Galvanized Cold-Formed Steel Bridges for Low-Volume Roads

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Typical installations, design practices, a wheel load distribution factor study, structural component tests, and a review of galvanized steel performance data are described for a galvanized cold-formed steel bridge system designed for low-volume roads. A number of these short-span bridges have been designed and built in Ecuador recently. Box sections used as longitudinal girders were shown analytically to provide better lateral distribution of wheel loads on bridge plank decks than conventional wide-flange beams or I-girders. Effective torsional stiffness of the box girders used in the study was determined from laboratory tests on two full-scale, 11.7-m-span, prototype box girders. Flexural tests were also conducted on the prototype girders. One girder was spliced by a field welding procedure and the other by a bolted splice design. Test results confirmed that strength and stiffness can be accurately predicted using state-of-the-art cold-formed steel design technology with some restrictions on geometry and welds. Durability performance data and life-cycle costing analyses on galvanized steel bridges in the United States indicate that the subject bridge system offers a durable, maintenance-free, and economical system when site environmental conditions are suitable for galvanized coatings.

Developing countries are seeking economical, easily erected, maintenance-free and durable bridges for their low-volume roads. Developed countries have similar needs for replacement or rehabilitation of aging low-volume bridges. GangaRao et al. (*1*) recently applied value engineering analysis techniques to low-volume bridges in the United States and concluded that the following parameters, with a weighting factor assigned, are the most important for selecting a low-volume bridge: material cost (23 percent), maintenance aspects (18 percent), durability (16 percent), service life (15 percent), availability (15 percent), and ease of construction (13 percent).

Applications, analysis, and component testing of an all-galvanized cold-formed steel bridge system designed for low-volume roads in Ecuador and other developing countries are described. In these countries, the hot-rolled steel structural shapes commonly used in developed countries for main structural members are not domestically produced and are expensive to import. Consequently, cold-formed sections of more readily available sheet and strip steel are often used for primary members such as longitudinal girders. Durability of the steel is enhanced by hot-dip galvanizing, which is available in many developing countries. Durability performance data and a life cycle cost study from the literature are reviewed.

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ALL-STEEL BRIDGES

For nearly 45 years, an all-steel bridge concept has been used for low-volume road bridges in the United States. The concept is based on the use of economical, mass-produced steel components: corrugated decking (bridge plank), hot-rolled wide-flange beam stringers (main longitudinal members), hot-rolled H-piles or cold-formed pipe piles and sheeting or bridge plank for substructures, and cold-formed guard rail for the safety railing. The individual steel components are relatively lightweight, making heavy lifting equipment unnecessary for erecting purposes, and are easily installed. Normally, except for a welder, no special skilled labor is required to construct these bridges. Except for the stringers and H-piles, the components typically have been hot-dip galvanized. The bridge plank deck is normally surfaced with a bituminous asphalt pavement after installation.

The components of the Ecuador bridge system (Figure 1) are similar to those of the U.S. system except for the primary structural members. Box sections similar to the prototype girder shown in Figure 2 replace the hot-rolled shapes used in U.S. bridges. They are fabricated by welding together two stiffened cold-formed channels. The channels are typically press brake-formed, but could be roll-formed. Shear and bearing stiffeners are welded inside before the channels are welded together with a lap between sections. The height-to-width ratio of the box sections varies, but typically is about 3 for longitudinal girders. Minimum thickness is 4.2 mm. The steel grade is equivalent to ASTM A36 or ASTM A570 Grade 36, depending on thickness. Cover plates are welded to the top and bottom flanges when required for bending strength and stiffness.

A rigorous welding quality control program based on American Welding Society standards for structural steel (D1.1) and sheet steel (D1.3) is used for all shop and field structural welding. All components are hot-dip galvanized following ASTM A123 or ASTM 153 specifications to provide a minimum of 610 g/m² coating weight (86 μm for steel thickness less than 6.35 mm, 99 μm for greater steel thickness). The decision to use galvanized components was influenced by the excellent performance of galvanized bridges in the United States.

Because of galvanizing-tank size limitations, sections are hot-dip galvanized in lengths not exceeding about 6 m. For the longer longitudinal girders, the galvanized coating is removed from the mating ends and the sections are joined by full-penetration groove welds. Zinc-rich paint is used to coat the splice welds. Box girders for bridge lengths to 28 m have been fabricated in this manner. A bolted splice design has

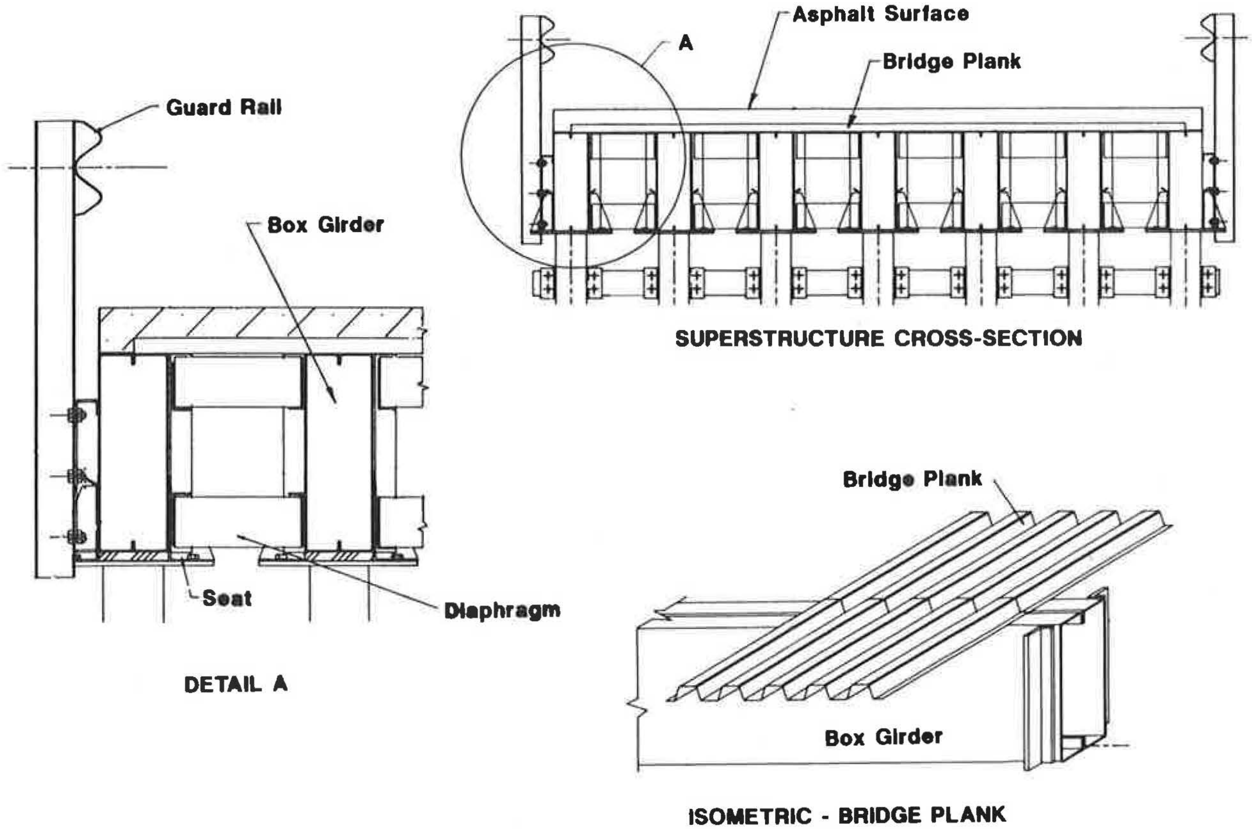


FIGURE 1 Cold-formed steel bridge details.

