

# Design and Construction of a Buried Precast Prestressed Concrete Arch



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*This paper describes the design, manufacture and construction of a buried precast, prestressed concrete arch, with a span length of 19.6 m (64.3 ft) and an overall length of 173 m (566 ft), which carries highway traffic over railway tracks near Obed in Alberta, Canada. In design, a nonlinear finite element analysis was undertaken to account for the interaction between the structure and surrounding soil, as the arch was subjected to backfill and highway loadings. This allowed the arch wall thicknesses to be reduced substantially when compared to the thicknesses used in conventional designs. The arch was erected over a Canadian National Rail main line without disturbing railway traffic. The structure was designed to provide Alberta Transportation and Utilities years of service, with maintenance costs lower than those normally associated with highway bridges.*

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**A** buried precast, prestressed concrete arch with a span of 19.6 m (64.3 ft) and an overall length of 173 m (566 ft) was recently constructed near Obed, Alberta, Canada, as part of the enlargement of the Yellowhead Highway (No. 16) between Edmonton and Jasper National Park. This arch carries highway traffic over two parallel tracks of the Canadian National Railway (see Fig. 1). The arch is believed to be the longest span structure of its type constructed over a railway line or highway in Canada.

A precast, prestressed concrete arch was selected for the railway overpass from considerations of highway traffic safety, initial cost and future maintenance costs for the structure, and the ease with which the highway could be



widened in the future. Railway traffic continued without interruption during the entire construction period.

This paper discusses the novel aspects of the arch design and describes the manufacture and installation of the arch. An important aspect of the paper is that the predicted behavior of the arch is compared to the observed behavior of the structure in the field.

The precast concrete arch was designed by Lamb McManus Associates Ltd. for Alberta Transportation and Utilities. The detailed geotechnical investigation, and the design of surface and subsurface drainage measures, were provided by the Geotechnical Services Section of Alberta Transportation and Utilities.

## SITE DESCRIPTION

Fig. 2 shows the roadway and track geometry at the location of the overpass. The highway consists of two eastbound and two westbound lanes, crossing the twin railway tracks. A perpendicular to the centerline of the highway median is on a 62½-degree skew to the centerline of the railway median.

At the location of the overpass, the roadway is on a curve with a 1200 m (3937 ft) radius and has a 5.9 percent superelevation. A portion of the railway track median is on a spiral curve, whereas the remainder is on tangent.

The geotechnical investigation for the site indicated that the soil stratigraphy below the level of the railway tracks consists of an approximately 9 m (29.5 ft) thick layer of very stiff, gravelly clay till of medium plasticity with a water content of 16 percent. The till is underlain by well-cemented silty sandstone bedrock. In one of the test holes, water was encountered 7.1 m (23.3 ft) below the original ground surface.

For the clay till, standard penetration test N-values varied from 26 to 94. The till is gravelly, sandy and silty, with a gray-brown color. Boulders were encountered in the clay till layer during drilling.

Prebored pressuremeter tests were carried out for the clay till. The tests gave Young's moduli values in the "working" range of 1700 to 3300 t/m<sup>2</sup>



Fig. 1. Completed arch.

(348 to 676 ksf). The values were lower than expected for till because the softer "loading" (rather than "unloading") portions of the pressuremeter curves were used in calculating moduli. A bulk modulus of about 2000 t/m<sup>2</sup> (410 ksf) was measured in the laboratory for the till. The sandstone bedrock is silty, fine grained and well-cemented, with a gray color.

## DESIGN ALTERNATIVES

Prior to undertaking the detailed design of the precast concrete arch, several design alternatives were considered for the overpass. To minimize snow drifting problems around the railway tracks in the winter, Canadian National Rail expressed a preference for the use of a continuous underground structure to support the highway.

The Bridge Engineering Branch of Alberta Transportation and Utilities also expressed a preference for an underground structure because it would be better suited to handling the complex roadway geometry at the overpass. They felt that a conventional bridge at the site could be a hazard during the winter, owing to the potential for preferential icing of the superelevated and curved deck surface. They also expressed a preference for structural alternatives that would allow for the easy future addition of one traffic lane in each direction on

the median side of the westbound and eastbound lanes.

The following alternatives were considered:

### 1. Buried structure with Reinforced Earth™ retaining walls —

This alternative would consist of precast, prestressed concrete box sections spanning between cast-in-place concrete footings, which would be supported on Reinforced Earth\* fill on either side of the tracks. The precast box sections would span perpendicular to the railway tracks, and would be covered by soil fill. Reinforced Earth walls would retain soil on either side of the tracks and at the entrances to the underground structure.

### 2. Buried structure with cast-in-place concrete retaining walls —

This alternative would consist of precast, prestressed concrete box sections spanning between ribbed concrete retaining walls. The retaining walls would be supported by spread footings below the level of the railway tracks. To resist the horizontal reaction from soil pressures at the bottom of the retaining walls, round HSS sections would be jacked under the railway tracks at the footing level. Reinforced Earth wing walls would be used at the entrances of the underground structure to retain soil.

\* Reinforced Earth™ is a system in which metal strips are introduced in the granular backfill behind retaining walls. The metal strips resist the tensile stresses that develop in the backfill owing to applied loads.



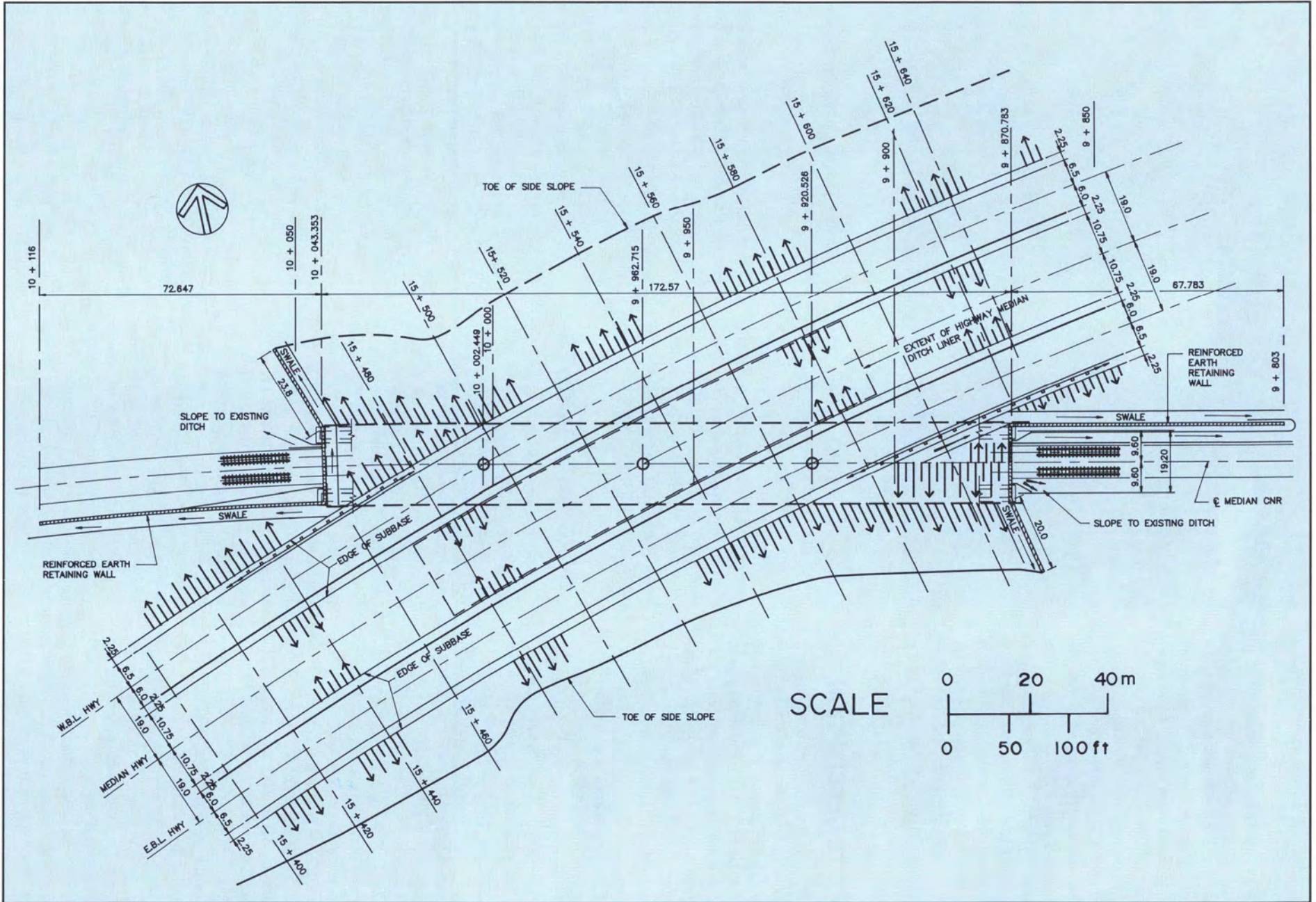


Fig. 2. Roadway and track plan at location of overpass.



