SAUVIE ISLAND BRIDGE REPLACEMENT

KENT CORDTZ

KIP COULTER

PHOTO UNAVAILABLE

IAN CANNON

BIOGRAPHY

Kent Cordtz, P.E., S.E. is the Bridge Discipline Director for David Evans and Associates, Inc. and is the engineer of record for the Sauvie Island Bridge. Mr. Cordtz obtained his Bachelor's and Master's degrees in civil engineering at San Jose State University, San Jose, California. He has been practicing bridge and structural engineering for 26 years, including 6 years with the Roseville, California office of David Evans and Associates, Inc. He is a licensed engineer in California, Nevada, Oregon, and Washington.

Kip Coulter, P.E. is the Bridge Discipline Leader for the Denver office of David Evans and Associates, Inc. and is the tied arch design leader for the Sauvie Island Bridge. Mr. Coulter obtained his Bachelor's degree in civil engineering at the University of Maryland, and has been a practicing bridge engineer for 42 years, including 6 years with David Evans and Associates, Inc. He is a licensed civil engineer in California, Washington, Colorado, and Montana.

Ian Cannon, P.E. is the Bridge Services Manager for Multnomah County, Oregon, and was the county's Senior Project Manager for the Sauvie Island Bridge replacement project. Mr. Cannon obtained his Bachelor's and Master's degrees in structural engineering at Washington State University, and has been a practicing bridge and marine engineer for 16 years, including 10 years with Multnomah County. He is a licensed professional engineer in Oregon and Washington.

SUMMARY

The Sauvie Island Bridge spans the Multnomah Channel of the Columbia River near Portland, Oregon. The five-span, 1177-foot replacement bridge features a 365-foot steel tied arch main span. Special design considerations included a unique radial cable pattern and provisions for internal redundancy in the tension tie beam. The bridge was assembled on a dock 9 miles from the project site, and then transported by barge to the site, where it was jacked into final position. This paper presents details of the design, fabrication, and erection of this signature structure.

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Kent Cordtz, P.E., S.E.: David Evans and Associates, Inc.
Kip Coulter, P.E., David Evans and Associates, Inc.
Ian Cannon, P.E.: Multnomah County

Figure 1. Photo-simulation of the 5-span replacement bridge

INTRODUCTION

The Sauvie Island Bridge is located in Multnomah County, approximately 10 miles north of downtown Portland, Oregon. The existing bridge provides the only vehicular access to Sauvie Island. The 24,000 acre island is bounded by the Columbia River, the Willamette River, and the Multnomah Channel. The bridge spans the Multnomah Channel and carries a mix of local residential, agricultural, industrial, and recreational traffic. Vessel traffic in the channel below ranges from pleasure craft to commercial vessels.

The existing 14 span, 1198-foot long bridge was constructed in 1950 and is eligible for listing on the National Register of Historic Places. The bridge spans the Multnomah Channel with a steel through-truss at the navigation span and deck trusses at the two side spans. The approaches consist of a total of 11 spans of reinforced concrete T-beams. The bridge has substandard roadway and sidewalk widths and excessive approach grades. In December of 2001, large cracks were found in the concrete approach spans. Emergency repairs were completed in early 2002. Even with the temporary repairs, the existing bridge is not adequate to meet the current needs of the island's farmers and industrial businesses. The existing bridge is classified as functionally obsolete and structurally deficient and is being replaced.

Figure 2. Existing Sauvie Island Bridge

David Evans and Associates, Inc. (DEA) completed the Tier I Bridge Siting Study for the new bridge in 2002 and the final design in

2005. The project was bid in 2005, and construction is scheduled for completion in 2008. The project has involved the public and other stakeholders in identifying critical issues and developing viable solutions to achieve a successful project. This process, led by Multnomah County, was conducted from the Siting Study through final design.

Replacement Bridge Alternatives

A total of 21 structure-type alternatives were initially identified for the replacement bridge. A screening process was implemented to determine the types that best satisfied the goals and objectives of the project. The screening process used four steps or "tiers" to eliminate alternatives that did not meet the specified criteria. The following summarizes the criteria for each of the four tiers:

1. Construction cost of the alternative must be within the budget limits established by Multnomah County based on available funding sources.
2. The alternatives advanced from Tier 1 must have a portion of the structure above the bridge deck to meet the requirements of the Oregon State Historic Preservation Office (SHPO).
3. The alternatives advanced from Tier 2 were evaluated based on initial cost, life-cycle costs, environmental sensitivity, and construction duration.
4. Members of the project team assessed the merits and disadvantages of the alternatives advanced from Tier 3. Public comments, concerns, and recommendations were considered. Discussions were held with the Multnomah County Board of Commissioners.

