Jointless Bridge Decks

EDWARD P. WASSERMAN

Today's bridge designer is striving essentially to achieve the same goals as his counterparts were 50 years ago: long term serviceability, low maintenance characteristics and economy of construction. While we have mastered new techniques such as welding, composite decks, hybrid girders, load factor and autostress designs, we still cling to many of the old ideas that lessen the potential for achieving our goals. One of the more important aspects of design, the reduction or elimination of roadway expansion devices and associated expansion bearings is overlooked consistently, or avoided.

Contractors, engineers, producers and fabricators of steel bridges should examine the possibilities for minimum or no joint structures to provide your clients the most durable and cost-effective product, as well as to remain competitive, as an industry, with alternate design materials.

HISTORY

In Tennessee DOT, a structural engineer can measure his ability by seeing how long a bridge he can design without inserting an expansion joint. In the last 20 years, nearly all our newer highway bridges up to several hundred feet have been designed with no joints, even at the abutments. If the structure is exceptionally long, we include joints, usually at the abutment only.

Joints and bearings are costly to buy and install. Eventually they are likely to allow water and salt to leak down onto the superstructure and pier caps below. Many of our most costly maintenance problems originated with leaky joints (Fig. 1). Expansion bearings inherently malfunction over time (Fig. 2). So we go to great lengths to eliminate both.

While 95% of our bridges are designed in-house, the no-joint or minimal-joint policy works well when a consultant designs a bridge for us, too. In a few cases, we have contractually absolved a consultant of liability in case problems should arise related to the absence of joints.

Why, if our no-joint approach is so good, have not bridges always been designed this way? Why so many joints?

Until computer and structural analysis programs arrived,

Figure 1

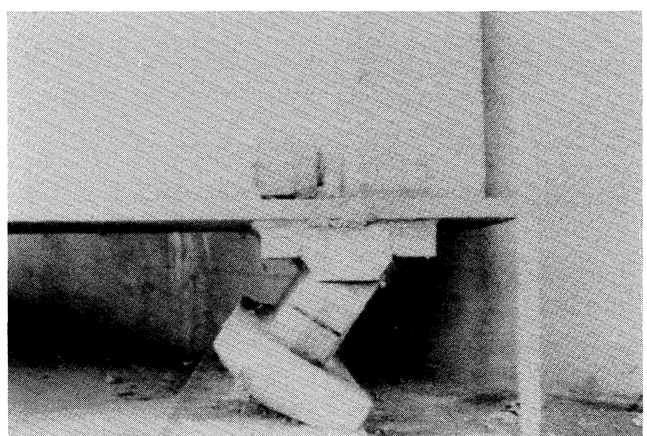

Figure 2

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figuring moment distributions of bridge loads at supports for continuous spans was extremely laborious. So, until the last 30 years or so, most designers chose to take the course of least resistance—several simple spans. Further, thermal movements must be dealt with in some manner; so why not the simple approach?

In Tennessee we began to re-examine this practice in 1956 when the Interstate Highway Trust Fund was created by Congress. We knew this would mean we would be installing hundreds of new multi-span overpasses at interchanges, for example. In the computer, we now had the tool to make designing continuous spans manageable. But how would a joint-free and bearing-free structure perform in the field? We started cautiously, building only a few. And the first ones were relatively short. When time proved a design was working, we got a bit bolder. First, we used *some* joints, but only at abutments. Then we eliminated even these. Our longest entirely jointless bridges are 927 ft (282 m) in concrete and 416 ft (127 m) in steel (Fig. 3).

Structural analysis of our no-joint bridges indicated we should have encountered problems, but we almost never have. One time, we tied the stub abutment of a bridge into rock, and the structure cracked near its end. But we were able to repair the bridge and install an expansion joint while the bridge was under traffic. The public never knew about it. That was one of our few problems. In effect we say, "So what if we get a crack across a bridge deck? Had we installed an expansion joint we would have had a manufactured crack of larger proportions and would certainly have encountered problems."

Back 10 years ago, the engineer of structures, Henry Derthick, wrote, "In the nearly 20 years we have been gaining courage trying longer and longer continuous units, we have never encountered a problem that could be attributed to the *absence* of a joint. If only we could make the same statement about the *presence* of a joint."

POLICY

Tennessee policy reads, "As a general rule, all bridges shall be continuous from end to end. There shall be no intermediate joints introduced in the bridge deck other than cold joints required for construction. This applies to both longitudinal and transverse joints."

Elaborating, "Structures must be designed to accommodate the movements and stresses caused by thermal expansion and contraction. Bridge designers should not accommodate these movements by using unnecessary bridge deck expansion joints and expansion bearings. This solution creates more problems than it solves. Structural deterioration due to leaking expansion joints and frozen expansion bearings usually means we have major bridge maintenance problems.

