rigidity for earthquake loads when liquification of embankment is assumed.

2. Thermal expansion is actually very small, and the backfill material around the abutment and the piling seems to yield sufficiently so that no distress is apparent. The piling was oriented to resist the force of earth pressure from the abutment backfill rather than the force of thermal expansion.

3. Temperature forces would act along the centerline of the roadway, not parallel to the pile web, and active soil pressure would act against the strong axis of the pile. Temperature effects are somewhat compensated for by predrilling for driven piles and filling the voids with pea gravel or sand.

No special treatments are usually given to backfill and pile cap on skewed bridges, and they might be constructed in the same way as on nonskewed bridges. As for the approach slab, it can be tied to the abutment with dowels or an expansion joint may be provided between the approach slab and the bridge slab. Some states put an expansion joint a certain distance behind the approach slab. In this case, the approach slab will act integrally with the abutment.

It has been more than 15 years since the first integral abutments on skewed bridges were constructed. No serious problems or distresses have yet been discovered. In view of the lack of theoretical and experimental research in this area, it is hoped that this survey will provide some useful empirical experience and information on the design of skewed bridges with integral abutments.

ACKNOWLEDGMENT

The study discussed in this paper was conducted by the Engineering Research Institute of Iowa State University and was sponsored by the Highway Division of the Iowa DOT through the Iowa Highway Research Board. We wish to express our gratitude to the bridge engineers in all 50 states for the 100 percent return rate on the survey questionnaire. In particular, special thanks go to those who provided additional information concerning their design and detailing procedures. We also wish to extend sincere appreciation to the engineers of the Iowa DOT for their support, cooperation, and counseling. Special thanks are extended to Bruce Johnson, Charles A. Pestotnik, Henry Gee, Vernon Marks, Kermit L. Dirks, and Wallace W. Sanders, Jr.

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Publication of this paper sponsored by Committee on General Structures.

Behavior of Abutment Piles in an Integral Abutment in Response to Bridge Movements

JAMES L. JORGENSON

A field study of the behavior of abutment piles for a bridge that has integral abutments, piers, concrete box girders, concrete deck, and six 75-ft spans is discussed. To compensate for anticipated thermal movements, two unique features were built into the bridge. Expansion joint material was placed between the back side of the abutment and the soil backfill, and compressible material was placed on the webs of the abutment piles to create low soil resistance to pile movement. Over a one-year period, monthly readings were taken of bridge length (by using steel tape), gap between soil backfill and back side of abutment, openings in the expansion joints on the concrete approach slabs, vertical elevation of abutments and piers, slope indicator readings on the four corner abutment piles, and temperatures of concrete deck and air. A formula involving air temperatures was developed to estimate the maximum change in bridge length due to thermal changes. The changes in bridge length agree with changes measured from steel tape and expansion joint openings. The study concluded that these changes did not result in equal abutment movements at each end of the bridge, and the maximum abutment movement resulted in stresses at the top of the piles sufficient to initiate a yield stress in the steel but not sufficient to form a plastic hinge. An analytic model was used to predict stresses in the abutment piles due to movements of the abutments.

Bridge engineers recognize that changes in air temperature result in changes in the temperature of bridge materials, which in turn result in movements of the bridge. So long as the bridge components (girders, piers, and abutments) are not restrained from movement, the movements do not create stresses in the bridge. However, if the girders, piers, and abutments are integral, thermal movements result in stresses in those bridge components. The bridge designer must then elect to use expansion joints, an integral structure, or some combination of the two.

My own observations and those of others (1) indicate that in many cases the expansion devices do not function as assumed by designers. This behavior has interested designers in eliminating the expansion devices and considering the use of integral structures.

A survey of bridge designers on the use of integral-abutment bridges (2) indicates that the maximum bridge length without an expansion joint is 400-450 ft for concrete structures. In addition, the induced stresses from thermal effects are recognized as being potentially significant.

The North Dakota State Highway Department has been using integral abutments on structures up to 350 ft in length. However, the Department is concerned about the magnitude of thermal movements and resulting stresses in long bridges. To respond to this need, the Department contracted with the Engineering Experiment Station of North Dakota State University to study the behavior of a 450-ft integral-abutment bridge in Cass County (see Figure 1).

The bridge studied is on Cass County Road 31, about 2 miles north of Fargo, and provides a crossing of the Cheyenne River. Construction took place between July 1978 and August 1979. In 1978 the old bridge was removed, rough grading was completed, all piles were driven, abutment and pier caps were poured, and the prestressed concrete girders were placed. In addition, at least one pier diaphragm and one intermediate diaphragm were poured. Work on placement of the concrete deck began in the spring of 1979, and the deck was poured in July. The remaining items were completed in time for the August 1979 opening to traffic.

The Cass County Bridge is a 450-ft-long concrete bridge with integral abutments and piers. There are no expansion joints on the bridge, but expansion joints are located in the approach slab about 30 ft from each end of the bridge.

