VALUE ENGINEERED BRIDGE DESIGNS IN BRITISH COLUMBIA

Don KENNEDY, Senior Bridge Engineer
David I. HARVEY, Manager, Bridge Engineering

Associated Engineering Ltd.
Burnaby, British Columbia, Canada.
Canada V5G 4M5

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Abstract

Recent initiatives on British Columbia Ministry of Transportation and Highways bridge projects have demonstrated that better value can be obtained through:

· Use of Value Engineering clauses in construction contracts, allowing contractors to offer cost-saving alternative designs tailored to specific project requirements.
· Design innovation.

The following examples from the Vancouver Island Highway Project are described in this paper:

· The Duke Point Underpass, a grade-separated interchange for the spur road to the new ferry terminal near Nanaimo, B.C. The bridge is an 80 m long by 17 m wide three-span curved-in-plan bridge crossing the existing Trans-Canada Highway. Originally designed and tendered as a cast-in-place concrete twin-box girder bridge, the contractor proposed a Value-Engineered alternative based on using three trapezoidal-section precast segmental box-girders. The segments were to be field spliced, post-tensioned, and topped with a composite concrete deck. The Ministry accepted the proposal, which was priced at 20% less than the original contract amount. The Duke Point Underpass is the first complete bridge to be Value-Engineered in the Province using the Ministry’s contract provisions. At $860 per m² it represents one of the lowest-cost bridges built in B.C. in recent years.

· Four bridges carrying the four-lane Inland Island Highway over watercourses were tendered with steel plate girder designs. The Bloedel Creek Bridge is a single 45 m span. The Wilfred Creek, Cowie Creek and Trent River Bridges are three-span designs with overall lengths of 65 m, 93 m, and 106 m. The two successful contractors commissioned Value Engineering design proposals using precast concrete girders. Innovative designs involving precast semi-continuous construction and hybrid prestressing systems were prepared. The designs were developed for maximum economy in concert with transportation and erection feasibility.

The opportunity created by the Value Engineering provisions required highly efficient, optimized precast concrete designs in order to compete with the structural steel solutions. The design concepts used were the result of the integration of design with construction, and the incentives created by the VE clauses to offer better value to the owner. The net total Value Engineering savings for the five bridges was $1.15 million. This savings was shared equally between the Ministry and the contractors.
INTRODUCTION

A major component of the B.C. Ministry of Transportation and Highways (MoTH) new construction program in recent years has been the Vancouver Island Highway. The 300 km-long $1.3 billion road has been designed to four-lane arterial and freeway standards. Some 150 bridges were needed for grade-separation and watercourse crossings.

In recent years, the bridge construction contracts have included Value Engineering clauses. These clauses encourage the contractor to propose alternative designs. Where these offer the Ministry a cost, schedule, or risk advantage, with no reduction in quality, they will be considered. The financial incentive offered to the contractor is 50% of the net savings. In addition, there can be an incentive to the contractor to use a preferred material or construction method.

Experience in applying the clauses suggests that savings can accrue through material competition, higher design efficiency, elimination of unnecessary features, and improved constructibility. The bridge projects described illustrate how existing structural systems can be developed and original designs revised to save costs. This was done without sacrificing durability, while maintaining the quality and elegance prevalent in successful bridges.

VALUE ENGINEERING

Recent Ministry bridge construction contracts have included Value Engineering clauses, along with an encouragement to submit any number of Value Engineering Proposals (VEPs) during the contract term. The Ministry will not accept VEPs that could negatively impact future maintenance costs, existing approvals, environmental or other agreements, or bridge aesthetics. The contractor cannot assume acceptance of any VEP during or after the tender period, or submit a claim based on a rejected VEP.

To be considered, a VEP must offer a tangible benefit to the owner. The substitution must be functionally equal or superior to the tendered design. It may reduce risk, initial or life-cycle costs, or offer a schedule advantage. VEPs may involve changes in materials, products, work methods, or construction sequence. In preparing VEPs, the original design criteria must be used. A change in contract conditions does not qualify as a VEP. In the cases described in this paper, all involved substantial or complete redesign of the tendered bridges.