"To eliminate these problems, it is our policy to design and construct bridges with continuous superstructures, with fixed and integral connection to substructures, and no bridge deck expansion joints unless absolutely necessary. When expansion joints are necessary, they will generally be provided only at abutments." Where joints are provided, these should be as few as possible. One exception where we place joints over piers is on a large river crossing at locations where the superstructure changes from concrete to steel.[1,2]

PRACTICE

In practice, this means that Tennessee builds steel superstructure bridges up to about 400 ft (120 m) long with no joints, even at the abutments; and concrete superstructure bridges of this type up to 800 ft (240 m) and sometimes longer. In both cases, this means a maximum movement at the bridge's ends of 2 in. (50 mm). For continuous decks longer than this, we use joints at the abutments. Our longest bridge built in one continuous deck without joints, except at the abutments, is the Kingsport Bridge, 2,800 ft

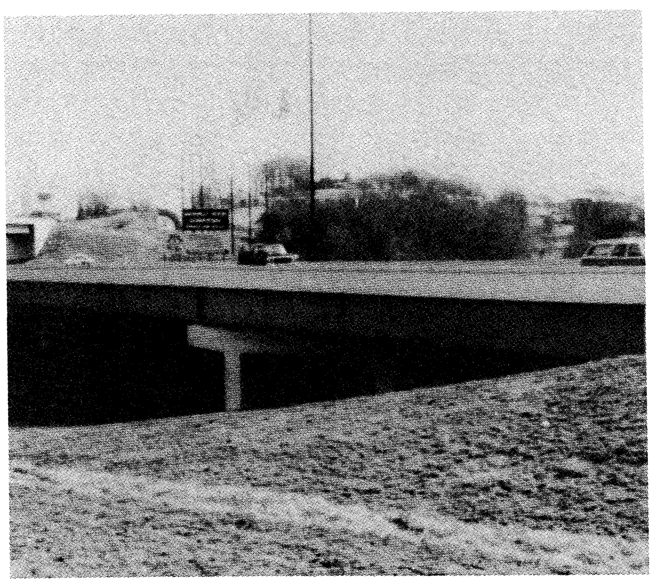


Figure 3A

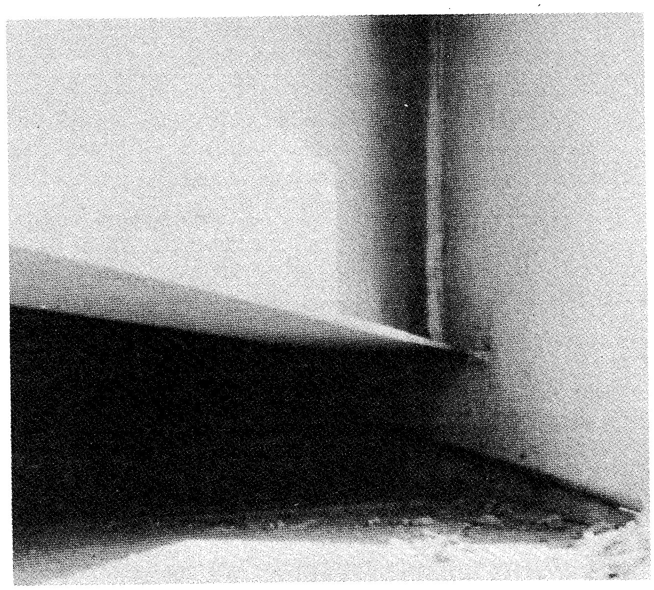


Figure 3B

(850 m). The University of Tennessee instrumented the bridge and found neither the deck elongation nor the superstructure stresses abnormal. All measured stress data were lower than predicted.

Exactly why, we do not know, but think we have some answers. One factor appears to be creep of the concrete. If concrete is expanded or contracted slowly, as by temperature changes, it creeps. Stresses due to shrinkage/expansion do not reach the levels predicted. To make theory better fit reality, in the case of concrete substructures, we have reduced the modulus of elasticity to one-third that used for dynamic loads.

In addition, temperature cycling of concrete bridges, it appears, reaches lower peaks than in steel. Apparently the greater mass of concrete provides a heat sink. Thus, its temperature tends not to rise as high nor as low as theory predicts. We design Tennessee bridges in concrete for a temperature range of 20°–90°F. (−7° to 32°C.), and steel superstructure bridges for a range of 0−120°F. (−18° to 49°C.). These values are consistent with the AASHTO Standard Specifications for Highway Bridges.

Based on these ranges and thermal coefficients of expansion for the respective materials, we design for 0.505 in. of movement/100 ft (0.420 mm/m) of span in concrete, and 0.936 in./100 ft (0.782 mm/m) in steel. While these rates give an edge to concrete bridges, steel designers, to be competitive, need to exercise economy in joint placement to the fullest extent.

Knowing what needs to be accomplished, how do we set about to reduce or eliminate expansion joints? Essentially, we (1) take advantage of pile translation and rotation capabilities, (2) modify foundation conditions if feasible, (3) take advantage of the reduced modulus of elasticity of concrete for long term loads, (4) allow hinges to form naturally or construct them, and (5) employ expansion bearings where necessary. Examples of how one employs these techniques are shown:

Δ = total thermal movement due to expansion and contraction (in.)

L = distance from theoretical fixed center of structure to the pier in question (ft)

ℓ = Pier height (ft)

M_B = Moment at base of column (ft-kips) due to $\Delta/2$ displacement at the top.

