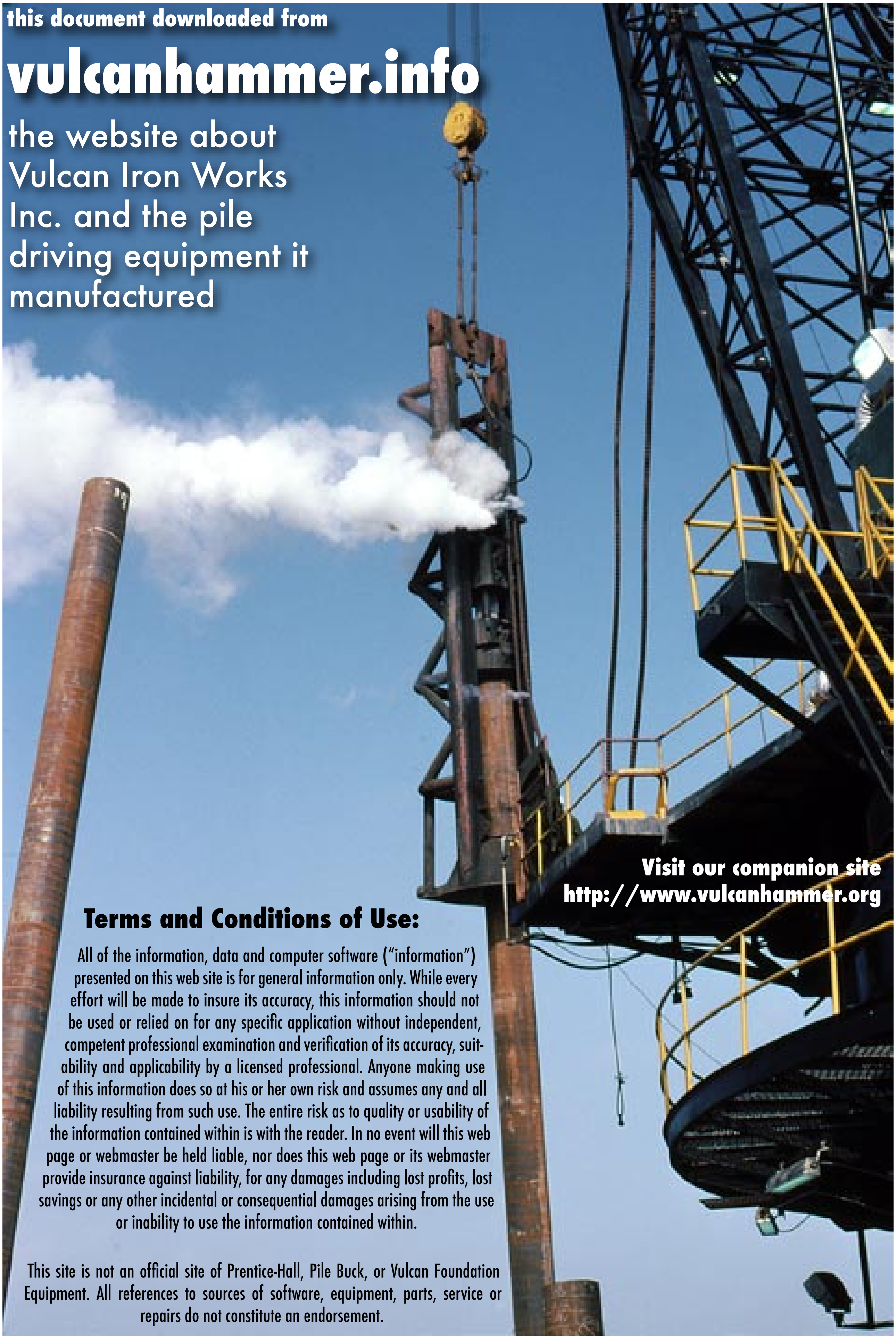

TEST PILES IN SAND AT HELENA, ARKANSAS

WAVE EQUATION ANALYSIS

by Ernest T. Mosley

Ernest T. Mosley assumed his present duties in the Civil Engineering Department of the Raymond Concrete Pile Division in New York in 1966. Before that he had worked for six years in the International Region in London in various capacities including sales, construction and engineering. Mr. Mosley first came to Raymond in 1956 as a Field Engineer and Assistant Job Superintendent. Later he became Job Superintendent and then Assistant District Manager in Kansas City. After that he served in various sales capacities in Syracuse and Detroit before being assigned to Pittsburgh as Assistant District Manager. Mr. Mosley is a graduate of the University of Texas (BSCE) and the University of Illinois (MSE) and a member of the American Society of Civil Engineers.

