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DIST. 14 REGION

W.P. No.

CONT. No. INSTRUMENT'N

W. O. No. 73-11099

STR. SITE No.

HWY. No. 11

LOCATION O.N.R O'HEAD - INSTRUMENTATION

No OF PAGES - —

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

31M-27

NEW LISKEARD RE-VISITED

Settlement, Pore Pressures,
Shear Strength Observations
in 1973

W.O. 73-11099 Dist 14

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Settlements, Pore Pressures and Shear Strength Observations at the Site of the Instrumented Section of the New Liskeard Embankment

Hwy. #11 District #14

W.O. 73-11099 W.P. 104-60-02

1. INTRODUCTION:

A limited scope field investigation was carried out during the period of August 13 to 17, 1973, at the instrumented section of the approach fills of the proposed O.N.R. overhead of Hwy. #11 in New Liskeard.

The purpose of the investigation was twofold. Firstly, it was to determine the increase of undrained shear strength of the varved clays beneath the embankment, and to observe existing settlements and pore pressures; secondly, to assist the University of Western Ontario in measuring the lateral pressures in the subsoils by means of hydrolic fracturing. The investigation was financed by the Ministry.

In the following sections a summary of the results of the first task of the investigation is given. The results of the hydrolic fracturing will be reported by the University of Western Ontario.

2. SETTLEMENTS:

2.1) A full scale spread footing was built in November 1965, and loaded to approximately 200 tons, being the estimated weight of a pier and one span of the proposed bridge. The bottom of the footing is six ft. below the top of fill, having four ft. fill under the base. Settlements under the footing in the order of 2.5 inches occurred during the first year after constructions. After 1966 up to the recent survey (August 1973) the settlements proceded with a constant rate, averaging 0.6 in./year (see Figure #1).

2.2) Settlements due to the reduced embankment loads were measured between the load removal in November 1965 and 1968. On account of the groundwater which inundated the instrument culvert, settlement augers and plates could not be surveyed during the recent visit. It appears, however, a fair assumption that the settlements measured in 1966 to 1968 have been proceeded with similar rate ever since. By extrapolating the time-settlement diagram, the yearly rate of settlement under the reduced embankment is postulated to be 0.6 in./year, exactly the same as measured under the test footing. It may be concluded, therefore, that settlements due to the footing load alone were completed within one year, and further settlements have been caused by the embankment load.

2.3) At Sta. 796 + 00 on the west side of the railway tracks, settlement augers were installed just below the embankment by drilling through the fill. The height of the fill at this location is still the original 17 ft.

The yearly rate of settlements beneath the center of the embankment between the years 1965 and 1968 was 1.4 in./year, and from 1968 to the present time it was 1.1 in./year. The change of rate was very likely a gradual one; since, however, there were no surveys carried out between 1968 and July 1973, this change could not be narrowed down.

The rate of settlement beneath the shoulder of the fill at the same station was 1.05 in./year between 1965 and 1968, and 0.85 in./year after 1968 (see Figure #2).

2.4) <u>Tentative Conclusions</u>:

The present rate of settlements at both sides of the railway tracks appears to be proportional to the imposed external loads. At the west side of the tracks, the height of fill is approximately twice the height of the east approach

...3

fill, and the rate of settlement under the west fill is also roughly double the one near the culvert. Consequently, it may be hypothesized that after removal of half of the fill at the east approach, there has been no overstressed zone under any part of the investigated fills.

Recent survey of the surface settlement plate S-6, some 111.1 ft. left of the ℄ in the shack, indicated that the ground level at this location was still approximately 1.3 in. higher than before the fill construction (see Figure #3). This finding may confirm that there has been some lateral "squeeze" of the subsoils due to the shear stresses beneath the original high embankment load.

3. PORE PRESSURES:

3.1) Only those piezometers placed in the shack, some 115 ft. left of ℄ , were found to be operational during the recent field work. Readings of these piezometers proved that there is still some excess pore pressure in this location as compared with the original distribution recorded before fill construction.

