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Christopher D. Eamon

Mississippi State University, Starkville, MS, christopher.eamon@wayne.edu

Andrzej S. Nowak

University of Michigan, Ann Arbor, MI

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Effect of Edge-Stiffening and Diaphragms on the Reliability of Bridge Girders

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Christopher D. Eamon¹, M. ASCE and Andrzej S. Nowak², F. ASCE

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Abstract

Secondary elements such as barriers, sidewalks, and diaphragms may affect the distribution of live load to bridge girders. The objective of this study is to evaluate their effect on girder reliability if these elements are designed to be sufficiently attached to the bridge so as not to detach under traffic live loads. Simple span, two lane structures are considered, with composite steel girders supporting a reinforced concrete deck. Several representative structures are selected, with various configurations of barriers, sidewalks and diaphragms. Bridge analysis is performed using a finite element procedure. Load and resistance parameters are treated as random variables. Random variables considered are composite girder flexural strength, secondary element stiffness, load magnitude (dead load and truck traffic live load), and live load position. It was found that typical combinations of secondary elements have a varying influence on girder reliability, depending on secondary element stiffness and bridge geometry. Suggestions are presented that can account for secondary elements and that provide a uniform level of reliability to bridge girders.

Introduction

It is well known that traditional analysis models used for bridge design do not accurately

1. Assistant Professor of Civil Engineering, Mississippi State University, MS, 39762-9546
2. Professor of Civil Engineering, University of Michigan, Ann Arbor, MI 48109-2125

predict actual structural behavior (Burdette and Goodpasture 1973; Wegmuller 1977; Buckle et al. 1985; Bakht and Jaeger 1992; Nowak and Kim 1998; Nowak et al. 1999). In particular, a significant discrepancy in behavior exists in the prediction of live load distribution to bridge girders. One recent study (Nowak et al. 1999) compared actual load distribution factors of short and mid-span bridges (20-30 m) tested in Michigan to the 1998 LRFD Code (AASHTO 1998) prediction (with two lanes loaded), and revealed differences of up to 35%. Analytical results suggest that larger discrepancies exist for longer-span structures (Mabsout et al. 1997; Eamon and Nowak 2002). Discrepancies exist primarily because traditional models do not account for features of actual bridges that may significantly affect load distribution. Features to be explored in this study are secondary elements such as barriers, sidewalks, and diaphragms.

Although secondary elements are not designed to redistribute live load, the available experimental data indicate that in fact they may, even at extreme overloads, as collected from proof load and bridge ultimate capacity tests (Nowak and Kim 1998; Nowak et al. 1999; Burdette and Goodpasture 1973). It must be noted that this data is limited and these elements should not be relied upon in design to participate in resisting vehicular live loads without further testing, as these components were not developed for this purpose. However, the experimental observations cited above lead to two issues that this paper will explore. First, as all bridges must have barriers, and many sidewalks, it may make sense to sufficiently increase their connectivity and take advantage of their presence, as the potential benefits of including these elements for consideration of live-load distribution may be significant. And second, as the AASHTO LRFD Code was calibrated without considering the effects of secondary elements, structures designed by this Code may have a variation in safety level, as these components clearly effect live load distribution even though not intended to. The objective of this study is to determine how the

reliability of bridge girders may be affected by secondary elements, and to suggest a procedure that may account for these effects.

Structures Considered

Fifteen bridges were considered in this study, with typical cross-sections as shown in Fig. 1. All bridges are simple span, two lane structures, with spans of 10 m, 20 m, 30 m, 40 m, and 50 m combined with girder spacing of 2 m, 3 m, and 4 m. The parameters are selected so that the considered structures satisfy the structural design requirements of the AASHTO LRFD (1998) Code. There is a growing trend to use fewer girders, hence the large 4 m girder spacing is also included in this study. For all of the structures, girders were composite steel and the deck was 230 mm thick reinforced concrete. Each structure was analyzed for six cases; 1) without any secondary elements; 2) with diaphragms only; 3) with barriers only; 4) with barriers and diaphragms; 5) with barriers and sidewalks; 6) with barriers, sidewalks, and diaphragms. Diaphragm, barrier, and sidewalk dimensions are based on current Michigan DOT designs, which are representative of many state DOTs. The concrete barrier considered is idealized as a rectangular cross-section with 340 mm width and 1000 mm height. Idealized dimensions were chosen such that the model barrier has the same stiffness value as the more commonly seen slanted base barrier used by Michigan DOT. The sidewalk has a width of 2000 mm, measured from the outside edge of the bridge deck to the interior edge of the sidewalk, and a height of 240 mm. The diaphragm is of the cross-bracing type and made of three steel angle shapes (two diagonals and one lower horizontal). These are 4x4x5/16 angles for 2 m girder spacing, 5x5x3/8 for 3 m girder spacing, and 6x6x3/8 for 4 m girder spacing. For the 10 m span bridge, one (transverse) row of diaphragms was placed between girders at midspan. For the 20-30 m spans,

two rows of diaphragms were equally spaced along the bridge length. For the 40-50 m span, three rows of diaphragms were used. A prior study indicated that increasing the number of diaphragms has little effect on load distribution at maximum moment position near midspan (Eamon and Nowak 2002). An important consideration is the degree of connectivity between bridge components (barrier, sidewalk, diaphragm), as if the elements detach under traffic loads, they can no longer aid in load re-distribution. As noted in the introduction, although these elements have performed favorably under substantial overloads, insufficient data exist to adequately assess the reliability of typical connection details, as these elements are not explicitly designed nor tested to aid in redistributing live loads. However, for the exploratory purposes of this study, they are modeled such that they remain positively attached to the bridge under a live load level that may cause a single girder failure.