The Raymond Concrete Pile Division of Raymond International Inc. is currently making intensive use of wave equation analyses for pile driving jobs. The results of these analyses have contributed toward a better understanding of what happens to a pile when it is driven into the ground.

At the present time the most useful data provided by wave equation solutions are stresses in the pile and driving resistance (blows/inch) for various values of soil resistance. This soil resistance is termed "ultimate resistance (RU)" and is indicative of pile capacity for the corresponding final driving resistance. It is not always equal to pile capacity since the resistance some types of soil offer to pile penetration change after the time of driving. If it increases, the phenomenon is commonly called "freeze." If it decreases, it is known as "relaxation." One must use experience and judgment in recognizing these soil conditions and in properly modifying the results obtained by wave equation solutions.

Since the best way of gaining this experience is by comparing wave equation solutions with full scale load tests to failure, Raymond has set up a program to study case histories making use of its extensive technical files. These files include reports of load tests made by the Raymond Company over a period of seventy years.

One of these reports covers the test pile program conducted by Raymond under the supervision of Ebasco Services in 1958 near Helena, Arkansas. The purpose was to determine pile capacities for several types considered suitable as foundation elements for the Helena Steam-Electric Station for the Arkansas Power and Light Company. The ultimate soil resistance of each test pile as calculated by wave equation solution has been compared with the load test data. The results of these comparisons are presented and analyzed herein.

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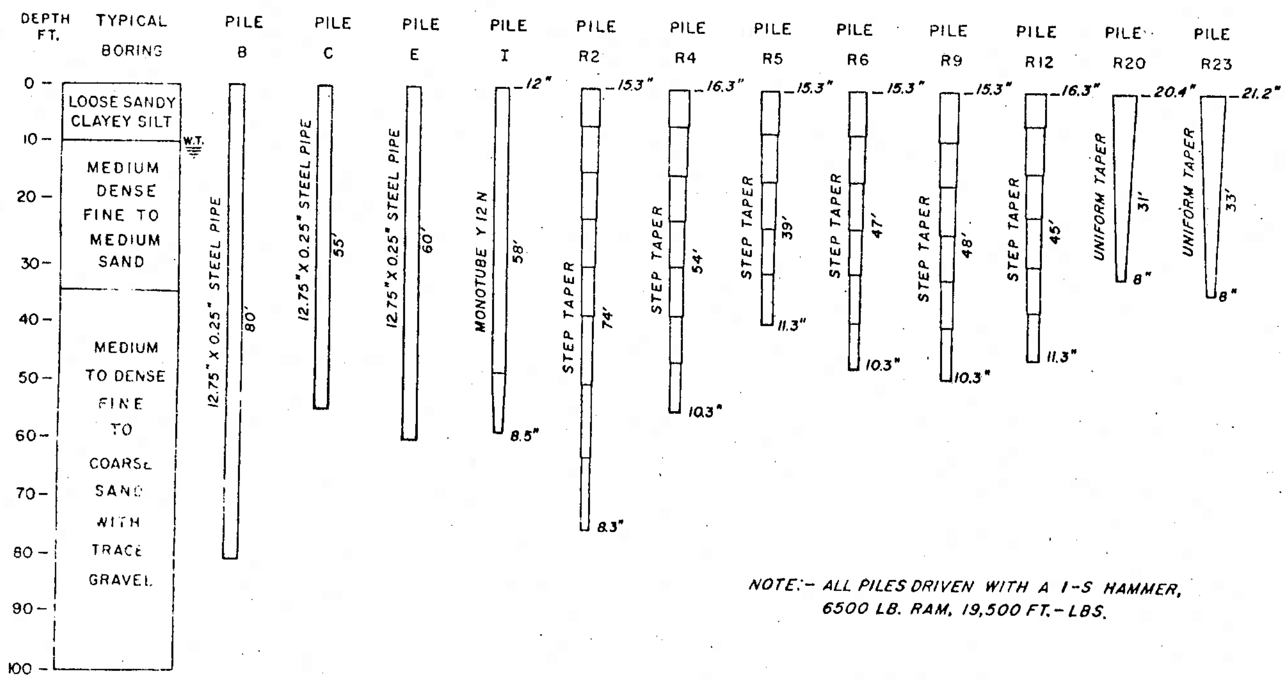

