Paper No. 322.

Foundation Problems at Karnafuli Project, Pakistan.

By
KHAJA AZEEMUDDIN[1]

1. Description of Project.

This is a multipurpose project across the river Karnafuli, in Pakistan, whose drainage area at the dam site is 4250 sq. miles, which is situated in a zone of heavy annual rainfall varying from 100 to 150 inches. The rain is mostly concentrated in the four monsoon months of June to September. It embraces hydropower generation of 160,000 KW, reclamation of 7,50,000 acres by drainage pumping, flood control for 350,000 acres, navigation, fisheries, etc. It involves the construction of an earthen dam 130 ft. high above the river bed and capable of storing 5.4 million acre feet of water, a spillway capable of discharging a flood of 5,25,000 cusecs, a power house to accomodate 4 units of 40,000 KW each and navigation facilities to handle 350,000 tons annually.

2. General Geology.

The Karnafuli basin lies within the area of Tipan and Surma formations which are the result of deposition by some great river in the Miocene and Pliocene periods. There geologically young sedimentary rocks are still in the process of being consolidated and compressed leterally by the great Tectonic forces which have raised the Himalayas. The sedimentary rocks of the region consist predominantly of layers of shale and sandstones ; the shales very from silt stones, soft shales, down to very hard calcarious shales whereas sandstones are normally softer, interpersed with a few hard layers in between. Gentle foldings can be observed in many places but these appear to have only a limited local effect. However, due to the sedimentary nature of these rocks and the effects of the foldings, very extensive boring operations were carried out in areas where the main structures are proposed to be located. In all 442 bore holes were drilled with an aggregate length of 53,405 ft.

Tbe foundations have to be placed on shale rock. It has compressive strength sufficiently greater than the moderate loads to be imposed by the proposed structures and sliding tests have shown exceptional shearing resistance.

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The Karnafuli basin is located in an area of moderate seismic activity, Severe earthquakes have been experienced in Assam and Burma In August 1950 an earthquake occured, of magnitude 8.25 to 8.50 on the Richter scale, whose epicentre was about 400 statute miles northeast of the dam site. All the structures have accordingly been designed for earthquake resistance bearing forces of 0·10 gravity applied horizontally.

3. Foundation of the Main Dam

The main dam is 1800 ft. long at the crest and 1500 at the bottom of the gorge. The river length proper is only about 500 ft., 'he rest consists of a berm at an average level of R.L. + 50. Shale rock is exposed on both the flanks. On the left bank, close to which the river is situated, the shale is exposed but it dips at a steep slope till it reaches a level of R L. —77 and then it rises and is again exposed on the surface at the right abutment hill. *Fig. 1* gives the cross section of different stratas along the centre line of the dam. It will be seen therefrom that the shale is overlain by loose fine medium sand, silty sand, blue clay and brown clay.

It will be costly to remove the overburden over the shale and cheaper methods of consolidation and resistance against permeability can be devised. For the purposes of design, it was necessary to carry out extensive tests of the physical properties of various soils. and particularly of their permeability and bearing capacity. The grain size analysis of the soils with their properties are given in Fig. 2.

From the above grain size analysis one is liable to conclude that the sand overlaying the shale will be watertight for the purposes of design requirements. However, in order to make certain. permeability tests were carried out at points P_1, P_2, P_3 and P_4. The general layout of these tests is given in Figs. 3 & 4. Before starting these tests the levels of the unlowered ground water table were recorded in the tubewell and in 9 piezometric tubes driven into the pervious foundations around the well. From the lowered ground water table the average permeability has been calculated. The co-efficient "K" of permeability was determined in two principal directions, one normal and the other parallel to the axis of the dam. The average figures are as follows :—

Pumping test No.	Normal to axis of dam (cm/sec)		Parallel to axis of dam (cm/sec)	
P_1	K	$3{\cdot}77 \times 10^{-2}$	K	$2{\cdot}63 \times 10^{-2}$
P_2	K	$6{\cdot}63 \times 10^{-2}$	K	$12{\cdot}34 \times 10^{-2}$
P_3	K	$19{\cdot}70 \times 10^{-2}$	K	$13{\cdot}5 \times 10^{-2}$
P_4	K	$10{\cdot}12 \times 10^{-2}$	K	$7{\cdot}72 \times 10^{-2}$

The free water level in all piezometric tubes was above the pervious strata. As the river here is still within the influence of tidal action, during all these pumping tests the water levels of the river were also noted simultaneously with that of piezometric pipes The ground water table followed practically the variations in the river levels.

The values obtained for the co-efficient "K" are rather unusual when compared with the grain size curves especially for a sand with hardly any admixture of gravel. The permeability tests carried out in the laboratory on samples of the same sand, gave the value of this co-efficient nearly 100 to 150 times smaller. The only conclusion that can be drawn is that in its original state the density of sand must be very low with a very high voids volume. This was further confirmed by the fact that the sand was flowing continuously into the bore casings, thereby preventing the performance of any standard penetration tests.

From these results it was falt that it may be feasible to grout this sandy layer. Cement grouting is not practicable because the average size of Portland cement particles are bigger than the voids of the sand to be grouted. Further the coarser cement particles are immediately retained by the sand, forming a thin impervious skin, thereby stopping even the finer particles of cement from further penetrating into the sand even under high pressures. Chemical grouting is feasible with a solution of silicate of soda (Na_2 Si O_3) having a determined concentration in degree Baume and by adding a determined amount of suitable electrolite for producing a siliceous gel But Chemical grouting is a very costly process even in countries where such chemicals are produced, Clay grouting is economical especially if Bentonite clay is available, whose size of particles is 0.2/u & less. Bentonite is known for its thixotropic properties. Bentonite clay is not found in East Pakistan, but other types of clay can also be used for grouting purposes, provided all the coarser particles are removed by sedimentation Laboratory investigations of the various clays showed that such a type of clay is available right at the dam site and what is termed as brown clay is quite satisfactory for this purpose, when it is separated by sedimentation from the fine sand and the coarsest silt particles included in the original clay, It was found that the clay so prepared with a density of about 40° BE can be stabilised by about 10% of Portland cement. The suspension was perfectly stabilised and only a negligible decantation of clay particles could be observed after the chemical reaction of the cement had started.

A borehole of 3" diameter was sunk right down to shale, 113 ft. below G.L., located at mid distance between pumping tests Nos. 3 & 4. At the lower part of the casing, a grouting pipe of 1½ inch diameter perforated over 2 ft. length was fixed. The clay slurry was prepared in large settling tanks and after separation of the coarse particles, agitation was done in a mixer from where the grout was forced with a Duplex pump allowing a maximum pressure of 220 lbs per sq. inch. Grouting was started with a clay slurry of 10° Baume and gradually increased upto 40° Baume. After grouting of 3,603 tons of dry clay, the grouting work

was temporarily stopped, due to lack of clay, and the work was resumed the next day by using a slurry of 17° Baume which was increased gradually to 50% Baume. A slurry of 50° Baume has already a consistency of a stiff cream. The starting grouting pressure was about 60 lbs/sq. inch and was increased upto 110 lbs/sq. inch. The increase in pressure, to a certain extent, is also due to the higher density of grout, involving a higher resistance in the sand.