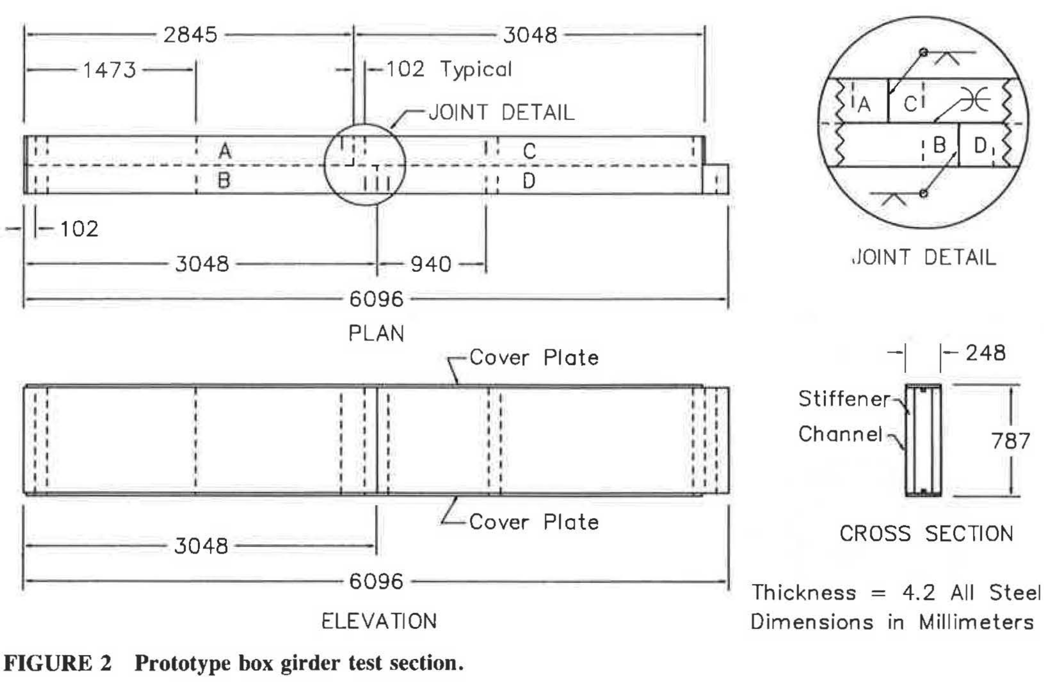


FIGURE 2 Prototype box girder test section.

also been developed and tested for use at sites where field welding is impractical.

Box girders have several distinct advantages over wide flange beams and I-girders in all-steel bridges. Less bracing is required for lateral stability of the compression flange, and lateral distribution of wheel loads is improved because of the high torsional stiffness of box sections. The double webs provide more uniform support for the bridge plank deck, plus more redundancy in the system, and the problem in I-girder bridges of the bridge plank effective span increasing under wheel load because of girder flange rotation is eliminated. The principal disadvantage of the box section is that it is not as efficient in vertical plane bending as the I-section.

GALVANIZED COLD-FORMED STEEL BRIDGE INSTALLATIONS

Ten galvanized cold-formed steel bridges have been locally fabricated and built on low-volume roads in Ecuador since 1987. Brief descriptions of two of the bridges follow.

Figure 3 shows the Colorado Bridge in the state of Chone, built in 1988. This 12.8-m-long bridge is designed to carry two lanes of traffic on a 7.0-m-wide deck supported by 10 longitudinal box girders. The girders are 780 mm deep by 250 mm wide and are fabricated from 4.2-mm-thick steel. The design was made for AASHTO HS20–44 truck loading. The abutment breastwalls are of tie-back anchored sheet piling. The abutment seat beams are cold-formed box sections supported by six hexagonal cold-formed steel columns mounted on a reinforced concrete pedestal footing.

Figure 4 shows the Zapallo Bridge in Chone, built in 1989. A delta truss substructure was used to shorten the effective span of this 26-m-long, 4.3-m-wide, single-lane bridge. The deck is supported by seven continuous box girders. The girders, with a 17.0-m center span, are about 650 mm deep by 230 mm wide and are fabricated from 5.5-mm-thick steel. The design loading was again AASHTO HS20–44. The vertical and inclined members of the 6.0-m-high delta trusses are fabricated box sections; the horizontal members are cold-formed channels. The trusses are mounted on a reinforced concrete pedestal footing. The sheet piling abutment breastwalls are tied back to deadman anchors at several levels.

FIGURE 3 Colorado Bridge, Ecuador.

FIGURE 4 Zapallo Bridge, Ecuador.

EVALUATION OF CURRENT LOW-VOLUME BRIDGE DESIGN PRACTICES

Low-volume bridges in the United States are currently designed using the same AASHTO specifications (*2*) as urban and Interstate highway bridges. Therefore, many low-volume bridge designs are believed to be overly conservative. The AASHTO specifications are also used for the Ecuador bridges, but are supplemented with the cold-formed steel structural design requirements for the American Iron and Steel Institute (AISI) specification (*3*) adjusted to AASHTO factors of safety.

GangaRao and Zelina (*4*) reviewed the AASHTO specifications recently to identify criteria that may be overly conservative or irrelevant for low-volume bridges. In addition to several geometric and functional criteria, they suggested that current fatigue and deflection requirements for structural design are excessively conservative.

For fatigue, they concluded that the current lowest level of AASHTO design cycles (100,000) used for low-volume bridges corresponds to an average daily truck traffic (ADTT) of 125 in one direction for a 50-year service life. Their conclusion was based on an observation that one AASHTO design cycle equals about 23 actual vehicle load cycles. They also concluded that an ADTT of 125 is about 10 times too high for typical U.S. low-volume roads and suggested that fatigue design be neglected entirely for low-volume bridges. However, fatigue checks have been used for the Ecuador bridges because of their lighter-gauge steel design.

For deflection, GangaRao and Zelina suggested that the live-load deflection criterion could be reduced from the current $L/800$ (L = span) limit to about $L/360$. A deflection limit of $L/500$ has been used for the Ecuador bridge design.

WHEEL LOAD DISTRIBUTION FACTORS

Another AASHTO design criterion may be overly conservative with regard to the Ecuador low-volume bridge designs: the wheel load distribution factor (DF) for longitudinal girders. In order to investigate this, an analytical study was undertaken. Finite-element elastic analysis techniques were used to include the structural interaction between the bridge plank deck, the longitudinal box girders, and the crossframes tying the girders together. The ANSYS (*5*) finite-element code was used for the models.

Three-dimensional orthogonal grid models (Figure 5) were developed and analyzed for 12.2- and 24.4-m spans, each with one- and two-lane deck widths. Girder spacing, controlled by the lightest bridge plank allowable span, was set at 780-mm centers. The box girders were modeled as a series of three-dimensional beam elements with an effective torsional stiffness determined by tests described in the following section. For comparison, standard hot-rolled wide-flange beams were modeled in the same manner.

