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Foundation Selection and Construction Performance - Clark Bridge Replacement

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SYNOPSIS: The paper describes the foundation investigation for the Clark Bridge Replacement, which spans the Mississippi River at Alton, Illinois. The subsurface investigation and the design considerations leading to the foundation piling selection are detailed. The construction performances of the selected H-piles and composite piles are described, including the use of pre-construction wave equation analyses to predict the performance of proposed pile hammers, and the use of the dynamic pile driving analyzer during construction to limit driving stresses and prevent pile damage.

INTRODUCTION

The old Clark Bridge, completed in 1928, is a two lane structure, which spans the Mississippi River about 20 miles north of St. Louis, between Alton, Illinois and St. Charles County, Missouri. Construction of the Clark Bridge Replacement, which is located about 1000 ft downstream from the existing bridge, began in mid-1990 and is scheduled to be essentially complete by December 1, 1993. Figure 1 shows the locations of the old and the new Clark Bridge.

The old Clark Bridge is 3640 ft long with a roadway only 20 ft wide. The super structure is a combination of concrete girder spans (Missouri Approach), steel through truss spans (Main Span), and steel deck truss spans (Illinois Approach) with a concrete deck throughout. The piers are concrete or steel columns on concrete footings or pedestals, all supported by timber piles. The main deficiencies that caused the need for a new bridge were structural deterioration and scour problems, as well as a significantly increased traffic demand and roadway deficiencies.

The new bridge will have a total length of 4620 ft, with 20 approach spans, which range between 115 ft and 200 ft in length. The main spans will be cable-stayed, with a center span of 756 ft and tail spans of 302 ft. The bridge deck will be composite concrete and steel, and the entire structure will be supported by steel H-piles. The new bridge will have four lanes with accommodations for a bicycle path in the outside shoulders. The structure is skewed 20 degrees with the river flow. Figure 2 shows a plan and elevation of the new Clark Bridge.