Figure 3. Photo-simulation of 8-span half-through arch

Initially, a weathering steel half-through arch with a 425-foot center span and two 155-foot side spans was identified as the preferred structure type for the main span. The aesthetics of the eight-span bridge with the half-through arch was preferred by many of the stakeholders in spite of its cost.

The rapidly rising costs of construction eventually forced the team to reconsider the other alternatives. A five-span alternative with a steel tied arch main span was eventually selected. The cost reduction was a result of the shorter main span, simplified details and fabrication, and reduction in the number of bents and deep drilled shafts.

STEEL TIED ARCH REPLACEMENT BRIDGE

The selected 5 span, 1177-foot long replacement bridge features a 365-foot weathering steel tied arch main span. The tied arch and its unique radial cable pattern satisfied the stakeholders' desire for an aesthetically pleasing bridge and solved the following engineering issues. The shallow depth of the tie girder met vertical clearance requirements over the navigation channel while eliminating a non-standard roadway profile. The tied arch met the requirement by the Oregon State Historic Preservation Office to construct a through type structure to mitigate the loss of the existing through-truss. The steel tied arch reduced the number of piers in the channel, significantly increased the permanent navigation opening, and met the Coast Guard permit requirement to construct the bridge over the navigation channel without any falsework and with minimal disruption to navigation.

Steel Erection

To attract the maximum number of bidders, the tied arch span was designed to be erected by either of two methods. The first method was cantilever erection using temporary towers and stay cables with material delivery and erection by barges. The towers would be located on the piers adjacent to the channel with backstays anchored to the approach spans. The second method was to assemble the tied arch and floor system offsite, deliver it to the site on barges, and erect it on the piers.

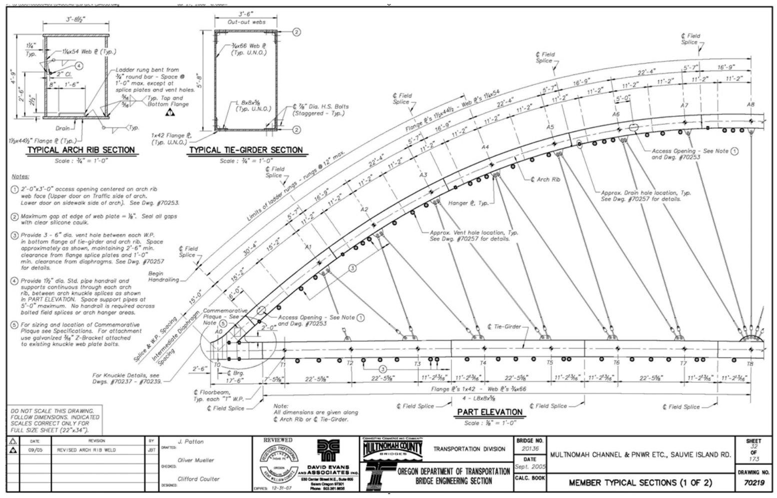

Figure 4. Tied Arch Detail

The Contractor selected the second method. This method allows for concurrent construction of the approaches and main span with a savings in schedule. The tied arch and floor system has been assembled on a dock at the Port of Portland located on the Willamette River approximately 9 miles from the project site. Temporary compression struts between the arch and tie girder have been installed to stiffen the structure during load-out and erection. The forms and reinforcement for the concrete deck will also be placed during initial assembly. While on the dock, the weight of the span will be transferred from the temporary cribbing to four skid beams located near the knuckles. The span will then be skidded from the dock onto self-propelled dollies on a single transport barge, and will be transported to the site in the low position. The bridge bearings will be attached to the structure prior to transporting to the site. Once at the site, the span will be raised approximately 60 feet by self-climbing jacks on the four barge-mounted jacking towers. The barge and arch span will then be positioned between the bents at high tide. The falling tide will lower the bridge onto temporary supports beneath the bearing seats at the correct final elevation, after which the bearings will be grouted in place. Temporary guides will position the bridge laterally and longitudinally as it is lowered onto the bearing seats.

