Indian Road Congress

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The Indian Roads Congress as a body does not hold itself responsible for statements made, or for opinions expressed, in the Papers or discussions in its Proceedings.

NOTICES AND ANNOUNCEMENTS

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I. REVISED RATES OF ENTRANCE FEE AND SUBSCRIPTION

The rates of entrance fee and annual subscription for Ordinary Members have been enhanced by the Council of the Indian Roads Congress as given below. This has been done most reluctantly, to meet the rising expenditure owing to the increased activities of the society.

Entrance Fee: An entrance fee shall be payable by Members on election on the following scale:—

	Rs.
Patron, Vice-Patrons and Honorary Members	Nil
Ordinary Members ...	25
Associate Members ...	120

Subscription: A yearly subscription shall be payable by Members on the following scale:—

	Rs.
Patron, Vice-Patrons and Honorary Members	Nil
Ordinary Members ...	25
Associate Members ...	120

Provided that a rebate of Rs. 5/- in case of Ordinary Members and Rs. 20/- in the case of Associate Members may be claimed if the subscriptions are paid within three months of the due date.

Note:—Part 2 of Volume XIV of the Journal of the Indian Roads Congress will be published shortly and, it may be mentioned that, it will be issued free of cost only to those Members (Ordinary and Associate) who have paid their arrears and subscription for 1949-50.

2. Members are requested to intimate change of address, if any, to the Secretary, Indian Roads Congress, so tha tcorrespondence, literature, publications etc., may be sent to their correct addresses.

3. Members who remit the amount by Money Orders are requested to give their full address with Roll No. and name in block letters in the Money Order Coupon to enable the I. R. C. Office to credit the amount to the correct account.

II. NEW ADMISSIONS

LIST OF THE MEMBERS OF THE INDIAN ROADS CONGRESS ENROLLED FROM 8-5-49 to 31-10-49.

Sr. No.	*Roll No.*	*Name.*	*Address.*
1262	1405	S. Subba Ra	Engineer-in-Charge, The Hindustan Construction Co. Ltd., Bhubaneswar, Dist. Puri. (Orissa).
1263	1406	R. Shunmuga-sundaram	Assistant Engineer, Highways, Sankeri-At-Salem. (Madras Province.)
1264	1407	S. P. Kapoor	S. D. O., P. W. D., Winterfield, Simla.
1265	1408	L. N. Kabiraj	Assistant Engineer, Damodar Valley Corporation, P. O. Bishangarh. (Hazaribagh), (Bihar).
1266	1409	G. L. Mital	District Engineer, P. W. D., B/R, Mathura. U.P.
1267	1410	Asim Mukherjee	Assistant Engineer, (Road Construction) W. & B., Lower Kadai Road, Berhampore. (West Bengal).
1268	1411	B. N. Chatto-padhya	Assistant Engineer, W. & B. Deptt., 10/A, Bepin Paul Road. Kalighat, Calcutta-26.
1269	1413	S. P. Bhatta-charyya	S. D. O. (W. B. Dept), Tamluk Construction Sub-Division, P. O. Tamluk, District Midnapore. (West Bengal).
1270	1414	K. V. Apte. (L.M.)	Director M/s. New United Construction and Engineering Co. Ltd., Industrial Assurance Building, Bombay 1.
1271	1415	M. D. Patel	Assistant Engineer, P. W. D. Construction Sub-Division, Bhadra, Ahmedabad.

Sr. No.	*Roll No.*	*Name.*	*Address.*
1272	1416	Prem Manohar	[illegible] Engineer, P. W. D. [illegible], 15 Mall Avenue, [illegible] (U.P.)
1273	1417	N [illegible]	[illegible]er (W. & B.) Mal- [illegible] Sub-Division, [illegible] Malda. (West [illegible]).
1274	1418	H Jeya [illegible]	Assistant Engineer, Highways, Sattur, Ramnad District. (Madras Province).
1275	1419	B. V. Reddy	Supervisor. (Highways). Kurnool East, Kurnool.
1276	1420	K. Sunder Naik	Assistant Engineer, Highways Department, Karur, District Trichinopoly. (Madras Province)
1277	1421	K. N. Basu	Executive Engineer, (W. & B.) Burdwan Dn., P. O. Hooghly.
1278	1422	S. M. Datta	Executive Engineer, Medical College Building Construction Division, P. O. Dibrugarh. (Assam.)
1279	1423	A. K. Srivastava	Assistant Engineer, P. W. D., Bahraich, U.P.
1280	1424	P. K. Ghoshal	Sub-Divisional Officer, W. & B. Deptt., Contai P. O. District Midnapore, (West Bengal).
1281	1425	K. C. Jain	P. A. to the Superintending Engineer, Calcutta Central Circle No: II, C.P.W.D., 15 1 Chowringhee Square, Calcutta.
1282	1426	P. C. Chatterjee	Assistant Engineer, (W&B Deptt) Krishnagar Construction Sub-Division, 2, Kantalpota Lane, Krishnagar, Dist. Nadia, (West Bengal).

Sr. No.	Roll No.	Name.	Address.
1283	1427	Albert Mayer	31, Union Square West, New York City, U. S. A.
1284	1428	K. Govindarajulu	Assistant Engineer, Highways, Rajampet, Cuddapah District.
1285	1429	B. Jayaraman	Assistant Engineer, (Bridges), 8, Dr. Nair Road, Theagarayanagar, Madras–17.
1286	1430	R. N. Joshi	C/o, S. B. Joshi & Co., Examiner Press Building, 35, Dalal Street, Fort, Bombay.
1287	1431	P. L. Lokagariwar	S. O. III in Engineer-in-Chief's Branch, A.H.Q., 97-C, Irwin Road, New Delhi.
1288	1432	Sujan Singh	Roads Engineer, Burmah Shell House, Connaught Circus, New Delhi 1.
1289	1433	S. P. Nigam	Assistant Engineer, P. W. D. (B. & R.), 13, Cement Concrete Road, Gorakhpur, U.P.
1290	1434	A. G. Tamizuddin	Assistant Engineer (Highways), Special Cement Concrete Sub-Division, Bellary. (Madras Province).
1291	1435	V. Subba Rao	Supervisor, P.W.D., C/o, Vemula Anjaneyulu Garu, Gudivada, Krishna District.
1292	1436	K. Subramania Chetty	Assistant Director, The Central Water Power Irrigation and Navigation Commission, Curzon Road, New Delhi.
1293	1437	K. V. Ahamad Koya	Section Officer, Highways, Taliparamba. (N. Malabar).

Associate Members

Sr. No.	Roll No.	Name.	Address.
1	34	A. R. P. Ramsay	Director, M/s. Tractors (India) Ltd, Victoria House, P.O. Box 323, Calcutta.
2	35	H. G. Hughes	Managing Director, Messrs. Armco (India) Ltd, P.O. Box 8989. 21, Esplanade Mansions, Calcutta.
3	36	A. Das Gupta	Works Manager, Indian Malleable Castings Ltd., Belghoria, 24 Parganas. (West Bengal).
4	37	William Harrison	Superintendent, Kumardhubi Engineering Works Ltd., Kumardhubi P.O. District Manbhum. (Bihar.)
5	38	F. C. Kraty	Director, Messrs. Marshall Sons & Co (India) Ltd., 99, Netaji Subhas Road, P. O. Box 22, Calcutta.
6	39	Basil Gray	Territorial Manager, C/o. Shalimar Tar Products (1935) Ltd, Block "M" Suraj Narain Building, Connaught Circus, New Delhi.

III. DEATH

The Council have received with regret intimation of the following death :—

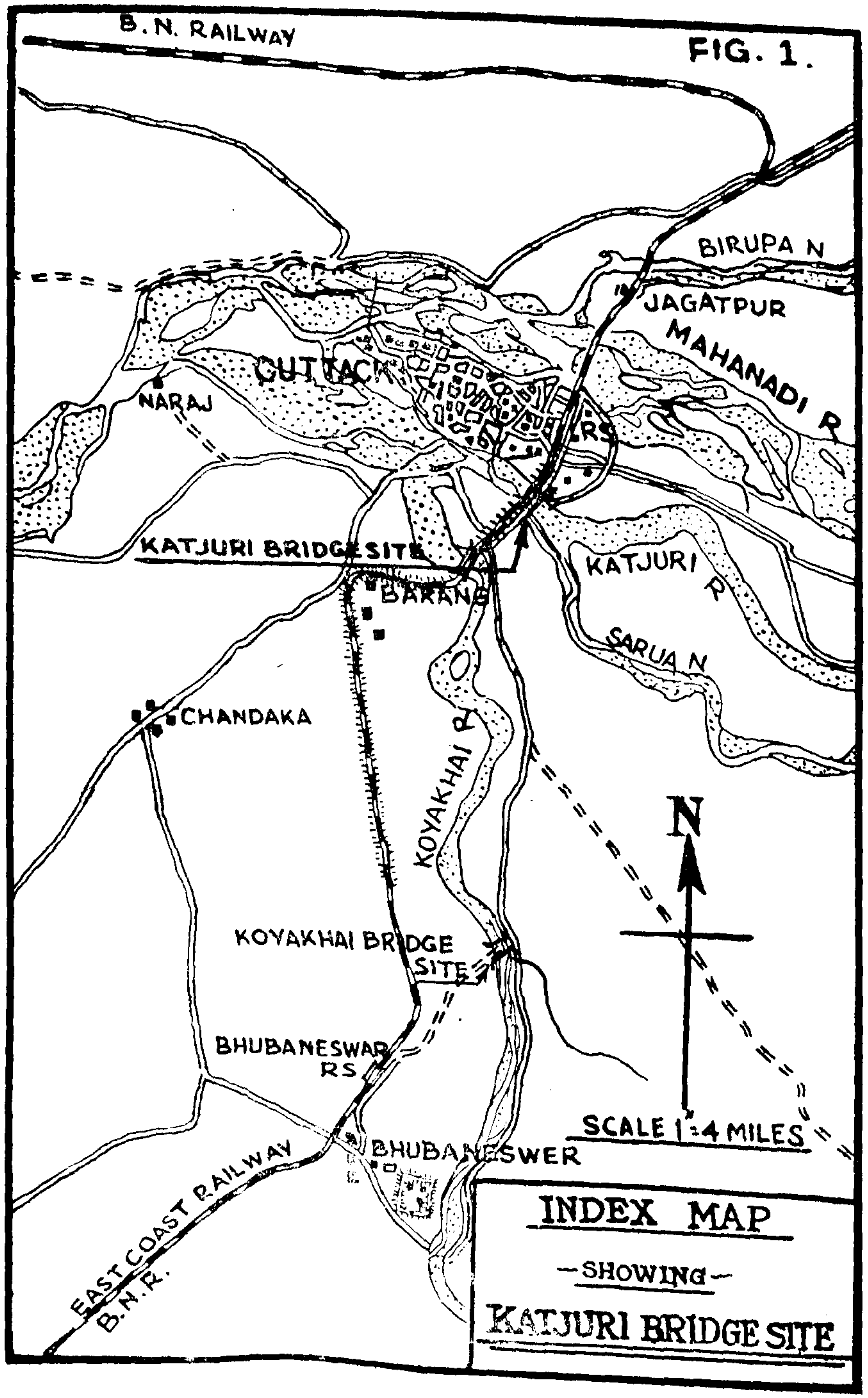
Sr. No.	Roll No.	Name.	Address.
1	1069	Bhagwat Prasad	Assistant Engineer, P. W. D., Ranchi. (Bihar).

IV. ADDRESSES OF MEMBERS WANTED

Communications addressed to the following members have been returned unclaimed by the Post Office. Members able to give information regarding the present address of any of those concerned are requested to forward particulars to Secretary, I. R. C.

Sr. No.	Roll No.	Name of Member.	Last known address.
1	1397	Ahmed Ovaise.	Assistant Engineer, P. W. D. B & R, 30 Munro Road, Agra Cantt.
2	453	Basu, N.	152, Ind. Rly Maint Co., C/o, 6. A. B. P. O.
3	779	Banerjee, Shishir Kumar.	Assistant Engineer, Patna Waterways Sub Division, Patna.
4	686	Bonsor, M. S.	Sub Divisional Officer, C o, The Garrison Engineer, Agartala, Tripura State.
5	622	Chakravarty, G. K. M.	State Engineer, Raigarh State, Raigarh.
6	576	Chowdhury, P. M.	District Engineer, P. W. D. Goona. (Gwalior).
7	575	Dixit, M. K.	C/o, Lashkar Municipal Committee, Lashkar, Gwalior.
8	1145	Dhruva R. C.	Executive Engineer, (Housing), Gaurd Room, Pochkhanwala Road, Worli, Bombay No. 18.
9	1279	Dubash, M. D.	No. 12, Rewa Mansion, Frere Road, Karachi.
10	20	Garewal, Sapuran Singh.	C o, The Ram Raj Ideal Town Co-operative Society, Ram Raj Mandi. (Saidapur-[illegible]). P. O. Besoma, District Meerut.
11	710	Ghosh, M. K.	Assistant Executive Engineer, Satbankura (Midnapore). (East Bengal).
12	700	Goswami, S. M.	Resident Engineer, C/o. Gannon Dunkerley & Co. Ltd., Secretariat Hill, Shillong. (Assam).
13	1225	Gulzar Singh, S.	Assistant Engineer, Designs, P. W. D. Building & Roads Branch, Simla-E.
14	894	Hanly, P. J.	15/1 Chowringhee Square, Calcutta.
15	229	Hodgson, E. S.	Overdeans Court, Diffen Hall near Farham Surrey, (England).
16	566	Jadhav, M. K.	Chief Engineer & State Architect, Ketkibag Bungalow, Camp Road, Baroda.
17	931	Khalique, M. V.	Ministry of Transport (Roads), New Delhi.
18	173	Kidar Nath.	32 Kaonli Road, Dehra-Dun. U. P.
19	24	Korni, M. A.	114 Regent Estate, Tolly Gunj, Calcutta.
20	998	Mathew K. George.	Supervisor, P. W. D., Trivandrum.
21	830	Mathur, J. K.	Assistant Engineer, Jodhpur Railway, Jodhpur.
22	206	Mufti. M. I. D.	Inspector of works and stores Central Command, Meerut.
23	183	Nawab Ahsan Yar Jung Bahadur(L.M.)	Afsar Manzil, Jublee Hill, Hyderabad-Deccan.
24	955	Parmeshri Dass.	Assistant Engineer, P.W.D., Azamgarh. U. P.

Sr. No.	Roll No.	Name of Member.	Last known address.
25	693	Rajaratnam, K. M. Mudaliar.	District Board Engineer, Kistna Dist. (Madras Presy.)
26	965	Rajgopalan, K. G.	Assistant Executive Engineer, Central P. W. D. Aviaton Sub-Division, Pudukottah Road, Trichinopoly. S. I. Rly.
27	844	Salukhe, V. B.	Guest House, Dewas Junior.
28	720	Shaw, M. (L. M.)	Executive Engineer, Central P. W. D., Indore.
29	1089	Uttam Singh. (L. M.)	Chief Engineer, Jaipur State, Jaipur.


B. N. RAILWAY
FIG. 1.
BIRUPA N
JAGATPUR
MAHANADI R
CUTTACK
NARAJ
RS
KATJURI BRIDGE SITE
KATJURI R
BARANG
SARUA N
CHANDAKA
KOYAKHAI R
N
KOYAKHAI BRIDGE SITE
BHUBANESWAR RS
SCALE 1"=4 MILES
BHUBANESWER
EAST COAST RAILWAY B.N.R.
INDEX MAP
—SHOWING—
KATJURI BRIDGE SITE

Another view of the Bridge under construction (the Railway Bridge is in the background)

shingle were found in the river bed. The exploratory borings were taken at the bridge site at 160 ft. apart and at 50 ft. on either side (upstream and downstream) of the centre line. The boring charts showing the details of the different strata met with, can be seen in plate I. When Artesian conditions were met with at the time of sinking some of the wells, deeper bores were put down 160 ft. apart on the centre line itself as shown in Plate II.

4. MAIN DETAILS OF THE BRIDGE

4.1. The bridge will have a superstructure of vibrated reinforced concrete balanced cantilever and suspended spans supported on piers and abutments made of 1 : 3 : 6 mass concrete, lightly reinforced. The foundation of each pier will be a twin well sunk to the approved depth. The width of the carriageway between kerbs will be 24 ft for vehicular traffic with 6 ft wide cantilever footpaths, raised 12 inches above the road level. The height of the embankments in the approaches at either end is about 25 ft., protected by stone pitching.

5. DATA FOR DESIGN AND SPECIFICATIONS OF WORKS

5.1. The Bridge Plan as shown in Plate III gives an idea of the general outline of the design of the bridge, omitting most of the details. The specifications laid down for the guidance of the contractors are attached as Appendix III.

6. PRELIMINARY ARRANGEMENTS

6.1. A siding was laid down at Cuttack railway station for unloading all materials which were arriving by rail. As a matter of fact all materials except sand, such as coal, cement, steel, stone chips, stone metal, gravel, plum stone etc., were brought by rail to Cuttack railway station and were carried from there by truck to the bridge site about 3 miles away.

6.2. Four quarries, described below, were taken on lease and labour camps set up at each.

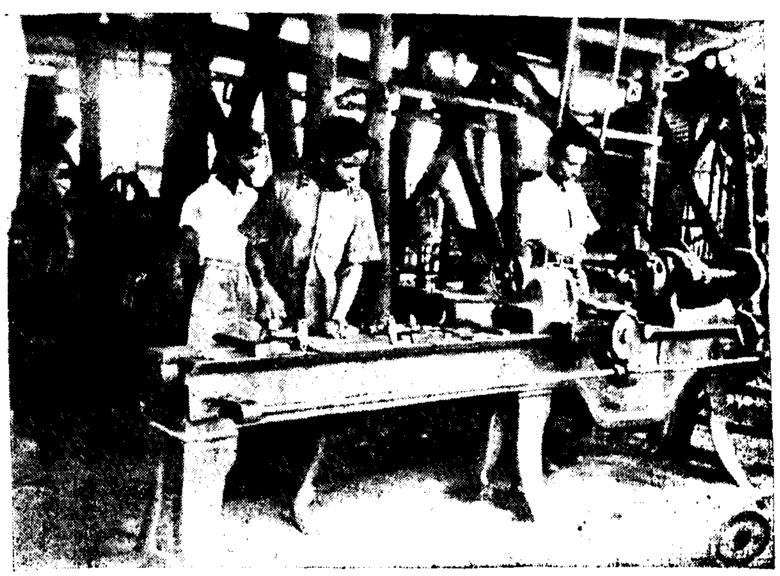
6.2.1. **Tapang Quarry.** This quarry, provides hard granite stone and is situated about 31 miles from Cuttack station on the Bengal Nagpur Railway. The Tapang railway station is situated about 3 miles from the quarry. Two Granulators were set up at this quarry to provide stone chips. Stone plums are also obtained from this quarry for the mass concrete work.

6.2.2. **Naraj Quarry.** This quarry is situated about 10 miles from the work site, but the route by which the metal is brought covers about 15 miles by boat and lorry. Mainly sand-stone is available in the Naraj Hill. The metal is taken to the river-side either by head load or by truck depending on the distance. There it is loaded in country boats, brought through the Mahanadi to the Taldanga canal and unloaded on the canal side at about a mile and a half from the work site, whence it is taken by truck.

6.2.3. **Haridaspur Quarry.** This quarry is situated at about 25 miles from Cuttack. Only hard granite stone chips and plums are being collected from this quarry. The stone is brought by rail from Haridaspur to Cuttack and then taken by truck to the bridge site.

6.2.4. **Hindol Road Quarry**. This quarry is situated about 49 miles from Cuttack on the local Bengal-Nagpur Railway line to Talcher. This quarry supplies gravel obtained by digging and screening. The gravel comes to the Hindol Road railway station by truck and then to Cuttack by rail.

6.3. A fairly big workshop has been built near the bridge site for steel fabrication, making shuttering, welding, cutting, drilling, etc., and manufacturing minor articles as well as for providing electricity for lighting and for the well sinking plant. Photograph 1 is a view of the workshop. Storage Godowns for cement and a



Photograph 1. A Corner of the Workshop

store yard have also been built near the Cuttack approach to the bridge.

6.4. The detail of equipment which was got together for the work, and is being used on it, is given in Appendix I.

6.5. An engineering appreciation of materials, tools, plant, and labour required for the whole work (including superstructure) will be found in Appendix II.

