

Structure Design and Construction on the Vail Pass Project

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Bridge construction on the 22.5-km (14-mile) long segment of I-70 through Vail Pass in Colorado presented unique problems to both designers and constructors because of the extraordinary care taken for environmental and aesthetic concerns. Span lengths and ratios, span-to-depth ratios, and structural member shapes were reviewed and established based largely on environmental and aesthetic criteria. Construction methods were established to minimize damage to the fragile terrain in the high-altitude environment. They included one of the first and most extensive uses of segmented post-tensioned concrete box-girder bridges in North America. Aesthetic, environmental, and engineering considerations also led to the design of a unique precast concrete retaining-wall system. The precast wall was alternated with a modification of a reinforced-earth-wall system of almost identical appearance.

Bridge and retaining walls used on the 22.5-km (14-mile) long segment of I-70 through Vail Pass were largely determined by the environmental constraints inherent in the design of the project. Environmental and aesthetic concerns played a significant part in the selection of the types and design of structures and the construction procedures used to erect them.

Early in the design process, the Division of Highways, Colorado Department of Highways, emphasized that the Vail Pass structures should be constructed so as to minimize damage to the natural terrain. Long cut-and-fill slopes were to be minimized as much as possible. In some cases, bridges and retaining walls were used to eliminate cut-and-fill slopes. In most cases, the total overall length of the structures was set by an effort to provide a smooth-appearing transition from roadway to bridge.

PRELIMINARY DESIGN

An April 1973 report (1) summarized the preliminary designs of the various types of bridges considered and made recommendations for the type of bridges to be used. The preliminary design studies were influenced largely by operational and environmental considerations: to provide safe driving conditions and minimize damage to the terrain. The objective of this conceptual study was to arrive at a recommended bridge type that would

1. Avoid during construction any terrain damage that would be difficult to remedy at such a high altitude,
2. Use a method of construction that would make maximum use of precast or prefabricated units,
3. Provide structures that would harmonize aesthetically with the forested mountain terrain and provide continuity of design and appearance over the entire length of the project,
4. Incorporate roadway approaches in such a manner as to avoid encroachment on streams, and
5. Provide a generous open space beneath bridges to encourage the growth of natural vegetation and accommodate the passage of hikers, skiers, and wildlife.

I-70 at Vail Pass is located at an altitude of 2590-3200 m (8500-10 500 ft) above mean sea level; winter driving conditions were thus of paramount concern. All

bridges were to be two-lane, one-directional structures with a total width of 13 m (42 ft). Maximum grades on the project are 7 percent, and maximum superelevation on the bridges is 8 percent. Several structures are on a curvilinear alignment, and some combine tangent, spiral, and circular curves all in one structure. One factor that affected the type of structure used was the relatively short construction season (five months a year) at Vail Pass elevations higher than 2740 m (9000 ft).

Seven types of bridges were considered and studied in detail in the preliminary design phase:

1. Short-span precast concrete girders,
2. Tubular steel space frames,
3. Welded plate girders,
4. Precast concrete earth-filled arches,
5. Long-span open-spandrel concrete arches,
6. Long-span welded plate box girders with concrete deck, and
7. Long-span precast, post-tensioned segmental concrete box girders.

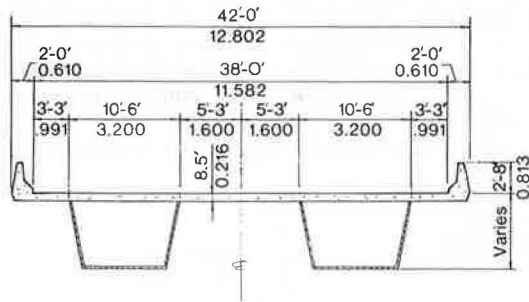
The first five were eliminated primarily on the basis of architectural considerations. Of the two remaining alternatives, the precast segmental concrete type of system seemed to satisfy all design criteria. Precast units can be manufactured at any time under carefully controlled conditions. This technique eliminates the problems associated with the short construction season, insofar as the superstructures are concerned. Cantilever construction of precast units requires a minimum of construction equipment on the ground in the immediate vicinity of the bridge. The void in the interior of the box girder also provides some insulation for the roadway surface.