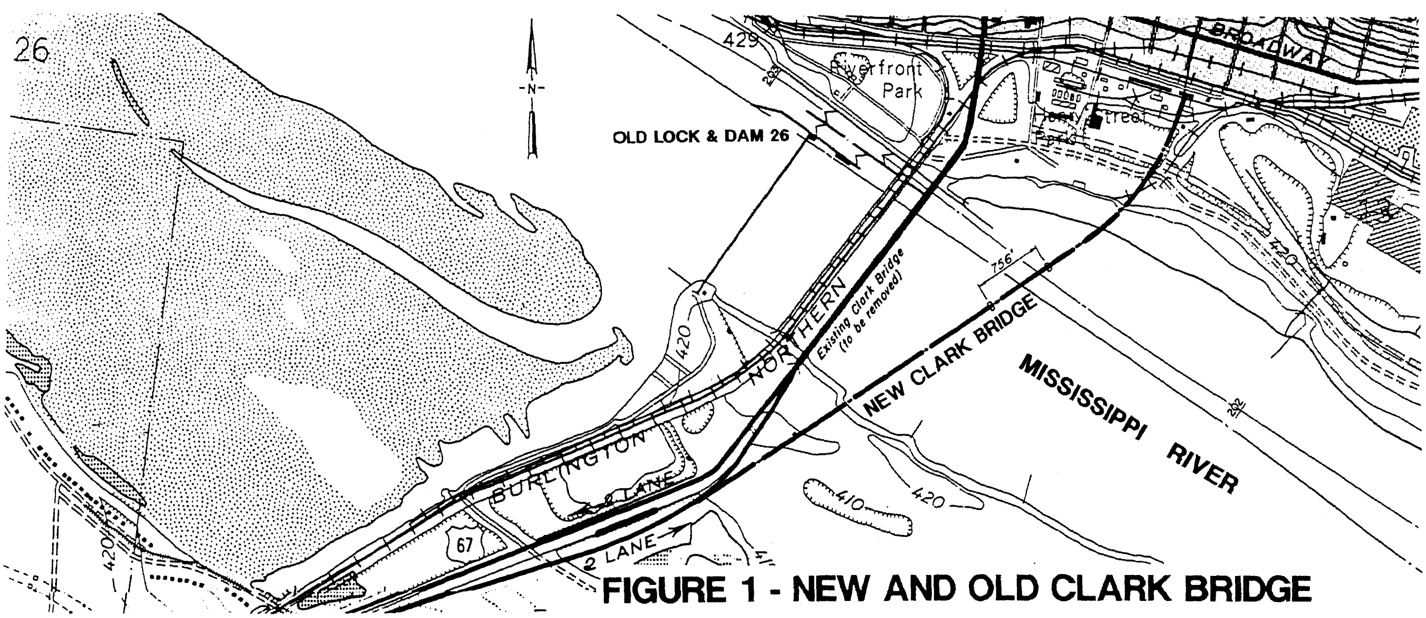

FIGURE 1 - NEW AND OLD CLARK BRIDGE

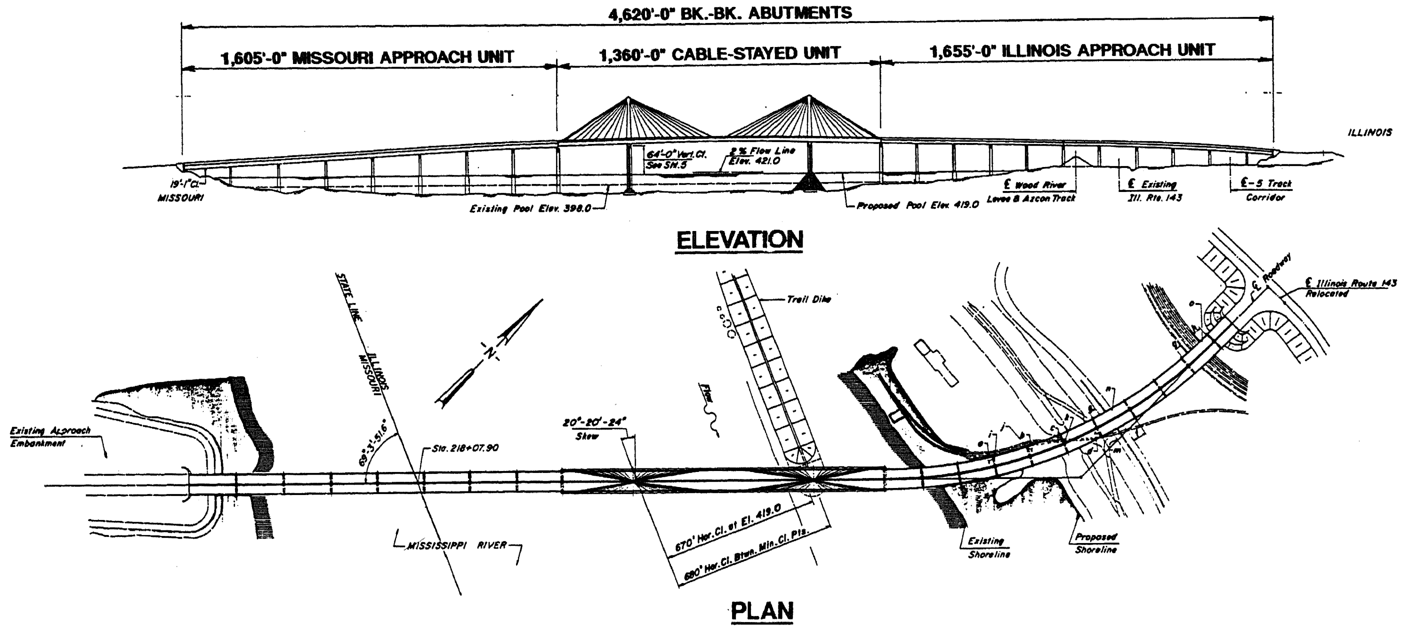

FIGURE 2 - NEW CLARK BRIDGE

SUBSURFACE INVESTIGATION

Geology

The project area lies in the southwestern edge of the Springfield Plain section of the Central Lowlands Physiographic Province. The surface deposits are glacial in origin and were deposited during the Pleistocene epoch. This area was covered by the Illinoian stage of glaciation. The more recent Wisconsinan ice front did not extend into this area. However, the near surface deposits of windblown silt (loess) and the outwash in the river valley are derived from the materials of Wisconsinan age glaciers.

Alton lies on the steep hillside overlooking the floodplain. Upstream from the new bridge on the Illinois side, high limestone bluffs rise above the river to elevations exceeding 550 ft M.S.L. The average elevation of the river floodplain at Alton is 415 ft M.S.L.