Arch Rib and Tie Girder

The arch span has a span-to-rise ratio of approximately 4.7:1. The span-to-rise ratio was selected on the basis of aesthetics and structural considerations, and results in a somewhat greater arch rise than would be obtained from the more conventional 5.0:1 ratio. The arch ribs and tie girders have span-to-depth ratios of approximately 76 and 63 respectively.

The arch ribs consist of welded box sections with internal diaphragms at the hanger locations. The depth of the rib was dictated by the minimum access openings through the diaphragms required by Multnomah County to facilitate access for periodic inspection.

Since complete fracture of either of the two tension tie girders could result in structure collapse, these main members are considered to be fracture-critical. The fracture-critical nature of the main tie girders was addressed by detailing the tie girders as fully bolted members without any welding. The tie girders are built-up steel box sections consisting of web and flange plates connected by bolted corner angles. This provides for internal redundancy, as a fracture in one plate cannot propagate to the entire cross section and lead to collapse. The structure was designed for the possibility of the complete fracture of a web or flange plate at any section of the tie girder.

Hanger Cables

The hanger cables consist of $2^{1}/_{2}$” diameter galvanized structural strand per ASTM A586 with Class A zinc coating on all strands. Non-adjustable cast steel open strand sockets are provided at the upper connection to the arch rib. Cable tensioning is performed at the open bridge sockets provided at the lower connection to the tie girder. Zinc spelter sockets are employed to attach the structural strand to the anchor sockets. This method of attaching structural strand cables to cast steel sockets has been in use for many years, and employs molten zinc to permanently join the individual wires of the structural strand in a conical “basket” in the cast steel socket. The resulting hanger cable assemblies provide a dependable and internally redundant tension member.

Figure 5. Open Bridge Sockets

The unique radial cable pattern was selected during the public involvement process purely on aesthetic value. It is not as structurally efficient or as stiff as a traditional vertical cable pattern or a cross cable pattern. In other recent tied arch projects, a crossed-cable pattern has been found to be most effective in stiffening the entire structural system and minimizing differential live load deflections, particularly when the live load is placed at one quarter point of the arch span. The resulting bending moments, stresses, and deflections in the arch rib and tie girder were minimized through proper definition of the arch rib geometry. This required an iterative process to develop the most efficient arch shape, as the cable forces, arch rib and tie girder moments, and deflections are extremely sensitive to the arch rib geometry. The final shape of the arch differs somewhat from the classic shape of a uniformly loaded arch, and also from the common approximation of that shape by a second-order parabola.

Floor System

The floor system consists of longitudinal stringers supported by transverse floor beams. Due to vertical clearance requirements over the navigation channel and roadway approach grade restrictions, the top of stringer and top of floor beam coincide. This results in the least structure depth and the lowest roadway profile.

The stringers are composite rolled wide flange sections with moment connections at the floor beams. The floor beam connections to the tie girders occur at hanger cable supports.

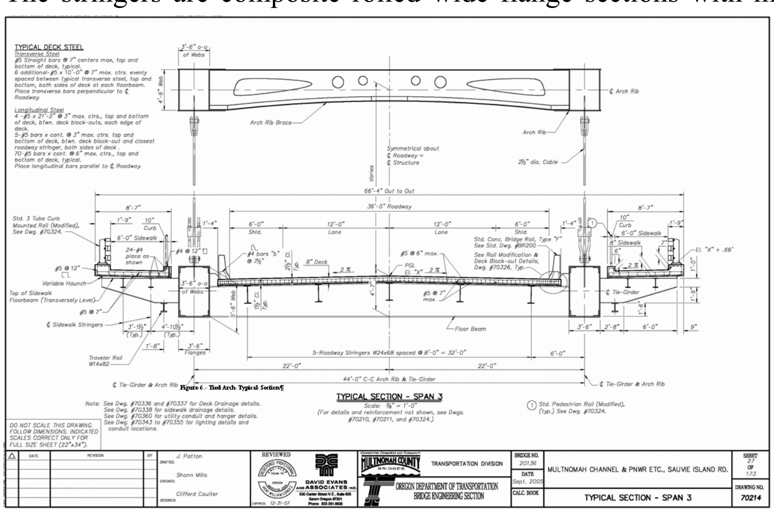

Figure 6. Tied Arch Typical Section

The floor beams are composite welded plate girders with moment connections to the tie girder. The floor beam connection prevents out-of-plane deformation of the floor beam web due to relative movement between the floor beam flange and the tie girder. The connection is intentionally flexible about the web of the floor beam in order to minimize weak axis bending. In addition, the deck slab is separated from the top flange of the floor beam near the connection to the tie girder.

