Composite Box-Girder Bridges During Construction

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A 1/20 scale-model study of simple-span girders was undertaken to examine the effect of a typical construction loading on both single open-section and quasi-closed-section girders and interconnected girders. Deflections, distortion, and stresses resulting from the torsional loading on these girder systems were observed. The response of an open, simple-span girder, braced to minimize distortion, can be predicted with available theories for mixed torsion. A quasi-closed girder generally behaves as a closed section and St. Venant shear stresses resist applied torsional loadings. However, the quasi-closed girder was found to have only 40 percent of the theoretical torsional stiffness value. Studies of interconnected boxes with end diaphragms and simple tie bracing at the one-third span points indicate that simple tie bracing significantly reduces the torsional rotation of open sections. This bracing can remain in place without affecting the appearance of the girder system. Bracing is necessary to maintain the stability of open sections during construction. Considering symmetry can help to minimize torsional construction loads on individual girders. Geometrically and structurally stable configurations can be designed for construction loadings by using simple bracing schemes.

Composite steel-concrete box-girder bridges have been favored in many locations in North America for intermediate-span bridge structures since design procedures were outlined in the mid-1960s (1, 2). Since these design procedures were accepted, simple- and continuous-span structures with individual spans varying from 20 to 100 m have been used for river crossings and grade separations at urban intersections.

The completed structure, which is aesthetically pleasing, consists of steel box girders made composite with a concrete deck slab. Economies in design are possible because the completed box-girder section has a higher torsional stiffness and a greater lateral distribution of live load than an I-beam structure with similar flexural strength. Additional economies are also possible in the fabrication and erection of box girders, compared to similar I-beam structures, because of the elimination of much of the wind and transverse diaphragm bracing.

A number of cross-sectional geometries of the completed structure are possible; the number of girders can vary from two to six or more depending on the plan geometry of the structure. Three typical cross-sectional geometries for two-box-girder systems, with single and double bearing arrangements and shallow and deep end-support diaphragms, are shown in Figure 1. Typical spacing of the centerline of the box girders varies from 4.3 to 6.7 m, and the depth of the cast-in-place concrete deck varies from 190 to 250 mm. The ratio of dead-load moment to live-load moment for box-girder structures will thus depend on the cross section, the depth of the deck, and the span arrangement chosen by the design engineer.

Experience with the construction of box-girder systems has indicated that the design specifications of 1968 to 1973 (1, 2, 3) do not provide design or construction engineers with sufficient guidance regarding the behavior of thin-walled flexible box girders during construction. The specifications do not clearly identify the need for intermediate diaphragms or cross-frames that would retain the cross-sectional geometry of the girder during fabrication and handling. Structural steel fabricators have often added bracing within the girders, but calculation procedures allowing for the design of such bracing are not given in the specifications or the design criteria (1, 2). The responsibility for the bracing of girders during construction is frequently given to the general contractor, who normally does not have access to the structural design calculations. Designing construction bracing systems without referring to the original design calculations can be difficult.

The current Canadian specification (4) does refer to forces acting during the construction phase and indicates that diaphragms (bracing) shall be used to cater for flexural distortional and warping stresses. But selection of calculation procedures for the construction-phase loading is left to the designer. This situation can lead to the use of box girders with excessive bracing and thus result in waste of material and labor.

Many of the construction difficulties mentioned above appear to have arisen from a misapplication of research data on composite box-girder systems. The research data that support the current design specifications for the analysis and proportioning of box-girder systems (5) appear to apply only to the characteristics of live load distribution for straight bridge spans of up to about 45 m. Bridge models at one-quarter scale were considered in the studies, but there was no consideration of dead-load similitude. Thus, dead-load force effects in an equivalent prototype structure were not fully represented in the research nor in the subsequent specifications (1, 2, 3).

This paper describes the possible loading configurations present during construction on a thin-walled box-girder system and examines the resulting stresses induced in typical torsionally open and torsionally quasi-closed box girders. The stress analyses are based on the results of 1/20 scale-model tests supplemented by a mixed torsion analysis of individual open and quasi-closed box sections. A variety of bracing systems are discussed that will minimize the additional longitudinal stresses caused by torsion in open box-girder systems.