Figure #4 shows the original pore pressures, indicating a hydrostatic deficiency, believed to be the results of a flow towards the Wabi River. The present piezometric pressures are still higher than the originals, the maximum excess pressure (Δu) being approximately seven ft. of water around the 50 - 60 ft. depth.

The variations of excess pore pressures (Δu) during the principal stages of fill construction, expressed in PSI, are shown on Figure #5. The initial pore pressure distribution on this diagram is considered to be zero. As it may be seen at the 60 ft. depth, the existing Δu is still approximately half of the value of the maximum observed Δu a few months after the construction of the high embankment.

It is to be noted that piezometers near the ground surface (P-41 and 42) have been effected by the seasonal variations of the free ground water level. No such variations were ever noted below the 45 ft. depth.

3.2) Conclusions:

Since the stress conditions in the soils have been permanently changed by the surrounding two embankments (fill for the present by-pass and the test fill), it is believed that pore pressures might never dissipate at this location to the original value's.

4. UNDRAINED SHEAR STRENGTHS:

4.1) It was reported by Stermac and al. that there had not been any strength increase under the embankment during the existence of the 18 ft. high fill (June 1963 to November 1965). This observation necessitated the change of the design and the subsequent removal of 8 ft. of fill. Some 7 months after the load decrease (July 1966), continuous field vane tests were again carried out near the ℄ of the reduced fill. These tests already showed a trend of increase of the undrained shear strengths. The trend was probably most noticable between elev. 605 ft. and 614 ft., within the weakest zone, where the strength increase was measured to be roughly 150 PSF.

4.2) Two boreholes numbered 73A and 73B were carried out by a hollow stem auger during the recent fieldwork, near the location of the previous holes. The bouldery fill was first excavated by a backhoe, then two pieces of 12 in. CIP were placed vertically in the excavation, upon which the hole was backfilled. The distance between the borings was 6 ft. In B.H. #73A 2 in. Shelby tube samples were taken at every 3 ft. In between the samples, vane tests were performed by means of a NX size M.T.C. standard vane apparatus. In B.H #73B, vane tests were carried out at those elevations where the Shelby tubes were taken, so that at the end of the borings we had vane test results at every 18 in., extending to elev. 567 ft. No samples were taken in B.H. #73B, but after each vane test the apparatus was pushed again 18 in., without, however, augering the hole. By performing vane tests at the latter depths, the effect of friction along the rod could be observed.

Figure #6 shows the variations of undrained shear strengths measured in 1964, 1966 and 1973.

4.3) <u>Conclusions</u>:

No strength increase took place in the varved clay subsoils during the two and a half year period, when the soil was loaded by the 18 ft. high fill. The reasons for this were discussed by Stermac and al. in 1966.

A gradual strength increase was observed after the removal of the fill, the increase being approximately 20 - 30% within the upper zones and 10 - 15% within the lower, approximately 7.5 years after the reduction of load.

This increase is still lower than the ultimate increase to be expected, the reason of which might be associated with the destruction of cementation bond in the clay under shear as reported recently by Lo (Behaviour of Embankment on Sensitive Clays Loaded Close to Failure. 1973).

A. K. Barsvary

AKB/ds — A. K. Barsvary, P. Eng.
August 31, 1973.