VEP’s are evaluated initially mainly on the basis of net cost saving. This is calculated by adding to the negotiated cost of the VEP:

- The contractor’s cost of developing the VEP.
- The owner’s cost of evaluating the VEP.
- Any resulting administrative or management costs to the owner.

The driving force for submissions is the Value Engineering Incentive. In these projects, this was 50% of the net saving being earned by the contractor. There are, however, disincentives, summarized as follows:
• The contractor’s cost in developing a VEP, for which he has no certainty of recovery.
• The time taken to prepare a complete design for a VEP submission by the contractor, his suppliers and his consultant.
• The time taken by the owner to evaluate, comment on, and negotiate a VEP, over which the contractor has no control.
• The period of delay during which the contractor can strictly only execute work unaffected by the VEP. He cannot, without risk, procure affected materials for either the original or alternative designs during this process.
• The uncertainty of obtaining acceptance of work carried out on components affected by the VEP prior to acceptance of the proposal.

The following bridges involved either partial or complete redesigns. All resulted from VEPs, which were accepted following minor adjustments requested by the Ministry. All involved the substitution of superstructure systems, along with either adjustments to the original substructure, or complete redesign.

The Bloedel Creek and Trent River Bridge contracts were won as part of a larger grading contract by Emil Anderson Construction. The Cowie and Wilfred Creek Bridge contracts were won by G.W. Carlson Construction. All four were scheduled for construction between late summer 1997 and spring 1998. The Duke Point Underpass was completed by Emil Anderson Construction in January, 1997.

**BLOEDEL CREEK BRIDGE**

This bridge is a single 45 m span, south of Courtenay, B.C. Twin 11.4 m wide roadways on separate superstructures cross 12 m above the creek at a 15° skew. The control line is on an 1800 m radius, with a deck superelevation of 3.4% (Figure 1).

The Bloedel Creek Bridge was originally designed using three steel plate girders per superstructure, 1.9 m deep, at 4.2 m centres. The deck was nominally 275 mm thick. Most of the ground and creek between abutments was not accessible for construction because of environmental restrictions.

The contractor wanted to reduce costs by using precast concrete girders. Studies indicated that this would be economical if a three-girder superstructure arrangement was maintained. This would, however, result in higher loads than concrete girders typically support over this span, and would stretch the limits of precast girder design.

The selected girder was a standard 2.3 m deep precast concrete I-section. At 46 m long, the precast girders weighed about 76,000 kg, well above road transportation limits. The precast concrete manufacturer’s plan was to transport the girders in one piece by barge from their plant in the Lower Mainland to a loading dock near the site, and then by off-highway forestry road to the site. Erection would be carried out using two 40 m-long launching trusses side by side, supported by falsework in front of the abutments.
In order to use the concrete three-girder system effectively, the superstructure needed reconfiguring. The 1 m wide girder top flanges permitted a 250 mm concrete deck to be used, with a girder spacing of 4.5 m. This spacing improved the deck overhang formwork. To provide sufficient flexural capacity, a hybrid prestressing system is used. Each girder incorporates the following:

- 44 straight 13 mm diameter pretensioned strands.
- 12 deflected 13 mm diameter pretensioned strands.
- 36 - 15 mm diameter post-tensioned strands, in three draped tendons of 12 strands each.

In order to maintain adequate cover in the 150 mm thick girder web, oval-section ducts were used. The post-tensioning was designed to be field-applied in two stages. Two tendons were tensioned prior to construction of the concrete deck, and one after the deck concrete had attained 20 MPa compressive strength. All tendon anchorages are located in the deck end-diaphragms, which were designed for this purpose. The cast-in-place diaphragms also serve as integral ballast walls. This avoids the need for precast girder end-blocks, which simplifies fabrication and, more importantly, reduces the weight of the girders.

This was important in order to maximize the span potential of the launching trusses. Having the end-diaphragms structurally effective prior to deck construction also helps to stabilize the girders. Two intermediate diaphragms per bridge are provided for erection stability. A minor benefit in live load distribution would also be achieved. To speed construction and save weight, structural steel sections were used, bolted to the concrete girders.