7. DESIGN AND CASTING OF WELLS

7.1. The Well.

7.1.1. The approved design for the foundation of piers consists of a twin well under each pier. The design of a twin well is illustrated in plate IV which shows that the overall size is 30 ft by 15 ft 6 in. The thickness of the steining is three

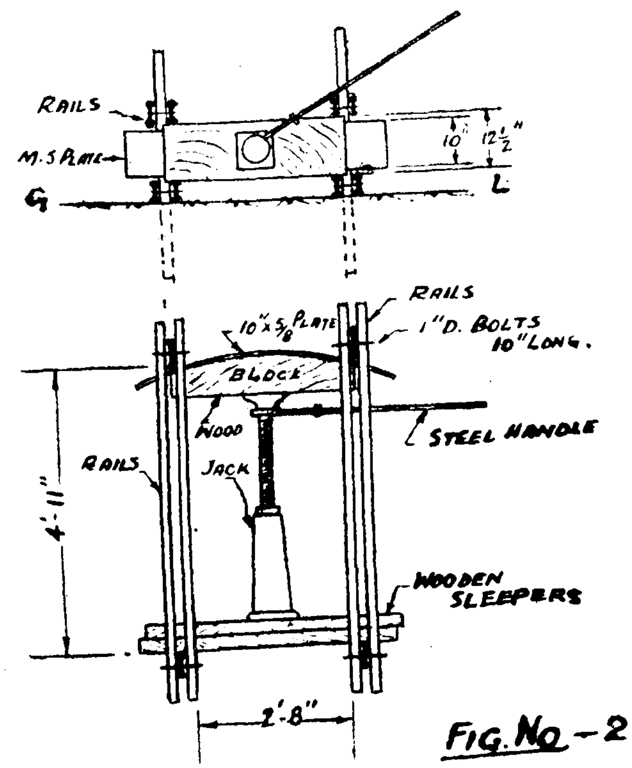


FIG. No-2

BENDING M.S. PLATE FOR CUTTING EDGE.

4 8

feet and the dredge holes are 9 ft. 6 in. diameter. Plate IV also gives the design of the massive reinforced concrete curb and the details of the steel cutting edge.

7.1.2. The cutting edge was built out of 10 in. by $\frac{5}{8}$ in. plates and 4 in. by $\frac{1}{2}$ in. equal angles, bent to the proper shape and electrically welded. Fig. 2 illustrates the simple equipment used for bending plates and angles. It consists of a screw jack working in a frame of steel rails and pressing the steel sections into shape by means of a timber batten cut to the proper curve.

7.1.3. Holes were drilled in the horizontal legs of the cutting edge angles for taking the vertical bond rods.

7.2. Form Work.

7.2.1. Well curb forms. The inside forms for the well curb were made out of curved wooden battens lined with sheet steel, six segments forming one part of the twin well. The outer forms consisted of steel plates 5 ft by 4 ft high with steel angles welded along the edges and the centre (for stiffening). Sixteen such pieces went round the complete outer periphery of a twin well.

7.2.2. Steining forms. The outer forms were the same as those for the curb. The inner forms were similar in design but with a smaller radius of curvature. Six pieces made up the inner circle.

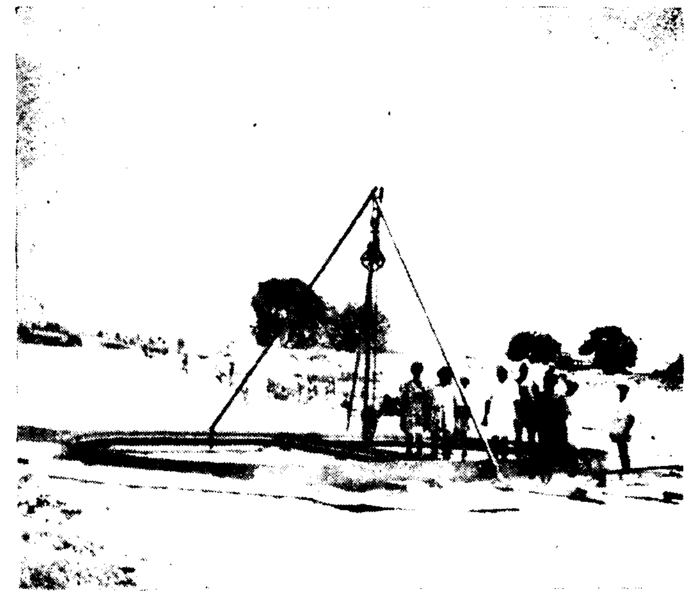
Two complete sets were required for uninterrupted casting on a twin well and enough forms were fabricated for simultaneous work on three twin wells.

7.3. Casting the Well Curb.

7.3.1. As the existing railway bridge was only 800 feet upstream, it was stipulated in the design data that the road bridge piers would be placed exactly in line with the piers of the railway-bridge so as to avoid undesirable hydraulic action. When commencing work on a pier, the cutting edge was laid in position with reference to the corresponding railway bridge pier and accurately levelled up on sand bags. Then the inside forms, the reinforcement and vertical bond rods, and the outside forms were fixed, in that order.

7.3.2. Photo 2 shows the cutting edge being fixed in position and Photo 3 shows the inner formwork fixed and the reinforcement being tied.

7.3.3. The curb was then cast in 1:2:4 cement concrete, the coarse aggregate being granite chips upto $\frac{3}{4}$ in. size and the fine aggregate, sand from the river-bed itself. The permissible slump was $\frac{1}{2}$ in. to 1 in.



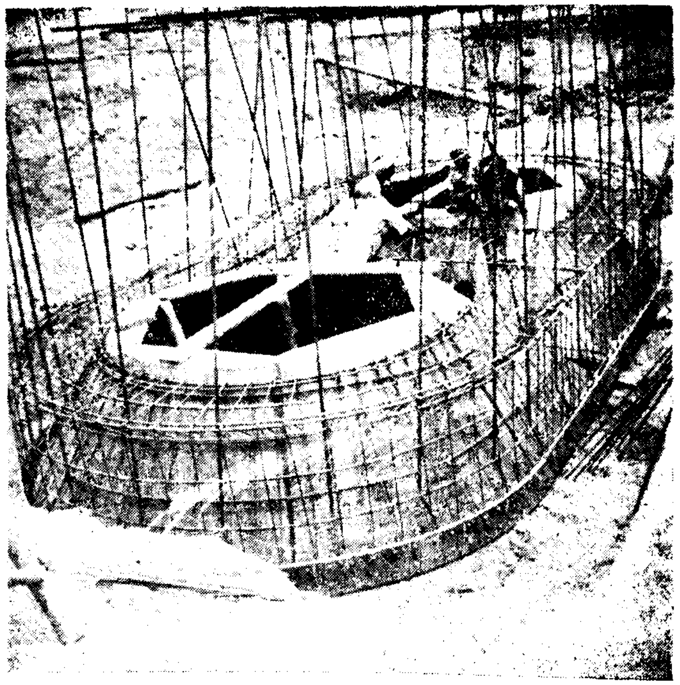
Photograph 2. **Aligning cutting edge.**

7.4. Casting The Steining.

7.4.1. Along with the well curb a 4-foot lift of steining was also cast. This consisted of 1:3:6 cement concrete with a $\frac{3}{4}$ in. slump on the average as for the curb. Plums of a maximum size of $\frac{1}{3}$ cu. ft. were placed in the steining concrete, spaced not less than six inches apart. This first section of 8 feet (4 ft curb and 4 ft steining) was sunk by manual dredging so that sinking might start on an even keel. Lifts of four feet at a time were added as sinking progressed.

8. CONCRETE MATERIALS AND TESTS

8.1. **Aggregates.** The coarse aggregate, including plums, was obtained from the sources mentioned in Section 6. Sand was obtained from the river-bed itself. This sand being moist, bulking due to moisture was determined and the addition of 20 per cent extra sand was authorized to compensate for bulking.



Photograph 3. Inner Shuttering and reinforcements of Curb.

8.2. Tests.

8.2.1. A slump test was made daily with the usual apparatus. The permissible limits for slump were laid pown as $\frac{1}{2}$ in. to 1 in. for all concrete work.

8.2.2. At every stage of work the compressive strength of concrete was tested. Two six-inch cube test blocks were taken simultaneously from a batch. One of these was cured for seven days and the other for 28 days. For the mix used in the well curbs (1 : 2 : 4) the strength was found to be 4200 lb. per sq. in. against the prescribed minimum of 3,375 lb. per sq. in. at 28 days curing. The corresponding test strength for 1 : 3 : 6 concrete was 2200 lb. per sq. in. Some of the test results are given in Table No. 1.

9. WELL SINKING

9.1. When the first eight feet had been sunk by manual labour and another 12 to 16 feet of steining had been added, the

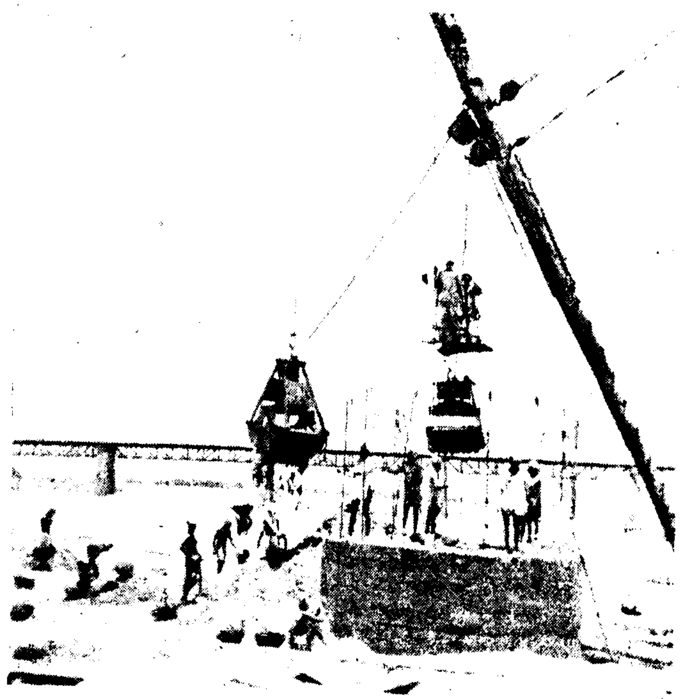
TABLE I

CHART FOR COMPRESSION TEST OF CONCRETE BLOCKS

Description	Date of moulding	Date of Testing	Period of curing	Area in Square inch	Crushing load in tons	Crushing load per square inch
Results obtained from Alipore Test House, Calcutta.						
1 : 3 : 6 mix $1\frac{1}{2}''$ stone aggregate steining P_3	1- 4-48	23-4-48	21 days	36.60	62	3794 lbs.
1 : 3 : 6 mix Steining P_3.	17- 4-48	10-5-48	21 ,,	37.21	41	2468 ,,
1 : 3 : 6 mix Steining P_3.	11- 5-48	31-5-48	20 ,,	36.84	47	2857 ,,
1 : 2 : 4 mix $\frac{3}{4}''$ aggregate abutment	17-12-48	15-1-49	23 ,,	36.24	72	4450 ,,
1 : 2 : 4 mix—well curb P_{10}.	3- 2-49	1-3-49	28 ,,	37.64	64	3809 ,,
Figures obtained by Avery testing machine at site.						
Well Cap P_1 1 : 2 : 4	11-4-49	9-5-49	28 days	36.00	65	4044 lbs.
Pier Cap P_2 ,,	15-4-49	13-5-49	28 ,,	36.375	83.5	5142 ,,
Pier Cap P_3 ,,	18-4-49	16-5-49	28 ,,	36.38	73	4500 ,,
Superstructure Downstream Girder 1 : 2 : 4	30-5-49	28-6-49	29 ,,	36.00	69	4293 ,,
Superstructure Deck slab 1:2:4	6-6-49	4-7-49	28 ,,	36.00	62.5	3889 ,,

dredging grab, worked by a steam winch, was brought into service. Photograph 4 shows the grabs at work. The well sank easily through the top strata of earth and sand but sinking became difficult when the cutting edge encountered stiff black clay. The usual expedients of dewatering, blasting and loading (kentledge) were restored to

9.2. **Blasting.** When dewatering was not very effective blasting with gelignite was tried. The function of the charge was to shake the well and thus reduce the frictional resistance to sinking. Incidentally, the blast cleared the sloping inner curb face of sticky clay, another possible cause of resistance to sinking.



Photograph 4. **Grabs at work.**

9.2.1. The use of an explosive for well sinking is not safe unless made carefully. A heavy charge could not be applied as that could possibly damage the steining. For the same reason a charge

was never applied with a sump of less than five feet, so as to ensure that only earth below the level of the cutting edge was disturbed and the curb was not damaged.

9.2.2. The best method of applying the explosive charge is not merely to let it down into the sump but to send down a diver to place it in the clay strata on the side of the sump. As an experienced diver was not available this could not be done and the former less satisfactory method was used.

9.3. Loading (Kentledge). Fig. 3. When the first two devices (viz., dewatering and blasting) failed to produce satisfactory results the well was loaded with steel rails or sand bags. As dredging had to go on at the same time the dredge holes had to be kept open. (Dewatering also went on simultaneously when possible). The rails were laid over sleepers placed on the top of the steining. When sand bags were used they were placed directly on the steining without sleepers and the kentledge load attained a height of 20 feet at times.

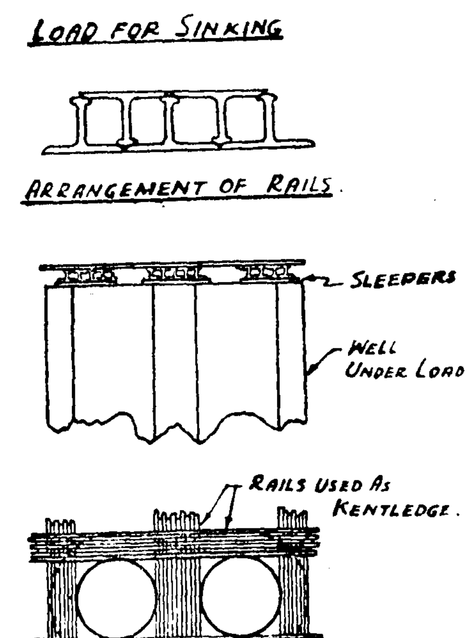


FIG. NO. 3.

9.4. **Special pumping arrangements.** When working in black sticky clay, grabbing was not satisfactory and did not bring up enough of material, particularly when water in the well was deep Dewatering had to be done while grabbing was in progress. There was no arrangement made for this in the first five twin wells —Piers 1 to 5—and the pump had to be removed for working the grab. In the subsequent piers this difficulty was overcome by making two vertical pumping holes, of 18 in. diameter each, in the steining with horizontal inlets 8 ft above the cutting edge (i.e., in the second 4 ft lift above the curb top). The arrangement is shown in Fig. 4. When a well is finally plugged these pumping holes are also filled up with 1 : 3 : 6 concrete under pressure, before placing the top plug.

ARRANGEMENT OF PUMP HOLES.

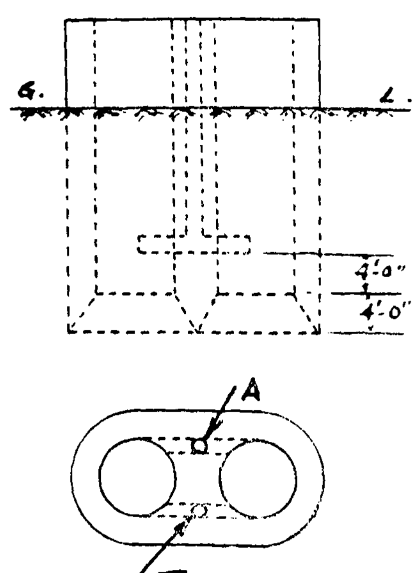


FIG. NO. 4.

10. SINKING PROBLEMS

10.1. Artesian Action.

10.1.1. The well curb of pier No. 2 was resting on stiff clay at R. L. 2.00 and there was a sump of 8 ft below the cutting edge level. As sinking was very difficult, some labourers were sent down with pickaxes to remove the clay. When they were at this job suddenly the strata underneath burst and water rushed in. The labourers came up and within two hours water stood at 51.00 in both chambers of the twin well and gradually rose to 55.20 while the water level in the river was only 47.00. Obviously the well had entered water bearing strata where hydraulic pressure was so great that it had pierced through about 16 foot thick clay strata still left below the sump. (The level of water bearing strata under P_2 is minus 22.00 ; vide Plate II).

10.1.2. Something similar happened in the case of P_3. From R. L. minus 23.00 the cutting edge suddenly sank to R. L. minus 30·00, after piercing two feet of clay and entering five feet into sand. Water rushed up and began to overflow the well, bringing sand with it.

10 1.3. Artesian action was encountered in all the wells of piers 1 to 6 at different stages. In all cases it came on rather suddenly although it could perhaps have been anticipated from the boring chart (Plate II) and precautions taken in advance.

10.1.4. When Artesian action started in a well, the water level rose until water began to overflow at a tremendous rate. De-watering by pumping was obviously out of question and the wells sank abruptly because of the disturbance in the water bearing strata under the cutting edge on account of the flow of water. Thus P_1 sank by 3 ft 6 in, P_2 by 11 ft, P_4 by 14 ft 6 in., and P_5 by 5 ft 6 in. In the case of P_4 there was a shift of 14 in. during this abrupt fall.

10.1.5. When so many wells began to spout there was a danger of the whole foundation getting undermined and the first concern was to stop the overflow. This was done by quickly building up a temporary steining in brickwork so as to contain the water and stop the flow. Table 2 gives the top level of the pervious strata and the water level inside the well, against the river water level of R. L. 47.00.

Table 2

	P_1	P_2	P_3	P_4	P_5	P_6
1. R.L. of pervious strata met	-16.00	-22.00	-25.00	-24.00	-28.00	-38.00
2. R. L. of water level in well	55.50	55.20	54.60	53.50	53.00	51.00

10.1.6. The Artesian action made further sinking of some wells necessary, after controlling the overflow, so as to get the bottom of the wells into undisturbed strata and plug them there.

10.1.7. When this temporary steining was broken (in P_2) for building up the concrete steining for further sinking below R.L. minus 30.00, water began to spout up again and the well sank suddenly by six inches. The brick steining was then left intact and encased in concrete. This was done in the other wells also.

10.1.8. Borings taken down to R. L. minus 60.00 showed that water bearing strata continued down to that level at least and possibly further down also.

10.1.9. During the attempts to trace out the source of the waters of the underground Artesian reservoir, the level of the water upstream of the Mahanadi anicut, four miles away, was found to be 68.31 while the Artesian water rose from R.L. 51.00 to 55.50 in the wells. Although chemical analyses of the two waters differed considerably, such discrepancies could be reasonably attributed to the passage of water through various strata from the anicut to the bridge site.

10.2. Cant.

10.2.1 Wells going out of plumb or getting displaced from the correct position are phenomena of frequent occurence in foundation work. The former of these defects, known as cant, could arise from a multiplicity of causes, e g:

(1) Irregular casting. If the outer surface of the concrete curb or steining is irregular, frictional resistance to sinking will vary, the well sinking unevenly and causing tilt.

(2) Eccentric or unequal dredging: If dredging is not done in the middle of the well or if the two parts of a twin well are dredged unequally, the well would sink more on one side than on the other and would develop a cant.

If dredging is controlled, by shifting the position of the grab from one side of the well to the other, or by dredging more in one part of the twin well than in the other, it is possible, within limits, to set right a slight cant. This method was successful to some extent when the depth was small. But when sinking had progressed more than 50 feet, this method became ineffective.

(3) Non-uniform strata. If the curb meets with strata of unequal strength along the periphery of the cutting edge, sinking will be unequal. The well will follow the line of least resistance and will tend to go down more on the side on which the strata is soft.

10.2.2 Methods to counteract cant.

One method, already mentioned, of setting right any tendency to cant was to regulate dredging. Another method, tried on pier No. 9, was to fix four *sal balli* props on the leaning side of the well. But when the well sank by another three inches all the four props gave way and the process was given up.

A third method used for rectifying cant was cantilever loading. Four rolled steel joists, 18 in. deep, were bolted on to the steining and loaded. The R.S.J's failed under a load of 350 tons, while the cant was not materially reduced (the reduction in cant was 0.03 ft.)

Many wells on this work have developed a cant and the problem of rectifying it remains unsolved. Although the cant in these wells was not so serious as to endanger the safety of the foundations, yet some technique has got to be evolved for avoiding or rectifying this defect as far as possible.

10.3. Shift.

10.3.1. Shift, i.e. displacement of a well from its correct position, is a phenomenon which goes hand in hand with cant. It was observed that wells did not cant or shift while they were going through the upper soft earth or through sand. Trouble started when the cutting edge came up against stiff black clay. The causes which led to cant also caused the well to shift. But there is no relationship between cant and shift so that the extent of one could not be inferred from the extent of the other. The following table will illustrate this point:

Pier No.	Cant across the well (feet.)	Shift (feet.)
4	0.15	1.2
9	0.75	0.5
13	0.60	2.5

10.3.2. The extent of cant would depend on the angle through which the well rotates while the shift would depend on this angle as well as on the position of the centre of rotation.

CE 17

10.4. Sand Blow.

10.4.1. Sometimes, while work was in progress, it was observed that the well would be suddenly choked with sand entering below the cutting edge. Such sand blows occurred only when sinking was going on in black clay and then too while work was actually in progress. No blows occurred when work was at a stand-still. Although occasionally a blow might occur during dredging, mostly they occurred during de-watering. The presence of water inside the well did not prevent sand blows and these were observed to occur even with 60 ft. of water inside the well.

