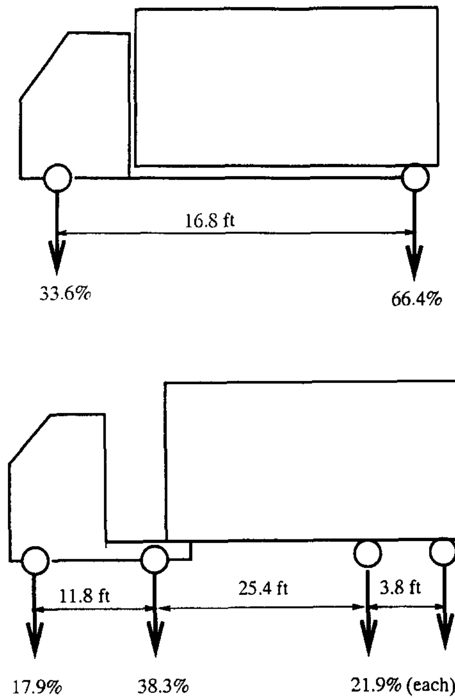

Fig. 6. Truck configuration.

As a result of comprehensive convergence studies [7] on slab-and-girder bridges, 14 elements in the traffic direction and two elements in the transverse direction (between girders) have been chosen.

BRIDGE NOMINAL RESISTANCE

Several slab-and-girder bridge models have been analyzed using the nonlinear finite element programs PCB3D. The program is a nonlinear finite element analysis program for reinforced and prestressed three-dimensional concrete frames and slab–beam systems, taking into account geometric and material nonlinearities, and the time-dependent effects of load and temperature history, creep, shrinkage, and aging of concrete, and relaxation of prestressing steel. The program is written in the FORTRAN IV language.

The mathematical formula used in this study to represent the stress–strain relationship of concrete is the one suggested by Kang [13]. The loading and reloading due to live load or temperature history are

Table 3. Input data at the midspan of a partially prestressed girder used in the parametric studies

$b_1 = 20$ in, $d_p = 49.43$ in
$b_2 = 96$ in, $d_{s1} = 52$ in
$b_3 = 26$ in, $d_{s6} = 3$ in
$b_w = 8$ in, $h_{f1} = 11$ in
$h_{f2} = 8$ in, $h_3 = 41.5$
$h = 54$ in
$f'_c = 500$ psi, $f'_{cs} = 3500$ psi
$A_p, A_{s1} = (5.51, 0)$ for PPR = 1
$A_p, A_{s1} = (3.98, 4.2)$ for PPR = 0.80
$A_p, A_{s1} = (2.75, 8.32)$ for PPR = 0.60
$f_{pu} = 270$ ksi, $f_{py} = 240$ ksi

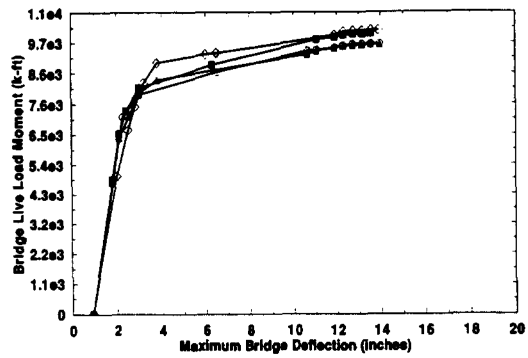

Fig. 7. Load–deflection curves for the 80-ft composite PPC bridge.

accounted for by a simple load reversal model of the stress–strain curve. The load reversal model utilized in this study is shown in Fig. 2. The properties of reinforcing steel, unlike concrete, generally are not dependent on environmental conditions or time. In this study a bilinear model which is symmetrical about the origin, as shown in Fig. 3 is used. A multilinear stress–strain curve, as shown in Fig. 4, is adopted for prestressed steel in this study. The slope of the unloading and reloading path is assumed to be the same as the initial modulus. Since prestressed steel is never subjected to compressive stresses, the compressive stress–strain curve is not considered. Since this study is concerned with slab-and-girder bridges, the basic bridge model consists of a reinforcing concrete slab supported by five prestressed concrete girders. The prestressed concrete griders are those of AASHTO IV and the cross-section of the bridge is shown in Fig. 5.

