CE 690

STATE-OF-THE-ART METHODS FOR DESIGN OF INTEGRAL BRIDGE ABUTMENTS

Submitted To:

Amde M. Wolde-Tinsae, Associate Professor
Department of Civil Engineering
Iowa State University

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By:

Bruce Johnson, Assistant Bridge Engineer
Federal Highway Administration
Iowa Division

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A = Empirical coefficient

a = Empirical constant

a_o = Horizontal angle of initial portion of a load-slip curve

B = Empirical coefficient

b = Pile width

C = Constant based on soil properties

c = Soil shear strength

d = Pile diameter

E = Elastic modulus

H = Depth below which the soil response is unaffected by the ground surface boundary

I = Pile moment of inertia about the loaded axis

K_a = Rankine coefficient of minimum active earth pressure

K_o = Coefficient of earth pressure at rest

k = Modulus of horizontal subgrade reaction

L = Portion of the bridge length affecting thermal expansion at one abutment

l_e = Effective length of the pile

M = Applied moment in the pile

$M(x)$ = Moment along the length of the pile

$M(l_e)$ = Moment at the point of fixity

m = Slope of the intermediate portion of a p-y curve for sandy soils

N = Standard penetration blowcount

N_c = Dimensionless bearing capacity factor

N_q = Dimensionless bearing capacity factor

n = Empirical constant

n_h = Constant of horizontal subgrade reaction

P = Lateral load at the pile top

P_m = Lateral soil resistance at a lateral deflection (y) of b/60

P_u = Ultimate lateral soil resistance

p = Soil resistance

p_p = Pile perimeter

Q = Axial pile load

q_f = Ultimate soil resistance

q_o = Effective vertical stress at the pile tip

x = Depth measured from the ground surface

y = Lateral deflection of the pile

y_k = Maximum deflection of the elastic portion of a p-y curve in sand

y_m = Maximum deflection of the parabolic portion of a p-y curve in sand

y_u = Ultimate lateral deflection

α = Coefficient of thermal expansion

β = Rankine angle of passive earth pressure

Δ = Lateral deflection at the top of the pile

δT = Allowable temperature drop of rise

ϵ = Soil strain in a standard triaxial test

γ = Average effective unit weight of a soil from the surface to depth (x)

ϕ = Soil friction angle

ρ = Vertical pile settlement

σ = Stress in the extreme fibers of the pile

τ_{max} = Ultimate soil shear resistance

θ = Rotation at the pile top

I. INTRODUCTION

1. Background

The routine use of integral abutments to tie bridge superstructures to foundation piling began in this country about 30 years ago.[19] Kansas, Missouri, Ohio, North Dakota, and Tennessee were some of the early users. This method of construction has steadily grown more popular. Today more than half of the state highway agencies have developed design criteria for bridges without expansion joint devices.

Most of the states using integral abutments began by building them on bridges less than 100 feet long. Allowable lengths were increased based on good performance of successful connection details. Full-scale field testing and sophisticated rational design methods were not commonly used as a basis for increasing allowable lengths. This led to wide variations in criteria for the use of integral abutments from state to state. In 1974 the variation in maximum allowable length for concrete bridges using integral abutments between Kansas and Missouri was 200 feet.[19] A survey conducted by the University of Missouri in 1973 indicated that allowable lengths for integral abutment concrete bridges in some states were 500 feet while only 100 feet in others.

The primary purpose for building integral abutments is to eliminate bridge deck expansion joints, thus reducing construction and maintenance costs. A sketch of a bridge with integral abutments is shown in FIGURE 1. Conventional bridge bearing devices often become ineffective and are susceptible to deterioration from roadway runoff through deck joints which are open or leak. A cross-section of a bridge with stub abutments and deck joints is shown in FIGURE 2.

CROSS-SECTION OF A BRIDGE WITH INTEGRAL ABUTMENTS

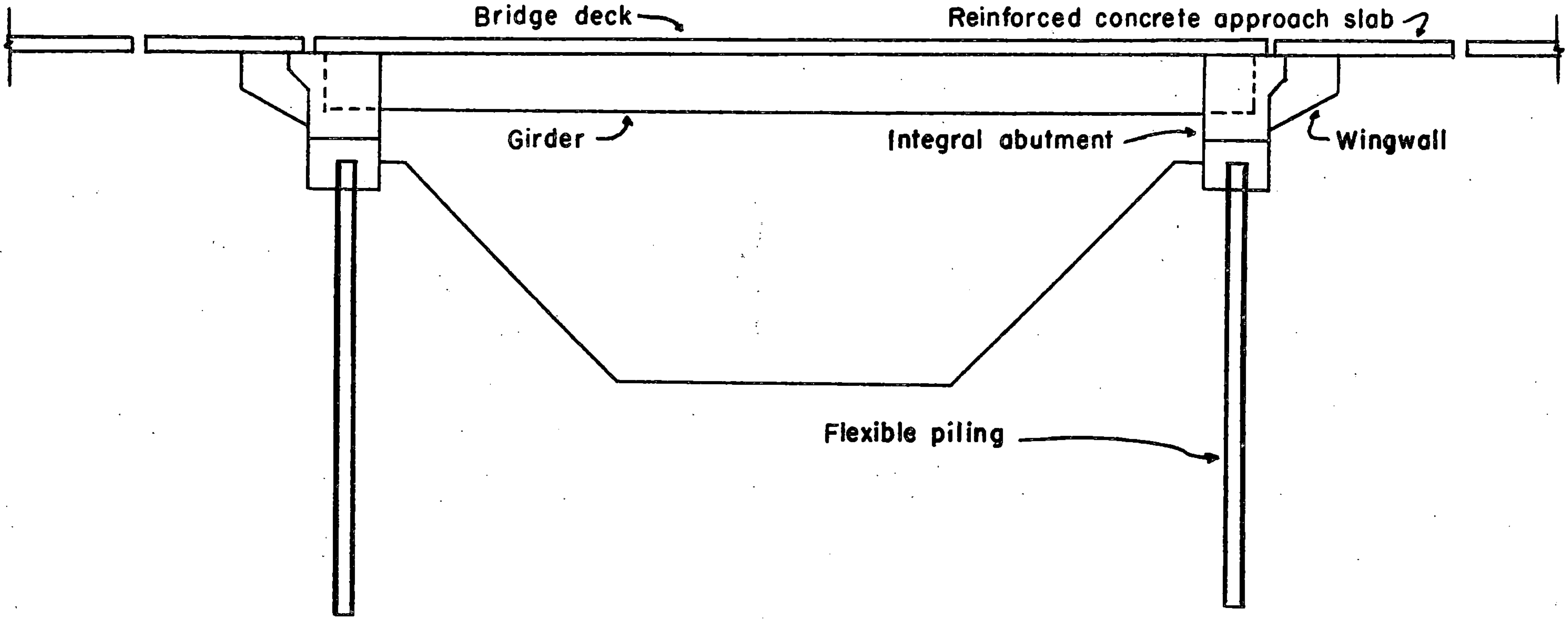


FIGURE 1

CROSS-SECTION OF A BRIDGE WITH EXPANSION JOINTS

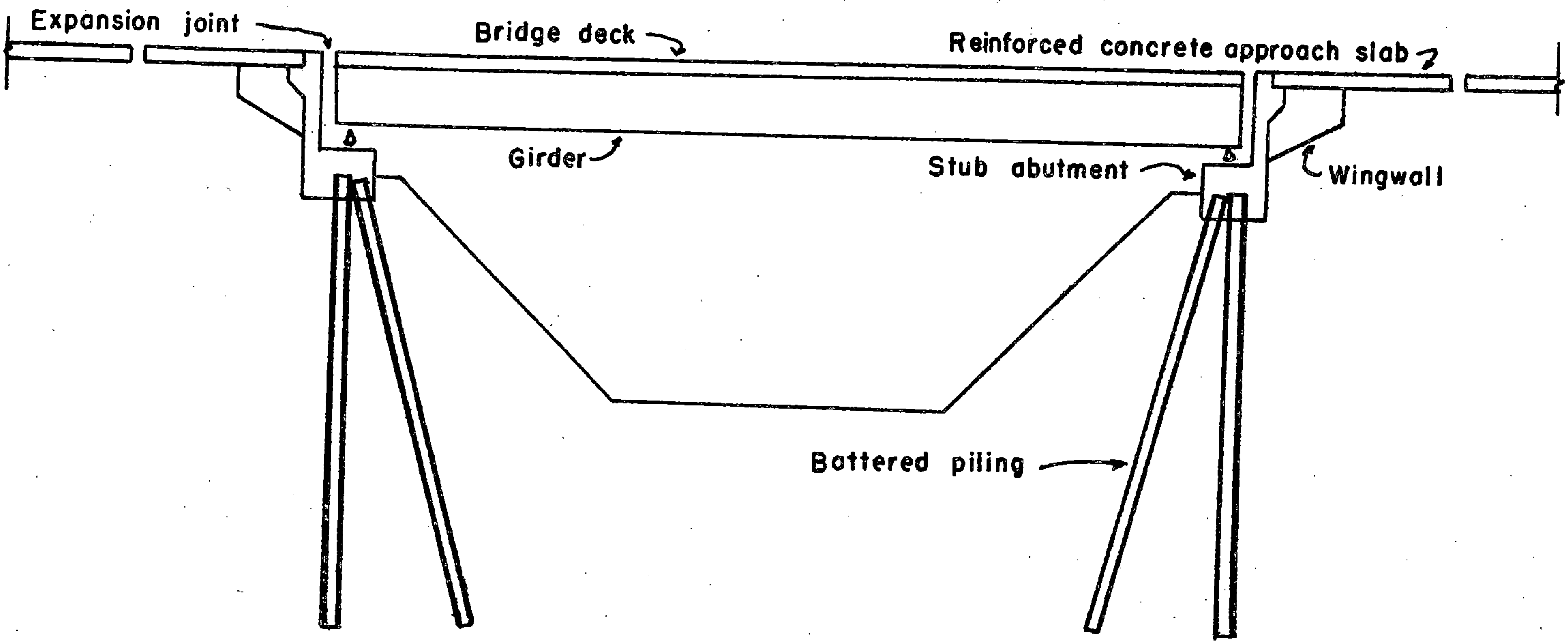


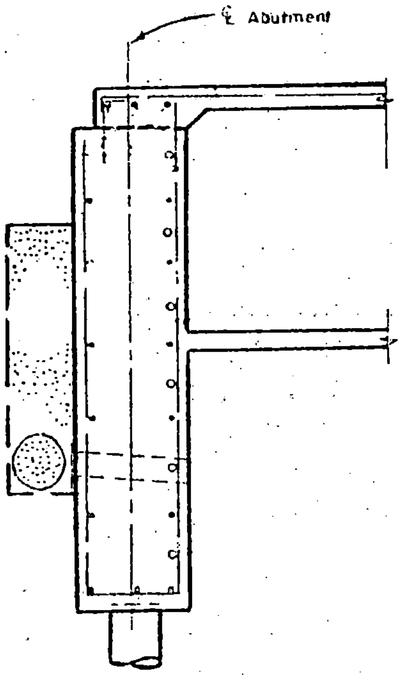
FIGURE 2

In an integral abutment bridge with flexible piling, the thermal stresses are transferred to the substructure via a rigid connection. Various construction details have been developed to accomplish the transfer as shown in FIGURE 3. The abutments contain sufficient bulk to be considered a rigid mass. A positive connection to the girder ends is generally provided by vertical and transverse reinforcing steel. This provides for full transfer of temperature variation and live load rotational displacements to the abutment piling.

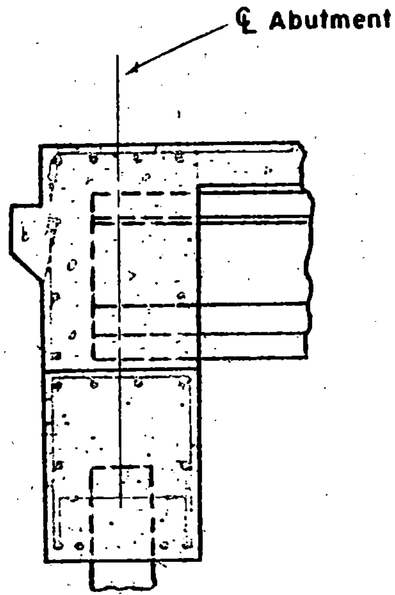
The semi-integral abutments shown in FIGURE 4 are designed to minimize the transfer of rotational displacements to the piling. They do transfer horizontal displacements, and they also allow elimination of the deck expansion joints. Rotation is generally accomplished by using a flexible bearing surface at a selected horizontal interface in the abutment. Allowing rotation at the pile top generally reduces pile loads.

The stresses in the abutment piling are dependent on the axial load (Q), lateral load at the top of the pile (P), rotation (θ) allowed at the abutment, stiffness (EI) of the pile, and resistance (p) of the soil (see FIGURE 5). Various simplifying assumptions can be made to allow a routine mathematical analysis of the system to be developed. An elastic solution based on statics can be obtained by assuming p = 0 and fixing the pile at some effective length (l_e) (see FIGURE 6). The point of fixity is assumed such that the lateral load-deflection response at the pile top is similar to that of the actual case considering soil support. Lengths of 10 feet and 10.5 feet have been used by some state highway agencies.[38,14] By assuming that the abutment is free to rotate and that the moment due to the axial load (Q) is very small compared to the bending moment caused by the lateral

INTEGRAL ABUTMENT DETAILS



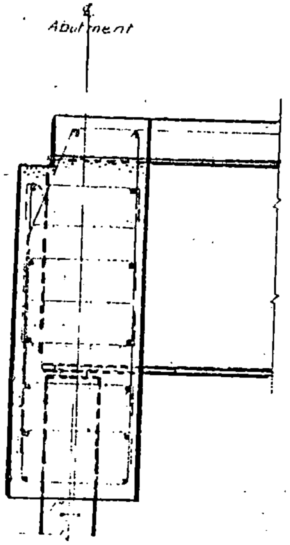
CALIFORNIA[6]



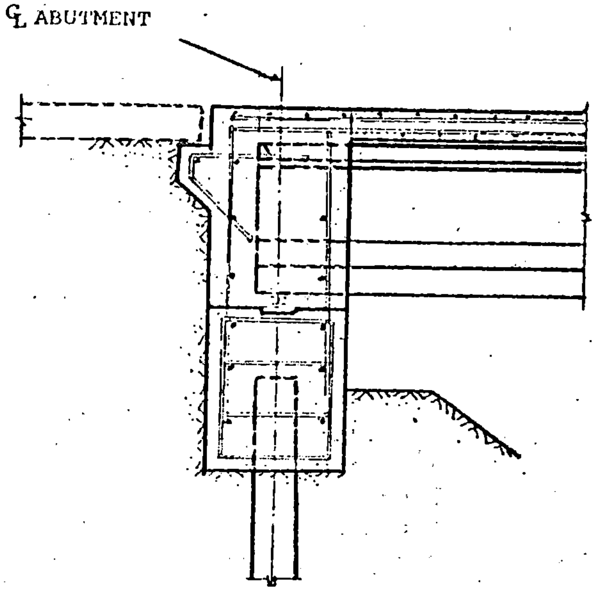
MISSOURI[5]

FIGURE 3

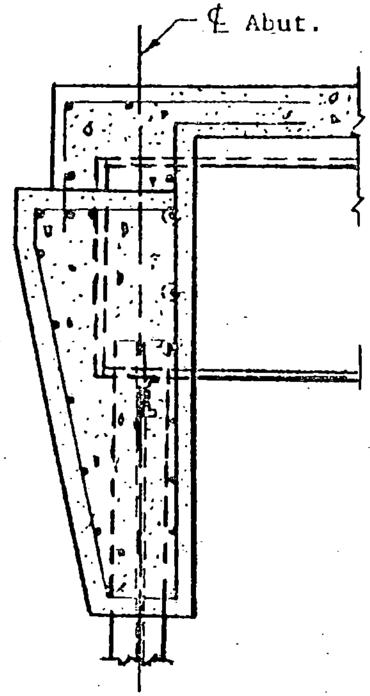
INTEGRAL ABUTMENT DETAILS



SOUTH DAKOTA[19]



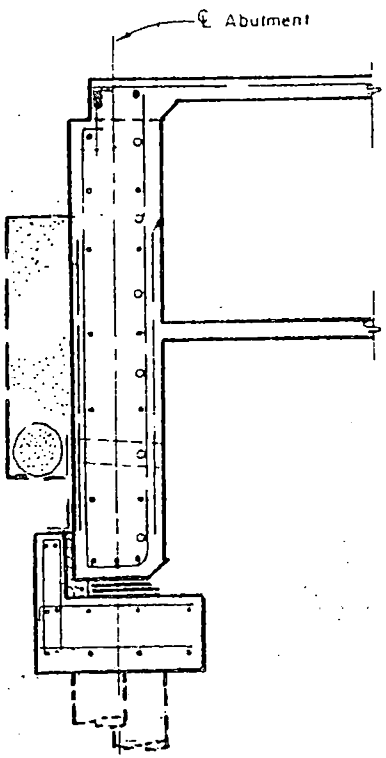
IOWA[13]



MISSOURI[12]

FIGURE 3 (CONT.)