The bridge plank deck was modeled as a series of three-dimensional beam elements also. The element lines were spaced 0.61 m apart, transverse to the girders, and equal to the width of individual bridge planks. At crossing point nodes, the plank elements were linked to the girder elements.

Crossframe bracing effects were simulated by applying displacement boundary conditions at selected locations. Boundary condition locations are shown by arrowheads in Figure 5.

The analysis model girders were designed for AASHTO HS20–44 truck loads. The wheel loads were placed as concentrated loads to produce maximum bending moments in the longitudinal girders.

On the basis of results of the analyses and torsional tests, new DF equations were recommended for bridge-plank-on-box-girder bridges. The recommended equations are compared with the current AASHTO (*2*) equations as follows (S = center-center spacing of girders in meters). Numerical results from the analysis models are compared with both sets of equations in Figure 6.

	Distribution Factor	
No. of Traffic Lanes	*Current AASHTO*	*Box Girder Recommendation*
1	$S/1.676$	$S/1.753$
2 or more	$S/1.372$	$S/1.600$

The recommended DF equations reduce the single-girder design load by about 4 to 14 percent. In order to use the equations, crossframe or other external diaphragm bracing must be used between box girders at end reactions and at intermediate points such that the spacing does not exceed 10 m. Also, internal plate diaphragms must be used inside the girders at the end reactions, and the bridge plank must be continuous across the deck width.

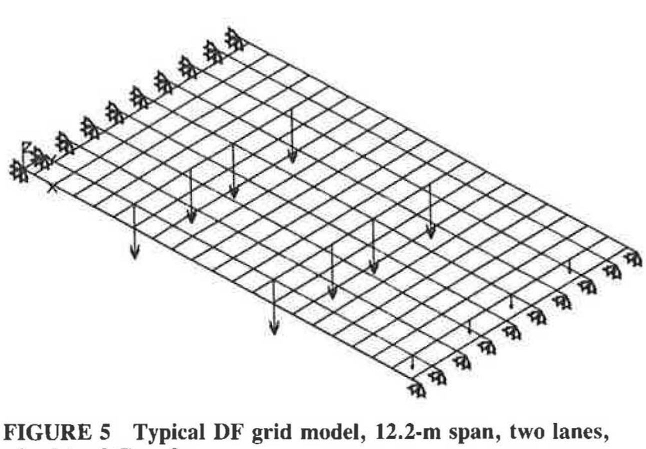


FIGURE 5 Typical DF grid model, 12.2-m span, two lanes, wheel load Case 2.

BOX GIRDER TORSIONAL AND FLEXURAL TESTS

Because no prior applications of cold-formed box sections to bridge girders were known, a testing program was developed to confirm design assumptions. The program included the following:

1. Fabrication of two prototype 12-m-long box girders—one with a bolted splice and one with a field-welded splice at midlength;
2. Torsional loading tests to determine effective torsional stiffness of the girders under several bracing conditions; and
3. Flexural loading tests to check flexural stiffness and ultimate moment capacity.

The tests were conducted at the Armco Research and Technology laboratory, Middletown, Ohio.

Four half-length girders (Figure 2) were designed and fabricated from 4.2-mm ASTM A569 steel with 248-MPa yield point. The box section was proportioned with the cover plate width-to-thickness ratio set at the maximum of 60 permitted by the AISI specifications. The ratio of web depth to thickness was set at 180, greater than the 170 maximum permitted by AASHTO but well below the maximum of 300 permitted by AISI for reinforced webs. The web stiffeners were sized according to AISI transverse stiffener criteria; the spacing was calculated to meet test requirements and was greater than that used for production girders.

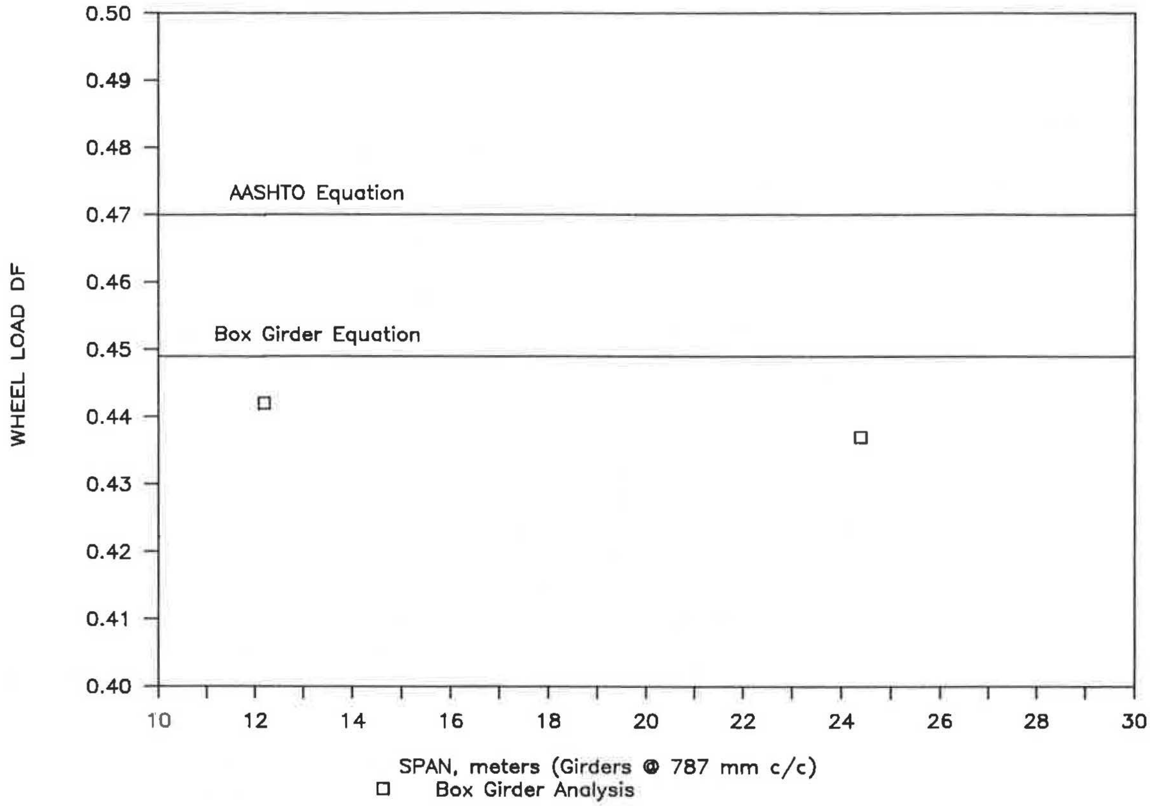
Two of the half-girders were joined with a prototype bolted splice design (Figure 7). Unlike a bolted splice for an I-girder, this splice offers a clear top flange for installing a bridge plank deck. The test splice was designed to provide bending and shear capacities of 90 and 75 percent, respectively, of the box net section capacities. The other two half-girders were joined by field welding procedures using stick electrodes (Figure 2, joint detail).

A finite-element analysis model of the welded splice girder was developed before testing to predict deflections and stresses for both the torsional and flexural loading tests. The ANSYS code was again used to develop the model (Figure 8) from plate elements.