Many of the advantages apparent in the segmental concrete system also apply to the steel box-girder system. The steel boxes can be erected in long segments, and the concrete deck can then be cast in place with minimum utilization of equipment on the ground underneath the bridge.

The preliminary studies led to the decision that relatively long-span prestressed concrete box-girder bridges should be designed for the project. It was also decided that the bridges should be erected by cantilever construction methods to minimize damage to the terrain in the immediate vicinity. Pier columns composed of precast elements supported on cast-in-place footings were recommended. It was also decided that all concrete exposed to view should be colored by adding iron oxide to the mix so that the structures would blend with the maroon sandstone color prevalent in the area.

The studies further indicated that the difference in cost between precast segmental bridges and structural steel box-girder bridges with a concrete deck would be very small. Consequently, steel box girders with cast-in-place concrete decks were recommended as an alternative to the primary recommendation of segmental concrete girders. This led to a decision to develop complete construction plans for and solicit bids on both structural types. Complete designs, plans, and

Figure 1. Typical section of steel box-girder bridge.



specifications were prepared for both alternatives for 17 bridges. Spans varied in number from two to five and in length from 9 to 79 m (30 to 260 ft).

STEEL BOX-GIRDER BRIDGES

Steel box-girder bridges were designed with a common cross section that included two welded steel box girders (see Figure 1). In all cases, the deck span between the top flanges of the box girders was the same. A length-to-depth ratio of approximately 25 was used. Each structure was designed with a constant depth and increased flange plate thicknesses in the areas of higher moment. Girders with variable depth were considered, but the architects preferred a constant depth and a high span-to-depth ratio to give the bridges a slim-line appearance.

American Association of State Highway and Transportation Officials (AASHTO) A-588 high-strength, self-weathering steel was used for the girders. A-36 steel was used for the diaphragms and stiffeners, which were left unpainted to minimize the time and cost of fabrication. The box girders were designed by using ultimate-strength procedures according to the 1976 AASHTO code and all applicable interim specifications. All stiffener plates were welded to the inside of the box girders to maintain a smooth exterior appearance. Diaphragms were used between the box-girder sections to provide the torsional stiffness required both during and after construction. The concrete deck was constructed of 21-MPa (3000-lbf/in²) normal-weight concrete and was made composite with the girders by shear studs welded to the top flange plates. All of the steel box girders were fabricated in Denver and transported to the jobsite by truck. The maximum length between field splices was approximately 33.5 m (110 ft). Field splices were made with high-strength bolted connections.

Abutments for the steel box-girder bridges were constructed of cast-in-place concrete with an ultimate strength of 21 MPa. Several of the abutments were on spread footings, and others were placed on steel point-bearing piles. Piers were designed as precast concrete segmental units to be posttensioned to a cast-in-place footing in the field. The contractors were given the option of casting the piers in place and reinforcing them with milled steel bars. In all cases in which steel superstructures were utilized, the cast-in-place procedure was used. The pier cross section was diamond shaped, in accordance with the preference of the architectural team. All bearing devices were of the cylindrical pot type and made of a confined elastomeric material. These devices were equipped with a stainless-steel sliding surface resting against a teflon-coated steel plate at substructure units where longitudinal movement was accommodated.

Concrete barrier curbs of the "Jersey" type were

cast in place on each edge of the deck. After all other construction items were completed, a 5-cm (2-in) asphalt wearing surface was placed on the concrete deck.

The steel box-girder bridges, with their self-weathering steel and colored concrete for deck, curbs, and substructure, resulted in a pleasing appearance that blends nicely with the soil and rock in the area.

PRECAST CONCRETE SEGMENTAL BOX-GIRDER BRIDGES

In 1972, when design of the Vail Pass bridges was started, the precast concrete segmental technique was quite new to American designers and constructors. The only such bridge built in this country up to that time was the Corpus Christi Bridge built by the Texas Highway Department. The cantilever erection technique and the precasting of bridge segments offered certain advantages that were particularly applicable to the Vail Pass project.

To understand the factors that influence the design of a segmental bridge built in cantilever, it is first necessary to consider the erection technique. Figures 2-11 show the erection procedure used for the precast segmental bridges. The abutments were designed to use cast-in-place concrete with an ultimate strength of 21 MPa (3000 lbf/in²). Piers were designed as precast segmental units with a diamond-shaped cross section. Pier footings of cast-in-place concrete were designed with ducts to allow the stringing of post-tensioning tendons for the pier shaft. Figures 2 and 3 show the erection of a typical pier.

