Report on Test Pile Program Conducted by Kansas and Missouri State Highway Departments

J.A. WILLIAMS, Bridge Engineer, Missouri State Highway Department

This report presents the primary reasons for development of a test pile program conducted by the Kansas and Missouri State Highway Departments, the procedures followed, and the results obtained in the driving and loading of two groups of four different types of piles. The U.S. Bureau of Public Roads and the two states involved agreed that because of the magnitude of pile requirements for the proposed structure, preliminary studies and investigations to predetermine comparative pile lengths and capacities would be worthwhile. It was hoped to obtain data that would justify the use of design loads for piles in excess of those permitted by the AASHO specifications, and thereby realize an appreciable economic saving in construction costs.

THE PLANNING and construction of structures in urban areas present many comlicated and costly problems. The Kansas and Missouri State Highway Departments ere confronted with such a problem in providing additional lanes for the existing tructure which connects Kansas City, Kans., and Kansas City, Mo. The existing tructure at this location consists of a truss over the Kansas River and some 8,000 ft I-beam spans averaging approximately 45 ft in length. This structure has four 10-ft affic lanes with a 6 in. wide median dividing the east-and west-bound traffic.

Interstate Route 70 has been routed over this structure and in order to meet Interate standards, it was necessary to widen the existing structure or build another ructure parallel to it. The anticipated traffic requires three lanes in each direction. ter a comparative cost estimate was made, the two states agreed that a separate ree-lane structure should be constructed parallel, but not tied to the existing structure. e existing structure would carry three lanes of traffic.

The central portion of the proposed structure consists of approximately 3,715 linear t of which 1,359 ft is in Kansas and 2,356 ft in Missouri. A joint agreement was tered into by the two states and a consulting firm was employed to design and prepare detail plans. At the time the preliminary plans for the proposed structure were ing discussed, structural steel was still scarce and it was agreed to construct a tinuously-reinforced concrete tee-girder structure with the spans matching the n arrangement of the existing structure. Because of the necessity for using short ns, the substructure costs for the proposed structure took on additional importance.

The central portion of the Intercity Viaduct, for which plans were being developed, located in the flood valleys of the Kansas River and the Missouri River, and not far m the junction of these two rivers. Preliminary sounding information brought out fact that at the Kansas end of the structure, bedrock was approximately 85 ft below und elevation and at the Missouri end it was approximately 65 ft to bedrock. The ings showed that the soil classification was fairly uniform for the entire structure,

the top 10 or 15 ft consisting of fine sand, the next 25 ft consisting of silty sand and sandy silt, and the balance consisting of fine to coarse sand and gravel. The bedrock was shale and limestone.

On the basis of this preliminary sounding data, it was evident that the substructure units should be placed on piling. The question then arose as to whether friction piling or point bearing piling would be the more economical. In accordance with AASHO specifications, the maximum assumed design load for a 14-in. cast-in-place concrete friction pile, without the benefit of extensive subsurface investigations or test loads, would be approximately 30 tons. With the benefit of subsurface investigations and past practices in the area, but without test loads, this maximum could conceivably be increased to 45 or 50 tons but normally not in excess thereof. AASHO specifications also limit the maximum design load for steel point bearing piles to 6,000 psi unless test loads indicate that larger loads can safely be sustained.

In view of the large number of substructure units involved in the project and the subsurface conditions as noted in the explorations, it was thought that a test pile program would be of considerable value. A thorough study was made of the proposed test pile program by the two state highway departments, the U.S. Bureau of Public Roads, and the consulting engineer's office. It was then agreed by everyone involved that a test pile program would be beneficial in determining the possible use of pile design loads in excess of those permitted by AASHO specifications and thus reducing the over-all foundation costs.

After a definite decision had been reached by all organizations involved, the consultant was requested to proceed with a test pile program and to investigate subsurface soil conditions in the vicinity of the pile driving site, and to determine the relative driving effort, pile length requirements, and bearing capacities for four types of bearing piles. The program was set up to obtain pertinent information not only for the Intercity Viaduct, but also to obtain additional data on friction piles which at some future time could be added to HRB Special Report 36, covering a study of the comparative behavior of friction piles (1958).

The consultant was authorized to set up a project, prepare specifications, and receive competitive bids for a program to consist of furnishing, driving, and test loadin four types of piles and for a split-barrel boring centered in the pile groups at each of two locations. One location was in Kansas and the other in Missouri. The program as originally proposed was to consist of one pile of each type at each of the two locatio but to satisfy the various pile manufacturers, three additional piles were added.

Location A, which was in Missouri, called for seven piles. The piles consisted of (1) $14\frac{1}{8}$-in. Union Metal Monotube Tapered Steel Pile, (1A) $12\frac{1}{4}$-in. Union Metal Mon tube Tapered Steel Pile, (2) $14\frac{3}{8}$-in. Raymond Step Tapered Pile, (3) $12\frac{3}{4}$-in. Armco Foundation Pipe Pile, (3A) 14-in. Armco Foundation Pipe Pile, (4) 12-in. BP 53 Stee H-Pile, and (4A) 12-in. BP 53 Steel H-Pile Driven to refusal. Piles (1), (1A), (2), (3), (3A), and (4) were driven to a depth of 55 ft below ground level and test loaded. Pile (4A) was driven to bedrock and test loaded.

Location B, which was in Kansas, called for six piles. The piles consisted of (5) $16\frac{3}{8}$-in. Raymond Step Tapered Pile, (6) $16\frac{1}{4}$-in. Union Metal Monotube Tapered Ste Pile, (7) 14-in. Armco Foundation Pipe Pile, (7A) 16-in. Armco Foundation Pipe Pil (8) 14 BP 73 Steel H-Pile, and (8A) 14 BP 73 Steel H-Pile driven to refusal. Piles (5), (6), (7), (7A), and (8) were driven to a depth of 54 ft 9 in. below ground level and test loaded. Pile (8A) was driven to refusal and test loaded.

Test piles fabricated by the Union Metal Manufacturing Company, Raymond Concr Pile Company, and Armco Drainage and Metal Products, Inc., were in accordance with manufacturer's standards and met the dimensions listed. The piles were locate and driven so that they could be used in the foundation of the permanent structure. The piles were all driven with a Modified Vulcan No. 1 Steam Hammer with a 6,500- ram and a rated energy of 19,500 ft-lb. The test piles were driven in accordance wi each respective pile manufacturer's recommended procedure, and each pile was dri to the specified depth without interruption.

After driving all piles at the two locations, the shells for the cast-in-place piles were inspected for shell condition and watertightness and then filled with concrete.

The seven-day compressive strength of the high-early strength concrete used in the piles was determined by testing companion 6- by 12-in. cylinder specimens and found to vary from 4,315 to 4,855 psi.

The contractor elected to apply the test loads concentrically to each test pile by using a 500-ton capacity hydraulic jack. Wide flange beams were secured to anchor piles to serve as a reaction for the jacking operations. Settlement of the pile was measured by two Ames dials reading to 0.001-in. located 180 deg apart, referenced to a fixed beam across the test site. Settlement readings were also made with a wire line arrangement customarily used by contractors.

The following procedure was followed by the contractor in handling the test load applications. The load was applied concentrically not earlier than 48 hr after the driving of the pile to be tested and anchor pile had been completed, and in no case were the cast-in-place piles loaded until the concrete had attained a compressive strength of 3,000 psi. The load was maintained at all times during the test by constant attention to load gage readings and jacking applications.

The load was applied in increments. The first application was approximately 50 tons per pile at Location A and 65 tons per pile at Location B. An increment load of approximately 25 tons per pile was applied not earlier than 1 hr after all measurable settlement of the initial loading had ceased. The least settlement considered measurable was 0.012 in. Additional 25-ton increments of load per pile were added after the measurable settlement of the previous load had ceased. The waiting period after measurable settlement had ceased was increased 1 hr for each additional load increment.

The loading for the first set of tests consisted of two 25-ton increments applied to each pile at Location A and three to each pile at Location B, in addition to the initial 50-ton and 65-ton applications. This made a total load of 100 tons at Location A and 140 tons at Location B. Yield point was reached at Location A by one pile before the 100-ton load was applied. Yield point was reached at Location B by four piles before the 140-ton load was applied. Yield point is defined as where the rate of gross settlement exceeds 0.03- in. per ton for the last increment of load applied.

All friction piles at Location A and B were unloaded after completion of the first test, or after failure, and the recovery noted. The load was removed in increments equal in magnitude to those applied with a 15-min interval between each increment. Immediately after the load was removed, the net recovery was observed and recorded. The recording of the recovery was continued for 3 hr after the load was removed.

The friction piles at Locations A and B that had not failed during the first test were given a second load test. The load was applied in increments of 25 tons. The load increments were applied at 15-min intervals until the magnitude of the previous load had been applied and thereafter at 2-hr intervals until the yield point was reached. After the second test, the piles were again unloaded and the recovery noted.

Pile (4A) at Location A and Pile (8A) at Location B, were the steel H-piles which were driven to refusal on rock. Pile (4A), the 12 BP 53 steel H-pile, was test loaded with an initial load of 50 tons and additional load increments of 25 tons each were applied at 2-hr intervals until the maximum load of 200 tons was applied. The maximum load was maintained for a period of 6 hr before the load was removed and recovery noted. Pile (8A), the 14 BP 73 steel H-pile, was test loaded with an initial load of 75 tons and additional load increments of 25 tons each were applied at 2-hr intervals until the maximum load of 300 tons was applied. The maximum load of 300 tons was maintained for 6 hr before the load was removed and recovery noted.

Borings were made at each of the test pile locations and extended from ground level to bedrock. Borings were made in accordance with the Proposed Method for Penetration Tests and Split-Barrel Sampling of Soil as published in "Procedures for Testing Soils," ASTM Committee D-18 (April 1, 1958). As previously stated, the soil classifications at Location A and Location B were quite similar and consisted of various gradations of sand with mixtures of silt and clay for the first 30 to 40 ft and coarse sand and gravel from there to bedrock. The ground water level was 28 ft below ground surface at Location A and 30 ft below ground surface at Location B.