M_T = Moment at top of column (ft-kips) due to $\Delta/2$ displacement at the top.

P = force required to produce $\Delta/2$ displacement (lbs.)

$E = 1.0 \times 10^6$ for concrete column in long term deflection (lbs./in.^{2})

I = Moment of inertia of column (ft^4)

For concrete structures: $\Delta = 0.00505L$

For steel structures: $\Delta = 0.00936L$

1. From the figure below, if the structure were a steel bridge with six 66 ft-8 in. spans, the total movement due to thermal effects at each abutment would be 1.87 in. If the abutments were sill-type and supported on one row of piles, that pile translation and/or rotation can accommodate the movement required without benefit of expansion joints. This can be accomplished by designing the connection of the abutment sill to exceed the moment capacity of the pile group or insuring the shear capacity of the sill exceeds the force required to displace the piles laterally.

2. If the sill abutments are bearing on in-situ rock or rock fill, modifications to build in the ability for translation by setting piling in pre-drilled oversize holes in solid rock or providing an earth core in rock embankments to allow the driving of piling can be used.

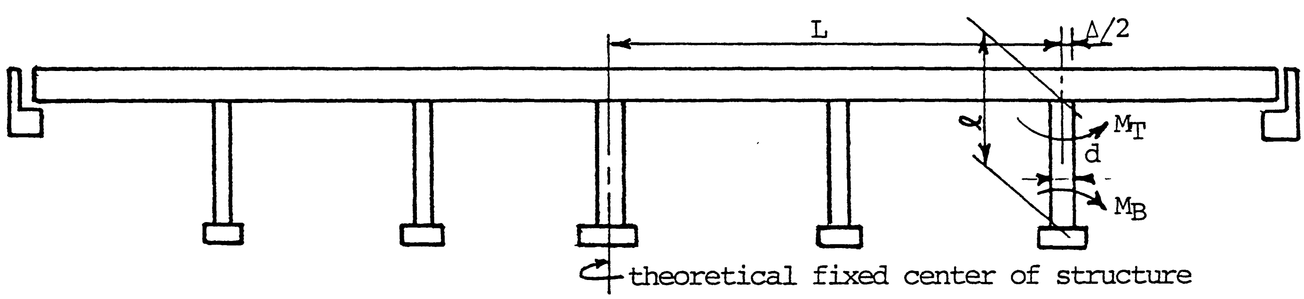
3–4. Using the long term modulus of elasticity for concrete in substructures can reduce applied moments by a factor of three (1,000,000 vs. 3,000,000). Consider a column free to translate and rotate at the top and fixed at its base, acting as a cantilever. It will develop a fixed end moment $M_B = P\ell/2 = P\ell$ (see notation above). For Pier 5 in the figure, the maximum deflection to be absorbed is $\Delta/2$ and the force that must be developed to cause the deflection:

$$P = \frac{3\ EI\Delta}{2\ell^3}$$

and the moment capacity of the column must be equal to:

$$M_B = \frac{3\ EI\Delta}{2\ell^2}$$

5. After calculation of the bending moment to be applied to the base of a column support, some decisions must be made concerning how the load is to be accommodated. Usually the decision is to reinforce the moments. However,



ıere are certain occasions where it is more beneficial to the tructure to introduce a hinge, thereby eliminating the ıoment build-up.

From Fig. 4, it is apparent that means of constructing ınges can range from very elementary to complex, deıending on needs of the designer. While the hinged column upport will usually have to be supported temporarily gainst overturning, the installation cost is minimal.

ı. A more practical solution to avoiding moment build-up lue to thermal stresses is introduction of an expansion ıearing at a support. While this is the most direct approach, onsideration of the cost and long-term maintenance probems caused by expansion bearings should be weighed. nevitably, expansion bearings of all types fail through ;auling, seizing, binding or overturning unless given peiodic attention.

Once we have succeeded in eliminating expansion joints ıt all intermediate substructures, we come to the most lifficult evaluation—whether to eliminate totally joints rom the bridge. The prime consideration is the ability of he abutments to move in response to thermal expansion ınd contraction. A typical pile-supported sill or perched abutment on flexible piling (Fig. 5) can deflect up to two in. with ease. The same sill-type abutment resting on, but not keyed into, rock, can tolerate only about ¼-in. Massive cantilever abutments cannot translate and therefore must be hinged as in Fig. 6, or an expansion device introduced.

When the decision to eliminate expansion joints at abutments is made, some fundamental questions must be satisfied: What will be the expected damage, if any? How can the damage be minimized or eliminated? What will be the effect of thermal expansion and contraction on the roadway approach?