	B	C	E	I	R2	R4	R5	R6	R9	R12	R20	R23
FINAL DRIVING RESISTANCE BLOWS PER INCH	3	1½	2	3	4	6	3	4½	4	5	5	7
TEST LOAD FAILURE TONS	120	60	104	80	170	185	125	140	140	140	140	240
WAVE EQUATION ULTIMATE RESISTANCE (RU) - TONS	100	70	86	95	140	190	105	150	135	164	152	200

Fig. 1. Summary of piles tested to failure

(Continued from preceding page)

A summary of the piles which were driven and subsequently tested to failure is shown in Figure 1. The test program included several other piles which were not tested to failure. Therefore, these have not been included in this study. A typical soil boring is shown at the left for comparison. The soil into which the piles were driven is essentially a medium to dense, fine to coarse sand, grading more dense and more coarse with depth. All piles were driven with a No. 1-S steam hammer having a 6,500 pound ram, a rated energy of 19,500 ft-lbs. and using a standard 6" wood capblock with grains vertical and confined by a steel ring. Although details are shown in subsequent figures, this summary includes the following data for quick reference and comparison: (1) the final driving resistance in blows per inch, (2) the test failure loads as evaluated by Ebasco's engineers and (3) the wave equation ultimate resistance.

All twelve piles shown in Figure 1 are either steel piles or steel mandrel driven piles. Although they are essentially the same in this respect their lengths, point diameters, and configurations vary considerably. Their configuration varies from no taper up to 0.4" per lineal foot. Point diameters varied from 8" to 12.75". Pile lengths varied from 31' to 80'. Final driving resistance varies from 1½ blows per inch up to 7 blows per inch. A comparison of the test failure loads clearly shows that other factors control pile capacities than pile length alone.

A wave equation solution was made for each of these piles. The factors governing these solutions are hammer ram size and velocity at impact, capblock stiffness, pile weight and stiffness, pile length, soil quake, soil damping resistance and mode of soil resistance. For all solutions the same hammer, capblock, soil quake, soil damping resistance and mode of soil resistance were used. The hammer was assumed to strike at 80% of its rated velocity. The soil parameters used were those recommended by E. A. L. Smith in A.S.C.E. Transactions Paper 3306, "Piledriving Analysis by the Wave Equation," (quake = 0.10" and damping resistance = 0.15 sec/ft. at point and 0.05 sec/ft. at side). For the sake of uniformity the mode of soil resistance for all piles has been assumed to be 50% at the point and 50% in friction spread uniformly over the embedded length of pile below a depth of 10 feet. This would appear reasonable since the sand becomes more dense and coarse with depth. One could just as well have made more sophisticated solutions by varying the mode of soil resistance according to the piles' shapes and sizes, and perhaps also according to their driving logs. However, experience has shown that any reasonable variation from what has been used here would not have caused major differences in results.

64% efficiency

The relationship between wave equation ultimate resistance and driving resistance in blows per inch is shown in Figure 2 for each of the test piles. The higher group of curves represents the mandrel driven piles, the lower group represents the relatively light pipe and monotube piles. It is interesting to note that up to about three blows per inch driving resistance there is very little difference between the light-weight piles and the mandrel driven piles. The reason for this appears to be that the soil is so soft compared to the stiffness of the piles that the effect of the latter is insignificant. Of more importance is the fact that the pile is moved a considerable distance by each blow. Thus for easy driving pile weight (inertia) is the more significant factor. These wave equation curves show that for a specific soil resistance (and easy driving) the light-weight piles actually drive easier. However, as the driving resistance increases beyond three blows per inch the heavier piles drive