The shape of the superstructure cross section is shown in Figure 4. The webs and bottom slab of the segmental box were increased in thickness in the areas over the piers to provide for the high shears and moments in this region. A concrete diaphragm was cast into the superstructure segment immediately above the pier to provide for the heavy shears from the reactions. The 1.2-m (4-ft) long segment that contained the diaphragm weighed approximately 50 Mg (55 tons). The more typical segments (shown in Figure 4) were 2.2 m (7.3 ft) in length and weighed approximately 36 Mg (40 tons) each.

The cantilever erection procedure is shown in Figures 5-11. The diaphragm segment is first placed on the pier cap and fastened down with Dywidag bars. Cantilever erection then begins by adding segments alternately to each end of the cantilever and coating the faying surfaces with epoxy just before temporary post-tensioning, which is applied until the two matching segments are tensioned to approximately 206 kPa (30 lbf/in²) across the adjoining faces. After each pair of segments has been attached to the cantilever, final post-tensioning tendons are stressed. This procedure is repeated until two cantilevers are erected, as shown in Figure 8.

A portion of the end spans next to the abutments is then placed on falsework to complete these shorter spans. The ratio of end-span length to interior-span length is about 0.6; this resulted in the final positive moments in the end span being approximately equal to the positive moments in the interior spans.

After the end span is completed and continuity prestressing is applied (as shown in Figure 10), the erection procedure is then repeated on the other piers in the bridge. Finally, the other end-span section is erected and the bridge is made continuous by the final post-tensioning operation (shown in Figure 11). Before final prestressing, however, the 20- to 25-cm (8- to 10-in) gap between abutting cantilever ends must be closed. This is accomplished by forming across the

gap and pouring concrete in place.

The precast concrete used had an ultimate strength at 28 days of 38 MPa (5500 lbf/in²). Prestressing steel was 18-MPa (2700-lbf/in²), 1.3-cm (0.5-in) diameter strands, 12 strands making up one tendon. Epoxy was applied to all match-cast joints to make the structure watertight and to transfer the shear forces from one segment to the next.

Figure 2. Placing of second pier segment.

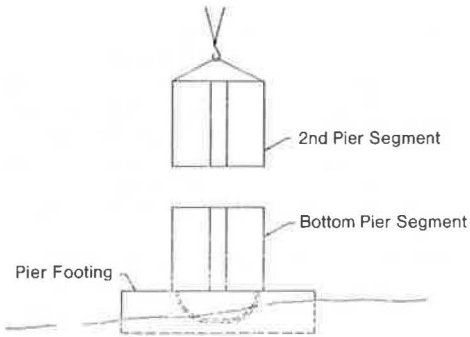


Figure 3. Completed pier with cast-in-place cap.

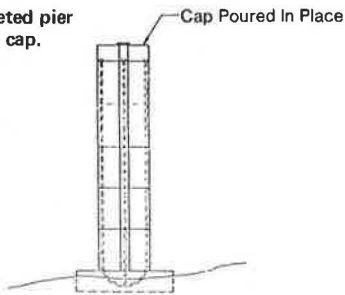


Figure 4. Section near midspan.

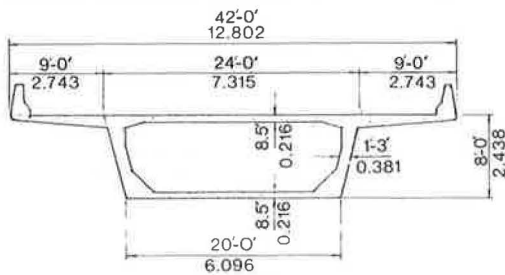
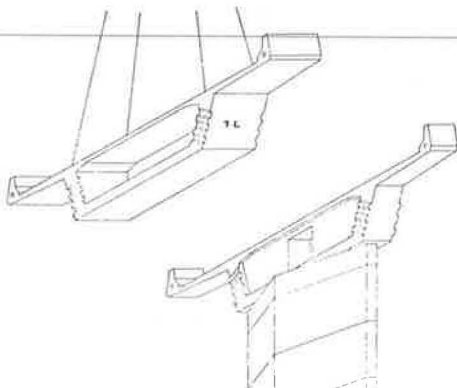


Figure 5. Lifting first (left) cantilever segment.



