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INDIAN ROADS CONGRESS

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NEW ADMISSIONS.

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ENROLLED FROM 1-10-1952 to 5-12-1952.

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1299	1696	Ghulam Rasool Laharwal	Assistant Engineer, P.W.D. B/R, Tanki Pora, Srinagar, Kashmir.
1300	1697	S. Nagarajan	Junior Engineer (Bridges), Highways, 31-Rly Colony, Egmore, Madras-8.
1301	1698	P.S. Padmanabhan	Assistant Executive Engineer, C.W. & P.C., Government of India, Hirakud Dam Project, C-12, Hirakud Camp, Via Sambalpur. (Orissa).
1302	1699	K. L. Nanda	Lecturer, Punjab Government School of Engineering, Nilokheri, District Karnal (Punjab).

DEATHS.

The Council have received with regret intimations of the following deaths :

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1	309	N. V. S. Murty.	Secretary to Government of Hyderabad, P. W. D., Hyderabad-14.
2	922	Vepa Krishnamurty.	Superintending Engineer, Vijayawada Circle, Vijayawada. (Madras State.)
3	1378	B. Krishna-Iyengar.	Executive Engineer, Bombay P.W.D., Sholapur. (Bombay State).

Paper No. 162

Recent Examples of Prestressed Concrete Bridges

by

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SYNOPSIS

The Paper gives a brief summary of the special reasons motivating the use of prestressed concrete in bridge construction. It is a reminder of the first tests carried out in Great Britain on a bridge girder, and deals more fully with the execution of two recently completed bridge structures, namely an 88-foot span Class 'I' road bridge, and a cantilever footbridge with a central span of 150 feet and two approach spans of 35 to 40 feet.

1. INTRODUCTION

1.1. It is not surprising to note that the technique of post-tensioning found its ready application in bridge construction as soon as it became known. The saying that a prestressed structure carried its own weight free of cost is particularly true in post-tensioned work, and this fact gains in importance when the structural weight forms a substantial part of the total load.

1.2. Another reason in favour of the early application of prestressing to bridgework is, perhaps, that many bridge structures are simply supported and are expected to behave as such. In a reinforced concrete framed building, the columns, beams, and slabs act together, and when using prestressed concrete instead of reinforced concrete, the designer is faced with the dilemma of either providing a less monolithic structure or alternatively adding the expense of full prestressing across all construction joints. No such dilemma occurs in bridge design where half-way solutions are, as far as possible, avoided. A simply supported bridge span is not expected to work more or less in conjunction with its supports. In fact, it is required to do precisely the opposite and roller bearings are often provided to ensure substantial freedom of movement. Similarly, a continuous bridge girder would have its reactions at supports determined as accurately as possible and full prestressing would be provided at these points. Prestressed concrete can, therefore, compete with other materials in bridge construction without any mental reservations as to the amount of continuity to be provided over and above assumptions made in the design.

2. EARLY EXAMPLES OF PRESTRESSED BRIDGES

2.1. It may be of interest to note that the oldest prestressed bridges in existence are quite large structures of spans of 100 feet, erected in Germany some twenty years ago.

2.2. In these structures, designed by Mautner according to Freyssinet's ideas, the bridge girders were cast in situ and the reinforcement was pre-tensioned. This unusual combination of in-situ concrete and pre-tensioned steel would hardly be employed nowadays with the many post-tensioning systems available. The pre-tensioned steel was anchored to the steel moulds which had to be sufficiently strong to take up the reaction in compression. The moulds were expensive but their cost was written off by many re-uses on several sites.

2.3. Mautner, who was with the Authors, also designed the first prestressed bridges in England. These structures, built for the Ministry of Transport, consist of precast girders manufactured in long lines with pre-tensioned steel wire. The making of these bridge units offered an opportunity to perform a full scale loading test carried out to destruction. This test gave valuable information on the safety factor against cracks and the ultimate safety factor to be expected in prestressed structures. It may be as well to give here a short summary of the performance obtained.

2.4. The beam selected for testing was a box-girder spanning 28 feet, with a depth of 18 inches and width of $17\frac{3}{4}$ inches. The vertical webs were 2 inches thick at midspan, and the top and bottom flanges $3\frac{1}{2}$ inches thick. The beam weighed 179 lb per ft run, and was designed for Ministry of Transport Standard loading. The prestressing was given by twentyfour 1/5th inch diameter wires, pre-tensioned to about 70 tons per sq. inch.

2.5. The first cracks appeared under a load equal to two and a quarter times the design load. These disappeared upon removal of the load and reappeared upon increasing the load to two and a half times the design load. Shear cracks also appeared at this stage of loading. Failure occurred by compression of the top flange when the superimposed bending moment reached 3.35 times the design bending moment for live load. Actual stress measurements were derived from deflection and strain-gauge readings recorded at various stages of loading. These stresses were found to agree very closely with stresses derived from computations. Pre-tensioning of the steel to 70 tons per sq. in. was considered to be too high by some. In fact, the beam failed by compression in the concrete, and the test was particularly useful in removing misapprehensions of an early failure of the beam through tension in the steel.

3. SAFETY FACTOR IN SHEAR

3.1. In the test on the 28 foot beam, the load was applied by means of four hydraulic jacks evenly distributed over the span. When the load reached a value equal to 2.7 times the design superimposed load, one of the jacks placed near a support gave out and the test proceeded with the two jacks at midspan. Although the failure occurred in bending, there were signs that a shear failure might have been obtained had the original loading arrangement been proceeded with.

3.2. It has been said that shear stresses are seldom critical in prestressed concrete; and, indeed, when a simply supported beam has been designed for bending, the main tensile stress at supports are usually found to be within the permissible limits. To take a numerical example, a shear stress of 340 lb per sq. in. under design load will produce a main tensile stress of 125 lb per sq. in. if combined with a horizontal pre-compression of 800 lb. per sq. in. at neutral axis. Under design load, therefore, the concrete will be working in tension to less than one third of its capacity which gives a comfortable margin of safety. On the other hand, when calculating the possible effects of overloading on the same beam, one finds that a 100 per cent increase of load, giving a shear stress of 640 lb per sq. in., will produce a main tensile stress of 390 lb per sq. in., which is more than three times the amount obtained under design load, and quite near to failure.

The computed increase in the main tensile stress under an overload is even more marked in cantilever beams where the section of maximum shear is also the section of maximum bending moment.

3.3. When the likelihood of overloading is very remote, as in a bridge, it seems perhaps natural to dimension a structure from the sole consideration of stresses under service. In cases where the load is not under special control, consideration will no doubt be given to the ultimate carrying capacity of the structure. A choice between these two alternatives is perhaps of particular significance from the point of view of shear strength.

3.4. It may be noted in this connection that post-tensioning methods are particularly effective in relieving stresses. The Freyssinet cables used in the two bridge structures described in this Paper can easily be bent to almost any curve so as to create actual shear forces acting in opposition to the shear forces induced by the deadweight and live load.

4. ADVANTAGES OF PRE-STRESSING

4.1. A description of two recently completed bridges may perhaps be preceded by a reminder of the several advantages consequent to the use of pre-stressing. These can be enumerated as follows:

4.1.1. **Prevention of cracks through bending.** In a pre-stressed structure, the tensile zone of a concrete section is subjected to a large pre-compression sufficient to cancel the tensile stress induced by deadweight and live loads. In the compression zone of the same section, the pre-stressing force, placed eccentrically, gives no pre-compression or even a slight pre-tension on the concrete which is automatically cancelled by the compressive stresses induced by deadweight alone. Taking the particular example of a simply supported span, the pre-stressing, placed at the bottom and acting in conjunction with the deadweight, creates a large compressive stress at the bottom edge and a slight compression at the top edge. When the live load is brought into action, the compressive stresses increase at the top and decrease at the bottom of the section so that the concrete remains under compression in all instances of loading.

4.1.2. **Prevention of cracks through shear.** In a normal reinforced concrete section, the main tensile stresses

caused by shear are equal in value to the calculated shear stresses and act at 45 degrees to the neutral axis thereby causing eventually the formation of shear cracks at right angles to their line of action. Under the combined effect of horizontal pre-stressing and vertical shear, the main tensile stresses are substantially reduced and can be brought down to a safe value. The danger of shear cracks is, therefore, effectively removed; furthermore, the curving upwards of the pre-stressing cables at supports provides the pre-stressing force with a vertical component acting in opposition to the vertical shear forces.

4.1.3. **Saving in concrete.** In pre-stressed structures, the concrete can be assimilated to a material equally good in tension and compression. It can, therefore, be used in the same way as a homogenous material such as steel or aluminium, and its permissible working stress can be closely related to the ultimate stresses derived from test cubes. In particular, a concrete showing an ultimate stress of 6000 lb per sq. in. may be used up to 2000 lb per sq. in., if a safety factor of 3 is deemed adequate. With the same quality of concrete in an ordinary reinforced concrete structure, the permissible compressive stress would be much lower, owing to the danger of tensile cracks extending into the compression zone. The use of good quality concrete, coupled with pre-stressing, allows, therefore, substantial reductions to be made in the concrete section with resulting savings in structural deadweight and foundation work.

4.1.4. **Saving in Steel.** It is known that reinforced concrete has little use for high tensile steel, the steel stresses having to be kept low in any case so as to limit the strain and guard against a widening of the cracks in the tensile zone. In pre-stressed concrete, on the other hand, the steel is stressed and strained on its own and quite independently from the surrounding concrete. The use of high tensile steel is therefore possible and, indeed, imperative in prestressed concrete since a large preliminary tension stress is required to reduce the relative importance of tension losses occasioned by shrinkage and creep in the concrete. Working steel stress in post-tensioned work amounts to some 60 tons per sq. in. as against 8 tons per sq. in. in reinforced concrete work and the saving in steel tonnage is correspondingly high.