Load Models

Bridge dead load and live load (truck traffic) are considered in this study. The load models are based on those developed for the calibration of the AASHTO LRFD Code (Nowak 1993 and 1999).

Dead load included the weight of the girders, deck slab, wearing surface, barriers, sidewalks, and diaphragms. Statistical parameters, the bias factor (λ) and coefficient of variation (V), are taken as $\lambda = 1.03$ and $V = 0.08$ for factory-made components (girders, diaphragms); $\lambda = 1.05$ and $V = 0.10$ for cast-in-place components (deck, barriers, sidewalks); and asphalt wearing surface is taken to have a mean value of 90 mm (3.5") with $V = 0.25$ (Nowak 1993 and 1999).

Live load parameters were derived from a truck survey used for calibration of the AASHTO LRFD Code (Nowak 1993, 1999). The model includes the parameters for a single lane and for

two traffic lanes. Based on the available data, this model assumes that every 15th truck on the bridge is accompanied by another truck side-by-side. It further assumes that with every 10th simultaneous occurrence (trucks side-by-side), the truck weights are partially correlated ($\rho=0.5$), and every 30th occurrence the truck weights are fully correlated ($\rho=1.0$). Moreover, with regard to multiple presence (multiple trucks in a single lane), every 50th truck is followed by another truck with distance between trucks from 4.5 to 30 m; every 150th truck is followed by a partially correlated truck (with regard to weight); and every 500th truck is followed by a fully correlated truck. The results of the model, as pertinent to this study, are as follows: for the single-lane loaded case, λ (ratio of actual moment to AASHTO LRFD HL-93 design moment) for a single lane varies from 1.35 at the shortest span (10 m) to 1.25 for longer spans (50 m), while V is 0.18 for all spans. For the two-lanes loaded case, λ for each truck varies from 1.2 at 10 m to 1.0 at 50 m (note λ for the total moment on the bridge would then be equivalent to 1.2×2 trucks = 2.4 at 10 m and $1.0 \times 2 = 2.0$ at 50 m), while V varies from 0.14 at 10 m to 0.18 at 50 m. For the bridges considered here, it was found that the two-lane loaded case dominates girder reliability index, to the extent that the single-lane loaded case can be neglected.

Recent field tests of 11 bridges conducted by the University of Michigan revealed a dynamic load factor of less than 0.10 for two heavily loaded trucks traveling side-by-side over various bridges (Nowak and Kim 1998; Nowak et al. 1999). Based on these results, the mean dynamic load factor is conservatively taken as 0.10 while the coefficient of variation is 0.80.

Structural Analysis Model

To determine the effects of secondary elements on girder distribution factor, structures were analyzed using the finite element method. Specific modeling details and results are fully

described by Eamon and Nowak (2002). In general, these models used 4-node Mindlin structural shell elements to represent bridge girder flanges and webs, and 8-node hexahedral elements to represent the deck, sidewalk, and barriers. Diaphragm members were modeled explicitly with Timoshenko beam elements that can account for (St. Venant's) torsion of sections of arbitrary shape. Standard material properties were used, and models were extensively verified by and calibrated to independent experimental data. It is conservatively assumed that secondary elements add no capacity but affect load distribution only. That is, they are included in the modeling when distributing live load to the girders, but they are not included when calculating girder capacity. Further, following AASHTO Code load distribution assumptions, additional reductions in girder distribution factor (GDF) in the inelastic range are conservatively ignored. The wheel patches of the AASHTO HS20 truck load configuration, or its design tandem, which governs on the 10m spans, were used for this analysis.

On each of the structures considered, two side-by-side trucks were positioned transversely until the maximum moment on any interior girder was found.

For the bridges considered in this study, it was found that structures with barriers and diaphragms can result in GDF reductions from 11-25%, while structures with barriers, sidewalks, and diaphragms can result in GDF reductions from 17-42% (Eamon and Nowak 2002). Most secondary element combinations tend to be more effective at closely-spaced girders and longer spans, and all elements are more effective on less-stiff bridges. Based on a parametric analysis of over 240 cases using the finite element models described above (Eamon and Nowak 2002), it was found that GDF can be well-approximated by a closed-form function of secondary element stiffness and bridge geometry. In general the empirical equations predict GDF within a few percent of the available data. GDF can be taken as follows:

$$\text{GDF} = (\text{GDF}_{\text{base}})[\alpha_d][\alpha_e] \quad (1)$$

Where:

$(\text{GDF}_{\text{base}})$ is GDF without secondary elements, which can be taken as the GDF formula in the AASHTO Code for interior girders on two-lane bridges (the LRFD version is recommended, as it is more accurate).

$[\alpha_D]$ is an adjustment factor for the presence of diaphragms:

$$\alpha_d = 1 + (L - 10)(S - 2) \left(\frac{K_d - 690}{490,000} \right) \quad (2)$$

$[\alpha_E]$ is an adjustment factor for the presence of edge-stiffening elements.