APPENDIX I

73-11099

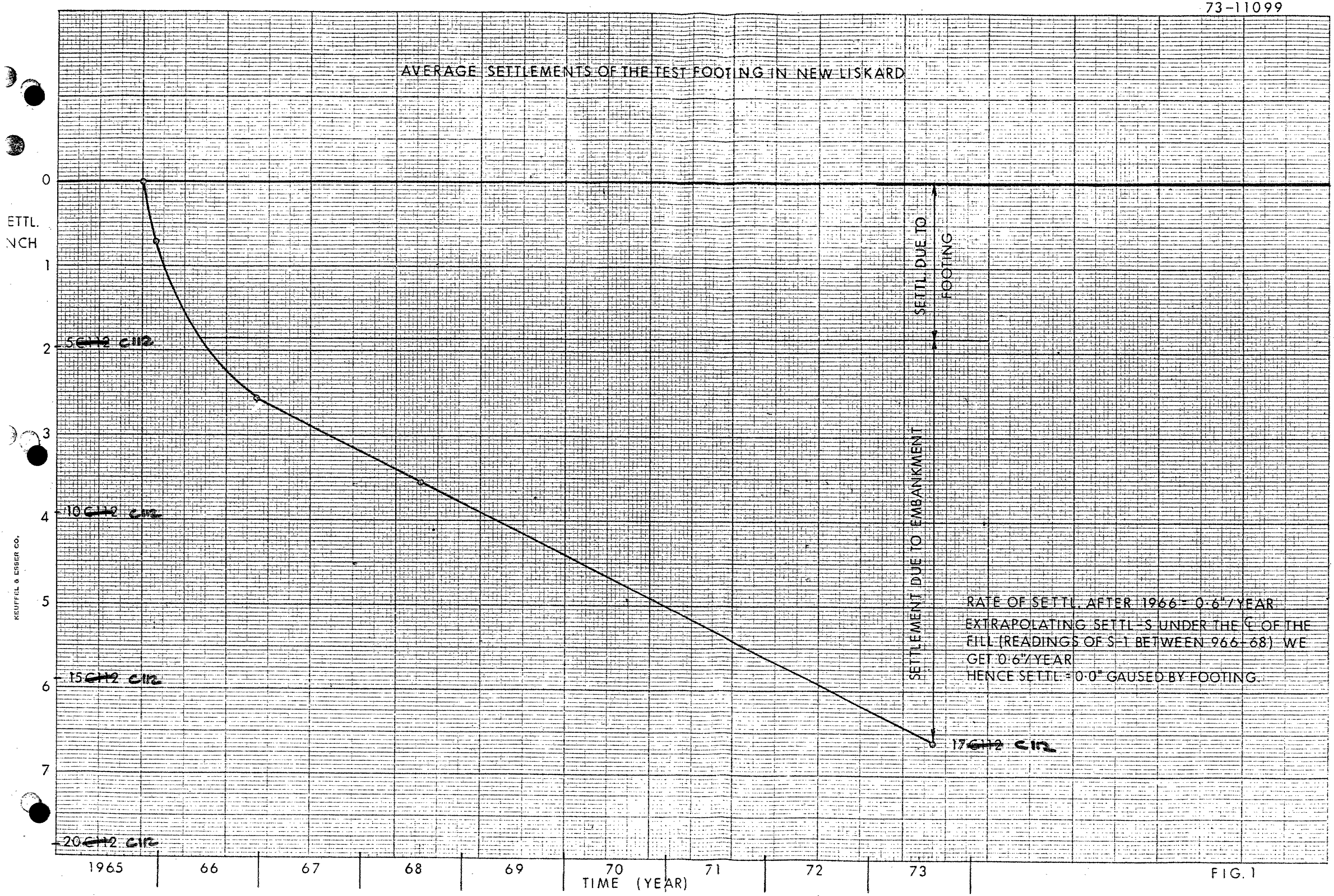