The answer to the first question is the most difficult. In Tennessee, we had mixed results. Most often, pile-supported sill and rock-supported, sill-type abutments will not develop cracking. In a few instances, minor cracking will occur either in the wingwall or sill, but these instances are not cause for alarm and create no serviceability problem. It is imperative that reinforcement commensurate to the task at hand be designed into the abutments. Typical details of connection and reinforcement are shown in Fig. 7. One fact is certain, if you intend to go the jointless route, approach the design in a workmanlike manner. Figure 8 is an early attempt to eliminate expansion joints at an abut-

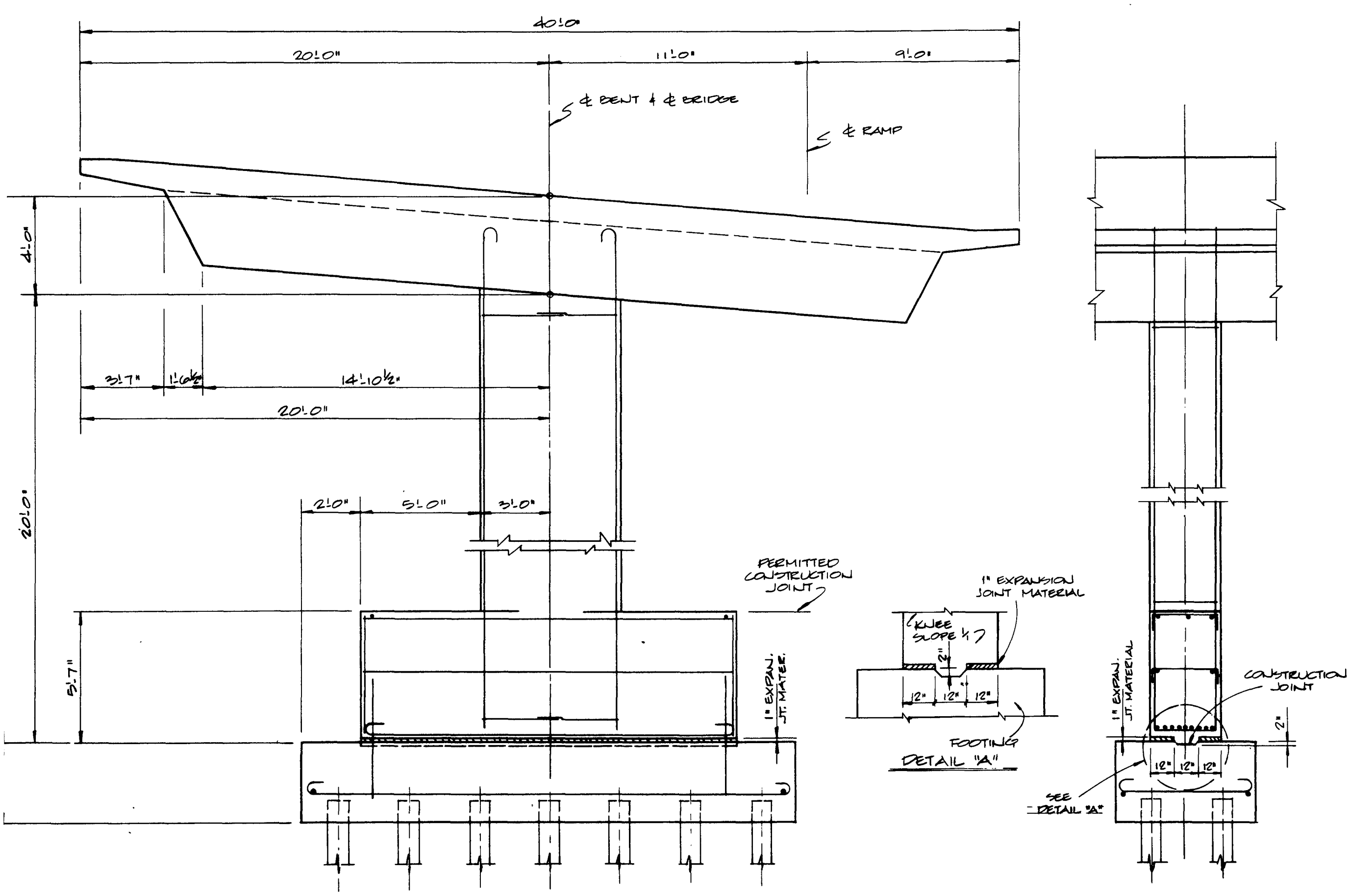