SEMI-INTEGRAL ABUTMENT DETAILS



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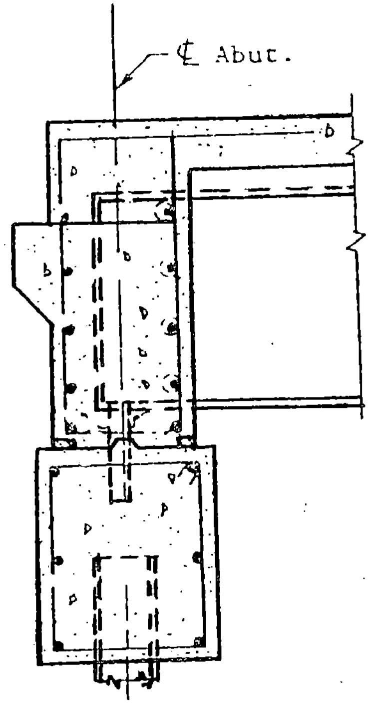


MISSOURI[12]

FIGURE 4

INTEGRAL ABUTMENT PILE LOADS

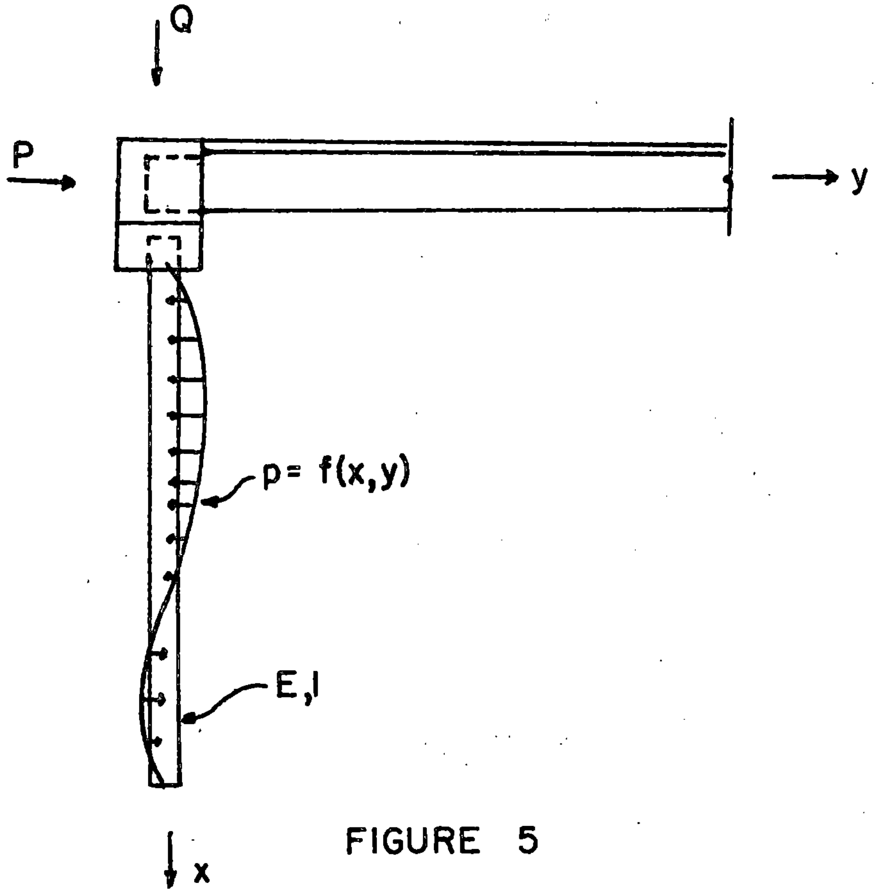


FIGURE 5

SIMPLIFIED PILE STRESS ANALYSIS

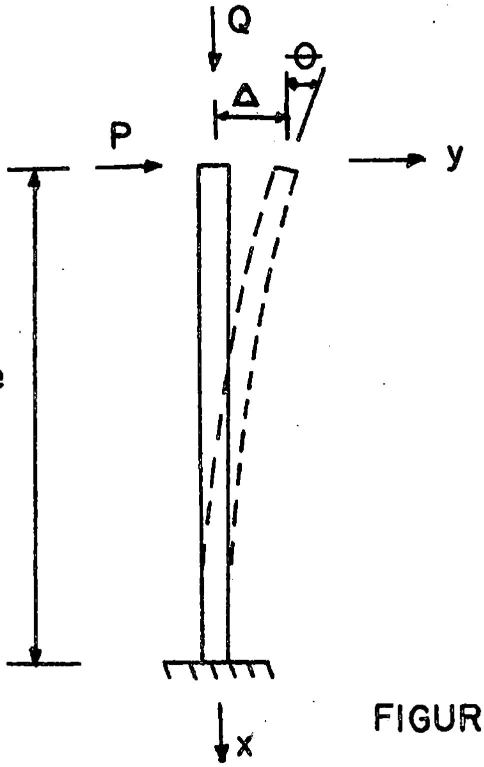


FIGURE 6

load (P), the following expressions result:

$$\Delta = Pl_e^3/3EI$$

$$M(x) = Px + Q(\Delta - y) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$M(l_e) = Pl_e = 3EI\Delta/(l_e)^2$$

Where:

$M(x)$ = Moment along the length of the pile

$M(l_e)$ = Moment at the point of fixity

x = Depth from the ground surface

y = Lateral deflection of the pile

Δ = Lateral deflection at the top of the pile

E = Elastic modulus of the pile

I = Pile moment of inertia about the loaded axis

In Iowa HP 10 x 42 steel piles are used predominantly in integral abutments with a 6.0 ksi vertical design load on bridges over 200 feet long. As an example, the stress in an HP 10 x 42 pile will be calculated ignoring soil support for an embedment length of 10 feet and a lateral deflection of 1 inch. The last two criteria are used by Tennessee to establish maximum allowable bridge lengths using integral abutments.

$$M(l_e) = 36.1 \text{ Ft-Kips}$$

$$\sigma = My/I + Q/A = 3Ey/(l_e)^2 + Q/A \quad \ldots (2)$$

$$\sigma = 30.4 + 6.0 = 36.4 \text{ ksi}$$

As shown by EQUATION 2 the piling stress can be decreased by minimizing the cross-sectional width of the pile. The stress for the next size smaller pile, an HP 10 x 36 (with $y = 4.079$), is 30.5 ksi. Changing the fixity condition at the pile top from "free" to "fixed" substantially increases the calculated stresses for a given lateral deflection at the top.

These simplified elastic equations indicate that the pile stresses are in the elastic range for movements of about 1 inch. A recent study in North Dakota included monitoring deflections in a 450-foot concrete box beam bridge. The total maximum movement including contraction and expansion was found to be about 2 inches at each abutment. When the soil resistance is included in the analysis, the calculated stress is reduced but still can be above yield.

The limit of allowable horizontal movement which will cause objectionable pile stresses has not been well defined. This is one reason why the wide variation in design criteria exists among the state highway agencies. A related question which may be equally difficult to answer is to define the level of objectionable stress in a pile. That is, can embedded piles give acceptable service operating at or near their yield strength? Experience in Tennessee and studies in North Dakota seem to indicate that they can.

2. Purpose

If thermal stresses can be accurately predicted and appropriately handled, the elimination of deck joints on as many bridges as possible is desirable. The current length limitation in Iowa for the use of integral abutments in concrete bridges is 265 feet. The first application with steel I-beam bridges in Iowa is currently under construction. These dual Interstate bridges are 263 feet in length.

The purpose of this study is to gain a better understanding of the behavior of integral abutments and to present background information for the Iowa Highway Research Project HR-227, "Piling Stresses in Bridges with Integral Abutments." The objective of the research study is to propose maximum bridge lengths for steel and concrete bridges for which integral abutments can safely be used.

3. Plan of Investigation

A survey questionnaire was prepared in cooperation with the Office of Bridge Design, Highway Division, Iowa Department of Transportation, to obtain information concerning the use and design of integral bridge abutments. Based on a review of the survey, several states were later contacted to gain a better understanding of successful design details and assess the performance of reletively long integral abutment bridges. Summaries of these telephone conversations with bridge engineers in Tennessee, Missouri, North Dakota, Kansas, and California are included in section II-4 of this report.

Most of the states which use integral abutments, as shown in APPENDIX I, have developed specific guidelines concerning allowable bridge lengths, design of the backwall, type of piling, etc. The basis of these guidelines is shown to be primarily empirical.

A brief review of available methods of mathematically representing the pile-soil system is conducted to determine what types of soil information are required. Methods of obtaining the soil data are discussed and limits are presented for use in the analysis.

Previous experimental studies have been conducted by Rowe,[34] Alizadeh and Davison,[1] Paduana and Yee,[36] South Dakota Department of Highways,[19] and North Dakota State University.[17] These projects were reviewed and compared to the possible methods of soil parament representation. Results are presented which may be significant to the current research project.

II. SURVEY OF CURRENT PRACTICE

1. Purpose

Surveys concerning the use of integral abutments have previously been conducted.[19,12] They have indicated that there are marked variations in design limitations and criteria for their use. Many states have not felt comfortable using a system which does not contain some "free space" for temperature variation displacements to occur.

Some of the variations among the states occur because of different temperature range criteria. Also, depending on the extent of de-icing salt use, some states may experience greater problems with bridge deck expansion joint devices than others. Naturally, it is difficult to justify altering existing construction techniques by either beginning the use of integral abutments or using them for much longer bridges, if the possibility of decreased distress and maintenance are not readily apparent.

The current survey was conducted to determine:

1. Various design criteria and limitations being used;
2. Assumptions being made regarding selected design parameters and appropriate level of analysis;
3. Specific construction details being used;
4. Changes in trends since previous surveys were taken; and
5. Long-term performance of bridges with integral abutments.

2. Questionnaire

The questionnaire was sent to the 50 states and Puerto Rico. Since the Direct Construction Office, Region 15, Federal Highway Administration is involved in bridge construction on Federally owned property, a questionnaire was also sent to the design department in Arlington, Virginia. A copy of the questionnaire and responses from each of these agencies are contained in APPENDIX I.

The survey questions were directed at limitations in bridge length, type, and skew. The states were also asked what assumptions were made in determining fixity conditions and loads for design of the piling and superstructure. A detail drawing of the type of integral abutment used in Iowa was included in the questionnaire.

It was hoped that some of the states using integral abutments had performed an analysis regarding anticipated movements and pile stresses. The questions regarding fixity and design loads were included to determine what level of analysis was felt to be appropriate.

Much of the progress in the use of integral abutments has come about by successive extention of limitations based on acceptable performance of prototype installations. In order to learn more from the several states who have pioneered the use of integral abutments, questions were asked regarding costs and performance.

3. Trends in Responses

Of the 52 responses received, 29 indicated that they use integral-type abutments. A few of these, such as New Mexico and Virginia, are just beginning to use them. Their first integral abutment bridge was either recently designed or currently under construction.

Of the 23 who did not use these abutments, there were 4 groups having similar responses.

1. Fourteen states have no plans to consider using this type of abutment.
2. Five states responded that they have not previously considered the possibility of fixing the girder ends to the abutments.

3. Three states have built some integral abutments or semi-integral endwalls, but currently do not use them in new bridge construction.
4. One state indicated that they were presently investigating the possibility of using integral abutments.

The following are some of the reasons given for avoiding the use of integral abutments:

1. The possibility of a gap forming between the backwall and the roadway fill (2 states);
2. Increased substructure loads (1 state);
3. The possible attenuation of a bump at the ends of the bridge (1 state);
4. The lack of a rational method for predicting behavior (1 state);
5. The possible additional stress on approach pavement joints (2 states); and
6. Cracking of the backwall due to superstructure end span rotation and contraction (2 states).

One of the purposes of this study is to present methods of analysis and design details which will reduce the potential ill-effects of these concerns. Many of the states currently using integral abutments have effectively solved most of these problems.

The following is a discussion of the responses received from states using integral abutments keyed to the question numbers of the survey. A summary of the responses is contained in APPENDIX I.

1. Most of the states using integral abutments do so because of cost savings. Typical designs use less piling, have simpler construction details, and eliminate expensive expansion joints.

Some states indicated that their primary concern was to eliminate problems with the expansion joint. A few said that simplicity of construction and lower maintenance costs were their motivation.

2. & 3. TABLE 1 shows bridge length limitations currently being used. In summary, 70 percent or more of those states using integral abutments feel comfortable within the following range of limitations: steel, 200-300 feet; concrete, 300-400 feet; and prestressed concrete, 300-450 feet. There are 3 states using longer limitations for each structure type. They typically have been building integral abutments longer than most states and have had good success with them. The move toward longer bridges is an attempt to achieve the good performance observed on shorter bridges for structures at the maximum practical length limit. This achieves the maximum benefit from what many regard as a very low maintenance, dependable abutment design.

The difference in concrete and steel length limitations reflects the greater propensity of steel to react to temperature changes. Although the coefficients of expansion are nearly equal for both materials, the relatively large mass of most concrete structures makes them less reactive to ambient temperature changes. This is reflected in the American Association of State Highway and Transportation Officials (AASHTO) design temperature variation, which is much lower for concrete.

TABLE 1

Maximum Length	Steel	Number of States Concrete	Prestressed
800		1	1
500		1	2
450		1	3
400	2	3	4
350	1	3	1
300	8	8	8
250	2	1	
200	5	1	2
150	1		
100		1	

INTEGRAL ABUTMENT BRIDGE LENGTH LIMITATIONS (1981)

4. Only a few states responded to the question regarding limitations on piling. Five states use only steel piling with integral abutments. Three others allow concrete and steel but not timber. No length limitations for timber piling were given by states other than Iowa. Timber piling is allowed in Iowa for bridges less than 200 feet in length. If the length is greater than 150 feet, the top of the pile which is embedded in the abutment is wrapped with 1/2 inch to 1 inch thick carpet padding material. This allows some rotation of the abutment, reducing the bending stress on the pile. Only 4 of the 29 agencies indicated that the webs of steel piles were placed perpendicular to the length of the bridge. In subsequent phone calls to a few other states, it was learned that others also follow this practice. At least 1 state began using integral abutments with steel piling placed in the usual orientation (with the pile web along the length of the bridge). This led to distress and cracking at the beam-abutment interface, and the state eventually began to rotate the piles by 90 degrees for greater flexibility. The writer believes that many states accept this as common practice and, therefore, did not mention it specifically.

5. & 6. Twenty-two states indicated that the superstructure was assumed pinned at the abutments. Five assumed partial fixity, and one assumed total fixity. Seventeen responses noted that at the pile top a pinned assumption was made, 4 reported a partial fixity assumption, and 5 states believe the pile top is totally fixed. Six of the states which assume a pinned condition

actually use a detail which is designed to eliminate moment constraint at the joint. In the absence of a detail which allows rotation, the appropriate assumption depends largely on the relative stiffnesses of the pile group and the end span superstructure. For example, if a single row of steel piling with their webs perpendicular to the length of the bridge was used with a very stiff superstructure, the joint would probably behave as if it were pinned in response to dead and live loads and as if it were fixed in response to temperature movements. If the stiffness of the pile group were increased, some degree of partial fixity would result depending on the ratio of stiffnesses.

7. Only a few states consider thermal, shrinkage, and soil pressure forces when calculating pile loads. Several states noted on the questionnaire that only vertical loads are used in design. Of those that do consider pile bending stresses, 8 use thermal forces, 3 use shrinkage forces, and 10 consider soil pressure.
8. Most states indicated that bending stresses in abutment piling were neglected. There were 3 states, however, that assumed a location for a point of zero moment and used combined bending and axial stresses. Also, prebored holes were used by three states to limit bending stresses by reducing the soil pressure.
9. Most states indicated that a free-draining backfill material is used behind the abutment. Some responses, however, indicated that problems were encountered such as undermining associated with granular soils. One state said, "Have recently experienced

problems with non-cohesive material behind this type of abutment. Backfill material should be cohesive and free from cobbles and boulders." Six other states use common roadway fill behind the abutment.

10. All except 4 states rest the approach pavement on the integral abutment. One state indicated that a positive tie connection was used to connect the slab. No comments regarding the practice of resting the slab on a pavement notch were noted. A few states indicated that they have experienced problems when reinforced approach slabs were not used.

11. & 12. All except 3 states reported lower construction and maintenance costs using integral abutments. One said costs were the same and 2 did not respond to the question. The following are some isolated comments that were made about construction and maintenance problems using integral abutments:

a. Longer wingwalls may be necessary with cast-in-place, post-tensioned bridges for backwall containment;

b. The proper compaction of backfill material is critical;

c. Careful consideration of drainage at the end of the bridge is necessary;

d. Wingwall concrete should be placed after stressing of cast-in-place, post-tensioned bridges;

e. The effects of elastic shortening after post-tensioning should be carefully considered, especially on single span bridges;

f. Proper placement of piles is more critical than for conventional abutments;

g. Wingwalls may need to be designed for heavier loads to prevent cracking;

h. Adequate pressure relief joints should be provided in the approach pavement to avoid interference with the functioning of the abutment;

i. Possible negative friction forces on the piles should be accounted for in the design; and

j. Wide bridges on high skews require special consideration including strengthening of diaphragms and wingwall-to-abutment connections.