The first precast segmental bridges were cast with the concrete rails, monolithically. Because of vertical alignment problems, however, the rails were omitted on subsequent precast sections and later cast in place after all post-tensioning was done in the completed structure. Two pot-bearing devices were used at each sub-structure support.

On the precast segmental bridges, the contractor

Figure 6. First left segment in place with temporary post-tensioning.

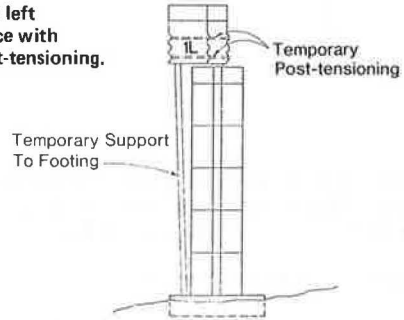


Figure 7. First right segment in place with temporary and permanent post-tensioning.

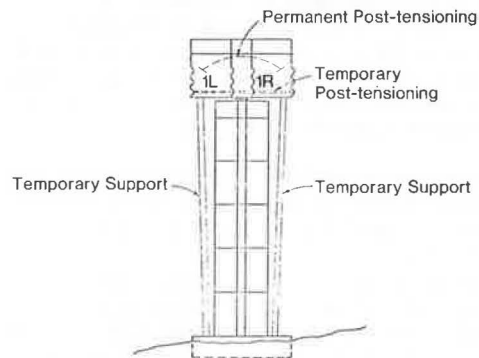


Figure 8. Completed cantilever.

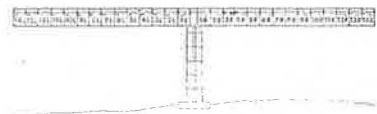


Figure 9. Segments near abutment 1 end on falsework before closure pour.

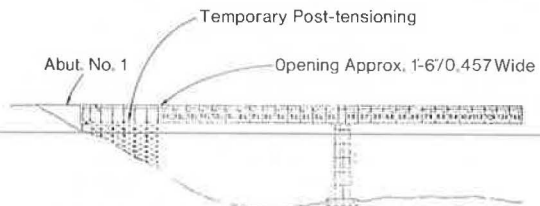


Figure 10. End span 1 complete with continuity prestressing (temporary post-tensioning removed).

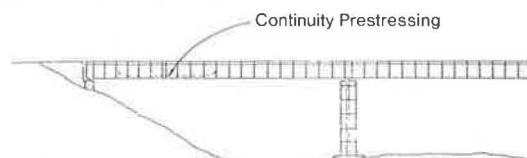
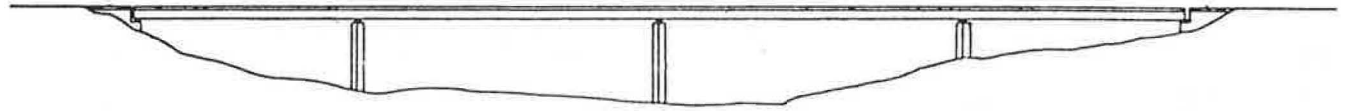


Figure 11. Completed bridge with all permanent post-tensioning in place.



used a 270-Mg (300-ton) track-mounted crane to lift the segments into position. The final construction specifications allowed a square area measuring 21 m (70 ft) on a side at the base of each pier on which the contractor could set his crane. From this location, the crane could reach to the end of each 32-m (104-ft) cantilever to set the last superstructure segment. This procedure required that the square area as well as a narrow access road be cleared under the bridges for the construction equipment. The original design concept called for overhead erection with either a high line or a gantry so that there would be no disturbance under the bridges. However, after each bridge site was reviewed and the cost associated with such an overhead erection scheme was considered, it was determined that the selected procedure was an acceptable compromise. With the exception noted above, the existing trees and larger foliage were left intact in the area of the bridge. Haul roads and platform areas for cranes were replanted after the completion of the construction to conform as closely as possible to their original condition.

