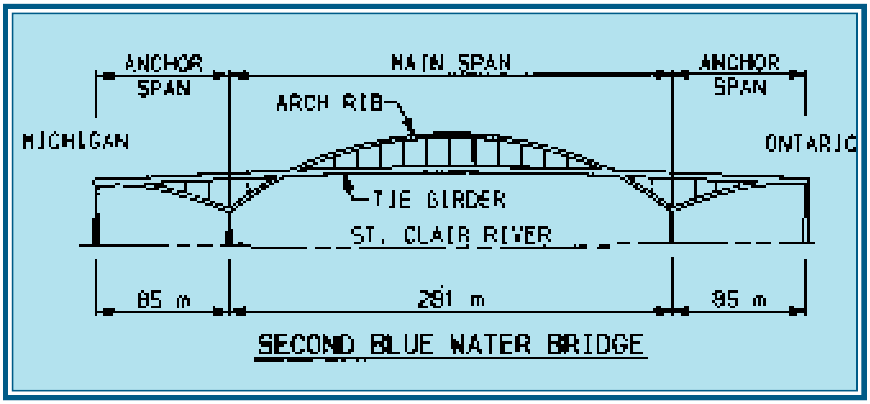


INTERNATIONAL CONNECTION

The Blue Water Bridge, which connects Port Huron in Michigan to Ontario, is one of the first major bridges to use LRFD and SI units

By Joseph E. Prickett, P.E., Brian D. Morgenstern, P.Eng., P.E., John M. Kulicki, P.E., P.Eng., Ph.D., and Roger A. Dorton, P.Eng., Ph.D.

As water leaves the southern tip of Lake Huron in the swift-moving Saint Clair River, it forms an international boundary between Ontario and Michigan. For almost 60 years international access between Port Huron, MI, and Point Edward, Ontario, has been provided by a cantilever truss bridge built near the north end of the river.

The Michigan Department of Transportation and The Blue Water Bridge Authority in Ontario jointly own and operate this cantilever bridge - each collects tolls from traffic entering the bridge, and traffic leaving the bridge passes through customs and immigration on each end.

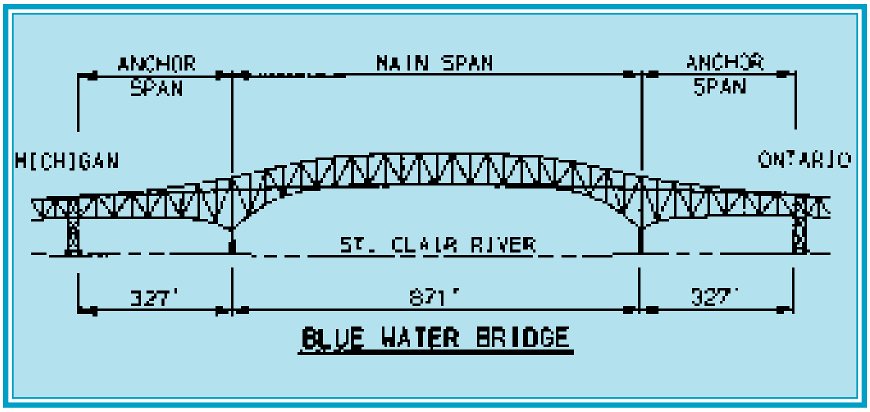
About 60 years ago, the firm of Modjeski and Masters, Inc. designed the river crossing and supervised its construction. This bridge is a Port Huron landmark and has become an integral part of the local heritage. The success of the crossing at this location has led to a steady growth of traffic and the anticipation of an additional bridge for a second crossing. Recent surges in the traffic volume led the Owners to begin work on the Second Blue Water Bridge years earlier than anticipated.

DESIGN OF THE NEW BRIDGE

The Joint Venture of Modjeski and Masters, Inc. from Pennsylvania and Buckland & Taylor, Ltd. from British Columbia were retained in 1993 by the Owners to develop studies and plans for the new bridge. The first phase of the work was to prepare engineering studies for the new crossing and develop a Study Report.