The following paragraphs describe the surficial geology (see Figure 3).

Floodplain Deposits: Floodplain deposits consist primarily of high to low plasticity clay, with varying amounts of silt, fine sand, and organic material. The clay is soft to firm, and the sand is very loose to loose.

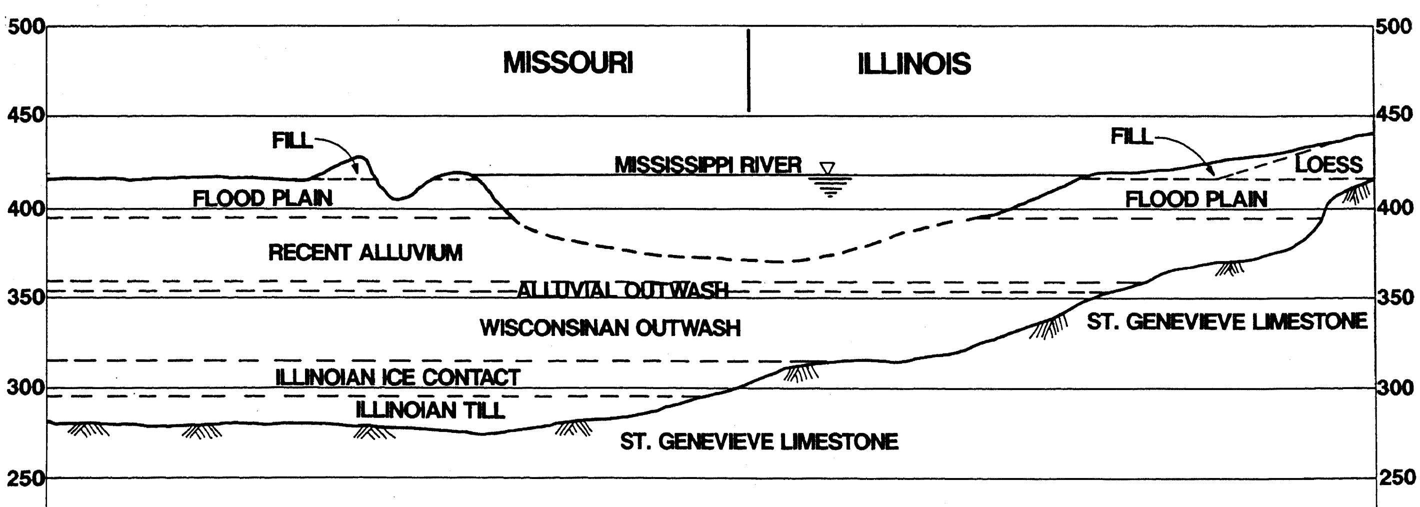

FIGURE 3 - GENERAL SURFICIAL GEOLOGY

consistent with previous results and only increased from 213 to 253 kips. It was concluded that correlations between CAPWAP and CASE Method capacity results and static test results were poor in this situation, and the static load results were relied upon.

Pile length for Test Pile #2 was calculated using the elastic deformation equation based on the static load test No. 1. Using the static load settlement curve at twice the design load and the corresponding settlement, pile length was estimated at 85 feet. Test Pile #2 was driven to 79 feet at a final driving resistance of 33 blows per foot and 23 psi gauge pressure. CAPWAP and CASE Method results indicated pile capacities were 20 percent less than required ultimate capacity. A static load test was conducted two days after Test Pile #2 was installed. Load test results as presented in Figure 21 indicate pile failure occurred at 416 kips. This is an approximate factor of safety of 3.0 on the 140 ton design pile capacity.