When only barriers are present:

$$\alpha_e = \left(\frac{120K_e + 1100}{100,000} \right) (S - 2) + \left(\frac{K_e}{1.5} \right)^{-0.06} \left(\frac{L}{10} \right)^{-0.08} \quad (3)$$

When barriers and sidewalks are present:

$$\alpha_e = 1.03 - \frac{L}{200} + \left(\frac{L}{800} \right) (S - 2) + (K_e - 1.5) \left(\frac{L^2}{20000} - \frac{L}{220} \right) \quad (4)$$

In these equations, K_d is the diaphragm stiffness ratio $K_d = \left(\frac{EI_g}{EI_d} \right)$, while K_e is the edge stiffness

ratio $K_e = \left(\frac{EI_s}{EI_u} \right)$. EI_g and EI_d are the rigidities of the interior composite girder and diaphragm,

respectively, while EI_s and EI_u are the rigidities of the stiffened and unstiffened exterior

composite girder. For EI_s , the entire sidewalk width and the barrier are included in the

calculation. EI_u is computed normally. Further accuracy can be obtained if the effective slab

width is adjusted for shear lag (Hambly 1976). S is the center-to-center girder spacing (meters) and L is the bridge span (meters).

The equations are verified only for two-lane, simple span, girder bridges with concrete decks, having adequate shear connection between components (including girder, deck, barrier, diaphragm, and sidewalks, if present), with girder spacing from 2.0 to 4.0 m, span from 10 to 50 m, deck thickness from 150 to 300mm, K_d from 10 to 630, and K_e from 1.5 to 20 for barriers and from 1.5 to 3.5 for barriers and sidewalks. If K_e exceeds 3.5 when barriers and sidewalks are present, it may be taken as 3.5. In no case should any of the factors be taken greater than unity. This may conservatively occur if extreme values are entered into the formulas. Sufficient data for shorter spans and more-closely spaced girders were not available. This data would also have to be collected and processed to extend the range of applicability of the equations.

Resistance Model

There are numerous random variables that affect the strength of a composite girder. These include material strengths, dimensions, and location of reinforcement in the composite section (classified as material properties and fabrication tolerances, FM). There are also uncertainties that result from simplified or approximate analysis methods (classified as the professional factor, P). A summary of these random variables and their parameters are given in Table 1, which are taken from the available literature (Ellingwood et al. 1980; Kennedy 1982; Siriaksorn 1980; Mirza and MacGregor 1979).

To determine the statistical parameters of composite girder moment resistance considering these variables, a Monte Carlo simulation was conducted. Practically, this involves randomly assigning values to the random variables (based on their statistical distributions), computing the

ultimate capacity of the resulting composite girder using the standard method, then repeating the process to obtain a sufficient number of capacity data, from which mean value and coefficient of variation can be calculated. The simulation was repeated, increasing the number of simulations each time, until further increases did not appreciably alter the results. This typically required from 1000 to 3000 simulations.

For FM the resulting bias factor is $\lambda = 1.07$ and coefficient of variation is $V = 0.08$. For P the bias factor is taken as $\lambda = 1.05$ and $V = 0.06$ (Nowak 1999). The final resistance parameters are $\lambda = 1.12$ and $V = 0.10$. The distribution function of resistance is approximately lognormal. The results are verified by the values obtained by Nowak (1999), and were the same as those used for the AASHTO LRFD Code calibration.

Secondary element and diaphragm stiffness were also initially included as random variables (due to variations in dimensions and material moduli). However, analysis indicated that these variations have an insignificant effect on girder load distribution (and hence girder reliability). This is primarily because girder load distribution is relatively insensitive to small variations in secondary element stiffness when typical sizes of these elements are used (Eamon & Nowak 2002). Therefore, these variations are not included further in the analysis.

Reliability Analysis

Girders are designed as close as possible to the AASHTO LRFD Code (1998) minimum specifications for moment strength design, and all considered girder resistances are no greater than 3% of that required by Code (and verified for shear capacity). Although the exact minimum required resistance was used for reliability analysis, a representative girder stiffness was needed to determine load distribution. Girders were not specifically designed to satisfy the “optional”

deflection limit state, as this would result in artificially high strength reliabilities. However, the LRFD Code does allow continuous barriers to be included in the analysis for serviceability checks. With this consideration, the bridges of this study also satisfy the optional deflection limit state as well. Note that for girders properly maintained and designed for moment resistance, shear does not typically govern composite steel girder reliability (Yamani 1992). The total mean load effect (moment) to the governing composite girder is found from the finite element model, where truck weights are multiplied by bias factor λ and dynamic load factor 1.10. Dead load is accounted for by including gravity effects in the analysis, while increasing component weights with their respective bias factors. Reliability index (β) is calculated using the Rackwitz-Fiessler procedure (Rackwitz and Fiessler 1978), an iterative process that approximates non-normal distributions with equivalent normal distributions at the design point. In this case, total load effect is normally distributed while resistance is lognormal.

Simulations indicated that the dominant live load case for the bridges considered both with and without (Nowak 1999) edge-stiffening elements was two simultaneous trucks side-by-side. Here, interior girders experience a significantly higher load effect than exterior girders, particularly when edge-stiffening elements are present. Although a single truck load case could in many cases (especially when edge-stiffening elements were absent) produce the greatest moment on an exterior girder, the resulting reliability of this case was significantly higher than the two-truck load case for interior girders. Thus overall bridge girder reliability is governed by interior girders and only these are considered further. The resulting reliability indices, using the models discussed above, are presented in Figures 3-10. In the figures, 'base' refers to the bridge without considering any secondary element effects. As expected, in this case the girder reliability indices are uniform and equal to the values computed by Nowak (1999), as the LRFD Code is

calibrated for these structures. Typical combinations of secondary elements (barrier + diaphragm and barrier + sidewalk + diaphragm), however, can increase the reliability index (from about 0.5 to 1), depending on element stiffness and bridge geometry.