TORSIONAL TESTS

For torsional loading, the test girders were set up on a structural testing floor, as shown in Figure 9. The girders were braced by crossframes fabricated for hot-rolled steel angles. A torsional loading frame (Figure 10) using two opposing-action hydraulic jacks was designed and fabricated to fit tightly around each girder. Dial gauges accurate to 0.025 mm were mounted at the end crossframes and on the load frame to measure horizontal deflections from which rotations were calculated.

Four bracing cases were tested. Case 1 had only the end crossframes in place. The midspan crossframe was added for Case 2. Because girder end warping distortions were observed in these two cases, internal plate diaphragms were welded to the bearing stiffeners at each girder end for Case 3. The midspan crossframe was removed for Case 4. Each case was



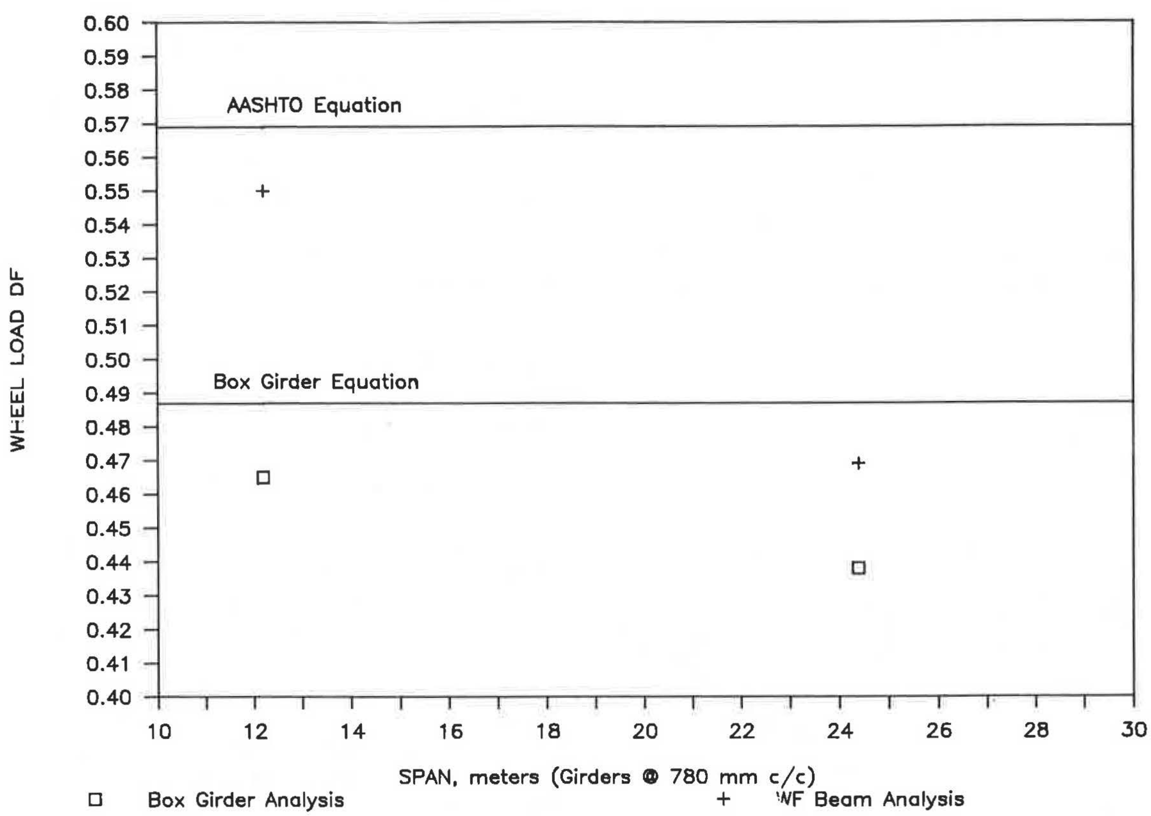


FIGURE 6 Wheel load distribution factors: (*top*) single lane and (*bottom*) two or more lanes.

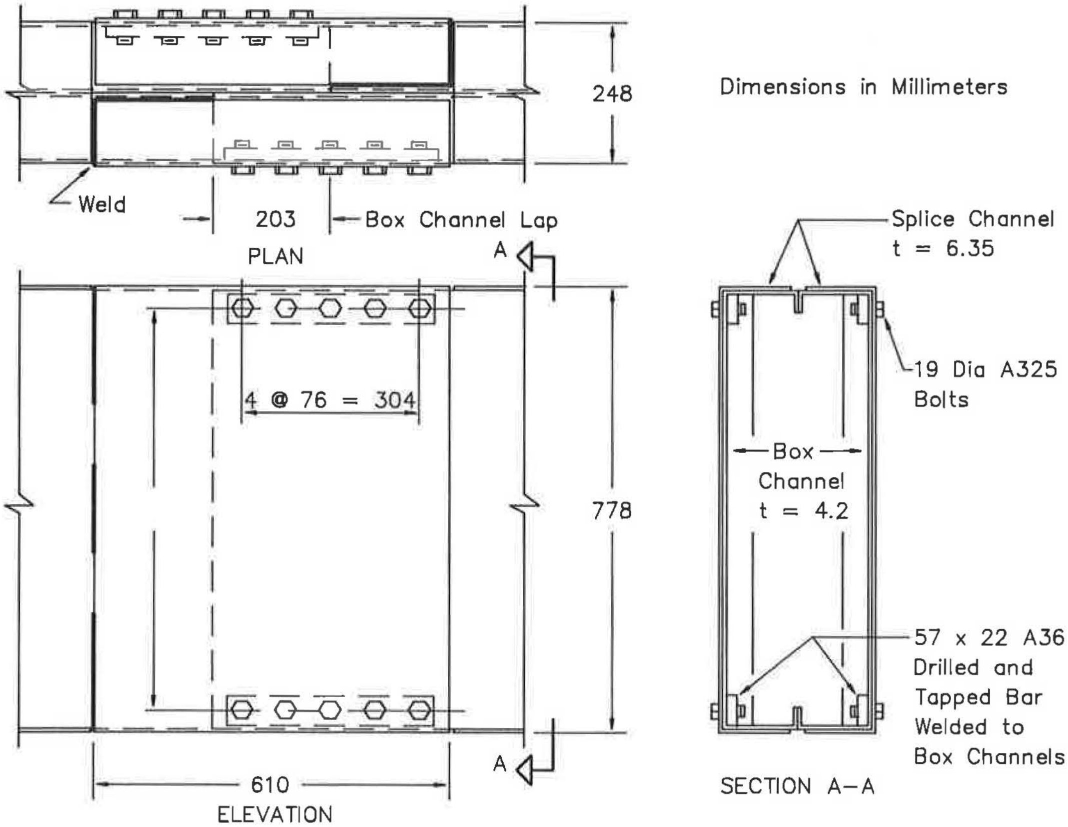


FIGURE 7 Prototype bolted splice.