For all segmental bridges, the contractors were given the option of either precasting the segments or casting them in place. In one project that contained four bridges, the contractor elected to cast the segments in place. Steel form travelers were mounted on the piers to hold the formwork required for casting the segments. After each segment was cast and post-tensioned, the form traveler was moved ahead and set up for the casting of the next segment. This procedure was repeated until the cantilevers reached midspan, which is much the same technique as that used for the precast segmental structures. The cast-in-place procedure has the advantage that all erection is done from overhead so that no equipment other than a light crane is needed on the ground in the vicinity of the bridge.

The same 38-MPa (5500-lbf/in²) concrete was used in the cast-in-place and the precast bridges. The post-tensioning system was made up of strength Dywidag bars rather than the strand used in the other bridges. All substructure units were also cast in place. Steel pot bearings were again used at the top of substructure units to support the box girders. A final touch on all concrete bridges was the application of a waterproof membrane under a 5-cm (2-in) asphalt overlay on the driving surface.

SPECIAL BRIDGES

At two locations, specific requirements dictated that bridges be designed without the preparation of alternative plans. One of these locations was a crossing over a canyon that had steep walls. In this case, a structure with a main span of approximately 55 m (180 ft) and flanking spans of about 9 m (30 ft) was selected. This resulted in an uplift at the abutment ends. Steel box girders were selected for this crossing since any support settlement might induce bending moments of a magnitude critical for the precast concrete structure. Because of their flexibility, the steel structures would not be overstressed by the redistribution of moments that could occur if there were minor support settlement. The cross section of the bridge superstructure for these steel bridges is the same as that of other steel

bridges in the Vail Pass project. Tiedowns at the end were constructed with prestressed rock anchors that were fastened to the abutment to hold it down.

The other location that required a special type of bridge was in an area where an underpass was required to accommodate the natural migration patterns of wildlife. The structures here, which are approximately 24 m (80 ft) in length, were made from cast-in-place concrete box girders that conformed to the shape of the precast segmental box girders used at other locations. The relatively short spans and the fact that these bridges were simply supported single spans made the precast segmental concept uneconomical.

RESULTS OF ALTERNATIVE BIDS

Of the 21 bridges designed and detailed for the project, 17 were completely designed and detailed in two alternative materials: steel and concrete. Contractors were then given both sets of plans and asked to bid on their choice. Table 1 gives data for the five construction contracts that involved the 17 bridges for which alternative designs were presented (approximately one year elapsed between the letting of the first and last contracts). Although there was considerable difference in the bid prices for steel versus concrete on individual projects, the total difference for all five projects, out of a grand total of over \$17 000 000, was less than \$80 000—a difference of less than 0.5 percent.

It is difficult to determine the overall savings in construction costs that accrued from the alternative bidding. The additional cost to produce a second set of drawings for a bridge designed for an alternative material was approximately 2.5 percent of the construction cost. In the opinion of the designers, the competitive bidding on the two alternative designs resulted in a savings of 7-10 percent of the construction cost.

RETAINING WALLS

One of the main objectives of the design effort was to avoid as far as possible damaging the natural terrain. Conventional cut-and-fill construction in steep, mountainous terrain always results in extensive excavation of back slopes that are practically impossible to revegetate. On the embankment side, the fill slope may chase the mountainside all the way to the bottom before "catching", covering acres of mountainside vegetation. Such was the case at Vail Pass. On major embankments, a fill slope of 1.5:1 is common, but at Vail Pass the natural slope was frequently this steep or steeper. Consequently, such embankments would have covered the entire mountainside from the roadway shoulder to the creek at the bottom. One of the recommendations made in the early design concept study (2) was that fill slopes be limited to a maximum of 3:1 to facilitate revegetation; later, 2:1 was accepted as the maximum. If fill slopes would not catch within a reasonable distance, they were eliminated by constructing a hillside structure that could be either a bridge or a retaining wall (see Figure 12). In a number of instances, an economic analysis indicated that a retaining wall would be preferable.

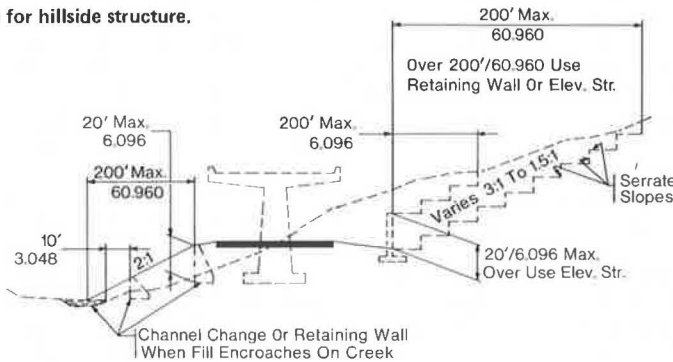
Table 1. Results of alternative bids on Vail Pass bridges.