The relation between permeability—"K" and void ratio "E" can be expressed fairly accurately by the equation,

$$K = A.\ E^B \quad (m/sec)$$

A in this formula represents the magnitude of the permeability of the soil, corresponding to a void ratio E=1 According to Slichter and Chardabellas, A can be determined by the following foamula (The units are in cms).

$$A = K = 1102.\ d^2_{10}$$

Where d_{10} is the effective grain size representing ten percent on the grain size diagram. We know that the average permeability in the field Kf is 9.5. 10^{-2} (cms/sec), and the permeability of the laboratory sample K_1 is 1.26. 10^{-3} (cms/sec), which has a porosity n_1 equivalent to 0.41 or $E_1 = 0.70$ From the grain size analysis the value of d_{10} if found to be 0.2 m, which gives the value of A as 4 4. 10^{-3}. Fig. 5 is a logarithmic curve giving the relation between K. and E. The curve is plotted from the two known points viz A which is the permeability corresponding to E=1 and the permeability K_1 corresponding to $E_1 = 0.7$. The value of B which is equivalent to tan a is found to be 2.25. This agrees with the experiments carried out by many authors, who determined that this value ranges between 2 and 5. From this curve the value of E_f is found to be 0 92 corresponding to the permeability of $K_f = 9.5\ 10^{-2}$ (cms/sec). The corresponding value of porosity in the field nf = 0.48.

The problem is therefore to reduce the percentage from 48 to 41 so that the permeability may be brought down to nearly 100 times smaller value, safe for the purposes of design. Further, what all is needed is to build an impervious curtain about 30 ft wide along the centre line of the dam. Fig. 6 gives the proposed layout of the bore holes and the expected sphere of their utility. There will be two rows of holes, 20 ft apart and the holes in each row at 8 ft centre to centre. After these holes are grouted, another series of holes will be drilled along the centre line, placed at 16 ft intervals. It is anticipated that due to adjacent grouting the central portion will also get filled up to a large extent and may be that instead of clay and cement, the slurry will have to be chemically treated with silicate of soda, converting it in a high speed rotating mixer to a thioxatropic suspension of clay gel. The total volume of work involved is 22,000 cft of boring and about 4000 tons of clay grouting, In addition to this grout curtain a blanket will be placed on the upstream side and at the downstream toe of the embankment relief wells will be constructed.

4. Compressive strengths of Brown and Blue Clay.

The Blue and Brown Clay overlay the sandy strata above the shale, having an average depth of 50 ft. The grain size analysis of these clays given in fig. 2, show that it is very necessary to determine their moisture content at different depths and also their load bearing capacity, because such fine clays often contain soft lenses in between. For this purpose, a 22″ dia. pipe was sunk by percussion and at regular depths of 6 ft. undisturbed soil samples were extracted with a special spoon sampler.

This sampler was 8″ diameter and samples of 18" in depth were taken out from a point 2 ft. below the bottom edge of the 22″ diameter casing. The bore holes were placed 30 ft. apart and each bore was taken down to a depth of 30 ft. The samples taken out were quickly sent to the Laboratory where tests were carried out without any delay.

The values of the strength of clays encountered at different horizons in different bores are given in Fig. 7. These results show that there is a comparatively weaker structure between the depths of 15 and 30 ft. A doubt was felt that possibly some error has crept in the method of taking samples and the earth in the sampler has been influenced during the sinking of 22" diameter bore pipe. In order to verify these results, a pit, 30 ft. deep, was dug near bore hole No. 67 and samples were taken out at regular intervals of 3 ft. and tested. The results are given in the curves of Fig. 7. These tests show that the difference may be upto 40%. It is felt that the values of previous results can be safely increased by 15 to 20%.

In order to build up the strength of those clays and to avoid the settlements being spread over a long period of time. It will be necessary to provide relief wells. Their location and spacing is being investigated. May be there will be a net work of holes about 2 ft. in diameter spaced at 50 ft. intervals and filled with sand and gravel ; more closer intervals may be needed for those located near the upstream and downstream toes.

5. Foundations of spillway.

Quite extensive boring operations were carried out in the spillway area As many as 150 bores were drilled aggregating to a total length of 20,921 ft. From these bores the underground geological cross section of the strata is plotted and is given in Fig. 8. It will be seen that between 2 layers of hard shales, there is a soft sandstone strata, which has at all costs to be avoided for any foundation work. From a geological point of view there can be no loose sand or sand stratum between two layers of compact shale because they were subjected to the same geological pressures. Moreover in this mass of soft sandstone nucleuses of hard crystallised sandstone are also found. From the presence of these nucleuses one is liable to conclude that some crystallisation process had already started but was interrupted later on for reasons we do not know. Further, this sandstone strata comes up to the surface on the hills beyond the extreme left bank of the spillway *vide* Fig. 9, which gives the underground sandstone contours. The main conside-

ration, governing the location and design of spillway and tail channel, was, that nowhere should there be a depth less than fifteen to twenty feet of hard shale above the sandstone contour.

Further, during those boring operations some absorption tests were carried out to determine the permeability of underground strata. The results of these tests are also given in Fig. 9. These results are quite at variance with labaratory tests on the sand recovered from the bore holes, which gives a permeability of 100 to 1000 times smaller in value. There is no relationship whatsoever between the permeability of this soft sandstone in situ and that of the sand having the same grain size curve. This leads one to conclude that some separation has taken place between the layers of sandstone and shale and with the presence and circulation of water in the contact zone probably the cementing agent of the sandstone is being removed progressively and disintegration is in progress. An effort was made to have a visual observation ; a large bore 22" in diameter was executed through the shale into the sandstone but as soon as the sandstone was reached the bore hole was rapidly filled with water preventing visual observation. This confirmed the original conclusions that the water comes mainly from the contact zone shales and sandstones ; further the lower part of the shale was also found to be fissured. Later another effort was made to confirm these doubts by grouting a series of holes located closeby and then by boring fresh test holes in between, to see where the cement grout has penetrated. These confirmed the previous conclusions and were further useful in determining the approximate distance between two holes necessary to provide a watertight curtain in front of the spillway. The general layout of these test holes is given in Fig. 10. These operations show that in order to have a continous curtain the distance between the succesive grout holes should not be more than 25 ft This grout curtain will be provided along the length of the spillway structure, besides the grouting of all the holes executed for investigation purposes, which were carefully preserved. Some of the results of the operations, carried so far, are given in the table 1, below :—

STATEMENT SHOWING CEMENT USED IN GROUTING OPERATION AT SPILLWAY SITE

S. No.	Boring No.	Depth of Boring in feet.	Cement consumed in the holes. in Tons.
1	4 S.P.	104·83′	5·5 Tons
2	5 S.P. (a)	128·0′	3·15 Tens
3	5 S.P. (b)	55·76′	0·5 Tons
4	96 S.P.	113·77′	13·85 Tons

S. No.	Boring No.	Depth of Boring in feet	Cement consumed in the holes. (Tons)
5	95 S.P.	114·6′	1·40 Tons
6	97 S.P.	106·23′	4·60 Tons
7	105 S.P.	113.77′	1·75 Tons
8	106 S.P.	113·50′	1·20 Tons
9	102 S.P.	108·85′	1.95 Tons
10	103 S.P.	124·59′	2·20 Tons
11	104 S.P.	108.85′	6·10 Tons
12	6 S.P.	175·33′	6·07 Tons
13	20 S.P.	121·85′	1·05 Tons
14	7 S.P.	114·0′	3·60 Tons
15	82 S.P.	98·36′	2·30 Tons
16	$\frac{84}{79}$ S.P.	98 36′	2·90 Tons
17	80 S.P.	98·36′	0·90 Tons
18	46 S.P.	118·69′	2·90 Tons
19	45 S.P.	101·63′	1·60 Tons
20	67 S.P.	118·08′	Nil
21	44 S.P.	120·82′	3·75 Tons
22	43 S.P.	119·01′	4·65 Tons