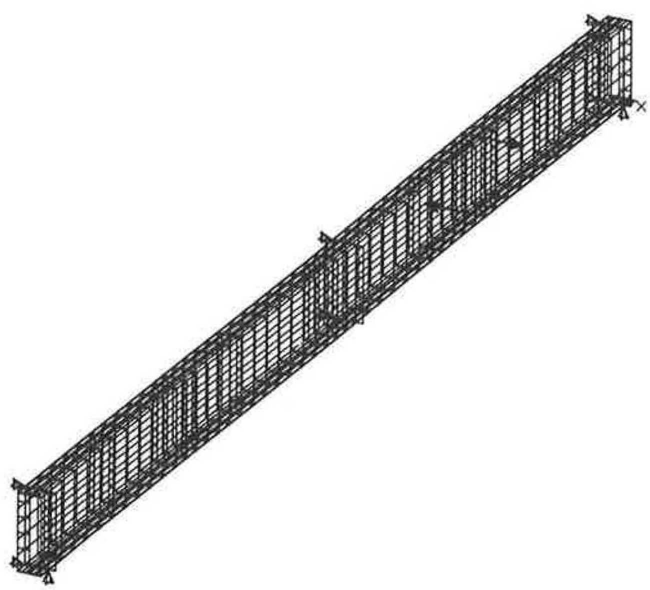


FIGURE 8 Test girder finite-element model. Torsional loading at north quarter-point end and midspan braces.

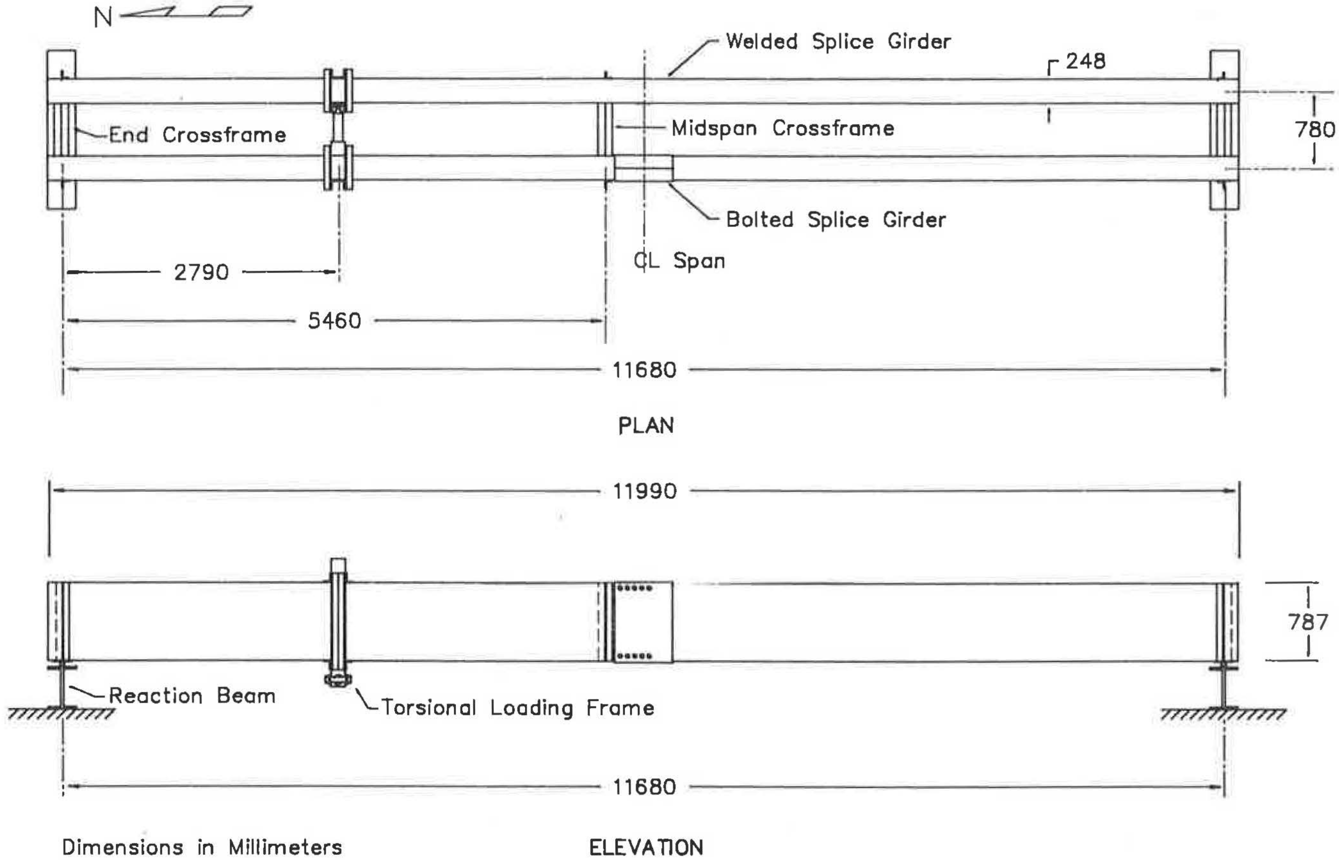


FIGURE 9 Torsional test setup.

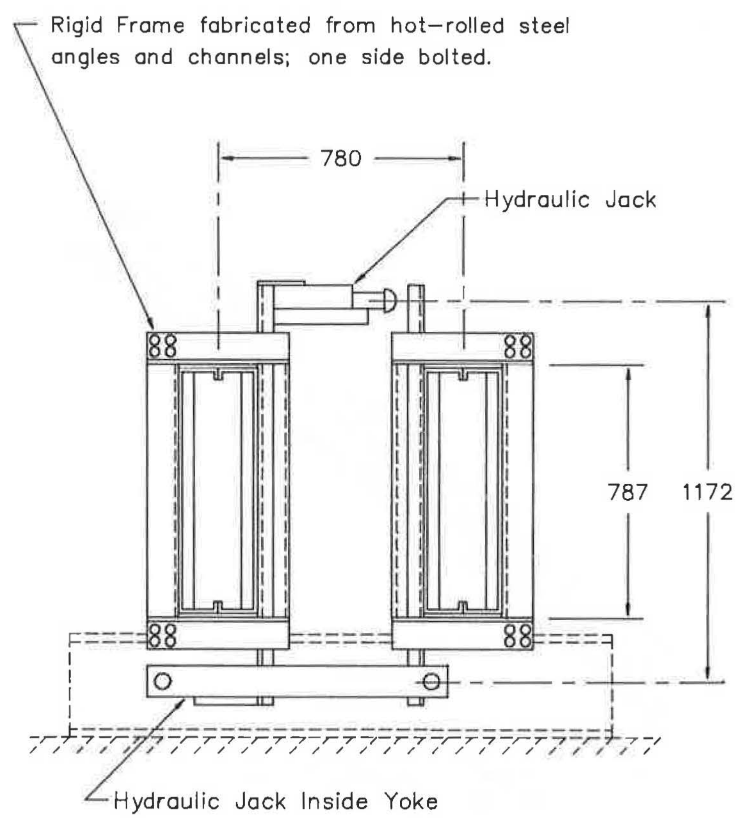


FIGURE 10 Torsional loading frame, sectional view (dimensions in millimeters).

loaded to approximately the maximum resisting torsional moment observed in the DF analysis models.

Net rotation at the land point is plotted as a function of the applied torsional moment (torque) in Figure 11. The internal diaphragms were effective in reducing warping, as can be seen by comparing Case 1 with Case 4 and Case 2 with Case 3 data in Figure 11. The diaphragms are used in all production box girders.

The average effective St. Venant torsional constants for the girders were backcalculated as 62 and 58 percent of theoretical for Cases 3 and 4, respectively. The 58 percent level was used in the distribution factor study to derive the box girder DF equations.