10.4.2. While sinking, the well disturbed the strata through which it passed and probably left such strata fissured. When de-watering was being done water from the surrounding strata would flow into the well under the cutting edge and sometimes sand from the sand strata above, being practically cohesionless, would flow in the wake of the water through these fissures and fill up the well.

10.4.3. By keeping a vigilant eye on the ground round the well sand blows could be predicted, for any major movement of earth at the bottom would be transmitted to the top surface. Any tendency to blow could be checked or aborted by promptly stopping the pumps as soon as any cracks or fissures were observed at the ground level. In the first few cases, however, such preliminary cracks remained unnoticed. Ditches as big as 20 ft. by 18 ft. by 4 ft. were observed at the ground level round wells P_4, P_7 and P_{10} while sand rose in the well to 22 ft. above the cutting edge.

10.4.4. Other typical cases of sand blow occurred when the well curb was going through sandy strata or mixed clay and sand. When the strata below the cutting edge was disturbed by dredging, a stage might come when the sides of the dredgehole or sump gave way suddenly and a blow occured. In such cases the grab was caught in the debris and, as in most cases the well also sank suddenly, the grab chain invariably gave way. In the case of well No. 5, it took almost four hours to locate the grab in the debris. As there was no diver at hand the following device was adopted.

10.4.5. An airlift pump was put down in the well to pump out the debris. At the same time a pulsometer pumped in water into the adjacent compartment of the twin well so as to force the water downwards. A current of water was thus set up from the latter half of the well under the central cutting edge into the former half. The debris was loosened by this current and was easily pumped up, releasing the grab which was hooked out.

11. BEARING CAPACITY OF THE SOIL.

11.1. In designing the bridge, the bearing capacity of the strata in which the wells were to be plugged was assumed at 7 tons per sq. foot, but no actual tests were made because the railway bridge, only 800 ft upstream, had been successfully supported for half a century by the strata at a corresponding depth and carrying a load slightly in excess of 7 tons per sq. foot.

11.2. When the stage for plugging the wells of P_1, P_2 and P_3 arrived, however, an effort was made to find the bearing capacity of the strata under these wells because they had been taken down much lower (as explained in 11.1) than the corresponding piers of the railway bridge. From the erratic behaviour of these wells during sinking, it was difficult to decide whether they could be safely plugged at the design depth of minus 15.00. Strata at this depth seemed to be soft, probably due to the upward seepage of water from the water bearing strata below. This seepage was found to have a softening effect on the soil upto 10 to 12 feet above the Artesian strata.

11.3. In order to test the bearing capacity of the soil, samples were sent to the Soil Research Stations at Madras and Karnal. In the case of P_2 samples were taken by means of a grab working at R.L. minus 19.00, but in the case of P_1 and P_3 which had already entered sandy strata, samples were taken from the heaps of excavated soil representing strata at about R.L. minus 11.00 and minus 22.00 respectively. The reports from the Research Stations suggested a bearing capacity, in an unconfined state, of 7·15 tons per sq. ft. in P_1, 5.5 tons per sq. ft. in P_2 and 2.6 tons per sq. ft. in P_3, with the comment that the procedure of taking the samples was defective. The samples should have been taken in such a way as to represent the actual conditions in which the soil existed in situ. Even in the case of P_2 the sample brought up had been disturbed by the grab and by its passage through the water in the well.

11.4. Later on un-successful attempts were made to obtain undisturbed samples by means of tools obtained from the Central Roads Organisation. These tools were found to be unsuitable for the granular and sticky soil in the foundations.

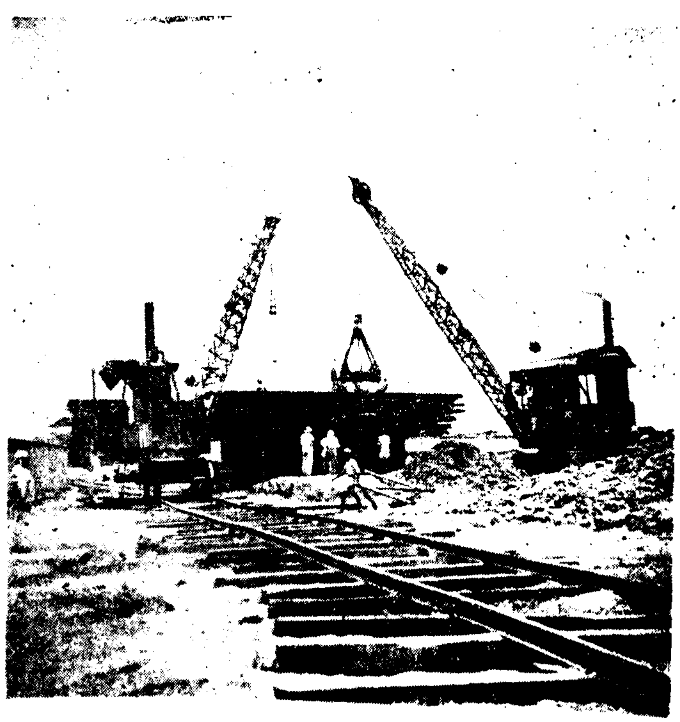
11.5. Finally, it was presumed that as even the disturbed soil sample had a bearing capacity of 7.15 tons per sq. ft. undisturbed strata in situ would be safe enough. However, in order to be quite sure of the safety of the foundations, loading tests were carried out.

12. TESTING OF WELLS

12.1. The wells of piers P_1, P_6 and P_7 have been tested so far, P_1 under a test load of 1000 tons and P_6 and P_7 under 1200 tons. The test loads were designed to give the same unit pressure under the steining as would develope under the whole area of the well plug and steining after the bridge came into use.

At the time of the tests the cutting edges of P_1, P_6 and P_7 were at R. L. minus 29.96, minus 51.69, and minus 36.11 respectively.

12.2. The test procedure was to fill the well with sand to 20 feet above the cutting edge for P_1 and 25 ft. for P_6 and P_7. The test load was then applied in stages of 100 tons with an interval of at least two hours after the addition of every 100 tons. Levels were taken at the start and after the application of every 100 tons. When the specified test load was reached the well was kept under observation for 24 hours. Photo No. 5 shows a test in progress.



Photograph 5. Test Loading with Rail Kentledge.

12.3. In the case of pier No. 1 the temporary brick steining, mentioned in para. No. 10.1.5, failed under the test load when it reached 700 tons. Wooden sleepers were then added round the brick steining to strengthen it and the test was completed under a load of 1000 tons.

12.4. In P_7 there was no settlement under the test load of 1200 tons kept on for three days.

12.5. In the case of P_6 the position was as follows:—

Load testing of P_7 and P_8 had been ordered and it was decided that if they were satisfactory, P_6 would be plugged at R. L. minus 53.00. At that time the cutting edge of P_6 was at R. L. minus 49.00. When an attempt was made at sinking it to minus 53.00 by de-watering, the well suddenly sank by 2 ft 3 in., the sump was filled up and the earth heaved up above the cutting edge by 18 in. On considering this behaviour it was decided to test this well also.

12.6. As de-watering for further sinking was considered undesirable, the following procedure was approved for testing P_6:—

(1) Dredge the well and make a two to three foot sump so as to leave no soft earth under the cutting edge.

(2) With a sump of 2 to 3 ft give two light charges of one gelignite stick at a time in order to sink the well if possible.

(3) Clean the well of any loose stuff produced by the blasts.

(4) Fill sand to 25 ft above the cutting edge.

(5) Apply a test load of 1200 tons.

12.7. Instructions 1 to 3 were not fully carried out by mistake and the well sank under test by half an inch under a load of 620 tons, and by another ½ in. under 670 tons. Under 1200 tons it had sunk by 0·22 ft. in all. The well was then kept under observation for three days; on the first day it sank 0·01 ft., but was steady for the next two days. The test was not, however, considered satisfactory as it had not been carried out according to instructions. The load test will now be repeated after sinking to R. L. minus 53.00.

12.8. The load testing of wells requires a lot of time. Employing two cranes and four lorries the time schedule is as follows:—

Loading 1200 tons.	...	7 days.
Observation.	...	3 days.
Unloading 1200 tons.	...	6 days.
Total:	...	16 days.

The tests were carried out in May 1949 at the fag end of the working season when the plant could be spared for the tests. Ordinarily during the working season, when plant would be required for other work, a test would take three weeks.

13. PLUGGING OF WELLS

13.1. A well should be plugged at a depth where the foundations could be reasonably expected to be safe against scour and where the soil under the well would have sufficient bearing capacity to take the loads coming on it. Generally speaking the completion plans of the fifty year old railway bridge upstream gave an indication of the likely scour depths. For bearing capacity it was felt that where a well was to be plugged in soft strata it would be load-tested. When a well was plugged in sandy strata (coarse sand) which, in a confined state, has great bearing capacity, it should be insured that there was adequate depth of sand below the plug. A depth of 15 ft. of sand below the plug was considered safe.

13.2. In P_6 and P_7, which were to be plugged in comparatively soft black clay, an attempt was made to de-water the wells completely and to increase the bearing area by undercutting the well curb all round and to form the plug in the shape of a bulb, somewhat like the lower end of a Franki pile. The aim was to make the concrete plug extend by one foot beyond the outer face of the well steining all round, and to a thickness of about three feet below the cutting edge. It was intended to place $\frac{1}{4}$ inch diameter rods spaced about 12 in. apart in the plug as near its bottom as possible. But when in well No. 7 the water level was depressed from R. L.+47.00 to R. L.+5.00 the well sank suddenly by 8 inches. Similarly well No. 6 suddenly went down by 2 ft 3 inches (vide para 12.5). Well No. 8 also behaved similarly. The attempt to form bulb plugs was then abandoned, and load testing was resorted to, as already described.

13.3. The design depth for the bottom plug in 1: 3: 6 cement concrete in a well is 5 ft. $4\frac{1}{2}$ in. above the cutting edge. In wells No. 1 to 6, however, it was decided to have 9 ft. $4\frac{1}{2}$ in. plugs in 1 : 2 : 4 cement concrete. Concrete of 1 : 3 : 6 mix was used in the plug in well No. 2 but water continued to ooze through it. It was felt that this was due to hydraulic pressure in the Artesian strata under the plug and a richer mix and a thicker plug were afterwards adopted.

13.4. In laying these plugs under water, concreting was done by means of a cage illustrated in Fig. 5. It is a cubical steel-plate box with two hinged lids at the bottom and a sliding lid at the top. The cage was charged with almost dry concrete and slowly let down into the well, the load being taken by the wire rope A, hook B, (which is hinged at C), rod D, and the two lid chains E. When

the cage touched the bottom of the well the wire rope A was slackened, the weighted lever came to the position shown dotted and released the rod D. Next the cage was pulled up. The four ropes, F, which remained slack going down then came into play and pulled up the cage. The lower lids opened and deposited the concrete.

CAGE FOR CONCRETING BOTTOM PLUG.

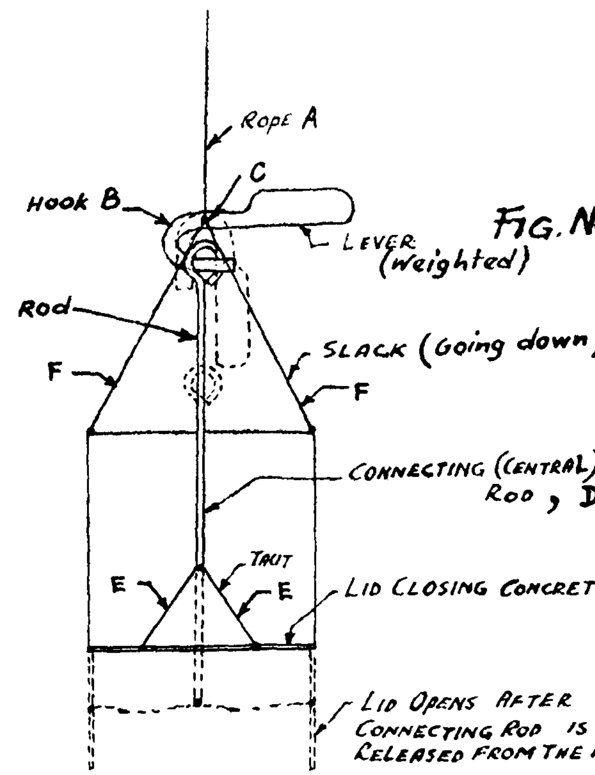


FIG. No. 5.

13.5. After laying the bottom plug, the well was filled with sand and the top one foot plugged with 1: 3: 6 cement concrete flush with the top of the steinings.

13.6. Over the top plug the well cap, 30 ft. by 16 ft. 6 in. by 2 ft., was laid with vibrated reinforced cement concrete, 1: 2: 4, with about 2 per cent reinforcement. The vertical steining rods were bent into the well cap.

13.7. The top R.L. of the well caps was at +50.00.

13.8. On the well caps, the piers are being built, boing just laid on the caps, and not tied down to them. The piers are placed exactly where they should be as originally designed, irrespective of the slight shift or twist of the wells. These piers P_1, P_2, P_3, and P_4, where the shift of the wells was nominal and negligible, were built according to plan. P_4 where the shift was 14½ in. still remains unbuilt.

14. POWER USED IN SINKING WELLS

14.1. The average power used in sinking the wells per foot depth has been worked out from the accounts maintained on the work. The power would naturally vary with the soils encountered and the efficiency of the equipment used. On the whole, well sinking on piers P_1, P_2, and P_3 took 14.9 H.P., working for one day, per foot in strata as shown on the boring chart in Plate II. Details are given in the Table No. 3.

15. A RUMOUR AND A SCARE.

15.1. Before ending this brief Paper, the author cannot help but refer to the rumour that sometimes spread over the whole of Orissa, that human sacrifice still formed an essential part in the construction of bridges across mighty rivers. The Presiding Deities of the Indian rivers are said to demand propitiation before they will permit any 'bonds' to be thrown across them. A rumour still persists that human sacrifices were made at the time the Mahanadi Railway Bridge was constructed. There is no accounting for the impressions to which the ignorant, superstitious mind clings nor of the manner in which rumour gains ground. Here was one instance: Because of illness of one of her children, a religious minded lady of Cuttack decided to offer prayers and the feeding of poor children formed a part of the ceremonies. One evening she sent her cook to get together some children for this purpose. While he was on this errand, Dame Rumour siezed upon the opportunity and some-how the impression suddenly got round that some one was collecting these boys to sacrifice them for the successful construction of the Kathjuri Bridge. Suddenly the poor cook found himself surrounded by a hostile crowd who started beating him up. As a result he was in the hospital for a month.

15.2. A possible explanation is that fatal accidents which sometimes happen in such big constructions are ascribed by the credulous to the irate River-God.

15.3. In order to counteract such wild rumours the Local Government had to issue a notice in the Press to remove this impression from the public mind.

TABLE 3

HORSE POWER USED FOR WELL SINKING ON PIERS NO. P_1, P_2 & P_3

Pier No.	Water Pump Capacity-6 H.P.		Lorries employed to carry Kentledge. Capacity-25 H.P.		Boiler, Cranes etc. 2 sets Capacity-10 H.P.		Air-Compressor for pumping. Capacity 20 H.P.		Total H.P. days	Length of steining in ft.
	Days worked	H.P.-days	Days worked	H.P.-days	Days worked	H.P.-days	Days worked	H.P.-days		
1	20	120	0	0	62×2=124	1240	13	260	1620	86
2	16	96	10	250	33×2=66	660	6	120	1126	96
3	12	72	12	300	43×2=86	860	10	200	1432	98
	Total	288		550		2760		580	4178	280

Therefore H. P.-days used for sinking per foot of well $= \frac{4178}{280} = 14.9$ H.P.-Days (approx.)

Appendix I

MACHINERY, TOOLS, AND PLANT

1. One 9.5 H.P. Cooper Engine is used for running the different machines in the workshop. It consumes 3 gallons of Diesel oil in 8 hours. There are two lathes, two drilling machines, a blower, a screw-cutting machine, and a hack-saw in the workshop, all worked by this engine. The capacity of one lathe is 10 ft and that of the other 6 ft. The power drill (3 H.P) is used for drilling holes in structural steel. The capacity of the 6-H.P. screw cutting machine is 4 inches. The hack-saw (2.H.P.) is used for cutting steel structurals upto 12 inches size. Rods, flats, and plates are cut by oxy-acetylene flame.

2. In addition to the above there are other machines as described below :—

2.1. Bandsaw (20 H.P) and Circular saw (12 H.P.) for cutting timber. These are worked by a diesel engine consuming 4 gallons fuel oil in 8 hours.

2.2. Welding sets :— Two units. They work at 20 H.P. and take up 300 amps. All the welding for the form work, the well curb cutting edge, and superstructure staging was done by these two sets.

2.3. Diesel generating sets :— Two units. 30 K.W. and 17.5 K.W., consuming about 16 gallons and 12 gallons fuel oil respectively in eight hours. The former is used for general lighting purposes and also for lighting up the area near the bridge site for night work, and the latter for running six drilling machines and other minor plant.

2.4. Air-Compressors;—Two units. One works at 55 H.P. and another at 90 H.P. delivering air at 100 lbs per square inch. Each consumes 16 gallons of diesel oil and two gallons of lubricating oil in eight hours.

The compressors have multifarious uses. They supply compressed air to vibrators, to the air-lift pump, and sometimes to the diver when he goes down the well. Hardly a day has passed, when the compressors remained idle since pumping water from the well is a most convenient way of sinking the well.

2.5. Boilers— There are eight boilers at two per pier (two wells). They work at 100 lbs. per square inch pressure supplying steam for the winches.

2.6. Grabs— There are eight grabs, of two types:

(a) Clam shell or circular plate grabs of the type of Bell's patent dredger.

(b) Tooth grabs or Grapple-dredgers—

The capacity of a plate type grab is 25 cu.ft., the diameter eight feet, and the weight 1 ton. The corresponding figures for a Tooth type grab are 20 cu. ft., six feet, and $\frac{3}{4}$ ton.

2.7. Water Pumps.

(i) Ram Pump and Centrifugal Pump—The capacity of each of these two pumps was 400 gallons per hour to a head of 60 ft at 10 H.P. These two pumps were used to pump water to the overhead galvanised iron tanks of 1600 gallons capacity at R.L. 110.00 on the left bank of the river from where water is supplied to the whole works by a network of galvanised pipes.

(ii) Sump pumps. The capacity of these three pumps is 10,000 gallons per hour each to a head of 40 ft. They are driven by compressed air at 100 lbs per sq. inch and are used for pumping water from the wells.

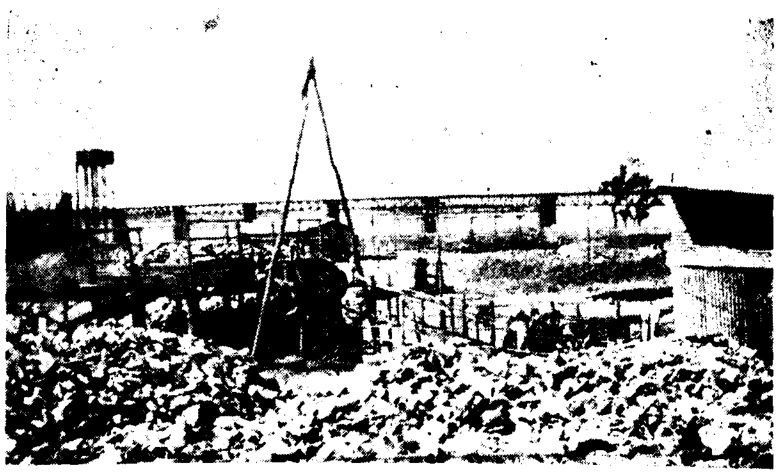
(iii) Airlift Pump—4 Nos. The capacity of these pumps is 20,000 gallons per hour each to a head of 90 to 100 ft. These are run by compressed air at 100 lbs. per square inch and are used for dewatering the wells. Being very effective they are almost always in use. But there is one drawback. If the suction is placed near the cutting edge, the compressed air disturbs the soil (especially when there is a sump). Further it required at least 20 ft. head of water to work, i.e., dewatering cannot be done completely for the last 20 ft. of water.

(iv) Pulsometer Steam Pump. The capacity of this pump is 25000 gallons per hour to a head of 90 ft. Being very efficient it was much used during the earlier stages. Later on difficulties arose regarding the supply of steam as the boilers were mostly required for other works and use of the pump was reduced.

2. 8. Cranes. Two Nos; one self-propelled. The capacity of each is 5 tons at 25 H. P. Length of jib-50 ft.

2.9. Crushers: Two Nos. They worked at 40 H. P. and can crush 2000 to 2500 cu. ft. in eight hours consuming 1 gallon of diesel oil and $\frac{1}{2}$ gallon of lubricating oil per hour. There are also two granulators, fixed up at Tapang quarry, with a capacity of 400 cu. ft. each of stone chips per day of 8 hours. Photo No. 6 shows a crusher at the bridge site.

