Geotechnical Investigation By Mechanical Boring

The following work has been completed by Eng. Geology and Geotechneques adminis.

(1) Six bore holes were drilled along the bridge axis; 3 borings in river bed and the other 3 on land one on the Khartoum side and two on the Al Fittaihab side.

(2) Total drilled depth in meters was 112 m consisting of:

- 57 m into sandy silt or sand layer, and
- 55 m into Nubian sandstone layer.

(3) Fourteen (14) standard penetration test were carried out.

(4) Thirteen (13) undisturbed samples and fifty four (54) disturbed samples were collected for the following laboratory tests.

- Sieve aanalysis
- Index properties
- Consolidation test
- Shearing test.

Detailed discussions are made from next page.

CHAPTER (1)

1/1 Introduction

The purpose of this work is a prefeasibility study for white Nile new Bridge foundation design .

The width of the river along the Bridge axis is 550 meters and depth to river bed ranges from few cms. to 1.5meters. The water gets deeper going from Khartoum direction towards Omdurman where at the center line gets its max. depth 1.5m (Sounding cross section No1).

Along the river axis the Nile silts, clays and fine sands and gravel are found to have a thickness ranging between 8m-12m.
This quaternery formation overlies the cretaceous Nubian sandstone formation which is mainly composed of consolidated clays ,sands , muds ,silts and the grains were cemented either by silica or ferrioxides on other hand .
This formation act as the most important aquifer for fresh groundwater throguh out the Sudan.

This report contains only the requested information by JIACA team but in the near future and after recieving the lab Data , consolidation test Parameters, shearing test factor and other data we are going to have a complete report with soil classification chart , plasticity chart and bearing cap. from both consolidation tests and standered penetration tests .

In appendix 3 the Geological map and the cross-section are compiled by Dr.E.M .Saeed (Hydrogeology of Khartoum province and Northern Gezira Area . March 1976) Bullitin NO 29. the cross -section were presented here as to give alight on the Nubian formation that we penetrate by all six borings along the Bridge axis.

1/2 Site Location

The site is located about two km south of the old white Nile Bridge . The bridge axis crosses the river at the entrance of Sunut forest.

1/3 Method of Investigation

A team from the Geology Researches Authrotiy of the Sudan equiped with Adico 36, Adico 40 drilling rigs , four floating tank (GRAS) and a barge from the river tranp.coop.started the boring on site at the first of June 1989 and finished the work on 23 of July 1989.

1/4 Climate

Khartoum is situated at the northern part of savanna belt .

It has ahot summer (April -October) with little rain fall and cold dry winter . The early summer weather is dry and unstable developing frequent severe dust storms known locally as (Habub) causing reduced visibility .

1/5 Geomorphology :-

1/5/1 Area West of the Nile

The Geomorphology of this area is some what different from the eastern one. The high relief lands forms developed through petrographic and finer structural control - the petrovariance of the relief are Merkhiyat jebels, (lat. 15^0 42N') long. 32^0 25'E made of clastic Nubian sandstone capped by aresis .Ferruqinous sandstone Jebel El Toriya composed of basalt (before quarrying) mades acircular mound .

Abu Shaaf range , lat.15^0 32'N long 31^0 51E) stands also due to the resis. of its cap beds which are highly ferruginous.

Sanddune ,the Qoz particularly well developed at the west, are saif -type alined in more or less N-S direction and extending up to 300 km with a width of about 7km e.g Qoz Abu dullu .

1/5/2 Area East of Niles

Geomorphic features noticed are attributed either to structural control or rock structural one.

Examples of the reflection of faulting , fracturing and corresponding topographic high are Jebel Ubeid Essid (latit $16^{0}12'$N and long 33^{0} $12'$E) and Jebel Habis Rocky range (lat .16^{0} $02'$N long. 33^{0} 09 E) both features trending SSW to NNS .

They form elongated ridges extonding up. to 12 km Jebel Surat e.g is a result of differential mechnial weathering which is a rock mass semi circular in shape composed of sandstone with acap of silicified sandstone .

Jebel sileitat on the other hand is of granitic composition standing conspicuous above the ground surface due to the relative deep weathering and erosion of the circum- adjacent rocks.

CHAPTER (2)

2.1 Geologic setting

With the exception of the granite gneiss complex to the north of Khartoum known as Sabaloka and the very out crops of basement at Jebel Awlia to the south of Khartoum and granitic outcrops at Jebel Silietat the region is covered by recent alluvium and Nubian formation .

2.2 The Alluvium

There are the white Nile and blue Nile deposits overlying the Nubian formation .They are formed of black clay and silt , occasionally containing calcareous concrelions .

2.3 The Nubian formation

Most of the Nubian sandstone outcrops, small hills and structural surfaces ,are on the western side of the Nile. The formation is characterized by cross-bedding sand sedimentation, ferruginous concretions lenses of koalinite of varied colours with a clear domination of red colour .When the rocks are stratified the beds are often horizontal , or slightly dipping and their thickness varies from some centimeters to several meters . The most common type of cross-bedding is the tabular plannar characterized by small inclined lamellae truncated by horizental surface (M.K.Omer 1975) In this detrital facies , four main sequences can be distinguished :

- the conglomerates and the gravels .
- the ferruginous concretions and crust .
- the sandstones.
- the clays .

2.3.1 The conglomerates and Gravels

They are intraformational flat bedded or lenses -

polyegenic-

They often cemented by arenaceous ferruginous material, their clasts are composed of rounded quarzitic gravels.The gravels are found sometime disseminated in the sand beds with out any preferential orientation.

2.3.2 The ferruginous concretions

These are fðund in the conglometrate beds ,or in the zone of contact between .sandy and mudstone laydr.They are dark red rounded ferrugionous ones. They are believed to be fragments coming from the disintegration of lat- eritichorizons, and then transported and deposited in the new orelying beds.

2.3.3 The ferruginous crust

They occure in the field in different ways sometimes are realy rich in iron, the Feo represent 53% . These ferru inous crusts occure in three way^{2}s^{3}(M.K.omer 1975).

a.At the foot of the sandstones oblique to them.

b.Between two strata in concordance with stratigication, , but then wedge out very quickly. Such crust correspond to old soils contemporaneous with the sedimentation.

c. Somtimes the crust envelope completely alense of sandstone.

2.3.4 The clays and Mudstone.

The clays occure in small beds but often in lenses of different thickness and of different horizontal extension. They may have adepth of only some meters-shmbat bridge ,report 1957 or adepth of some hudred meters.

- Gedlogical Research's Authority of the Sddan open file reports Merikh clup borehole 1000ft, Omdrman Mosque borehole 1974.

Inspite of the importance of vention extention of the clay lensses their horizental extention is rather limited ,and they seem to be tøø important to be deposited in rivers and too deep to be deposited in small lakes.

- The more important lenses in Khartoum -Omdurman region are localized in azone having approximatly adirection NW-SE ,From this zone, northward andsouthward ,the sediments generally become coarser ,the gravels and conglomerater more abundant

(for example Jebel Aulia in the south and Jebel Abu Waleidat in the north of Khartoum M.K Omer 1975.

- The clay is a koalinitic clay contain in very fine crystallized quartz and geothite , and free of gravels and coarse sands .

2.3.5 Sandstones

The sandstones are generally of siliceous nature . They are composed of quartz grains and very few grains of other minerals . The cement is either cilicous or argillo- ferruginous In certain sandstones - Merkhiyathills - the cement is less a bundant and composed of silty material .In the different types, of the sandstones , the clay fraction was extracted and its mineralogical nature was determined by X-ray diffracture methods (M.K.Omer1975) It was found that a well crystallized kaolinite is the unique clay mineral and some traces of illite were detected in only one sample .Acomplete analysis was also carried out on representative samples of the different types of standstones of the Nubian formation using x-ray methods .The clay and standstone are essentially composed of the following:-

- Well crystallized quartz very abundant
- Geothite
- fraction inferior to 2 microns is composed of kaolinite only.
- traces of geothite -not alway present.

CHAPTER (3)

3. Geological Setting of Site under question

3.1 Boring NO1

This is located at 310m from the western bank of the white Nile towards the centre of river the depth of boring is 30 meters.

The formation penetrated by the borehole is as follows- see appendix 2-

- silt, clays , sands ,gravels
- mudstones - sands stones

3.1.1 Silt and clays

These are plastic dark black containing miacateous matter also and $caco_3$ concretions .This deposit goes down to 9 meter and some times mixed with fine angular pure quarzitic sand e.g from 3.0m- 3,30m. The colour of the deposits changes between red to black and this due to iron solution concentration At 9m the formation changes for coarse sand and fine to medium grained gravels these sands and gravels are resulted from the disintegration and deep weathering of a Nubian formation , the grains are angular to subangular indicating that transportion distance is very short .

3.1.2 Mudstones andsandstones

This formation starts at 9m and up to 12.20m the mudstone is found rich in sand and with various colours reddish - dark brown , yellowish and Pinkish - Rianbow colours and mostly this due to iron solution and organic matter concentrations . The sandstone starts at depth 12.20 m with various colours - whitch , yellowish grey,yellowish and reddish and this due iron solution staining .

This formation goes friably and deeply weathered up 28m in depth and it becomes hard and sounding The grain size ranges between fine and medium angular to subangular .

3.2 Boring NO 2

This borehole is located on the western bank of river i.e 30cm from bank in water the depth to river bed is about 60cm boring depth is 19.0m - for logging see appendix 2-

- silt ,clays ,sand
- mudstone and sandstones

3.2.1 silt clay and sands

The fine grained sands brown in colour starts from o uptto 1.70 ,then follows the silty clay dark in colour and plastic and up to 5.0 meters the clays becomes more sticky changes its colour to reddish due iron solations ,from 5 meters the clays start tohave sands at 7 meters the sand becomes very pure quarziti grains at 9m the mudstones starts in friable and highly weathered conditions.

3.2.2 Mudstones and sandstones:

This continues from 9-12,20 as usual pinkish madstone follow by yellowish then reddish ores due to iron axides staining and patching ,the rock is plastic probably due presence of koalinite or illite groups also it shows amanner of hardness. The sandstones begins at 12.20m up to 19.0m where at the first meters it yellwish highly weathered then at - 17.m the colour changes to reddish and the rock is hard and sounding .

3.3 Boring NO 3

The borehole is located in the eastern bank of ,the white Nile the total depth is 15.0m the boreholes log is as follows :-

- Clays , sands ,silts, gravels
- mudstone , sandstone.

3.3.1 Clays, silts, sands and gravels

From zero down to 6.om we have alight yellowish silty clays these are typical river deposits and they contains little black mica and river shells from 6.0m down 8.o we struck alight brown fine sand .

3.3.2 Mudstones

From 8.0 down to 15.0m we penetrate a various colours mudstone yellowish ,brown ,pinkish and whitel. these muds contains little sands and they are hard and sticky when they are saturated with water.

3.4 Boring No4

This one is located at Omdurman side about 50m away from boring NO 2 which on shore .The formation penetrated by this boring is composed of.

- clay ,silt ,sand
- mudstones

3.4.1 clay,silts and sands

These as previously stated they belong to quaternery deposits , they resemple the white deposits. From 0.00m in depth upto...5.0m in depth we penetrate a highly compact and sticky silty clay . The clays have iron oxides staining and batch's sometime they are dark black due to organic matter .

Also they contains shell fraqment, and micacious flacks , from 5.0m the clay begin to have fine sands and are dominent and clays show alittle presence also the silts are dominent .

3.4.2 Mudstone

These start at 10.0 m in depth up to 17.30m, at the first meters they have in sands ,but bellow 12.0m They are free of sands , they show variaty of colours , the dominent ones are yellowish , pinkish reddish and whitch due to organic matters , iron oxide solution and koalinite mineralization.

3.5 Boring NO 5

This borehole is located about 50m East of boring NO.3 It lies an land Elmugran side . The total depth is 17.30 m.The soil penetrated by this boring is as follows:-

- Clay ,silts , sands
- Mudstones.

3.5.1 Clays ,silts and sands

These deposits started from o level down to 7,20 m they are brown silty sand with little micacious materials they show a' matter elasticity some times the silts are dominent over the sand and Vica versa. From 7,20 m down to 7.0m we penetrate avery plastic brown clays where we took our undisturbed sample.

From 7.0 down 12.70m we met silty sands , puee fine sand and gravely sands there of coarse are disintegration product of Nubian formation.

3.5.2 Mudstones & sandstones

At depth 12.70 asandy yellowish very friable mudstones appear in the drilling water these muds goes down to 14m in depth , then we penetrade a very friable highly weathered yellowish sandstone as we fail to get core samp,e using core barrel except a five cm sample , the disturbed samples that we got were from water circulation all the above fact indicate that the rock is highly weathered this formation goes down to 17.30m in depth where we stop drilling according to a written note from JIACA team .

3.6 Boring NO6

This borehole is located at Omudurman side about 1.0 km away to the West of boring NO.4 .There is an aoutcrop of Nubian sandstone formation 25m to West of boring NO.6.

In the floading season the area of boring NO.6 is total covered with water and the river reach the past mentioned outcrop (25m W.of B.H.NO.6). the soil penetrated by this boring is a follows .

- clays (koalinite ,) sands ,gravels.
- Mudstones sands stones and conglomenrates .

3.6.1 Clays ,sands and gravels

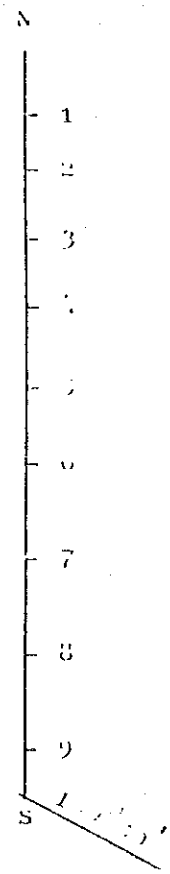
From zero down to 6.0m we penetrate gravely sand formation the sands ranges from fine upt to coarse grained they are angular in shape the gravels are fine and angular in shape the formation contains rock fragment (sandstones, mudstone and ferrugonous crust).
There fragment are transported by the river and deposited. Down to 7.5m we have sandy gravely clays from 7.5m to 10.0 m we have fine to coarse gravely sands at 11.0m in depth we penetrate a lense of koalinite where we have been stucked the lense is about one meter thick.
From 10.0 m down to 14.0 we penetrate gravely aand formation the sands ranges between fine and coarse some time we have pure sands from 14.0 down to 15.0 we stracka yellowish sandy mudstones.

4.Boring location and leveling

Refereing to the mean sea level and from abolt or N.E sie of Omdurman bridge (Point NO 8732 of value 380.880). Nine points of equal distances 150m and running in N.S direction were been selected starting from the bolt .The values of these points were tabulated as follow

P.1	P.2	P.3	P.4	P.5	P.6	P.7	P.8	P.[illegible]
3,5.56	375.02	375.81	375.92	375.5	375.55	375.71	[illegible]	[illegible]

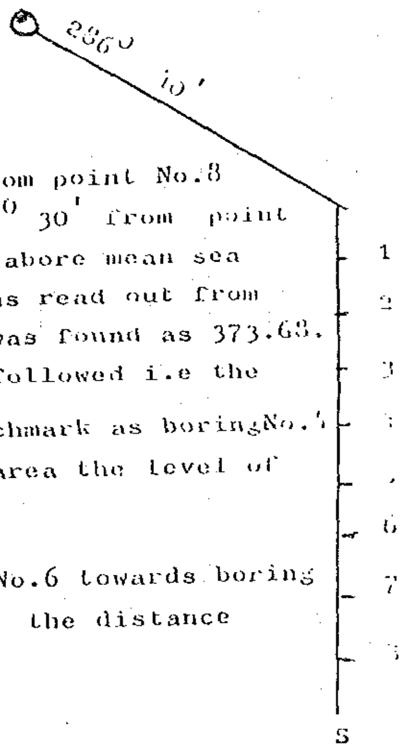
From Point No.9 to B H No.5
The direction of $150^{0}\ 50'$ and distance of 175 were been calculated .The level of this bore hole is found to be 375,18 the bearing from bashole No.3 was $273^{0}\ 2[illegible]'$ and it is 374.92m abore sea level . The same meth[illegible] was carried for Omdurman side where .We have a benchmark No.9 of high 784.714.Aseries of points with b 100m distance were considered starting from the Benchmark towards boring No.4 The bearing of these points from the Benchmark is 286^{0} 40 then the points were ploteo in NS direction as we see.



P.1	P.2	P.3	P.4	P.5	P.6	P.7	P.8
37 .72	375.07	375.56	375.67	375.23	375.11	375.9[illegible]	37[illegible]

Benchmark No.9

384.714



Boring No.4 is of 374.96
bearing in south direction from point No.8
boring No.2 is located at 251° 30' from point
No.8 and its level is 374.8m abore mean sea
level .Abearing of 250° 46 was read out from
boring NO .1 and It . level was found as 373.68.
For boring No.6 The same is followed i.e the

leveling starts from the benchmark as boringNo.4
and 2 were located in fload area the level of
boring No.6 was 375.74.

The direction from borehole No.6 towards boring
No.5 was found to be 265° 30' the distance
was 667.6m.

HOLE NO 5

RECORD OF S.P.T

DEPTH	LOG	BLOW Number 1st 6"	2rd 6"	3rd 6"	Total per 18"	N Value per 12"=300
0_70		1	-	-	-	-
1_1.70		1	-	-	-	-
2_2.70		28	-	-	-	-
3_3.30		8	11	-	19	15
3.30_3.60		3	5	-	8	5
4_5			Circulation			
5_6						
6_7						
7_8						
8_9		Core	friable	circulation		
9_10.60						
10.60_12.10						
12.10_13.60						
13.60.15.15						
15.15_16.70						
16.70.18.20						

RECORD OF S.P.T

HOLE No.2

Depth	L O G	Blow Number			total per 18	N value per 12 30cm
		Ist 6	2nd 6	3rd 6		
0-1		1	2	2	5	3
1 - 1.70		1	1	2	4	3
2 - 2.70		4	8	10	22	7
3- 3.70		3	5	8	16	17
4- 4.70		2	3	3	8	5
5- 5.30		2	4	-	6	4
50- 5.60		7	10	-	17	11
5.60-5.90		3	6	-	9	6
6- 6.70		6	13	14	33	11
7. 7.70		5	10	11	26	17
8 -8.70		9	10	12	31 .	27
9 -9.70		36	35	60	131	87
9.70-11.45						
11.45- 13						
13- 15						
15- 16						
16- 17.55						
17.55.19						

RECORD OF S.P.T HOLE No.3

Depth	LOG	BLOW NUMBER 1st 6"	2nd 6"	3rd 6	Total per 18"	N Value per 12" 300
1 ______ 150		6	4	5	15	10
2 ______ 2.70		4	3	3	10	7
3 ______ 3.50		1	3	2	6	4
5 ______ 5.30		3	-	-	3	2
6 ______ 6.70		2	2	11	15	10
7 ______ 7.30		3	7	30	40	27
8 ______ 8.30						
9 ______ 9.60		58	-	-	58	39
9 60 ______ 11.20						
1 .20 ______ 12.45						
1[illegible].45 ______ 14.45						
14.45 ______ 15.10						

HOLE No 4

RECORD OF S.P.T

DEPTH	LOG	BLOW NUMBER			total per 18"	N value per12"= 300
		1st 6"	2rd 6"	3rd 6"		
0_1		7	13	18	38	25
1_1.70		4	4	5	13	9
2_2.70		1	2	4	7	4
3_3.30		4	4	_	8	5
3.60_3.90		-	-	-		
4_4.70		3	4	4	11	7
5_5.70		3	2	2	7	4
6_6.70		2	4	7	13	9
7_7.70		7	8	15	30	20
8_8.70		10	15	20	45	30
9_9.70		14	20	33	67	45
10_10.30		39	Water circulation core			
10.30. 10.50						
10.60_11.50						
11.50_13						

HOLE No 5

RECORD OF S.P.T

DEPTH	LOG	BLOW Number 1st 6"	2rd 6"	3rd 6"	Total per 18"	N value per 12"=300
o_1.70		3	4	3	10	7
2_2.70		3	3	4	10	7
4_4.70		8	11	13	32	21
5_5.30		6	8	-	14	9
6_6.70		2	2	1	4	3
7_7.30		5	6	-	11	7
7.30_7.60		2	3	1	5	3
8_8.70		3	4	7	14	9
9_9.70		2	2	1	5	3
10_10.70		11	5	8	24	8
11_11.70		Water circulation sample				
12_12.70		31	54		85	57
13_13.50						
13.50_14.50						
14.50_15.50						
15.50_16.30						
16.30_17.30						

HOLE No 6

RECORD OF S.P.T

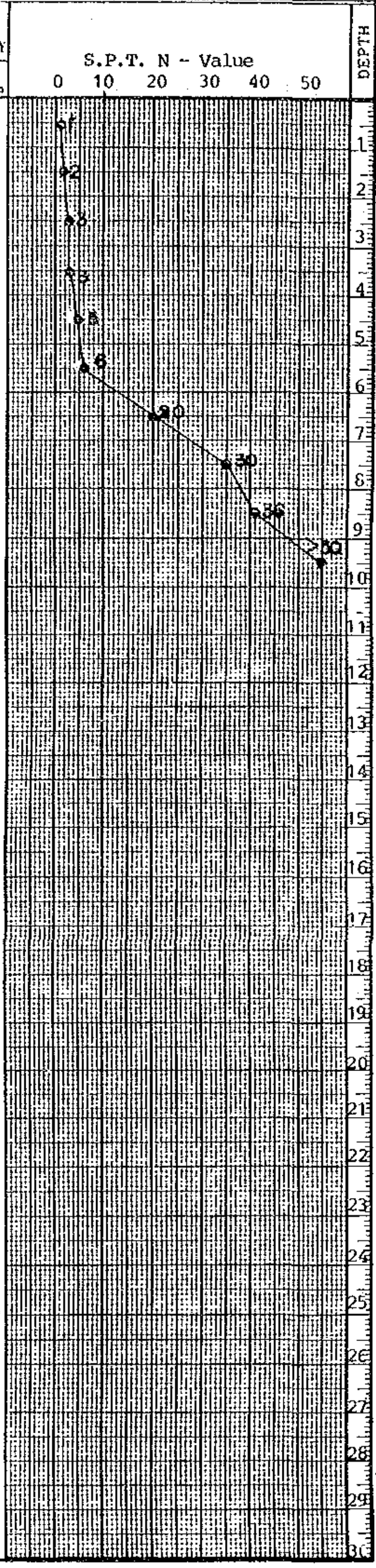
EPTH	L O G	BLOW 1st 6"	Number 2rd 6"	3rd 6"	Total per 10"	N value per 12"a500
0.50_0.95		3	10	23	36	24
0.95_1.45		14	18	33	65	40
1.45_2.50		-	-	-	-	-
3_3.70		4	3	7	14	9
3.70_5.70		5	7	16	28	19
15.70_6.70		11	22	50	83	55
6.70_7.70		-	-	-	-	-
7.70_8.70		27	66		93	21
8.70_9.50		15	30	18	63	42
9.50_10		-	-	-	-	-
106_11		Core				
11_12						
12_13						
13_14						
14_15						
15.10						

DRILL LOG

HOLE NO. B-1 SHEET NO. 1 OF 1

PROJECT	The F/S on the Construction of the New White Nile Bridge			DEPTH	30 M	ELEVATION	373.3
SITE	St.No 27+62 (in the river)	COORDINATE	: :	INCLINATION	Vertical	DRILL RIG	LNB - 200
AVERAGE CORE RECOVERY	70%	DATE	FROM 7 Jul TO 12 Jul	DRILLED	Mr. Yashin	LOGGED	Mr.S.Ikeda

DATE	DEPTH	ELEVATION	ROCK TYPE OR FORMATION		COLUMN SECTION	DESCRIPTION	BIT & DIAMETER	GROUNDWATER LEVEL	CORE RECOVERY %	cm
	4.0	369.3	Alluvial Layer	Clay		0.0 - 4.0 Clay. Current river deposit. Homogeneous. Very soft and high plasticity.		GL + 1.7m		
	6.0	367.3	Alluvial Layer	Clay		4.0 - 6.0 Clay. Silt fraction a little. Slightly soft consistency.				
	9.3	364.0	Alluvial Layer	Sand		6.0 - 9.3 Sand. Homogeneous fine sand. Poor grain size distribution. Medium density.				
	30.0	343.3	Nubian Formation	Sand stone Inter laid Mud stone		9.3 - 30.0 Sandstone. Basal rock layer. Consists of medium grain sandstone. Partially, mudstone interlaid. Weathered and ferrificated. Moderately solidified. Stiff enough for bridge foundation. 9.3 to 13.0 m. Decomposed and loosened. 15 to 16 m. Hravily ferrificated, and very hard. 25 to 30 m. Gradually sound and hard.				



HOLE NO.

LOG FORM-B

* R.Q.D is Rock Quality Designation, R.Q.D=(Total length of cylindric cores longer than 10 cm)/(Total core length) x 100%
* LUGEON VALUE is l/min/m under injection water pressure of 10kg/cm²
* DEPTH and ELEVATION are in meter
* DIAMETER is in millimeter

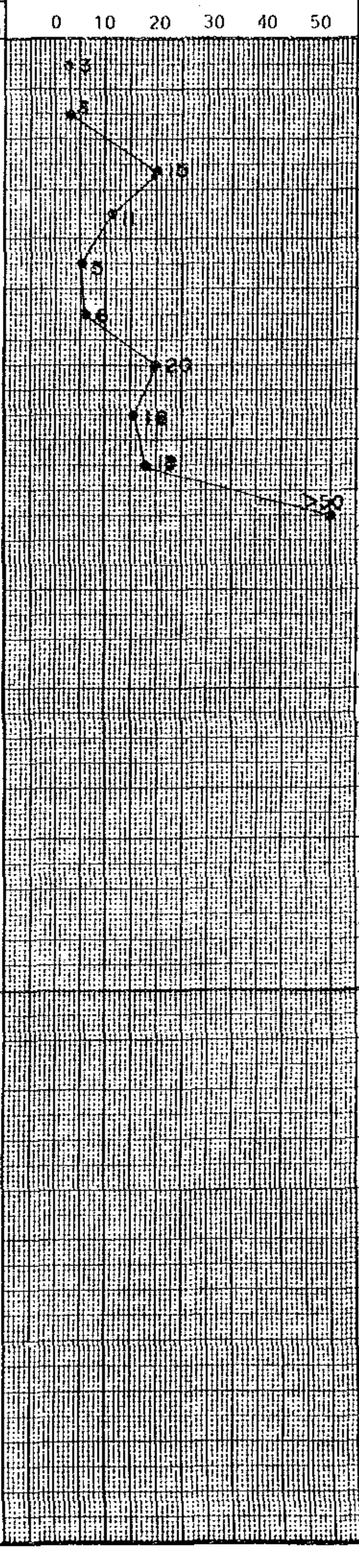
DRILL LOG

HOLE NO. B - 2 SHEET NO. 1 OF 1

PROJECT	The F/S on the Construction of the New White Nile Bridge			DEPTH	19 M	ELEVATION	374.3 m
SITE	Left Bank (in the water)	COORDINATE	: :	INCLINATION	Vertical	DRILL RIG	ENB - 200
AVERAGE CORE RECOVERY	65%	DATE	FROM 21 Jun TO 25 Jun	DRILLED	Mr. Zuheil	LOGGED	Mr. S. Ikeda

DATE	DEPTH	ELEVATION	ROCK TYPE OR FORMATION		COLUMN SECTION	DESCRIPTION	BIT & DIAMETER	GROUNDWATER LEVEL	CORE RECOVERY % / cm
	6.0	368.3	Alluvial Layer	Clay		0.0 - 6.0 Clay Silt fraction 10 to 15%. Homogeneous. High plasticity. Medium hard in consistency.		GL + 50cm	
	9.0	365.3	Alluvial Layer	Sand		6.0 - 9.0 Sand Fine to Medium in grain size distribution. Uniform, homogeneous.			
	13.0	361.3	Nubian Formation	Mud stone		9.0 - 13.0 Mudstone. Stratified. Decomposed and weathered Partially iron oxide gatheres. Weakly consolidated.			
	15.0	359.3	Nubian Formation	Loose Sand stone		13.0 - 15.0 Loose Sandstone Decomposed and loosened. Fragile.			
	19.0	355.3	Nubian Formation	Sand stone		15.0 - 19.0 Sandstone. Medium grain. Decomposed and weathered Slightly weak in consolidation.			

S.P.T. N ~ Value



HOLE NO.

LOG FORM-B

*R.Q.D is Rock Quality Designation, R.Q.D=(Total length of cylindric cores longer than 10 cm)/(Total core length) x 100%
*LUGEON VALUE is l/min/m under injection water pressure of 10kg/cm²
*DEPTH and ELEVATION are in meter
*DIAMETER is in millimeter

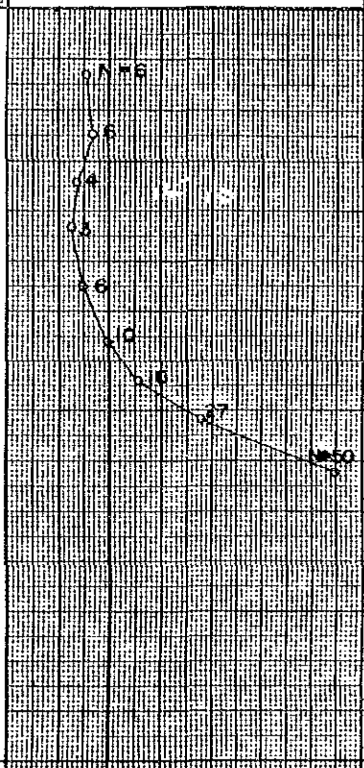
DRILL LOG

HOLE NO. B-3 SHEET NO. 1 OF 1

PROJECT	The F/S on the Construction of the New white Nile Bridge			DEPTH	15.0 M	ELEVATION	374.1
SITE	Right Bank (in the River)	COORDINATE	: :	INCLINATION	Vertical	DRILL RIG	INB - 200
AVERAGE CORE RECOVERY	70%	DATE	FROM 7 Jun TO 15 Jun	DRILLED	Mr. Zuhail	LOGGED	Mr.S.Ikeda

DATE	DEPTH	ELEVATION	ROCK TYPE OR FORMATION		COLUMN SECTION	DESCRIPTION	BIT & DIAMETER	GROUNDWATER LEVEL	CORE RECOVERY %	cm
	7.0	361.1	Alluvial Layer	Clay		0.0 - 7.0 Clay Silt fraction 10 to 15%. Homogeneous. High plasticity. Slightly soft.		GL + 50cm		
	9.6	364.5	Alluvial Layer	Sand		7.0 - 9.6 Sand. Very fine and uniform in grain size. Medium density.				
	15.0	359.1	Nubian Formation	Sand-stone		9.6 - 15.0 Base Rock. Sandstone in alternation with Mudstone. Weathered and decomposed. Sufficiently tight for foundation.				

S.P.T. N - Value



HOLE NO.

LOG FORM-B

*R.Q.D is Rock Quality Designation, R.Q.D=(Total length of cylindric cores longer than 10 cm)/(Total core length) × 100%
*LUGEON VALUE is l/min/m under injection water pressure of 10kg/cm²
*DEPTH and ELEVATION are in meter
*DIAMETER is in millimeter

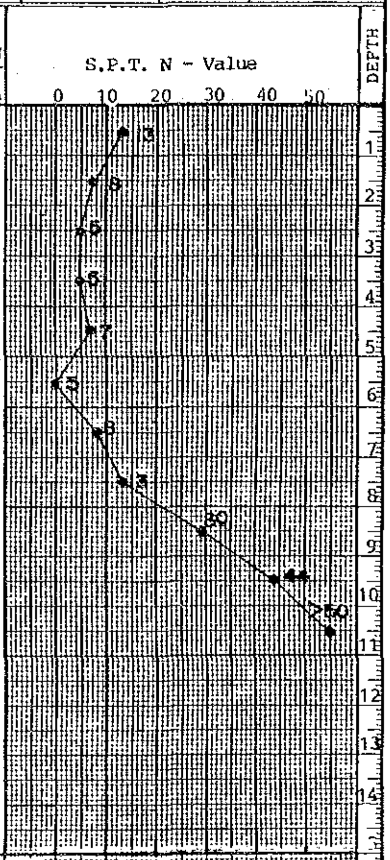
Appendix 3 (3)

DRILL LOG

HOLE NO. B-4 SHEET NO. 1 OF 1

PROJECT	The F/S on the Construction of the New White Nile Bridge			DEPTH	15 M	ELEVATION	375.7
SITE	Left Bank Abatment	COORDINATE	: :	INCLINATION	VERTICAL	DRILL RIG	ENB - 200
AVERAGE CORE RECOVERY	80%	DATE	FROM 24 Jun TO 30 Jun	DRILLED	Mr. Yasin	LOGGED	Mr.S.Ikeda

DATE	DEPTH	ELEVATION	ROCK TYPE OR FORMATION		COLUMN SECTION	DESCRIPTION	BIT & DIAMETER	GROUNDWATER LEVEL	CORE RECOVERY %	cm
	6.5	369.2	Alluvial Layer	Clay		0.0 - 6.5 Clay. Silt fraction a little. Homogeneous. High plasticity. Fairly well consolidated due to desiccation. Medium to hard in consistency.				
	10.0	365.7		Sand		6.5 - 10.0 Sand. Fine to medium grain. Homogeneous. High density.		GL - 6.5M		
	15.0	360.7	Nubian Formation	Mud-stone		10.0 - 15.0 Mudstone. Base rock layers. Interbedded with sand-stone. Weathered and decomposed. Partially hematite, iron oxide.				



HOLE NO.

LOG FORM-B

*R.Q.D is Rock Quality Designation, R.Q.D= (Total length of cylindric cores longer than 10 cm)/(Total core length) x 100%
*LUGEON VALUE is l/min/m under injection water pressure of 10kg/cm²
*DEPTH and ELEVATION are in meter
*DIAMETER is in millimeter

Appendix 3 (4)

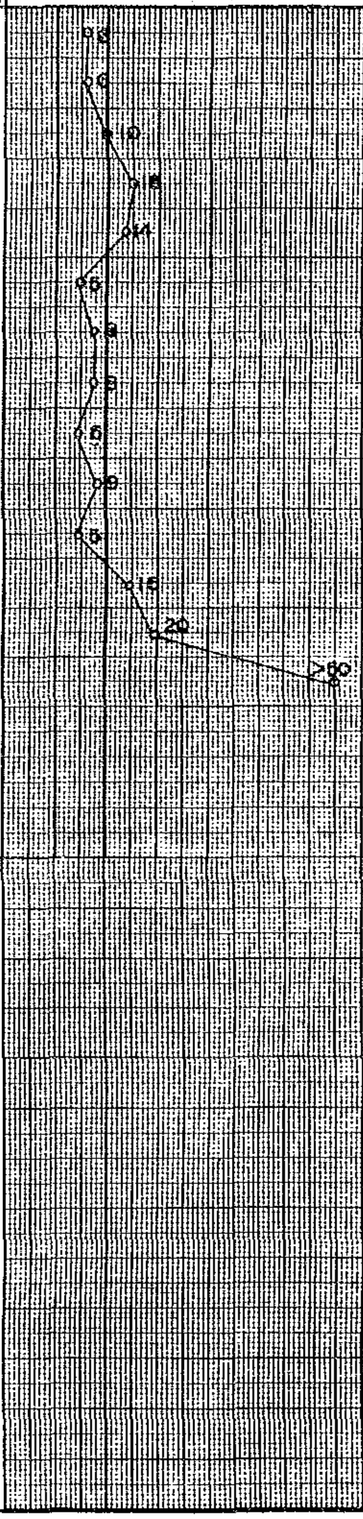
DRILL LOG

HOLE NO. B-5 SHEET NO. 1 OF 1

PROJECT	The F/S on the Construction of the New White Nile Bridge			DEPTH	17 m	ELEVATION	376.4 m
SITE	Right Bank (Abatment)	COORDINATE	: :	INCLINATION	Vertical	DRILL RIG	ENB - 200
AVERAGE CORE RECOVERY	75%	DATE	FROM 18 Jun TO 21 Jun	DRILLED	Mr. Yasin	LOGGED	Mr.S.Ikeda

DATE	DEPTH	ELEVATION	ROCK TYPE OR FORMATION		COLUMN SECTION	DESCRIPTION	BIT & DIAMETER	GROUNDWATER LEVEL	CORE RECOVERY (%)	CORE RECOVERY (cm)
	9.5	366.9	Alluvial layer	Clay		0.0 - 9.5 Clay. Silt fraction a little. Homogeneous. High Plasticity. Up to 5 m, Hard consistency due to desiccation. From 5 m to 9.5 Saturated. Soft consistency.		GL- 5.7 m		
	12.0	364.4	Alluvial layer	Sand		9.5 - 12.0 Sand Fine to medium in grain size. Hompgeneous.				
	14.5	361.9	Nubian Formation	Sand & Gravel		12.0 - 14.5 Sand & Gravel Heavily decomposed Nubian sandstone.				
	17.0	359.4	Nubian Formation	Sand-stone		14.5 - 17.0 Sandstone. Alternated with mudstone Slightly fragile.				

S.P.T. N - Value



HOLE NO.

LOG FORM-B

* R.Q.D is Rock Quality Designation, R.Q.D = (Total length of cylindric cores longer than 10 cm)/(Total core length) x 100%
* LUGEON VALUE is l/min/m under injection water pressure of 10kg/cm²
* DEPTH and ELEVATION are in meter
* DIAMETER is in millimeter

WHITE NILE BRIDGE
INVESTIGATION BORE HOLE
BORE HOLE NO.6

DEPTH (M)

WATER LEVEL

Depth (M)		DESCRIPTION
1		Medium to coarse sand and some fragment mudstone
2		Yellowish mudstone
3		Sand with flakes of mica
4		Coarse aand and medium gravel
5		Weathered sandstone
6		Yellowish sandy clay and fragment mudstone
7		Reddish sand -yellowish clay with some fine gravel
8		Fine to medium yellowish sand
9		Medium to coarse sand with fine to medium gravel
10		Yellowish -highty weathered sandstone
11		Koaline -very sticky with hard compact sandstone
12		Yellowish sand (river deposit)
13		Medium to coarse sand and medium gravelk
14		Fine sand
15		Conlomerate with some coarse gravel
16		
17		
18		
19		

Laboratory Test

INTRODUCTION :

This report describes the laboratory soil investigation undertaken by the Building and Road Research Institute, (BRRI) of Khartoum University (Faculty of Eng.) on behalf of JICA of Japan.

The aim of the investigation, is to provide geotechnical data required for the feasibility of the New White Nile Bridge.

The soil samples were sampled and collected by the Geological Department of the Ministry of Engergy and Minig and transported to the BRRI laboratory in disturbed and undisturbed forms.

The testing procedures followed were in general conformance with those recommended in British Standard BS1377:1975, Institute of Civil Engineers, London. A description of each test is given below :-

Atterburg Limits:

The method used in the determination of the liquid limit (LL) is by using the casagrande apparatus.

The plastic limit (PL) is also determined according to BS 1377:(1975) .

The LL and PL values are given in table 1.

Specific Grabity GS.

This was determined according to BS 1377:1975 (bottle) method. The Gs values are given in table 1.

Linear Shrinkage

Linear shrinkage (L.S.) is measured by using a mould 15cm long and IXI cm cross- section. The mould was first filled by a soil paste mixed at a moisture equal to the plastic limit. The paste was oven dried and its length was measured. The L.S. was then computed as the eduction in length expressed as a percentage of the original length.

Natural Moisture Content (NMC) and Dry Density

The moisture content and dry density of the undisturbed samples were determined following BS 1377:1975 methods. They are given in table(1)

Uncontined Compressive Strength UCS.

This was determined in the trianial machine according to BS 1377:1975. Seven UCS tests were carried out, five tests for undisturbed samples and two for yemoulded samples compacted to 90% OMC. The results are shown in Appendix B.

Shear Strength

Unconsolidated - undrained (UU) triaxial tests were performed on seven samples to determine the shear strength parameters C (Cohesion) and ϕ (angle of internal friction). Five samples were tested at field placement conditions (i.e NMC and filed density) and two compacted at 90% OMC. For each sample, three 38mm diameter specimens were tested after being subjected to contining pressures approximately ranging between 70-200 KN/m^2.

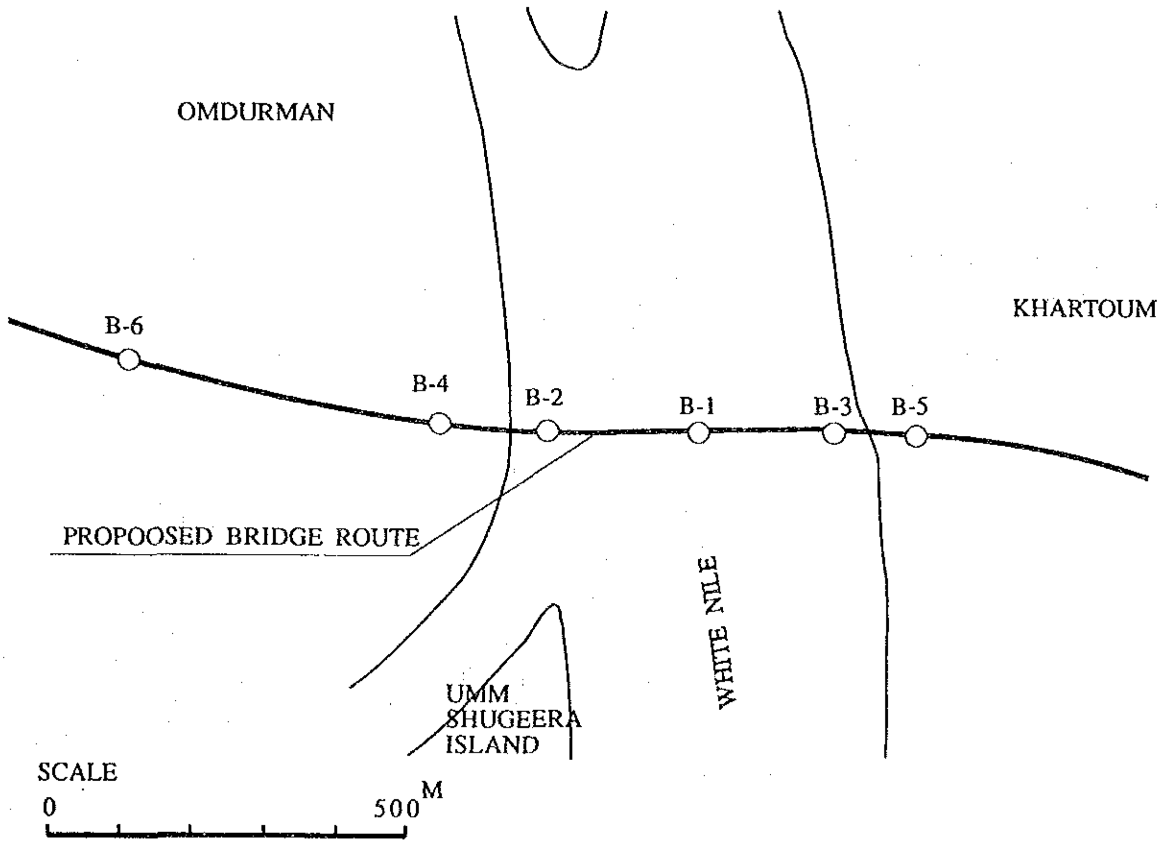
The Mohr circles and shear failure envelopes were plotted and the values of c and $\emptyset$ were determined for each soil sample as shown in Appendix C.