Lateral Bracing

Bottom lateral bracing consists of WT sections arranged in a "K" pattern. Top lateral bracing consists of built-up variable depth welded I sections arranged in an "X" or diamond pattern. A unique aesthetic feature was incorporated into the design of the top lateral bracing. The lateral bracing webs have large cut-outs that resemble the wing markings of a local butterfly and increase the visual transparency of the structure.

Corrosion Protection

One of the project goals was to minimize long-term maintenance costs such as painting of steel members. Weathering steel per AASHTO M270, Grade 50W (ASTM A709, Grade 50W) is used throughout the main span. Weathering steel has been used successfully in the Portland area in girder bridges and also in exposed trusses.

The project incorporates numerous details to minimize corrosion concerns related to the use of weathering steel in a wet climate.

In the arch ribs and tie girders, potential condensation and water intrusion is diverted to drains, vent holes are provided in the bottom flanges, and caulking is provided at specified locations such as the gap between the lower hanger plate and the top flange of the tie girder, and on the "uphill" side of outer field splice plates.

As an additional measure of protection, the following portions of the weathering steel are painted:

- Interior of the arch ribs
- Interior and exterior of the tie girders and knuckles
- Pin hole areas of arch hanger plates
- Outer ends of floor beams at connections to the tie girders
- Ends of longitudinal stringers at deck expansion joints

The interior surfaces are painted a light color to facilitate inspection. The exterior surfaces are painted a color similar to uncoated weathering steel.

The hanger cables are galvanized structural strand with Class A zinc coating applied to each individual wire. The sockets, stirrup rods, and pins are galvanized. Contact surfaces between galvanized components and weathering steel are painted.

Design Issues

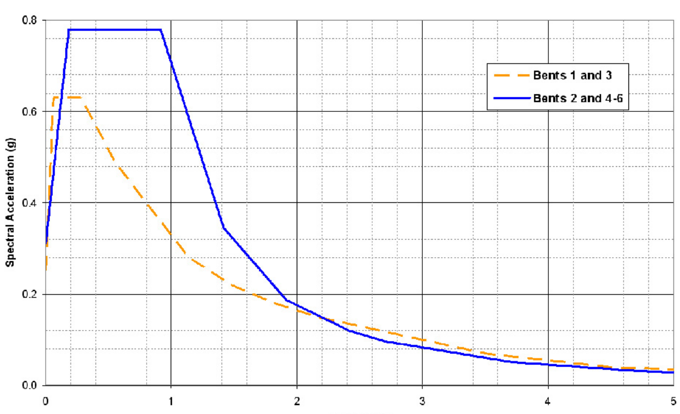

Figure 7. Spectral Acceleration Response Spectra (5% Damping)

The bridge is designed in accordance with the AASHTO LRFD Bridge Design Specifications, 3rd Edition, 2004 (LRFD). Supplemental requirements included permit loadings per Oregon Department of Transportation (ODOT) and Multnomah County.

The tie girders and knuckles are designated as fracture critical members (FCM) by ODOT. Redundancy was explicitly incorporated into the design to preclude failure of the tie girder due to fracture of a flange or web plate. The bolted built-up tie girder was designed for the loss of a single web or flange plate using a special Extreme Event load combination. Extreme Event III as specified by the Pennsylvania Department of

Transportation, developed under FHWA direction, was used for this evaluation as LRFD lacks specific adopted requirements in this respect.

Bridge hanger cable assemblies, and the hanger plates to which they are attached, are designed with a minimum factor of safety of 4.0 for breaking strength versus calculated unfactored dead load plus live load including impact. Bridge hanger cable assemblies are also designed for the loss or replacement of any one cable with a minimum factor of safety of 3.0 for breaking strength versus calculated unfactored dead load plus live load including impact.

