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RESEARCH IN SOIL MECHANICS
AT THE UNIVERSITY OF MANCHESTER.

YOUR REF.

MELDONS,
FLETSAND ROAD,
WILMSLOW,
CHESHIRE.
TEL: WILMSLOW 3093.

30th March, 1963

Messrs. Edgar Morton & Partner,
Consulting Engineering Geologists,
Styperson House,
Adlington,
Macclesfield.

Dear Mr. Morton,

Proposed Storage Reservoir at
Havant Thicket

I write to report on the results of the soil mechanics tests and their influence on the design and construction of an earth embankment 60 ft high.

1. A report of the numerical values of the soil mechanics tests carried out at the University is enclosed. You will also have the results of tests made by Messrs. Pre-Piling Surveys Ltd.
2. Considering first the strength of the London Clay on which the dam will be founded, Fig.1 shows the shear strength of undrained undisturbed samples plotted against depth. The results of tests made initially at the standard rate of 2% per minute have been neglected. The presence of fissures in the London Clay requires a slower rate of test in order to ensure uniformity throughout the sample and consequently a rate of 1% per hour was used for the majority of the tests and Messrs. Pre-Piling Surveys Ltd were instructed to do likewise.
3. Considering an average Downstream slope of $2\frac{1}{2}/1$ as might arise with a slope of $2\frac{1}{2}/1$ and a berm, the bank would fail at a shear of $6\frac{1}{2}$ p.s.i., and a strength of 10 p.s.i. would be required to ensure an adequate factor of safety of 1.5. It follows that either the bank must have an extended toe or the foundation requires consolidation during construction by means of sand drains. From Fig.1 the minimum depth of the drains would be 18 ft.
4. The presence of thin layers of fine silt in the laminations of the clay, as noted in paragraph 7(b) of your report dated 18th Dec. 1962, and as observed in the samples tested, could result in a spread of pore water pressure towards the toe. Consequently the use of a minimum number of sand drains through these silt layers is essential.
5. In order to design a drainage system and subsequently to provide adequate control during construction, measurements were made of the effective strength parameters of the foundation clays. Fig.2 shows the angle of shearing resistance in terms of effective stresses, ϕ' , plotted against the height above the Basal Beds. The commonly quoted value for London Clay is 20° and this value was found in places near the surface. With depth the value of ϕ' at this site was found to increase to over 30° . The average value of the cohesion intercept c' for all samples was 300 p.s.f. Both Figs 1 and 2 illustrate that the minimum strength of the foundation clays occurs just below the surface. Exceptionally low values of ϕ' of 13° have been found at B.F.4. The previous minimum value which I have met anywhere has been 15° and this is a matter for further investigation.

6. Fig.3 shows the observed pore water pressures at the peak effective stress ratio for a range of possible values of the major principal total stress. The results from Figs 2 and 3 have been used to make an effective stress analysis. This shows that, in the absence of drainage, the water pressure in the foundation would rise above the level of the bank near the crest, and that the factor of safety would be just below unity on completion. While this calculation supports that made using the undrained shear strengths it also indicates that 60% of the pore water pressure must be dissipated under the full 60 ft height of the bank in order to achieve a factor of safety of 1.5.

7. Owing to the presence of fissures and laminations the necessary spacing of the drains may be wider than that predicted on the basis of consolidation tests on small samples. The measured values of the coefficient of consolidation of samples tested under horizontal drainage in the pressure range to be expected from the bank lay between 25 and 85 ft²/yr. Below 15 ft depth the presence of silt layers should increase the real speed of pressure dissipation. The necessary spacing also depends on the value of ϕ' and on the rate of construction. On the basis both of calculations and of observations and experience on other sites of a similar nature I would suggest at this stage that drains be installed at 30 ft centres over the full depth of the London Clay, with intermediate drains at 15 ft centres to a depth of 15 ft below the surface.

8. Where the height of the dam exceeds 50 ft the drains should penetrate through the Basal Beds. They may then also control seepage uplift if the permeability of the Basal Beds is sufficiently low. This area applies to some 600 ft width in the centre of the valley. Between 50 and 35 ft height the depth of the deeper drains may be progressively reduced to 15 ft at a bank height of 35 ft. At this height the area is some 1400 ft wide. Elsewhere no foundation consolidation is necessary.

9. In view of the low strength of the surface clays it is nevertheless advisable to continue the use of a ground drainage blanket over the whole area of the seat of the bank, independent of its height.

10. In Figs 4 and 5 I have indicated the sort of design which meets the conclusions of your report and the present findings. In order to avoid the use of a deep cut-off trench through the Basal Beds and in order to employ sand drains over the foundation in the central area of the site, it is necessary to seal the clay core into the clay foundation near the Upstream toe. A horizontal clay blanket joins the central clay core to the partial cut-off. The latter is shown carried down 14 ft on the side of an open excavation.