5. THE CASTLE BRIDGE IN ABERCRAVE, NEAR SWANSEA

5.1. The Site and other Data

5.1.1. The new Castle Bridge, recently completed in Abercrave, near Swansea, provides a crossing over the river Tawe on a road diversion that eliminates a dangerous crossing over the same river through an existing narrow bowstring-girder bridge, made of structural steel (Fig. 1). This prestressed concrete structure is believed to be the largest yet constructed in Great Britain for the full Ministry of Transport standard loading. It comprises a 22-feet carriageway for Class I* standard loading road and two footpaths, 10 feet wide, over a clear span of 80 feet.

5.1.2. An arched rib design was first considered, especially in view of the nature of the ground, which was reputed to be rock. The appointed Engineers, however, had already built out the first two post-tensioned bridge schemes in Great Britain with results coming fully up to expectation and consideration was therefore given to a prestressed design for the new Castle bridge. No restrictions were given as to structural depth, and the structure took the shape of eight precast and prestressed, deep girders carrying a reinforced concrete decking cast in situ and containing a number of post-tensioned parabolic cables. The girders were to be simply supported and so required a far smaller amount of foundation work than would have been necessitated by the arched rib design. The large saving in foundations became fully apparent when the excavation revealed that a thrust-bearing stratum was not available at normal depths and that an arched bridge would have required either raking piles or deep massive footings sufficiently heavy to provide resistance to arch thrust through frictional forces.

5.1.3. As things stood, the simply supported girders required only the normal type of mass concrete retaining wall, with wing walls.

5.2. Main Girders

5.2.1. The eight precast girders are of an I section (Plate I). These are 85 feet long and 5 feet 6 inch deep, with a unit weight of $38\frac{1}{2}$ tons. Top and bottom flanges are 24 inches and 20 inches wide respectively and the web is 8 inches thick. There are 20 Freyssinet cables in each of the intermediate beams and 14 cables in each of the edge beams, each cable containing twelve wires of

* 220 lb/sq. ft. plus a K. E. L. of 2700 lb./ft. width of roadway, the footpath loading being 100 lb/sq. ft.

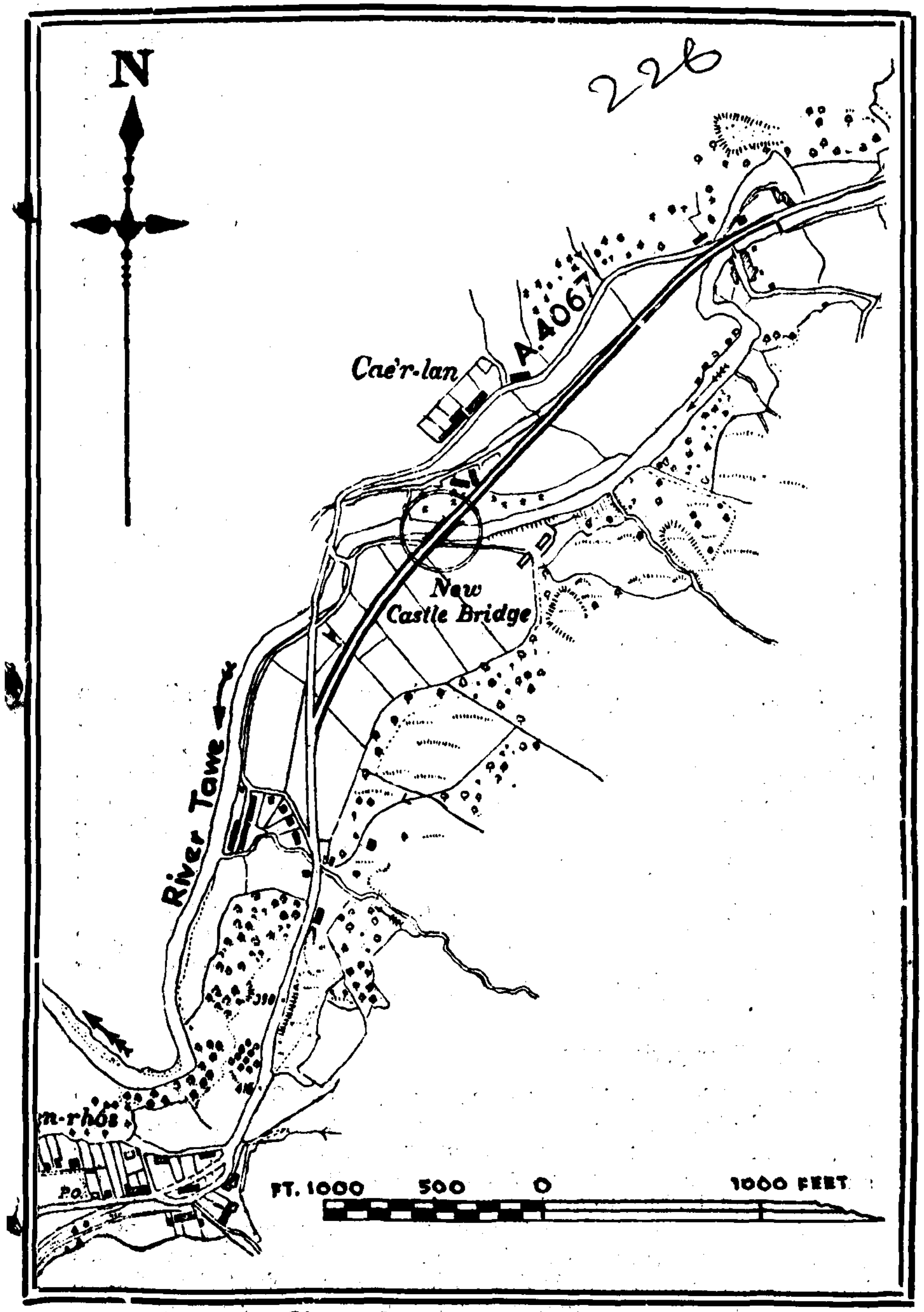


Fig. 1. Site of Castle Bridge at Abercrave.

0.2 inch diameter. The cables are placed close together in the bottom flange at mid-span and are given a continuous upwards curve towards both supports thereby providing an effective resistance to shear forces. Each cable is provided at either support with a Freyssinet anchor cone and the 20 (or 14) cones at each girder end are spaced regularly throughout the sectional depth so as to distribute the end pre-compression as uniformly as possible.

5.2.2. The web was given an 8 inch thickness in order to give sufficient cover to the curved cables. A 4 inch thickness would have been sufficient to resist the shear stresses and there are many examples to show that the casting of a 4-inch thick member is a practical proposition. In the present case, however, it was found that an increase to 8 inch would facilitate the placing and curving of the cable and bring about an appreciable reduction in frictional losses. A fish-bellied girder, with a bottom flange curving upwards and containing all the cables throughout the length, would have facilitated the placing of cables and allowed a reduction of the web thickness to some 4 inches, but this shape of girder was not considered suitable in view of the pronounced skew on the bridge.

5.3. Deck Slab

5.3.1. The deck slab is reinforced concrete cast in situ to a thickness of 8 inches between the top flanges of the precast beams. These flanges were provided with mild steel bars projecting horizontally on either side and lapping with the slab reinforcement and their vertical faces were cast to a slight taper to ensure the transfer of shear forces from slab to beam. The road surfacing consists of a layer of ordinary concrete laid to falls with a wearing surface of rock asphalt 2 inch thick The deck slab proper covers five bays measuring 6 feet 2 inches between centres of girders and is designed for a future extension of the carriageway to a width of 30 feet, as against the present 22 feet width, with a corresponding reduction in the width of footpaths.

5.4. Footpath Bays

5.4.1. The two side bays, measuring 4 feet 8½ inches between centres of girders, are used as service ducts. A reinforced concrete slab, spanning between the lower flanges of the girders, gives support to the various mains, and the duct roof is formed by removable slabs bearing on chasses provided in the top flanges. A reinforced concrete parapet coping, concrete kerb, and paving slabs on sand filling, complete the construction of the footpath bays. The services duct is lined with mastic asphalt three-fourth inch thick and the railing is of light mild steel construction.

5.5. Cross Girders

5.5.1. The overall carrying capacity of a wide bridge is substantially increased by the provision of cross girders designed to distribute occasional heavy loads between main girders. It is felt that these cross girders need not be very numerous, their proper functioning depending mainly on their individual rigidity. Four or five cross girders seem to be amply sufficient on any span to ensure a proper working of the bridge structure as a whole and, in the case under review, these were placed 15 feet apart and given the whole available depth of 5 feet 6 inch with a thickness of 9 inches. They were cast in situ, after the beams had been placed, and their mild steel reinforcement was made to lap with starter bars left projecting from the beams.

5.5.2. Consideration was given at first to a transverse prestressing of the cross girders and deck slab but it was found that this would not result in any saving and this part of the structure was accordingly carried out in ordinary reinforced concrete.

5.6. Bearers

5.6.1. The provision of rocker and roller bearings, recommended in Great Britain for spans greater than 50 feet, becomes all the more essential in prestressed concrete construction as it enables the concrete to take its strain under prestressing. In the work described, a roller bearing was provided at one end of each beam in the shape of a 5 inch diameter steel roller, 2 ft. 8 in. long, with top and bottom steel plates 1½ inch thick, designed to distribute the concrete compression in the beam and on the abutment. The rocking arrangement at the opposite beam end consists of two distribution plates 1½ inch thick, the under plate fixed to the abutment being made slightly convex on its top face. This comparatively simple arrangement is sufficient to ensure freedom of movement in the structure under the influence of temperature variations and delayed shrinkage and creep in the concrete.