Compaction and California Bearing Ratio (CBR) Tests:

These were conducted for two samples, according to BS 1377:1975. (2.5 kg yammer dynamic compaction and normal CBR test) . The two compaction are illustrated in Appendix D and the CBR results are given in table 2.

Consolidation Test

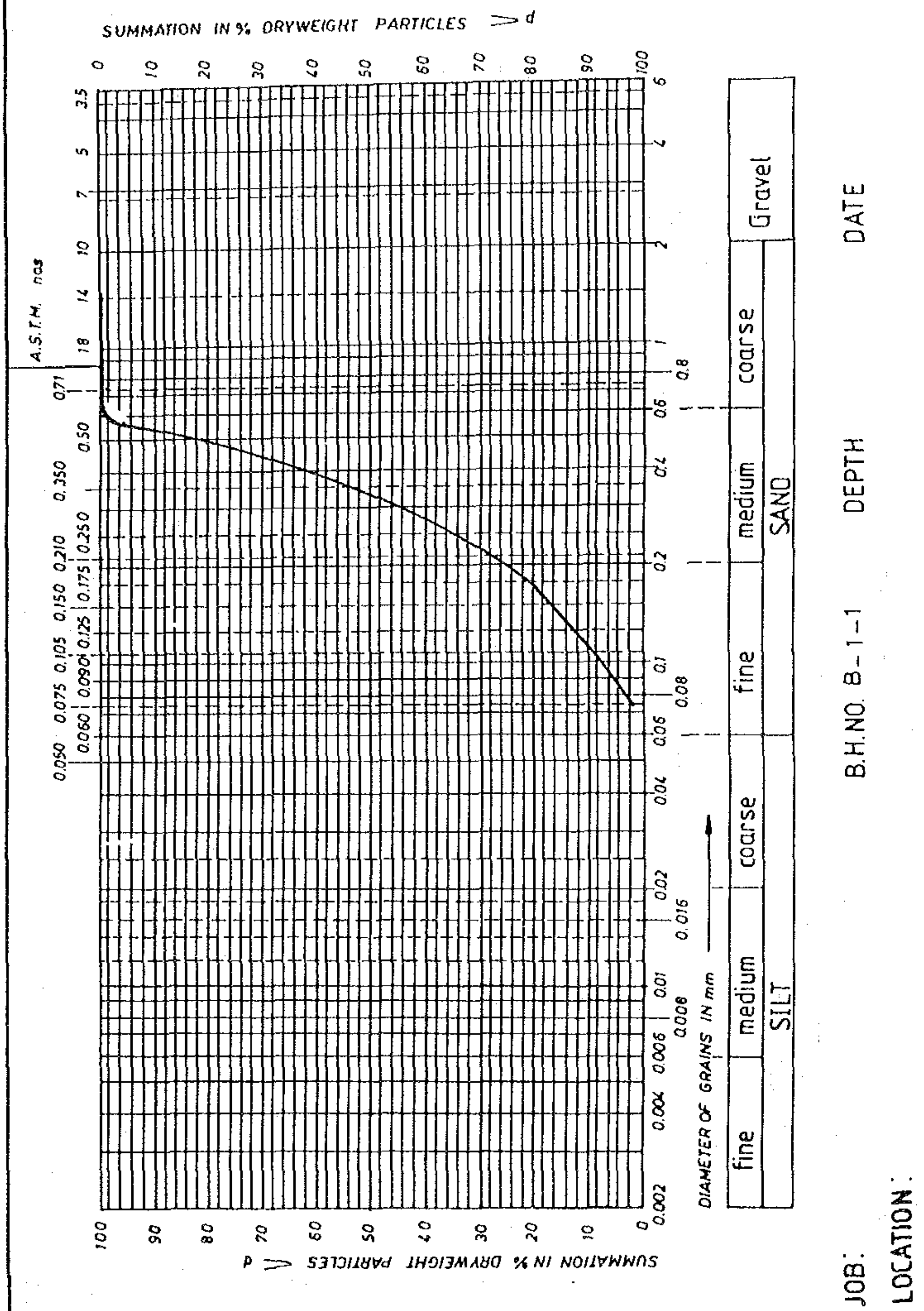
Consolidation tests were made on 7 soil samples, five at natural moisture content and dry density and two compacted at 90% OMC. The change in sample thickness was plotted against the applied pressure to obtain the consolidation curves shown in Appendix D.



PLAN

APPENDIX " A "

PARTICLE SIZE DISTRIBUTION CURVES



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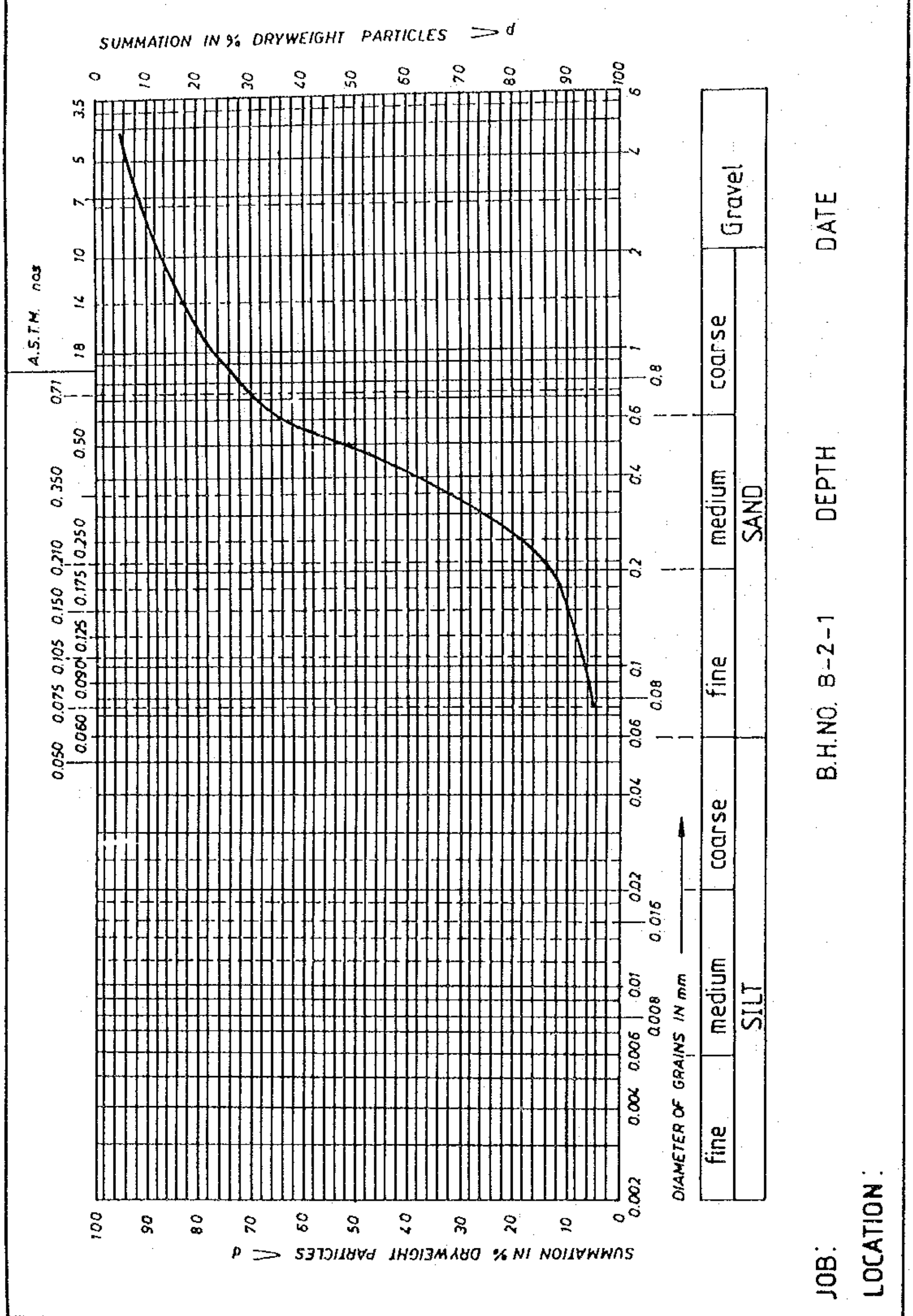
GRAINSIZE DISTRIBUTION

NOTES



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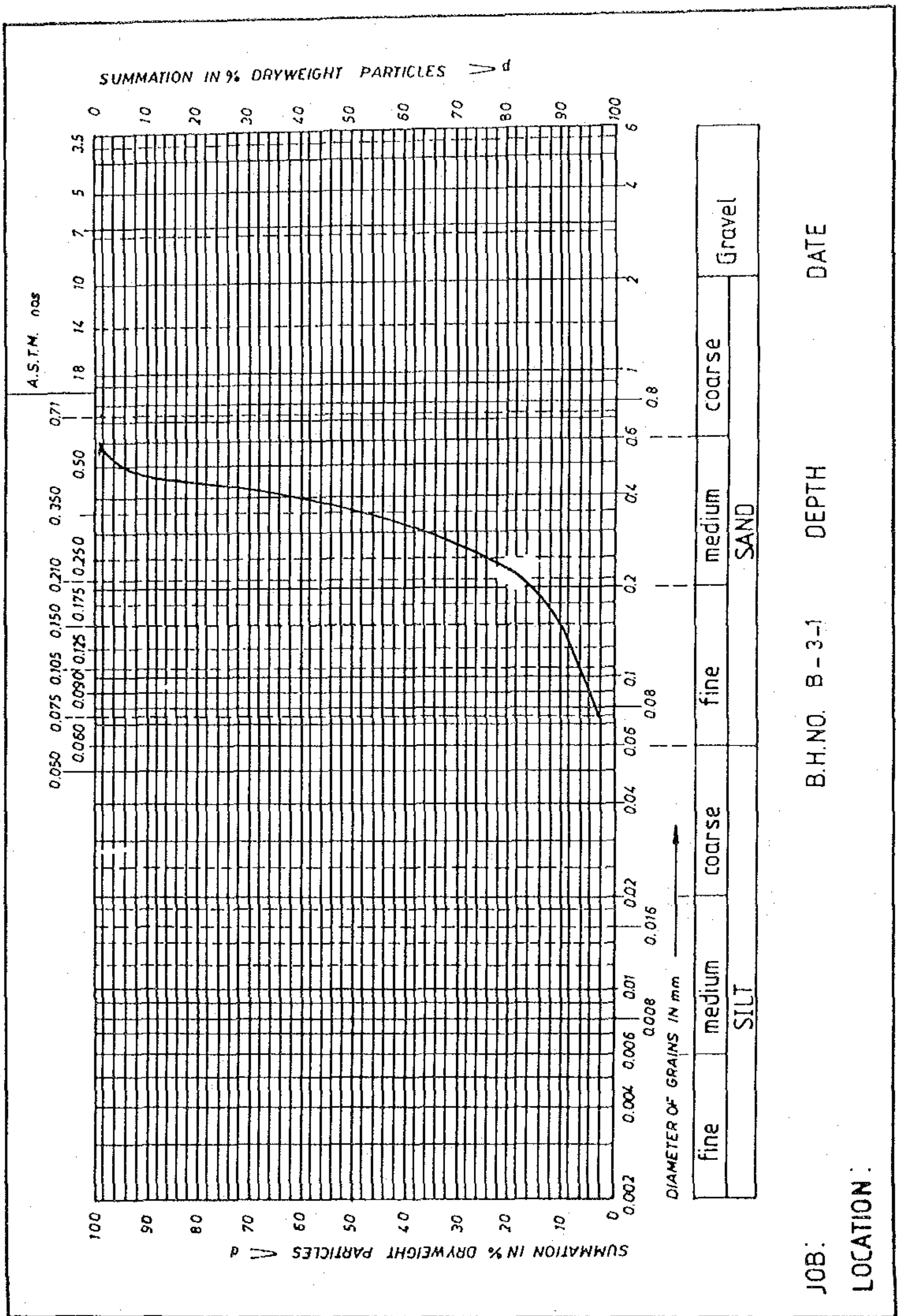
GRAINSIZE DISTRIBUTION	NOTES



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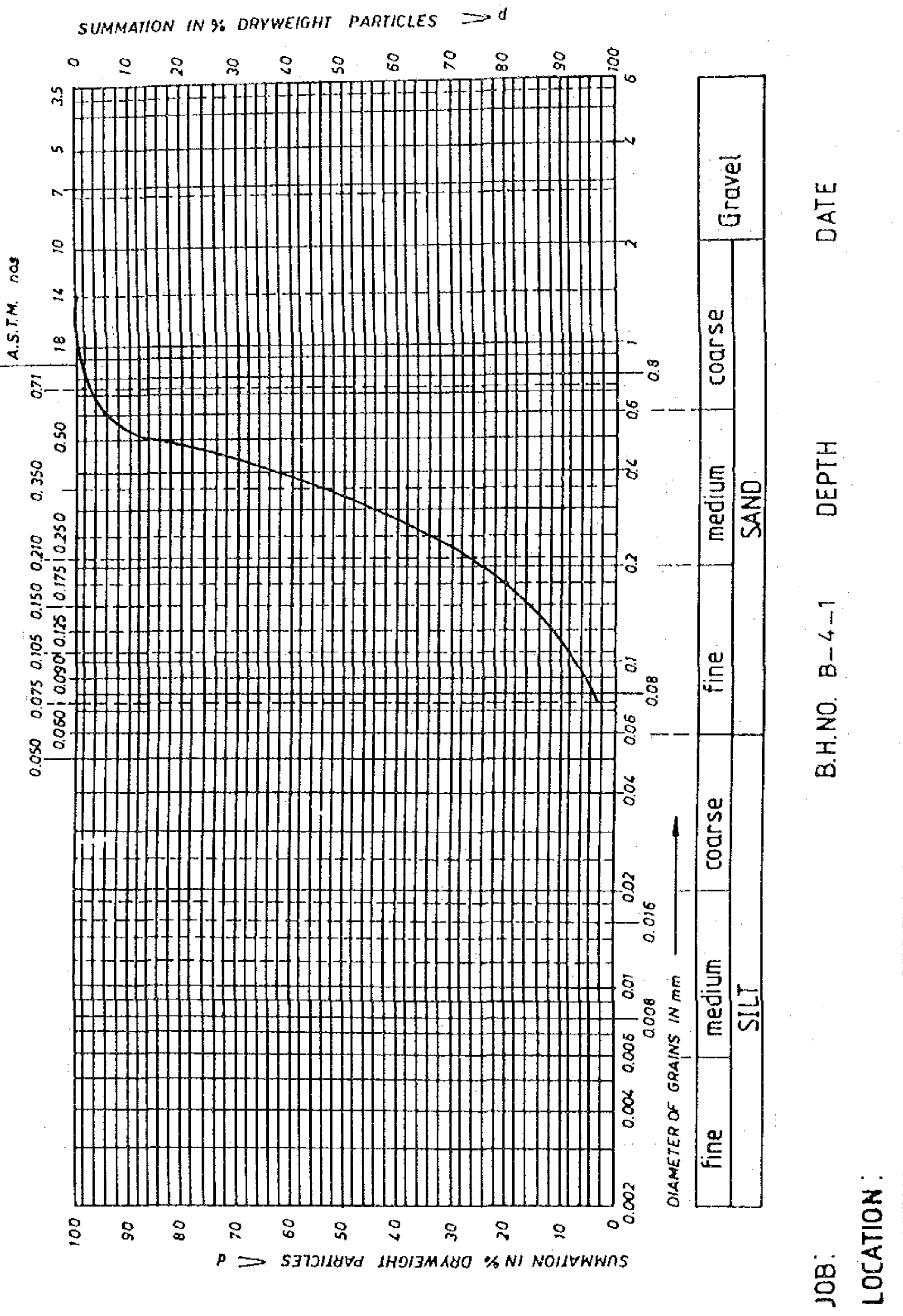
GRAINSIZE DISTRIBUTION	NOTES



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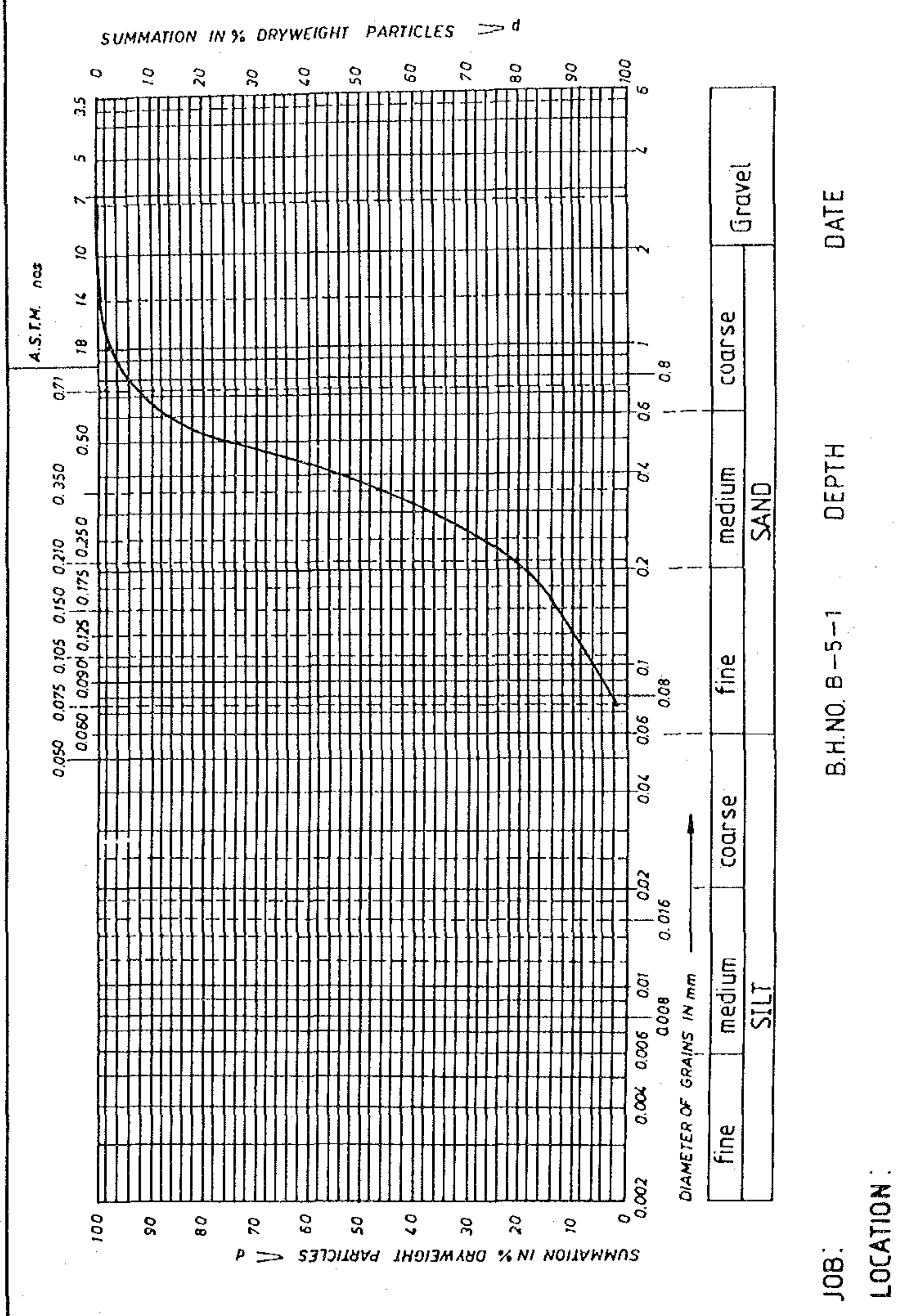
GRAINSIZE DISTRIBUTION	NOTES



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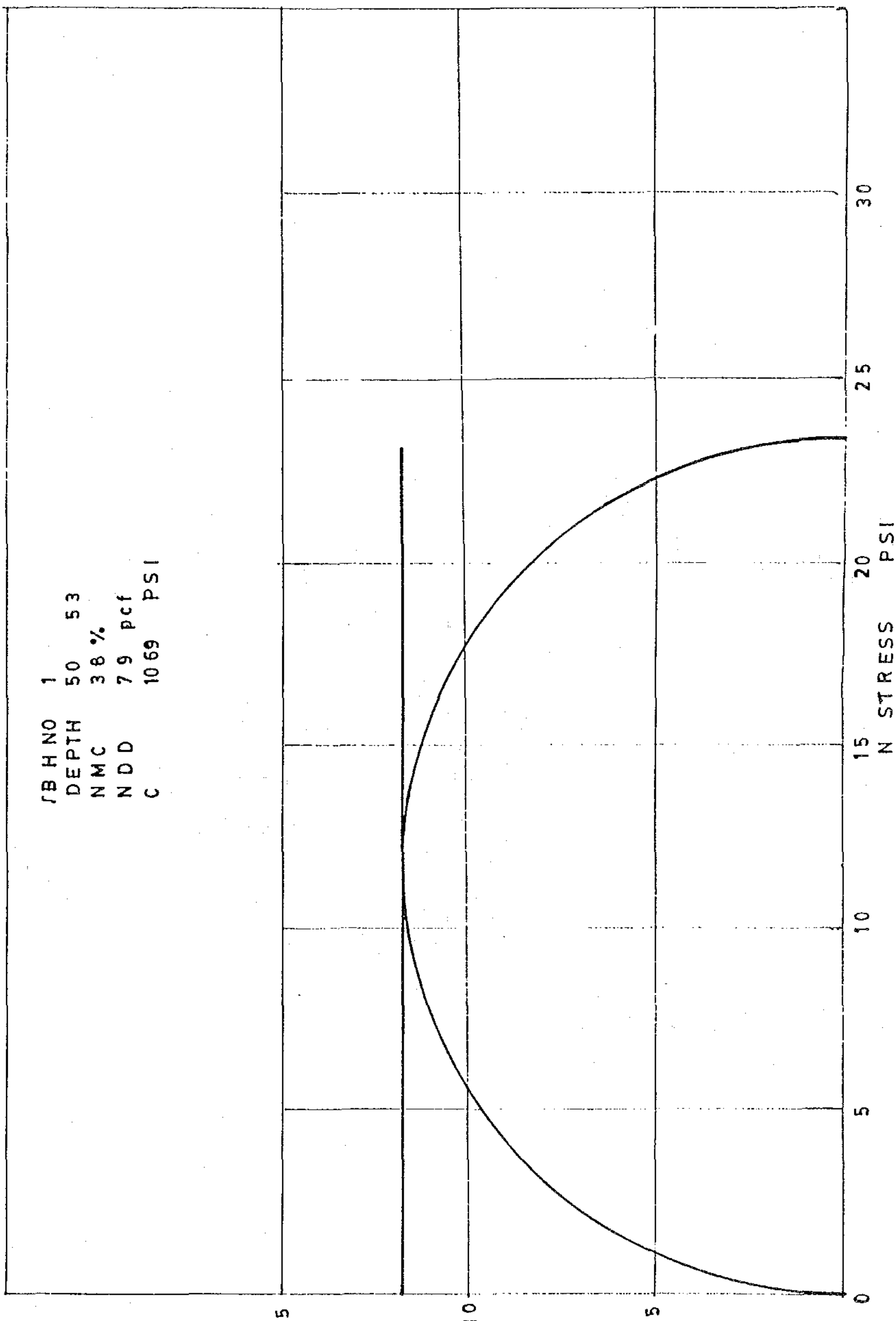
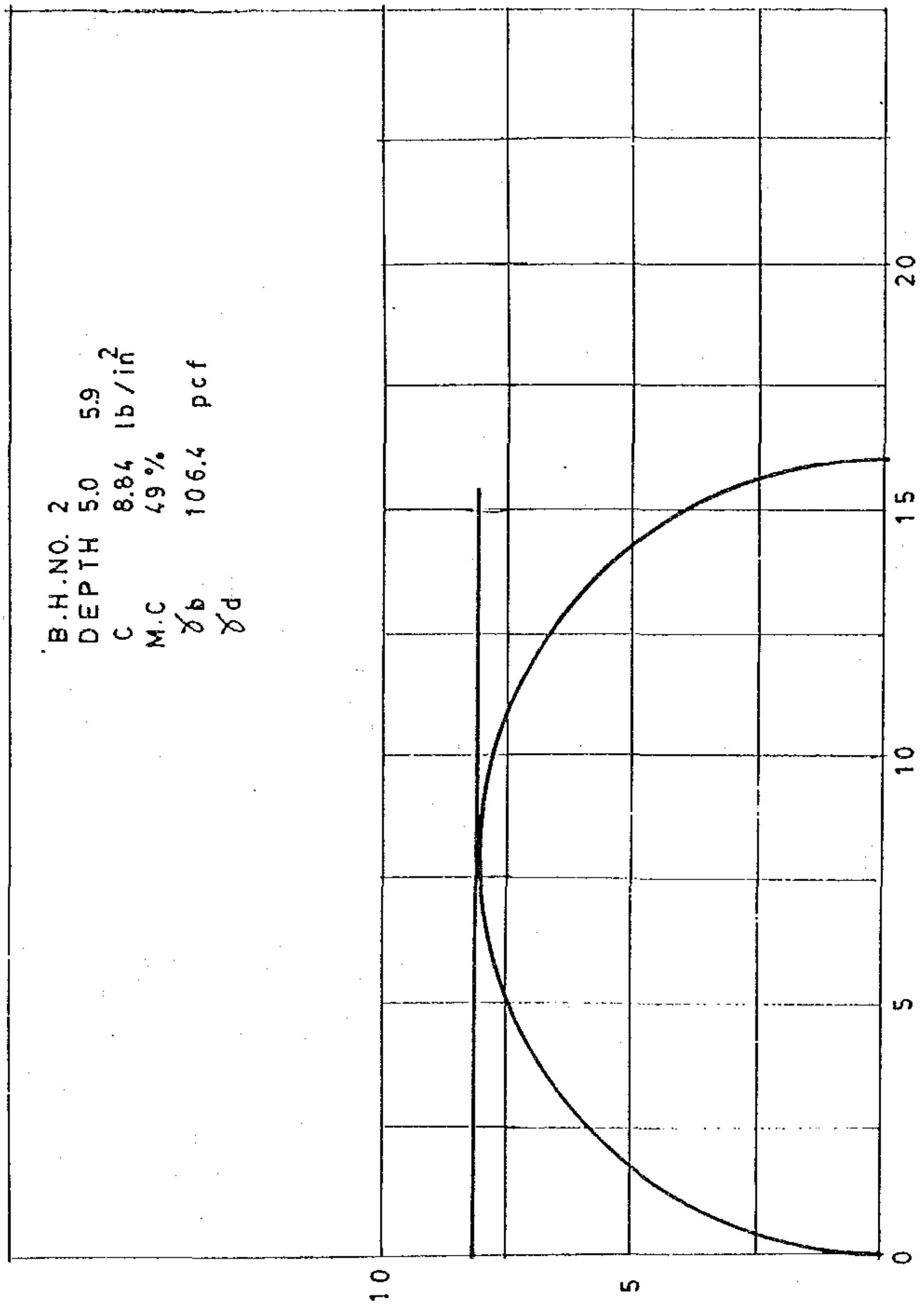
GRAINSIZE DISTRIBUTION	NOTES

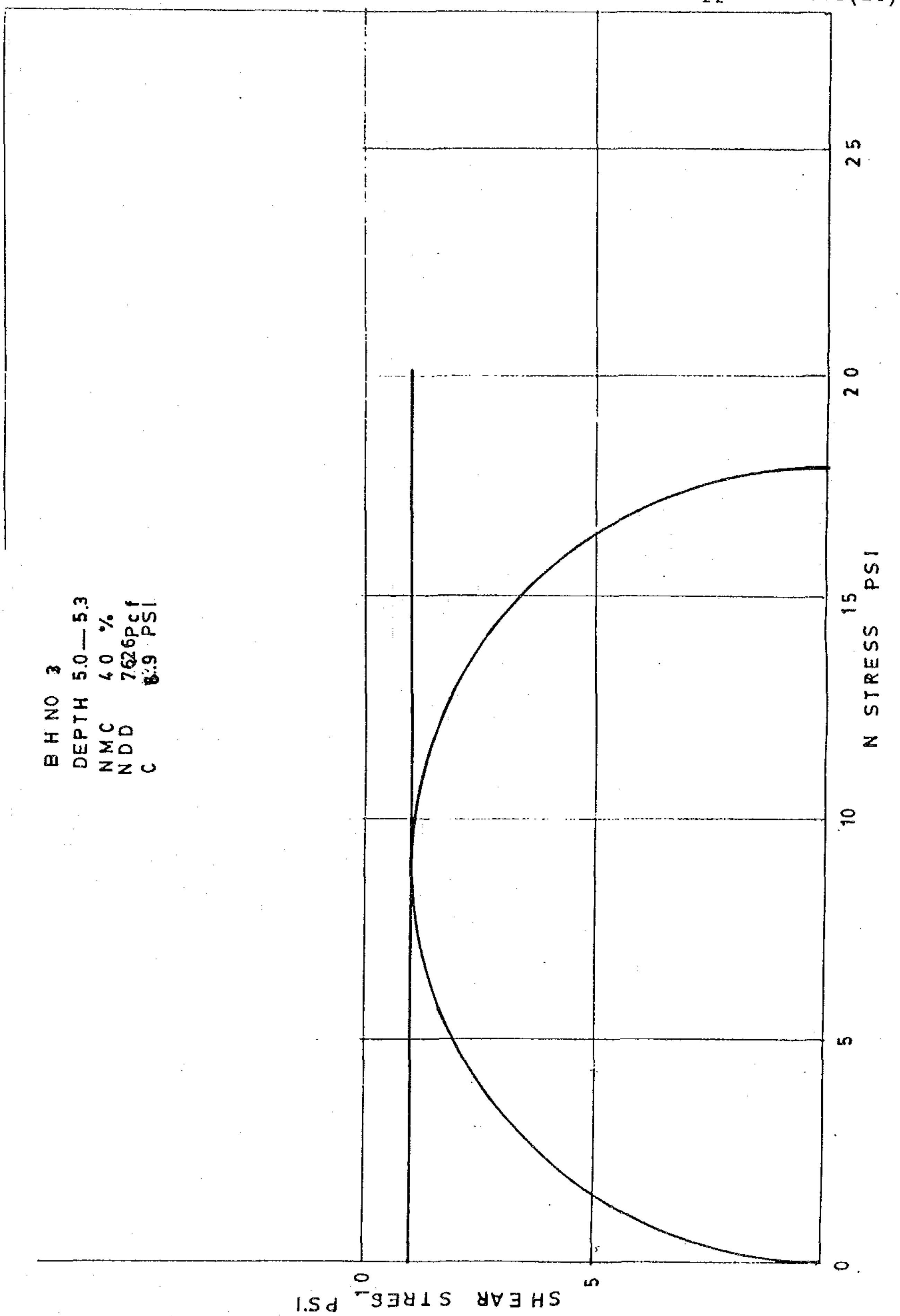
APPENDIX "B"

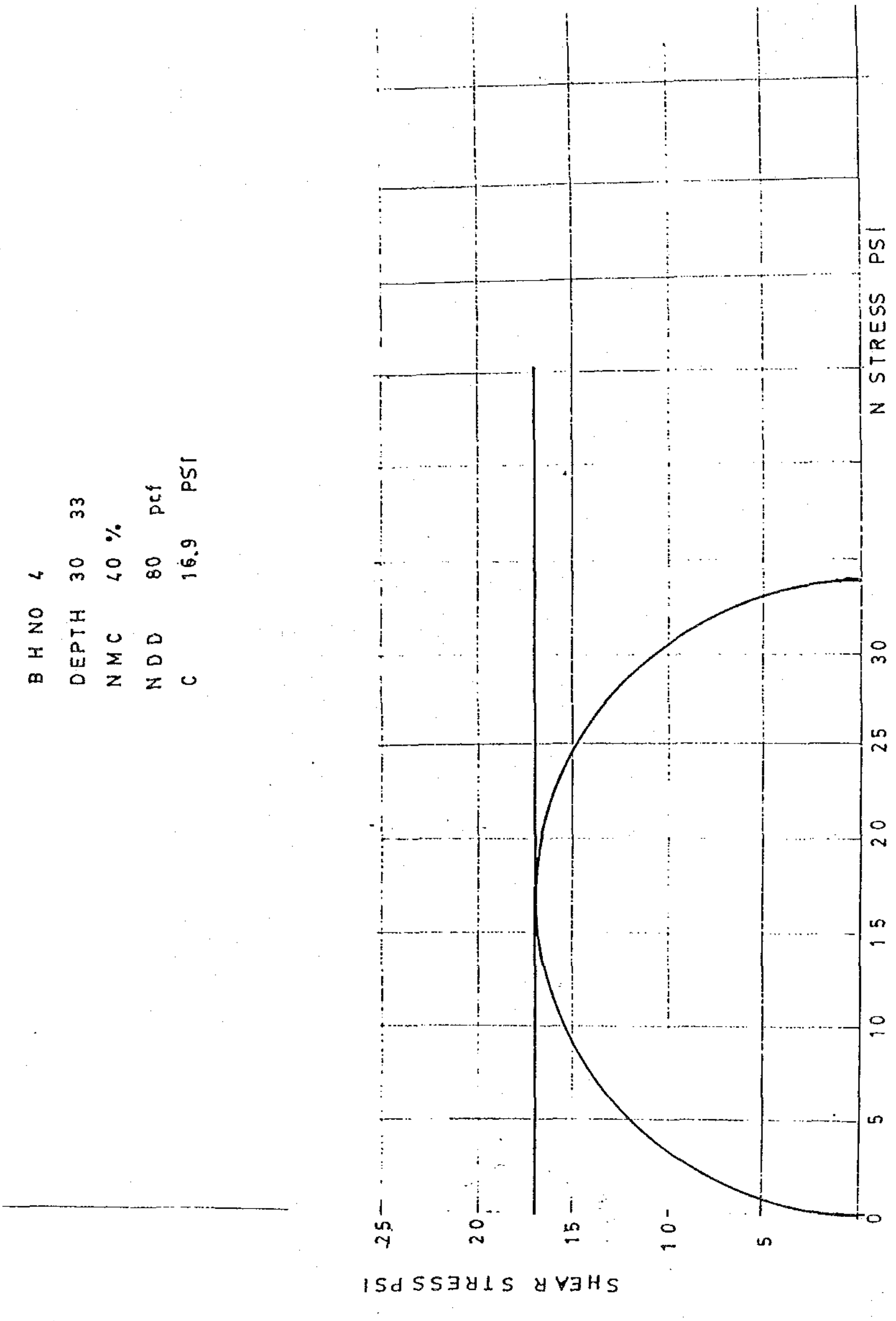
UNCONFINED COMPRESSIVE STRENGTH GRAPHS

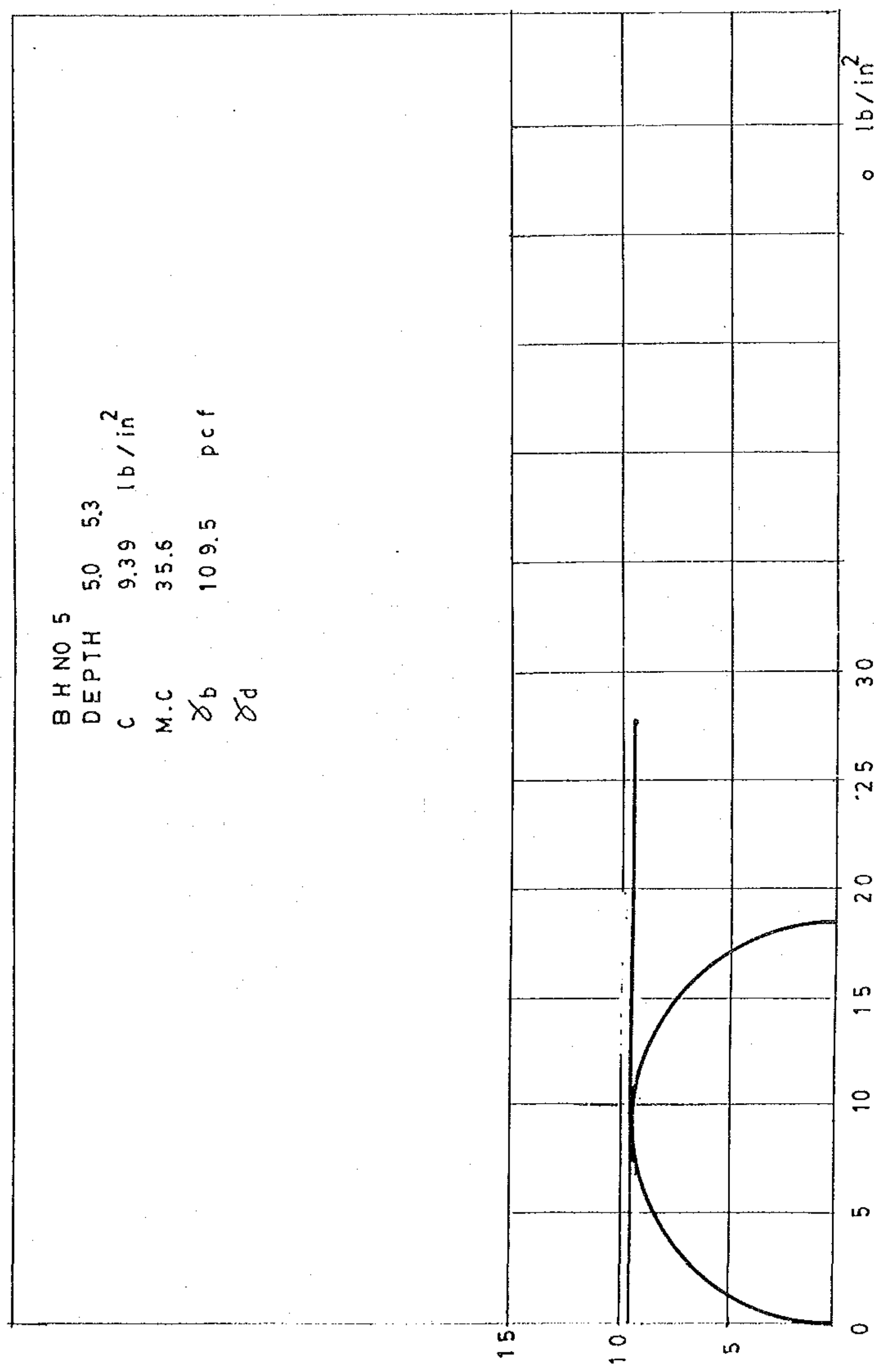


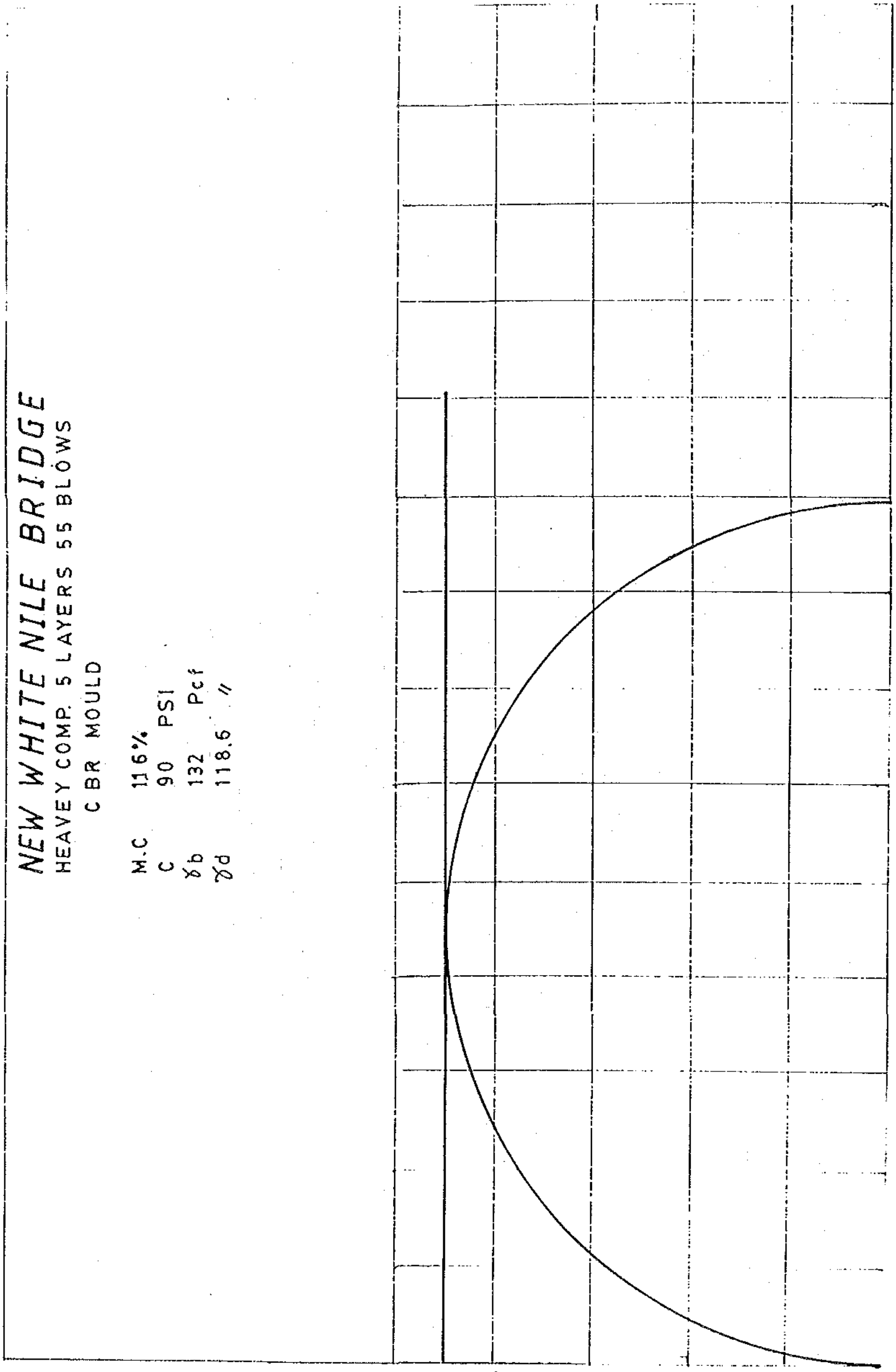
BH NO 1
DEPTH 50 53
NMC 38 %
NDD 79 pcf
C 1069 PSI
N STRESS PSI
0
5
10
15
20
25
30
SHEAR STERSS PSI
15
10
5