FLEXURAL TESTS

For flexural loading, the girders were set up individually on the structural testing floor (Figure 12). Loads were applied equally at the one-third span points using an MTS closed-loop, two-channel hydraulic test system (Figure 13). The test girder compression flange was braced laterally at the load points by knife-edge channel braces bolted to the load frames. Linear electrical transducers were used to measure vertical deflections to the nearest 0.025 mm at the loadpoints and at midspan. Loads were held constant while the transducer readings were taken.

Midspan deflection versus the ratio of applied load to ultimate load is plotted for both test girders and compared with the ANSYS model predictions, as shown in Figure 14. Flexural stiffness calculated from these plots was 95 and 87 percent of the ANSYS prediction for the welded and bolted splice girders, respectively. Bolt slip accounted for the lower stiffness of the bearing-type bolted-splice design. A reduction factor is applied to the effective box section moment of inertia for deflection calculations when bolted splices are used in production bridges.

The ultimate moment capacity of both test girders was controlled by the box section compression cover plate buckling strength. The actual yield point of the bolted splice channel steel was higher than anticipated, so the splice moment capacity was actually greater than that of the box section. Ultimate moment capacity was reached at 1.76 and 1.77 times actual allowable moment capacity for the welded and bolted splice girders, respectively, with the compression cover plate continuously welded along its longitudinal edges. Because this factor of safety level was slightly under the AASHTO nominal of 1.82, design criteria restricting (a) the cover plate maximum width-to-thickness ratio, (b) the spacing of intermittent cover plate welds, and (c) the maximum depth-to-thickness ratio of the webs were implemented to ensure a minimum factor of safety of 1.82 for production girders.

PERFORMANCE OF GALVANIZED STEEL IN BRIDGES

Although galvanizing has been in commercial use for over 200 years, it has been specified for entire bridge structures in North America only since the early 1960s. The first U.S. bridge with every structural member, fastener, and other steel components hot-dip galvanized was the Stearns Bayou bridge in Ottawa County, Michigan, constructed in 1966 (*6*). Since that time, several hundred all-galvanized bridges have been built in the United States.

The Stearns Bayou bridge was inspected in 1986 (*7*). It is in a rural atmosphere, is about 2 m above fresh water for most of its 128-m length, and is subject to winter salting. After 20 years of service, the beams and diaphragms were free of rust or stain. Coating thicknesses ranged from 76 to 140 μm (112 μm average). The bearing pads exhibited stain on areas subject to standing water and road deicing salts but still had coating thicknesses averaging 89 μm.

A monitoring program (*8*) comparing side-by-side galvanized and painted bridges in a suburban environment with deicing salt exposure has been in progress since 1970 in Indiana. Zinc-rich paint was used on some areas of the galvanized bridge. The last report (1987) indicated that both the galvanized and zinc-rich paint coatings were in good condition after 17 years of service. Minimum galvanized coating thickness was 94 μm. White rust (zinc oxidation) was showing on diaphragms adjacent to expansion joints and on bearing components. The painted bridge was recoated after 14 years and at 17 years demonstrated corrosion on diaphragms adjacent to expansion joints and on bearing components.

Atmospheric exposure test data on zinc and galvanized steel panels have been published for a number of locations in the United States, United Kingdom, and the Panama Canal Zone (*9*). These data indicate that the weight loss of zinc plates is linear with time and also correlates well with the weight loss of galvanized panels. Thus the performance of a hot-dip galvanized bridge at a particular site may be predicted by exposing weighed zinc or galvanized steel plates at the site for 2 to 3 years.

The following is a partial list of published estimates of years of coating life (years to rusting) for an 86-μm galvanized coating thickness (*9*).

Location	*Environment*	*Zinc Corrosion Rate (μm/yr)*	*Coating Life (yr)*
Miraflores, Panama Canal Zone	Mild marine	1.135	76.1
Limon Bay, Panama Canal Zone	Marine	2.654	32.5
Daytona Beach, Fla.	Marine	1.996	43.3
Brazos River, Tex.	Industrial marine	1.839	47.0
Middletown, Ohio	Semi-industrial	1.224	70.5
Phoenix, Ariz.	Rural desert	0.295	293

Galvanized coating provides both barrier and cathodic protection to the steel. The zinc provides sacrificial (cathodic) protection to the base metal when it is exposed by scratching or gouging. An experiment that demonstrated this was reported by Coburn (*9*). A galvanized steel plate was scored to the base metal with varying line widths and exposed to an industrial environment for 56 months. Visual inspection revealed no red rust in exposed widths up to 1.6 mm. Some surface rusting was demonstrated at an exposed width of 12.7 mm.

LIFE CYCLE COSTING OF GALVANIZED BRIDGES

Life cycle costs should be considered when a coating is chosen for steel bridges. The cost of galvanizing must be related to field performance to determine life cycle costs.