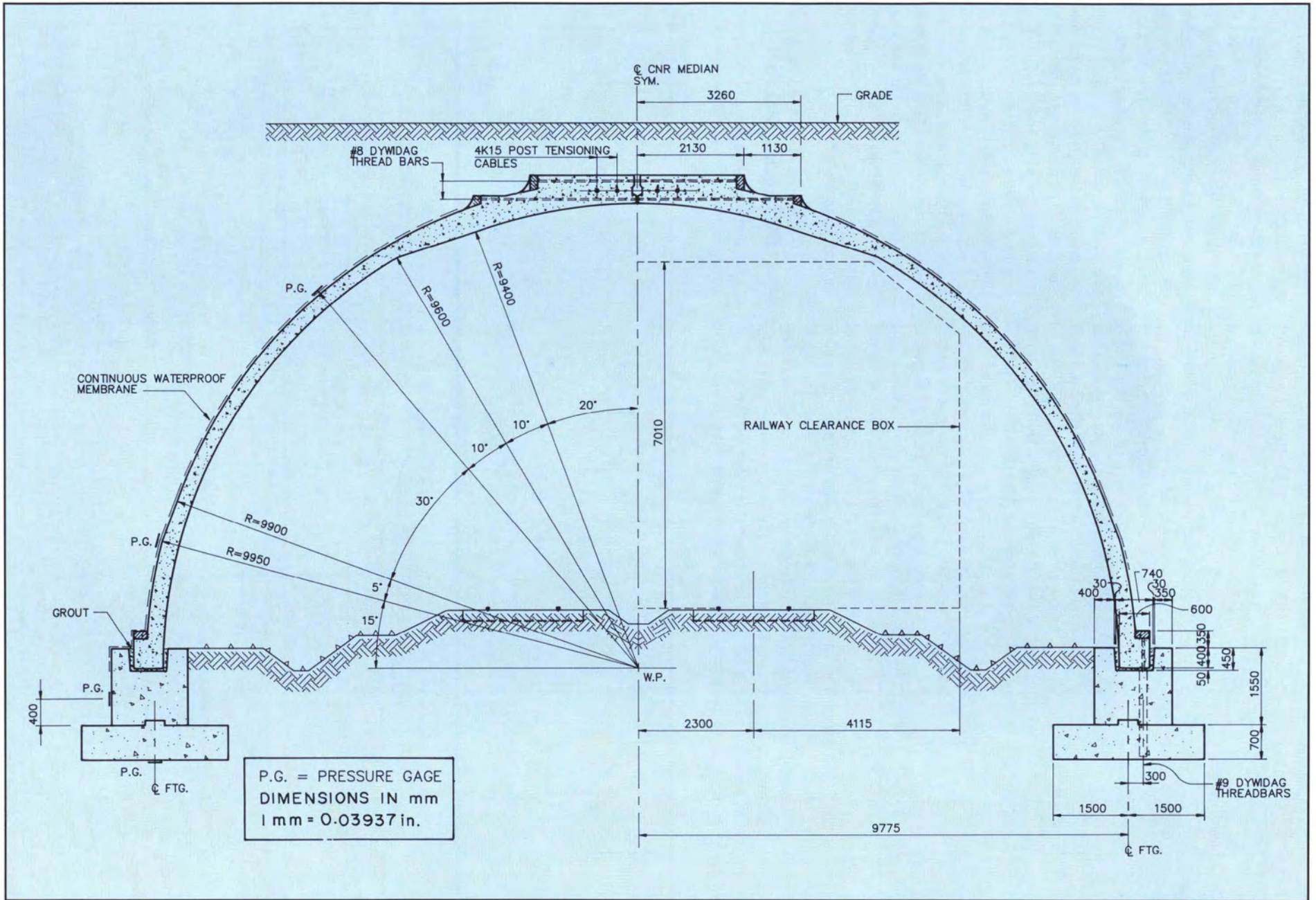


Fig. 3. Arch cross section.



Table 1. Cost comparisons (1987 Canadian dollars).

Alternative designs	Estimated construction cost
1. Buried structure with Reinforced Earth retaining walls	\$4,394,000
2. Buried structure with cast-in-place concrete retaining walls	\$4,849,000
3. Precast concrete arch	\$3,721,000
4. Bridge structure with Reinforced Earth abutments	\$3,516,000
5. Conventional bridge structure	\$3,758,000

**3. Precast concrete arch** — Shown by Figs. 1 and 3, this alternative would consist of two precast concrete arcs of a circle that are connected together at the top to form an arch. The arch would rely on reinforced concrete

footings below the level of the tracks for vertical support and soil pressures on the sides of the arch segments for lateral support. The arch segments would be cast in easy-to-handle lengths and erected sequentially. Head walls with a cast-in-place concrete facing in front of Reinforced Earth and Reinforced Earth wing walls would be used at the entrances to the arch structure to retain the approach and roadway fills.

**4. Bridge superstructure with Reinforced Earth abutments** — This alternative would consist of twin single-span bridge superstructures extending parallel to the roadway between Reinforced Earth abutments. One superstructure would be required for the eastbound lanes and a second for the westbound lanes. To achieve the proper roadway profile and grades, steel girders with variable thickness cast-in-place concrete decks would be used for the bridge superstructures. The potential for settlements of the superstructures would be reduced by

supporting the substructures on piles at the abutments.

**5. Conventional bridge structure** — This alternative would consist of three-span, continuous steel girders supported by conventional piers and abutments. One superstructure would be required to span parallel to the eastbound lanes, and a second, parallel to the westbound lanes.

Table 1 presents the estimated construction costs for the five alternatives at the time the preliminary engineering report for the project was prepared. From consideration of initial cost, traffic safety and future maintenance costs, the precast, prestressed concrete arch alternative was selected for the overpass. The bid cost for the total project in 1988 was \$3,900,000 (Canadian), including \$1,799,000 for the precast concrete work.

## DETAILED DESIGN

By conventional design, a concrete arch subjected to highway traffic

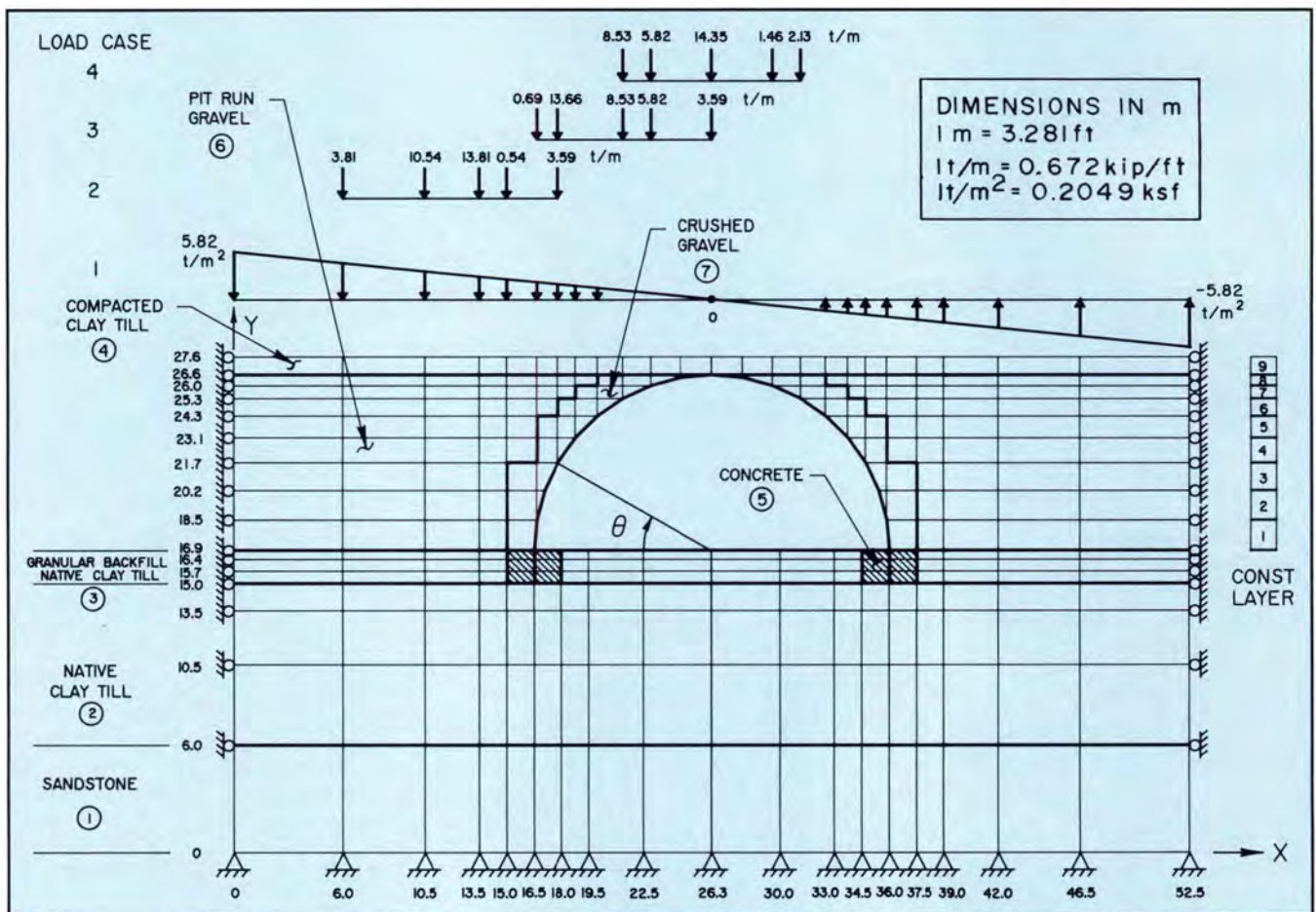


Fig. 4. Finite element mesh.

with 0.4 to 2.5 m (1.3 to 8.2 ft) of fill above it with the span shown in Fig. 3 would have had wall thicknesses in the range of 600 to 900 mm (24 to 36 in.). These large thicknesses would be required to resist the bending moments from soil pressures and highway traffic loadings, with relatively small deformations.