In late 1993, a Study Report was issued indicating that six bridge forms would be suitable for the new crossing:

- steel cable-stayed bridge;
- concrete cable-stayed bridge;
- a duplicate truss;
- a parallel chord truss;
- a simple span tied arch; and
- a continuous tied arch.



Shown at top is the Second Blue Water Bridge under construction with the First Blue Water Bridge pictured in the background. Pictured above is a line drawing of the First Blue Water Bridge.

Discussions, evaluations and economics of each of these types were presented in the Study Report to the owners with a matrix recommended for evaluating and choosing a type to be carried into final design.

The preliminary cross-section was established as a three-lane deck with sidewalk, traffic barriers and pedestrian railing. The preferred alignment adjacent to the existing bridge was set. All project documents would be completed in S.I. units. The design would conform to the new AASHTO LRFD Bridge Design Specifications and major provisions of the Ontario Highway Bridge Design Code (OHBDC), with the LRFD being the primary specification.

SELECTED STRUCTURE

For the main river crossing, a continuous tied arch was selected with approaches of box girders and multi-girder spans. A requirement for the main bridge construction was that the work be divided equally between the Owners, and that the construction be equally divided between a contractor from Canada and one from the United States in a joint venture contract, which dictated that at least two fabricators and two steel suppliers would be required. A considerable effort was required during the design to ensure that the details, materials, standards and procedures in the plans were proper for construction in both countries.

The main span deck is reinforced concrete for three traffic lanes and a pedestrian sidewalk. The slab thickness was set at 7" in order to reduce dead load on the main span. The roadway has a bituminous wearing surface, and the sidewalk has a latex modified concrete overlay. The stringers are rolled beam sections, made continuous and composite with the deck slab. The floorbeams are welded I-sections with welded transverse stiffeners. Steep roadway grades (4.65 percent) and channel clearance requirements resulted in a shallow superstructure depth. This limited the available web depth for the floorbeams, resulting in the need for intermediate floorbeams between vertical locations. Welded I-members were used for the floor system lateral bracing.

Under dead load only, an uplift condition would occur at the anchor span end bearings. A counterweight was added to provide a positive reaction under all loading conditions, except the most extreme live load case, and the bearings here are designed to resist the uplift resulting from that case. The floor system at the anchor end required modification to accept a concrete counterweight. Intermediate stringers were added to the typical cross-section in the two end panels. Stringer depth was increased to 36" in these two panels to support the concrete mass. The stringers were coped over the floorbeams to accommodate their increased depth.

Power driven, rail-mounted platform travelers provide access to the underside of the deck and floor system. One traveler rests near the anchor piers at each end of the bridge, and each is capable of traversing the entire length of the arch structure. Access to the remainder of the bridge is provided by an integrated system of crosswalks, ladders, stairways, railings and handropes. The special consideration given to access inside the tie girder resulted in forced ventilation, adequate lighting and special surface finishing of the interior.

The arch has proven to be a successful form of span for many years. The primary load path used by an arch is the curve of the arch itself, where the shape of the arch is ideally the shape of the moment diagram caused by the loading. The load is basically carried in compression by the arch, and at the supports a horizontal force is needed to resist the compression of the arch. A large number of steel arch bridge spans have been built, and many of these are simple spans using a horizontal steel tie member from end-to-end of the arch to resist the horizontal force of the arch. Less than a half dozen continuous tied arch bridges have been previously used in North America.

When the vertical load on the arch varies, some flexural strength and stiffness is required since the arch cannot change its shape to accommodate the change. The tie member is commonly supported by the arch so they can act together flexurally. In a tied arch, the bridge floor is commonly attached to the tie.

Since the tie girder and arch rib act together flexurally, it is possible to choose, by selective proportioning, the member which will carry most of the flexural stresses. For this design, the tie girder, which directly supports the bridge deck, was chosen to be the principal member and it is proportioned to be considerably stiffer than the arch rib.