The prestressing provides sufficient girder capacity for transportation and handling. The girder strength is supplemented by the first stage of field post-tensioning in order to provide sufficient resistance for deck concrete placement, the largest load component. Additional post-tensioning is then added to provide sufficient capacity to resist imposed loading.

By using the hybrid prestressing system, the following is achieved - which would not be possible using a conventional pretensioned girder:

- More total prestressing applied to each girder.
- Greater strand eccentricity near mid-span.
- Greater strand deflection, vertical prestress component, and shear capacity near the girder ends.
- Better control of stresses throughout the loading stages.

This approach, coupled with a high specified strength of 70 MPa at 56 days, permitted the three precast concrete girder concept to be used.

The additional 400 mm of structural depth compared to the original design was not an issue. The clearance over the creek below was controlled by the highway profile, and remained adequate. The abutments were adjusted to accommodate the revised superstructure details. The original footings were located on bridge end fill or bedrock. The footings were adequate to support the additional superstructure weight. The integral ballast walls reduce the overturning effects inherent in the original design, such that footing sizes could be reduced, despite the additional weight.
The Bloedel Creek Bridge is scheduled for completion in June 1998.

**WILFRED CREEK BRIDGE**

The Wilfred Creek Bridge is one of the few multi-span fully integral abutment bridges in B.C. The twin 65 m long superstructures are continuous over spans of 17 m, 29 m, and 19 m, and cross 15 m above the creek at a skew of 15°. The deck widths are 11.9 m northbound and 12.9 m southbound. The additional width provided sufficient sight distance around the 1000 m radius curve. The deck has 4.8% superelevation (Figure 2). The bridge is supported on driven steel pipe piles and concrete piers. Pier piles were specified in the original design as scour protection. These were not changed in the VEP. The site is immediately south of Courtenay, B.C.

The bridge was originally designed with four 1.2 m-deep continuous steel plate girders per superstructure, and a 225 mm thick concrete deck. The bridge was redesigned using pretensioned, continuous concrete I-girders. These are 1.7 m deep and are spaced at 3.5 m or 3.8 m centres, similar to the steel girders. To reduce the quantity of deck reinforcement, the selected deck thickness was increased to 250 mm. The additional superstructure depth was not an issue.

The contractor was installing the piles during the preparation of the Value Engineering Proposal. As a result, only superstructure changes could be contemplated, and no opportunity was available to investigate a different number of girders. With a maximum weight of 32 000 kg, the girders are readily transportable. As the girders and deck are inherently stiff and only lightly stressed, no intermediate diaphragms are necessary. The centre span units were positioned using a steel launching truss resting on the intermediate support bents. This was the same truss used to erect all four of the precast I-girder bridges discussed in this paper. Two 165-ton and one 440 ton cranes were used for the erection. Economies were undoubtedly gained by the two contractors in the erection methods and equipment mobilized for these four bridges.

In order to provide the maximum structural efficiency and constructibility, the simply-supported girders were placed directly on the abutments and piers, while projecting reinforcement was embedded into the concrete support-diaphragms. No bearings or joints are used. After placing the support-diaphragm concrete, the girders became continuous for subsequent imposed loads, including the deck concrete. This novel approach provides a more efficient structure to resist the deck self-weight, while also stabilizing the girders. The tensile stresses near the girder ends are controlled by deflecting a high proportion of the pretensioning strands. Twelve of the twenty-eight 13 mm diameter strands in the centre spans are deflected. Projecting overlapping rebars resist the support moments.

The concept of stiff, built-in concrete girders allows the deck concrete to be placed in one continuous operation. Minor deck cracking was observed in the weeks following deck concrete placing, but was not attributable to the deck construction sequence. The cracking was described as substantially less (wider spacing and less crack width) than often observed in decks on modern continuous steel girders. It was not perceived as an issue by the Ministry. Crack widths are expected to reduce as creep strains occur over time.
The girders are shorter versions of the Cowie Creek girders (see below), and therefore, experience comparatively low stresses. A 28-day concrete strength of 40 MPa was specified. To accommodate the deeper superstructure, the abutment wall depth was increased and the intermediate pier columns were shortened. In supporting the heavier superstructure, the pile loads increased only slightly, and the pier columns were found to have adequate seismic load resistance without modification. In light of the increased demands, pile capacities - based on driving records - were independently reviewed by a geotechnical engineer, and found adequate. The bridge is scheduled for completion in June 1998.