5.7. Quality of Materials and Design Stresses

5.7.1. The steel wire used in the manufacturing of the prestressing cables was high tensile carbon steel wire, rolled to one quarter of an inch diameter, and cold drawn to one fifth of an inch diameter. The cables were delivered to the site ready made, each cable consisting of twelve wires grouped circumferentially around a central hollow core and sheathed in a light gauge steel tubing or in a plastic hose, the total diameter being just over one inch. Tensile tests performed on several samples showed an ultimate stress of not less than 100 tons per sq. in. with a 0.1 per cent proof stress of 75 to 80 tons per sq. in. The cables were tensioned to 70 tons per sq. in., and it was assumed that losses caused by creep and shrinkage would leave a permanent prestressing force of 56 tons per sq. in. equivalent to 21 tons per cable.

5.7.2. The concrete was mixed in the following proportions:

112 lb of rapid hardening cement;
150 lb of washed channel sand;
105 lb of half-inch Pennant Stone; and
135 lb of three-fourth inch Pennant Stone.

5.7.3. The half and three-fourth inch aggregates were measured separately in order to obtain more consistent results but the precautions taken in this instance are not typical of the prevailing prestressed concrete practice, which consists generally in making one single batch of the stone element.

The sand and stone were sent to a laboratory for grading and chemical analysis as this was found to be more expedient than the usual site tests.

The type of mixer used on the job made it necessary to mix the concrete rather wet, in fact to a water-cement ratio of 0.56. In spite of this large quantity of water, the seven days cube strength amounted to 6000 lb per sq. in. or more, which made it feasible to prestress the beams seven days after casting. As shown by the computations, the highest stress on the concrete occurred immediately after completion of the prestressing and this amounted to 2200 lb per sq. in. that is about one third of the ultimate concrete strength.

5.7.4. Much has been written with regard to the necessity of obtaining early concrete strengths, as they provide the best proof of the quality of the material and allow the prestressing to be carried out without lengthy delays. In order to speed up the job, one of the beams on the Castle bridge was prestressed at seven days after the cube test had shown a strength greater than 6000 lb per sq. in. The camber caused by prestressing amounted to $2\frac{1}{8}$ inches at midspan, whereas similar beams, prestressed at 21 days, gave a camber of a quarter inch only. As the camber is a measure of the deferred deformation of the concrete, upon which depend the relaxation losses in the steel, it became obvious that the maturing time could be of some importance, quite apart from considerations of strength.

5.7.5. There is no doubt that the high water-cement ratio contributed in part to the large camber. In view of the experience gained on this and other jobs, it is felt that a $1:1\frac{1}{2}:3$ concrete, having a water-cement ratio of 0.45 or less, and left to mature two weeks, will be suitable for prestressing if its strength at fourteen days is near enough 5000 lb per sq. in.

5.8. Execution

5.8.1. Consideration was first given to the possibility of casting the beams on the bank and launching them across the river by means of masts. Although this method has been used in several cases, it was found more expedient, in this particular instance, to erect a temporary steel bridge crossing the river between abutments placed on the central line of the new work and made wide enough to cast one pair of beams. Only one set of beam shuttering, made in lengths of 12 feet, was used for the whole job. Two beam soffits were erected on the temporary bridge and operations took place in the following order:

(a) Two precast reinforced concrete slabs (See Photo 1), 7 inches thick, 2 feet wide, and 5 feet 6 inches high, were placed vertically, one at each end of soffit No. 1. These slabs, which contained the Freyssinet female anchor cones built-in (See Photo 2) at the required slopes, were used as a permanent end shuttering to the beam.

The cables, delivered to the site complete with sheath, were then placed on a parabolic curve rising from the bottom at

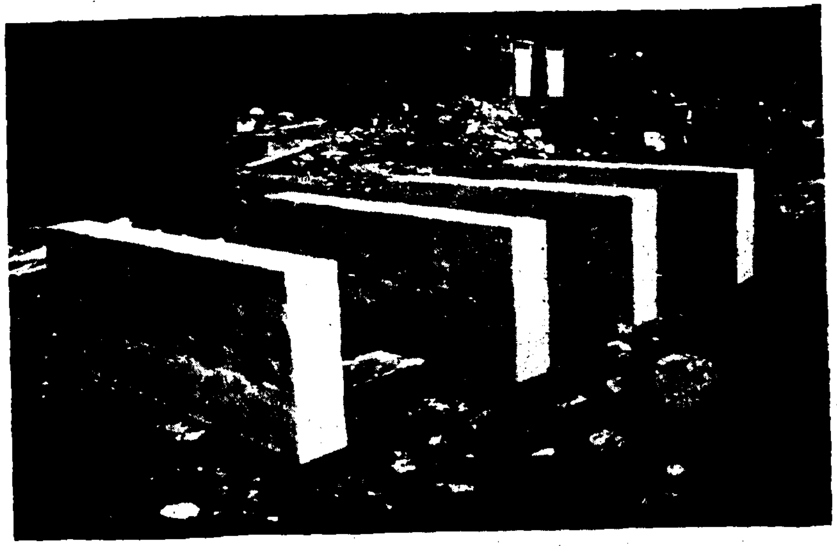


Photo 1. **Abercrave Bridge—Precast Anchor blocks**

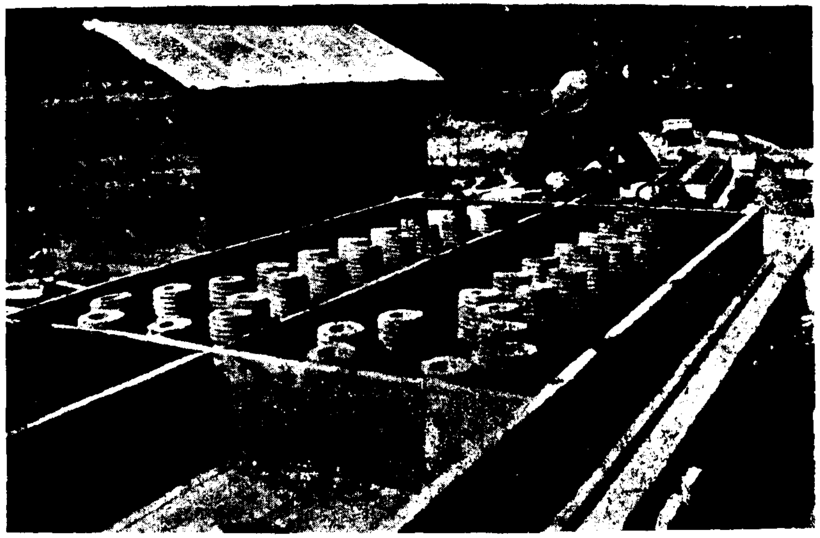


Photo 2. **Abercrave Bridge—Making precast anchor blocks with Freyssinet cones built in.**

midspan to their respective anchorage cones at each end (See Photo 3). They were made to pass through the cones and projected 2 feet 6 inches beyond. The sheathing itself stopped short of the anchor block and a bitumastic compound was used as a sealer at this point to prevent the entry of cement grout into the female anchorages. The cables were held in their respective positions by binding them to the vertical three-eighth inch diameter stirrups provided at 2 feet centres for this purpose.

The side shuttering was then adjusted in position and the 20 cubic yards of concrete which one beam contains were cast in the time interval of 6 to 8 hours, without interruption. A weigh batcher and tilting drum mixer were used and the concrete was brought at hand by means of a monorail transporter running the length of the beam. Both external and internal vibrators were used, the former to better effect.

The side shuttering was stripped the next day and erected on soffit No. 2, and the beam left to mature.

(b) In most beams, the tensioning of cables took place after a maturing time of 10 to 14 days. A male cone was inserted at each end of the cable to be tensioned; one double acting Freyssinet jack was brought to bear at each end against the face of the

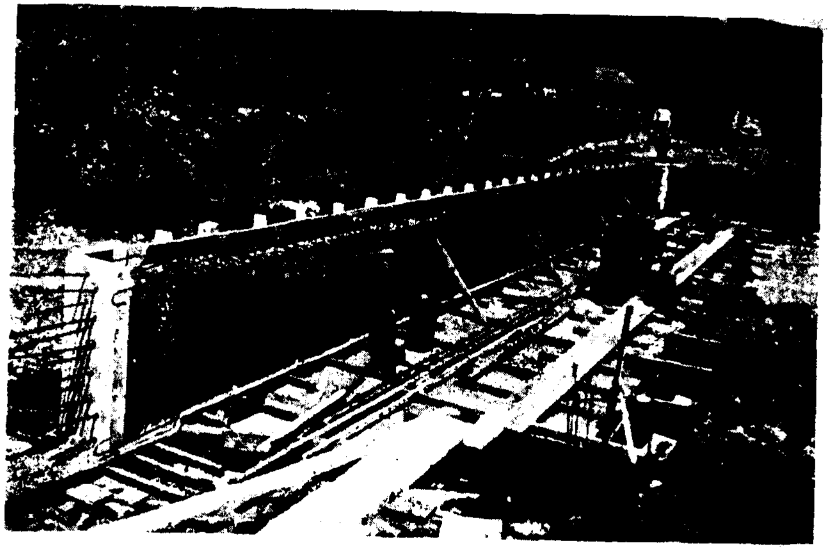


Photo 3. **Abercrave Bridge—Manufacture of precast beam. The view shows shuttering on one side, anchor blocks at end, and parabolic cables.**

beams and the projecting wires were fixed to it by means of steel wedges (See Photo 4). The jacks, operated by a Tangye pump, Photo 5, induced the required tension in the cables by leaning on to the beam and their secondary action drove the male cone to refusal, thereby locking the cable under permanent tension. The jack was then disconnected and re-fastened to the next cable. The cables were tensioned in a given order, so as to avoid lateral bending, and 5 inch elongation was obtained on each of them.