This bridge layout consists of several basic segments. The main support framing consists of the end segments made up of the anchor spans (85 meters) and those portions of the main span extending from the main pier to the knuckle joints (36 meters). These support the middle segment or main arch (209 meters) between the knuckle joints. These are basically independent, closed units, except for the flexural continuity of the tie girder and arch rib at the knuckle joint. Steel vertical columns and hangers connect the arch rib and the tie girder. Steel floorbeams supporting the steel floor stringers are attached to the tie girders.

Arch Details

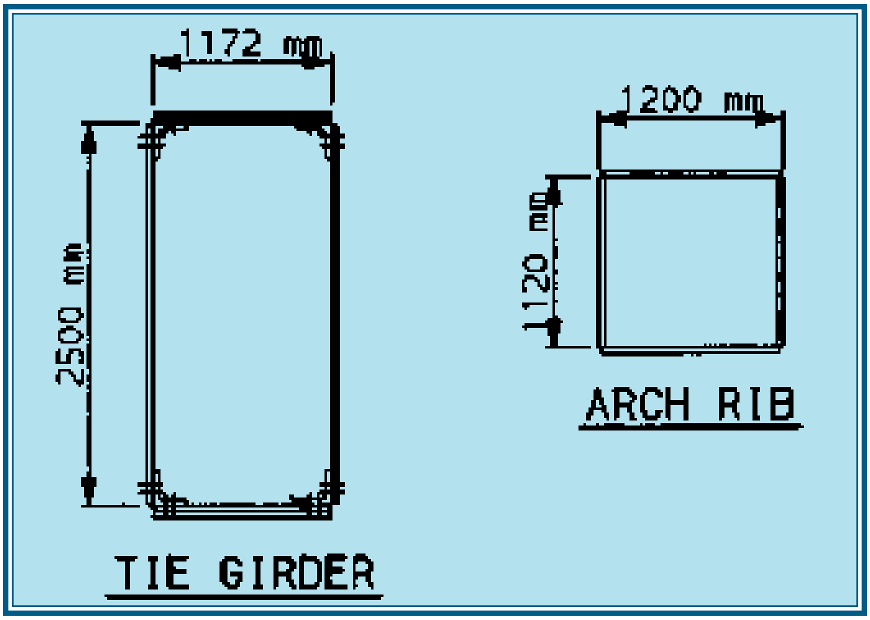
The continuous tied arch requires a number of special design considerations, as do the LRFD requirements, and the demands of the Owners for this crossing.

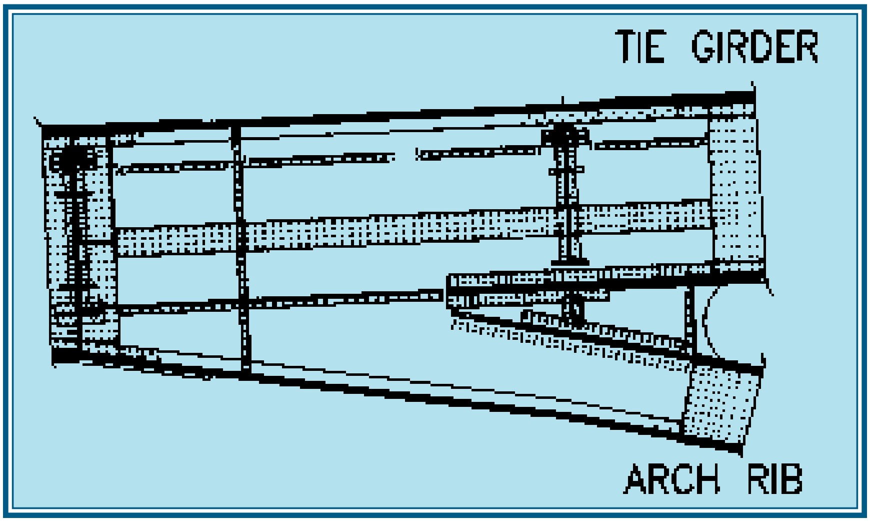
The arch rib is a box about 1.2 meters on a side made of welded steel plates, and near each end is a welded closure plate to seal the main length of the arch rib members. These sealed sections are not painted, but they have been partially evacuated, then filled with dried air and sealed. Pressure test points are located in the end portions of the members for long-term monitoring of the interior pressure.