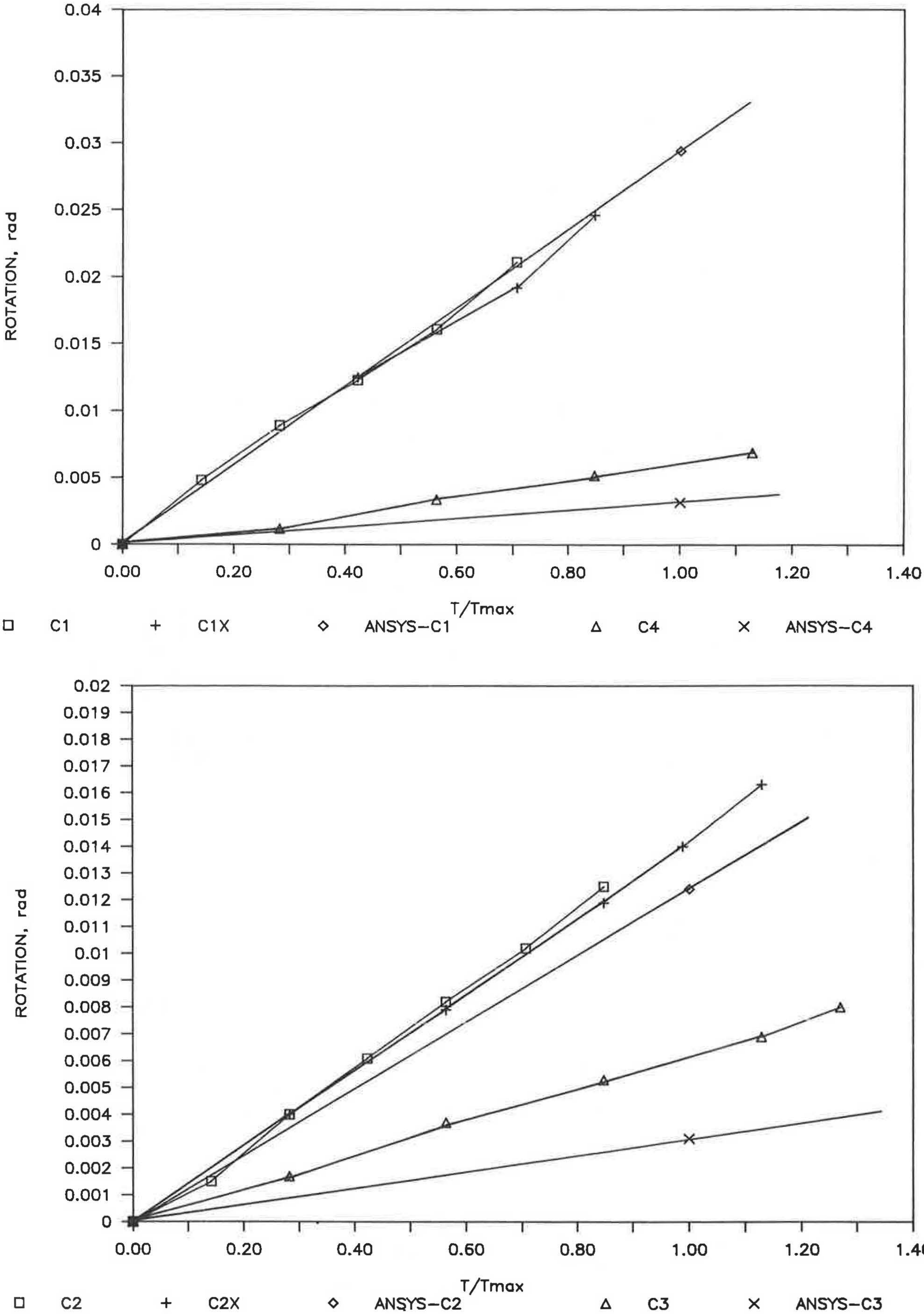


FIGURE 11 Torsional test results: (*top*) end bracing only and (*bottom*) end and midspan bracing.

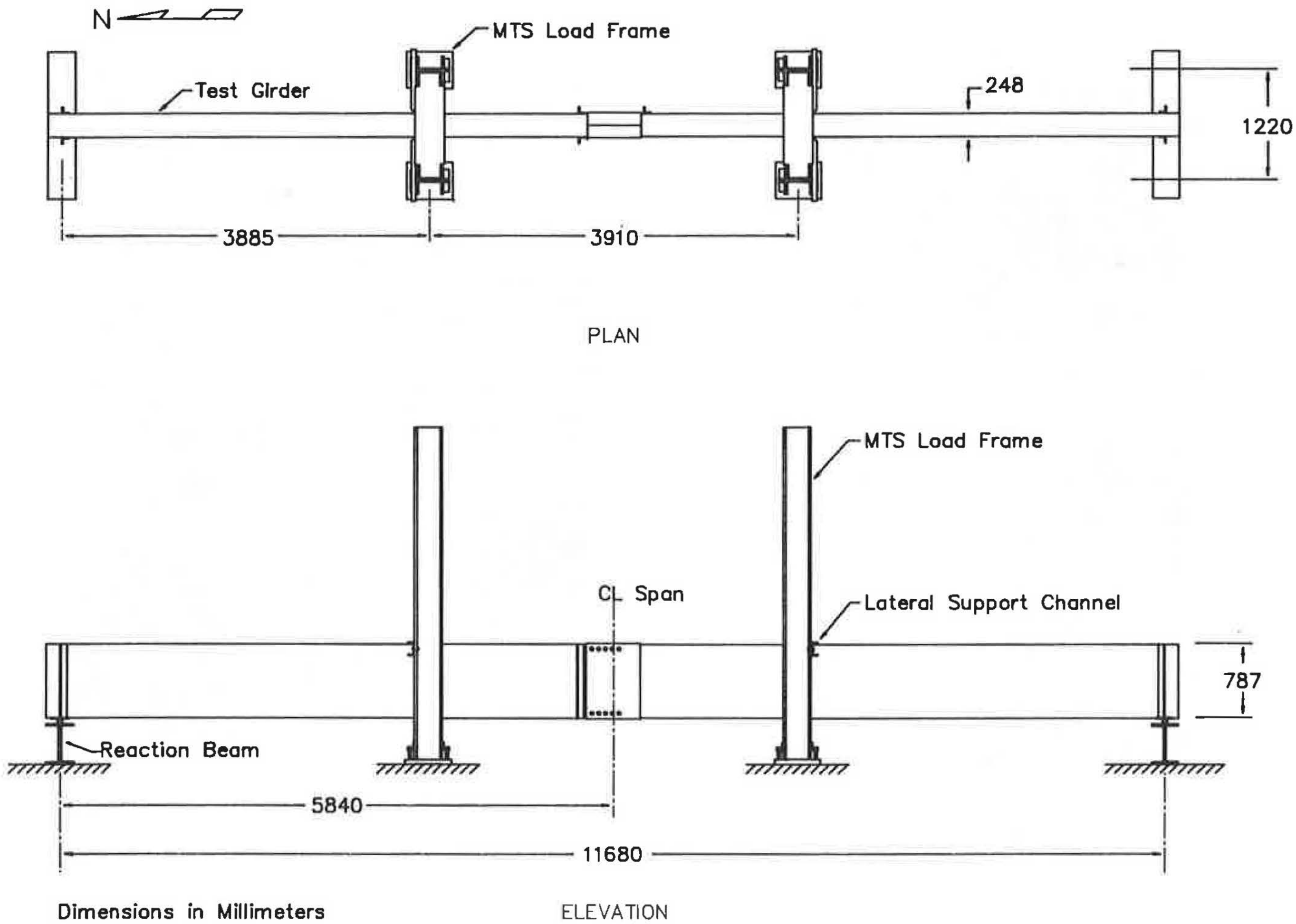


FIGURE 12 Flexural test setup.

A detailed discussion of life cycle costing is provided by Coburn (*9*).

The results of a study on the Stearns Bayou bridge show that when the bridge was built in 1966, the total galvanizing cost was $8,750 (*6*). The painting estimate was $8,600 including blast and shop cleaning plus field painting. Assuming a liberal 20-year service life for the paint and a 5 percent annual inflation rate, the cost of painting was estimated at $17,000 after the first 20 years, $46,000 after 40 years, and $322,000 after 80 years. The galvanized coating life was estimated at about 80 years (*7*). The additional $150 galvanizing cost resulted in a 27 percent annual return on investment by postponing recoating costs.

CONCLUSIONS

Typical installations, design practices, wheel load distribution factors, structural component tests, and galvanized steel performance data have been described for a new galvanized cold-formed steel bridge system. A number of these short-span bridges have been designed and built recently on low-volume roads in Ecuador.

Key structural components of the system are the cold-formed steel box sections used as main longitudinal girders. These provide greater redundancy and were shown by finite-element analyses to provide better wheel load distribution than conventional wide-flange beam or I-girders. Full-scale structural tests confirmed that strength and stiffness of the box girders can be accurately predicted using state-of-the-art cold-formed steel design technology with some restrictions on geometry and welds.

Durability performance data and life cycle costing analyses on galvanized steel bridges in the United States indicate that the Ecuadorian bridge design offers a durable, maintenance-free, and economical system when site environmental conditions are suitable for galvanized coatings.