NOTATION

The following notation was used in this study of construction loading on box-girder bridges:

b = average width of girder
D = spacing of distortional bracing
e = eccentricity of loading with respect to centerline of girder
e_f = eccentricity caused by formwork
e_p = eccentricity caused by finishing plant
e_v = eccentricity caused by vertical load
e_w = eccentricity caused by wind
h = height of girder
I_x = moment of inertia about x axis
I_y = moment of inertia about y axis
I_ω = sectorial moment of inertia
l, L = span length
m = distributed torsional moment
m_c = moment caused by concrete
m_f = moment caused by formwork
m_v = moment caused by vertical load
m_w = moment caused by wind
W_c = weight of concrete per unit length

Figure 1. Typical cross-sectional geometries for three box-girder systems.

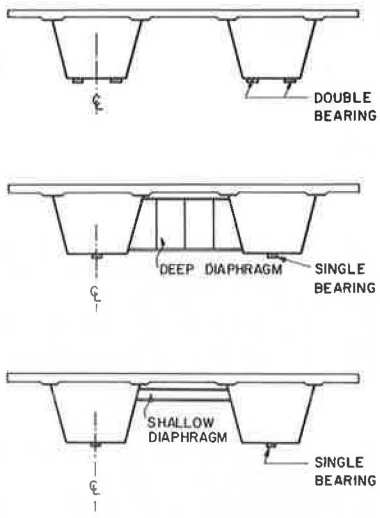
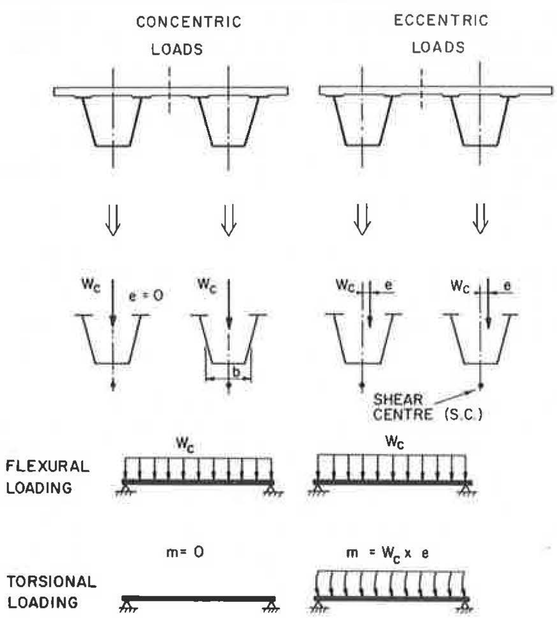


Figure 2. Loading as a result of concrete deck.



W_v = vertical load per unit length
W_w = wind load per unit length

CONSTRUCTION LOADINGS

As part of the design process the design engineer selects a sectional geometry and establishes the position of the concrete deck relative to the centerlines of the individual box girders and the complete box system. Two examples of cross-sectional arrangements are shown in Figure 2. In the concentric-loading case, the weight of the fresh concrete results in only flexural stresses. The compression flanges of each box girder require bracing to avoid the lateral buckling that results from the combined action of vertical load and the horizontal component of the force in the web.

As a result of geometric or other constraints, the designer may choose a cross-sectional geometry in which the deck concrete gives rise to eccentric loading of the individual girders in the simplified box-girder structure (Figure 2). This case gives rise to a uniformly distributed load (W_c) located at a transverse eccentricity (e) with respect to the centerline of individual box girders and results in both flexural and torsional loadings as shown in Figure 2.

The end reactions provide a restraint for shear caused by the vertical loading. Restraint for the torsional loading is also required and can easily be provided as part of the design of end-support systems 1 and 2 in Figure 1. The shallow diaphragm and single-support case (end-support system 3 in Figure 1) requires special design and detailing to ensure adequate strength and stiffness for torsional loading.

Other construction loadings that give rise to predictable torsional effects during design include wind loading of the exposed girders, the finishing machine, and the formwork. Additional torsional loadings result from the concrete-handling systems chosen by the general contractor and the unsymmetrical placing of deck concrete. These loadings can be minimized by careful specification.

