Three creek crossings of a new freeway in southwestern British Columbia (B.C.) have recently been completed using precast prestressed concrete to form buried arch culverts (Fig. 1). The concept of precast concrete culverts is not new, but these structures are novel in terms of size and construction details.

The location of the project is shown in Fig. 2. The new highway was constructed to provide a better route from the coast to central B.C. and the rest of Canada than provided by the previous Highway 1 through the Fraser Canyon.

It was constructed on an accelerated schedule to be open in the spring of 1986 for EXPO 86 in Vancouver. The particular section of highway on which these crossings are located is roughly in the middle of the Cascade Mountain Range as it extends into the Coast Range in B.C. The elevations of the crossings are between 750 to 900 m (2400 to 3000 ft) above sea level. The special physical characteristics of the topography, the logistics of working in this somewhat remote site and the construction schedule all influenced the design of the structure for the crossings.

A profile of the arch culverts that were constructed is shown in Fig. 3. At 20 m (65 ft) span, the opening width is approximately 2 m (6.5 ft) wider than what is claimed to be the world's largest ARMCO Superspan multiplate recently completed in Ontario. The opening

Note: This article is based on a paper presented at the International Conference on Concrete in Transportation, Vancouver, British Columbia, September 1986, cosponsored by the American Concrete Institute and the Canadian Portland Cement Association.
Describes three highway creek crossings recently completed in British Columbia using large span segmental precast post-tensioned concrete culverts. Includes an overview of project constraints, design and construction aspects, and cost factors, with comments on the circumstances where it may be beneficial for engineers to consider large buried structures as alternatives to short span bridges.

Thomson & Associates Ltd. (THOMSON) was under assignment to the B.C. Ministry of Transportation and Highways (MOTH) for the design of this 8 km (5 mile) section of highway in the Boston Bar Creek Valley.

Initially, most design considerations pointed to a conventional multiplate culvert to direct the creek under the highway. Hydrological assessment indi-
icated the design flood discharge of the creek could be handled by a 7 to 8 m (24 to 27 ft) diameter full circle pipe. Also, it was very desirable to avoid bridge decks at this mountainous location. The high potential for icing caused the MOTH to impose restrictions of no curves or spirals as well as minimum grades to be designed for bridge decks.

There was considerable difficulty in finding a satisfactory alignment in the narrow creek valley that met these criteria for bridges. In addition, the obvious advantages of lower maintenance strongly favored the selection of buried structures for the crossings. A further and very strong factor was also the fact that a multiplate culvert can usually be constructed at a cost considerably less than a bridge.

Environment factors played an important role in the planning stage and design process. Boston Bar Creek is a prime habitat for trout, and fisheries authorities considered even a 7 m (23 ft) diameter pipe to be a “dark hole” barrier to migrating fish when the length was as long as 65 m (213 ft). In addition, there are some active debris torrent paths and avalanches intersecting the creek immediately upstream of the crossing sites, and large stumps and logs were seen in the creek bed. Fisheries concerns and the ominous piles of debris in the creek tended to steer the selection of structures for the crossing to bridges.

The concept of remaining with a buried structure but extending its size to pass the expected debris and to allow sufficient light for the fish was agreed to by the MOTH. The 20 m (65 ft) span was selected to allow passage of logs at flood. With a circular arch shape giving an opening height of 6 m (20 ft), fisheries’ approval was received based on a limiting length of 65 m (213 ft).

The first concept for such a large culvert was an enlarged version of the corrugated multiplate, open bottomed cul-

![Map showing location of project.](image-url)
vert. However, there were several difficulties with this idea. First, there was no precedent of such a large multiplate ever being constructed.

After some experience with previous difficulties in erecting large multiplate arches, the MOTH, the geotechnical consultant and THOMSON felt that such a corrugated steel arch would be critically sensitive to the care taken in backfill, the quality of the backfill, and would require a thick cover over the crest for live loads. These large, flexible arches essentially achieve their strength from a thick soil arch. This would require extremely close supervision and possibly a great quantity of imported backfill.

The second objection was doubt as to the ability of a supplier to meet the delivery schedule for three large structures. Finally, to construct a large multiplate steel culvert, it would have been necessary to provide closely spaced temporary supports under the plates until a minimum layer of backfill was placed. This was impractical, since there was insufficient space in the valley to relocate the creek around the construction site.