4. Review of Details and Design of Selected States

Telephone visits were conducted with 5 states to discuss in greater depth the items covered on the questionnaire and to become more familiar with their design rationale for integral abutments. They were Tennessee, Missouri, North Dakota, Kansas, California, and Iowa. Some of the items covered in the visits are discussed below.

a. Tennessee[38]

Tennessee has extensive experience with integral abutment construction and performance. It is estimated that over 300 steel and 700 concrete bridges have been built with integral abutments. Mr. Ed Wasserman, Engineer of Structures, Tennessee Department of Transportation, indicated that the state was very pleased with the performance of these structures and has noted no undue stress on the abutments.

The maximum length limits using integral abutments were arrived at by setting a limit of expansion or contraction of 1 inch. This figure was developed empirically over a period of several years. By using a simplified column analysis with an unsupported length of 10 feet the state calculated the piling stresses to be just slightly over yield when deflected only 1 inch. Tennessee uses the average AASHTO temperature change of 35° F for concrete structures and 60° F for steel. The maximum bridge lengths (2L) for this allowable deflection (Δ) are about 800 feet for steel and 400 feet for concrete.

$$L_{\text{concrete}} = \frac{\Delta}{\alpha_c(\delta T)_c} = \frac{1/12}{(.0000060)(35)} = 396 \text{ feet}$$

$$L_{\text{steel}} = \frac{\Delta}{\alpha_s(\delta T)_s} = \frac{1/12}{(.0000065)(60)} = 214 \text{ Feet.} \quad (3)$$

Where:

α_c = Coefficient of thermal expansion for concrete (AASHTO)

$(\delta T)_c$ = Allowable temperature drop or rise for concrete (AASHTO)

α_s = Coefficient of thermal expansion for steel (AASHTO)

$(\delta T)_s$ = Allowable temperature drop or rise for steel (AASHTO)

Tennessee has not completed any research work to verify the assumptions used to develop design criteria other than observing the good performance of constructed bridges. Abutment details used by Tennessee are very similar to Iowa's. Timber piles are not used.

b. <u>Kansas</u>[39]

Kansas has not participated in formal research activities to formulate design criteria for integral abutments. The length limitations and details used have been developed empirically through many years of experience. The following length limitations have been established: steel, 300 feet; concrete, 350 feet; and prestressed, 300 feet. Mr. Earl Wilkinsen, Bridge Engineer, Kansas State Highway Commission, indicated that a few cast-in-place bridges up to 450 feet long had been built in the past with integral abutments, but this is not the general rule.

Point-bearing steel piles with 9000 psi allowable bearing are used most often. Some concrete filled steel shell piling or prestressed concrete piles are occasionally specified.

c. Missouri[25]

Missouri had planned to instrument the piling of an integral abutment several years ago but was unable to do so because of construction timing. No other investigations of integral abutments have since been planned.

Criteria for use of integral abutments have been developed primarily from following the success of other states, notably Tennessee. The maximum length limit for steel bridges has recently been increased from 300 to 400 feet. Over 100 concrete bridges (mostly prestressed) and over 40 steel bridges have been built with integral abutments over a period of 12-15 years.

d. North Dakota[11]

North Dakota has built over 300 bridges with integral abutments. Most of these have concrete superstructures. They have had good performance except in two areas. First, the superstructure was originally connected to the backwall with dowell bars which were placed with insufficient cover. In some places the concrete over the dowell bars on the inside face of the backwall cracked due to thermal forces caused by contraction of the superstructure. Second, the piles were originally placed with the webs parallel to the long axis of the bridge. Using this orientation caused some distress in the backwall since the piles offered relatively large resistance to lateral bridge movements. The problem was eliminated when the piles were installed with the webs perpendicular to the long axis of the bridge.

North Dakota was an early user of integral abutments. Their design criteria is based mainly on their own experience. No formal analysis methods are employed to calculate stresses in the piles. Steel and concrete bridges are currently limited to 300 feet while prestressed bridges are built up to 450 feet in length.

Last year the state built a 450-foot prestressed concrete box beam bridge on a 0 degree skew near Fargo, North Dakota. The piling in the integral abutments were instrumented with strain gauges and had inclinometer tubes attached. Dr. Jim Jorganson, Civil Engineering Department, North Dakota State University, was commissioned to monitor the movements and strains in the bridge for one year. He will have a preliminary report prepared late this summer. It appears that the maximum total movement at each end is about 2 inches.[17] This is equivalent to a temperature variation of about 117° F.

The installation contains a unique feature which was designed by Moore Engineering, West Fargo, North Dakota. A special expansion joint material several inches thick is placed behind the abutment backwall. Behind it is a sheet of corrugated metal. The mechanism is designed to reduce passive earth pressures on the abutment and to help reduce the formation of a void space upon contraction of the superstructure. The system is shown in FIGURE 18 and discussed further in Section III-6.

e. California[7]

California has engaged in several projects investigating the performance of laterally loaded piles in bridge embankments. This work has been done at California State University at Sacramento, and by the California Department of Transportation, Bridge Department, and will be described more fully in the literature review. The research was able to suggest a correlation between the coefficient of subgrade reaction used in an elastic design method to the standard penetration blowcount. Maximum bending moments in steel H-piles were predicted within 15 percent of measured values.

California does not analyze pile stresses due to bending at each bridge site. Guidelines have been developed to aid designers in determining the type of abutment to use. They are currently using integral abutments with concrete bridges up to 320 feet long. Because of the effects of elastic shortening on application of post-tensioning forces, the length limitation for prestressed bridges is about 100 feet less. Design of the endwall is based on specified horizontal loads depending on the type of piling used (see APPENDIX II).

f. Iowa[14]

Iowa began building integral abutments on concrete bridges in 1965. One of the first was on Stange Road over Squaw Creek in Ames. This prestressed beam bridge is about 230 feet long with no skew. The writer visited this bridge in August 1981 to determine if any apparent distress was evident. Both approaches were generally in good shape with no major cracking noted. The abutment walls, wingwalls, and beams showed no thermal movement related cracking or distress.

Mr. Henry Gee, Structural Engineer, Office of Bridge, Iowa Department of Transportation, inspected at least 20 integral abutment bridges yearly for about 5 years after construction. They varied in length from 138 to 245 feet with skews from 0 to 23 degrees. The inspections were terminated since no distress or problems were found which related to the lack of expansion joints in the superstructure.

Iowa's length limitation for integral abutments in concrete bridges is 265 feet. This is based on an allowable bending stress of 55 percent of yield plus a 30 percent overstress since the loading is due to temperature affects. The moment in the pile was found by a rigid frame analysis which considered the relative stiffness of the superstructure and the piling. The piles were assumed to have an effective length of 10.5 feet, and the soil resistance was not considered. The analysis showed that the allowable pile deflection was about 3/8 inch.

5. <u>Summary</u>

There is wide variation in design assumptions and limitations among the various states in their approach to the use of integral abutments. This is largely due to the empirical basis for development of current design criteria. Some states, such as Tennessee and Iowa, have used traditional statics analysis methods for a beam or beam-column to estimate piling stresses. It is recognized, however, that assumptions concerning end fixity and soil reaction may substantially affect the results. A simple rational method of accurately predicting pile stresses would be a valuable addition to the current state-of-the-art in integral abutment design.

Those who use integral abutments are generally satisfied with performance and believe they are economical. Some problems have been reported, however, concerning secondary effects of inevitable lateral displacements at the abutment. These include abutment, wingwall, pavement, distress, and backfill erosion. Only a few states noted that any difficulty had been encountered (see "Comments" section in APPENDIX I). Other states reported that solutions have been developed for most of the ill-effects of abutment movements. They include: (1) additional reinforcing and concrete cover in the abutment, (2) more effective pavement joints which allow thermal movements to occur, and (3) positive control of bridge deck and roadway drainage. From the comments of most states, the writer infers that the benefits from using integral abutments are sufficient to justify the additional care in detailing to make them function properly.

Very little work has been done to monitor the actual behavior of integral abutments except in checking for obvious signs of distress in visible elements of the bridge. The research work being done in North Dakota to monitor actual strains and pile displacements in an actual integral abutment installation is one of very few full-scale projects. It is reported on more fully in section III-6 of this report.

Several states have been progressively increasing length limitations for the use of integral abutments over the last 30 years. Improvements in details have also taken place which generally can eliminate the possibility of serious distress occurring with abutment movements of up to 1 inch. These progressive steps in state-of-the-art bridge engineering have occurred over the past thirty years and are primarily the result of the observance of satisfactory performance in actual installations.

III. LITERATURE REVIEW

1. Analytical Approaches

Several analytical studies[2,9,21,22,29,31] have been made of the laterally loaded pile problem. They are primarily based on Hetenyi's formulation for beams on elastic foundation.[16] Most of the formulations assume an elastic soil response, although some have included inelastic soil behavior by using an iterative or step-wise solution.

The two most promising solutions are the finite difference method and the finite element method. They are step-wise formulations which can consider two-dimensional soil reaction variations. Both methods require a computer for solution.

The finite difference method involves the solution of the basic differential equation of the laterally loaded pile at preselected node points along the pile length.

$$EI \frac{\partial^4 y}{\partial x^4} + Q \frac{\partial^2 y}{\partial x^2} + p = 0 \quad \text{. . . . .(4)}$$

Where:

x = Depth from the top of the pile

E = Modulus of elasticity of the pile

I = Pile moment of inertia

Q = Axial load on the pile

Lateral variations in the soil resistance (p) are handled by assuming a value, solving for the deflection (y), and then iterating until a preselected p-y curve (see FIGURE 7) for the node is satisfied.

The finite element solution generally uses beam-type elements with

RESISTANCE-DISPLACEMENT (p-y) CURVES

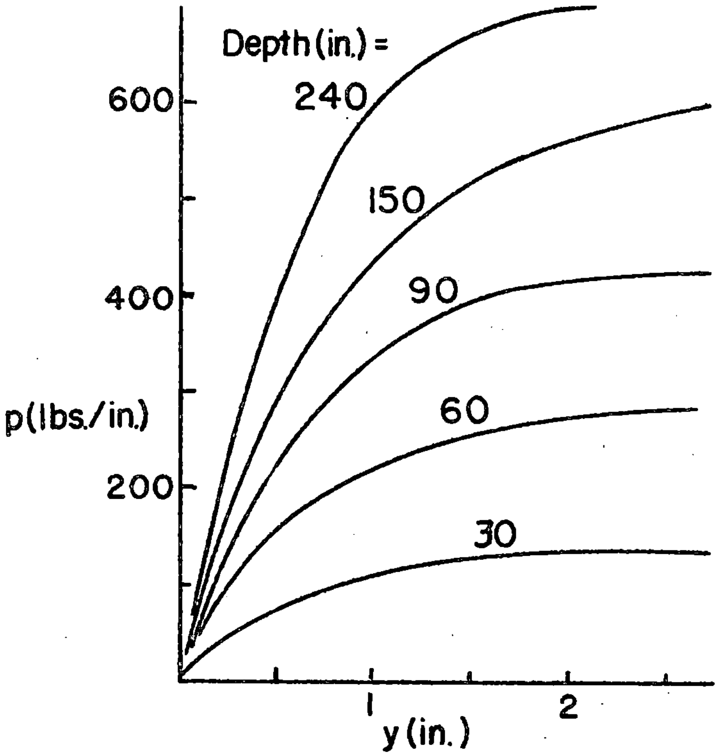


FIGURE 7[21]

three degrees of freedom (x and y translation and in-plane rotation). Lateral soil springs are used to model the soil structure interaction characteristics. The spring values are adjusted after iterative solutions for pile deflections are compared with given p-y curves. After the soil resistance values are determined to the desired precision, the final structural stiffness matrix is formed, displacements are calculated, and element forces and stresses can then be evaluated.

The finite element solution has the ability to consider variable shear transfer to the soil by each element along the pile length. A typical curve showing the load transfer to soil versus axial displacement for various depths is shown in FIGURE 8.[2]

After each iterative displacement calculation, the vertical movement due to axial strain in the pile is subtracted from the total deflection to find the pile element slip. The load matrix is revised with the new element friction load obtained by entering the load-slip diagram for the appropriate depth.[2] The cycling continues until the current and preceding slip values agree to a specified precision.

2. <u>Development of Load-Displacement (p-y) Curves</u>

Probably the most accurate method of developing p-y curves is to use sensitive instruments to measure pile deflection and earth pressure directly in a full-scale lateral load test. Although the necessary equipment could probably be obtained given the level of current technology, the method would be expensive and time consuming.

Another potentially accurate method is to place electric strain gauges along the length of the pile. After calculating pile stresses and bending

LOAD - SLIP CURVES

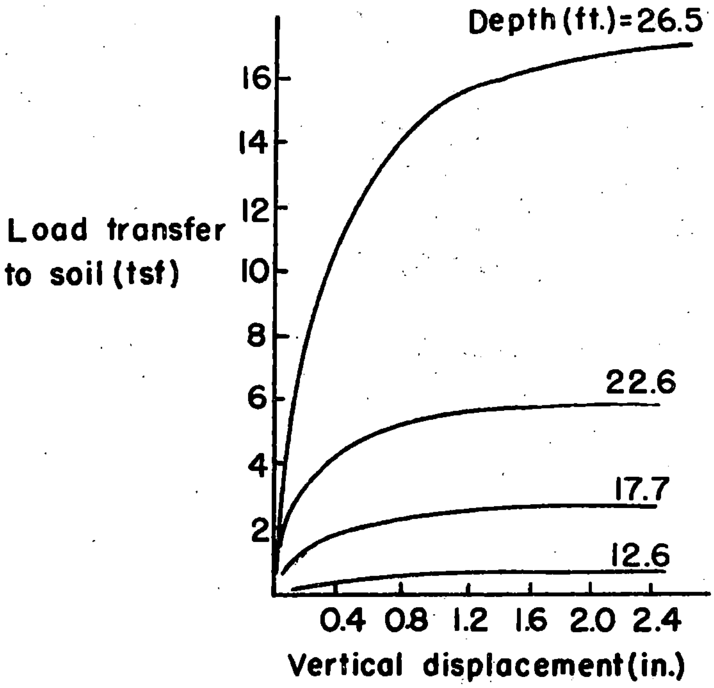


FIGURE 8[2]

moments from the strain readings, the soil pressure (p) and lateral displacement (y) can be found from EQUATIONS 5 and 6.

$$y = \iint M/EI \, dx \quad \ldots \ldots \ldots \ldots \ldots \quad (5)$$

$$p = d^2M/dx^2 \quad \ldots \ldots \ldots \ldots \ldots \quad (6)$$

Where:

M = Applied moment in the pile

This method is also quite expensive and requires extreme care in taking measurements since the deflection is extremely sensitive to variations in the bending moment.[33]

It is possible to obtain approximate values for p-y variations along the pile by knowing the load, moment, deflection, and rotation at the top of a test pile. This simple test requires only that a pile be driven beyond the point below which the soil has no appreciable affect on pile-top deflections and a lateral load be applied while measurements are periodically recorded. The method is based on Reese and Matlock's non-dimensional solutions[31] which assume a linear variation of soil modulus with depth. Relatively accurate information can be obtained, but the method[30] does require actual field measurements to be taken.

Several investigators[20,32,33] have attempted to correlate a lateral load-deflection response with laboratory soil tests. The form of the equation normally used is shown in EQUATION 7.

$$p = Cy^{1/n} \quad \ldots \ldots \ldots \ldots \ldots \ldots \quad (7)$$

Where:

C = A constant which varies on soil properties

n = A constant which varies with the type of soil

Possible functional relations and values for C and n are shown in TABLE 2. The following specific values for a soft clay have been suggested by Matlock[20]:

$$C = P_u/2(y_{50})^{1/3} \quad \text{. . . . . (8)}$$

Where:

$$P_u = \begin{cases} 3cb+\gamma bx+cx/2 \\ 9cb \end{cases} \quad \text{(use smaller value)} \quad \begin{matrix} \text{. . . (8a)} \\ \text{. . . (8b)} \end{matrix}$$

$$y_{50} = 2.5b\epsilon_{50} \quad \text{. . . . . (8c)}$$

y_{50} = Displacement at 50 percent of the maximum deviator stress

ϵ_{50} = Strain at 50 percent of the maximum deviator stress

The Iowa Department of Transportation's current soil investigation procedure at bridge sites includes taking a split tube sample if compressible layers are found in the area of the approach fill. Soil strength, unit weight, and compressibility data are routinely obtained on these samples by performing triaxial, density, and consolidation tests. If three split tube samples were taken, sufficient information would be available to predict the soil response with reasonable accuracy to a depth of about 15 feet. Since soil conditions below about 15 feet have little affect on bending stresses in laterally loaded piles,[28,1] sample depths of 3, 7, and 12 feet would seem to be convenient choices.

If stiff clay is encountered, the equations are modified slightly. Generally, ϵ_{50} will be somewhat lower and the exponent is changed from 1/3 to 1/4.