(c) After completion of the prestressing, the beam was lifted from its soffit by two 25-ton jacks and lowered at each end on a two wheeled bogey carried by a rail running along and on top of each abutment. (Photo 6.) Two 5-Ton horizontal jacks, leaning against a stop fixed to the rail, were used to move the beam along the abutment to its final position, Photo 7, where it was jacked down on its prepared roller and rocker bearings.

5.8.2. The beams were cast at the rate of one pair every five weeks and, with the four pairs in final position, the construction of deck slab, service duct, footpath, roadway, and guardrail, proceeded in the usual way.

5.8.3. **Grouting the cables**. There remains to be mentioned the grouting of the cables which was made by inserting a nozzle

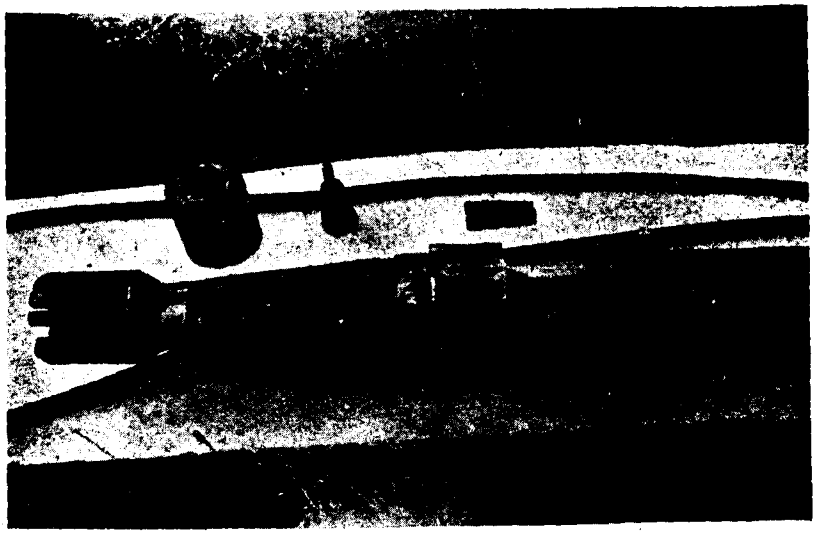


Photo 4. Abercrave Bridge—The Freyssinet jack, anchor block (male and female), steelwedge for jack and completed cables.

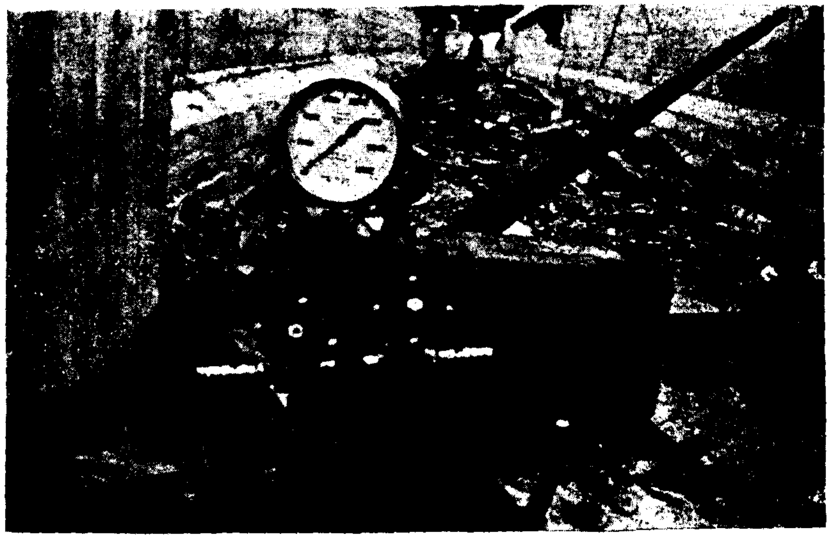


Photo 5. Abercrave Bridge—High Pressure hand pump for perating Freyssinet jacks.

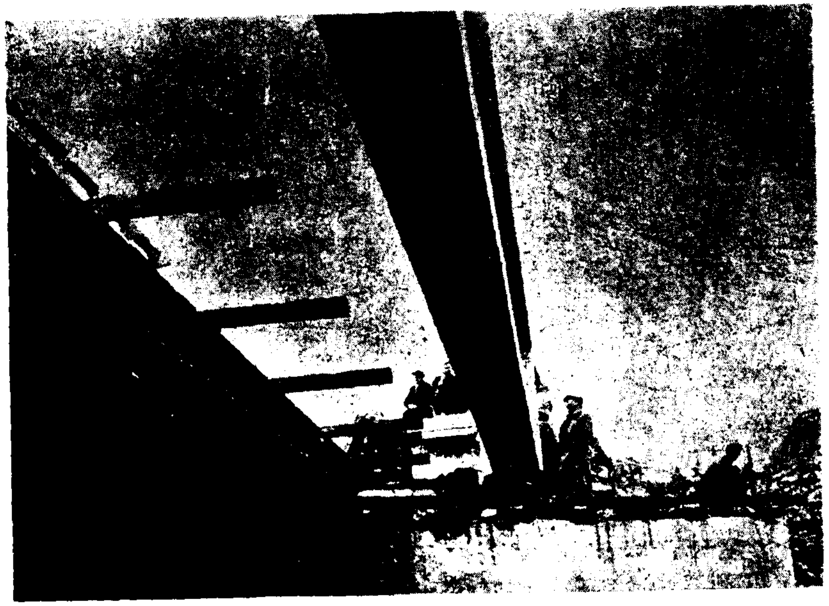


Photo. 6: Abercrave Bridge—Precast girder placed on bogeys, being moved sideways.

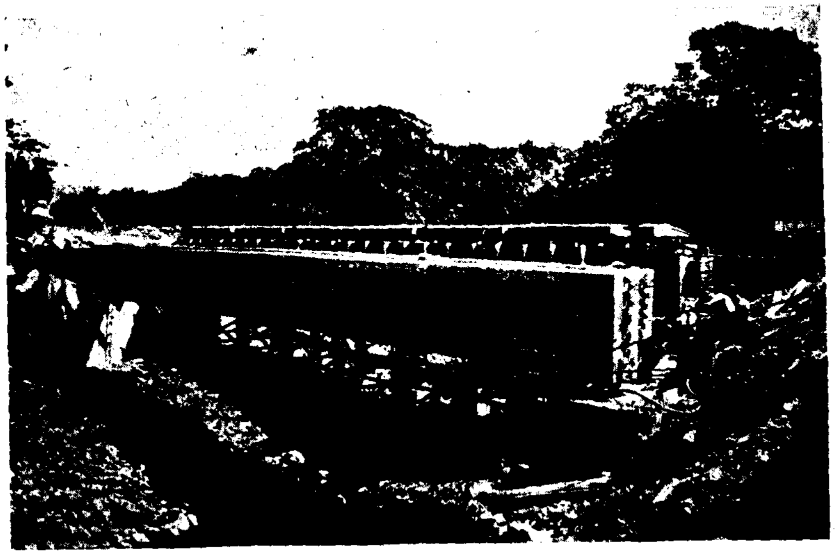


Photo. 7: Abercrave Bridge—Precast beam completed and moved to its final position

at one end through a circular hole left in the male cone and injecting cement grout from a colloidal mixer. The prepared grout was poured into a cylinder and the required injecting pressure was given by a piston operated by water. The hollow cables were first filled with water from end to end and the water was chased out at the far end and replaced by the pressure grout in order to avoid air locks.

A mixing of one hundredweight of cement to six gallons of water appears to be suitable for the injecting grout.

5.9. Costs

5.9.1. The foundation level of this bridge is comparatively shallow and costs were therefore, not high. Including foundations the cost per square foot of deck was about £ 7 (or ninetythree rupees approximately)

6. THE CASTLE FOOTBRIDGE OVER THE SEVERN, AT SHREWSBURY

6.1. Terms of Reference and Site Data

6.1.1. The original steel suspension Castle Bridge having reached the end of its useful life, a decision was taken to erecta new work in its place (Fig 2), and the Borough Authorities invited tenders for design and construction of a footbridge in any suitable material. The terms of reference were economy of construction and maintenance, eventual life, suitability to surroundings, and speed of erection. These conditions could easily be fulfilled by prestressed concrete and among the several tenders received for structural steel, reinforced concrete, and prestressed concrete, a prestressed concrete design was finally selected as being advantageous on all counts. This design, which it is proposed to describe in some detail, was the result of close collaboration between the tendering contractors, their Consulting Engineers, and Consulting Architects.

6.1.2. The original suspension bridge towers were supported on two masonry piers placed 150 feet apart in the bed of the river. These piers were naturally used to carry the new structure which took the shape of a three-span construction, the central span being 150 feet between supports and the side spans 35 feet from central lines of support to the banks.