2.10. Shingle washers:—2 Nos. They work at 10 H. P., output being 1000 c.ft. in eight hours. Water and unwashed metal were introduced at one end of the drum having spiral plates in side which was rotated slowly by an engine. Washed metal and muddy water came out at the other end when the water flowed away leaving the clean metal.



Photograph 6. **Stone crusher at Bridge site.**

2.11. Concrete Mixers: Eight Nos. There were two types of concrete mixer:

(i) Rotating-drum type- 4 Nos., ½ cu. yd. capacity. These work at 10 H. P. consuming three gallons of diesel oil and ½ gallon of lubricating oil in eight hours. Generally six to seven hours were required to cast four feet of the steining wall of 1088 cu. ft. approximately.

(ii) Tilting-drum type- 4 Nos., capacity 10/7 with 10 H. P., consumption three gallons of diesel oil and ½ gallon of lubricating oil in eight hours and out put 1088 cu. ft. in six to seven hours.

2.12. Derricks:—Six derricks with winches were used in the place of cranes to facilitate sinking operation. The derricks were built up of two wooden poles 25 ft each and joined by four bull-headed rails 10 ft long & 7/8 inch diameter bolts. The jibs were 60 ft long composed of two 30 ft wood poles and joined in the same manner. Generally 8 to 10 hours were required for 20 khalasis to assemble one jib and derrick.

2.13. Avery Testing Machine (Portable).

The machine is very handy and is specially designed for carrying out compression tests on cubes of various materials. The load is applied by means of a hydraulic cylinder and pump mounted on a strong, one-piece cast steel frame. The oil reservoir is contained in the base of the machine and a double-plunger hand-pump fixed to the side of the frame. A release valve is provided for lowering the ram to the bottom of the cylinder.

The load is indicated on a pressure gauge fitted with a lagless maximum pointer, and graduated from zero to 100 tons by divisions of 0. 5 ton. An additional gauge was also provided with a stop valve with graduation upto 40 tons by divisions of 0. 2 ton.

The machine was received in April, 1949. Prior to that all test cubes were sent to the Alipore Test House, Calcutta, for testing.

Appendix II

Engineering Appreciation of Materials, Tools, Plant & Labour required for Kathjuri Bridge Construction.

Serial No.	Item	Number or Quantity
	A. MATERIALS	
1	Cement	5500 tons
2	Steel	3000 tons including 1500 tons of pig iron and rails for kentledge.
3	Coal	2000 tons.
4	Stone chips, metal and plum stone	9,00,000 Cu. ft.
	B. TRANSPORT	
1	Wagons	2,600 Nos. for collection of materials, tools & plant.
2	Trucks	20 Nos. -do-
3	Country boats	5 Nos. -do-
	C. TOOLS & PLANT.	
1	Crane	2 Nos.
2	Derrick set	6 Nos.
3	Winch	8 sets.
4	Boiler	8 Nos.
5	Pumps	
	(a) Sump pump	3 Nos.
	(b) Air-lift pump	4 Nos.
	(c) Pulsometer pump	1 No.
6	Air Compressor	2 Nos.
7	Concrete Mixer	8 Nos.
8	Diver's Pump	1 No.
9	Big Screw Jack	6 Nos.
10	Water Supply pump with Engine	2 sets.
11	Electric Generating Set	2 Nos.
12	Welding set	2 Nos.
13	Stone Crusher	2 sets.
14	Gravel Washer	2 Nos.
15	Workshop Machines	
	(a) Lathe	2 sets.
	(b) Drilling Machine	2 sets,
	(c) Band Saw	2 sets.
	(d) Screw Cutting Machine	1 set.
	(e) Hacksaw	1 set (Power driven)
16	Ram Pump and Centrifugal Pump	2 sets for water supply.
	D. LABOUR AND STAFF.	1500 Nos. (including skilled and unskilled labour)

Appendix III

Extract from the Specifications and Data for the Design of the Bridge

The bridge shall consist of 1: 2: 4 reinforced concrete balanced cantilever and suspended span superstructure, supported on adequate piers and abutments made of 1: 3: 6 lightly reinforced mass cement concrete or brick or stone masonry of a twin well of adequate dimensions founded at the correct depth and constructed to approved specifications. The abutment foundations shall consist of concrete of adequate dimensions, founded at the correct depth and of approved specifications, executed in open digging.

The following shall be the requirements in designing the bridge :—

(a) Accommodation for traffic :—

(i) One roadway 24 ft. clear width between kerbs.

(ii) Two footpaths of 6 ft. clear width each—one on each side of the road.

(b) Numbers of spans and lengths :—

17 main spans of 160 ft. each centre to centre of piers.

2 End Cantilever spans of 54 feet each.

(c) Levels and heights on the bridge :—

(i) Finished level of caps of piers and lowest part of span bearings—not lower than R. L. 81.5.

(ii) Finished level of the lowest edge or face of the superstructure at the middle of the main spans—not lower than R. L. 91.5.

(iii) The crown of the road over the whole of the bridge to be horizontal—as low as possible but not lower than R. L. 100.75.

(iv) Footpaths shall be twelve inches higher than the adjacent road.

(v) Top of handrails shall be three feet higher than the adjacent footpaths.

(vi) Tops of steining of well foundations not higher than six inches above the lowest river water level in the dry season.

(vii) Base of well foundations of piers at R. L. (—) 35.00 in case of 12 piers and at somewhat higher levels for the remaining six piers. (This has subsequently been changed due to the unreliable behaviour of wells during sinking—Author).

(viii) Base of abutment foundations at R.L. 64.50.

(d) Bearings:—Of adequate design to provide suitably for good distribution of load expansion and contraction due to temperature and for rotational and other effects due to deflection.

(e) The design shall be good for the following standards of loading, stresses, physical conditions and other paraphernalia of design practice:—

(i) Hydraulic data:—

Observed H. F. L. — R. L. 79.00

Estimated discharge — 577,000 cusecs.

Estimated mean Velocity in the unobstructed Channel — 5 ft. per second.

Estimated scour level at piers in the main stream - R. L. 6. 5.

Estimated scour level at other points —

Twice the difference between H. F. L. and dry season bed level.

(ii) Live loads for roadway :—

2 traffic lanes of Class A (since changed to class AA — Author) loading with impact as prescribed in clauses 205 and 209 of the I. R. C. revised draft Bridge Code (1946).

(iii) Live loads for footpaths as prescribed in clause 207 of I. R. C. revised draft Bridge Code (1946).

(iv) Wind load, horizontal load due to water currents, longitudinal forces, buoyancy effects, earth pressure effects, temperature effects, deformation and secondary stresses and erection stresses in accordance with clauses 210, 211, 212, 214, 215, 216, 217, 218, 219 of I. R. C. Revised Draft Bridge Code (1946). No allowance need be made for Seismic stresses.

(v) Handrails to be designed in accordance with requirements of clause 112 of the same code.

(vi) Temperature variation — plus & minus 30°F.

(vii) Modular Ratio for reinforced concrete design — 15.

(viii) Permissible stresses shall be taken as under:— Compressive stress in 1 : 2 : 4 concrete in reinforced concrete members subjected to direct compression—600 lbs. per sq. inch. Shear stress in 1 : 2 : 4 reinforced cement concrete — 75 lbs. per sq. inch.

Maximum extreme fibre compressive stress in concrete of reinforced concrete members subjected to flexure — 750 lbs. per sq. inch.

Tensile stresses in steel reinforcement in reinforced concrete beams, cantilevers and slabs — 16,000 lbs. per sq. inch.

Bond stress between 1 : 2 : 4 concrete and Steel — 100 lbs. per sq. inch but when the reinforcement is in the form of plain bars used to resist tensile stresses induced by bending the bond stress calculated from the eqation —

$$\text{Bond stress} = \frac{\text{Total shear of section}}{\text{arm of moment of resistance} \times \text{perimeter of bars.}}$$

shall not exceed 200 lbs. per sq. inch.

Average pressure intensity on foundation soil at R. L. (—) 35.00 is 7 tons per sq. ft.

The latter may be increased for greater and shall be decreased for lesser, depths of foundations at the rate of 0.05 ton per sq. ft. for every foot of difference in the depth.

No reliance shall be placed on the effectiveness of skin friction to decrease pressure. The pressure intensities over foundations soil so arrived at shall not at any point be exceeded by more than 25% under the worst combination of loads.

Pressure on soil in foundation of abutments—1 ton per sq. ft.

Pillars marking the exact line finally adopted for the bridge shall be erected about 60 ft. beyond the abutments at each end of the bridge to enable a check being maintained on the centre line throughout the execution. A sufficient number of stable masonry bench marks and other reference marks shall also be provided at suitable locations to enable a complete check being maintained on the correctness of various levels and positions. The exact positions of the abutments, piers and foundations shall be most accurately determined and adhered to using the best steel tapes and other suitably accurate survey instruments.

FOUNDATIONS.

1. General Design :— Each pier shall be founded on twin wells built over reinforced concrete well curbs sunk to approved levels vide bridge plan. In the event of these levels being varied under the orders of the Chief Engineer, plus or minus adjusment shall be made from the contract price as may be necessary at the time of payment. Foundations for abutments shall be executed in open digging and taken down to levels indicated in the specification.

The design of the curbs, the diameter of the wells, the thickness of the steining and its reinforcement shall be adequate to ensure trouble free sinking without much use of kentledge and to safely withstand stresses that may come upon the steining as also upon the base of the foundations under the worst combination of loads during and after execution of work. The design shall also be adequate to allow for accidental accentricity and shocks during sinking.

The diameters of wells, and thickness of steining shown in the plan shall be taken to be provisional but minimum, they shall be suitably increased if found necessary on detailed examination or to suit materials employed or methods of working and no extra payment will be due on this account. All vertical reinforcement in the steining shall be properly connected into the body of the reinforced concrete curb at the bottom.

2. Well curbs :— Shall consist of suitably designed reinforced concrete 1:2:4 with strong steel cutting edges efficiently anchored to the body of the reinforced concrete. Rods anchored in the body of the curb shall be left projecting upwards to form the vertical reinforcement of the steining. The amount of reinforcement in the reinforced concrete well curb covered by the contract is not exceeding 1½%. The designs shall be suitable for the diameter and width of steining adopted for the wells and shall be otherwise sufficiently rigid and appropriate to enable trouble free placing and sinking.

3. Steining of the wells:—shall consist of either of the following materials at the option of the contractor suitably reinforced with steel bars not exceeding ¼ % of the steining by volume :

(a) 1 : 3 : 6 Cement concrete. Round plums each not exceeding ½ cubic foot shall be permitted in steining of the wells 3′ or more in thickness provided that no plum comes closer than 9″ clear to any face and 6″ clear to any other plum. No plums shall be allowed in steinings less than 3 ft. in thickness.

(b) First class burnt bricks laid in cement mortar 1 : 6 or best kankar lime mortar 1 : 2.

(c) Coursed rubble stone masonry first sort in cement mortar 1 : 6 or best kankar lime mortar 1 : 2. The minimum thickness for cement concrete shall be 8′ and may have to be greater for other materials.

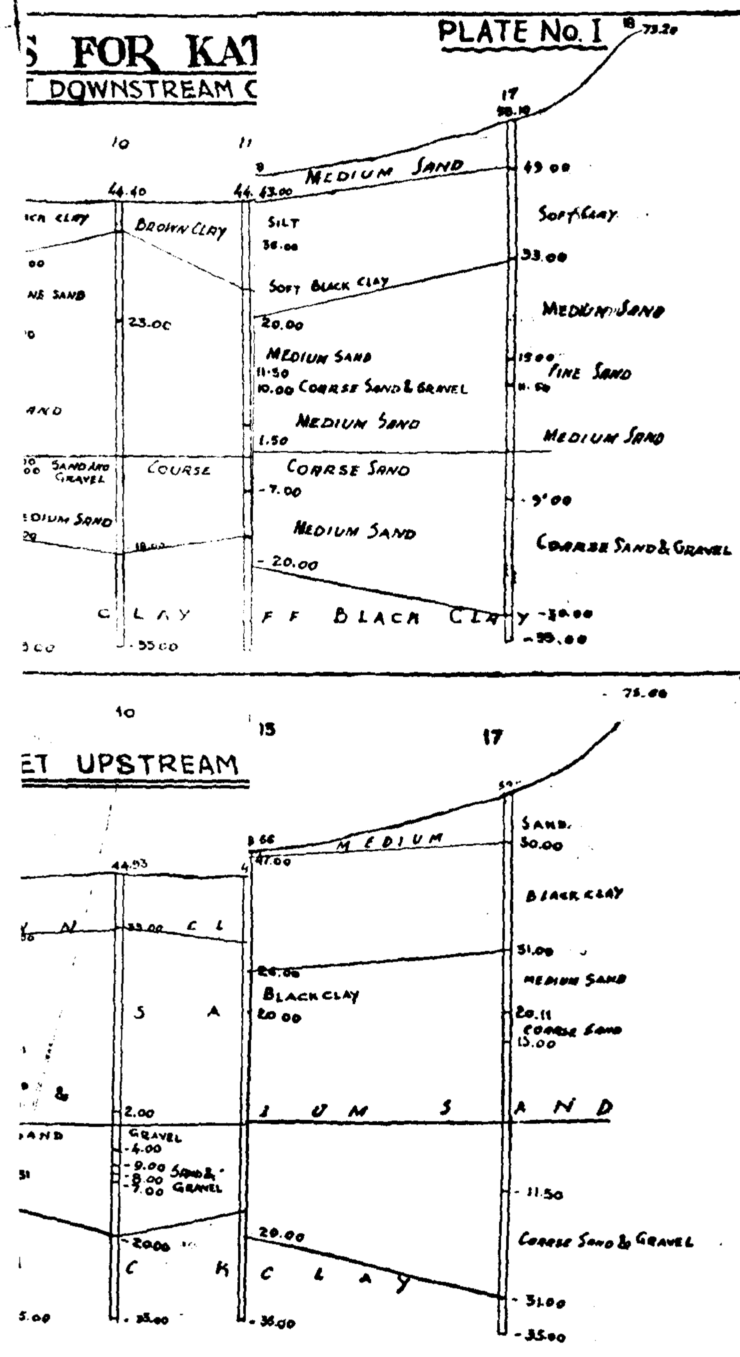
In case of use of masonry steinings, the mortar sorrounding the steel reinforcement in the steining to a minimum distance of ¾ inch shall be cement mortar not poorer than 1 : 3.

The actual design adopted shall in every case be suitable for the material used.

4. Construction and sinking of wells.

The twin curb shall be placed or cast truly in position after excavating in the open as far as it is practicable to do so. The steining shall then be built upon it by stages to convenient levels and the twin well sunk true and vertical in the usual manner with the help of dredgers and other plant. Any tendency to a departure from the vertical or true position during sinking should be immediately investigated and corrected with the help of divers and / or special plant where necessary. When the twin well has reached its predetermined bottom level (or earlier in the case of unforeseen happenings), the strata pierced and the behaviour of the well during sinking shall be reported to the Chief Engineer for orders whether it is to be sunk further or treated otherwise. It may be necessary on occasions to test the bearing capacity in cases where the predetermined depth is not reached, by loading the well to the design load plus additional load to compensate for the frictional resistance of the river bed material upto 3 Cwt. per sq. ft. of well surface upto the point of estimated maximum scour. The Chief Engineer may at his discretion prescribe an alternative method of testing. When the twin well has been sunk to the full required depth and has been passed as true and suitable, the bottom shall be cleared up and levelled as far as practicable and plugged with 1 : 3 : 6 cement concrete to a minimum thickness of 4 ft. and in any case sufficient to ensure that the top of the plug is at least 1 ft. above the top of the curb so as to cover the entire curb portion and a portion of the base of the steining. This concrete shall be laid in still water by lowering the mixed concrete in a specially designed water-tight emptying bucket. After the plug has set, the twin well shall be filled up with clean sand rammed, upto the top level of the steining less the thickness of the top plugs which latter shall not be less than one foot thick and shall consist of 1 : 3 : 6

cement concrete. The twin well of each pier shall then be completely covered by a properly designed reinforced concrete 1 : 2 : 4 cap slab of adequate thickness with reinforcement upto 2 %. The foundation of the abutments shall be not less than 2 ft. in thickness and shall consist of lime concrete, or cement concrete 1 : 4 : 8.



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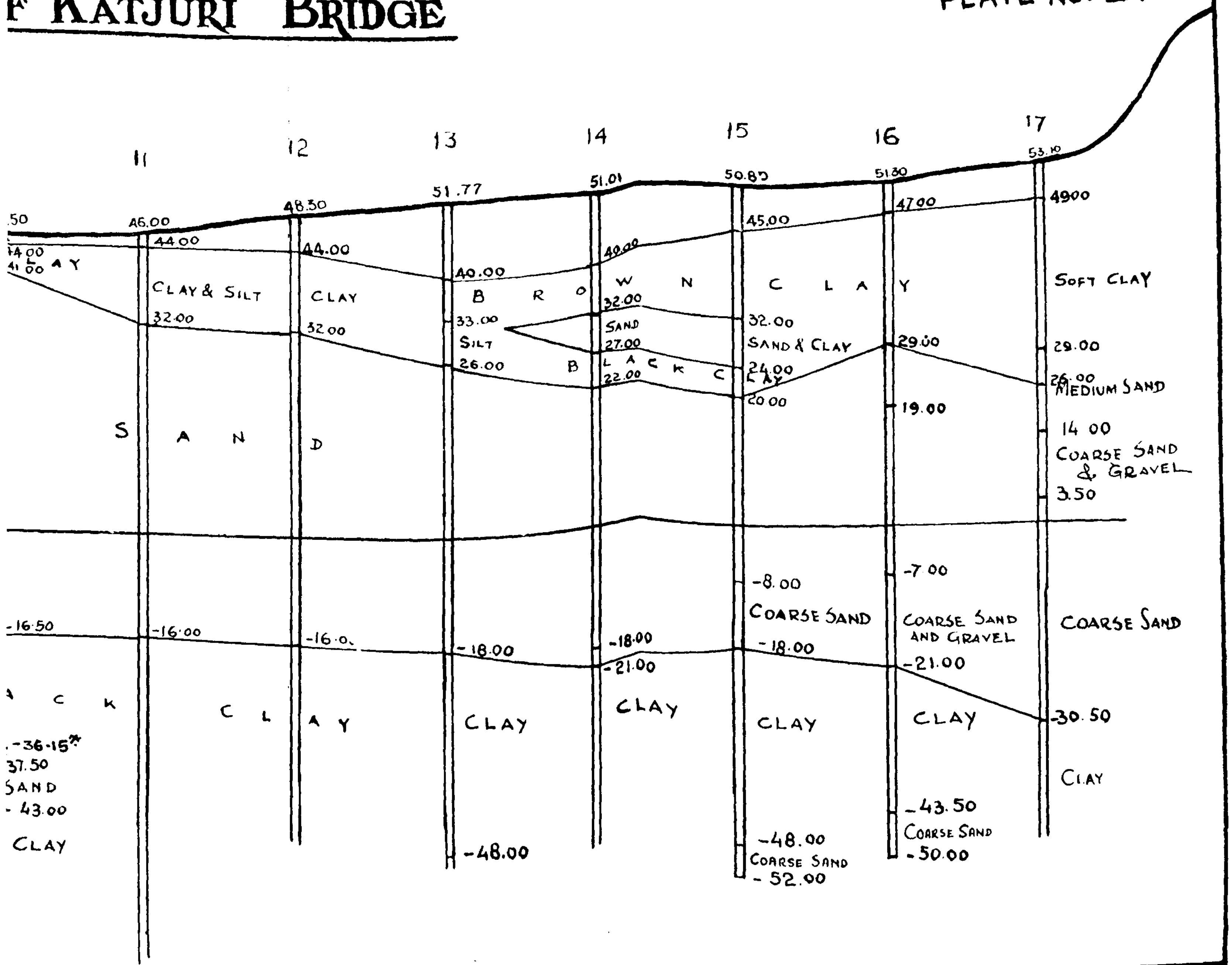

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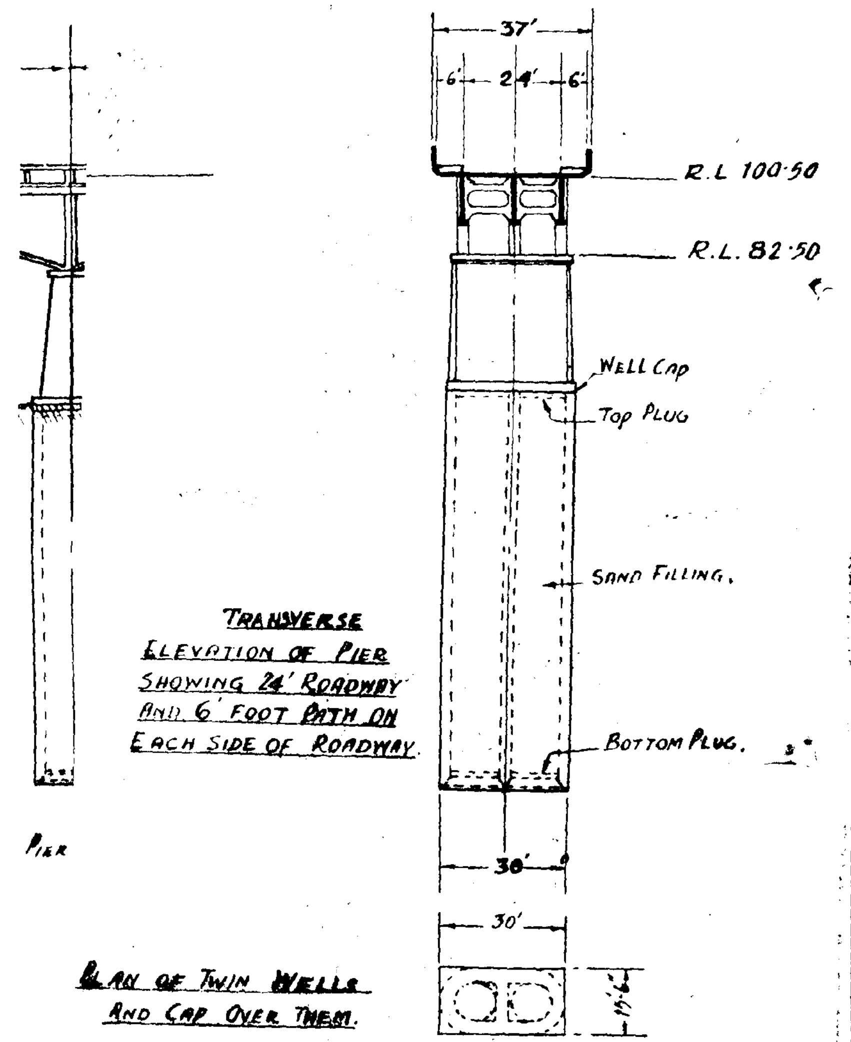


TRANSVERSE ELEVATION OF PIER SHOWING 24' ROADWAY AND 6' FOOT PATH ON EACH SIDE OF ROADWAY.