To evaluate the structural system reliability of a bridge system, the system live load and resistance need to be defined clearly and then evaluated. Both

Table 4. Results of ultimate resistance of an 80 ft bridge system (PPR = 0.80)

Event i	r_1(k-ft)	r_2(k-ft)	r_3(k-ft)	r_4(k-ft)
1	9991	8791	10,487	7784
2	10,282	8526	10,320	6908
3	9827	9234	10,172	7438
4	10,406	8718	10,120	7452
5	10,080	9111	10,300	7343
6	9986	8952	10,334	7747
7	10,254	8569	10,450	8002
Mean	10,118	8843	10,312	7525
Standard deviation	203	268	134	358

Table 5. Results of the system reliability (PPR = 0.80)

Statistical property	Component				
	R_n	$L+I$	D_1	D_2	R
Mean	8763	2504	2720	3288	14,711
Standard deviation	274	451	272	263	640

$\beta_{sys} = 7.53$.

Table 6. Results of ultimate resistance of an 80 ft bridge system (PPR = 1.0)

Event i	r_1(k-ft)	r_2(k-ft)	r_3(k-ft)	r_4(k-ft)
1	11,034	9707	11,587	8601
2	11,356	9413	11,402	7633
3	10,854	10,196	11,239	8219
4	11,493	9627	11,181	8234
5	11,133	10,060	11,380	8114
6	11,029	9885	11,418	8560
7	11,325	9462	11,546	8842
Mean	11,175	9613	11,393	8315
Standard deviation	225	377	147	396

system resistance and live load depend on the truck configuration, transverse position of the truck, and the simultaneous number of trucks occupying the bridge deck.

In this paper, only a single truck unit is considered when analyzing the bridge system. Further, four cases of possible truck placement are considered. The cases are

1. A single unit in the right lane with probability of occurrence (P_c) equal to 0.133.
2. A semi-trailer in the left lane with $P_c = 0.533$.
3. A single unit in the left lane with $P_c = 0.067$.
4. A semi-trailer in the left lane with $P_c = 0.267$.

The configuration of both the single unit and the semi-trailer are shown in Fig. 6. In general, the bridge resistance increases if more trucks are placed on the bridge. Since the bridge resistance and live load are correlated, it is difficult to predict which case of loading will control.

To establish the statistical characteristics of the system resistance, seven events for each truck type and position (case) are considered. The characteristics of each event are obtained from simulating truck weight, axle spacing and truck position (in the transverse direction). The truck weight is assumed to be normally distributed, the axle spacing is uniformly distributed and the truck position is log-normal with a mean value of 3 ft from the edge of the lane. For each event a nonlinear finite element analysis of the bridge is performed and the solution is obtained through a load control strategy. For each case, the mean and standard deviation are calculated using

$$\mu_{rj} = \frac{1}{n} \sum_{i=1}^{n} r_{ij} \tag{3}$$

Table 7. Results of the system reliability (PPR = 1.0)

Statistical property	Component R_n	$L+I$	D_1	D_2	R
Mean	9593	2504	2720	3288	15,601
Standard deviation	346	451	272	263	702

$\beta_{sys} = 8.29$.

Table 8. Results of ultimate resistance of an 80 ft bridge system (PPR = 0.6)

Event i	r_1(k-ft)	r_2(k-ft)	r_3(k-ft)	r_4(k-ft)
1	9484	8349	9960	7393
2	9760	8097	9802	6561
3	9328	8769	9661	7065
4	9878	8279	9612	7078
5	9568	8652	9783	6975
6	9479	8501	9815	7358
7	9734	8138	9925	7601
Mean	9605	8398	9794	7147
Standard deviation	193	254	127	340

Table 9. Results of the system reliability (PPR = 0.6)

Statistical property	Component R_n	$L+I$	D_1	D_2	R
Mean	8318	2504	2720	3288	14,326
Standard deviation	260	451	272	263	619

$\beta_{sys} = 7.15$.

$$\sigma_{rj} = \sqrt{\frac{1}{n} \sum_{i=1}^{n} r_{ij}^2 - \mu_{rj}^2} \tag{4}$$

in which n is the number of events (7) and r_{ij} is the ultimate truck moment that the bridge can withstand for case i and event j. The total mean and standard deviation of the bridge moment resistance (beyond dead load moment) considering all cases is given by

$$\mu_r = \sum_{j=1}^{4} P_{cj} \mu_{rj} \tag{5}$$

$$\sigma_r = \sum_{j=1}^{4} P_{cj} \sigma_{rj}. \tag{6}$$

RELIABILITY INDEX FOR ULTIMATE BRIDGE STRENGTH

The failure function which includes the bridge resistance, the combined live and impact moments and dead load moments of the cast-in-place slab, and the weight of the precast girders is given as

$$g = R - (L + I) - (D_1 + D_2) \tag{7}$$

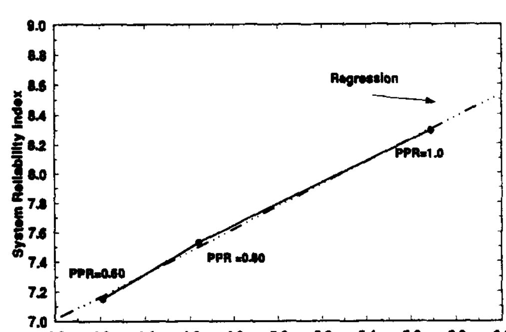

Fig. 8. System versus girder reliability index.