It is not possible to give a complete summary of the driving record of each pile,

but a summary is given of the load in ton at yield point for each pile as well as the capacity of the pile as computed by the following formula

$$P = \frac{2WH}{2000\left(S + 0.1\frac{w}{W}\right)} = \frac{19.5}{\frac{12}{B} + \frac{0.1w}{W}}$$

in which

P = capacity in tons;
H = height of fall of the hammer in feet;
S = average penetration in inches per blow;
B = blows per foot of penetration;
w = weight of pile, mandrel and cap in pounds; and
W = weight of striking part of hammer in pounds.

Pile No. 1—14⅛-in. Union Metal Monotube Tapered Steel Pile. Yield point load 115 tons. Formula capacity 48.5 tons. Factor of safety 2.37 (Fig. 1).

Pile No. 1A—12¼-in. Union Metal Monotube Tapered Steel Pile. Yield point load 103 tons. Formula capacity 48.9 tons. Factor of safety 2.11 (Fig. 2).

Pile No. 2—14⅜-in. Raymond Step Tapered Pile. Yield point load 145 tons. Formula capacity 26.0 tons. Factor of safety 5.57 (Fig. 3).

Pile No. 3—12¾-in. Armco Pipe Pile. Yield point load 85 tons. Formula capacity 33.6 tons. Factor of safety 2.53 (Fig. 4).

Pile No. 3A—14-in. Armco Pipe Pile. Yield point load 105 tons. Formula capacity 40.1 tons. Factor of safety 2.62 (Fig. 5).

Pile No. 4—12 BP 53 Steel H-Pile in Friction. Yield point load 110 tons. Formula capacity 26.7 tons. Factor of safety 4.12 (Fig. 6).

Pile No. 4A—12 BP 53 point bearing and driven to refusal at more than 200 tons (Fig. 7).

Pile No. 5—16⅜-in. Raymond Step Tapered Pile. Yield point load 165 tons. Formula capacity 39.8 tons. Factor of safety 4.14 (Fig. 8).

Pile No. 6—16¼-in. Union Metal Monotube Tapered Steel Pile. Yield point load 124 tons. Formula capacity 78 tons. Factor of safety 1.59 (Fig. 9).

Pile No. 7—14-in. Armco Pipe Pile. Yield point load 110 tons. Formula capacity 52.2 tons. Factor of safety 2.11 (Fig. 10).

Pile No. 7A—16-in. Armco Pipe Pile. Yield point load 123 tons. Formula capacit[y] 63.2 tons. Factor of safety 1.95 (Fig. 11).

Pile No. 8—14 BP 73 Steel H-Pile in Friction. Yield point load 116 tons. Formul[a] capacity 43.7 tons. Factor of safety 2.65 (Fig. 12).

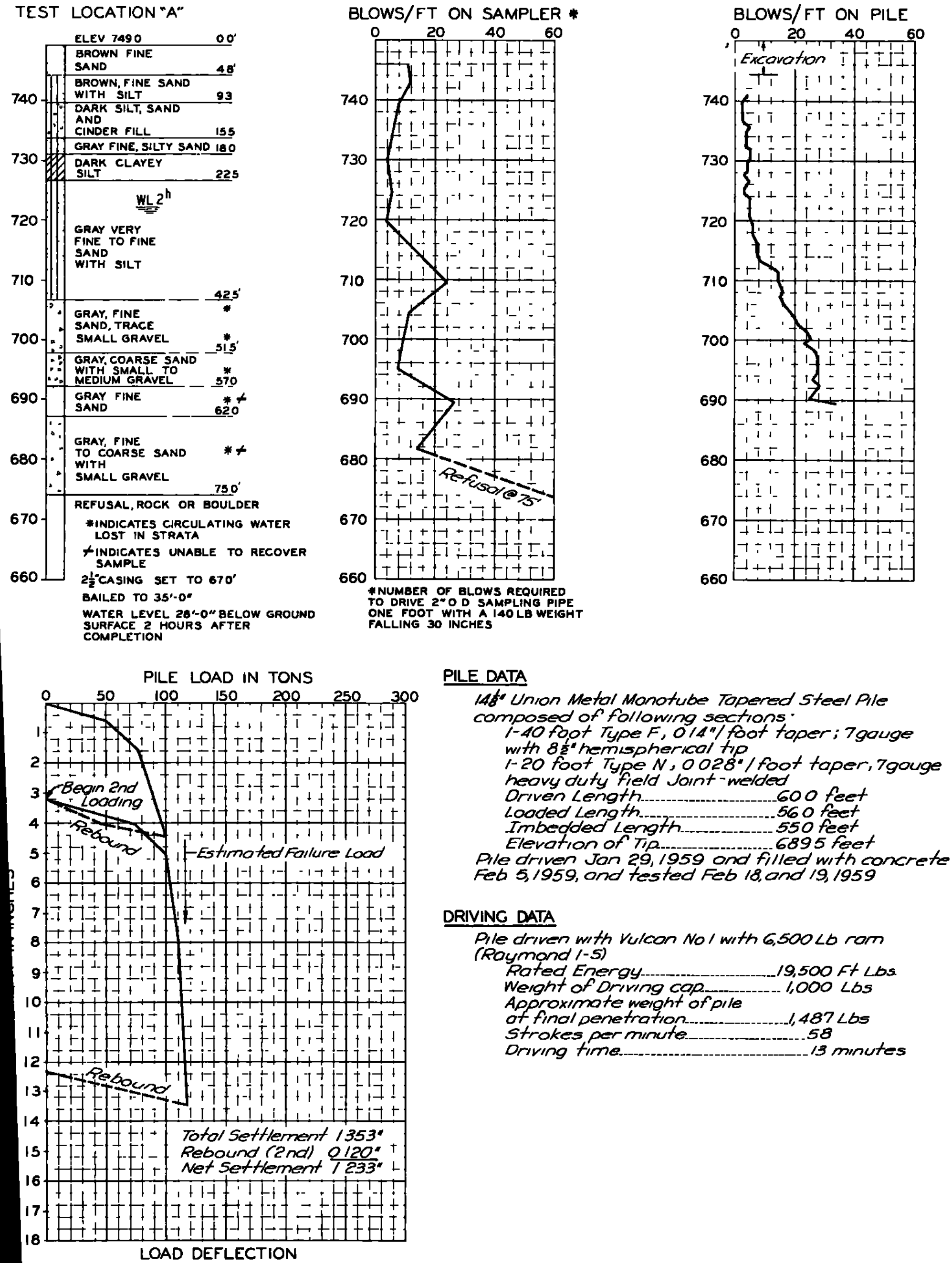
Pile No. 8A—14 BP 73 Steel Pile point bearing driven to refusal at more than 300 tons (Fig. 13).

To complete an economic comparison of foundation types, it was necessary to determine appropriate lengths of friction piles required for a selected range of design loads. This range was arbitrarily selected to vary from 30 to 70 tons per pile in 10-ton increments. The length of pile for a given design load had to be determined in so[me] manner from the results of the load tests.

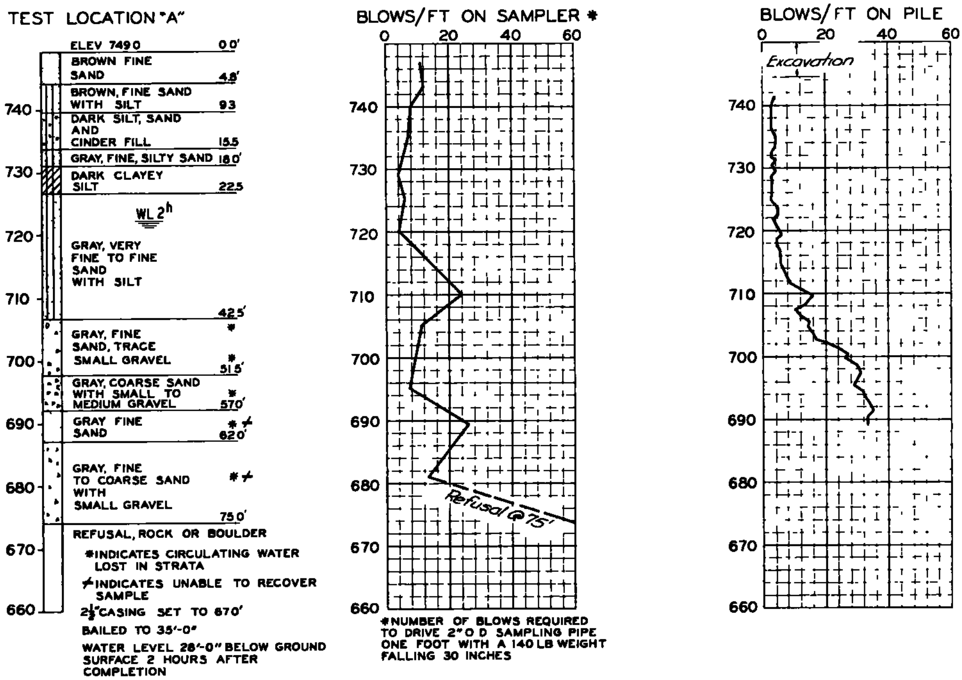
One approach which was considered was to assign friction values to the subsurface soils based on available boring and test data, thus estimating the depth of penetration required to provide the desired design load plus a factor of safety.

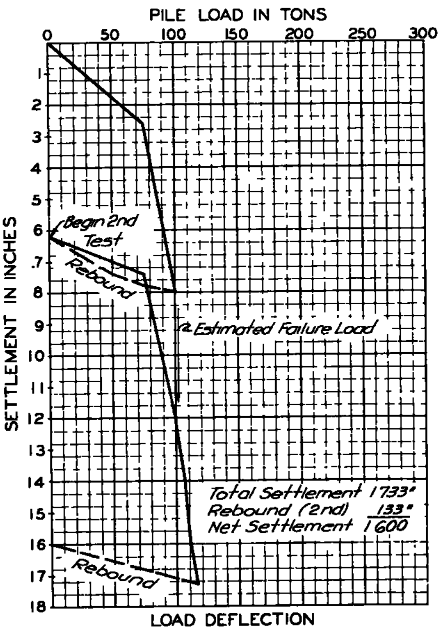
The method actually used was based on the Danish Formula wherein the ultimate test loads, plastic set values and driving records obtained during the program were incorporated in the pile length determinations. In arriving at this formula, the resistance of the embedded surface of the pile is disregarded and it is assumed that the point resistance varies with the downward increment of the pile point. The materials of the pile and the driving cushion are assumed to be perfectly elastic. Inertia force[s] in the soil and energy losses due to irreversible derformations, except of the soil, are disregarded. This formula may be written

$$aWH = \tfrac{1}{2}\,QSo + QS$$



gure 1. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location A, Pile No. 1).