PLAN OF TWIN WELLS AND CAP OVER THEM.

LIA

Paper No. 145.

"Stability of Densified Soil Under Adverse Moisture Conditions."*

By Sachindra Nath Gupta.

TABLE OF CONTENTS

1. FOREWORD

The Author had the opportunity, under the auspices of the University of Utah, U. S. A. in 1947-48, of undertaking an original study of the effects of excessive moisture on the stability of compacted soil. The research was conducted on two different soil samples of different classifications. Four specimens of each sample were prepared with varying compactive efforts, the moisture content in each case being the corresponding optimum, and were subjected to continuous soaking simulating field conditions. This Paper presents a brief description of the investigation.

*Received for publication on 26-9-49—Secy. I. R. C.

2. INTRODUCTION

2.1. Only twenty years ago, the profession of civil engineering had but vague ideas on the behaviour of soils. Many failures were attributed to "poor subgrade" without entering into details as to what was the real cause of failure.

2.2. The investigations at that time had the character of a professional adventure with doubtful prospects of success. After years of research it became obvious that

(a) earth is never uniform

(b) its properties are too complex for rigorous theoretical treatment, and

(c) a mathematical solution is extremely difficult.

2.3. Extensive research developed in this field a few years later, when two new developments were added –

(i) A fixed classification schedule and a test procedure to classify soils were adopted, and

(ii) a mathematical theory of stability with some conception of granular soil, cohesive soil, and moisture content of soil, was established.

2.4. Almost simultaneously, to be precise in 1933, R.R. Proctor of Los Angeles created a landmark by furnishing specific relations between moisture content and density of soil.

2.5. The California Division of Highways, who earlier developed their own method and equipment for determining the optimum moisture content for optimum compaction, adopted standard soil control procedures correlating soil compaction, bearing ratio, and expansion tests, to an optimum limit which could be simulated in actual field operations within the range of economic expenditure and practicability.

3. STABILISATION OF SOIL DEFINED AND DISCUSSED

3.1. Soil stabilisation, though the term is rather loosely used, now covers the entire field from consolidation of clay and adobe soils at optimum moisture without admixture to the latest developments in emulsified asphalt and soil cement mixtures.

3.2. Soil stabilisation, as defined by C. A. Hogentogler formerly of the U. S. Bureau of Public Roads, is "The process

of giving natural soils enough abrasive resistance and shear strength to accommodate traffic or loads under prevalent weather conditions without detrimental deformation. The essential consideration in stabilisation is to provide the combination of internal friction and cohesion required to furnish the soil with high shearing strength. It is well known that the denser the soil, the greater is its stability. The methods employed include the use of admixtures, compaction, and laboratory control. Optimum water content is fundamental with gradation. Admixtures may be soil materials, deliquiscent chemicals, solutions of electrolytes, soluble cementitious chemicals, primers and neutralisers, and insoluble binders."

3.3. The essential features of stabilisation include prevention of clay, silt, and loamy soils becoming detrimentally wet, incorporation of granular materials in clay soils, addition of cohesive binder to granular soils, or combination of one or more of these.

3.4. There are different methods of accomplishing stabilisation of soil which are classified as follows :—

(1) Mechanical methods viz., compaction or densification.

(2) Stabilisation by inert or chemical admixtures, or cementitious materials.

3.5. The difference between compaction and densification lies in the control of the soil water. In the former, neither the thickness of layers nor the moisture content is critical, the density produced being qualitative. In the latter the moisture content is critical and the corresponding density is quantitative in character.

3.6. The stability of densified soil water mixture plays a very important role in the design of low cost flexible pavements. Irrespective of the combined or individual strength of subgrade, sub-base, and base, at the time of construction, its future value depends on the amount of water that can enter the various layers when subjected to inundation.

3.7. It is a known fact that, within certain limits of plasticity, the strength of a plastic soil mixture varies inversely as its moisture content. There are some soils which would be satisfactory if compacted by using more compactive effort than ordinarily applied. Local conditions will determine whether a soil requiring excessive compactive effort is more economical than one which compacts readily but costs more to procure.

4. THEORY OF COMPACTION

4.1. Three theories advanced so far to explain the soil compaction phenomenon are as follows:—

(a) **Proctor's theory or Lubrication Theory.**
(b) Theory of Moisture Films.
(c) Surface Tension Theory.

Mechanics of soil water mixture is fundamental in most of these theories.

4.2. The most plausible theory outlined by R. R. Proctor[1] says that the ordinary soil mass is composed of gravels, sands, silts, and clays, and compaction is the act of forcing the fine grains into the voids between the larger grains. Proctor contends that water coats the surface of soil grains and serves as a lubricant, reducing the resistance due to internal friction between the soil grains.

4.3. As the moisture content is increased, a point will be reached where the compactive force will overcome this resistance and will cause the fines to be forced into the voids between the larger grains, and the maximum density will be obtained. This is the optimum density at the optimum moisture content, which is the lubrication limit, under a particular compactive effort.

4.4. As moisture is increased beyond this point, the compactive force compresses the small volume of entrapped air in the soil mass. This compression gives rise to hydrostatic pressure, which tends to separate the soil grains by partial flotation, thereby causing a reduction in density.

4.5. The second theory, advanced by C. A. Hogentogler Jr[2]., indicates that, if the moisture content was low and the moisture films very thin, there would be high resistance between the soil grains as the thin films are extremely cohesive. As the moisture is increased, these films would become thicker and less cohesive, enabling them to serve as lubricants. Near the maximum density, these films reach a thickness which gives a cohesive strength at the points of contact that just fails to balance the compacting force. This allows the compacting force to arrange the fines efficiently into the voids of the larger grains and results in maximum density.

4.6. As the optimum moisture is exceeded, the increase in film thickness causes a corresponding increase in the separation of the soil grains and, consequently a decrease in density. Also, as the films thicken, their strength is decreased, resulting in decreased

[1]. R. R. Proctor - "Fundamental Principles of Soil Compaction." *Engineering News Record.* Vol. 111, 1933.

[2]. C. A. Hogentogler Jr. "Essentials of soil compaction"—Proceedings Highway Research Board, Vol. 16, pp. 309—1936.

stability of the compacted mass, which is reflected by the rapidly decreasing penetration resistance.

4.7. The third theory is based on that property of soil moisture known as surface tension. This theory contends that the moisture throughout the soil mass creates surface tensions which aid the compacting force in pressing the soil grains into closer contact. It is the conventional explanation of the shrinkage of fine-grained soils and also the explanation of their cohesiveness.

5. THE REASONS FOR COMPACTION AND ITS EFFECTS

5.1. There are three basic reasons why a soil in a subgrade or in an embankment is compacted :—

(a) Reduction in permeability,
(b) Reduction of settlements,
(c) Increase in stability or shearing strength.

5.2. The most logical deduction with regard to the benefits of compaction is that, with increased density, a soil will have less water content, and this is desirable if a soil is subject to saturation after compaction.

5.3. On bearing tests it has been found that, by compacting a subgrade, the bearing capacity may be increased as much as 150 per cent in some cases, which is certainly a strong point in favour of compaction.

6. EFFECTS OF MOISTURE ON COMPACTION

6.1. The frictional resistance between soil particles determines the plasticity of a soil. The addition of water to a very dry soil reduces the capillary force, and this has two effects: (1) the particles spring apart slightly and (2) the frictional force between them is reduced. The soil is then more plastic and has expanded slightly in volume. This effect continues with the addition of water to the soil until the forces of capillarity are almost entirely released and no further soil expansion occurs. The limit of lubrication is reached when the voids become filled with water.

6.2. The most important principle of compaction is the effect of moisture content of a soil upon the density to which it may be compacted. For a given method of compaction of a soil, a very low moisture content may result in a hard fill having practically no plasticity, while a slightly higher moisture content produces a soil of greater density that contains less voids and is more plastic. This effect continues until the moisture content, with a small amount of contained air that the compaction process cannot remove, becomes just sufficient to fill the voids when the compaction process is completed. A higher moisture content limits the compaction of the soil to a point at which the voids equal the volume of the contained air and water resulting in a compacted soil with more voids, less density, and increased plasticity. This effect continues with the increase in moisture until the soil becomes too plastic to sustain compactive equipment.

6.3. In order to study the effect of moisture on the compaction of soil, let a typical density moisure curve be examined. (Fig. 1). A soil compacted at six per cent moisture will, to all appearances, be an extremely stable compacted mass. However, in this condition, as may be seen from the figure, the mass has about twenty-six per cent voids by volume and can take on water up to about 13.2 per cent moisture by weight, when it becomes saturated. At 13.2 per cent moisture, the material would be relatively unstable, as indicated by a penetration resistance of practically zero.

6.4. The soil compacted at optimum moisture (8.5 per cent) may be compared. This soil is not nearly so firm as the one compacted dry as indicated by the penetration resistance of about 2,350 lbs per sq. inch. But it can take on water up to about 10.7 per cent only. At this saturated moisture, the indicated penetration resistance is about 330 lbs per sq. inch, or some 33 times firmer than the same soil compacted dry.

6.5. This comparison shows how variations in placement moisture affect the predicted stability at saturation This analysis is largely synthetic in nature and may not be strictly true, as the materials in a structure would consolidate to some extent as construction progresses. Consequently even before the structure is saturated, the consolidation of the soils especially those at the optimum moisture or above, probably would increase in density to such an extent that the final (saturated) moisture would be limited to even less than that indicated. Materials placed dry and hard have been observed to settle very little under average load conditions and take their settlement as they are saturated On the other hand the settlement of material at optimum moisture is not appreciably affected by the saturating water.

6.6. Observations made both in the laboratory and the field show that materials placed dry show much more final settlement than materials placed at the optimum density and moisture. The settlement curve has about the same shape as the compaction curve and shows the minimum settlement to occur at nearly the optimum conditions.

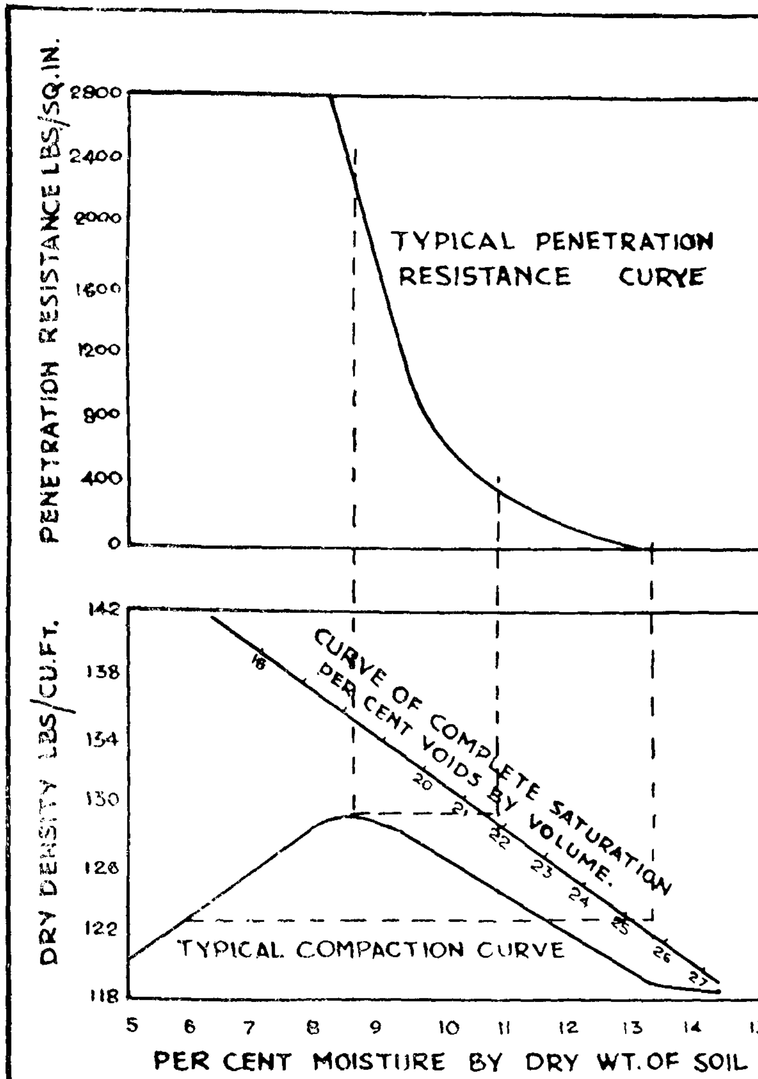


Fig.-1 EFFECTS OF MOISTURE UPON COMPACTED DENSITY.

6.7. Another important consideration of the effect of variation in placement moisture is the permeability of the soil mass. Observations have shown extremely wide variation in the permeability of the soil mass, depending upon placement moisture conditions. In general, these observations have shown that soils compacted dry have percolation rates that are many times (as much as several hundred times for some soils) the percolation rates of the same soils compacted wet. Undoubtedly this is true for a short time, but whether or not it remains true in a structure is a question that is yet to be answered. The important consideration is the rapid saturation of the structure.

7. EFFECTS OF VARIATION IN THE COMPACTIVE FORCE

7.1. The foregoing paragraphs have shown the trends in variation that are produced by changing moisture conditions assuming the compacting force to be constant. Observations have shown, as will be described later, that these trends are still relatively true as the compacting force is changed.

7.2. Many observations have been made in which the compaction has been applied in different manners. But all of them gave the same general trends. They indicated in general that –

(a) The density obtained is a function of the work applied.

(b) Within limits, the method of applying the work is not important.

(c) Within limits, increased roller weight is more effective.

(d) Increase in density for increased roller trips up to ten or twelve passes has been favourable but beyond this, comparatively small.

7.3. Experiments have established that for each compactive force there is a different optimum moisture. As the compactive force is increased, the optimum density becomes higher and the optimum moisture gets less.

7.4. The increased compaction not only gives more favourable density but gives a much more stable mass, as indicated by the penetration resistance.

7.5. Variations in moisture content will occur even with most efficient construction control operations. Variations from optimum moisture are relatively less harmful when heavy rollers are used. However, if light rollers are used, the moisture must be rigidly controlled to give satisfactory density. The modern trend towards adoption of heavy rolling equipment is thus explained.

7.6. Another advantage gained through increased compaction is exhibited by laboratory observations of settlement shown graphically in Fig 2. There is a significant reduction in settlement as the compaction is increased.

7.7. An appreciable decrease in permeability is observed in materials compacted with increased effort but the difference seems to be less pronounced than that noted with changes in placement moisture. Specimens compacted to higher densities with higher compactive efforts present higher penetration resistance. This seems to indicate an increase of the saturated stability. This phenomenon will be brought out in greater detail in the investigation that follows.

8. SWELL OR LOSS OF STABILITY IN WATER

8.1. If the free surface of a layer of plastic or semi-solid clay is covered with water, the clay expands. This swelling represents the elastic expansion of the clay caused by the elimination of the surface tension of the capillary water. Water enters through the free surface and flows into the interior, impelled by the hydrostatic stress difference between the surface and the interior, and so causes the increase in bulk.

8.2. If, during a compression test, the water is completely drained off the surface and the external load then removed, the volume of the clay remains unchanged, because expansion would require increase of the moisture content and there is no free water available. On the other hand, when the volume remains unchanged, equilibrium is maintained by the surface tension of the capillary water taking the place of the external load. Like a rubber skin it opposes any tendency to expansion.

8.3. The extent to which water will be absorbed depends upon both the capillary properties and the degree of cohesion possessed by the soil.

8.4. Sufficient cohesion existing between the soil particles will prevent the entrance of water in an amount sufficient to cause the soil to lose stability, unless the soil is manipulated. The amount of cohesion possessed by a soil of given constituents depends upon both the moisture content and state of compaction of the soil. Therefore, the relative amount to which the soil will expand depends upon both the degree of consolidation and the moisture content of the soil before wetting.

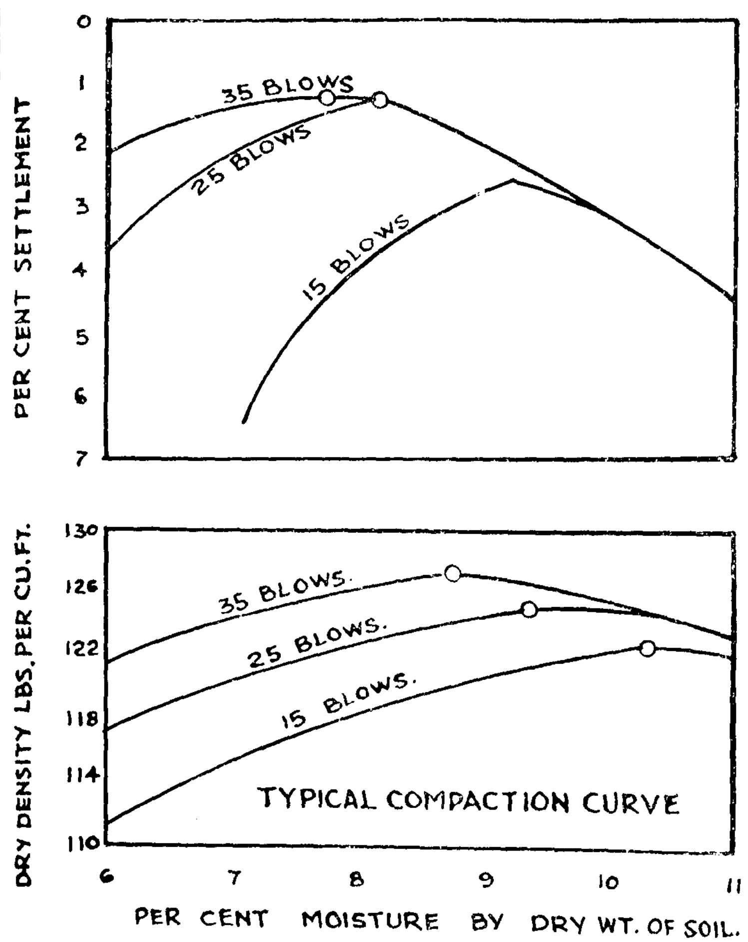


Fig.-2 EFFECTS OF AMOUNT OF COMPACTION AND MOISTURE UPON SETTLEMENTS.

8.5. In the investigation that follows soil samples were compacted with varying compactive efforts and moisture contents and thereafter subjected to continuous soaking to observe loss of stability in each case.

9. PREVIOUS WORK IN THE FIELD

9.1. In the 1930's, Proctor carried out laboratory compaction tests on more than two hundred different soils, each of which was compacted at several moisture contents by the process which has now been standardised as the Proctor Procedure.

9.2. This procedure was repeated with all the specimens with increasing moisture contents until there was no longer any increase in density of the compacted mass. This gave the condition of maximum density under this particular compactive effort and the moisture content at which this density occurred was called the "Optimum Moisture Content." In order to obtain a complete moisture-density relationship curve beyond the optimum density and optimum moisture, the test was repeated at least two times more with increasing moisture content beyond the optimum.