A transverse section through the bridge deck is shown in Figure 2. Prestressed concrete box girders were used to support a poured-in-place concrete deck. The girders and deck were designed to act as a monolithic unit even over the piers (see Figure 3). The concrete curb is tied into the concrete deck; however, the curb does have expansion joints at 15-ft intervals.

Continuity at the piers is also shown in Figure 3. The steel pile, pier cap, diaphragm, concrete girder, and concrete deck were all reinforced to behave as a single unit. For the center three piers, the piles are oriented with their strong direction of bending in the longitudinal direction of the bridge. A concrete wall was placed between the pile in each pier. The piles in the piers adjacent to the abutments have their weak direction of bending in the longitudinal direction of the bridge.

A section through the abutment is shown in Figure 4. As with the piers, the pile cap, diaphragm, concrete girder, and concrete deck are reinforced to act as a single monolithic unit. The pile is oriented with its weak direction of bending in the longitudinal direction of the bridge and is reinforced within the abutment cap and diaphragm to transmit the full plastic movement of the pile. In anticipation of thermal changes in the length of the bridge, a pressure relief system was set up between the back side of the abutment and the backfill soil. The system consists of 3-1/3-in-wide by 4-in-thick pressure relief strips placed vertically at about a 4-ft spacing on the back side of the abutment. Corrugated metal was used to retain the granular backfill behind the pressure relief strips. The material used for pressure relief strips was "Pressure Relief Joint", manufactured by W.R. Meadows. According to data sheet 324 on this product, the material will recover 96 percent of its thickness after being compressed to 50 percent of its thickness.

To permit longitudinal movement of the abutment piles without generating significant resistance to movement, a 2-in layer of compressible material was placed on each side of the web of the pile. The soil was predrilled to a 16-in diameter and a 20-ft depth. Compressible material was glued to the pile; after pile driving, the void space was filled with sand (see Figure 5). The compressible material was Ray-Lite with a density of 1.25 pcf and a compressive strength of 8-16 psi.

There is an expansion joint in the approach slab at 20 ft from each end of the bridge. One end of the approach is tied into the bridge abutment, and the other rests on the smooth surface of a supporting slab. As the bridge changes in length, the expansion joint will open and close.

The soil profile consisted of a glacial-lake-deposited clay to a 100-ft depth underlaid by glacial till. The soil profile under the north abutment was as follows: 15 ft of fat clay of medium stiffness, 6 ft of soft silty clay, 4 ft of stiff fat clay, 70 ft of fat clay of medium stiffness, and 17 ft of very stiff sandy clay, followed by silty sand that was very dense. The piles were about 110 ft long and extended into the silty sand.

MEASUREMENTS OF BRIDGE MOVEMENTS

Measurements of bridge movements were taken during the period from August 8, 1979, through September 7, 1980. Some measurements were taken prior to August 8, but they were not complete sets of data. The readings were taken at about one-month intervals except during September 6 and 7, when they were taken at 6-h intervals. (Because early morning temperatures will not contain the highest daily temperature, it was decided to take additional readings at about 6-h intervals over a 24-h period.)

Readings were taken early in the morning just after daylight. When the sun is shining on the bridge, the exposed concrete surface increases in temperature at a higher rate than the remaining concrete. Hence, the sun causes unequal temperatures within the concrete. After sundown, the concrete temperature is influenced by the current temperature of the concrete and the air temperature. By daybreak, the air will have had the best possible chance to equalize the temperature throughout the concrete. Other researchers have found the temperature of the concrete to be fairly uniform at daybreak (3).

Change in Bridge Length

The length of the bridge was measured by placing a 500-ft-long steel tape on top of the concrete curb and measuring the distance between markers, which were cast into the concrete near the ends of the curb. The tape length was corrected for temperature change with the air temperatures taken as the tape temperature.

One way to determine the change in length of the bridge would be to sum the movements of the abutments. This is shown in Figure 6 along with the change in bridge length determined by tape measure-

Figure 1. Plan and elevation of integral-abutment bridge.

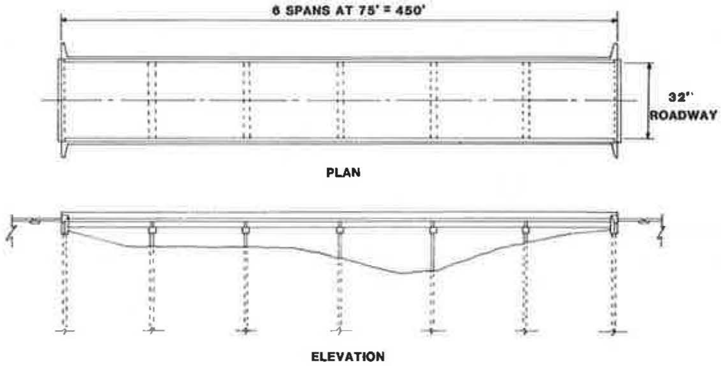


Figure 2. Transverse section through deck.