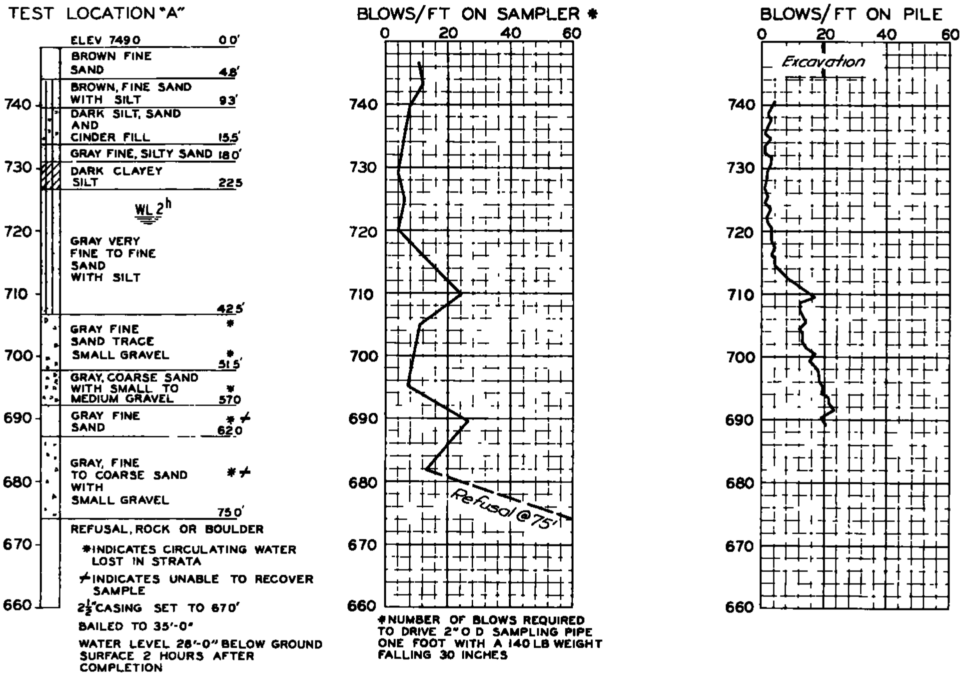
PILE DATA

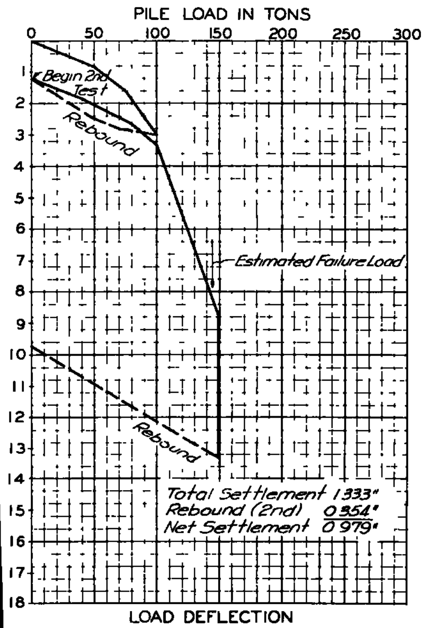
12¾" Union Metal Monotube Tapered Steel Pile - composed of following sections
1-30 foot Type F, 0.14"/foot taper, 7 gauge, hemispherical tip
1-30 foot Type N, 0.028"/foot taper, 7 gauge
Driven length .... 60.0 feet
Loaded length .... 56.2 feet
Imbedded length .... 55.0 feet
Elevation of tip 689.3
Pile driven Jan 30, 1959 and filled with concrete Feb 5, 1959, and test loaded Feb 16 and 17, 1959

DRIVING DATA

Pile driven with Vulcan No 1 with 6500 Lb ram (Raymond 1-S)
Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile at final penetration .... 1,383 Lbs
Strokes per minute 58
Driving time .... 12 minutes

Figure 2. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Tes[t] Pile Program, Location A, Pile No. 1A).





PILE DATA

14⅜" Raymond Step Tapered Pile - composed of following sections -

1-8 foot Raymond designation #000 (8⅛" tip diameter, 14 gauge) with welded flat steel plate to close bottom
1-8 foot Raymond designation #00 (14 gauge)
1-8 foot Raymond designation #0 (14 gauge)
1-8 foot Raymond designation #1 (16 gauge)
1-8 foot Raymond designation #2 (16 gauge)
1-8 foot Raymond designation #3 (16 gauge)
1-8 foot Raymond designation #4 (18 gauge)
Driven length .... 56.0 feet
Loaded length .... 56.2 feet (Built up)
Imbedded length .... 55.2 feet
Elevation of tip 689.3

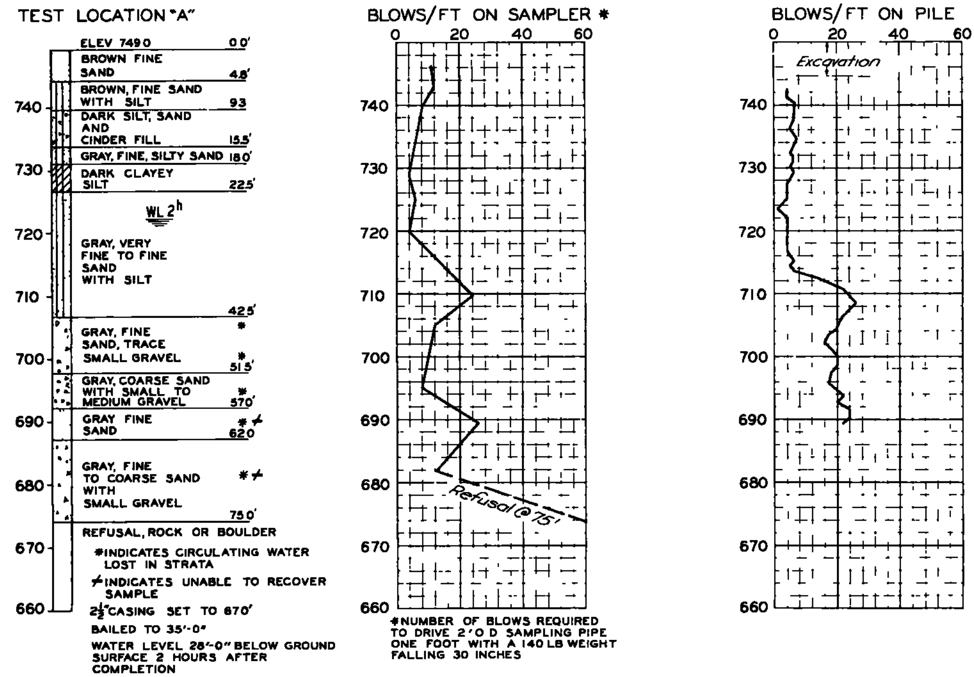
Pile driven Jan 30, 1959 and filled with concrete Feb 5, 1959, and test loaded, Feb 19 and 20, 1959.

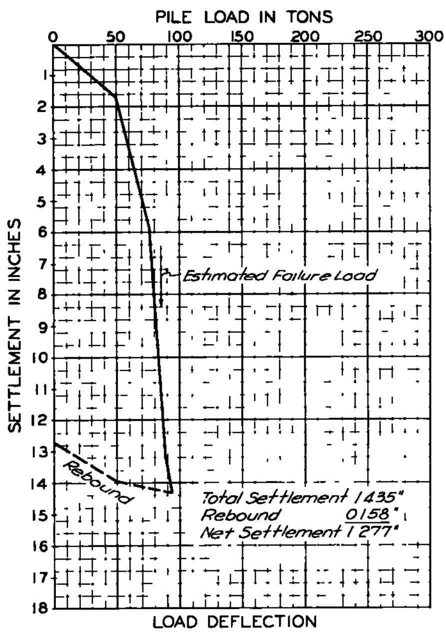
DRIVING DATA

Pile driven with Vulcan No 1 with 6500 lb ram (Raymond 1-S)

Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile and mandrel at final penetration .... 8,564 Lbs
Strokes per minute 58
Driving time .... 10 minutes

Figure 3. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location A, Pile No. 2).





