The Strutted Box Widening Method for Prestressed Concrete Segmental Bridges

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This paper introduces the strutted box widening method (SBWM), a system that allows a two-lane segmental bridge to be designed and constructed so that it can be easily widened into a three- or four-lane bridge at any time in the future. This solution is attractive because widening only needs to occur if and when traffic volumes warrant it. Two examples demonstrate how the SBWM can be used to widen a variable-depth cast-in-place segmental bridge and a constant-depth precast segmental bridge. Design and construction considerations of the SBWM are addressed, and the advantages and disadvantages of the SBWM are outlined. Two particularly appealing potential applications are described.

Two bridge examples introduce the strutted box widening method (SBWM). The first example is a variable-depth cast-in-place segmental bridge, while the second is a constant-depth precast segmental bridge. Both are built by the balanced cantilever method of construction. The concept can be applied to both precast and cast-in-place segmental bridges built by a variety of construction methods, including the balanced cantilever, span-by-span, incremental launch, and heavy lift methods.

Figs. 1 and 2 illustrate the SBWM concept for a variable-depth bridge and a constant-depth bridge, respectively. Note that the lane configurations are identical, although the structural system varies somewhat. Let us define three stages of construction and arbitrarily assign dates to these stages of construction. The initial construction of the bridge is two lanes plus shoulders in Year 2003 (Stage 1). The bridge is widened to three lanes plus shoulders in Year 2020 (Stage 2) and to four lanes plus shoulders in Year 2060 (Stage 3). The resulting deck widths are 13.8 m (45.3 ft) in Stage 1, 17.5 m (57.4 ft) in Stage 2, and 21.7 m (71.2 ft) in Stage 3.

What makes the SBWM such an attractive solution is its flexibility. Widening only needs to take place when traffic volumes warrant it. If the traffic volumes do not increase as fast as projected, widening can be delayed as long as necessary. If the traffic volumes increase faster than expected, the bridge can be widened from two to four lanes directly (i.e., it is not necessary to have an intermediate three-lane bridge). The configuration of the bridge at the end of its service life can be two, three, or four lanes.

The SBWM concept in essence is quite simple. During Stage 2 construction, exterior compression struts are installed and the deck slab is widened. Additional transverse internal prestressing tendons and longitudinal external prestressing tendons are installed and stressed. Widening from Stage 2 to Stage 3 construction is similar. The deck slab is again extended (cantilevered), and additional transverse internal prestressing tendons and longitudinal external prestressing tendons are installed and stressed.

The structural system varies depending on whether the bridge is variable depth or constant depth. A variable-depth bridge (Fig. 1) has all compression struts at the same distance from and the same angle to the deck surface. This means that interior compression struts are required to balance the exterior compression struts. This also means that web bending will occur due to unbalanced live load. A constant-depth bridge (Fig. 2) does not require interior compression struts since the force in the exterior compression struts is transferred through the bottom slab and taken in torsion. Hence, a constant-depth bridge is structurally more efficient and easier to design than a variable-depth bridge. However, both are still very good structural solutions. In order for the compression struts to be effective, they need to have an angle from the deck surface of at least 30 degrees. This means that the section depth may be governed by the compression strut geometry rather than the span-to-depth ratios.

Although strutted boxes have been widely used in the past as solutions for both segmental bridges and cable-stayed bridges, this represents the first time (to the author's knowledge) that the strutted box solution has been used as the basis for the future widening of a segmental bridge.

Podolny and Muller[1] describe a particularly interesting strutted box segmental bridge. The Kochertal Bridge in Germany is a nine-span constant-depth cast-in-place segmental bridge. The bridge has typical span lengths of 138 m (452.8 ft) and a section depth of 7.0 m (23.0 ft) (which gives a span-to-depth ratio of 19.7). The bridge is built in two stages. Stage 1 consists of constructing in balanced cantilever a box girder that has a deck width of 13.1 m (43.0 ft) and a soffit width of 8.6 m (28.2 ft). Stage 2 consists of casting 8.83 m (29.0 ft) cantilevers on a series of precast struts to give a total deck width of 30.76 m (100.9 ft) (to accommodate six lanes of traffic). Although the bridge is built in two stages, this was done to facilitate construction, rather than to accommodate future widening.

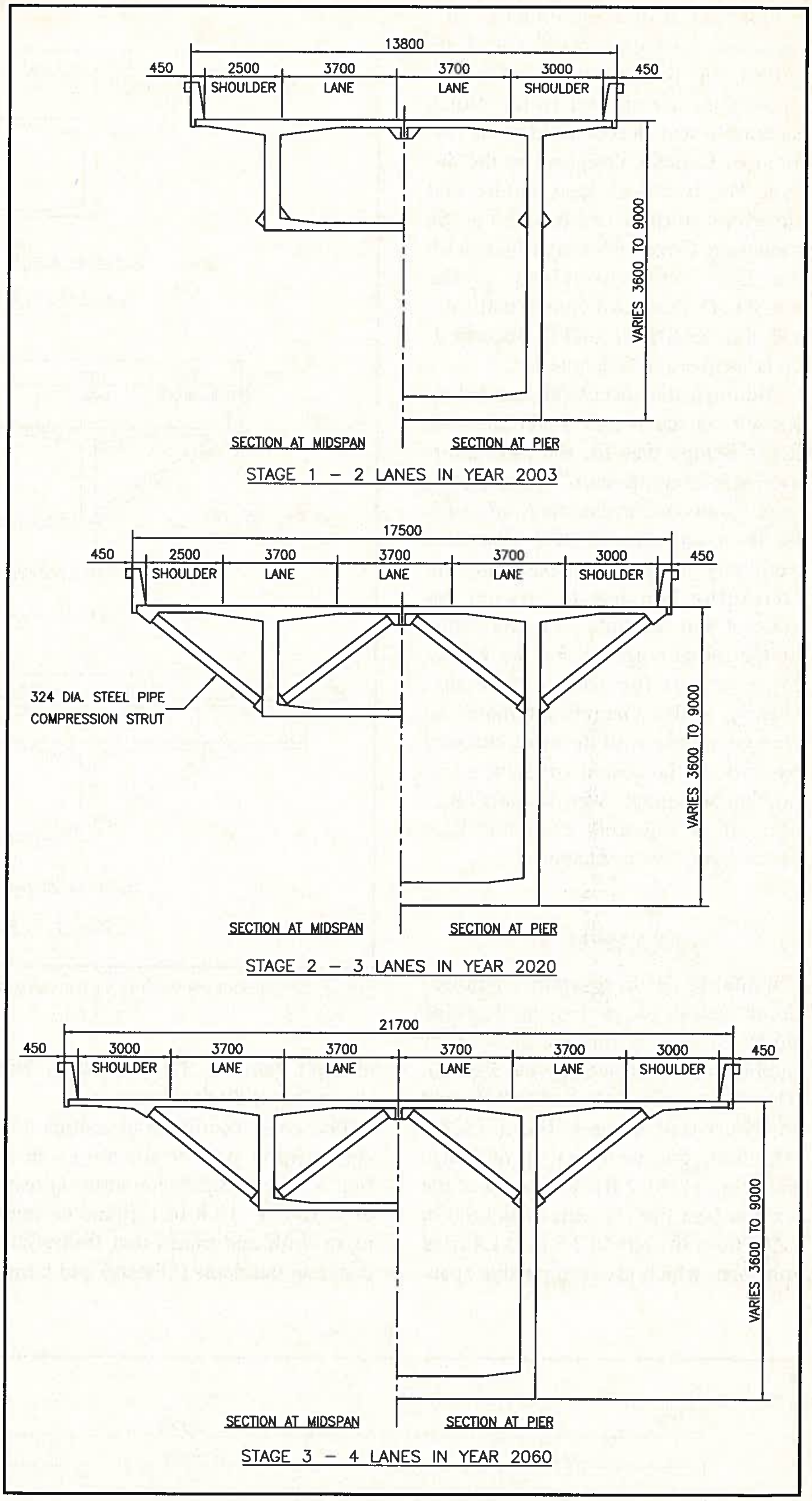


Fig. 1. Cross section widening for variable-depth bridge (Example 1).

The two bridge examples in this paper were designed for live load according to the Canadian Code[2,3] because they are similar to the North Saskatchewan River Bridge in Edmonton, Canada, designed by the author. The live load, lane widths, and shoulder widths are based on the Canadian Code. A comparison with the live load provisions of the AASHTO Standard Specifications[4] and the AASHTO LRFD Specifications[5] is given in Example 1.

Although the successful low bid by Kiewit on the North Saskatchewan River Bridge was for the steel alternate, representatives of Kiewit indicated to the author that their bid prices for the concrete and steel alternates were very close. This is especially encouraging because the owner requested that the bids be based solely on the initial cost, and not the widening costs or the life-cycle costs. Clearly, if the concrete alternate can be competitive with the steel alternate based on initial cost, it will have a significant advantage over the steel alternate when widening costs and life-cycle costs are considered.

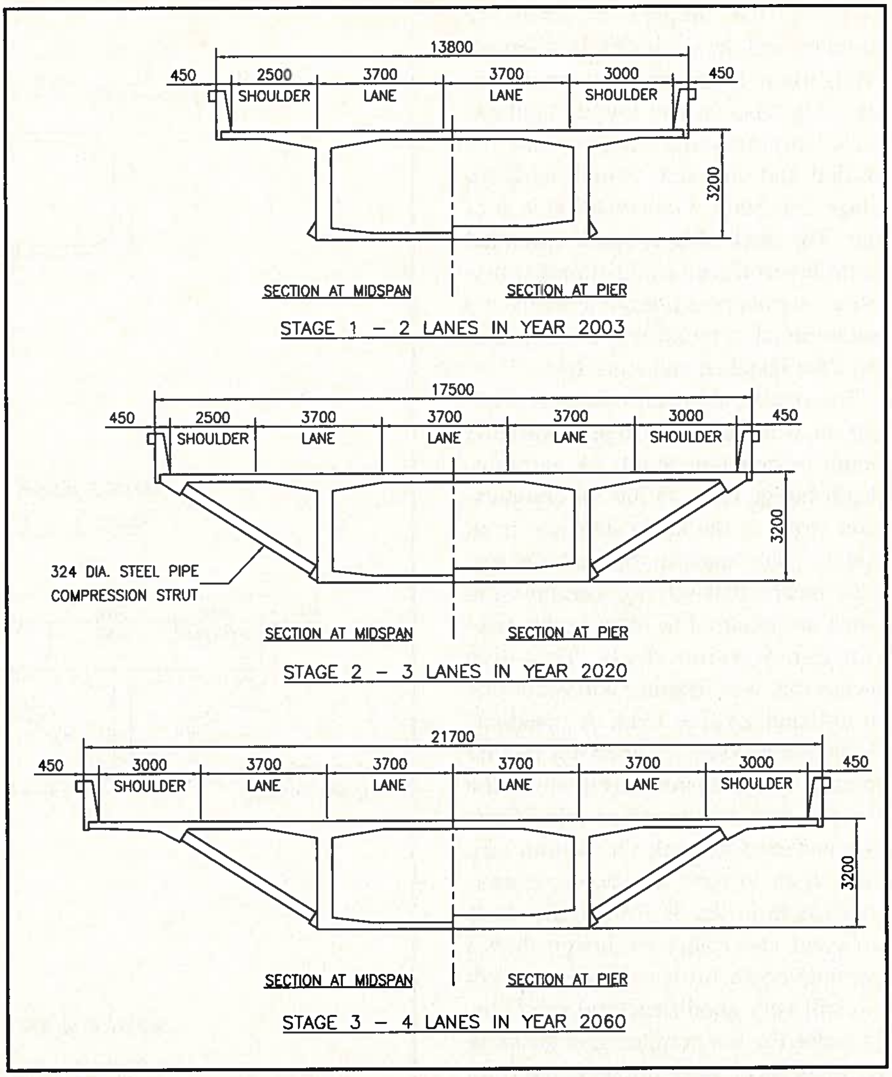


Fig. 2. Cross section widening for constant-depth bridge (Example 2).