Construction-stage loadings for a number of Canadian box-girder bridges were examined. Typical vertical loads resulting from concrete plus formwork were found to be between 25 and 40 kN/m, and the horizontal loads due to wind were found to be between 2 and 3.5 kN/m. These load types result in shear and bending moment diagrams that have forms familiar to designers. The flexural loads act eccentrically to the shear center and give rise to torsional loading. The ratio (e_v/b) of the eccentricity of the vertical loads from the centerline of the girder (e_v) to the average girder width (b) varies from 0 to 0.14 (the shear center lies on the axis of symmetry). The analysis of the typical box-girder sections indicates that the vertical and horizontal loads and associated torsional loads vary from structure to structure. The values of torsional loading are controlled by the design engineer's choice of cross-sectional geometry.

MODEL STUDIES

A series of scale-model tests of thin-walled members was initiated to develop a more complete understanding of the distortional and warping stresses that occur in eccentrically loaded box-girder systems during construction. These tests relate to the response to flexural and torsional loading of both single box girders and multiple interconnected box girders.

To establish the dimensions of the scale-model structures, the geometries of a series of prototype steel box-girder bridges provided by the Canadian Steel Industries Construction Council were examined; typical ratios obtained for base width to height, top width to height, and width to girder spacing are given in Table 1. In addition, ratios of width or height to thickness were also calculated for top and bottom flange plates and webs. These data were used to proportion a cross-sectional geometry representative of a typical structure with unstiffened webs (Figure 3). The cross-sectional geometry of the resulting model structure, scaled to $\frac{1}{20}$, is shown in Figure 4. To develop this geometry it was necessary to make minor adjustments to the results of the nondimensional study of prototype structures.

Figure 5 shows the two types of box girders that were developed by using the basic model geometry: (a) a torsionally open section that included distortional bracing to minimize possible cross-sectional distortion of the model under the action of torsional load and (b) a torsionally quasi-closed section, which was the result of including top chord bracing in the model. The distortional bracing was located at a spacing of three member depths for both types of box girders (Figure 6) and is representative of prototype structures (Table 1). The plan arrangement of the top chord bracing for the quasi-closed box girders is also shown in Figure 6. Rods were

Table 1. Typical prototype geometries of steel box-girder bridges.

		Span		Girder						
Bridge	Type	1 (m)	2 (m)	b (m)	h (m)	b/h (m)	e_v (mm)	e_v/b Ratio	Web Slope	D/h
a	C[a]	42.4	42.7	2.06	1.68	1.23	–	–	4.4/1	9.2
b	C[a]	64.0	77.7	2.67	2.90	0.92	172	0.064	4.2/1	2.7
c	S[b]	63.4	–	2.01	2.01	1.00	120	0.060	4.3/1	3.9
d	C[a]	59.4	76.2	2.29	2.44	0.94	122	0.044	5.3/1	3.1
e	S[b]	42.7	–	2.19	1.62	1.34	254	0.116	4.6/1	4.4
f	C[a]	44.5	56.4	2.40	2.26	1.06	229	0.095	2.4/1	5.0
g	C[a]	61.0	85.3	2.21	2.49	0.89	343	0.138	6.5/1	–

[a]Continuous-span system. [b]Simple-span system.

Figure 3. Cross section of typical prototype geometry derived from Table 1.

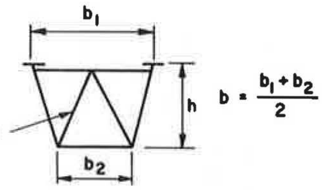


Figure 4. Cross-sectional geometry of prototype and model sections.

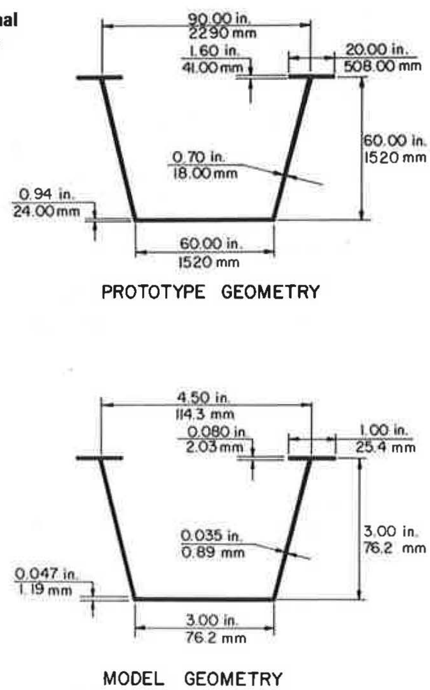


Figure 5. Open and quasi-closed members.