PILE DATA

12¾" Armco Foundation Pipe Pile, spiral welded 60 foot length, ¼" wall thickness End closed with flat steel plate

Driven length .... 60 feet
Loaded length .... 55.6 feet
Imbedded length .... 55.0 feet
Elevation of tip 689.9

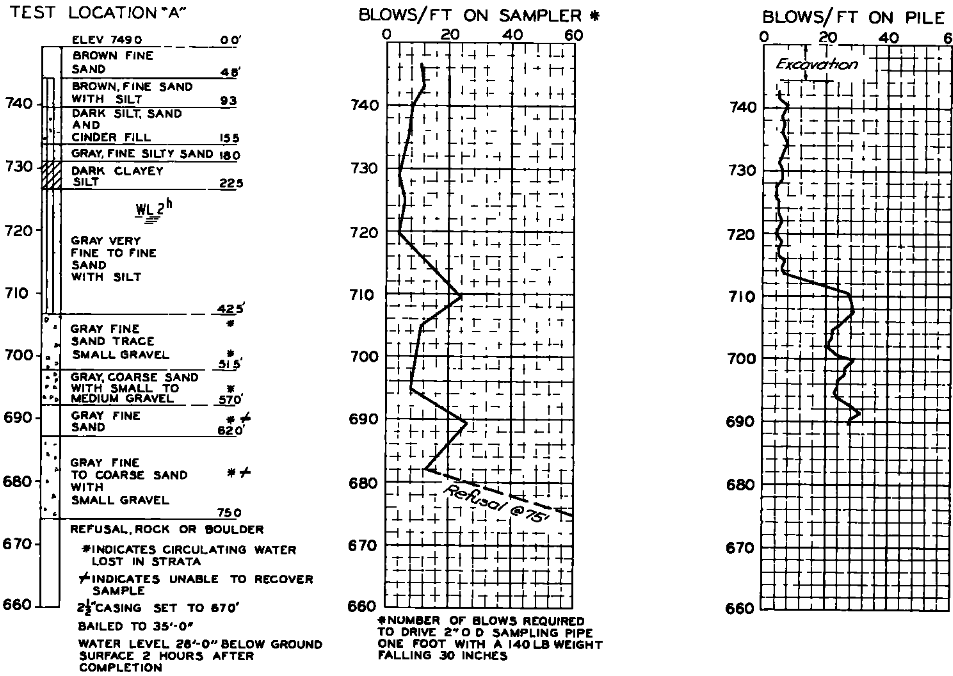
Pile driven Jan 30, 1959 and filled with concrete Feb 5, 1959, and test loaded on Feb 12 and 13, 1959

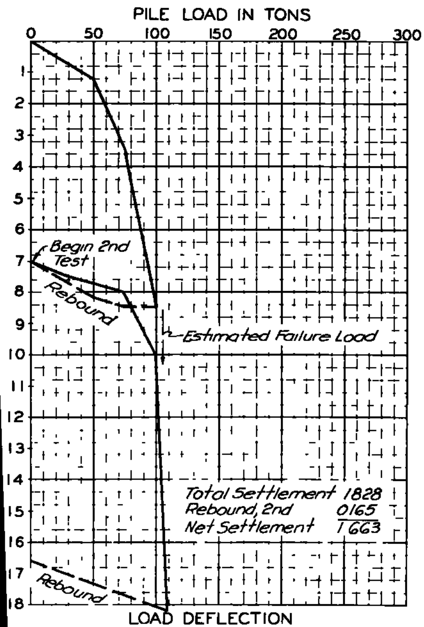
DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-S)

Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile at final penetration .... 1,510 Lbs
Strokes per minute 58
Driving time .... 11 minutes

Figure 4. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location A, Pile No. 3).





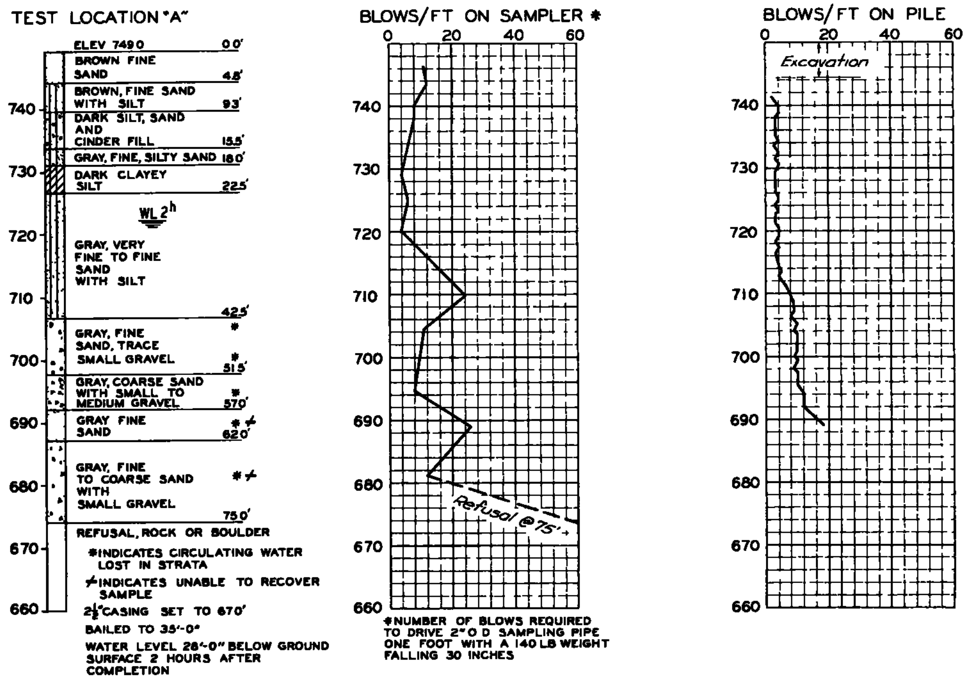
PILE DATA

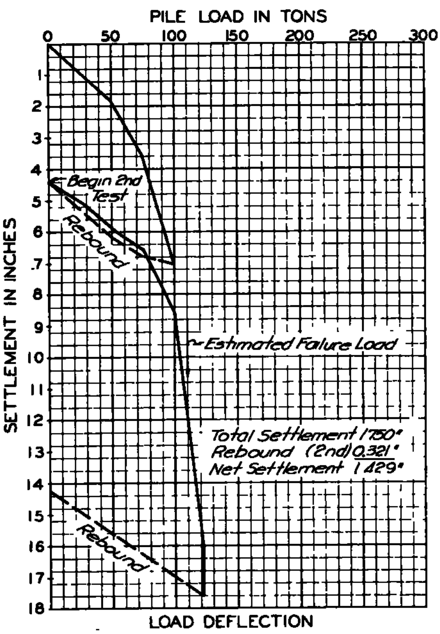
14" Armco Foundation Pipe Pile, spiral welded 60' length, ¼" wall thickness End closed with flat steel plate
Driven length .... 60 feet
Loaded length .... 55.3 feet
Imbedded length .... 55.0 feet
Elevation of tip 690.2
Pile driven Jan 30, 1959 and filled with concrete Feb 5, 1959, and test loaded on Feb 16 and 17, 1959

DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-5)
Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile at final penetration .... 1,660 Lbs
Strokes per minute 58
Driving time .... 16 minutes

Figure 5. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location A, Pile No. 3A).





PILE DATA

12" BP 53 Steel H Pile 56 foot length, steel meeting requirements ASTM Desig A-7

Driven length	*56 feet*
Loaded length	*55.7 feet*
Imbedded length	*55.0 feet*
Elevation of tip	*689.8*

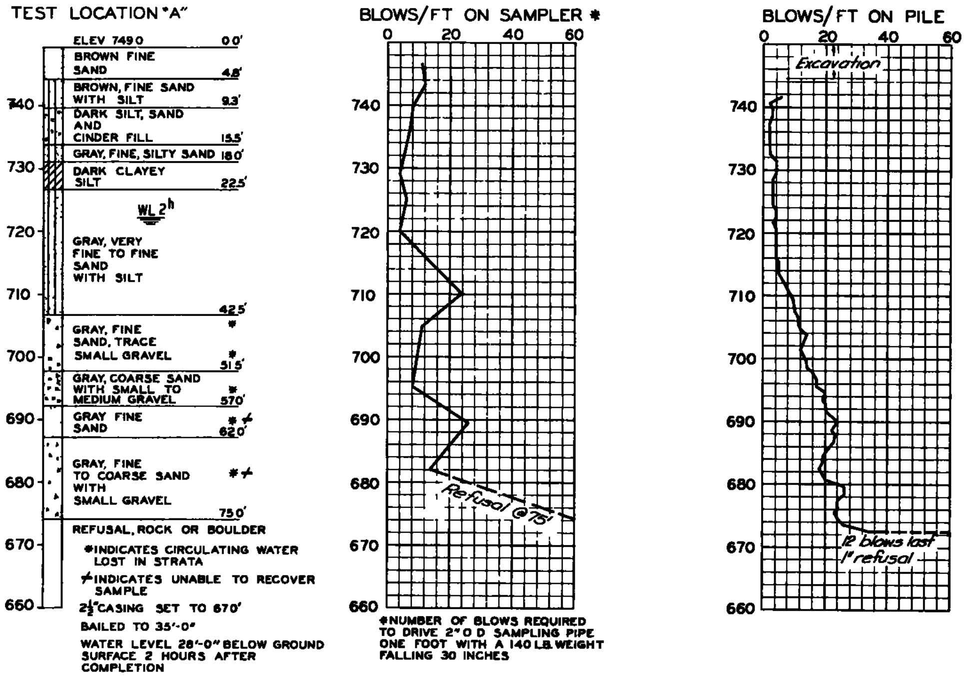
Pile driven Feb 2, 1959, and tested on Feb 13 and 14, 1959

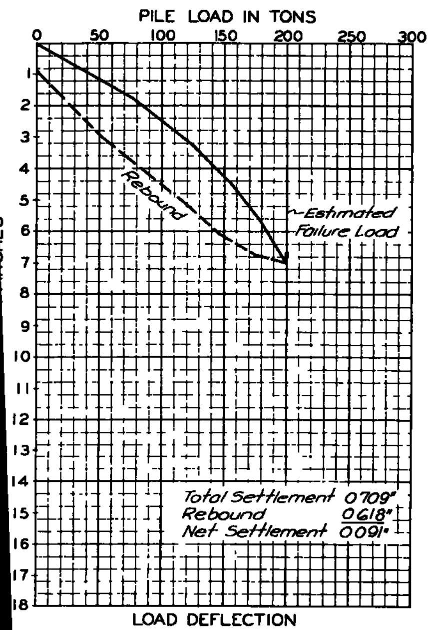
DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-S)

Rated Energy	*19,500 Ft Lbs*
Weight of driving cap	*1,000 Lbs*
Approximate weight of pile at final penetration	*2,968 Lbs*
Strokes per minute	*58*
Driving time	*8 minutes*

Figure 6. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Te Pile Program, Location A, Pile No. 4).