Figure 4A

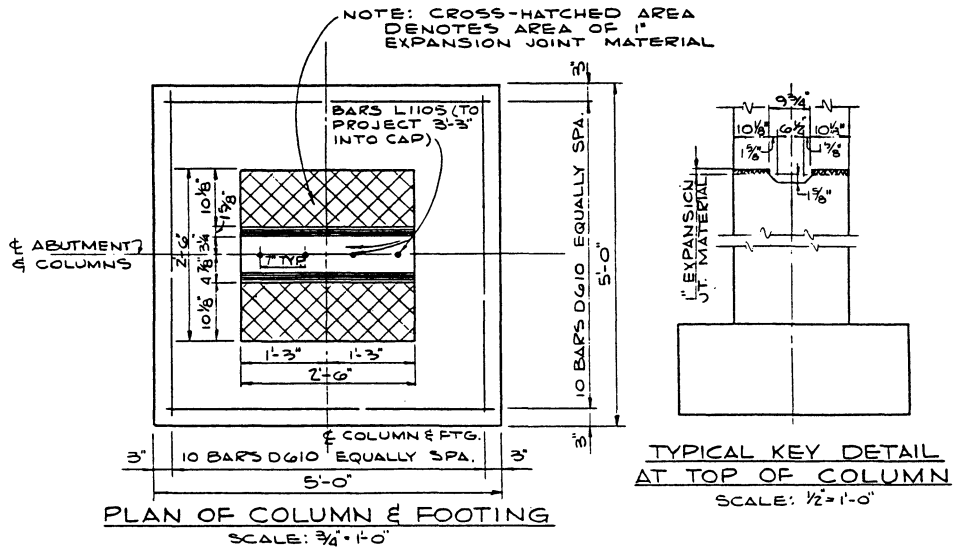


Figure 4B

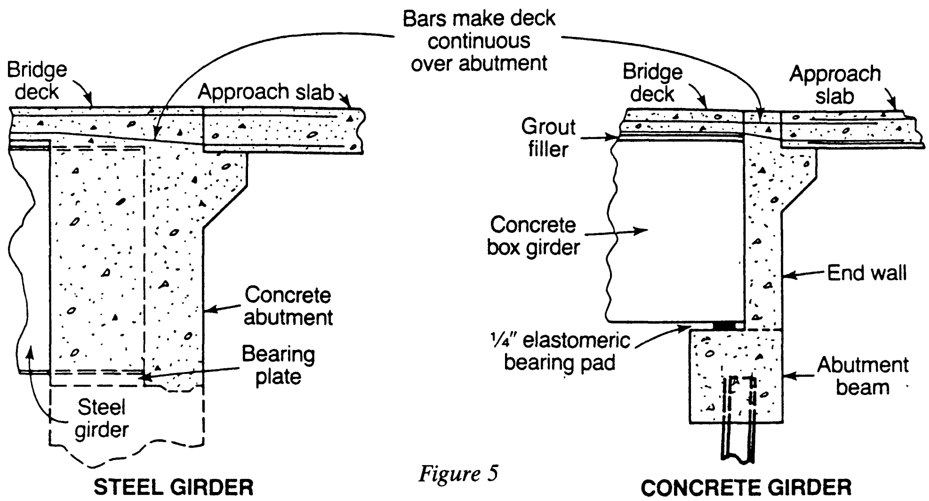


Figure 5

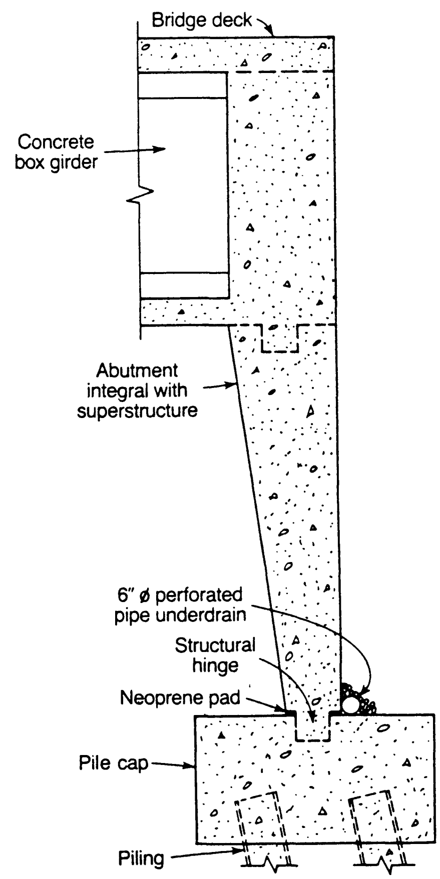


Figure 6

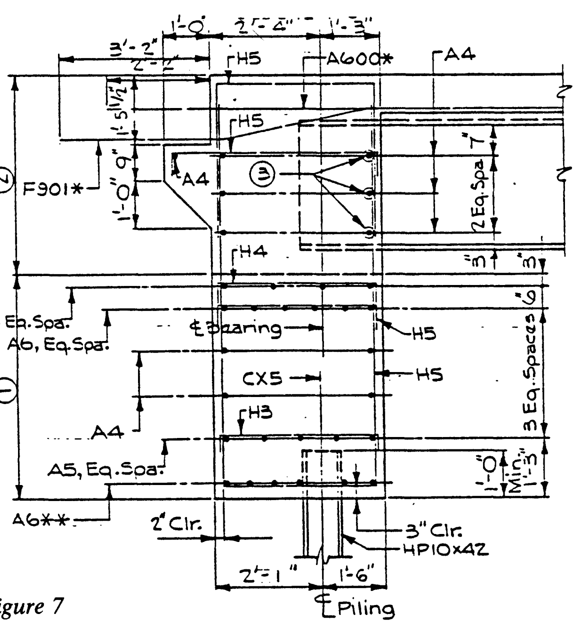


Figure 7

ment without due regard to consequence. In this instance, the structure was designed and constructed to incorporate joints at the abutments. However, a last minute decision was made, after construction of the abutment and erection of the girders, to eliminate the joints. As indicated, results were less than satisfactory. Note, however, what you see is the result of 10 years service. Had joints been installed, they doubtless would have had to be repaired or replaced by now.