The precast concrete arch used for the railway overpass near Obed, Alberta, has wall thicknesses that are in the range of 300 mm (11.8 in.) (see Fig. 3). To achieve these substantial savings in materials, transportation costs and erection costs, it was assumed that the arch would interact with the surrounding soil to resist loads. The arch is relatively flexible so that large bending moments do not develop as the structure displaces and interacts with the surrounding backfill.

In the early 1980s, soil-structure interaction was considered in the design of precast concrete arches for the first time by Hebden.<sup>1</sup> Hebden designed three thin-walled precast concrete arches for the Coquihalla Highway in British Columbia, Canada. These arches have 20 m (65.6 ft) spans with 250 mm (9.8 in.) thick wall sections, and carry the highway over creeks. In contrast to the clay till at the Obed site, the footings for the Coquihalla arches were founded on rock.

Subsequent to the design of the Coquihalla culverts and the Obed culvert considered in this paper, the Reinforced Earth Company Ltd.<sup>2,3</sup> has introduced the trademarked TechSpan System to North America. The TechSpan System consists of 150 to 350 mm (6 to 14 in.) thick precast concrete segments which are erected to form a three-point hinged arch. A Reinforced Earth Company Ltd.<sup>3</sup> brochure illustrates a 12 m (39.4 ft) wide structure over a railway in Madrid, Spain.

### Computer Model

The Obed arch considered herein, and the Coquihalla arches, were analyzed using the NLSSIP computer program developed by Byrne and Duncan.<sup>4</sup> This program uses the finite element method to account for the nonlinear response of the soil-arch system. The soil was modeled by two-

dimensional isoparametric finite elements, assuming plane strain and isotropic behavior. The concrete arch was modeled by beam elements.

Fig. 4 shows the finite element model of the Obed arch and surrounding soil. The arch was loaded incrementally by assuming that the soil was placed sequentially in construction layers 1 through 9. Within each load increment, the structural response was assumed to be linear. To approximate nonlinear and stress-dependent soil behavior, the elastic properties of the soil were adjusted in accordance with calculated stress levels after the placement of each construction layer. Although geometric nonlinearities were accounted for, the arch beam elements were assumed to have the same elastic properties for all load increments.

The soil response was represented by a tangent Young's modulus,  $E_t$ , and a tangent bulk modulus,  $B_t$ , using the following expressions:

$$E_t = K p_a \left[ \frac{\sigma_3}{p_a} \right]^n \times \left[ 1 - \frac{R_f (1 - \sin \phi) (\sigma_1 - \sigma_3)}{2c \cos \phi + 2\sigma_3 \sin \phi} \right]^2 \quad (1)$$

and

$$B_t = K_b p_a \left[ \frac{\sigma_3}{p_a} \right]^m \quad (2)$$

In Eq. (1),  $K$  is the modulus number,  $n$  is the modulus exponent,  $R_f$  is the failure ratio,  $p_a$  is atmospheric pressure expressed in the same units as stresses,  $\sigma_1$  and  $\sigma_3$  are the major and minor principal stresses, and  $c$  and  $\phi$  are the Mohr-Coulomb strength parameters of cohesion and angle of internal friction. The development of Eq. (1) is explained by Duncan and Chang.<sup>5</sup>

Duncan et al.<sup>6</sup> indicate that the angle of internal friction to be used in Eq. (1) decreases in proportion to minor principal stress in accordance with the following expression:

$$\phi = \phi_o - \Delta\phi \log_{10} \left[ \frac{\sigma_3}{p_a} \right] \quad (3)$$

where  $\phi_o$  is the value of the angle of internal friction at a minor principal

stress equal to atmospheric pressure and  $\Delta\phi$  is the reduction in the angle of internal friction for a tenfold increase in the minor principal stress.

In Eq. (2),  $K_b$  is the bulk modulus number and  $m$  is the bulk modulus exponent. The development of Eq. (2) is explained by Duncan et al.<sup>6</sup>

Table 2 presents the parameters recommended by the Geotechnical Services Section of Alberta Transportation and Utilities for the various types of soil shown in the finite element model in Fig. 4. The earth pressure coefficients given in the table were used to evaluate the initial stresses in the foundation and each newly placed layer of soil during backfilling.

To account for various degrees of compaction, three sets of soil parameters were considered for the pit run gravel backfill. A controlled pit run gravel backfill was specified for the project, since it was necessary to achieve both adequate compaction and drainage around the arch structure.

Table 3 presents representative values of the tangent Young's modulus, tangent bulk modulus and angle of internal friction for the soil materials around the arch. These values were calculated for major and minor principal stresses equal to atmospheric pressure, which is equal to 10.33 t/m<sup>2</sup> (2.12 ksf).

The Young's and bulk moduli implied by the parameters given in Table 2 for Material 2, native till, were about 2 to 2.5 times greater than those justified by the laboratory and field test results referred to previously.

Although the test results were judged to be too low by the geotechnical specialist, the strength of the culvert was also checked assuming a modulus number of 400, a modulus exponent of 0, a failure ratio of 0.75, a bulk modulus number of 200, a bulk modulus exponent of 0, a cohesion of 3 t/m<sup>2</sup> (615 psf), an angle of internal friction of 22.5 degrees at a minor principal stress equal to atmospheric pressure, a reduction in angle of internal friction of 0, and an earth pressure coefficient of 0.7 for the native till.

For the concrete arch, an effective moment of inertia of 0.25 times the value calculated from the gross concrete section was used in the computer



Table 2. Properties of soils, concrete and gravel.

Parameter	Material*								
	1	2	3	4	5	6			7
	Sandstone	Native clay till	Soil around footings	Compacted clay till	Concrete	Pit run gravel			Crushed gravel
						Standard Proctor density (percent)			
100+	95	90							
Unit weight (t/m <sup>3</sup> )	2.14	2.14	2.24	2.24	2.4	2.32	2.24	2.16	2.16
Unit weight (pcf)	134	134	140	140	150	145	140	135	135
Modulus number	4000	820	450	450	259,000	1200	850	500	300
Modulus exponent	0.5	0	0.5	0.5	0	0.34	0.5	0.4	0.4
Failure ratio	0.6	0.75	0.7	0.7	0	0.6	0.7	0.7	0.7
Bulk modulus number	2400	450	175	175	124,000	720	450	175	75
Bulk modulus exponent	0.25	0	0.3	0.3	0	0.25	0.25	0.2	0.2
Cohesion (t/m <sup>2</sup> )	2.55	0	0	0	214	0	0	0	0
Cohesion (psf)	522	0	0	0	43,800	0	0	0	0
Angle of internal friction (degrees)	50	37	32	32	15	51	41	42	35
Reduction in angle of internal friction (degrees)	9	12	8	8	2	9	4	2	5
Earth pressure coefficient	0.48	0.48	0.5	0.7	0	1.5	0.75	0.5	0.75

\* See Fig. 4 for location in computer model.

Table 3. Representative values of moduli and angle of internal friction.

Parameter	Material*			
	1 Sandstone	2 Native clay till	4 Compacted clay till	6 Pit run gravel (Density 95 percent)
Tangent Young's modulus (t/m <sup>2</sup> )	41,300	8470	4650	8780
Tangent Young's modulus (ksf)	8470	1740	952	1800
Tangent bulk modulus (t/m <sup>2</sup> )	24,800	4650	1810	4650
Tangent bulk modulus (ksf)	5080	952	370	952
Angle of internal friction (degrees)	50	37	32	41

\* See Fig. 4 for location in computer model.

model for the beam elements (see Fig. 4). An effective moment of inertia was used to account for increases in flexibility owing to cracking, creep and shrinkage effects.

In the computer model, the footings were allowed to deflect and rotate as the surrounding soil elements deformed under load.

For strength calculations, load factors of 1.3 and 0.98 were used for soil

placed in layers above the footings and for the self-weight of the arch, respectively. Load Case 1 in Fig. 4 is for unbalanced fill on the arch. The truck loads shown for Load Cases 2, 3 and 4 are for a MS300 (HS33.7) truck, assuming a load factor of 2.17 and a 3.7 m (12 ft) lane width. Although the road crosses on a skew, for design it was conservatively assumed that trucks cross perpendicular to the lon-

gitudinal axis of the arch.

To account for a total backfill depth of 2.5 m (8.2 ft) above the crown of the arch at certain locations along the length of the structure, for some of the results presented, an additional surcharge load of 3.36 t/m<sup>2</sup> (0.69 ksf) was added to the loads shown in Fig. 4. This surcharge load accounts for 1.5 m (4.9 ft) more backfill than is represented by the model shown in Fig. 4.

In the analysis, the displacements and stresses in the soil and the displacements, forces and moments in the arch were recorded after each layer of soil (from Layers 1 through 9 in Fig. 4) was added. This simulates the response of the soil-structure system as backfill was placed. The corresponding displacements, stresses, forces and moments were also recorded after each load case was considered.