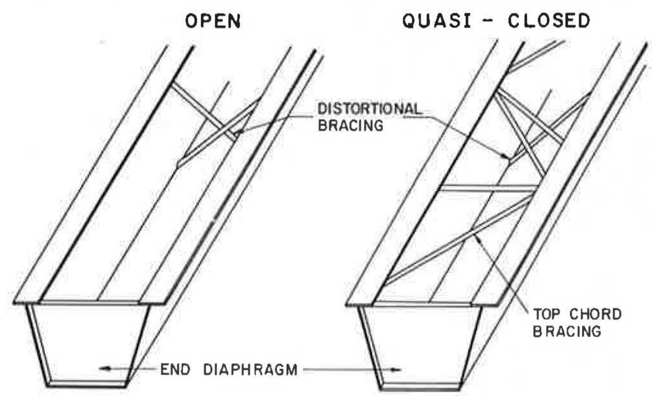
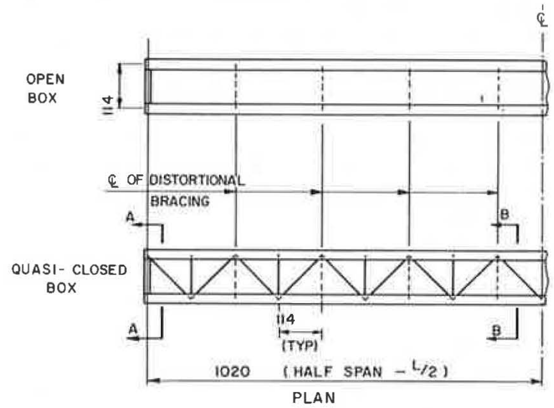


Figure 6. Plan of model girders.

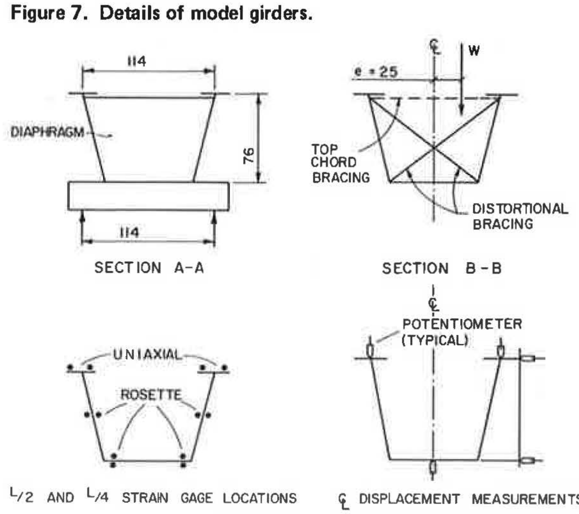


used throughout for the bracing members and were designed by using a maximum slenderness ratio of approximately 200. The models were carefully fabricated to minimize distortion and imperfections caused by welding. An aluminum jig was used to ensure that the desired geometry was maintained (6).

The models have a span length of 2030 mm and a total length of 2057 mm. The span-to-depth ratio of the simply supported system is approximately 27. The reaction system provided at each end of the models (Section A-A in Figure 7) consists of two load cells, a cross beam, and a solid end diaphragm that allows for transfer of shear forces from the webs of the model to the support and retains the sectional geometry at the supports. The load cells were located on the cross beam to coincide with the axes of the top flanges of the model. The load cells do not restrain flexural rotation or warping of the section, but the end diaphragms do provide a restraint on warping.

Loading was applied directly to the top flanges of the

Figure 7. Details of model girders.



model to simulate the effects of the placement of concrete deck. The load was applied to the model through two sets of wiffle-tree systems arranged so that load was distributed to 16 equal point loads on each flange. The wiffle tree did not restrain horizontal displacement. The intensity of this equivalent uniform load could be varied between each flange so that the resulting line of action of the total load is eccentric to the centerline of the model and the eccentric loadings illustrated in Figure 2 can be modeled. For convenience, an eccentricity of 25 mm was used in testing (Section B-B in Figure 7).

Models were instrumented with strain gauges at the midspan and quarterspan points and with displacement potentiometers at midspan. The strain gauges (Figure 7) allow longitudinal and transverse strain measurements to be made, and the potentiometer readings yield vertical and horizontal displacements as well as rotations of the model.