TABLE 2

	Soft Clay	Firm Clay	Sand
n	3	4	f(ϕ,K_o,x,b,γ)
C	f(c,x,b,γ)	f(c,x,b,γ)	f(ϕ,K_o,x,b,γ)

Where:

c = Shear strength at depth x

x = Depth from the ground surface to the p-y curve

b = Width of the pile

γ = Average effective unit weight from the surface to x

K_o = 0.4

ϕ = Soil friction angle

CONSTANTS USED IN p-y RELATIONSHIPS

Above a certain depth (H) the ultimate lateral soil resistance (P_u) is given by:

$$P_u = A\gamma x\left[\frac{K_o x\tan\phi\sin\beta}{\tan(\beta-\phi)\cos\alpha} + \frac{\tan\beta}{\tan(\beta-\phi)}(b + x\tan\beta\tan\alpha) + K_o x\tan\beta(\tan\phi\sin\beta-\tan\alpha) - K_a b\right] \quad (9)$$

Where:

γ = Average effective unit weight from the surface to x

ϕ = Friction angle of the soil

x = Depth from surface to point where p-y curve is desired

α = $\phi/2$

β = $45 + \phi/2$

K_o = 0.4

K_a = $\tan^2(45-\phi/2)$

b = Pile width

A,B = Empirical coefficients varying with the depth to width ratio as shown in FIGURES 9 and 10, respectively

$$H = \frac{b\cos\alpha\left[K_a\tan^7\beta\tan(\beta-\phi) + K_o\tan\phi\tan^3\beta\tan(\beta-\phi)-1\right]}{K_o\tan\phi\cos\beta+\tan\beta\tan\alpha\cos\alpha+K_o\tan(\beta-\phi)(\tan\phi\sin\beta-\tan\alpha)\cos\alpha} \quad (9a)$$

H = 11.4 for ϕ = 30 and 7.77 for ϕ = 20

This formulation is based on a passive wedge-type failure assumed to occur near the ground surface. The resulting static equilibrium equation for the lateral force against the wedge is differentiated with respect to the depth to obtain the expression for soil resistance per unit length of the pile.

For depths well below the ground surface the soil is assumed to fail

"A" COEFFICIENT CHART

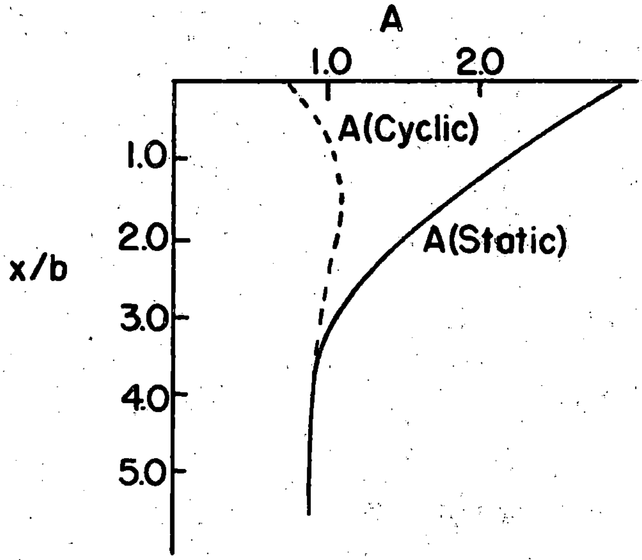


FIGURE 9[33]

"B" COEFFICIENT CHART

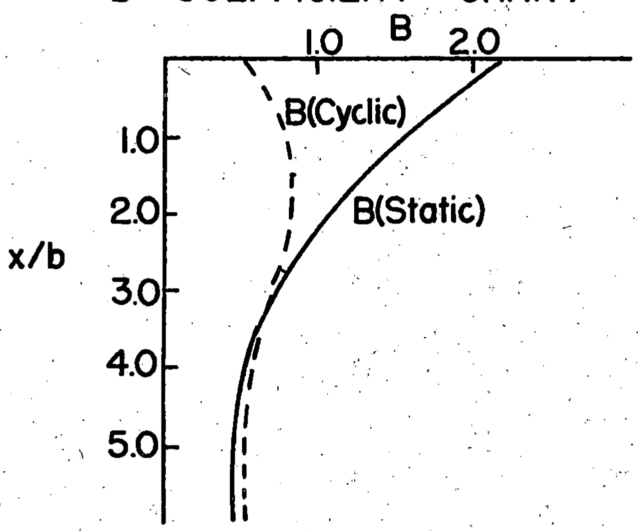


FIGURE 10[33]

by flowing horizontally in a rectangular section around the pile. Active earth pressure is assumed to be the minimum pressure adjacent to the pile. The total soil resistance at depths greater than H is calculated using Mohr-Coulomb theory and is given by

$$P_u = AK_a b\gamma x(\tan^8\beta - 1) + AK_o b\gamma x\tan\phi\tan^4\beta. \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (9b)$$

An intermediate value (P_m) on the p-y curve can also be calculated using either EQUATION 9 or 9b if the coefficient B (see FIGURE 10) is used in place of A. The value of P_m is located on the curve (see FIGURE 11) where y=b/60.

3. <u>Example p-y Curve</u>

To illustrate the procedure further, a set of p-y curves will be developed for a fine sand. For use in this example, the sand will be taken to have a standard penetration blowcount (N) of 15. Based on the given N value, the sand will be assumed to have medium relative density and moderate strength. In this case, values of 105 pounds per cubic foot and 30 degrees will be used for effective unit weight and friction angle, respectively. Using EQUATION 9a, the H value is 11.4. Selecting x equal to 3 feet, EQUATION 9 yields P_u equal to 184 pounds per inch and P_m equal to 104 pounds per inch.

The initial straight portion of the p-y curve is defined by the modulus of subgrade reaction (k), where $k = n_h x$, and n_h is the constant of horizontal subgrade reaction.[36] An appropriate value for n_h is selected from TABLE 3. Since the results are reletively sensitive to the value selected, correlation with field tests is desireable.

APPROXIMATE p-y CURVE FOR A FINE SAND

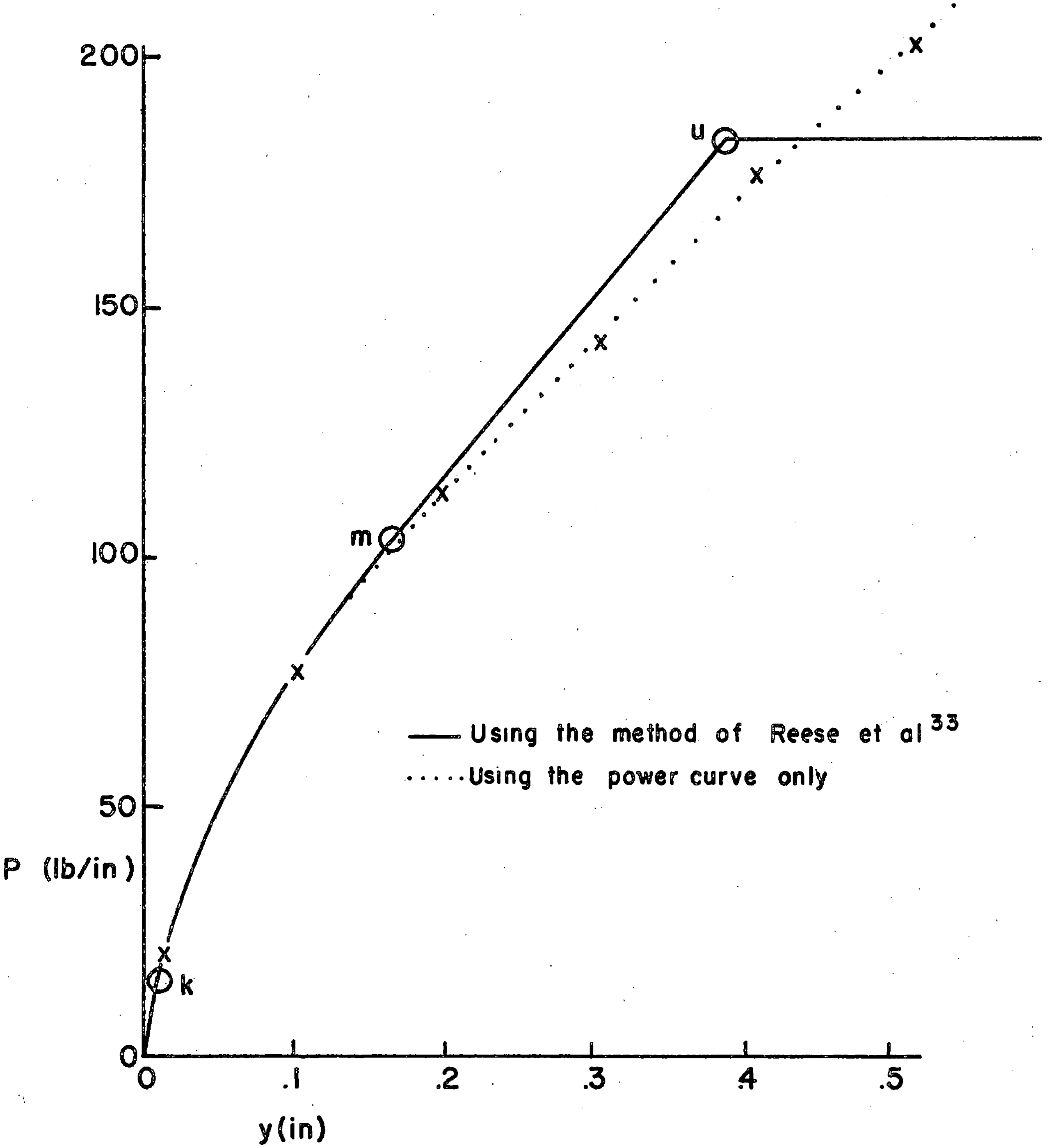


FIGURE 11

TABLE 3

Reletive Density	Loose	Medium	Dense
Recommended n_h lb/in^3	20	60	125

RECOMMENDED n_h VALUES[33]

The general shape of the curve is shown in FIGURE 11. Points m and u are established at:

$$y_m = b/60 \text{ and } y_u = 3b/80 \text{ . . . .(10)}$$

Point k is located at:

$$y_k = (C/n_h x)^{n/n-1} \text{ . . . .(11)}$$

Where:

$$C = P_m/y_m{}^{1/n}$$

$$n = P_m/my_m$$

$$m = (P_u - P_m)/y_u - y_m).$$

In this example the following values are obtained using the above equations and the assumed values of effective unit weight and friction angle for a fine sand:

y_u = .375 inches

y_m = .167 inches

m = 385.0 pounds per square inch

n = 1.62

C = 314.0 pounds per inch

y_k = .0065 inches

The portion of the curve between k and m is defined by $p = 314y^{1/1.64}$.

With these values and the selection of n_h as 60 pounds per square inch, the p-y curve shown in FIGURE 11 is completely defined. If y_k is less than 10 percent of y_m, a reasonably accurate curve may be obtained by using only the power curve ($p = Cy^{1/n}$). The dotted line in FIGURE 11 shows that this simplification yields nearly the same curve except at higher values of y, where it is conservative. The effects of this approach would have to be investigated over the range of values of interest before implementing it fully.

This example development of a p-y curve is based on average characteristics of fine sand as shown on the Iowa Department of Transportation Foundation Soils Information Chart (see APPENDIX III). Similar analyses could be performed for the other soils shown on the chart using assumed average values of unit weight and strength from blowcount correlations in the literature.[40] If more accurate curves are desired for specific field locations, soil samples should be obtained and tested.

This method is based on field tests in submerged granular soils. Its use for soils above the water table may require the selection of higher values of n_h.

Some simplifying techniques can be used to ease the development of p-y curves in clays. Rewriting EQUATION 8a (found on page 34) in its more familiar form yields:

$$P_u = (3 + \gamma x/c + .5x/b)cb \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (12)$$

Assuming a conservative value for γ/c of 0.2 and selecting b equal to 0.833 (for an HP 10 x 42 pile), the equation becomes:

$$P_u = (2.5 + 0.86x)c \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (13)$$

EQUATION 8b can be written for an HP 10 x 42 pile as:

$$P_u = 7.5c \quad \ldots \quad (14)$$

Therefore, EQUATION 13 controls to a depth of 5.8 feet. Thereafter, equation 14 begins yielding a lower value of P_u.

If ϵ_{50} in EQUATION 8c is taken as 0.02 for soft clays,[10] the constant C can be written as:

$$C = \begin{cases} (4.6 + 1.6x)c & x \leq 5.8 \text{ feet} \\ 13.8c & x > 5.8 \text{ feet} \end{cases} \quad \ldots \quad (15)$$

and therefore:

$$p = \begin{cases} (4.6 + 1.6x)cy^{1/3} & x \leq 5.8 \text{ feet} \\ 13.8cy^{1/3} & x > 5.8 \text{ feet} \end{cases} \quad \ldots \quad (16)$$

A similar development can be done for stiff clay taking ϵ_{50} as 0.005 so that:

$$p = \begin{cases} (4.7 + 1.6x)cy^{1/4} & x \leq 5.7 \text{ feet} \\ 13.8cy^{1/4} & x > 5.7 \text{ feet} \end{cases} \quad \ldots \quad (17)$$

This approximate formulation is good for 10 inch piles only. It is useful, however, since only a shear strength value is needed to develop p-y curves for various depths. In an effort to develop a method of predicting average shear strength values for common surface soils in Iowa, historical soil test records from the Iowa Department of Transportation were studied by the writer. Soil test data from split tube samples were available from locations throughout the state. However, the writer selected data from 19 sites in 4 Iowa counties (Blackhawk, Benton, Buchanan, and Linn) for further study. Values of standard penetration blowcount (N) and shear strength (c) were fit to a simple linear prediction model. The following best fit equation had a correlation of 0.82 with the actual data:

$$c = 97.0\ N+114.0 \quad \text{pounds per square foot} \quad . . . . (18)$$

Where:

c = Shear strength

N = Standard penetration blowcount

This simplified procedure should allow quick calculation of approximate p-y curves based only on readily available N values. If this method were to be routinely used, further study should be done to verify and improve the shear strength prediction model and to further limit the γ/c ratio for soft and firm clays.

4. Development of Load-Slip Curves

The vertical load on a pile can be carried by shear transfer to the adjacent soil and by bearing at the end point. Numerous methods have been proposed for estimating the ultimate end-bearing resistance of an embedded pile.[40] There are large variations in the results from these methods in part since they are based on different failure modes. The skin resistance can be estimated by methods proposed by Meyerhof,[24] Tomlinson,[37] and Seed and Reese.[40] Their procedures involve empirical relationships derived from pile load tests.

The basic expression for the ultimate soil resistance for point-bearing of a pile in clay is:

$$q_f = cN_c + q_oN_q \quad . . . . (19)$$

Where:

q_f = Ultimate soil resistance

c = Shear strength

q_o = Effective vertical stress at the pile tip

N_c, N_q = Dimensionless bearing capacity factors

$N_c = (N_q - 1)\cot \phi$

The strength parameters for a typical glacial clay[15] in Iowa may be represented by c = 1400 pounds per square foot and $\phi = 9^\circ$. Using an N_q of 3 as recommended by Meyerhof[40] and an assumed average buoyant unit weight for the overburden of 65 pounds per square foot, a 40-foot pile has an ultimate end-bearing of 25 kips per square foot. Using an HP 10 x 42 pile as is common in Iowa, the ultimate point load is about 2.2 kips. Iowa glacial clay deposits can yield much higher bearing values than this, but on the average the point resistance can be neglected for the purpose of this study. Certainly if the pile is founded in alluvial silts or soft clay soils, the end-bearing is also negligible.

The point load in sandy soils can be estimated using the traditional bearing capacity formula with appropriate estimates of the shear strength and density. Meyerhof[40] has also proposed an empirical method for use in granular soils.

$$q_f = 8N \quad \text{. . . . .} (20)$$

Where:

q_f = Ultimate soil resistance

N = Standard penetration blowcount

For example, gravelly sand as shown in the Iowa Department of Transportation Foundation Soils Information Chart has an average N value of 21. The point load using a HP 10 x 42 pile is 14.5 kips. Alternately, under the same assumptions used in the glacial clay example and assuming a friction angle of 35 degrees ($N_q = 49$)[3] for the gravelly sand, the bearing capacity formula yields an ultimate point load of 11 kips. Unless the friction angle and soil density are known at a specific site, EQUATION 20 can be used as a satisfactory approximation.

Point-bearing piles which are properly seated in bedrock can normally be assumed capable of carrying allowable pile loads with little or no displacement. That is, they behave like elastic columns.[40] This limits the amount of skin resistance that can develop. Some shear load transfer will occur, however, due to elastic shortening of the pile.

The following is a typical note included on bridge foundation plans by the Iowa Department of Transportation to assure proper seating of point-bearing piles:

> "Steel HP 10 x 42 point-bearing piling shall be driven to practical refusal and seated in sound rock. Seating shall be done with a diesel hammer with a ram weight of at least 2,700 pounds delivering at least 19,000-foot pounds of energy or a gravity hammer having an effective weight of at least 4,500 pounds and driving energy of not less than 36,000-foot pounds nor more than 40,000-foot pounds."[26]

The design bearing value is also normally specified. In Iowa it is limited to a load causing an axial stress of 6,000 pounds per square inch when used in an integral abutment. Under these conditions it may be assumed that virtually no settlement of the pile tip occurs.

Some investigators[40] have assumed that the distribution of skin friction along the length of the pile is parabolic for a floating point pile (see FIGURE 12). This is intuitively reasonable if the shear transfer is considered to be a function of the pile displacement and available shear resistance, which vary inversely along the length of the pile to some point where the resistance may reach a maximum. For practical problems the distribution can be assumed to be linear to a depth of about 15 pile diameters where a maximum value of shear resistance can be taken.[40] This is shown in FIGURE 12 as a dashed line. Meyerhof[24] has related this maximum value to the standard penetration blowcount (N).

$$\tau_{max} = .02\ N \text{ kips per square foot} \quad . . . . . (21)$$

Where:

τ_{max} = Ultimate soil shear resistance

Tomlinson[37] has presented a method to estimate the maximum value using the soil shear strength (c).

$$\tau_{max} = acp_p \quad . . . . (22)$$

Where:

p_p = Pile perimeter

a = 0.7 for most applications in soft clay. (Other suggested values are contained in the literature.)