### Results of Computer Analysis

Fig. 5 presents the total factored bending moments for the concrete arch as the soil layers are added during backfilling. These moments were calculated using the Table 2 soil proper-



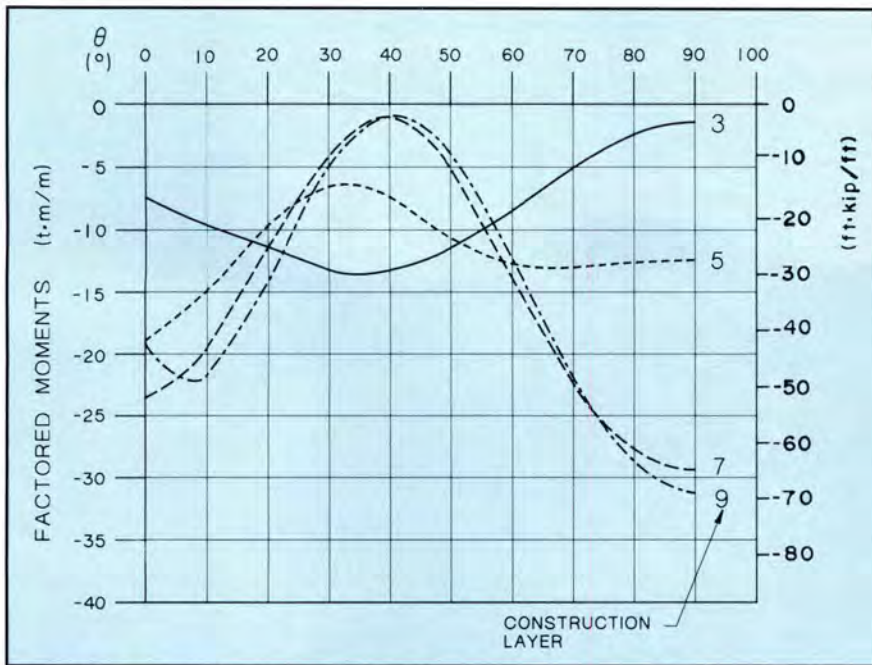


Fig. 5. Sample bending moments during backfilling.

ties with pit run gravel backfill compacted to 95 percent of Standard Proctor density. The angle  $\theta$  is measured relative to the top of the left-hand footing in Fig. 4. Moments are positive if they result in compression on the outside fiber of the arch.

Fig. 6 shows the incremental factored bending moments for the unbalanced soil and truck loadings described in Fig. 4. Although a nonlinear analysis was used, the incremental moments of Fig. 6 can be added to the moments for Layer 9 in Fig. 5 to obtain an approximation of the combined moments

for the various loading conditions.

One might anticipate that rock is required below the footings of an arch to offer enough restraint to sustain negative moments at the junction between the bottom of the arch and the tops of the footings. It might be anticipated that the clay till founding material at Obed is not stiff or strong enough to resist negative moments.

Even though the Obed structure is not founded in rock, the geometry and stiffness of the arch and surrounding soil are such that negative moments can be sustained at the junction be-

tween the bottom of the arch and the tops of the footings (see Fig. 5).

The factored axial forces in the arch following the placement of backfill Layer 9 were 58, 70 and 123 t/m (39, 47 and 83 kip/ft) at angles of 90, 40 and 0 degrees, respectively. The incremental factored axial forces for Load Case 4 (see Fig. 4) were 18, 25 and 19 t/m (12, 17 and 13 kip/ft) at the respective positions on the arch. Load Case 4 resulted in larger incremental axial forces than did Load Cases 2 and 3.

Fig. 7 shows the computed soil stresses at working load levels assuming that 2.5 m (8.2 ft) of backfill has been placed over the crown of the arch. By way of comparison, assuming an earth pressure coefficient of 0.5, by conventional calculations one would expect horizontal soil pressures varying from 0 at the ground surface to 13.7 t/m<sup>2</sup> (2.81 ksf) at the level of Point 139 in Fig. 7.

An estimate of the soil pressures under the footings can be made by summing the weight of the soil above the footings and arch, the weight of the arch and the weight of the footings, and dividing this total by the sum of the footing areas. By these conventional calculations, the soil pressure would be approximately 56.3 t/m<sup>2</sup> (11.5 ksf), assuming 2.5 m (8.2 ft) of soil is above the arch crown. The vertical pressures shown in Fig. 7 are approximately 20 percent lower than the pressures obtained by conventional calculations.

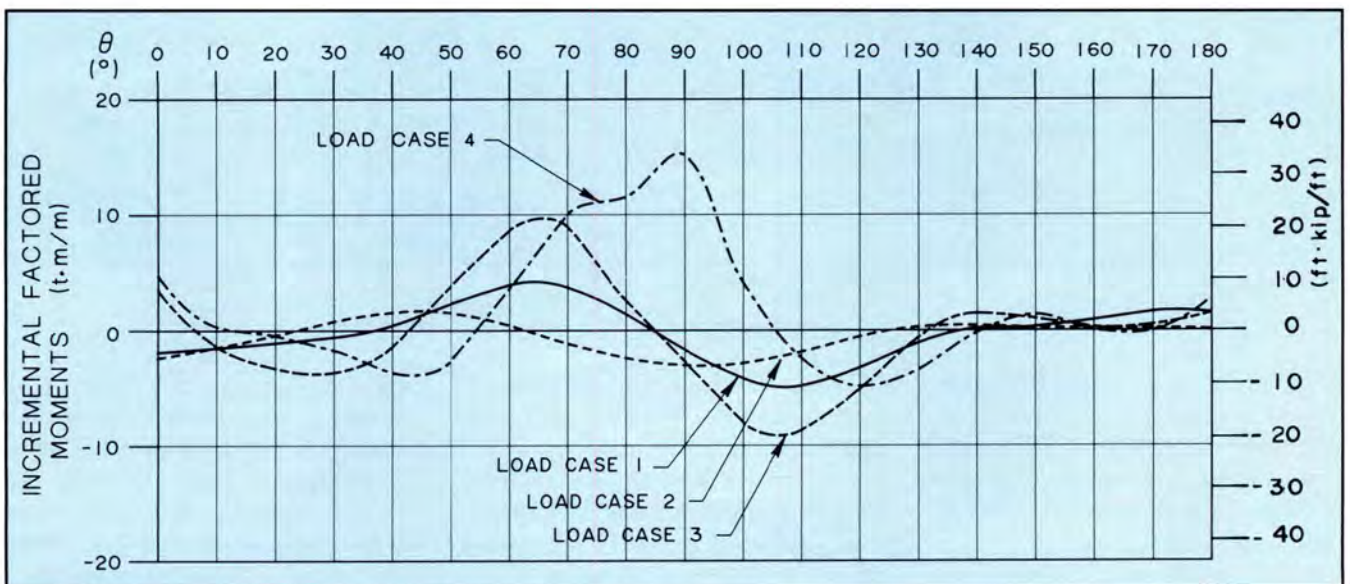


Fig. 6. Sample bending moments from unbalanced soil and truck loading.



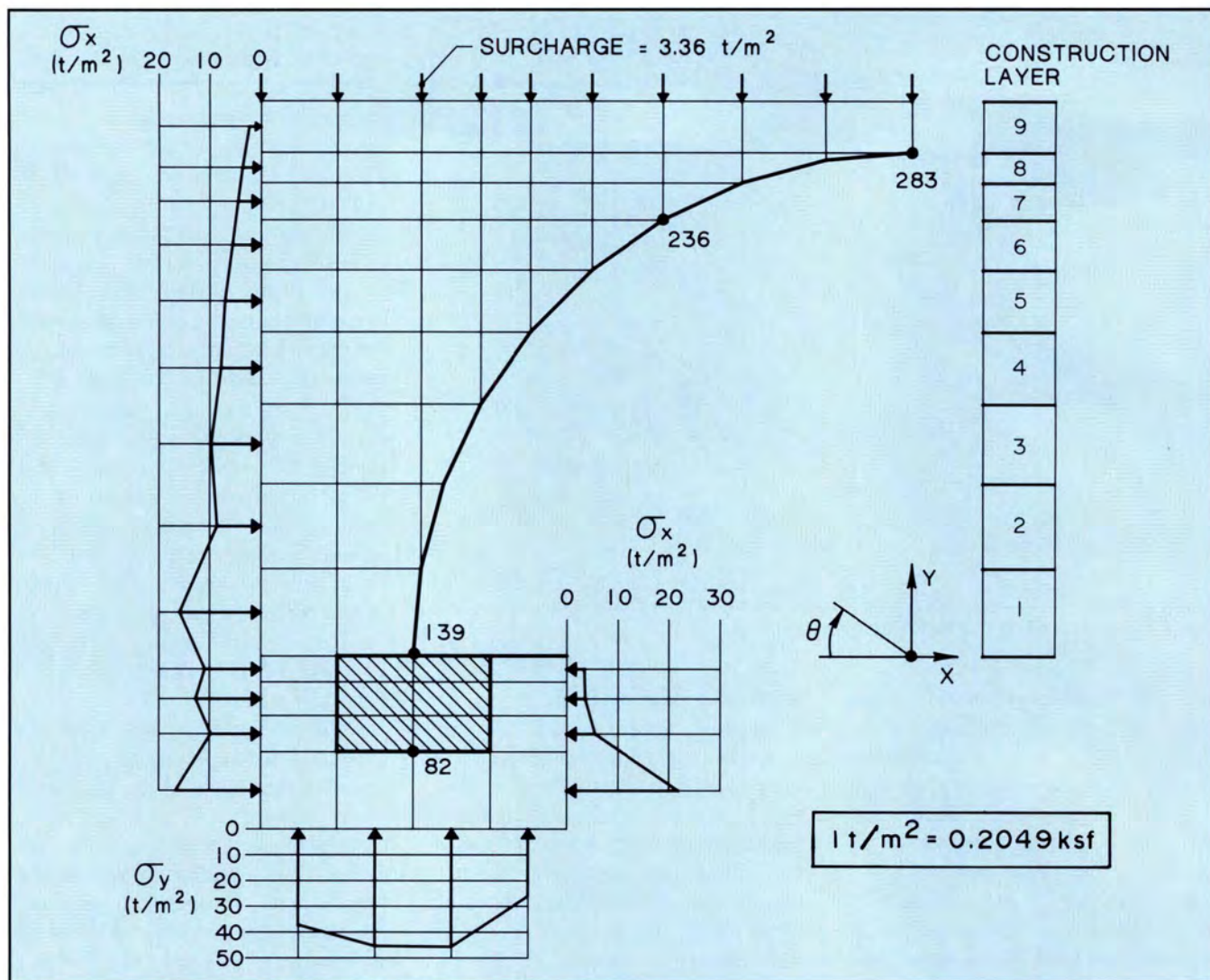


Fig. 7. Soil pressures for backfill 2.5 m (8.2 ft) above crown.