FIG. 1

73-11099

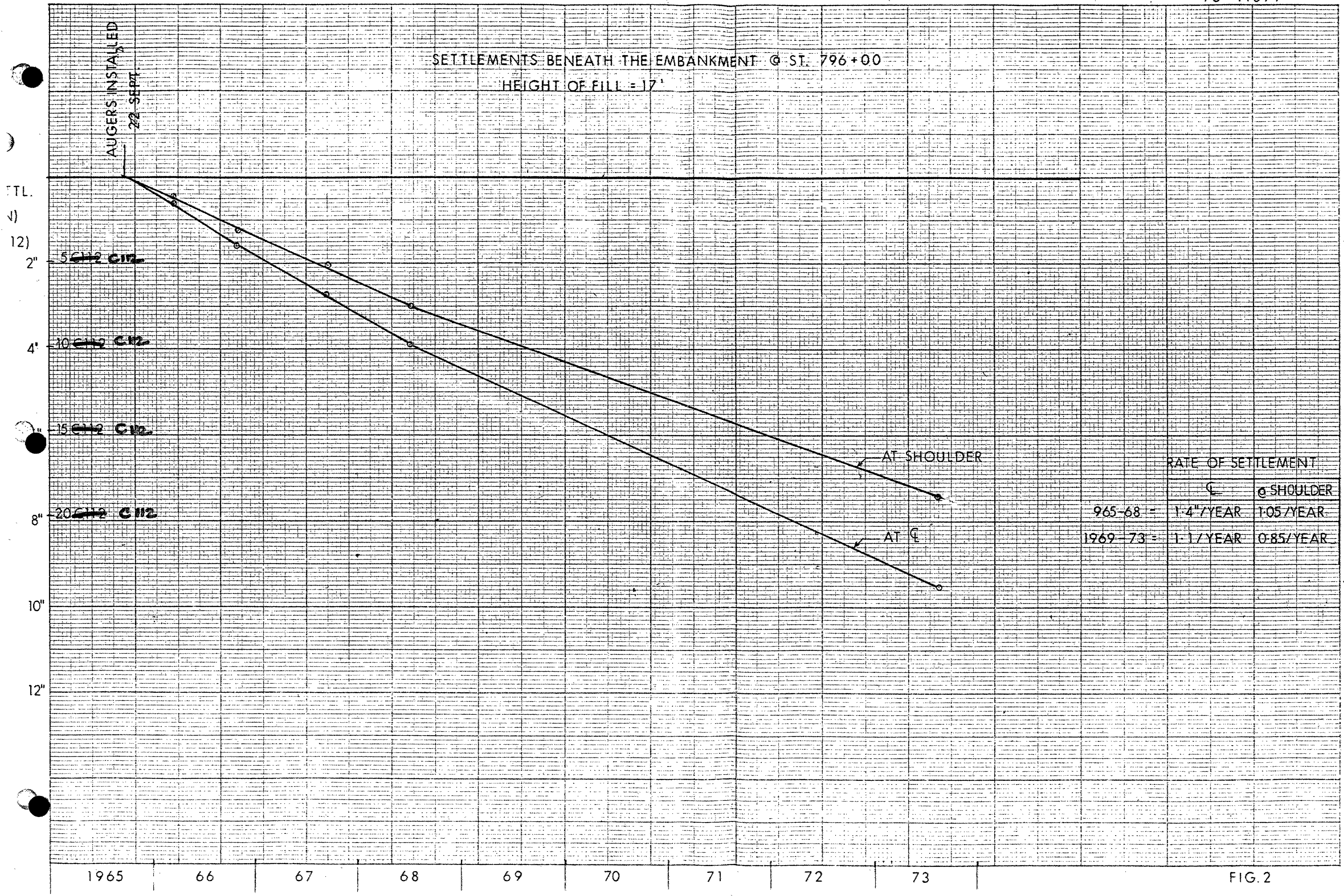