6. Power House.

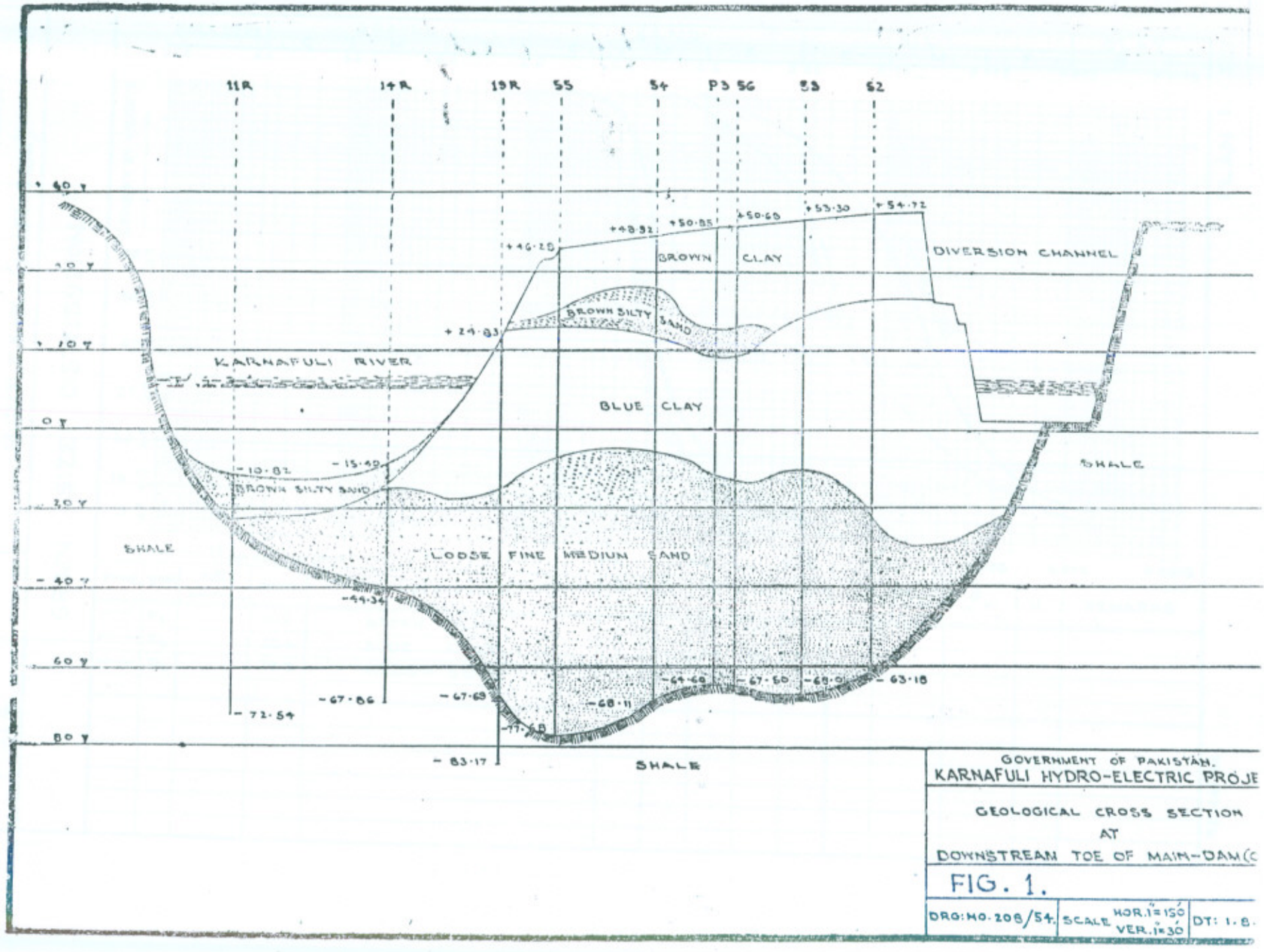
The power house is located half a mile away from the dam site, where the river passes closeby, after taking a long detour round the hills. The tentative layout of the Power House and the geological cross sections at the site are given in Figs, 11 and 12. The lowest foundation level below the Turbines is about R. L.—50. It is therefore necessary that at the site selected the sandstone strata should not be above R. L.—70 so as to give 20 ft. depth of solid shale rock above the sandstone. From the

figures given above it will be seen that the exact location had to be very carefully determined by means of a number of bore holes, in order to satisfy the above requirements.

The foundation problems of Karnafuli Project are interesting because of the presence of weaker or pervious strata below comparatively stronger and impervious rocks or clay. The problem was to consolidate these strata for the design of safe structures. In order to carefully locate the various structures and determine the methods to be followed in solving their foundation problems, very large exploratory work was done. The total length of investigation holes drilled is more than 10 miles, but the cost involved has been more than repaid due to the solutions that were possible for various foundations problems in the light of information so gained.

SUMMARY

The Karnafuli Project, in Pakistan, is a multipurpose project envisaging 160,000 KW of Hydro-Power generation and reclamation of 750,000 acres. It involves the construction of a dam 130 ft. high with a storage of 5·4 million acre feet and flood disposal works for 5,25,000 cusecs. The dam site is situated in a region of geologically young sedimentary rocks belonging to Miocene and Pliocene periods, consisting of various grades of shales and soft sandstone. In order to develope safe and economical designs nearly 10 miles long borings were done and this paper deals with some of the details proposed to be adopted in the design of important sturctures like Dam, Spillway and Power House. At the Main Dam the shale rock is overlain with a heavy depth of sand and clay. For economical considerations it is proposed to let these remain in site and to provide a clay grout curtain along with a blanket and relief-wells. This design was developed by carrying out pumping tests and experimental grouting. The second problem was the deterioration of the clay strength at different horizons and the methods to be adopted for building up the strength of clay at weaker spots It the case of spillway it was found that below hard shale and at its junction with soft sandstone there is a plane of seperation due to disintegration; although this is not conitnuous but has to be filled with grout. Extensive borings were done in order to plot the geological cross section of underground strata and experimental tests were carried out to determine the location and spacing of bore holes necessary for grouting this plane of separation. In the case of Power House. due to deep depth of foundation below the centre line of turbines, the location had to be very carefully fixed so that there will still remain a minimum depth of hard shale above the sandstones. These fouudation problems are interesting because of the presence of a weaker or pervious strata below comparatively stronger and impervious rocks of clay.



GOVERNMENT OF PAKISTAN.
KARNAFULI HYDRO-ELECTRIC PROJE

GEOLOGICAL CROSS SECTION
AT
DOWNSTREAM TOE OF MAIN-DAM (C

FIG. 1.