Table 4. Calculated displacements at locations around arch.

Point*	Total				Incremental	
	Soil at crown (Layer 8)		Soil 2.5 m above crown		Truck load at crown (Load Case 4)	
	X (mm)	Y (mm)	X (mm)	Y (mm)	X (mm)	Y (mm)
82	9.2	-26.5	7.8	-43.1	-1.1	-1.8
139	4.5	-26.5	1.2	-43.1	-1.2	-1.8
236	12.4	-8.8	9.7	-33.2	0.0	-5.8
283	0.0	26.4	0.0	-6.0	1.3	-8.7

\* See Fig. 7 for locations of displacements.  
 Note: 1 mm = 0.03937 in.; 1 m = 3.281 ft

The following are possible reasons why the vertical soil pressures under the footings determined from the computer analysis are smaller than the pressures obtained from conventional calculations:

1. The footings are founded on clay till, which is more compressible than

the rock often present for arch structures. As the Obed structure was back-filled, the computer analysis accounted for the settlement of the footings relative to the adjacent soil. Some of the weight of the soil above the structure was transferred by arching action and shear stresses to the

adjacent backfill material.

2. The pressures from the computer analysis are determined at the centers of the finite elements, rather than at locations directly under the footings. If a finer finite element mesh had been used, higher vertical pressures may have been determined from the analy-



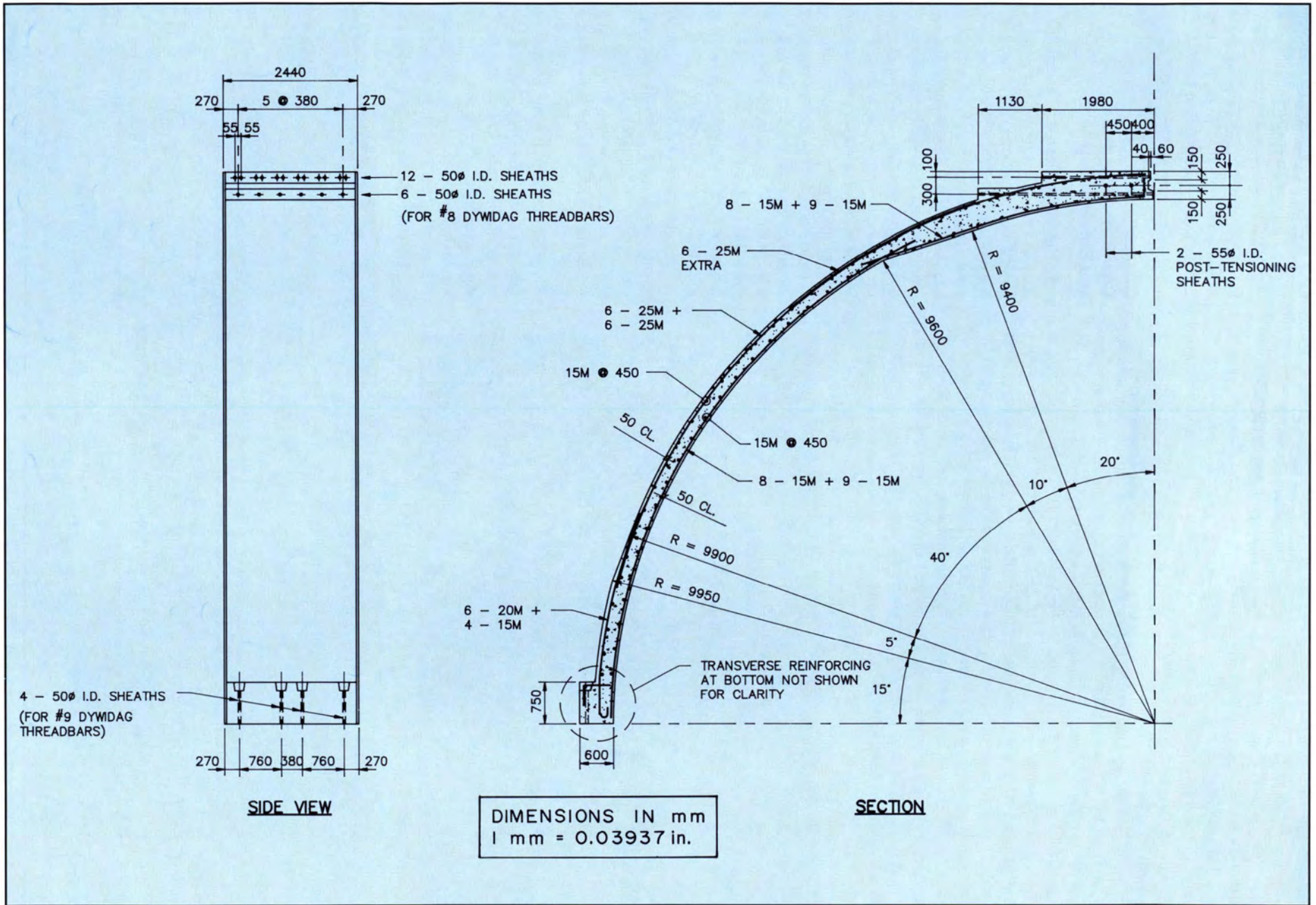


Fig. 8. Typical precast concrete arch segments.



sis directly below the footings.

Working level displacements of the culvert during backfilling were also calculated by the computer program. Table 4 lists displacements at locations around the arch and at the footing for various stages of construction. The points referenced in Table 4 are shown in Fig. 7. The same soil properties were used in the analysis undertaken to prepare Table 4 and Fig. 7.

Upon backfilling to the level of the arch crown (Layer 8 in Fig. 7), the total calculated footing settlement was 26.5 mm (1.04 in.) and the upward movement at the crown was 26.4 mm (1.04 in.). With 2.5 m (8.2 ft) of backfill, the total calculated footing settlement increased to 43.1 mm (1.70 in.), but the crown moved 6.0 mm (0.24 in.)

below its initial position prior to the start of backfilling. As shown by Load Case 4 in Table 4, the calculated incremental displacements for truck loading were relatively small.

### Arch Design

Fig. 8 shows the reinforcing steel arrangement used for a typical arch segment. Concrete with a minimum 35 MPa (5080 psi) compressive strength at 28 days and reinforcing steel with a 400 MPa (58 ksi) yield strength were used to construct the arch segments.

The reinforcing steel was selected so that the moments and axial forces calculated from a large number of computer analyses for various soil conditions were within the moment-axial

force interaction diagrams for the arch wall at all sections. In general, the computed axial forces were well below the axial capacity of the arch sections at balanced load conditions, ensuring ductile behavior near failure loads.

### Connections at the Crown and Footings and Between Segments

In the initial design, the structure was assumed to be a three-point hinged arch. Computer calculations indicated that both wall bending moments and displacements would have been unacceptably large for the three-point hinged system. To reduce moments and deflections for final design, the adjacent arch segments were assumed to be continuous at the crown and rigidly connected to the footings.

Figs. 3 and 8 show the thickening of the arch segments at the crown and the 414 MPa (60 ksi) grade Dywidag threadbars which were used in conjunction with a grouted longitudinal joint to achieve continuity.

The connection of the arch segments to the conventionally reinforced concrete strip footings is also illustrated in Figs. 3 and 8. By connecting the arch segments to the footings with grouted longitudinal joints, negative moments could be developed at the bottom of the arch as the weight of soil and lateral earth pressures caused the footings to rotate relative to the arch during backfilling.

Note that the Dywidag threadbars transfer the tensile forces at the inter-



Fig. 9. Footing construction.

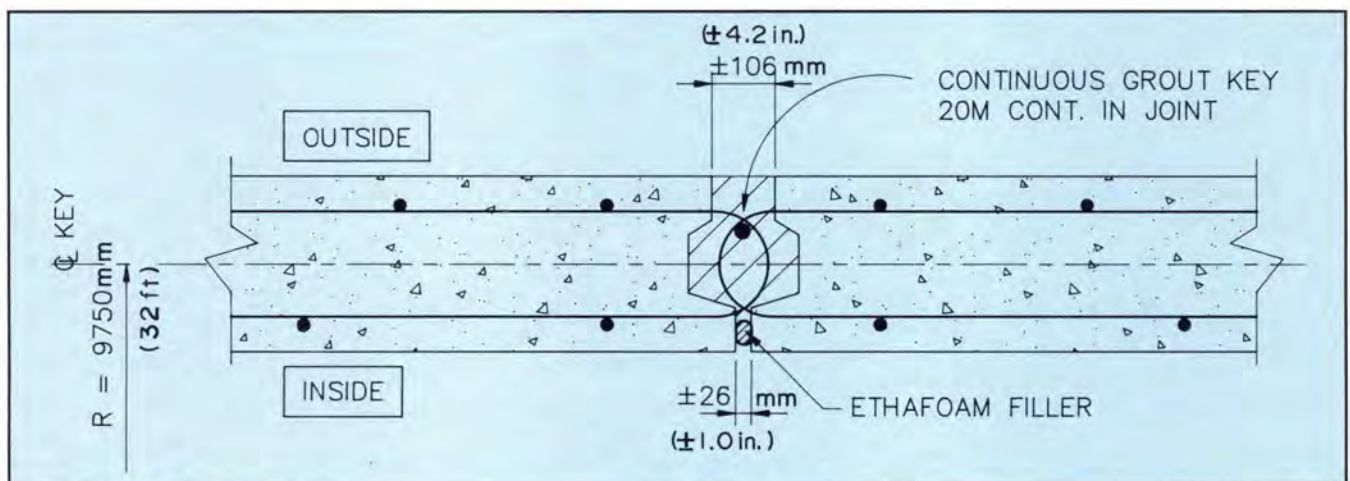


Fig. 10. Typical arch circumferential joint.



faces between the precast concrete segments and footing pedestals to anchor plates near the tops of the "knobs" at the bottoms of the segments (see Fig. 8). These tensile forces were transferred to the outer layers of the reinforcing steel in the arch segment walls by concrete truss action.