There are several apparent trends in the data. Figures 6-10, which plots the variation of β with girder spacing, indicate that secondary elements increase reliability to a greater degree on structures with closely-spaced girders. This matches the results predicted from the finite element analysis, whereby the edge-stiffening elements have shown to be most effective in aiding load distribution at bridges with closely-spaced girders. Although diaphragms roughly have the reverse result to a lesser degree, becoming more effective at wider girder spacings, their effect on load distribution is overpowered by edge-stiffening elements.

Figures 3-5, which plot reliability index as a function of bridge span, indicates a peak β close to the 20m span structure. This trend is caused by the interaction of GDF and the changing proportion of dead load to total load as span length changes. Edge-stiffening and diaphragms primarily benefit live load distribution. These elements have an increasing influence on live load distribution as span increases (the elements are more effective on the less-stiff spans, which tend to be longer). However, as bridge span increases, dead load accounts for a larger proportion of the total load effect. Thus, as span increases, the effects of secondary elements on the total load decrease.

The result is that the shortest spans benefit little, as secondary elements affect live load distribution only marginally at short spans. The longest spans also benefit little, as although live load distribution is affected significantly here, dead load accounts for a large proportion of the total load. Mid-range spans thus benefit most from secondary elements, where the spans are long enough so that the effect of secondary elements on live load distribution is significant, but short

enough that live load remains a large portion of the total load effect. This is shown in Figure 11, which for a 3m girder spacing bridge with barrier, sidewalk, and diaphragm, plots the two opposing trends of secondary element effectiveness (GDF_{base}/GDF_{actual}) and proportion of live load to total load effect ($LL/(LL+DL)$). Also on the graph is an indication of the interaction of these two trends, total girder resistance divided by the total load ($Resistance / Load$), which peaks around 20m (as does reliability index).

Recommendations

To provide a uniform level of reliability to bridges designed with positively-connected secondary elements, the load distribution effects should be accounted for. One way that this can be done is by adjusting girder distribution factor appropriately, an approach which would be compatible with existing LRFD Code format and is straightforward to implement. The GDF equations presented above may be used for this purpose, but can be simplified greatly if the following observations are considered:

1) Diaphragms were found in most cases to have little effect on load distribution and hence reliability. It would therefore be reasonable to neglect equation (2).

2) The equations presented were formulated to match results for a wide range of parameters, many of which are not typically used in current bridge design practice (for example, a wide girder spacing on a short span bridge, or very stiff/very flexible secondary elements). Results of the most typical design geometries (in terms of span, girder spacing, and secondary element stiffness combinations) are given in Figures 12-14.

3) The complexity of the equations may imply a predictive accuracy which does not exist for actual structures, as other factors specific to particular structure may affect load distribution as

well (partial bearing fixity, deterioration, other alterations to ideal boundary conditions and stiffness parameters).

From the observations stated above and taken from Figures 12-14, the following load distribution effects can be expected and can be recommended for typical secondary element configurations when these elements are positively connected to the structure:

- Diaphragm alone -- no reduction in GDF
- Barriers -- reduce GDF by 10%
- Sidewalks -- at 10m and shorter spans, no reduction in GDF; at 25m and greater spans, reduce GDF by 10% or 15% for girder spacing of 3m or 2m, respectively.

Note that barrier and sidewalk GDF reductions are additive. Linear interpolation is reasonable.

If structures are designed or evaluated considering the load distribution effects of secondary elements, the reliability indices will become more uniform from one structure to the next. However, the overall effect is that design moment is lowered and thus reliability is lower than that intended. Therefore, an additional load factor γ is needed to account for this. Reliability indices adjusted for the load distribution effects of secondary elements with $\gamma = 1.15$ are given in Figures 12-14 (indicated as “NEW bar. + dia.” and “NEW bar. + s.w. + dia.” on the graphs). Here the values for base, barrier + diaphragm, and barrier + sidewalk + diaphragm are copied from Figures 3-5 for comparison. In this case, γ is chosen such that the reliability index for the bridge geometry that was considered to represent the ‘target’ index for the AASHTO LRFD Code calibration (18m span, 1.8m girder spacing; see Nowak 1999) is unchanged. Notice on the graphs that, when secondary elements are accounted for, the target bridge happens to represent the most reliable of the cases considered. This is by coincidence, but results in the majority of

the structures to have β 's raised above their previous levels. This is the most conservative target, but may not be the best choice if the safety of the majority of existing bridge designs is deemed satisfactory. Clearly, other choices of γ are also possible which may represent a more desirable 'average' safety level.

It should be emphasized that the process presented does not alter the currently accepted safety level of existing bridges, but rather strives to increase the uniformity in safety level from one structure to the next. Therefore, even though the load distribution effects of secondary elements are not considered in traditional bridge design procedures, no increases in live loads are warranted for analyses that do include these elements, as doing so would lower the safety level of new bridge designs. This is because, as noted earlier, although secondary elements are not designed to affect load distribution, they do so even at extreme overloads, and thus affect girder reliability when not intended to.

