Autostress: Tennessee’s Experience

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Tennessee has embraced the concept of autostress design and has two structures in service currently with two other structures under design. Confidence has been established in the structural adequacy of autostress through the experiences of the two in-service bridges and the demonstrated toughness of the Great River Road Bridge while under severe circumstances. Although autostress-designed rolled beams produce an economical steel bridge through reduction in materials and fabrication costs, they face strong competition from precast prestressed beams except where weight, shallow depth, and compact shipping lengths are controlling factors.

Tennessee’s experience with structural steel usage has generally mirrored national trends over the past four decades. In the 1950s and early 1960s, structural steel held a stable competitive position in the bridge construction market. In span ranges of 60 to 80 ft it could compete with cast-in-place or precast prestressed concrete. In greater span lengths it could compete with cast-in-place or precast prestressed concrete. In the later 1960s and early 1970s, steel began to lose ground on several fronts. Antiquated plants and escalating wages reduced output efficiency at the mills and in the fabricating plants of the steel industry. Meanwhile, new prestressed concrete plants, requiring far less capital outlay and using nonunion labor, were springing up in strategic locations near the concentrations of construction activity. Pressured girders could literally be poured, cured, stressed, and stockpiled within weeks after the issuance of a work order. Further, on-the-spot partial payment for stockpiled materials for the completed beams left the fabricators in a highly solvent position with virtually no cash flow problems. Reinforced hollow box girders replaced tee beams as longer span requirements and heavier loads were introduced. Because the preponderance of construction was on new location, the need to avoid falsework was diminished. Again, as with prestressed concrete, few skilled crafts were required and material payments to contractors were almost concurrent with placement. Soon steel was relegated to spans of more than 120 ft and a sizable share of the market was lost.

The steel industry’s efforts to promote more efficient steel use through load factor design were met stride for stride by the concrete industry so that, to date, steel has made no significant inroads toward recapturing the short-to-medium-span range market.

The Tennessee Department of Transportation (DOT), for example, has on several occasions prepared alternate plans for load-factor-designed, steel plate girders with spans in the 130- to 150-ft range, to bid against precast prestressed box beams in the 110- to 120-ft range. Yet steel has scored no victories, even when the alternatives were for approach spans of large river crossings where the main spans were constructed in steel.

What was and is needed for steel to compete in a wider market is a nonlabor-intensive, quickly fabricated girder that is virtually free of fatigue-generating details that reduce its pound-for-pound efficiency. Autostress design goes a long way toward fitting this need. Now it is possible to design a non-cover plated rolled shape girder using more economical sizes that offer significant savings to the customer. Publication of the American Association of State Highway and Transportation Officials (AASHTO) Guide Specification for Alternate Load-Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections (I) in 1985 could well be a watershed date in the history of the steel industry.

**TENNESSEE’S EXPERIENCE**

Since adoption of the Guide specifications, Tennessee has produced three Autostress designs and is currently working on a fourth. All of these designs have been rolled beams. The Great River Road Bridge over the Obion River in extreme West Tennessee was the initial design attempt, and represents the longest structure (Figure 1). Composed of two end spans of 100 ft and seven interior spans at 112 ft, the total length was 984 ft. The cross-section (Figure 2) is composed of six W36 × 182 A-572 rolled beams spaced on 8-ft, 4-in. centers supporting a 48-ft roadway. Diaphragms at interior supports and at intermediate locations are composed of MC 18 × 42.7 A-36 Channels. Laminated elastomeric pads are used throughout for both fixed and expansion bearings. The bridge contains 25.9 lb of steel/ft².

Unfortunately this will be the second bridge of similar design constructed at the site. Construction of the original bridge (Figure 3) begun in September 1987 was nearing completion in April 1988. During that month, as a result of an ongoing historical period of drought in the Mississippi Valley, the Obion River dropped in elevation from 256 to 233. In May 1988 the department discovered that Pier 3 had begun migration toward the River, causing a sag in the deck between Piers 2 and 4. Subsequently, installed inclinometers in the banks suggested a deep-seated slide was occurring approximately 45 ft below the banks at the interface between the overlying silts and a dense sand lens. Between May and June, the Mississippi River at the mouth of the Obion River, 1 mi away, dropped an additional 10 ft to elevation 223. Long tension cracks formed in the Obion’s banks all along the river for several miles. Removal of 25 ft of overburden temporarily checked the slide. By December 1988 the Mississippi had...
TOTAL LENGTH OF BRIDGE = 984'-0"

FIGURE 1  Great River Road over Obion River.