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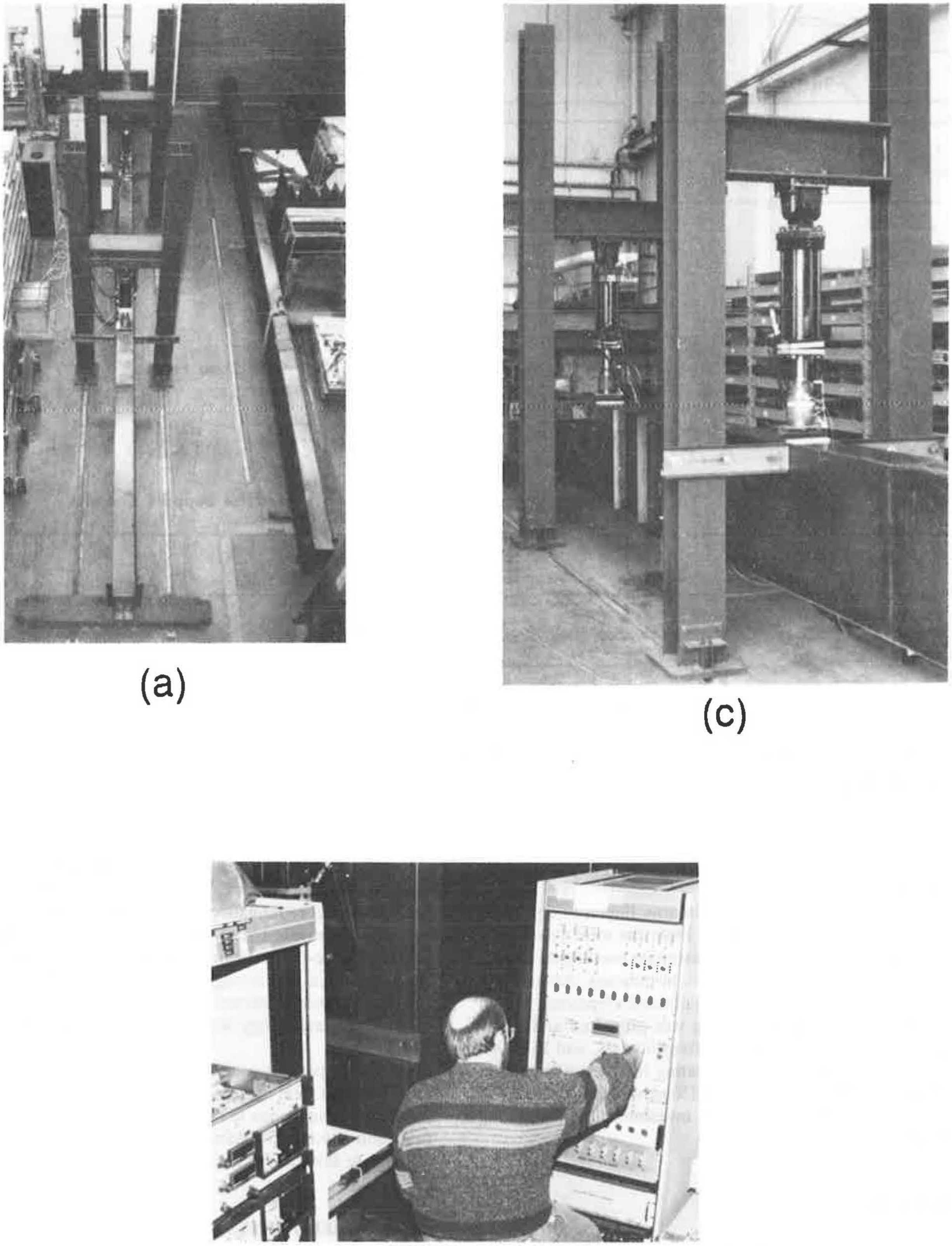

FIGURE 13 Flexural test equipment: (a) view from above, looking south; (b) left—load control unit, right—deflection read-out system; and (c) load reaction frames, actuators (jacks), and lateral braces (looking north).

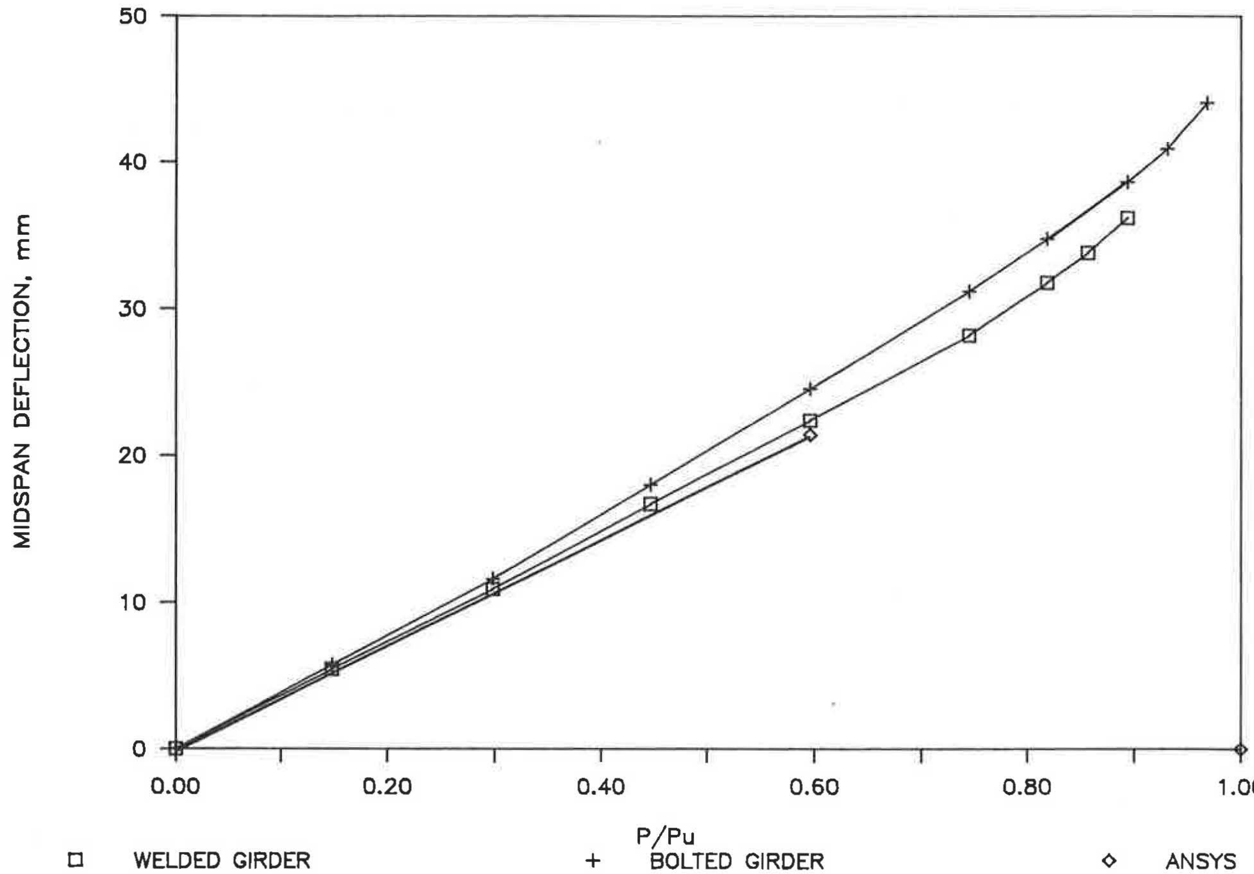

FIGURE 14 Flexural test results for prototype cold-formed box girders compared with ANSYS model predictions.

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