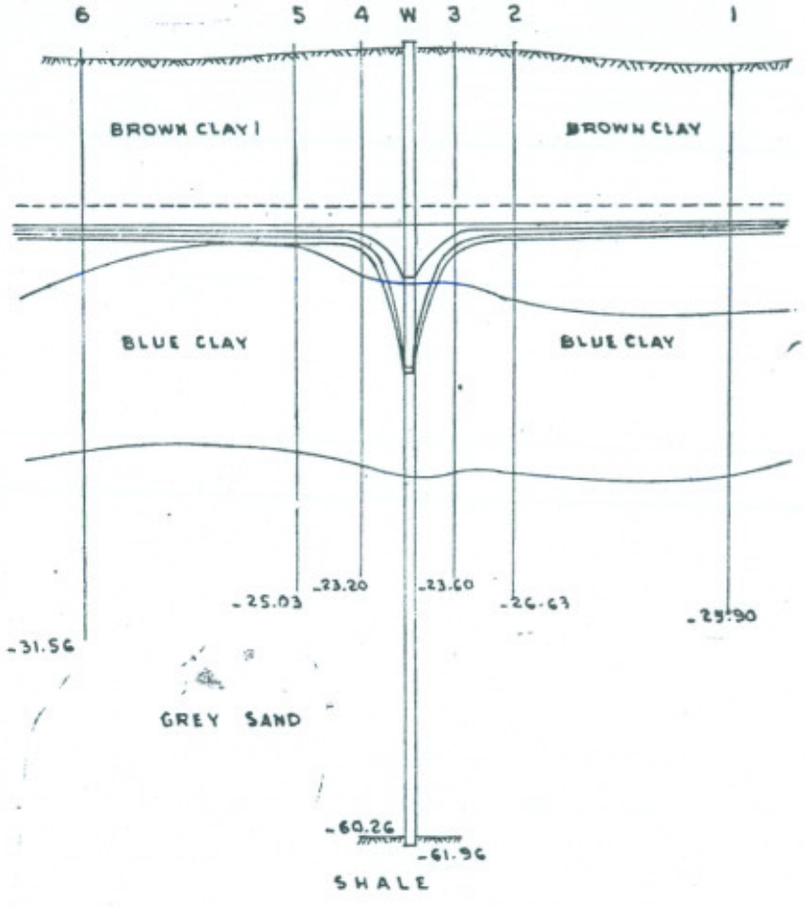
DRG: NO. 208/54. | SCALE HOR. 1" = 150' VER. 1" = 30' | DT: 1.8.

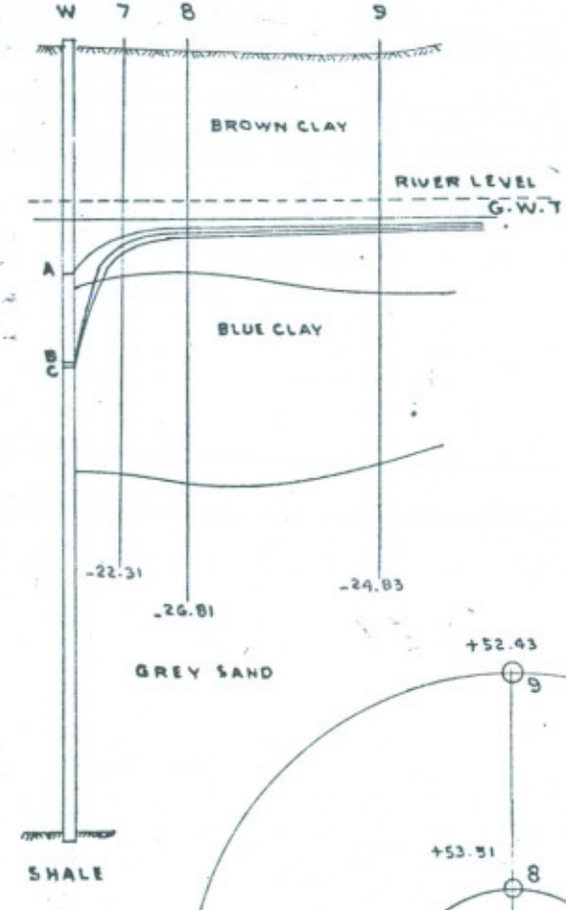
GOVERNMENT OF PAKISTAN
KARNAFULI HYDRO-ELECTRIC PROJECT

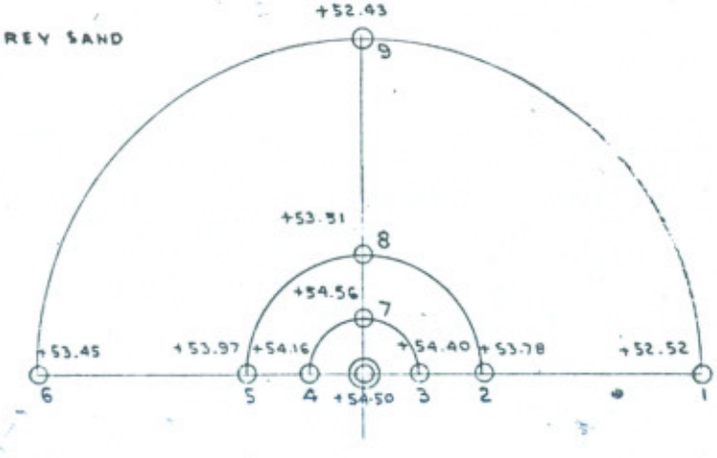
PUMPING TEST No. 4.