in which L, I, D_1, and D_2 are the live, impact, slab dead, and precast girder dead load moments, respectively. A five-girder composite prestressed concrete bridge with a span of 80-ft is analyzed using the PCB3D nonlinear finite element program. A cross-section of the bridge is shown in Fig. 5. The input

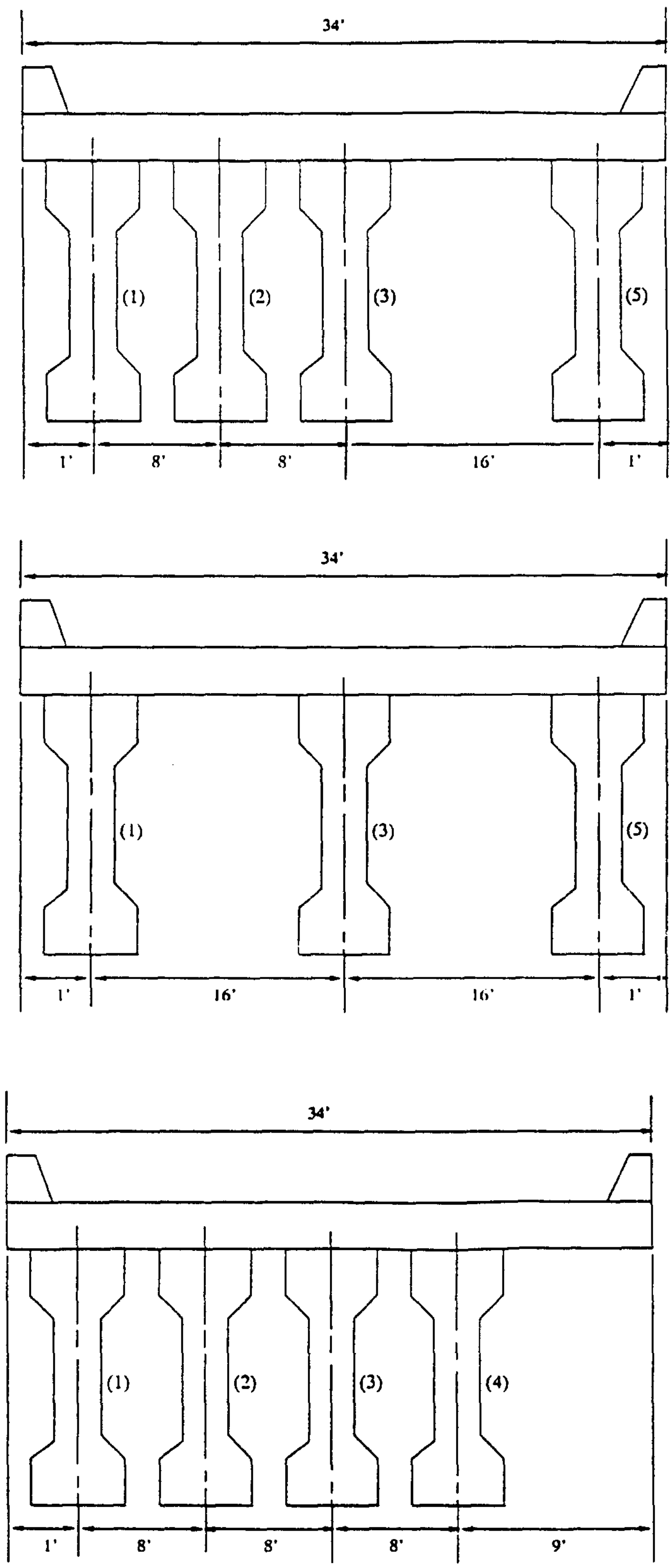

Fig. 9. Bridge configurations.

data at the midspan sections of each girder used are given in Table 3.

Figure 7 shows four of the seven simulated load–deflection curves for the 80-ft composite prestressed concrete bridge subjected to a single unit truck in the right lane. This figure illustrates the variation in the bridge resistance as a result of the uncertainty in the weight, position, and axle spacing of the truck. The outputs for the bridge resistance under a different truck placement together with the mean and standard deviation (PPR = 0.80) are presented in Table 4. The final results of the system reliability are shown in Table 5. The net system reliability index for this bridge is equal to 7.53.