PILE DATA

12" BP 53 Steel H Pile 75 foot length, steel meeting requirements ASTM Desig A-7

Driven length .... 75.0 feet
Loaded length .... 72.3 feet
Imbedded length .... 71.3 feet
Elevation of tip 672.3

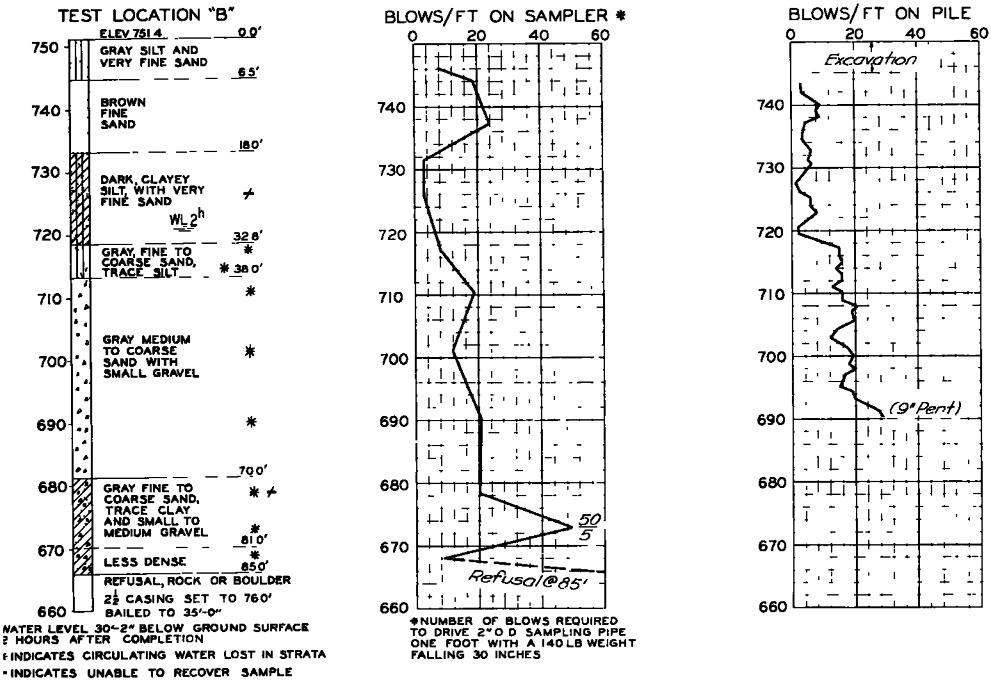
Pile driven Feb 2, 1959 and tested on Feb 20, 21 and 22, 1959

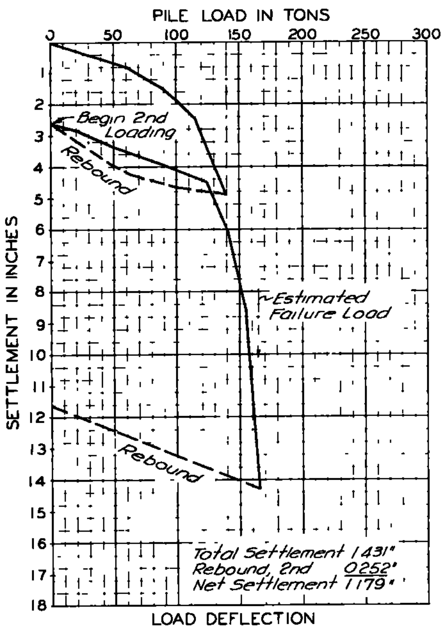
DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-S)

Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile at final penetration .... 3,975 Lbs
Strokes per minute 58
Driving time .... 17 minutes

ure 7. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location A, Pile No. 4A).





PILE DATA

16$\frac{3}{8}$" Raymond Step Tapered Pile - composed of following sections

1-8 foot Raymond designation #0,* 14 gauge
1-8 foot Raymond designation #1, 14 gauge
1-8 foot Raymond designation #2, 14 gauge
1-8 foot Raymond designation #3, 16 gauge
1-8 foot Raymond designation #4, 16 gauge
1-8 foot Raymond designation #5, 16 gauge
1-8 foot Raymond designation #6, 18 gauge
Driven length .... 56.0 feet
Loaded length .... 56.0 feet
Imbedded length .... 54.8 feet
Elevation of tip .... 690.7 MSL

Pile driven Feb 11, 1959, filled with concrete Feb 13, 1959 and test-loaded on Feb 25 and 26, 1959

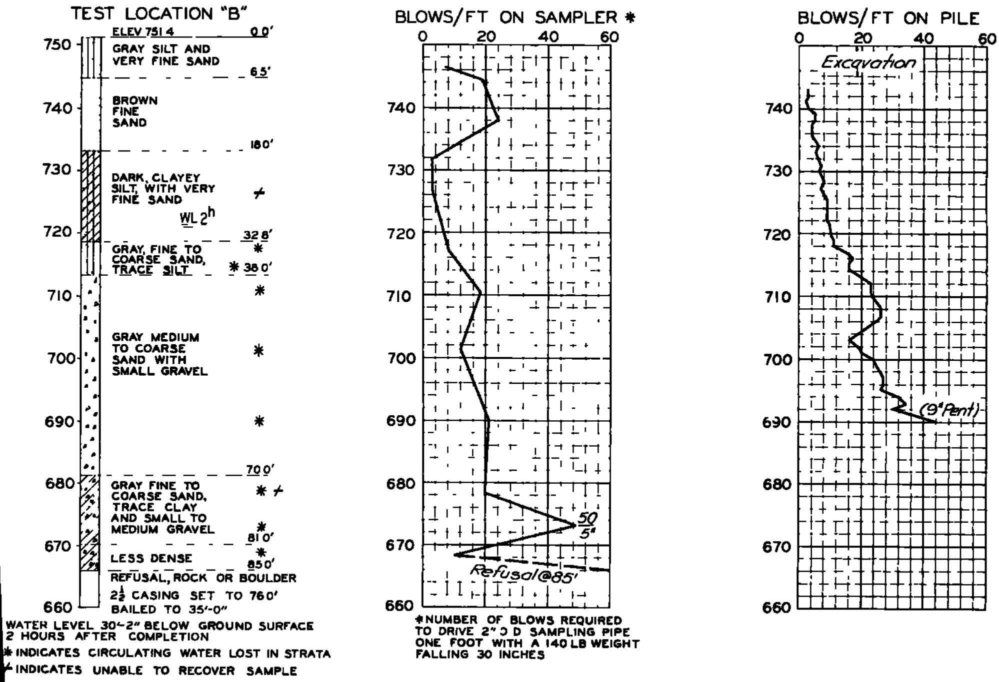
* 11¼" tip diameter with welded flat steel plate to close bottom

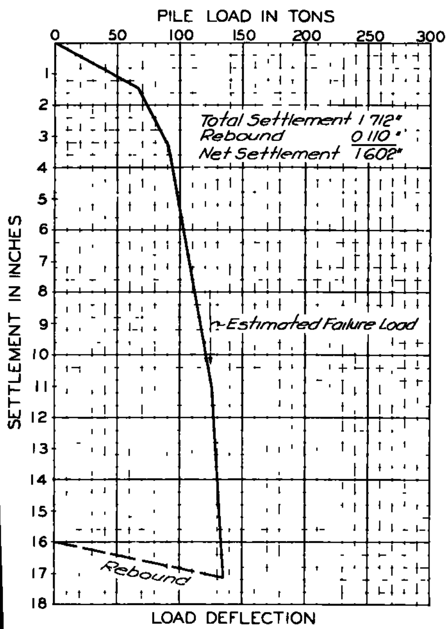
DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-5)

Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile and mandrel at final penetration .... 10,048 Lbs
Strokes per minute 58
Driving time .... 13 minutes

Figure 8. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location B, Pile No. 5).





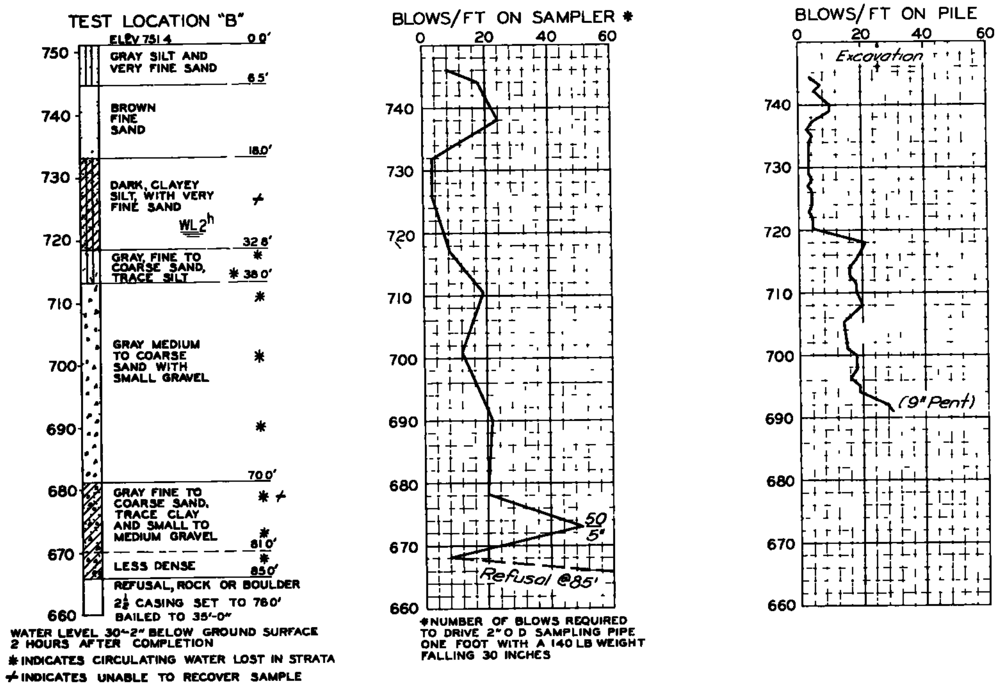
PILE DATA

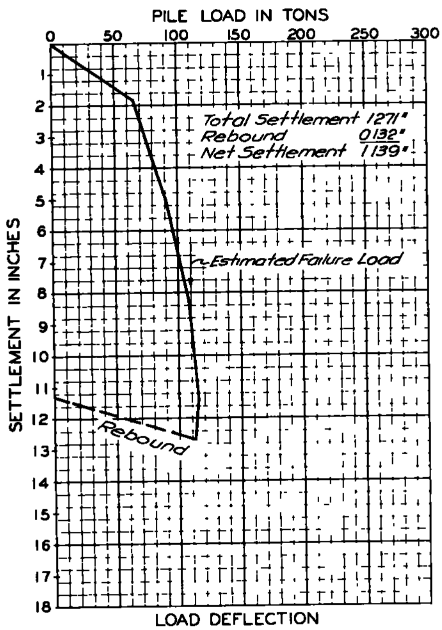
16¾" Union Metal Monotube Tapered Steel Pile-composed of following sections
1-33 foot Type J, 0 25"/foot taper, 7 gauge with hemispherical tip
1-25 foot Type N, 0 28"/foot taper, 7 gauge heavy duty field Joint, welded
Driven length .... 58 0 feet
Loaded length .... 56 4 feet
Imbedded length .... 54 9 feet
Elevation of tip 690 6
Pile driven Feb 11, 1959 and filled with concrete Feb 13, 1959, and tested on Feb 27 and 28, 1959

DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-5)
Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile at final penetration .... 1,670 Lbs
Strokes per minute 58
Driving time .... 15 minutes

gure 9. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location B, Pile No. 6).