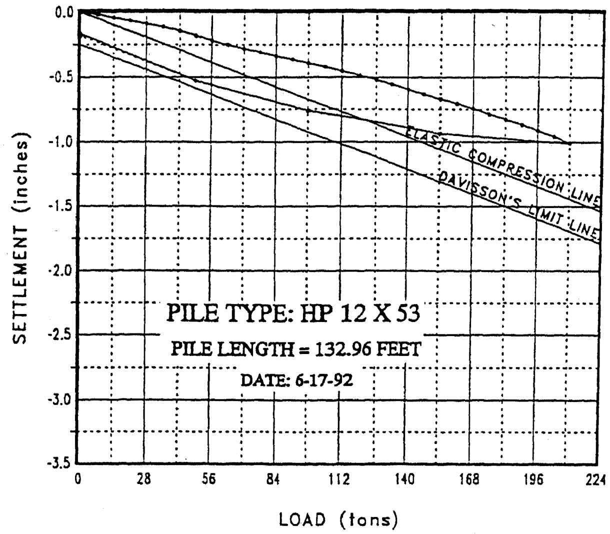

FIGURE 20. LOAD TEST NO.1 MO. ABUTMENT

TABLE 6
LOAD TEST PILES
MISSOURI ABUTMENT

PILE #	DRIVING STATUS	DATE TESTED	APPROXIMATE PENETRATION DEPTH (ft)	DRIVING RESISTANCE (bl/ft)	AVERAGE ENERGY TRANSFER TO PILE HEAD (ft–lbs)	CALCULATED PILE HEAD COMPRESSION STRESS (ksi)	MAXIMUM CAPWAP CALCULATED COMPRESSION STRESS (ksi)	MOBILIZED CASE METHOD CAPACITY PREDICTION RMX, J=0.60 (kips)	MOBILIZED CAPWAP CAPACITY ESTIMATE (kips)	STATIC LOAD TEST RESULT (kips)
LTP #1	EOID	6/12/92	70	22	10,570	17.92	18.97	179	180	––
	EOR #2	6/15/92	130	42	12,280	20.35	20.79	213	250	––
	N/A	6/17/92	130	––	––	––	––	––	––	420+
	BOR #3	6/18/92	130	7/1"	8,900	18.39	––	253*	––	––
LTP #2	EOID	6/22/92	69	33	11,110	20.27	21.26	232	247	––
	N/A	6/24/92	69	––	––	––	––	––	––	416

NOTES: EOID = END OF INITIAL DRIVE
BOR = BEGINNING OF RESTRIKE
* = INDICATES CASE DAMPING FACTOR OF J = 0.25 WAS USED FOR FIELD EVALUATION
+ = INDICATES FAILURE DID NOT OCCUR DURING STATIC LOAD TEST
LTP = LOAD TEST PILE

TABLE 7
LOAD TEST PILES
MISSOURI ABUTMENT

PILE #	APPROXIMATE PILE PENETRATION (ft)	SMITH DAMPING (sec/ft)		SOIL QUAKE (inches)		MOBILIZED CAPWAI CAPACITY (kips)			STATIC LOAD TEST CAPACITY (kips)
		SHAFT	TOE	SHAFT	TOE	SHAFT	TOE	TOTAL	
LTP #1	70	0.08	0.10	0.10	0.35	145	35	180	–––
LTP #1	130	0.10	0.18	0.08	0.27	222	28	250	420+
LPT #2	69	0.08	0.07	0.10	0.30	221	25	246	416

NOTES: + = INDICATES FAILURE DID NOT OCCUR DURING STATIC LOAD TEST
LTP = LOAD TEST PILE

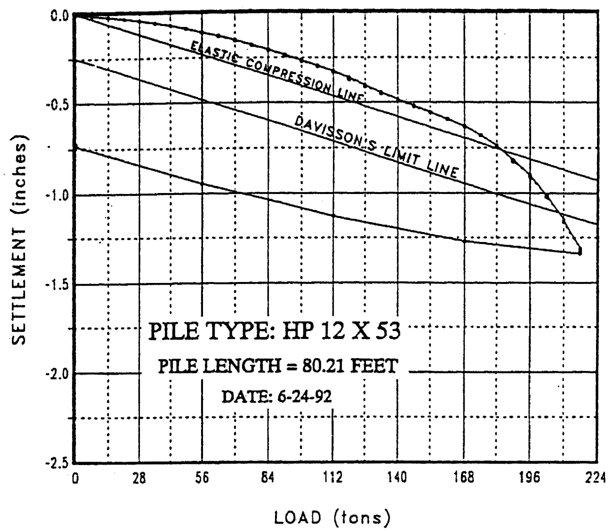

FIGURE 21. LOAD TEST NO.2 MO. ABUTMENT

In general, dynamic pile capacity estimates were 40 percent less than the ultimate static pile capacities. Dynamic shaft resistance was estimated at 186 kips and was fairly consistent with the estimated 224 kips static shaft resistance. Dynamic end bearing was estimated at 30 kips, significantly less than the static load end bearing of 192 kips. STS Consultants theorized, and IDOT concurred, that under dynamic conditions where high accelerations existed near the H-pile toe, full soil resistance through arching between the webs and flanges was not experienced. As a result, end bearing resistance obtained during dynamic testing was a very conservative estimate.