6.1.3. If the job had been planned on entirely new lines, an effort would have been made to increase substantially the side spans and reduce the central opening in order to botain a better balance, structurally speaking. A central

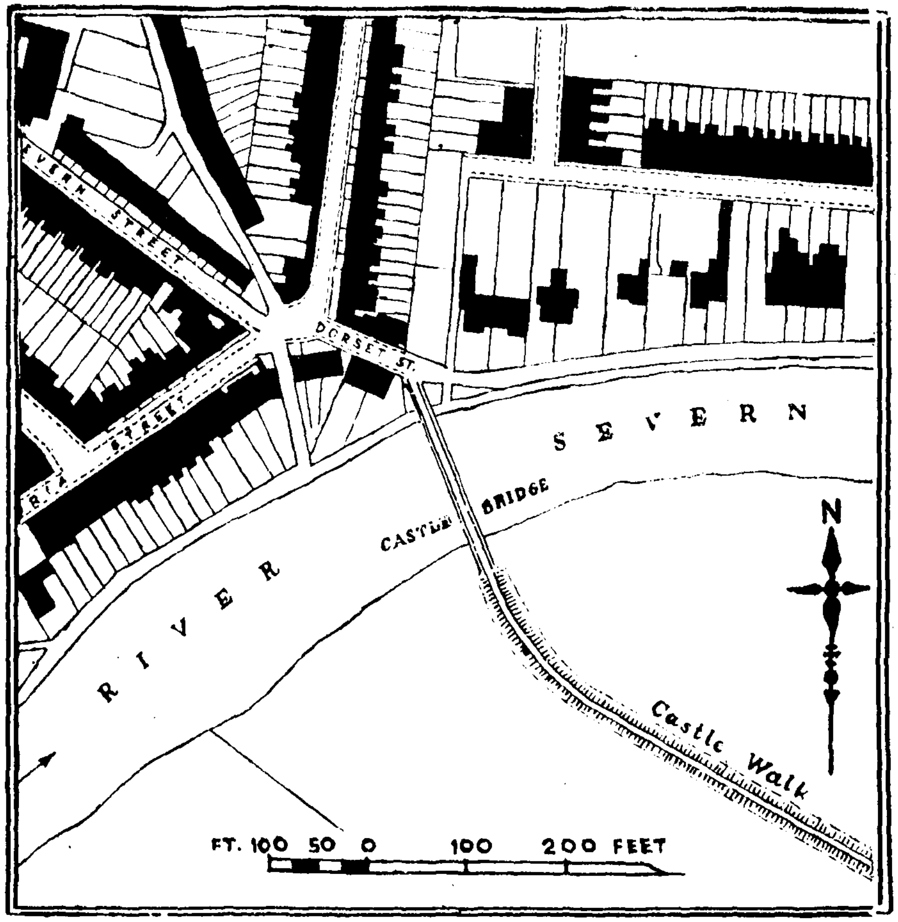


Fig. 2: Site of the Castle Foot-bridge over the Severn.

Photo 8: Castle Bridge Shrewsbury—General View.

span longer than the approaches by about one third would have been nearer to the most economical proportions. As a 150 feet central span was, in fact, so much greater than the approach spans, the provision of a heavy permanent counterweight on each bank was necessary. These counterweights tied the structure to each bank, as it would have been expensive to put them on rollers. In view of this, it was necessary to cater for temperature movements by breaking the continuity at the central span and providing for simply supported girders at midspan, resting on two cantilevers. The suspended girders could be precast on the bank and hoisted in position and this design fell in with the wishes expressed by the Contractors to avoid temporary piling and staging at midstream.

6.1.4. A general elevation of the final work is shown in Plate II. The suspended span measures 60 feet between central lines of support. It rests on two cantilever spans projecting 45 feet beyond their pier supports and anchored to their counterpoise blocks at 35 feet 6 inches back from these same supports.

The work measures 247 feet over the counterpoise blocks.

In the central span, the soffit line starts from the pier supports at the level of the maximum flood and assumes a parabolic arched curve rising to 7 feet above flood level at midstream.

The road level starts from existing road levels at the bank and ascends in a parabola to a maximum rise of 5.4 feet at midstream.

The structural depth is 7 feet 3 inches at pier supports and 3 feet at midspan. A 4 feet 6 inches depth is provided at the counterpoise block, mainly for shear resistance and the soffit line at the approaches is slightly curved to harmonise with the structural lines of the centre span.

6.2. Suspended Span

6.2.1. It was decided at an early stage to erect the suspended span by means of precast prestressed girders, with a precast slab deck. The individual weight of girders was to be kept within 10 tons for handling purposes. Four girders are provided on an overall deck width of 12 feet 6 inches and they are given a channel section instead of the more familiar I section. The channel section was selected in this case since two beams could be placed back to back on the central line of the deck, with the two outer beams presenting to the view a plain vertical face which, it was felt, would improve the appearance of the structure.

Photo 9: Castle Bridge, Shrewsbury—General View.

6.2.2. The girder depth is 2 feet 9 inches at midspan, increasing to 3 feet 5 inches at supports. The flanges are 5 inches thick and 12 inches wide and the web 6½ inches thick. The prestressing is given by six Freyssinet cables of twelve 0.2 inch diameter wires each. The width to span ratio being only one in sixty, special care was taken to place the cables on a vertical line passing through the centroid of the section as a slight horizontal eccentricity of the prestressing force would have caused a considerable lateral deflection. It was this consideration that led to the adoption of a 6½ inches thickness for the web, whereas a much smaller thickness would have been sufficient for strength.

The beams contain a secondary mild steel link reinforcement, sufficient to bind the pre-stressing cables in position.

6.2.3. **Bearings**. Each beam is provided with a 2 inch diameter steel roller bearing at one end and a steel rocker bearing at the other end.

6.2.4. Vertical web stiffeners were provided at 9 feet centres with horizontal starter bars left at the top. Struts measuring 9 inches x 9 inches were cast in situ at these points to tie the beams in pairs.

6.2.5. **Services.** Small precast reinforced concrete members, resting on the bottom flanges, were added at 3 feet centres to support a 9 inches diameter gas main, a 6 inches diameter water pipe, four 3 inches diameter electric cables and a telephone duct.

6.2.6. **Deck.** The space between each pair of beams is spanned by precast deck slabs, 3 inches thick. These are connected together by a 1 foot 6 inches band of deck cast in situ on top of the inner beams and also by casting in situ of the two outer kerbs supporting the railing.

6.3. Cantilever Span

6.3.1. The cantilever beams measure 80 feet 6 inches in length, including a 45 feet projection beyond the pier support and towards midstream. Three beams are provided on a deck width of 12 feet 6 inches. The central beam, rectangular in section, is 1 foot 4 inches thick and its depth varies to a maximum of 7 feet over the pier. The two outer beams, also rectangular, are 8 inches thick and their depth varies as in the central beam. Small projections are provided at the top of these beams, giving to the precast deck slab the same seating area as that given by the precast girders in suspended span.

6.3.2. There are ten pre-stressing cables in each of the outer beams and twenty in the central one. These cables are placed at

Photo 10: **Castle Bridge, Shrewsbury—Placing prestressing cables in cantilever beams.**

the top of the section over pier supports and curve down on either side towards the beam ends where the anchorage cones are placed. (See photos 10 and 11). In this particular case, the beam ends were not large enough to accommodate all the anchorage cones some of which were placed at the soffit. The beam ends, containing their anchorage cones, were made in the form of precast blocks to facilitate construction but the beams themselves were cast in situ.

6.3.3. The cantilever beams are tied together at the top by small cross beams placed 9 feet apart; and bottom cross beams, placed 3 feet apart, are used to carry the services. The 3 inch precast deck slabs, in situ filling, and kerbs are identical to the corresponding items in the suspended span.

6.3.4. A 3½ inch diameter steel roller, 11 feet 6 inches long, provided as bearing over the pier allows the cantilever construction to pivot freely about its pier support and affords complete freedom of movement in respect of temperature and prestressing strains. The roller is sandwiched between two steel plates which distribute the bearing pressure on the concrete. The sheathed cables and anchorage cones were delivered to the site ready made.

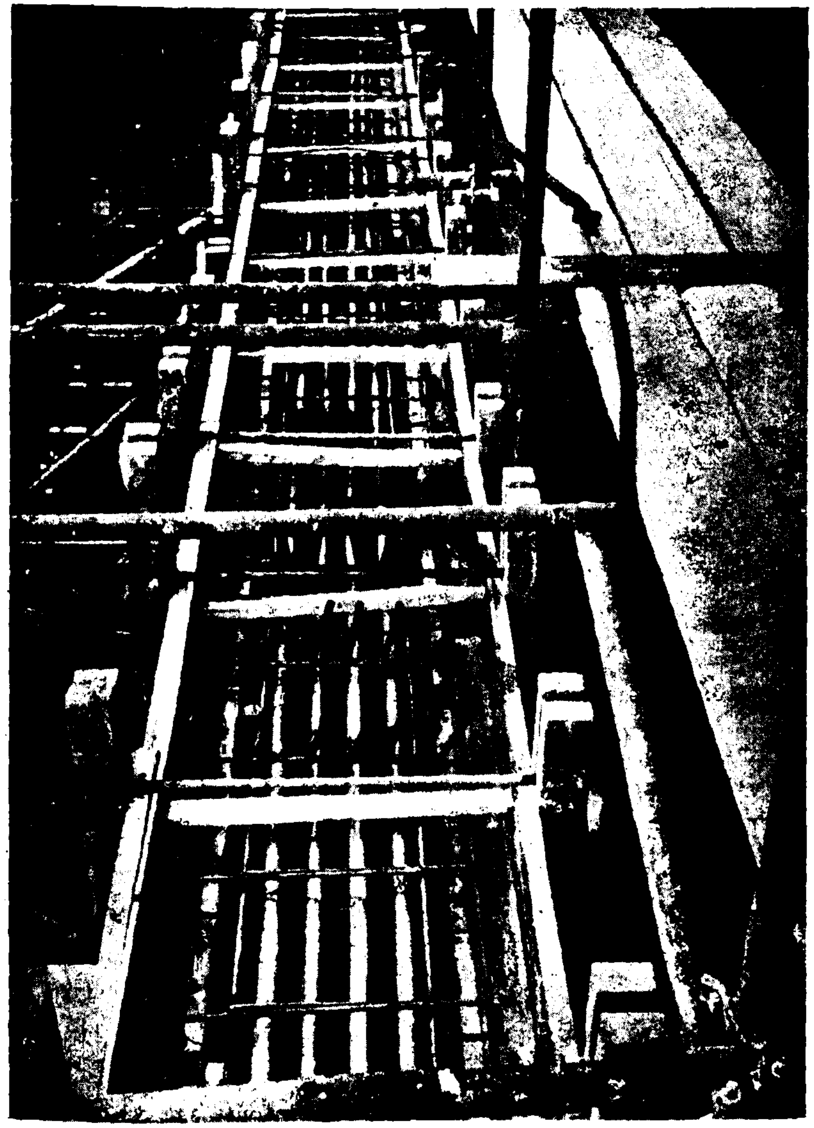


Photo 11: **Castle Bridge, Shrewsbury—Cables in cantilever beam on centre line of bridge.**

6.4. Counterpoise Blocks

6.4.1. Each counterpoise block, made of mass concrete, measures 13 feet by 13 feet in plan and 10 feet 6 inches in depth. It is made sufficiently heavy to prevent uplift even when submerged by exceptional floods. Had this heavy concrete block been allowed to rest squarely on its base, it would have provided, in some unknown measure, a degree of fixity to the beam which would in turn have affected both the stress produced by direct loads and the counter stress created by prestressing. It was found desirable in this case to determine the problem clearly by restoring complete freedom to slope deflection and this was done simply by allowing the counterpoise block to rock on its base.