11. Considering now the fill within the bank, the clay core, blanket and cut-off could be composed of London Clay. The surface weathered clays may have shear strengths between 5 and 14 p.s.i. and at their mean strength of 10 p.s.i. should be soft enough to form an impermeable seal and yet firm enough to be laid by mechanical plant. However clays at strengths less than 8 p.s.i. may have to be rejected, being too soft for mechanical plant. I am particularly concerned to find natural water contents as much as 14% wet of the Proctor Optimum, see item (4) of the University soil test report. Also Fig.6 shows that the low density and high water content at Optimum for these weathered London clays come outside the range of materials used on many other sites. This in itself is not necessarily a disadvantage but together with the low strengths and the high Liquid Limit values in the region of 72% prove the necessity for special care in the use of these materials.

12. Where silt laminations occur, containing water, the borrow material will soften after extraction. This may be the cause of the high water contents found on samples from B.H.s 4 and 10, with the exception of B.H.10, 1 $\frac{1}{2}$ -3 ft which may be locally dry at the surface. These materials would be quite unsuitable as main fill material. Furthermore the exceptionally low values of ϕ' would require flat slopes even with drainage. In this connection the test results on compacted samples by Messrs. Pre-Piling Surveys Ltd yield improbable values of ϕ' of 90.

13. For these reasons I have asked for the undisturbed samples obtained from the Reading Beds in order to investigate their suitability for use as the main rolled fill. They were not previously investigated owing to their depth below the surface near the seat of the bank. However they might be won from an area North of your line AA on Plan E. The detailed placement of these and the Basal Bed materials within the bank will in any event depend on your estimate of the relative quantities available and on considerations of haulage.

14. At this stage therefore the provisional profiles, Figs 4 and 5, indicate the main considerations to be borne in mind namely,

- 1) The use of a partial upstream cut-off.
- 11) The use of drainage by sand drains and drainage blankets to control water pressures in the fill and foundation, owing to the wet nature of these clays and their low drained strengths.
- 111) In order to allow time for the drainage to operate it may well prove necessary to restrict the raising of the central area of the embankment to 45 ft in the first season.

15. During construction it is essential that water pressures in the foundation and in the fill be measured by means of piezometers and at this stage I estimate that a total number of 25 to 30 on one section would be required.

I hope that this information may prove sufficient to allow general progress to be made with the scheme. Prior to the finalisation of contract drawings and the specification you may feel it would be advisable to have a number of pits opened in the borrow areas which we could inspect and if necessary test in greater detail.

Yours sincerely,



Encl. 5 sheets of diagrams
Soil test report.

The Department of Engineering,
The University of Manchester,
Oxford Road,
Manchester, 13.

March, 1963.

Soil Mechanics Test Report on Undisturbed Samples from
Havant Thicket Reservoir Site, Portsmouth and Gosport Water Co.

1. Standard Undrained Triaxial Compression Tests on $1\frac{1}{2}$ " Dia.
Samples using frictionless end platens. Testing rate 2%
strain per minute. Fissured London Clay.

B.M.	Depth ft	Water Content %	Density p.c.f.	Cell Pressure p.s.i.	Deviator Stress p.s.i.	Mean shear strength p.s.i.
1.	1 $\frac{1}{2}$ -3	32.1 31.4 31.6	121.5 121.0 120.0	0 20 40	17.8 31.5 23.8	12.1
1.	11 $\frac{1}{2}$ -13	28.7 29.0 28.1	124.5 122.2 124.0	0 20 40	11.4 16.7 14.8	7.1
1.	26 $\frac{1}{2}$ -28	27.6 25.6	125.0 125.0 126.0	0 20 40	14.3 19.0 29.0	10.4
1.	31 $\frac{1}{2}$ -33	26.0 26.8	124.5 126.0 124.5	0 20 40	15.5 16.7 32.0	10.7
1.	36 $\frac{1}{2}$ -38	27.8 27.6	124.5 124.5 126.0	0 20 40	15.2 25.3 31.5	12.0
1.	41 $\frac{1}{2}$ -43	27.9 25.6 25.8	127.0 125.5 127.5	0 20 40	23.8 45.4 18.1	14.5

As above but rate reduced to 0.0006" per min. and with filter
papers to speed distribution of water pressure.

1.	46 $\frac{1}{2}$ -48	25.6 25.2 26.1	129.0 130.0 129.0	20 40 60	30.4 29.6 34.0	15.7
2.	1-2 $\frac{1}{2}$	35.0 35.0 31.6	118 117 119	20 40 60	10.0 12.9 14.2	6.2
2.	6 $\frac{1}{2}$ -8	30.1 29.5 28.3	121 121 121	20 40 60	26.6 19.1 23.9	11.6
2.	13 $\frac{1}{2}$ -15	31.7 31.8 32.0	122 122 121	20 40 60	22.2 20.0 14.2	9.4

1) Continued.