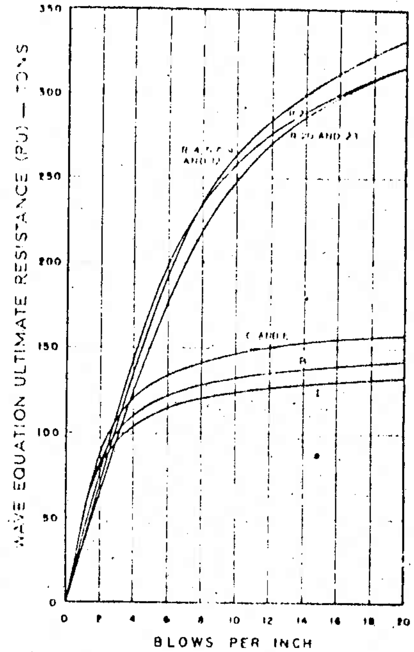

Fig. 2. Wave equation ultimate resistance vs. driving resistance

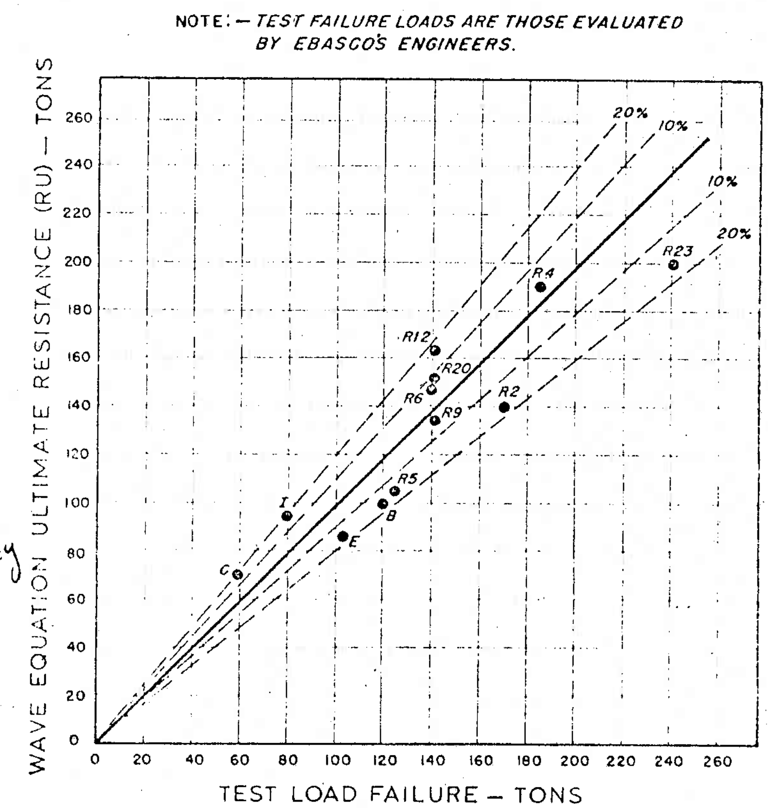

Fig. 3. Wave equation ultimate resistance vs. test load failure

easier for any specific soil resistance. The reason appears to be that for hard driving the soil is quite stiff compared to the stiffness of the piles. Thus, the relative stiffness of various types of piles is quite significant. Since the driving is hard the pile moves only a small distance with each blow of the hammer. Thus, pile weight (inertia) is not so significant for hard driving. On this particular job most of the piles were stopped at relatively low driving resistances. Thus, there is little difference between wave equation solutions for the two groups of piles.

A comparison of ultimate resistance calculated by wave equation solution with test failure loads is shown in Figure 3, from which it may be seen that in no case did the wave equation ultimate resistance vary from the actual test failure load by more than 20%.

The load-settlement curves for most of these piles did not show a definite load at which failure occurred. Refer to Figures 5 through 8. There are several different methods of selecting the failure load. The test failure loads as evaluated by Ebasco's engineers have been used in Figure 3 as a basis for comparing ultimate resistances (RU) calculated by wave equation solutions. The RU values are based upon the exact final driving resistance recorded for each pile. In order to appreciate the sensitivity of (1) final driving resistance and (2) selection of test failure load, Figure 4 has been plotted to show the effect on RU of a variation of plus or minus ½ blow/inch in final driving resistance, and the effect on the test failure load of using two arbitrary methods for its selection. The lower value has been taken as that load at which the slope of the load-settlement curve is 100 tons/inch. The upper value is the maximum estimated sustainable load without continued settlement. It can be seen that for low driving resistances the RU value is very sensitive to the final driving resistance.

These comparisons indicate that for the type of soil encountered at this site and for these types of piles the wave equation solution gives a reasonably good prediction of ultimate pile capacity without the necessity of making modifications for soil freeze or relaxation.