EXAMPLE 1

Consider a three-span variable-depth cast-in-place segmental bridge built by the balanced cantilever method of construction (see Fig. 3). The main span is 160 m (525.0 ft), and the end spans are each 100 m (328.1 ft), giving an overall length of bridge of 360 m (1181.2 ft). The depth of the section (see Fig. 1) varies from 9.0 m (29.5 ft) at the pier to 3.6 m (11.8 ft) at midspan, which gives respective span-to-depth ratios of 17.8 and 44.4. The strut angle is 39 degrees.

The cross section dimensions and compression strut details are given in Fig. 4. The compression struts consist of 0.324 m (12.8 in.) diameter steel pipes with end plates that frame into concrete buildouts (blisters) and transfer the load directly to the webs from longitudinal beams at the deck level. Note that there are two triangular-shaped exterior longitudinal T-beams and one rectangular-shaped interior longitudinal T-beam.

The delineation between Stages 1, 2, and 3 construction is indicated in

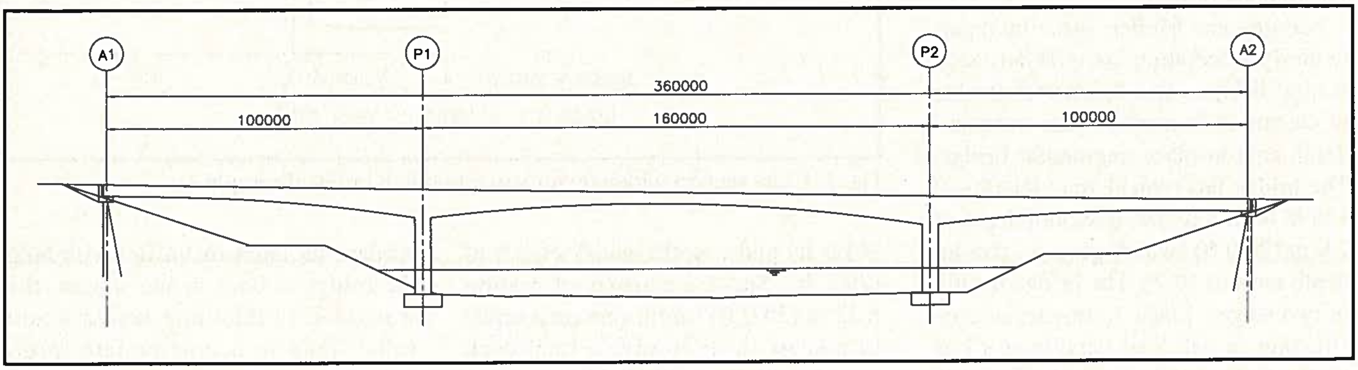


Fig. 3. Bridge elevation (Example 1).

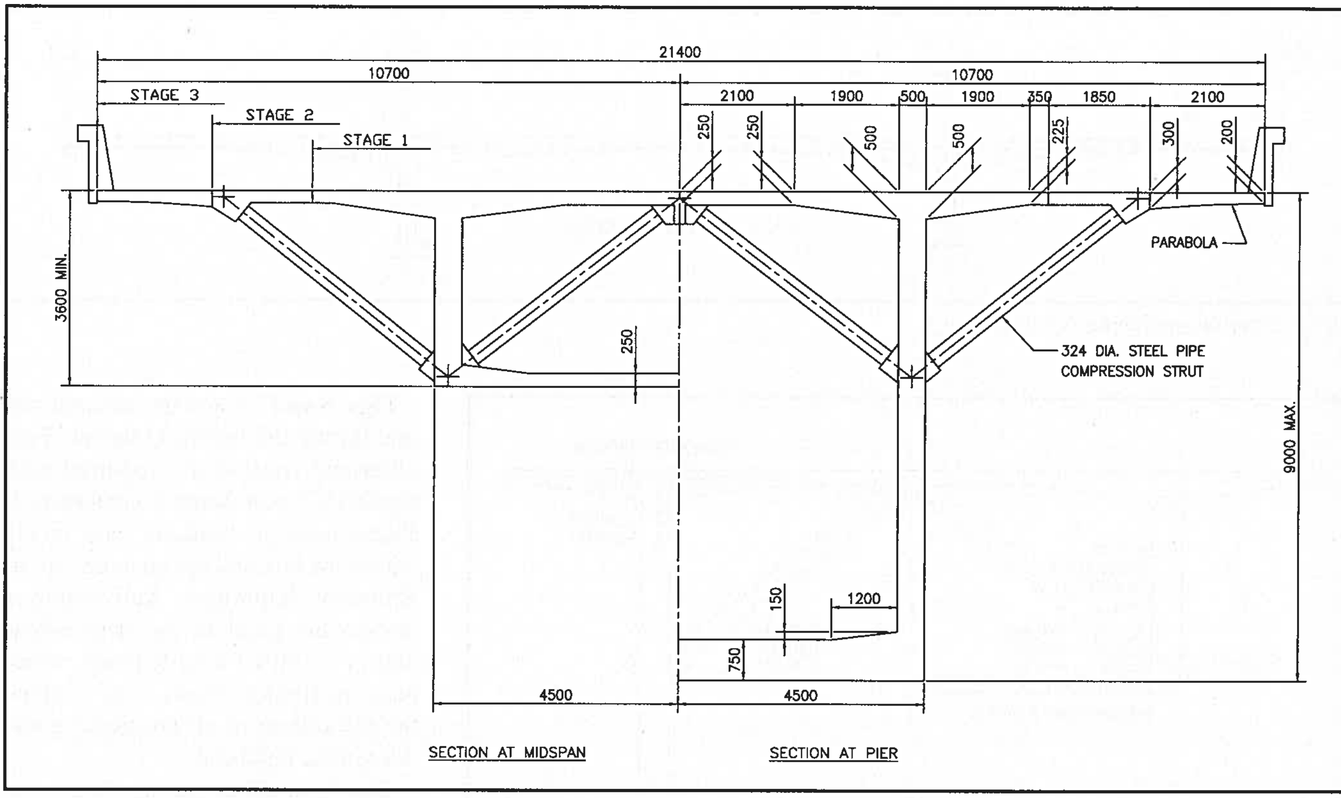


Fig. 4. Cross section dimensions (Example 1).

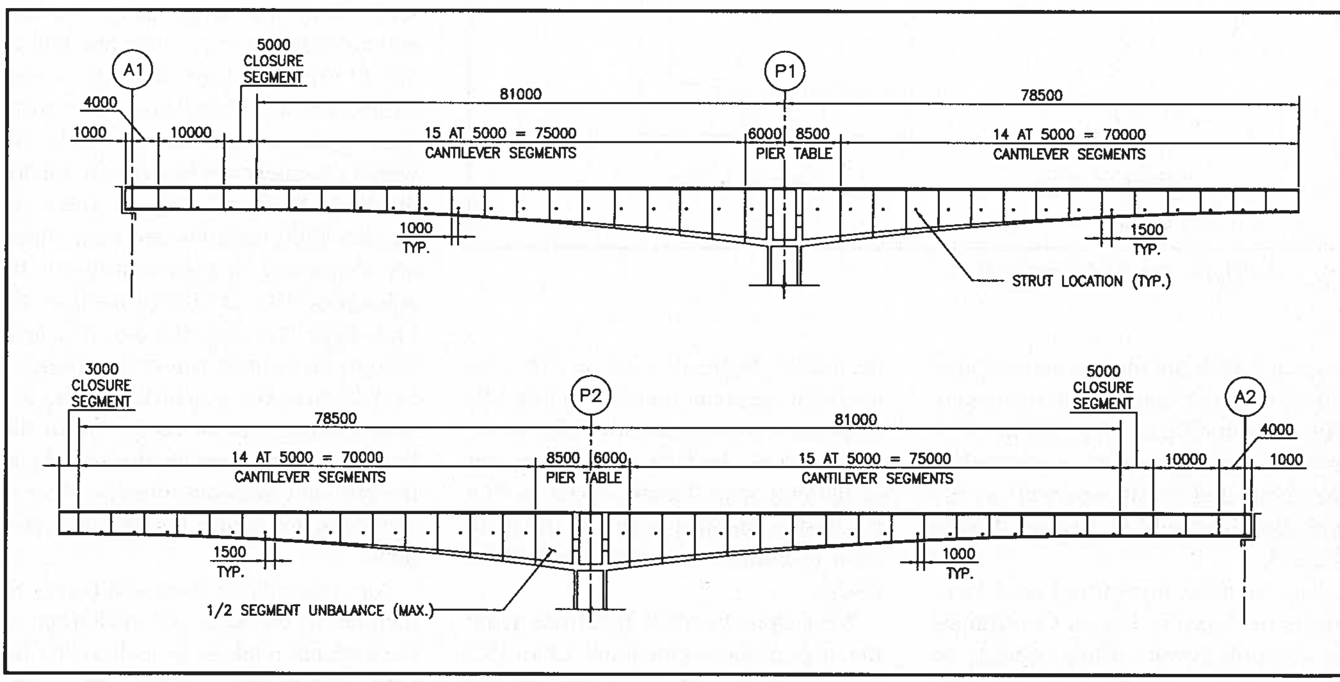


Fig. 5. Segment layout and strut locations (Example 1).

Fig. 4. The top slab thickness for Stage 1 varies from 0.250 m (9.8 in.) at the center of the box to 0.500 m (19.7 in.) at the web and 0.225 m (8.9 in.) at the end of the cantilever. The fillet width and thickness dimensions shown are those required to accommodate the cantilever tendons. The top slab thickness for Stage 2 is constant at 0.225 m (8.9 in.), while the top slab thickness for Stage 3 varies parabolically from 0.300 to 0.200 m (11.8 to 7.9 in.).

The soffit width is 9.0 m (29.5 ft). The bottom slab thickness varies from 0.250 m (9.8 in.) at midspan to 0.750 m (29.5 in.) at the pier. The midspan bottom slab thickness and fillet width and thickness dimensions are those required to accommodate the bottom span tendons. The pier bottom slab thickness is that required to resist the maximum compression during balanced cantilever construction (which

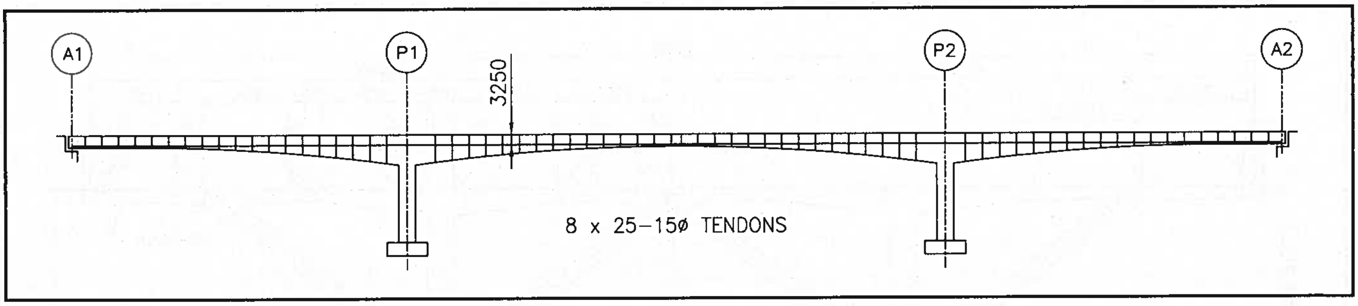


Fig. 6. External tendon layout (Example 1).