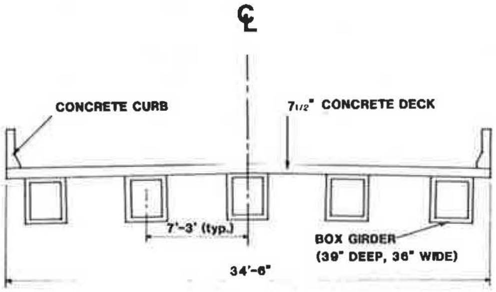


Figure 3. Longitudinal section through pier.

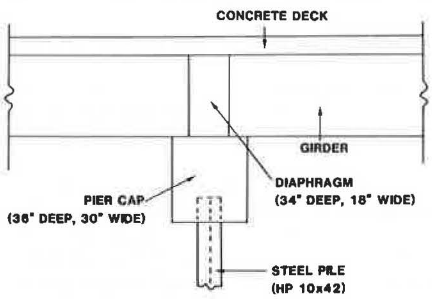


Figure 4. Section through abutment.

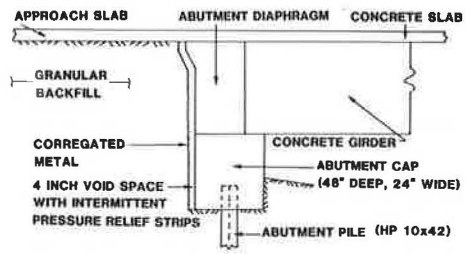
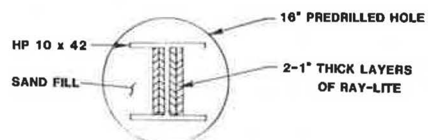


Figure 5. Compressible material on abutment piles.



ment. One line shows the sum of movements in the expansion joints on the approach slabs. Another line shows the sum of the changes in gap openings between the abutment and the backfill. Note that the readings are about equal except for the tape readings at 12:00 p.m. Because the expansion joints are located 20 ft from the end of the bridge, changes in the temperature of the approach slab will cause expansion joint readings to be slightly different.

Bridge Temperature

Air temperature versus average concrete deck temperature for the 24-h period is shown in Figure 7. Air temperature (67°) and deck temperature (65°) were about equal at 7:00 a.m. During the morning there was a 17° rise in air temperature but only a 5° rise in deck temperature. The following 6 h brought the opposite effect--a 10° rise in air temperature and a 23° rise in deck temperature. The same pattern occurs for a drop in temperature--i.e., the change in deck temperature lagging the change in air temperature. The 7:00 a.m. readings indicated the deck and air temperature to be within 2° of each other.

An interesting question is whether the change in deck temperature can explain the change in length of the bridge. Information on this question is presented in Figure 8. One line represents the change in bridge length determined by tape measurement. The other line shows a calculated change in length based on change in deck temperature. A thermal coefficient of $6x10^{-6}$/°F was used in the calculations. Nearly equal changes in length occur at 12:00 midnight and 7:00 a.m. the next morning. The largest difference occurs at 6:00 p.m. The results suggest that the deck temperature is not the temperature of the entire bridge at least during portions of the day. The slab is open to the direct sun while the box girders are shielded from the direct sun. The lower temperature of the box girders accounts for the reduced change in length of the bridge.

Over a one-year period, tape measurements taken indicate that, for the nine readings shown in Figure 9, the average error in readings was 0.40 in and the range was 0.0-1.1 in. There is a correlation between these measurements and the data obtained from slope indicators fixed to each of the corner piles in each abutment, although individual slope indicator readings may contain errors.

What are the maximum measured movements of each abutment during the one-year period? By using the measured value of the expansion joint opening as the best indicator for the north abutment movement, a value of 0.73 in was obtained on February 26, 1980. With 2.34 in as the change in bridge length for that date, the south abutment moved 1.60 in. On January

Figure 6. Change in length of bridge: abutment movements.

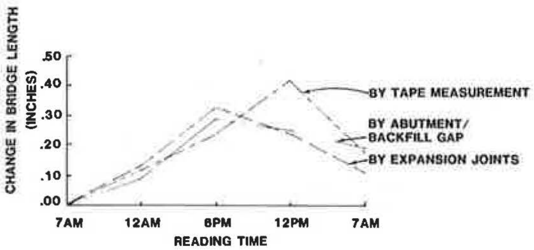


Figure 7. Air temperature versus concrete deck temperature over one day.

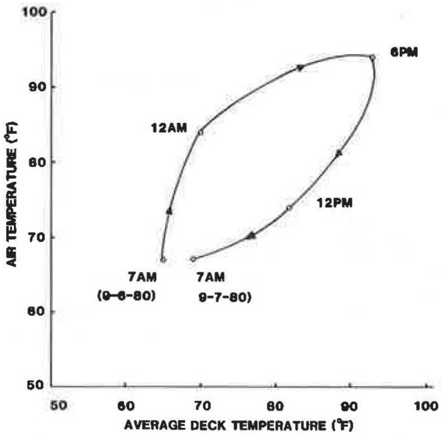
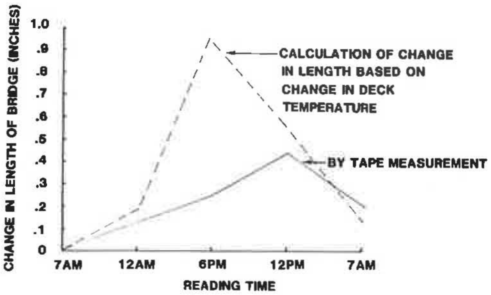


Figure 8. Change in length of bridge: deck temperature.