**COWIE CREEK BRIDGE**

This bridge is a three-span 92 m-long bridge south of Courtenay, B.C. (Figure 3). Twin 11.4 m wide roadways on separate superstructures cross 20 m above the creek at a skew of 6°. The main span is 37.5 m, with 27.5 m end spans. The control line is on a tangent. The deck crossfall varies, however, as superelevation run-offs encroach onto both ends of the deck.

The Cowie Creek Bridge was originally designed using four 1.2 m-deep continuous steel plate girders per superstructure, at 3.25 m centres. A 225 mm thick cast-in-place concrete deck is used. The bridge length was reduced by using large end fills. The area beneath the main span was not accessible for construction purposes.

The contractor wished to replace the steel girders with prestressed concrete girders, by adapting the system used for Wilfred Creek Bridge. It was determined that a similar system could be employed for the two bridges. The 1.7 m deep prestressed girders, weighing up to 42 000 kg, are just within highway transportation weight and length restrictions. The centre span units could be positioned using a launching truss resting on the pier caps.

It proved to be just possible to design the precast girders for the negative end moments generated by the weight of the concrete deck. Ten overlapping 30M hooked bars project from the top flanges into the concrete pier diaphragms. The precast concrete girders experience significantly higher stresses than at Wilfred Creek. Twenty six straight strands and twenty deflected strands of 13 mm diameter are used in the centre span girders. A 56-day strength of 70 MPa was required. The precaster was able to achieve the required compressive strengths at all stages without resorting to “high performance” concrete.

A single intermediate diaphragm of steel angle bracing is used at the midpoint of the centre span to stabilize the girders during construction. The 0.5 m additional depth of the concrete superstructure was readily accommodated by lowering the pier caps and abutment seats. The inherent stiffness of the precast girders permitted the deck concrete to be placed in one continuous operation.

Integral ballast walls are used at each end of the 92 m-long superstructure. No bearings are used at the monolithic pier connections, but laminated rubber bearings support the girders at abutments.

A re-analysis of the bridge was carried out for seismic loading arising from the heavier concrete superstructure. No modification of the columns or footings was required. The re-design of all substructure components considered seismic demands arising from plastic hinging of columns. An
increase in lateral sliding resistance of the bank-seat abutments was provided by placing two 0.9 m deep shear keys beneath the footings. The rationale was to have the re-designed abutments begin to slide at the same seismic accelerations as the original design. This was judged necessary, and easily done, to partly satisfy the criterion of equivalent performance. The bridge construction is scheduled for completion in July 1998.

**TRENT RIVER BRIDGE**

The three-span, 106 m long Trent River Bridge is located 750 m north of the Bloedel Creek Bridge (Figure 4). Twin 11.4 m wide roadways on separate superstructures cross 24 m above the creek at a skew of about 22°. The 1800 m horizontal curve requires a deck superelevation of 3.4 %.

This bridge was originally designed using three 1.6 m-deep continuous steel plate girders per superstructure, at varying centres of approximately 4.2 m. The deck had a minimum thickness of 275 mm. No construction access was available in the area beneath the main spans.

The contractor wished to replace the steel girders with precast concrete. A rapid assessment indicated that to achieve the maximum cost saving, the original three-girder arrangement would have to be retained. By increasing the girder spacing to a constant 4.4 m, a deck thickness of 250 mm could be used, reducing the superstructure weight and simplifying deck forming.

The 40 m main span girders could be placed by using two launching trusses supported on the pier caps. Standard 2 m deep precast I-girders could be employed, but would need to be adapted to the high levels of loading than they would typically experience. Their maximum length of 39 m and weight of 53,500 kg precluded highway transportation. However, by using barges and industrial roads, the girders could be routed to site.