PILE DATA

14" Armco Foundation Pipe Pile, spiral welded 60 foot length, 5/16" wall thickness End closed with flat steel plate

Driven length .... 60.0 feet
Loaded length .... 56.5 feet
Imbedded length .... 55.0 feet
Elevation of tip 690.5

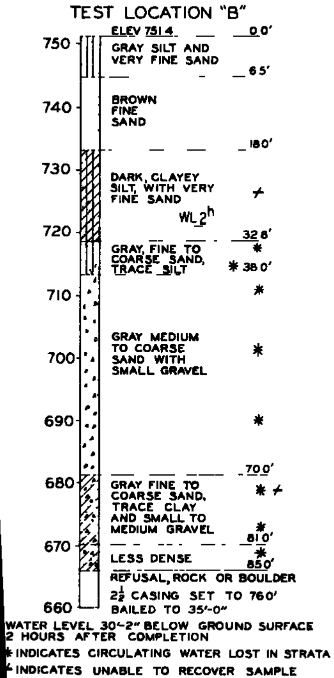
Pile driven Feb 11, 1959 and filled with concrete Feb 13, 1959, and tested on Feb 25, 1959

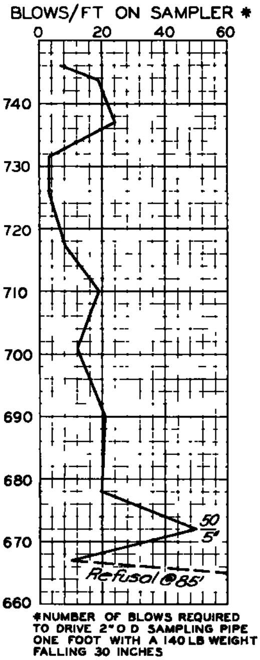
DRIVING DATA

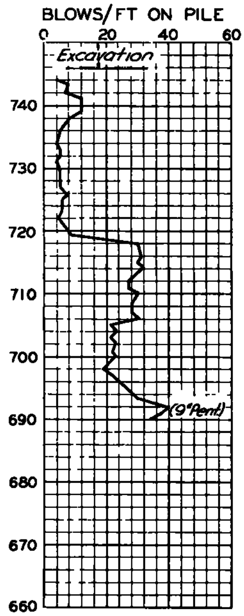
Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-S)

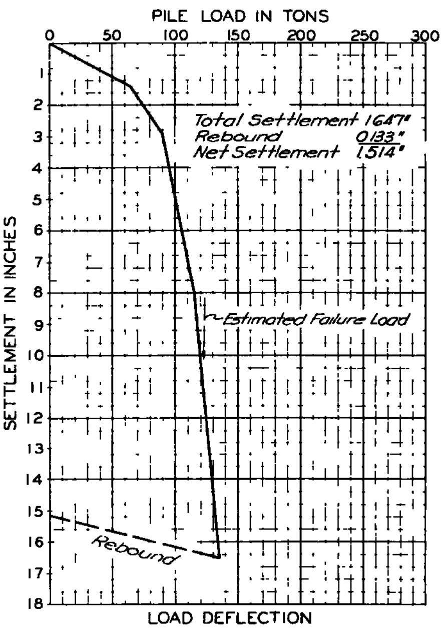
Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile at final penetration .... 1,660 Lbs
Strokes per minute 58
Driving time .... 12 minutes

Figure 10. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Te[st] Pile Program, Location B, Pile No. 7).









PILE DATA

16" Armco Foundation Pipe Pile - spiral welded 60 foot length 5/16" wall thickness End closed with a flat steel plate

Driven length	*60.0 feet*
Loaded length	*56.3 feet*
Imbedded length	*54.8 feet*
Elevation of tip	*690.7*

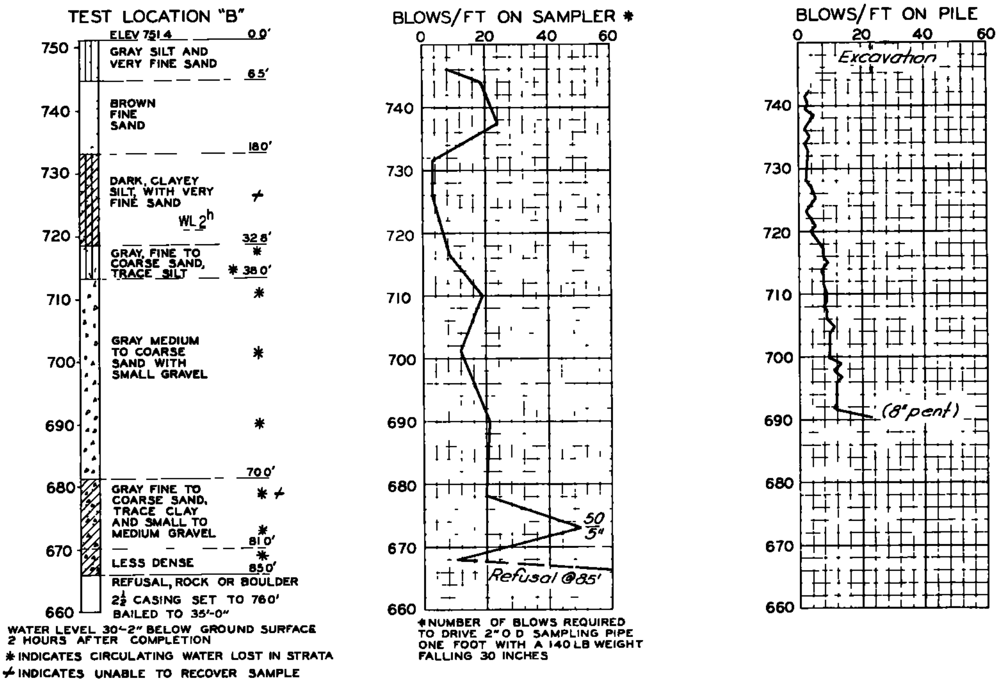
Pile driven Feb 11, 1959 and filled with concrete Feb 13, 1959, and tested Feb 24, and 25, 1959

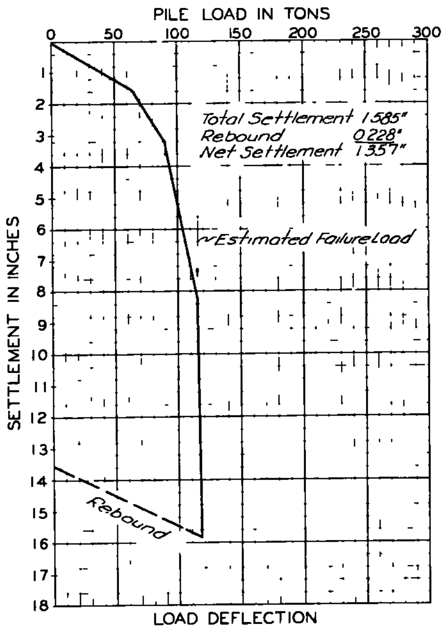
DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb. ram (Raymond 1-S)

Rated Energy	*19,500 Ft Lbs*
Weight of driving cap	*1,000 Lbs*
Approximate weight of pile at final penetration	*1,900 Lbs*
Strokes per minute	*58*
Driving time	*17 minutes*

gure 11. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location B, Pile No. 7A).





PILE DATA

14" BP 73 Steel H Pile 59' length, steel meeting requirements ASTM - Desig A-7

Driven length	59.0 feet
Loaded length	56.8 feet
Imbedded length	55.3 feet
Elevation of tip	690.2

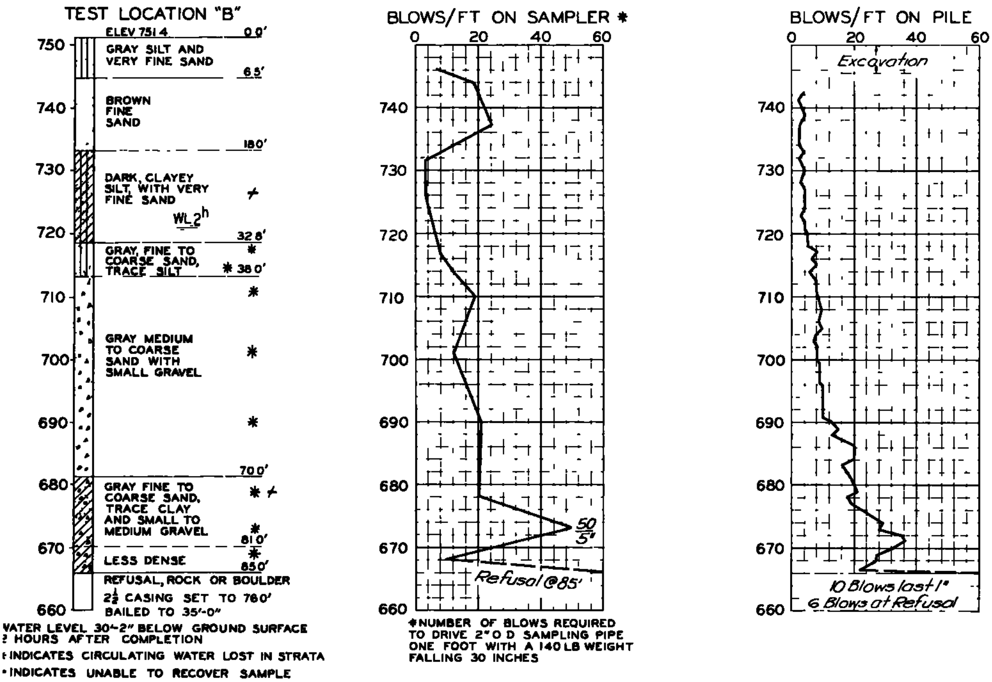
Pile driven Feb 11, 1959 and tested Feb 26, 1959