The tie girder is a steel box built up by bolting and it consists of steel plates with corner connecting angles - it is about 1.2 meters wide by 2.5 meters deep. The tie girder is the tension member that provides the sole horizontal support for the entire arch. In addition, it provides most of the flexural resistance of the arch segments. It is the quintessential Fracture Critical Member and mitigation measures were proposed in the Study Report to make this tied arch structure, then under Federal moratorium, acceptable to the Owners. Clearly, mitigation measures translate into additional, but necessary, costs. If the tie were a steel box assembled by welding, it is possible that, under the impact of varying loading, a crack might propagate across the entire member (using the welds as a path from one plate to the other). For this reason, it was decided that the tie girder would not contain any welding; rather, it would be assembled by high-strength bolts. Even so, a potential crack could propagate across one of the plates or elements of the tie girder. As a safeguard, the tie girder was proportioned so that it could withstand the loss of any one plate or element. These measures gave the tie girder the internal redundancy desired and removed the specter of Fracture Critical fabrication requirements.

Temperature changes and loads on the bridge cause movements at the supports. The main arch bearing in Ontario is fixed against sliding, and all the others are designed for longitudinal sliding. The capacity for sliding is provided by incorporating Teflon on polished stainless steel within the bearing. At the Michigan main pier, the bearing design accommodates movement of over 300 mm. contraction and over 400 mm. expansion. A tough flexible disk in compression is a part of the bearing's support for vertical load, and permits the small rotation which takes place at the support joints.

In addition to the bolting used to assemble the tie girder, all the member connections are made with high- strength bolts, and all the high-strength bolts are galvanized. The paint system used on the bridge consists of a coat of zinc-rich primer, a second coat of epoxy, and a top coat of light grey urethane. The primer was shop applied to all surfaces of completed members, and the final two coats were shop applied to all, except faying surfaces at field connections.





Shown above, from top to bottom: arch and cross-sections; elevation of "Y" joint; and shop assembly of "Y" joint.

Special Details

Several locations on the arch required special study to develop arrangements and details which satisfied the requirements of structural adequacy and orderly flow of forces, aesthetics, maintenance, and fabrication and erection.

At the ends of the bridge, the tie girder and the arch rib merge into a single variable-depth box member for several panels. In merging the two members, a large compression from the arch rib combines with a large tension from the tie girder to form a moment in the single member. The interrupted flanges of the rib and tie are continued well into the joint to help accomplish this transfer. The sizes of the plates became so large it was necessary to add a longitudinal field splice along the joint to make the sizes manageable.

The support joint at the main pier was a location made difficult by the fact that it is a major support for the bridge, and also because of the large angle change in the arch rib. The general arrangement provides continuity for the heavily loaded arch rib plates in the large weldment at the base of the column. The vertical sides of the arch rib are backed up by the vertical plate of the column; the bottom flange of the arch rib bears on the bearing plate; and a special plate was added inside the column to back up the top flanges of the rib. Large welds at close spacing is used, and the plans required the Contractor to assemble and fabricate one joint just to evaluate the distortion from the welding. If satisfactory, the trial assembly could, in fact, be used on the bridge.

Coming riverward from the main pier, the rising arch rib intersects the tie girder. This, again, is a location where large forces must be carefully carried. It was decided to carry the rib forces through the joint in a rib section, and build the tie girder to pass by it. At adjacent splices, the rib was reduced in width by two plate thicknesses so it would fit between the webs of the tie girder. Additional web plates were added to the tie girder, and special connections carried the flange strength and material outboard of the tie girder webs, so it was possible to create the opening in the tie girder needed by the rib.

The forces during erection were evaluated in designing the arch, as required by the LRFD Specifications. It was found that several of the shorter verticals were overstressed when erected due to the flexure caused by the combination of shop camber and erection loads. It was decided to pin these several members during erection and then make the final bolted connections after the bridge had been swung.