As a designer gaining confidence in joint elimination, one inevitably becomes bolder, stretching and testing the limits, most often successfully, sometimes not, but one learns. Occasionally, one has the urge to over-reach his calculations. Figure 9 is one of 8 or 10 such examples tried in Tennessee. This 218-ft steel, box-girder structure is actually a 3-span continuous unit with short end spans fixed to develop the necessary resisting moments that allow the slender 178-ft central span to function at a depth of only 35 in. Fixity was achieved by integrally attaching the girders to the massive coffin abutments filled with crushed stone. The fixity was achieved at the price of eliminating expansion devices and expensive hold-down bearings. If one calculated the forces needed to displace the abutments and the stress build-up in the girders, he would conclude the task is impossible. Yet this bridge has functioned well for seven years with no signs of distress to the abutments or superstructure. How does it function? We can't say for certain—but it works!

When all bridge joints are eliminated, the effect of abutment movement on the approach pavement must be considered. Where we have concrete pavement approaching a concrete deck, we install a compression seal joint between them. Where interface is asphalt to concrete, no special treatment is necessary. This will eventually cause some

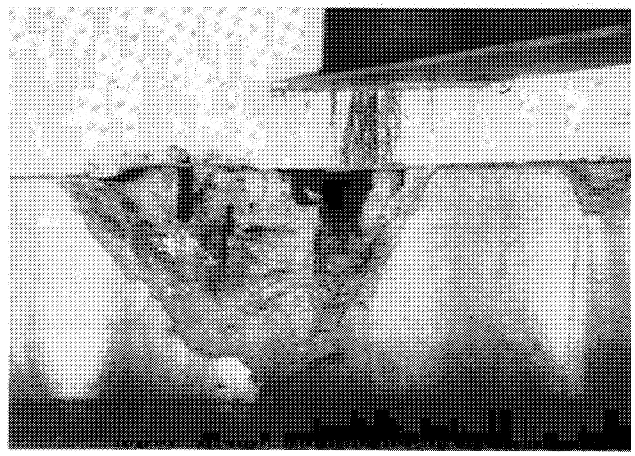


Figure 8

local pavement failure and a bump. However, this is minor problem compared to joint maintenance.

After discussing the design of jointless bridges at lengtl the designer should pay some attention to rehabilitatio and retro-fitting of existing structures. As we indicatec joints cause problems for bridges, and at some point in tim the piper must be paid. Tennessee has developed sever; procedures for rehabilitating economically existing struc tures that allow for easy elimination of joints and a degree if not total continuity. Again, start by moving cautiously, a indicated in Fig. 10, achieving full continuity by removin parts of the slab over supports and installing splice plates t girders. This is time-consuming and expensive work.

For moderate length spans, we progressed to removal c part of the top slab to splice-in tension reinforcement fc the slab, but no tension flange splice plates were installec