30, 1980, the change in bridge length was measured by tape to be 0.37 in greater than on February 26, 1980. Hence, the south abutment could have moved 1.96 in depending on the expansion joint opening, which was not measured that day due to ice in the joint.

Two sources of temperature measurements were used: the air temperature and the temperature of the concrete deck. A plot of these two temperatures is shown in Figure 10. If the air and deck temperatures were equal, all points would be on the diagonal line. The temperatures would not necessarily be equal since fluctuations in air temperature take place much quicker than fluctuations in deck temperature. However, the fact that air temperatures decreased to less than 17°F while the deck temperature remained at 17°F is questionable. Did the deck temperature stay at 17°F or did the temperature reading equipment malfunction in that temperature range? That question can best be answered by determining which temperature (air or deck) relates to change in bridge length.

This information was calculated and is shown in Figure 11. Two lines represent the calculated change in bridge length based on changes in air temperature and deck temperature. The third line is the change in length of the bridge based on tape measurement. For the two temperatures in question in January and February, the changes in deck temperature do not account for the changes in bridge length. Hence, the equipment used to measure deck temperature malfunctioned at temperatures below 17°F. Figure 11 shows the closeness of change in length based on air temperature at sunrise.

Temperature Influence on Length

The maximum change in bridge length due to temperature change can be estimated by first calculating the change in length from dawn on the coldest day of the year to dawn on the hottest day of the year and then adding an estimate of the change in length during the hottest day of the year. For the one 24-h period studied, the dawn air temperature was 67°F and the maximum air temperature was 94°F. However, the measured change in length during that air temperature change was about 0.27 in (Figure 6), which is equivalent to an average temperature change in the bridge of about 8.3°F. That is, for a 27° change in air temperature, about one-third of that change (8.3°) resulted in change in bridge length.

MEASUREMENT OF ABUTMENT MOVEMENTS

Three independent measurements were taken on the longitudinal movements of the abutments. They were (a) the gap between the backfill and the abutment, (b) the opening of the expansion joint on the approach slabs, and (c) slope indicator measurements on the piles in the abutments. Do the three methods provide consistent readings on abutment movements? That question is answered by the data shown in Figure 12. Abutment movements from 7:00 a.m. on September 6, 1980, are plotted for each approximate 6-h interval during a 24-h period. Note that the average backfill to abutment gap and average expansion joint openings provide nearly equal displacements whereas the displacements from the average of the slope indicator readings are much larger. Two reasons are suggested as to why the slope indicator readings do not agree with the other readings. The first is that the slope indicator displacements are calculated on the assumption that the deflection of the pile at the 35-ft depth does not change. A second reason is that each displacement at the top of the pile (actually 3 ft below the deck surface) results from the difference among 18 sets of readings. If any one of those readings were in error, the displacement would be in error.

Movements Between Abutments and Soil Backfill

As Figure 4 shows, there is a partial void space between the solid side of the abutment and the backfill. This void space is held open by using pressure relief strips between the abutment and the backfill. When the abutment was poured, four steel pipes were cast in the form to provide openings through the abutment into the void space. The width of the opening of the void space is measured by placing a rod through the opening in the abutment and measuring the extension of the rod from the abutment to the corrugated steel. Four readings were taken on each abutment.

As stated earlier, there is an expansion joint in the approach slab at each end of the bridge. Measurements were made on the size of openings in the joint at the north end of the bridge. Three readings were taken, one on each side and one in the middle of the slab.

Figure 9. Change in length of bridge over one year.

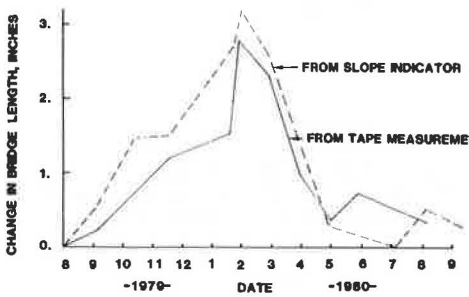


Figure 10. Air and concrete deck temperatures over one year.