The challenge was to design the girders. The system used for Cowie Creek Bridge was investigated. Insufficient flexural capacity was available both at mid-span and at the intermediate supports. The solution was to supplement the pretensioned segments by applying field post-tensioning from end to end. In this way, a higher total level of prestressing could be applied, thereby increasing girder load capacity without increasing the number of girders.

Each girder has sixteen straight and twelve deflected 13 mm diameter pretensioning strands. These strands provide strength for transportation and handling while reducing the required quantity of post-tensioning. Four ducts per girders are provided, each containing seven 15 mm diameter strands. The ducts are oval-section, 55 mm by 75 mm, in order to maintain adequate cover in the 140 mm thick girder web. The ducts are field-spliced prior to casting the concrete diaphragms and subsequently post-tensioned. The tendon anchorages are located in the cast-in-place ballast walls. All tendons are tensioned prior to deck construction, which is carried out in one continuous concreting operation.

The specified girder concrete strength is 70 MPa at 56 days. As well as using the maximum flexural capacity of the girders at mid-span, the maximum achievable shear capacity was required near the intermediate supports. Full advantage of pretensioning, post-tensioning, the concrete strength, and the vertical component of prestressing was taken. The Modified Compression Field Theory contained
in the draft CHBDC was used. Mobilizing both the maximum flexural and shear capacities is a good indication that the design achieved maximum efficiency, and stretched the limits of precast concrete bridge girder technology.

To enhance stability during construction a single intermediate diaphragm of galvanized steel WT bracing is provided at the centre of the main span. The additional depth of the concrete girders was acceptable and did not require any revision to the highway profile. No bearings are used at the monolithic pier connections, but laminated rubber bearings support the girders at the bank-seat abutments. Although the intermediate piers did not require strengthening to resist seismic loads from the heavier superstructure, shear keys were added beneath the footings of the abutments to increase their sliding resistance.

The substructure of the Trent River Bridge was also included in the Value Engineering Proposal. An evaluation demonstrated that the spacing of the pier columns could be reduced to better balance the loading on the caps and footings. Additional savings were realized by reducing the size of the pier footings. All substructure components were detailed to resist seismic demands associated with plastic hinging of the columns.

The largest cost saving, however, was achieved by eliminating the forty-eight, vertical 32 mm diameter threadbar ground anchors originally detailed. This was done by lowering the footings by up to 1 m, thus keying them into the site bedrock. Footing sizes were detailed to resist seismic demands from column hinging, and in fact were reduced in the heavier VEP design. The Trent River Bridge is scheduled for completion in July 1998.

**DUKE POINT UNDERPASS**

This bridge is part of the grade-separated interchange between the Trans-Canada Highway and the spur road to the Duke point Ferry Terminal near Nanaimo, B.C. The bridge carries three traffic lanes on a 16.7 m wide deck over the four-lane divided highway. The bridge is continuous over its 21 m, 38 m and 21 m spans. The plan alignment is part radius, part spiral, with a minimum of 5.6 % deck superelevation.

The bridge was originally designed as two 1.5 m deep twin-cell cast-in-place post-tensioned concrete trapezoidal box girders supported by flared concrete piers. The contractor wanted to build a precast concrete girder bridge. Because of aesthetic considerations and a depth restriction, I-section girders were not acceptable. As a result, a 1.4 m deep precast concrete trapezoidal open-top box girder was used for the Value Engineering Proposal. By using three of the box girders at 6 m centres, a useful cost saving could be achieved. Additional costs could be saved by supporting the girders on circular discrete-column piers (no pier cap beams), while eliminating the steel pipe piles originally used to support the east pier and abutment. Instead, footing subexcavation and structural fill replacement was employed.

The superstructure design uses five segments per girder connected by cast-in-place concrete splices and field post-tensioning. The 21 m long centre span segments are lightly pretensioned, while the remainder are provided with reinforcement to control handling stresses. The 16 m-long pier segments
include local bottom flange thickening near the piers and an integral internal diaphragm. The girders are supported by pot bearings on top of the columns and laminated rubber bearing at the abutments. No intermediate or pier diaphragms are used. The end-diaphragms retain the approach fills and accommodate the post-tensioning anchorages.