B.H.	Depth ft	Water Content %	Density p.c.f.	Cell Pressure p.s.i.	Deviator Stress p.s.i.	Mean Shear Strength p.s.i.
2	16 $\frac{1}{2}$ -18	29.1 28.1 28.1	124 124 124	20 40 60	15.2 19.1 17.0	8.5
2	21 $\frac{1}{2}$ -23	29.2 28.0 27.5	125 127 125	20 40 60	27.5 33.8 27.1	14.7
2	26 $\frac{1}{2}$ -28	23.7 25.6 24.5	125 123 124	20 40 60	25.1 29.0 27.5	13.6
4	1 $\frac{1}{2}$ -3	35.6 36.1 35.0	117 117 116	20 40 60	16.0 8.5 10.5	5.8
4	4 $\frac{1}{2}$ -6	29.8 32.1 29.8	121 121 121	20 40 60	12.8 12.8 15.2	6.8
4	7 $\frac{1}{2}$ -9	31.2 31.6 31.5	120 121 121	20 40 60	15.8 24.8 23.5	10.7
4	13 $\frac{1}{2}$ -15	26.6 26.5 27.4	122 123 122	20 40 60	28.6 28.2 26.6	13.9

2. Consolidated-undrained triaxial compression tests with pore water pressure measurement on 1 $\frac{1}{2}$ " samples. Porous discs and frictionless end platens used with filter papers. Dead loading to equilibrium. Five hours to failure.

B.H.	Depth ft	Water Content %	Cell Pressure p.s.i.	Deviator p.s.i.	Sum p.s.i.	c' p.s.i.	ϕ'
4	4 $\frac{1}{2}$ -6	33.0	10 30 50	12.3 21.7 29.1	28.1 67.1 109.1	3.1	13.0
4	7 $\frac{1}{2}$ -9	32.0	10 30 50	16.1 25.8 33.4	31.1 70.2 109.6	4.6	13.5
10	6 $\frac{1}{2}$ -8 Remoulded		10 30 50	12.1 26.0 41.8	28.3 67.2 109.6	1.0	21.1

3. Oedometer Consolidation Tests on 3" Dia. Samples taken in a Horizontal Direction and using Single Drainage Direction over 1" length.

B.H.	Depth ft	Water Content %	Pressure tons/ft ²	Strain	Coefficient of Consolidation. ft ² /yr	
2	6 $\frac{1}{2}$ -8	30.2	$\frac{1}{2}$ 1 2 4	0.0032 0.0045 0.0113 0.0290	85 30	Fissured Brown Clay
2	13 $\frac{1}{2}$ -15	28.2	$\frac{1}{2}$ 1 2 4	-0.0008 0.0011 0.0077 0.0208	48 25	Fissured and Laminated Brown Clay
2	16 $\frac{1}{2}$ -18	30.2	$\frac{1}{2}$ 1 2 4	0.0069 0.0105 0.0190 0.0350	47 32	Ditto
4	7 $\frac{1}{2}$ -9	30.5	$\frac{1}{2}$ 1 2 4	0.0070 0.0112 0.0243 0.0392	250 31	Ditto
4	13 $\frac{1}{2}$ -15	29.4	$\frac{1}{2}$ 1 2 4	0.0008 0.0030 0.0110 0.0278	500 86	Friable Brown Clay and Yellow Patches.