Project ^a	Bridge	Number of Spans	Length (m)	Low Steel Bid (\$)	Cost per Square Meter of Steel (\$)	Low Concrete Bid (\$)	Cost per Square Meter of Concrete (\$)
1	F-11-AX	4	222	5 992 155	517.98	5 527 318	447.83
	F-11-AW	5	268				
	F-11-AV	4	210				
	F-11-AU	4	204				
Total			904				
2	F-12-AK	3	67	3 777 549	544.56	4 111 170	592.57
	F-12-AM	2	73				
	F-12-AN	3	107				
	F-12-AO	3	112				
	F-12-AP	3	183				
Total			542				
3	F-11-AP	3	94	994 347	479.00	1 053 364	507.43
	F-11-AO	2	68				
Total			162				
4	F-11-AN	4	225	4 257 771	445.75	4 108 057	430.14
	F-11-AM	4	227				
	F-11-AL	4	157				
	F-11-AK	3	137				
Total			746				
5	F-12-AT	4	221	2 298 409	405.70	2 598 938	458.78
	F-12-AS	4	221				
Total			442				
Total			2796	17 320 231	483.85	17 398 847	486.00

Note: 1 mm = 3.3 ft.

^aProjects are listed in the order they are advertised for bids.

Figure 12. Criteria for hillside structure.



The criteria for retaining walls established in the design concept studies can be summarized as follows (2):

The design goal for all retaining walls should be a structure that is visually compatible with the existing terrain. Wall faces should be textured or patterned to avoid the smooth, flat surfaces foreign to the natural terrain. Again, the color of materials used should blend with the natural environment. All retaining wall schemes should consider the effective use of vegetation to partially conceal wall surfaces and break up the larger expanses.

Wall heights should be scaled to the terrain from the roadway. The 6 m (20 ft) established as the maximum height to determine backslope distances should not imply that a 6-m (20-ft) high wall will be located immediately adjacent to the roadway. In such a case three walls, 1.8 to 2.1-m (6 to 7-ft) high and terraced to provide ledges for native shrubs and small trees, would be preferred over the single wall.

In short, the wall should be aesthetically compatible with its surroundings.

Conventional reinforced concrete cantilever and counterfort walls were ruled out for heights greater than 6 m (20 ft) on the basis of incompatible appearance and unduly high construction costs. The patented Reinforced Earth Company system with concrete facing panels was considered, since this system could be erected in a setback pattern so as to form terraces

at intervals to support vegetation. The architectural consultants believed, however, that some other system should be devised to give a more interesting appearance to the exposed surfaces.

An early drawing from the design concept report (see Figure 13) shows an idea for a precast unit conceived during the design concept studies. In later phases, however, stability calculations indicated that the required dimensions of the unit would make the system uneconomical. Another system was then devised that consisted of two simple precast concrete elements that could be easily erected in a manner not completely unlike that used to construct the old miner's crib. An L-shaped precast concrete member called a tieback is used; this has a long horizontal leg that extends into the fill and a vertical leg that extends upward to provide a reaction for the earth-retaining panel that spans the 3-m (10-ft) space between tiebacks. For aesthetic and structural reasons, the earth-retaining facing panels were designed as parabolic curved units. The architectural consultants believed that the idea had aesthetic merit; from an engineering standpoint, it would have the added virtue of being very simple to cast and erect. But it soon became apparent that the soil mechanics involved were somewhat complex and that a

Figure 13. Typical precast concrete retaining-wall unit.