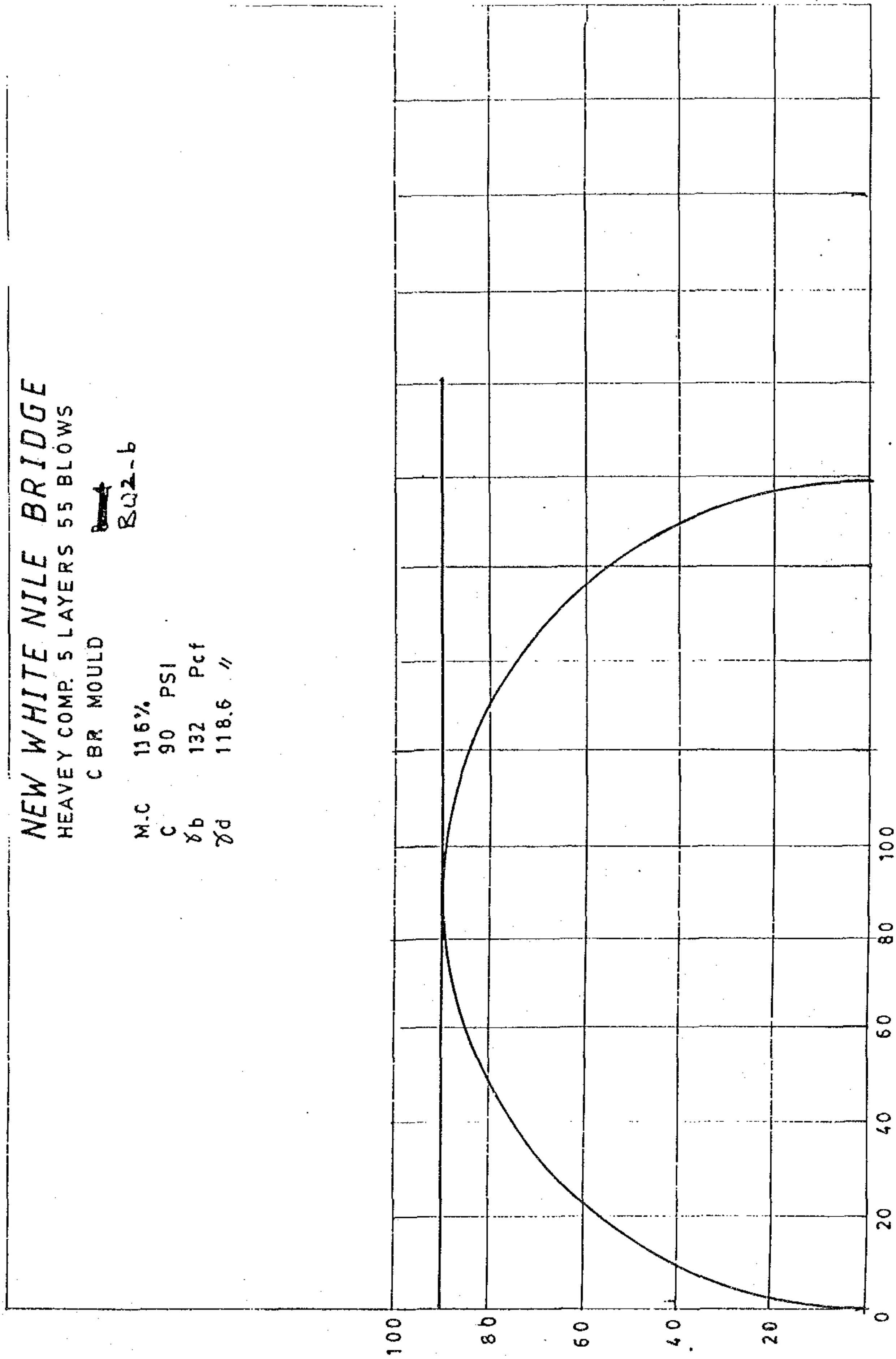

B.H.NO. 2
DEPTH 5.0
C 8.84 5.9 lb/in²
M.C 49 %
γb 106.4 pcf
γd
0
5
10
15
20
10
5


B H NO 3
DEPTH 5.0 — 5.3
NMC 4 0 %
NDD 7626Pcf
C
N STRESS PSI
0
5
10
15
20
25
SH EAR STRES PSI
0
5
10


B H NO 4
DEPTH 30 33
N M C 40 %
N D D 80 pcf
C 16.9 PSI
N STRESS PSI
0
5
10
15
20
25
30
SHEAR STRESS PSI
5
10
15
20
25

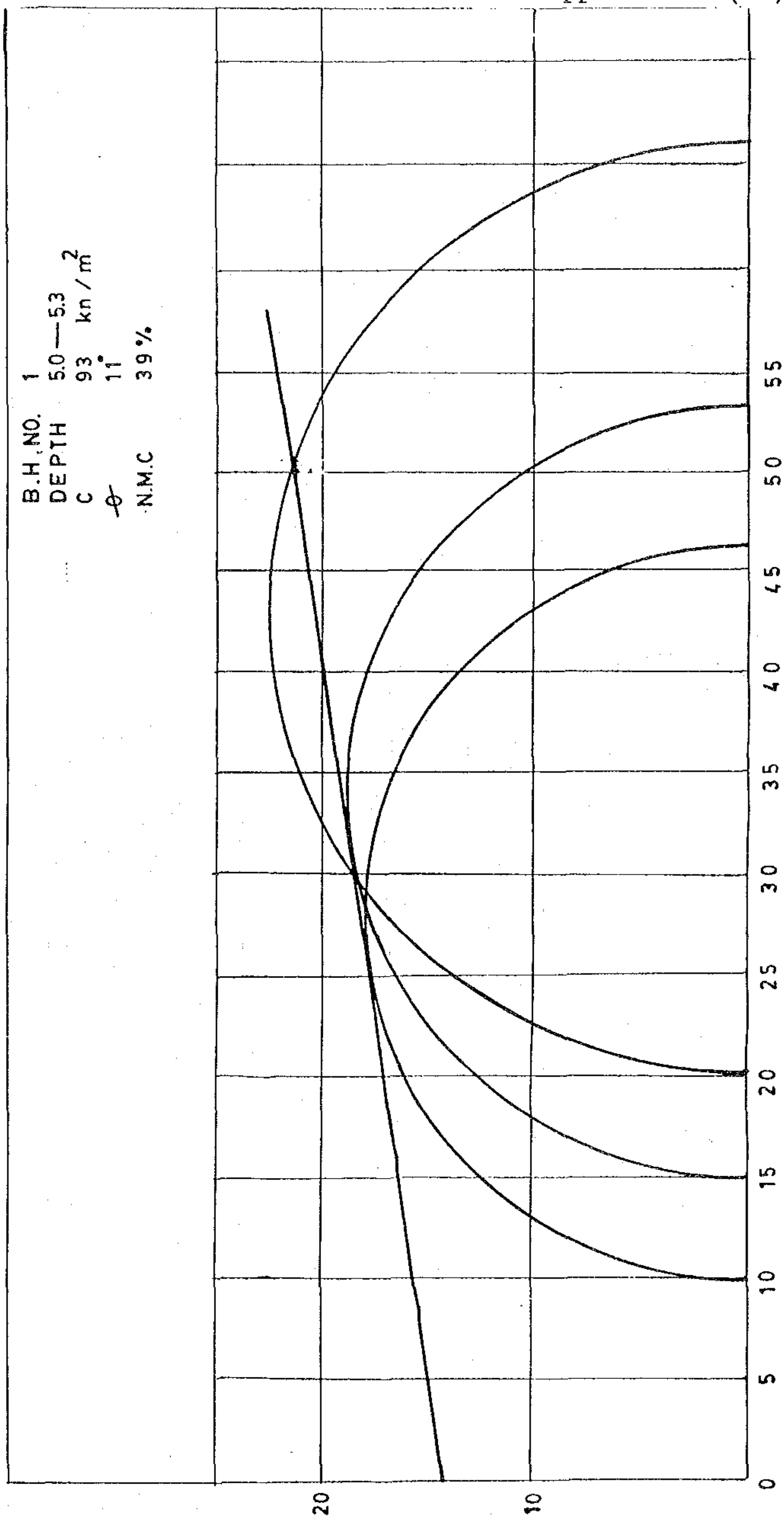

B H NO 5
DEPTH 5.0 5.3
C 9.39 lb/in2
M.C 35.6
γb 109.5 pcf
γd
0
5
10
15
20
25
30
o lb/in2
15
10
5
0

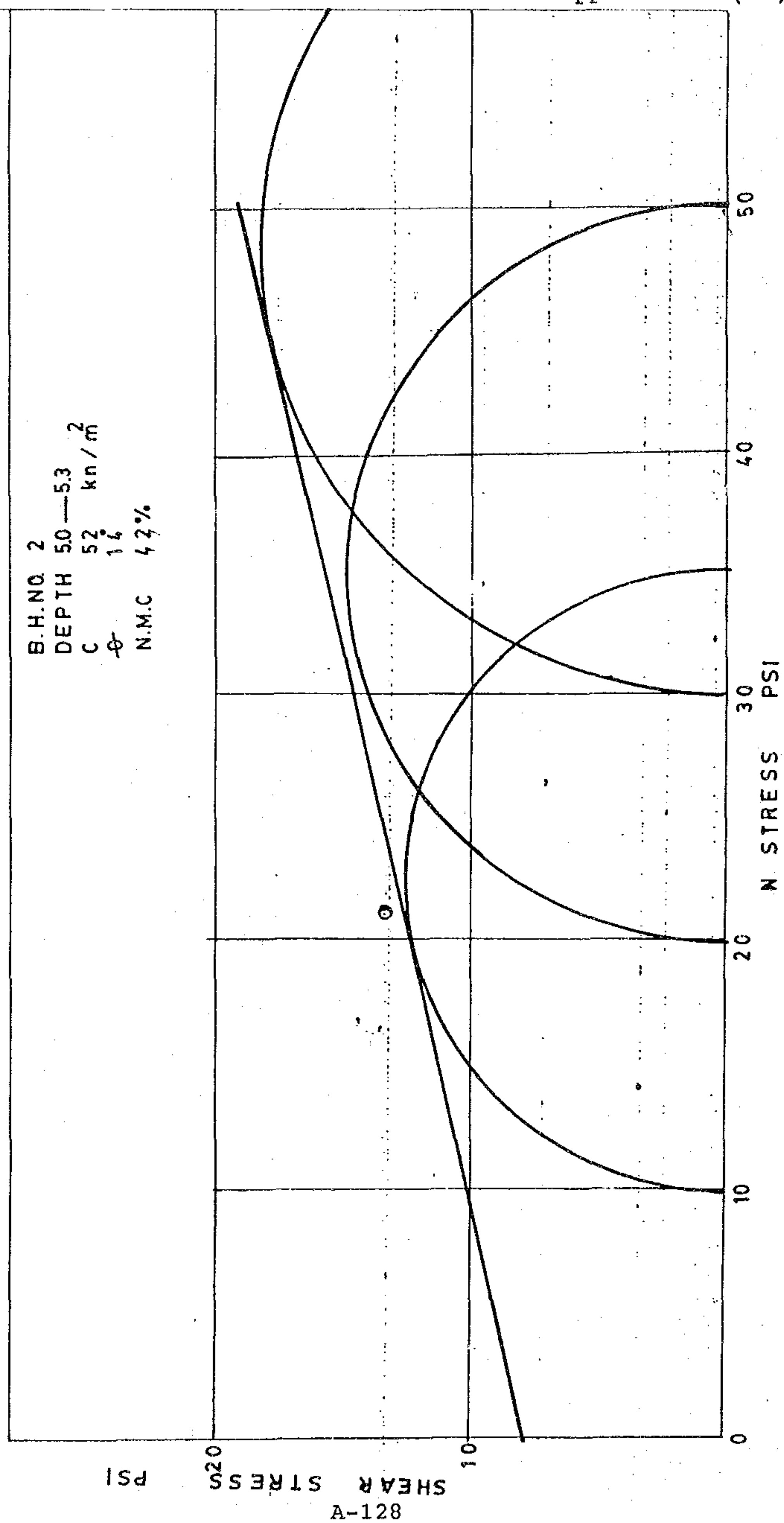

NEW WHITE NILE BRIDGE
HEAVEY COMP. 5 LAYERS 55 BLOWS
C BR MOULD
M.C 11.6%
C 90 PSI
γb 132 Pcf
γd 118.6 〃

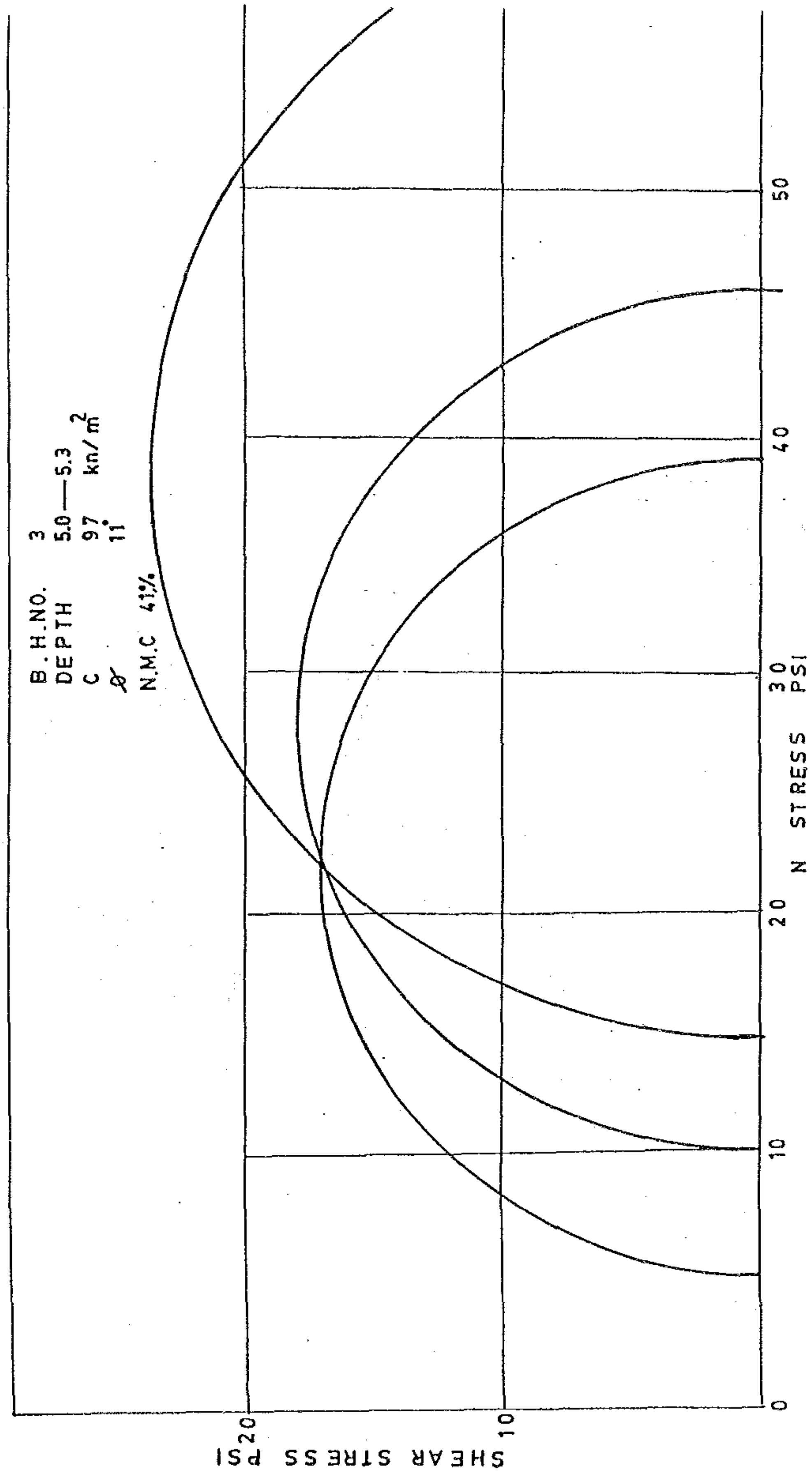

NEW WHITE NILE BRIDGE
HEAVEY COMP. 5 LAYERS 55 BLOWS
C BR MOULD
BQ2-b
M.C 13.6%
C 90 PSI
γb 132 Pcf
γd 118.6 //
100
80
60
40
20
0
20
40
60
80
100

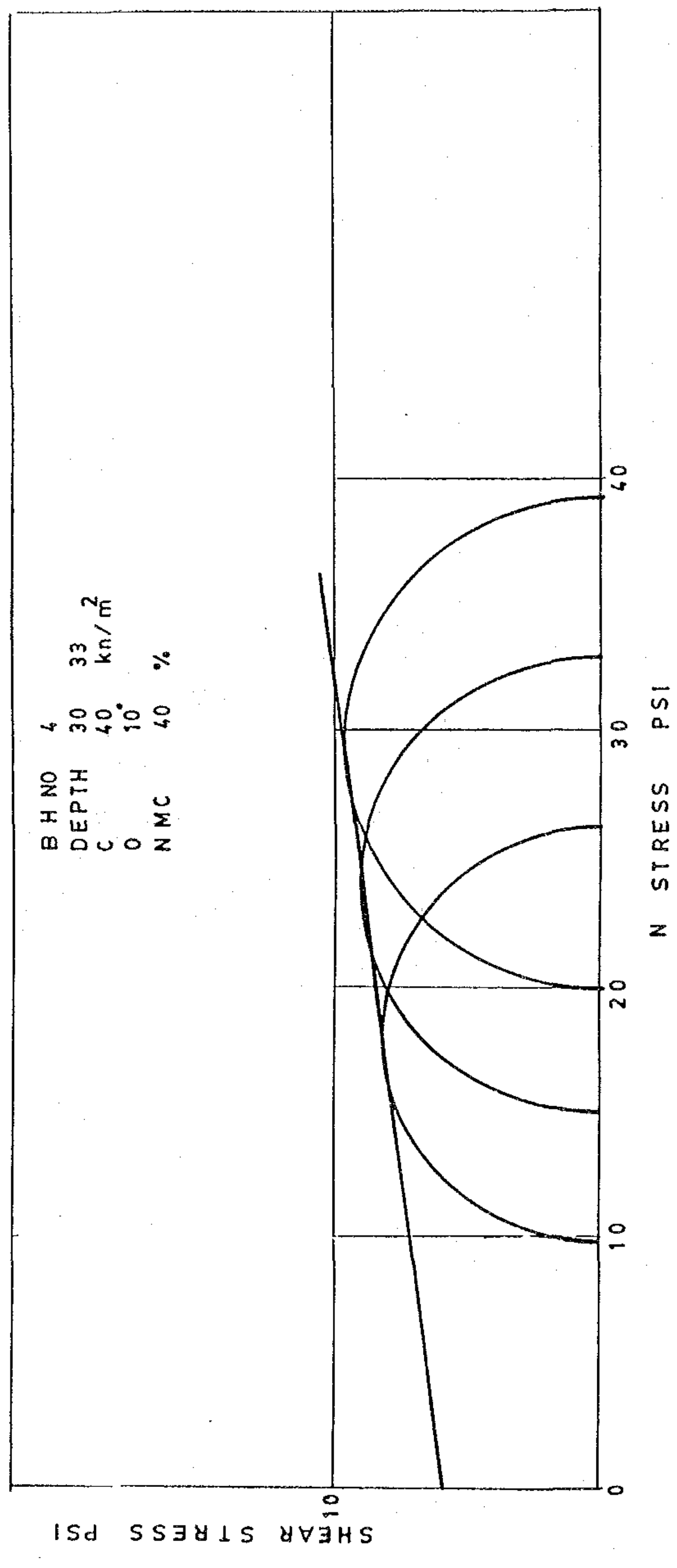
APPENDIX "C "

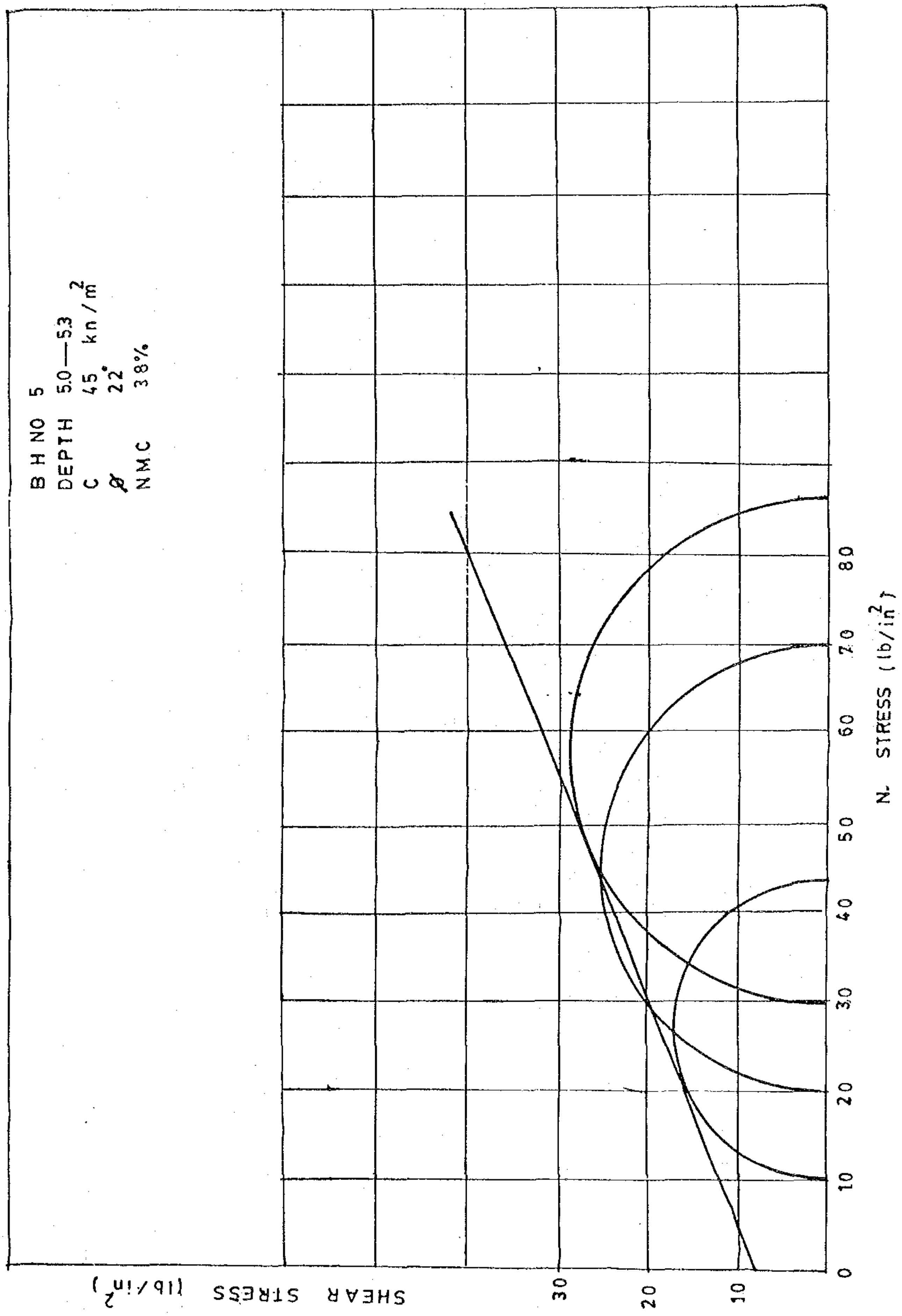
UU TRIAXIAL SHEAR STRENGTH TESTS GRAPHS

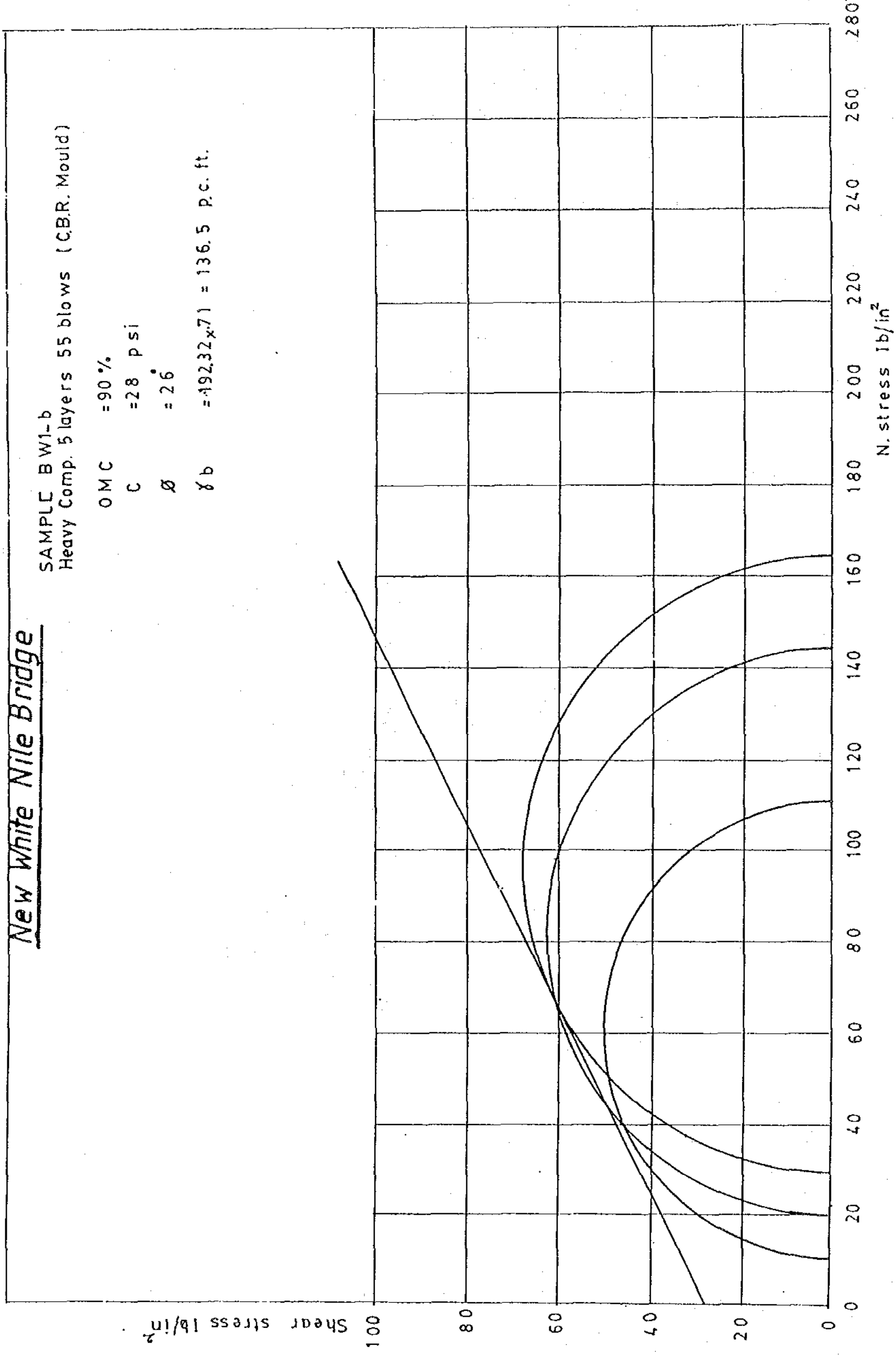

B.H.NO. 1
DEPTH 5.0 — 5.3
C 93 kn/m2
φ 11°
N.M.C 39%
0
5
10
15
20
25
30
35
40
45
50
55
20
10

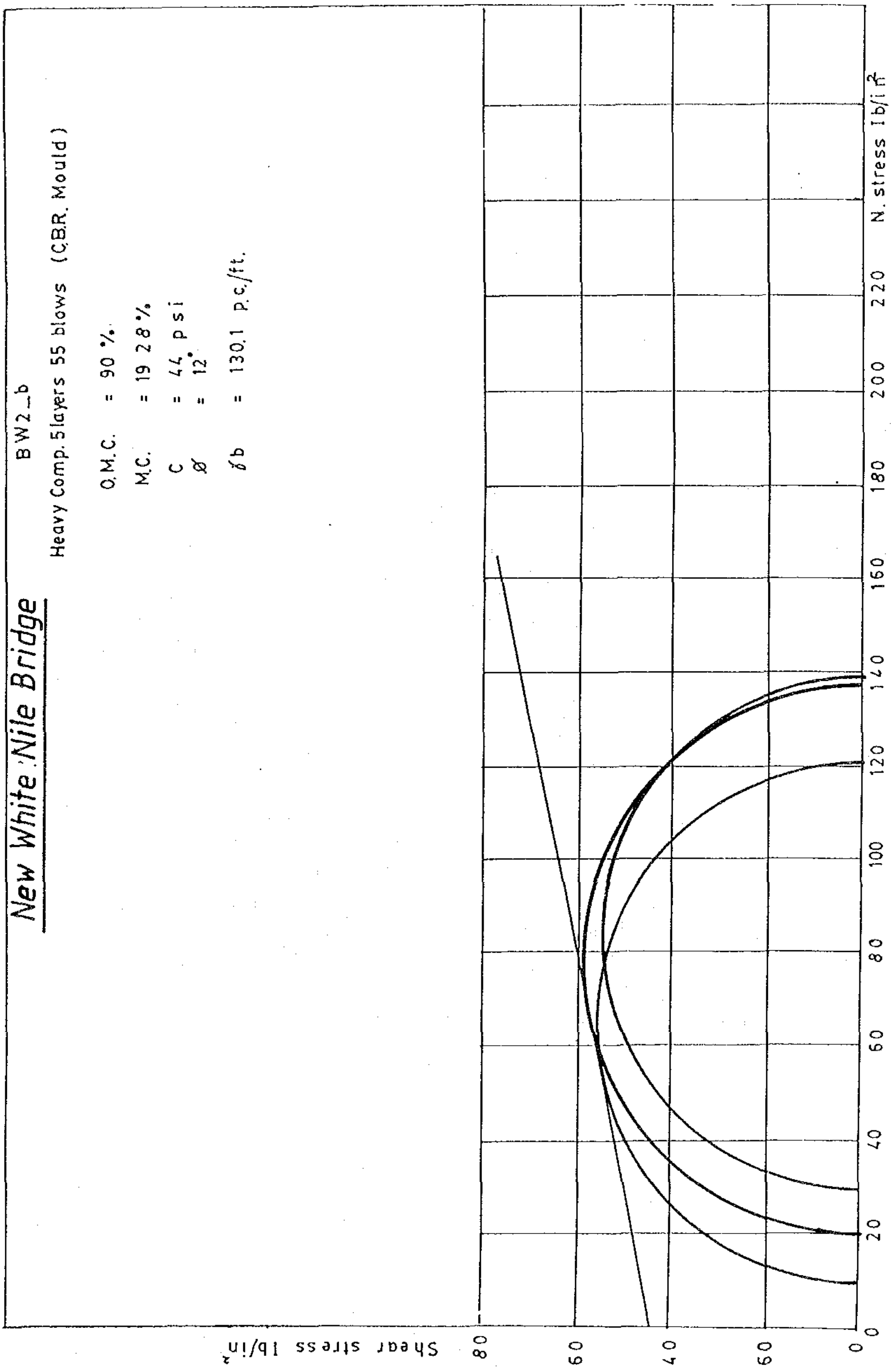

B.H.NO 2
DEPTH 5.0—5.3
C 52 kn/m2
φ 14°
N.M.C 42%
N STRESS PSI
SHEAR STRESS PSI
0
10
20
30
40
50


B.H.NO. 3
DEPTH 5.0—5.3
C 97 kn/m2
ø 11°
N.M.C 41%
N STRESS PSI
0
10
20
30
40
50
SHEAR STRESS PSI
10
20


B H NO 4
DEPTH 30 33
C 40 kn/m2
O 10°
NMC 40 %
N STRESS PSI
SHEAR STRESS PSI
0
10
20
30
40

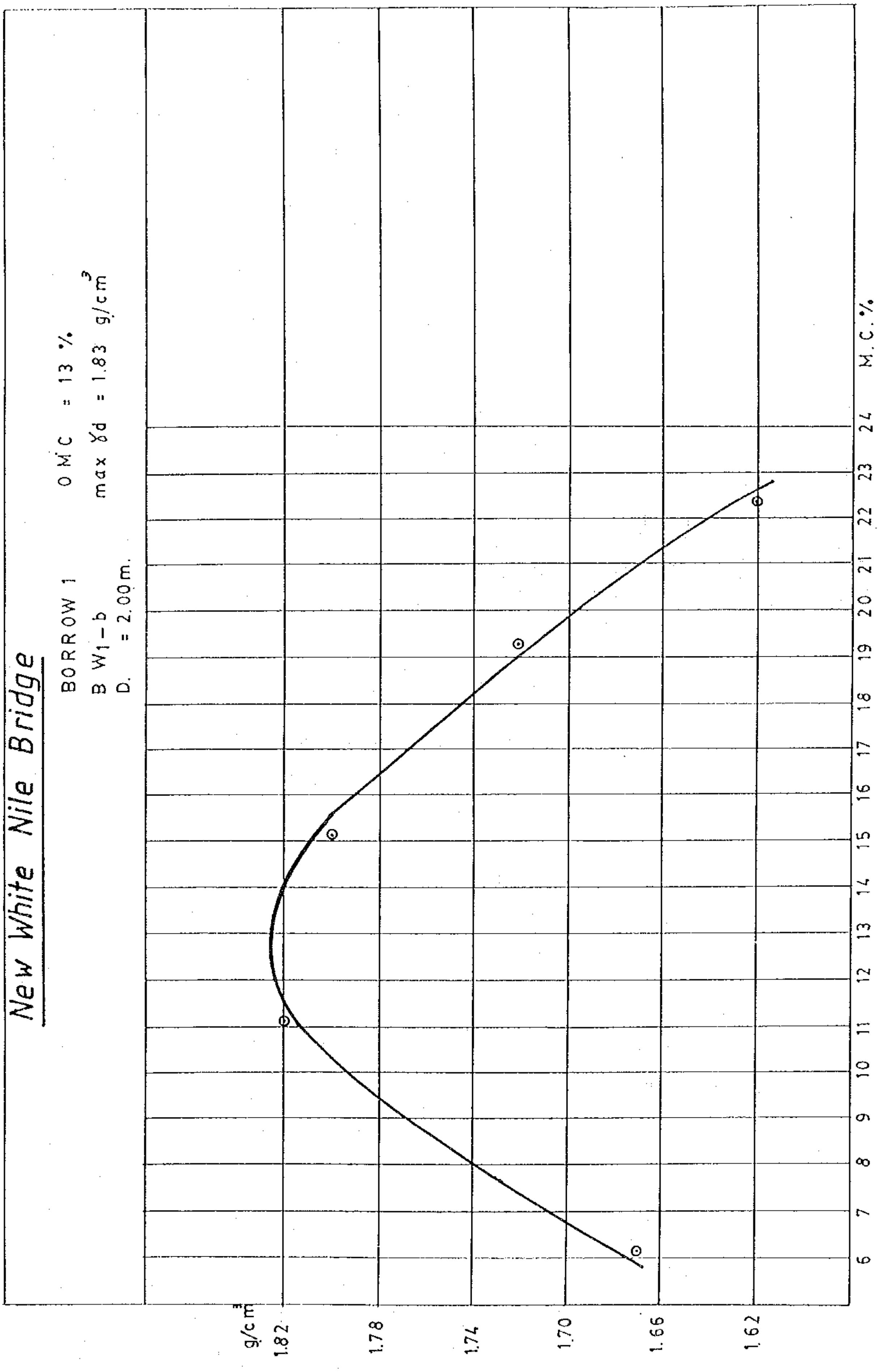

B H NO 5
DEPTH 5.0—5.3
C 45 kn/m2
ø 22°
NMC 38%
N. STRESS (lb/in2)
SHEAR STRESS (lb/in2)
0
10
20
30
40
50
60
70
80

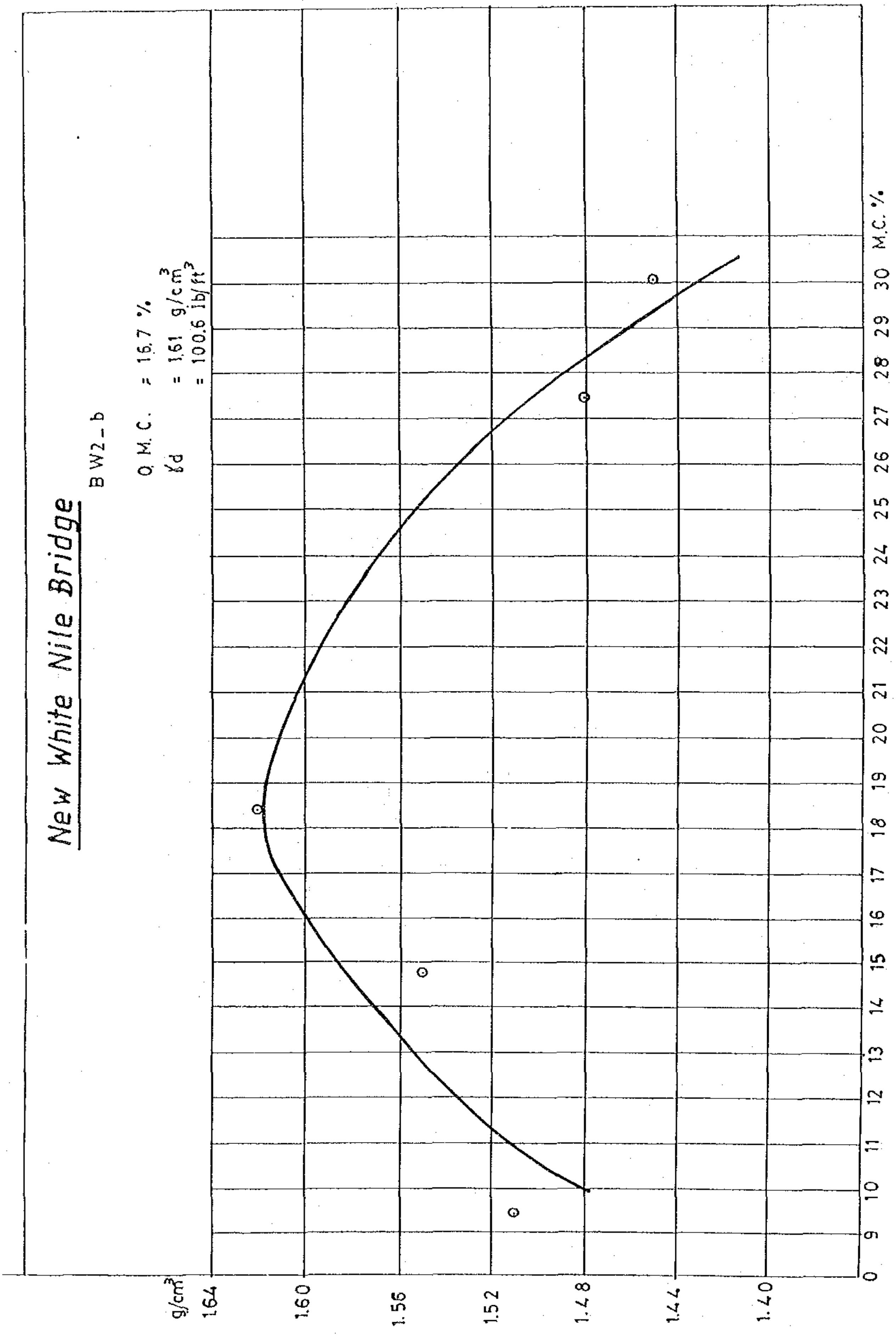

New White Nile Bridge
SAMPLE BW1-b
Heavy Comp. 5 layers 55 blows (C.B.R. Mould)
OMC =90 %
C =28 psi
Ø = 26°
γb =192.32x.71 = 136.5 p.c. ft.
Shear stress lb/in²
N. stress lb/in²
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280


New White Nile Bridge
BW2_b
Heavy Comp. 5 layers 55 blows (C.B.R. Mould)
O.M.C. = 90 %
M.C. = 19 28 %
C = 44 psi
Ø = 12°
γb = 130.1 p.c./ft.
N. stress lb/in²
Shear stress lb/in²

APPENDIX " D "

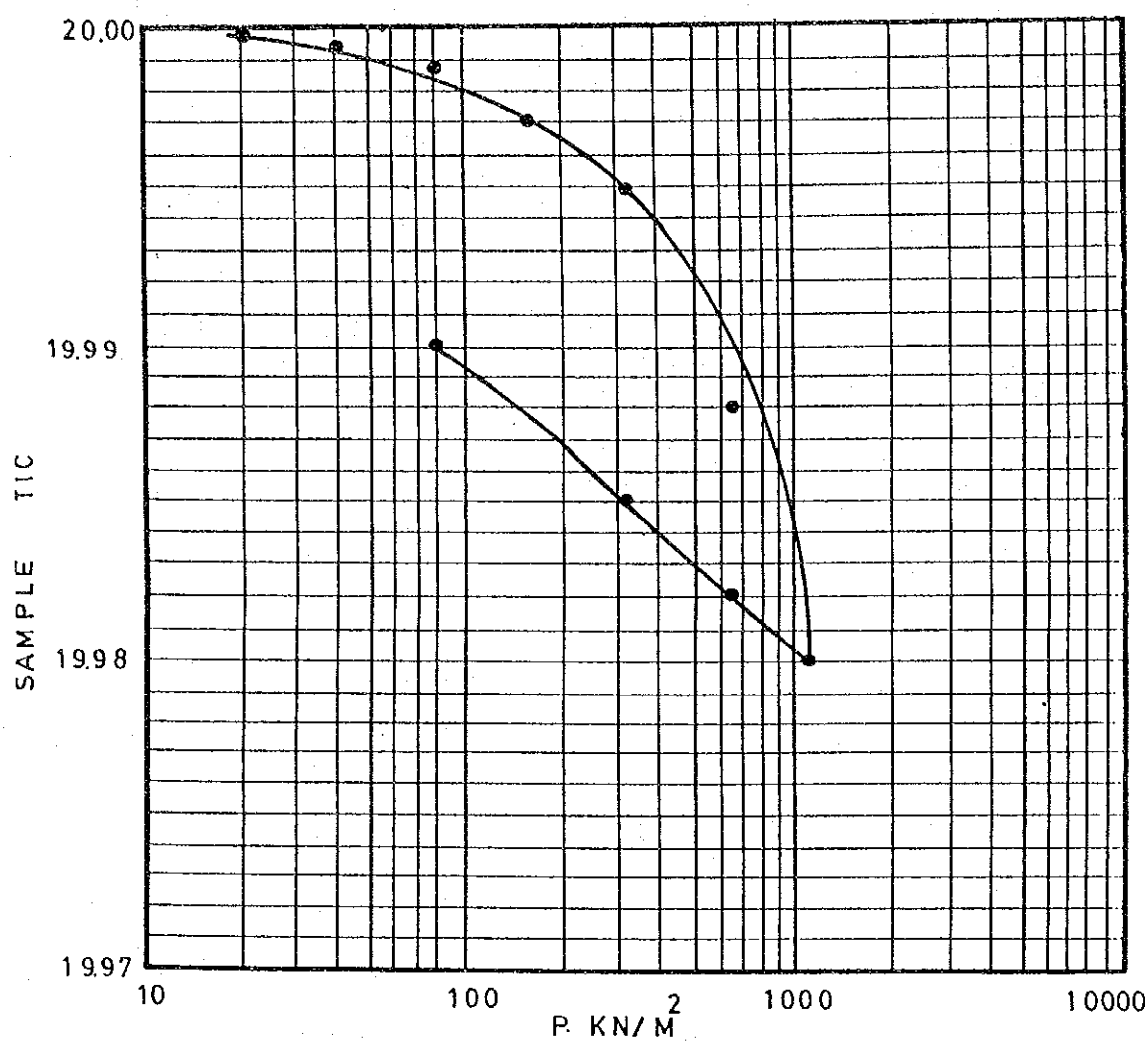
COMPACTION CURVES


New White Nile Bridge
BORROW 1
B W1 – b
D. = 2.00 m.
OMC = 13 %
max γd = 1.83 g/cm³
M.C.%
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
g/cm³
1.82
1.78
1.74
1.70
1.66
1.62