FIGURE 2  Typical section of Great River Road over Obion River.

dropped to an all-time low of elevation 219. The banks subsequently began moving again. Finding no method of stabilization that could be undertaken while the bridge remained, it was decided to demolish and reconstruct it.

In the summer of 1989, before demolition, Piers 1, 2, and 3 were all migrating toward the River. Pier 3 had migrated out of plumb some 12 ft, was tilting transversely, and had settled more than 4 ft. For safety, Pier 3 was pulled down. Despite the loss of support at Pier 3 the autostress-designed superstructure, which now spanned a distance of 224 ft, refused to buckle at the adjacent supports (Figure 4). The estimated dead load moment at Piers 2 and 4 exceeded the original total design moment by a factor of 1.6, attesting to the reserve capacity of Autostress designs.

Big Opossum Creek Road over the Buffalo River, which is of a later design, is the Tennessee DOT’s first autostress-designed bridge to be completed. This is a four-span continuous structure 90/111/111/90 with a total length of 402-ft (Figure 5). The cross section is composed of three W36 x 170 A572 beams, spaced on 8-ft centers supporting a 22-ft roadway (Figure 6). Beams at Abutment No. 1, which is keyed into rock, are supported on laminated elastomeric expansion bearings. All piers are equipped with fixed, ½-in. 70 durometer neoprene pads. The beams are integral with Abutment No. 2.

The Big Opossum Bridge has the following statistics:

Structural steel/ft²: 22.51 lb/ft²
Cost of steel in place and painted: $0.61/lb
Superstructure cost = $25.65/ft²
Total cost = $35.88/ft²

When the Tennessee DOT was confronted with the challenge to bridge State Route 42 over the Wolf River (Figures 7, 8), it turned again to Autostress design techniques. The bridge is an integral part of the improved road between Byrdstown, Tennessee, and the Kentucky state line, 3½ mi northeast.

FIGURE 4  Great River Road/Obion River after removal of Pier 3.

FIGURE 5  Big Opossum Road/Buffalo River.
The construction of an economical bridge in the hard-to-reach location presented difficult problems. The long, winding roads to the site demanded that a design would have to use components that could be shipped in manageable pieces. The erection scheme and the movement of lightweight erection equipment to the site demanded the designers' careful attention.

Preliminary design investigations included (a) precast-prestressed concrete box beams, (b) welded steel plate girders designed by load factor design (LFD), (c) cover-plated rolled structural steel beams designed by LFD, and (d) rolled structural steel beams designed by the Autostress design procedures. The answer to the problem was a four-span rolled structural steel beam bridge designed by Autostress procedures. This design has six continuous lines of W36 rolled structural steel beams with a composite concrete deck slab. The bridge has spans of 101-128-128-101 ft. The two-lane roadway is 44 ft wide and is some 87 ft above the flow line of the river. Unlike the prismatic beams used for the Great River Road and the Big Opossum Road bridges, the superstructure's beams in the positive moment areas are W36 x 170 sections, and sections over the piers are W36 x 194 (Figure 8-11). All of the W36 material conforms to AASHTO M223 (ASTM A572, Grade 50) material. Because Autostress procedures were used, cover plates were not needed over the pier sections. The shipping requirements and the constructability of the bridge were managed by keeping the maximum length of a fabricated section to 72 ft and maximum lifting weight to only 12,240 lb (see the Appendix for details and sample calculations of this bridge).

The Tennessee DOT achieved an economical solution by using Autostress design procedures. The Wolf River crossing represents the longest spans yet built in Tennessee using rolled sections. The structure contains only 25.3 lb of structural steel/ft² of deck area. The bridge was bid in place and painted, for 58¢/lb. The superstructure bid breakdown cost was $26.52/ft² of deck area and the total bridge cost was $40.76/ft².