Fig. 9 shows the corrugated pipes that were cast into the footings to accept the Dywidag threadbars from the arch segments.

Adjacent arch segments were also connected together circumferentially by grouted joints, as shown in Fig. 10.

Although not relied upon to carry loads, the complete arch was tied together at the crown by four 4K15 longitudinal post-tensioning cables (see Fig. 3). These cables help to prevent circumferential shrinkage cracks from forming in the arch and assist in transferring vehicular loads between adjacent segments.

## PLANT MANUFACTURING

The 140 segments for the arch were manufactured in Edmonton by Con-Force Structures Limited over a period of seven months, starting in the summer of 1988. In peak manufacturing periods, one segment was cast each day. The segments had a width of 2440 mm (96 in.) and a mass of slightly less than 35 t (77.2 kip weight).

The reinforcing steel for a given arch segment was tied on a wood jig the day before casting. It took approximately 12 man-hours to tie the steel for one segment.

The next day, the reinforcing steel was lifted by crane into the steel form shown in Fig. 11. The reinforcing steel was adjusted, final reinforcing bars were added, and the segment was cast by expending approximately 40 man-hours.

The segment was cured overnight. The next day, the segment was removed from the form by means of a four-point crane lift, and placed on the plant floor in the same position as it was cast. To reduce stresses, the ends of the segments were connected together by using reinforcing bars to form a tied arch. These reinforcing bar ties also maintained the shape of the arch segments from the time the segments were removed from the form until they were erected.



Fig. 11. Form for arch segments.



Fig. 12. Trucking of arch segment.

The segment was then moved to the plant yard for storage. Near winter's end, the segments were shipped to the project site.

## SITE ERECTION

In February of 1989, the arch segments were trucked a distance of 240 km (150 miles) from Edmonton to the Obed site. Fig. 12 shows the trucking of one of the segments. The reinforcing bars used to connect the ends of the segment together to form a tied arch are apparent in the photograph.

As shown in Fig. 13, the segments were erected in pairs without falsework using two conventional 81.6 t (90 Imperial ton) cranes. Canadian National Rail site representatives cooperated with the precast concrete contractor during the

erection of the arch segments, allowing at least one window of two hours each day for erection.

Even though erection could not proceed within 30 minutes of the passage of a train, during peak construction periods it was possible to erect four pairs of segments in one day. The erection of the entire structure was completed in about one month.

By way of comparison, the erection procedure used for the Obed arch was different from that used for the Coquihalla project (Hebden<sup>1</sup>). For the Coquihalla arches, the segments were erected on top of a structural steel falsework system.

The Obed arch segments were held in position by neoprene pads and blocking at the footings, and temporary connections at the crown. Imme-





Fig. 13. Erection of arch segments.



Fig. 14. Arch segment grouted into footing pedestal.

diately following erection, the structure was a three-point hinged arch. Because the arch was erected during the winter, it was not possible to grout the bottoms of the segments into the tops of the footings, to grout the longitudinal joints between the adjacent segments, or to grout the circumferential joints between segments as erection proceeded.

At approximately 1100 m (3610 ft) above sea level, Obed is at the highest elevation on the Yellowhead Highway. Because of the relatively high elevation and northern latitude, there are relatively few frost-free days per year at the site. At the time of bidding, the

precast concrete contractor planned to employ proper winter curing procedures for the grouted joints. Fortunately, the footings of the arch, and joints between the arch segments, were grouted without difficulty in a period of about six weeks in the summer of 1989.

Fig. 14 shows the base of the arch after it was grouted into the footing pedestal.

### Deflections

Because the arch segments were relatively thin and flexible, the crown of the arch deflected downward by ap-

proximately 170 mm (6.7 in.) immediately after erection. This deflection increased by approximately 43 mm (1.7 in.) before the segments were grouted together in the summer of 1989. By this time, a longitudinal trough had formed along the crown of the structure. In addition, 0.2 mm (0.0079 in.) wide longitudinal cracks spaced at about 200 mm (8 in.) on center formed on the outside faces of the arch segments. The deflection and longitudinal cracking of the arch during erection were completely predictable.

The design drawings should have specified an upward camber for the arch to counteract deflections owing to self-weight. Alternatively, the tops of the arch segments could have been cast with slopes so that water would run off the crown of the structure after the segments were grouted together.

## MOISTURE CONTROL AND EARTHWORK

Moisture control after the completion of the project was a major design consideration. Care was taken in the design to detail the arch to prevent moisture from leaking through the structure. This would prevent moisture from forming icicles on the arch roof during cold weather, and from collecting at the level of the railway tracks. The ditches on either side of the railway tracks were crowned within the arch to prevent water from ponding in the structure.

Fig. 15 shows some of the measures which were taken to control moisture. A 300 mm (12 in.) thick impervious clay till layer was specified as the last lift of backfill over the arch. An impervious liner was provided for the portion of the highway ditch over the arch (see Fig. 1 for the extent of the ditch liner). Also, a continuous waterproof membrane was applied over the arch structure. A free-draining layer of crushed gravel was specified around the arch to help drain moisture to the weeping tile pipes on the soil side of the footings.

To prevent moisture from reaching the arch footings and possibly softening or weakening the founding soil, the pedestals were backfilled with clay till. Also, the railway ditches and the soil



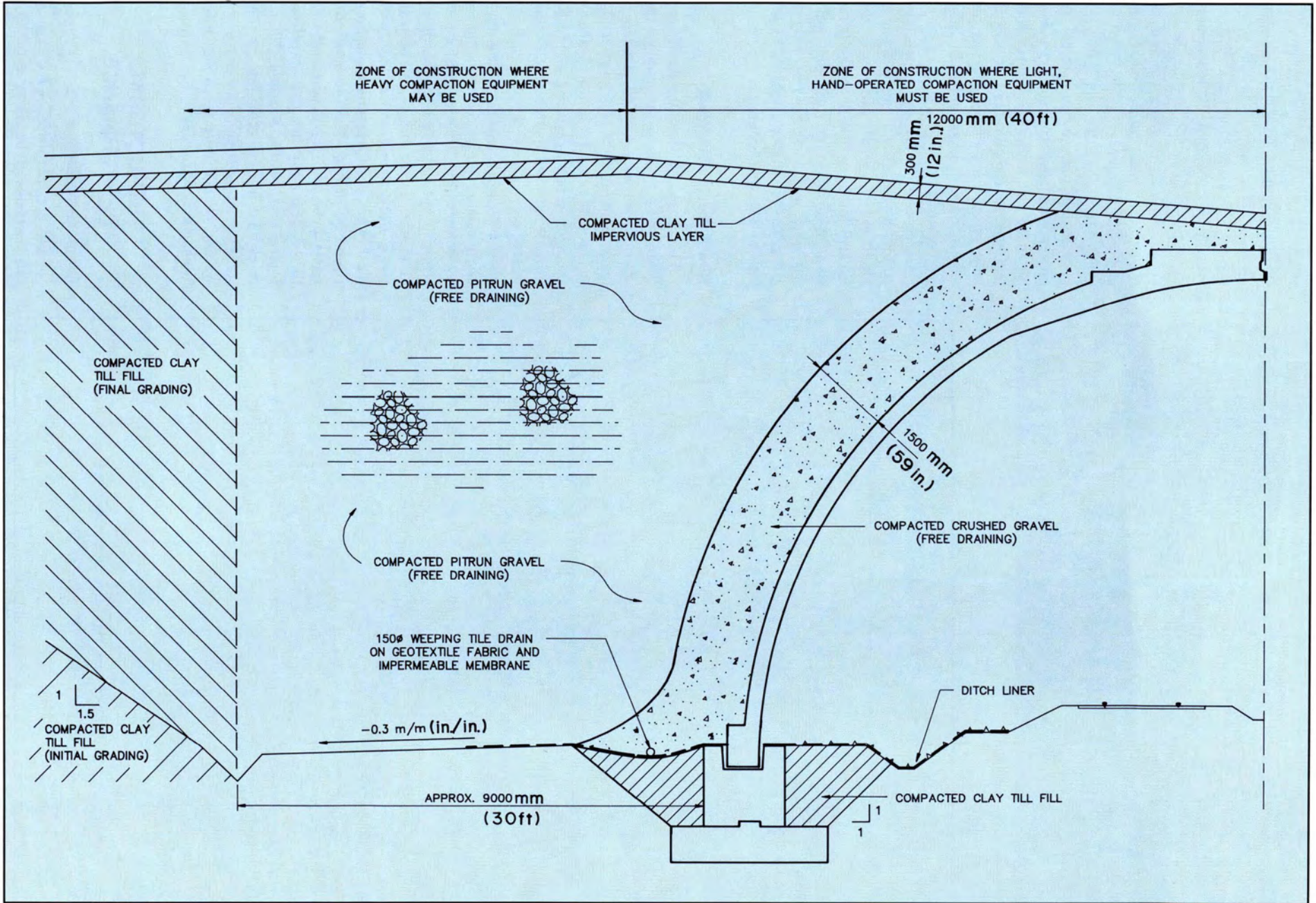


Fig. 15. Moisture control and backfill types.