New White Nile Bridge
B W2_b
O.M.C. = 16.7 %
γd = 1.61 g/cm³
= 100.6 lb/ft³
M.C. %
g/cm³

APPENDIX " E "

CONSOLIDATION CURVES



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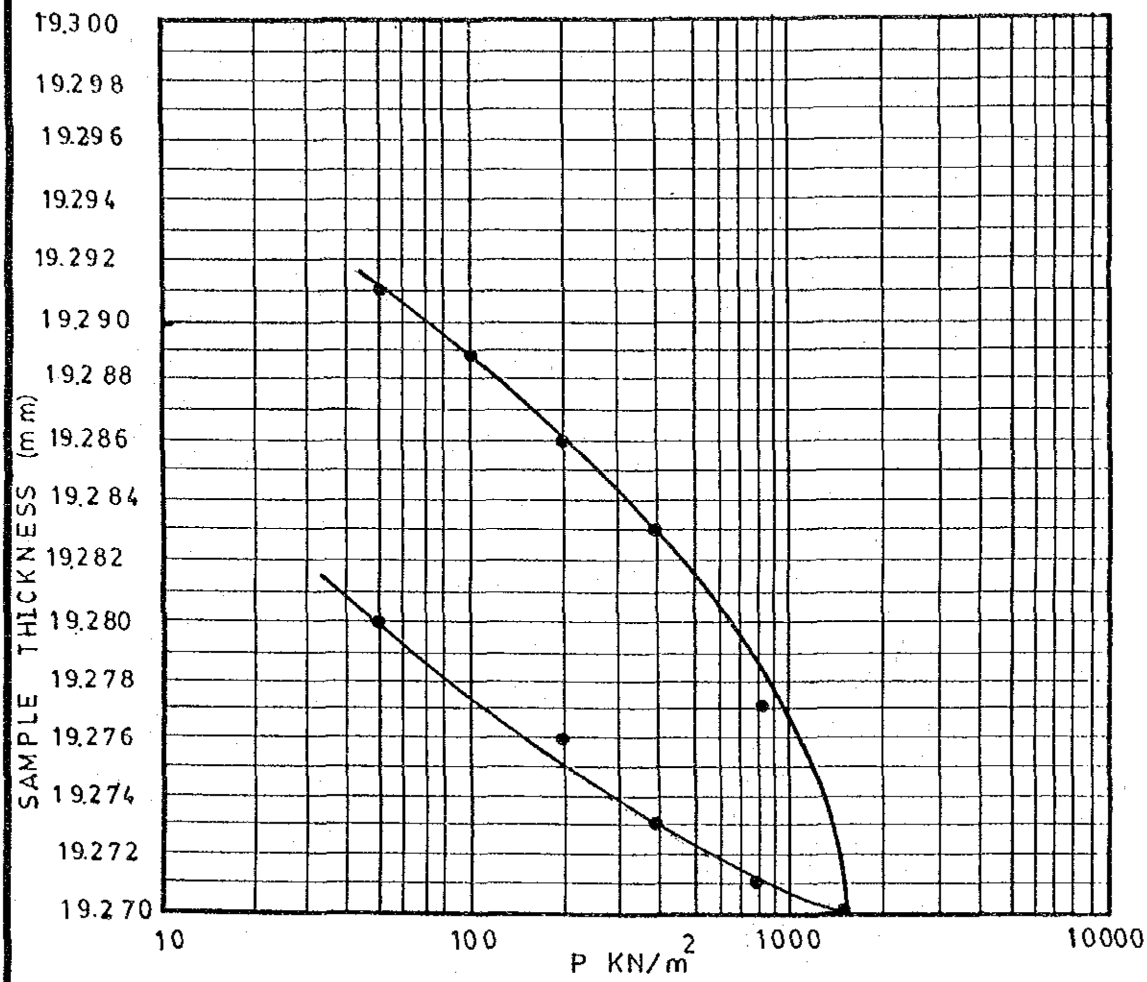
CONSOLIDATION TEST RESULTS

JOB:

DATE:

B.H.NO. 1	APPLIED PRESSURE KN/M²	M_V (M^2/MN)	C_V ($M^2/YEAR$)
DEPTH(m)			
5.0 — 5.3	100 — 200	1.7×10^{-3}	10.07



BUILDING & ROAD RESEARCH INSTITUTE
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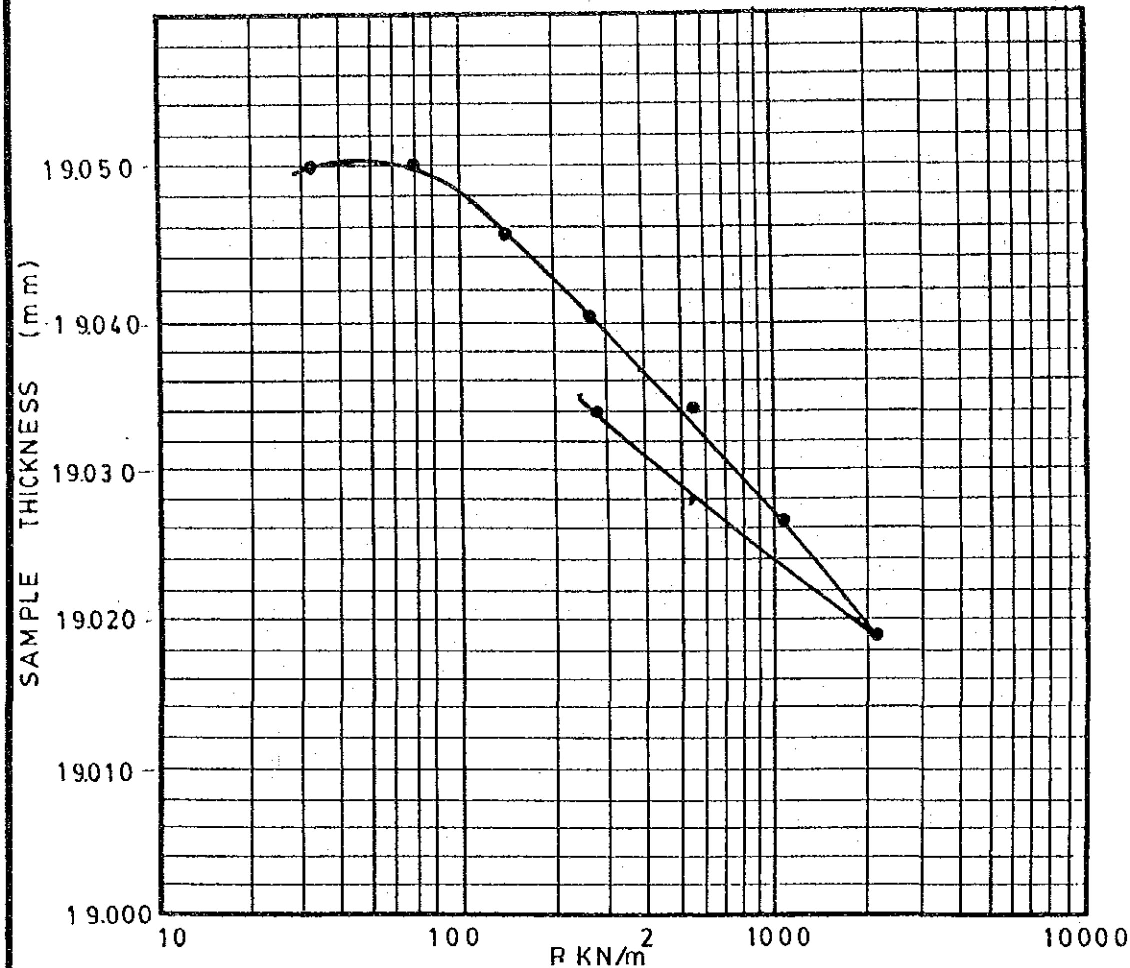
CONSOLIDATION TEST RESULTS

JOB: DATE:

B.H.NO. 2	APPLIED PRESSURE KN/M^2	M_v (M^2/MN)	C_v (M^2/YEAR)
DEPTH(m)			
5.0 — 5.3	100 — 200	2.33 X 10^{3}	11.45



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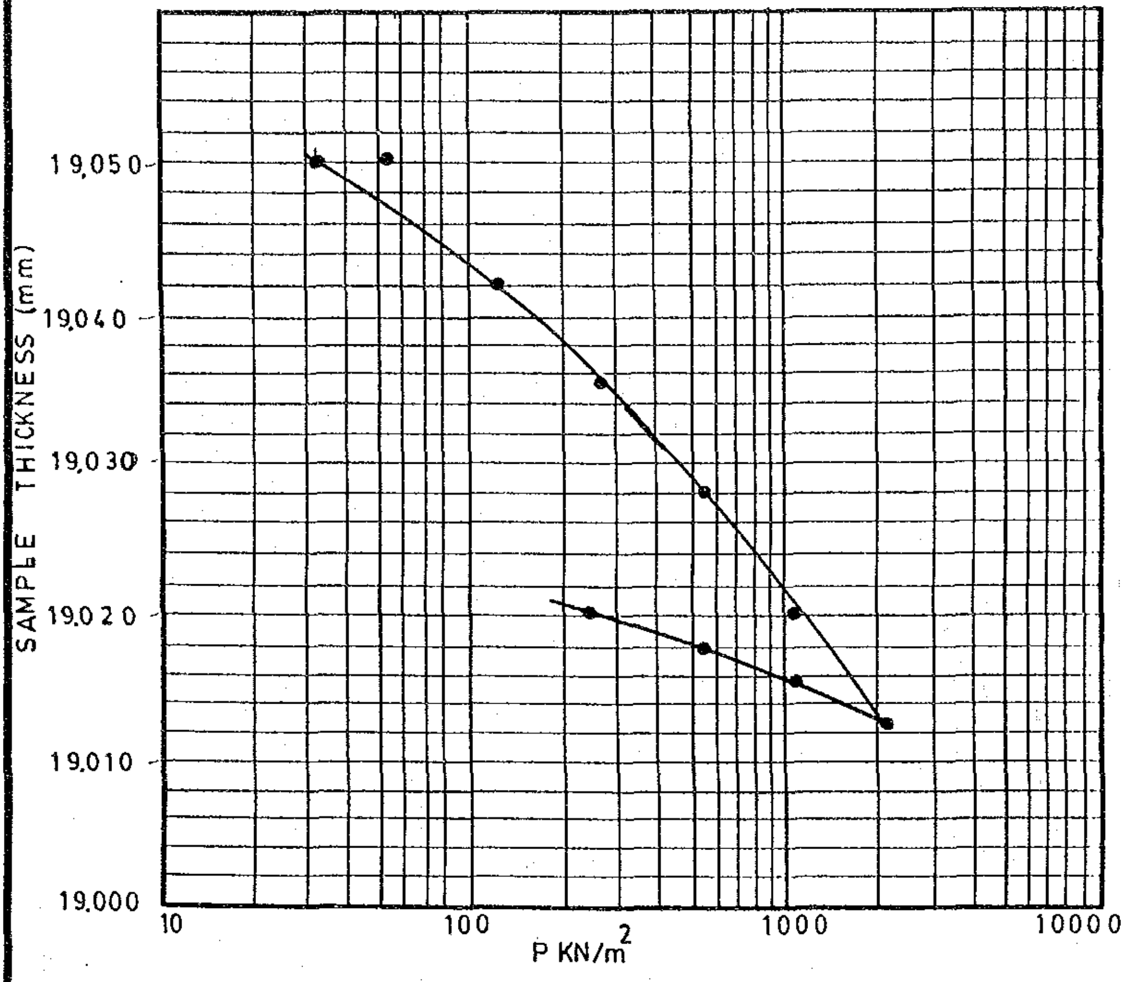
CONSOLIDATION TEST RESULTS

JOB:

DATE:

B.H.NO. 3	APPLIED PRESSURE KN/M²	M_V (M²/MN)	C_V (M²/YEAR)
DEPTH(m)			
5.0 — 5.3	100 — 200	2.3×10^{-3}	15.7

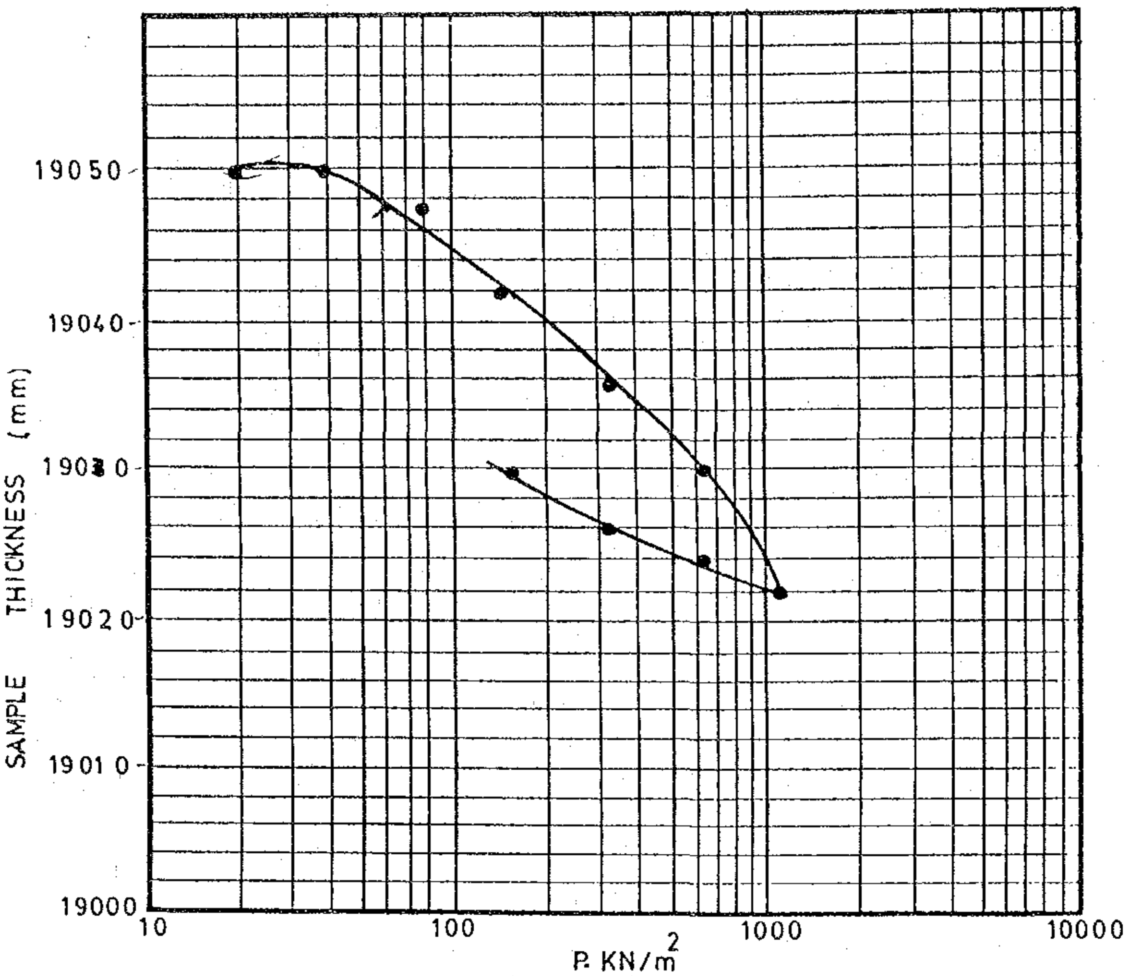


BUILDING & ROAD RESEARCH INSTITUTE
UNIVERSITY OF KHARTOUM
CONSOLIDATION TEST RESULTS

JOB:

DATE:

B.H.NO. 4	APPLIED PRESSURE KN/M^2	M_V (M^2/MN)	C_V $(M^2/YEAR)$
DEPTH(m)			
3.0 — 3.3	100 — 200	2.83×10^{-3}	17.9

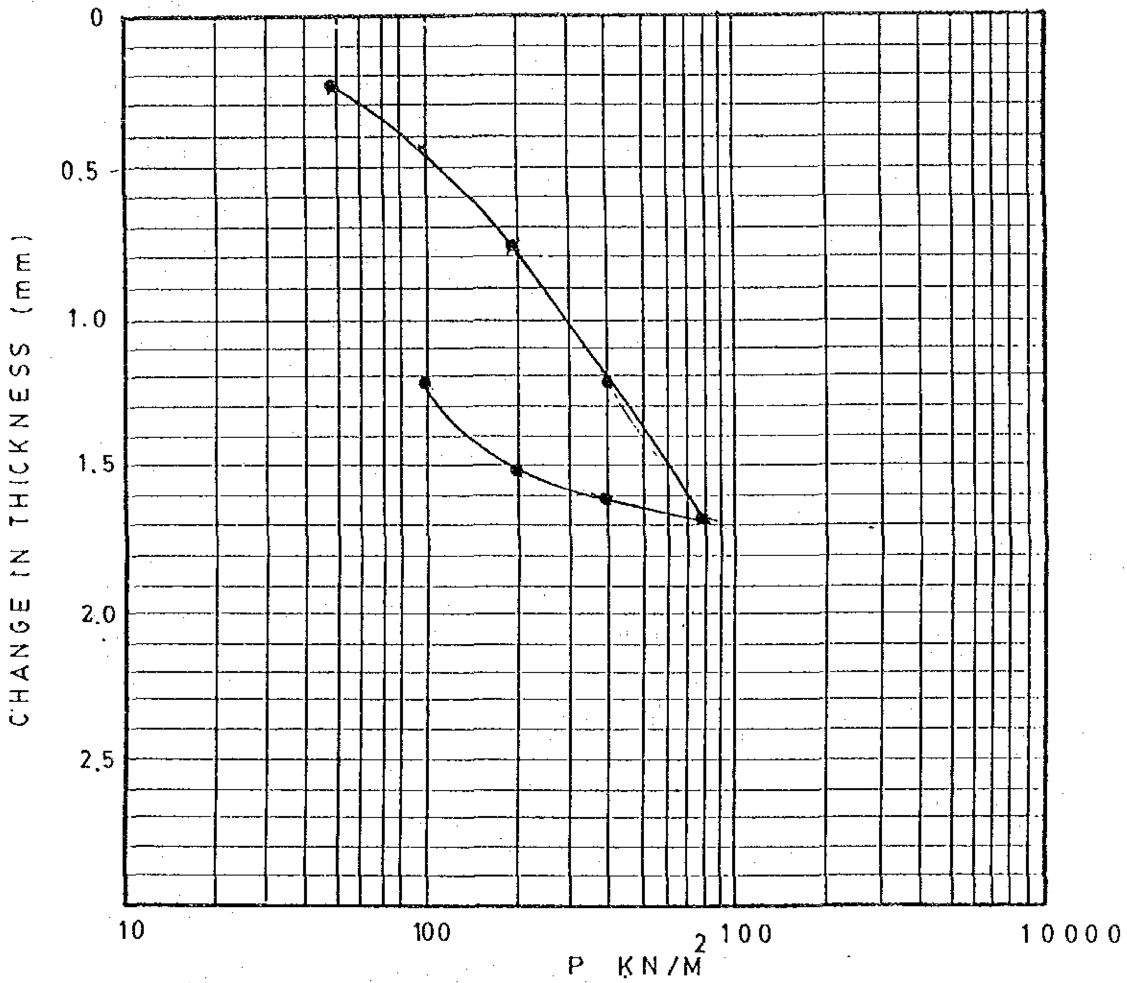


BUILDING & ROAD RESEARCH INSTITUTE
UNIVERSITY OF KHARTOUM
CONSOLIDATION TEST RESULTS

JOB:

DATE:

B.H.NO. 5	APPLIED PRESSURE KN/M²	M_V (M^2/MN)	C_V (M^2/YEAR)
DEPTH(m)			
5.0 — 5.3	100 — 200	3.3×10^{-3}	11.16

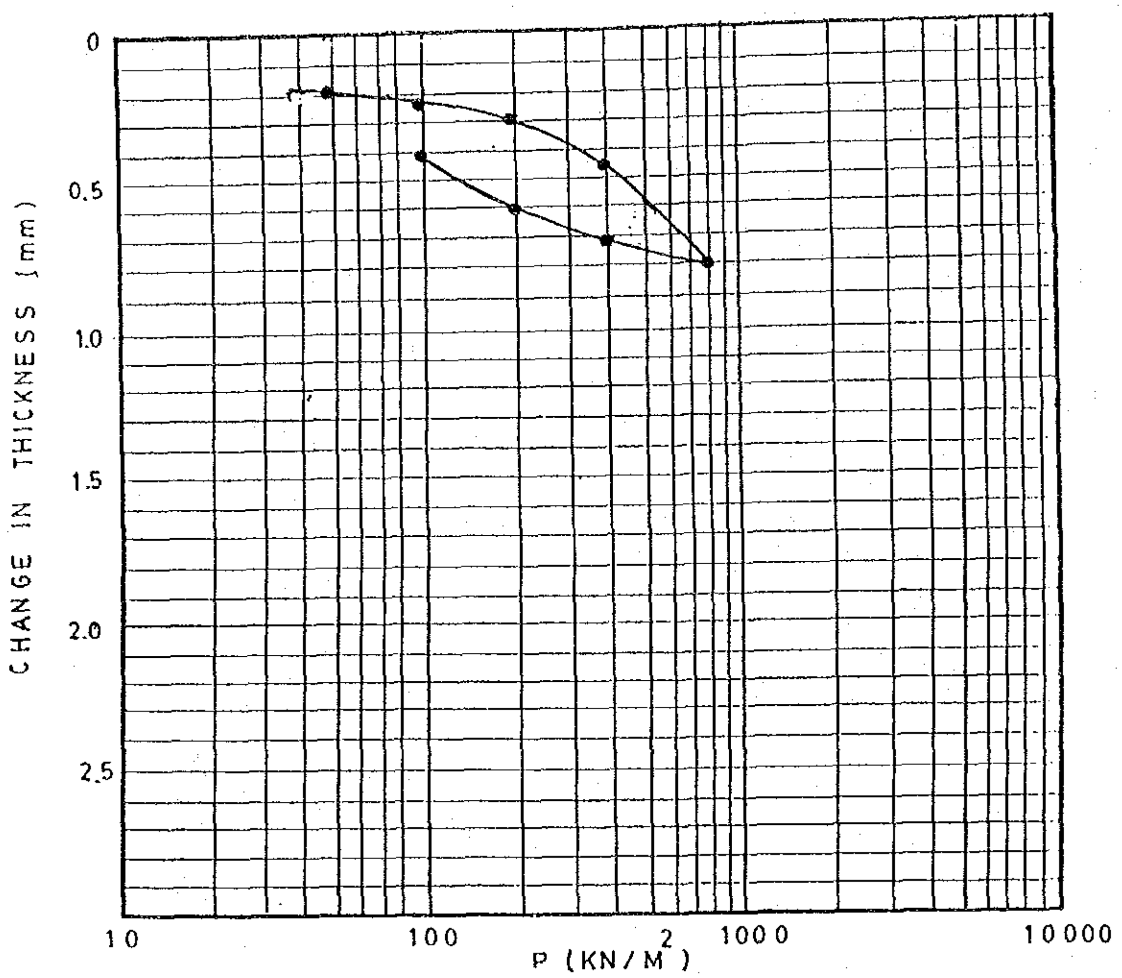


BUILDING & ROAD RESEARCH INSTITUTE
UNIVERSITY OF KHARTOUM
CONSOLIDATION TEST RESULTS

JOB:

DATE:

B.H.NO. BW1 b	APPLIED PRESSURE KN/M²	M_V (M^2/MN)	C_V (M^2/YEAR)
DEPTH(m)	100——200	0.176	



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UNIVERSITY OF KHARTOUM
CONSOLIDATION TEST RESULTS

JOB:

DATE:

B.H.NO. BW 2-b	APPLIED PRESSURE KN/M^2	M_V (M^2/MN)	C_V (M^2/YEAR)
DEPTH(m)	100 — 200	0.0 43	

Table (1)

a) Undistarbed Sample Basic Engineering Properties

Sample	Depth m	LL %	P.I. %	Gs	NMC %	NDD PCf
B - 1	5.0 - 5.3	36	14	2.63	39	79.06
B - 2	5.0 - 5.30	43	16	2.59	42	71.17
B - 3	5.0 - 5.3	38	15	2.64	41	76.26
B - 4	3.0 - 3.3	61	37	2.61	40	80.00
B - 5	5.0 - 5.3	55	24	2.63	38	80.74

b) Linear Shrinkage

Sample	Depth m	L.S. %
B - 1	5 - 5.3	10
B - 2	5 - 5.3	12
B - 3	5 - 5.3	12
B - 4	5 - 5.3	19
B - 5	5 - 5.3	16

Table (2)

CBR values for Sample Compacted at 90% OMC and Soaked for four days.

Sample	OMC %	MDD gm/ cm	CBR %
BW1 - b	13	1.83	2.8
BW1 - b	19	1.62	2.3

Embankment Study

A high embankment is planned for part of the road section where the subsoil consists of soft clay. Therefore, a study on the embankment is made below to examine settlement and slip failure.

(1) Consolidation Settlement

According to the boring results, it is judged that the layer from ground surface to 4 meters deep is liable to settlement by consolidation.

Settlement is estimated for the position of embankment center (roadway center) with various embankment heights h = 6 m, 8 m, 10 m and 12m by means of the following formula:

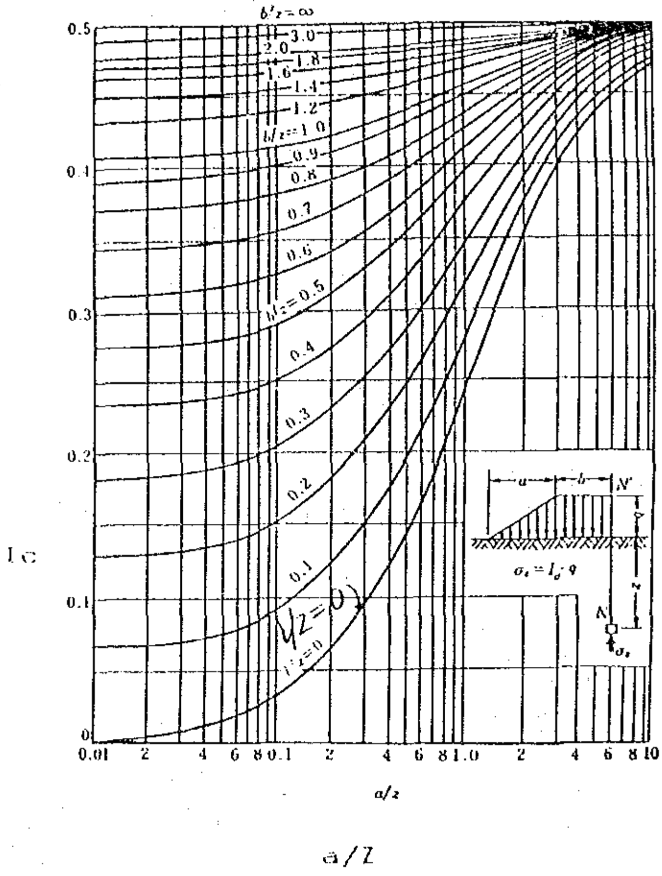
$$S = H \times \frac{Cc}{1 + eo} \times \log_{10}\left(\frac{Po + dP}{Po}\right)$$

where, S : Amount of consolidation settlement
H : Thickness of settlement layer
Cc: Compression index
eo: Void ratio
Po: Initial pressure
dP: Increment stress caused by embankment, which is given by the Osterberg's graph as to the stress of embankment weight as shown in the following figure.

Those values related to calculation of settlement are empirically decided as follows taking into consideration the results of mechanical borings and soil tests.

H = 4 m
Cc = 0.3
eo = 0.4
Po = 5 t/m2

The increment stress dP is obtained by the Osterberg's graph on the premise that unit weight of embankment is 1.8 t/m3. The values are tabulated in the following table.



σ_z = Ic × q

q = r_t × h

r_t = Unit Wt.

h = Embank. Ht.

Figure 3.5 Osterberg's Graph

Increment Stresses

Embankment Height m	dP t/m2
6	11.0
8	14.0
10	18.0
12	23.0

The settlement amount is obtained as shown in the following table and figure.

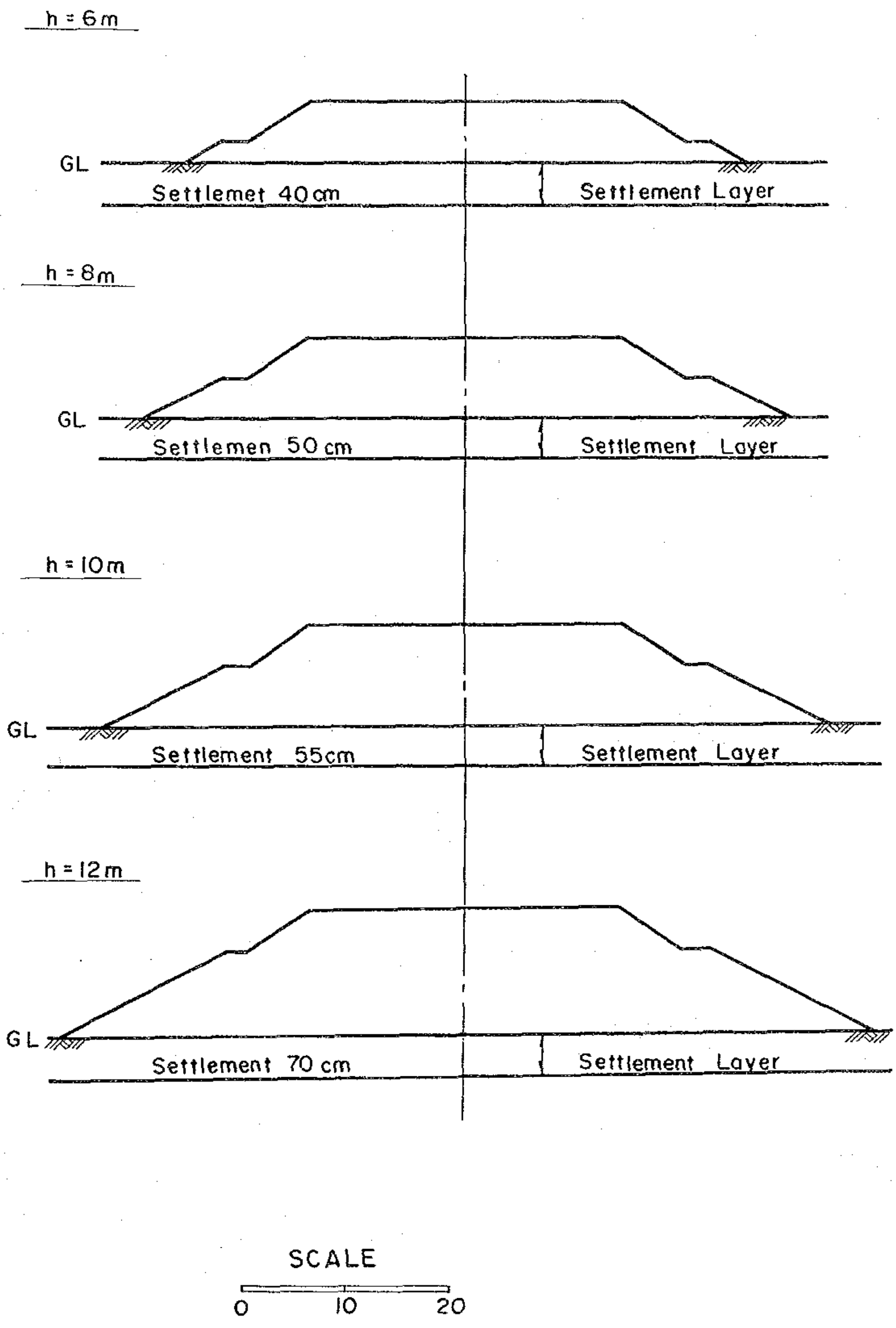
Consolidation Settlement By Embankment Heights

Embankment Height	Settlement
6 m	40 cm
8 m	50 cm
10 m	55 cm
12 m	70 cm

On the other hand, time in relation to settlement is calculated by the following formula:

$$t = \frac{Tv \times H'^{2}}{Cv}$$

where, t : Required time for the final settlement
Tv : Time factor (theory of consolidation)
H' : Thickness of stratum related to drain
Cv : Coefficient of consolidation.


h = 6m
GL
Settlemet 40cm
Settlement Layer
h = 8m
GL
Settlemen 50 cm
Settlement Layer
h = 10m
GL
Settlement 55cm
Settlement Layer
h = 12m
GL
Settlement 70 cm
Settlement Layer
SCALE
0
10
20

In case of 80 % settlement, Tv = 0.567, H' = 4 m and Cv = 200 cm2/day, required time t is obtained as follows:

$$t = \frac{0.567 \times 400^2}{200} = 450 \text{ days}$$

(2) Slip Failure (Circular Slip Failure)

If the subsoil is composed of homogeneous, circular slip safety can be calculated by the following formula:

$$Fs = \frac{C}{_t \times h} \times n$$

where, Fs : Safety factor
C : Cohesive strength of subsoil
$_t$: Unit weight of embankment
h : Embankment height
n : Coefficient of stability

n is decided by the parameters with regard to embankment such as trapezoid of embankment (a,b) and depth of slip surface (z) shown in the next page.

As a result of iteration, the following is obtained when C = 3 t/m2, $_t$ = 1.8 t/m3 :

Safety Factor By Embankment Height
(Without Any Countermeasure)

Embankment Height	Safety Factor of Circular Slip
8 m	1.3
10 m	1.01
12 m	0.93

Consequently, it is concluded that the maximum height of embankment without improvement be 8.0 meters.

Counter weight embankment is often employed as a countermeasure against slip in order to increase the value of safety factor. Detailed calculation is required, when embankment height more than 8 meters is planned with counter weight embankment, by dividing the circular section into appropriate strips (division method) as shown in the following figure.

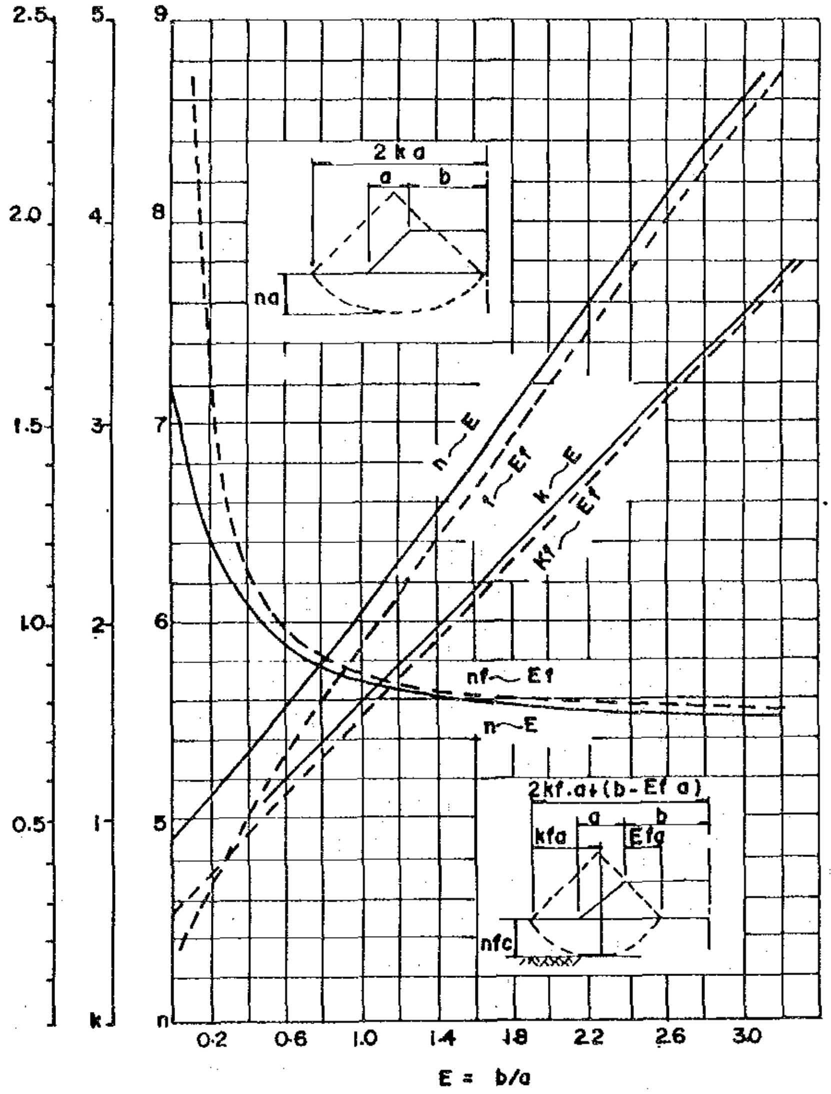
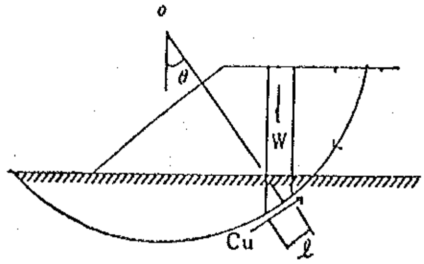


Figure 3.7 Graph for Deciding the Coefficient of Stability of Circular Slip Safety



Calculation of Circular Slip Safety by Division Method

Where,
- Fs : Safety factor, on the premise of long time stability Fs > 1.2
- Cu : Cohesive strength in slip surface
- Ø : Internal angle
- W : Weight of section
- W' : Weight of section reduced by buoyancy
- l : Length of slip surface of each section
- Wc : Counter weight

As a result of calculation, counter-weights of 30 and 60 t/m2 are required for embankment heights of h = 10 m and 12 m respectively as shown in the next page.

(3) Embankment Height to be recommended

Based on the discussions above in (1) and (2), the following is concluded as the recommendation for maximum embankment heights for the proposed road.

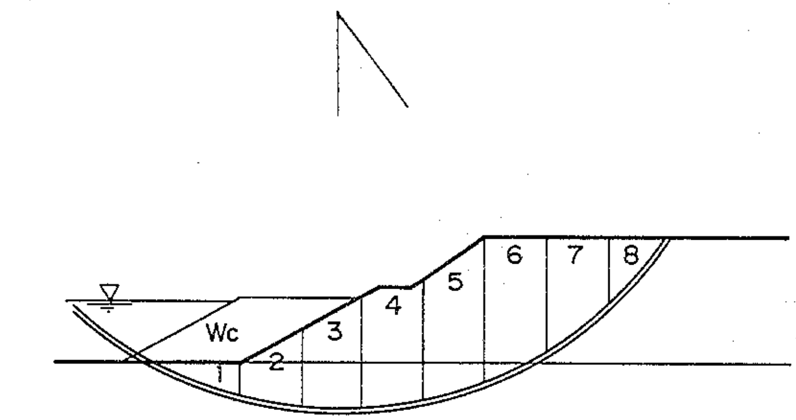
<u>If no countermeasure against circular slips</u>

Maximum Embankment Height = 8.0 m

<u>If counter weight embankment</u>

Maximum Embankment Height = 10.0 m

Although the embankment height might be theoretically possible up to 12 meters or more if the counter-weight is increased, such a high embankment is impractical since consolidation settlement of more than 70 cm settlement would occur in two (2) years.



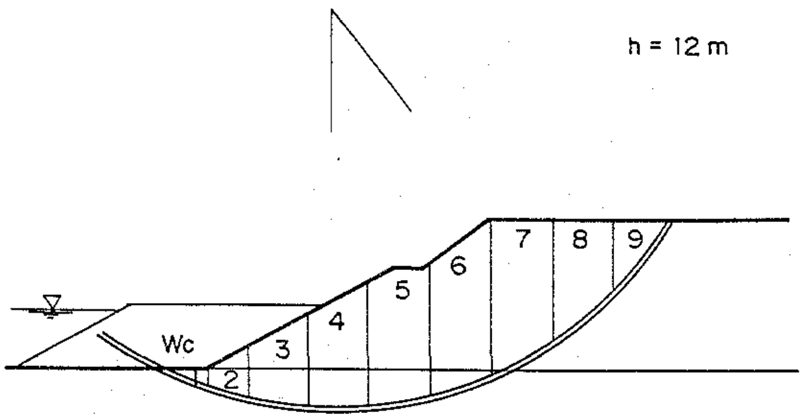
h = 10 m

	1	2	3	4	5	6	7	8	Σ	
Cu x 1	27.	15	15	15	15	18	12	16	133	
W cos Q x tan ø u	0	0	0	0	0	0	1.4	1.2	2.4	135.4
W sin	$\overline{5.9}$	$\overline{6.9}$	$\overline{2.1}$	11	25	35	43	40	139	139

$$Fs = \frac{\Sigma(Cu \times 1 + W\cos Q \times \tan \phi u)\ Wc}{\Sigma \sin Q} \geqq 1.2$$

where Wc : Counter weight

$$\frac{135.4 + Wc}{139} \geqq 1.2 \quad \therefore Wc \geqq 30$$



h = 12 m

	1	2	3	4	5	6	7	8	9	Σ	
Cu x 1	12	11.5	15	15	15	16.5	12	12	16	125	
W cos Θ x tan ø u	0	0	0	0	0	0	2.5	1.8	0.8	5.1	130
W sin Θ	$\overline{3.1}$	$\overline{7.2}$	$\overline{8.4}$	2.4	15.4	30	48	50	27	156	156

$$Fs = \frac{\Sigma(Cu \times 1 + W\cos\Theta \times \tan \phi u) + Wc}{\Sigma \sin \Theta} \geqq 1.2$$

$$\frac{130 + Wc}{156} \geqq 1.2 \quad \therefore Wc \geqq 57 = 60$$

Further Work Program for Detailed Design

1 Bridge Foundation

As a result of the present boring, the depth to the basal rock layer (Nubian formation) was nearly 15m. Since interval of boring is about 250m in the river, this is insufficient to confirm its correct depth at every pier position. Therefore, check borings will be required for the detailed design.