COST COMPARISONS

The Autostress design was a clear choice in the case of the State Route 42 crossing. The precast-prestressed concrete box girder preliminary investigation indicated an 11 percent greater cost. In addition, the concrete system involved sections 114 ft in length and weighing 79,000 lb. The Autostress solution compared to the welded steel plate girder scheme designed with LFD indicated a potential savings of 30 percent in girder costs. The cover-plated rolled structural steel sections designed by LFD were approximately 10 percent to 12 percent heavier in comparison to the Autostress design. In addition, the cover plates would have increased fabrication costs.

Cost comparisons, in the absence of actual head-to-head bidding of alternative structure types, are tricky at best. However, comparisons between the State Route 42 Autostress design and a load factor-designed welded plate girder present a good opportunity to examine differences between it and another Tennessee bridge design, Pleasant Ridge Road over State Route 145.

The Pleasant Ridge Road structure is 452-ft long with spans of 101/125/125/101 (Figure 12). The bridge is composed of three continuous welded-plate girders, with 54-in.-deep webs spaced 8-ft on centers and supporting an 8-in.-thick composite deck. The maximum lift of girder pieces was about 80 ft. The bridge, with a roadway width of 22 ft, was designed by the load factor method and also contained 25.3 lb of structural steel/ft² of
deck area. Let in August 1986, the cost of the structural steel was 98¢/lb, in place and painted. Closer analysis of all bids for the two bridges shows that the average price/lb of steel for the Pleasant Ridge structure was 95¢ and for the State Route 42 structure was 65¢; 32 percent less cost for the Autostress design. Coincidentally, the two projects, with nine and seven bidders, respectively, had three common bidders. The average price/lb for these common bidders was 93¢ for the Pleasant Ridge structure and 68¢ for the State Route 42 structure, or 27 percent less for the Autostress design. It would be fair to conclude that Autostress-designed rolled beam can be anticipated to cost about 30 percent less than a load factor-designed welded plate girder. The biggest portion of the savings is obviously in fabrication costs.

CAN AUTOSTRESS STEEL GIRDERS COMPETE AGAINST PRESTRESSED CONCRETE BEAMS?

To date, scheduling and other factors have not allowed Tennessee to prepare alternate plans to allow direct bidding competition between Autostress-designed rolled beams and precast prestressed concrete beams of the same span. Previous load factor-designed rolled beams have not been successful against precast prestressed concrete, even when bid as approach spans to larger steel main-span units.

The Tennessee DOT has also begun to use precast prestressed concrete bulb-tee girders with span capacity up to 140 ft. Although a definite price trend has not been established, it is estimated that the bulb-tee beams on about 8- to 9-ft centers for 125-ft spans will cost approximately $120 to $130/linear ft. Comparably, this infers that the girders used on the State Route 42 bridge would have to cost no more than 62¢ to 66¢/lb, in-place, to compete. At current prices, the same steel rolled beams are estimated to be 72¢ to 74¢/lb. The conclusion, therefore, is that today the Autostress beams will be competitive only where the complexities of the site dictate light girder sections for erection or shorter lengths for transportability. Other economical uses of Autostress designs can be made when an increase in the cost of steel can be offset by a reduction in roadway approach cost because of shallower superstructure. This reduction depends on site conditions.
Tennessee has yet to produce an Autostress-designed welded plate girder. However this will be a future consideration in its search for economical structures to fit every need.

**SUMMARY**

Tennessee has embraced the concept of Autostress design and has two structures in service currently with two other structures under design. Confidence has been established in the structural adequacy of Autostress through the experiences of the two in-service bridges and the demonstrated toughness of the Great River Road Bridge while under severe circumstances.

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**REFERENCE**


**APPENDIX**

*State Route 42 Over Wolfe River, Picket County*

**ROLLED BEAM AUTOSTRESS DESIGN**

**TYPICAL X-SECTION**

![Diagram of typical X-section](image)

**DESIGN FEATURES**

- **FOUR SPAN:** 101'-128'-128'-101'
- **COMPOSITE DESIGN FULL LENGTH**
- **STRUCTURAL STEEL:** Beams - Grade 50
  Diaphragms and Stiffeners - Grade 36
- **CONCRETE:** $f'c = 3000$ PSI
- **REBAR:** $F_y = 60$ KSI
- **LOADING:** HS20-44
- **COMPOSITE DEAD LOAD:** Rails - 453 PLF Each
  Future Overlay - 27 PSF
- **LLDF = 1.200** **IMPACT = 1.221**