Each girder is provided with six tendons of eleven 15 mm diameter strands. One additional strand per tendon could have been added if needed in the field, although this proved to be unnecessary. Only ten strands per tendon were assumed effective in the design. The eleventh strand is included to avoid the provision for future post-tensioning specified by the Ministry. Inside the box girders, 50 mm thick precast stay-in-place forms were used. The deck was cast in one operation and then post-tensioned longitudinally.

The bridge was completed in January 1997 and opened to traffic a few months later. Despite the time taken to redesign the bridge and obtain approval for the VEP, no delay in the construction schedule occurred.

COST SAVINGS

The following are net Value Engineering savings. Under the contract Value Engineering clauses, the VEP savings are shared equally between the Ministry and the contractor. The gross design savings, i.e., prior to deduction of the cost of assessment, cost of VEP preparation, and engineering redesign, is about 30% higher:

Table 1 - VEP Cost Savings

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Tender Cost ($)</th>
<th>VEP Savings ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bloedel Creek</td>
<td>1,400,000</td>
<td>97,000</td>
</tr>
<tr>
<td>Creek</td>
<td>2,000,000</td>
<td>120,000</td>
</tr>
<tr>
<td>Cowie Creek</td>
<td>2,300,000</td>
<td>230,000</td>
</tr>
<tr>
<td>Trent River</td>
<td>2,800,000</td>
<td>400,000</td>
</tr>
<tr>
<td>Duke Point</td>
<td>1,500,000</td>
<td>300,000</td>
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<tr>
<td>Total</td>
<td>10,000,000</td>
<td>1,147,000</td>
</tr>
</tbody>
</table>

DISCUSSION

The VEP savings on these bridges range from 6% to 20% of the tender prices. In all cases, precast concrete girders were used to replace steel girders, except for the Duke Point Underpass where cast-in-place concrete was originally used. It should be noted, however, that for the three bridges with the lowest cost savings, only the girders were changed. When this happens, an engineering evaluation of the complete structure and some redetailing is necessary as the superstructure depth and weight is
frequently different.

The largest savings occurred with the redesigns for the Duke Point Underpass and the Trent River Bridge. For these structures, the complete bridge was re-engineered. Savings in the substructure were significant and were similar to superstructure savings.

If the VEP savings are viewed as the gross difference in cost of the materials changed, the savings are highly significant. That this can be achieved is perhaps surprising. The bridges, it should be realized, have already been subject to careful evaluation and design optimization by the original designers. In addition, the Ministry or their Project Managers reviews consultant designs for quality and cost-control. Savings over approximately 10%, however, are regarded as appreciable. The savings enable more bridges to be built for a given budget.

Why, therefore, are significant savings possible? The answer, we believe, is because there is no infallible system for designing the most cost-effective bridge. The incentive of 50% of the VEP savings will drive an aggressive contractor to find a better way. In these cases, it involved stretching the recognized limits of precast concrete girder technology. The contractors, however, take a significant risk in deferring construction while he prepares a VEP, during which time affected work is either delayed or performed speculatively. The Ministry may also take on financial or schedule risk in evaluating the VEP's.

By allowing the contractor to take ownership of the design via Value Engineering, the cost and schedule advantages of design/build are available. The resulting stimulus to be creative and develop a solution geared to the contractor’s available resources, while introducing competition from a different construction material, often permits significant savings to be found. By opening up the bridge design as well as construction to competition, better value can be delivered by industry to the owner. Always desirable, it is especially important for publicly-funded projects.

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**Owner's Representative:** Vancouver Island Highway Project Management Team.
**Contractor:** Wilfred Creek Bridge, Cowie Creek Bridge: Carlson Construction Ltd.
         Bloedel Creek Bridge, Trent River Bridge
         Duke Point Underpass: Emil Anderson Construction Ltd.
**Precast Concrete Manufacturer and Erection:** Con-Force Structures Ltd.