The results can be expanded to show that the static test results account for arching to form a square plug at the H-pile toe. Eliminating the tip bearing from the CAPWAP dynamic calculations and including the full 12 inch x 12 inch base value of 192 kips accounts for the difference in dynamic and static tests. However, this explanation does not account for the shaft resistance, which should be much higher with the longer pile. Extrapolating the load test for the longer pile to failure would result in a load of over 600 kips. The previous explanation does not account for the shaft resistance difference, which would lead to such a high capacity. The difference in static and dynamic results must lie in an unmeasured value, possibly due to the H-pile shape in combination with the reduction in effective pile surface during dynamic loading in granular soils. The dynamic and static loading conditions appear to have different failure modes in these granular soils supporting H-piles.

Production pile driving criteria were established by wave equation results and static load test results. Using the static load test results at twice the design load and the equation for elastic deformation, a 75 foot production pile length was estimated. In addition, a minimum final driving resistance of 30 blows per foot and a minimum gauge pressure of 20 psi was required.

Production piles were driven through the 31 foot embankment into the dense underlying sands. Driven lengths varied between 69 to 76 feet with tip elevations near 360.0 and 365.0. Final driving resistance ranged between 23 to 34 blows per foot, and hammer gauge pressure varied between 18 and 20 psi.

Illinois Approach

The Illinois Approach is 1,655 feet long and supported by Piers 12 through 21 and the North Abutment. Piers 12 through 17 are supported on 14 x 117 H-piles. Pile lengths range from 84 to 110 feet. The H-piles were designed to have a 206 ton design pile capacity and be driven to refusal on bedrock. Both pile sections were rolled from ASTM A-36 steel. The FHWA recommended limit of 0.9 times the yield strength of the pile was used as the maximum allowable driving stress.

Piers 18 through 21 and the North Abutment are supported on 14 x 89 H-piles. These pile lengths ranged from 58 to 82 feet. The 14 x 89 H-piles were required to be driven to a 157 ton design pile capacity and to refusal on bedrock.

Pile Hammer and Wave Equation: Piles for Piers 12, 13, 14, 15, 18, 19, and 20 were driven with an IHC S-70 hydrohammer. The IHC S-70 has a 7,700 pound ram with a maximum stroke of 6.69 feet. A hammer cushion is not used with the IHC hammers. The manufacturer's maximum rated energy is 51,000 foot-pounds, and the minimum rated energy is 1,450 foot-pounds. The driving system for Pier 21 and the North Abutment will consist of an MKTDE70B single acting diesel.

The wave equation analysis assumed a triangular soil resistance distribution over the lower 80 percent of the pile length and a constant shaft resistance of 300 kips. CAPWAP analyses performed on piles at the main span and Missouri approach provided representative damping and quake parameters. Smith damping factors for shaft and toe were 0.10 and 0.08 seconds per foot, respectively. Soil quake factors selected for shaft and toe were 0.10 inches and 0.09 inches, respectively.

Using the maximum hammer stroke for both pile sections at 2.0 and 2.5 factors of safety, wave equation analyses indicated compressive stresses were as high as 39 ksi, well over the yield stress of A36 steel. Analyses run with reduced hammer strokes and energy levels indicated compressive stresses were within the FHWA guidelines. As a result, a maximum hammer stroke of 5 feet and a maximum energy of 31,900 foot-pounds for the 14 x 117 H-piles indicated compressive stresses would not exceed FHWA guidelines. The 14 x 89 H-pile analyses indicated the hammer stroke should not exceed 4.5 feet, and the energy to the pile should not exceed 27,600 foot-pounds. A 10 percent loss between the energy transferred to the pile head and the IHC readout panel energy was assumed. Results of the wave equation analyses are provided in Table 8.