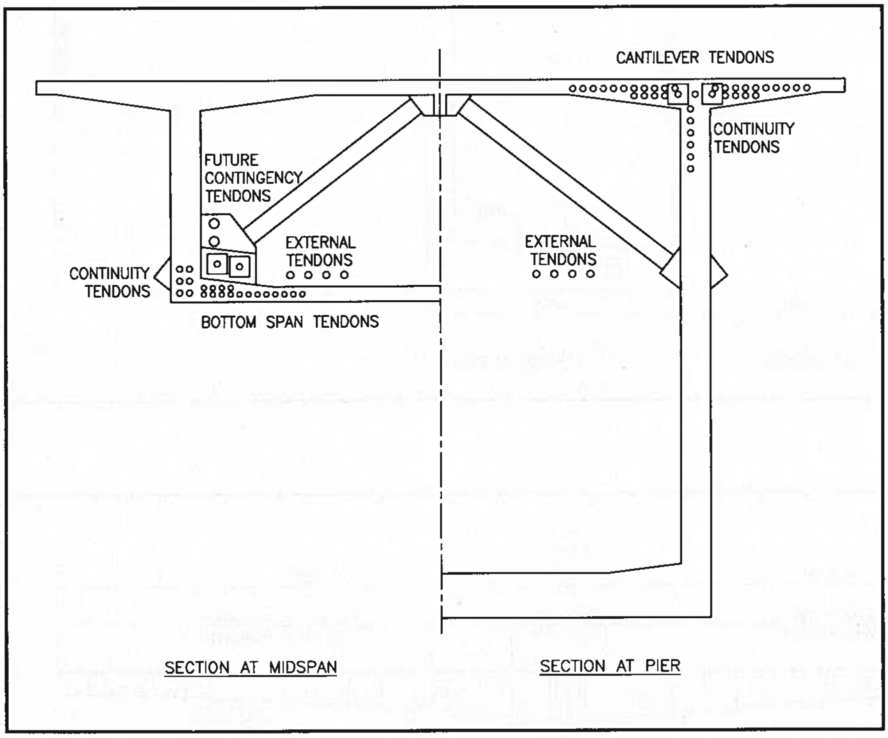


Fig. 7. Bulkhead details (Example 1).

is greater than the maximum compression due to the Stage 3 service loads). The web thickness of 0.500 m (19.7 in.) has been chosen to accommodate the shear and torsion as well as the web bending which occurs during Stage 3.

The segment layout and strut locations are shown in Fig. 5. Constrained by the form traveler lifting capacity on the one hand and the desire to minimize the total number of segments on the other, segments of 5.0 m (16.4 ft) length have been chosen. A one-half segment unbalance is built into the pier table — 6.0 versus 8.5 m (19.7 versus 27.9 ft) — to minimize the unbalanced moment that is transferred into the pier and foundation. Hence, there are 15 cantilever segments on one side of the cantilever and 14 on the other. There is a 5.0 m (16.4 ft) abutment segment, two 5.0 m (16.4 ft) segments constructed on falsework, and a 5.0 m (16.4 ft) closure segment in the end span. Finally, a 3.0 m (9.8 ft) closure segment is cast in the main span to complete construction of the bridge.

Note that the strut locations from the edge of the segment are 1.0 m (3.3 ft) on one side of the cantilever and 1.5 m (4.9 ft) on the other. This simplifies the form traveler operations since the strut locations are at the same location for each form traveler. This also gives the compression struts a uniform spacing of 5.0 m (16.4 ft) for the length of the bridge, which enhances the appearance of the bridge and increases the structural efficiency of the strutted box.

Figs. 6 and 7 show the external tendon layout and bulkhead details. Four 25-strand tendons are required each for Stage 2 and Stage 3 construction. These straight tendons pass freely within the box and are anchored in the abutment diaphragm. Anti-vibration devices are required for these simple 360 m (1180 ft) long tendons, which have no friction losses. Fig. 7 shows the disposition of all prestressing tendons in the bulkhead.

The strutted box is not overly congested despite the fact that there is a substantial amount of prestressing steel. There are 30 cantilever tendons per web near the pier corresponding to the 29 segments plus pier table being cantilevered. There are 16 bottom span tendons per web near midspan, which are anchored in pairs in anchor blocks at the strut locations. There are six continuity tendons per web, which are anchored in pier/abutment diaphragms. The continuity tendons are in a single row near the pier for shear reasons and in two rows near midspan to maximize the eccentricity. The external tendons pass freely within the box, as can be seen in the section at the pier and midspan. Finally, there is provision for future contingency tendons.

An open cellular abutment is recommended to facilitate the anchorage of the external tendons as well as the future contingency tendons. It is also suggested that interior lighting (which utilizes a portable generator) be installed inside the box girder during Stage 1 construction to facilitate installation of longitudinal external prestressing tendons for Stages 2 and 3 construction, as well as periodic inspection and maintenance for the life of the bridge.

The bending moment diagrams for

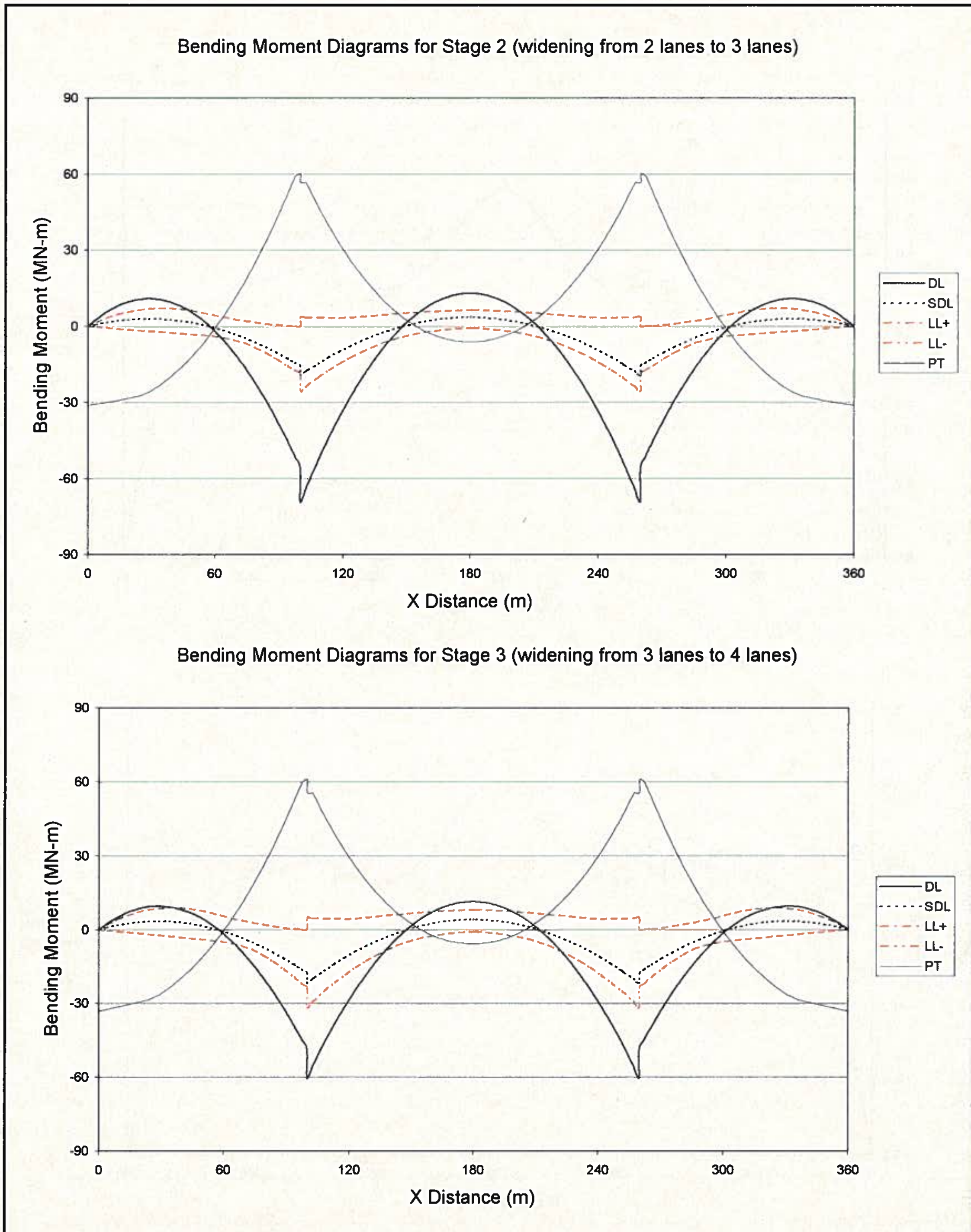
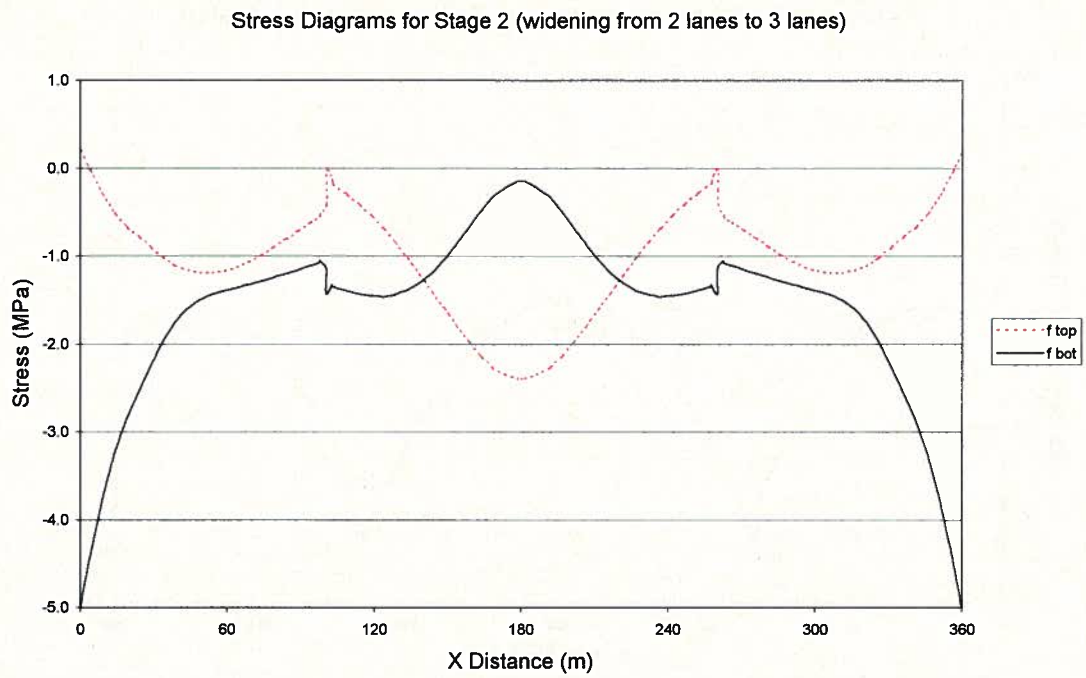


Fig. 8. Bending moment diagrams (Example 1).

the Stages 2 and 3 widenings are shown in Fig. 8. Individual moment diagrams due to dead load, superimposed dead load, positive live load, negative live load, and prestressing are plotted. In proportioning the prestressing, a good first step is to load-balance the dead load moment and allow the axial compression to offset the superimposed dead load and live load moments. Example 2 discusses an alternate method of proportioning the prestressing so that there is a reserve of compression everywhere.

The stress diagrams for the Stages 2 and 3 widenings are shown in Fig. 9. The stresses at the top and bottom of the section are shown. Note that the reserve of compression varies from approximately 0 to 2.4 MPa (0 to 348 psi) for most of the bridge, except near the ends, where the reserve reaches 5 MPa (725 psi) at the bottom and a very small tension at the top.