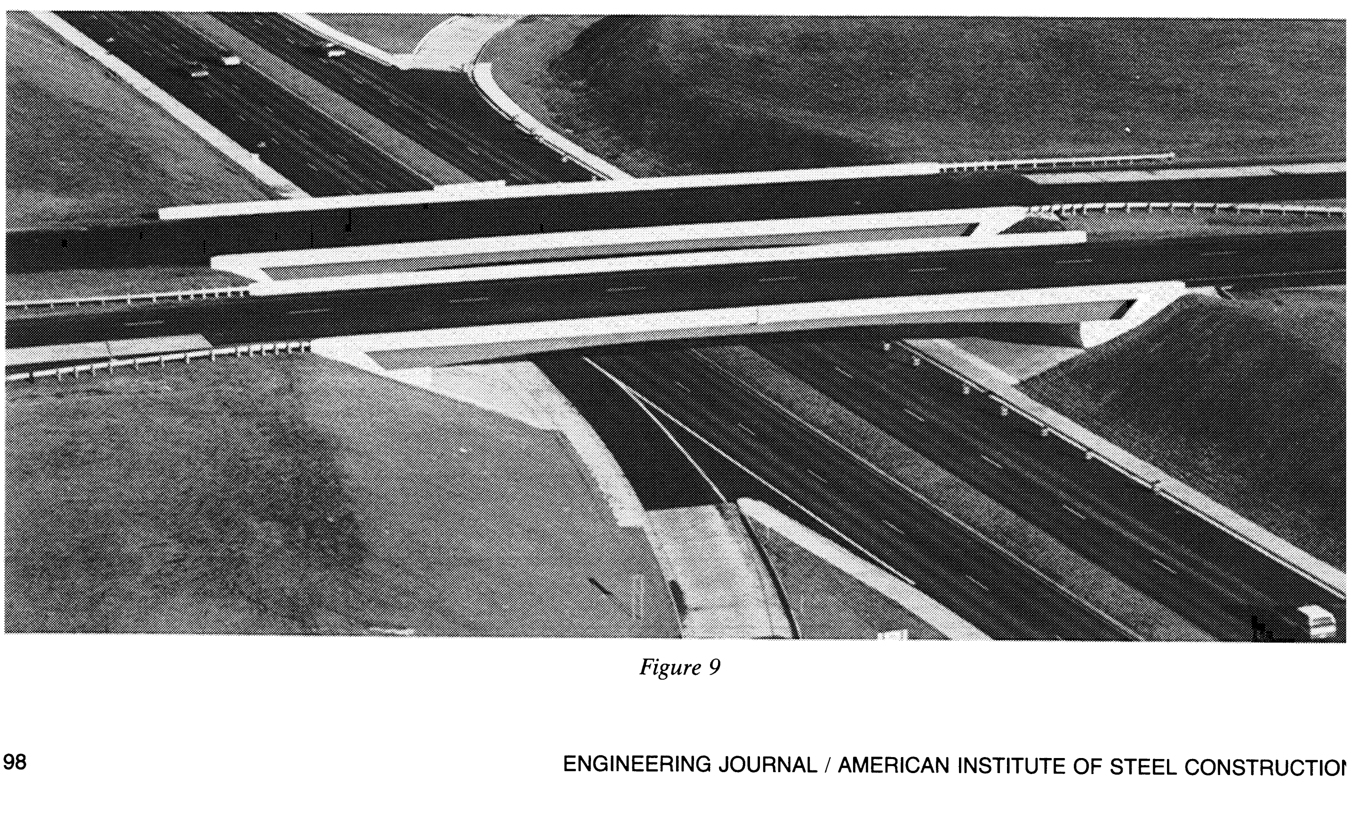
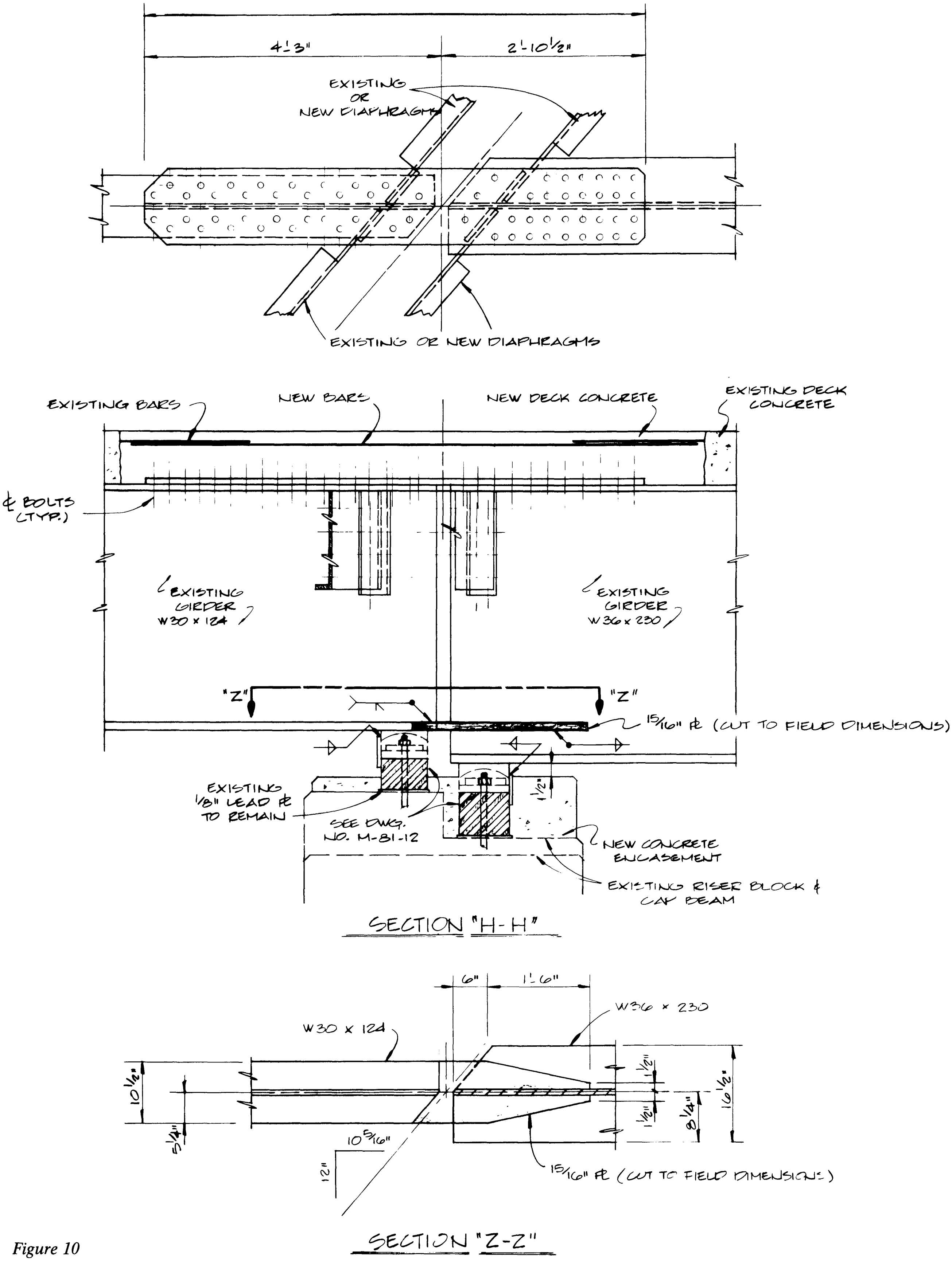