Fig. 16. Backfilling of arch.

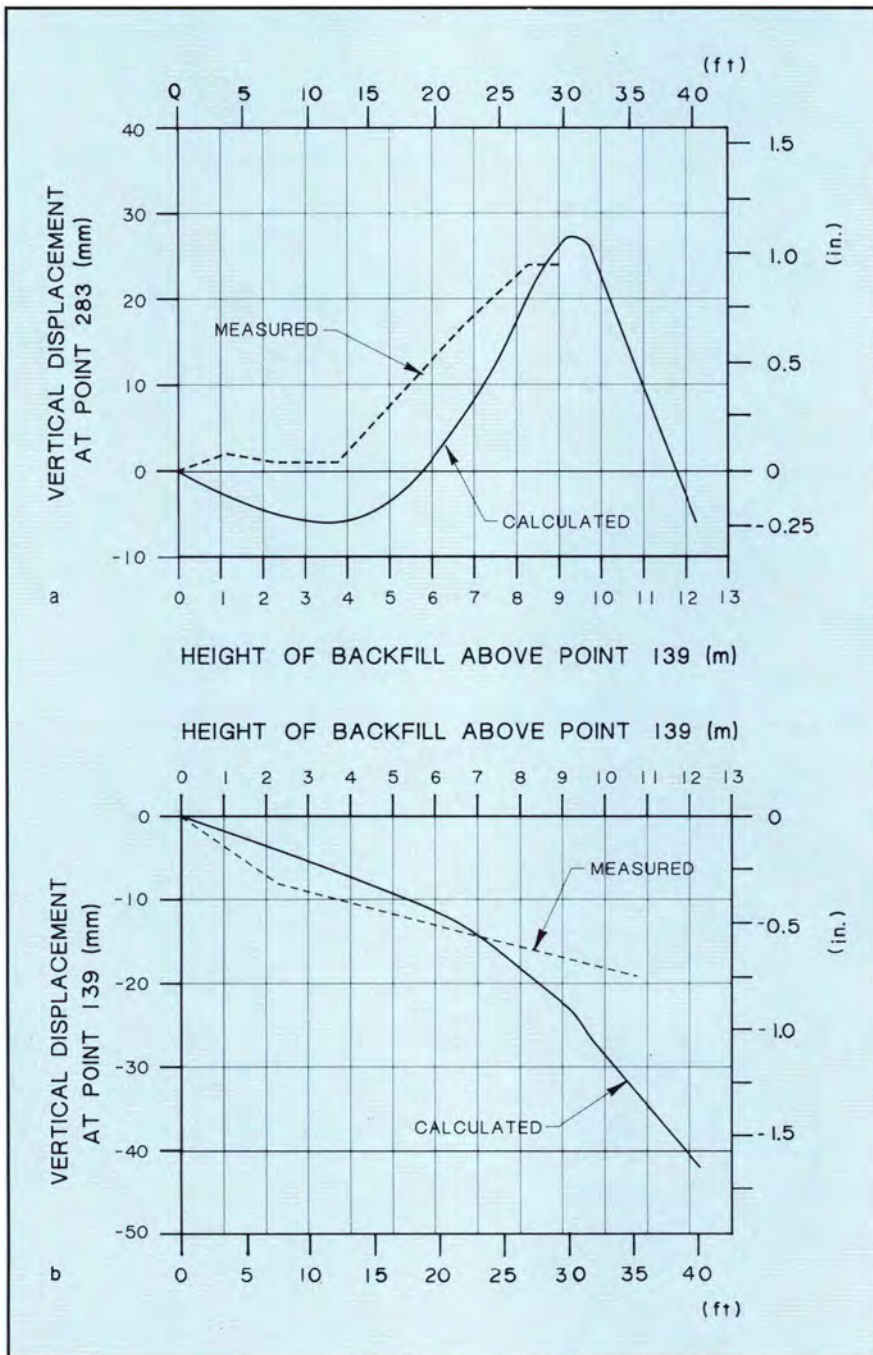


Fig. 17. Comparison of measured and calculated displacements.

below the weeping tile drains were covered with impermeable membranes.

Fig. 15 also shows the initial grading of the site and the backfill types specified for the arch. To reduce moments during construction, heavy compaction equipment was not allowed in the zone immediately beside and above the arch.

Fig. 16 shows the arch after the backfill was partially completed. To prevent large moments from developing in the arch walls, the specifications required that the elevations of backfill on opposite sides of the structure not vary by more than 200 mm (8 in.) during construction.

Because of the relatively short construction season and wet weather at Obed, only a small portion of the backfill for the arch was placed in the summer of 1989. The remaining backfill work was completed by July of 1990. The backfilling operation took a total of 64 days to complete.

### Reinforced Earth Walls

Figs. 2 and 16 show the Reinforced Earth wing walls at the ends of the arch. These walls were constructed as the backfilling of the arch progressed.

The soil behind the head walls of the arch was also strengthened by using Reinforced Earth strips. In contrast to conventional Reinforced Earth walls with precast concrete face panels, the head walls were constructed by connecting the soil reinforcing strips to horizontal 20 M reinforcing steel bars at the backs of the wall faces. Filter fabric and welded wire mesh were placed behind these horizontal 20 M bar walers to retain the granular backfill. Upon completion of the backfilling of the arch and Reinforced Earth walls, 300 mm (12 in.) thick cast-in-place concrete facings were poured in front of the horizontal walers, welded wire mesh and filter cloth at the head walls.

## STRUCTURAL PERFORMANCE

### Displacements

Field measurements of the vertical displacements of the crown of the arch and the tops of footing pedestals were



made at various stages during backfilling — by the Alberta Research Council (Sadoway and Christison<sup>7</sup>), Alberta Transportation and Utilities, and a representative of the contractor — using conventional surveying equipment. These measured displacements are compared to the calculated displacements in Figs. 17(a) and 17(b) for Points 283 and 139, respectively, shown in Fig. 7. The displacements were calculated by means of the finite element analysis described previously using the Table 2 soil properties, with pit run gravel compacted to 95 percent of Standard Proctor density.

Relatively small changes in displacements occurred between October 21, 1989, and May 7, 1990, when backfilling operations were stopped at a height of 2.3 m (7.5 ft) for the winter. These small changes in displacements were neglected in plotting the measured displacements in Fig. 17. It was reasoned that the displacements which occurred over the winter were likely caused by such variables as seasonal moisture changes or frost in the soil, and should not be compared to the displacements resulting from the backfilling of the arch structure.

Because the measured and calculated displacements compare favorably, it can be concluded that the analytical model used herein adequately predicted the response of the arch to the loads caused by backfilling.

### Soil Pressures

Instrumentation to measure soil pressures was installed at two locations along the length of the arch. At each location, eight pressure gauges were installed, with four placed symmetrically on either side of the centerline of the Canadian National Rail median. Fig. 3 shows the positioning of the gauges on one side of the centerline of the median.

The pressure gauges (Sadoway and Christison<sup>7</sup>), consist of two circular stainless steel plates, approximately 230 mm (9.1 in.) in diameter, welded together around their periphery and spaced apart by a narrow cavity filled with antifreeze. High-pressure stainless steel tubing connects the cavities to strain gauge sensors. Pressures act-

ing on the gauges are converted to electrical signals which are transmitted to a readout device.

Soil pressures were measured at various stages during backfilling, initially by the Alberta Research Council (Sadoway and Christison<sup>7</sup>) and at later stages by Alberta Transportation and Utilities. Fig. 18 compares measured and calculated soil pressures at the four positions on the structure shown in Fig. 3. The same soil properties were used in the calculations undertaken to prepare Figs. 17 and 18.

Of the 16 pressure gauges installed, 14 remained functional over the four-year period for which field measurements were made. The measured pressures plotted in Fig. 18 represent average pressures from a total of either three or four gauges at two locations along the length of the arch structure. Even though the soil pressure measurements were made over a relatively long period of time by two different groups under field conditions, the measured soil pressures compare reasonably well with the calculated pressures in Fig. 18.

It is interesting to note that the vertical soil pressures at the bottoms of the arch footings appeared to increase with time between October 30, 1989, and June 4, 1990, while backfilling was halted for the winter. The pressures appeared to increase again after July 13, 1990, upon completion of backfilling.

These increases in footing soil pressures may have occurred because the shear stresses that developed in the soil around the footings as backfill was placed dissipated with time, transferring an increased portion of the gravity loads to the footings. Additional load may also have been transferred to the arch structure by shear stresses as the backfill settled around the structure, or as the founding clay till below the backfill compressed.

The measured vertical footing pressures [see Fig. 18(a)] appeared to approach the pressure estimated by conventional calculations. As noted previously, by conventional calculations, the footing pressures are determined by summing the weight of the soil above the footings and arch, the weight of the arch and the weight of the footings, and dividing this total by

the sum of the footing areas.