Table 1 gives a comparison of the live load, including the number of traffic lanes and the reduction in live load intensity as provided by the Canadian Code[2,3] and two AASHTO Specifications.[4,5] According to the Canadian Code, the bridge has to be designed for three, four, and six lanes with reduction factors of 0.80, 0.70, and 0.55, respectively. The AASHTO Standard and AASHTO LRFD Specifications require the bridge to be designed for



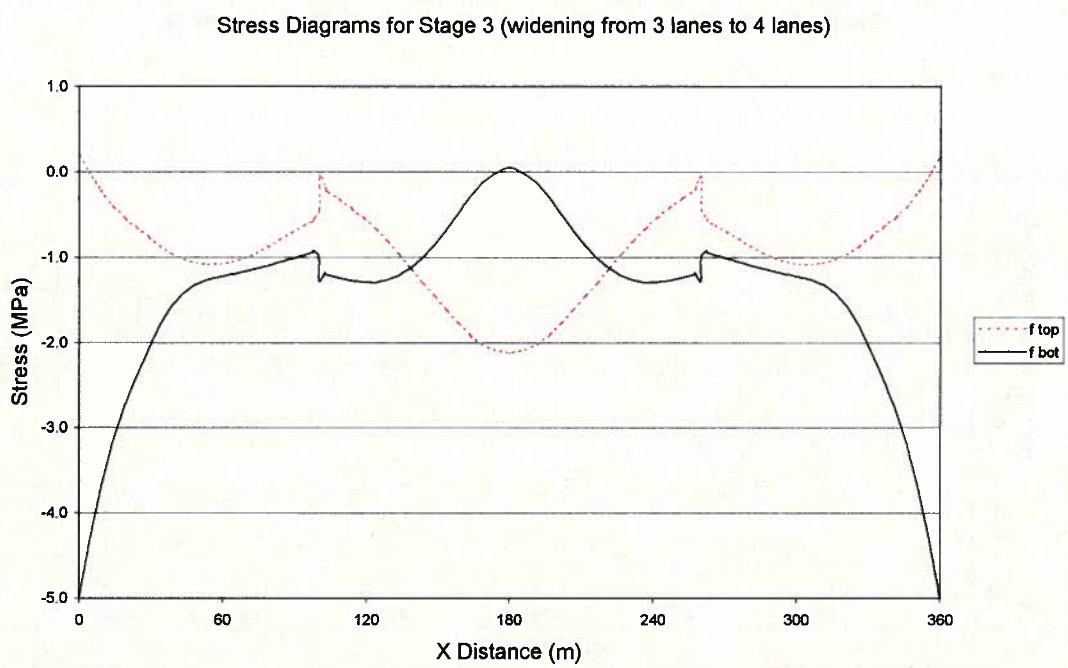


Fig. 9. Stress diagrams (Example 1).

three, four, and five lanes; the reduction factors given by the AASHTO Standard Specifications are 0.90, 0.75, and 0.75, while those given by the AASHTO LRFD Specifications are 0.85, 0.65, and 0.65.

What is of interest here is the *increase* in live load (as a percentage) for each code when going from Stage 1 to Stage 2 and Stage 3. When widening from two to three lanes (Stage 2), the increase in live load is 16.7 percent according to the Canadian Code and 11.1 percent according to the AASHTO Standard Specifications to achieve a 50 percent increase in traffic capacity. When widening from two to four lanes (Stages 2 and 3 combined), the increase in live load is 37.5 percent according to the Canadian Code and 38.9 percent according to the AASHTO Standard Specifications to achieve a 100 percent increase in traffic capacity. When the AASHTO LRFD Specifications are considered, the results are even better at 2.0 and 27.5 percent. In all cases, this represents a very good return (in traffic capacity) over investment (in structural capacity).

Table 2 gives a load summary comparison when widening from two to three lanes (Stage 2). The loads, moments, and shears are given for the initial two-lane bridge, the one-lane widening, and the resulting three-lane bridge. The increase is expressed as a percentage of the one-lane widening over the two-lane bridge. Note that the dead load of 391.15 kN/m (26.8 k/ft) is the average load per length for this variable-depth bridge. Looking at individual components, the dead load increase is 6.8 percent, the superimposed dead load increase is 13.0 percent, and the live load increase is 16.7 percent. Although the live load increase is 16.7 percent, the traffic capacity increase is 50 percent, or three times this value. Since the live load is a small fraction of the dead load, the required increase in superstructure moment and shear capacity varies from 8.6 to 10.0 percent, while the traffic capacity increase is 50 percent, or five times this value.

Also given in Table 2 are a load summary comparison when widening from two to four lanes (Stages 2 and 3 combined). Here, the loads, moments, and shears are given for the initial two-lane bridge, the two-lane widening, and the resulting four-lane bridge. Again, the increase is expressed as a percentage of the two-lane widening over the two-lane bridge. Looking at individual components, the dead load increase is 12.7 percent, the superimposed dead load increase is 27.9 percent, and the live load increase is 37.5 percent. In this table, the required increase in superstructure moment and shear capacity varies from 17.1 to 20.2 percent, while the traffic capacity increase is 100 percent, or five times this value.

The SBWM is a very effective solu-

Table 1. Live load comparison.

Construction stage	Live load (lanes)	Increase (percent)
	Canadian Code[2,3]	
Stage 1	$3 \times 0.80 = 2.40$	—
Stage 2	$4 \times 0.70 = 2.80$	0.40/2.40 = 16.7 percent
Stage 3	$6 \times 0.55 = 3.30$	0.90/2.40 = 37.5 percent
	AASHTO Standard Specifications[4]	
Stage 1	$3 \times 0.90 = 2.70$	—
Stage 2	$4 \times 0.75 = 3.00$	0.30/2.70 = 11.1 percent
Stage 3	$5 \times 0.75 = 3.75$	1.05/2.70 = 38.9 percent
	AASHTO LRFD Specifications[5]	
Stage1	$3 \times 0.85 = 2.55$	—
Stage 2	$4 \times 0.65 = 2.60$	0.05/2.55 = 2.0 percent
Stage 3	$5 \times 0.65 = 3.25$	0.70/2.55 = 27.5 percent

Table 2. Load summary comparison for Example 1.

		Widen from two to three lanes (Stage 2)				Widen from two to four lanes (Stages 2 and 3 combined)			
		Two-lane bridge	One-lane widening	Three-lane bridge	Increase (percent)	Two-lane bridge	Two-lane widening	Four-lane bridge	Increase (percent)
Loads	DL (kN/m)	391.150	26.450	417.600	6.8 percent	391.150	49.508	440.658	12.7 percent
	SDL (kN/m)	56.142	7.326	63.468	13.0 percent	56.142	15.642	71.784	27.9 percent
	LL (lanes)	$3 \times 0.80 = 2.40$	0.40	$4 \times 0.70 = 2.80$	16.7 percent	$3 \times 0.80 = 2.40$	0.90	$6 \times 0.55 = 3.30$	37.5 percent
Moment, Pier 1L (MN-m)	DL	–754.245	–55.863	–810.108		–754.245	–104.562	–858.807	
	SDL	–118.5728	–15.473	–134.045		–118.573	–33.036	–151.609	
	LL	–113.746	–18.958	–132.704		–113.746	–42.655	–156.401	
	Total	–986.564	–90.293	–1076.857	9.2 percent	–986.564	–180.253	–1166.817	18.3 percent
Moment, Pier 1R (MN-m)	DL	–929.624	–69.084	–998.708		–929.624	–129.308	–1058.932	
	SDL	–146.6348	–19.134	–165.769		–146.635	–40.855	–187.489	
	LL	–152.018	–25.336	–177.354		–152.018	–57.006	–209.024	
	Total	–1228.277	–113.554	–1341.831	9.2 percent	–1228.277	–227.168	–1455.445	18.5 percent
Moment, Span 2 (MN-m)	DL	163.207	12.986	176.193		163.207	24.306	187.513	
	SDL	27.563	3.597	31.160		27.563	7.680	35.243	
	LL	38.514	6.419	44.933		38.514	14.443	52.957	
	Total	229.284	23.002	252.286	10.0 percent	229.284	46.429	275.713	20.2 percent
Shear, Abut 1 (MN)	DL	–11.345	–0.766	–12.111		–11.345	–1.434	–12.779	
	SDL	–1.626	–0.212	–1.838		–1.626	–0.453	–2.079	
	LL	–2.552	–0.425	–2.977		–2.552	–0.957	–3.509	
	Total	–15.523	–1.403	–16.926	9.0 percent	–15.523	–2.844	–18.367	18.3 percent
Shear, Pier 1L (MN)	DL	27.931	1.880	29.811		27.931	3.519	31.450	
	SDL	3.990	0.521	4.511		3.990	1.112	5.102	
	LL	4.097	0.683	4.780		4.097	1.537	5.634	
	Total	36.018	3.084	39.102	8.6 percent	36.018	6.168	42.186	17.1 percent
Shear, Pier 1R (MN)	DL	–31.131	–2.115	–33.246		–31.131	–3.959	–35.090	
	SDL	–4.489	–0.586	–5.075		–4.489	–1.251	–5.740	
	LL	–4.690	–0.782	–5.472		–4.690	–1.759	–6.449	
	Total	–40.310	–3.483	–43.793	8.6 percent	–40.310	–6.968	–47.279	17.3 percent

Note: 1 kN/m = 68.58 lb/ft; 1 MN-m = 738.1 kip-ft; 1 MN = 225 kips.

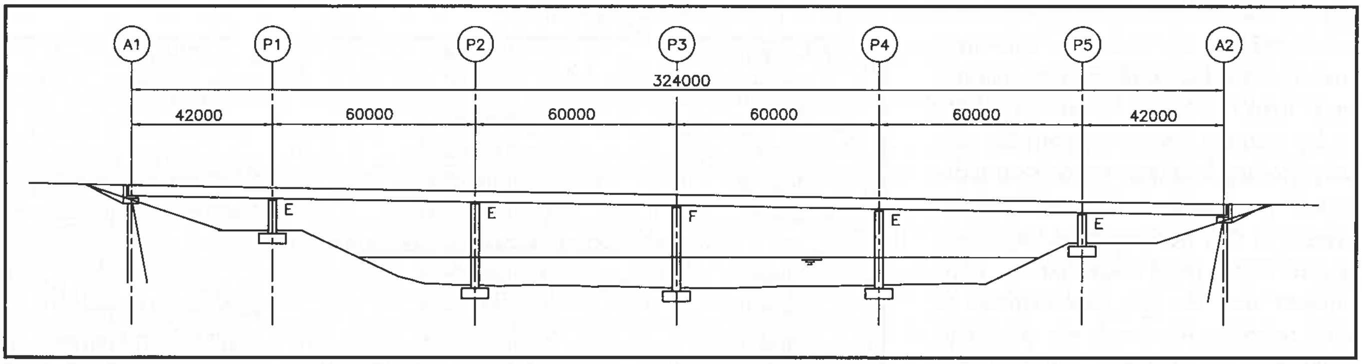


Fig. 10. Bridge elevation (Example 2).

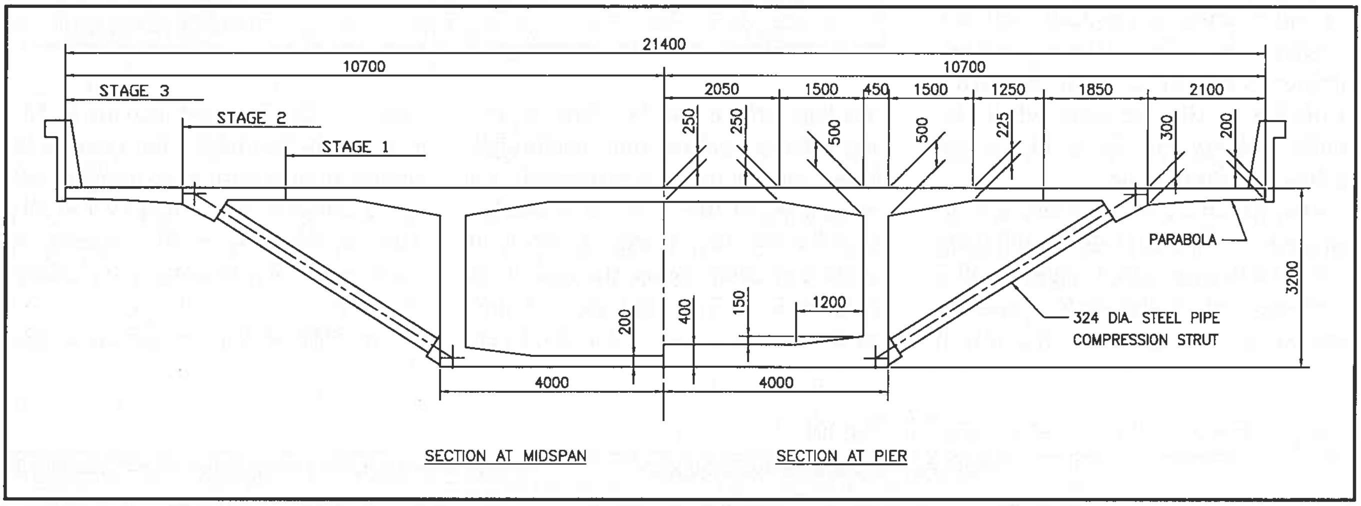


Fig. 11. Cross section dimensions (Example 2).

tion in this example, because for an additional investment of 10 or 20 percent in superstructure moment and shear capacity, the respective return is 50 or 100 percent in traffic capacity.