A buried bridge type of structure, on a straight span covered with a layer of fill, as well as a reinforced steel plate arch, were also briefly investigated but were considered uneconomical.

An initial investigation of the use of cast-in-place concrete for the arch culverts was carried out. The points in favor of cast-in-place concrete were the constant cross section being suitable for a travelling form and the relative freedom of slab thickness. However, this freedom was not really a great advantage, since a high quality, dense concrete was desired, in any case, for durability. Cast-in-place concrete was eventually rejected due to the fact that most of the casting work would have had to take place in the worst winter weather conditions, and the sand available locally was felt to be too coarse for a high strength, dense concrete.

Precast concrete was finally chosen for its ability to guarantee excellent
quality durable concrete and the fact that production could take place in controlled plant conditions during the winter. There appeared to be a better probability of achieving the scheduled completion date with precast arches as opposed to cast-in-place concrete. Good dimensional control and close supervision were also advantages foreseen for precast concrete. A preliminary cost comparison showed precast concrete to be slightly more economical, but as the degree of confidence in the unit prices was not high for these fairly unusual structures, the recommendation for precast concrete was based on technical

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**Fig. 4.** Arch footings showing base connection of precast segments.

**Fig. 5.** Finite element mesh model of soil-culvert interaction.
Design Considerations

Due to the tight time constraint, it was necessary to proceed with the footings in the spring of 1984, before the detailed design of the arch slabs was completed. The width of a solid cast-in-place strip footing was designed for the vertical loading. The depth was dictated by making the underside of the base at a level below the expected scour depth. A diagram of a typical footing is shown in Fig. 4.

A large portion of the length of each footing was to be on solid rock but some areas required bearing on granular material. For these areas, a bearing pressure of 950 kPa (10 tons per sq ft) was used.

A rigid connection between the arch slabs and the footings was employed. This was accomplished by setting large 55 mm (#18) diameter Dywidag, Grade 400 (60 ksi) thread bars in voids cast into the footing, as shown in Fig. 4. Initially, it was anticipated that the base of the slabs would bear on a strip of elastomeric bearing material, creating essentially a pinned base connection. However, subsequent analysis showed that with the relatively thin slab section desired for precast economy, base rotations were quite high.

Joint rotation in this case is caused by moments built up as backfill is added and compacted from the footing to the crest. The rotation and a high axial load resulting from the arch effect made design of a suitable elastomeric bearing material impractical. A steel rocker arrangement was rejected as being costly and too difficult to protect against corrosion, so a fixed base arrangement was adopted.

Analysis of the arch slabs was done using a finite element computer simulation to model the interaction of soil and structure. The mesh of elements is shown in Fig. 5, with the heavy line to the left indicating one special case in-
investigated with a rock boundary near the structure. The particular program used had the capability to account for inelastic behavior so limit state design methods were used. Load factors were applied to soil density and live load that were generally consistent with Canadian bridge design codes. Assistance with the computer analysis as well as in the appropriate assignment of soil parameters was received from the Civil Engineering Department of the University of British Columbia.

A diagram of moments and axial forces resulting from the analysis is shown in Fig. 6. The somewhat unusual moment pattern seemed to develop as backfill layers were brought up in a staggered fashion. A 1 m (3.28 ft) difference in level was used as a pessimistic estimate of grade control. Once the backfill cleared the crest, the moment pattern stayed nearly constant and the axial force increased with successive layers of fill. This agreed fairly closely with the classic compression arch under uniform load theory.

The values for moments and axial forces are for the worst condition of the three crossing sites with 9 m (30 ft) of fill over the crest. The other two site conditions were for a shallow cover of 2 m (6.5 ft) over the crest which was simply an intermediate case in the analysis.

It was anticipated that during construction there would be a need to haul fill material over two of the sites. To accommodate this, a live load of a two-axle 93,000 kg (100 ton) off-road haul truck was checked during the computer simulation, when the fill cover was 2 m (6.5 ft). The moment curve on the bottom of the figure is the maximum influence of such a truck.

By comparison, it can be seen that the moments incurred by the heavy axle loads are much less significant than the effects of the rising backfill. In fact, at the crest the negative or hogging moment does not change sign under the live load so a crest joint with top steel only, as shown in Fig. 7, was employed. The transverse connection bars at the crest were also 55 m (#18) diameter Dywidag thread bars at the same spacing as the base/footing connection. The top bars were grouted into pipe sleeves and were set but not stressed.