Two series of model tests were carried out to examine (a) single members under combined torsional and flexural loading and (b) the response of interconnected members when the loaded member was restrained by bracing.

Response of Single Box Girders to Eccentric Loading

Both the open and the quasi-closed members (Figure 5) were studied under uniform eccentric loading. The eccentricity value chosen (25 mm) corresponds to an e_v/b ratio of 0.27, which is approximately twice the maximum e_v/b ratio noted for prototype structures during construction (Table 1).

Figure 8 illustrates the applied load versus midspan rotation response for both the open and the quasi-closed members. The location of the shear center (the center of rotation) for both the open and the quasi-closed members was found to lie outside the section and generally in the position shown in Figure 8. This result was expected for the open section; studies continue on calculation procedures for the location of the shear center of a quasi-closed box section.

The load-rotation response (Figure 8) of the open section is nonlinear but recoverable. The nonlinear response results from the increase in the torsional load resulting from the horizontal deflection of the relatively flexible thin-walled open section and the corresponding increase in torque (this is similar to the $P - \delta$ effect in eccentrically loaded slender columns). The initial portion of the load-deflection curve of the open section corresponds to the first-order analysis value, and the second-order analysis agrees favorably with the latter portion of the curve when the additional torque caused by the horizontal deflection of the member is included.

The load-rotation response of the quasi-closed member is linearly elastic and indicates that the torsional stiffness of this member is approximately 20 times that of the open-section member. The theoretical stiffness value for a quasi-closed section (8) is 50 times the open-section value. A similar ratio for theoretical to observed rotation has been observed for quasi-closed girders with span-to-depth ratios of 10 (7). Although the torsional stiffness value of the quasi-closed section is significantly larger than that of the open section, the reasons for the stiffness value for the quasi-closed section being less than the theoretical value are presently being examined.

The open-section member is subjected to two types of longitudinal stress: (a) flexural bending and (b) warping restraint. Because of the low torsional resistance of an open section, torsion in sections similar to that shown in Figure 4 is carried primarily by restrained warping. For the model section, the loading pattern, and the span studied, the total torque is resisted by both the warping restraint (warping torsional moment) of 85 percent and a St. Venant torsional moment of 15 percent. The longitudinal stresses developed by the warping torsional moment are significant for the extreme case of $e_v/b = 0.27$. Calculated values based on analyses developed by Vlasov (8) and Kollbrunner and Basler (9) were outlined by Harris (10). The results obtained from the mixed torsion analysis compare closely with Harris' observed values.

The longitudinal stress results for the quasi-closed section are basically those predicted by elementary beam theory. The quasi-closed section acts as a closed section and the torque is resisted by a St. Venant torsional moment.

Prototype Response

Because the results obtained from the model studies do not apply directly to a prototype structure, the prototype section (Figure 4) was analyzed for combined vertical concrete and formwork loading to develop an understanding of the stresses and displacements that might develop in such a structure. An eccentricity ratio (e_v/b) of 0.15 was used to determine an eccentricity of 285 mm for the vertical load from the centerline.

Section properties for the prototype span are as follows: Shear center distance below the geometric centroid = 1550 mm, $I_x = 5.58\ (10^{10})\ mm^4$, $I_y = 1.13\ (10^{11})\ mm^4$, and $I_\omega = 1.69\ (10^{16})\ mm^6$. These properties were used in a mixed torsion analysis to give the following deformations caused by the eccentric line load (e_v = 285 mm) of 30 kN/m over an unsupported span length of 39.6 m: Midspan rotation = 0.0674 rad, midspan vertical deflection = 81 mm, and top-flange horizontal deflection at midspan = 157 mm. Girder movements of such a magnitude during construction would be unacceptable to the design engineer. Bracing of the torsionally open girder is necessary. The horizontal displacement of such a girder would be reduced by approximately $1/20$ if the eccentrically loaded open-section box girder were converted to a quasi-closed system or if the midspan section were restrained from rotating by suitable bracing.

Values for bending, warping, and total longitudinal stresses are shown in Figure 9 (in megapascals) for the loading case considered. The flexural longitudinal stresses are amplified appreciably when the open-section member is eccentrically loaded without intermediate

Figure 8. Rotation versus applied load for model girders.