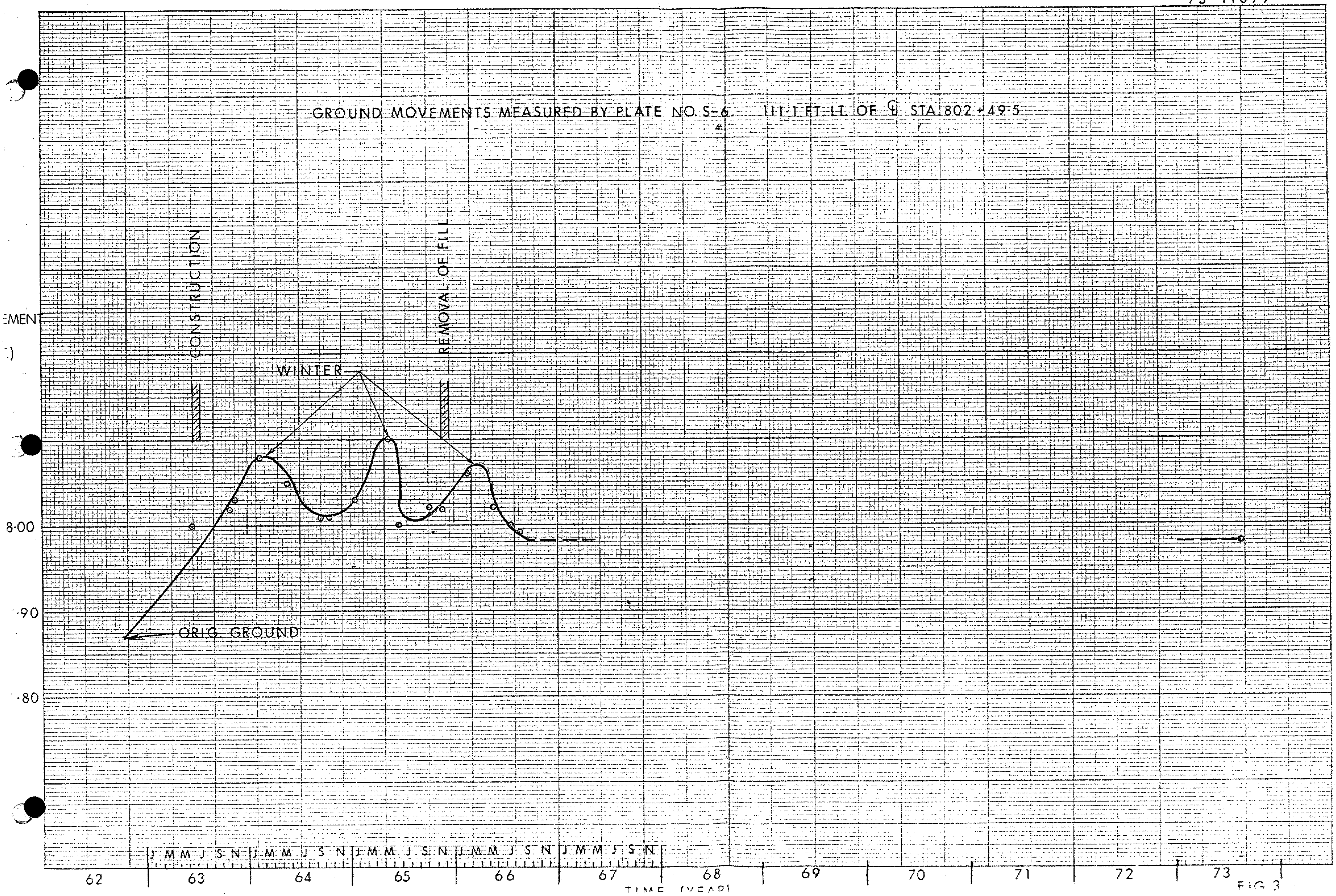