4. Standard Proctor Compaction Tests

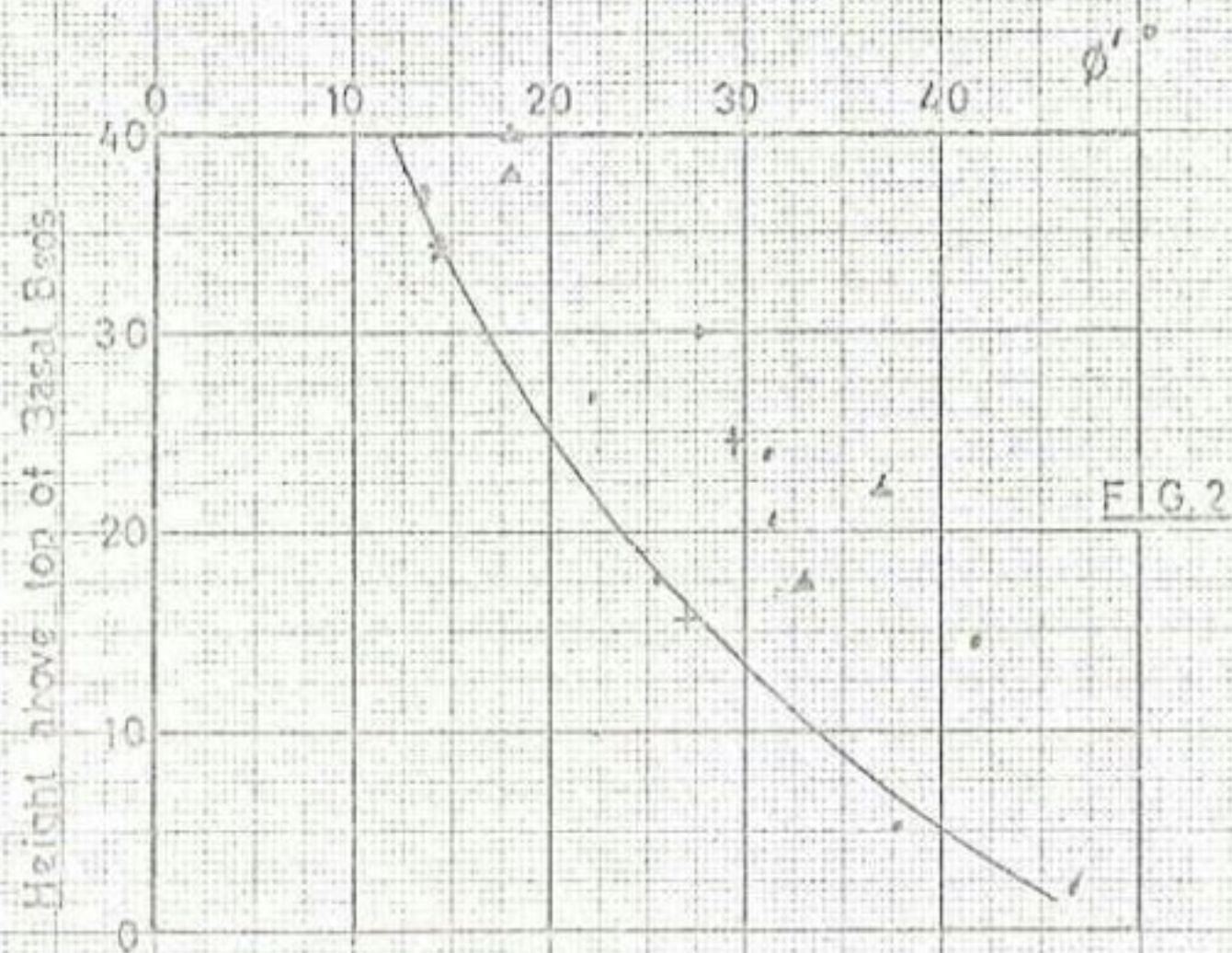
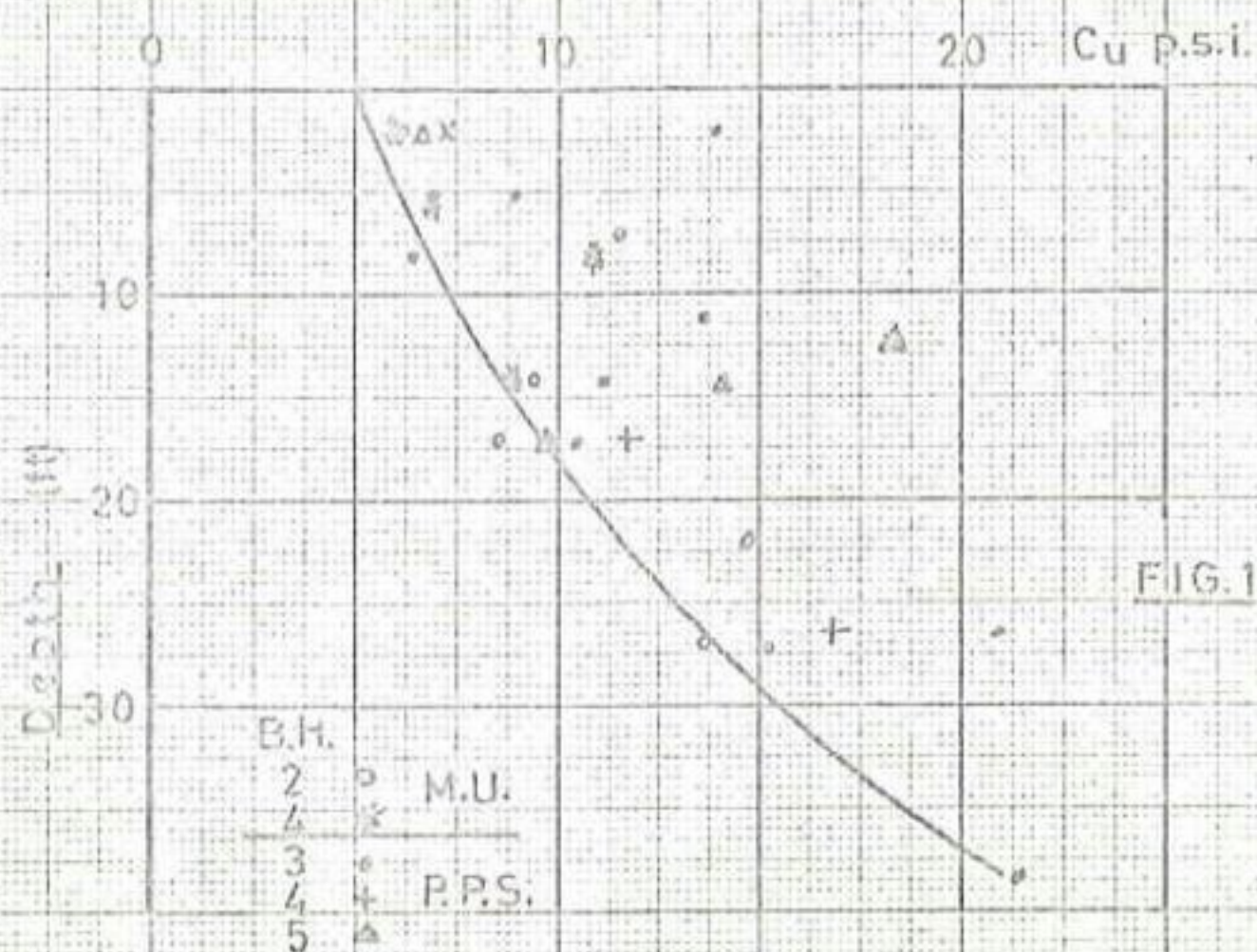
B.H.	Depth ft	Water Content %	Dry Density p.c.f.	Optimum Values % p.c.f.		Natural Water Content %
4	1 $\frac{1}{2}$ -3	22.3 23.2 24.5 26.0 28.8	90.0 92.8 93.4 92.5 88.5	24.5	93.4	36.5
4	4 $\frac{1}{2}$ -6	20.3 23.4 23.5 25.1 27.4 31.3	89.9 94.9 95.8 95.5 93.1 87.2	24.0	96.0	30.1
10	1 $\frac{1}{2}$ -3	20.0 22.5 24.5 27.5 28.8	96.0 97.0 96.2 93.5 90.8	24.5	96.2	26.9