These two methods[24,37] were compared by the writer to empirical data developed by the Iowa Department of Transportation[35] for routine pile length design as shown in TABLE 4. The previously described blowcount-shear

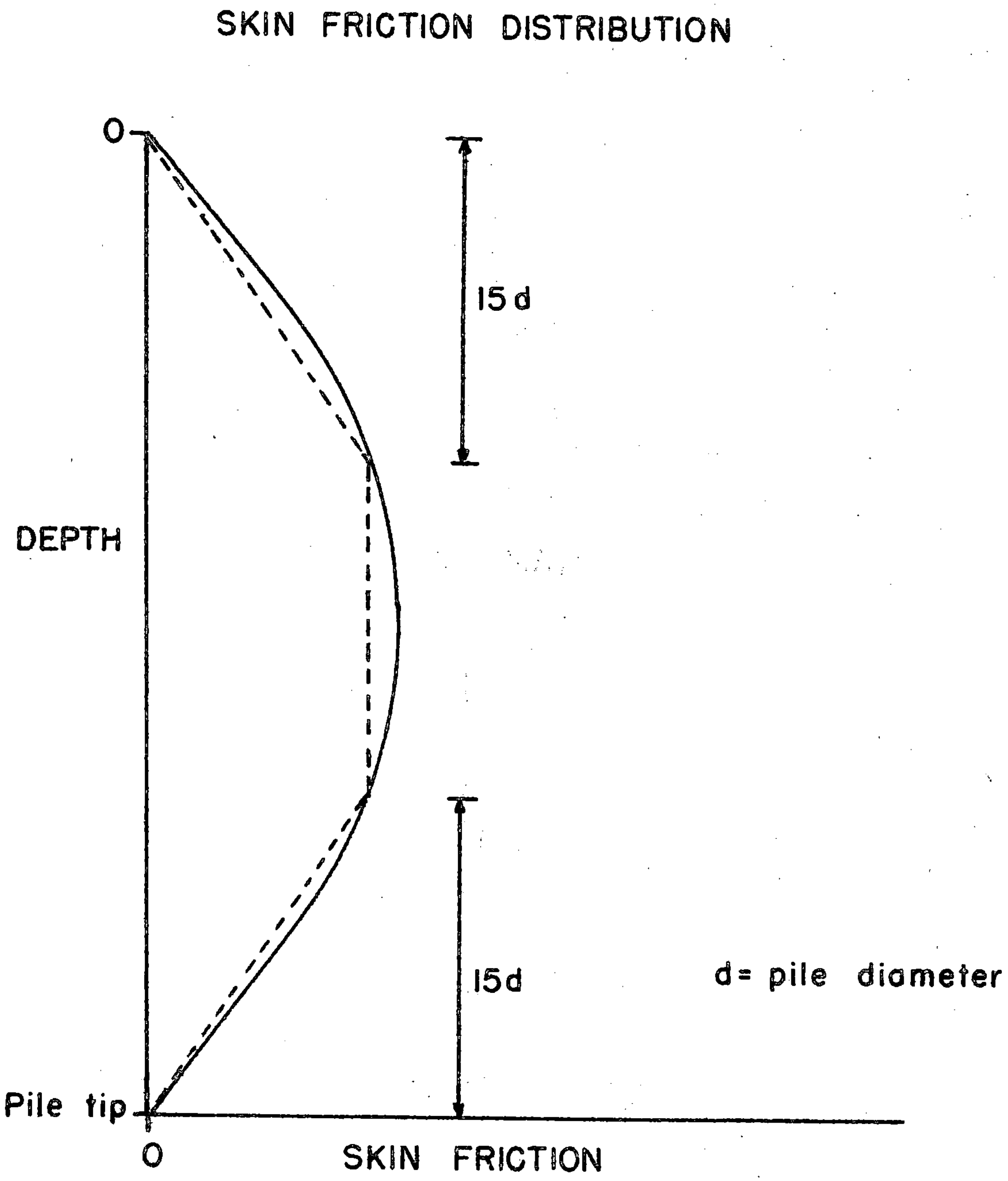

SKIN FRICTION DISTRIBUTION
0
DEPTH
15 d
15d
d= pile diameter
Pile tip
0
SKIN FRICTION

FIGURE 12

TABLE 4

	Ave N-value	Steel H-Pile			16" Concrete Pile		
		Iowa DOT	Meyerhof	Tomlinson	Iowa DOT	Meyerhof	Tomlinson
Very soft silty clay	1	.8	.27	.49	2.0	.44	1.1
Soft silty clay	3	.8	.82	.94	2.0	1.3	2.2
Stiff silty clay	6	1.6	1.6	1.6	3.2	2.6	3.7
Stiff silt	5	1.6	1.4	1.4	3.2	2.2	3.2
Stiff sandy silt	5	1.6	1.4	1.4	3.6	2.2	3.2
Stiff sandy clay	6	2.4	1.6	1.6	3.6	2.6	3.7
Silty sand	8	2.8	2.2		4.0	3.5	
Clayey sand	13	2.4	3.6		4.0	5.7	
Fine sand	15	2.4	4.1		4.4	6.6	
Course sand	20	3.6	5.5		4.8	8.7	
Gravelly sand	21	3.6	5.7		6.4	9.2	

Ultimate Soil Shear Resistance (kips/linear foot of pile) strength correlation was used to establish c for use in Tomlinson's[37] formula. The values used by Iowa are based on numerous pile load tests, many of which were taken to yield.[14] The shear resistance was assumed to be equal at all depths within a given soil type layer. Values were first developed from tests in predominately one soil type. Once some of the values were established, others could be obtained from tests in multi-layered soils. For the purposes of this study, it is recommended that the Iowa Department of Transportation values be used.

In many of the pile load tests conducted by the Iowa Department of Transportation the yield point was taken at a vertical settlement of 0.2 inches. Notable exceptions to this were long piles driven through a thick layer of soft soil which had high yield displacements (up to 1.5 inches) and point bearing piles which had very low yield displacements (as low as 0.04 inches). For pile load testing currently conducted by Iowa, the yield point is defined as the point where settlement is no longer proportional to the load and shows a marked deviation from normal. The Department of Transportation soil engineering staff believe that testing under this criteria tends to support the 0.2 inch yield point for most Iowa soils.[14]

This value represents the gross displacement at the top of the pile. However, it can be used to estimate the point where the maximum load transfer to the soil occurs if elastic shortening of the pile is accounted for by using an arbitrary reduction of 0.05 inch. This is the elastic shortening of an HP 10 x 42 steel pile loaded at half the normal allowable load (37 tons) in Iowa at a point halfway down a 40-foot embedment length.

5. Example Load-Slip Curves

Based on the foregoing empirical data, the load-slip relationships shown in FIGURE 13 are believed to represent upper and lower bounds that can be used in a mathematical analysis of soil-pile interaction. These bounds represent conditions that may likely be found near the ground surface in Iowa.

The only points identified precisely are the points of maximum load transfer. The shape of the curve is assumed. The exact shape could be obtained by conducting load tests on instrumented piles. This was done by Coyle and Reese[8] who developed the curves in FIGURE 14 based on the analysis of pile responses over a wide geographic area.

Kezdi[18] used a semi-empirical law to describe the load-slip behavior of piles in granular soils. He used information from a measured shear transfer versus slip curve to predict a pile load-settlement curve. If the load-settlement curve was available, the method could be used in reverse to estimate the slope in the initial portion of the load-slip curves shown in FIGURE 13. The following equation was used by Kezdi[18] to describe the response of a pile during a load test:

$$Q = Q_o(1-\exp(-k\rho/\rho_o-\rho)) \qquad (23)$$

Where:

Q = Load on the pile

Q_o = Ultimate pile load

ρ = Settlement

ρ_o = Settlement corresponding to P_o

k = $\rho_o \tan a_o$

a_o = Horizontal angle of the initial slope of the load-slip curve (see FIGURE 15)

APPROXIMATE LOAD-SLIP CURVES

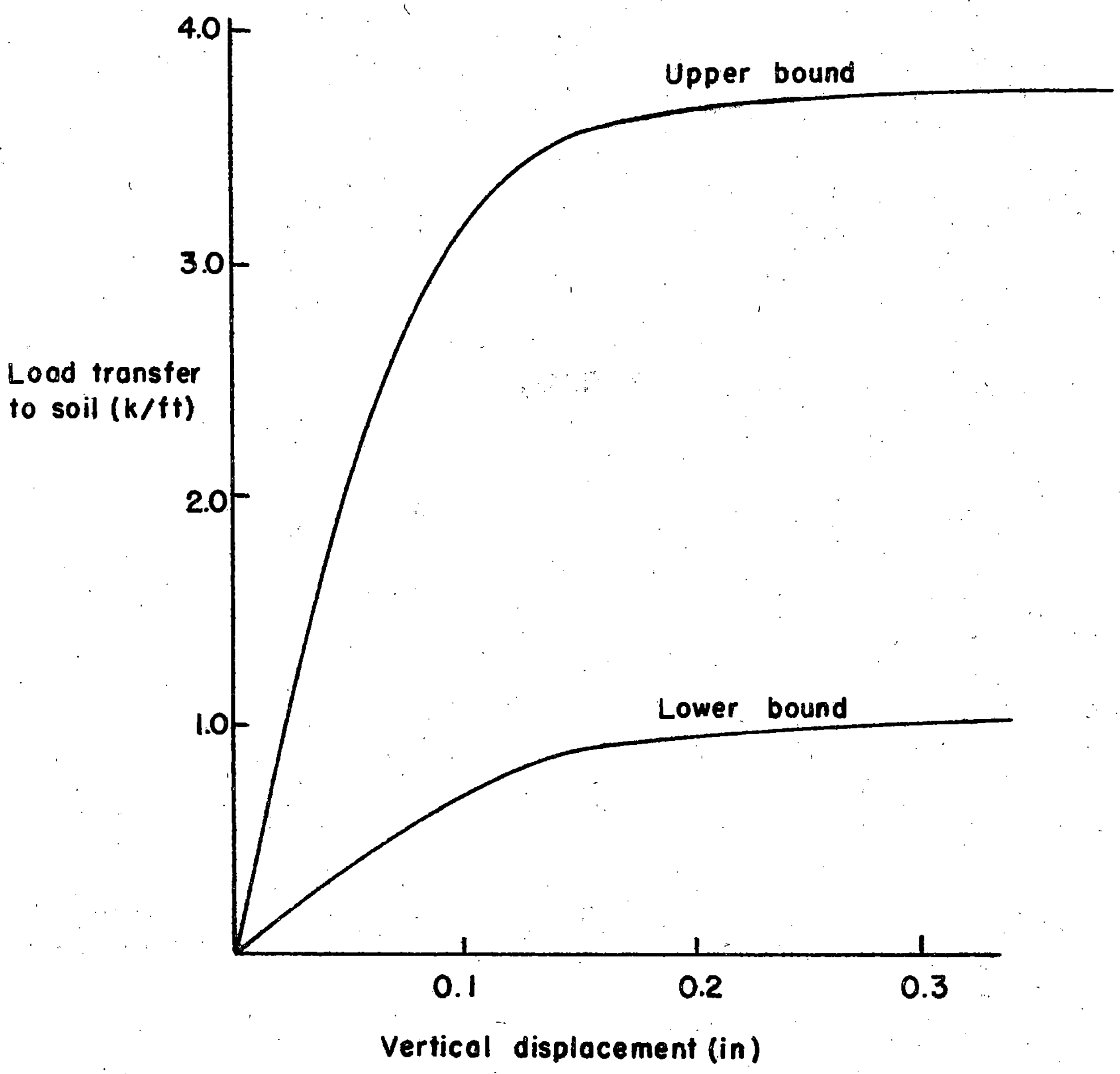


FIGURE 13

EMPIRICAL LOAD-SLIP CURVES

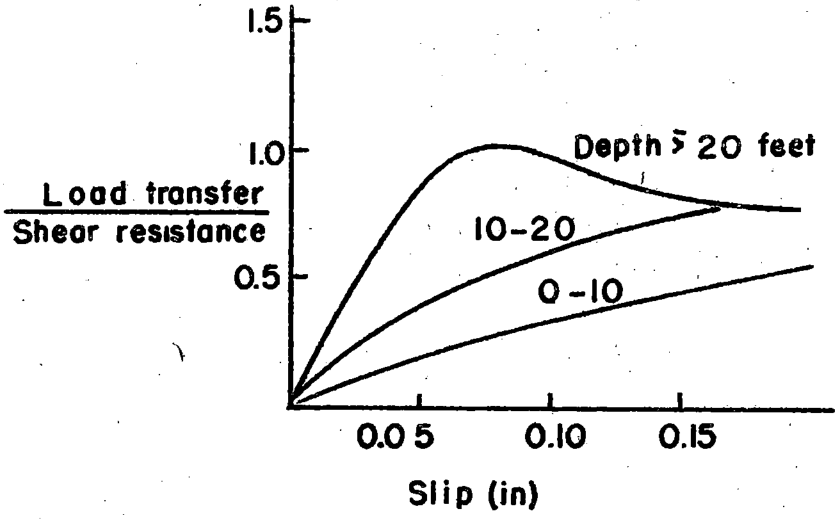


FIGURE 14[2]

INITIAL SLOPE ESTIMATION FOR
LOAD-SLIP CURVES

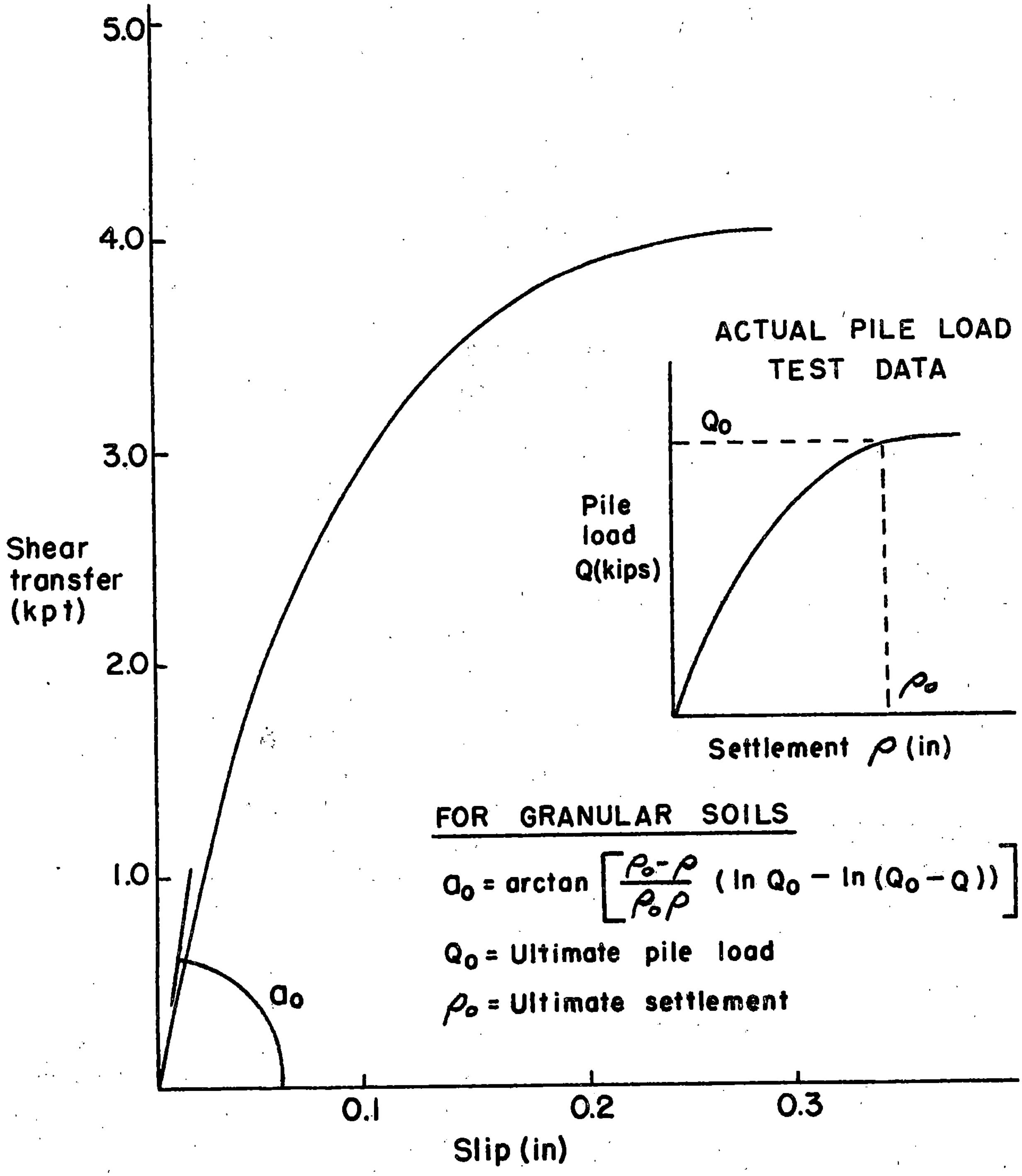


FIGURE 15

This equation can be solved for a_o as shown in FIGURE 15. By knowing the yield point and any other point on the pile load test curve, an estimate of the initial slope of the load-slip curve can be obtained. In an effort to estimate this angle for the soils described on the Iowa Department of Transportation soils chart, actual pile load test records were reviewed by the writer. Several tests were selected where the pile was embedded in predominately one soil type. Values of the angle a_o were calculated using the load and settlement values at yield and at a point about one half of yield. The results are shown in TABLE 5. Note that the writer has extended the use of the method to apply it to predominately cohesive soils. This was done only for academic interest since Kezdi's[18] original work included correlations with granular soils only.