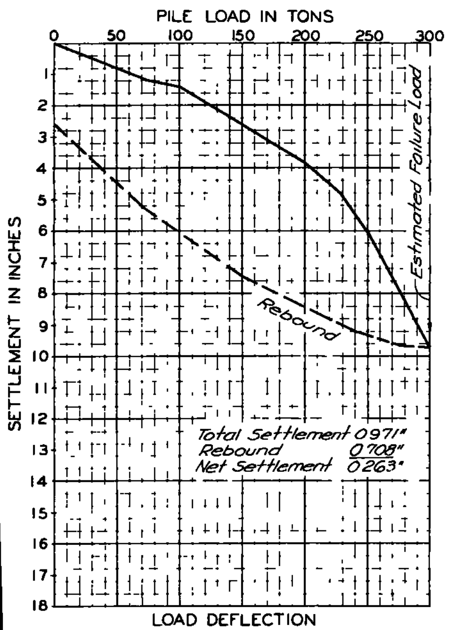
DRIVING DATA

Pile driven with Vulcan No 1 with 6,500 Lb ram (Raymond 1-S)

Rated Energy	19,500 Ft Lbs
Weight of driving cap	1,000 Lbs
Approximate weight of pile at final penetration	4,300 Lbs
Strokes per minute	58
Driving time	8 minutes

Figure 12. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location B, Pile No. 8).





PILE DATA

14" BP-73 Steel H Pile 85' length, steel meeting requirements ASTM Desig A-7

Driven length .... 85.0 feet
Loaded length .... 79.9 feet
Imbedded length .... 78.5 feet
Elevation of tip 667.1
Pile driven Feb 11, 1959

DRIVING DATA

Pile driven with Vulcan No 1 with 6500 Lb ram (Raymond 1-S)

Rated Energy .... 19,500 Ft Lbs
Weight of driving cap .... 1,000 Lbs
Approximate weight of pile at final penetration .... 6,200 Lbs
Strokes per minute 58
Driving time .... 19 minutes

gure 13. Intercity viaduct connecting Kansas City, Mo., and Kansas City, Kan. (Test Pile Program, Location B, Pile No. 8A).

in which

a = coefficient of energy loss in driving system (1.0 for drop hammer and 0.8 for all others);
W = weight of hammer;
H = height of fall of hammer;
Q = bearing capacity of pile (ton);
So = dynamic compression of pile with a fixed point; and
S = total plastic set per blow.

Any consistent set of units may be used in the formula.

This formula can be expanded as follows:

$$aWH = \tfrac{1}{2}\,QSo + QS = Q\left(\frac{So}{2} + S\right)$$

If the dynamic compression of the pile is taken as

$$\left(2aWH\,\frac{L}{AE}\right)^{1/2}$$

Then

$$Q = \frac{aWH}{\frac{S0}{2} + S} = \frac{aWH}{\left(\frac{aWHL}{2AE}\right)^{1/2} + S}$$

Let

$$K = \left(\frac{aWHL}{2AE}\right)^{1/2}$$

Then

$$Q = \frac{aWH}{K+S}$$

For a test pile

$$Q\ (\text{actual}) = \frac{aWH}{K\ (\text{actual}) + S\ (\text{actual})}$$

The actual K value for each pile, using the ultimate test loads and sets previously recorded, is obtained by direct computation.

$$K\ (\text{actual}) = \frac{aWH - Q(\text{actual}) \times S(\text{actual})}{Q(\text{actual})}$$

Although this value of K would be reduced for a length of pile shorter than the test pile, it is conservative to assume it constant. Using this value of K, the plastic set required to obtain bearing capacities of twice the design loads of 30, 40, 50, 60, and 70 tons per pile is determined from the formula

$$Q = \frac{aWH}{K+S}$$

The set value thus determined for design loads of 30, 40, 50, 60, and 70 tons per pile was converted to blows per foot and from an inspection of the driving records of the driven test piles, the length of pile was determined. For estimating purposes, the average lengths of cast-in-place friction pile for the design loads were

Location	30-Ton (ft)	40-Ton (ft)	50-Ton (ft)	60-Ton (ft)	70-To[n] (ft)
A	33	40	47	51	55
B	28	30	43	55	55

Comparative cost studies were made for the various design loadings of both fricti[on] piles and point bearing piles. For purposes of this comparison, preliminary design[s] were made for all 136 footings included in the central portion of the Intercity Viaduct

and quantities determined for all foundation items. The items for which quantities were determined and the unit prices applied in the comparison are

Structural excavation at $5.00 per cubic yard.
Footing concrete at $60.00 per cubic yard.
Reinforcing steel at $0.15 per pound.
Cast-in-place concrete piles and 12 BP 53 steel piles at $6.00 per linear foot.
14 BP 73 steel piles at $7.50 per linear foot.

As previously mentioned, various design loadings were assumed. For friction-type pile footings, design loadings of 30, 40, 50, 60, and 70 tons per pile were investigated, using pile lengths that were previously discussed. For the point bearing pile-type footings, design loadings of 6, 9, and 12 kips per square inch of cross-sectional area were used, with both 12 BP 53 and 14 BP 73 steel piles.

It is recognized that cost comparisons are always subject to criticism from the standpoint of the unit prices assumed. In this particular case, however, the cost of piles represents 70 to 80 percent of the total cost of the foundation. For this reason, the unit prices used for excavation, concrete and reinforcing steel would have no significant effect on the comparisons. Likewise, any variations in the cost per foot for piles would not materially change the comparisons.

As a result of the test pile program, the relative driving effort and pile lengths required was determined and the bearing capacities of four types of bearing pile in friction were obtained. It was found that the comparative results obtained by using the four types of piles in friction could be quite different in types of soil different from those which were encountered at this location. As previously mentioned in this report, the top 10 or 15 ft consisted of fine sand, the next 25 ft consisted of silty sand and sandy silt, and the balance consisted of fine to coarse sand and gravel.

Applying the unit prices which were tabulated previously, this test program shows that for the central portion of the Intercity Viaduct, consisting of 136 footings, the most economical plan was to use cast-in-place friction piles with a design load of 70 tons per pile. In second place with an increased cost of a little more than ½ of 1 percent was a footing designed for 4 steel pilings either 12 BP 53 or 14 BP 73 and using a maximum bearing value of 12,000 psi.

Discussion

F. M. FULLER, Raymond Concrete Pile Company—In conjunction with the test program reported by the author the pile contractor at his own expense and under the supervision of an outside firm of consulting engineers drove and load tested additional Raymond Step Taper piles.

Four of these were driven as friction piles approximately 56 ft in length using nominal point diameters ranging from 9 to 12 in. Two of these piles were driven at each test site in close proximity to the contract test piles reported by the author. The results of these tests are summarized in Table 1 and it will be noted that ultimate loads ranged from 160 to 175 tons.

In the supervising engineer's report based on the results of these tests it was concluded that the allowable load on the Step Taper friction piles is not affected materially by varying the tip diameter. Also it was their conclusion that a minimum design load of 70 to 80 tons for a 56-ft long Step Taper pile is justified.

As the author points out, the original purpose of this test program was to make an economic comparison between friction and end-bearing piles. When it became evident that end-bearing steel H piles would be used regardless of the test results and furthermore that only one type of pile would be tested in end-bearing, the contractor proceeded with a 300-ton test on an end-bearing Step Taper pile at test site A (Pile No. A5A).

This pile had a nominal 8-in. diameter and was driven approximately 72 ft to a final resistance of 10 blows per inch. Under 300-ton test (maximum capacity of testing equip-

TABLE 1

INTERCITY VIADUCT—CENTRAL PORTION—KANSAS CITY TEST PILE PROGRAM
(Additional Tests on Raymond Piles)

Location	Pile No.	Pile Type	Point Dia. (in.)	Length (ft)	Final Blows per in.	Ultimate Load (tons)
A	2A	Step taper	$10\frac{3}{8}$	56	3	165
	2B	Step taper	$9\frac{3}{8}$	56	2	165
	5A	Step taper	$8\frac{1}{8}$	72	10	over 300
B	5A	Step taper	$11\frac{3}{8}$	56	4	160
	5B	Step taper	$12\frac{3}{8}$	56	5	175

ment) the gross settlement was 0.83 in. and the net settlement after rebound was 0.36 in. Compared to this the 14 BP 73 steel H-pile driven to refusal had a gross settlement of 0.97 in. and a net of 0.26 in. under 300-ton test.

It was concluded by the supervising engineers that for the Step Taper end-bearing pile a minimum design load of 100 tons could certainly be used.

The complete data on all additional Step Taper piles tested were immediately reporte to all parties concerned.

Table 2 gives the estimated lengths for the different type piles under various design loads as prepared by the highway departments' consulting engineers. Also given in Table 2 is the table of average lengths at each of the two test sites which was included by the author. An examination of these tabulated data will reveal the following:

1. The Step Taper piles were the only piles considered by the engineers to have a design capacity of 70 tons.
2. In all cases where other piles are in the same load group with Step Taper piles, the estimated length for the Step Taper piles is equal to or less than the average length established for that load group.