Conclusions

The reliability of bridge girders was determined considering the effects of barriers, sidewalks, and diaphragms positively connected to the structure. The results indicate that significant differences in reliability may exist between structures with and without these elements. The target safety level can be obtained by accounting for secondary elements in design. A practical solution, in lieu of conducting a finite element analysis for each bridge designed or evaluated, is to adjust the current LRFD GDF formula. Such an approach is compatible with existing LRFD Code format and is straightforward to implement. Additional considerations come with this task, however, such as insuring the initial and continuing contribution of secondary element stiffness to the structure. Not only would this involve verifying the rigor of initial detailing, but also the

long-term effects of stiffness deterioration. These tasks are significant and beyond the scope of this study.

Although this study focused on simple span structures, which represent the majority of existing steel bridges (FHWA 2001), many modern structures are continuous. As girders in continuous span bridges typically experience lower maximum load effects and slightly better transverse load distribution, the expectation is that these structures would display somewhat less reliability index variation caused by secondary elements. Trends would remain similar, however. As the current design trend is to use fewer bridge girders, this study explored structures with large girder spacings. Most (older) existing structures have smaller girder spacings (1.5-1.8 m). As overall load distribution considering secondary elements tends to be more uniform with smaller girder spacings, results here can be considered conservative for these structures.

For now, even without accounting for secondary elements in design, it is clear that girder bridges in general have better load distribution than assumed. The load distribution equations and recommendations presented can be used without reference to reliability index, such as to approximate load distribution effects without conducting a more rigorous (but potentially more accurate) analysis. Results might be used for service load evaluations such as those for fatigue investigations, for example. As load distribution results here are based primarily on calibrated numerical models, however, significant additional experimental studies, particularly with loads at girder ultimate capacity levels, are warranted before any codification can be recommended.

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Table 1. Resistance Random Variables

Fig. 1. Idealized Bridge Cross-Sections

Fig. 2. Typical Finite Element Model

Fig. 3. Reliability Index as a Function of Span, 2m Girder Spacing

Fig. 4. Reliability Index as a Function of Span, 3m Girder Spacing

Fig. 5. Reliability Index as a Function of Span, 4m Girder Spacing

Fig. 6. Reliability Index as a Function of Girder Spacing, 10m Span

Fig. 7. Reliability Index as a Function of Girder Spacing, 20m Span

Fig. 8. Reliability Index as a Function of Girder Spacing, 30m Span

Fig. 9. Reliability Index as a Function of Girder Spacing, 40m Span

Fig. 10. Reliability Index as a Function of Girder Spacing, 50m Span

Fig. 11. Typical Load Proportion as a Function of Span

Fig. 12. Typical Moment Reduction due to Diaphragms

Fig. 13. Typical Moment Reduction due to Barriers

Fig. 14. Typical Moment Reduction due to Barriers and Sidewalks

Fig. 15. New Reliability Indices as a Function of Span, 2m Girder Spacing

Fig. 16. New Reliability Indices as a Function of Span, 3m Girder Spacing

Fig. 17. New Reliability Indices as a Function of Span, 4m Girder Spacing

Table 1. Resistance Model Random Variables

Random Variable*	Bias Factor λ	Coefficient of Variation V
Concrete		
Compressive Strength	0.95-1.08	0.15-0.18
Deck Thickness (t)	1.0	0.4/t
Barrier Height (h)	1.0	0.4/h
Barrier Width (w)	1.0	0.4/w
Sidewalk Thickness (s)	1.0	0.4/s
Sidewalk Width (l)	1.0	0.4/l
Reinforcing Steel		
Yield Strength	1.03	0.022
Area of Rebar	1.0	0.015
Distance, steel to slab top (d)	1.0	0.7/d
Girder and Diaphragm Steel		
Yield Strength	1.06-1.13	0.10-0.12

*Strength distributions are lognormal; geometry distributions are normal.