It is noted that the reliability index for each girder is equal to 4.64. The increase in the system reliability over the girder reliability reflects the level of redundancy that a composite prestressed girder bridge has. The results for PPR = 1.0 and 0.6 are presented in Tables 6–9. The ratios of the system reliability to the girder reliability when PPR = 1.0 and 0.60 are 1.48 and 1.71, respectively.

The relation between β_{sys} and β_{girder} is slightly nonlinear and depends on the PPR. In addition, increasing β_{girder} decreases the ratio of $\beta_{sys}/\beta_{girder}$ as shown in Fig. 8.

REDUNDANCY OF PRESTRESSED CONCRETE BRIDGES

In general, girder bridges have the ability to redistribute the applied loads due to accidental or local damage (reduction in nominal resistance parameters). The redundancy of a bridge system can be evaluated from both deterministic and probabilistic points of view. Deterministic formulations ignore the variabilities associated with the resistance and load parameters. A more realistic measure for a bridge redundancy can be formulated based on the reliability theory. This measure should relate the safety of the structure at various damage states to that of the undamaged bridge. A simple redundancy factor is proposed as

$$R = \frac{\beta_i}{\beta_0}, \tag{8}$$

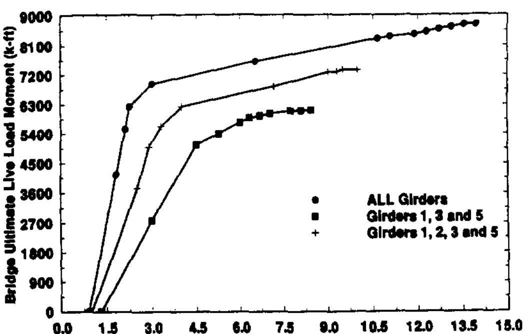

Fig. 10. Effect of accidental damage on the bridge maximum live load moment.

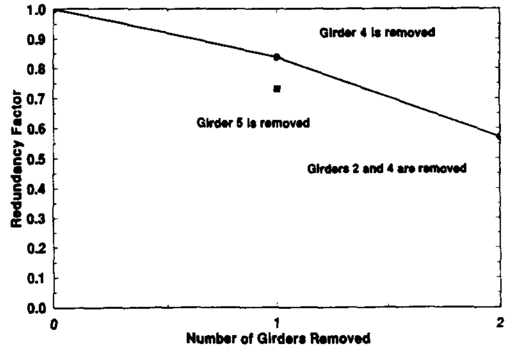

Fig. 11. The effect of accidental damage on bridge redundancy.

where β_i and β_0 are the damaged and undamaged system reliability indices. The redundancy factor R is always less or equal to 1. When $R = 1$, the structural damage has no influence on the bridge behavior. Since β_i depends on the bridge resistance, the definition of R is also dependent upon the truck configuration, truck position, material behavior, and finally the location of the damaged member(s).

Effect of accidental damage

To investigate the effects of accidental damage on the redundancy factor, some girder are removed completely from the structure. Three cases are investigated, namely, removing girder 4, removing girders 2 and 4, and finally removing the exterior girder 5 (see Fig. 9). The bridge maximum live load moment capacity versus the bridge deflection for the cases when internal girder are removed is shown in Fig. 10. The maximum live load moment of the three-girder bridge is 6120 (k-ft) while that of the four-girder bridge (interior girder 4 is removed) is 7345 (k-ft). The redundancy factors for these bridges are shown in Fig. 11. To investigate the importance of the exterior and interior girders, a four-girder bridge was created by removing girder 5 (exterior girder) as shown in Fig. 9. The redundancy factor is presented in Fig. 11. The figure clearly shows that removing the exterior girder reduces the reliability more than removing an interior girder.

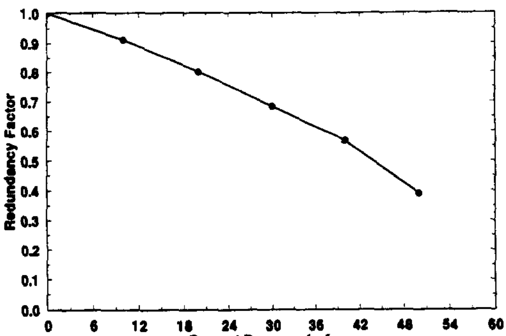

Fig. 12. The effect of local damage on the bridge redundancy.