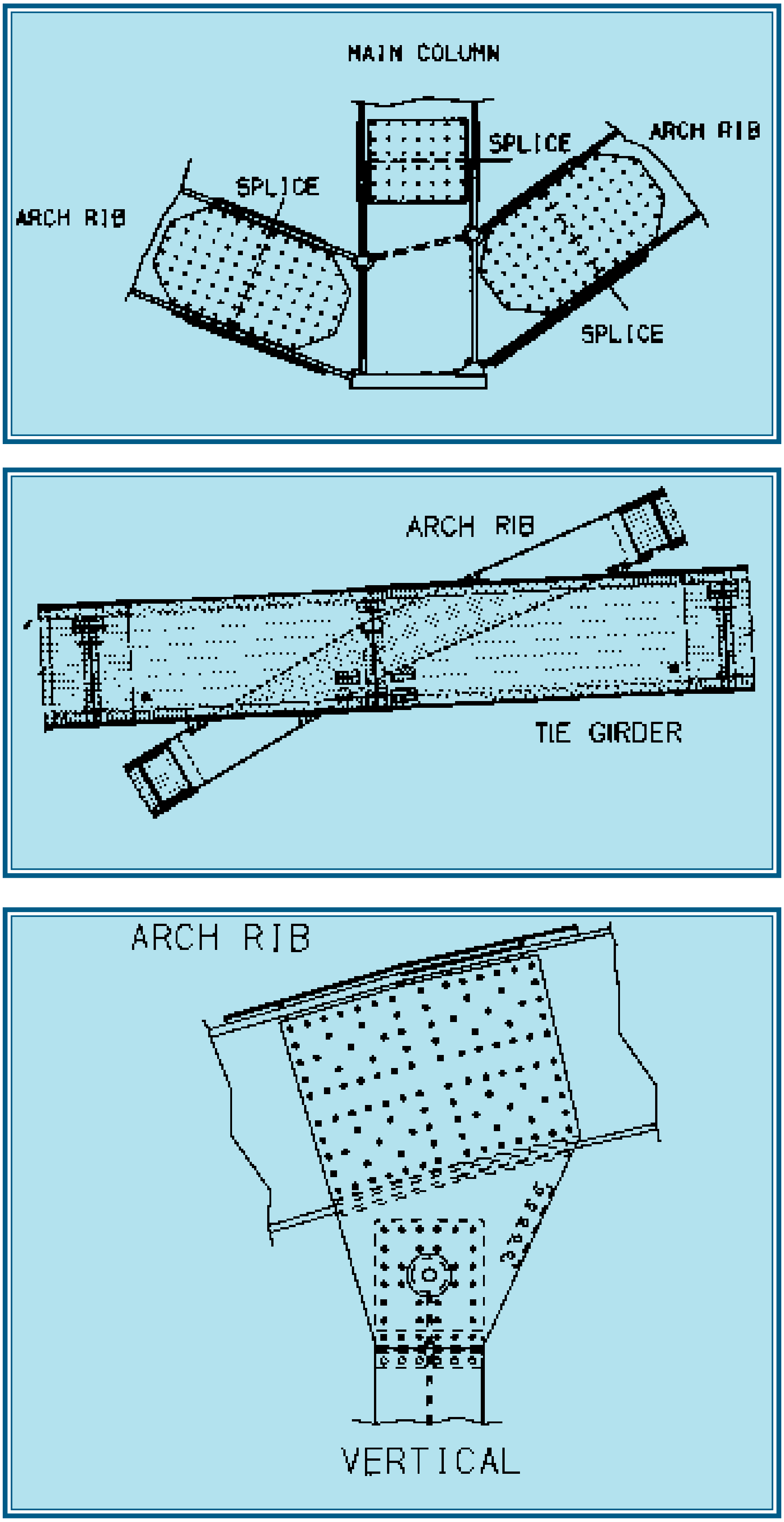
Arch Erection

Erection from the water, which would have been difficult due to the speed of the current, was banned by the Coast Guard. In accordance with the LRFD Specifications, the design plans included a feasible erection procedure. This plan first erected the anchor span using temporary bents and then placed a falsework tower over the main pier. Erection of the main span was accomplished by cantilevering from this point, using stays secured at the anchor pier and passing over the tower to support the river span sections.