4) Continued.

B.H.	Depth ft	Water Content %	Dry Density p.c.f.	Optimum Values % p.c.f.		Natural Water Content %
10	6 $\frac{1}{2}$ -8	19.4 21.4 24.7 26.6 30.1	97.4 98.0 96.8 94.3 89.3	21.4	98.0	35.1 to 42.5
Brown Clay Fissured and Laminated						
10.	16 $\frac{1}{2}$ -18	18.4 20.4 24.8 27.2 30.6	95.4 96.8 98.0 93.5 88.5	22.5	97.2	38.1 to 39.4
As above						

5. Identification Tests.

B.H.	Depth ft	Liquid Limit %	Plastic Limit %	% Clay %	Natural Liquidity Index
10	1 $\frac{1}{2}$ -3	72	28	-	-0.02
10	6 $\frac{1}{2}$ -8	63	26	46	0.25 - 0.45
10	16 $\frac{1}{2}$ -18	73	27	51	0.20 - 0.27
Disturbed sample of Reading Bed Red Clay		59	22		



FOUNDATION SHEAR STRENGTHS

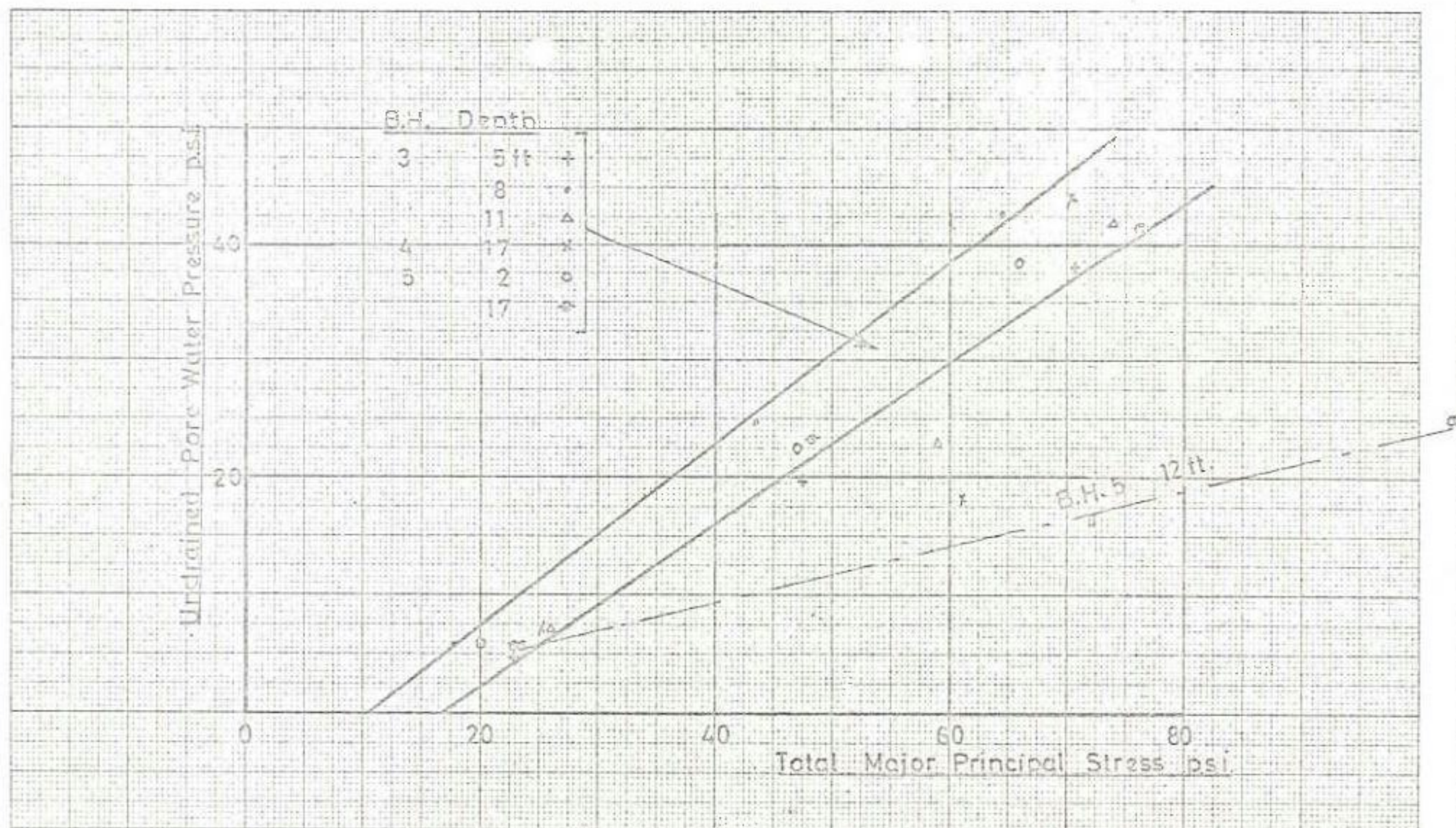


FIG. 3.