FIG. 4

DRG. NO: 214/54 | SCALE 1: 300 | DATE 11.9.54



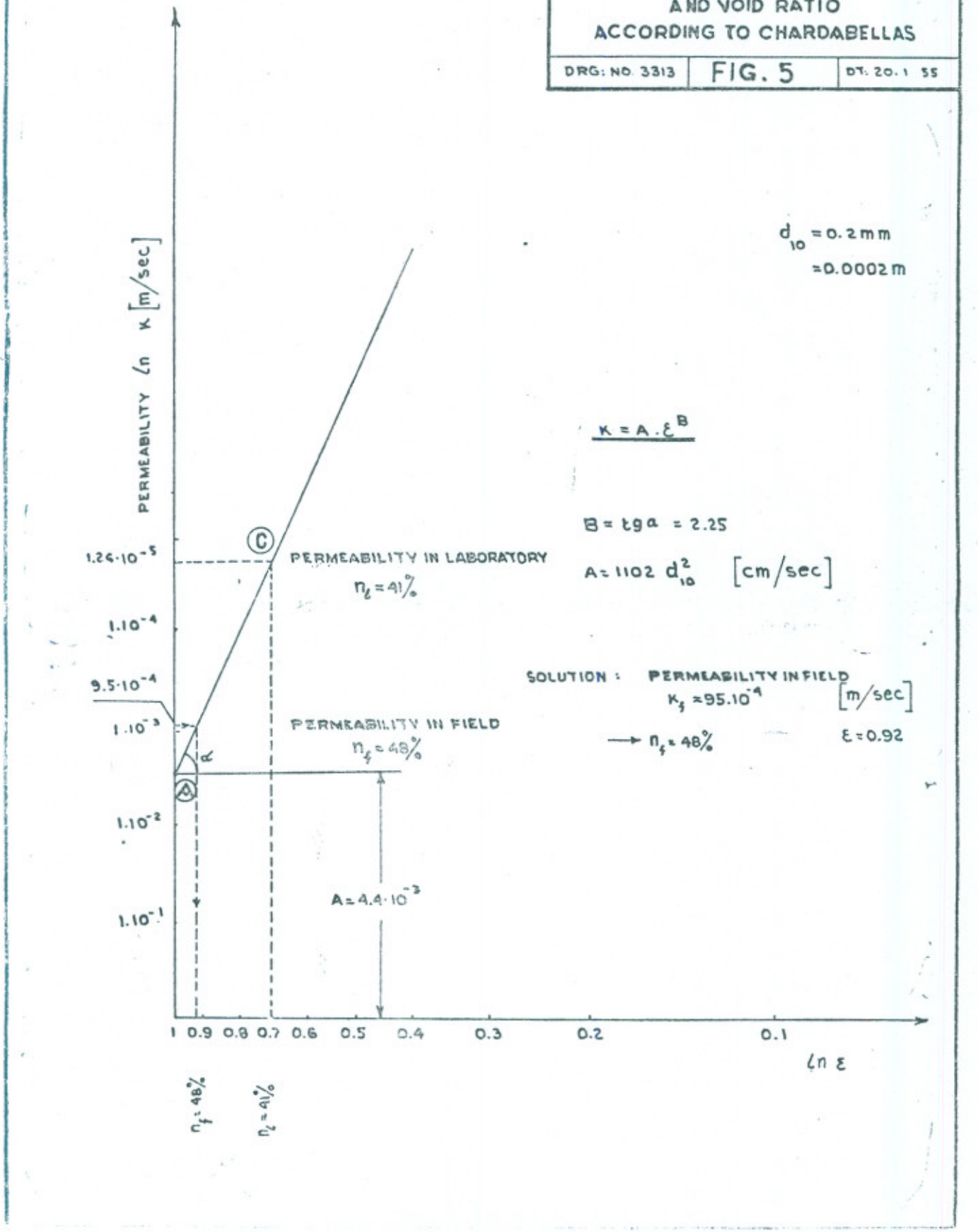


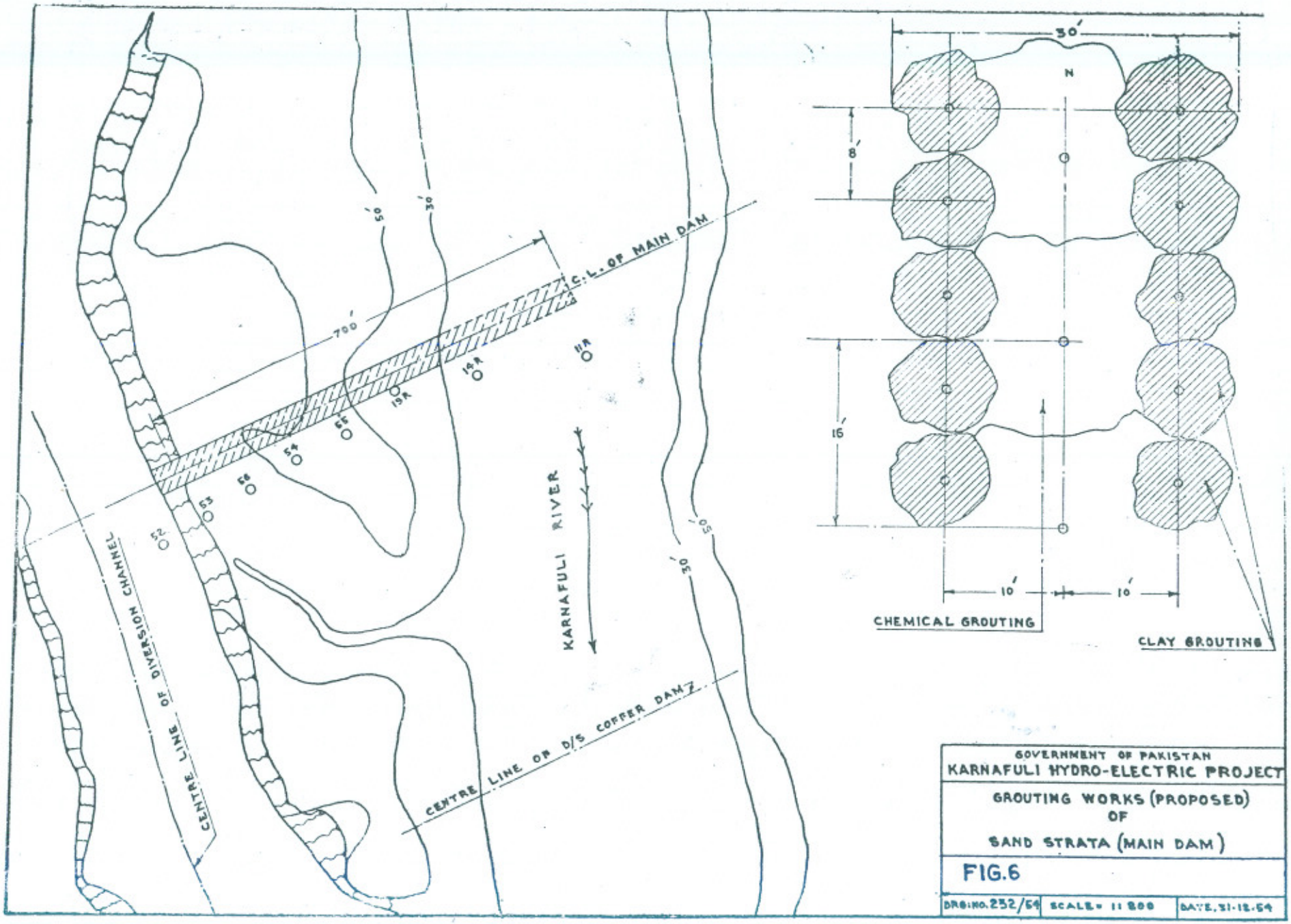


GOVERNMENT OF PAKISTAN
KARNAFULI HYDRO-ELECTRIC PROJECT

RELATION BETWEEN PERMEABILITY AND VOID RATIO ACCORDING TO CHARDABELLAS

DRG: NO. 3313 | FIG. 5 | DT. 20.1.55





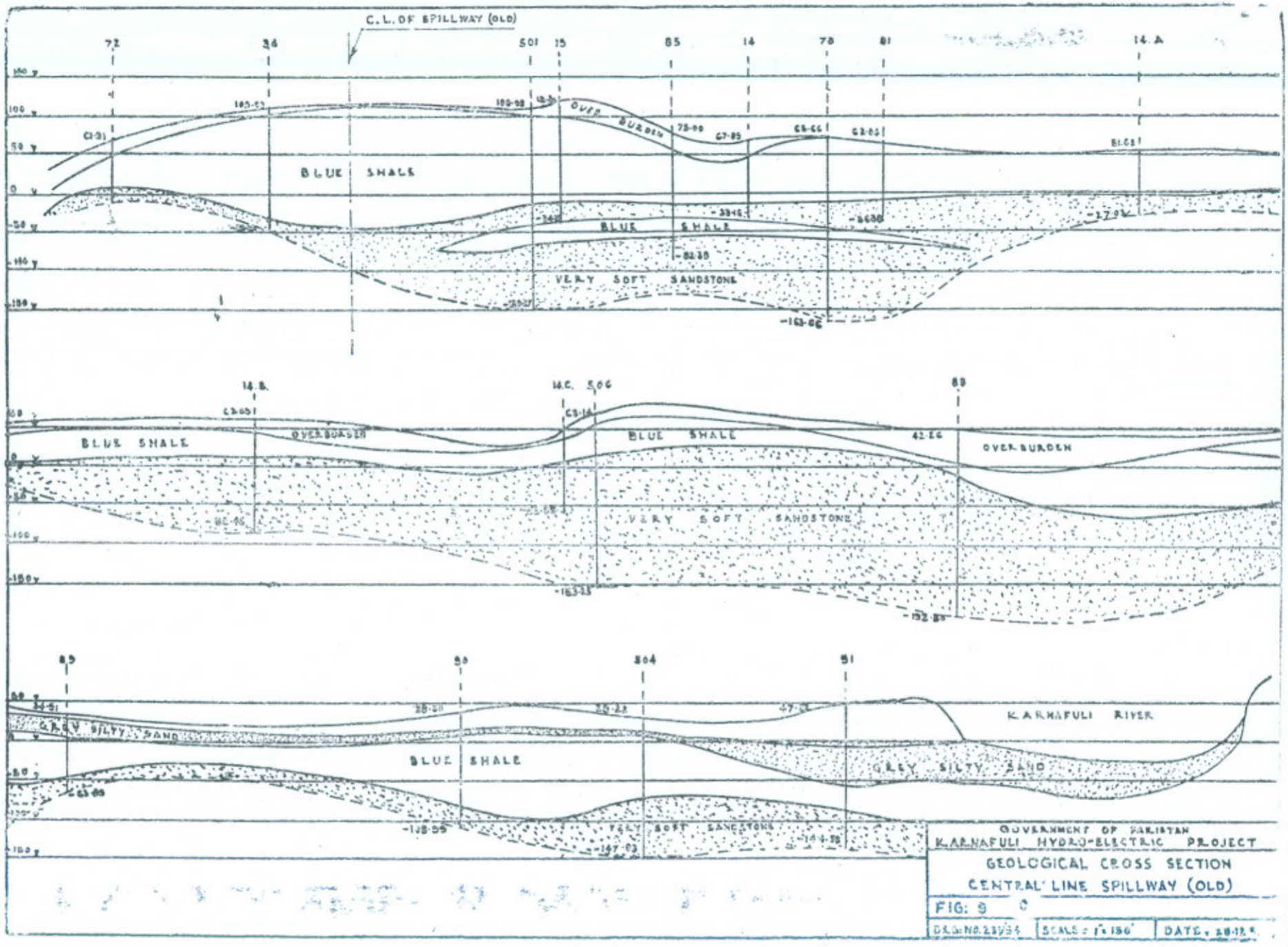
GOVERNMENT OF PAKISTAN

KARNAFULI HYDRO-ELECTRIC PROJECT

GROUTING WORKS (PROPOSED)
OF
SAND STRATA (MAIN DAM)

FIG.6