Each half of the arch was erected by a contractor from that country, and each elected to use the basic procedure shown in the plans, with minor modifications. To handle the uplift created by the cantilevering special, tie rods were set into the anchor pier footing and for attachment to the tie girder at the point of the stay attachment.

Approaches

Three continuous steel box girder spans are used for the flanking spans placed at each end of the tied arch to provide visual continuity, and beyond



Shown at left, from top to bottom: main pier joint; intersection of rib and tie; and detail of column relief joint.

that the approaches vary in their makeup to suit local conditions. The flanking spans are supported by three box tube girders about 2.1 meters deep. These girders are composite with the concrete deck, and a plane of bracing is provided for the top flanges. The Michigan side has three spans of 61 meters, and the Ontario portion has three spans of 54 meters.

Beyond the flanking spans, the Michigan Approach continues with 1.8 meter deep precast prestressed concrete girders in spans ranging from 28 to 36 meters. The girders are made continuous for live load in those spans where the roadway and cross-section are uniform. The last span is a simple span of steel girders. Near the Michigan Plaza a special crossover ramp is provided to permit traffic to access the proper lanes under specific conditions of operation, and this ramp is framed with composite prestressed concrete and steel girders.

Substructure

All the piers are of reinforced concrete founded on steel 'H' piles driven to rock. The main piers have a column under each of the arches, and are connected at the top by a strut. The remaining are hammerhead piers, except at the crossover and locations adjacent to the Plaza where the bridge widens to make a hammerhead impracticable.

Division of work

The main river crossing and the flanking spans are in a single set of plans and arranged so that each Owner and each country's contractor is responsible for construction to the center of the main river crossing. A separate plan set is prepared for each country's approach for administration by that country's Owner. Thus, the construction of the main bridge and approaches forms three separate contracts: the Main Bridge and Flanking spans; the Michigan Approach, and the Ontario Approach. It became necessary that some portions of the contracts overlap and the Contractors for each approach will continue some portion of the work to the centerline of the river: placing the bridge deck overlay, and installation of light standards and signal devices.

A number of special provisions had been anticipated for the project, as is common for a structure of this magnitude and complexity. After the project had been divided into three separate contracts, for a number of independent contractors working simultaneously on the same bridge performing overlapping operations, it was realized construction coordination could present special problems in the area of specifications. As a result, standalone specifications were prepared for the main bridge and flanking spans, which was the first contract completed, and these were appropriately modified for the subsequent approach contracts. These specifications were prepared in S.I. coordinating the requirements of Michigan and Ontario.

LRFD Specification

The bridge was designed using the *AASHTO LRFD Bridge Design Specifications, S.I. Units, First Edition, 1994*. The completed edition was released shortly before the start of final design began. Specific project Design Criteria, begun during the Study Phase, were developed and shown on the plans. These began by establishing the LRFD as the basis for the design and continued with further definition and refinement, all specific to this project.

The LRFD Specifications require designers to explicitly consider the importance, redundancy and ductility of the structure and its components. These features enter the design process through the load multipliers shown in Table 1.

The loadings and traffic patterns on the existing bridge had been studied by Modjeski and Masters, Inc. previously, and a fairly common condition had been observed—bumper-to-bumper traffic with a high proportion of trucks over the full-length of the main bridge and approaches, all waiting to pass immigration and customs. The experience with this bridge was one of the reasons that the LRFD Specifications contains a 'STRENGTH II' load condition where a special loading, applicable to a specific bridge is used. The special loading condition as selected and included in the Design Criteria consists of loading any two lanes uniformly with an intensity of 24 kN/m centered in each lane with no concurrent load in the third lane or sidewalk, no superimposed concentrated loads, and no impact.