Figure 9



4'-3"
2'-10½"
EXISTING OR NEW DIAPHRAGMS
EXISTING OR NEW DIAPHRAGMS
EXISTING BARS
NEW BARS
NEW DECK CONCRETE
EXISTING DECK CONCRETE
℄ BOLTS (TYP.)
℄ EXISTING GIRDER W30 x 124
℄ EXISTING GIRDER W36 x 230
"Z"
"Z"
15/16" PL (CUT TO FIELD DIMENSIONS)
EXISTING 1/8" LEAD PL TO REMAIN
SEE DWG. NO. M-81-12
1½"
NEW CONCRETE ENCASEMENT
EXISTING RISER BLOCK & CAP BEAM
SECTION "H-H"
6"
1'-6"
W36 x 230
W30 x 124
10½"
5¼"
10 5/16"
12"
1½"
1½"
8¼"
16½"
15/16" PL (CUT TO FIELD DIMENSIONS)
SECTION "Z-Z"

Figure 10

only a compression splice to the bottom flange was added. This worked very well, especially where deteriorated slabs required concrete removal.

Current practice (Fig. 11) uses a 4½-in. reinforced concrete overlay on the existing concrete deck. Prior to placement, the existing deck is scarified to a depth of 1-in., remaining delaminated concrete areas removed and the deck surface sandblasted and cleaned. Negative moment reinforcement is placed over the joints at intermediate supports and the overlay poured. Depending upon the type of girders and length of span, beam end modifications vary. For short-span, cast-in-place and prestressed concrete girders, no additional modifications are employed. For longer concrete spans, compression blocks are poured between adjacent beam ends. Similarly, for short-span steel, no modification is required. For longer steel spans, compression flanges are jointed. In some cases, bearings must be modified. In most cases, the ends of slabs at abutments and backwalls are modified for moment connections to elimi nate end of bridge joints.

Where the foregoing modifications are employed, i structure widening is to be added, the new section will be o fully continuous composite construction.

In summary, Tennessee's 20-yr. experiment has provei that, for thermal movements up to 2-in., both immediat construction savings and long-term maintenance saving can be realized by total elimination of joints. Further join elimination promotes extended serviceability and leads t more efficient, aesthetically pleasing bridges.

REFERENCES

1. *Tennessee Department of Transportation* Structura Memorandum No. 45 *Division of Structures.*
2. *Loveall, C. L.* Jointless Bridge Decks *Civil Engi neering, November 1985.*

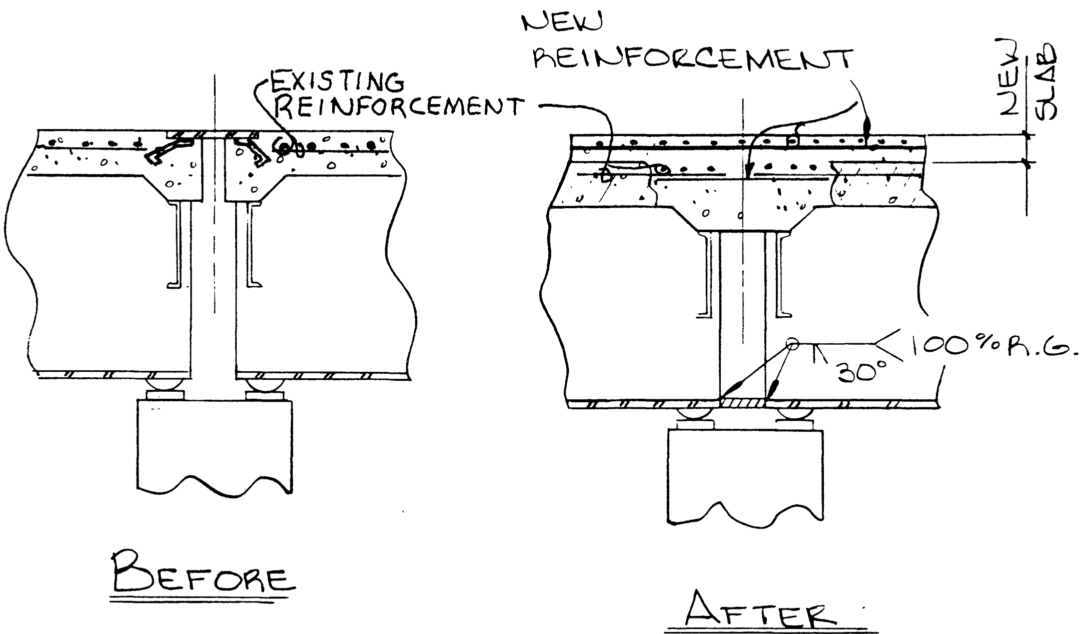


Figure 11