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BEARING CAPACITY OF TAPER PILES AT MAADI SATELLITE PROJECT, CAIRO

BY

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ABSTRACT:

One of the largest pile driving jobs on which high capacity Raymond piles have been used in Egypt, that Maadi Satellite project, Maadi, Cairo. More than 4000 piles of step taper Raymond piles were driven by Misr-Raymond company, to support high rise towers.

This article analyiz the methods of prediction of load capacity compared to full scale tests and gives a useful recommendations.

INTRODUCTION:

The basic problem of computation of load capacity of a deep foundation can be formulated as follows: A cylindrical shaft is placed by some means to a depth inside a soil mass of known physical properties. A static, vertical, central load is applied at the top and increased until a shear failure in the soil is produced. To be determined is the ultimate load which this foundation can support.

If the surrounding soil is cohesionless silt or sand, pile driving may cause soil densification, which is most pronounced in the imediate vicinity of pile shaft and extends in gradually diminishing intensity over a zone extending between one to two pile diameters around the pile shaft. The driving process is also accompanied by increases in horizontal groung stress, and changes in vertical stress in pile vecinity. In very dense cohesionless soils, such as sand or gravel, loosening may take place in some zones, along with substantial grain crushing and densification in the imediate vecinity of the pile. (According to Kerisel (1962) some of the test piles in dense sand were excavated and pulled out with a hull of highly densified, crushed material that resembled a fine-grained sandstone). In such soils there are permenent changes in horizontal as well as in vertical ground stress that can be very pronounced. Hard driving can leave large residual stresses in both the pile and the soil, consideration of which may be essential for understanding the behaviour of the pile-soil system. As the piles are often made in groups the picture is further complicated by the complex and not always well understood effect of placing of adjacent piles. For these and other reasons the problem under consideration poses difficulties unparalleled in other common soil mechanics problems. A general solution of the problem is not yet available and will be hard to formulate and solve.

Soil Conditions:

The location of Maadi Satellite Project is about 10 Km south Cairo, along the Nile river. At that site the soil consists of medium claysy silt from the ground surface down to a depth of 1.5 m follows by loose, fine to medium sand to 14 m from surface then dense medium to coarse sand up to the end of borings 30 m from surface, ground water table was located about 1.5 m from surface. Fig. 1 shows the general soil profile.

Bearing Capacity Analysis by Wave Equation:

A thorough review of analytical solutions to the one-dimensional wave equation and their applications to impact pile driving problems is given elsewhere (2).

The general wave equation for a pile surrounded by a medium providing side and end resistance to the acceleration:

$$\frac{\partial^2 u}{\partial t^2} - \frac{E}{\rho}\frac{\partial^2 u}{\partial y^2} \pm R = 0$$

Initial conditions at $t = t_0$:

$$U(x,t) = U_o(x)$$

$$\frac{\partial u}{\partial t}(x,t) = V_o(x), \quad R(x,t) = R_o(x)$$

Boundary conditions:

$$U(x,t) \text{ or } \frac{\partial u}{\partial x}(x,t) \text{ at } x = 0$$

$$U(x,t) \text{ or } \frac{\partial u}{\partial x}(x,t) \text{ at } x = L$$

The exact solution of such a system of equations is available only for a few special cases.

The development of high-speed-digitel computers made solutions of large system easily accomplished.

The greatest succes in wave equation applications may be gained in the selection of the most effective driving system.

The major uncertainties that have been found are in usage of the method to determine pile bearing capacities and the related question of whether or not penetration is possible under very hard driving. The inaccurancies found in applying the wave equation solutions are generally attributed to the inadequency of the assumed rheological model of soil resistence. The nature of pile-soil interaction behaviour is extremely complex. As the pile penetrates, discontinous shear deformations develop at the pile tip and along the shaft. Simulataneously the material beneath the tip is compressed and displaced as the region adjacent to the tip is sheared and deformed in extension. As the pile penetrates further the soil along the shaft is severely sheared. Soil exhibit an extremely complex deformation and failure behaviour in the laboratory under well-controlled stress deformation conditions. Soil behaviour may be a function of stress/deformation history, streee level, streespath, and (particularly important for impact driving behaviour) deformation rate. The developmented end dissipation of excess pore fluid pressures effects the effective stresses, deformation and failure response of the soil. It is essential, therefore, to recognize that the determination of reprezentative soil parameters as input to a pile driving analysis is truly a crude exercize of engineering judgement.