Table 1: Load Multipliers

Component	η	η_D	η_R	η_I
Tied Arch Superstructure	1.10	1.00	1.05	1.05
Tied Arch Piers	1.05	1.00	1.00	1.00
Piles (groups of 8 or more) Approach	1.00	1.00	0.95	1.05
Superstructure & Piers	1.05	1.00	1.00	1.05

One of the early considerations had to do with the singular importance of the tie girder since it is a tension member essential to the entire bridge, and it must have redundancy as described previously.

In accordance with the LRFD requirements, the design included the studies required to develop a satisfactory construction sequence for the tied arch. None of the arch segments carries its load as an arch until the segment is 'closed', or joined with the tie. Until that is achieved, the members are all merely beams requiring falsework and temporary support. The detailed erection sequence is shown in the plans and includes: the staging; the falsework and temporary bracing locations and loadings; deflections; and procedures for making closures of the several segments of the tied arch. The final design and detailing of the permanent members of the tied arch was checked and adjusted, if required, to accommodate the loadings from the construction sequence.

The Design Criteria indicates the requirements for deck replacability, and the plans include staging diagrams for the feasible procedure applicable to each part of the bridge.

SI Units

The S.I. system has been in use in Canada for a number of years, and in the United States many engineers are working toward adopting the S.I. system, which will soon be mandatory for Federally-funded highway projects. The requirement that this bridge be jointly designed by U.S. and Canadian engineers, and be jointly built by U.S. and Canadian contractors was a compelling reason that the plans would be presented in S.I. units.

Some recent projects have been planned so that the field measurements and the office design work would be made in customary English units, and the conversion to S.I. would be a separate step to take place as the final drawings are made, so that the completed drawings contain S.I. units. For the Second Blue Water Bridge project, it was decided that all the engineering work, including measurements, studies, design, and plans would be in S.I. units. The initial survey which set the project monuments and the project coordinate system used S.I. units, as did the following geotechnical and aerodynamic studies and reports.

The timing of this project fell in the awkward interim stage during which the engineering community of the United States was preparing to begin using S.I. units, and there was still some differences of opinion as to the 'standard' way of presenting and using the S.I. units. Therefore, it was important that a consistent procedure or standard be adopted for this project in the beginning, and that this standard be explicitly shown in the plans.

The project plans define the project dimensions and units, as well as indicating some of the conventions used in presenting these values. Project dimensions were given in millimeters (mm), and values over five digits long were written using a space instead of a comma to break the number in clusters of three. Elevations and coordinates were given in meters (m) and stations were given in kilometers (km). Forces were tabulated in kiloNewtons (kN), and mass was given in kilograms (kg). The units for stresses, temperature, and bending moment were given as megaPascals, degrees Celsius, and kiloNewton-meters. A table was provided for conversion between S.I. and customary U.S. values for this project. Since conversion of values where 'weight' is referred to in the customary U.S. system may be somewhat ambiguous, the plans define 'weight,' for this project as being synonymous with mass, to be measured in kilograms or tonnes, where a tonne equals 1000 kilograms.

Preparing for SI

Several important steps required in starting design work in S.I. include familiarization, general re-tooling, and accumulation of resource and availability information. To some extent, these involve repeating the stages engineers have gone through during the years of accumulating experience and expertise.

Tables for converting between S.I. and English units, and general booklets describing the S.I. system are good starting points in familiarization, but the objective is to develop a 'feel' for S.I. units, and this is obtained by actually using S.I. values and quantities.

In the design office and drafting room, re-tooling has been relatively simple and inexpensive. The Architect's scale with subdivisions showing feet and inches in fractions of an inch were replaced with S.I. scales for reading and scaling from drawings. The 1/4-inch grids on the calculation pads were replaced with grids of 5 mm. The CAD computer software was modified for drafting in S.I. instead of feet and inches, and the design software was modified as required for the different units, and the different format required.