6.5. Quality of Material and Design Stresses

6.5.1. Particulars of materials and stresses are similar to those given previously. (Vide Para 5.7.) Stress diagrams at the centre of the suspended span and over the pier are given in Plate III. It will be realised that the stresses caused by prestressing can never act alone. The prestressing causes the beam to camber and lift itself off its temporary support, thereby bringing the deadweight into action. In particular, the large tensile stress of 1080 lb per sq. in. caused by prestressing at the bottom edge of the cantilever section is not alarming in any way since the compression induced at this same point by the deadweight is sufficient to counteract it.

6.6. Execution

6.6.1. The method devised by the Contractors for erecting the suspended span is particularly interesting. The four, 60 feet channel sections were manufactured on the bank at about 100 feet upstream of the new bridge. A cutting was excavated in the bank, sloping towards the river to form a slipway (Photo 12). A concrete slab was laid and the beams were cast and prestressed in a manner similar to that adopted for the Castle bridge previously described. The beams were then rolled head-on down the slipway towards the water edge where a timber-piled gantry had been built. The downwards movement was controlled by a pair of winches, one pulling and the other retarding the beam. The gantry with its overhead chain block was then used to load one beam into two pairs of pontoons which were floated downstream to the bridge site. The cantilever structure had meanwhile been cast in situ. Two five ton lorry mounted cranes, weighing 16 tons each, travelled under their own power to the ends of the cantilevers and hoisted the beam into its final position. (Photo 13). The whole operation for each beam took just over half a day.

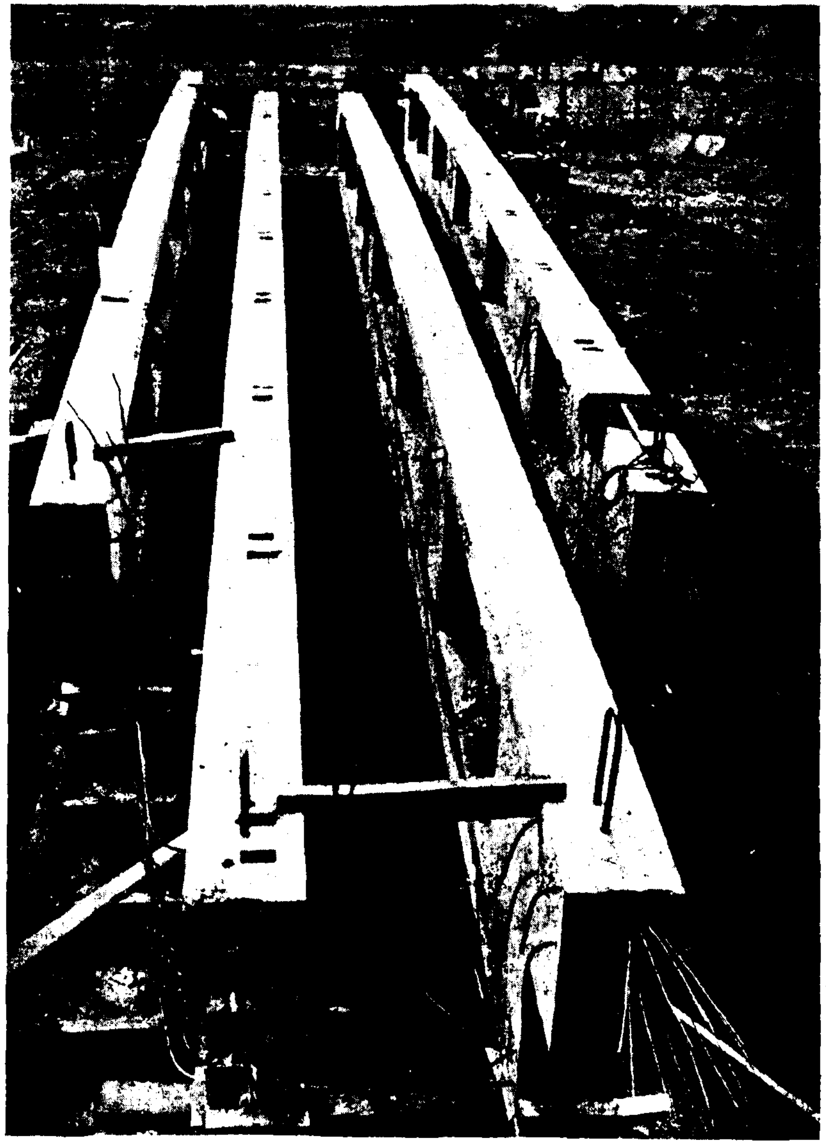


Photo 12: Castle Bridge Shrewsbury—Suspended span girders ready for launching.

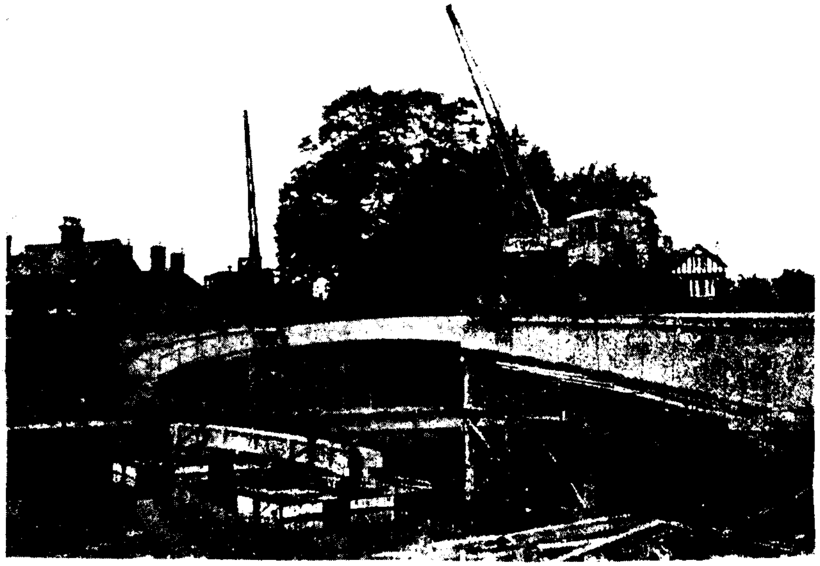


Photo 13: Castle bridge Shrewsbury—Lifting of suspended spans girders with two cranes placed at nose of cantilever beams.

6.7. Costs

6.7.1. As has already been stated, the old foundations had been utilised in the construction of this bridge. The cost was £ 3-15-0 or about fifty rupees per square foot of deck exclusive of foundations.

7. ACKNOWLEDGMENTS

7.1. The New Castle Bridge at Abercrave was built under the authority of Mr. J. M. Richards, M. I. Mun. E., County Surveyor of Brecon. Messrs. R. M. Douglas (Contractors) Ltd., were the Contractors, and Messrs. L. G. Mouchel & Partners Ltd. Civil Engineers, the Consulting Engineers.

7.2. Mr. F. R. Dinnis, A.M.I.C.E., A.M.I.Mun.E., Borough Surveyor of Shrewsbury, authorised the erection of the New Castle Bridge over the Severn, on a tender sponsored by the Contractors, Messrs. Taylor Woodrow Construction Ltd., Messrs. L. G. Mouchel & Partners Ltd., Civil Engineers, and The Prestressed Concrete Co., acted as Consulting Engineers to the Contractors, who secured also the advice of Messrs. T. P. Bennett & Son, Consulting Architects.