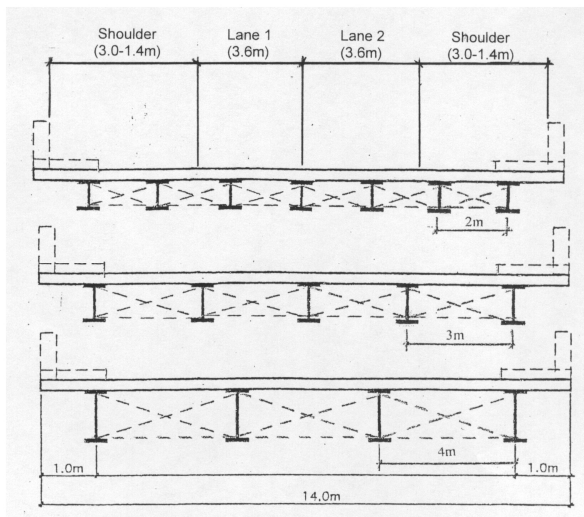


Figure 1. Idealized Bridge Cross-Sections

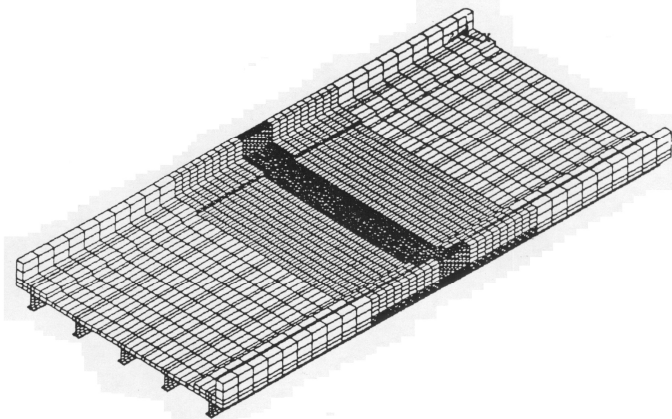


Figure 2. Typical Finite Element Model

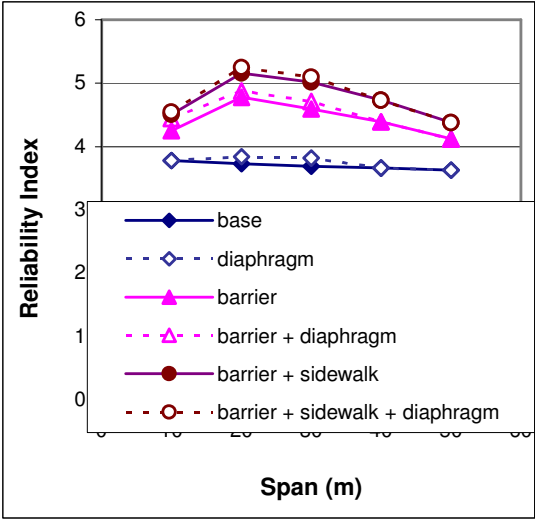


Fig. 3. Reliability Index as a Function of Span, 2m Girder Spacing

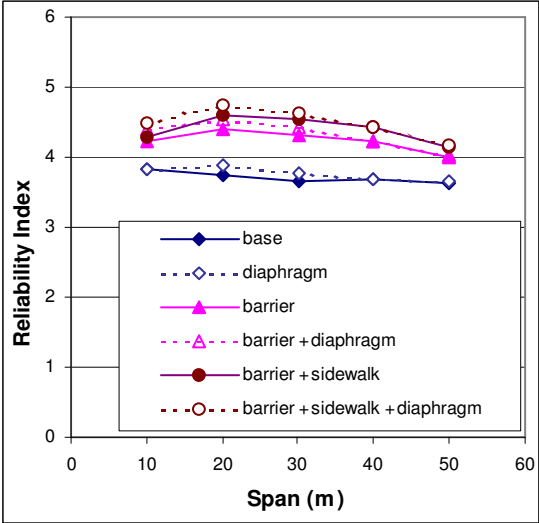


Fig. 4. Reliability Index as a Function of Span, 3m Girder Spacing

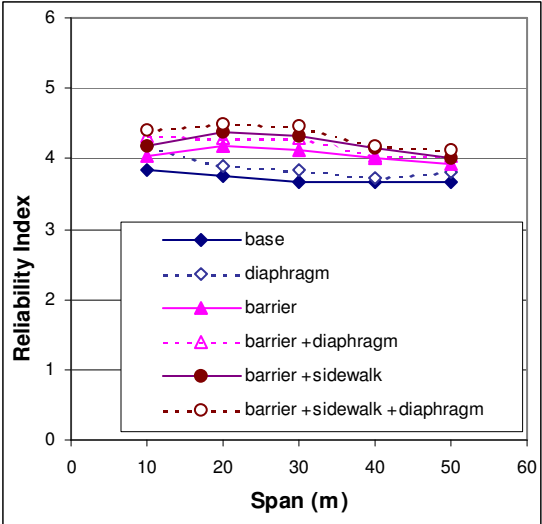


Fig. 5. Reliability Index as a Function of Span, 4m Girder Spacing

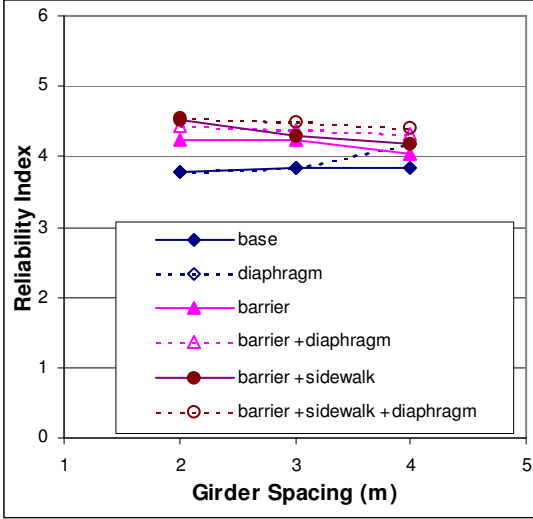


Fig. 6. Reliability Index as a Function of Girder Spacing, 10m Span

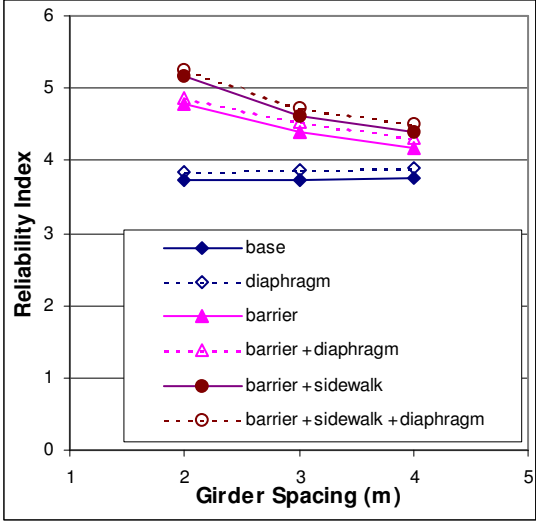


Fig. 7. Reliability Index as a Function of Girder Spacing, 20m Span