EXAMPLE 2

Consider a six-span constant-depth precast segmental bridge built by the balanced cantilever method of construction (see Fig. 10). The interior spans are 60 m (196.9 ft), while the exterior spans are 42 m (137.8 ft), giving an overall length of bridge of 324 m (1063.0 ft). The depth of the section (see Fig. 2) is 3.2 m (10.5 ft), which gives a span-to-depth ratio of 18.8 and a strut angle of 33 degrees. The section depth is governed by the compression strut geometry rather than the span-to-depth ratio in this example.

The cross section dimensions and compression strut details are given in Fig. 11. The compression struts consist of 0.324 m (12.8 in.) diameter steel pipes with end plates that frame into concrete buildouts (blisters) and transfer the load directly to the bottom slab from triangular shaped exterior longitudinal T-beams at the deck level. The top slab thickness for Stage 1 varies from 0.250 m (9.8 in.) at the center of the box to 0.500 m (19.7 in.) at the web and 0.225 m (8.9 in.) at the end of the cantilever. The top slab thickness for Stage 2 is constant at 0.225 m (8.9 in.), while the top slab thickness for Stage 3 varies parabolically from 0.300 to 0.200 m (11.8 to 7.9 in.).

The soffit width is 8.0 m (26.2 ft). The bottom slab thickness varies from 0.200 m (7.9 in.) at midspan to 0.400 m (15.7 in.) at the pier. The midspan bottom slab thickness is a minimum value, while the pier bottom slab thickness is that required to accommodate the maximum compression during balanced cantilever construction (which is greater than the maximum compression due to the Stage 3 service loads). The web thickness is 0.450 m (17.7 in.).

The segment layout and strut locations are shown in Fig. 12. Each balanced cantilever has a 1.8 m (5.9 ft) pier segment and eight 3.6 m (11.8 ft) typical segments on each side of the cantilever. Closure segments are 0.6 m (2.0 ft). The end spans have a 1.5 m (4.9 ft) abutment segment and three 3.6 m (11.8 ft) typical segments which are assembled on falsework. The construction sequence proceeds from one end of the bridge to the other, building cantilevers P1 to P5 in succession, and pouring closures in the backspan as construction proceeds. At the completion of construction, the bridge superstructure sits on fixed bearings at P3 and expansion bearings everywhere else (see Fig. 10). Note that the strut locations are always 2.1 m (6.9 ft) away from the edge of the segment. This simplifies the casting cell operations since the strut locations are always at the same location in the cast-

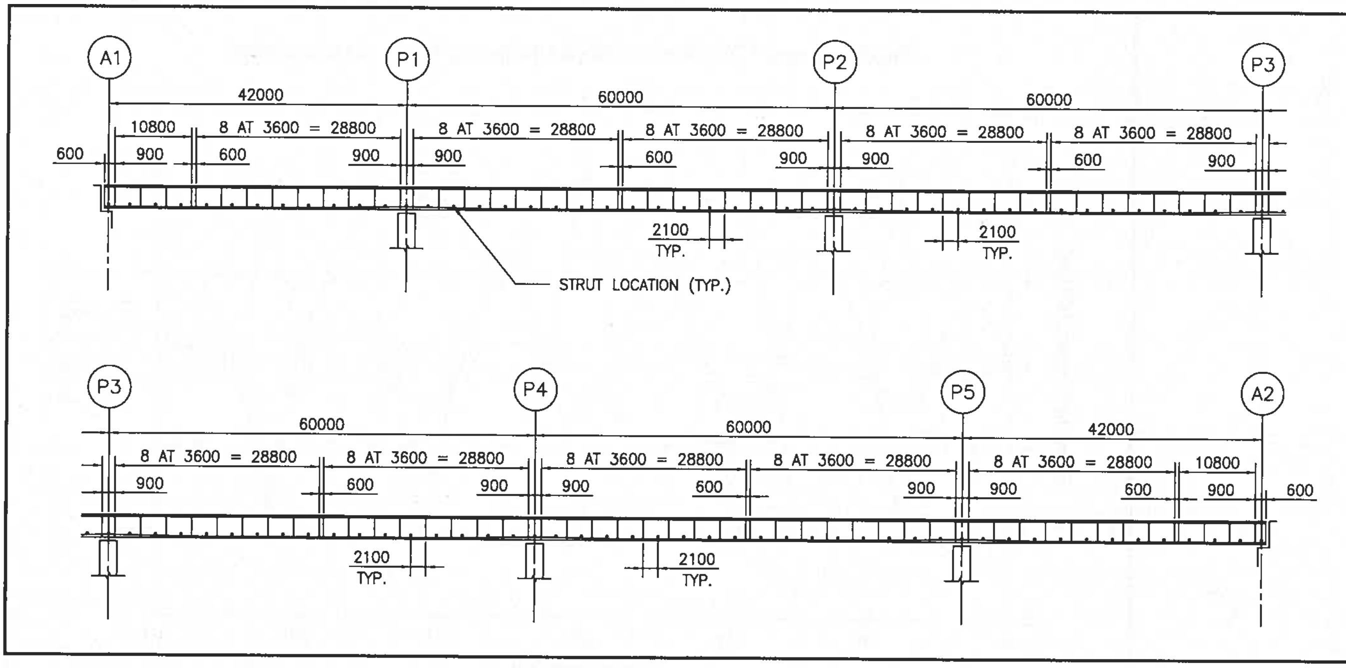


Fig. 12. Segment layout and strut locations (Example 2).

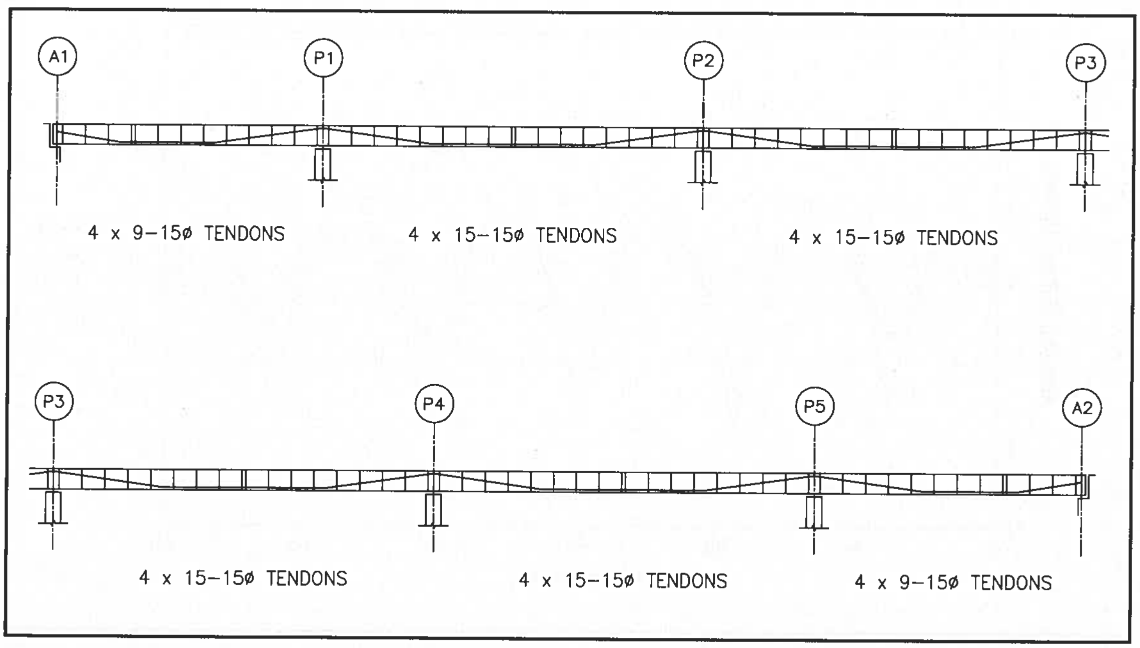


Fig. 13. External tendon layout (Example 2).

ing cells for the typical segments. This also gives the compression struts a uniform spacing of 3.6 m (11.8 ft) for the length of the bridge.

Figs. 13 and 14 show the external tendon layout and bulkhead details. Stage 2 and Stage 3 construction each requires two 15-strand tendons in the interior spans and two 9-strand tendons in the exterior spans. These draped tendons are held down at the deviation diaphragms and anchored at the pier/abutment diaphragms. Fig. 14

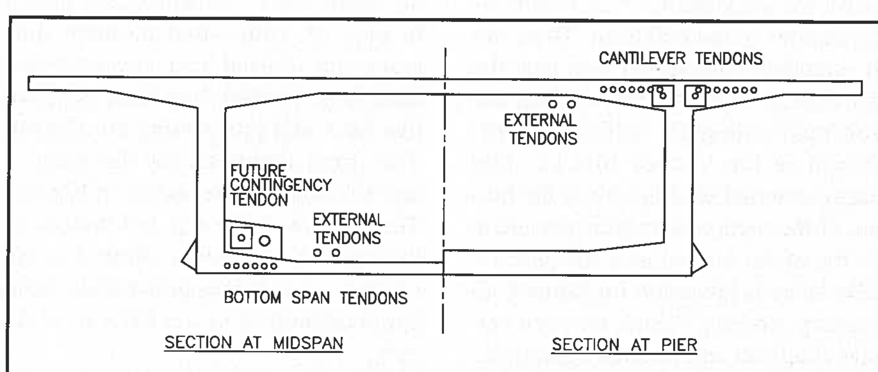


Fig. 14. Bulkhead details (Example 2).