DRG.NO. 232/54 | SCALE= 1" = 800' | DATE. 31-12-54

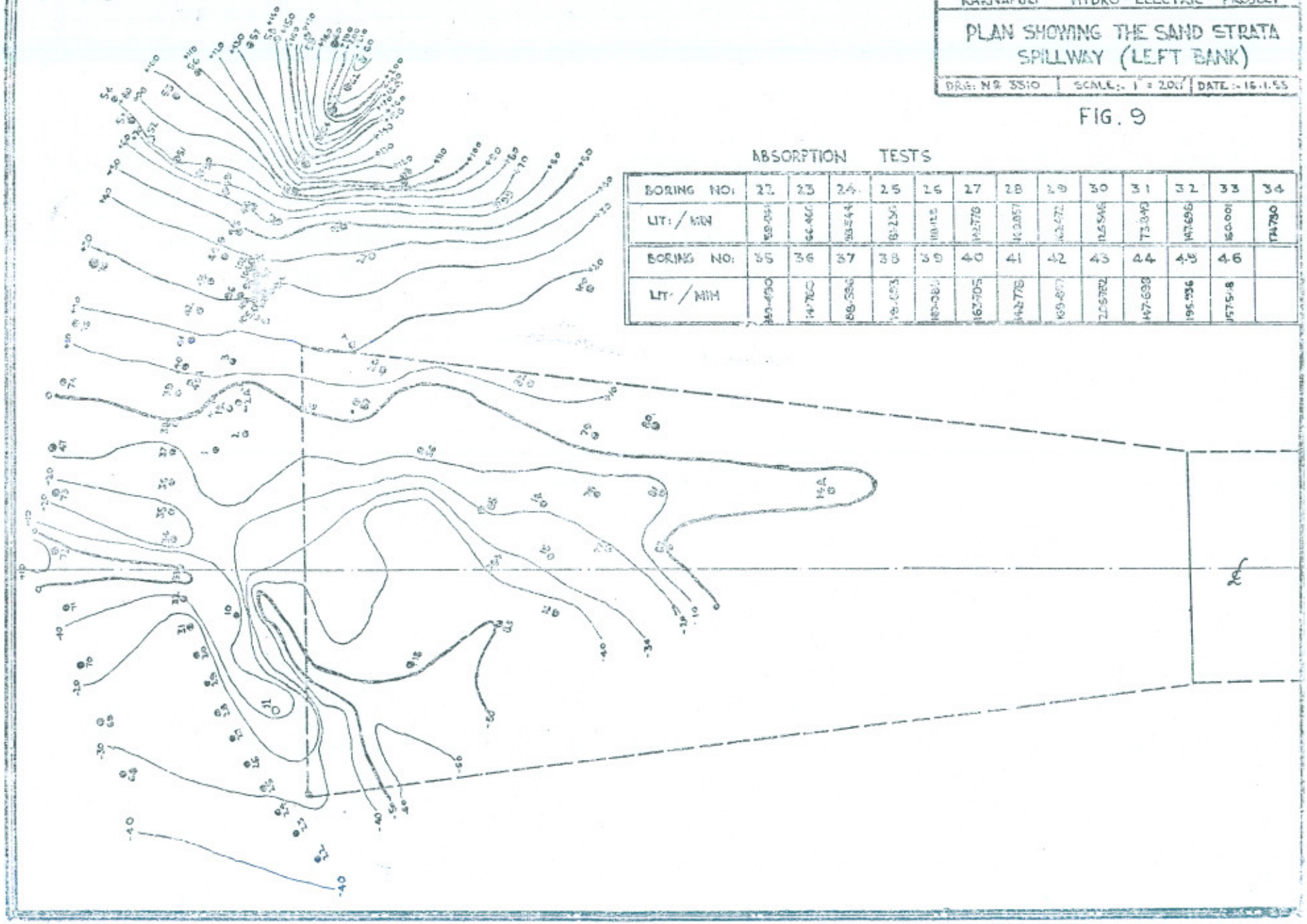

C.L. OF SPILLWAY (OLD)
7.2
3.4
5.01
1.5
8.5
1.4
7.8
8.1
14.A
OVER BURDEN
BLUE SHALE
VERY SOFT SANDSTONE
14.B.
14.C.
5.06
8.9
OVER BURDEN
BLUE SHALE
VERY SOFT SANDSTONE
8.5
5.0
8.04
5.1
KARNAFULI RIVER
GREY SILTY SAND
BLUE SHALE
VERY SOFT SANDSTONE
GREY SILTY SAND
GOVERNMENT OF PAKISTAN
KARNAFULI HYDRO-ELECTRIC PROJECT
GEOLOGICAL CROSS SECTION
CENTRAL LINE SPILLWAY (OLD)
FIG: 9
DRG. NO. 23/34
SCALE: 1" = 150'
DATE: 28-12-5