A number of steel tied arches have experienced excessive live load deflections. The critical condition for live load deflection is when the live load is placed on one-half of the span. The loaded half of the span deflects downward while the other half deflects upwards. For this project, the design was based on a deflection limit of L/800 where L is the length between the bearing and the point of inflection. This equates to a live load deflection limit equal to the total span length divided by 1600. Refinement of the arch rib geometry resulted in live load deflections even less than this limit.

Seismic effects were a consideration in the bridge design. The peak ground acceleration for the 1000-year earthquake is 0.28g. Two site specific response spectra were developed for the project due to the highly variable subsurface conditions.

The design of the main members of the arch span was not controlled by seismic effects. Seismic forces and movements were, however, a major consideration in the design of the restraining features and seat lengths.

Fixed bearings are provided at one end of the arch span and guided bearings at the other. The bearings were designed to transfer the seismic forces from the arch span to the supporting bents. The maximum seismic forces on the bearings were 1500 kips. The bearing design forces are the unreduced seismic forces from the controlling response spectrum analysis including a 100%-30% combination of orthogonal effects.

A secondary system provides redundancy in the case of a bearing failure. Concrete shear keys are provided at each bent to restrain transverse movements and lateral seismic forces. The seat lengths were sized to provide more than twice the maximum longitudinal seismic movement to prevent unseating. Cable restrainer units consisting of twenty ¾” diameter galvanized wire strand cables were also provided across each expansion joint to tie the arch span to the bent to restrain longitudinal seismic movements.

Construction Loading Considerations

The radial cable pattern results in two cables intersecting at each floor beam. The more vertical cable in each pair tends to carry more of the gravity loads. The hanger cables are designed to remain in tension under all loading conditions. If the dead load were to be applied to the completed structure in a single stage, the more vertical cables would have a larger tension force than the more inclined cables. The more inclined cables would become slack under certain loading conditions. This requires manually stressing the cables or altering the construction sequence to obtain the minimum cable tensions. The following sequence provided in the plans eliminated the need to stress the cables:

- Assemble arch span on temporary supports. All cables are slack.
- Remove slack from the more inclined cables.
- Remove temporary supports under tie girders to load the more inclined cables.
- Remove slack from the more vertical cables.
- Place concrete deck and remaining superimposed dead loads to load all cables.

The plans allowed the Contractor to modify the erection sequence and cambers, if necessary. The Contractor developed an erection and cable installation sequence that differed from the assumed sequence but achieved the specified cambers and cable forces. The Contractor’s sequence required stressing each of the cables.

Maintaining the plan camber allowed the shop drawings and a portion of the steel fabrication to be completed prior to development and approval of the arch erection and load out/jack up submittals. Temporary compression struts installed between the arch and the tie girder were employed to stiffen the structure during load-out and installation.

The adequacy of the structure for the Contractor's proposed erection sequence and load-out/jack-up procedure was verified during the review process. In addition to the analysis performed by the Contractor's engineer, DEA independently verified the erection stresses and cambers. The main members were adequate without major modification. Strengthening was typically limited to localized areas such as at temporary strut connections and jacking points.

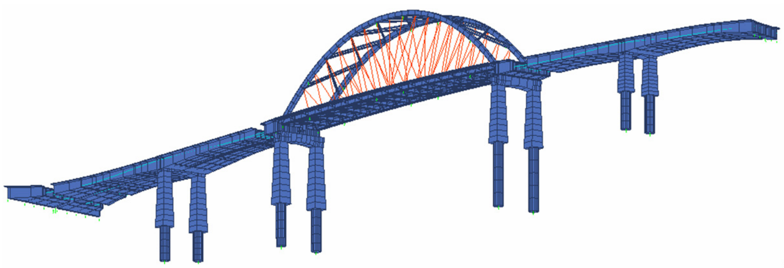

Figure 8. RM2000 Global Model of Replacement Bridge

Analysis Methods

The "global" analysis of the bridge was performed using the computer program RM2000. RM2000 is a specialty program developed for the construction stage analysis of bridges. The 3-dimensional analysis included the following:

- Construction stages including erection of steel members, hanger cable installation and stressing, concrete deck placement sequence, and time-dependent effects including creep and shrinkage.
- Live load including HL-93, permit vehicles, and sidewalk loading. A total of four ODOT permit vehicles and one Multnomah County permit vehicle were evaluated in addition to the HL-93 loading. Gross weight of the permit vehicles ranged from 98 kips to 228 kips.
- Dynamic analysis for seismic loading.
- Miscellaneous loadings such as wind and thermal as well as the prestressing effects on the approach spans.
- Effects of non-linear cable stiffness.