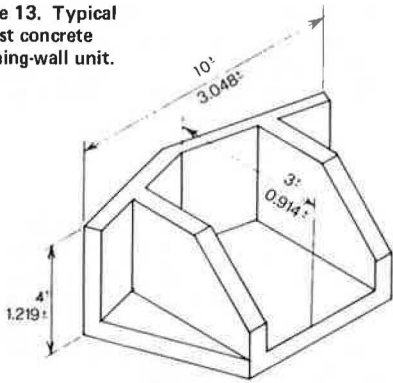
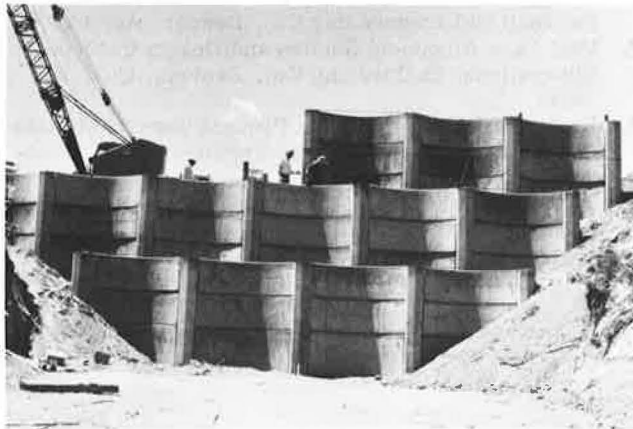


Figure 14. Sedalia test wall under construction.



considerable amount of development work would be necessary.

The analysis departed somewhat from the conventional, so it was decided to confirm the calculations by a full-scale test to confirm design assumptions and the analysis. A four-tier wall 10 m (32 ft) high that consisted of 31 tiebacks and 81 facing panels was designed. This wall was erected at a test site near Sedalia, Colorado (see Figure 14). Pressure cells were installed throughout the backfill to measure pressures acting at various points. Deflection measurements were made on each of the tiebacks to determine settlement and horizontal displacement. The test data verified most of the assumptions and generally validated the analysis. On some points, however, the data were inconclusive, and a need for additional investigation was indicated. The results of these tests are given elsewhere (3).

Engineers from the state and federal approving agencies reviewed the report. There was a lack of consensus as to whether the method of analysis was acceptable. Since construction schedules did not permit the time to resolve these differences, it was decided to use a different analysis in designing the structures to be constructed at Vail Pass, and this resulted in longer (and more costly) tiebacks for the lower tiers.

In an effort to resolve the differences of opinion that had arisen over the design analysis, it was decided that a major section of a wall should be instrumented during construction and the data collected to further evaluate the analysis procedure. A six-tier, 15-m (50-ft) high retaining wall was designed and constructed in the vicinity of station 498 (see Figure 6 in the paper by Milhollin elsewhere in this Record). This wall was instrumented

to collect values that would be used in determining the safety factor against sliding and pressures acting on the face of the wall (the principal values in contention). The results of these tests, which are reported elsewhere (4), indicated that the pressures acting on the face of the wall were very low in comparison with those calculated by the Rankine formula. This was also characteristic of the information collected on the Sedalia test wall. The low pressures on the face appeared to result from a combination of the reinforcing effects of the tiebacks in the tier above, the setbacks, and soil arching within the embankment. Measured pressures followed the Rankine pressure distribution for only the first tier. In the upper tier of the wall, at a 15-m (50-ft) embankment height, the average face pressures were only about 30 percent of the Rankine calculated pressure.

In summary, it was the designer's contention that tiebacks of equal length in all tiers would result in a wall that could appropriately be analyzed as a battered wall. The opposing viewpoint was that a vertical wall analysis should be used and that tiebacks of varying lengths should be embedded in the fill so that their ends lay in a vertical plane. It is the designer's opinion that the results of both tests substantiate the design analysis based on the battered-wall concept and that this approach should be used on future designs to take advantage of the savings inherent in the shorter tiebacks. To further document design procedures, an eminent geotechnical engineer was retained to make an independent review. It was also his conclusion that the battered-wall concept was valid.

The Reinforced Earth Company suggested that competition might be sharpened if the plans included the reinforced-earth system as an alternative. They offered to redesign the facing panels to be practically identical in appearance to the precast units. On the plans for the projects subsequent to the one on which the above-mentioned wall at station 498 was constructed, the reinforced-earth system was included as an alternative. As a result, the entire Vail Pass project as completed includes a 6172-m² (66 432-ft²) exposed face area of the precast concrete tieback type of wall, and 14 406 m² (155 066 ft²) of the reinforced-earth type of wall with curved face panels. In addition, conventional flat-faced reinforced-earth walls were constructed in those areas where a high vertical wall was required. An attempt was made to evaluate the comparative costs of the two wall systems, but the results were inconclusive. It is our opinion that, if the precast concrete tieback wall is designed on the battered-wall concept and as contractors gain experience in its erection, it will prove to be quite economical.