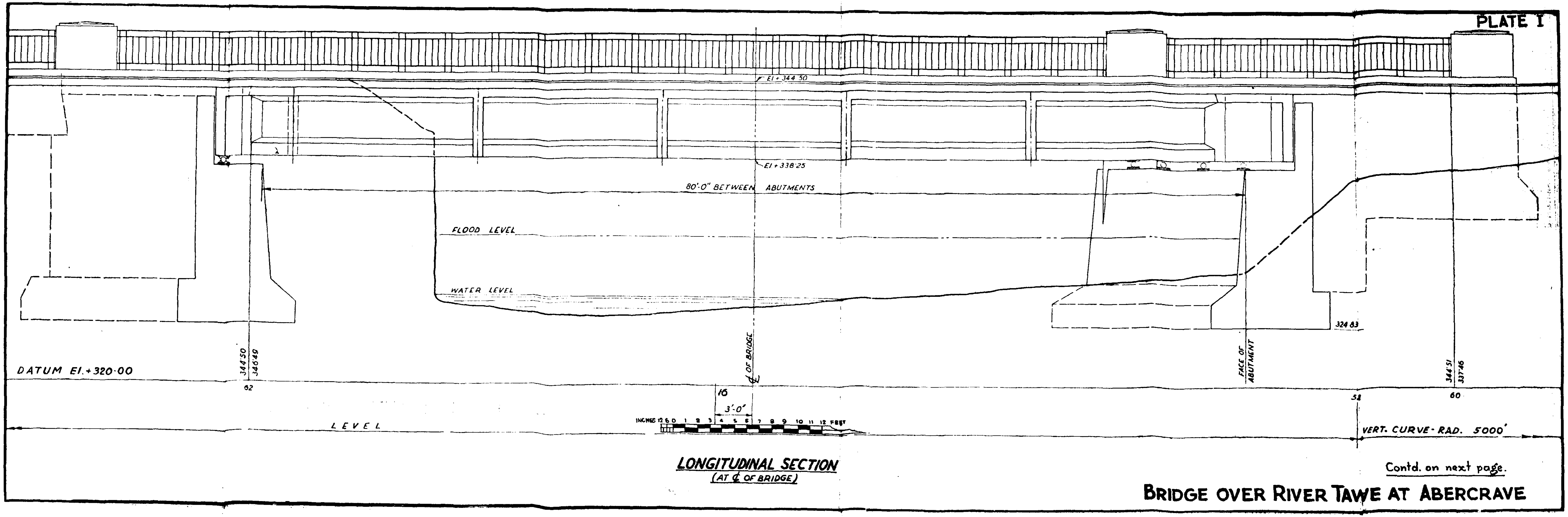
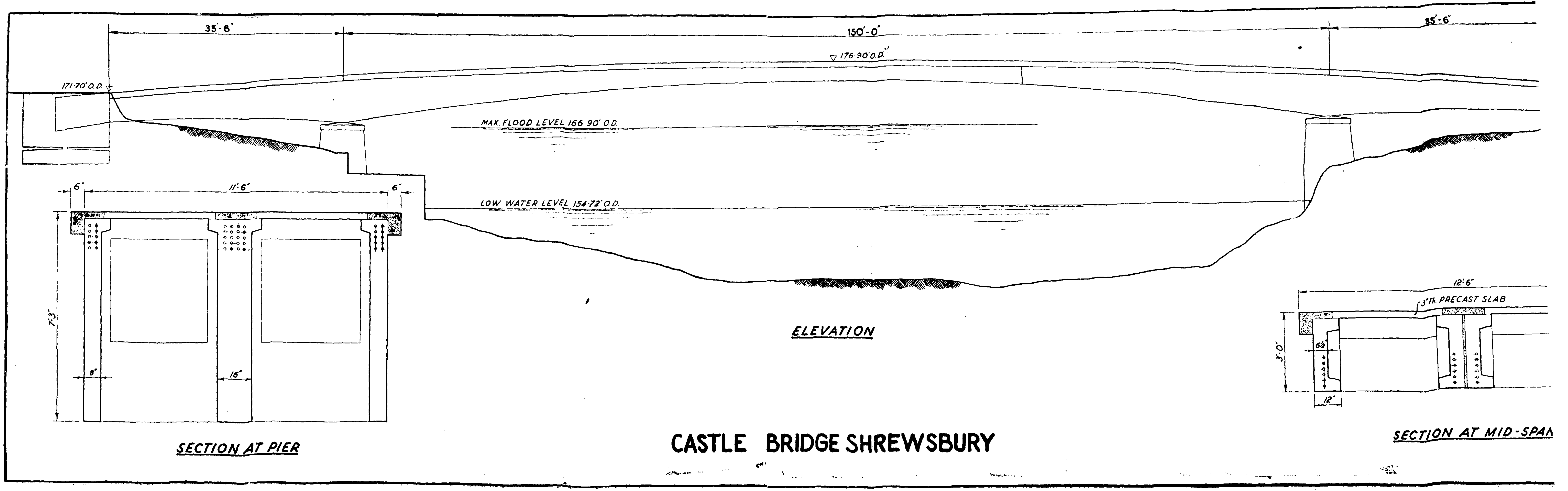


PLATE I
El + 344 50
El + 338 25
80'-0" BETWEEN ABUTMENTS
FLOOD LEVEL
WATER LEVEL
324 83
DATUM El. + 320·00
344 50
346 49
62
℄ OF BRIDGE
FACE OF ABUTMENT
344 51
337 46
58
60
16
3'-0"
LEVEL
VERT. CURVE-RAD. 5000'
LONGITUDINAL SECTION
(AT ℄ OF BRIDGE)
Contd. on next page.
BRIDGE OVER RIVER TAWE AT ABERCRAVE



SECTION AT PIER

CASTLE BRIDGE SHREWSBURY

SECTION AT MID-SPAN

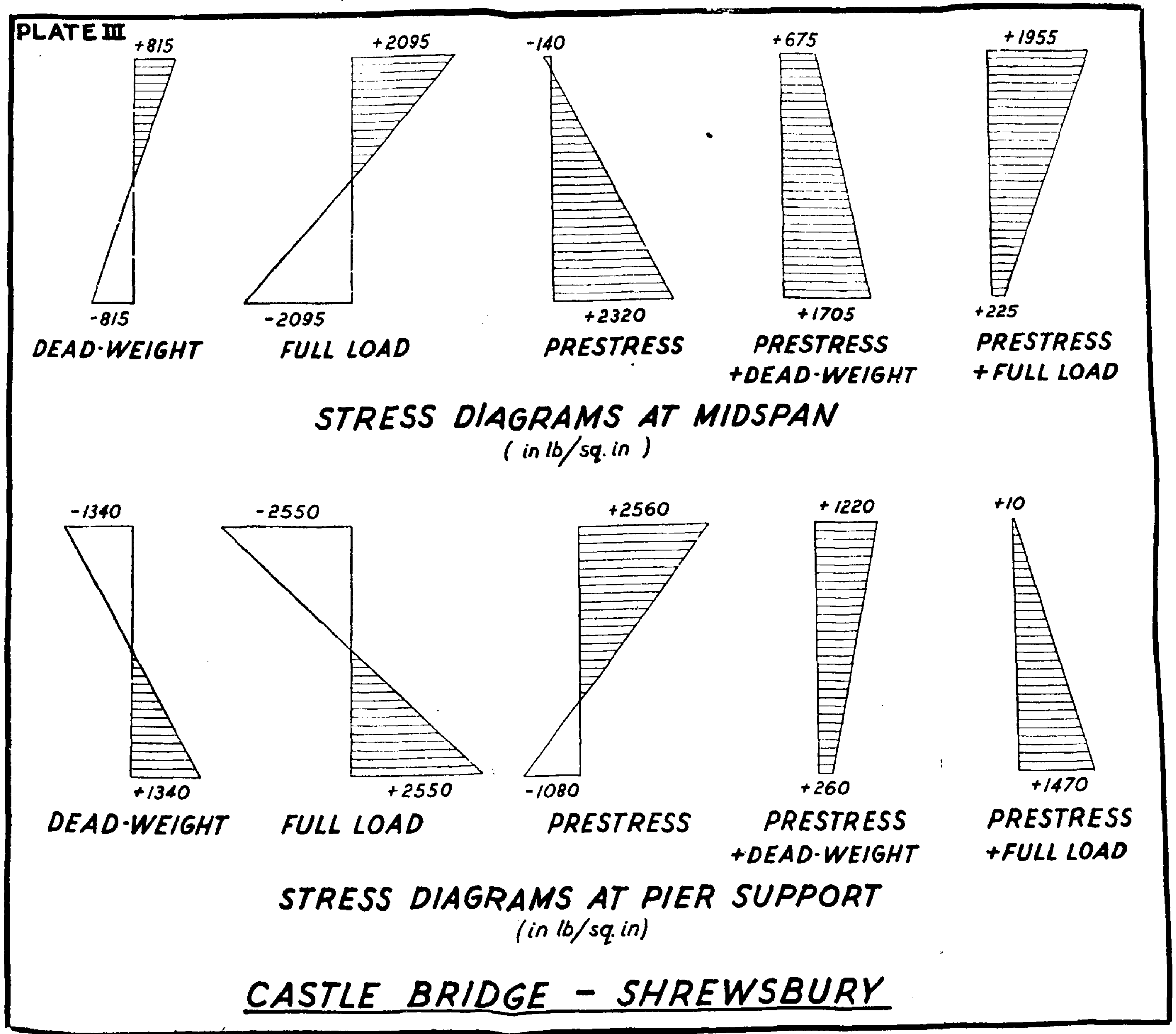


PLATE III
+815
-815
DEAD-WEIGHT
+2095
-2095
FULL LOAD
-140
+2320
PRESTRESS
+675
+1705
PRESTRESS +DEAD-WEIGHT
+1955
+225
PRESTRESS +FULL LOAD
STRESS DIAGRAMS AT MIDSPAN
(in lb/sq. in)
-1340
+1340
DEAD-WEIGHT
-2550
+2550
FULL LOAD
+2560
-1080
PRESTRESS
+1220
+260
PRESTRESS +DEAD-WEIGHT
+10
+1470
PRESTRESS +FULL LOAD
STRESS DIAGRAMS AT PIER SUPPORT
(in lb/sq. in)
CASTLE BRIDGE - SHREWSBURY

Paper No. 163.

"Soil-Cement in Road Construction"

by

K. F. ANTIA, B.SC. (ENG.) HONS., M.I.E., A.M.I. STR. E., F.P.W.I.

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SYNOPSIS

The Paper describes in detail the properties of soil-cement and advocates its use in road construction as the most economic specification for certain purposes. As soil forms more than 85 per cent of soil-cement the importance and methods of soil classification are discussed. Performance tests and field control factors are given, as well as some rough and ready methods for quick tests of soil-cement. After describing the various methods of soil-cement construction the author gives a suggested code of practice.