GOVERNMENT OF PAKISTAN
KARNAFULI HYDRO-ELECTRIC PROJECT

PLAN SHOWING THE SAND STRATA SPILLWAY (LEFT BANK)

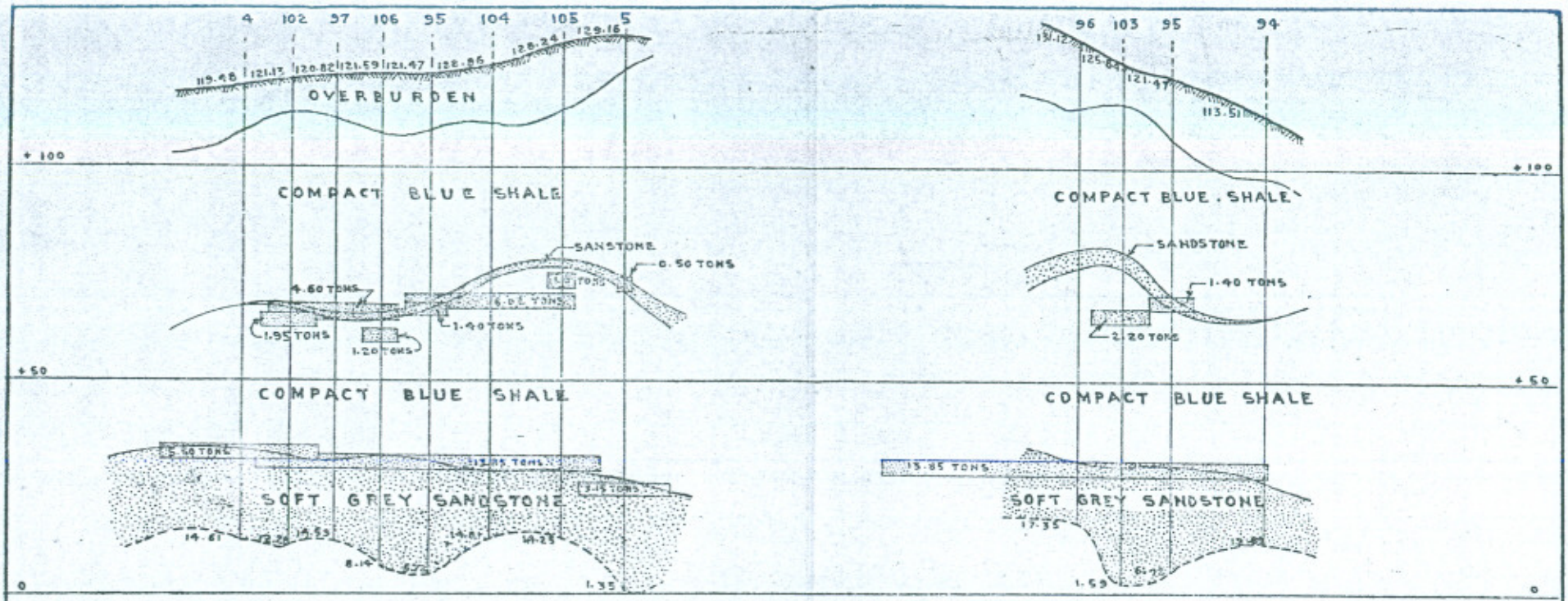
DRG: No 3510 | SCALE:- 1" = 20'-0" | DATE :- 16.1.55

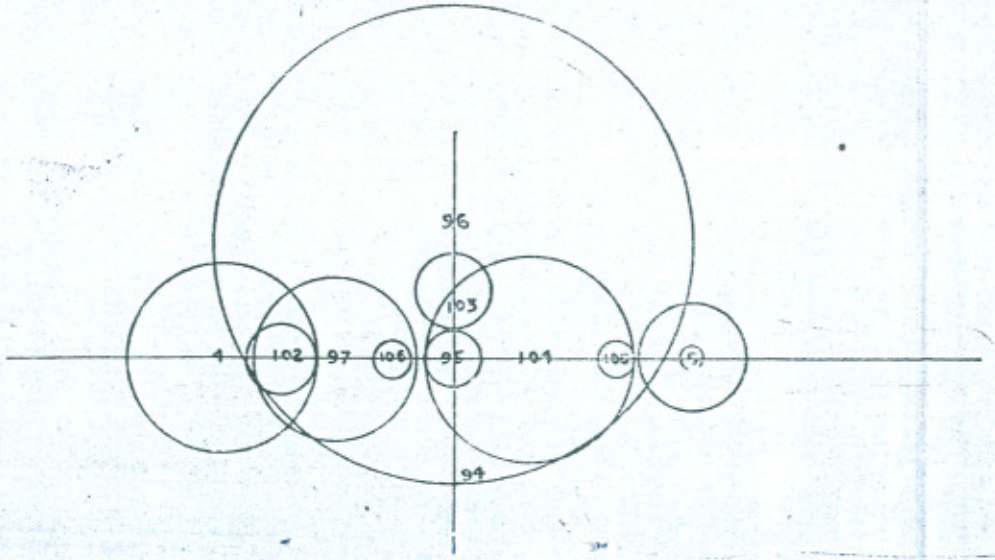
FIG. 9

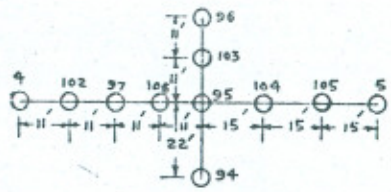


ABSORPTION TESTS

BORING NO:	22	23	24	25	26	27	28	29	30	31	32	33	34
LIT: / MIN	59-054	66-460	93-544	18-234	18-112	142778	142857	62-472	125-548	73-349	147696	150-001	174790
BORING NO:	35	36	37	38	39	40	41	42	43	44	45	46	
LIT: / MIN	159-490	147-760	186-586	19-093	180-381	167-705	142778	159-497	225-782	167-698	195-436	157-5-8	