6. Previous Research

a. California[27]

California began informal studies of some of their long structures without expansion joints about 15 years ago. Their efforts consisted of identifying appropriate structures and conducting periodic inspections to monitor performance. Twenty-seven bridges were studied. They varied in length from 269 feet to 566 feet. About 18 of the bridges had integral abutments while the others had semi-integral. An example of a typical inspection record[4] is shown in FIGURE 16.

Although a final report on this study will not be available until 1982, the Structures Office, California Department of Transportation, has reported the following interim findings:[7]

1. There is no apparent distress at end bent columns;

TABLE 5

Soil Description	a_o (degrees)
Very soft silty clay	*70
Soft silty clay	*72
Stiff silty clay	*74
Stiff silt	*74
Stiff sandy silt	*74
Stiff sandy clay	*74
Silty sand	75
Clayey sand	76
Fine sand	78
Coarse sand	80
Gravelly sand	82

* Kezdi's[18] semi-empirical law was correlated to load tests in granular soils only (see text).

LOAD-SLIP CURVE INITIAL SLOPE RECOMMENDATIONS

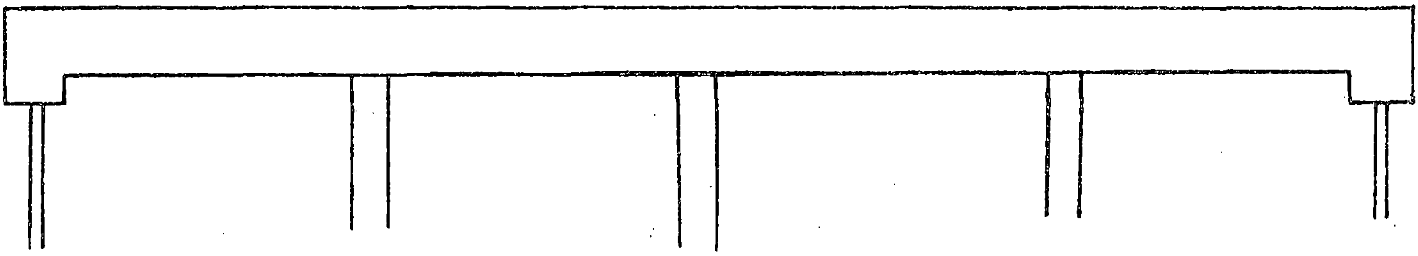
SAMPLE

INSPECTION RECORD
OF
STRUCTURES WITHOUT EXPANSION JOINTS

Date 5-1-67

Br 53-1671 Name Fairfax On Ramp Co-Rte LA-10

Type RCB Length 352' Skew Var. Year Built 1964



ELEVATION

APPROACH PAVEMENT

Type: AC

Condition: The Westerly approach appears to have been patched twice, it is now in good condition. Easterly approach has settled slightly, it has never been patched. A 1/16" wide transverse crack has occured in the Easterly approach about 8' from the abutment for most of the width. The crack has been filled with latex.

STRUCTURAL DEFECTS

Space between structure and PCC curb: 1/2" Westerly, 3/8" Easterly.

Deck surface has a few transverse cracks over the bents, otherwise crack free.

No cracks found in soffit, webs, abutment walls, or columns.

There is a 1/2" crack between fill and backwall of Westerly abutment.

COMMENTS

Traffic volume appears to be light to moderate.

FIGURE 16[4]

2. There is no cracking on girder soffits related to the lack of deck joints;
3. No structural distress is apparent at the abutments;
4. Some problems have occurred from erosion and piping of abutment support soils due to small amounts of water flowing down behind the abutments; and
5. There are no apparent deck cracking problems associated with expansion stresses.

The interim report recommends that a reinforced concrete approach slab be used with all jointless structures.

In 1971 and 1972 the California Department of Transportation and the Federal Highway Administration sponsored a research project to correlate theoretical solutions for laterally loaded piles to full-scale field tests in bridge embankments. Most of the work was done by Mr. W. S. Yee at the University of California at Sacramento.

Mr. Yee worked with two available solutions for laterally loaded piles. The first was the non-dimensional solutions with soil modulus proportional to depth developed by Reese and Matlock.[31] This method allows analysis of variable fixity conditions at the pile top and can be used in an iterative solution for other than linear variations of the soil modulus. Mr. Yee also used the finite difference solution to the general differential equation. Since the pile is separated into small elements in this solution, any discrete variation in the soil modulus can be accommodated.

In Mr. Yee's study, however, a linear variation was assumed. The coefficient of soil modulus (n_h) was determined by measuring the deflection and rotation at the top of a laterally loaded pile as described by Davisson.[9]

Load tests were performed on instrumented piling at 3 actual bridge construction sites. Using strain gauge measurements, the moment in the pile was calculated and compared to calculated moments using the experimentally determined n_h value. A typical example of the results is shown in FIGURE 17.

Mr. Yee[42] concluded that:

1. Reliable predictions of bending moments and pile stresses could be found using experimentally determined n_h values and either the non-dimensional solution or the finite difference method;
2. The use of a linear variation in soil modulus with depth is a good approximation;
3. The influence of the soil below about 12 to 20 feet on pile stresses was practically negligible; and
4. The effective length of the pile was about 15 feet for a free-head condition and about 21 feet for a fixed-head condition.

The results of this research were used to develop guidelines for the use of integral abutments in California. They are used when up to 1 1/2 inches of total movement due to thermal forces is expected in a reinforced concrete bridge. Also to avoid rotation problems at the abutment, the end span is limited to 160 feet. The use of integral abutments is limited on prestressed bridges to those where the elastic shortening due to post-tensioning is less than 3/8 inch, and the end span is less than 115 feet (see APPENDIX II).

CALCULATED VERSUS EXPERIMENTALLY DETERMINED PILE MOMENTS

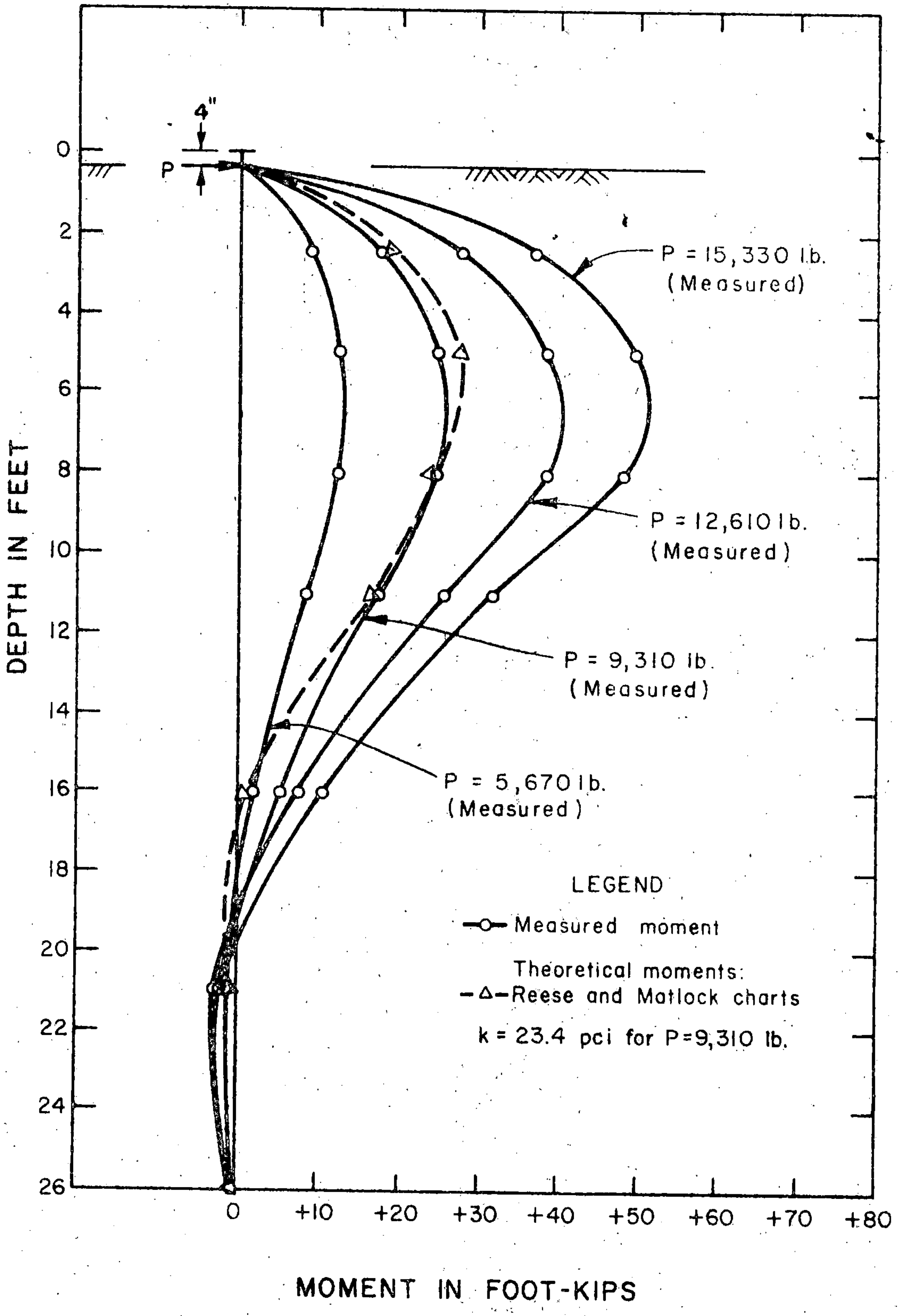


FIGURE 17[42]

b. Missouri[12]

In 1972 the University of Missouri conducted a survey and feasibility study of integral and semi-integral abutments. The work was sponsored by the Missouri State Highway Department and the Federal Highway Administration.[12] The survey was undertaken to determine current design methods and limitations used by state highway agencies. The study was made to determine the feasibility of instrumenting a jointless bridge to obtain thermal induced stresses.

The survey indicated that 13 states were using integral abutments with steel bridges and 24 with concrete bridges. The distribution of length limitations was as shown in TABLE 6. Three states allowed the use of integral abutments for non-skewed bridges only; none used them with skews over 30 degrees.

The survey concluded that:

1. The use of superstructures connected to flexible substructures was becoming generally acceptable;
2. Design limitations were more restrictive for steel bridges than concrete;
3. There was no simple design criteria which accounted for shrinkage, creep, temperature, or substructure flexibility;
4. Induced stresses resulting from thermal effects, creep, shrinkage, backfill movement, etc., are recognized by bridge engineers as potentially significant, but there is a wide variance in method for considering them; and
5. Bridge design engineers are interested in induced stresses and associated problems, are generally uncertain as to the

TABLE 6

Maximum Length (feet)	Number of States Steel	Concrete
100	2	4
200	8	6
300	2	7
400		2
450		2
500		1

INTEGRAL ABUTMENT BRIDGE LENGTH LIMITATIONS (1972) significance of and suitable methods for consideration of these stresses, and would welcome a simple, rational design criteria and specific recommendations as to design details.

In the feasibility study a temperature distribution model was developed and superstructure stresses were calculated for a wide range of temperature variations. The non-dimensional solutions for laterally loaded piles developed by Reese and Matlock[31] were used with an assumed value of the modulus of soil reaction. Instrumentation procedures were recommended for a field test to verify the theoretical results. The field test, however, was not carried out and no further work has been done on the project.

c. <u>South Dakota</u>[19]

In 1973 South Dakota State University conducted full-scale model tests on integral abutments to determine induced stresses in the superstructure and the upper portion on the piling. The model consisted of two HP 10 x 42 steel piles on 8-foot 6-inch centers cast into a rigid concrete abutment with 2 plate girders about 26 feet long. The 32-foot piles were driven into silty clay over glacial till to a bearing capacity of 23 tons. The pile tops were welded to the bottom flanges of the girders.

Various lateral displacements within plus or minus 1 inch were induced at the abutment by jacking at the free end during four construction stages. The results of interest are with the slab and backfill in place. Strains were measured corresponding to stresses of up to 42 kips per square inch in the piling. This occurred just below the bottom of the concrete abutment. Several conclusions were drawn by the investigators. They were called qualitative results which would require further study to verify.

1. Stresses were induced into the girders which in some cases were additive with dead and live load stresses. The induced stresses were generally within the 40 percent overstress allowed by AASHTO.
2. Horizontal movements over about 1/2 inch will cause yielding in the piles.
3. Free draining backfill is recommended since frozen soil against the abutment can greatly increase induced girder stresses by limiting free movement.
4. The use of approach slabs which allow rotation and translation of the abutment and, if possible, avoid continuing compaction of the backfill by traffic is recommended.

As part of this study a questionnaire was sent to 10 states in the North Central part of the United States. Two trends can be identified when the survey is compared to the responses of these states to the survey recently conducted by Iowa. Four of the states (Idaho, Missouri, North Dakota, and South Dakota) have substantially increased their length limitations for use with integral abutments. Four of the states (Iowa, Kansas, Nebraska, and Wisconsin) have retained the same limits and 2 states still do not routinely use integral abutments. Also of interest is the fact that 3 of the states have begun to routinely use integral abutments with steel bridges since 1973; 4 of them already did and 1 still does not.

d. North Dakota[17]

A recently constructed county road bridge near Fargo, North Dakota, was instrumented and monitored for temperature induced stresses by North Dakota State University. The study is being conducted by Dr. J. Jorgenson, Chairman of the Civil Engineering Department, and is sponsored by the State Highway Department.

The bridge is a 450-foot by 30-foot prestressed concrete box girder with six 75-foot spans and no skew angle. It was built in August 1979 on a very low volume gravel road.

The bridge was designed by Moore Engineering, West Fargo, North Dakota. Since the bridge length was at the limit for the use of integral abutments in North Dakota, a unique system was used to limit the passive earth pressure on the backwall. A diagrammatic representation of the abutment is shown in FIGURE 18.

The purpose of the expansion joint material behind the abutment is to hold back the soil during thermal contraction of the superstructure and to provide a collapsible mass to work against during expansion. Dr. Jorgenson informed the writer that the maximum lateral movement measured at the pile top has been about 2 inches. No distress has been noted which could reasonably be attributed to this movement. Dr. Jorgenson also reported that the bridge approach to superstructure transition was still very smooth.

The piling are founded in a deep glacial clay layer. Soft clay deposits exist near the surface and down to the limit of influence on the temperature stresses in the pile. Actual stresses in the piles are being determined from strain gauge readings for various temperature

SKETCH OF MOORE ENGINEERING INTEGRAL ABUTMENT SYSTEM[17]

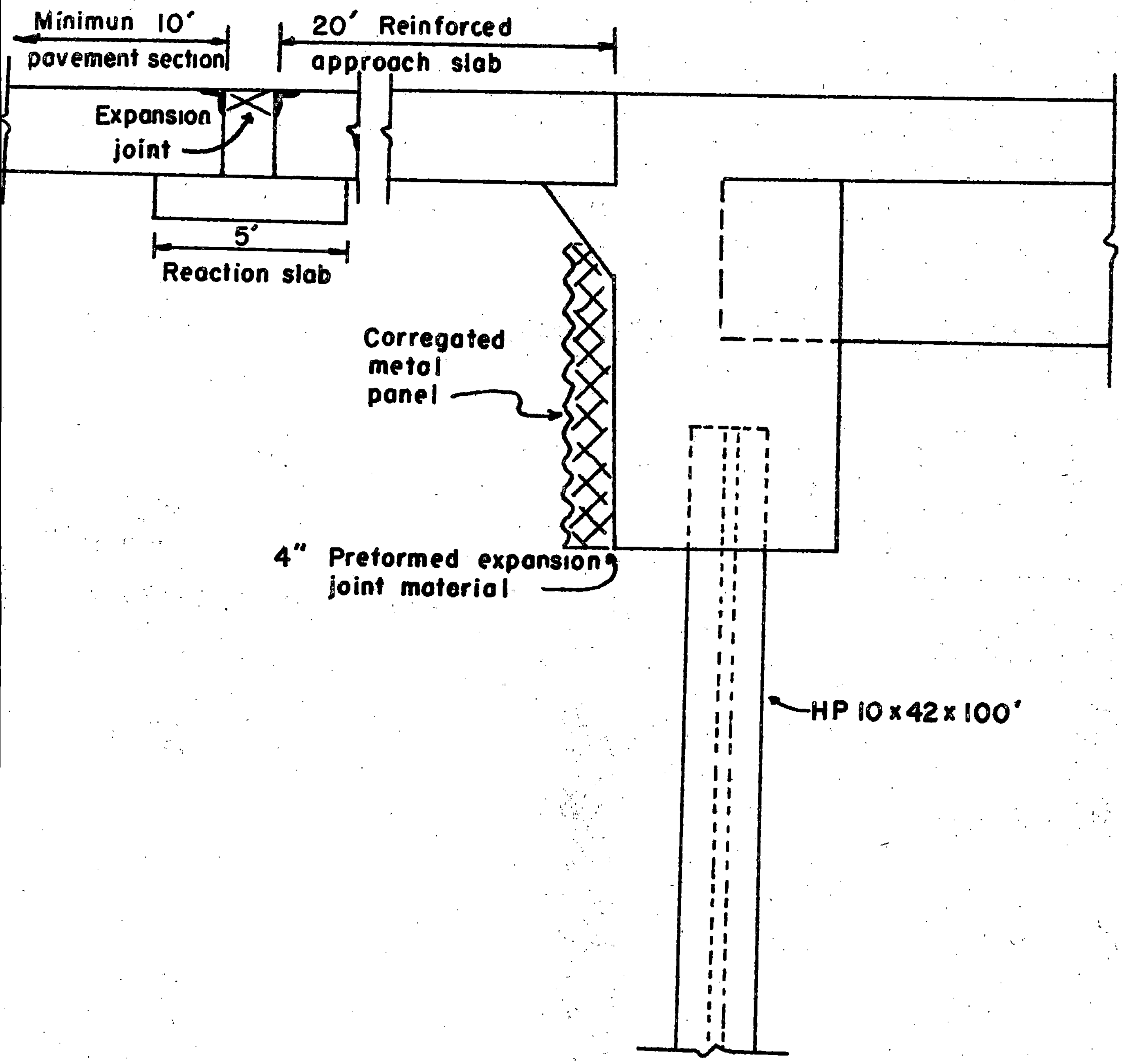


FIGURE 18

ranges throughout the year. The results of this analysis will be available in the late summer of 1981. Based on the results of the South Dakota study, it seems likely that the piles are being stressed above yield with the reported off-center deflections of up to 1 inch occurring.