To protect the Colorado Department of Highways from possible future litigation, the precast wall was patented (U.S. Patent 4,050,254). The department was granted royalty-free license to use the system. It can extend this license to other government agencies for use of the wall on highway projects constructed with federal-aid funds.

SUMMARY AND CONCLUSIONS

Bridge construction on the Vail Pass portion of I-70 presented some unique problems to both designers and constructors because of the extraordinary care taken for environmental and aesthetic concerns. Span lengths, span ratios, span-to-depth ratios, and structural member shapes were reviewed and established largely on the basis of environmental and aesthetic criteria. Construction methods were established to minimize damage to the fragile terrain in the high altitude of the

Vail Pass area. Careful selection and treatment of materials—e.g., the use of self-weathering steel box girders and the use of color in the concrete—were undertaken to blend the bridges into the natural color scheme of their surroundings.

Without these concerns for environmental protection, it is likely that the cantilever construction of segmental concrete box girders would not have been attempted. The experience gained in this popular European construction technique on the Vail Pass project, and on other projects more recently initiated in this country, will help to promote its more widespread use in future U.S. bridge projects. The alternative bidding technique used on the Vail Pass bridges indicates that, in the matter of cost, the segmental concrete structures can compete quite favorably with the more conventional types of structures.

Where retaining walls were applicable, environmental and aesthetic concerns also resulted in extraordinary measures to minimize damage to the natural terrain. A unique precast concrete retaining-wall unit was developed that was aesthetically acceptable and easy to cast and erect. As an alternative, panels of the patented reinforced-earth type of wall were redesigned to be practically identical to the precast units. An attempt was made to evaluate the comparative costs of the two wall systems, but the results were inconclusive. The cost of the precast concrete tieback walls can be significantly reduced if they are designed on the battered-wall concept.

The various structures at Vail Pass as they appear today provide evidence that major highway structures can indeed be constructed without serious damage to

the environment. The Vail Pass bridges and retaining walls will stand for many years as testimony to the sincere efforts of various government agencies and designers involved in the project to preserve the aesthetic appeal of the area.

ACKNOWLEDGMENT

We would like to acknowledge the following firms and individuals who were involved in the various aspects of the Vail Pass project discussed in this paper: Frank Lloyd Wright Foundation, H. J. Meehen Engineering Company, James R. Libby and Associates, F. Dwayne Nielson of International Engineering Company, and Ralph B. Peck, geotechnical consultant.

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**A.B. Milhollin was with the Denver office of International Engineering Company when this research was performed.*

Cast-in-Place Segmental Bridges in the Vail Pass Project

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The design and construction of four cast-in-place segmental bridges that carry the Vail Pass section of I-70 over Miller and Black Gore Creeks are discussed. The free-cantilever method of construction is detailed. Measures dictated by special environmental and time restrictions on construction are also examined.

This paper describes the design and construction of four bridges that carry the Vail Pass segment of I-70, located about 11 km (7 miles) southeast of Vail. Two of these structures span Miller Creek and the other two Black Gore Creek.

The configurations and cross sections of the bridges are shown in Figures 1 and 2. The spans are relatively short, but the strict environmental constraints specified in the contract documents limited possible construction techniques.

The owner prepared two completely different designs for bidding purposes—one with a steel superstructure and the other with a prestressed concrete superstructure. Although the concrete design was based on precast segmental construction, the cast-in-place method was allowed if the contractor preferred. To blend the

structures with their surroundings, a light pink concrete color was specified to match the large natural rock outcroppings in the area.

Final construction was based on the cast-in-place segmental method, and the construction sequence was slightly different from that suggested in the original design. The piers, which varied in height from 6 to 21 m (20 to 70 ft), were cast-in-place hollow sections prestressed vertically. The substructure was constructed according to the original contract plans.

Construction of the superstructure of the Miller Creek bridges started in May 1977 and was completed in the first week of September. In August, construction of segments on falsework was started on the Black Gore Creek bridges; the cantilever segments of these bridges were started after the form travelers had been removed from the Miller Creek bridges. Construction of the Black Gore Creek bridges was completed in the second week of November 1977.