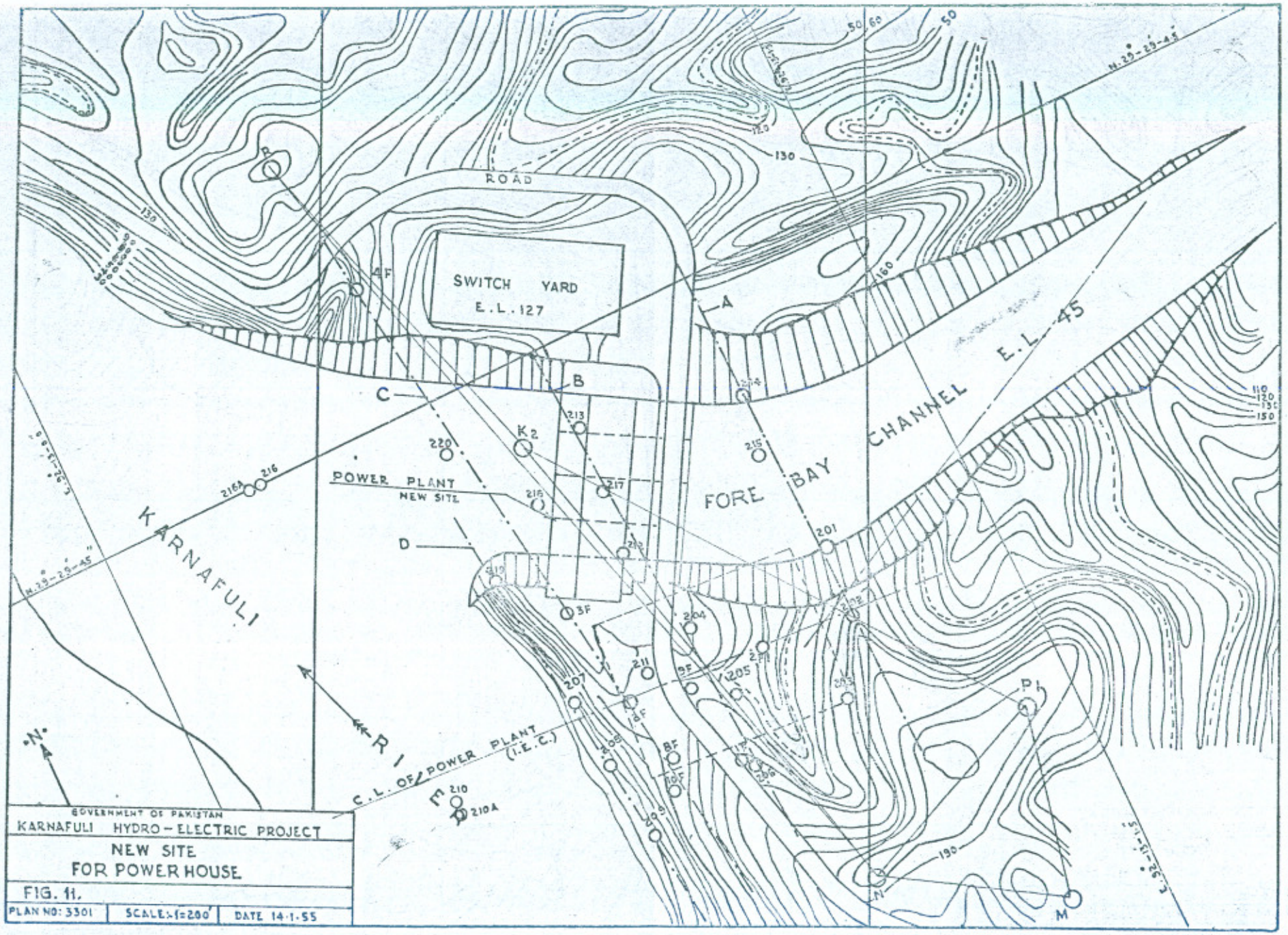
GOVERNMENT OF PAKISTAN

KARNAFULI HYDRO-ELECTRIC PROJECT

GROUTING TEST
AT
SPILLWY SITE
(ABSORPTION DIAGRAMMS)

FIG. 10

DRG: NO. 224/54	SCALE. 1:400	DATE. 29.11.54


ROAD
SWITCH YARD
E.L. 127
FORE BAY
CHANNEL E.L. 45
POWER PLANT
NEW SITE
KARNAFULI
C.L. OF POWER PLANT (I.E.C.)
A
B
C
D
K2
K1
P1
M
N
1F
2F
3F
BF
GOVERNMENT OF PAKISTAN
KARNAFULI HYDRO-ELECTRIC PROJECT
NEW SITE
FOR POWER HOUSE
FIG. 11.
PLAN NO: 3301
SCALE: 1"=200'
DATE 14-1-55

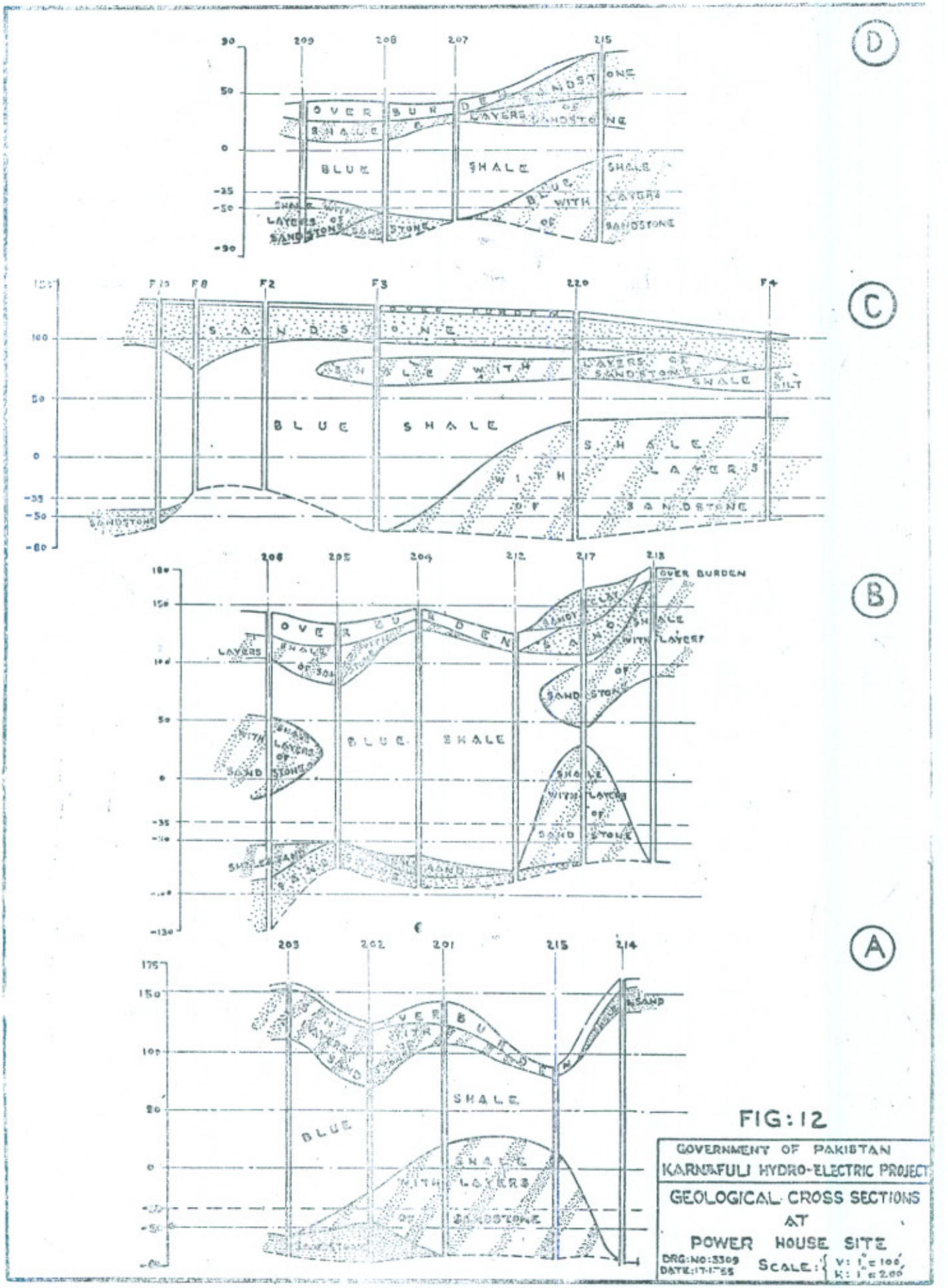


FIG: 12

GOVERNMENT OF PAKISTAN
KARNAFULI HYDRO-ELECTRIC PROJECT

GEOLOGICAL CROSS SECTIONS AT POWER HOUSE SITE

DRG: NO: 3309
DATE: 17-1-55

SCALE: V: 1" = 100'
H: 1" = 200'