The 3-dimensional model captured the interaction of the floor system/concrete deck with the tie girder in resisting the thrust from the arch ribs.

RM2000 was used to model the structure during the design phase of the project. RM2000 reports member forces and stresses as well as deflections. Member capacities were verified using post-processors developed by DEA.

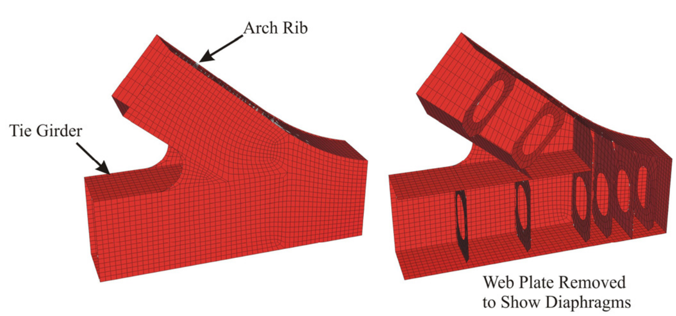

Figure 9. ABAQUS Model of Arch Knuckle

RM2000 was also used during the construction phase to analyze the structure for the Contractor's proposed erection sequence and load-out/jack-up procedure.

The "local" analysis of the arch knuckle was performed using ABAQUS. ABAQUS is a finite element analysis program geared towards the solution of complex problems. The local analysis captured the complex behavior of the arch knuckle and verified the adequacy of the design.

CONSTRUCTION

Construction began in 2005 and is scheduled for completion in 2008. Steel fabrication was completed in the spring of 2007. The fabricator employed a progressive assembly technique at his shop to complete final fit-up of all major steel members. Following shop fabrication and assembly, the components were dismantled and shipped to the Port of Portland dock on the Willamette River where the final assembly was completed. The tie girder was assembled to the correct cambered geometry on timber blocking supported on the dock, followed by arch assembly from temporary shoring towers supported on the tie girder. Following steel assembly, the hanger cables were installed and tensioned to the specified initial tension. Final load-out onto the barge for shipment to the jobsite is scheduled for October 2007.

Figure 11. Installation of permanent casing for drilled shafts

Figure 10. Progressive assembly of arch span at fabricator's shop

Construction of the concrete substructure and post-tensioned box girder approach spans progressed at the site while the arch fabrication and assembly was underway. The eight 10-foot diameter drilled shafts at the bents and the two 6-foot diameter drilled shafts at the island abutment have been constructed. Each drilled shaft was constructed with a permanent steel casing fitted with a cutting shoe and installed with an oscillator. Shaft lengths up to 163 feet were required to reach bedrock. Construction of concrete columns, cross beams, and abutments is nearly complete. The concrete box girder superstructure is being constructed on falsework supported by temporary steel pipe piling in the river channel.

CONCLUSION

Once complete, the new Sauvie Island Bridge will provide the required capacity to support the heavy vehicles operated by the island's agricultural and industrial businesses while also providing for safe bicycle, pedestrian, and truck use. The steel tied arch meets the project's stringent engineering and permitting requirements and also satisfies the aesthetic desires of the stakeholders.

Figure 12. Completed Span on Port of Portland Dock

PROJECT DATA

Owner:	Multnomah County, Oregon
Design firm:	David Evans and Associates, Inc., Portland, OR
General contractor:	Max J. Kuney Company, Spokane, WA
Steel fabricator:	Fought and Co., Tigard, OR
Steel erector:	Schneider Up, East Olympia, WA
Erection consultant:	T.Y. Lin International, Olympia, WA
Load out, jack up, and install:	Dix Corporation, Spokane, WA
	Norsar, LLC, Everett, WA
Project cost (bid):	$37,900,000
Bridge costs (bid):	$26,400,000 (including bridge removal and temporary works)
Steel tonnage:	1250 tons (121 pounds per square foot)
Hanger Cables (bid):	$500,000
Steel Cost (bid)	$7,275,000 ($2.92 per pound) including erection