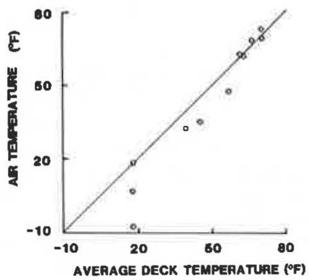
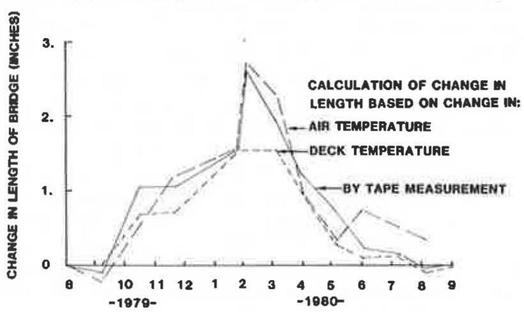


Figure 11. Change in length of bridge: temperature readings.



Vertical Movement and Displacement of Abutments, Piers, and Piles

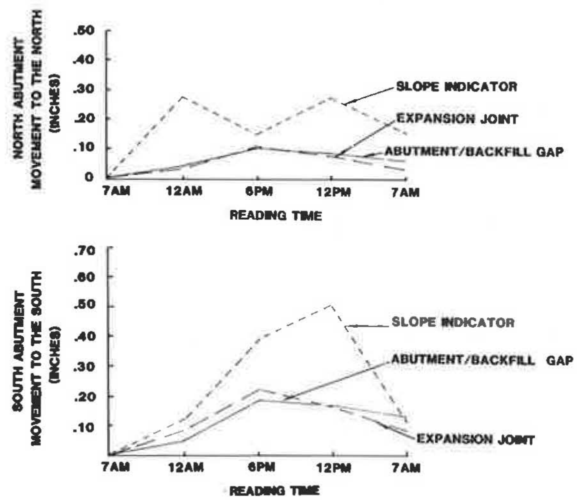
To determine any changes in the elevation of the piers and abutments, two permanent benchmarks were constructed and a level circuit was run each time bridge data were collected. Elevation changes over the year were less than 0.03 ft.

A slope indicator casing was attached to each edge pile of each abutment. Readings were taken periodically to measure the slope of the pile. These slopes were used as a measure of pile movement as well as of bending stresses in the pile.

The casing extended from the top of the concrete deck to depths of 31-35 ft and had pairs of grooves in perpendicular planes. One pair of grooves was oriented to the weak plane of the pile. The casing was placed on the piling by holding angles welded in place before pile driving. When the driving was completed, a section was added to the casing that permitted it to extend to the top surface of the concrete deck. The slope indicator casing was encased in concrete throughout the height of the abutment.

The series 200-B slope indicator instrument from the Slope Indicator Company was used to measure the slope or the casing. All readings were taken at 2-ft intervals with the instrument in the plane of weak direction of the pile, which is also the longitudinal plane of the bridge. If, at a particular point on the casing, the slope changes with time, that is an indication of pile movement. The magnitude of pile movement between any two slope readings is determined by multiplying the length of the pile between readings by the change in slope of the lower reading. If it is assumed that there is no movement at the bottom end of the pile, then the displacements at each reading point along the casing can be assumed to determine the displaced position for the casing.

Figure 12. Abutment movements over one day.



MEASUREMENT OF STRESSES IN PILES AND CONCRETE TEMPERATURE

To determine the bending stresses in the piles, electrical resistance strain gages were attached to the two edge piles on the north abutment, wired, and moisture-protected in the laboratory. After the piles were driven, the wires were placed in plastic pipe inside the concrete and brought to a junction box encased in the abutment wing wall. Stable readings were observed in the laboratory check of the gages and again in the fall of 1978 after the abutment was poured. The next spring the area was flooded to a level above all the strain gages. Following the flood, the readings for most gages would not stabilize. Due to the erratic readings, the electrical resistance strain gage data were not used.

Four thermocouples were installed in the concrete deck. They were of copper constantan material manufactured by Honeywell. A Model 199-1F digital thermometer manufactured by Omega Engineering was used to read the temperature. The thermocouples were checked for accuracy at 40°F and 70°F. The thermocouples were located midway between the edge and adjacent girders at about 25 ft from each end of the bridge.

The thermocouples were installed by first forming a void space in the slab. When the slab forms were removed, the thermocouples were inserted and the remaining space was filled with concrete.

ANALYTIC MODEL DEVELOPED TO MEASURE STRESSES IN PILES

A secondary objective of this bridge study was to develop a model to measure the stresses in the abutment piles. Pile stresses depend on the relative

Figure 13. Model for calculating pile stresses.