7. Summary

Based on a review of available literature, the most attractive analytical approaches to obtaining solutions for pile stresses in integral abutments are iterative methods using a finite difference or finite element formulation. Both methods require knowledge of soil parameters to predict load-slip and resistance-displacement (p-y) relations.

The writer has presented recommendations for use in the development of p-y and load-slip curves to be used in analytical model for investigating pile stresses in integral abutments. TABLE 7 shows the recommended range of ultimate shear resistance (τ_{max}) and initial horizontal angle (a_o) of the load-slip curve.

The range of soil strengths recommended for developement of p-y curves is shown in TABLE 8. Other relationships for granular soils or intermediate strength cohesive soils can be developed as described in the body of the report, if desired.

Previous research work in the area of integral abutments includes:

1. Surveys of detailing and design criteria used by the state highway agencies;
2. Full-scale model tests; and
3. Monitoring performance of actual bridge installations.

The survey conducted by the University of Missouri in 1972 showed that

TABLE 7

	Maximum	Minimum
τ max(kips/foot)	3.6	0.8
a_o (degrees)	82	70

RECOMMENDED LOAD-SLIP PARAMETERS

TABLE 8

Standard Penetration Blowcount	Corresponding Shear Strength (psf) (equation (11))	Recommended p-y Relationship (equation (10))
1	210	$p=\begin{cases}(11.8 + 4.0x)y^{1/4} & \text{for } x<5.7' \\ 33.8y^{1/4} & \text{for } x\geq 5.7'\end{cases}$
25	2500	$p=\begin{cases}(1 + x/3)y^{1/3} & \text{for } x<5.8' \\ 2.9y^{1/3} & \text{for } x\geq 5.8'\end{cases}$

RECOMMENDED p-y RELATIONSHIPS

the use of integral abutments in highway bridges was a generally accepted practice. Although no simple rational design method was available, some states were building bridges up to 500 feet long without expansion devices. The few problems reported were judged to be of no greater magnitude than those experienced when movable supports and expansion devices are used.

The full-scale model tests in South Dakota in 1973 showed that for lateral pile top deflections over 1/2 inch, the stress in the upper portion of the pile may be at yield. The tests results indicated that the use of approach slabs that allow rotation and translation of the abutment was advisable. Free draining backfill was recommended in cold climate areas. Research on full-scale bridge abutments in California in 1973 showed that the non-dimensional solution as proposed by Reese and Matlock[31] and the finite element formulation could accurately predict piling stresses. The effective length of laterally loaded piles was shown to vary from 15 to 21 feet. The results of this work were used by the California Department of Transportation to develop design criteria for integral abutments which are still in use today.

The performance monitoring of an integral abutment bridge in North Dakota is still underway. With measured total deflections at the pile top of about 2 inches, the abutment appears to be functioning properly with no movement related distress.

IV. CONCLUSIONS

The responses to the nationwide survey indicate that a majority of the state highway agencies use integral abutments and are pleased with their performance. Three more states are using them with concrete bridges today than did in 1973 and 10 more are using them with steel bridges.

Most states that use integral abutments have increased their maximum allowable bridge length since 1973. Length limitations in 1973 were on the order of 200 to 300 feet. Several states are now building concrete bridges over 400 feet long without expansion joints. Many would like to increase their length limitations but are concerned about possible additional abutment distress, approach pavement failures, and overstressed piling.

Problems mentioned by some of the states seemed to be restricted to only a few respondents. Others noted that they had experienced problems but had since implemented effective solutions. Wingwalls which had cracked at the backwall interface are now being designed for greater loads with more reinforcing. Some erosion and backfill containment problems are being solved by using longer wingwalls. Many states noted the importance of using an adequate approach slab. Positive containment of runoff at the bridge ends can also help keep backfill problems to a minimum. End span rotation problems can be reduced by limiting the length of the endspan.

Several states said that unknown piling stresses were a deterrent to the use of integral abutments. Studies have indeed shown that under normal temperature variations, piling in integral abutments of long bridges will be stressed to yield. This may occur with lateral movements of as little as 1/2 inch.

Several states are now building bridges with integral abutments that can have greater potential movements. As shown by the survey, the piling stress due to thermal movements is generally ignored since no simple rational method of analysis is readily available. Some states have assumed simplified fixity conditions and effective lengths of the piles in order to calculate stresses, but they realize that the results are only approximations of the actual conditions.

Analytical methods are available that can accurately predict pile response to lateral loads, but they generally require a full-scale testing program to supply the needed soil information. Soil parameters can be estimated from standard laboratory tests, but the results are much less accurate. Another approach is to develop bounds for the soil information and analyze each critical combination of input data. To this end these limits have been established for typical Iowa soils and presented in this paper. It is hoped that this data will enable accurate analyses to be performed for use in the development of easy to use design charts capable of predicting safe integral abutment bridge designs for given soil conditions.

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VI. APPENDIX I

Questionnaire for Bridges with Integral Abutments

and

Summary of Responses

Part 1. Responses to all questions except number 4

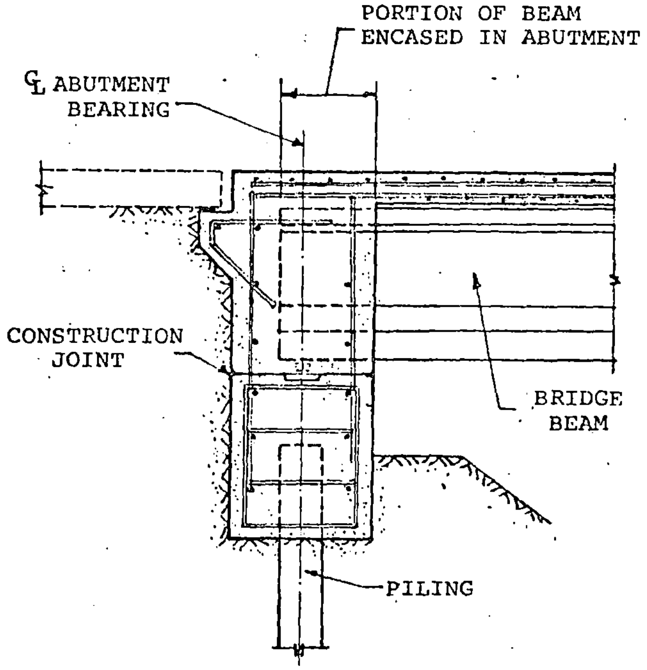
Part 2. Responses to question 4

Part 3. Additional comments made by some of the states

Note: States not listed in Part 1 answered no to question 1 and, therefore, did not complete the remainder of the questionnaire.

QUESTIONNAIRE FOR BRIDGES WITH INTEGRAL ABUTMENTS

1. Do you use bridge designs with integral abutments and without expansion devices, similar to the following sketch? yes _____ no _____ Primary (one) reason why, or why not: ______________________________.
   If the answer is no, skip the remainder of this questionnaire and please return.



2. With what type of bridges do you use integral abutments?

   steel ____ prestressed concrete ____ poured-in-place concrete _____

3. What are your maximum length limits (in feet)?

	0°	0° - 15°	15° - 30°	30° < skew
steel	______	______	______	______
prestressed concrete	______	______	______	______
poured-in-place concrete	______	______	______	______
	______	______	______	______

4. What limits, if any, do you place on the piles? (bearing vs. friction, soil type etc.)

   steel pile ______________________________
   timber pile ______________________________
   concrete pile ______________________________

5. What type of structural assumption is made for the end of the girder?

   pinned (moment equal zero) ______
   fixed (rotation equal zero) ______
   partially restrained ______ { restrained by pile ____ / restrained by soil on abut. ____ }

   other assumptions ______________________________

6. What type of structural assumption is made for the top of the pile?

   pinned (moment equal zero) ____ Is the joint detailed as a pin? ____
   fixed (rotation equal zero) ____
   partially restrained ____ { restrained by girder ____ ; restrained by soil on abut. ____ }
   other assumptions ______________________

7. What loads do you include when calculating pile stress?

   thermal ____ temperature range ______________
   shrinkage ____
   soil pressure on abutment face ____

8. How is bending accounted for in the pile?

   Neglect or assume bending stresses do not affect pile performance ____
   Assume location of pile inflection point and analyze pile as bending member ____
   Reduce bending by prebored hole ____
   Other ______________________

9. What type of backfill material do you specify on the backside of the abutment?

   ____
   ____
   ____

10. Does the approach pavement rest directly on the abutment? yes ____ no ____

11. Briefly evaluate the performance of integral abutment bridges in your state. (Compare to bridges with expansion devices).

    Construction
      relative cost more ____ same ____ less ____
      special problems ______________________
      ____

    Maintenance
      relative costs more ____ same ____ less ____
      special problems ______________________
      ____

Please return to: Lowell Greimann
420 Town Engineering
Iowa State University
Ames, Iowa 50011

PART 1, SUMMARY OF RESPONSES TO QUESTIONS 1, 2, 3, 5, 6, and 7

		Steel			Concrete			Prestressed			Girder	Pile	Pile Loads		
			Length			Length			Length		End	Top			Soil
State	Reason	Use	≤30*	>30*	Use	≤30*	>30*	Use	≤30*	>30*	Fixity	Fixity	Thermal	Shrinkage	Pressure
AL	Cost	Y	300	---	Y	---	115	Y	416	104	Pin	Pin	Y	N	Y
AZ	Maint	Y	253	N	Y	330	N	Y	404	N	Pin	Pin	Y	Y	Y
CA	Cost	Y	---	---	Y	320	320	Y	230	230	Pin	P.Res	N	N	N
CO	Cost	Y	200	---	Y	400	---	Y	400	---	Pin	Pin	N	N	Y
CT	---	Y	200	---	N	---	---	N	---	---	Pin	Fix	Y	N	N
GA	El.Jt	Y	300	---	Y	300	---	Y	300	---	Pin	---	N	N	N
IA	Cost	N	---	---	Y	265	---	Y	265	---	Pin	Fix	Y	N	N
ID	Cost	Y	200	N	Y	400	N	Y	400	N	Pin	Pin	N	N	N
IN	Cost	N	---	---	Y	---	100	N	---	---	---	---	N	N	N
KS	El.Jt	Y	300	300	Y	350	350	Y	300	300	Pin	Pin	Y	Y	N
KY	Cost	N	N	N	Y	300	N	Y	300	N	Fix	Fix	Y	N	Y
MO	El.Jt	Y	400	---	Y	400	400	Y	500	500	Pin	Pin	N	N	N
MT	Cost	Y	300	N	Y	100	N	Y	300	N	Pin	Pin	N	N	Y
ND	Maint	Y	350	---	Y	350	---	Y	450	---	Pin	Fix	N	N	N
NE	El.Jt	Y	300	---	N	300	---	Y	N	---	Pin	Pin	Y	N	N
NM	El.Jt	Y	---	---	Y	---	---	Y	---	---	P.Res.	P.Res.	Y	Y	Y
NY	Cost	Y	305	---	---	---	---	---	---	---	Pin	---	Y	N	N
OH	Cost	Y	300	300	Y	300	300	Y	300	300	Pin	Pin	N	N	N
OK	---	Y	200	N	Y	200	N	Y	200	N	P.Res.	P.Res.	N	N	N
OR	El.Jt	Y	N	N	Y	350	300	Y	350	300	Pin	Pin	N	N	N
SD	Cost	Y	320	---	Y	450	---	Y	450	---	Pin	Fix	N	N	N
TN	El.Jt	Y	400	400	Y	800	800	Y	800	800	Pin	Pin	N	N	N
UT	El.Jt	Y	300	250	N	---	---	Y	300	250	Pin	Pin	N	N	N
VA	Simp.	Y	242	---	N	---	---	Y	454	---	Pin	Pin	N	N	Y
VT	Cost	Y	150	100	N	N	N	N	N	N	P.Res.	P.Res.	Y	N	N
WA	Cost	N	---	---	Y	350	---	N	---	---	Pin	Pin	N	N	N
WS	Cost	Y	200	200	Y	300	N	Y	300	300	P.Res.	Fix	N	N	N
WY	Simp.	Y	300	300	Y	500	500	Y	500	500	Pin	Pin	N	N	N
R15	El.Jt	N	N	N	Y	270	160	Y	300	240	P.Res.	Pin	N	N	N

Y Yes
N No
--- No Response
* Bridge skew in degrees

PART 1, SUMMARY OF RESPONSES TO QUESTIONS 8, 9, 10, and 11

State	Pile Bending Neglect	Pile Bending Infl. Pt.	Pile Bending Prebore	Backfill	Approach Pavmt. on Abutment	Construction Cost More	Construction Cost Same	Construction Cost Less	Maintenance Cost More	Maintenance Cost Same	Maintenance Cost Less
AL	Y	Y	N	Gran.	N	N	N	Y	N	N	Y
AZ	Y	N	N	Cohes.	Y	N	N	Y	N	N	Y
CA	Y	N	N	Perv.	Y	N	N	Y	N	N	Y
CO	Y	N	Y	Gran.	Y	N	N	Y	N	N	Y
CT	Y	N	N	Perv.	Y	N	N	Y	N	N	Y
GA	Y	N	N	Rd.Fill	Y	N	N	Y	N	N	Y
IA	N	N	Y	Gran.	Y	N	N	Y	N	N	Y
ID	Y	N	N	Rd.Fill	Y	N	N	Y	N	N	Y
IN	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
KS	Y	N	N	Rd.Fill	Y	N	N	Y	N	N	Y
KY	N	Y	Y	Gran.	N	N	N	Y	N	N	Y
MO	Y	N	N	Rd.Fill	Y	N	N	Y	N	N	Y
MT	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
ND	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
NE	Y	N	N	Rd.Fill	Y	N	Y	N	N	N	Y
NM	N	Y	N	Rd.Fill	Y-N	N	N	N	N	N	N
NY	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
OH	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
OK	Y	N	N	---	Y	N	N	Y	N	N	Y
OR	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
SD	N	N	Y	Gran.	Y	N	N	Y	N	N	Y
TN	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
UT	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
VA	Y	N	N	Gran.	N	N	N	N	N	N	N
VT	Y	N	N	---	N	N	N	Y	N	N	Y
WA	N	N	N	Gran.	Y	N	N	Y	N	N	Y
WS	Y	N	N	Gran.	N	N	N	Y	N	N	Y
WY	Y	N	N	Gran.	Y	N	N	Y	N	N	Y
R15	Y	N	N	Perv.	Y	N	N	Y	N	Y	N

Y Yes
N No
--- No Response

PART 2, SUMMARY OF RESPONSES TO QUESTION 4

State	Steel	Timber	Concrete
AL	*	*	*
AZ	9 ksi in Brg., <9 ksi in Fric.	Not used	In friction only
CA	Assume 5 kips Lat. Resis./pile	Same as steel	13 k. Lat. R./pile
CO	*	Not used	Not used
CT	Use in bearing	---	---
GA	Use in weak axis	Not used	Not used
IA	Use in weak axis, Fric. only	Use if Br. Length<150'	Not used
ID	*	Not used	Not used
IN	Use H-pile or shell	---	---
KS	Mostly used in bearing	Mostly used in bearing	Mostly used in Brg.
KY	Use in Brg. or friction	---	Used in friction
MO	10' minimum length	Not used	Used in friction
MT	9 ksi in bearing	Used in friction	Not used
ND	*	*	*
NE	Used in weak axis	---	---
NM	Use steel only	Not used	Not used
NY	*	Not used	*
OH	*	Not used	*
OK	Use in bearing	Not used	Not used
OR	*	Not used	*
SD	*	*	*
TN	*	Not used	*
UT	Use in single row	Use in single row	Use in single row
VA	Upper portion allowed to flex	---	---
VT	15' minimum length	Not used	Not used
WA	Use in bearing or friction	Use in Brg. or Fric.	Use in Brg. or Fric.
WS	Use in bearing or friction	Use in friction	Use in Brg. or Fric.
WY	Use in bearing or friction	Not used	Not used
R15	Use in weak axis	Not used	Not used

*   No Limitations
--- No Response

PART 3, SUMMARY OF ADDITIONAL COMMENTS MADE BY SOME OF THE STATES.