The slabs themselves were designed as plain reinforced sections under axial load with a ductility ratio of not less than three.

As can be seen in Fig. 7, the crest flange-type joint formed a significant ridge beam when all of the arch pairs were grouted together. This ridge beam was post-tensioned with four 15 mm (0.6 in) diameter strands in each of eight ducts. This post-tensioning served a dual purpose. When cover was shallow,
the stressed crest beam acted to spread concentrated axle loads parallel to the axis of the culvert, thus assisting in the structure’s ability to handle concentrated loads.

After the depth of fill increased to the extent that concentrated loads are distributed by the fill cover, the longitudinal stressing served to secure the pairs of arch segments together against the spreading forces of soil friction. It was difficult to quantify the latter effect so the stressing was sized on the basis of the beam action under the concentrated live loads mentioned above.

All joints between precast arch slab segments, and between segments and the footing, were grouted except for the transverse joints running between the footing and edge of the ridge beam. A plastic foam sealing strip was used in this area. It was felt that there would be an advantage in keeping the whole culvert structure as torsionally soft as possible for the instance that the end of one strip footing went from a rock support to granular, and some differential twisting of the structure resulted.

**Construction Aspects**

As mentioned above, the strip footings were constructed in the fall of 1984, when the arch slab design was being finalized and precast bids arranged. The excavation, footing construction and riprap work was performed by the road grade contractor under the direction of the MOTH. Riprap was initially used to contain the stream with dykes between the footing locations, to allow dewatering the excavation. When the footings were cast, the riprap was moved to form scour protection. Fig. 8, below, shows the completed footings and riprap scour protection.

Casting of the arch slab segments was carried out at a precast plant in Langley, B.C. in the Fraser Valley, approximately 150 km (100 miles) from the site. The 148 precast segments were cast from January to March 1985. In order to meet the schedule required, three forms were used on a one-day cycle for each. The forms consisted of concrete arches themselves, with steel side forms and steel end block forms. The half arch segments could be cast “on the flat”

Fig. 8. Footing and scour protection for culvert.
since the base angle of each piece was approximately 30 deg to the horizontal. The low slump concrete mix used would rest at that angle. The forms are shown in Fig. 9.

Each segment was 12.2 m (40 ft) long on the chord of the arc, and 2.5 m (8.2 ft) wide. The thickness was generally 250 mm (10 in.) except at the base, where the thickness was flared to 350 mm (14 in.) over approximately one quarter of the arc for the high negative moment in this area. At 25 tonnes (27 tons), each segment would just fit onto a standard flat bed truck at legal load.

Specified concrete strength for the precast segments was 50 MPa (7250 psi) at 28 days, but the actual average strength achieved by the precast contractor, APS (Architectural Precast Structures Ltd.), was considerably higher than this. The generous strength was required for early handling capability.

Reinforcement was welded wire mesh in equal size at both surfaces. This was done to bracket the close spaced waves in the moment curves shown in Fig. 5, which were actually shifting slightly in position with alternating backfill layers. The welded mesh also gave the advantage of short development length for these moment waves and better retention than bars for pulling out of tension steel on the inside surface of the curve. Supplemental bars were used for the high moment area at the footing.

The erection scheme worked out by the precast contractor was quite ingenious and effective. A steel truss frame was fabricated to span between inside edges of the footings with a width of one segment. The truss, as can be seen in Fig. 10, was designed to support the top corners of a pair of arch segments and had positions for two hydraulic hand jacks on each side at the top. A 181 tonne (200 ton) capacity mobile crane was stationed to the side of one footing and swung the precast arch pieces from the truck to position on the footings and the truss, as shown in Fig. 11.

When properly aligned, the hydraulic jacks supporting the crest edge of the units were lowered, causing the vertical abutting surfaces of each pair to come into contact. The arch pair were then

Fig. 9. Plant casting of arch segments.
Fig. 10. Erection truss used for mounting arch segment.

Fig. 11. Partially erected arch culvert.
self supporting. Following placement of one pair of segments, the truss was advanced along the footing to the next position.

Erection started in April 1985 and was completed in June 1985. Initially, the erection crew could only achieve two or three segments per day. Once familiarity with the sequence was gained, they quickly advanced to an average of eight to ten segments per day (five pairs), with a maximum of eleven per day. The limitation on erection speed became the ability to deliver precast segments to the site on the busy construction road.