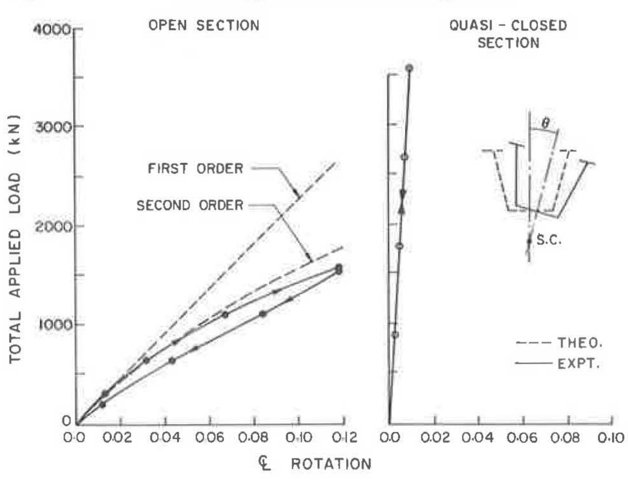


Figure 9. Longitudinal stress values for typical prototype structure.

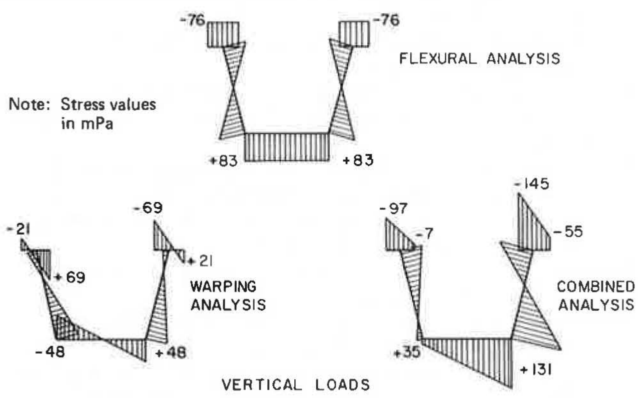


Figure 10. Interconnected model box-girder systems.

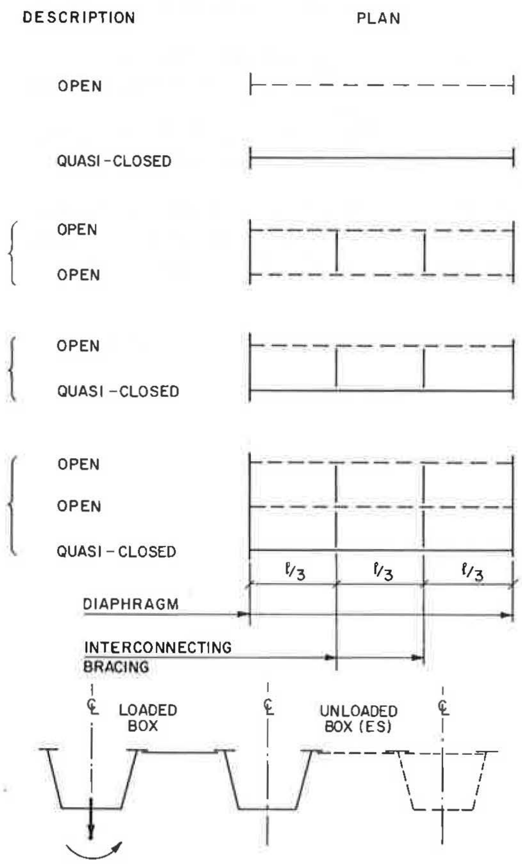
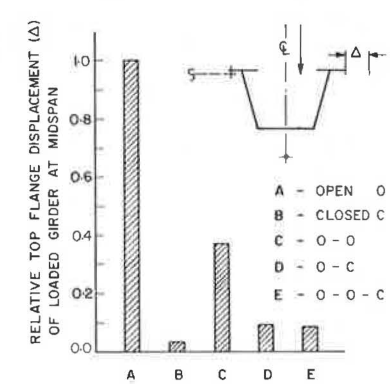


Figure 11. Relative horizontal displacements of open, quasi-closed, and interconnected box-girder systems.



bracing within the span. An analysis of horizontal wind-force effects assumed a horizontal wind load of 1.2 kPa on the vertical face of the girder. The maximum wind stress is 20 MPa, approximately one-quarter of the gravity load bending stress. Wind-induced rotations are approximately one-third of the eccentric gravity load rotation.