Soil disturbence due to pile installation may drastically affect the resistance to penetration during driving and the static behaviour, on well. For sensitive clays the remolded strength may be substantially less than undisturbed" strengths such that a driven pile may shows very little resistance during driving or immediately after driving. A strength regain and, therefore, an increase pile capacity may develop as reconsolidation (excess pore pressure dissipation) of the disturbed soil proceeds with time. Strength regain may continue in some sensitive cohesive soils over several years as a result of consolidation and thixotropic effects. This strength regain as commonly described as pile set-up" or freeze". Dense deposits of fine cohensionless soils may develop negative pore pressures during pile driving, giving high transiant strengths.

A pile load test or subsequent redriving after these excess pore pressures dissipate often reveals a much lower resistance to penetration then the initial driving resistance would indicate.

The development of high resistance during high rate of deformation loading that do not prevail under static loading condition is commonly termed "relaxation". In many cases the actual soil resistance to penetration during driving bears little resemblance to the resistance observed during a load test performed after transient phenomena have passed.

Bearing Capacity For Soil Data:

The ultimate capacity P_u of the pile consists of skin friction P_s and point bearing P_p.

Consequently:

$$P_u = P_s + P_p$$

For calculation purposes it is generally assumed that the skin friction resistance and the point resistance can be determined separately and that these two factors do not affect each other. Test results reported by Cambefort (1953), Kezdi (1957) and Stuart, Hanna and Naylor (1960) show however that the skin friction resistance affects the point resistence for piles which have been driven through cohesionless soils. However this influence is in most cases small and can be neglected. Very small axial deformations are generally necessary to mobilize completely the skin friction resistance along a pile as observed, among others, by Muller (1939), Schenck(1951), Zweck (1953), D Appolonia Romualdi (1963), D Appolobia Hribar (1963) and Weele (1964).

In contrast relatively large deformations are required to mobilize the maximum point resistence of piles which are driven into cohesionless soils. Therefore the largest part of the applied loads while at high load levels the largest part is carried by point resistence (Mansur & Kaufman, 1958, Mohan, Jain & Kumar, 1963).

Skin resistance:

The skin resistence has been evaluated, since the turn of this cetury, similar to the resistance to sliding of a rigid body in contact with soil. For sand this implies an assumption that should be proportional to the everage overburden pressure alonge the skin.

$$P_s = A_s . K_s . \tan\delta \; . q_s \; .....$$

A_s = area surface.
δ = friction angle pile/soil
q_s = overburden pressure.

In which K_s is dimensionless factor represents the ratio of effective horizontal to overburden stress, and ranging between active to passive pressure.

Point resistance:

It has been suggested by Caquot (1934) and Buisman (1935) that the point resistance of piles in sand should be proportional to initial overburden pressure at the level of the pile point:

$$P_p = A_p . q . N_q \; ....$$

A_p = area of pile point
q = overburden at tip.

In which N_q represents the bearing capacity factor for a deep foundation. Numerous theorstical and semi-emperical curves for N_q as unique function of the angle of internal friction $\emptyset$ have been proposed.

Bearing Capacity Analysis by Nordlund Equation:

Nordlund (1963) has developed an empirical method which takes into account the volume of soil displaced by the pile, the material of the pile, and the shape of the pile. This equation not valid for a pile for which jetting or predrilling as in this study.

$$P_u = N_q A \, P_D + \sum_{d=o}^{d=D} K_\delta \, P_d \, \sin\delta \;\; C_d \, \Delta d$$

in which P_u = ultimate bearing capacity, N_q = dimensionless factor
A = Area of pile point, P_D = overburden pressure to a depth D
K_δ = dimensionless factor, δ = friction angle pile/soil
C = perimeter encompassing the pile.