Of the factors used in these wave equation solutions the physical properties of the hammer, capblock, and piles are considered to be fairly accurate. However, the soil parameters are known to be no more than reasonable estimates. When more is known of the manner in which these parameters act under dynamic loading a more sophisticated wave equation solution can be made. In the meantime wave equation solutions can be usefully employed providing good judgment is used in their interpretation. The results of the case history studies referred to earlier are expected to contribute much toward this end. □

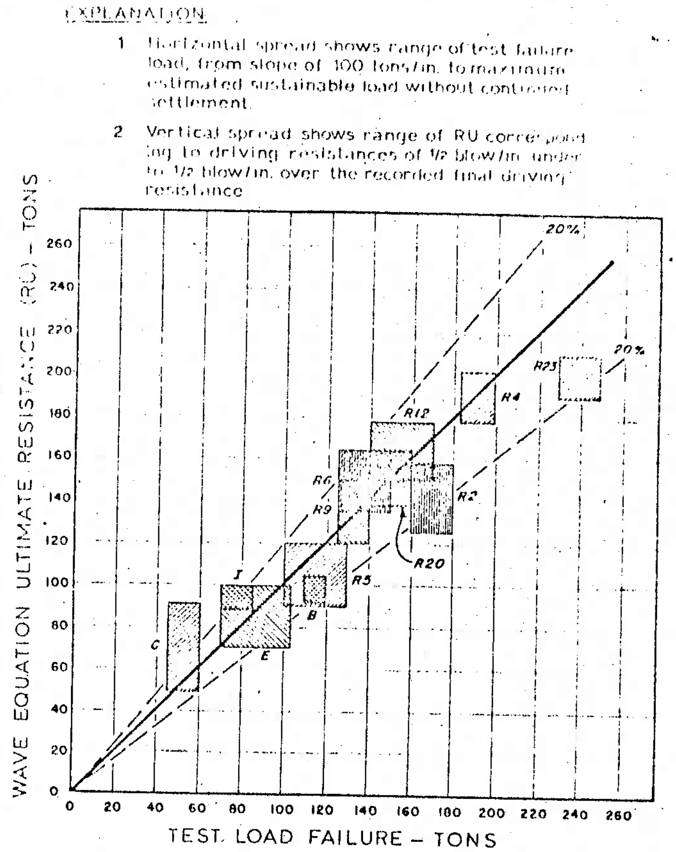

Fig. 4. Wave equation ultimate resistance vs. test load failure

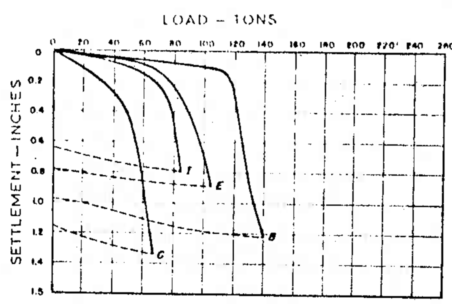

Fig. 5. Test load vs. settlement

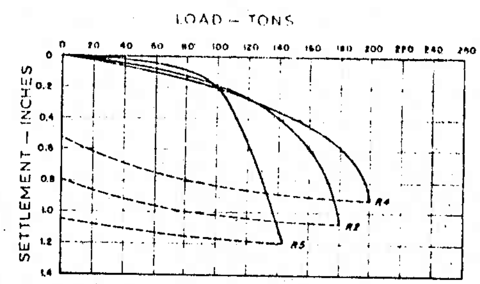

Fig. 6. Test load vs. settlement

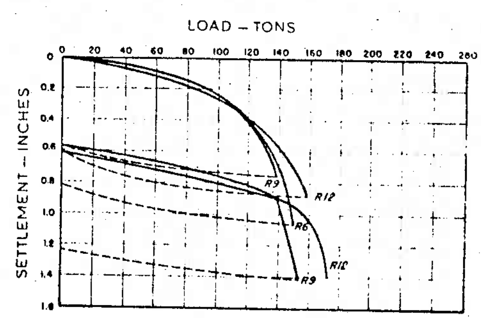

Fig. 7. Test load vs. settlement

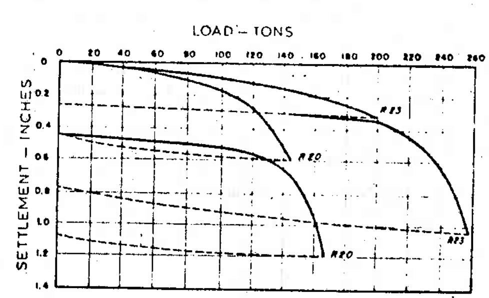

Fig. 8. Test load vs. settlement