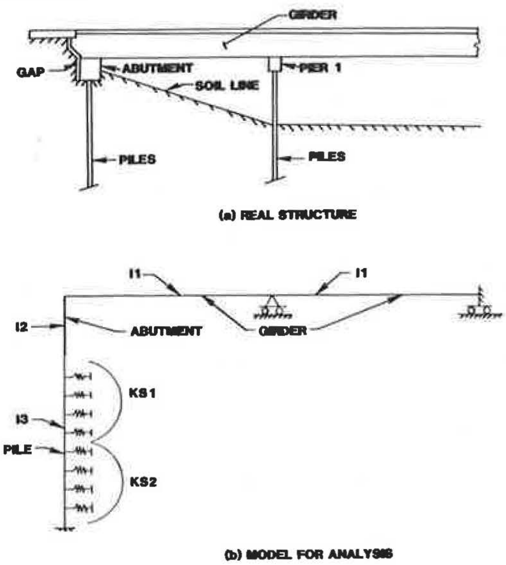
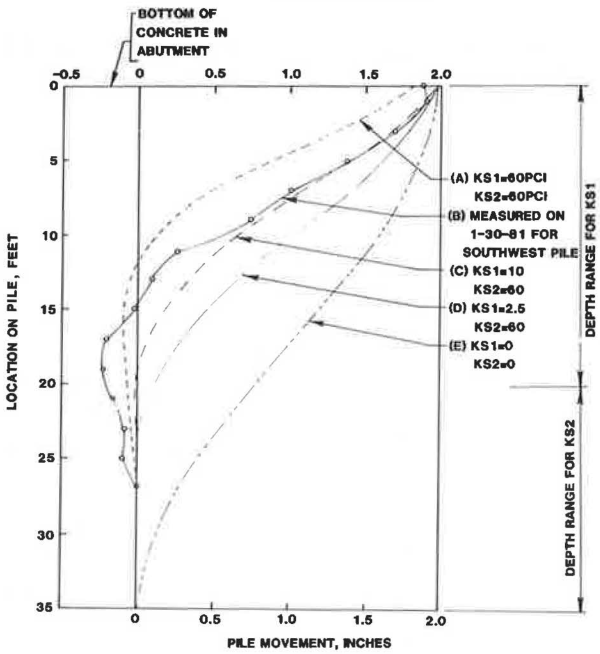


Figure 14. Measured and calculated pile movements.



stiffness of bridge, abutment, pile, and soil as well as the abutment and pile movement and method of pile installation. To analyze this situation, it is necessary to devise mathematical models that describe the behavior of each element in the problem.

The model used is shown in Figure 13. Figure 13a is a sketch of the real structure that shows the bridge girder, abutment, piles, soil line, and first pier. The gap between the abutment and the backfill soil is also shown. Figure 13b illustrates the mathematical model. The beams are on rollers in order to place a specified displacement in the abutment. I1 represents the moment of inertia of that portion of the bridge deck that reacts with a single pile, and I2 is the moment of inertia of an equivalent portion of the abutment. The moment of inertia of a single pile is represented by I3. The model is considered to follow elastic behavior.

The remaining parameter in the model is the modulus of subgrade reaction, which is represented by the symbol KS. The modulus of subgrade reaction is a measure of the load-deformation relation for soil. It is the ratio of the stress on a loaded plate divided by the magnitude of the displacement of the plate into the soil. Bowles (4) indicates that the best method of obtaining the KS for the soil is to conduct a lateral load test on the pile. In this case a load test was not conducted. Other estimates are that, for clays, KS is primarily dependent on the unconfined compressive strength of the clay. Soil tests in the region of the pile under study give an average unconfined compressive strength of 1.50 kips/ft^2. This was multiplied by 72 (from Bowles) to obtain a modulus of subgrade reaction in kips per cubic foot. After changing to pounds per cubic inch (pci) units, 60 pci was used where the pile was in contact with clay.

The KS for that portion of the pile with compressible material attached to the web depends on the load-displacement relations for the compressible material, the sand around the pile, and the clay around the sand. Load-displacement tests were run on the two 1-in-thick layers of Ray-Lite. For a displacement up to 0.17 in, the KS was 42 pci; for displacements from 0.17 in, the KS was 13 pci. Load-displacement tests were not run on the sand or clay. The KS for the combined materials will be less than what it is for any one of the materials. With that as background, two values of KS--2.5 and 10.0 pci--were used as estimates where compressible material was attached to the pile.

Pile Displacement

Earlier it was pointed out that the maximum recorded movement for the south abutment occurred on January 30, 1980. The movement of the west pile on the south abutment for that date is plotted in Figure 14 along with calculated pile movements by using the model shown in Figure 13 and different values of modulus of subgrade reaction.

The left side of Figure 14 indicates the location on the pile measured from the bottom of the concrete in the abutment. That point is about 8 ft below the roadway. Measurements of pile displacements were taken with the slope indicators to a depth of 24 ft below the bottom of the concrete. The calculation mode was extended to 35 ft below the bottom of the abutment concrete. Modulus-of-subgrade-reaction springs were placed at a 2-ft spacing along the pile. The left side of the figure indicates the range in depth over which the two modulus-of-subgrade reactions apply.

The solid line indicates the measured location of the pile on January 30, 1980. The remaining four lines are calculated pile movements based on different values of KS. All calculated movements are based on a 1.96-in movement of the top of the abutment. Movement will extend over the entire length of the pile if the soil provides very little resistance to the movement--i.e., a low modulus of subgrade reaction, as shown by curve E. In contrast, for high values of KS (60 pci in both regions), the pile movements are very small, as shown by curve A. Curves C and D are for what was considered to be reasonable values for the subgrade modulus--i.e., from 2.5 to 10 pci for the top 20 ft of pile and 60 pci for the remaining depth. Note that curve C closely matches the measured pile movement down to the 9-ft depth. Models could be developed to obtain closer agreement at lower pile depths, but this was not pursued since the pile stresses at those depths are likely to be less than those near the top of the pile.