**SECTION CAPACITIES**

**NEGATIVE MOMENT SECTION - W36x194**

- $M_y = 3157$ K-FT
- $M_p = 3999$ K-FT

**POSITIVE MOMENT SECTION - W36x170**

- $M_y = 3536$ K-FT for $n = 10$
- $M_y = 3171$ K-FT for $n = 30$
- $M_p = 4714$ K-FT

**COMPACT SECTION REQUIREMENTS WERE MET FOR BOTH GIRDER SECTIONS.**
DESIGN MOMENTS (from continuous beam program)

<table>
<thead>
<tr>
<th>SPAN POINT</th>
<th>1.4 (4.6)</th>
<th>2.0 (4.0)</th>
<th>2.5 (3.5)</th>
<th>3.0</th>
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<tr>
<td>NON-COMPOSITE DEAD LOAD</td>
<td>649</td>
<td>-1315</td>
<td>628</td>
<td>-1367</td>
</tr>
<tr>
<td>COMPOSITE DEAD LOAD</td>
<td>248</td>
<td>-429</td>
<td>259</td>
<td>-459</td>
</tr>
<tr>
<td>LL+I</td>
<td>957(T)</td>
<td>-837(L)</td>
<td>1019(T)</td>
<td>-916(L)</td>
</tr>
<tr>
<td>SERVICE LOAD DL + (LL+I)</td>
<td>1854</td>
<td>-2581</td>
<td>1906</td>
<td>-2742</td>
</tr>
<tr>
<td>OVERLOAD DL + (5/3)LL+I</td>
<td>2492</td>
<td>-3139</td>
<td>2585</td>
<td>-3353</td>
</tr>
<tr>
<td>MAX DESIGN LOAD 1.3(DL + (5/3)LL+I)</td>
<td>3240</td>
<td>-4080</td>
<td>3360</td>
<td>-4360</td>
</tr>
</tbody>
</table>

OVERLOAD DESIGN at Pier 2

\[ \text{Mdl-nc} = 1367 = 0.43 < 0.6 \longrightarrow \text{Lower Limit of } M/M_{\text{pn}} \]

from Inelastic Rotation Curve

Non-composite dead load produces no plastic deformation in the non-composite section.

Steel Stress @ Overload = 3353(12) = 53.1 KSI Steel will yield.

Determine Point A \[ M_{\text{overload}} = 3353 \]
\[ M_{\text{p}} = 3999 \]

Determine Point B \[ \text{Total Rotation of the Pier due to the Elastic Overload Moment assuming free hinges at the piers.} \]

FOR LANE LOADING:

CONCENTRATED LOAD ---\> \( P = (5/3)(1.200)(1.221)(9) = 22 \text{ KIPS} \)

UNIFORM LOAD ---\> \( DL = 1.323 \text{ K/FT includes slab, rails, future overlay, beam, details, and haunch.} \)

\( LL = 0.32(5/3)(1.200)(1.221) = 0.781 \text{ K/FT} \)

\[ \text{Mdl-nc} = 1367 = 0.34 \]
\[ M_{\text{p}} = 3999 \]

INTERSECTS COMPOSITE INELASTIC ROTATION CURVE AT 0.7 MRADS

GRAPHICAL REPRESENTATION
\[ \theta = \frac{3}{24EI} + \frac{Pab(L+a)}{6LEI} \]

\[ \theta_{(SPAN \ 2)} = \frac{(1.323 + 0.781)(128)(144) + 22(76.8)(51.2)(128 + 76.8)144}{24(29000)(27150)} \]

\[ m_c = \frac{66.3 (0.84)}{75.6} = 0.74 \]

\[ \theta_{(TOTAL)} = \theta_{(SPAN \ 2)} + \theta_{(SPAN \ 3)} = 75.6 \text{ MRADS} \]

FROM ATTACHED GRAPH, \[ M_{auto} = 0.05 \]

\[ M_{auto} = 0.05(3999) = 200 \text{ K-FT} \]

THIS AUTOMOMENT AT PIER 2 MUST BE DISTRIBUTED ACROSS THE STRUCTURE.
\[ M_{CL3} + 2Md(L3 + L4) + MeL4 = 0 \quad \text{Me} = 0 \text{ AT END SUPPORT} \]

\[ 200(128) + 2Md(128 + 101) = 0 \]

\[ Md = -56 \text{ K-FT} \]