FIG. 2



GROUND MOVEMENTS MEASURED BY PLATE NO. S-6. 111.1 FT. LT. OF ℄ STA. 802+49.5
CONSTRUCTION
REMOVAL OF FILL
WINTER
ORIG. GROUND
EMENT
8·00
·90
·80
J M M J S N
62
63
64
65
66
67
68
69
70
71
72
73
TIME (YEAR)

FIG 3

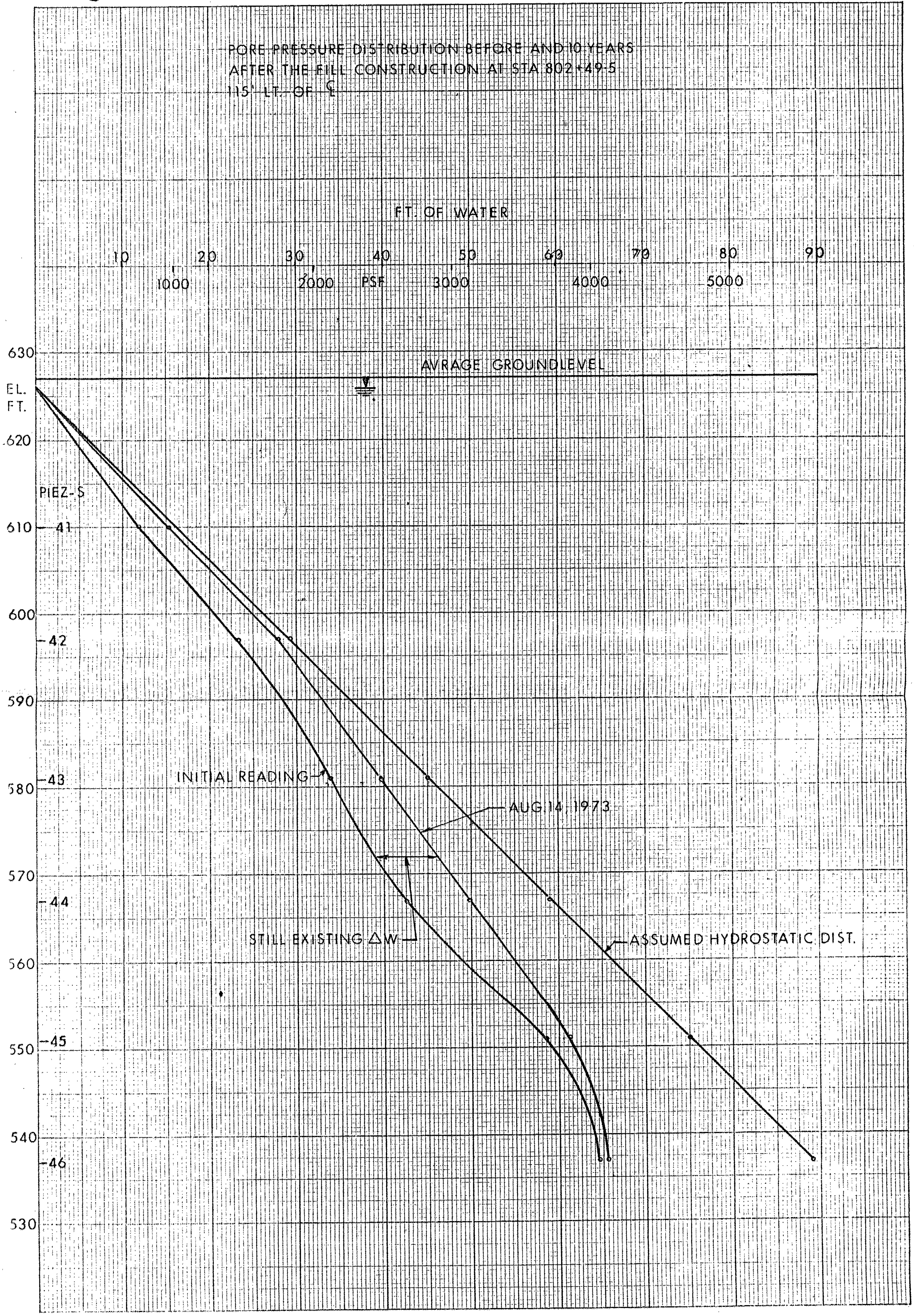

PORE-PRESSURE DISTRIBUTION BEFORE AND 10 YEARS AFTER THE FILL CONSTRUCTION AT STA 802+49.5 115' LT. OF ℄
FT. OF WATER
10 20 30 40 50 60 70 80 90
1000 2000 PSF 3000 4000 5000
630
AVRAGE GROUNDLEVEL
EL. FT.
620
PIEZ-S
610
-41
600
-42
590
INITIAL READING
-43
580
AUG 14 1973
570
-44
STILL EXISTING ΔW
ASSUMED HYDROSTATIC DIST.
560
-45
550
540
-46
530