9.3. The specific gravity of the soil particles was determined in order to establish the position of the zero air-voids curve. The method of obtaining this was important, due to its effect on the calculation of the quantity of water that the compacted soil might absorb.

9.4. The maximum unit weight obtained by the Proctor procedure at optimum water content is maximum for that compactive effort only. It is well-known that the compacted unit weight obtainable at a given water content can be varied considerably by changing the number of blows, or the thickness of the layers, or the rigidity of the support upon which the compaction is performed.

9.5. The effect of the support on which the compaction is performed has an important bearing on the accuracy of the results obtained. In the engineering laboratory of the National Park Service (U. S. A.), considerable variation has been found in the compacted unit weight depending on whether the compaction cylinder rests on the concrete floor, over the leg of a table, or in the centre of the table. If a similar support were used by all laboratories results would be much more comparable.

9.6. The California Bearing Ratio tests were confined to determining the optimum moisture and maximum consolidation obtainable under a load of 2,000 lbs. per square inch and need not be discussed here.

9.7. The Research Division of the Texas Highway Department, in co-operation with the Engineering Experiment Station of the Agricultural and Mechanical College of Texas, has made extensive laboratory investigation on what actually happens to compacted soils of different types under different conditions present in a highway subgrade. One conclusion arrived at was stated as follows:—

"Within certain limits, reduction in the moisture content of clay and increase in the compacting force applied, increase the compressive strength and points are reached where further changes in those directions cause a reduction in the compressive strength of the soil structure."

9.8. With the objective of investigating the possibilities of obtaining correlating information on density, strength and moisture content of soils, Richard L. Sloane, conducted comprehensive tests at the Engineering Experiment Station, University of Utah during 1946.

9.9. The four soil samples used in the investigation were classified as A-4, A-2-4, A-3, and A-4-6 (Appendix I) on the basis of liquid limit, plastic limit, mechanical analysis, and capillary potential test results. The Proctor and the California Bearing Ratio tests were also performed for the purpose of correlation of the different controlling factors. The major portion of research was devoted to investigating, for each soil, the density and strength relationships through the workable moisture range (dust to mud extremes) and through a wide range of varying compactive efforts, starting with twenty-five blows per layer, the Proctor standard, and increasing in uniform increments to as much as eight hundred blows per layer as dictated by information gained in the previous experiments.

9.10. The experimental works done at the University of Utah taken as a whole, lead to the following conclusions :—

1. The Proctor and the C. B. R. methods of compaction do not reach the ultimate obtainable density. Possibilities for higher bearing power and density still exist.

2. Higher "Ultimate Density" for a given soil can be obtained with optimum compactive effort and at an optimum moisture content corresponding to that compactive effort.

3. With the "Ultimate Density", bearing power of the magnitude of six or seven times that which ordinarily would be obtained will result.

10. THIS INVESTIGATION — SCOPE

10.1. The foregoing discussions, and more particularly the University of Utah Research, have shown that a density higher than that obtained in Proctor or C. B. R. methods can be obtained. This consideration seems to be useful in design and construction of highway and airport subgrades. It has, however, to be seen whether this higher compactive effort can be developed economically with the existing or future highway equipment. Moreover, the knowledge of the maximum strength or density, as obtained through initial compaction, is not adequate for final adoption in design because it is a known fact that the initial density and strength will be adversely affected by the presence of excess moisture caused by seepage of surface water, or capillarity of underground water, or by inundation. It is a matter of further investigation to determine the actual nature and extent of the adverse effect caused by excessive moisture.

10.2. This investigation attempted to throw some light on the behaviour of compacted soil under excessive moisture. It was contemplated to test two different samples of soil, making four specimens from each, compacted in the 1/30 cubic foot mould under varying static loads, ranging from Proctor to C. B. R. limits, in equal increments with the corresponding theoretical optimum moisture content in each case. The compressive strength would be determined before and after the specimens were subjected to continuous soaking till saturation. An attempt was then made to correlate the compactive effort, and loss of stability due to swelling. It was thought possible to throw some light on the following questions :—

(a) What is the nature and extent of swell in a soil sample under continuous soaking?

(b) In what way is the swell a function of compaction?

(c) Is swell a function of the initial moisture content of the specimen and if so, how?

(d) Is swell a function of time and if so to what extent?

(e) What relationship is there between the loss of stability under water and the initial compactive effect?

(f) What should be the optimum compaction for a given soil.

10.3. In order to reply these questions, the laboratory tests should be conducted under conditions that would closely simulate the field conditions. With that objective the following assumptions were made :—

(a) The subgrade could be initially compacted to predetermined optimum density with optimum compactive effort.

(b) Soil within the subgrade is supported on all sides except at the top and, therefore, resembles confined soil.

(c) The subgrade soil sustains a uniform load represented by the base courses and the pavement above.

(d) The subgrade is saturated with surface or ground water, which takes into account the worst condition for design purposes.

10.4. The field conditions were simulated in the laboratory in this investigation in the following way :—

(a) The sample was compacted under a varying static load in the compression machine.

(b) A surcharge load equivalent to the weight of an eight-inch pavement was placed on top of the compacted specimen while soaking.

(c) The specimen with surcharge weight confined in the mould was surrounded with water to observe the volumetric change and loss of compressive strength.

11. SAMPLES TESTED

11.1. The investigation was conducted with two different soil samples marked S-I and S-II. Sample S-I was obtained through the State Road Commission of Utah and S-II from the excavations of the new Book Store for the University of Utah.

11.2. Both the samples were pulverised, care being taken not to crush the individual soil particles, and then allowed to dry at the room temperature. Classification tests were run on both the samples. The soil S-I belonged to P. R.A. classification A-2 whilst the second sample fell into classification A-2-4. A summary report of the classification is given in Appendix II. The data and results of different classification tests can also be found in Appendix II. Sample S-I was a poorly graded, plastic, sandy soil having 9·6 per cent gravel and a very small percentage of silt and clay in it, while sample S-II was a plastic loam with 2·98 per cent gravel.

11.3. The hygroscopic moisture of both the samples was determined prior to the compaction and swell tests in each case. This determination was important in as much as, while preparing the samples at optimum moisture content for each compactive effort, the hygroscopic moisture had to be taken into account because of the fact that the optimum moisture content lay in a critical range and too much variation might have indicated results least expected.

12. APPARATUS USED

12.1. So far as the classification tests were concerned, the standard apparatus was used for the main investigating tests. In making specimens with varying compactive efforts at their corresponding optimum moisture contents, moulds similar to the Proctor mould were used. These moulds were also used in the University of Utah Research described in the foregoing paragraphs. A descriptive section of this mould is given in Fig. 3. The main cylinder, made of black steel, has an internal diameter of four inches and an inside height of 4.59 inches. The cylinder can be securely fixed to a base plate six inches square by means of two bolts and nuts passing through two angle iron cleats welded on to the cylinder opposite each other.

12.2. The collar, 2¼ inches high and with 4 in. internal diameter, is fastened to the main cylinder on to a V-notch by means of three thumb screws passing through three vertical flat straps spaced at 120 degrees, welded to the collar. This is very important so as to prevent displacement or pushing up of the collar under the pressure of the compacting machine or due to swell.

12.3. A lead surcharge of ten pounds to fit inside the collar, forms a part of this equipment and simulates the weight of an eight-inch pavement on a subgrade. This load measures 3.9 inches in diameter and 2.25 inches high in the form of a solid cylinder. A stud, having a circular head, is permanently seated at the center of the surcharge load, to provide a bearing surface for the extensometer rod. To allow access of water above the sample and below the surcharge weight, 1/16 inch by 1/16 inch slots are cut 1/2 inch apart on the under-surface of the load.

12.4. The extensometer dial to read the swell is fitted to a device which can be seated over the cylinder. This device consists of a dural plate with the three steel posts half notched and spaced equidistant from each other. The whole assembly of the apparatus is shown in Photograph No. 1

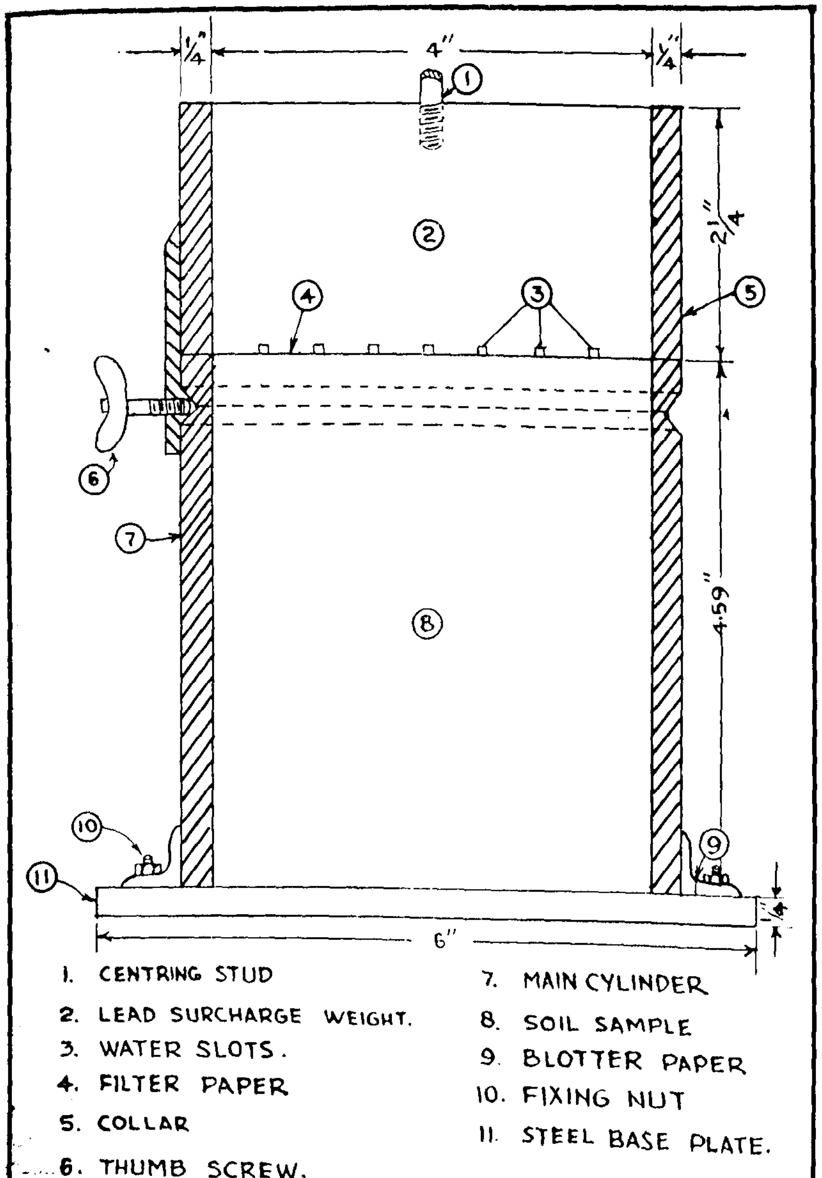
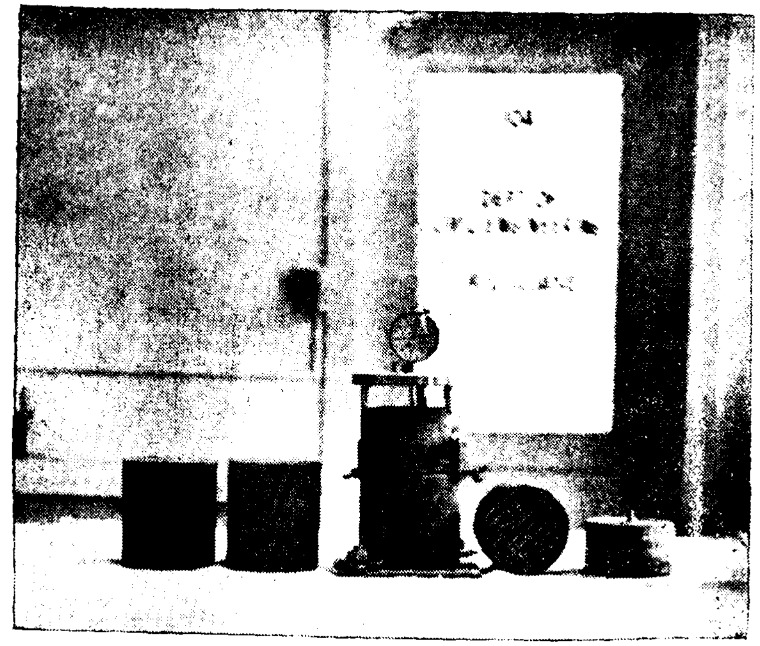


Fig-3 DESCRIPTIVE SECTION OF CYLINDER.



Photograph 1. **Apparatus For Soaking Test. Extruded Samples, extensometer reading assembly. Surcharge weights.**

13. METHOD OF INVESTIGATION

13.1. The main investigating tests were conducted on eight specimens, four from each sample. The procedure was exactly the same for either sample. This was as follows :

13.2. After the classification tests had been run, the sample of minus 4 size particles was tested for optimum moisture content under the Proctor compacting effort of twenty-five blows from a rammer weighing 5½ pounds with a drop of twelve inches in each of the three layers to fill the Proctor mould. Instead of using the hand rammer originally used by Proctor, the automatic tamper as shown in Photograph No. 2 was used.

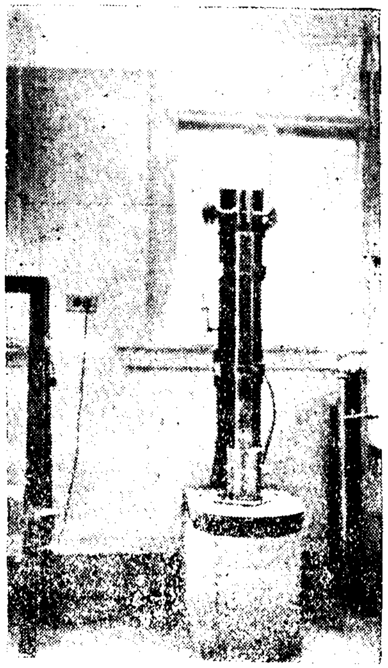
13.3. When the Proctor optimum moisture had been determined, it was time to run the main tests of this investigation. It was decided to prepare four specimens from each sample compacted with different compactive efforts varying from Proctor to C. B. R. In order to do that, the following assumptions were made :—

(a) Proctor compactive effort is equivalent to a static load of five hundred pounds per sq. inch.

(b) C. B. R. compactive effort is equivalent to a static load of two thousand pounds per sq. inch.

(c) For Proctor optimum moisture content less than fifteen per cent, the corresponding C. B. R. optimum for the same soil is 1.5 per cent less, and for a Proctor optimum above fifteen per cent, the corresponding C. B. R. optimum is two per cent less.

(d) The optimum moisture varies uniformly between these two limits with varying compactive efforts but in inverse proportion.

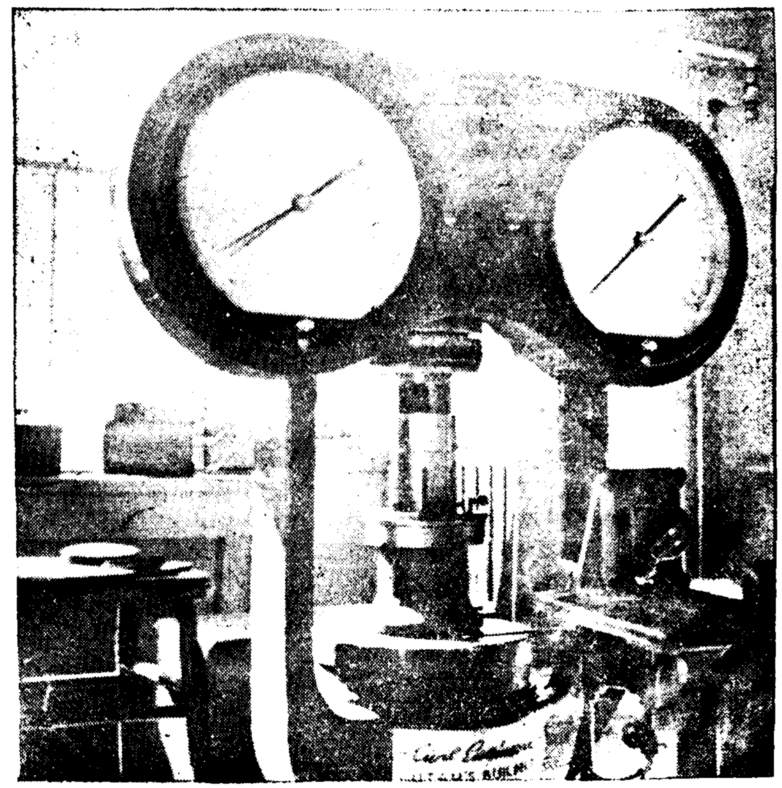


Photograph 2. **Automatic Tamper.**

13.4. Four specimens were made from each sample compacted at two thousand, fifteen hundred, one thousand, and five hundred pounds per square inch, setting the sample in each case at the corresponding optimum moisture content calculated on the basis

of the above assumptions and taking into account the hygroscopic moisture present in the sample and the moisture that may be lost by evaporation at the time of mixing.

13.5. A 2500 gram sample was weighed to an accuracy of one gram in a shallow mixing pan. The requisite quantity of water, to make up the optimum moisture content after it was mixed, was then mixed thoroughly with the soil sample and tampered for a few minutes under cover of a damp cloth to prevent evaporation. The soil was then tamped into the tared mould and then compacted to maximum dry weight per cubic foot under the four varying loads of five hundred, one thousand, fifteen hundred, and two thousand pounds per square inch through the hydraulic compression machine shown in Photograph No. 3.



Photograph 3. **Hydraulic Compression Machine Compacting Soil in Mould.**

13.6. In applying the load, the head of the compression machine was operated at the rate of 0.05 inch per minute The maximum compacting load in each case was maintained on the sample for one minute and then released.

13.7. The mould was then removed from the testing machine and, after removing the detachable collar, the excess height of the specimen above the six inch mould was scraped off. The mould with specimen was weighed to find out the weight of the compacted soil specimen.

13.8. The specimen was extruded from the mould and tested in the compression machine for the ultimate crushing strength. Samples from the specimens were saved, dried in an oven and weighed before and after drying to find out the actual moisture content after compaction. This also gave the dry density of the specimens.

13.9. After having determined the crushing strengths of the four compacted specimens of the same sample, the process was repeated to produce four similar compacted specimens again. This time, however, a thick blotting paper was placed over the base plate and below the cylinder to facilitate percolation of water while the specimens were placed in water.

13.10. After the excess earth above six inches of the cylinder was struck off and the whole thing weighed, a filter paper was placed on the top of the specimen and the collar refitted in position. The load surcharge of ten pounds was softly put on the filter paper and the initial extensometer reading was taken and recorded. The cylinder with the specimen was then gently placed in a pan of water, keeping the water level about one inch above the specimen as shown in Photograph No. 4.

13.11. Extensometer readings were taken every day to measure the swell or increase in volume till the extensometer recorded on further increase. The cylinders were taken out one by one, surface dried, and then weighed to find out the moisture absorbed. The specimens were then extruded from the mould and tested at once in the compression machine for ultimate crushing strength. Samples of specimens were saved to find out the moisture content by drying.

14. RESULTS OBTAINED

14.1. Samples of the data of all the tests are included in the appendices. Appendix II covers the classification tests of both the samples while Appendix III covers the main investigating tests starting from the Proctor test for determining the optimum

Photograph 4. Specimen in Soaking Pan.

moisture content at twenty-five blows. For facility of reading the results at a glance, these have been entered in Tables I and II in the following pages.

14.2. For facility of discussion, the following graphs have been drawn with the help of the data obtained.

Figs. 4 and 5–Cumulative expansions in per cent against time in days. (Plate I)

Figs. 6 and 7–Maximum swell and optimum density against different compacting efforts. (Plate I)

Figs. 8 and 9-Crushing loads in tons per square foot before and after swell against the corresponding compactive efforts. (Plate II)

Fig 10 – Loss of stability due to soaking against different compactive efforts. (Plate II)

Fig. 11 – Percentages of water absorbed before and after saturation against different compactive efforts. (Plate II)

14.3. The loss of stability has been computed and shown in Tables III and IV.

TABLE I

SUMMARY OF RESULTS

Sample Number	S-I
Class of Soil	Sandy Soil
Appearance	Grayish Brown
Classification (PRA)	A-2
Proctor Optimum Moisture, per cent	15.2

Cylinder Number	I	II	III	IV
Compactive Effort, /Sq. In.	2000	1500	1000	500
Initial Moisture, per cent	13.0	14.0	14.6	15.0
Achieved Dry Density, /Cu. Ft.	124.4	122.2	120.1	118.4
Density after Swell	120.4	120.1	118.8	118.0
Lineal Swell in inches	0.0537	0.0435	0.0315	0.0317
Per Cent Volume Increase	1.172	0.950	0.686	0.691
Total Water Absorbed, gms.	14	11	12	25
Time of Saturation, Days	21	24	24	24
Final Moisture, % Dry Weight	16.0	15.9	16.0	15.5
Ultimate Bearing Stress before Soaking, Tons/Sq. Ft.	8.02	6.90	6·03	4.87
Ultimate Bearing Stress after Soaking, Tons/Sq. Ft.	5.44	5.16	4.58	3.73

TABLE II

SUMMARY OF RESULTS

Sample Number	S-II
Class of Soil	Loam
Appearance	Brownish Gray
Classification (PRA)	A-2-4
Proctor Optimum Moisture, per cent	17.5

Cylinder Number	I	II	III	IV
Compactive Effort, /Sq. In.	*2000*	*1500*	*1000*	*500*
Initial Moisture, per cent	15.0	15.6	16.2	17.2
Achieved Dry Density, /Cu. Ft.	120.3	118.2	116.5	114.1
Density after Swell	116.8	116.2	114.8	112.2
Per Cent Volume Increase	1.525	1.192	0.567	0.538
Total Water Absorbed, gms.	20	17	12	13
Time of Saturation, Days	17	17	16	16
Final Moisture, % Dry Weight	16.9	17.5	17.7	18.2
Ultimate Bearing Stress Before Soaking, Tons/Sq. Ft.	10.75	9.26	6.59	5.27
Ultimate bearing Stress after Soaking, Tons/Sq. Ft.	5.44	4.87	4.58	3.73

TABLE III

LOSS OF STABILITY AFTER SOAKING

Sample No. S-I

Area of Specimen 12.57 Sq. In.