Load Tests:

A serial of load tests (15 tests) were conducted; lengthes of piles from 11 m. to 21 m. driven to refusal according to wave equation some of load tests were done on redriven and retaped piles. All tests were done according to ASTM designation.

Ultimate Load Criterion:

Various ultimate load critria, all empirical in nature, have been proposed and used by different researchers and design organizations (see, for example Chellis, 1964); a survey is presented in Table 2.

The auther suggested to use criterion 1b in the following form: Unless the load-settlement curve of a pile shows a definite peak load, the ultimate load is defined as the load causing total settlement of the pile point equal to 10% of the point diameter.

Computation of Ultimate Loads:

Ultimate loads of all tests were computed by all methods except Norllund and listed in Table 1.

Interpretation of Test Results and Conclusions:

By using a safety factor 3 to get the working load from soil data and a safety factor 1½ to get the working load from load test, and comparing all results with the wave equation load, it observed that the working load from soil data is conservative in general, and the results of wave equation are nearer to observed.

A considerable amount of research including well instrumented observation on full size tests are needed for safer and more economical design in future.

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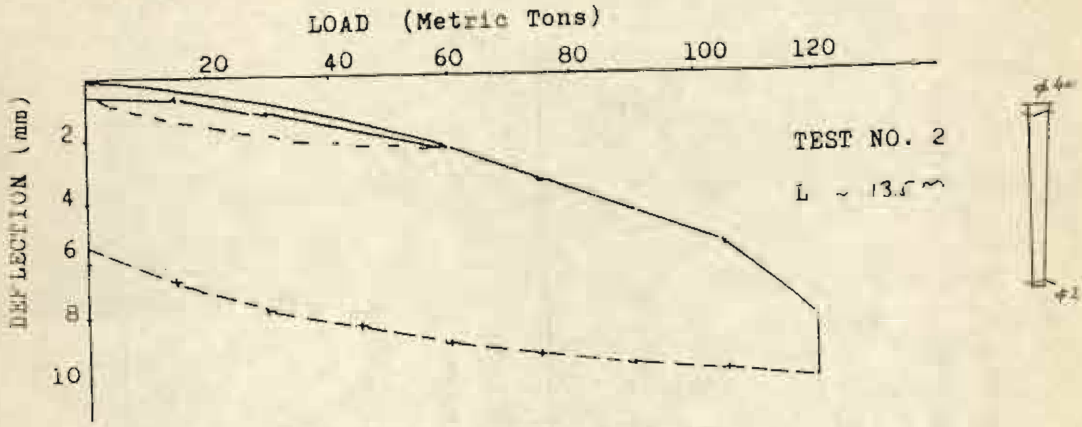
Table 1: Comparisson of observed and computed bearing capacity.

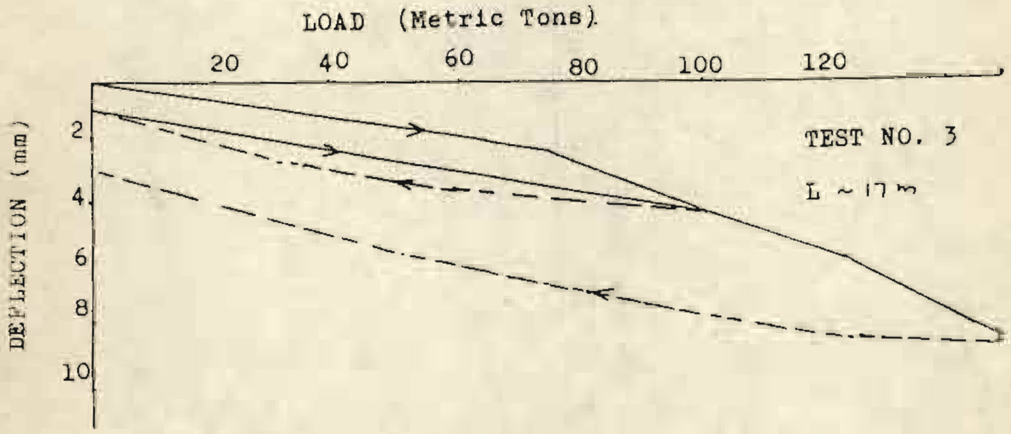
Test No.	Wave equ. working load t.	Ultimat load from soil data t.	Observed ultimat load t.	Working load from soil data t.	Observed working load t.
2	60	192	120	64	79
3	100	240	135	80	89
4	60	213	90	71	60
5	100	300	200	100	130
7	100	300	200	100	130
8	100	280	200	94	130
9	100	255	150	85	100
11	100	225	150	75	100
12	60	155	95	52	63
13	100	249	150	85	100
14	100	200	150	67	100
15	100	220	150	73	100