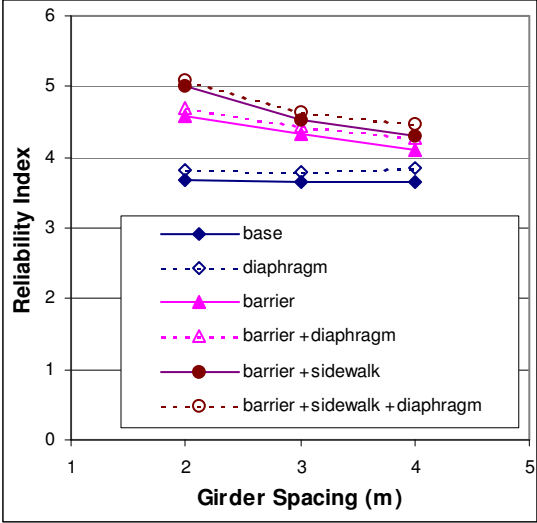


Fig. 8. Reliability Index as a Function of Girder Spacing, 30m Span

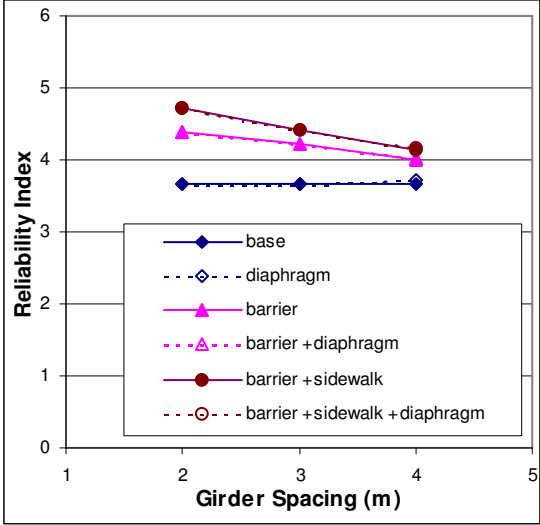


Fig. 9. Reliability Index as a Function of Girder Spacing, 40m Span

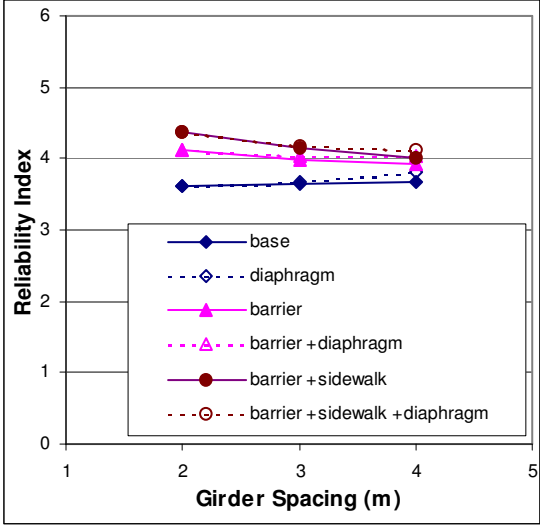


Fig. 10. Reliability Index as a Function of Girder Spacing, 50m Span

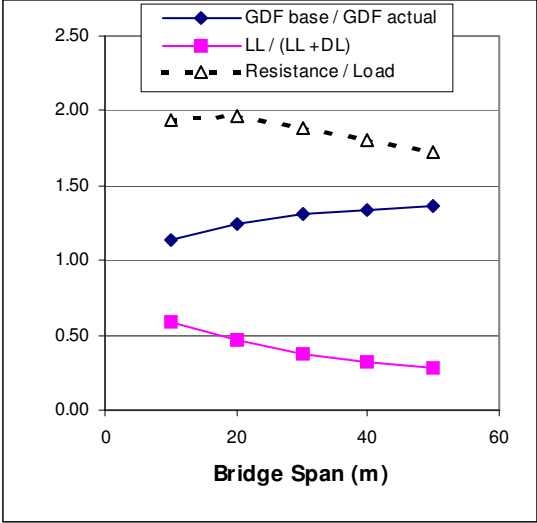


Fig. 11. Typical Load Proportion as a Function of Span

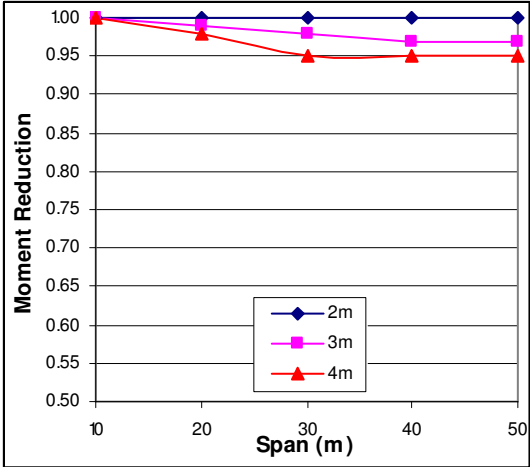


Fig. 12. Typical Moment Reduction due to Diaphragms

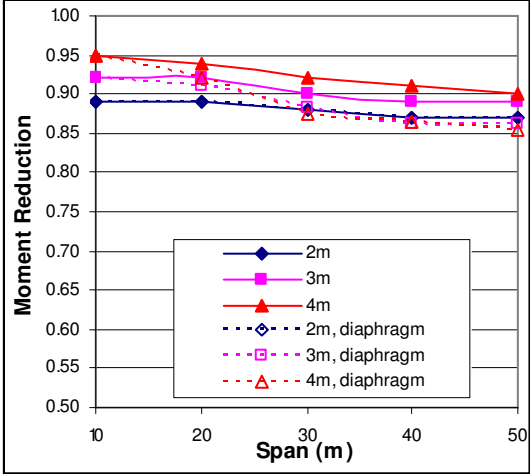


Fig. 13. Typical Moment Reduction due to Barriers

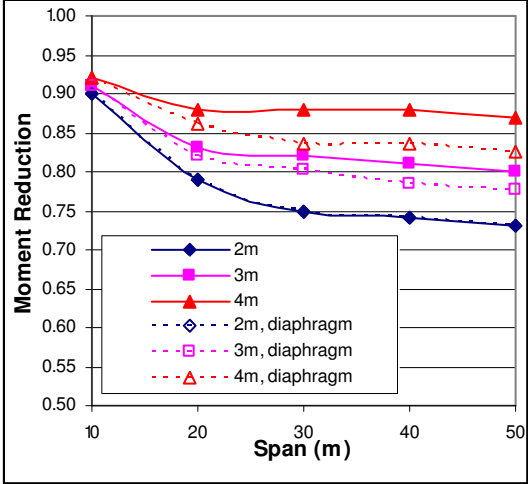