TABLE 8
WAVE EQUATION ANALYSES
ILLINOIS APPROACH
PIERS 12, 13, 14, 15, 18, 19, 20

ANALYSIS #	PILE SECTION	ANALYSIS TYPE	HAMMER STROKE (ft)	WAVE EQUATION PREDICTED COMPRESSION STRESS (ksi)		WAVE EQUATION PREDICTED DRIVING RESISTANCE (bl/ft) FOR CAPACITY		WAVE EQUATION PREDICTED ENERGY TRANSFER TO PILE HEAD (ft-lbs) AT CAPACITY	
				628 kips	785 kips	628 kips	785 kips	628 kips	785 kips
1	HP 14 X 89	STANDARD	6.69	38.4	38.8	63	144	41,000	41,000
2	HP 14 X 89	VARIABLE STROKE	3.0–6.69	25.4–38.4	---	335–63	---	18,400–41,000	---
3	HP 14 X 89	VARIABLE STROKE	3.5–6.69	---	27.8–38.8	---	869–143	---	21,500–41,000
4	HP 14 X 89	STANDARD	4.50	31.6	31.6	139	358	27,700	27,600
ANALYSIS #	PILE SECTION	ANALYSIS TYPE	HAMMER STROKE (ft)	WAVE EQUATION PREDICTED COMPRESSION STRESS (ksi)		WAVE EQUATION PREDICTED DRIVING RESISTANCE (bl/ft) FOR CAPACITY		WAVE EQUATION PREDICTED ENERGY TRANSFER TO PILE HEAD (ft-lbs) AT CAPACITY	
				824 kips	1030 kips	824 kips	1030 kips	824 kips	1030 kips
5	HP 14 X 117	STANDARD	6.69	37.0	36.8	89	190	42,500	42,600
6	HP 14 X 117	VARIABLE STROKE	3.5–6.69	26.1–37.0	---	315–89	---	22,400–42,500	---
7	HP 14 X 117	VARIABLE STROKE	4.0–6.69	---	28.0–36.8	---	766–189	---	25,600–42,600
8	HP 14 X 117	STANDARD	5.0	31.5	31.9	157	357	31,800	31,900

Test Pile and Production Driving: The pile driving analyzer was used on select test piles at each pier along the Illinois Approach. The results of the CAPWAP and PDA analysis are provided in Table 9. Using the wave equation results as the established driving criteria, the test piles were driven to refusal on or near the limestone bedrock. The hydrohammer was operated at maximum stroke. IHC panel readout energies at final driving ranged from 29 to 38 kips per foot and were consistent with wave equation values. Results from the PDA and the CAPWAP indicated the maximum compressive stresses were exceeded by as much as 14 percent. To avoid overstressing the production piles at Piers 18, 19, and 20, it was recommended that the driving hammer be operated around 30,000 foot-pounds or less and 36,000 foot-pounds or less for Piers 12 through 15. Final driving resistance varied from 160 to 300 blows per foot and was consistent with the predicted wave equation values of 157 to 357 blows per foot. No pile damage from hard driving in the cobble zones or on bedrock was detected. A refined wave equation analysis using soil resistance distribution, soil quake, and damping parameters from the CAPWAP analyses was performed for pile capacities with 2 and 2.5 factors of safety. A wave equation bearing graph was generated from this analysis as reference for production pile driving criteria. One example of this graph is provided in Figure 22 for combined results from Piers 18 and 19.