Volume of Specimen 0.335 Cu. Ft.

Compactive Effort /Sq. In.	Set Up At Moisture per cent	Crushing Load before Soaking		Crushing Load after Swell		Loss of Stability	
		Lbs.	Tons/ Ft.^{2}	Lbs.	Tons/ Ft.^{2}	Tons/ Ft.^{2}	per cent
2000	13.0	1400	8.02	950	5.44	2.58	31.4
1500	14.0	1200	6.90	900	5.16	1.74	25.2
1000	14.6	1050	6.03	800	4.58	1.45	24.5
500	15.0	850	4.87	650	3.73	1.14	29.5

TABLE IV

LOSS OF STABILITY AFTER SOAKING

Sample No S–II

Area of Specimen – 12.57 Sq. In.

Volume of Specimen – 0.335 Cu. Ft.

Compactive Effort per Sq. Inch	Set up at Moisture per cent	Crushing Load before soaking		Crushing Load after Swell		Loss of Stability	
		Lbs.	Tons/ Ft.^{2}	Lbs.	Tons/ Ft.^{2}	Tons/ Ft.^{2}	per cent
2000	15.0	1870	10.75	950	5.44	5.31	50.4
1500	15.6	1700	9.26	850	4 87	4.39	43.8
1000	16.2	1150	6.59	800	4.58	2.01	30.5
500	17.2	920	5.27	650	3.73	1.54	29.2

15. DISCUSSION OF RESULTS

15.1. In order to bring out the full meaning and the implications of the results and the data obtained and thereby to facilitate discussion and the ultimate solution of the problem or problems, it was decided to draw a number of graphs showing the relationships between swell and time, swell and compactive effort, density (before and after swell) and compactive effort, ultimate bearing power (before and after swell) and compactive effort, and last and perhaps the most important in this investigation, the relationship between loss of stability in water and the compactive effort. As has already been stated, the moisture content was not critical in this investigation.

15.2. It is known that each compactive effort has its own optimum moisture content which produces best results under adverse conditions. With this knowledge, all the test specimens in this investigation were set up at an initial moisture equal to the optimum moisture corresponding to the respective compactive effort. It had, however, not been verified whether the Proctor compactive effort was equivalent to a static load of five hundred pounds per square inch with these two particular soil types This was, however, not very important as it was possible to evaluate the results of the four compactive efforts without any reference to the Proctor compactive effort.

15.3. What was really important was whether the figures for optimum moisture content, assumed for the different compactive efforts, were really the optimum. If not, the results were likely to vary. If the moisture had been below optimum in any one case, more water would have been absorbed resulting in more swell and apparently more loss in stability. If above optimum, the swell would be less and so also the loss in stability. It was, however, assumed with previous knowledge in the field that the optimum moistures assumed were correct within reproduceable field limits.

15.4. The results and data were, therefore, discussed keeping those possible handicaps in mind. The following points presented themselves for discussion:

(a) Amount and nature of swell or expansion under adverse moisture conditions and how far expansion was a function of time.

(b) Whether saturation moisture was a function of initial compaction and density.

(c) How dry density for varying compactive efforts was affected by saturation.

(d) Effects of saturation on the ultimate bearing stress before and after swell as compared to different compactive efforts.

15.5. After extensive consideration had been given to these aspects, the question naturally arose-- and the reply to this question was the aim of this investigation--**what was the optimum compactive effort to which one or both the soil types should be subjected in order to produce the best results so far as stability under adverse moisture condition was concerned?**

15.6. With these questions in mind, it was obvious that the factors of swell, time, density, compressive strength, and stability had to be treated separately on the basis of the information furnished by the corresponding curves.

15.7. **Expansion Data:** The volume-increase or expansion or swell which resulted when the best specimens were brought from their moulded to saturated condition by water-soaking is shown in tabular form in Appendix III This expansion was plotted against time, in number of days, for the four specimens of the same sample in Figs. 4 and 5 Plate I) In Figs 6 and 7 (Plate I) the maximum swell has been shown against the compactive efforts. Soil S-I exhibited a volume-increase of 1.17 per cent and soil S-II 1.525 per cent. All these curves indicate the same trend, that the higher the compactive effort the greater is the expansion. In order to obtain a confirmation a third test was run under identical conditions but with the same result.

15.8. This was not in actual conformity with the findings of the University of Utah Experiment Station. The reason mainly lies in the difference in conditions under which the two sets of experiments were done. First, Utah moulds had blotting paper linings inside when the specimens were compacted, which reduced the frictional resistance at the sides and did not allow any elastic rebound to develop. Secondly, Utah made a series of tests with each compactive effort with varying moisture contents so that the optimum moisture contents for each compactive effort were known. But in this investigation the optimum moisture content for a particular compactive effort was assumed and could not be verified. If it were less than the optimum, considerable difference in results would follow. Particularly, higher compactive effort with a moisture content below the optimum would result in an accumulation of stresses at the skin which would induce elastic rebound at the slightest lubrication by the soaking water and consequent release of strain.

15.9. Russell, Worsham, and Andrews, however, indicated[1] that for a given moisture content at the time of compaction, the volume-increase of embankment material, upon the addition of moisture, varies directly with its compactive density. The tests they conducted were at low range of compactive efforts and were not identical with this investigation. However, there was further scope for thorough investigation in this line.

15.10. Another indication of these test results was that the expansion occured mostly within the first week of soaking after which the percentage of expansion was very small compared to the expansion that occured in the first seven days. Expansion was maximum in the first day of the entire soaking period.

15.11. **Saturated Moisture Data**: The moisture contents of all the specimens of both the samples increased when they were saturated. This moisture content of saturated specimens is expressed as per cent of their oven dry weight in Fig. 11, (Plate) II and plotted against the compactive effort. The additional moisture absorbed was three per cent dry weight for S-I and one and nine-tenths per cent for S-II at the initial compactive effort, whereas, at the highest compactive effort, it was five-tenths per cent and one per cent in turn. The data indicates that saturated moisture content was almost the same irrespective of the moulded moisture, moulded dry density, and compactive effort.

15.12. Russell, Worsham and Andrews indicated that, for a constant weight density, the saturated moisture varies directly with the moulded moisture, the density being a direct function of the compactive effort; and for a constant moulded moisture, the saturated moisture varies inversely with the wet moulded density. The present investigation differs from this in that both moulded density and moulded moisture varied simultaneously at the range producing conditions which made the saturated moisture almost the same in each case.

15.13. This also would not seem to be in full conformity with the findings of the Utah Research. The reasons appeared to be the same as explained in para 5.8 under "Expansion Data." This was, however, an interesting find and merits further comprehensive investigation.

1. R. K. Andrews, H. W. Russell, W. B. Worsham, "Influence of Initial Moisture and Density on the Volume Change and Strength Characteristic of Two Typical Illinois Soils," *Proceedings of the Highway Research Board*, Vol. XXVI (December 1946), P. 544.

15.14. **Dry Density**: The dry densities of all the specimens of both the samples both before and after soaking have been plotted against the compactive efforts and shown in Figs. 6 and 7 (Plate I). These indicated, without exception, that the dry density was a direct function of the compactivee ffort. The Utah Research had, however, shown that there is an ultimate limit to this increase in density with the increase in compactive effort but this was beyond the scope of this investigation.

15.15. It also indicated that stability after swell was a direct function of the initial density, though proportionate loss was greater with higher compactive efforts. Part of the discrepancy was explained by the experimental procedure indicated by the "expansion data" and part by the fact that initial higher density means more soil particles per unit volume and consequently more pores. As all pore spaces are affected by saturation, decrease in density is likely to be greater. Moreover, if the theory of elastic rebound is a fact in this case, the pore spaces will automatically increase whether there is sufficient water or not.

15.16. The highest density obtained on S-I sample was 124.4 pounds per cubic foot and that on S-II sample was 120.4 pounds per cubic foot. The decrease in density after soaking was 4·0 pounds and 3·5 respectively.

15.17. **Crushing Strength**: The crushing strength of the compacted specimens of both the samples before and after swell have been plotted against the compactive efforts in Figs. 6 and 7 (Plate I). Both curves follow a similar trend and indicate that the higher the compactive effort at its corresponding optimum moisture content, the greater is the crushing strength both before swell and after swell. This also supports the findings of the University of Utah Engineering Experiment Station described in the foregoing chapter.

15.18. The loss of stability as a result of soaking has also been shown in Fig. 10(PlateII) plotted against the compactive efforts. The results are noted briefly in Tables III and IV. These curves indicate that loss in stability varies directly with compactive effort though the ultimate strength is higher with higher compactive effort even after swell. This is in conformity with the findings under expansion data. So what explains the latter will explain this phenomenon too.

16. CONCLUSION

16.1. Having thus discussed the various aspects of this investigation, the main problem will now be discussed. It has been seen in all the graphs that both the soil types follow the same trend more or less. From the trend in loss of stability and also that of swell, the 1,000 pounds per square inch compactive effort seems to be critical. Adverse effects of moisture are more pronounced beyond this range whereas below this range lower stability is obtained due to lower density. So this compactive effort seems to be the optimum in both the soil-types depending on the economy of operations and field conditions.

16.2. The major conclusions that can be inferred from the test data are, therefore, as follows :

(a) The higher the compactive effort the lower is the optimum moisture for the optimum density.

(b) Optimum density increases with the compactive effort.

(c) The higher the compactive effort at the corresponding optimum moisture content, the greater is the ultimate bearing stress or crushing strength of the moulded specimen before and after soaking.

(d) Loss in stability is higher with greater compaction; the resulting stability, however, still remains higher with the higher compactive effort.

(e) The compactive effort of 1,000 pounds per square inch seems to be the optimum for the two soil types under investigation, since the samples are least affected by water at this effort.

(f) The volume-increase relation with compactive effort needs to be studied in the light of the observations made in the discussions. It is advisable to run a series of tests with the same compactive effort, varying the moisture content in each test so that the optimum moisture corresponding to each compactive effort is known for proper correlation. It is also necessary to eliminate the causes of elastic rebound.

(g) The two soil types tested were very similar to each other, the classification being A-2 and A-2-4, as would be evident from the summary report on classification tests, Appendix II. From the summary of results as given in Tables I and II it is also seen that they followed the same pattern and almost the same behaviour under adverse water conditions. So, if anything is predicted, it applies to both the soil types and not to any one particularly.

16.3. No particular density or crushing strength was desired in this investigation because these increased with the compactive effort. Soil S-I showed a maximum ultimate compressive strength of 8.02 tons per square foot which reduced to 5.44 tons per square foot after recording a loss of 31.4 per cent, whereas, soil S II developed a maximum ultimate compressive strength of 10.75 tons per square foot which reduced to 5.27 tons per square foot recording a loss of 50.4 per cent. These clearly indicated that it was not an economic proposition to compact these two soil types to 2,000 pounds per square inch if stability under moisture condition was the aim. On the other hand, a reference to Tables III and IV will show that conditions at the compactive effort of 1,000 pounds per square inch recorded the lowest loss in stability, which was only 24.5 and 30.5 per cent for soil types S-1 and S-II respectively. It was for this reason that 1,000 pounds per square inch had been declared as the optimum compactive effort for these soil types. But, as has already been pointed out, it would not be wise to predict any definite pattern or behavior to any particular soil type from the results of this limited investigation under conditions explained before in this discussion.

17. PRACTICAL ASPECT OF THE INVESTIGATION

17.1. It has been stated in the foregoing pages that modern trend is towards use of heavier rollers in compaction of soil for the following reasons :—

(a) They produce greater density and therefore greater stability with higher bearing power.

(b) They permit efficient handling of construction equipment due to decrease in placement moisture.

(c) They permit less rigid moisture control.

But there is a limit to which the compacting effort or the weight of roller can be increased with increased advantage. Coming to the laboratory research, it means that there is a compactive effort beyond which there is no corresponding increase in optimum density and/or beyond which the corresponding loss in stability due to saturation is disproportionately excessive.

17.2. But it is quite likely that this optimum compactive effort, as found in the laboratory, can never be achieved in field conditions except at an enormous cost. The limitations are imposed by

(a) the capacity of the mechanical equipment and

(b) excessive increase in capital cost and operating charges of equipment.

17.3. The actual or practical optimum compactive effort should be a compromise between the two which would enable the construction of a pavement of lesser thickness but longer life at an economic cost.

17.4. The aim of the investigation was to find a laboratory optimum compactive effort for different types of soils from the stability point of view and to predict whether it was possible to find an optimum compactive effort for a particular type of soil without taking up laboratory tests in each individual cases. A large number of investigatiors is necessary before it will be possible to evolve a mathematical formula by means of which the optimum compactive effort of a particular soil type can be worked out depending on gradation and other physical factors.

17.5. In actual field operations, there are three methods of compaction by rolling which are usually considered economic. Each has got its own specific advantage

(a) Smooth steel-tyred roller.

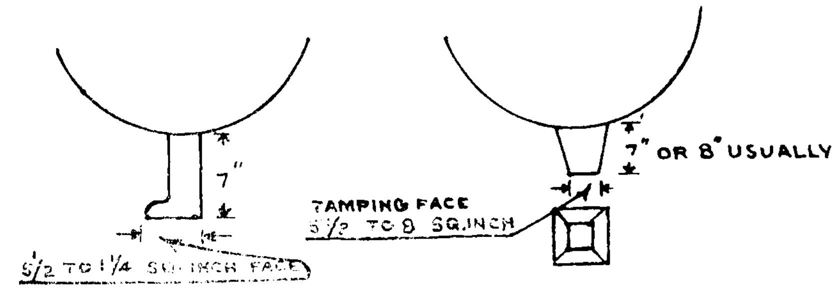
(b) Rubber-tyred roller.

(c) Sheepsfoot tamping roller.

17.6. Sheepsfoot rollers compact soils from the bottom upwards fairly uniformly and the compaction is controlled in the field by density method and not by Proctor penetration needle. Since the sheepsfoot rollers always leave one inch to two inches of loose soil above, these rollers do not produce compaction planes as are seen in the case of rolling with flat tyred rollers or even to some extent by rubber-tyred rollers.

17.7. With average equipment, the average maximum thickness of soil that can be compacted with sheepsfoot rollers in one operation is 8 in. loose. If 12 in. is to be compacted, it is necessary to have three times as heavy an equipment as in the former case. The U.S.A. War Department once tested with a 30 ton, 8 feet diameter, roller 12 feet long, which could compact two feet thickness of soil. But the cost was prohibitive and the method proved impractical.

17.8. Sheepsfoot tamping rollers are steel drums on the outside of which are attached short metal rods shaped similar to a sheep's foot as shown in sketch (a) opposite. This design of feet is, however, obsolete now-a-days. The present design is shown in sketch (b).



(a) (b)

17.9. The rollers may be oscillating or non-oscillating. The weight of a single drum varies from 3,000 pounds to as much as 19,300 pounds, empty. Provision is usally made for filling the drum with water and/or sand in order to add to its weight. The feet are staggered to distribute the weight and give a better tamping effect. The size and number of sheeps feet, that is length and cross section at edge, is specified to get a definite foot pressure on each individual tamping face. The compactive effort can thus be increased to the required extent. The compactive effort is simply calculated as below :

$$\frac{\text{Weight of roller}}{\text{No. of teeth in one line} \times \text{area of each toothface}}$$

= foot print pressure per square inch. The equivalent static presure is dependent on the speed of travel of the rollers. A travel speed of about 2½ miles per hour is, however, usual.

17.10. It is common practice to "walk the machine out of the ground", which is effected by making trip after trip over the same area until the feet no longer penetrate.

17.11. In average earth, 6 to 11 trips over the same area will usually give good compaction depending on the type of soil, moisture content, and unit weight of compacting roller. The final compaction or rolling is always done by the smooth-tyred roller.

Appendix I

CLASSIFICATION OF SOILS

According to U. S. A. Public Roads Administration.

In this system of classification, soils have been classified as to their engineering properties into eight groups designated A1 to A8 as follows:

Group A-1. Well-graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface-treated or when used as a base for relatively thin-wearing courses.

Group A-2. Coarse and fine materials, improper grading or inferior binder. Highly stable when fairly dry. Likely to soften at high-water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer, or to become loose and dusty in long-continued dry weather.

Group A-3. Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Not likely to heave because of frost nor to shrink or expand an appreciable amount. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements.

Group A-4. Silt soils without coarse material and with no appreciable amount of sticky colloidal clay. Has a tendency to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm-riding surface which rebounds but little upon the removal of load. Likely to cause cracking in rigid pavements as a result of frost heaving, and failure in flexible pavements because of low supporting value.

Group A-5. Similar to Group A-4 but furnishes highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterward.

Group A-6. Clay soils without coarse material. In stiff or soft plastic state this group absorbs additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams or cause failure owing to sliding in high

fills. Furnishes firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are likely to cause cracking in rigid pavements.

Group A-7. Similar to Group A-6, but at certain moisture contents deforms quickly under load and rebounds appreciably upon removal of load, as do subgrades of Group A-5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in Group A-6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterward. May contain lime or associated chemicals productive of flocculation in soils.

Group A-8. Very soft peat and muck, incapable of supporting a road surface without being previously compacted.

Appendix II

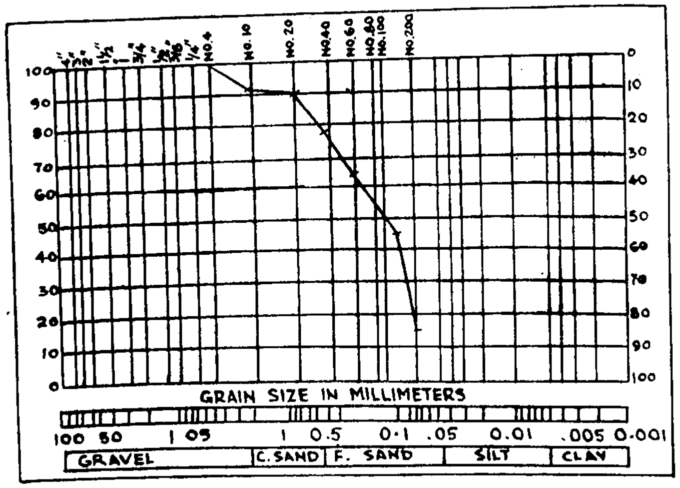
CLASSIFICAION TESTS

Summary of Data and Computations.

SUMMARY REPORT

TABLE No. 5.

SAMPLE NO. S.1. TYPE & SOURCE OF MATERIAL SANDY SOIL, UTAH



GRADING CHART

SCREEN ANALYSIS

SCREEN	% PASSING
3"	——
2"	——
1½"	——
1"	——
¾"	——
⅜"	100
NO.4	98·0
NO.10	90·4
NO.20	77·4
NO.40	63·6
NO.200	7·4

CAPILLARY MOISTURE CONTENT

GRAVEL 9·6%

BINDER ANALYSIS

COARSE SAND	26·8%
FINE SAND	56·2%
SILT & CLAY	7·4%

CLASSIFICATION PRA A-2

CHARACTERISTICS OF MATERIAL

PASSING THE NO 10 SIEVE

CAPILLARY RISE 1·4 INCHES.

PASSING THE NO.40 SIEVE

LIQUID LIMITS 26·2. P. I. RATIO. 0·37.