FIG. 4

73-[illegible]099

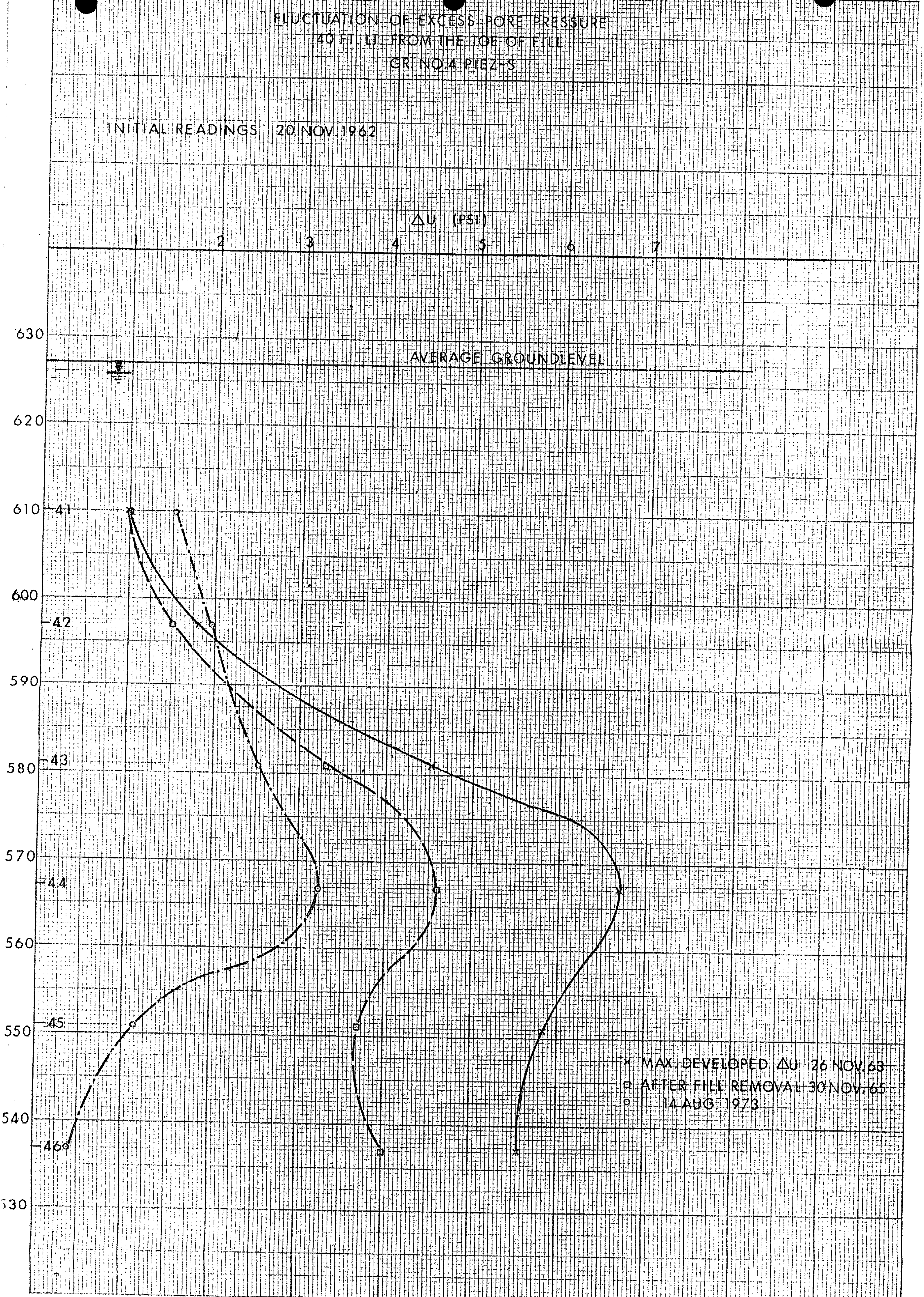

FLUCTUATION OF EXCESS PORE PRESSURE
40 FT. LT. FROM THE TOE OF FILL
GR. NO.4 PIEZ-S
INITIAL READINGS 20 NOV. 1962
ΔU (PSI)
1
2
3
4
5
6
7
AVERAGE GROUNDLEVEL
630
620
610
600
590
580
570
560
550
540
530
41
42
43
44
45
46
× MAX. DEVELOPED ΔU 26 NOV. 63
□ AFTER FILL REMOVAL 30 NOV. 65
⊙ 14 AUG. 1973

FIG. 5

COMPARISON OF VINE TEST RESULTS OF

BH'S 1C & D AND BH'S 73 A & B

DATES	LOCATIONS						
OCT. 64	1 C	802+70	3' LT. ℄	2 A	802+53	14' RT. ℄	JULY 66
	1 D	802+70	3' LT. ℄	2 B	802+50	14' RT. ℄	
AUG. 73	73 A	802+70	13' LT. ℄				
	73 B	802+65	13' LT. ℄				