Interconnected Model Box-Girder Studies

A series of tests was carried out on the box-girder arrangements shown in plan form in Figure 10. These were

1. A pair of connected open girders,
2. An open girder connected to a quasi-closed girder, and
3. A pair of connected open girders braced to a quasi-closed girder.

In each case one open box girder was loaded eccentrically and connected to the adjacent member(s) at the third points by a simple bar (Figure 10). Diaphragms were provided at the support points to ensure an adequate torsional restraint at the supports.

The loading case considered in Figure 10 assumes that only one girder is eccentrically loaded and that adjacent girders are subjected to a nearly concentric loading. The simple tie will not provide any restraint to torsional loading for the case of two connected open girders if the girders are eccentrically loaded so as to rotate in the same direction.

Figure 11 shows relative horizontal displacements measured at the elevation of the top flange for a single open and a single quasi-closed box girder as well as for three interconnected cases. Even in the case of two torsionally open box girders, a positive structural connection between two such girders will reduce horizontal displacements caused by eccentric loading. These horizontal displacements are reduced by more than 15 for systems that incorporate a single quasi-closed box girder as part of the multigirder system.

The single-tie bracing was chosen as the simplest and cheapest torsional restraint. Such a member may be retained in a completed structure without influencing the appearance.

DISCUSSION OF RESULTS

The rotation and horizontal movements observed during the construction of box girders that form part of a composite box-girder bridge structure can be attributed to a variety of torsional loadings, the sources of which include the concrete deck, wind, the finishing equipment, and the formwork. A torsional restraint at the supports is thus necessary to ensure overall stability of the member during construction. Additional bracing will frequently be necessary to minimize later movements and to ensure that large stresses are not built into the box-girder system. A similar conclusion was reached by Poellot (11).

Bracing should be supplied for a torsionally open cross section during construction to ensure that horizontal movements and associated longitudinal warping stresses are minimized. A number of possible bracing schemes are shown in preliminary form in Figure 10, and the relative deformations of these schemes under torsional loading are shown in Figure 11.

CONCLUSIONS

The probable causes of torsional loading of composite box-girder bridge structures during construction have been discussed. These torsional loadings can be identified at the design stage so that structures with adequate torsional restraint at the supports can be proportioned by the design engineer. Model studies were supplemented by the results of mathematical analyses to predict the response of torsionally open and torsionally quasi-closed sections to combined flexure and torsion.

Bracing is obviously required between supports to maintain the stability of open-section members during construction. Geometrically and structurally stable configurations can be designed for construction loadings by using a variety of simple bracing schemes.

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Effects of Diaphragms on Lateral Load Distribution in Beam-Slab Bridges

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The effect of diaphragms on the lateral distribution of live load in simple-span beam-slab bridges with prestressed concrete I-beams and without skew is presented. The computer-based analysis used the finite-element method for two previously field-tested bridges with span lengths of 21.8 and 20.9 m (71.5 and 68.5 ft). The first part of the investigation dealt with the extent of the participation of the midspan diaphragms in lateral load distribution. It was found that the reinforced-concrete midspan diaphragms contribute only about 20 to 30 percent of their stiffness to load distribution. In addition, when all design lanes are loaded the contribution of the diaphragms is negligible. The second phase of the research dealt with the effect of the use of multiple diaphragms on lateral load distribution. Numerical comparisons were made for cases in which the superstructure had a midspan diaphragm and diaphragms at third, quarter, and fifth points. When the vehicle was located so as to produce maximum bending moment in the bridge, it was found that the increase in the number of diaphragms does not necessarily correspond to a more even distribution of loads at midspan. It was also found that, if all the design lanes are loaded, the contribution of diaphragms is negligible regardless of the number of diaphragms used.

Lateral distribution of live load in simple-span beam-slab highway bridges with prestressed concrete I-beams and without skew is one of the critical aspects in the design of these bridge superstructures. Until recently provisions for load distribution were far from realistic (2, 3, 6). Recent investigations have refined provisions for the lateral distribution of live load not only in right bridges but in skewed bridges as well (3,6). One of the major design issues for bridge engineers, however, remains unresolved: the contribution of midspan diaphragms (or third-span, depending on the span length) used in highway bridges.

This paper, which provides a summary of the findings of an extensive analytical research project on load distribution in beam-slab bridges (3), including sufficient qualitative information for use by designers, attempts to answer two basic questions: