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INTEGRAL ABUTMENTS
FOR
STEEL BRIDGES

by

Edward P. Wasserman, P.E. and John Houston Walker, P.E.

Structures Division
Tennessee Department of Transportation

INTRODUCTION

Today's bridge designers are essentially striving to achieve the same goals as their counterparts were 70 years ago: long-term serviceability, low maintenance characteristics, and economy of construction. While new techniques have been mastered such as welding, composite decks, Load Factor and Autostress designs, designers still cling to many old ideas that lessen the potential for achieving their goals. One of the more important aspects of design — reduction or elimination of roadway expansion devices and associated bearings — is consistently overlooked or avoided by many bridge-design practitioners. As a result, this chapter on jointless bridges utilizing integral abutments has been developed to discuss issues ranging from their historical development through their design implementation to help provide the necessary understanding for today's bridge engineer.

Historical Developments

In May of 1930, Professor Hardy Cross published a paper in the Proceedings of the American Society of Civil Engineers(1) that proved to be a landmark in the field of structural engineering. The paper presented a simple and straightforward method for the analysis of continuous beams and frames by means of moment distribution. Prior to introduction of the Hardy Cross Method, almost all bridges were designed as a series of structurally determinate simple spans.

Moment distribution was immediately accepted by progressive structural engineers, and with its use, continuous bridges and bridges with more than one continuous unit began to appear. Most of these structures were standardized units, because although the Hardy Cross Method was a significant advancement, it still required time consuming and tedious hand calculations. The real boom in the design and construction of continuous bridges began with the widespread use of the computer.
According to Martin P. Burke, Jr., the Ohio Department of Transportation was one of the first agencies to initiate the routine use of continuous construction in the 1930's\(^2\). Utilizing riveted field splices, adjacent spans were linked to construct fully continuous steel stringer bridges. By 1934, the Department had devised its first field butt-welded splice. In the late 1950's, high-strength bolted field splices were introduced and became the method of choice for field erection by 1963. In a 1989 survey\(^3\), 87 percent of 30 responding states reported the routine use of continuous structures for short and medium spans.

Along with the development of continuity, the use of integral abutments to achieve jointless bridges progressed. Ohio, South Dakota and Oregon appear to have pioneered the use of jointless bridges in concrete in the 1930's and 1940's, with California following suit in the mid-1950's. With the National Interstate Highway System construction boom in the late 1950's and early 1960's, the use of continuous jointless bridge construction began a real growth spurt. By the mid-1960's, Tennessee and five other states had adopted continuous bridges with integral abutments as standard construction.

**Reasons for Jointless Construction**

Over the past thirty years, engineers have become more aware of the pitfalls associated with the use of expansion (cycle control) joints and expansion bearings. Joints are expensive to buy, install, maintain and repair. Repair costs can run as high as replacement costs. Successive paving will ultimately require that joints be replaced or raised. Even waterproof joints will leak over time, allowing water — salt-laden or otherwise — to pour through the joint accelerating corrosion damage to girder ends, bearings and supporting reinforced concrete substructures. Accumulated dirt, rocks and trash fill elastomeric glands leading to failure. Hardware for joints can be damaged and loosened by snow plows and the relentless pounding of heavy traffic. Broken hardware can become a hazard to motorists and a liability to owners.

Bearings are also expensive to buy and install and more costly to replace. Over time, steel bearings may tip over and/or seize up due to loss of lubrication or buildup of corrosion. Elastomeric bearings can split and rupture due to unanticipated movements, or ratchet out of position.

Teflon sliding surfaces are fragile for bridge applications and can fail prematurely due to excessive wear from dirt and other contaminants, or due to poor fabrication and construction tolerances. Pot bearings also suffer frequent damage due to poor fabrication and construction techniques.

Joints and malfunctioning expansion bearings can also lead to unanticipated structural damage. The presence of joints can facilitate abutment overturning due to inadequate resistance to active earth pressures, live-load surcharging or approach pavement growth. Joints can even facilitate the settlement of pile-supported abutments.

In 1985, the Federal Highway Administration published a report on tolerable movement criteria for highway bridges\(^4\). The report was the culmination of a five-year study conducted by West Virginia University, which examined 314 bridges in the United States and Canada. A total of
580 abutments were examined. Over 75 percent of these abutments experienced movement, contrary to their designers intent. In the words of the report:

"The magnitude of the vertical movements tended to be substantially greater than the horizontal movements. This can be explained, in part, by the fact that in many instances the abutments moved inward until they became jammed against the beams or girders which acted as struts, thus preventing further horizontal movements. For those sill type abutments that had no backwalls, the horizontal movements were often substantially larger, with abutments moving inward until the beams were, in effect, extruded out behind the abutments."

The conclusion to be inferred is that providing for thermal movements by means of expansion joints and bearings does not avoid maintenance problems; rather, the provision of these items can often facilitate such problems.

In the early 1960's, bridge maintenance requirements were studied nationwide. It was determined that joints and bearings are a major source of bridge maintenance problems. While casting about for solutions to joint and bearing maintenance problems, many engineers became aware that bridges constructed without joints were outperforming jointed bridges by remaining in service for longer periods without the requirement for maintenance. While some cracking in abutments occurred, these cracks were not detrimental to serviceability. Jointless bridges were generally not plagued with the same distress evident in jointed bridges.

It was also acknowledged that integral abutment bridges were being constructed at a lesser first cost. The reasons for the lesser cost of jointless bridges with integral abutments go deeper than the obvious benefits of lesser material cost and the elimination of expansion joints and bearings. Some of the more important reasons are summarized below.

**Design Efficiency**

Tangible efficiencies are achieved in substructure design due to an increase in the number of supports over which longitudinal and transverse superstructure loads may be distributed. For example, the longitudinal load distribution for the bent supporting a two-span bridge is reduced by 67 percent when integral abutments rather than expansion abutments are used. Depending upon the type of bearings planned for an expansion abutment, transverse loadings on the same bent can be reduced by 67 percent as well.

**Added Redundancy And Capacity For Catastrophic Events**

Integral abutments provide added redundancy and capacity for all types of catastrophic events. In designing for seismic events, considerable material reductions can be achieved through the use of integral abutments by negating the need for enlarged seat widths and restrainers. Further, the use of integral abutments eliminates loss of girder support; the most common cause of damage to bridges in seismic events. Tests in Japan\(^\text{5}\) have demonstrated that significant improvements in damping capacity are realized when integral abutments are used. When integral abutments move rapidly, passive pressures engage which act to absorb significant amounts of energy. This
mobilized damping force is increased by the soil/pile interaction and remains effective throughout each full cycle of seismically induced movement.

As stated in the Final Report of FHWA/RD-86/102, Seismic Design of Highway Bridge Foundations Vol. II(5), integral abutments are the preferred design feature for more active seismic regions. Joints introduce a potential collapse mechanism into the overall bridge structure. Integral abutments have consistently performed well in actual seismic events and have significantly reduced or avoided problems of backwall and bearing damage that are associated with seat-type jointed abutments. The damping arising from soil-abutment interaction has been proven to significantly reduce the lateral loads taken by intermediate substructure columns and footings. Tests on several short (less than 200 ft) bridges, listed in the FHWA report, found as much as 15 percent damping for the longitudinal mode of response of the bridge deck system. The report also recommends that integral abutments be proportioned to restrict displacements to 4 in. or less to minimize damage (see p. 138, ibid., for design procedure).

**Enhanced Load Distribution For Girders At Bridge Ends**

Integral abutments provide substantial reserve capacity to resist potentially damaging overloads by distributing loads along the continuous and full-depth diaphragm at bridge ends.

**Enhanced Protection For Weathering Steel Girders**

An FHWA Technical Advisory T5140.22(6) provides specific guidance and recommendations for the use of uncoated weathering steel in highway bridges. The Technical Advisory (TA) recommends that girder ends under expansion joints receive special attention in the form of initial and maintenance painting. The TA further recommends that joints be eliminated. With the exception of initial corrosion protection for those portions of the girder embedded in the concrete end diaphragm and those portions about 1 ft in front of the diaphragm, no coating is required at the time of construction or subsequently thereafter when integral abutments are used. A recently completed report, Performance of Weathering Steel in Highway Bridges - A Third Phase Report published by the American Iron and Steel Institute(7), underlines the success of jointless weathering steel bridges with integral abutments and other forms of jointless construction.

**Rapid Construction**

With integral abutments, only one row of vertical (not battered) piles are used and fewer piles are needed. The entire end diaphragm/backwall can be cast simultaneously and with less forming. Fewer parts are required. Scheduling problems with suppliers and manufacturers are avoided.

**Tolerance Problems Are Reduced**

The close tolerances required when utilizing expansion bearings and joints are eliminated with the use of integral abutments. Bridge seats need not conform exactly to girder flange slope and
camber corrections, since the girder loads are ultimately carried by the concrete comprising the end diaphragm. Minor mislocation of the abutments creates no fit-up problems.

**Greater End Span Ratio Ranges**

For normal expansion bearing conditions, the ratio of the end-span to the adjacent interior-span length must be held to approximately 0.6, unless uplift conditions are to be accommodated. If uplift can occur, expensive hold-down devices must be added to expansion bearings. Utilizing integral abutments allows for much shorter end spans, if desired, since the abutment acts as a counterweight and the uplift capacity of the piling may be used. By adjusting the pouring sequence to cast the abutment around the girder ends first, computed uplift due to dead loads can be eliminated.

**PRACTICES OF DESIGN AGENCIES**

Surveys taken by Wolde-Tinsae, Greiman(8,9) and Burke(3) indicate that 28 states, plus FHWA Region 15, have developed guidelines for the design and construction of integral abutment bridges.

A summary of the surveys reveals the following practices:

<table>
<thead>
<tr>
<th>DESIGN CONSIDERATIONS AND ASSUMPTIONS</th>
<th>NUMBER OF RESPONDENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Length¹</td>
<td>250 ft 7</td>
</tr>
<tr>
<td></td>
<td>300 ft 11</td>
</tr>
<tr>
<td></td>
<td>400 ft 3</td>
</tr>
<tr>
<td>Calculate Pile Stresses Due to Lateral Movement</td>
<td>Yes 6</td>
</tr>
<tr>
<td></td>
<td>No 23</td>
</tr>
<tr>
<td>Assumed Pile Head Condition Hinged</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>Fixed 6</td>
</tr>
<tr>
<td></td>
<td>Partially Restrained 7</td>
</tr>
<tr>
<td>Utilize Approach Slab? Yes 20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No 9</td>
</tr>
<tr>
<td>Backfill Material Granular 20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Other 9</td>
</tr>
<tr>
<td>Maximum Skew Angle 2⁵ Unspecified 20</td>
<td></td>
</tr>
<tr>
<td>15° 5</td>
<td></td>
</tr>
<tr>
<td>30° 5</td>
<td></td>
</tr>
<tr>
<td>1 Not all respondents have designed jointless steel bridges</td>
<td></td>
</tr>
<tr>
<td>2 Measured from a line perpendicular to the longitudinal axis of the bridge</td>
<td></td>
</tr>
</tbody>
</table>

**TOLERABLE MOVEMENTS AND LENGTH LIMITATIONS**

The previously cited study(4), also published in 1983 in the Transportation Research Board *Transportation Research Record* 903(10), analyzed data collected on 420 bridges in 39 states and
the District of Columbia over an eight year period. The study showed that about 1/3 of the bridges had abutments that experienced horizontal movements. Half the full-height abutments moved horizontally, with movements ranging from 0.1 in to 8.0 in. (2.1 in. average). Twenty-six percent of the perched (stub- or sill-type) abutments experienced horizontal movements ranging from 0.3 in. to 14.4 in. (average 2.9 in.). Sixty-two percent of the spill-through abutments experienced movements ranging from 0.5 in. to 8.8 in. (2.4 in. average). Structural damage related to horizontal movements is described in the study as misalignment of bearings and superstructure, beams jammed against abutments, girders overrunning abutments (no backwall present), cracks in the superstructure (beams and slabs), damage to expansion joints, mislocation of steel bearings, deformed elastomeric pads, and cracking of bridge seats. Primarily, the reported damage was related to bearings and expansion joints and not to the abutments themselves. Approximately 2/3 of the damage was to joints and 1/3 of the damage was to bearings. Abutment damage from horizontal movement occurred with greater frequency for movements of 2 in. or greater. It should be noted, however, that none of the abutments sampled were designed to undergo any movements.

What can be deduced from the foregoing statistics? Despite the designers' intent to avoid damage from thermal effects by providing expansion joints and bearings, damage occurred 31 percent of the time at an average horizontal displacement of 2.55 in.

On the other hand, well over 1000 bridges with integral abutments have been constructed in the state of Tennessee alone, with calculated movements ranging from ¼ in. to 2 in. Properly designed, integral abutments can easily accommodate up to 2 in. of total movement and more.

**FICTION AND FACTS ABOUT INTEGRAL ABUTMENTS**

In a paper published in 1992, titled *Performance Evaluation of Integral Abutment Bridges*\(^{(11)}\), the results of a survey suggested many reasons for not utilizing jointless bridges.

The reasons, with rebuttal, are discussed below:

1. "*Increased earth load can cause abutment cracking.*" In reality, the components of integral abutments are substantially more rugged than abutments for jointed bridges. Details, similar to those found later in this chapter, have provided many years of service with essentially no abutment cracking. Experience and tests demonstrate that passive earth pressures are no match for the forces developed by the mobilized inertia of expanding bridge superstructures.

2. "*Skews greater than 20° (measured from a line perpendicular to the axis of a bridge) cannot be accommodated.*" The results of several surveys conducted between 1982 and 1992 show that five states regularly utilized jointless abutments for skews up to 30°, and one state has successfully utilized skews up to 70°. Observations indicate that sharp skews are problematic to bridges of all types, with or without joints. In a paper on movements and forces in skewed bridges\(^{(12)}\), a valid case was made that the force required to stabilize rotation of the abutments on a 30° skewed abutment approaches 50 percent of the passive pressure on the backwall; for a 45° skewed abutment, 70 percent of the total passive
pressure would be required. These values far exceed the frictional resistance of the backfill against the abutment backwall or the shearing resistance of the backfill. The deficit force must first be developed either by shear keys and/or bearings, and then be transmitted through the abutments, ultimately to the foundation system.

A paper presented by Dr. Charles Roeder and Shashi Moorty\textsuperscript{13} discusses further the phenomenon associated with skewed bridges; both tangent and curved. The report points out that when full-depth joints become clogged with dirt and debris, longitudinal thermal movements induce large transverse movements. Additionally, the orientation of the structure can cause erratic behavior. Even straight orthogonal bridges have experienced transverse movements. East-West oriented bridges in a northern latitude exhibit transverse movements due to the low angle of the sun, which can cause direct radiation and heating of the southernmost girder. This heating is sufficient to cause differential expansion across the bridge cross section. Further, North-South orientations at lower latitudes have caused bridges to experience similar problems from exposure to morning and evening sun. Lastly, wide skewed bridges which have joints can experience unrestrained transverse movements directly proportional to the bridge dimensions. Short, wide, skewed bridges will have unrestrained transverse movements nearly as large or larger than its longitudinal movements. Severe damage can occur as these unanticipated movements work on bearings, unidirectional joints, anchor bolts and surrounding concrete. As the structure softens due to distress, movements increase and damage worsens. These eventualities are not normally anticipated in design. Alternatively, utilizing integral abutment connections allows the entire backwall and deck to be mobilized to transmit the horizontal forces. Clearly, integral abutments provide a more rugged mechanism for transmitting the forces to the abutment support system. Further, the sharper the skew, the more desirable it is to use integral abutments.

Similarly, curved-girder bridges exhibit even more unpredictability than skewed bridges. It is unclear whether the primary thermal movements follow the chord between abutments, consecutive supports, or the local tangent at abutments. This uncertainty can lead to misaligned bearings and unidirectional joints resulting in damage, as described above. Field measurements\textsuperscript{13} have indicated that thermal expansion can sometimes be radial depending on the slenderness of the supporting piers. Again, integral abutments can mitigate any effects of miscalculated or unintended movements.

3. "Integral abutments can only be applied to short bridges." This may or may not be misunderstood depending on one's definition of short. Steel bridges up to 536 ft in length have been constructed. A review of the National Bridge Inventory System in 1992 revealed that 80 percent of all bridges in the NBIS are less than 180 ft in total length. About 90 percent are less than 400 ft in length. This means that most all bridges have the potential to be jointless, given the proper conditions of flexibility at the abutments.

4. "Cracks develop at the interface of abutment backwalls or approach pavements with the asphalt roadway paving. This causes a bump." With properly detailed approach pavements, discussed later, this will not be the case. For small movements, up to $\frac{1}{2}$ in. total, special details or approach pavements may not be needed. Further, the primary sources of a bump
are independent of whether or not joints are used at abutments and are independent of abutment support, i.e., piles or spread footings. Any abrupt or even small vertical differential movement over a relatively short horizontal distance can cause a bump. Eventually, when approach roadways are re-paved, a bump will be created, unless joints are raised at great expense. In contrast, subsequent paving may be carried over jointless bridges without interruption. One fact is certain; if there is a joint, there will be a bump.

5. "Integral abutment bridges are limited to pile-supported abutments." While pile-supported abutments are desirable, stub abutments with up to ¼ in. of total movements have been successfully used for spread footings supported on, but not keyed into rock. Additionally, tall abutments that are hinged at their base have been used for movements up to 2 in.

6. "Cranes cannot go close to abutments to place beams since backfill is placed after beams are set. Therefore, large booms are required." The most common method of construction, even for bridges with joints, is to set beams prior to pouring backwalls. Further, if the backwalls for expansion abutments are poured and backfilled before setting the beams, the abutments will be at a greater risk of displacement resulting from crane loadings than jointless abutments, which have no backwalls at the time of beam erection.

7. "Good details for tying approach slabs to the abutments are not available. Longer than normal approach pavements are required." Successful details will be presented later in this chapter. The length of approach pavements, usually between 15 and 25 ft, is generally the same as the length used for jointed abutments.

8. "Integral abutments limit future modifications, such as widening." Actually, integral abutments significantly simplify widenings. There are no expansion joints to match and no difficult temperature settings to make. As a rule, the connections between existing and widened abutments are more rugged.

9. "Cracks in the slab, end diaphragms or wingwalls are possible." Properly detailed, cracks should not exhibit themselves in end diaphragms or abutment wings; if cracks do occur, they will be minor. With respect to transverse cracks in slabs near abutments, it can be expected that the normal amount of slab cracking will occur, just as cracks will exhibit themselves over interior supports of any continuous structure. Where slabs are subject to tension, the design is based on a cracked-section analysis with respect to the slab.

10. "Wingwalls cannot be tied to the abutment." Most states utilize wings that are integral with the abutments in jointless bridges. Tennessee has made wings integral with the abutments for over thirty years. Wings up to 10 ft tall and 20 ft in length have been employed successfully in a cantilever configuration. It is recommended that integral wings be oriented parallel to the axis of the bridge in order to take maximum advantage of the bending strength of the wings, and to offer the least area exposed to passive soil resistance during the expansion phase.

11. "Erosion of the approach embankment caused by water intrusion can be a problem." This is a problem that can exhibit itself independent of whether the abutments are integral or
expansion type. The primary source of water intrusion is embankment settlement. Once settlement occurs, water is free to enter the fill via separations adjacent to the abutment wings and/or the backwall depending on wingwall orientation. This intrusion of water can also occur at abutments with or without approach pavements. It is true that the differential movement of the abutment backwall or approach pavement with respect to the roadway pavement can cause gaps through which water may pass. Water intrusion from all these sources can be mitigated with proper detailing. However, periodic maintenance may be required whether or not expansion joints are provided at abutments. Suggested details are presented later in this chapter.

INTEGRAL ABUTMENT DESIGN

While integral abutments have been used successfully for 50 years, their implementation has been anything but an exact science, but rather a matter of intuition, experimentation and observation. Inspection of many bridges with failed expansion bearings has revealed that anticipated catastrophic damage has not always occurred. The ability of bents and pile-supported abutments to accommodate thermal movements has often been underrated. Despite the lack of analytical tools, engineers have been pushing the envelope by constructing longer and longer jointless bridges, thus building on the lessons learned.

The reason that exact design approaches have not been fully developed is that the analysis of a pile under lateral loads is a problem in soil-structure interaction. Since the deflected shape of the loaded pile is dependent upon the soil response, and in turn, the soil response is a function of pile deflection, the system response cannot be determined by the traditional rules of static equilibrium. Further, soil response is a non-linear function of pile deflection. The ultimate problem for the structural engineer is the determination of the practical point of fixity of the buried pile.

In recent years, elasto-plastic soil/structure analysis tools have allowed engineers to better correlate mathematically what they have known to be achievable based on years of experience. Several methods have been developed that attempt to model soil-pile interaction\(^{14,15,16,17,18}\). However, the most promising method of analysis is found in Report No. FHWA-5A-91-048, COM624P - Laterally Loaded Pile Analysis Program For The Microcomputer, Version 2.0\(^{19}\).

The methods used in Reference 19 recognize that the solution to the problem of laterally loaded piles requires: 1) differential equations to obtain pile deflections and, 2) iteration, since soil response is a non-linear function of the pile deflection along the length of the pile. Further, the solutions presented recognize that as the backfill is acted upon for several cycles, it becomes remolded. Thus, an array of load-deflection, moment and shear conditions can be investigated.

Important to the solution is the development of a pseudo modulus of elasticity for the embankment soils that are acted upon by piles subjected to lateral loads. The most popular technique used in the United States is the \(p-y\) method. Using this procedure, pile response is obtained by an interactive solution of a fourth-order differential equation using finite-difference techniques. The soil response is described by a family of non-linear curves (\(p-y\) curves) that
compute soil resistance $p$ as a function of pile deflection $y$. A thorough discussion of the procedure can be found in Reference 19.

As an example of how an analysis can be carried out, consider the bridge identified in Figures 1a and 1b:

- Calculate the thermal movement demand:
  
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temperature Range</td>
<td>$0^\circ - 120^\circ$ F</td>
</tr>
<tr>
<td>Coefficient of Expansion</td>
<td>0.0000065/degrees F</td>
</tr>
<tr>
<td>Length of Structure</td>
<td>426 ft</td>
</tr>
</tbody>
</table>
  
  Total movement demand = $0.0000065(120)(426) = 0.332$ ft (4 in.)

  For a structure with equal intermediate bent stiffnesses, the movement demand will be equal; therefore, the movement at each end will be 2 in. With the girders set at a temperature of $60^\circ$ F, the required movement demand at each abutment will be $\pm 1$ in.

- Calculate the plastic-moment capacity of the embedded pile:

  HP 10 x 42 piling embedded 12 in. into the abutment beam is selected, with the piling oriented for bending about the strong axis. Two key calculations are required to assess the adequacy of the abutment and pile system to function as needed. As will be demonstrated later, the pile chosen must be able to develop a sizable resisting moment at the top in order to achieve double curvature within its design length under a 1 in. deflection. Using a free head condition at the top of the pile and analyzing the pile as a cantilever maximizes the pile capacity at a displacement of 0.68 in.

  HP 10 x 42 pile properties:

  - $F_y = 36$ ksi
  - $A_s = 12.4$ in.$^2$
  - $S_x = 43.4$ in.$^3$
  - $r = 4.13$ in.
  - $Z_x = 48.3$ in.$^3$
  - $b_t = 10.075$ in.

  Analysis of the fixed head condition, using the COM624P program, indicates plastic rotation of the pile will occur at a displacement of 0.68 in. Therefore, all calculations for pile capacity under combined loads, with displacements greater than 0.68 in., assign a maximum moment to the top of the pile (base of the abutment beam) as follows:

  $M_p = F_y Z_x$

  $= 36(48.3) = 1,738$ kip-in.
The second calculation centers on the ability to develop the plastic-moment capacity of the pile within the embedded length of the pile penetrating the cap. Research conducted by Burdette, Jones and Fricke\(^{(20)}\) determined that the concrete bearing capacity around large inserts can reach \(3.78 \text{f}_c'\) in bearing on average.

Referring to Figure 2, check the ability of the pile head to develop the plastic-moment capacity. Calculate the maximum couple developed by the concrete in compression over a width equal to that of the HP 10 x 42 flange:

Let \(f_{cb}\) represent the bearing strength of the concrete around the embedded pile head:

\[
C_1 = C_2 = 0.85 f_{cb} ab \\
= 0.85 f_{cb} (0.85)(6)(10.075) \\
= 43.68 f_{cb} \text{ in.}^2
\]

\[
Z = 12 - 5.1 = 6.9 \text{ in.} \\
\text{Couple} = 43.68 f_{cb} (6.9) \\
= 301.4 f_{cb} \text{ in.}^3
\]

Determine \(f_{cb}\) required to develop the plastic-moment capacity in the pile:

\[
M_p = 301.4 f_{cb} \text{ in.}^3 \\
1,738 = 301.4 f_{cb} \\
f_{cb} = 5.77 \text{ ksi}
\]

Assuming \(f_c' = 3.0 \text{ ksi}\):

\[
\frac{f_{cb}}{f_c'} = \frac{5.77}{3.0} = 1.92 << 3.78
\]

Therefore, the plastic-moment capacity can be developed in the pile head with a safety factor of nearly 2.0.

Having established the ability to develop the plastic-moment capacity of the HP 10 x 42 pile at its top, the COM624P program is utilized to develop the deflected shape of the pile under specified conditions. Figure 3 is a schematic representation of the soil profile through which the abutment piles are driven, as well as the loads to be applied to the abutment/pile system. The resulting \(p-y\) curves for a 1 in. displacement of the HP 10 x 42 piling, measured at the ground line, are shown in Figure 4. Curves for points on the pile measured 60 in. and 120 in. below the ground line are depicted for comparison. Predicted pile deflections versus depth of pile penetration, for specified magnitudes of displacement at the ground line, are shown in Figure 5 (1 in. and \(1\frac{1}{2}\) in. displacements are depicted for comparison). The corresponding pile moments versus depth, corresponding to the displacements shown in Figure 5, are plotted in Figure 6. Also shown in Figure 6 are the unbraced lengths of pile to be investigated in the capacity...
calculations that follow. These unbraced lengths are determined from identification of the points of zero moment at varying depths of pile embedment.

Table 1 contains the applicable group load cases for the abutment piles of the example bridge. These are the loads applicable only to the length of pile under investigation between points of zero moment, as identified in Figure 6. It should be noted that the thermal moments and resulting factored loads are unique to the soil profile shown in Figure 3, as well as to the 1 in. and 1½ in. lateral displacements of the abutment and pile head.

The resulting factored loads for the various group loading conditions will be compared to the column interaction diagram for the pile that must be generated. Using the Load Factor Design provisions of the AASHTO Standard Specifications for Highway Bridges\(^{(21)}\), determine the column interaction diagram for an HP 10 x 42 pile oriented for bending about the strong axis.

- Calculate the column capacities from AASHTO Article 10.54 and develop the resulting interaction diagram for an unbraced length of \(L_c = 12.1\) ft:

\[
\frac{KL_c}{r} \leq \sqrt{2\pi^2 \frac{E}{F_y}} \tag{10-152}
\]

\[
\frac{KL_c}{r} = \frac{0.875(12.1)(12)}{4.13} = 30.76
\]

\[
\sqrt{2\pi^2 \frac{E}{F_y}} = 126.1 \geq 30.76
\]

\[
\therefore F_{cr} = F_y \left[1 - \frac{F_y}{4\pi^2 E} \left(\frac{KL_c}{r}\right)^2\right] \tag{10-151}
\]

\[
F_{cr} = 36 \left[1 - \frac{36}{4\pi^2 (29,000)} (30.76)^2\right] = 34.93 \text{ ksi}
\]

\[
P_u = 0.85A_s F_{cr} \tag{10-150}
\]

\[
P_u = 0.85(12.4)(34.93) = 368.2 \text{ kips}
\]

\[
M_p = F_y Z_x = 36(48.3) = 1,738 \text{ kip-in}
\]

\[
M_u = F_y S_x = 36(43.4) = 1,562 \text{ kip-in}
\]

\[
F_e = \frac{E\pi^2}{\left(\frac{KL_c}{r}\right)^2} \tag{10-157}
\]

\[
F_e = \frac{(29,000)\pi^2}{(30.76)^2} = 302.5 \text{ ksi}
\]

Using the foregoing values and Equations (10-155) or (10-156), the interaction diagram for an HP 10 x 42 pile oriented for bending about the strong axis is created and is shown in
Figure 8. Superimposed on this interaction diagram are the group loadings from Table 1. The data points used to develop the interaction diagram are summarized in Table 2.

As demonstrated from an examination of Figure 8, the HP 10 x 42 piling is capable of withstanding a 1 in. displacement at its head and performing adequately as a compression member in bending when driven through stiff clay. It will be noted, however, that the example requires that the abutment be cast integrally at or very near a beam temperature of 60° F in order to function as designed. This may not be a condition for which the designer can or wishes to exercise control. A wider temperature range over which the casting of the integral abutment can be accomplished is attainable, but at a price. The alternatives are either to select a larger pile or to reduce the vertical loading by increasing the total number of piles. In this example, the use of four extra piles was investigated increasing the total number from seven to eleven. The COM624P program was used to develop the geometric shape of the HP 10 x 42 piling for a 1½ in. displacement in either direction. From the results, a revised column interaction diagram, shown in Figure 8a, was produced. As can be seen, satisfactory performance can be achieved by adding the four piles. In doing so, the temperature range over which the abutment may be cast has been extended to 60° F; that is, from 30° F to 90° F.

To provide an overview of the ability of HP 10 x 42 piling to perform satisfactorily in strong-axis bending, other soil types — listed in Table 3 — were substituted for the top 20 ft of material shown in Figure 3. Figures 9 through 11 show the resulting interaction diagrams. Note that only piles driven through hard clay (Figure 10) failed to provide satisfactory performance. Where hard clay is anticipated to be used as roadway embankment, the pile would need to be driven through a pre-bored hole approximately twice the diameter of the pile. The pre-bored hole is recommended to be 8 ft in depth below the abutment beam and backfilled with loose sand.

- Calculate the adequacy of the backwall to resist passive pressure due to expansion, which can be conservatively accomplished by assuming a uniformly increasing load applied to a simple beam (see AISC Beam Diagrams and Formulas, Case 2(22)).

Referring to Figure 7, the simple span is defined from the bottom of the approach slab to the bottom of the abutment beams; in this case, a distance of 9.8 ft.

\[
P_p = \frac{1}{2} \gamma h^2 \left[ \frac{1 + \sin \phi}{1 - \sin \phi} \right]
\]

\[
= \frac{1}{2} (130)(9.8)^2 \left[ \frac{1 + \sin 35^\circ}{1 - \sin 35^\circ} \right]
\]

\[
= 23.03 \text{ kips/ft of wall}
\]

\[
P_u = 1.3(23.03) = 29.97 \text{ kips/ft of wall}
\]

Check vertical wall bending:
\[ M_u = \left[ \frac{2 P_u l}{9 \sqrt{3}} \right] = 0.13 P_u l \]
\[ = 0.13(29.97)(9.8) = 38.18 \text{ kip-ft} \]

From calculations not shown,
\[ \phi M_n = 46.48 \text{ kip-ft} > 38.18 \text{ kip-ft} \quad \text{ok} \]

Check horizontal bending of the backwall between beams:

Referring to Figures 12 and 13:

\[ M_u = \frac{P_u l^2}{12} = \frac{29.97(11.75)^2}{12} \]
\[ M_u = 344.8 \text{ kip-ft} \]
\[ b = 9.8(12) = 117.6 \text{ in.} \]
\[ d = 33.19 \text{ in.} \]
\[ A_s = 10 - \#6 \text{ bars} \]
\[ A_s = 10(0.44) = 4.4 \text{ in.}^2 \]

From calculations not shown:
\[ \phi M_n = 6485 \text{ kip-ft} > 344.5 \text{ kip-ft} \quad \text{ok} \]

Check shear:

\[ V_u = \frac{P_u l}{2} = \frac{29.94(11.75)}{2} = 175.9 \text{ kips} \]
\[ V_c = 2 \sqrt{f_c} b_w d = 2 \sqrt{3000(117.6)(33.19)} / 1000 \]
\[ V_c = 427.6 \text{ kips} \]
\[ \phi V_c = 0.85(427.6) = 363.4 \text{ kips} > V_u = 175.9 \text{ kips} \quad \text{ok} \]

**INTEGRAL ABUTMENT DETAILS**

Components of jointless bridges generally are subjected to the same forces as other continuous bridges with expansion joints at their ends. Exceptions to this rule apply only when integral abutments are tall and the structure is designed as a frame.

The most desirable end conditions for an integral abutment are the stub or propped-pile cap type (Figure 13), which provides the greatest flexibility and hence, offers the least resistance to cyclic thermal movements. Under these conditions, only the abutment piling and wings are subjected to
higher stresses. These stresses have proved through the years to have not caused unacceptable distress.

Using the pile-supported stub-type abutment, steel-girder bridges up to 400 ft in length may be easily constructed. Longer steel bridges may be constructed, with due consideration given to the forces and movements involved. The details and discussions in this section, however, are specifically applicable to overall lengths of structure up to 426 ft.

**Pile Configuration**

Piles driven vertically and in only one row are highly recommended. In this manner, the greatest amount of flexibility is achieved to accommodate cyclic thermal movements. Likewise, in seismic events, the dampening forces are engaged to the largest extent by the embankment backfill rather than by the cap and piling, which will reduce the damage resulting from large displacements.

**Pile Orientation**

A survey taken in 1983\(^9\) demonstrated that states differ in opinion and practice with regard to pile orientation. Fifteen states orient the piling so that the direction of thermal movement causes bending about the strong axis of the pile. Thirteen others orient the piling so that the direction of movement causes bending about the weak axis of the pile. Both methods have proven to be satisfactory to the respective agencies. Orienting the piling for weak-axis bending offers the least resistance and facilitates pile-head bending for fixed head conditions. However, due to the potential for flange buckling, the total lateral displacement that can be accommodated is more limited than when the piling is oriented for strong-axis bending.

**Anchorage of Beams to Pile Cap**

Steel beams, being more sensitive to temperature changes than concrete beams, should be connected to the pile caps with anchor bolts prior to making integral connections. Fortunately, steel beams are easily adaptable to these connections.

Two details have been used successfully. The first involves placing the beams on \(\frac{1}{4}\) in. plain elastomeric pads (Figure 14). Anchor bolts pass from the abutment pile cap through both the pad and the bottom flange of the beam or girder. The second method uses taller projecting anchor bolts equipped with double nuts; one above and one below the flange (Figure 15). The latter method provides better control over the grade of the beam and requires less precision in preparing the bridge seats of the pile cap.

Both details provide a very desirable feature in that the superstructure and pile cap can move together, avoiding damage to the freshly poured concrete when the integral connection is made to lock the superstructure and abutment together. It is also recommended that a portion of the reinforcing bars located in the front face of the abutment pass continuously through the girder webs as shown in Figures 14 and 15.
Approach Pavements

Due to the difficulties in obtaining proper embankment and backfill compaction around abutments, approach pavements are recommended; especially for new construction. Approach pavements offer many benefits other than acting as a bridge between the abutment and more densely compacted embankments. Approach pavements provide a transition from the approach to the bridge if embankment settlement occurs. Such transitions provide a smooth ride, thereby reducing impact loads to the bridge. Approach pavements also provide greater load distribution at bridge ends, which aids in reducing damage to the abutments; especially from overweight vehicles. Finally, properly detailed approach pavements help control roadway drainage, thus preventing erosion of the abutment backfill or freeze/thaw damage resulting from saturated backfill.

The approach slab must be anchored into the abutment backwall so that it moves in concert with the bridge. Otherwise, cyclic expansions will force the slab to move with the bridge without a mechanism to pull it back when the bridge contracts. As debris fills the resulting opening, repeated cycles will ratchet the slab off its support. The anchorage used to fasten the approach slab should be detailed to act as a hinge so that the slab can rotate downward without distress as the embankment settles. Figures 16a through 16d depict desirable features of approach pavements.

Backfill

The survey discussed earlier (9) indicated that porous, granular backfill is used by 75 percent of the respondents. The selection of this type of backfill offers two benefits: 1) such material is more easily compacted in close spaces, and 2) the material aids in carrying any water intrusion away from the abutments. Well-graded material is desirable. Uniformly graded material does not compact well and provides less interlocking of particles, thus acting more like marbles.

Drainage

The use of a vertical stone column about two feet in width is recommended, with a height reaching from the bottom of the abutment beam or pile cap to the top of the roadway subgrade. This drain should be placed between the abutment backwall and the embankment backfill and should wrap around the backwall — between the parallel wingwalls and the roadway embankment — since any settlement of the approach pavement will create a gap through which surface runoff will flow. A perforated drain pipe, overlying an impervious layer of soil or plastic, should be placed at the base of the vertical stone column and should be sloped to provide drainage away from the abutment area.

Provisions for Expansion

In all cases where the approach roadway or a ramp is constructed of concrete, provisions for an expansion joint must be provided. Where the anticipated total movement at an abutment exceeds
½ in. and the approach roadway is asphalt, an expansion joint should be considered. The reason for the latter is that larger movements can damage asphalt adjacent to the end of the approach pavement in the expansion cycle. During the contraction phase, a significant gap is created through which water can infiltrate the subgrade. If regular maintenance can be arranged to fill this gap with a suitable joint sealer in cold weather, no joint will be needed.

If expansion joints are provided, the joints should only be located at the roadway end of the approach pavement. It is a certainty that the joint system will fail at some future time. If the joint is located between the abutment backwall and the approach pavement, then the slab jacking process mentioned above will occur.

It is recommended that joints similar to the one detailed in Figure 16d be used, and not joints that contain metal hardware for anchorage. This will avoid the problem of replacing or raising the joint should subsequent paving projects dictate that an overlay be placed on the bridge. The joint shown in Figure 16d may simply be replicated in the same manner in which it was originally installed atop the existing joint.

CONSTRUCTION SEQUENCE

The following sequence is recommended when constructing steel bridges with integral abutments to reduce the effects of thermal movements on fresh concrete and to control moments induced into the supporting pile system:

1. Drive the piling and pour the pile cap to the required bridge seat elevation. Install one of the desired anchoring systems described earlier. Pour the pile caps for the wingwalls concurrently.

2. Set the beams/girders and anchor them to the abutment. Slotted holes in the bottom flanges are recommended to aid in the erection since the temperature will vary from the time that the anchors are set in the cap to the time that the girders are fully erected. Do not fully tighten the anchor nuts at this time; instead, leave free play for further dead-load rotations.

3. Pour the bridge deck in the desired sequence excluding the abutment backwall/diaphragm and the last portion of the bridge deck equal to the backwall/diaphragm width. In this manner, all dead-load slab rotations will occur prior to lock-up, and no dead-load moments will be transferred to the supporting piles.

4. Tighten the anchor nuts and pour the backwall/diaphragm full height. Since no backfilling has occurred to this point, the abutment is free to move without overcoming passive pressures against the backwall/diaphragm. The wingwalls may also be poured concurrently.

5. Place the vertical drain system and backfill in 6-in. lifts until the desired subgrade elevation is reached. Place a bond breaker on the abutment surfaces in contact with the approach pavement.
6. Pour the approach pavement starting at the end away from the abutment and progressing toward the backwall. If it can be so controlled, approach pavements should be poured in the early morning so that the superstructure is expanding, and therefore, not placing the slab in tension.
REFERENCES


Other references of interest:


DENOTES LOCATIONS OF PLOTTED P-Y CURVES.

STIFF CLAY
C = 11 lb/in^2
E50 = 0.005
K = 1000 lb/in^3
y = 124 lb/ft^3

20' = 240 in.

STIFF CLAY
C = 5 lb/in^2
E50 = 0.01
K = 300 lb/in^2
y = 124 lb/ft^3

Water Table 32' = 384 in.

SAND
\( \phi = 33^\circ \)
K = 92 lb/in^3
y = 52 lb/ft^3

40' = 480 in.

SAND
\( \phi = 29^\circ \)
K = 40 lb/in^3
y = 52 lb/ft^3

60' = 720 in.

SOIL PROFILE

Figure 3
Deflection (Inches)

Depth (Inches) (100's)

M_{top} = M_{p} = 144.9 \text{ k-ft}

Figure 5
STEEL COLUMN INTERACTION DIAGRAM
LOAD FACTOR ANALYSIS
HP 10 X 42 PILE  X-X AXIS

STIFF CLAY
(Refer to Figure 3)

AXIAL LOAD (K)

MOMENT (K-FT)

UNBRACED LENGTH =  12.1  FT.

Figure 8
STEEL COLUMN INTERACTION DIAGRAM
LOAD FACTOR ANALYSIS
HP 10 X 42 PILE  X-X AXIS

AXIAL LOAD (K)

MOMENT (K-FT)

UNBRACED LENGTH = 15.7 FT.

Figure 9
STEEL COLUMN INTERACTION DIAGRAM
LOAD FACTOR ANALYSIS
HP 10 X 42 PILE  X-X AXIS

AXIAL LOAD (K)

MOMENT (K-FT)

UNBRACED LENGTH = 8.9 FT.

Figure 10
STEEL COLUMN INTERACTION DIAGRAM
LOAD FACTOR ANALYSIS
HP 10 X 42 PILE  X–X AXIS

AXIAL LOAD (K)

MOMENT (K–FT)

UNBRACED LENGTH = 12.6 FT.

Figure 11
Figure 12
Figure 13
Figure 16a

PLAN
(90° SKEM)

Figure 16b

SECTION B - B

NOTE: SLAB TO BE POURED DIRECTLY ON MINERAL AGGREGATE BASE STONE.
LIMITS OF MINERAL AGGREGATE

SECTION A - A

*NOTE: WHEN BRIDGE END DRAINS ARE REQUIRED, ANY REINFORCING STEEL INTERFERING WITH BRIDGE END DRAIN SHALL BE CUT IN FIELD.

NOTE: TOP OF SLAB TO CONFORM TO ROADWAY SLOPE AND GRADE.

Figure 16c

JOINT SEAL SYSTEM

BACKER ROD SHALL BE A CLOSED CELL NON-GASSING FOAM MATERIAL CAPABLE OF WITHSTANDING ELEVATED TEMPERATURE RESULTING FROM THE REACTION OF THE TWO COMPONENT JOINT SEAL SYSTEM.

2 LAYERS FELT PAPER

2"x8" STYROFOAM FORMING STRIP

SECTION C - C

Figure 16d
LOAD FACTOR GROUP LOADINGS

SERVICE LOADS FROM DESIGN NOTES:

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<th>P (K)</th>
<th>M (K-FT)</th>
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<td>DEAD LOAD</td>
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APPLICABLE AASHTO GROUP LOADINGS:

GROUP I = 1.3*(DL + 1.67 * LL)
GROUP IV = 1.3*(DL + LL + (R+S+T))
GROUP V = 1.25*(DL + (R + S + T))
GROUP VI = 1.25 *(DL + LL + (R + S + T))

FACTORED LOADS:

STIFF CLAY:

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<tr>
<td>V</td>
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<td>VI</td>
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<tr>
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HARD CLAY:

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FACTORED LOADS

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SAND:

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FACTORED LOADS

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Table 1
\[ \text{Mp} = FY'Z = 1,738,800 \quad \text{IN-LB} = 144.9 \quad \text{K-FT} \]

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<td>73,629.8</td>
<td>1,225,360.6</td>
<td>1,401,388.5</td>
<td>1,225,360.6</td>
<td>102.12</td>
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<td>55.2</td>
<td>55,222.3</td>
<td>1,308,485.1</td>
<td>1,485,741.4</td>
<td>1,308,485.1</td>
<td>109.04</td>
</tr>
<tr>
<td>36.8</td>
<td>36,814.9</td>
<td>1,392,356.6</td>
<td>1,570,094.3</td>
<td>1,392,356.6</td>
<td>116.03</td>
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<tr>
<td>18.4</td>
<td>18,407.4</td>
<td>1,476,994.9</td>
<td>1,654,447.1</td>
<td>1,476,994.9</td>
<td>123.08</td>
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<td>0.0</td>
<td>0.0</td>
<td>1,562,400.0</td>
<td>1,738,800.0</td>
<td>1,562,400.0</td>
<td>130.20</td>
</tr>
</tbody>
</table>

Table 2
LATERALLY LOADED PILES
USING COM624P PROGRAM

COMPARISON OF RESULTS

FOR 1° DEFLECTION:

HP 10 X 42 PILE

<table>
<thead>
<tr>
<th>SOIL TYPE</th>
<th>C or Phi (psi or deg)</th>
<th>k (pci)</th>
<th>E50</th>
<th>LATERAL FORCE (k)</th>
<th>MAXIMUM MOM. (K FT)</th>
<th>DIST. TO FIXITY (FT)</th>
<th>UNBRACED LENGTH (FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STIFF CLAY</td>
<td>11 psi</td>
<td>1000</td>
<td>0.005</td>
<td>41.7</td>
<td>62.3</td>
<td>12.4</td>
<td>12.1</td>
</tr>
<tr>
<td>MED. CLAY</td>
<td>4 psi</td>
<td>100</td>
<td>0.010</td>
<td>25.9</td>
<td>38.4</td>
<td>17.5</td>
<td>15.7</td>
</tr>
<tr>
<td>HARD CLAY</td>
<td>3 psi</td>
<td>800</td>
<td>0.004</td>
<td>94.6</td>
<td>125.8</td>
<td>8.3</td>
<td>8.9</td>
</tr>
<tr>
<td>SAND</td>
<td>29 deg</td>
<td>50</td>
<td>---</td>
<td>36.8</td>
<td>68.7</td>
<td>11.5</td>
<td>12.6</td>
</tr>
</tbody>
</table>

NOTES: 1. LATERAL FORCE IS THE FORCE REQUIRED TO PRODUCE 1° DEFLECTION AT PILE HEAD.
2. UNBRACED LENGTH IS MEASURED BETWEEN POINTS OF MOMENT CONTRAFLEXURE.
3. MAXIMUM MOMENT IS BELOW PILE HEAD, WITHIN THE UNBRACED LENGTH.

Table 3