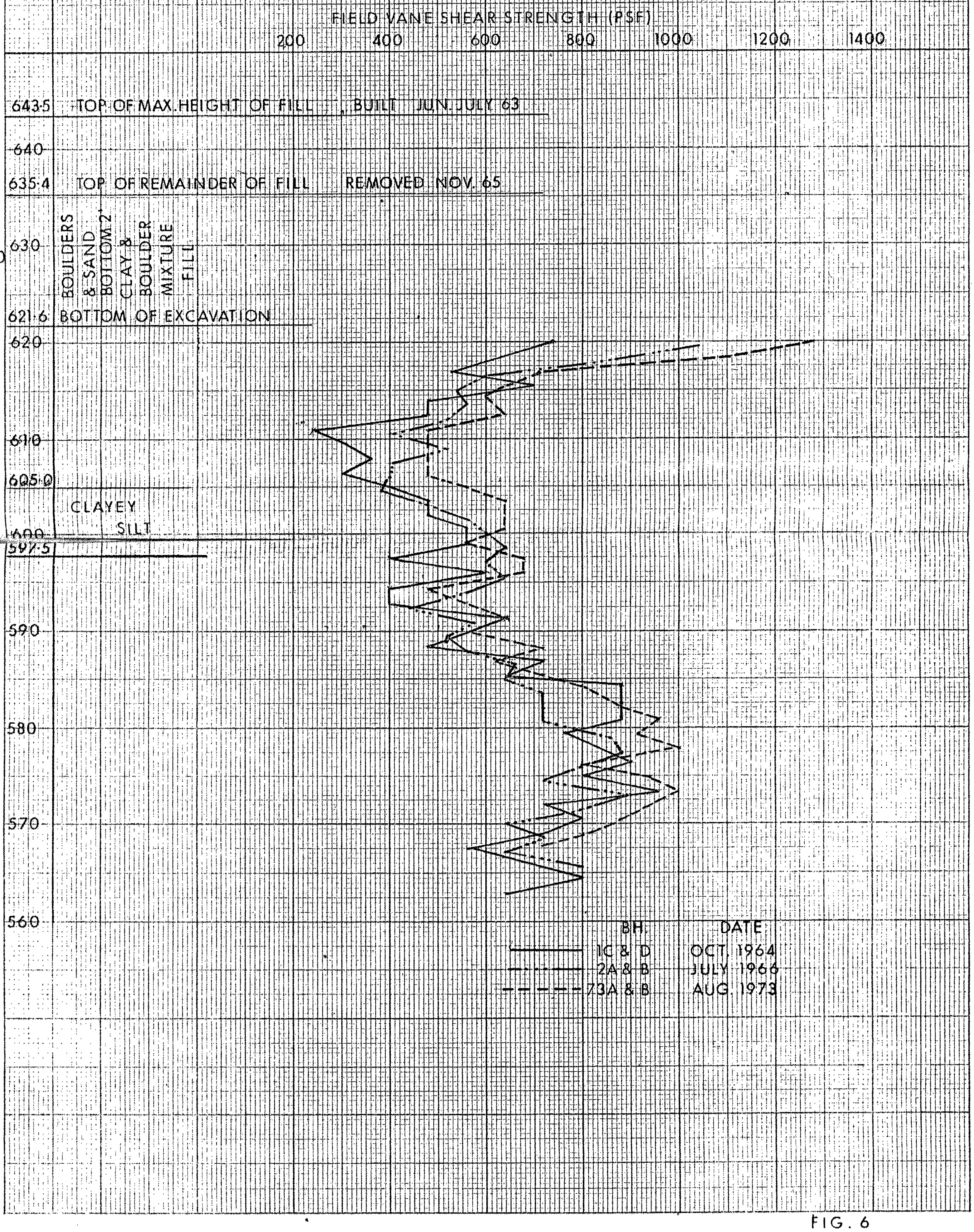


FIG. 6