2 Subsoil Investigation In The Rainy Season

Present subsoil investigation has been carried out late in the dry season. During this time, the stronger conditions have been tested. Normally, as the embankment site is covered with flood, the natural subsoil condition should be examined in the rainy season. Therefore investigation should be repeated with more comprehensive examination by test boring, Duch cone sounding, undisturbed sampling, and soil tests late in the rainy season, when the soils are in their weakest condition. In case of soil test, the consolidation factor and shearing properties of C.D condition (Consolidated and Drained) should be carefully examined.

3 Basalt Quarry

The basalt is a promising rock material in quality. Considering the present condition of quarry site, the outcrop hill appears small, therefore, check boring is required to confirm the estimated amount of it.

4 Embankment Material

From the viewpoint of compaction in construction, the water content of material is a key factor. In particular, the natural water content in the rainy season is very important for studying the engineering properties in relation to the optimum conditions for compaction. Consequently, the following tests are required:

- Sampling and soil tests of the natural conditions in rainy season
- Compaction tests
- Mechanical tests on the condition of probable optimum compaction (Consolidation, Unconfined compressive strength, Shearing and C.B.R).pa

5 Survey of Desert Sand

The estimated amount of the desert sand, which is used for sand material of concrete, shall be confirmed by the test pitting and sieving in a large area.

3.4.6 Concrete Test

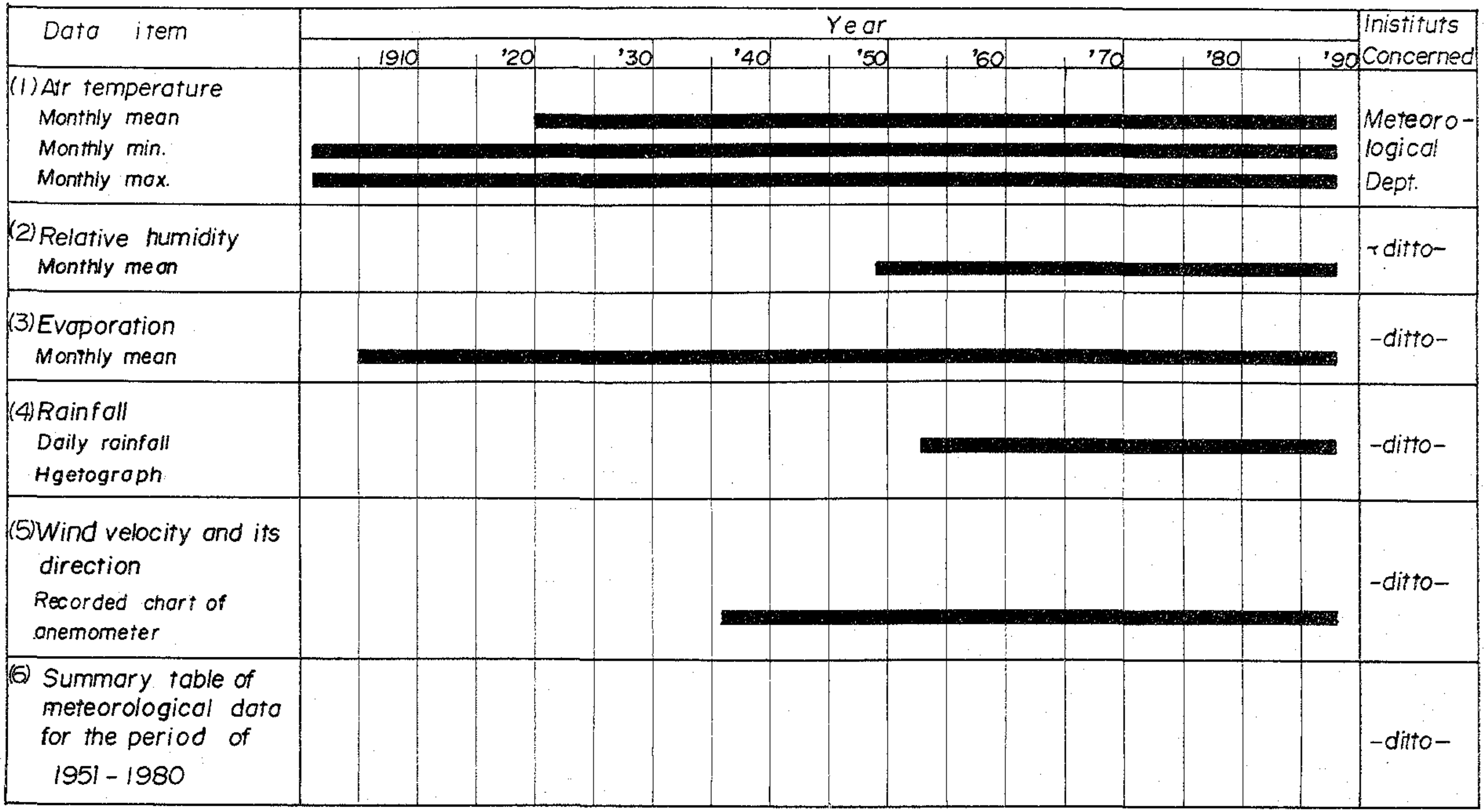
The following items shall be examined for concrete:

(1) Tests of mixtures using desert sand as the fine aggregate

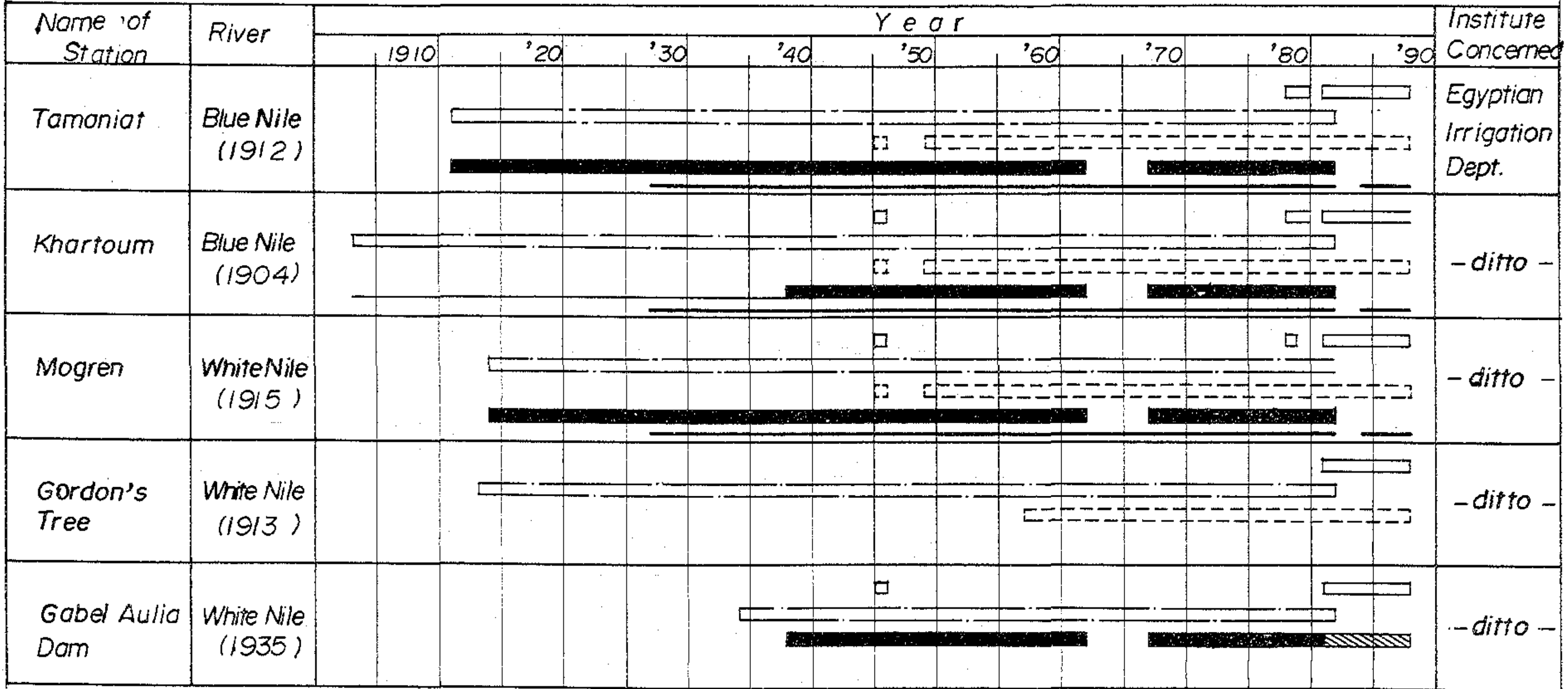
(2) Finding out the most suitable additives

(3) Curing in the hot and dry condition (60 deg.C in temperature and 50 % in humidity in the wind flow)

Inventory of Meteorological Data Collected



Inventory of Hydrological Data Collected

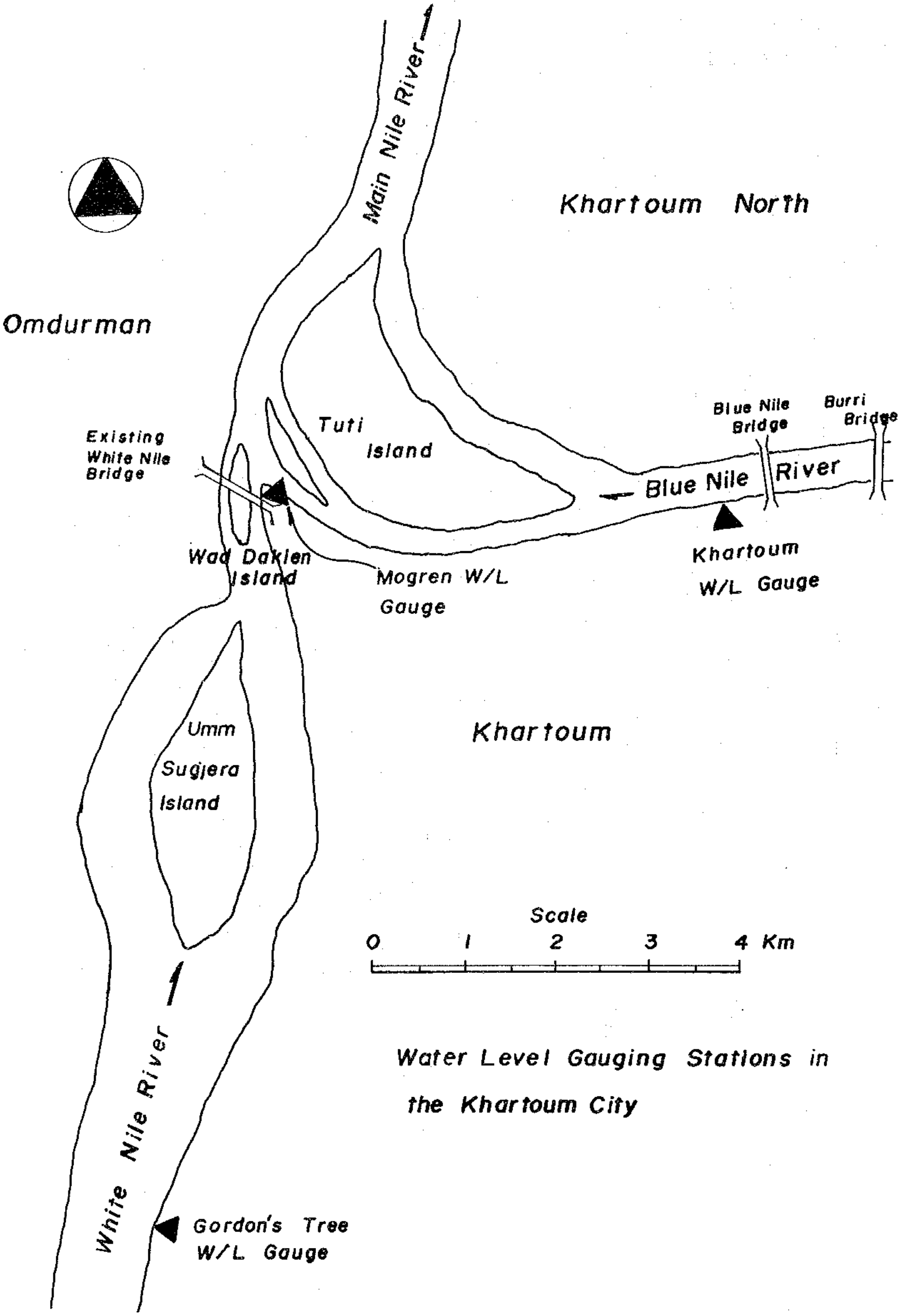


Legend

Water Level
- Daily mean
- Ten day mean
- Annual max.

Dischcarge
- Daily mean
- Ten-day mean
- Discharge Measurement

Note Figures in parenthis mean the installed year of station



Water Level Gauging Stations in the Khartoum City

Appendix 6.8

Meteorological Records at Khartoum Observatory

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Mean/Total	Period
						M o n t h								
(1) Air temperature (°C)														
- Monthly mean	23.2	24.2	27.9	31.1	28.6	33.7	31.3	30.3	31.1	31.6	27.9	24.5	28.8	1921-1988
- Minimum	6.0	6.9	9.3	11.4	17.5	19.8	17.6	18.0	16.0	16.5	11.0	6.0	-	1902-1988
- Maximum	40.1	43.3	45.2	47.2	47.3	47.7	47.0	43.5	45.3	44.7	42.0	40.2	-	1902-1988
(2) Relative humidity														
(%)	27	22	18	17	20	29	44	52	43	28	27	30	30	1949-1988
(3) Monthly Rainfall														
- Mean (mm)	0	0	0	0	4	6	42	71	23	4	1	0	151	1951-1986
- Rainy days	0	0	0	0	1	2	6	7	4	1	0	0	22	1901-1988
(4) Evaporation														
(mm/day)	13.6	16.2	19.3	21.0	19.9	18.6	14.7	11.7	13.5	15.6	15.7	13.4	16.1	1906-1988
(5) Wind Velocity (m/hour)														
- Prevailing direction	N	N	N	N	SSW	SSW	SSW	S	SSW	N	N	N	-	1971-1980
- Mean velocity (m/s)	4.5	4.5	4.9	4.0	3.1	4.0	4.0	3.6	3.6	3.1	4.0	4.5	4.0	1971-1980
- Maximum velocity (m/s)	13.4	15.6	14.3	20.6	21.0	25.5	22.8	34.9	25.5	22.3	12.9	13.4	-	1938-1988

Source : Meteorological Department, Ministry of Defence

Annual Highest Water Level at Mogran (1/2)

Order	Year	Date Month	Date Day	Gauge Reading (m)	Water Level (Rl.m)	Exceedance Probability (Weible Plot)
1	1946	Sep.	2	17.26	379.96	1%(75-year)
2	1988	Aug.	28	17.05	379.75	3%(30-year)
3	1917	Sep.	17	17.00	379.70	4%
4	1954	Aug.	31	16.80	379.50	5%(20-year)
5	1975	Sep.	20	16.78	379.48	7%
6	1961	Sep.	4	16.74	379.44	8%
7	1964	Aug.		16.74	379.44	9%(10-year)
8	1958	Sep.	1	16.70	379.40	11%
9	1959	Sep.	3	16.65	379.35	12%
10	1963	Aug.	3	16.57	379.27	13%
11	1970	Aug.	30	16.48	379.18	15%
12	1938	Sep.		16.45	379.15	16%
13	1955	Sep.	4	16.44	379.14	17%
14	1916	Sep.		16.43	379.13	19%
15	1985	Sep.	9	16.32	379.02	20%(5-year)
16	1957	Aug.	29	16.30	379.00	21%
17	1929	Sep.		16.27	378.97	23%
18	1934	Sep.		16.26	378.96	24%
19	1969	Aug.	26	16.24	378.94	25%
20	1953	Aug.	30	16.23	378.93	27%
21	1960	Aug.	27	16.18	378.88	28%
22	1971	Aug.	25	16.18	378.88	29%
23	1956	Aug.	14	16.18	378.88	31%
24	1974	Aug.	22	16.17	378.87	32%
25	1935	Aug.		16.12	378.82	33%
26	1947	Sep.		16.12	378.82	35%
27	1977	Aug.	17	16.12	378.82	36%
28	1952	Aug.	26	16.10	378.80	37%
29	1936	Sep.		16.09	378.79	39%
30	1967	Aug.	29	16.08	378.78	40%
31	1937	Aug.		16.08	378.78	41%
32	1942	Aug.		16.06	378.76	43%
33	1966	Sep.	19	16.00	378.70	44%
34	1962	Aug.	29	15.97	378.67	45%
35	1923	Aug.		15.94	378.64	47%
36	1924	Sep.		15.92	378.62	48%
37	1922	Sep.		15.91	378.61	49%(2-year)
38	1928	Sep.		15.90	378.60	51%
39	1933	Sep.		15.90	378.60	52%
40	1926	Aug.		15.85	378.55	53%
41	1919	Sep.		15.84	378.54	55%
42	1973	Aug.	26	15.78	378.48	56%
43	1932	Sep.		15.76	378.46	57%
44	1945	Sep.		15.76	378.46	59%
45	1951	Sep.	2	15.74	378.44	60%

Annual Highest Water Level at Mogran (2/2)

Order	Year	Date Month	Date Day	Gauge Reading (m)	Water Level (Rl.m)	Exceedance Probability (Weible Plot)
46	1943	Sep.		15.71	378.41	61%
47	1950	Aug.	14	15.70	378.40	63%
48	1949	Sep.		15.66	378.36	64%
49	1931	Sep.		15.65	378.35	65%
50	1983	Sep.	7	15.65	378.35	67%
51	1978	Aug.	22	15.60	378.30	68%
52	1981	Sep.	3	15.58	378.28	69%
53	1921	Sep.		15.55	378.25	71%
54	1927	Aug.		15.53	378.23	72%
55	1980	Aug.	20	15.50	378.20	73%
56	1968	Aug.	16	15.48	378.18	75%
57	1918	Sep.		15.46	378.16	76%
58	1948	Aug.		15.45	378.15	77%
59	1965	Aug.	23	15.38	378.08	79%
60	1930	Aug.		15.38	378.08	80%
61	1976	Aug.	26	15.33	378.03	81%
62	1939	Aug.		15.29	377.99	83%
63	1944	Aug.		15.28	377.98	84%
64	1920	Aug.		15.26	377.96	85%
65	1915	Sep.		15.24	377.94	87%
66	1940	Aug.		15.21	377.91	88%
67	1925	Aug.		15.20	377.90	89%
68	1979	Aug.	23	15.14	377.84	91%
69	1987	Aug.	29	15.10	377.80	92%
70	1982	Aug.	27	15.00	377.70	93%
71	1941	Sep.		14.87	377.57	95%
72	1986	Sep.	3	14.68	377.38	96%
73	1972	Aug.	31	14.39	377.09	97%
74	1984	Aug.	26	13.46	376.16	99%

Note : The above are based on ten-day and daily mean data.

Source : Ministry of Irrigation

Annual Lowest Water Level at Mogren

Order	Year	Date Month	Day	Gauge Reading (m)	Water Level (Rl.m)
1	1945	May		9.92	372.62
2	1987	Mar.	6	10.00	372.70
3	1946	May	29	10.08	372.78
4	1951	May		10.10	372.80
5	1940	May		10.13	372.83
6	1988	Feb.	24	10.14	372.84
7	1943	May		10.19	372.89
8	1944	May		10.20	372.90
9	1985	Feb.	26	10.20	372.90
10	1986	June.	15	10.20	372.90
11	1938	May		10.22	372.92
12	1942	May		10.23	372.93
13	1953	May		10.30	373.00
14	1952	May		10.32	373.02
15	1983	Mar.	7	10.36	373.06
16	1984	June	6	10.39	373.09
17	1954	May		10.42	373.12
18	1961	June		10.44	373.14
19	1948	May		10.45	373.15
20	1973	Mar.		10.45	373.15
21	1978	Mar.		10.45	373.15
22	1980	Mar.		10.48	373.18
23	1958	May		10.49	373.19
24	1949	May		10.54	373.24
25	1979	Feb.	11	10.54	373.24
26	1966	Mar.		10.60	373.30
27	1974	Feb.		10.62	373.32
28	1982	Mar.	7	10.62	373.32
29	1939	Mar.		10.63	373.33
30	1970	Apr.		10.63	373.33
31	1975	June		10.65	373.35
32	1981	Feb.		10.66	373.36
33	1976	Mar.		10.68	373.38
34	1960	May		10.70	373.40
35	1950	June		10.73	373.43
36	1959	May		10.75	373.45
37	1977	Mar.		10.82	373.52
38	1969	June		10.84	373.54
39	1972	Mar.		10.84	373.54
40	1968	Mar.		10.87	373.57
41	1971	Mar.		10.91	373.61
42	1963	Mar.		10.93	373.63
43	1964	Mar.		10.95	373.65
44	1967	Apr.		11.05	373.75
45	1955	June		11.07	373.77
46	1947	May		11.08	373.78
47	1957	Feb.		11.13	373.83
48	1965	Apr.		11.30	374.00
49	1962	May		11.38	374.08
50	1918	May		11.40	374.10
51	1956	May		11.44	374.14

Note : The above are based on Ten-day and daily mean data.
Source : Ministry of Irrigation

Annual Maximum Discharges at Downstream of Gebel Aulia Dam

Year	Month	Discharge mil.m^3 /day	m^3/sec
1948	Oct.	91.9	1,064
1949	Oct.	96.1	1,112
1950	Nov.	89.9	1,041
1951	Jan.	86.7	1,003
1952	Feb.	80.0	926
1953	Mar.	84.4	977
1954	Nov.	94.7	1,096
1955	Jan.	84.7	980
1956	Nov.	89.5	1,036
1957	May	90.2	1,044
1958	Mar.	84.9	983
1959	Sep.	86.5	1,001
1960	Apr.	80.1	927
1961	Nov.	94.9	1,098
1962	May	95.1	1,101
1963	-	-	-
1964	-	-	-
1965	-	-	-
1966	-	-	-
1967	-	-	-
1968	Jan.	110.0	1,273
1969	Apr.	120.0	1,389
1970	Apr.	120.0	1,389
1971	Apr.	120.0	1,389
1972	Apr.	120.0	1,389
1973	Oct.	109.0	1,262
1974	Oct.	108.0	1,250
1975	Nov.	126.0	1,458
1976	May	123.0	1,424
1977	Sep.	108.0	1,250
1978	Oct.	105.0	1,215
1979	Apr.	120.0	1,389
1980	May	112.0	1,296
1981	Oct.	110.0	1,273
1982	Dec.	101.0	1,169
1983	Mar.	105.0	1,215
1984	Jan.	119.0	1,377
1985	Dec.	125.0	1,447
1986	Oct.	106.0	1,227
1987	Apr.	85.0	984
1988	Dec.	131.0	1,516
Maximum			1,516

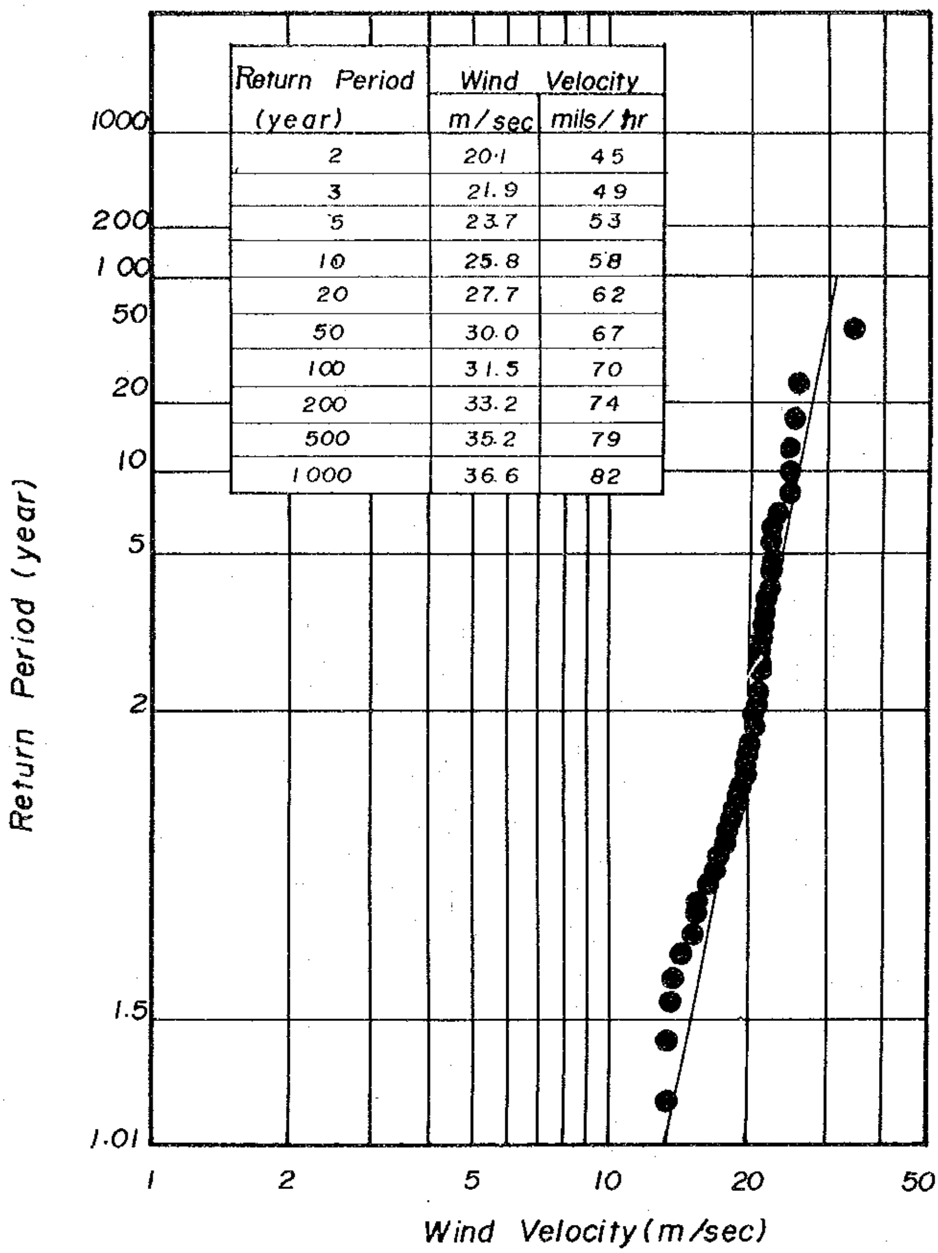
Source : Ministry of Irrigation

Appendix 6.12

Annual Maximum Wind Velocity at Khartoum

Year	Date	Direction	Wind Velocity (m/sec)
1938	Aug. 22	E	17.4
1939	Apr. 5	SSW	16.5
1940	Aug. 20	EES	20.1
1941	June 25	S	21.9
1942	Aug. 30	SE	13.4
1943	Aug. 23	S	13.9
1944	Jun. 16	SE	17.9
1945	Sep. 16	E	15.6
1946	Jul. 10	E	13.4
1947	Aug. 2	W	13.9
1948	Aug. 9	E	15.2
1949	Aug. 24	EES	22.3
1950	Aug. 20	E	24.6
1951	Aug. 30	E	22.8
1952	Aug. 25	NE	22.8
1953	Sep. 6	E	22.8
1954	Sep. 6	-	21.0
1955	Jul. 16	E	21.9
1956	Sep. 2	NE	21.9
1957	Aug. 12	E	34.9
1958	Jul. 25	E	22.3
1959	Aug. 10	E	19.2
1960	Jun. 20	S	19.7
1961	Jul. 24	E	22.3
1962	Jun. 19	E	25.0
1963	Jul. 10	E	22.8
1964	Jul. 20	E	21.5
1965	Sep. 3	E	25.5
1966	Jun. 9	EES	22.8
1967	May 9	S	21.0
1968	Sep. 2	E	21.9
1969	Jun. 26	E	21.5
1970	Sep. 5	E	18.8
1971	Sep. 7	S	20.1
1972	Aug. 25	NE	22.8
1973	July 30	S	25.0
1974	Sep. 15	S	20.6
1975	Sep. 21	SSW	26.4
1976	Sep. 6	S	21.0
1977	Aug. 20	E	23.7
1978	July 12	SE	14.7
1979	June 15	ESE	20.1
1980	June 3	NE	17.9
1981	July 29	-	22.3
1982	Aug. 18	E	20.6
1983	July 9	S	17.0
1984	July 23	S	15.6
1985	Aug. 10	ESE	22.3
1986	- -	-	-
1987	Sep. 16	SSE	19.0
1988	Sep. 1	SEE	18.0
Mean			20.5

Source : Meteorological Department,
Ministry of Defence



Return Period (year)	Wind Velocity m/sec	Wind Velocity mils/hr
2	20.1	45
3	21.9	49
5	23.7	53
10	25.8	58
20	27.7	62
50	30.0	67
100	31.5	70
200	33.2	74
500	35.2	79
1000	36.6	82

Frequency Curve Annual Maximum Wind Velocity

Source : The Study Team

Estimation of Wind Current Wave

1. Estimation Condition

According to Bretschneider's method, wind velocity, average water depth and fetch characterize wind current wave. In the study, the following figures are set for estimation of wave height (H):

- Wind velocity	(U)	31.5	m/sec
- Water depth	(h)	6	m
- Fetch	(F)	11,000	m
- Gravitational acceleration	(g)	9.8	m/sec^2

2. Calculation

(1) Wave Height

Bretschneider prepared the relation between wave height, wind velocity, fetch and water depth as shown in Figure 1. According to this figure, it is easy to derive wave height by calculating (gF/U^2) and (gF/U^2) and reading (gH/U^2) from the figure.

Results are as below:

$gh/U^2 = 9.8 \times 6.0 / 31.5^2 = 6 \times 10^{-2}$

$gF/U^2 = 9.8 \times 11{,}000 / 31.5^2 = 109$

From the figure, gH/U^2 is read at 2×10^{-2} and wave height are calculated as follow:

$$
\begin{aligned}
H &= U^2 \times 2 \times 10^{-2} / g \\
&= 31.5^2 \times 2 \times 10^{-2} / 9.8 \\
&= 2.02
\end{aligned}
$$

H calculated is rounded up to 2.1 m of wave height.

(2) Wave Length

Firstly, wave period is needed to estimate wave length. Bretschneider proposed the following procedures:

- Assuming H equivalent wave height of deep water wave, fetch (F) is read at 8,000 m from wind velocity (U) and wave height (H), based on Figure 2.

- Inputting this fetch to the following equation for estimation of period (T), prepared in S.M.B. Method:

$$gT/2/3.14/U = 1.37\ (1-(1+0.008(gF/U^2)^{1/3})^{-5})$$
$$= 1.37\ (1-(1+0.008(9.8x8000/31.5^2)^{1/3})^{-5})$$
$$= 0.21$$

$$T = 0.21 \times 2 \times 3.14 \times 31.5 / 9.8$$
$$= 4.3 \text{ second}$$

Wave length (L) is calculated based on the following equation, prepared on the basis of Small Amplitude Wave Theory:

$$L = (g/2/3.14) \times T^2$$
$$= (9.8/2/3.14) \times 4.3^2$$
$$= 29.1 \text{ m}$$

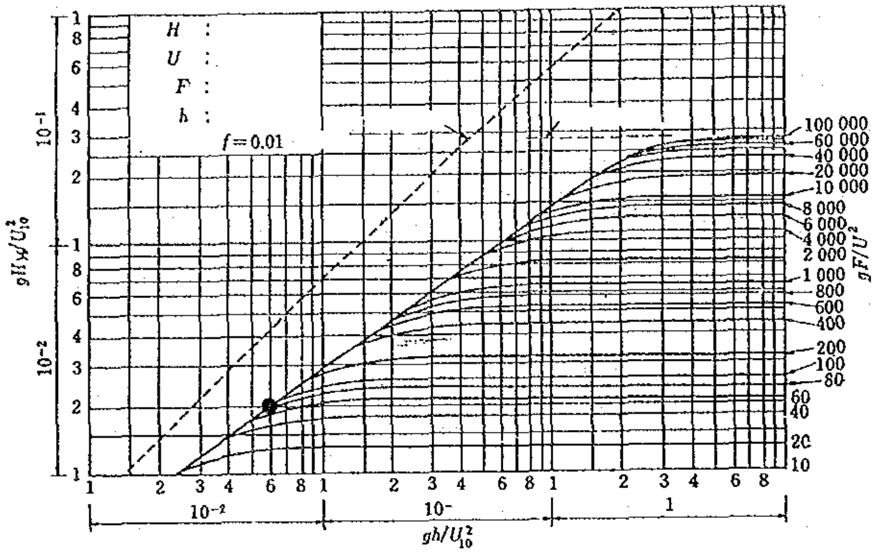


Figure 1

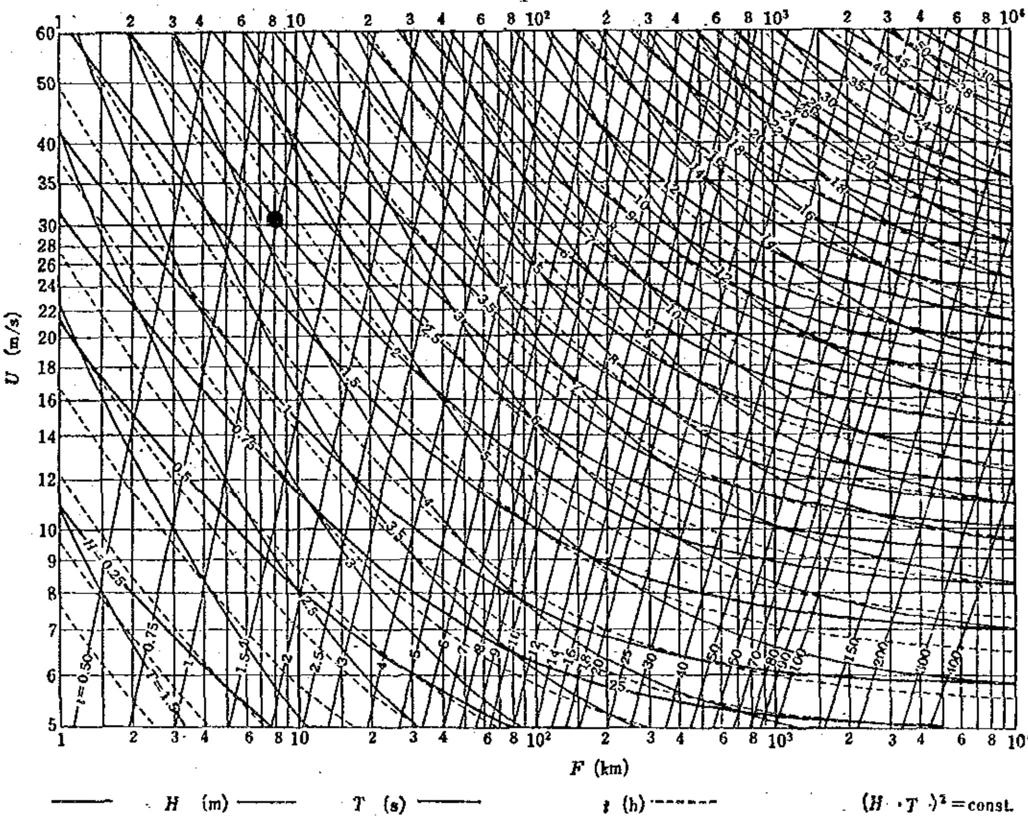
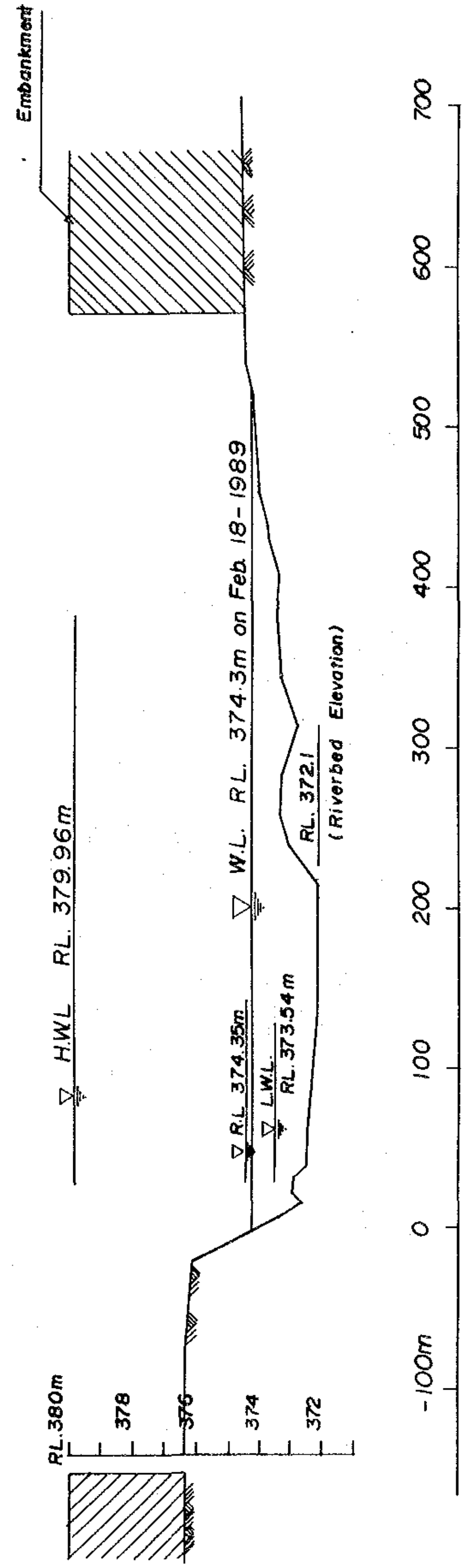
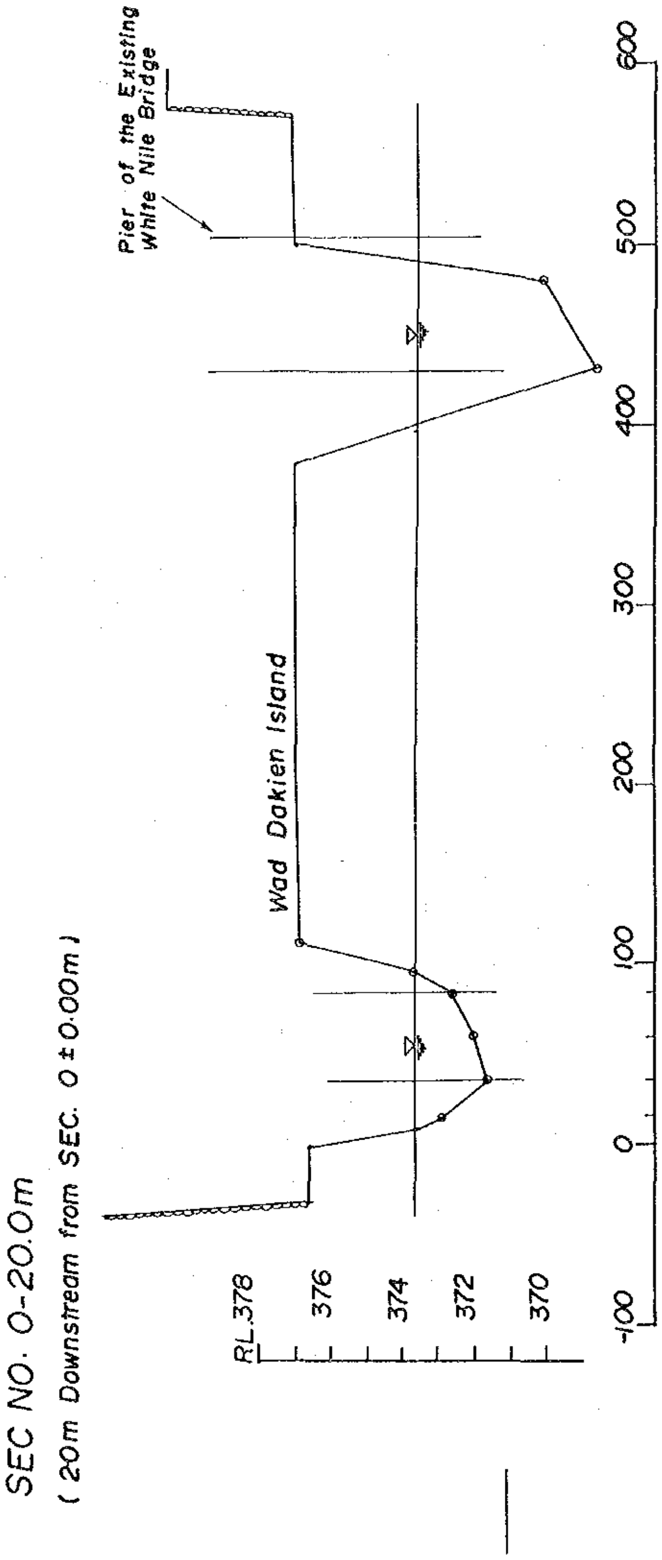