When the procedures described herein are used for the design of similar arches in the future, it is recommended that the maximum of the pressures calculated by conventional procedures or by finite element analyses be used for proportioning footings.

### Axial Forces and Moments

The axial forces and moments in the arch were not evaluated as part of the field investigation. The reason for this is that the results of conventional strain gauge instrumentation would have been difficult to interpret. When strain gauges are mounted on reinforcing bars, it is difficult to determine how much of the moment at a section between cracks is resisted by the reinforcing steel and how much is carried by the concrete.

We believe that the actual moments are reasonably close to the predicted factored moments divided by the appropriate load factor, since no distress has been observed in the arch structure.

## SUMMARY AND CONCLUSIONS

The precast, prestressed concrete arch in Obed, Alberta, is now open to highway traffic. The performance of the structure during backfilling was predicted by a nonlinear finite element computer program which was used to analyze the interaction between the arch and surrounding soil. The results of the computer calculations for structural displacements and soil stresses were verified by field measurements. An analysis of the type used for the Obed arch is recommended for underground structures where it is desired to save construction materials by accounting for the ability of the soil and structure to act together when resisting loads.

The arch was erected over twin railway tracks without placing constraints on the railway traffic. Railway traffic proceeded without interruption during the entire construction period.

Because the potential for roadway icing in the winter months will be diminished, the underground arch should prove to be safer for highway traffic than a conventional bridge



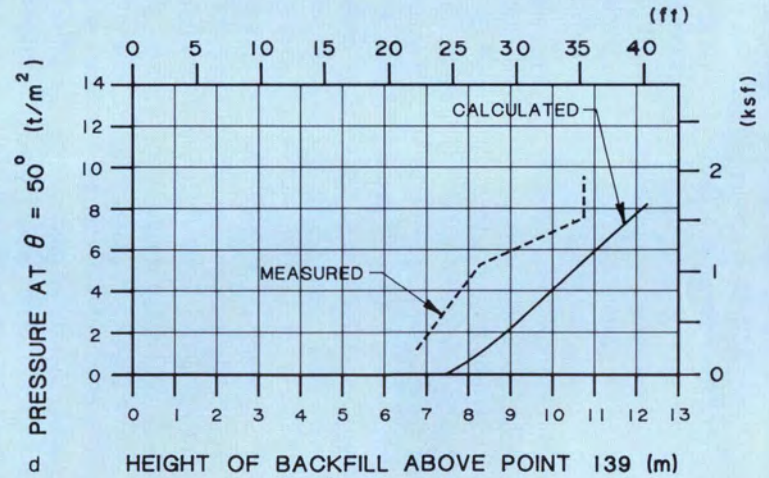
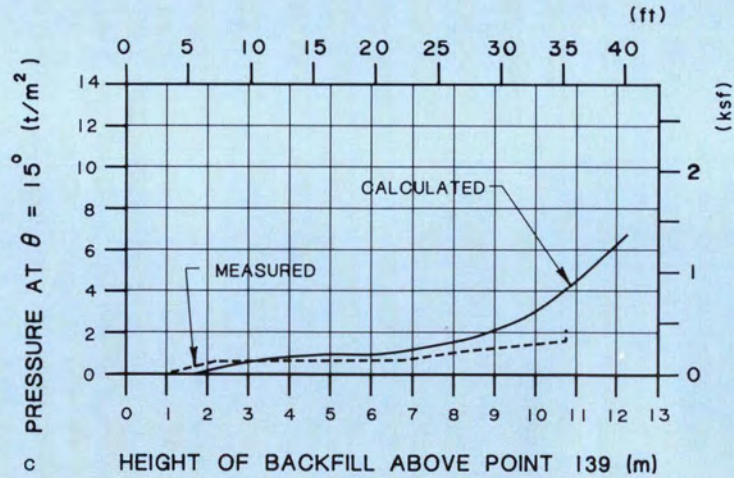
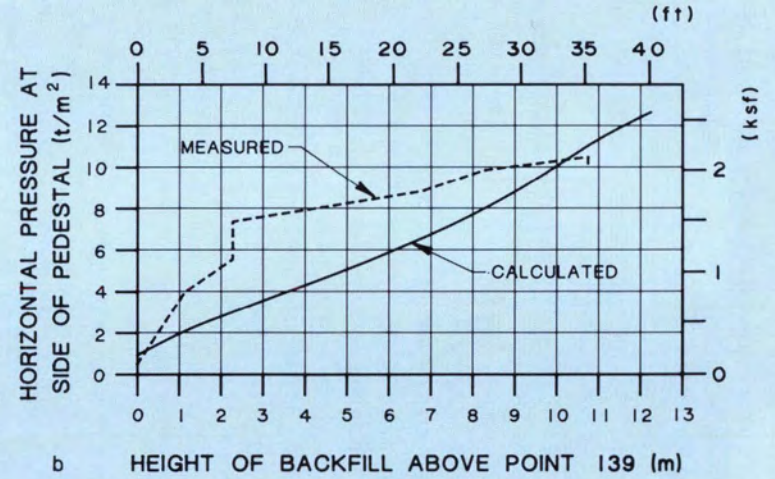
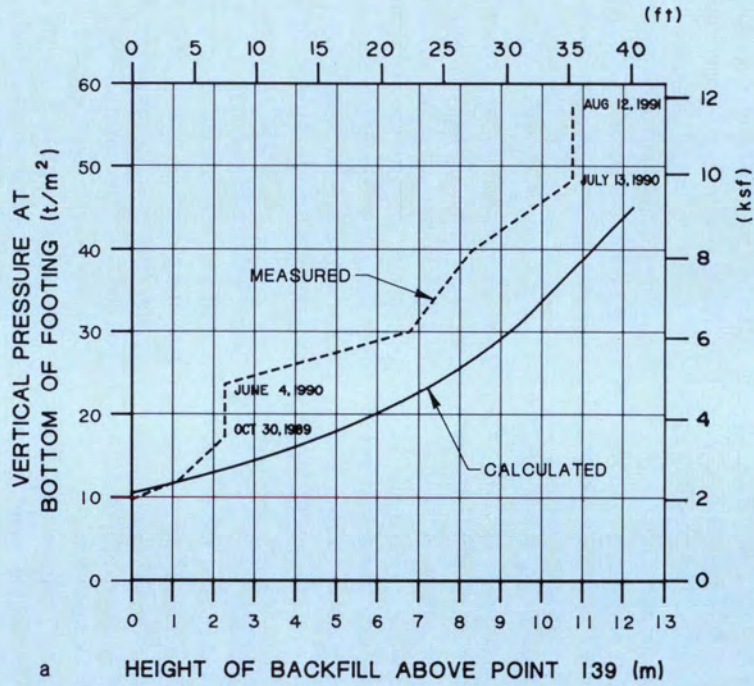


Fig. 18. Comparison of measured and calculated soil pressures.



structure. Future maintenance costs should also be reduced because the arch structure is not directly exposed to roadway salts.

Transportation planners and design engineers should consider using buried precast concrete arch structures for highway crossings where, in the past, conventional bridges have presented problems related to construction, maintenance or safety.

## ACKNOWLEDGMENT

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V. A. Dyaljee and M. Pariti were responsible for providing the design-

ers with geotechnical design parameters. Dr. Dyaljee also provided the authors with the results of displacement and pressure gauge measurements.

D. L. Anderson and P. M. Byrne of The University of British Columbia reviewed certain aspects of the arch analysis and design. Representatives of the Canadian National Railway provided their requirements relative to the waterproofing of the arch and site drainage.

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The authors also wish to acknowledge the thoughtful suggestions for improvement made by the PCI

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## Credits

Owner: Alberta Transportation and Utilities, Edmonton, Alberta, Canada

Structural Engineer: Lamb McManus and Associates Ltd., Edmonton, Alberta, Canada

Geotechnical Consultants:

- Alberta Transportation and Utilities, Edmonton, Alberta, Canada
- EBA Engineering Consultants Ltd., Edmonton, Alberta, Canada

General Contractor: Richardson Bros. (Olds) Ltd., Olds, Alberta, Canada

Precast Concrete Contractor: Con-Force Structures Limited, Edmonton, Alberta, Canada

Reinforced Earth Subcontractor: Reinforced Earth Company Ltd., Rexdale, Ontario, Canada

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## APPENDIX — SI CONVERSION FACTORS AND CONSTANTS

### Conversion Factors

- 1 millimeter (1 mm) = 0.03937 in.
- 1 meter (1 m) = 3.281 ft
- 1 kilometer (1 km) = 0.622 miles
- 1 kilogram (1 kg) = 2.205 lb
- 1 megapascal (1 MPa) = 145 psi
- 1 metric tonne (1 t) = 1000 kg = 2205 lb = 2.205 kips
- 1 Imperial ton = 2000 lb = 2.0 kips
- 1 kilonewton (1 kN) = 224.8 lb = 0.2248 kips

Atmospheric pressure = 10.33 t/m<sup>2</sup>  
= 2.117 ksf

### Truck Loads

The Canadian MS truck has the same wheel and axial arrangement as the AASHTO HS truck.<sup>8</sup> The total load on the first two axles of the truck in kilonewtons, kN, is given by the number following MS. To convert from kN to t, divide by the acceleration of gravity, which in the SI system is 9.81 m/s<sup>2</sup>.

### Cost

\$1.00 Canadian = \$0.78 U.S. at the time of publication.

Metric reinforcing bar areas.

Bar	Area	
	(mm <sup>2</sup> )	(in. <sup>2</sup> )
15 M	200	0.310
20 M	300	0.465
25 M	500	0.775