PLASTIC INDEX 9·8. P.I. L.L

DRY DENSITY LB/FT.^{3} SWELL ·69 – –1·17%

MOISTURE CONTENTS

SET UP AT – 13·7% AFTER SWELL 16·0% TOP INCH X%

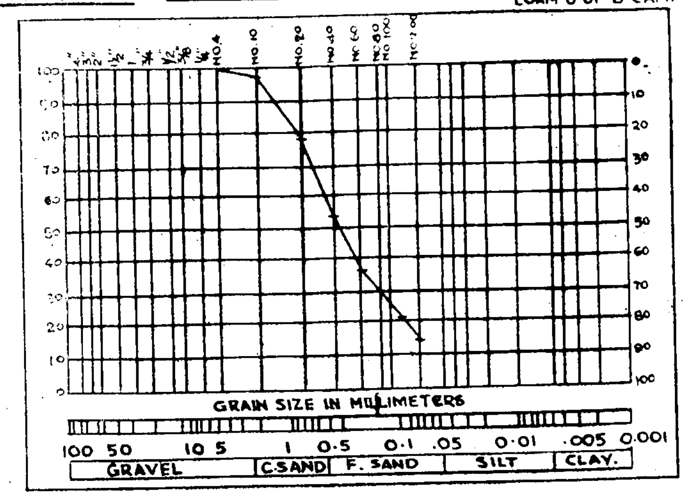
PROCTOR DATA OPTIMUM MOISTURE 15·2% DENSITY 115.4 LB/FT.^{3}

SPECIFIC GRAVITY: 2·66.

SUMMARY REPORT

TABLE No. 6

SAMPLE No. S-II TYPE & SOURCE OF MATERIAL LOAM U OF U CAMPUS



GRADING CHART

SCREEN ANALYSIS

SCREEN	% PASSING
3"	——
2"	——
1½"	——
1"	——
¾"	——
⅜"	——
NO.4	100
NO. 10	97·02
NO. 20	78·07
NO. 40	52·94
NO. 200	14·73

CAPILLARY MOISTURE CONTENT.

GRAVEL – 2·98%

BINDER ANALYSIS

COARSE SAND	44·08%
FINE SAND	38·21%
SILT & CLAY	14·73%

CLASSIFICATION PRA.A-2-4

CHARACTERISTICS OF MATERIAL.

PASSING THE No. 10 SIEVE

CAPILLARY RISE 16 INCHES.

PASSING THE NO. 40 SIEVE

LIQUID LIMIT 30·5 P.I. RATIO 0·37

PLASTIC INDEX 11·3 P.I. L.L.

DRY DENSITY LB/FT^3 SWELL ·5 – – –1·5%

MOISTURE CONTENTS

SET UP AT 15·5% AFTER SWELL 16·9% TOP INCH X%

PROCTOR DATA; OPTIMUM MOISTURE 17·5% DENSITY 112·2 LB/FT^3.

SPECIFIC GRAVITY 2·59.

(15)

TABLE VII

APPARENT SPECIFIC GRAVITY

Sample No.	S-I	S-II
Test No.	S-I-1	S-II-1
Weight of Flask, grams	135.40	70.33
Weight of Soil and Flask	318.97	120.33
Weight of Dry Soil	183.57	50.00
Weight of Soil, Flask and Distilled Water	749.75	299.91
Volume of Flask	500.00	198.90
Volume of Fluid	430.78	179.58
Volume of Soil Particles	69.22	19.32
Apparent Specific Gravity	2.66	2.59

TABLE VIII

MECHANICAL ANALYSIS

Sample No. S-I

Test No. S-I-2 Weight of Sample... 100.0 gms.

Sieve Number	Particle Size, mm.	Retained Weight, gms.	Retained Percent	Cumulative Per cent	
				Retained	Passing
3/8in.	9.520	...	...	...	
4	4.760	2.00	2.0	2.0	100.0
10	2.000	7.60	7.6	9.6	98.0
20	0.840	13.00	13.0	22.6	90.4
40	0.420	13.80	13.8	36.4	77.4
60	0.250	18.70	18.7	55.1	63.6
140	0.105	29.60	29.6	84.7	44.9
200	0.074	7.90	7.9	92.6	15.3
Pass	200	7.40	7.4		
	Total	100.00	100.00		

TABLE IX

LIQUID LIMIT

Sample No. S-I

Test No. S-1

Trial No.	1	2	3
Bottle No.	5	6	9
Number of Blows	64	28	18
Weighing Bottle Empty	42.30	41.69	41.89
Weighing Bottle, Plus Wet Soil	55.88	56.21	55.60
Weight of Wet Soil	13.58	14.52	13.71
Weighing Bottle Plus Dry Soil	53.32	53.27	52.66
Weight of Dry Soil	11.02	11.58	10.77
Moisture Content Per Cent	23.2	25.3	27.3
Liquid Limit from Graph		26.2	

[A similar test for soil sample S-II gave a L. L. of 30.5. The details as above are omitted here-Ed.]

TABLE X

PLASTIC LIMIT

Sample No. S–I

Test No. S–1–4

Trial No.	1	2	3
Can No.	13	14	16
Weight of Can, Empty	41.31	41.52	42.24
Weight of Can, Plus Wet Soil	42.78	42.75	44.16
Weight of Wet Soil	1.47	1.23	1.92
Weight of Can, Plus Dry Soil	42.57	42.58	43.89
Weight of Dry Soil	1.26	1.06	1.65
Moisture Content	0.21	0.17	0.27
Moisture Content Per Cent	16.7	16.1	16.4

Mean Value 16.4 per cent

Plasticity Index = 26.2 – 16.4 = 9.8

$$\frac{\text{P. I.}}{\text{L.L.}} = \frac{9.8}{26.2} = 0.37.$$

Similar tests for soil S-II gave the following values :—

Mean Value = 19.2 per cent.

Plasticity Index = 30.5 – 19.2 = 11.3

$$\frac{\text{P.I.}}{\text{L.L.}} = \frac{11.3}{30.9} = 0.37,$$

TABLE XI

SHRINKAGE FACTOR

Sample No. S–I

Test No. S–I—5

Trial No.	1	2	3
Weight of Porcelain Milk Dish	9.90	10.17	9.39
Weight of Porcelain Milk Dish Plus Wet Soil	35.88	33.85	36.13
Weight of Wet Soil	25.98	23.68	26.74
Weight of Porcelain Milk Dish Plus Dry Soil	29.83	28.03	29.61
Weight of Dry Soil	19.93	17.86	20.22
Moisture Content Per cent	30.20	32.50	32.50
Volume of Porcelain Milk Dish or Wet Soil	14.20	12.50	14.30
Volume of Dry Soil Pat	11.10	9.50	11.2
Shrinkage in per cent of Weight of Dry Soil	15.6	16.8	15.4
Mean Value	15.9 per cent		

Similarly for S. II. Mean Value = 22.1 per cent.

TABLE XII

FIELD MOISTURE EQUIVALENT

Sample No. S–I

Test No. S–I—6

Trial No.	1	2	3
Can No.	13	14	16
Weight of Can, Empty	41.31	41.52	42.24
Weight of Can Plus Wet Soil	58.29	71.87	71.05
Weight of Wet Soil	16.98	30.35	28.81
Weight of Can Plus Dry Soil	55.35	66.67	66.05
Weight of Dry soil	14.04	25.15	23.81
Moisture Content	2.94	5.20	5.00
Moisture Content Per Cent	20.90	20.70	20.90
Mean Value =	20.8 per cent		

CAPILLARY POTENTIAL

(HOGENTOGLER — BARBER CAPILLARIMETER)

Sample No. S–I

Test No. S–1—7

Trial No.	1	2	3
Capillary Rise	16″	14″	14″
Mean Value =	14 inches		

Similarly for S-II

Mean value of F. M. E = 29.3 per cent

Capillary Potential = 16 inches

Appendix III

Main Investigating Tests

Summary of Data and Computations

TABLE XIII

PROCTOR OPTIMUM MOISTURE

Sample No. S-I

Test No. S-I—8

Volume of Cylinder 4″ Dia. by 4.59″ high = 0.0335 Cu. Ft.

Degree of Compaction—25 blows per layer

Trial No.	1	2	3	4	5
Weight of Cylinder, Empty, gms.	4483	4483	4483	4483	4483
Weight of Cylinder Plus Wet Soil	6288	6429	6490	6480	6443
Weight of Wet Soil	1805	1946	2007	1997	1960
Wet Density Pounds Cu. Ft.	119.2	128.5	132.6	131.9	129.4
Can No.	5	6	7	9	10
Weight of Can, Empty	42.32	41.71	41.22	41.91	41.76
Weight of Can Plus Wet Soil	126.51	99.22	108.44	90.47	109.42
Weight of Can Plus Dry Soil	117.77	92.49	99.77	83.61	99.12
Weight of Dry Soil	75.45	50.78	58.55	41.70	57.36
Moisture Content	8.74	6.73	8.67	6.86	10.30
Moisture Content per cent Dry Weight	11.6	13.3	14.8	16.4	17.9
Dry Density. Pounds/Cu. Ft.	102.8	111.9	115.0	113.1	109.8

Optimum Moisture Content from graph = 15.2 per cent

Optimum Dry Density = 115.4 pounds per Cu. Ft.

For S-II O. M. C = 17.5 per cent.

O. D. D = 112.3 per cubic foot.

TABLE XIV

UNCONFINED COMPRESSION TEST BEFORE SWELL

Sample No. S-I

Test No. S-I—9

Volume of Cylinder—4″ Dia. by 4.59″ high = 0.0335 Cu. Ft.

Area of Specimen—12.57 Square Inches

Cylinder No.	I	II	III	IV
Weight of No. 4 Sample, gms.	2600	2600	2600	2600
Hygroscopic moisture, per cent	1.0	1.0	1.0	1.0
Set up at moisture per cent	13.7	14.2	14.7	15.2
Compactive Effort Lbs. in²	2000	1500	1000	500
Weight of Mould, Empty, gms.	4481	4481	4481	4481
Weight of Mould plus Compacted Soil	6559	6561	6552	6541
Weight of compacted soil	2078	2080	2071	2060
Wet Density, No./Cu. Ft.	137.2	137.4	136.8	136.0
Crushing Load, Pounds	1400	1200	1050	850
Crushing Strength, Tons/Sq.Ft.	8.02	6.90	6.03	4.87
Can No.	5	6	69	11
Weight of Can, Empty	42.32	41.71	41.91	41.76
Weight of Can plus Wet Soil	66.55	77.01	81.70	85.76
Weight of Can plus Dry Soil	63.38	72.36	76.40	79.75
Weight of Dry Soil	21.06	30.65	34.49	37.99
Moisture Content per cent	15.0	15.2	15.4	15.85
Dry Density	119.3	119.3	118.5	117.3

(Figures for S-II are omitted here-Ed.)

TABLE XV

CONTINUOUS SOAKING TEST—I

Sample No. S–I

Test No. S–I—10

Volume of Cylinder—4″ Dia. by 4.59″ high = 0.335 Cu. Ft.

Area of Specimen—12.57 Square Inches

Cylinder No.	I	II	III	IV
Weight of No. 4 Sample, gms.	2500	2500	2500	2500
Hygroscopic Moisture, per cent	1.0	1.0	1.0	1.0
Set up at Moisture, per cent	13.7	14.2	14.7	15.2
Compactive Effort, Lbs./in.^{2}	2000	1500	1000	500
Weight of Mould, Empty, gms.	4546	4590	4559	4567
Weight of Mould plus Compacted Soil	6675	6699	6644	6630
Weight of Compacted Soil	2129	2109	2085	2063
Wet Density, lb./Cu. Ft.	140.6	139.3	137.7	136.2
Can No.	11	12	14	16
Weight of can, Empty, gms.	41.76	41.45	41.55	42.26
Weight of Can, Plus Wet Soil	55.96	56.30	55.25	54.19
Weight of Can, Plus Dry Soil	54.33	54.48	53.51	52.64
Weight of Dry Soil	12.57	13.03	11.96	10.38
Moisture Content, per cent	13.0	14.0	14.6	15.0
Dry Density	124.4	122.2	120.1	118.4

(Figures for S-II-are omitted here-Ed.)

TABLE XVI

CONTINUOUS SOAKING TEST—II

Sample No. S–I

Test S–I—10 Cyl. No. I Set Up At 13.7 % Moisture

No. Days	Dial Reading	Change in.	Cum. Change	% Incr. in Vol.	Wt. Grams	Incr. in Wt. & H_2O	Calc. Den.
0	0.4563	0.0000	0.0000	0.0000	2129		124.4
1	0.5017	0.0454	0.0454	0.989			
2	0.5051	0.0034	0.0488	1.063			
3	0.5085	0.0034	0.0522	1.129			
4	0.5090	0.0005	0.0527	1.147			
5	0.5095	0.0005	0.0532	1.158			
6	0.5096	0.0001	0.0533	1.160			
7	0.5100	0.0004	0.0537	1.172			
10	0.5100	0.0000	0.0537	1.172			
18	0.5100	0.0000	0.0537	1.172			
20	0.5100	0.0000	0.0537	1.172			
21	0.5100	0.0000	0.0537	1.172	2143	14	120.4

Compactive Effort 2,000 pounds per square inch.

Optimum Moisture Content—13.7 per cent.

Figures for other compactive efforts and for tests repeated on soil S. II are omitted here-Ed.

TABLE XVII
CONTINUOUS SOAKING TEST—III
STRENGTH AFTER SWELL

Sample No. S-I
Test No. S-I-10

Cylinder No.	I	II	III	IV
Set Up at Moisture, per cent	13.0	14.0	14.6	15.0
Compactive Effort, Lbs./in.²	2000	1500	1000	500
Weight of Mould plus Soil after Swell	6689	6710	6656	6655
Weight of Mould, Empty	4546	4590	4559	4567
Weight of Soil after Swell	2143	2120	2097	2088
Weight of Soil before Swell	2129	2109	2085	2063
Weight of Water Absorbed	14	11	12	25
Total Water after Swell, gms.	290.8	306.3	316.4	334.5
Crushing Load, Pounds	950	900	800	650
Area of Sample, Sq. In.	12.57	12.57	12.57	12.57
Strength in Tons/Sq. Ft.	5.44	5.16	4.58	3.73
Can No.	11	9	6	5
Weight of Can, Empty	41.76	41.91	41.71	42.32
Weight of Can, Plus Wet Soil	92.85	158.70	154.45	96.10
Weight of Can, Plus Dry Soil	85.81	142.72	138.84	88.87
Weight of Dry Soil	44.05	100.81	97.13	46.55
Moisture Content, per cent	16.0	15.9	16.0	15.5

TABLE XVIII
CONTINUOUS SOAKING TEST—III
STRENGTH AFTER SWELL

Sample No. S-II

Test No. S-II—10

Cylinder No.	I	II	III	IV
Set Up at Moisture, per cent	15.0	15.6	16.2	17.2
Compactive Effort, Lbs./in.²	2000	1500	1000	500
Weight of Mould, plus Soil After Swell	5542	5509	5522	5469
Weight of Mould, Empty	3426	3422	3460	3432
Weight of Soil after Swell	2116	2087	2062	2037
Weight of Soil before Swell	2096	2070	2050	2024
Weight of Water Absorbed	20	17	12	13
Total Water after Swell, gms.	334.4	339.9	344.1	361.1
Crushing Load, Pounds	950	850	800	650
Area of Sample, Sq. In.	12.57	12.57	12.57	12.57
Strength in Tons/Sq. Ft.	5.44	4.87	4.58	3.73
Can No.	5	6	10	13
Weight of Can, Empty	42.31	41.90	42.39	41.34
Weight of Can, Plus Wet Soil	89.11	93.39	115.17	108.92
Weight of Can, Plus Dry Soil	82.35	85.74	104.24	98.52
Weight of Dry Soil	40.04	43:84	61.85	57.18
Moisture Content, per cent	16.9	17.5	17.7	18.2

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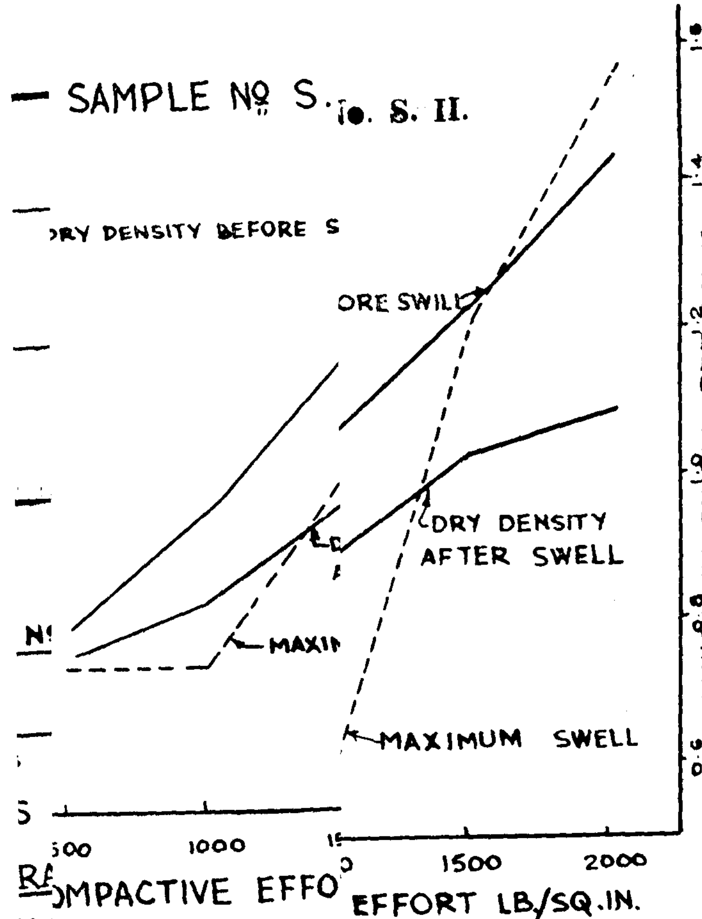

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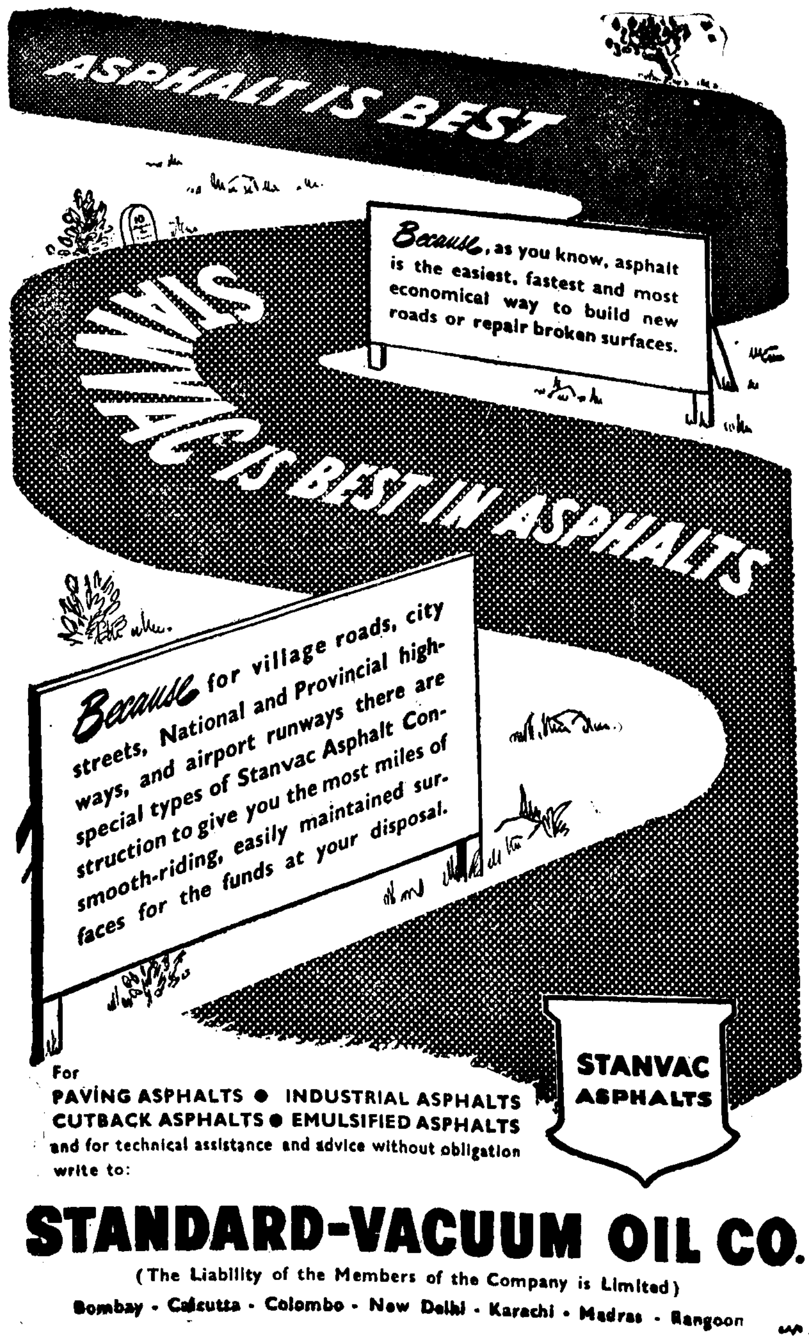
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