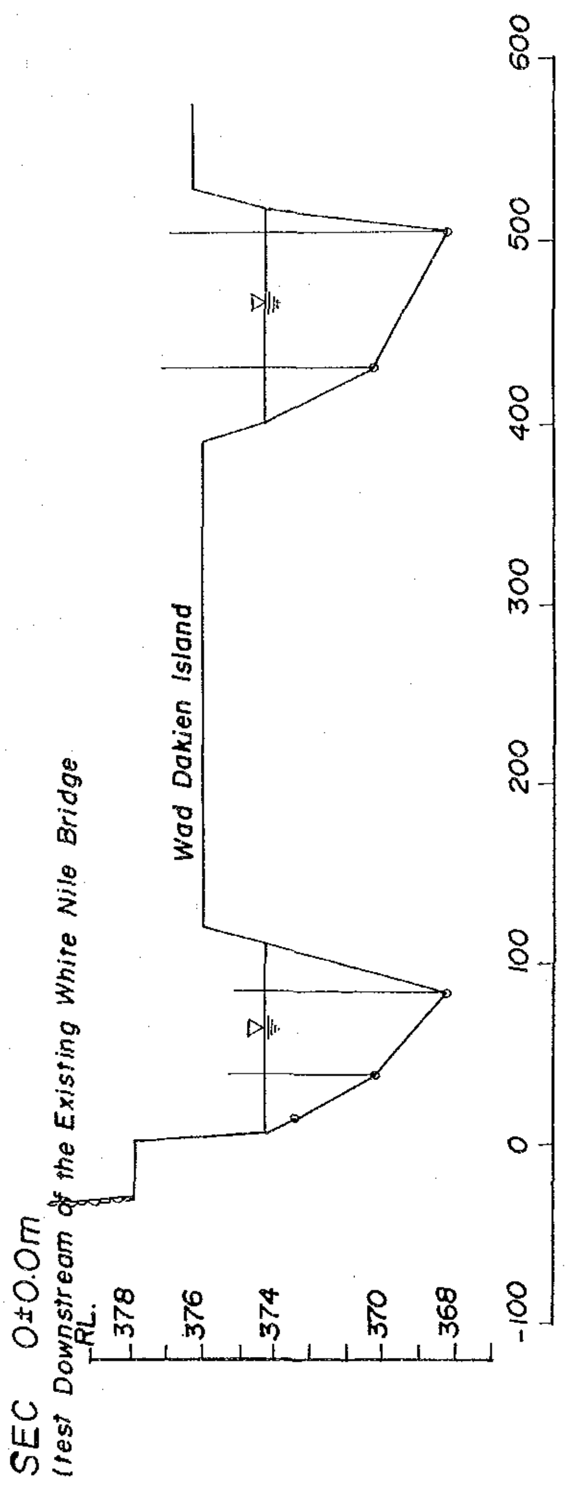
Figure 2

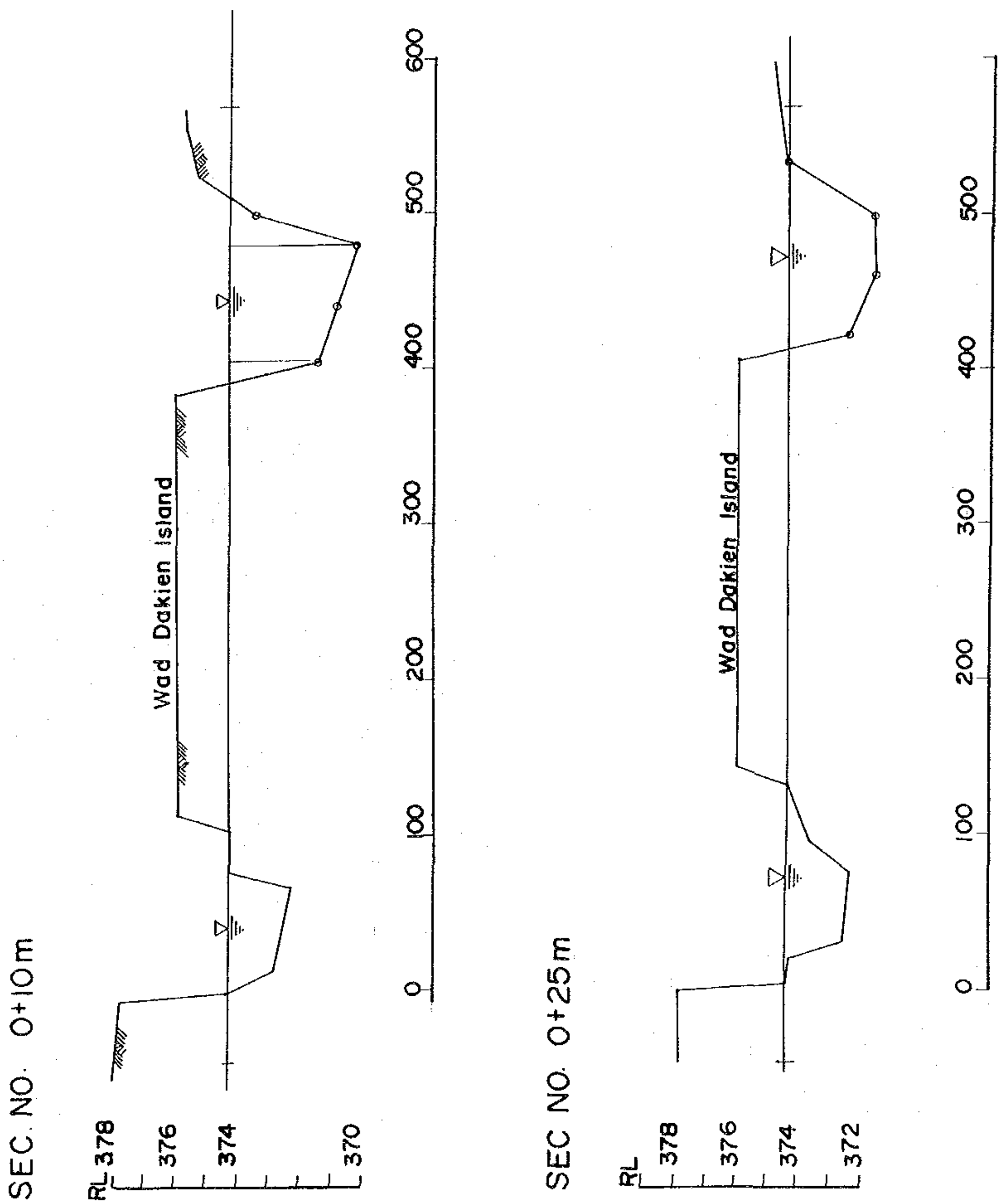


Source : The Study Team

River Cross-section at the Proposed Bridge Site


SEC NO. 0-20.0m
(20m Downstream from SEC. 0±0.00m)
Pier of the Existing White Nile Bridge
Wad Dakien Island
RL.378
376
374
372
370
-100
0
100
200
300
400
500
600


SEC 0±0.0m
Downstream of the Existing White Nile Bridge
Wad Dakien Island
RL.
378
376
374
370
368
-100
0
100
200
300
400
500
600

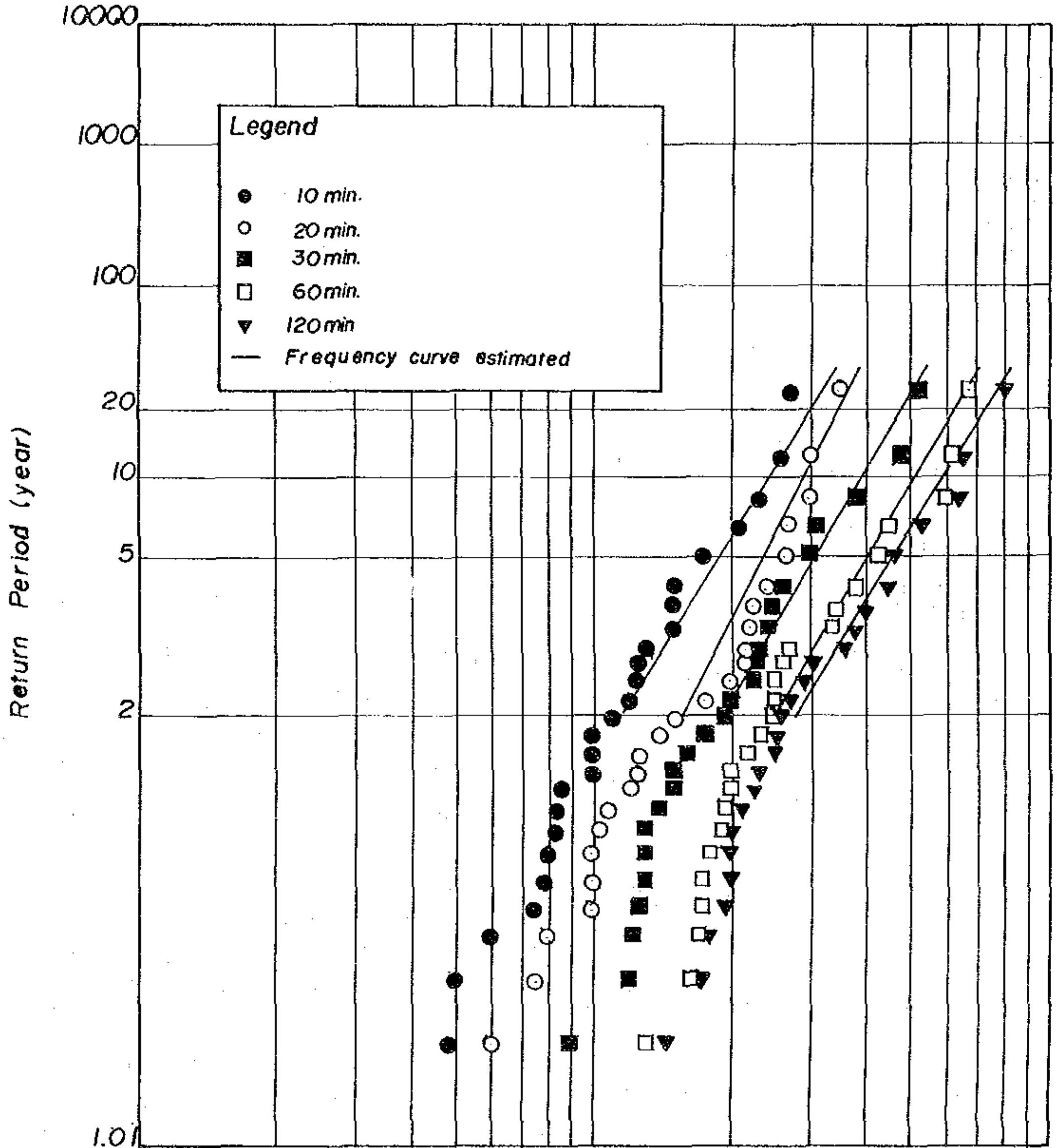


River Cross Section around the Project Site

Annual Maximum Rainfall Amount and Its Duration

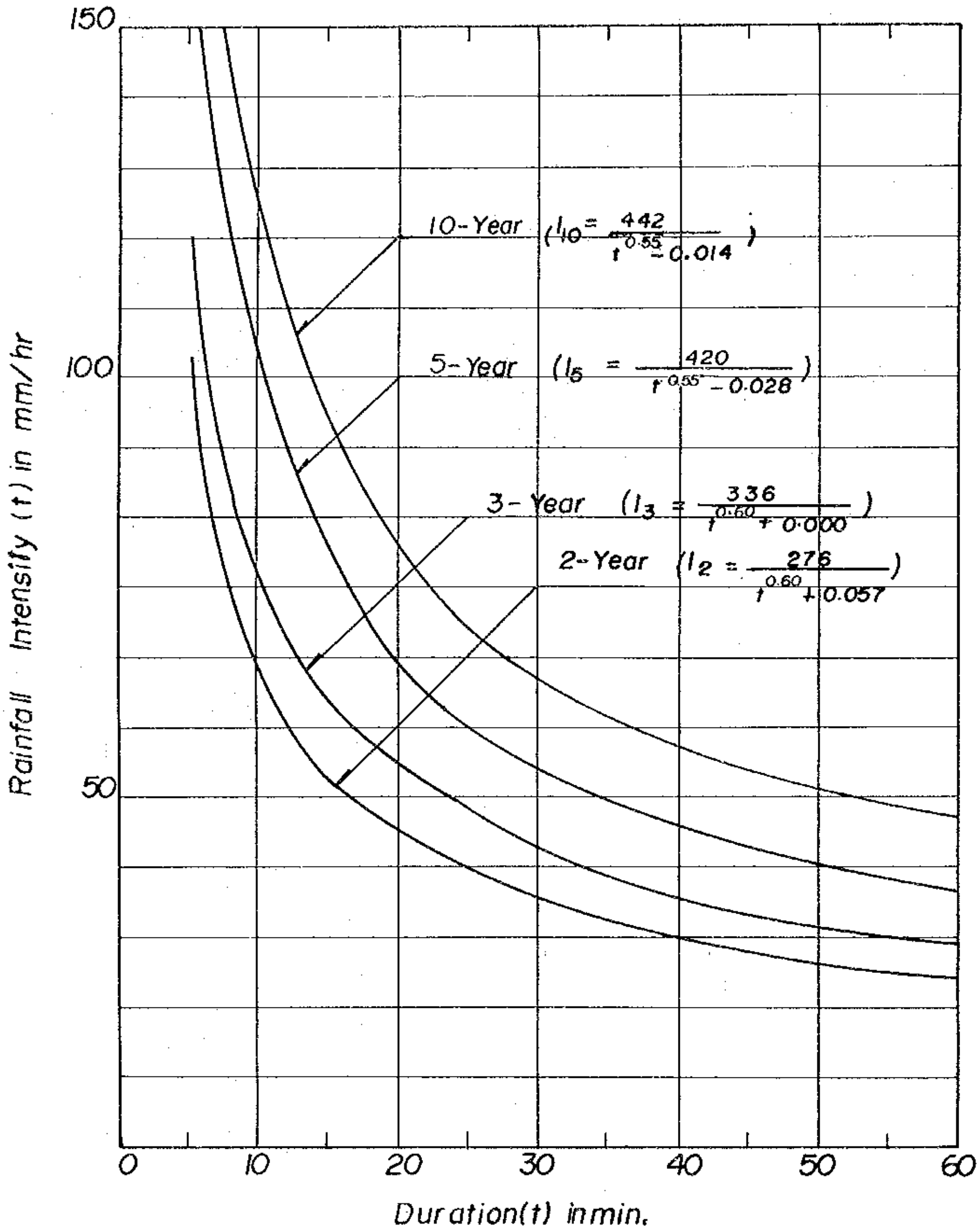
Year	Date	Rainfall Amount (mm) Duration (min.) 10	20	30	60	120	180
1951	Aug.19	4.9	6.0	8.6	16.6	17.6	17.6
1952	–	–	–	–	–	–	–
1953	Aug.27	15.0	21.5	25.5	26.8	27.0	27.0
1954	–	–	–	–	–	–	–
1955	–	–	–	–	–	–	–
1956	Sep.30,Aug.11	10.0	12.5	19.5	27.0	29.5	30.5
1957	Aug.10	11.0	15.0	13.0	34.3	35.5	36.2
1958	Aug.14	6.0	10.0	15.0	18.0	20.0	20.0
1959	Sep. 5	13.0	21.8	23.1	25.1	26.1	27.1
1960	Aug. 3	8.0	12.0	15.0	19.5	19.5	19.5
1961	July 8	12.5	30.0	37.5	67.5	79.0	79.0
1962	Sep. 3	26.3	27.0	31.0	34.0	40.0	46.0
1963	Aug.10,Sep.24	7.9	8.0	13.0	25.0	25.2	25.2
1964	Aug.14	8.5	10.0	13.0	19.0	22.5	24.3
1965	Aug.24	15.0	22.5	23.0	23.5	28.9	32.5
1966	Aug. 1	5.0	7.5	12.5	17.5	20.8	28.3
1967	Aug.15	27.5	35.0	52.5	61.0	64.8	68.7
1968	Aug.14	8.3	13.8	17.5	21.7	38.3	44.9
1969	Aug.17	10.0	12.5	12.7	13.0	14.3	14.3
1970	–	–	–	–	–	–	–
1971	Aug.13	10.0	10.2	15.8	17.1	17.1	17.1
1972	Aug.25	23.2	30.0	43.0	62.5	64.0	64.0
1973	Aug. 2	12.0	24.0	25.1	25.5	25.5	25.5
1974	–	–	–	–	–	–	–
1975	Aug. 3	7.5	10.0	12.5	17.5	20.0	22.0
1976	Aug.31	12.5	20.0	22.5	37.5	52.5	55.0
1977	Aug.20	17.1	22.1	24.6	42.5	45.5	49.2
1978	July19	15.0	17.5	20.0	20.0	20.0	20.0
1979	Aug.15	8.2	10.6	14.0	20.0	23.0	25.0
1980	–	–	–	–	–	–	–
1981	Aug.29	21.3	27.0	30.0	44.5	44.5	44.5

Source : Meteorological Department,
Ministry of Defence



Source : The Study Team

Frequency Curve of Annual Maximum
Rainfall at Khartoum



Source : The Study Team

Relation between Probable Rainfall Intensity and Duration

Estimation of Wave-Run-Up Height

Wave-run-up height is estimated according to the Saville's equivalent slope method below:

1. Determination of Wave Breaking Point

Wave height coming wave to bank slope and wave breaking facility and its length estimated in Appendix V- are as below:

- Wave height	(H)	2.1	m
- Wave length	(L)	29.1	m

From the above values, H/L is worked out at 0.07. According to Figure 1 and 0.07 of H/L, hb/H (hb : wave breaking depth) is read at 1.3 and hb is calculated at 2.7 m.

2. Wave-Run-Up Height

As shown in Figure 2, the wave-run-up height is assumed firstly. Secondarily, cot a and H/L are calculated and R/H (R : wave-run-up height) is read from Figure 3. Finally, R is calculated. This procedure will be repeated until the assumed and calculated values indicate same value.

In this study, wave-run-up height is worked out at 2 m based on the following values:

- Assumed wave-run-up height	2.0	m
- cot a	3.8	
- H/L	0.73	
- R/H	0.9	
- R	2.0	

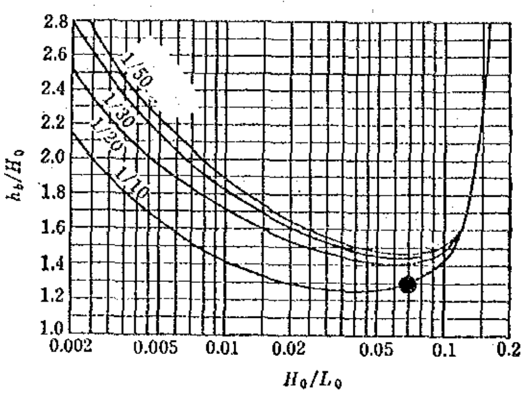


Figure 1

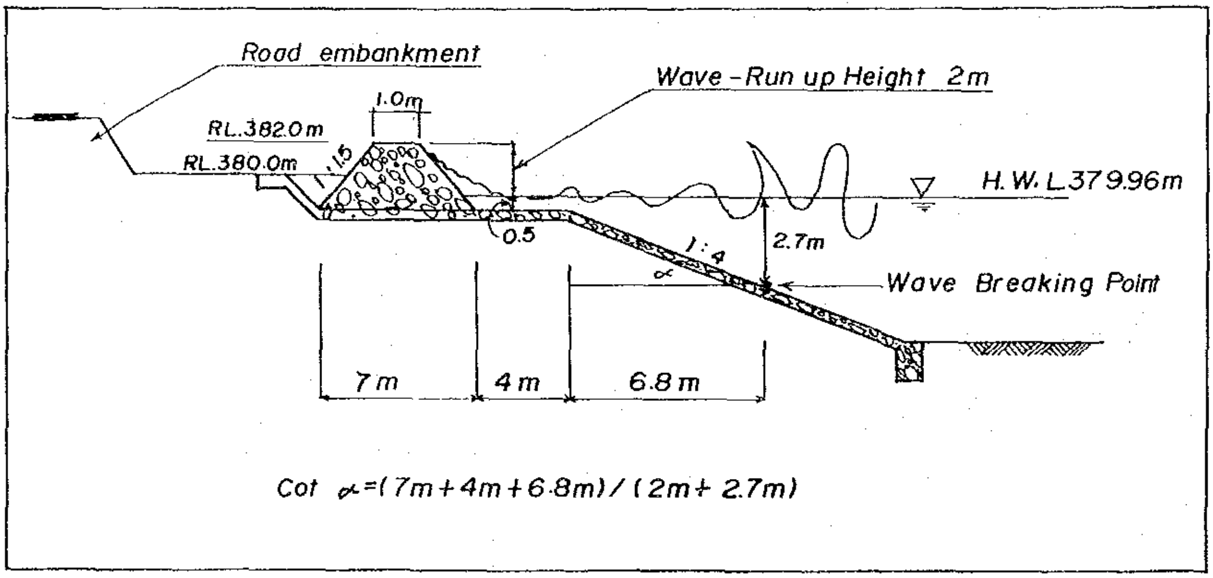


Figure 2

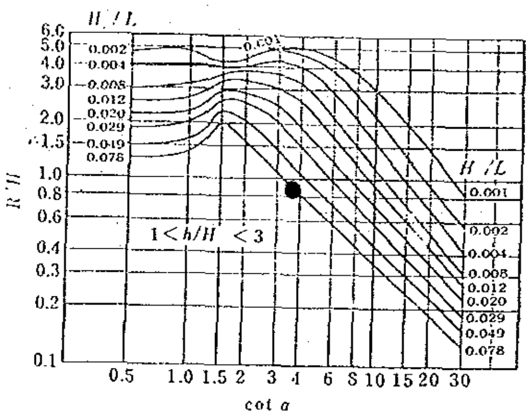
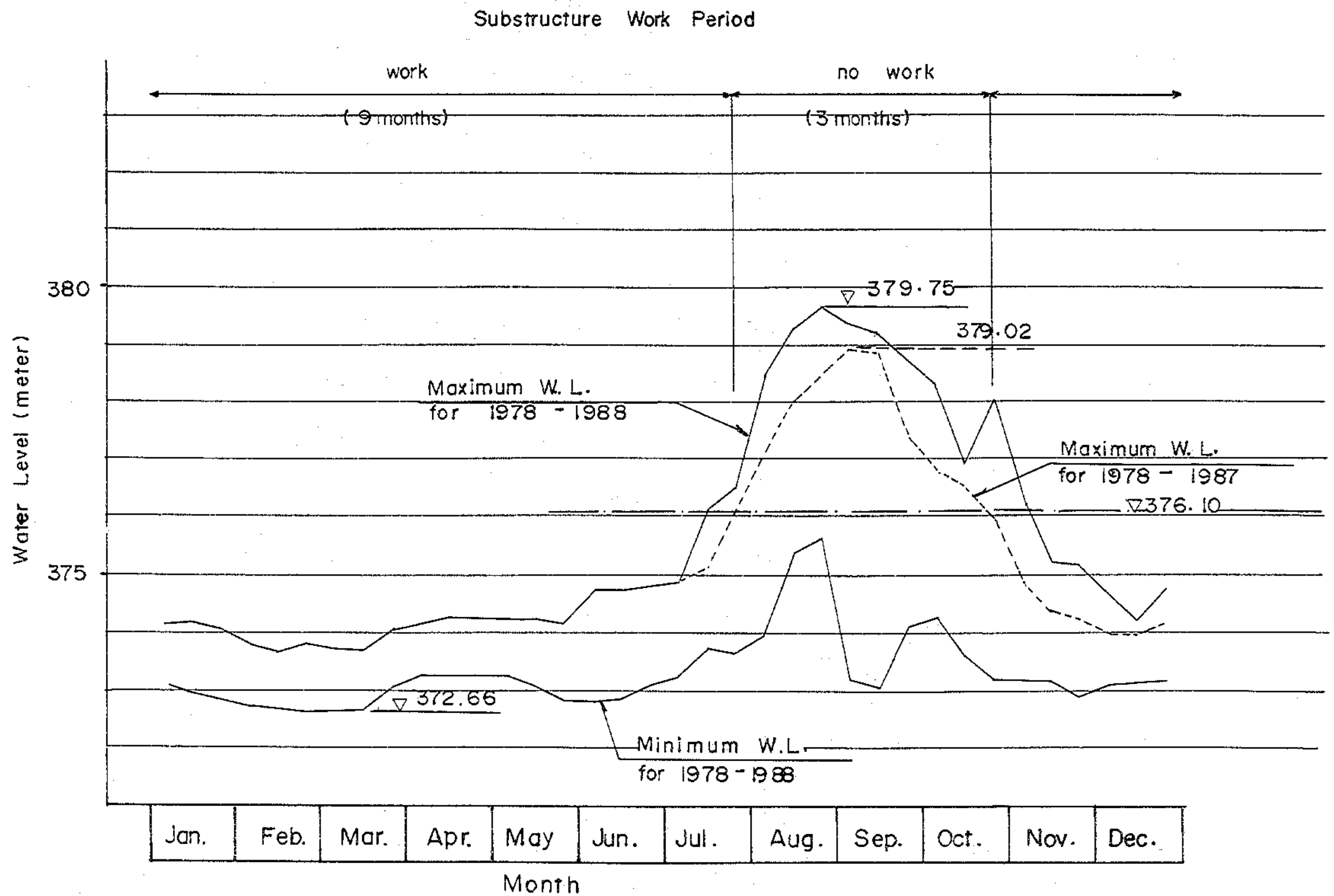


Figure 3



note The level of zero(0) is indicated mean sea level at Alexandria
Maximum and minimum water level at Mogren G.S.
For 1978 1988

Data Source : Ministry of Irrigation

Preliminary Bridge Engineering

Table of Contents

DISTRIBUTION FUNCTION
of OVERLOADED AXELS
KHARTOUM - WAD MADANI Section

Axel Load (ton) > - <=	Number of Survey AXELS	PERCENTAGES RBPC 1988 prelimi	HOIS 1986	CUMUL RBPC 1988 prelimi	HOIS 1986
10 - 11	194	29.39	15.97	29.39	15.97
11 - 12	162	24.55	18.47	53.94	34.44
12 - 13	127	19.24	15.18	73.18	49.62
13 - 14	80	12.12	13.11	85.30	62.73
14 - 15	49	7.42	11.42	92.72	74.15
15 - 16	21	3.18	9.35	95.90	83.50
16 - 17	16	2.42	7.79	98.32	91.29
17 - 18	5	0.76	4.64	99.08	95.93
18 - 19	1	0.15	1.94	99.23	97.87
19 - 20	4	0.61	0.95	99.84	98.82
20 - 21	1	0.15	0.46	99.99	99.28
21 - 22	0	0.00	0.46	99.99	99.70
22 - 23	0	0.00	0.23	99.99	99.93
23 - 24	0	0.00	0.04	99.99	99.97
24 - 25	0	0.00	0.00	99.99	99.97
25 - 26	0	0.00	0.03	99.99	100.00
26 - 27	0	0.00	0.00	99.99	100.00
27 - 28	0	0.00	0.00	99.99	100.00
28 - 29	0	0.00	0.00	99.99	100.00
29 - 30	0	0.00	0.00	99.99	100.00
*** Total ***	660	99.99	100.00		

Note: HOIS is shown " HIGHWAY ORGANIZATION AND INVESTMENT STUDY.
Ministry of Finance and Economic Planning.

(source : AXLE LOAD CONTROL SYSTEM axel load survey ROADS and BRIDGES PUBLIC CORPORATION October 1988).

TOTAL WEIGHT DISTRIBUTION FUNCTION
OF SURVEYED CONVOYS
KHARTOUM - WAD MADANI Section

Total Weight (ton) > - <=	Number of survey convoys	Average Weight Classe (ton)	PERCENTAGE per Convoys (%)	PERCENTAGE per Weight (%)	CUMUL percentage Convoys (%)	CUMUL percentage Weight (%)
10 - 13	0	0.00	0.00	0.00	0.00	0.00
13 - 16	1	14.20	0.36	0.10	0.36	0.10
16 - 19	1	18.00	0.36	0.13	0.72	0.23
19 - 22	2	19.50	0.72	0.28	1.44	0.51
22 - 25	1	22.80	0.36	0.16	1.80	0.67
25 - 28	0	0.00	0.00	0.00	1.80	0.67
28 - 31	4	29.75	1.45	0.86	3.25	1.53
31 - 34	18	33.04	6.52	4.29	9.77	5.82
34 - 37	20	35.87	7.25	5.17	17.02	10.99
37 - 40	33	38.54	11.96	9.17	28.98	20.16
40 - 43	30	41.91	10.87	9.07	39.85	29.23
43 - 46	18	44.51	6.52	5.78	46.37	35.01
46 - 49	12	47.77	4.35	4.13	50.72	39.14
49 - 52	20	50.46	7.25	7.28	57.97	46.42
52 - 55	19	53.68	6.88	7.36	64.85	53.78
55 - 58	25	56.46	9.06	10.18	73.91	62.96
58 - 61	13	59.97	4.71	5.62	78.62	69.58
61 - 64	14	62.50	5.07	6.31	83.69	75.89
64 - 67	10	65.64	3.62	4.73	87.31	80.62
67 - 70	11	68.65	3.99	5.45	91.30	86.07
70 - 73	9	71.51	3.26	4.64	94.56	90.71
73 - 76	3	75.00	1.09	1.62	95.65	92.33
76 - 79	1	76.60	0.36	0.55	96.01	92.88
79 - 82	2	80.70	0.72	1.16	96.73	94.04
82 - 85	2	83.90	0.72	1.21	97.45	95.25
85 - 88	1	86.00	0.36	0.62	97.81	95.87
88 - 91	4	90.25	1.45	2.60	99.26	98.47
91 - 94	1	93.60	0.36	0.68	99.62	99.15
94 - 97	0	0.00	0.00	0.00	99.62	99.15
97 - 100	0	0.00	0.00	0.00	99.62	99.15
100 - 103	0	0.00	0.00	0.00	99.62	99.15
103 - 106	0	0.00	0.00	0.00	99.62	99.15
106 - 109	0	0.00	0.00	0.00	99.62	99.15
109 - 112	0	0.00	0.00	0.00	99.62	99.15
112 - 115	1	114.40	0.36	0.83	99.98	99.98
115 - 118	0	0.00	0.00	0.00	99.98	99.98
118 - 121	0	0.00	0.00	0.00	99.98	99.98
121 -	0	0.00	0.00	0.00	99.98	99.98
Total	276		99.98	99.98		

Note: HOIS is shown "HIGHWAY CORGANIZATION AND INVESTMENT STUDY.
Ministry of Finance and Economic Planning".

(source: AXLE LOAD CONTROL SYSTEM axle load survey
ROADS and BRIDGES PUBLIC CORPORATION october 1988).

Comparative table of loads indicated in international standards

Kind of load	Japan Road Association Standards (J.R.A.)	AASHŌTŌ	British Standards B.S.

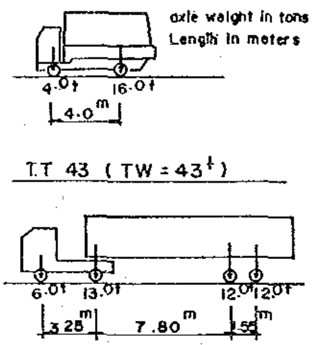
1) Dead Load

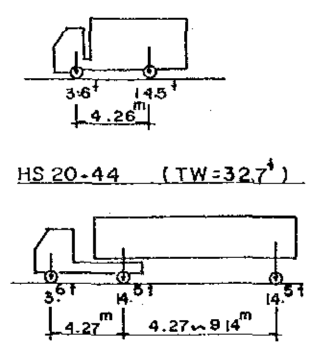
Materials weights in tons per cubic meters

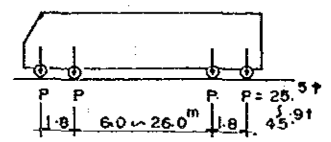
Materials		J. R. A.	AASHŌTŌ	B.S.
Steel or cast steel		7.85	7.85	no statement
Cast iron		7.25	7.21	
Concrete	Plain	2.35	2.40	
	Reinforced	2.50		
Asphalt pavement		2.30	2.40	

2) Live Load

J.R.A.	AASHŌTŌ	B.S.
T-20 (Total weight 20^t)	H 20-44 (TW = 18.1^t)	HA
TT 43 (TW = 43^t)	HS 20-44 (TW = 32.7^t)	HB 25-45 (TW = $102 \sim 184^t$)