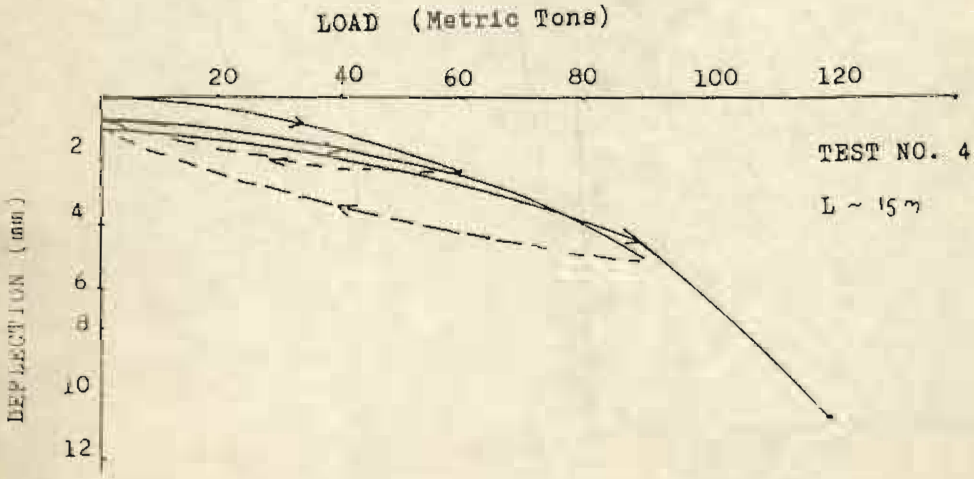
Fig. 1

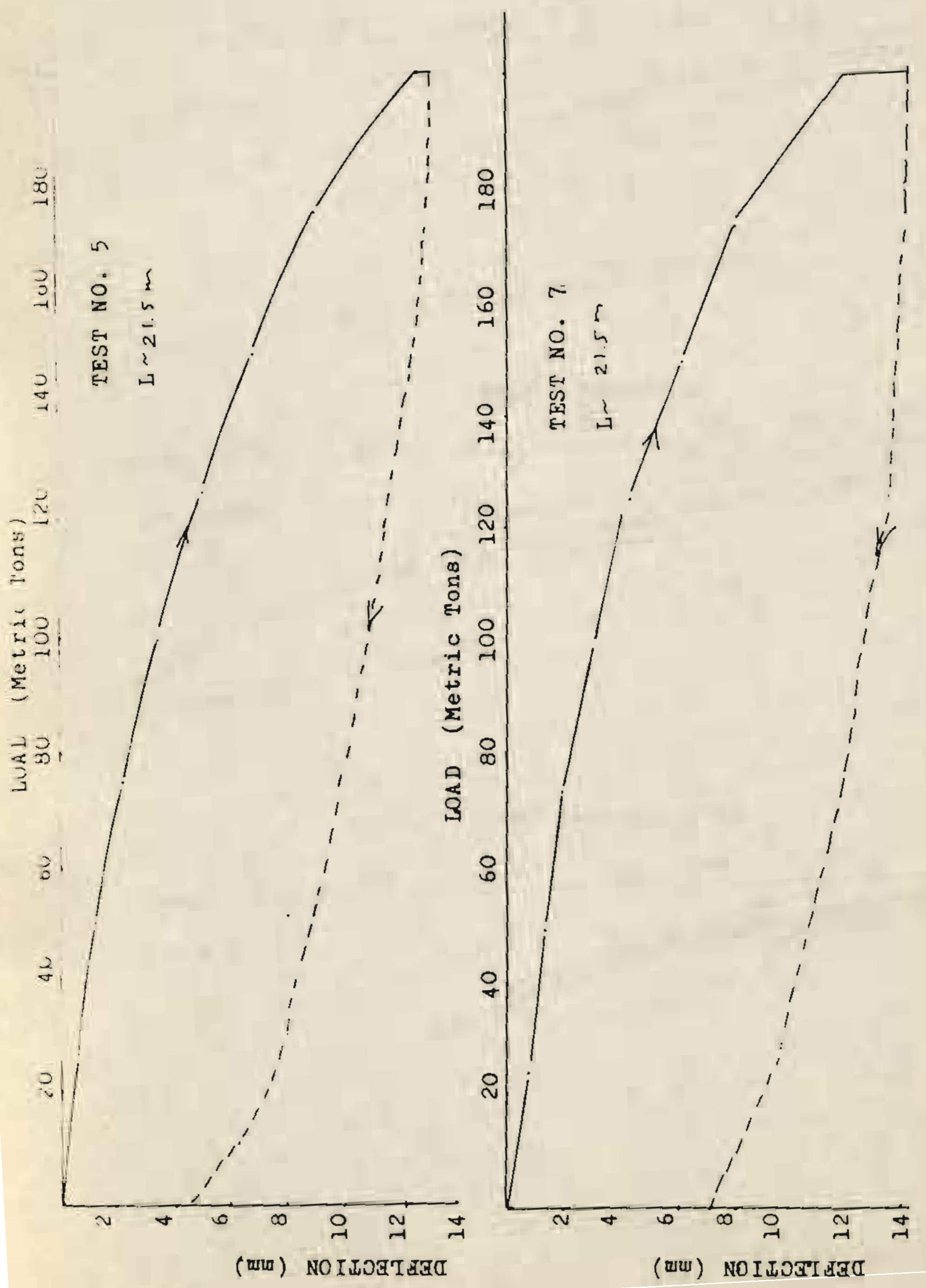
GENERAL SOIL PROFILE

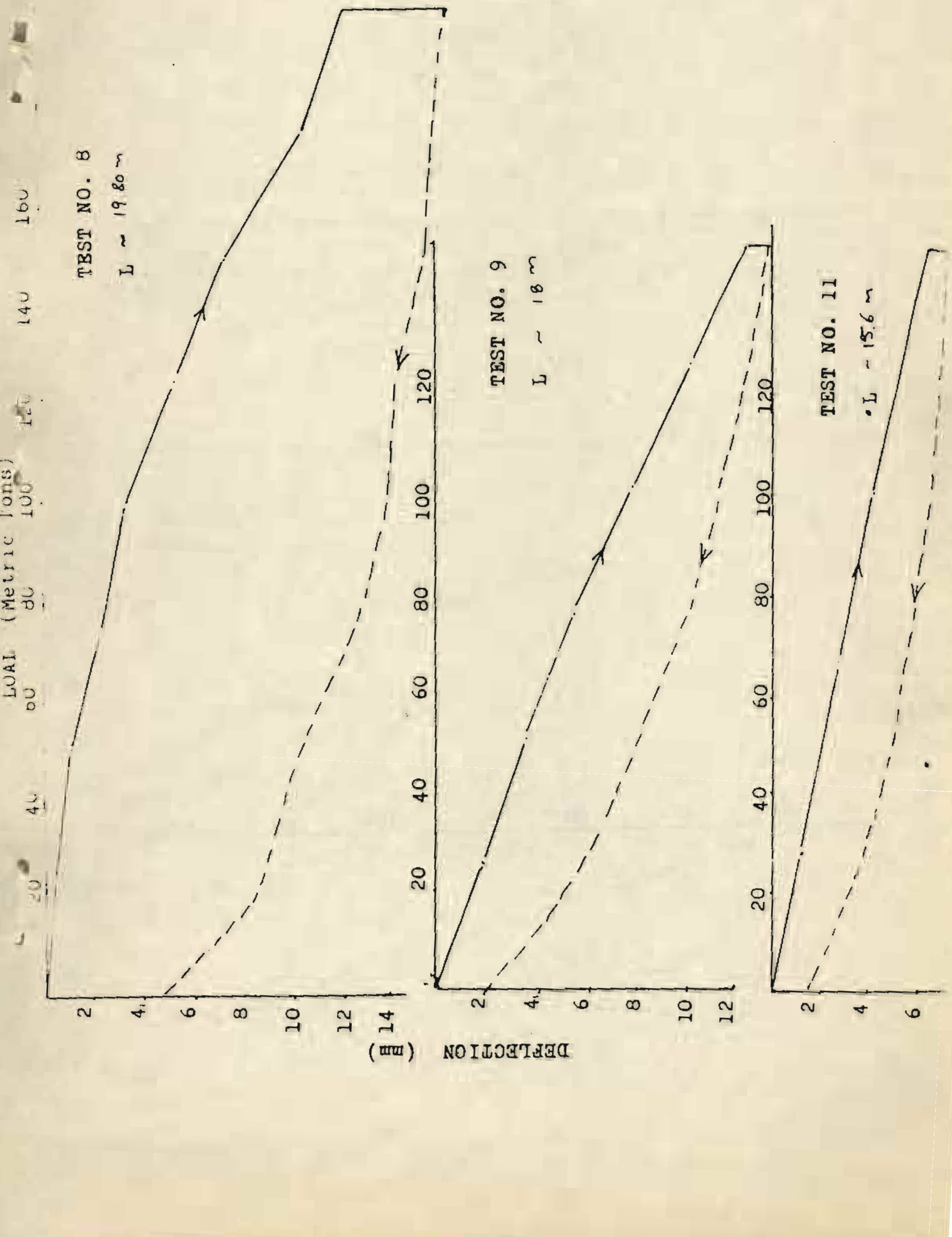
Depth m	SPT	Strata Log	Strata Elev.	Description	Water Table	Remarks
1	8, 10			Dark gray med. clayey silt.		
2, 3, 4	8			Brownish gray loose, fine to med. sand.		
5, 6, 7, 8	6, 10, 9, 6			Grayish br. loose, f. to med. sand.		
9, 10, 11, 12, 13	8, 10			Br. gray loose, med. to coarse sand, tr. of f. gravel.		
14, 15, 16	22			Grayish br. med. to c. sand.		
17, 18, 19, 20	17, 24			Gray med. to f. sand.		
22, 24				Gr. br. med. to c. sand.		
26	16, 20			Br. gr. med. to c. sand.		
28				Br. gr. dense, med. to c. sand.		