Similarly, the Automoment at Piers 1 and 3 are determined and distributed across the structure. By superposition, the final Automoment diagram for the structure becomes:

\[ M_{pe} = 3669 \text{ K-FT} \]

\[ M_{max} = 4360 \text{ K-FT} \]

The difference between the effective plastic moment capacity at the piers and the maximum design load negative moment is redistributed to the positive moment portions in the same way that the Automoments were previously.

\[ M_{max} = 3360 + 412 = 3772 \text{ K-FT} < M_p = 4714 \text{ K-FT} \]

SECTION IS ADEQUATE FOR MAXIMUM DESIGN LOAD.
LATERAL BRACING

\[ \text{Lb} = \left[ 3.6 - 2.2 \left( \frac{M_1}{M_2} \right) \right] \times 10 \times r \]

\[ \text{Fy} \]

\[ \text{r} = 2.56 \]

For the end spans:

\[ \frac{9F}{\text{UNIFORM LOAD}} = \text{DL} + 0.32F \]

\[ \text{Mpe} = 3669 \text{ K-FT} \]

\[ \text{F} = 1.3(22) = 28.6 \text{ K} \quad \text{DL} = 1.3(1.323) = 1.72 \text{ K-FT} \]

Summing moments about the pier gives:

\[ \text{R ABUT} = 119.3 \text{ K} \]

Assuming \( \text{Lb} = 10 \text{ FT} \),

\[ \text{M} = -1947 \text{ K-FT} \]

\[ \text{Lb} = \left[ 3.6 - 2.2\left( \frac{-1947}{-3669} \right) \right] \times 10 \times 2.56 \]

\[ \text{Lb} = 10.38 \text{ FT} > 10 \text{ FT} \quad \text{OK} \]

The spacing in the 2nd bay is assumed 14 FT.

\[ \text{Mu} = \frac{\text{FyS}}{4\pi^2} \left[ 1 - \frac{3\text{Fy}}{4\pi^2(29000)} \left( \frac{14(12)}{5.0175} \right)^2 \right] \]

Since reinforcing makes section unsymmetrical, use 0.9b'.

\[ \text{M2} = 2068 \text{ K-FT} \quad \text{M1} = 786 \text{ K-FT} \]

\[ 0.7\text{M2} > \text{M1} \quad \text{Increase Mu by 20\%}, \text{ not to exceed CyS} \]

\[ \text{Mu} = 1.2(50)757.75 \left[ 1 - \frac{3(50)}{4\pi^2(29000)} \left( \frac{14(12)}{5.0175} \right)^2 \right] \]

\[ \text{Mu} = 3232 \text{ K-FT} \]

\[ \text{FyS} = 757.75(50) = 3157 \text{ K-FT} \quad \text{CONTROLS} \]

\[ \text{Mu} > \text{M2} \times 14 \text{ FT}. \text{Spacing is satisfactory.} \]

The interior spans are checked in a similar manner.
LIVE LOAD DEFLECTION

\[ \frac{L}{600} = 1.515 \text{ IN (END SPANS)} \]
\[ \Delta = 1.920 \text{ IN (INTERIOR SPANS)} \]

From continuous beam program, with three lanes equally distributed and fully composite girders:
\[ \Delta = 1.362 \text{ IN (END SPANS)} \]
\[ \Delta = 1.927 \text{ IN (INTERIOR SPANS)} \]

SECTION IS SATISFACTORY FOR LIVE LOAD DEFLECTIONS.

OTHER DESIGN ITEMS CHECKED BUT NOT SHOWN HERE:

UPLIFT AT ABUTMENTS
FATIGUE
CAMBER FOR DEFLECTIONS DUE TO DEAD LOAD AND AUTOMOMENTS
WIND LOADS
SHEAR CONNECTORS
SPILCES
BEARING STIFFENERS

SUMMARY

TOTAL STRUCTURAL STEEL WEIGHT \( \rightarrow \) 532,945 LB.
STEEL WEIGHT PER SQ. FT. OF DECK \( \rightarrow \) 25.3 LB/SF