The precast segments were lowered over the base connection dowels as shown in Fig. 12. The large Dywidag bars were set in plastic concrete in 200 mm (8 in.) diameter voids immediately prior to placing the segment so that the bars could be shifted slightly to accommodate tolerances. Steel shims were used under the precast base and between the base and the thrust surface on the footings to control alignment. When a pair of arches were in place and self supporting, the top connection bars were installed.

After all precast arch pairs were erected, and prior to the grouting of joint surfaces, strand groups for the post-tensioned tendons in the ridge beam were drawn through interconnected ducts. Joint grout was excluded from the ducts by neoprene foam pads glued to the precast element at the plant in a similar manner as strips set to seal the bottom and sides of grouted volumes. After several days curing of the joint grout the strands were stressed and grouted.

While the precast contractor was completing the finishing work and stressing the culverts, the grading contractor returned to complete the headwalls. Following curing of the headwall concrete, the backfill operation was initiated by the grading contractor.

Specifications for backfill under the grading contract called for a 1.0 m (3.28 ft) minimum thickness of graded, select structural backfill around the culvert. This layer, shown in Figs. 13 and 14, was primarily for protection of the surface of the concrete rather than a structural soil arch.

The backfill was placed and compacted in 460 mm (18 in.) lifts evenly on both sides of the culvert. This is the same procedure as generally used with a large corrugated plate structure. The resultant stresses induced in the arch are probably somewhat less than those predicted by the 1.0 m (3.28 ft) difference in elevation side-to-side as anticipated in the computer analysis. The completed grade over one of the culverts is shown in Figs. 15 and 16.

Concluding Remarks

From the experience of this project, large span precast segmental arch culverts are feasible and economically competitive with bridges and other types of construction. Advantages of a buried structure such as virtual absence of maintenance, no deck icing problems
Fig. 13. Beginning of backfilling culvert.

Fig. 14. Later stage of backfilling culvert.
Fig. 15. Completed culvert with vehicular traffic.

Fig. 16. Another view of completed culvert with vehicular traffic.
and greater freedom of road alignment, compared to bridges, can be achieved even in fairly remote areas.

Final costs of the Coquihalla culverts versus the estimated costs for bridges indicate that in shallower fill conditions, such as creek to grade separations of 6 to 8 m (19.7 to 26.2 ft), the cost of a segmental precast culvert is approximately the same as for a bridge. However, for higher fill situations such as creek to grade separations of greater than 8 m (26.2 ft), a segmental precast culvert can offer significant savings over a bridge. The highest level culvert of the three on the Coquihalla project, with streambed to grade separation of 15 m (49.2 ft), was completed for approximately one-half the estimated bridge cost at the project site.

The fixed contract (1984 prices) for the supply and erection of the precast concrete arches was $966,160 (Canadian) for all three culverts. This bid price converts to a unit cost of $5222 per lineal meter. The approximate cost of the footings for all three culverts was $750,000. The structural backfill layer amounted to $90,000 with an additional $50,000 for general backfill.

The precast segmental culverts are inherently tolerant of high live loads in combination with shallow cover depth. They would be well suited to railway crossings and off-highway haul road crossings. Compared to corrugated steel plate culverts, the precast concrete culverts are much more tolerant of backfill quality and compaction, and are expected to have a considerably longer service life due to the dense high strength concrete possible with precast segments.

This project won a Special Jury Award in the 1986 PCI Professional Design Awards Program.

Credits

Project Name:
(1) Boston Bar Creek Culvert, Station 363 (Boulder site).
(2) Boston Bar Creek Culvert, Station 389 (Miranda site).
(3) Boston Bar Creek Culvert, Station 426 (Box Canyon site).

Owner:
Province of British Columbia, Ministry of Transportation and Highways, Victoria, British Columbia.
T. R. Johnson, PE, Deputy Minister.


Structural Design: R. H. Hebden, PE.

Overall Project Manager: V. D. Cripps, PE.

Construction Supervisor: J. G. McCall, PE.


Specialist Computer Assistance: Dr. D. Anderson, PE; Dr. N. Nathan, PE; Dr. P. Byrne, PE. All Professors of Civil Engineering, University of British Columbia, Vancouver, British Columbia.