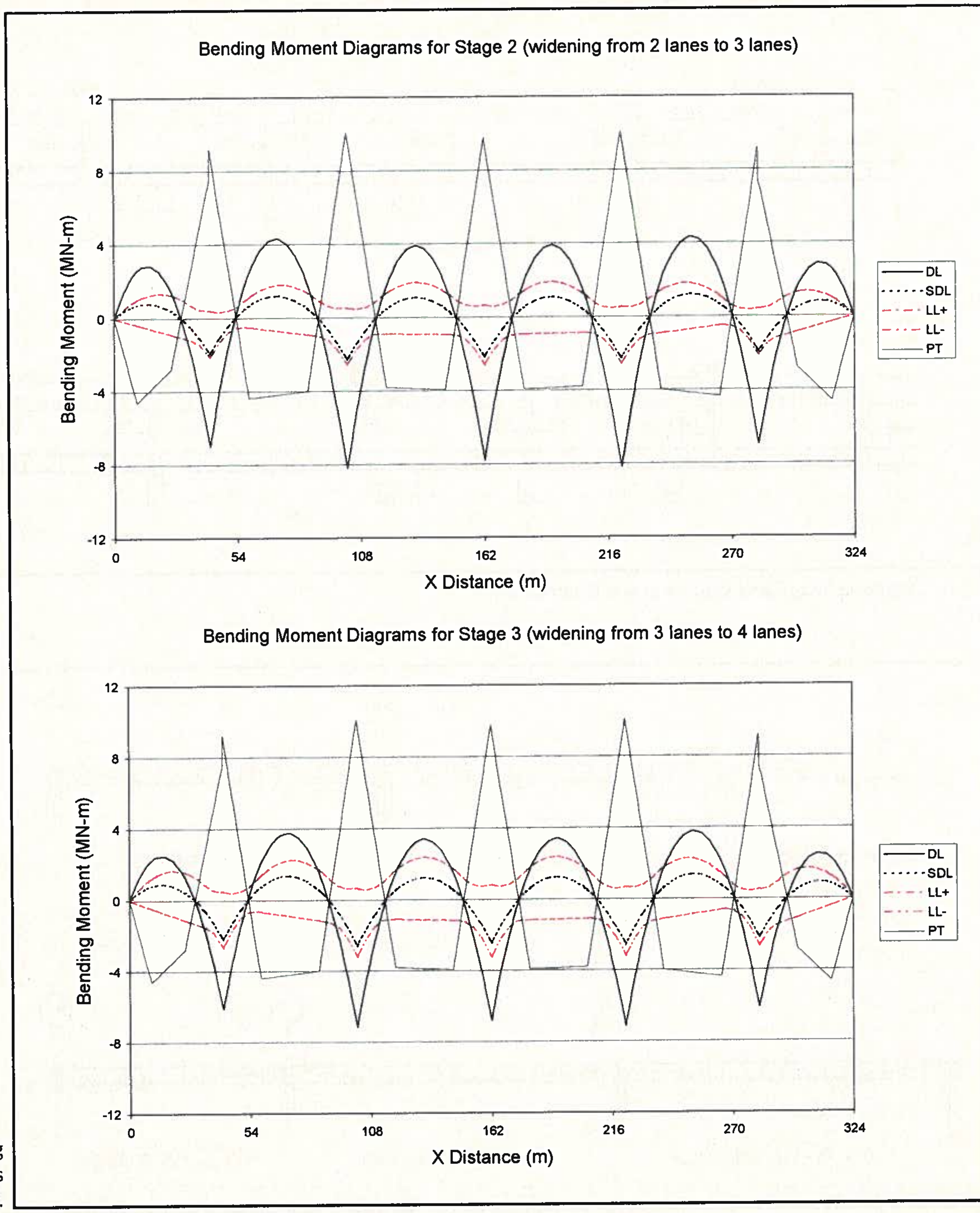
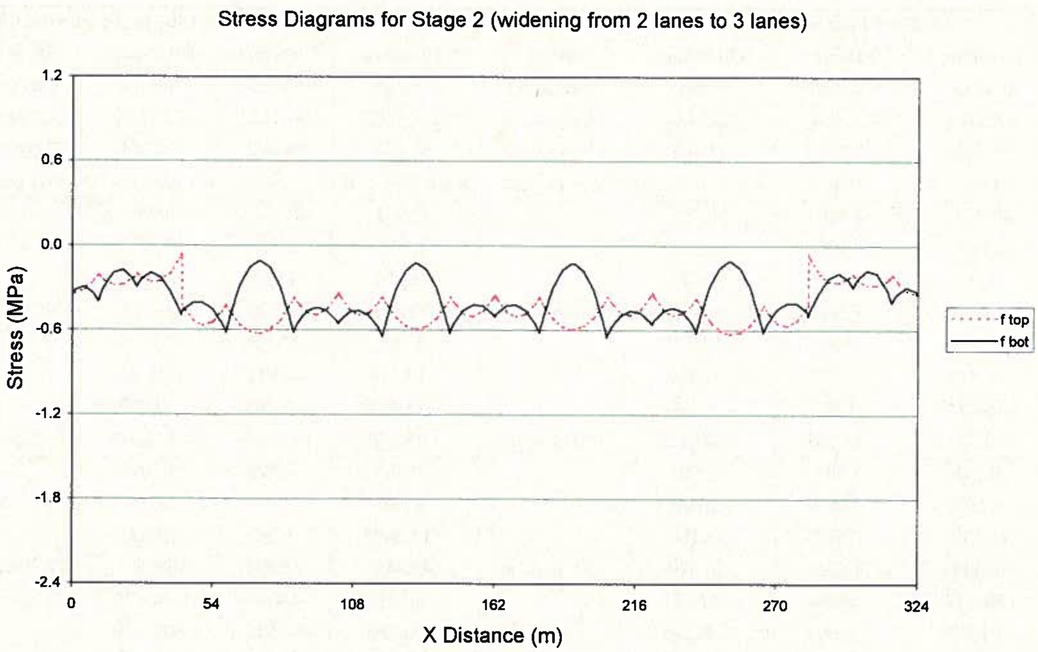


Fig. 15. Bending moment diagrams (Example 2).

shows the disposition of all prestressing tendons in the bulkhead. There are 14 cantilever tendons per web near the pier and six bottom span tendons per web near midspan, which are anchored in the anchor blocks. The draped external tendons are at the bottom of the section near midspan and at the top of the section near the pier. Finally, there is provision for future contingency tendons. Again, an open cellular abutment and interior lighting are recommended.

The bending moment diagrams for the Stage 2 and 3 widenings are shown in Fig. 15. Individual moment diagrams due to dead load, superimposed dead load, positive live load, negative live load, and prestressing are plotted. The stress diagrams for the Stage 2 and 3 widenings are shown in Fig. 16. The stresses at the top and bottom of the section are shown. Note that the reserve of compression varies from approximately 0 to 0.6 MPa (0 to 87 psi).

The prestressing has been proportioned here by combining the results of various analyses in a spreadsheet, which is linked to an interactive graphical plot of the stresses (Fig. 16). The results of the dead load, superimposed dead load, and live load analyses (as well as thermal analyses,[6] including positive thermal gradient, negative thermal gradient, temperature rise, and temperature fall) are imported into the spreadsheet, along with unit load cases of draped prestressing in the exterior spans and the interior spans. In this way, multipliers for the exterior span prestressing and interior



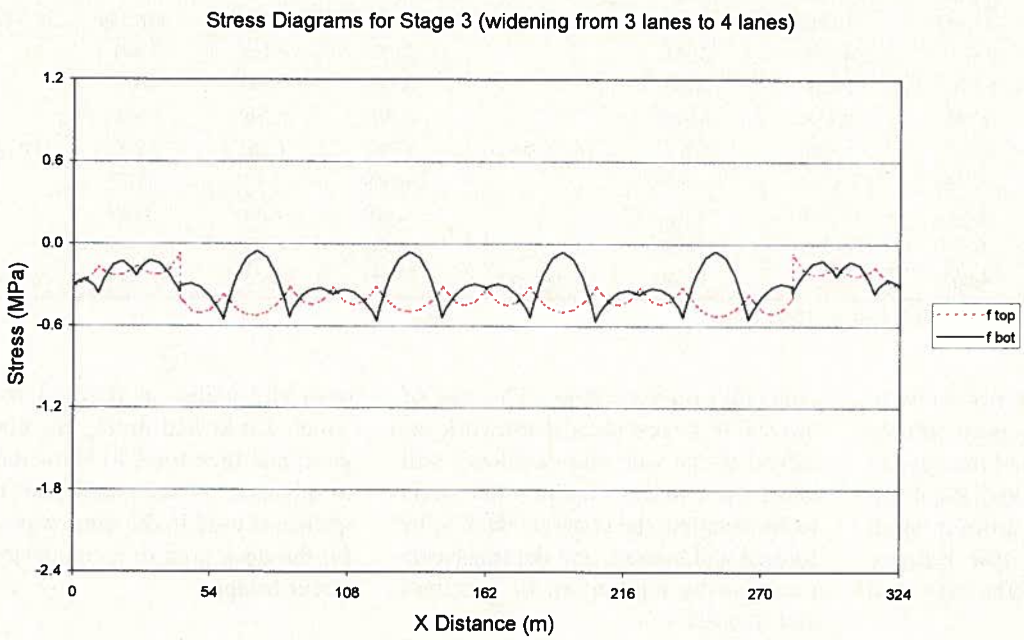


Fig. 16. Stress diagrams (Example 2).

span prestressing are adjusted to immediately see the effect on the stresses. The optimum amount of prestressing is proportioned to have a reserve of compression everywhere at the top and bottom of the section while using the least amount of prestressing.

It is interesting to compare the loads, moments, and shears in Example 2 (Table 3) with those in Example 1 (Table 2). When widening from two to three lanes, the dead load increase is 13.1 percent in Example 2, whereas it was only 6.8 percent in Example 1. This makes sense because the overall dead load is smaller for a bridge having shorter spans, meaning that the percentage increase of the widened bridge is larger. The superimposed dead load increase and the live load increase are the same in both examples at 13.0 and 16.7 percent, respectively. Again, this makes sense because the superimposed dead load and live load are the same in both examples. The net effect is that the required increase in superstructure moment and shear capacity varies from 13.6 to 13.9 percent in Example 2, whereas it varies from 8.6 to 10.0 percent in Example 1. When widening from two to four lanes, the required increase in superstructure moment and shear capacity varies from 27.1 to 28.1 percent in Example 2, whereas it varies from 17.1 to 20.2 percent in Example 1.

To summarize, Examples 1 and 2 require an increase in superstructure moment and shear capacity of 10 and 14 percent, respectively, to achieve a 50 percent increase in traffic capacity

Table 3. Load summary comparison for Example 2.

		Widen from two to three lanes (Stage 2)				Widen from two to four lanes (Stages 2 and 3 combined)			
		Two-lane bridge	One-lane widening	Three-lane bridge	Increase (percent)	Two-lane bridge	Two-lane widening	Four-lane bridge	Increase (percent)
Loads	DL (kN/m)	202.030	26.450	228.480	13.1 percent	202.030	49.508	251.538	24.5 percent
	SDL (kN/m)	56.142	7.326	63.468	13.0 percent	56.142	15.642	71.784	27.9 percent
	LL (lanes)	3 × 0.80 = 2.40	0.40	4 × 0.70 = 2.80	16.7 percent	3 × 0.80 = 2.40	0.90	6 × 0.55 = 3.30	37.5 percent
Moment, Span 2 (MN-m)	DL	32.921	4.310	37.231		32.921	8.067	40.988	
	SDL	9.148	1.194	10.342		9.148	2.549	11.697	
	LL	10.754	1.792	12.547		10.754	4.033	14.787	
	Total	52.823	7.296	60.119	13.8 percent	52.823	14.649	67.472	27.7 percent
Moment, Pier 2 (MN-m)	DL	–62.672	–8.205	–70.877		–62.672	–15.358	–78.030	
	SDL	–17.416	–2.273	–19.688		–17.416	–4.852	–22.268	
	LL	–15.434	–2.572	–18.007		–15.434	–5.788	–21.222	
	Total	–95.522	–13.050	–108.572	13.7 percent	–95.522	–25.998	–121.520	27.2 percent
Moment, Span 3 (MN-m)	DL	29.795	3.901	33.696		29.795	7.301	37.097	
	SDL	8.280	1.080	9.360		8.280	2.307	10.587	
	LL	11.369	1.895	13.264		11.369	4.263	15.632	
	Total	49.444	6.876	56.320	13.9 percent	49.444	13.872	63.315	28.1 percent
Moment, Pier 3 (MN-m)	DL	–59.577	–7.800	–67.377		–59.577	–14.600	–74.177	
	SDL	–16.556	–2.160	–18.716		–16.556	–4.613	–21.169	
	LL	–15.600	–2.600	–18.200		–15.600	–5.850	–21.450	
	Total	–91.733	–12.560	–104.294	13.7 percent	–91.733	–25.062	–116.796	27.3 percent
Shear, Abut 1 (MN)	DL	2.971	0.389	3.361		2.971	0.728	3.700	
	SDL	0.826	0.108	0.933		0.826	0.230	1.056	
	LL	0.796	0.133	0.928		0.796	0.298	1.094	
	Total	4.593	0.629	5.222	13.7 percent	4.593	1.257	5.849	27.4 percent
Shear, Pier 3L (MN)	DL	–6.009	–0.787	–6.796		–6.009	–1.473	–7.482	
	SDL	–1.670	–0.218	–1.888		–1.670	–0.465	–2.135	
	LL	–1.374	–0.229	–1.603		–1.374	–0.515	–1.889	
	Total	–9.053	–1.234	–10.287	13.6 percent	–9.053	–2.453	–11.506	27.1 percent

Note: 1 kN/m = 68.58 lb/ft; 1 MN-m = 738.1 kip-ft; 1 MN = 225 kips.

(or 20 and 28 percent, respectively, to achieve a 100 percent increase in traffic capacity). The required increase in superstructure moment and shear capacity is smaller for longer span bridges than for shorter span bridges. The SBWM solution works very well in both examples.

CONSTRUCTION CONSIDERATIONS

One of the fundamental decisions that has to be made during a bridge widening project is whether to allow traffic on the bridge during construction. This issue also comes into play when widening the bridge by the SBWM.