With reference to (1) this means that the cost comparison between end-bearing and 70-ton friction piles was actually based on steel H-piles versus Raymond Step Taper piles. The cost analysis for cast-in-place piles was based on a unit pile price of $6. per lineal foot. This resulted in the 70-ton Step Taper piles costing about $18,000 les than the steel H-piles. The actual unit cost for the Step Taper piles, as guaranteed to the owners, was considerably less than the estimated unit cost. This difference in unit prices could conservatively be set at $1.00 per lineal foot. Therefore, using

TABLE 2

INTERCITY VIADUCT—CENTRAL PORTION—KANSAS CITY TEST PILE PROGRAM
(Data from Engineer's Report)

Location	Pile No.	Pile Type	Estimated Length (ft) for Design Loads of 30 Tons	40 Tons	50 Tons	60 Tons	70 Tons
A	1	$14\frac{1}{8}$" Monotube	32	39	46	55	-
	1A	$12\frac{1}{4}$" Monotube	33	42	49	-	-
	2	$14\frac{3}{8}$" Raymond step taper	32	34	42	46	51
	3	$12\frac{3}{4}$" Armco pipe	32	34	-	-	-
	3A	14" Armco pipe	32	33	34	-	-
	4	12 BP 53	37	50	55	-	-
B	5	$16\frac{3}{8}$" Raymond step taper	28	29	37	54	56
	6	$16\frac{1}{4}$" Monotube	28	32	51	57	-
	7	14" Armco pipe	26	37	53	-	-
	7A	16" Armco pipe	26	27	28	54	-
	8	14 BP 73	46	54	55	-	-
A		Cast-in-place friction[1]	33	40	47	51	55
B		Cast-in-place friction[1]	28	30	43	55	55

[1]For estimating purposes, average length of cast-in-place friction pile (H-piles excluded as frictio piles) for the design loads.

$5.00 per lineal foot as the cost of the 70-ton cast-in-place piles the savings over the end-bearing H-piles would actually be about $54,000.

With reference to (2) the estimated lengths for the Step Taper piles were, in some cases, 5 to 6 ft shorter than the average length for that load group. It should be noted that this difference would never show up in any comparison of pile cost based on the same average length for both tapered and constant section piles.

Also it should be noted that in every case, except the 70-ton group, the average length used was equal to or less than the longest pile in the group (excluding H-piles which were not considered as friction piles). In other words, in every case there would be some pile type driving considerably longer than the "average." This also would not show up in any comparison of pile costs based on "average lengths."

In Table 3 are assembled data from the engineer's report to compare the estimated pile lengths based on the Danish formula; the length for each pile at which the various capacities were theoretically attained as determined by the modified Eytelwein formula; and the required pile length on a proportionate basis using the actual capacity at the 56-ft length. Also shown are the final Eytelwein formula capacities based on the driving resistance for each pile together with the actual capacities based on the ultimate test loads after applying a factor of safety of 2.

Obviously, there is little agreement between the Danish formula, the Eytelwein formula, and the actual results. For example:

1. Pile No. 2 (Step Taper) according to the Eytelwein formula developed a theoreti-
:al capacity of only 27.8 tons which was the lowest formula capacity of all friction
piles (excluding the H-pile). Yet, the Step Taper piles actually developed the highest
capacity as determined by load test.
2. In spite of the final test results pile No. 2 (Step Taper) and No. 3 (12¾-in. pipe)
were given equal estimated lengths for both 30- and 40-ton loads.
3. Pile No. 3A (14-in. pipe) and Pile No. 7A (16-in. pipe) were each given basical-
y the same estimated lengths for 30-, 40- and 50-ton design capacities. For a friction
ile of this type this is not reasonable.
4. All piles (except steel H) in each test group (1 to 3A and 5 to 7A) were assigned
asically the same estimated lengths for a 30-ton design capacity. However, the test
esults definitely indicate a difference in load-carrying ability between the various
pe piles of the same length.

TABLE 3

INTERCITY VIADUCT—CENTRAL PORTION—KANSAS CITY TEST PILE PROGRAM
Comparison of Estimated Lengths by Danish Formula (D) vs Length by Eytelwein Formula (D)[1] vs Estimated Length by Proportionate Method (P)[2]
For Various Design Loads[3]

e .	Type	30 Tons D E P (ft)	40 Tons D E P (ft)	50 Tons D E P (ft)	60 Tons D E P (ft)	70 Tons D E P (ft)	Final Eytel. Formula Capacity (tons)	Actual Capacity by Test FS = 2 (tons)
	Monotube 14 in.	32 41 29	39 47 39	46 - 49	55 - 58	- - 68	48.5	57.5
	Monotube 12 in.	33 43 33	42 44 44	49 53 55	- - 65	- - 76	48.9	51.5
	Step taper 14 in.	32 - 23	34 - 31	42 - 39	46 - 46	51 - 54	26.0	72.5
	Pipe 12 in.	32 34 40	34 - 53	- - 66	- - 79	- - 92	33.6	42.5
	Pipe 14 in.	32 33 32	33 34 43	34 - 53	- - 64	- - 75	40.1	52.5
	H 12 in.	37 - 31	50 - 41	55 - 51	- - 61	- - 71	26.7	55.0
	Step taper 16 in.	28 54 20	29 - 27	37 - 34	54 - 41	56 - 48	39.8	82.5
	Monotube 16 in.	28 32 27	32 48 36	51 54 45	57 55 54	- 55 63	78.0	62.0
	Pipe 14 in.	26 36 31	27 53 41	53 55 51	- - 61	- - 71	52.2	55.0
	Pipe 16 in.	26 27 27	27 27 36	28 53 46	54 55 55	- - 64	63.2	61.5
	H 14 in.	46 55 29	54 55 39	55 - 48	- - 58	- - 68	43.7	58.0

gth by Eytelwein formula based on driving log of test pile.
gth by Proportionate Method based on actual capacity for 56-ft pile.
entry indicates capacity not attainable under method used to determine required length.

An examination of Table 3 and the author's explication of the Danish formula reveal that some very important factors which influence pile driving are disregarded and some very broad assumptions are made. Although the K factor in the Danish formula is related to elastic energy losses, its value in each case was determined by the actual ultimate capacity and final set of the pile. A comparison of final resistance versus actual capacity by load test indicates no correlation. Yet the determination of pile lengths by this formula is based on the results of the driving log for each test pile. Thus, the beneficial effect of the weight and rigidity of certain piles in efficiently transmitting the hammer energy (for example, piles 2 and 5) is entirely disregarded. The high energy losses due to elastic deformations in the light-weight piles is demonstrated by the substantial increase in the driving resistance when the pile tip enters a denser soil. However, according to the Danish formula this would indicate greater capacity which is just the reverse of that proved by the test program.

With reference to Tables 3 and 4 this is indicated for pile 7A which can be classified as a light pile (34 lb per foot as compared to the weight of pile No. 5—180 lb per foot). When this pile encountered the dense stratum 27 ft below ground surface the blow count jumped from 9 blows per foot to 30 blows per foot. Therefore, any capacity which required up to 30 blows per foot according to the Danish formula would be satisfied with this 27-ft pile. In comparison, for the heavy step tapered pile No. 5 the blow count went from 9 to 15 when the pile point encountered the same stratum.

It should be evident, therefore, that predicting pile lengths by some formula where the effects of various pile characteristics are not considered can be very misleading.

Table 4 summarized data from the engineer's report together with supplemental information. Here the safe load as determined by the modified Eytelwein formula (use by the engineers) is compared to the safe load as determined by the Engineering News formula. It will be noted that for all of the light piles (Nos. 1, 1-A, 3, 3-A, 4, 6, 7, and 7-A) the allowable load by the Eytelwein formula is higher than that by the Engineering News formula. Conversely, for the heavy piles (Nos. 2, 5 and 8) the safe load by Eytelwein formula is less than that by Engineering News. In the modified Eytelwei formula the factor 0.1 is multiplied by the ratio of weight of pile to weight of hammer ram. Obviously, this formula favors a light pile and penalizes a heavy pile. Also, it should be obvious from the test results that this formula is basically wrong. The results of this formula are actually contrary to the experience on this test program in which the heavy Step Taper piles carried the highest ultimate load. It is evident there fore, that the elastic energy loss in the light piles is much more serious than inertia

TABLE 4

INTERCITY VIADUCT—CENTRAL PORTION—KANSAS CITY TEST PILE PROGRAM
(Data from Engineer's Report)

Pile No.	Pile Type[1]	Diameter - Inches Butt	Tip	Pile Wgt. (lb)[2]	Avg. Wgt. (per ft-lb)	Final Res. (blows/ft)	Safe Load by Formula[3] (-tons) EYT	EN	Actual Yie Load by Test (-to
1	Monotube	14⅛	8	1,487	27	33	48.5	42	115
1A	Monotube	12¼	8	1,383	25	33	48.9	42	103
2	Step taper	14⅜	8	8,564	153	20	26.0	27	145
3	Pipe	12¾	12¾	1,510	27	22	33.6	30	85
3A	Pipe	14	14	1,660	30	27	40.1	36	105
4	Steel H	12	12	2,968	53	18	26.7	25	110
5	Steel taper	16⅜	10	10,048	180	37	39.8	46	165
6	Monotube	16¼	8	1,670	30	58	78.0	63	124
7	Pipe	14	14	1,660	30	36	52.2	45	110
7A	Pipe	16	16	1,900	34	44	63.2	53	123
8	Steel H	14	14	4,300	73	33	43.7	50	116

[1]Monotube furnished by Union Metal Mfg. Co.; step taper furnished by Raymond Concrete Pile Co., pipe furnished by Armco Drainage & Metal Products, Inc.; all pipes 56 ft long and driven with 19,500 ft-lb hammer.
[2]Add 1,000 lb for weight of driving cap.
[3]EYT = Modified Eytelwein formula used by engineers.
EN = Engineering News formula.

in heavy piles. The superior driving characteristics of a heavy rigid pile (using an adequate size hammer) has been clearly demonstrated on very many pile tests under a variety of sub-soil conditions throughout the country.

The sub-soil at the Intercity Viaduct test sites could generally be classified as granular or cohesionless. This soil type is very commonly found and therefore the results of these tests are not only applicable to the Viaduct site but could be applied elsewhere. Inasmuch as the soil profile is quite common to all parts of the country these tests represent a valuable contribution with wide application. Comparable results could be expected elsewhere.

It is interesting to note that the test load reaction was obtained by jacking against anchor piles. These were Raymond Step Taper piles approximately 70 ft in length. In some cases uplift loads of over 100 tons per pile were resisted.

HRB:OR-412