Figure 15. Bending moment in pile.

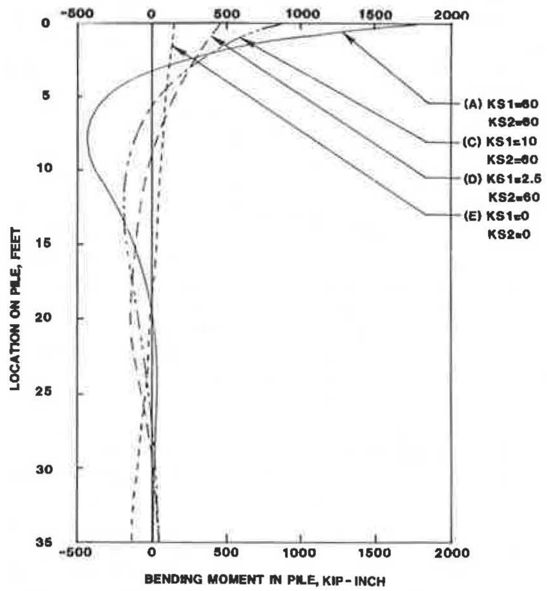
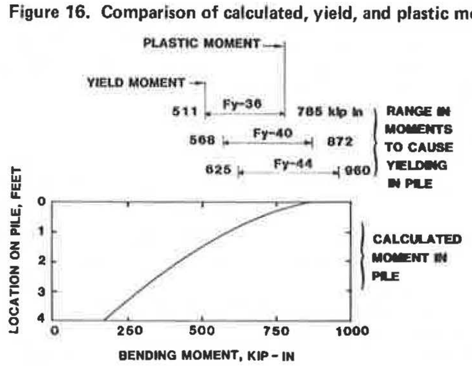


Figure 16. Comparison of calculated, yield, and plastic moments.



Pile Stresses

Pile bending moments for four sets of soil constants are shown in Figure 15. For the more firm soil (KS = 60 pci), the maximum moment is 1842 kip•in. The maximum moment at the top of the pile is controlled primarily by the modulus of subgrade reaction KS1. Holding KS1 constant and doubling KS2 will have less than a 5 percent influence on the maximum moment. At the top of the pile, the maximum moment causes tension on the inside edge of the flange. Going down the pile, the moment goes to zero and increases to cause tension on the outside edge of the flange. These maximum moments are about 23 percent of the top maximum moments and occur at 7-10 ft below the top of the pile.

Bending moments are related to bending stress by F = M/S, where F is the bending stress, M is the bending moment, and S is the elastic section modulus for the pile. HP10x42 piles were used in the abutment with the minor axis of the pile in line with the longitudinal axis of the bridge. The elastic section modulus for the pile is 14.2 pci. Based on the maximum bending moments from Figure 15, the maximum bending stresses are 9.6, 33.4, 59.3, and 129.7 ksi. Since some of these moments are beyond the yield strength for the steel, the assumed elastic behavior for the analysis model was not correct.

Bending moments for that case that most closely fits the actual pile deflections are shown in Figure 16. The lower portion of Figure 16 shows a plot of the calculated moment in the pile, and the upper portion shows a plot of the range in moments to cause yielding in the pile for different yield strengths of steel. The yield moment occurs when the outer fiber of the flange reaches the yield stress. The plastic moment occurs when the strain has been sufficient to cause yielding at middepth of the member. The guaranteed minimum yield strength (F_y) for the pile steel was 36 ksi. Very little steel would be produced at that level, and the average value would be closer to 40 or 44 ksi. If 44 ksi is used as the yield strength of the steel, then yielding took place in less than the top 1 ft of the pile, and at the top of the pile the yielding was only on each outer one-fourth depth of the flange.

Are the calculated stresses in Figure 16 correct, since the model was based on an elastic analysis and the stress near the top of the pile was beyond the elastic limit? The error is believed to be minor because the yielding was not sufficient to form a plastic hinge.

CONCLUSIONS

Based on the one year of measurements at the bridge and study of the data, the following conclusions were made:

1. The maximum change in length of the bridge due to thermal change can be estimated by using a temperature change equal to

$$DT = T1 - 72 + (T3 - T1)/3 \quad (1)$$

where

T1 = air temperature at dawn on the hottest day,
T2 = air temperature at dawn on the coldest day, and
T3 = maximum air temperature on the hottest day.