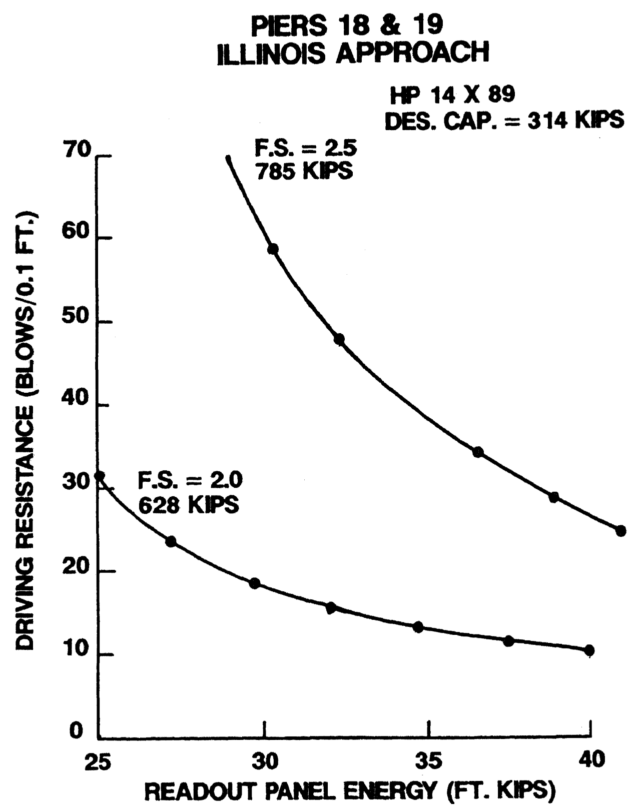

FIG. 22. REFINED WAVE EQUATION RESULTS

TABLE 9
ILLINOIS APPROACH
PIERS 12, 13, 14, 15, 18, 19, 20 TEST PILE RESULTS

TEST PILE #	BRIDGE PIER	PILE HAMMER	DELIVERED(1) HAMMER ENERGY (ft-lbs)	AVG ENERGY(2) TRANSFERRED TO PILE HEAD (ft-lbs)	AVG ENERGY TRANSFER RATIO	FINAL DRIVING RESISTANCE (blows/ft)	MAXIMUM PILE HEAD COMPRESSION STRESS FROM PDA (ksi)	MAXIMUM COMPRESSION STRESS & LOCATION FROM CAPWAP-C ANALYSIS (ksi)	MAXIMUM CASE METHOD CAPACITY (kips) & J = .70	CAPWAP-C TOTAL	CAPACITY SHAFT	(KIPS) TOE	SMITH DAMPING SHAFT (sec/ft)	SMITH DAMPING TOE (sec/ft)	QUAKES SHAFT (inches)	QUAKES TOE (inches)
TP-2	12	IHC S-70	36,000	35,130	97%	20/0.0	29.7	30.1 (at toe)	979	1080	240	840	0.19	0.04	0.07	0.16
TP-4	13	IHC S-70	33,100	30,750	93%	30/0.1	30.8	25.1 (at toe)	821	864	280	584	0.19	0.07	0.10	0.16
TP-5	14	IHC S-70	35,100	33,070	94%	16/0.1	30.0	31.7 (at toe)	931	898	167	731	0.19	0.07	0.08	0.17
TP-6	14	IHC S-70	36,100	33,360	92%	16/0.0	29.8	32.2 (at toe)	945	980	200	780	0.14	0.04	0.10	0.18
TP-7	15	IHC S-70	37,300	35,300	95%	15/0.0	32.4	33.3 (at toe)	965	980	375	605	0.21	0.03	0.10	0.11
TP-8	15	IHC S-70	38,000	34,720	91%	19/0.1	30.2	31.4 (at toe)	862	924	300	624	0.18	0.03	0.10	0.12
TP-13	18	IHC S-70	34,200	32,150	94%	12/0.0	37.8	39.5 (at toe)	952	805	177	628	0.15	0.19	0.08	0.11
TP-14	18	IHC S-70	32,400	29,980	92%	6/0.0	34.1	30.8 (at toe)	794	819	254	565	0.18	0.11	0.08	0.09
TP-15	19	IHC S-70	29,500	27,000	91%	8/0.0	33.9	34.8 (at toe)	897	854	204	650	0.08	0.14	0.08	0.10
TP-16	19	IHC S-70	30,200	23,960	79%	12/0.0	29.5	31.4 (at toe)	689	650	149	501	0.08	0.12	0.08	0.12
TP-17	20	IHC S-70	35,000	29,300	84%	16/0.0	31.8	34.9 (at toe)	852	855	167	688	0.10	0.08	0.08	0.14
TP-18	20	IHC S-70	33,000	28,500	86%	15/0.0	31.3	39.4 (at toe)	948	861	213	648	0.10	0.15	0.08	0.08
TP-19	20	IHC S-70	33,000	28,200	85%	N/A	29.8	30.7 (at toe)	659	589	214	375	0.13	0.17	0.10	0.22