FOUNDATION PORE PRESSURES

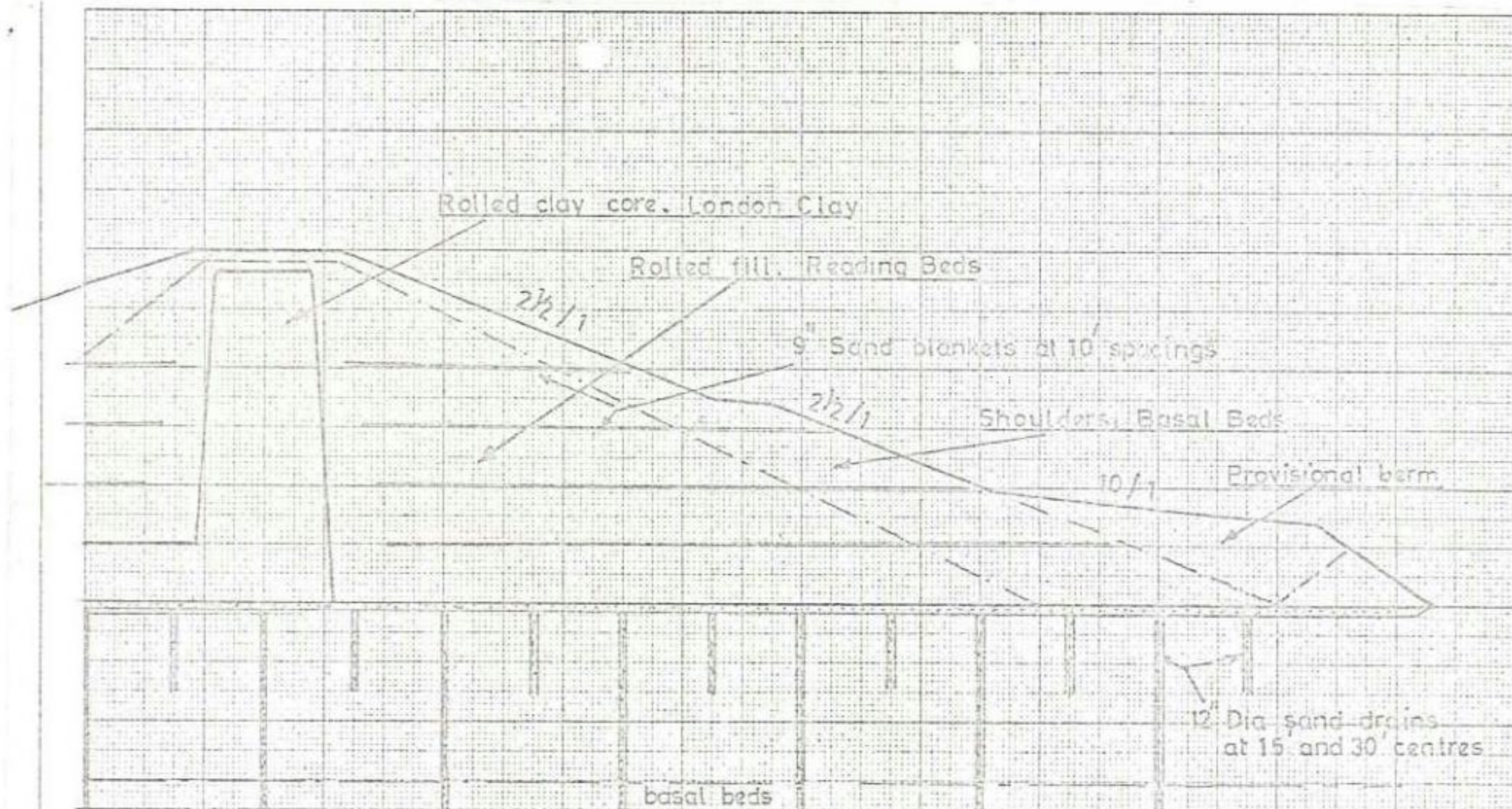


FIG. 4 HAVANT THICKET RESERVOIR. PROVISIONAL DOWNSTREAM PROFILE

Scale 1 inch = 20 feet

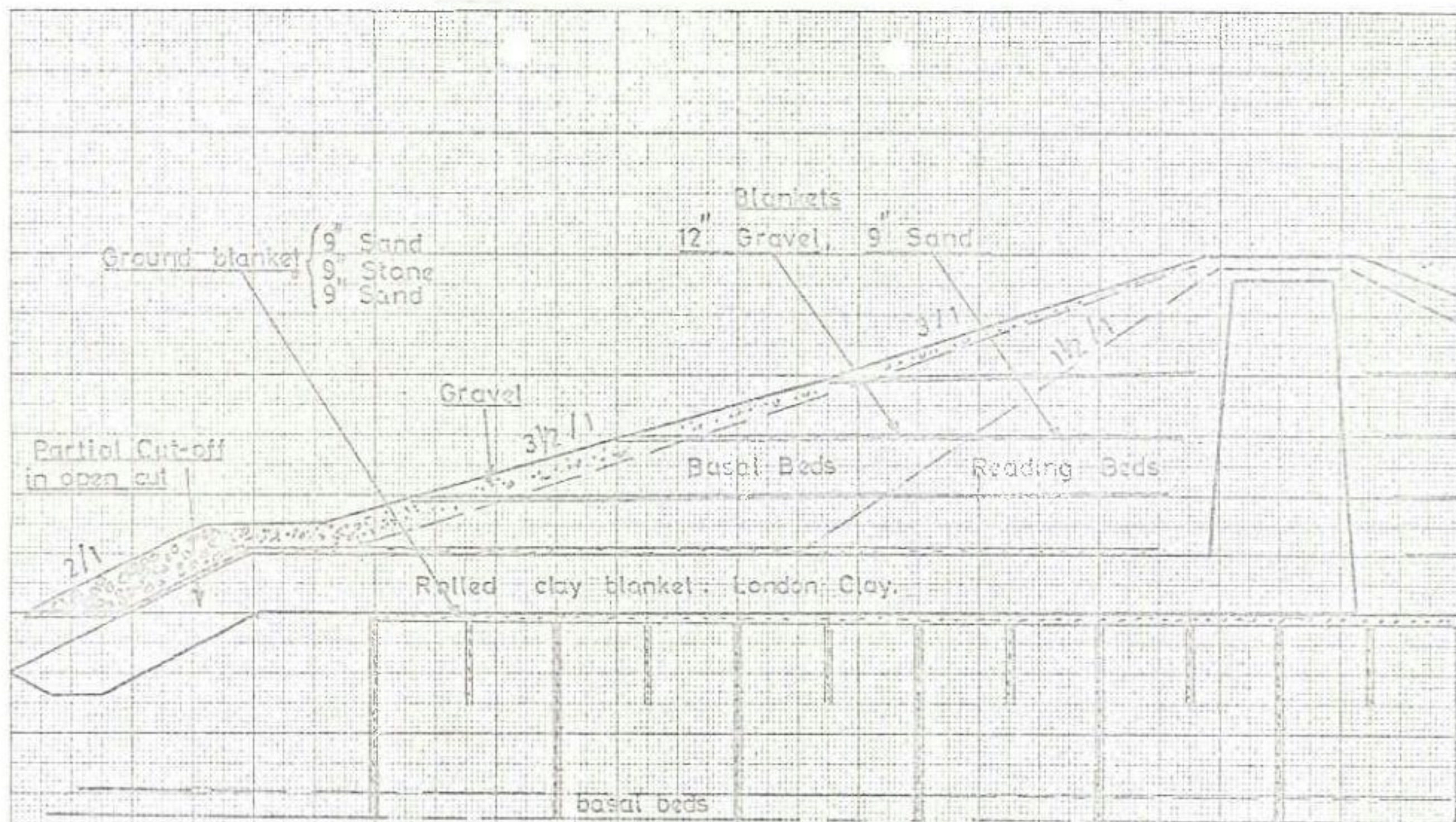


FIG. 5 HAVANT THICKET RESERVOIR. PROVISIONAL UPSTREAM PROFILE

Scale 1 inch = 20 feet

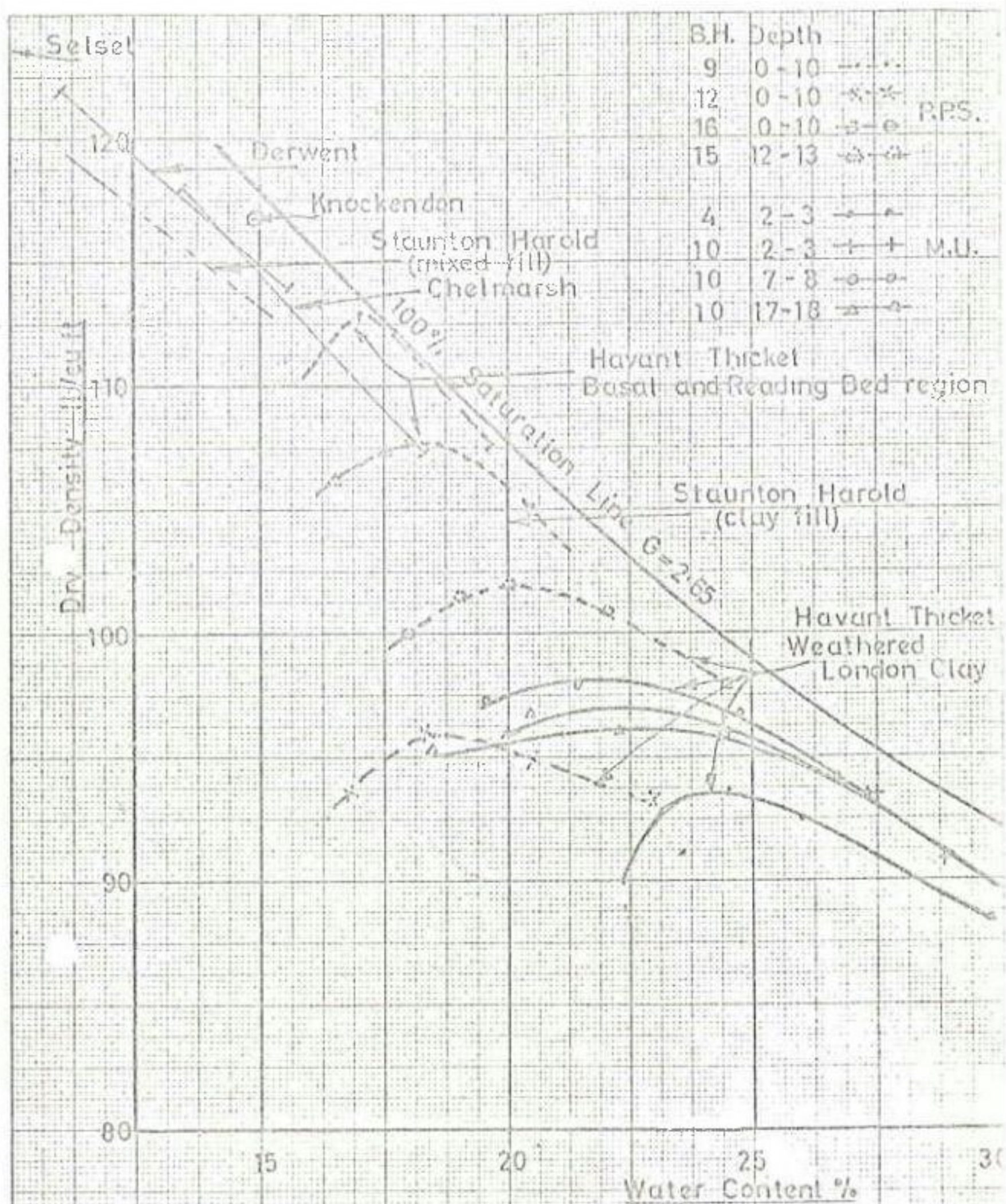


FIG. 6 . COMPACTION CURVES