If traffic is allowed on the bridge during widening, detours will not be required, but construction will proceed more slowly since it will occur adjacent to traffic. Barriers will be required to separate the traffic from the construction activities. The use of moveable knee-braced formwork attached to the web and cantilever will allow the exterior compression struts to be installed, the concrete deck to be formed and poured, and the transverse prestressing tendons to be installed and stressed.

If traffic is not allowed on the bridge during widening, construction will proceed much more rapidly, but the traffic will have to be detoured. The erection truss shown in Fig. 17 can be used to widen the bridge directly from two to four lanes (or from two to three lanes or from three to four lanes as described previously). The truss facilitates the installation of the exterior compression struts. It has suspended forms at the ends for the Stage 2 and Stage 3 concrete deck pours. The ends of the truss have safety rails as well as extra room for installing and stressing the transverse prestressing tendons. The truss bears over the webs on Hilman rollers, which are locked during the concrete pour, and then freed to allow the truss to advance. A staggered pouring sequence is used in the same way as that for the deck pour of a composite steel girder bridge.

DESIGN CONSIDERATIONS

Longitudinal Flexure

The bridge has to be designed for three stages of construction. By keeping the design for each stage of construction independent as outlined below, the additional design work is not substantial.

Stage 1 requires the traditional design of a segmental bridge where the construction sequence and the time-dependent effects of creep, shrinkage, and relaxation are considered. Prestressing is proportioned to facilitate segmental construction and to accom-

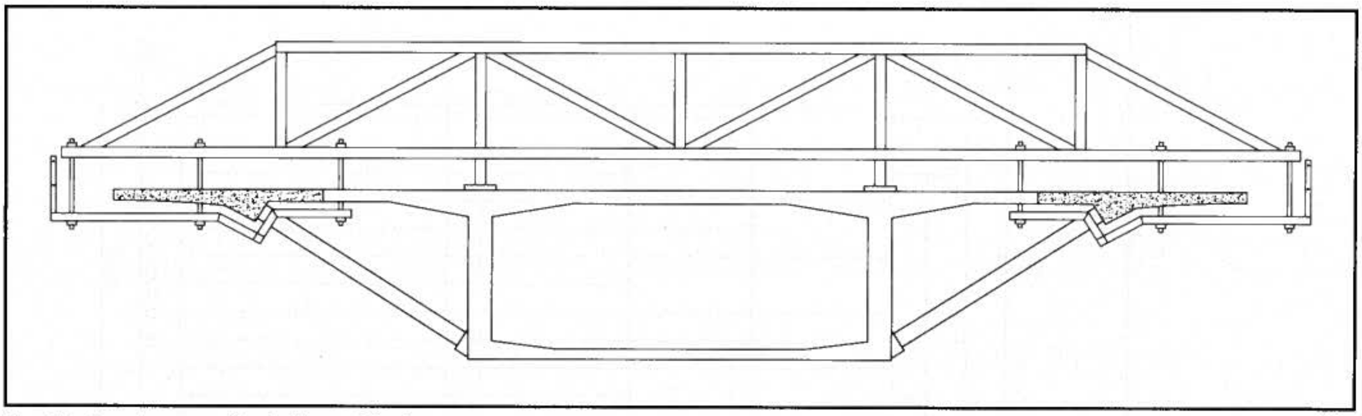


Fig. 17. Erection truss for bridge widening.

modate the redistribution of stresses. There should be a reserve of compression at the top and bottom of the section at all locations under the effects of all service loads, both at the initial opening of the bridge and after final long-term losses have occurred.

Stage 2 requires the design for the incremental loads of Stage 2, and can be designed elastically because it occurs so far in the future that the time-dependent effects of Stage 1 have little or no effect. Thus, the Stage 2 incremental loads and the Stage 2 section properties are applied to an elastic analysis of the completed bridge to allow the prestressing to be proportioned so as to have a reserve of compression at the top and bottom of the section at all locations.

Stage 3 requires the design for the incremental loads of Stage 3 and can be designed in a similar manner to Stage 2.

Validation of the final design requires analysis for all three stages of construction in a 2D (or even a 3D) time-dependent computer program with the assumed dates of construction (the assumed dates will need to be replaced by the actual dates in the analysis when widening occurs). There should be a reserve of compression at the top and bottom of the section at all locations at any time during the lifetime of the bridge.

It is also necessary to check the ultimate strength design for all three stages of construction. The ultimate strength at Stage 3 is provided by the internal tendons of Stage 1 along with the external tendons of Stage 2 and Stage 3.

Longitudinal Shear and Torsion

The webs have to be designed for shear due to dead load, superimposed dead load, and live load, as well as for torsion due to eccentric live load and overturning wind. Stage 3 is the controlling case in the design for shear and torsion. It is important that the Resal effect be considered in the design. (Note that the Resal effect, which is explained in books on segmental box girder design, relates to the reduction in shear force by the vertical component of the inclined compression flange.) The webs also have to be designed for web bending (as discussed in the next section). Of course, the top and bottom slabs also have to be designed for torsion.

Design for shear and torsion can be performed by the traditional approach or by the strut-and-tie method. It is also desirable that the principal stresses be checked to ensure that cracking of the webs does not occur.

Transverse Design

The transverse analysis for Example 1 is facilitated by the creation of two computer models:

Finite Element Model — One-half of the interior span is modeled for Stage 3 construction. The model includes plate elements for the top slab, webs, and bottom slab, along with offset beam elements with rigid links for the longitudinal beams and beam elements for the compression struts. The appropriate end boundary conditions are applied. It has been found that a 1.0 x 1.0 m (3.3 x 3.3 ft) mesh size is reasonable (although local refinement to a 0.25 x 0.25 m (0.8 x 0.8 ft) mesh size at the location of concentrated loads is also worthwhile).

Plane Frame Model — A 5.0 m (16.4 ft) unit length of the cross section is modeled for each of the three stages of construction. By having the unit length of the cross section correspond to the spacing of compression struts, the model can be run both with and without compression struts to study the enveloping cases. A number of different depths of cross section with different locations of compression struts needs to be considered. The effects of concentrated wheel loads are determined by using influence surfaces as usual.

The individual elements are designed based on the results of the finite element analyses and the plane frame analyses. These elements include the longitudinal beams, the compression struts, the web reinforcement, and the top slab transverse prestressing.

Longitudinal Beams — The two triangular-shaped exterior longitudinal T-beams and one rectangular-shaped interior longitudinal T-beam take the load from the deck and transfer it through the compression struts to the web. The longitudinal beams thus reduce the deck spans in the transverse direction, which reduces the overall amount of prestressing and reinforcement in the deck. The longitudinal beams are reinforced with flexural and shear reinforcement.

Compression Struts — The compression struts can consist of either 324 mm (12.8 in.) diameter steel pipes (as described previously) having a wall thickness of 13 mm (0.5 in.), or

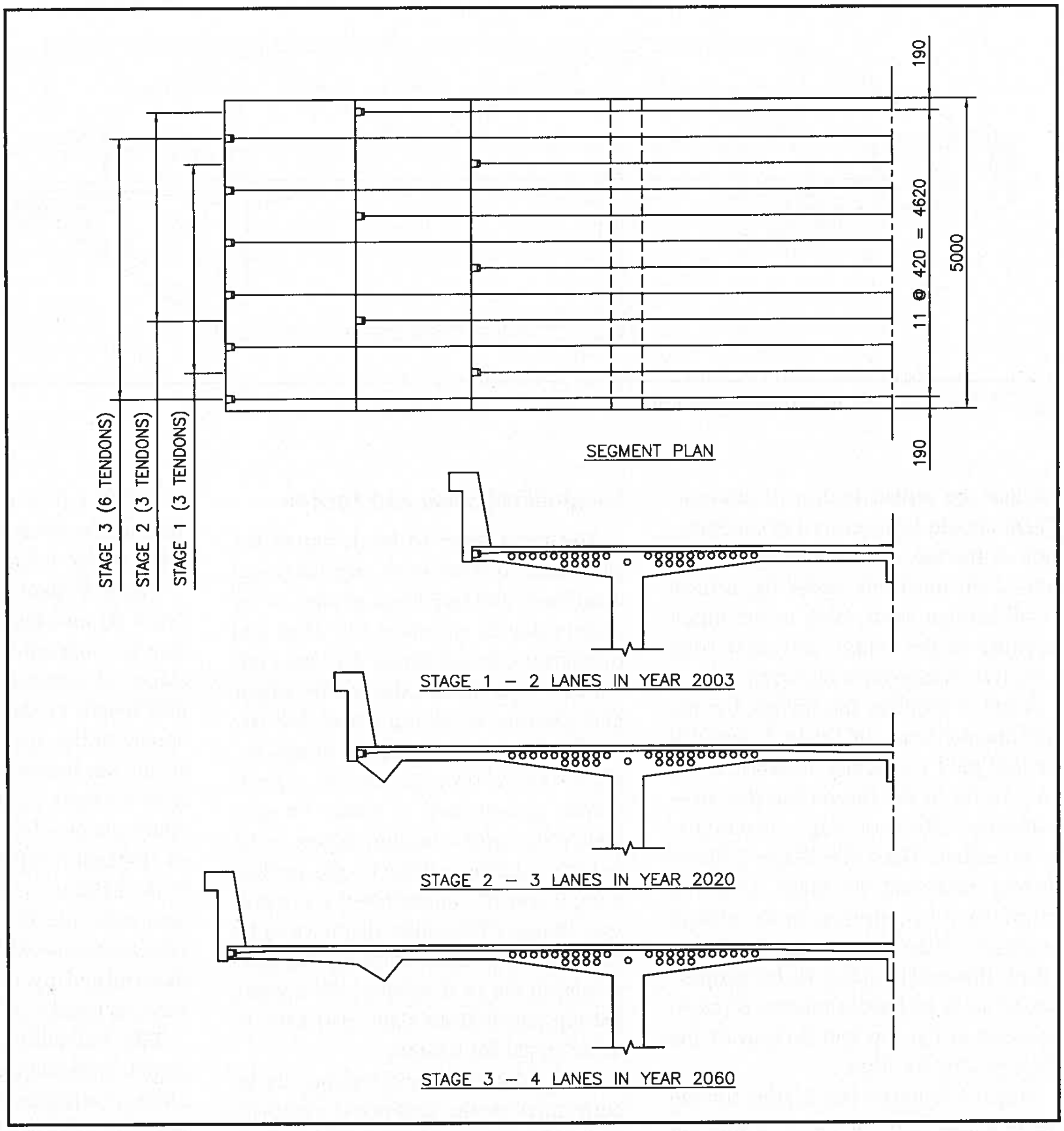


Fig. 18. Transverse tendon details.

400 x 400 mm (15.7 x 15.7 in.) precast concrete elements. They are designed for the axial forces given by the analyses along with the corresponding shear forces and bending moments.

Web Reinforcement — The web reinforcement is designed for longitudinal shear and torsion (as previously described) in addition to web bending. Web bending is due to the unbalanced live load being transmitted through the compression struts to the webs. The maximum inner face web bending in the right web occurs when live loads are placed on both cantilevers, whereas the maximum outer face web bending in the right web occurs when the live loads are placed on the right cantilever as well as between the rectangular-shaped interior longitudinal T-beam and the right web. Influence lines have been found to be very useful in positioning live loads so as to produce the maximum effects.

The finite element analyses have shown that the maximum web bending occurs at the location of the strut, and dies out very quickly at any longitudinal distance away from the strut. It is, thus, recommended that additional web bending reinforcement be banded locally on either side of the strut in addition to the shear and torsion reinforcement. For this example, this means that inner face web bending requires an additional 13 No. 15M reinforcing bars (13 No. 5 bars) at 150 mm (5.9 in.) centered about the strut and bundled with the shear and torsion bars which are also spaced at 150 mm (5.9 in.). The web bending bars are thus placed 900 mm (3.0 ft) on either side of the strut. No additional outer face web bending reinforcement is required for this example.