Designer's work requires resource information with data and availability information about standard materials such as, steel rolled shapes and plates, reinforcing bars, fasteners, and the like. In the United States documentation on products in the English system have been accumulated over the years. One method of presenting plans in S.I. units is to make a design and selection of materials in English units, and make a soft conversion to S.I. values. In this system, all the English units are multiplied by S.I. conversion factors and used directly. Where possible, it is desired to use a hard conversion which uses rounded S.I. values. Tables, brochures and other information is available from professional and trade organizations, and U.S. manufacturers and suppliers indicating construction materials in hard converted S.I. units.

Working in SI

In proportioning the bridge members, the designers were careful to choose from the published list of plate thicknesses. The manufacturer's published tables of S.I. rolled shapes were at hand for selection of sizes. For the tied arch span, 24 mm dia. H.S. bolt was chosen as a fastener.

While developing details for the bridge contacts with the manufacturers revealed that they have not generally made the changeover to standard S.I. rolled shapes, and, therefore, the S.I. sizes indicated in their publications are generally not, in fact, actually available. For this reason, the rolled shapes used for the stringers and connection angles were dimensioned based on soft conversions of the English equivalent.

A U.S. supplier of S.I.-sized reinforcing was difficult to locate. One major manufacturer explained that they exported a great deal of S.I.-sized reinforcing, but this was all made to the foreign standards where the bar diameter is in even S.I. units. The CRSI standard S.I. sizes, as used in Canada and the United States, produce rounded S.I. areas, but the bar diameters are not in even S.I. units. A supplier was found in Pennsylvania who is furnishing S.I. reinforcing for a new Federal building in Washington, D.C.

High-strength S.I. bolts are not readily available from U.S. manufacturers. In addition, steel fabricators have not yet retooled to accept S.I. fasteners. The contractors requested that the nearest English equivalent bolt (1") be used in lieu of the bolt diameter specified on the plans (24 mm). The Owners approved the use of English equivalent fastener size with the stipulation that all details be reviewed with respect to net section to ensure no components were overstressed with the larger diameter bolts.

Although manufacturers furnish catalogs, brochures and data on S.I. supplies for structural use, there has been very limited requirement for them to-date, and they are either in very short supply or non-existent in the United States. As the need and use of S.I. materials becomes a reality and starts to grow, their availability will increase in response.

Grand Opening

In mid-July 1997, the Second Blue Water Bridge opened with a two-day celebration. By some estimates, as many as 300,000 people walked across the bridge. Several days later the bridge was opened to two-way traffic and the venerable First Blue Water Bridge was taken out of service for rehabilitation. When it returns to service in mid-1999, each bridge will be one way, and the total lane capacity will be increased from one lane each way to three lanes each way.

Joseph E. Prickett, P.E., is a senior associate and John M. Kulicki, P.E., P.Eng., Ph.D., president and chief engineer with Modjeski and Masters, and Brian D. Morgenstern, P.Eng., P.E., is a principal and Roger A. Dorton, P.Eng., Ph.D., is manager of the Ontario office of Buckland & Taylor.

Bridge Details

Total length of bridge:	6,109 feet
Number of spans:	39
Total weight of main span:	20,084 tons
Structural steel in river span:	14,000 tons
Bolts:	350,000
Total design drawings:	760
Shop drawings:	~ 2,000

Project Team

- Owners: Michigan Department of Transportation; The Blue Water Bridge Authority
- Designers: Modjeski and Masters, Inc.; Buckland & Taylor, Ltd.
- Environmental Consultant: Giffels Associates, Ltd.
- General Contractors: PCL/McCarthy Joint Venture
- Fabricators: PDM of Wausau, Wisc., and of Eau Claire, Wisc.; Canron, Inc. (Canron/Midwest)
- Erection: Midwest Constructors; Traylor Brothers
- Painting: Michigan Multi-Coat Shop System (shop coat by Blastetch Corportion and touch-up/joints by Hartman Walsh Paint Co.)

Bearings/Expansion Joints: H.S.C. Canada, Inc.; D.S. Brown