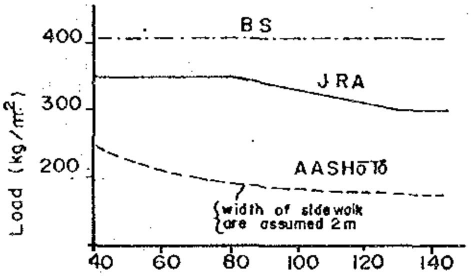




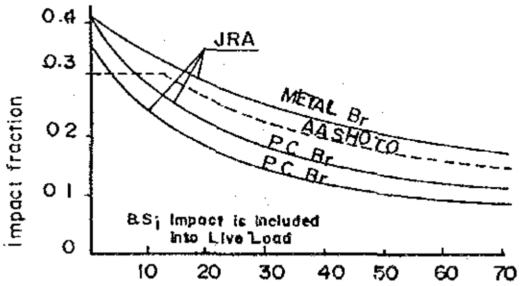


3) Sidewalk Load

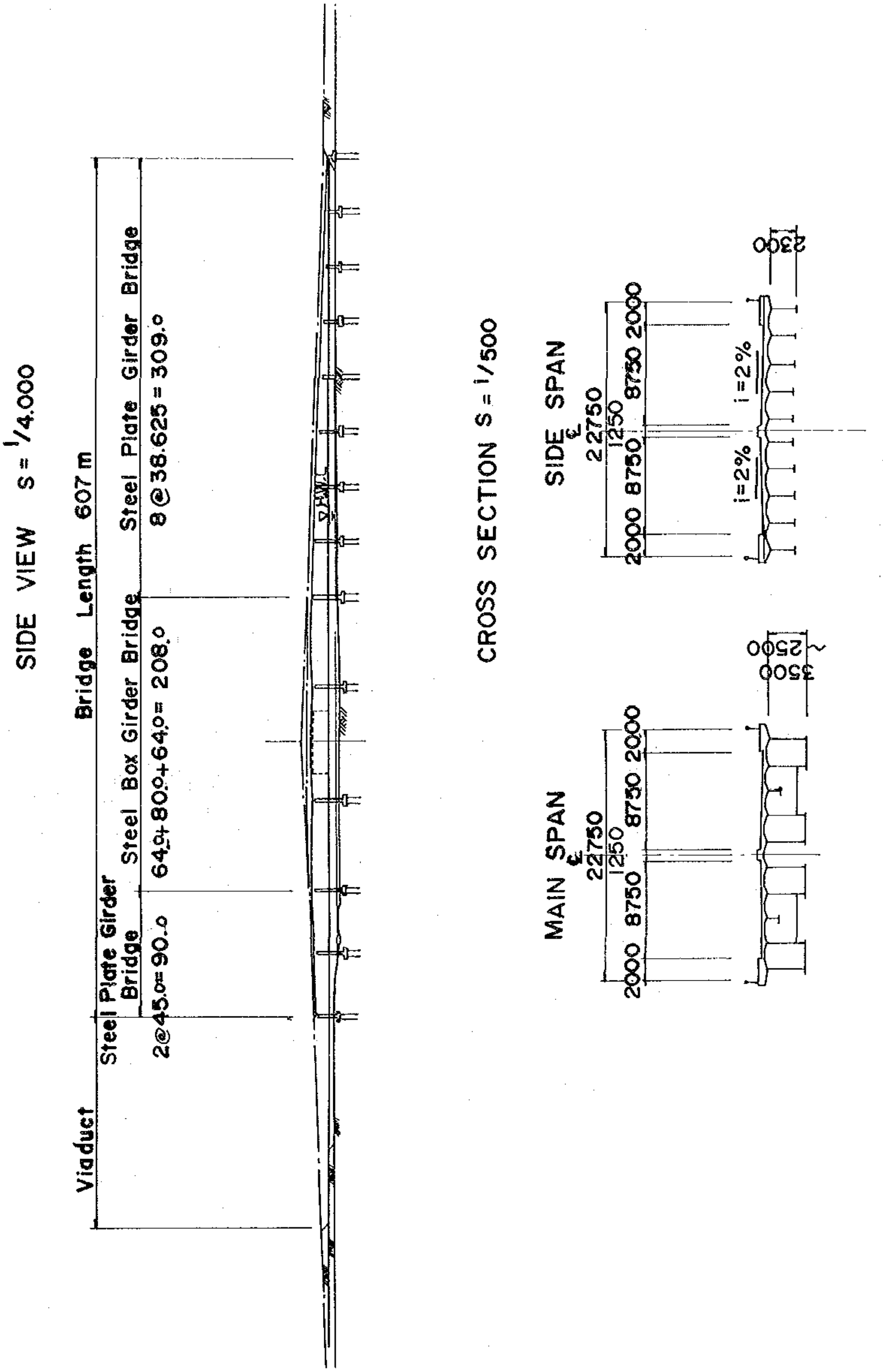
4) Impact



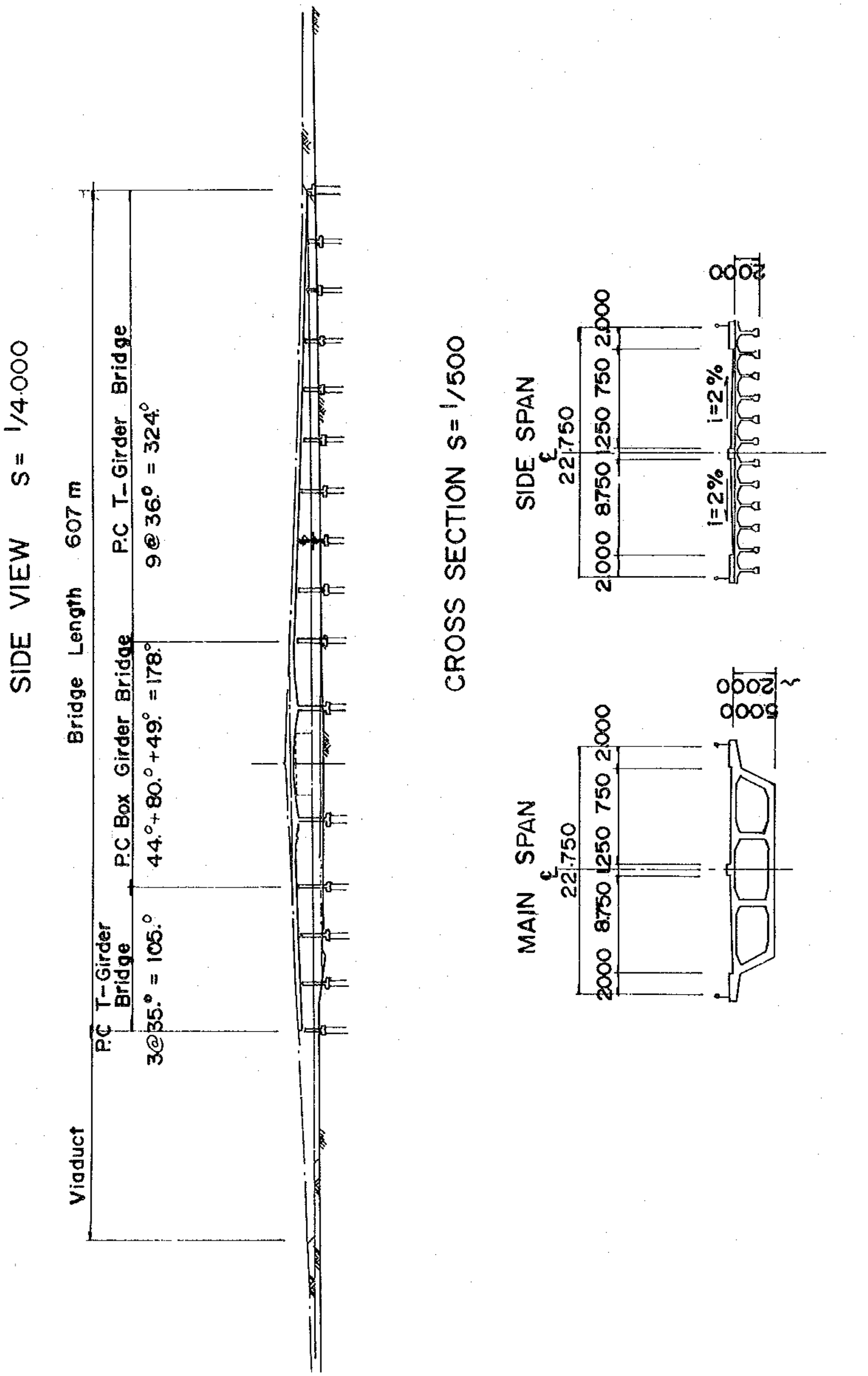
Sidewalk Load



Impact

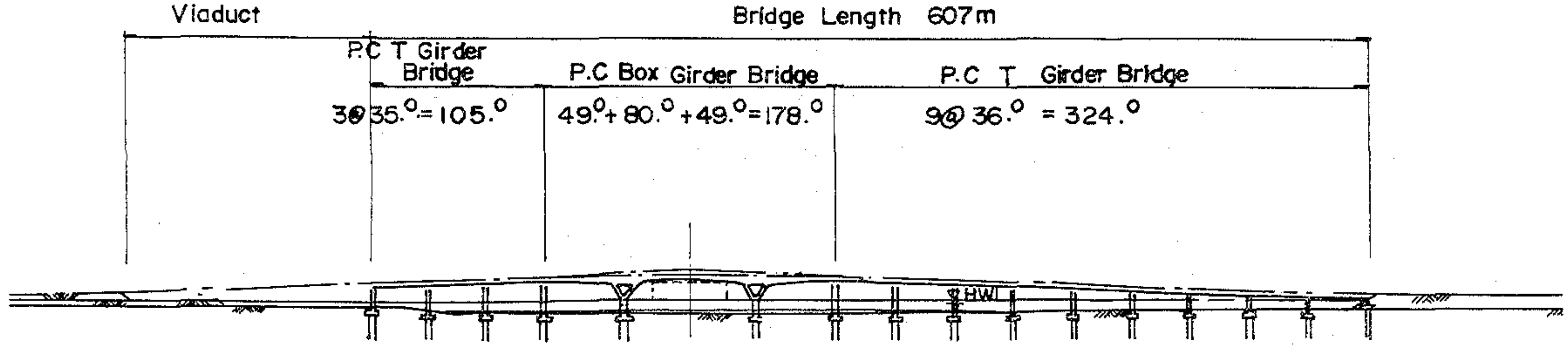


Steel Box Girder and Steel Plate Girder



P.C Box Girder (T-Type Pier) and P.C T-Girder

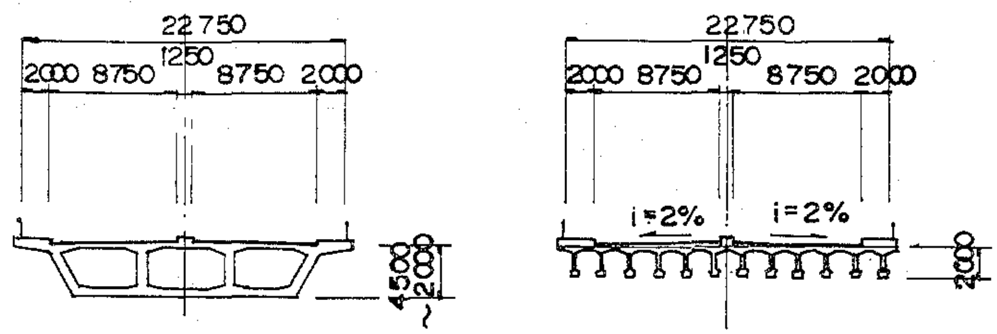
SIDE VIEW S=1/4.000



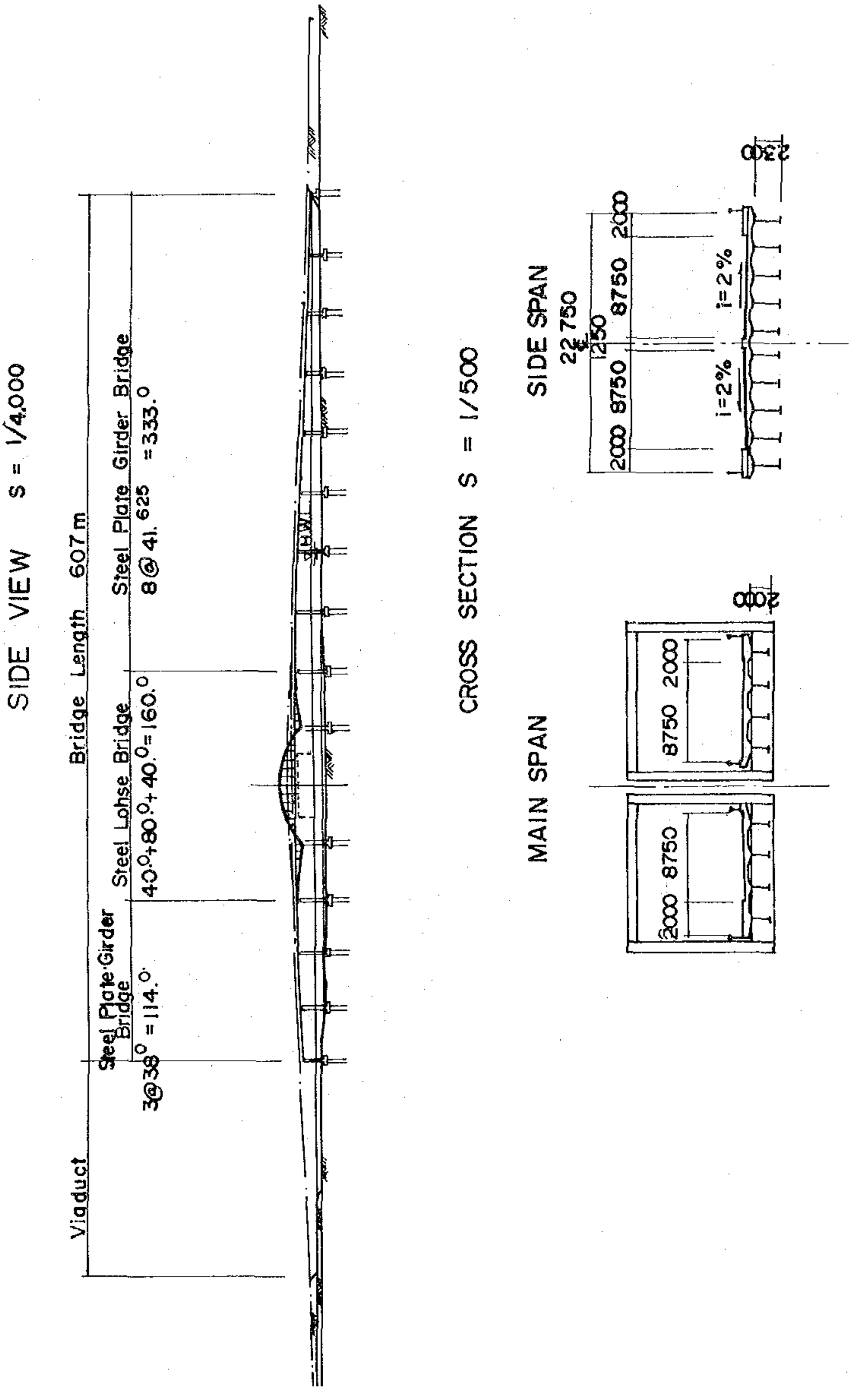
CROSS SECTION S=1/500

MAIN SPAN

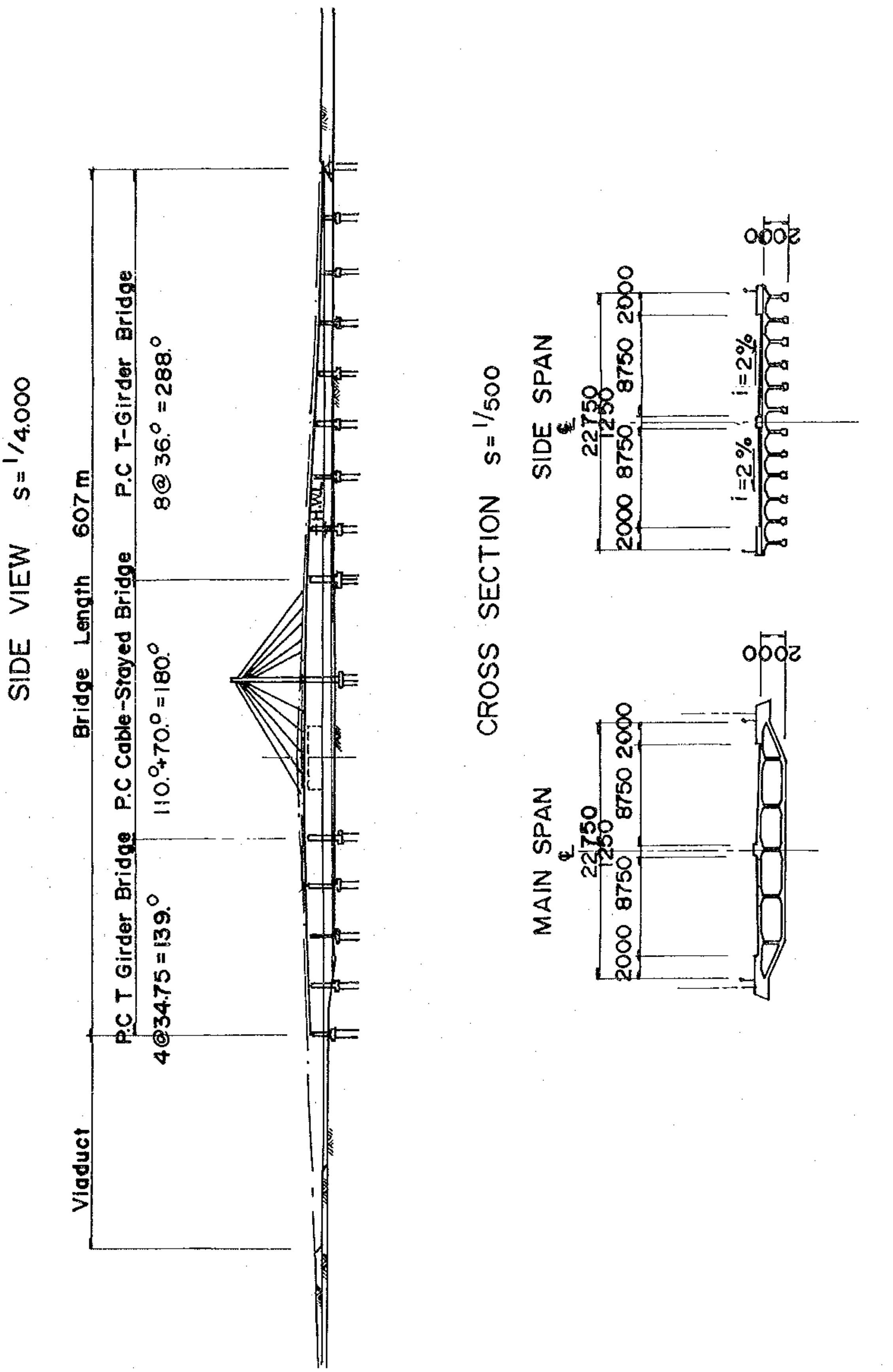
SIDE SPAN



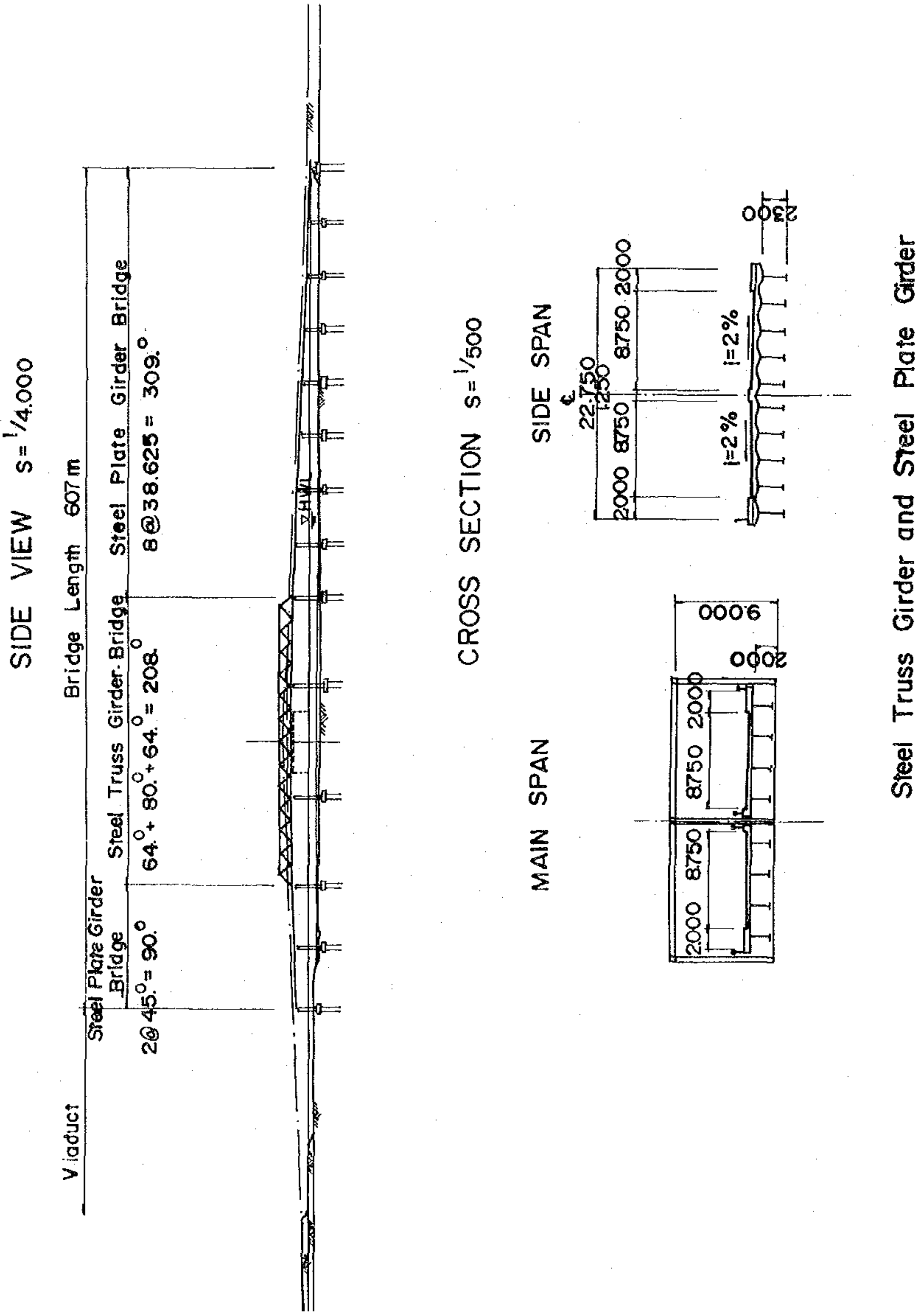
P.C Box Girder (V-Type Pier) and P.C T-Girder



Steel Lohse and Steel Plate Girder



P.C Cable-Stayed and P.C T-Girder


SIDE VIEW s= 1/4,000
Bridge Length 607 m
Viaduct
Steel Plate Girder Bridge
2@45.0= 90.0
Steel Truss Girder Bridge
64.0+ 80.0+ 64.0= 208.0
Steel Plate Girder Bridge
8@38.625 = 309.0
▽HWL
CROSS SECTION s= 1/500
MAIN SPAN
2000 8750 8750 2000
2000
9,000
SIDE SPAN
2000 8750 8750 2000
22.750
1250
i=2%
i=2%
2300
Steel Truss Girder and Steel Plate Girder

Structural Layout of PC Box Girder with V-leg Pier

1. Structural Layout of Box Girder

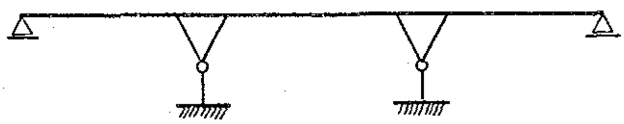
The structural layouts for the main span were examined based on the following conditions:

a) The bending moment according to variation of structural layouts is to be minimized
b) The displacement or restriction of the girder due to temperature fluctuation, drying shrinkage and creep is to be minimized.
c) The bridge surface is to be continuous in a direction of the bridge axis
d) The forces transmitted to the foundations is to be minimized
e) The structural layout is to be superior from the aesthetic viewpoint.

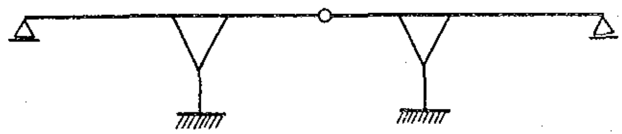
Variation of the structural layouts are shown as follows:

(1) Rigid Frame without Hinge

(2) Rigid Frame with Hinge at the V-Type Pier



(3) Rigid Frame with Center Hinge



Layouts of Rigid Frame for Main Span

(1) Rigid Frame without Hinge

The superstructure is a continuous girder type for the V-type pier and the bottoms of its pier are firmly fixed in the river bed. Therefore, the forces occur by dead and live loadings and fluctuation of temperature is relatively larger than other rigid frame types. Driving condition is also comfortable. The overall view is attractive by the features of the static indeterminate structural type. Lastly the lesser expansion joint is also an advantage of this type.

(2) Rigid Frame with Hinge at the V-Type Pier

The superstructure is the same as the above type (1), however, two hinges are provided below the V-type pier. The bottoms of the piers are also firmly fixed in the river bed. Since the upper portion of this structure is hinged at the bottom of V-legs, bending moment at the midspan of the superstructure is larger by about 20 % in comparison with the above type (1). Driving condition is also comfortable. The disadvantage is costly maintenance of bearing shoes and poorer aesthetics.

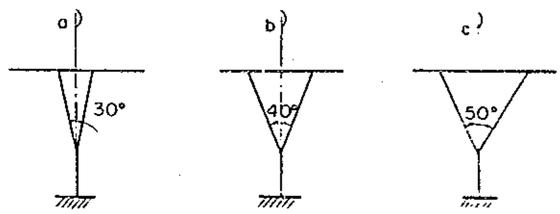
(3) Rigid Frame with Center Hinge

The superstructure is the rigid frame with a center hinge. The bottoms of the V-type piers are also firmly fixed in the river bed. The bending moment is comparatively greater than type (1) and (2) around the connected points between the superstructure and the top of the V-legs. The disadvantageous point is high maintenance cost for the center hinge. The edge of cantilevered girder must be carefully constructed because the defection frequently occurs at the center hinge as the Shambat Bridge.

Consequently, type (1) rigid frame without hinge is recommended. The reasons are less displacement, comfortable driving and less maintenance cost.

2 Variation of V-Legs

In this study three kinds of variation are examined for V-legs shown in the following figure. The wider the angle at the bottom of V-leg is, the greater the sectorial forces in the V-leg becomes when transmitted by the dead and live loadings. In addition, The wider V-legs not only reduce the girder depth but also improve the aesthetics. Regarding to the construction, it makes little difference during construction among these three types. Therefore, Type (b) is recommended.



Variation of V-Legs

Results of Structural Calculation of Prestressed Concrete Box Girder with V-leg Pier

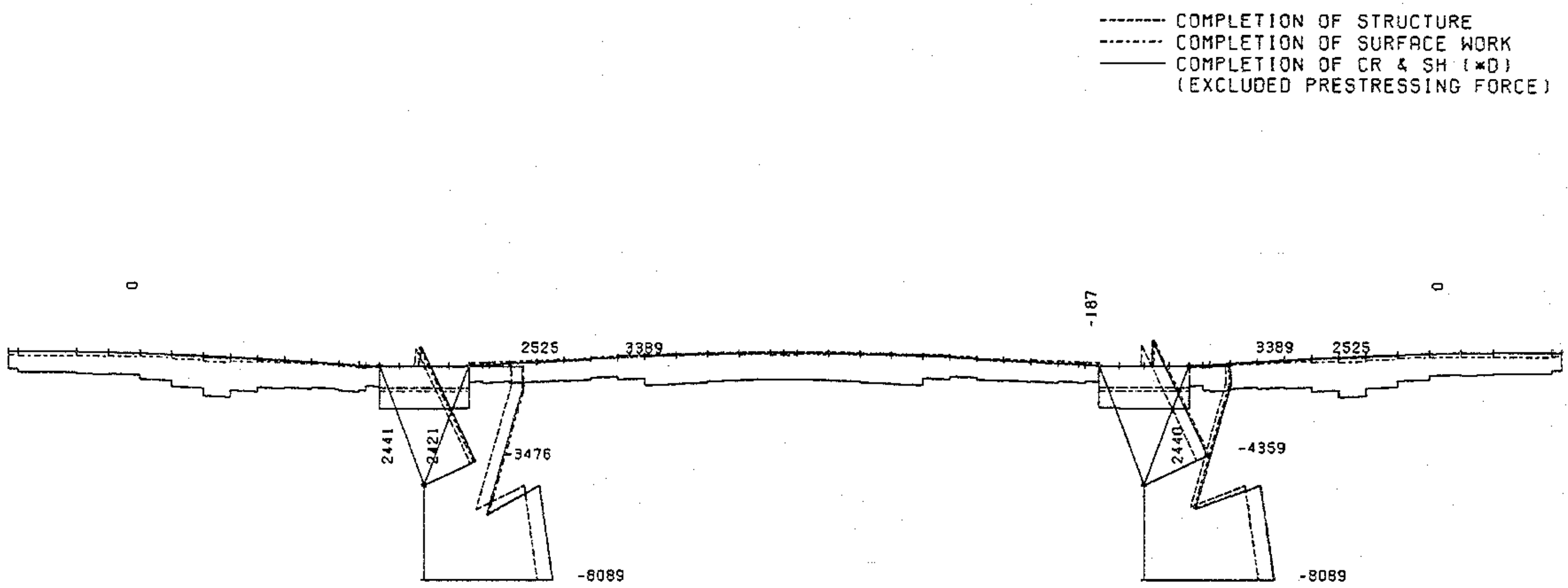
Calculation was conducted based on the design criteria as discussed in Main Report.

Results are shown in diagrams of axial forces, bending moments, shearing forces from next page.

These results attached hereinafter are prepared only for reference purposes for further steps of detailed design.

THE NEW WHITE NILE BRIDGE
NORMAL FORCE AFTER CONSTRUCTION

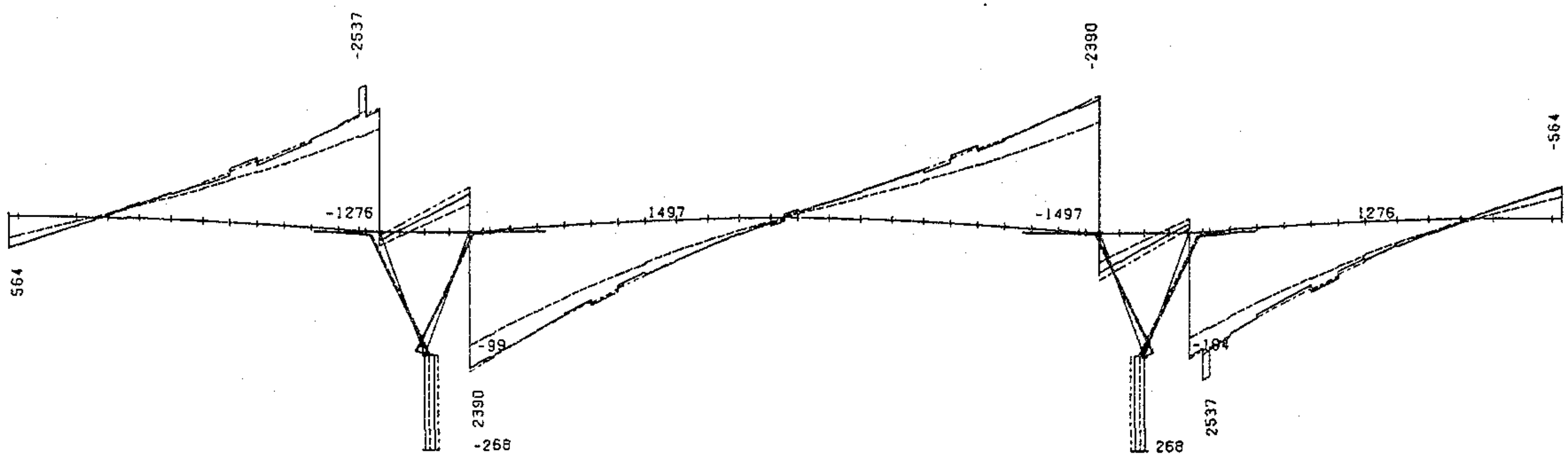
SCALE=1/ 716
—— 4044.5 TON



THE NEW WHITE NILE BRIDGE
SHEARING FORCE AFTER CONSTRUCTION

SCALE=1/ 716
—— 1268.8 TON

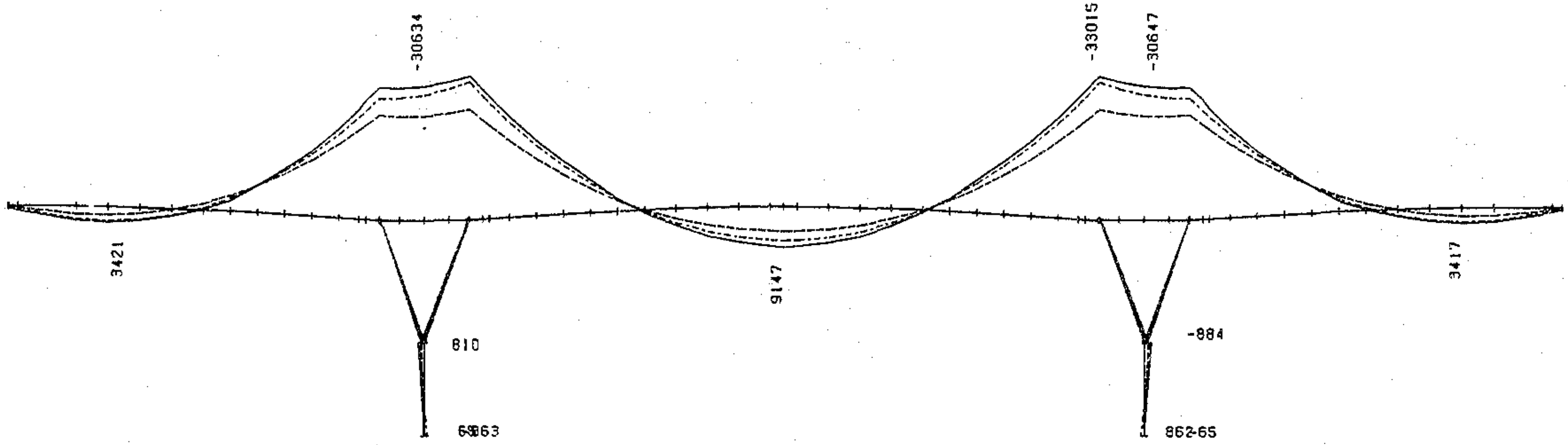
-------- COMPLETION OF STRUCTURE
-·-·-·-· COMPLETION OF SURFACE WORK
——— COMPLETION OF CR & SH (∗D)
(EXCLUDED PRESTRESSING FORCE)



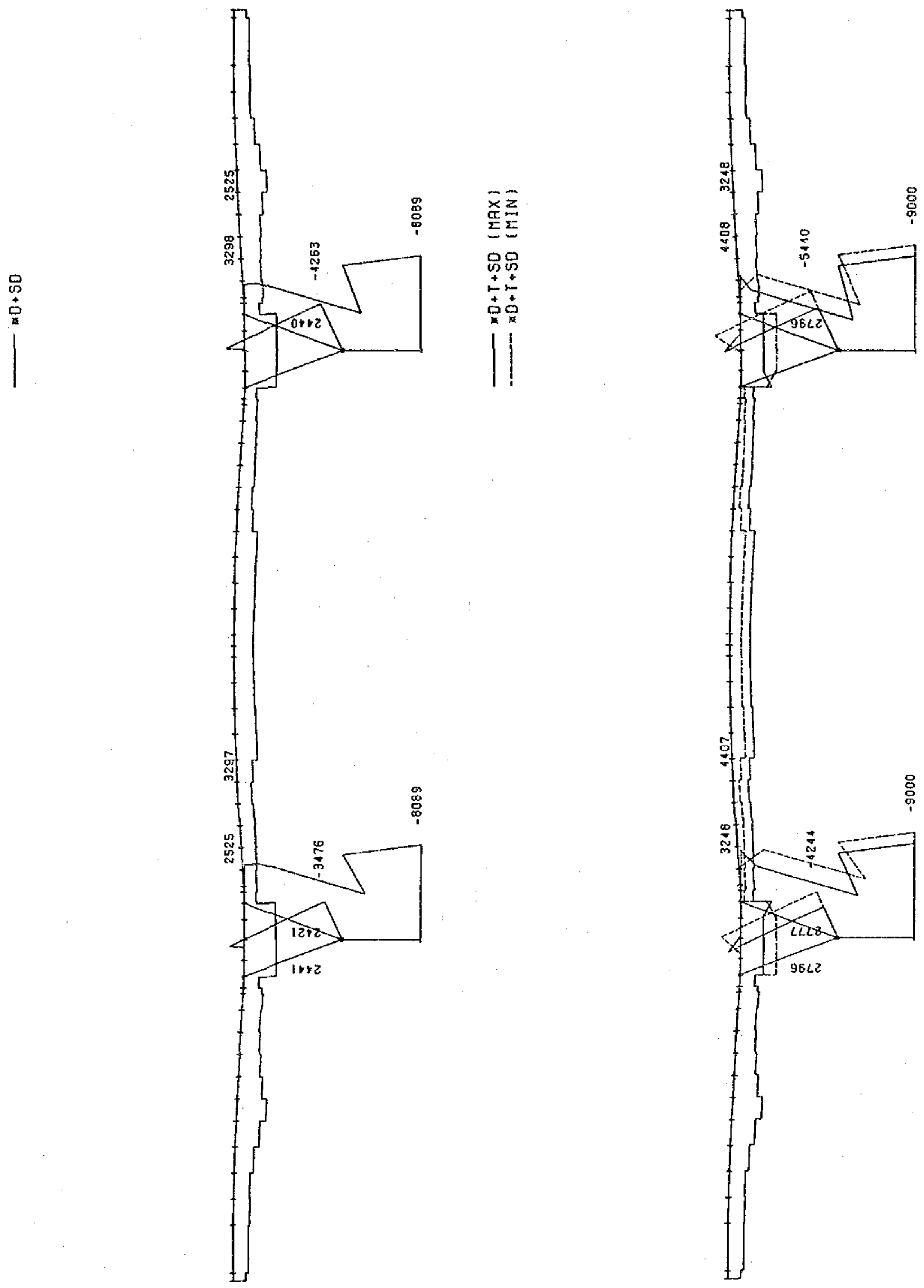
THE NEW WHITE NILE BRIDGE
BENDING MOMENT AFTER CONSTRUCTION

SCALE=1/ 716
—— 16507.8 TM

-------- COMPLETION OF STRUCTURE
-·-·-·-· COMPLETION OF SURFACE WORK
——— COMPLETION OF CR & SH (*D)
(EXCLUDED PRESTRESSING FORCE)

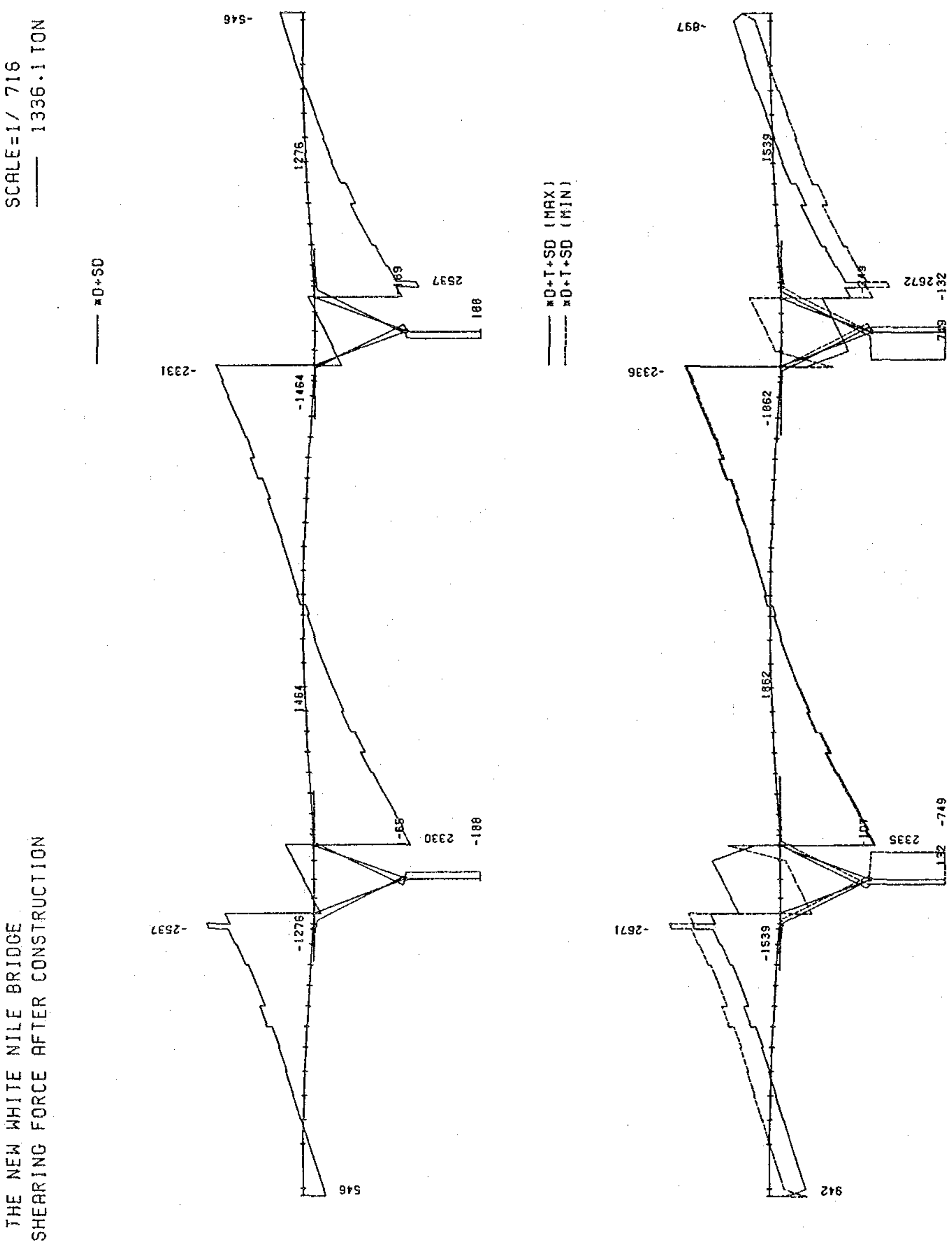


SCALE=1/ 716
4500.0 TON



THE NEW WHITE NILE BRIDGE
NORMAL FORCE AFTER CONSTRUCTION

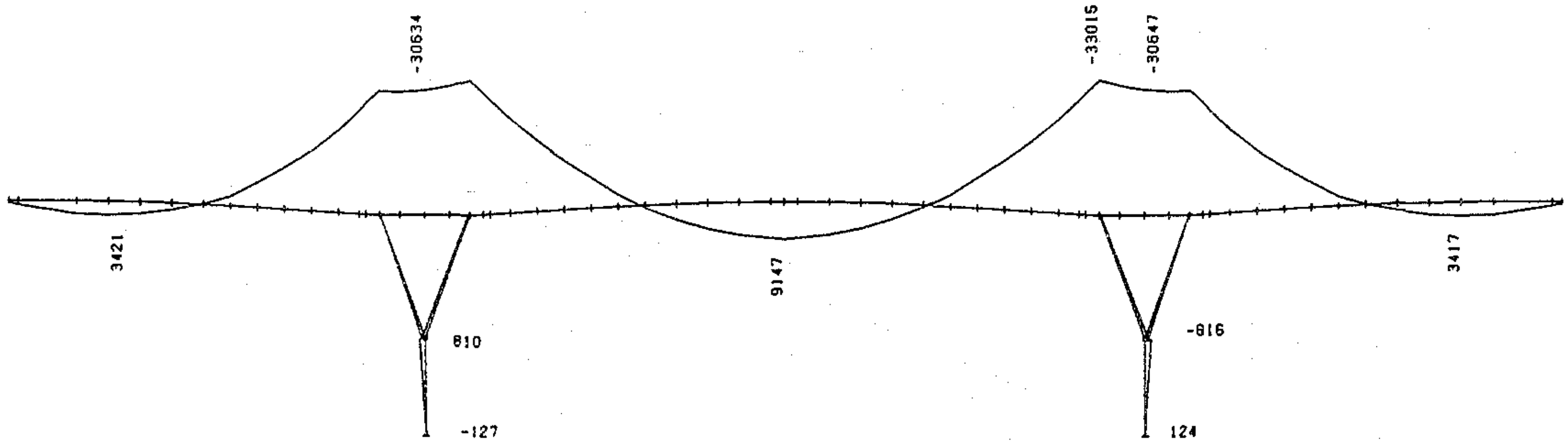
Appendix 7.6(6)



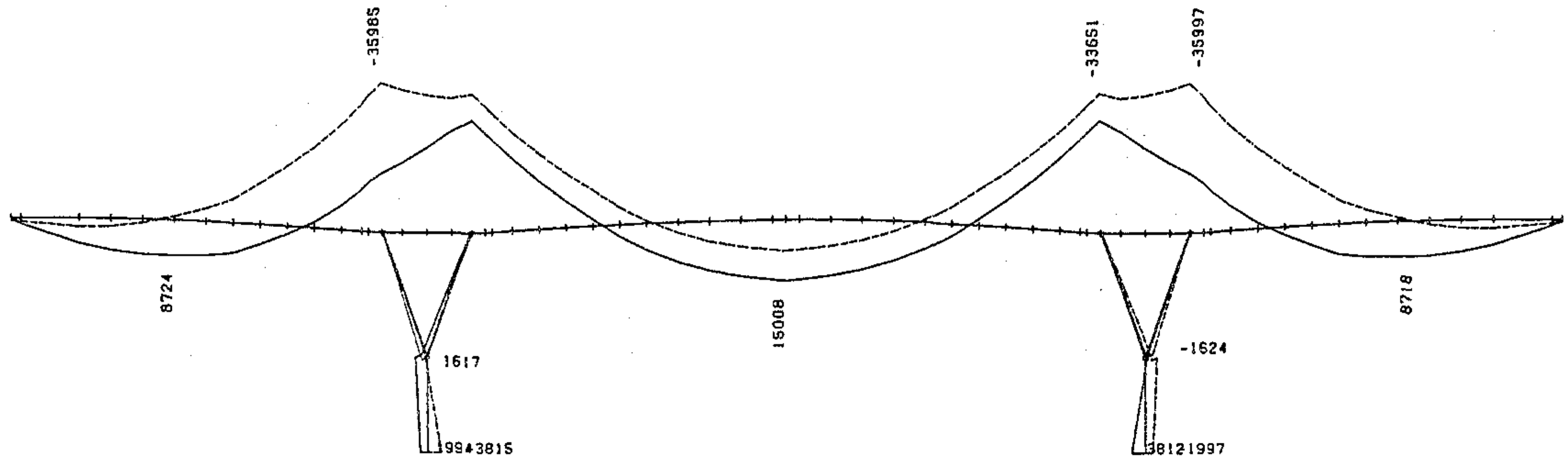
THE NEW WHITE NILE BRIDGE
BENDING MOMENT AFTER CONSTRUCTION

SCALE=1/ 716
—— 17998.9 TM

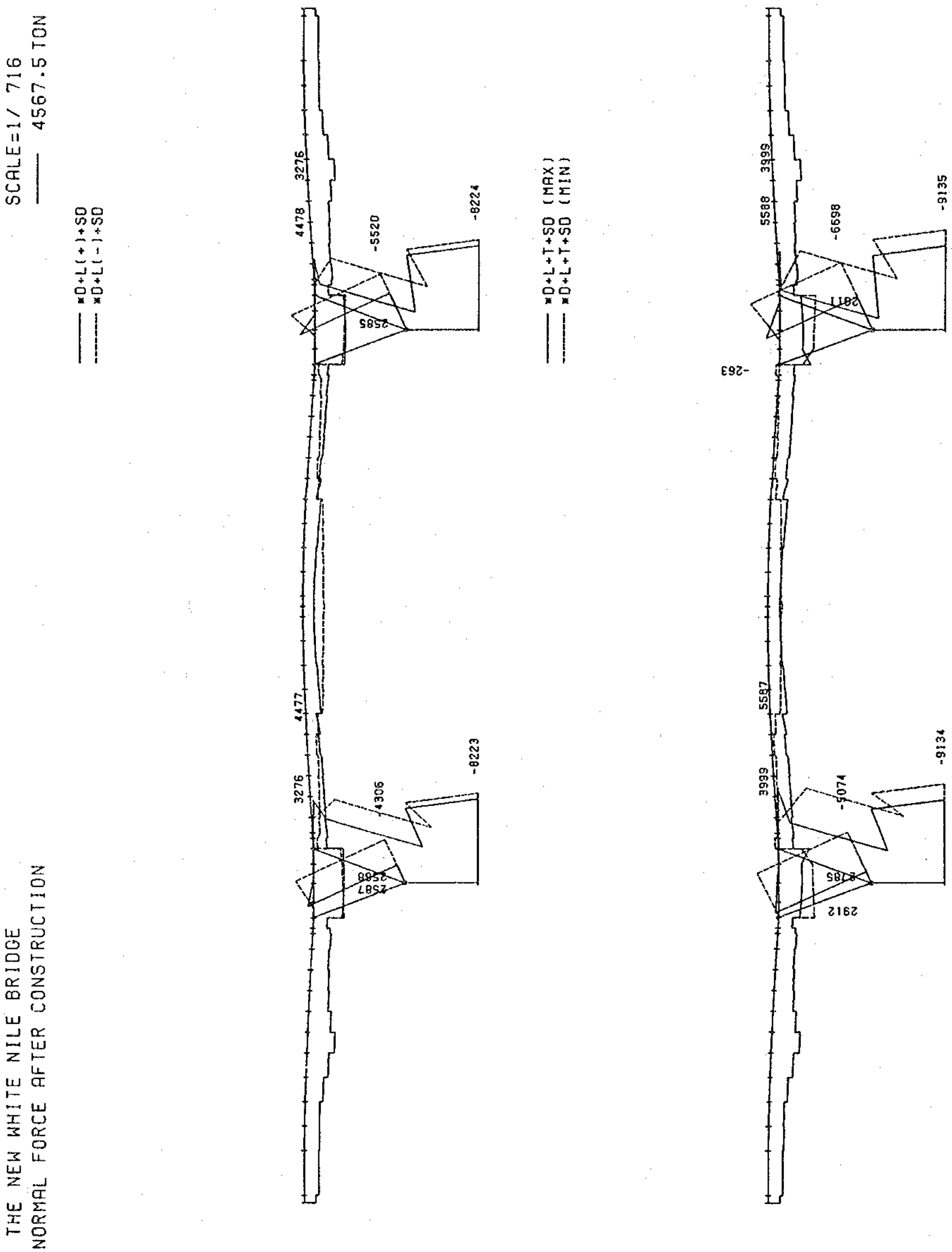
—— *D+SD



—— *D+T+SD (MAX)
- - - *D+T+SD (MIN)

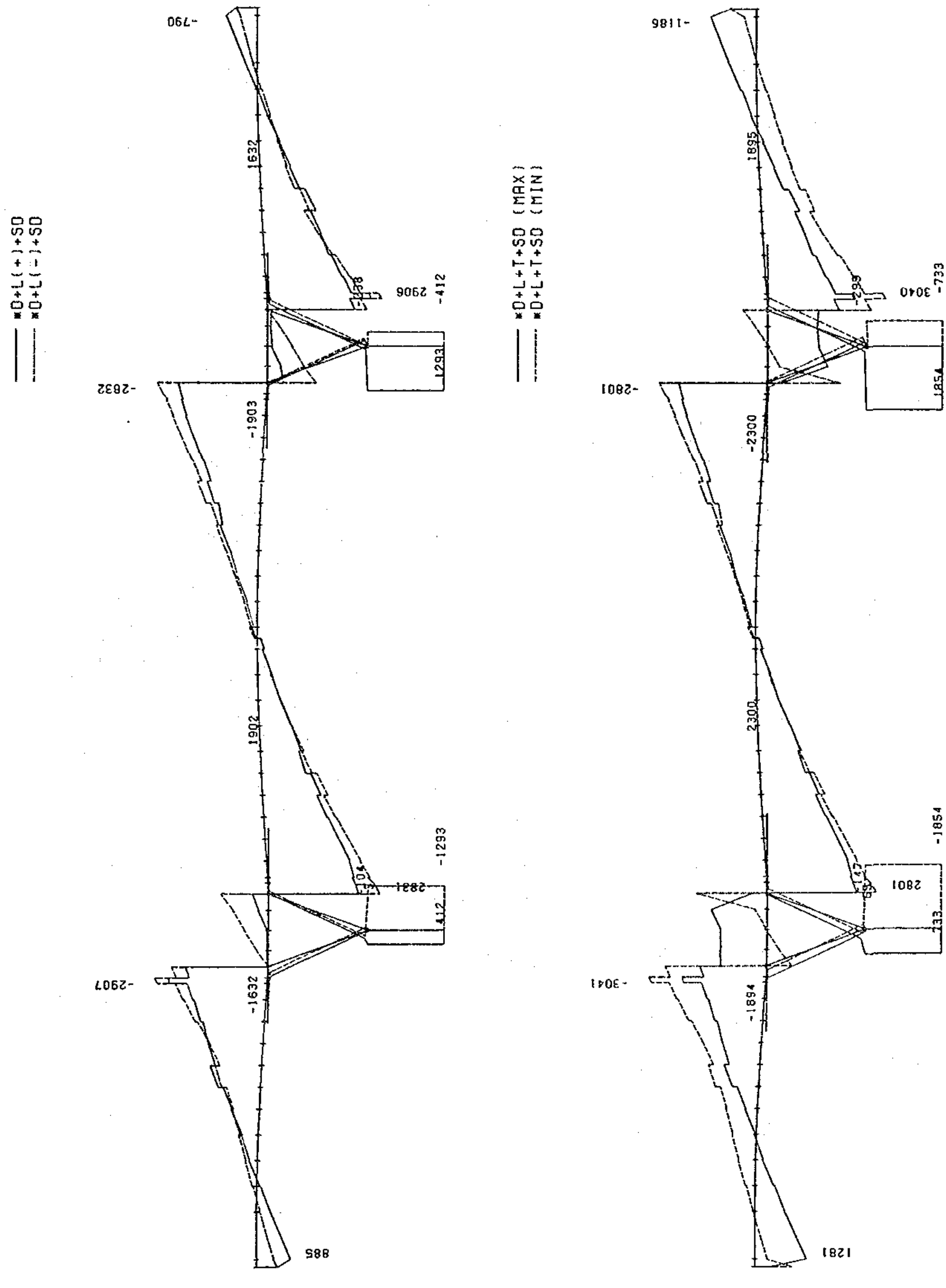


Appendix 7.6(8)