2. The above change in bridge length agrees well with changes in length determined from tape measurement and measurements of openings in expansion joints.
3. The change in bridge length did not result in equal movement at the two ends of the bridge. At the point of maximum bridge shortening, the south abutment moved in 1.96 in and the north abutment moved in 0.74 in from their initial August positions.
4. In one year, the gap between the abutment and the backfill closed about 0.5 in on the north abutment and 0.75 in on the south abutment.
5. The vertical movements of the abutments and piers were nearly zero.
6. The method of measuring pile stresses failed. However, pile stresses were calculated and resulted in pile displacements that matched those from the slope indicator measurements. For the maximum measured abutment movement of 1.96 in, the stress at the top of the pile was sufficient to initiate a yield stress in the steel but not sufficient to cause the formation of a plastic hinge.
7. An analytic model was used to predict stresses in the piles due to movements of the abutments. The two parameters that have the most influence on pile stresses are the amount of abutment movement and the modulus of subgrade reaction near the upper portion of the pile.

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Publication of this paper sponsored by Committee on Foundations of Bridges and Other Structures.

Effective Coefficient of Friction of Steel Bridge Bearings

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A study to determine experimentally the effective coefficient of friction of four classes of steel bridge bearings used by the Oklahoma Department of Transportation is reported. As-built, rusted, and in situ (debris at the moving surfaces) conditions were tested by using full-scale bearings under normal loads to 250 000 lb. In addition, the effects of manufacturing tolerances on bearing performance were analyzed. From the tests it was found that unturned pipe rollers exhibit the lowest effective coefficient of friction of the four rolling devices tested. For turned pipe rollers it was found that the equivalent coefficient of friction is a function of the amount of horizontal movement from the centerline. A geometric explanation was devised, and excellent agreement between predicted and measured results was achieved. Tests with a pintle rocker showed that fabrication inaccuracies, especially in the sole plate socket radius, can significantly affect the performance and effective coefficient of friction of the bearing. In all cases, tests with rusted bearing plates or with sand spread over the lower bearing plate showed significant increases in the effective coefficient of friction.

Expansion and contraction caused by temperature changes, deflection, relative support settlement, creep, and other factors will produce motion in a bridge. The movement is very slow, but the forces involved can be tremendous and usually are accommodated by bearings at piers or abutments. If the bridge does not have the ability to move, because either it does not have a bearing or the bearing is not working, it pushes and tears at its supports until it achieves the ability to move.

Even if the bearing is working properly, horizontal force is transmitted to the pier or abutment through friction caused by relative motion of the bearing parts or by eccentric loading of the bearing as found in certain "pipe" bearings. This force must be accommodated in the design of the supporting structure; if not, structural damage can occur.

The purpose of this study was to determine experimentally the effective coefficient of friction of several classes of bridge bearings used by the Oklahoma Department of Transportation (ODOT). Both as-built conditions and simulated conditions, as found after several years of use, were used in the testing program. A thorough literature search revealed that very few studies of the behavior of complete bearing assemblies have been conducted and that specification provisions have been based on classic values of coefficients of friction between sliding parts without regard to effects of manufacturing tolerances or environmental effects. This study is an attempt to assess these effects and to provide guidelines to establish accurate estimates of horizontal force requirements for the class of bearings tested.

For the purpose of this study, the effective coefficient of friction (μ_{eff}) is defined as

$$\mu_{eff} = F/N \qquad (1)$$

where F is the horizontal force to overcome the resistance to allow motion and N is the normal force applied to the bearing. The value of F was determined experimentally for the entire assembly for an applied normal force N, from which μ_{eff} is calculated.

BACKGROUND

Many types of bearing devices are used to accommodate bridge movement: single rollers, groups of rollers, rockers, elastomeric pads, sliding plates, sliding tetrafluorethylene (TFE), etc. In general, bridge bearings can be classed in two categories: elastomeric and mechanical (1). According to a recent National Cooperative Highway Research Program synthesis on the design, fabrication, construction, and maintenance of bridge bearings (2), the elastomeric bearing pad is perhaps the best expansion bearing because it is unaffected by weather (e.g., it has no moving parts to freeze), has nothing to corrode, is low in cost, and requires almost no maintenance. However, elastomeric bearing pads are limited to 700 psi for vertical load capacity and 3 in for horizontal movement and their success depends on the quality of the material. On the other hand, for mechanical bearings the movements and rotations are accommodated by rolling, rocking, or sliding actions, usually on metal parts that can accommodate much larger bearing pressures. Furthermore, mechanical bearing devices can be designed for virtually unlimited horizontal motion (2).

One of the simplest types of mechanical bearing is the roller or "pipe roller", simply a piece of steel pipe with a stiffener as shown in Figure 1a. The load-carrying capacity of the roller is a function of its radius and can be found from the following formula (3): For diameters up to 25 in,

$$P = [(F_y - 13\,000)/20\,000]\ 600\ d \qquad (2)$$

and for diameters from 25 to 125 in,

$$P = [(F_y - 13\,000)/20\,000]\ 3000\ \sqrt{d} \qquad (3)$$

where

P = allowable bearing (lb/linear in),
d = outside diameter of the roller (in), and
F_y = minimum yield point in tension of the steel in the roller or bearing plate, whichever is the smaller (psi).