NOTES:
(1) BASED ON W.H. FOR MKT HAMMER OR READOUT ON IHC CONTROL PANEL
(2) BASED ON AVERAGE OF FINAL 20 HAMMER BLOWS

Production piles for Piers 12, 13, 14, 15, 18, 19, and 20 were driven to refusal in the limestone bedrock. Piles penetrated the sand and gravel glacial deposits with little difficulty since very few cobble and boulder zones were encountered. To penetrate the hard driving zones, HB 77600B hard bite pile points were attached to the piling. PDA and wave equation results were used to establish driving criteria. The 14 x 89 H-piles for Piers 12 through 15 were driven to tip elevations of 298 to 314. The 14 x 117 H-piles for Piers 18, 19, and 20 were driven to tip elevations of 299.6 to 313.5. Final driving resistance for the 14 x 117 H-piles ranged between 180 to 250 blows per foot and had driving energies between 35 to 37 kips per foot. The 14 x 89 H-piles had final driving resistances that ranged between 180 to 200 blows per foot and driving energies of 33 to 37 kips per foot. Generally, these values were within 5 to 30 percent of the predicted values from the wave equation and the PDA. Based on driving resistance values, design ultimate capacities were verified by the wave equation bearing graph.

SUMMARY

High capacity H-piles driven with large pile hammers to refusal on bedrock were used to support the main spans and Illinois approach spans. The piles penetrated the dense sands and gravels and even thick cobble and boulder deposits with relative ease. Reinforced pile tips were used to protect the ends of piles driven hard into the limestone bedrock. Several piles were extracted for inspection and showed no damaged.

Preconstruction wave equation analyses were successfully used to evaluate and ultimately approve (in conjunction with pile driving analyzer results during construction) a Eropean-made pile driver. The "hydrohammer" was not conducive to standard IDOT methods of evaluation because of its much more efficient than normal ability to transmit energy from the hammer to the pile top. The pile driving analyzer (PDA) was also successfully used to estimate compressive stresses during test pile driving, so that driving criteria to limit pile damage could be developed for production piles. CAPWAP analyses were used in conjunction with the PDA to develop "refined wave equation" generated curves relating energy delivered to the pile to blow counts for a specific pile capacity. This information was used as guidance during construction, in addition to standard IDOT refusal driving criteria.

The major concern for the composite piles driven for the Missouri approach piers was that the high energy needed to advance the large diameter pipe sections may overstress the smaller H-pile stinger, especially during final driving on bedrock. Again, preconstruction wave equation, PDA, and CAPWAP analyses were used to determine an appropriate driving energy to avoid overstressing of the stinger. A closed-end pipe section (at the pipe-stinger juncture) was used initially, but after problems developed, it was determined that an open-end pipe at the juncture would facilitate more efficient production driving.

At the Missouri abutment, medium capacity friction H-piles were designed, and a static load test was required by the specifications. Driving of the load test pile was controlled by wave equation analyses and the PDA, and the resulting driven length was over twice the estimated length (by static capacity calculation methods). After the first static load test indicated an ultimate load of well over three times the design load, a second pile was driven to a shorter length (closer to the originally estimated length). Even though the PDA and CAPWAP analyses indicated an ultimate capacity of less than twice the design load, the second pile was load-tested to failure at three times the design load. The production piles were then driven to lengths 5 feet to 10 feet less than the second load test pile.

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REFERENCES

Fruco Engineers Incorporated (1973), for the U.S. Army Engineers District, St. Louis, "Over Water Steel H-Pile Driving and Testing Program, Lock and Dam 26 (Replacement)."

Fruco Engineers Incorporated (1982), "Feasibility Study of Guide Wall Caisson Foundations - Lock and Dam No. 26 (Replacement)."

Fruco Engineers Incorporated (1983), "Caisson Installations for Upstream Guide Wall - Lock and Dam No. 26 Replacement."

Gobel, Raushe, Likens Incorporated for Illinois Department of Transportation (July 1992 - Present), Wave Equation Analyses Reports for the North Abutment and Piers 20 and 21; and Dynamic Pile Testing Reports for Piers 6, 7, 20 and 21.

Hanson Engineers Incorporated and Globetrotters Engineering Corporation (1989), "Geotechnical Report for the Clark Bridge Replacement Over the Mississippi River."

Hanson, W.E., and Harms, J.E. (1990), "The Old and New Clark Bridge."

Shannon and Wilson, Inc. (1981), "Lock Pile Installation Study, Lock and Dam No. 26 (Replacement), Mississippi River, Alton, Illinois."

Soil Testing Services Incorporated for IDOT (July 1990 - July 1992), Preconstruction Wave Equation Reports and Dynamic Pile Testing Summary Reports for the Main Span, the Missouri Approach (Piers 1-6), the Illinois Approach (Piers 12-19), and the Missouri Abutment.