Top Slab Transverse Prestressing — Fig. 18 shows the transverse prestressing tendon layout in plan and section for the three stages of construction. Each tendon consists of four strands in a flat duct. There are twelve transverse prestressing tendons in each segment. Three tendons are stressed in Stage 1, three in Stage 2, and six in Stage 3. The tendon profile is at the top of the deck near the web for negative moment, and in the middle of the deck near midspan for both positive and negative moment. The stresses and ultimate strength are checked at the critical locations for all three stages of construction.

Another prestressing scheme to consider is banding the transverse tendons in plan near the locations of the compression struts. In this way, there will be more transverse tendons near the compression struts and fewer transverse tendons away from the compression struts. This will allow the transverse prestressing to more closely follow the results of the finite element analyses.

The transverse prestressing ducts for the Stages 2 and 3 widenings need to be installed during Stages 1 and 2 construction. These plastic ducts need to be protected against moisture intrusion, which can lead to freezing and cracking of the deck concrete. One suggestion is to fill the ducts with grease or foam and cap the ends. The grease or foam would be flushed out when widening occurs. Another suggestion is to place continuous foam backer rod in the empty ducts for the entire length of each tendon and cap the ends. Freezing water would simply compress the backer rod and not crack the concrete. The backer rod would be removed when widening occurs. At any rate, protection of ducts for the Stages 2 and 3 widenings is very important for the durability of the structure.

Shear Lag

The AASHTO Segmental Guide Specifications[7] state that the effective flange width may be determined (1) by elastic procedures such as the finite element method, (2) by provisions given in the Ontario Highway Bridge Design Code,[8] or (3) by provisions given in the AASHTO Segmental Guide Specifications.

The overall efficiency of the cross section predicted by the Ontario Code drops from a range of 94.9 to 100.0 percent in Example 1 to a range of 60.5 to 97.1 percent in Example 2. This dramatic drop is expected because shear lag is more severe for wide short-span bridges than for wide long-span bridges. The results validate the use of the full cross section in the final design for Example 1, and suggest that the effective widths need to be reduced in the final design for Example 2. Inclining the compression struts in plan as well as in elevation for shorter span bridges may be a useful method for reducing the amount of shear lag.

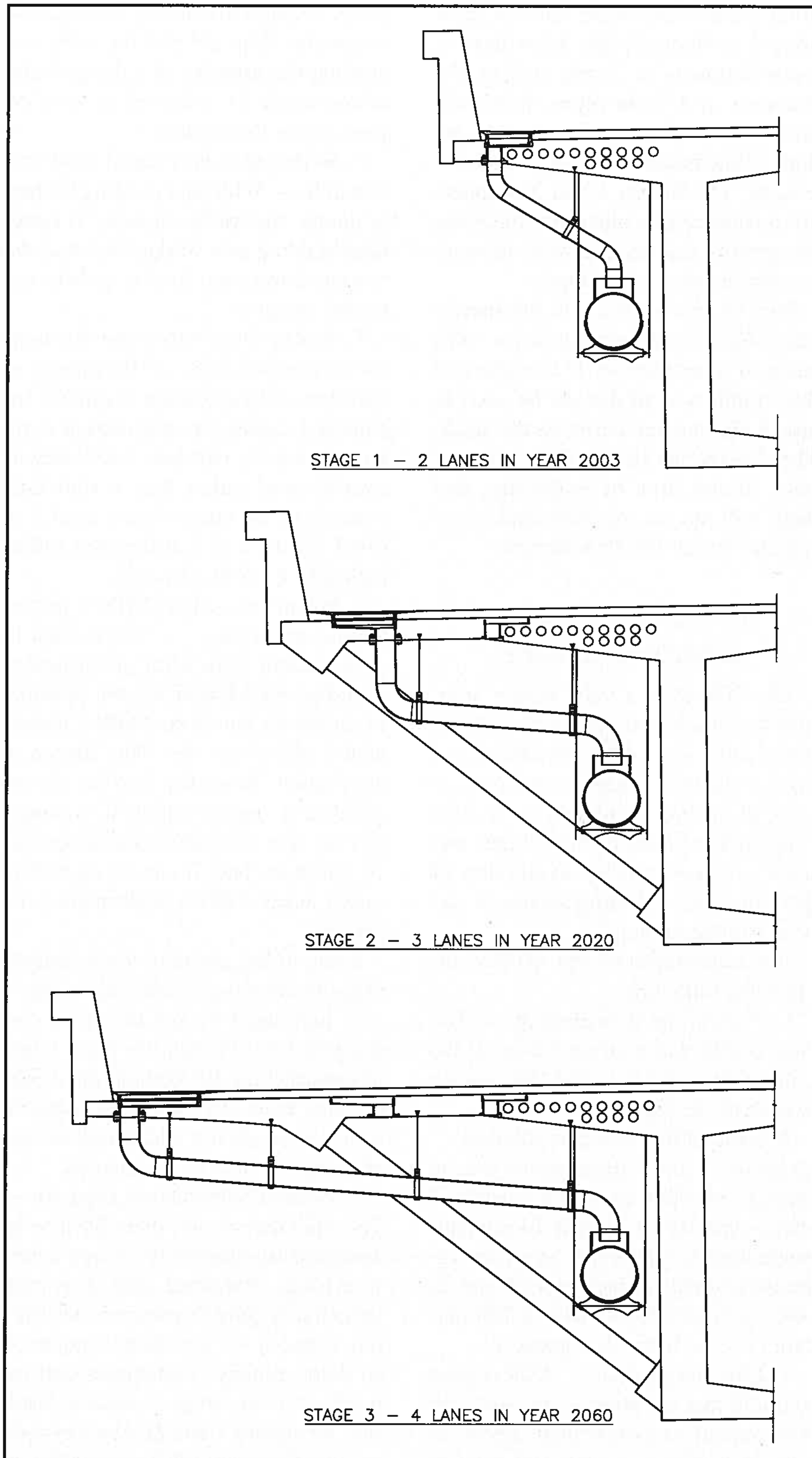


Fig. 19. Deck drainage details.

Miscellaneous Details

Fig. 19 shows the deck drainage details for the three stages of construction. The Stage 1 scuppers are located to avoid both the longitudinal and transverse tendon ducts. The deck drain pipes dump water into an open hopper collector pipe. In widening from Stages 1 to 3, the size of the scuppers and drain pipes increases, since the volume of water to be drained increases as the deck area increases. The Stages 1 and 2 scuppers are abandoned in place by removing the grating and carefully concreting the remainder of the scupper.

Fig. 19 also shows that the barrier curbs have to be relocated for each stage of construction. It is suggested that stainless steel dowels be used to attach the barrier curbs to the deck. The dowels can be cut flush with the deck at the time of widening, and there will not be any corrodable reinforcement near the deck surface.

ADVANTAGES AND DISADVANTAGES

The SBWM is a valid bridge solution that has a wide range of applications, and it should be considered during the planning stages for any project. Consideration requires the careful evaluation of both its advantages and disadvantages, and an evaluation of the possible widening needs of the bridge in the future.

The advantages of the SBWM include the following:

1. Flexibility in widening — The final configuration of the bridge at the conclusion of its useful life can be two, three, or four lanes.

2. Structurally efficient solution — Examples 1 and 2 demonstrate that an increase in superstructure moment and shear capacity of 10 and 14 percent, respectively, gives a 50 percent increase in traffic capacity (or 20 and 28 percent, respectively, gives a 100 percent increase in traffic capacity).

3. Low initial cost — With respect to traditional bid projects, this solution will appeal to government agencies who are under increasing pressure to make their construction budgets go further. This will make sense to taxpayers who will not question why a four-lane bridge has been built when hardly enough traffic exists for a two-lane bridge. With regard to design-build projects, this especially makes sense because financing becomes so important. Why not pay for easily expanding the capacity of a design-build bridge when it is required in 20 or 60 years rather than today?

4. Reduced incremental cost and schedule — Widening existing bridges to double the traffic capacity is better than building new bridges because the construction cost and schedule are greatly reduced.

5. Reduced prestressing for cantilever construction — The amount of cantilever prestressing required for balanced cantilever construction is reduced, since a two-lane wide deck is cantilevered rather than a four-lane wide deck. In other words, a 13.5 m (44.3 ft) deck is cantilevered rather than a 21.4 m (70.2 ft) deck.

6. Potential to satisfy FHWA performance objectives — When used in conjunction with high performance concrete, the SBWM has the potential to satisfy all ten of the FHWA performance objectives[9] for "The Bridge of the Future." It readily satisfies the requirement "easily widened or adaptable to new demands," and Reference 10 discusses how it can be applied to satisfy other FHWA performance objectives.

Some of the potential disadvantages of the method include the following:

1. Increased design effort — The designer has to design for three stages of construction. By keeping the design for each stage of construction independent, however, the additional amount of design work is not substantial.

2. Greater substructure capacity — The foundations and piers have to be designed to support the Stage 3 service loads. Balanced cantilever construction requires increased substructure capacity for unbalanced moments, so there already is adequate load capacity for the Stage 3 service loads (i.e., Examples 1 and 2). Span-by-span construction (and other methods) normally require additional substructure capacity in Stage 1 to support the Stage 3 service loads.

3. Greater shear capacity — The shear reinforcement has to be provided in Stage 1 to support the Stage 3 service loads. For the two examples given here, the additional shear capacity required in Stage 1 varies from 18.3 percent for the variable-depth bridge having 160 m (525.0 ft) spans (Table 2) to 27.4 percent for the constant-depth bridge having 60 m (196.9 ft) spans (Table 3). In the best-case scenario, the web thickness required for Stage 1 and Stage 3 would be the same, meaning that only additional shear reinforcement would be required. In the worst-case scenario, a larger web thickness is required for Stage 3 than for Stage 1, meaning that the additional concrete required for the web would have to be supported by additional shear reinforcing and longitudinal prestressing. This additional concrete, reinforcement, and prestressing is wasted if widening to Stage 3 never occurs. [Note that in Example 2, the vertical component of the prestressing counteracts the demand for an increased web thickness, while in Example 1, an increased web thickness is provided — i.e., 450 versus 500 mm (17.7 versus 19.7 in.).]

4. Provision for future prestressing — Although the strands and anchorages for the transverse internal tendons and longitudinal external tendons are installed and stressed in Stage 2 and Stage 3 as they are required, provision for this future prestressing has to made in Stage 1. This means that ducts for transverse tendons have to be installed and protected. This also means that blisters for longitudinal tendon deviation and anchorage (as well as all anchorage zone reinforcement) have to be provided in Stage 1.

5. Possible need to close bridge during widening — If the erection truss described in this paper is used, the bridge will have to be closed during widening. If moveable formwork attached to the web and cantilever is used, however, the bridge may not need to be closed.

TWO POTENTIAL APPLICATIONS

Two potential applications of the SBWM are particularly appealing – namely, for major long-span river crossings and for long median-based expressways.

Suppose that a major long-span river crossing needs to be designed and constructed. Present traffic estimates are that the bridge requires only one lane in each direction (Stage 1). It will take a significant budget and construction effort to build this bridge. The owner can construct this bridge to have the lowest initial cost, or can spend slightly more to allow the bridge to be widened in the future to have two lanes in each direction (Stage 3).

Long portions of elevated median based expressways can be constructed in congested urban environments where right-of-way acquisition is prohibitive. The elevated structure can be constructed and widened in the future, as discussed in this paper, or it can be constructed directly to its ultimate configuration (three or four lanes). The SBWM allows a very wide elevated superstructure [21.7 m (71.2 ft)] to be constructed while having a relatively small substructure footprint in the median.

CONCLUDING REMARKS

The strutted box widening method (SBWM) introduced in this paper allows a two-lane segmental bridge to be designed and constructed so that it can be easily widened into a three-lane or four-lane bridge at any time in the future. This is an attractive solution since widening only needs to be done when and if traffic volumes warrant it.

The author believes that the strutted box widening method should be given serious consideration by government agencies and design-build consortia starting at the planning stage on any project. It is an excellent solution for building an economic and efficient bridge to handle current traffic volumes, while at the same time planning ahead to build cost-effective and schedule-effective bridge widenings to handle future traffic volumes.

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