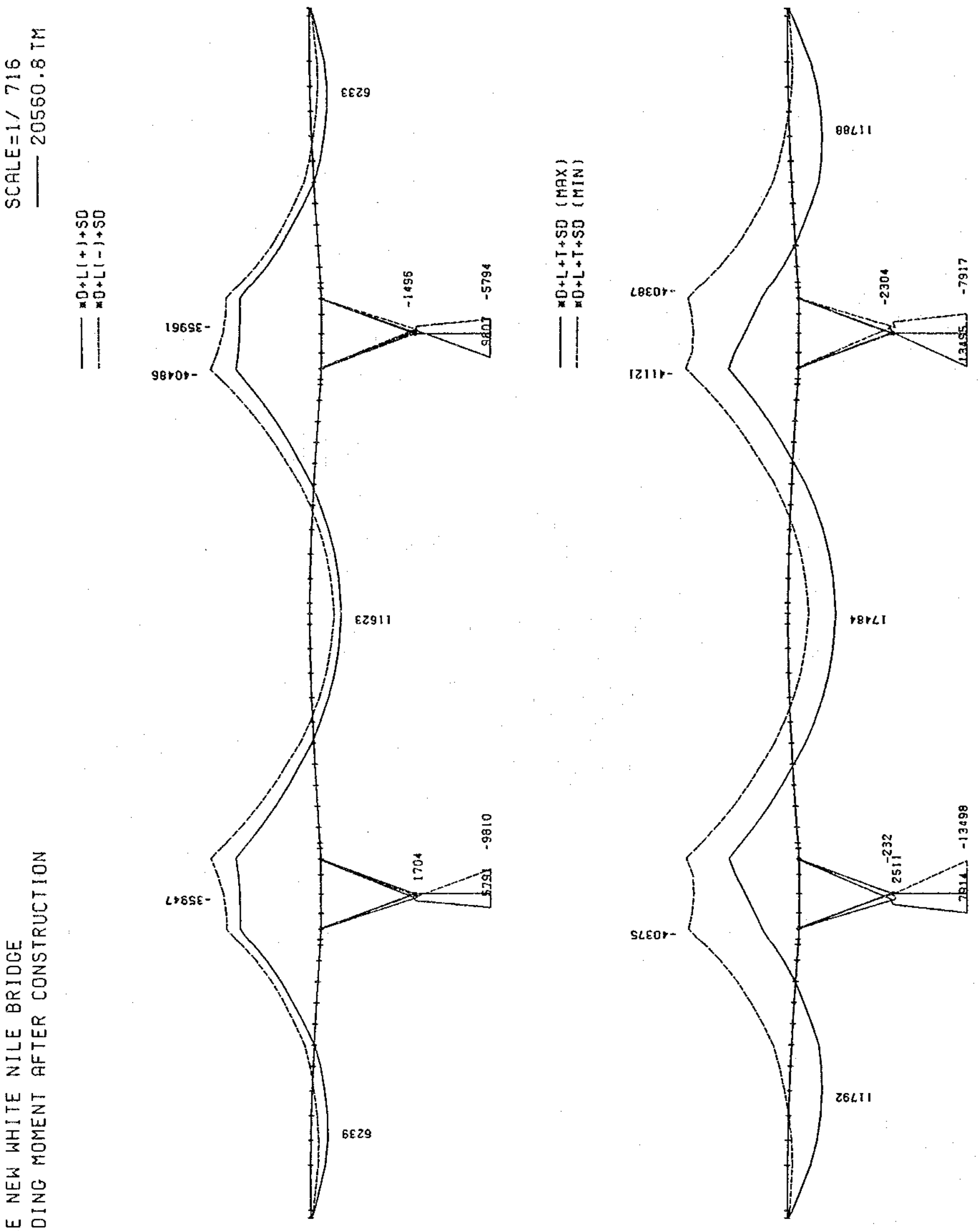


THE NEW WHITE NILE BRIDGE
SHEARING FORCE AFTER CONSTRUCTION

SCALE=1/ 716
—— 1520.9 TON

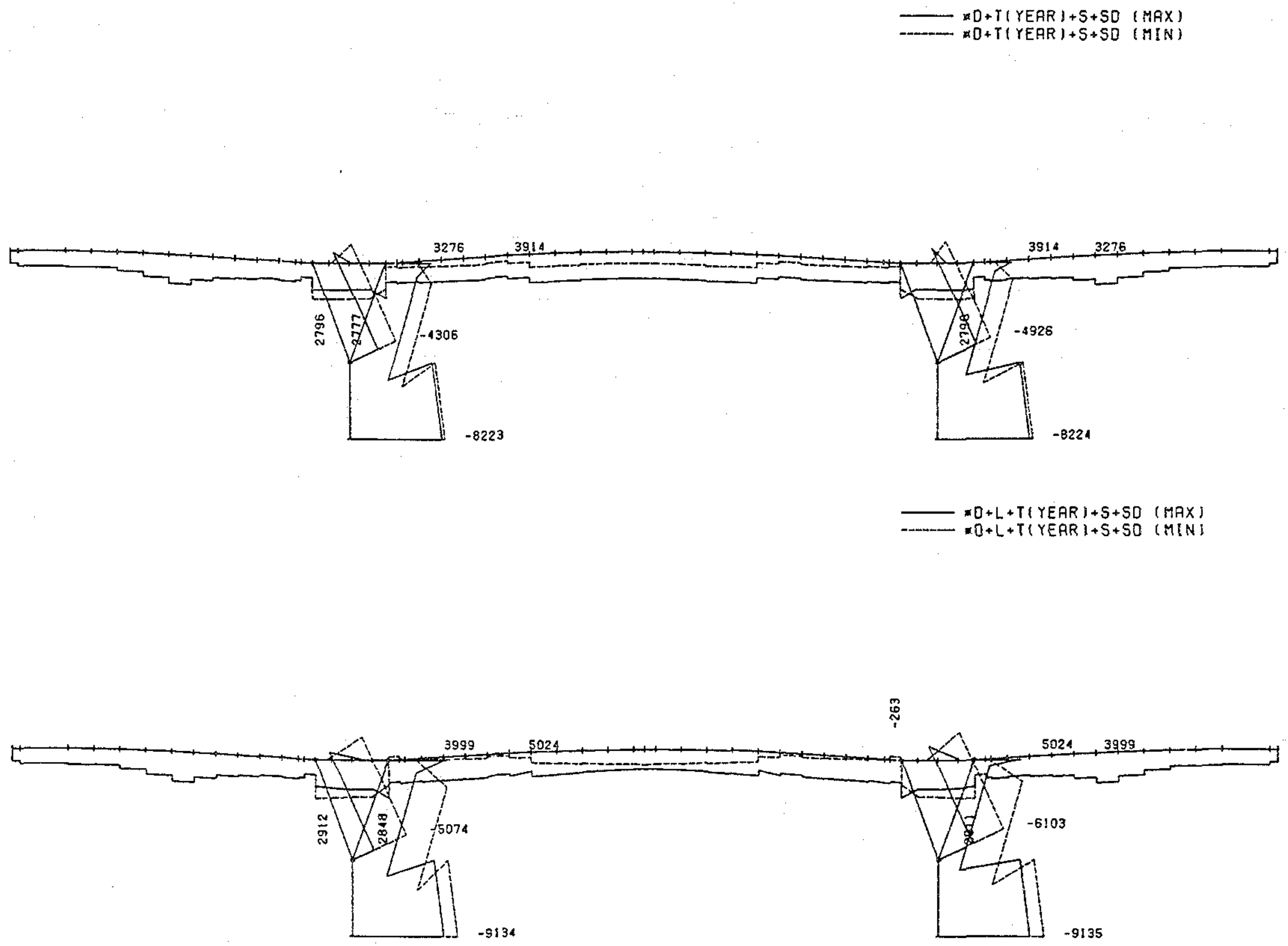


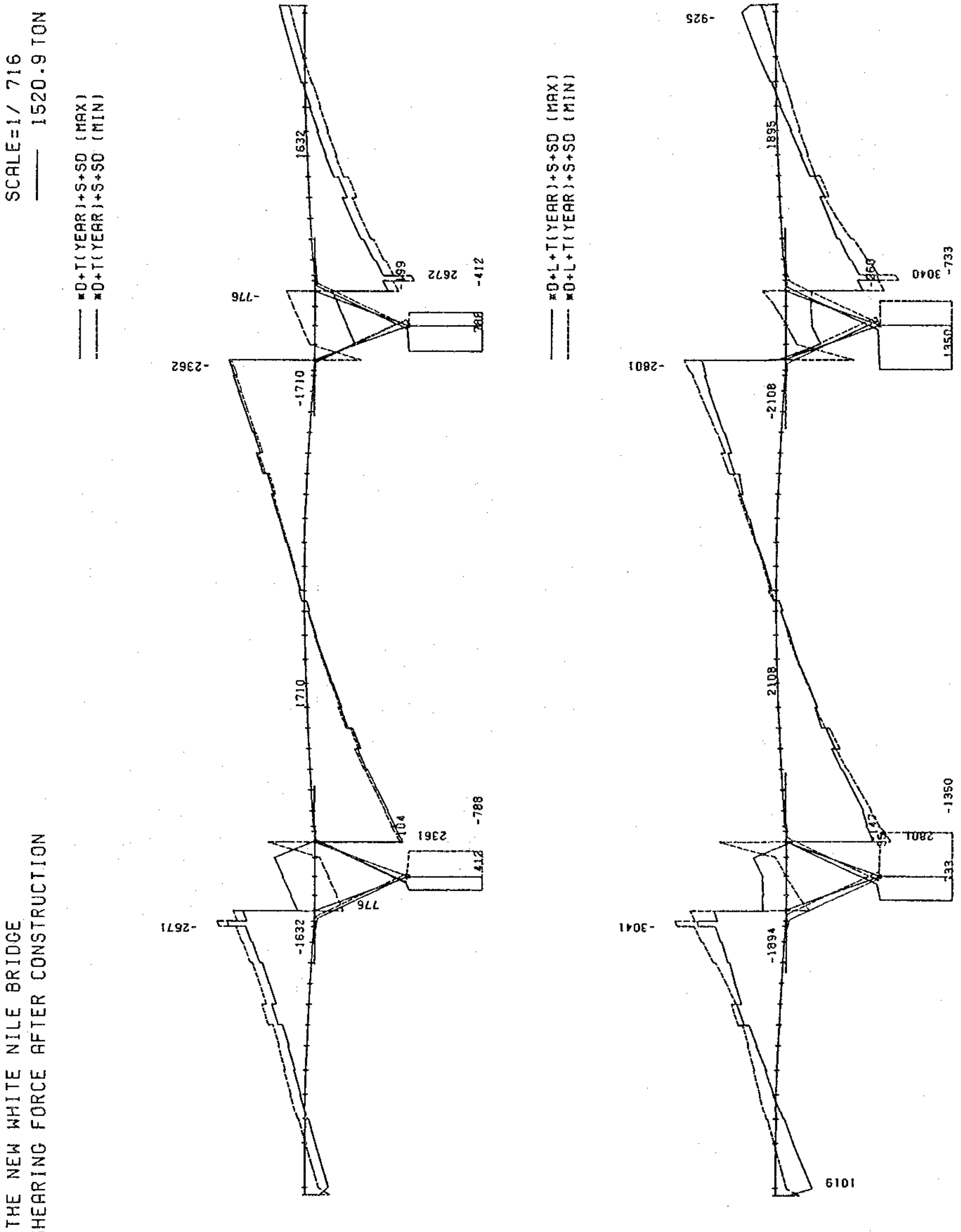
Appendix 7.6(10)



THE NEW WHITE NILE BRIDGE
NORMAL FORCE AFTER CONSTRUCTION

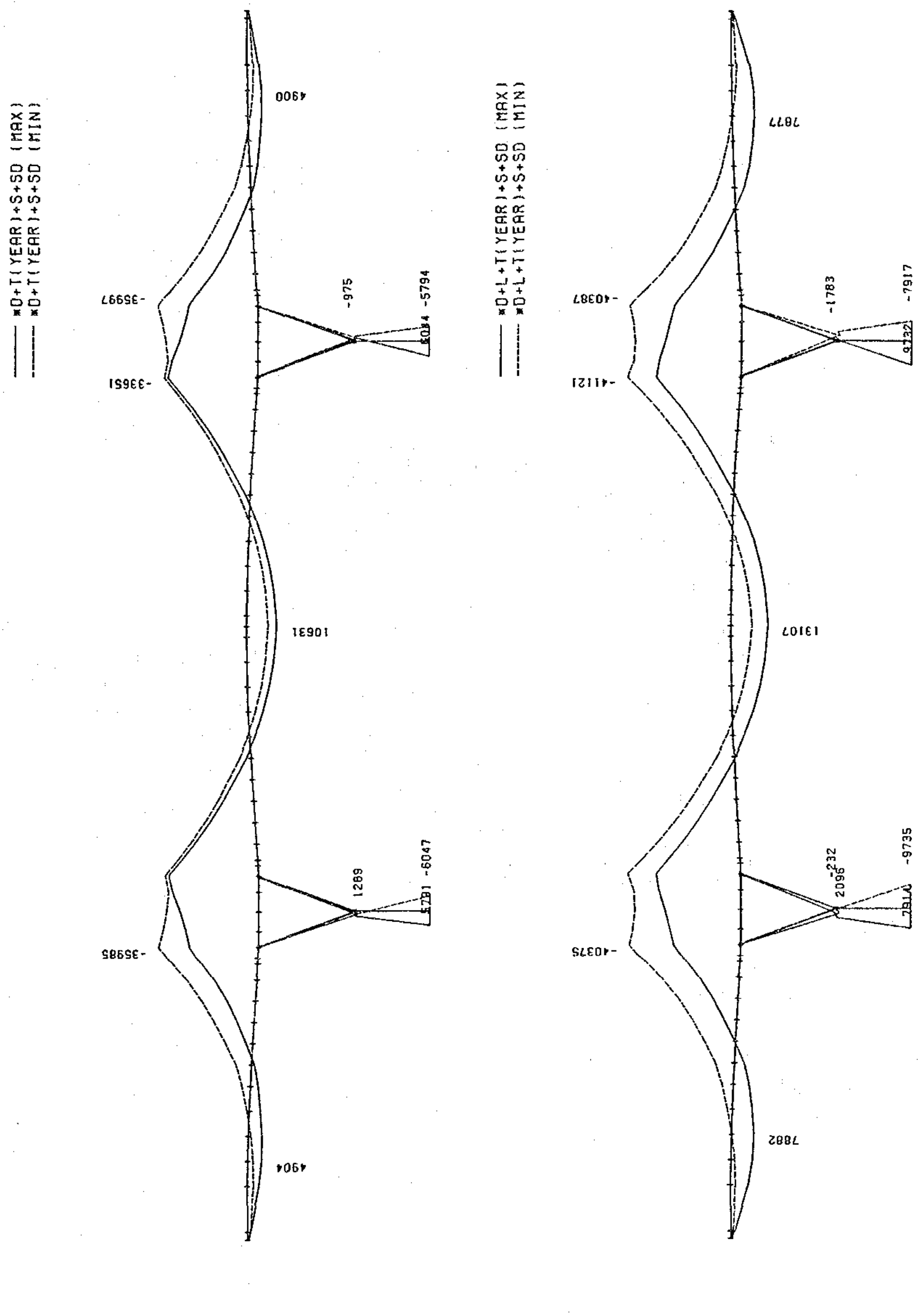
SCALE=1/ 716
—— 4567.5 TON




THE NEW WHITE NILE BRIDGE
SHEARING FORCE AFTER CONSTRUCTION
SCALE=1/ 716
1520.9 TON
*D+T(YEAR)+S+SD (MAX)
*D+T(YEAR)+S+SD (MIN)
*D+L+T(YEAR)+S+SD (MAX)
*D+L+T(YEAR)+S+SD (MIN)
-2671
-1632
776
2361
104
412
-788
1710
-2362
-1710
-776
2672
199
288
-412
1632
-3041
-1894
2801
143
133
-1350
2108
-2801
-2108
3040
260
1350
-733
1895
-925
1019

Appendix 7.6(13)

THE NEW WHITE NILE BRIDGE
BENDING MOMENT AFTER CONSTRUCTION



Structural Studies on PC Box Girder

1 Thickness of Sectional Components of Box Girder

(1) Top Slab

The top slab width is determined by the interval of supporting points of the deck slab. The width and thickness of the top slab of the box girder usually gives a sufficient area for the compressive and tensile sections of the positive and negative moments in the box girder. Therefore, the top slab thickness is determined by the minimum requirements of the size of the prestressing steel (tendons, sheaths) arrangement and their anchoring.

(2) Web

The web thickness normally makes almost no contribution to the rigidity of the box section. Therefore, the web thickness is determined by such requirements as space for the prestressing steel arrangement and its anchoring and the sectional area to withstand the bending and shearing force.

(3) Bottom Slab

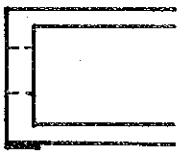
The bottom slab thickness is determined by such requirements as the required area for the compressive and tensile sections of the positive and negative moments in the box girder, the size of prestressing steel arrangement and its anchoring.

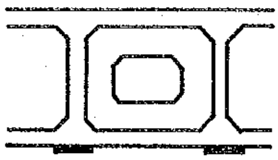
(2) Diaphragms

Diaphragms with special forming works in the box girder greatly influence the construction period and costs. As with the single box girder with sufficient rigidity, it is desirable to minimize its members. The types of diaphragm have many variations as described below:

- At the end point

  The diaphragm provided at the end point of the box girder should transmit the reaction from the superstructure to the bearings as shown in the following figure.

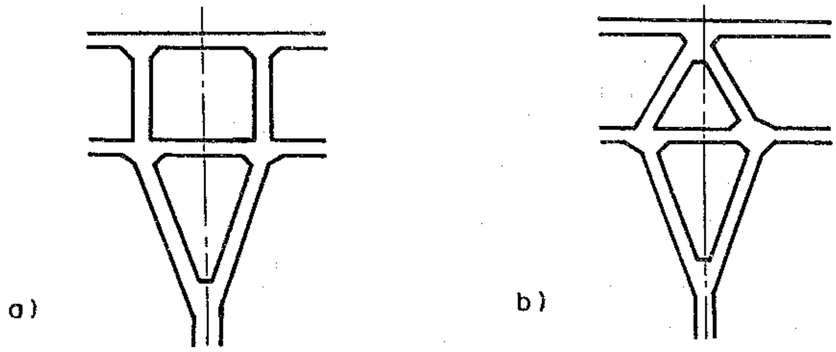




End Point Diaphragm

- At the intermediate support

  The diaphragm provided on the intermediate support should transmit the shear force smoothly to the substructure shown in the following figure.



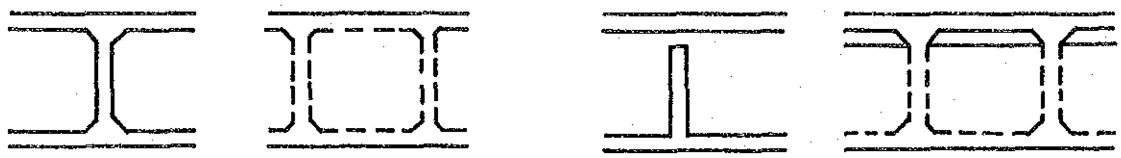
Intermediate Support Diaphragm

Theoretically, type (b) with two diagonal diaphragm would be effective in transmit the shear force, but type (a) is recommended from construction considerations.

- In a Span

  Since the diaphragm in a span is normally cast in place after completion of the box girder, Type (b) is more convenient than Type (a) as the forms can be removed to the outside of the girder. Type (a) and Type (b) are shown as follows:

Type (a) Type (b)



Diaphragm in a Span

- Open Space

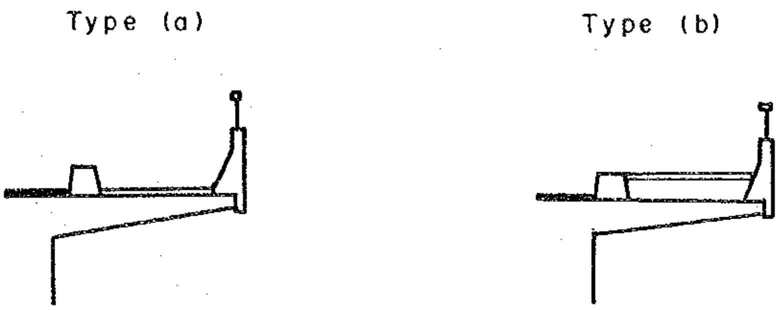
  The open space of a diaphragm is for the removal of forms from the completed box girder whether temporary or permanent space. To avoid damage from stress concentration, it is desirable to make the open space as small as possible.

- Arrangement of Diaphragm

  The diaphragm should be provided not only over the supports but within the spans. Where a diaphragm is provided in the span, it is effective to arrange it at the middle of the span. However, if the span is long, it is recommended that additional intermediate diaphragm be arranged at intervals of about 40 m. For single box girder and multi-box girder bridges, diaphragms should be provided over each support and within each span.

(3) Type of Side Walk

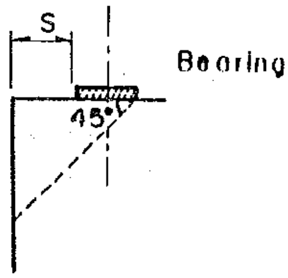
The width of the sidewalk depends on the number of persons crossing the proposed bridge on foot or by bicycle. Moreover, whether the sidewalk is to be provided on both sides or on one or other side of the bridge has to be determined by consideration of the bridge's location in relation to the urban center. From the viewpoint of the structure, the moment force due to the self-weight of the sidewalk should not have much influence on the main girder. In addition, the sidewalk plays a role in protecting the structure the vehicle collision. It is also desirable to be instalalation of utility appurtenances. Therefore Type (b) shown in Fig. 5.18 was adopted from the viewpoint of safety, maintenance, construction cost,and aesthetics.



Type of Sidewalk

(4) Bearing Seat

In the direction of the bridge axis, the bearing seat guarantees sufficient space for the bearings to be installed with the gaps (S) between the girders or the end of the girders and the parapet wall of the abutment. The bearing bed, which is transmitting the force from the superstructure to the substructure, should contribute to avoiding stress concentration. The following figure and the formulas below give guidelines to the width (S) from the edge of bearing to the front of substructure.



Bearing Seat

S = 20 + 0.5 x L ....in case of L<100 m

S = 30 + 0.4 x L ....in case of L>100 m

Where, L : Span length (m)

Structural Studies on PC-I Girder for Side Span

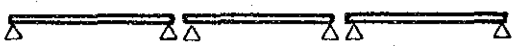
1 Structural Layout

The structural layouts for the side span are examined based on the following conditions:

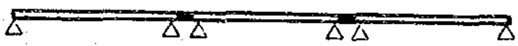
a) reduction of number of expansion joints due to a large number of queued piers with short span length
b) cost minimum of maintenance
c) easier construction

Three type variations for the layouts are depicted in the following figure.

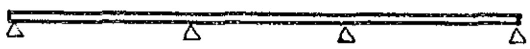
(1) Simple Span Girder



(2) Connecting Girder



(3) Continuous Girder



Layouts of I-Girder for Side Span

(1) Simple Span Girder

The simple span girders are placed on two bearings. Expansion joints connect the girders each other, which requires many expansion joints. Therefore, the simple span girder results in discomfort driving and also requires a lot of maintenance.

(2) Connecting Girder

Girders are placed on two bearings like the single girder, however, rebars connect adjacent girders and concrete fills the space between girders. The structure of the connecting girder is comparatively complex, but the number of expansion joints can be reduced.

(3) Continuous Girder

The continuous type girder has no joints and gives comfortable driving. However, this structure is complex and costly because the prestressing cables are provided in the upper portion of the girders at the pier.

In consequence, type (2) is recommended mainly from the viewpoints of reducing expansion joints and lowering the maintenance cost.

(2) Girder Depth

The girder depth of PC - I girder is usually computed by ratio depth and span length from 1/18 to 1/20 on condition that l is 20m to 40m. In this study, the designed side span length is to be 36m and the girder depth is to be 1.8m to 2.0m. However, 2.1m is adopted for the girder depth from the view point of HA and HB loadings and concrete strength, considering the availability at the bridge site.

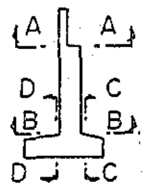
Results of Stress Culculation

Stresses in the specific sections shown in the following figure were culculated and summarizes from next page.

RESULTS OF STRESSES CALCULATION (1)

Substructure		A1 Abutment				A2 Abutment			
Section for Checking Stresses		A-A	B-B	C-C	D-D	A-A	B-B	C-C	D-D
Sectional Force	Bending Moment M (+m)	3.0	81.8	499.8	1,175.3	9.2	33.1	351.7	277.0
	Axial Force N (+)	-	44.7	-	-	-	50.4	-	-
	Shearing Force S (+)	4.0	25.6	2,236.9	828.9	6.3	14.0	1,743.4	178.0
Width of Section	B (cm)	100	100	2,775	2,775	100	100	2,775	2,775
Thickness of Section	H (")	50	150	150	150	50	150	150	150
Concrete Cover	d (")	40	140	135	140	40	140	135	140
Arrangement of Main R-bar	Dia. (mm) - Pitch (mm)	D16ctc250	D22ctc125	D16ctc125	D19ctc125	D19ctc125	D16ctc250	D16ctc250	D16ctc250
	Nos. of Row of R-bar	1	1	1	1	1	1	1	1
	Amount (cm2), As	7.94	30.97	436.92	633.17	22.92	7.94	7.94	7.94
(kg/cm2) Stresses	Compression of Concrete, Fc	18.9	41.2	12.3	23.4	38.5	19.7	11.7	8.7
	Tension of R-bar, Fs	1,028	1,404	898	1,420	1,135	462	1,244	936
	Shearing of Concrete, T	1.0	1.8	5.8	1.9	1.6	1.0	4.5	0.4
(kg/cm2) Allowable Stresses	Fca	80	-	80	-	80	-	80	-
	Fsa	1,800	-	1,600	-	1,800	-	1,600	-
	Ta	3.9	-	7.8	6.4	3.9	-	7.8	6.4

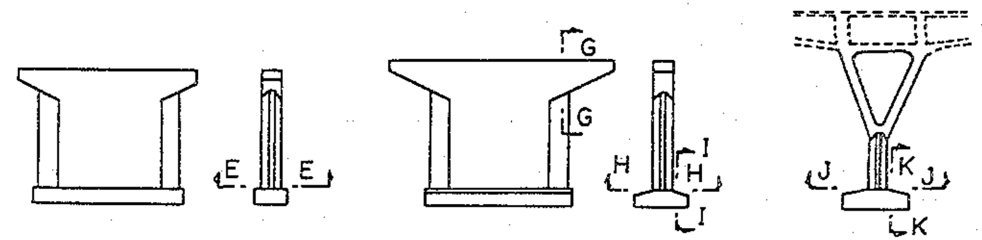
Note: 1. Position of Section



Abutment

RESULTS OF STRESSES CALCULATION (2)

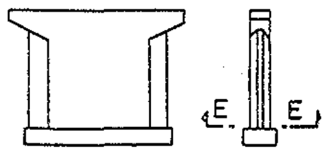
Substructure		P1 Pier	P9 Pier	P10 Pier			P13 Pier		
Section for Checking Stresses		E-E	E-E	G-G	H-H	I-I	G-G	H-H	I-I
Sectional Force	Bending Moment $M^{(t \cdot m)}$	250	453	793.5	491.6	625.6	1148.5	623.3	486.9
	Axial Force $N^{(t)}$	1,479	1,451	-	2,306.5	-	-	3,404.1	-
	Shearing Force $S^{(t)}$	18.3	30.6	202.8	32.6	1,223.2	214.0	30.6	1,811.9
Width of Section	$B^{(cm)}$	1,175	1,275	200	1,310	1,550	250	1,185	1,550
Thickness of Section	$H^{(")}$	160	120	389	200	150	424	250	150
Concrete Cover	$d^{(")}$	10	10	14	10	15	14	10	15
Arrangement of Main R-bar	Dia.$^{(mm)}$ - Pitch$^{(mm)}$	D16ctc250	D16ctc250	D32ctc125 D32ctc250	D16ctc250	D19ctc125	D32ctc125 D32ctc250	D16ctc250	D16ctc125
	Nos. of Row of R-bar	1	1	2	1	1	2	1	1
	Amount $^{(cm2)}$, As	91.36	99.30	182.67	105.26	352.40	230.52	95.33	244.28
(kg/cm2) Stresses	Compression of Concrete, Fc	-	25	25.9	14.2	22.3	25.9	16.3	13.1
	Tension of R-bar, Fs	-	88	1,271	57	1,358	1,329	103	1,133
	Shearing of Concrete, T	-	0.2	2.7	0.1	5.3	2.0	0.1	6.1
(kg/cm2) Allowable Stresses	Fca	-	100	80	100	80	80	100	80
	Fsa	-	2,250	1,800	2,250	1,600	1,800	2,250	1,600
	Ta	-	4.9	6.8	4.9	7.8	6.6	9.8	7.8



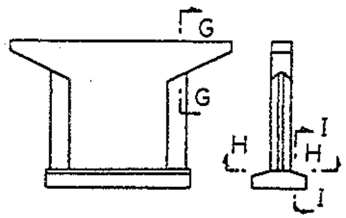
P1 ~ P9 Pier   P10~P13, P16 ~ P24 Pier   P14, P15 Pier

RESULTS OF STRESSES CALCULATION (3)

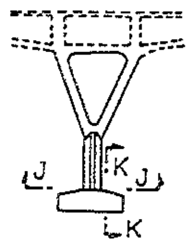
Substructure		P14 Pier		P17 Pier		
Section for Checking Stresses		K-K	L-L	G-G	H-H	I-I
Sectional Force	Bending Moment M (+m)	9,958.0	801.33	1,207.3	937.5	848.3
	Axial Force N (+)	8,430.0	-	-	3,131.2	-
	Shearing Force S (+)	1,287.0	-	356.0	50.0	1,628.1
Width of Section	B (cm)	1,455	100	200	1,310	1,550
Thickness of Section	H (")	250	300	389	200	150
Concrete Cover	d (")	15	15	15	10	15
Arrangement of Main R-bar	Dia. (mm) - Pitch (mm)	D32ctc125 D32ctc250	D38ctc125 D38ctc125	D32ctc125 D32ctc125	D16ctc250	D22ctc125
	Nos. of Row of R-bar	2	2	2	1	1
	Amount (cm2), As	1,373.97	182.40	238.26	105.26	476.13
(kg/cm2) Stresses	Compression of Concrete, Fc	86.2	67.3	35.7	22.2	28.4
	Tension of R-bar, Fs	1,145	1,814	1,513	36	1,429
	Shearing of Concrete, T	3.8	-	4.8	0.2	7.3
(kg/cm2) Allowable Stresses	Fca	92	92	80	100	80
	Fsa	2,070	2,070	1,800	2,250	1,600
	Ta	4.5	-	6.8	7.6	7.8



P1 ~ P9 Pier



P10~P13, P16 ~ P24 Pier



P14, P15 Pier

Appendix 7.9(4)

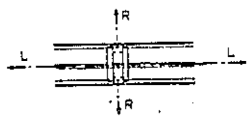
Stability Calculation of Pile Foundation

Stability calculation of piles are shown on the subsequent pages along with induced stresses in pile sections.

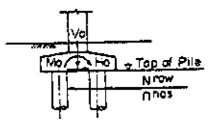
Notation of the summary table are as indicated below:

RESULTS OF STABILITY CALCULATION OF PILE FOUNDATION (1)

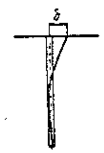
Substructure		A1 Abutment		A2 Abutment		P7 Pier		P9 Pier		P10 Pier		
Direction of Examination		L-L		L-L		L-L		L-L		L-L		R-R
Examination Case		N	B	N	B	B	B	N	B	N	B	N
Acting Force at Top of Pile	Horizontal Force Ho (t)	602	821	483	508	0	18	0	31	0	33	26
	Moment Mo (tm)	1,053	1,223	572	711	0	278	0	499	35	541	75
	Vertical Force Vo (t)	3,746	3,746	3,407	3,407	1,722	1,722	1,703	"	2,727	"	2,736
Nos. of Pile Row	N	2	"	2	"	1	"	1	"	2	"	4
Nos. of Pile	n	16	"	12	"	5	"	5	"	8	"	"
Horizontal Deflection of Pile Top	d (cm)	1.2	1.3	1.0	1.1	0	0.4	0	0.8	0.0	0.2	0.0
	Allowable da	1.5	"	1.5	"	1.5	"	1.5	"	1.5	"	"
Reaction of Pile	R max. (t)	330	337	358	366	334	"	332	"	343	375	347
	R min. (")	138	132	210	202	-	-	-	-	339	307	337
	Allowable Ra	414	517	414	517	414	517	414	517	414	517	414
Bending Moment of Pile	Mt (tm)	83	82	68	68	-	56	-	100	1.5	15	9.2
	Mm (")	106	108	78	82	-	57	-	103	"	20	7.0
	lm (m)	5.4	5.4	4.7	4.7	-	1.1	-	1.0	0	2.6	5.2
Main Rebar of Pile	Dia (mm) - pitch (mm)	D32-141	"	D22-177	"	D22-283	"	D22-236	"	D22-283	"	"
	Nos. of Row of Rebar	1.5	"	2	"	1	"	1	"	1	"	"
	Amount As (cm2)	238.3	"	61.94	"	38.71	"	46.45	"	38.71	"	"
Stress of Pile	Compression of Concrete Fc (kg/cm2)	79	81	72	74	-	63	-	100	-	42	-
	Tension of Rebar Fs (")	878	898	914	948	-	781	-	1,170	-	595	-
	Shearing of Concrete T (")	6	6	5	5	-	1	-	2	-	0.6	-
Allowable Stress of Pile	Fca (kg/cm2)	80	100	80	100	-	100	-	100	-	100	-
	Fsa (")	1,600	2,250	1,600	2,250	-	2,250	-	2,250	-	2,250	-
	Ta (")	17	21	17	21	-	7.4	-	7.4	-	7.4	-



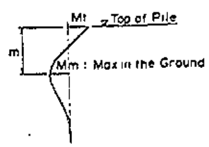
Direction of Examination



Acting Force



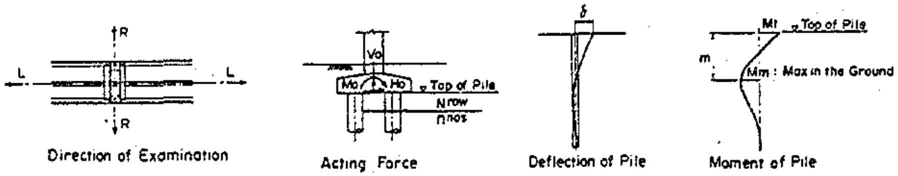
Deflection of Pile



Moment of Pile

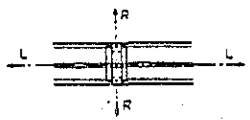
RESULTS OF STABILITY CALCULATION OF PILE FOUNDATION (2)

Substructure		P13 Pier				P14 Pier			P17 Pier		
Direction of Examination		L-L			R-R	L-L			L-L		R-R
Examination Case		N	B	T	N	N	T	C	N	B	N
Acting Force at Top of Pile	Horizontal Force Ho (t)	0	31	0	138	693	1,287	824	0	50	55
	Moment Mo (tm)	22	685	22	442	5,473	12,532	6,177	0	1,013	149
	Vertical Force Vo (t)	3,926	"	"	3,953	11,186	11,053	11,186	3,552	"	3,563
Nos. of Pile Row	N	2	"	"	6	5	"	"	2	"	5
Nos. of Pile	n	12	"	"	"	35	"	"	10	"	"
Horizontal Deflection of Pile Top	d (cm)	0.0	0.2	0.0	0.1	0.3	0.5	0.3	0.0	0.4	0.1
	Allowable da	1.5	"	"	"	1.5	"	2.5	1.5	"	"
Reaction of Pile	R max. (t)	328	355	328	352	382	452	391	355	409	366
	R min. (")	326	299	326	307	257	180	248	-	302	346
	Allowable Ra	414	517	476	414	414	476	621	414	517	414
Bending Moment of Pile	Mt (tm)	0.6	13	0.6	32	33	55	41	-	19	18
	Mm (")	"	19	"	33	45	83	53	-	33	19
	lm (m)	0	3.0	0	4.5	4.3	"	"	-	4.1	5.1
Main Rebar of Pile	Dia (mm) - pitch (mm)	D22-283	"	"	"	D22-141	"	"	D22-283	"	"
	Nos. of Row of Rebar	1	"	"	"	1	"	"	1	"	"
	Amount As (cm2)	38.71	"	"	"	77.42	"	"	38.71	"	"
Stress of Pile	Compression of Concrete Fc (kg/cm2)	-	-	-	48	-	80	-	-	53	-
	Tension of Rebar Fs (")	-	-	-	653	-	1,037	-	-	721	-
	Shearing of Concrete T (")	-	-	-	1.4	-	4.3	-	-	0.6	-
Allowable Stress of Pile	Fca (kg/cm2)	-	-	-	80	-	92	-	-	100	-
	Fsa (")	-	-	-	1,600	-	2,070	-	-	2,250	-
	Ta (")	-	-	-	3.9	-	4.5	-	-	4.9	-

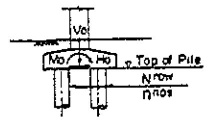


RESULTS OF STABILITY CALCULATION OF PILE FOUNDATION (3)

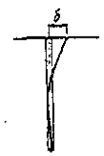
Substructure			P24 Pier		
Direction of Examination			L-L		R-R
Examination Case			N	B	N
Acting Force at Top of Pile	Horizontal Force	Ho (t)	0	50	55
	Moment	Mo (tm)	0	463	149
	Vertical Force	Vo (t)	2,831	"	2,842
Nos. of Pile Row	N		2	"	4
Nos. of Pile	n		8	"	"
Horizontal Deflection of Pile Top	d (cm)		0	0.3	0.1
	Allowable da		1.5	"	"
Reaction of Pile	R max. (t)		354	387	366
	R min. (")		-	321	345
	Allowable Ra		414	517	414
Bending Moment of Pile	Mt (tm)		-	7	20
	Mm (")		-	18	15
	lm (m)		-	4.2	5.2
Main Rebar of Pile	Dia (mm) - pitch (mm)		D22-283	"	"
	Nos. of Row of Rebar		1	"	"
	Amount	As (cm2)	38.71	"	"
Stress of Pile	Compression of Concrete	Fc (kg/cm2)	-	42	-
	Tension of Rebar	Fs (")	-	599	-
	Shearing of Concrete	T (")	-	0.7	-
Allowable Stress of Pile	Fca (kg/cm2)		-	100	-
	Fsa (")		-	2,250	-
	Ta (")		-	4.9	-



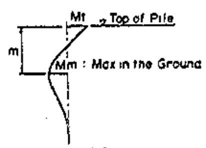
Direction of Examination



Acting Force



Deflection of Pile



Moment of Pile

Preliminary Road Engineering

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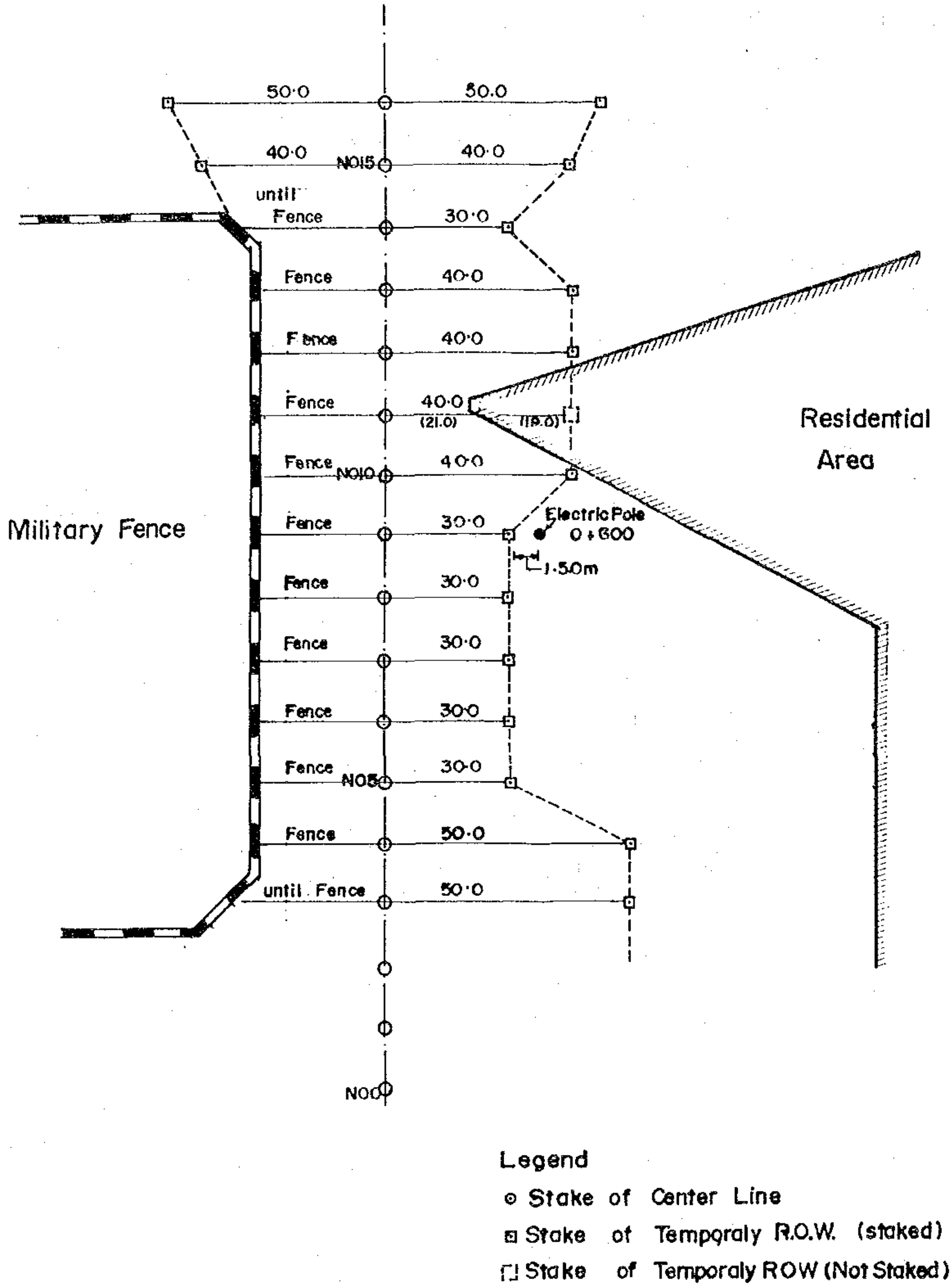

Location of Temporary Right of Way