Fig. 14. Typical Moment Reduction due to Barriers and Sidewalks

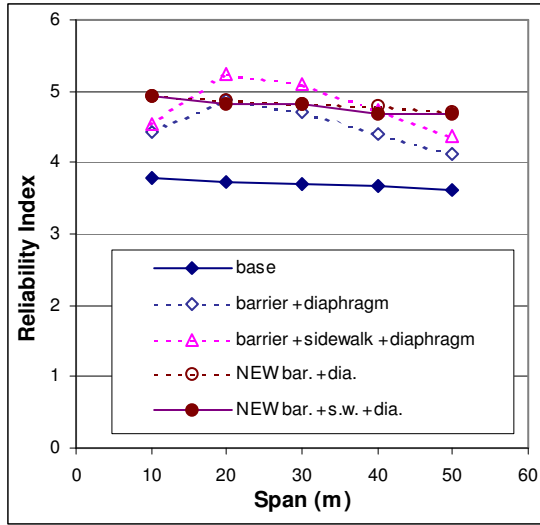


Fig. 15. New Reliability Indices as a Function of Span, 2m Girder Spacing

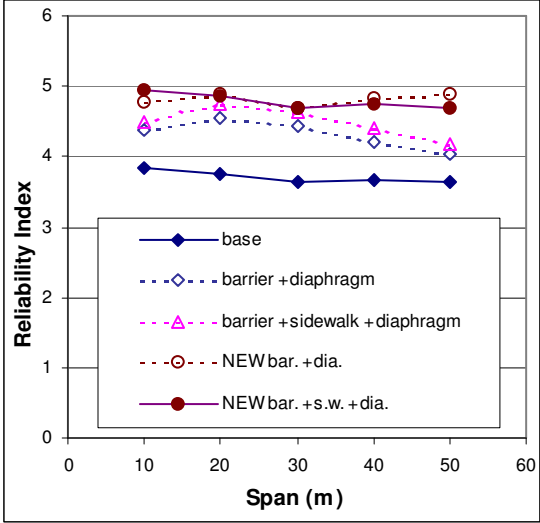


Fig. 16. New Reliability Indices as a Function of Span, 3m Girder Spacing

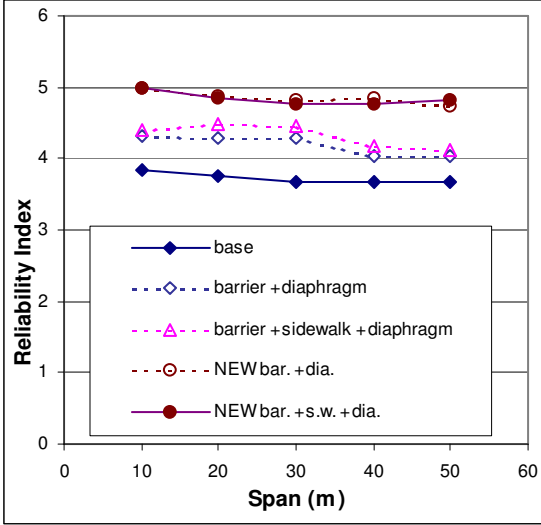


Fig. 17. New Reliability Indices as a Function of Span, 4m Girder Spacing

Notation List

K_d Diaphragm stiffness ratio

K_e Edge stiffness ratio

L Girder spacing (m)

S Bridge span (m)

V Coefficient of Variation

α_d GDF adjustment factor for diaphragms

α_e GDF adjustment factor for edge-stiffening elements

β Reliability Index

λ Bias factor