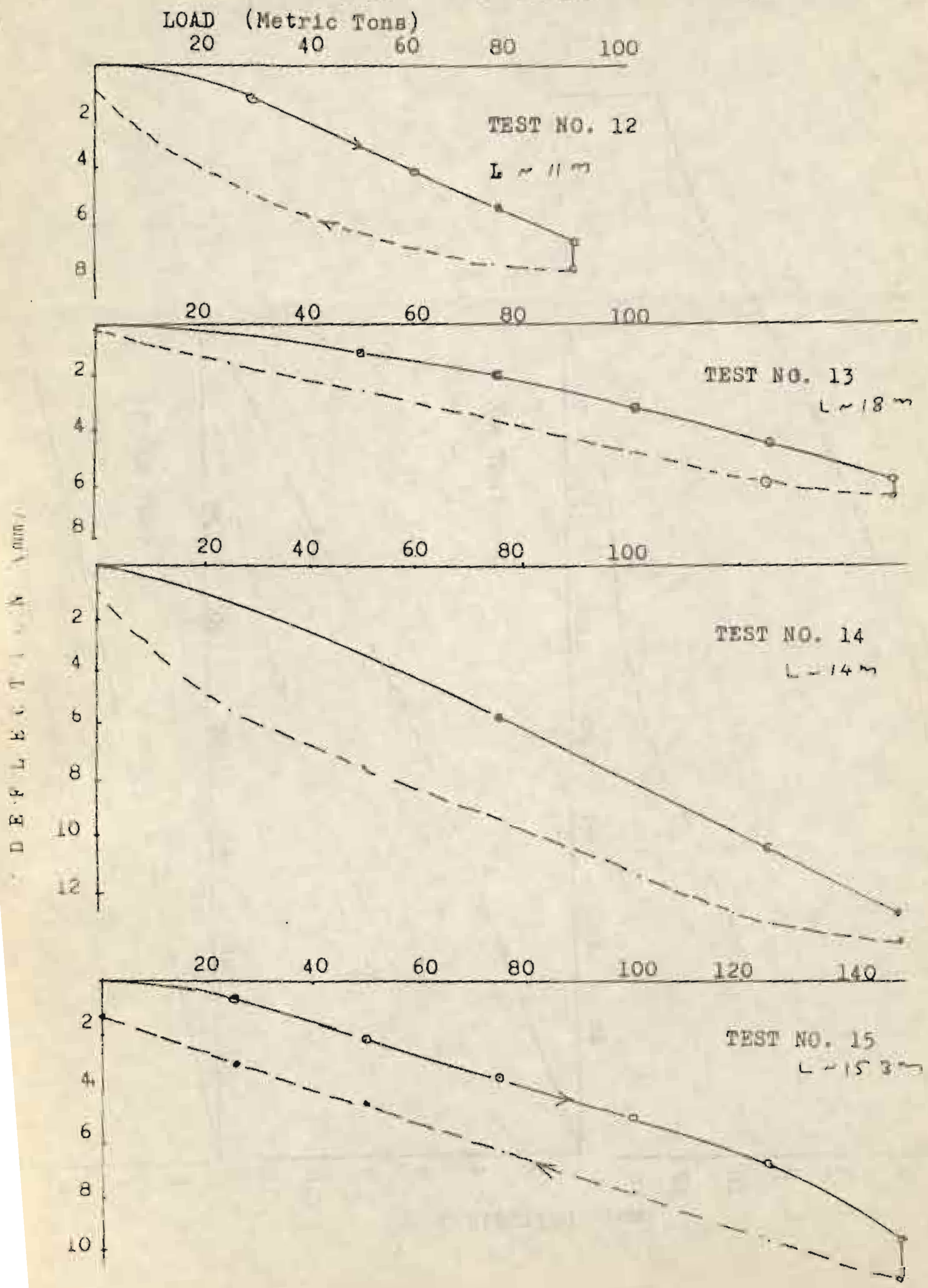

LOAD (Metric Tons)
20 40 60 80 100 120
DEFLECTION (mm)
2 4 6 8 10
TEST NO. 2
L ~ 13.5 m


LOAD (Metric Tons)
20 40 60 80 100 120
DEFLECTION (mm)
2 4 6 8 10
TEST NO. 3
L ~ 17 m


LOAD (Metric Tons)
20 40 60 80 100 120
DEFLECTION (mm)
2 4 6 8 10 12
TEST NO. 4
L ~ 15 m


TEST NO. 5
L ~ 21.5 m
LOAD (Metric Tons)
DEFLECTION (mm)
TEST NO. 7
L ~ 21.5 m
LOAD (Metric Tons)
DEFLECTION (mm)


LOAD (Metric Tons)
DEFLECTION (mm)
TEST NO. 8
L ~ 19.80 m
TEST NO. 9
L ~ 18 m
TEST NO. 11
L ~ 15.6 m


LOAD (Metric Tons)
TEST NO. 12
L ~ 11 m
TEST NO. 13
L ~ 18 m
TEST NO. 14
L ~ 14 m
TEST NO. 15
L ~ 15.3 m