Arizona

The additional lateral movement associated with this system, particularly with cast-in-place, post-tensioned concrete box girders, dictates longer wingwalls for backfill containment and the careful compaction of backfill material. Also, an adequate drainage system must be provided to prevent surface runoff from entering voids created at the ends of the wings and approach slabs; otherwise, progressive erosion of the approach embankment and under the approach slab occurs.

Alaska

No special construction or maintenance problems were noted.

California

The abutment is not stable when standing alone during construction if the backwall height is too great. Wingwalls must be cast after stressing of cast-in-place prestress construction to avoid rotation and translation of walls. If soils don't yield, piling absorbs a large amount of prestressing force resulting in a large rotation at abutments and a large downward deflection in the span. This has been a particular problem with simple span cast-in-place prestress construction.

Colorado

We do have some problems with settlement of backfill behind the abutment and cracks in the asphalt pavement, but the problems are much less than the problems associated with snowplows and bridge expansion devices and bearing devices.

Connecticut

We have constructed one bridge to date and are very satisfied with it.

Georgia

Have had a problem with cracks in the wingwalls.

Idaho

Some problems have resulted from failing to provide adequate expansion joints in concrete approach pavements, but such problems are not peculiar to design concept under consideration. Problems are to be expected if the bridge is long, has no expansion joints anywhere, is a steel bridge, is on a substantial skew, or a combination of the foregoing. If used with discretion, the design concept is good in that it saves initial and maintenance costs of expansion joints.

Kentucky

No special construction or maintenance problems have been reported.

Missouri

We limit integral abutment bridges to a 40 degree skew.

Montana

No special contruction problems noted. Integral abutment bridges probably require a little more maintenance due to embankment settlement.

Nebraska

Maintenance can be a problem if no concrete approach slab is provided.

New York

We assume that construction costs are lower because of simpler abutment forming details and fewer piles. Setting the girders directly on the piles created some alignment difficulty for the contractor. In the future we plan to use a detail similar to the detail shown in No. 1 of your questionnaire.

The continuous approach slab on a 125 foot single-span steel bridge built in 1980 has cracked at the rear face of the backwall. It is a tight crack that runs tha full width of the slab but does not appear to be detrimental. To date, no detectable cracking has occured in the backwalls and the abutments seem to be functioning as designed.

Ohio

As yet, no significant construction or maintenance problems have been noted.

Oklahoma

Integral abutments are used only on bridges with zero skew.

South Dakota

With steel bridges and longer concrete, we still utilize an expansion device in the approach slab system. Savings is in bearings and piling. Sill or abutment does not have to be designed for overturning loads.

For most steel bridges and longer concrete, we feel it is necessary to attach the approach slab with integral curb and gutter to the bridge. Without this provision, severe erosion around the wings can result and problems with approach fill settlement are increased.

Utah

No special construction or maintenance problems have been noted.

Vermont

Some minor approach settlement is anticipated.

Washington

Sometimes the piles may not end up in a straight line and at the right location. Some maintenance problems with downdrag and settlement have been noted.

Wisconsin

Cracking of diaphragms has been noted on bridges with large skews (greater than 20 degrees) and/or with long abutments. We limit integral abutment bridges to 40 degree skews.

Wyoming

No special construction or maintenance problems have been noted.

FHWA Region 15

We noted a problem with pavement cracking at bridge ends. This has since been eliminated with the use of approach slabs.

VII. APPENDIX II

Memorandum to Designers, Office of Structures Design, California Department of Transportation

This memorandum was attached to California's response to the integral abutment questionnaire. It describes California's criteria for the use of end diaphragm abutments, which includes both integral and semi-integral types. Examples of details used by California are shown in FIGURES 1 and 2 in the body the this report.

MEMO TO DESIGNERS:

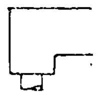
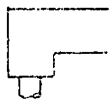
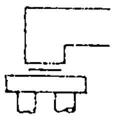
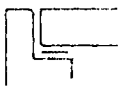
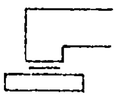
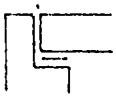
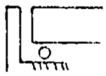
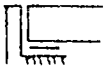
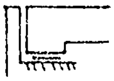
The end diaphragm is an integral part of the bridge superstructure. Frequently this diaphragm is extended below the soffit of the superstructure to rest directly on piles or on a footing. This type of support is an "End Diaphragm Abutment." The discussion here will be limited to those situations where the diaphragm is fixed at the soffit and in effect is a cantilever beam between the soffit and the base which rests on piles or a footing.

Structure Movement:

Thermal movements are easily absorbed by this abutment. Concrete bridges of 400 feet between abutments, when conventionally reinforced, have shown no evidence of distress even though the end diaphragms rested directly on piles.

Elastic shortening due to post tensioning, however, is rapid and must be provided for in the abutment design when the initial shortening due to stressing exceeds 3/8". When the span adjacent to the abutment exceeds about 160 feet, there could be an additional problem of rotation. To minimize the damage to the abutments of single span post tensioned structures due to earthquake, both abutments should be on sliding supports when that is the recommended treatment (See table below).

Below are listed some guidelines for use in providing for abutment movement. The limits shown are by no means absolute, but illustrate a conservative approach to the problem. Seat-type abutments are advisable where movement ratings are equal or greater than 1-1/2 inches.

SUPPORT TYPE	LIMITING CONDITION Initial shortening due to stressing or length of end span	RECOMMENDED TREATMENT	
		Prestressed	Conventional
Driven Piles	0 to 3/8" (Spans up to 115')	 No special treatment	 No special treatment
	3/8" to 1" (Spans 116' to 160')	 or  Sheet metal over neoprene strip or elastomeric pad.	
CIDH Piles	0 to 1" (Spans up to 160')		
Spread Footing	0 to 1" (Spans up to 160')	 or  Sheet metal over neoprene strip or elastomeric pad.	
All Types	Over 1" (Spans over 160') (Conventional - when M.R. ≥ 1½")	 Roller or   Elastomeric Pad	

Previous Memo dated 8-25-71

Restraining Forces:

Listed below are assigned values for resistance offered by various end conditions. This force is applied at the base of the end diaphragm to determine the proper reinforcement. The values shown do not take into account the special situations where very long piles or small limber piles offer little resistance to longitudinal movement. Note that earthquake longitudinal force may govern over those shown below. See Section 2-25 Bridge Planning & Design Manual, Volume I.

Abutment Type	Design Logit. Force
End Diaphragm on CIDH piles	*25 kips per pile
End Diaphragm on Concrete Driven Piles	*20 kips per pile
End Diaphragm on 45T Steel Piles	*15 kips per pile
End Diaphragm on Neoprene Strip or Pads	15% of dead load
End Diaphragm on Rollers	5% of dead load

*These values are intended for use in the design of end diaphragm only. For determining the number of piles required for longitudinal force, see Section 4-15.8(3) of Bridge Planning & Design Manual, Vol. I.

Earthquake Forces:

Shear keys must be added to provide resistance to transverse and longitudinal earthquake forces acting on the structure. These normally will be placed behind and at the ends of the abutment wall on narrow structures. On wide structures, additional keys may be located in the interior. One half inch expansion joint filler should be specified at the sides of all keys to minimize the danger of binding. For earthquake design forces, see Section 2-25.2, Bridge Planning & Design Manual, Vol. I. For key sizes and key reinforcement, see Section 1, Bridge Planning & Design Manual, Volume III.

Drainage

1. No pervious material collector or weep holes required for flat slab bridges.
2. Continuous pervious backfill material collector and weep holes may be used for abutments in fills or well drained cuts in desert locations and at sites where a 5-ft level berm is specified.

November 15, 1973 5-2

Drainage (Cont'd.)

End Slope Treatment	Weep Hole Discharge
Unprotected berm	Directly on unprotected berm
Bib slope paving	C. spacer or groove in paved surface
Full slope paving	On spacer on groove in paved surface

3. Continuous permeable material and Perforated Steel Pipe collector discharging into Corrugated Steel Pipe overside drains should be used for all other abutments.

4. Corrugated Steel Pipe overside drains must be coordinated with road plans. If there is no discharge system and no collector ditch, the outfall must be located away from the toe of slope to prevent erosion of the end slope.

5. Abutment drainage systems should be coordinated with the slope paving. See Memo to Designers 5-10.

Backfill Placement

Unless there are special soil conditions or unusual structure geometrics, the designer need not specify the method or timing of backfill placement. Passive resistance of soil in front of the end diaphragm offers little restriction to structure movement due to stressing. Nor will the active pressure of backfill behind the end diaphragm materially alter the stress pattern even if the fill is completed at one abutment before being started at the other.

Suggested Details:

Sketches showing suggested abutment details are located in Bridge Planning and Design Manual, Volume IV, Detailer's Guide.

G. A. Hood

G. A. Hood

W. J. Jurkovich

GGF:bt

VIII. APPENDIX III

Iowa Department of Transportation

Foundation Soils Information Chart

FOUNDATION SOILS INFORMATION CHART

A majority of the bridge foundations designed by the Highway Division, Iowa Department of Transportation rest upon piling which derive their support primarily from the shear strength of the surrounding soil rather than from end bearing. Economical and safe design of such foundations requires a knowledge of the bearing capacity of the foundation soils. A chart for pile length determination based upon the available information and experience was first introduced in 1958. This chart provided a feasible method of selecting pile lengths which effectively reduced pile cut-off. As more information becomes available, it is necessary that the "Foundation Soils Information Chart", used for estimating pile lengths, be periodically updated.

A total of 234 pile load tests have been performed since 1950. To properly evaluate the information, the tests were categorized as (a) pile tested to yield, and (b) pile tested to bearing. Of the total, 117 pile load tests were grouped into the "pile tested to yield" category. To evaluate the bearing capacity of foundation soils the piles tested to yield were reviewed, excluding the inconclusive tests. Sufficient numbers of conclusive tests are available for review.

The pile load tests performed on piles founded in only one foundation soil have enabled establishing a definite bearing value for that soil. Pile tests on certain soils have indicated a need for change in the bearing values given in the previous charts.

All available foundation soil information has been evaluated and incorporated in the revised design chart. Blow count values (N-Values*) obtained from standard penetration tests performed on foundation soils and bedrocks have been included in the chart and in the additional recommendations. Statistical analysis was used to determine the mean value and standard deviation for blow counts on all soils.

Evaluation of pile load tests performed upon tapered steel shell piles on the I-129 project at the Missouri River crossing south of Sioux City indicate that the bearing value of the tapered pile in cohesionless foundation soils is greater than the bearing for parallel sided pile. However, the bearing value for tapered piling is not as high as originally indicated by the test loads made at the Council Bluffs viaduct. The additional column for steel shell piles has been left in the revised chart but the

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*N-Value: The number of blows required by a 140 lbs. hammer with a free fall of 30 in. to drive a 2 in. O.D. by 1-3/8 in. I.D. split tube sampler 1.0 ft. into the soil.

value have been reduced. According to Peck* the effect of taper pile in unconsolidated cohesive soils does not increase the bearing capacity of the pile.

The attached "Foundation Soils Information Chart" gives the allowable friction bearing per foot length of pile for different types of piles in different foundation soils. The chart and the methods of pile length determination described on subsequent pages will allow the designer to effectively select adequate pile lengths. To make effective use of the chart, the sounding nomenclature should compare with the chart nomenclature. The revised chart and the information contained herein will be subject to change as additional information becomes available.

The hammer formulas used for pile driving during construction shall conform to the Standard Specifications and current Supplemental Specifications unless otherwise specified. The present hammer formulas are used as a check for pile bearing during construction. When the formula bearing for a pile is less than the design bearing, a pile load test should be secured.

The "Foundation Soils Information Chart" is intended to be an effective aid in selecting proper pile lengths. At stream crossings where scour may be a problem, tip penetration should

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*Peck, Ralph B.: A Study of the Comparative Behavior of Friction Piles: Washington, D.C.: Highway Research Board: Special Report #36: 1958.

be specified. Preliminary Bridge Design will determine the approximate scour depth.

Where compressible (unconsolidated) soils are under a fill, the fill should be predrilled, and drag forces calculated in accordance with the method described elsewhere.

A steel test pile in Johnson County was tested by pulling. The resultant allowable bearing value for very firm glacial clay fill was 0.3 tons per foot in uplift.

Soil Description	Mean N-Value	Range* of N-Value	Estimated Allowable Bearing Value for Friction Piles in Tons per Foot (Factor of Safety = 2.0)								
			Wood Pile	Steel "H" Pile	Concrete Pile		Steel Shell Pile				
							Parallel Sided				Tapered
					16"	14"	18"	14"	12"	10"	12"(Av.)
Alluvium or Loess											
Very soft silty clay	1	0-1	0.3	0.2	0.5	0.4	0.3	0.3	0.3	0.2	
Soft silty clay	3	2-4	0.3	0.2	0.5	0.4	0.3	0.3	0.3	0.2	
Stiff silty clay	6	4-8	0.5**	0.4	0.8	0.7	0.5	0.5	0.4	0.4	
Stiff silt	5	3-7	0.5	0.4	0.8	0.7	0.5	0.5	0.4	0.4	
Stiff sandy silt	5	4-8	0.5	0.4	0.9	0.8	0.5	0.5	0.4	0.4	
Stiff sandy clay	6	4-8	0.7	0.6	0.9	0.8	0.6	0.6	0.5	0.4	
Silty sand	8	3-13	0.8	0.7	1.0	0.9	0.6	0.6	0.5	0.4	
Clayey sand	13	6-20	0.7	0.6	1.0	0.9	0.6	0.6	0.5	0.4	
Fine sand	15	8-22	1.0	0.6	1.1	1.0	0.7	0.7	0.6	0.5	0.9
Coarse sand	20	12-28	1.2	0.9	1.2	1.1	0.9	0.9	0.8	0.6	1.2
Gravelly sand	21	11-31	1.6	0.9	1.6	1.6	1.3	1.2	1.0	0.9	1.6
Glacial Clays											
Firm silty clay	11	7-15	1.0	0.7	0.9	0.8	0.7	0.6			
Firm silty gl. clay	11	7-15	1.0	0.8	1.0	0.9	0.7	0.6			
Firm clay (Gumbotil)	12	9-15	1.0	1.0	1.0	0.9	0.7	0.6			
Firm glacial clay	11	7-15	1.4	0.9	1.1	1.0	0.9	0.8			
Firm sandy gl. clay	13	9-17	1.4	0.9	1.1	1.1	0.9	0.8			
Firm-very firm gl. clay	14	11-17	1.4	1.2	1.2	1.1	0.9	0.8			
Very firm gl. clay	24	17-31	1.6	1.4	1.6	1.7	1.4	1.3			
Very firm sandy gl. clay	25	15-35	1.6	1.4	1.6	1.6	1.4	1.3			

*Range = Mean ± 1 Std. Deviation

**Underlined values determined from pile load tests to yield.

Note: Glacial Soils with N-values greater than 35 and granular soils with N-values greater than 50 MUST be given special consideration.

Date: January, 1967
Revised: June, 1976

ADDITIONAL RECOMMENDATIONS

1. Do not end a pile in a foundation material for which N-Value is 4 or less.
2. For wood friction piles, calculate the pile length from the total estimated safe bearing based on the design load and select the nearest pile length in multiples of 5 feet.
3. For a steel pile, the allowable load over the cross sectional area of the tip of the pile shall not exceed the following:

   6,000 psi in bedrocks for which N = 20 - 200

   9,000 psi in bedrocks for which N = 200 or more
4. When driving steel pile into bedrock, the following penetration is recommended:

   8 ft. to 12 ft. in broken limestone, where practicable.

   8 ft. to 12 ft. in shale or firm shale (N=20 to 50).

   4 ft. to 10 ft. in medium hard shale, hard shale or silt stone (N=50 to 200).

   3 ft. to 6 ft. in sandstone, siltstone, or hard shale (N=200 or more).

   1 ft. to 3 ft. in solid limestone.
5. If spread footing foundations are considered for a structure, additional core borings should be obtained to determine the allowable bearing value of the foundation material. In

absence of any other data, the allowable bearing value may be adopted from the following table:

Bedrock	Average N-Value	Allowable Bearing Value, tons/sq. ft.
Shale	16	2
Firm Shale	25	3
Med. Hard Shale	50+	5
Hard Shale	50+	5
Siltstone	50+	5
Sandstone	50+	5
Limestone	100+	10

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I express thanks to all those in the highway agencies who helped to make this report complete by returning the survey questionnaire. I offer a special thanks to those who provided additional information concerning their design and detailing procedures.