1. INTRODUCTION

1.1. Description of soil-cement

1.1.1. Soil-cement is an intimate mixture of measured amounts of Portland cement, pulverised soil, and water, adequately compacted to a high density to form a hard, stable, and durable mass. The manner in which cement acts as a binding material in a soil-cement mixture is a matter of speculation. In the opinion of Housel, it forms a granular soil structure with little more than enough cement paste to join the particles at their points of contact. The soil structure is spot-welded, so to speak.

1.1.2. After the satisfactory hydration and crystallization of cement on hardening, an examination of the material reveals inter-connected grains of soils and hydrated cement in a somewhat chain like structure held rigidly in place forming a hard and stable mass.

1.1.3. In contrast with concrete, where the cement mortar coats the aggregate, in soil-cement the cement is coated by the soil ; for this reason a different result, as may be expected, is obtained.

1.1.4. Apart from the hard and durable soil-cement structural material, cement may also be used as an admixture to change the undesirable characteristics of certain clayey soils to a marked degree, modifying it to a more favourable construction material. The essential difference is that, while the soil-cement behaves as a rigid structural material throughout its life, in the cement modified soil, behaviour as a hardened mass is only for a limited time, depending on the severity of the weathering agencies. The mixture eventually disintegrates into a granular mass and reverts to the state of admixture before hardening. In the following discussion, reference is made only to the hard, durable soil-cement structural material.

1.2. Behaviour as a road material

1.2.1. As early as 1917 soil and cement mixed with some chemicals were being tried for use as a road material for road construction in the U.S.A. In 1922 soil and cement, in combination with certain proprietary chemicals, were mixed with large quantities of water and used for making roadways in the States of Iowa and South Dakota. These were followed by field experiments in other States where cement was added to the soil forming the road bed to increase its stability. Although the results of these early experiments were not perfect, sufficient information was obtained to indicate that soil and cement could be intimately

mixed to produce satisfactory construction for light traffic roads and that further investigation of these mixtures might establish basic data regarding their use. In 1935 the Portland Cement Association undertook an extensive research programme to establish the fundamental characteristics of soil-cement mixtures. More than three thousand soils covering the range found in the United States were examined with the result that a scientific procedure that ensured dependable result was established.

1.2.2. Generally, soil-cement bases are regarded as being flexible, but they have considerable slab strength which permits the wheel loadings to be distributed over a far greater area than with other types of flexible pavements of the same thickness. It has been the general experience that even on the poorest of subgrades a 6 in. soil-cement layer has at least the same load distributing capacity as one 8 in. layer of crushed stone, first class gravel, broken stone or macadam or other types of stabilized base. Soil-cement has not, however, so far achieved widespread use as a surfacing material owing to the fact that during rainy weather it has not withstood continued heavy loading nor has it been able to prevent the formation of pot-holes and ravelling caused by the abrasive action of traffic. Hence it has been used mainly (a) as a base course on which a suitable wearing surface has been provided ; and (b) as a sub-base in concrete construction where it has provided an excellent surface on which to place the concrete and has enabled the required thickness of concrete to be reduced.

1.3. Extent of use abroad and in India

1.3.1. Real progress with this form of construction for roads has been made chiefly in the U.S.A and Australia, though considerable laboratory study and experimental work are in progress in many other countries. Upto the end of June 1952, 90 million square yards had been paved with soil-cement in U. S. A. In December, 1951 the area of soil cement pavement was 85 million square yards distributed as follows :

Roads and Streets	59,654,326 sq. yd. equivalent to 8,474 miles of 12 ft wide road
Airports	23,001,750 sq. yd.
Miscellaneous	3,056,831 sq. yd.

1.3.2. In **Australia**, up to the end of 1951, more than 7,000,000 sq. yd. of soil-cement pavement had been laid. This figure is impressive when it is realised that most of it has been laid since 1945.

• The figure for the United Kingdom is 662,000 sq. yd.

1.3.3. In India soil-cement as a constructional material for roads has not been used though it has been used for low-cost housing in the Punjab and in Bangalore. For roads and pavements the following experimental stretches have been tried.

(*a*) At Kusmi aerodrome near Gorakhpur in Uttar Pradesh a soil-cement strip, 30 feet wide and 60 feet long and 6 in. compacted depth, was laid in August 1942. The soil was light, sandy loam with a clay content of 12 per cent; the percentage of cement used was 8.33 by volume and an optimum moisture content of 15 per cent by weight was adopted. The equipment used was of local manufacture, such as indigenous hoes, hand tampers and a light roller. The material was mixed at the side of the road on mixing platforms and laid on the consolidated subgrade and compacted. The strip was loaded for test a month after laying. Immediately before the test there had been a heavy shower. The test load on 8 points in the pavement, of 8 tons per sq. ft., transmitted through 6 in. wooden cubes did not produce any signs of subsidence or cracking. Continuing the loading to 14 tons per sq. ft., only at two spots there were signs of settlement, these being due to uneven levelling of the supports. The load on one of them was left for a further period of 18 hours and, on removal of the load, it was found that there was only local sinking without any signs of cracking in the neighbourhood.

(*b*) A soil-cement base course of fairly good length has been laid recently on the Chidambaram—Bhuvanagiri road, the foundation for the 4 in. 1 : 2 : 4 concrete road slab comprising of two 4 in. layers of soil-cement topped with 4 inches of water bound macadam. The bottom 4 in. layer is stabilised with two per cent cement by volume and the next 4 in. layer with 4 per cent cement by volume, the soil in both cases being sandy. The work was completed in 1950. (Fig.1.)

(*c*) On the Ramnad—Mandapam road, a 3 in. layer of soil-cement with 8 per cent cement by volume and a sandy soil was adopted as a base for laying 1½ in. thick asphaltic concrete. Two miles of this road had been completed at the time of writing this paper. (Fig.1.)

(*d*) Also, on the Marina in Madras, several small stretches have been attempted for the side-walk with thickness ranging from ½ in. to 3 in. using varying percentages

of cement and of blending with borrowed material. According to reports, the results of these experiments do not appear to have been satisfactory.

1.4. Life and scope of soil-cement

1.4.1. The role of soil-cement in the works referred to in the previous section has been (a) either as a base course for concrete and bituminous pavements; or (b) as a structural pavement with a very thin coat of concrete or bitumen for light duty roads. Its use has been so well established, that specifications with minimum requirements of compressive strength have been evolved

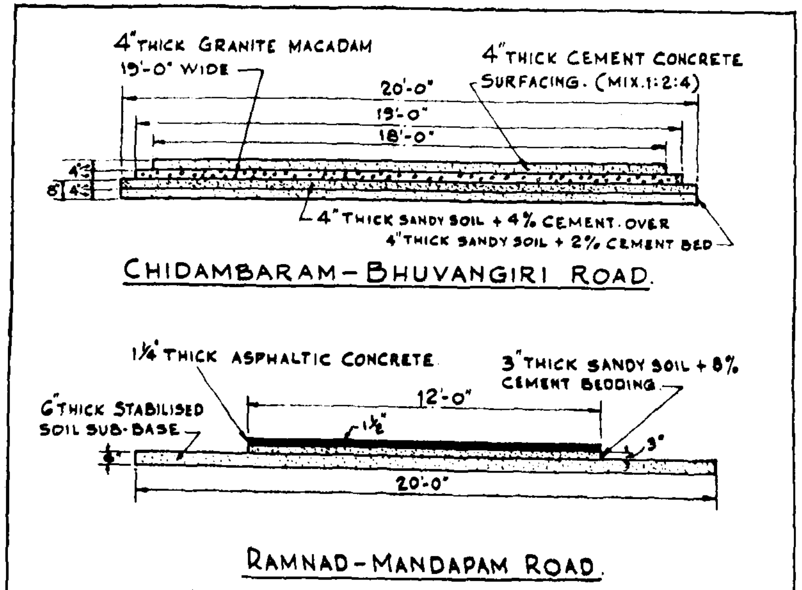


FIG.1

and are frequently used for soil-cement work. Soil-cement pavements constructed abroad have withstood the test of time and have given satisfactory performance to a degree even better than most other road materials. The following are but a few examples:

(*a*) About 10,000 sq. yd. of roads, 9 ft. wide, giving access to small holdings owned by the Surrey County Council were re-surfaced in 1943-44 with a layer composed of a 4 in. compacted thickness of clayey sand bound with 6 per cent of Portland cement and surface dressed with tar and chippings. The cost of the work was much less than the cost of water-bound macadam and it has been found to be less costly in maintenance.

(*b*) New roads have been constructed in Surrey using a sub-base of stabilised soil, cement being used as the binder, with both concrete and tar-macadam surfacings. The results in all cases have been satisfactory and the cost less than that of hard-core sub-bases.

(*c*) A new carriage-way was constructed in Surrey in 1943-44 having a 1 : $2\frac{1}{2}$: 5 unreinforced concrete surfacing 3 in. thick compacted on a sub-base of stabilised soil 8 in. thick compacted. The surfacing was mechanically compacted and was laid almost immediately after the laying of the sub-base so as to obtain a key. The sub-base consisted of a clay-sand from a nearby pit, mixed in a paddle type mixer with 6 per cent Portland cement. The footpath on this new road was constructed with the same cement-soil mix compacted to a thickness of 2 in. Although the road is used by heavy lorries its condition after so many years is satisfactory.

(*d*) A 6 in. pavement of soil-cement made of 10 per cent cement mixed with sandy clay soil in the Oberon Street in Sydney, Australia, was laid in 1949; since then it has been withstanding very heavy traffic, including double decker buses.

(*e*) A runway of $3\frac{1}{2}$ in. thick soil-cement with 8.8 per cent cement, laid on a layer of 3 in. thick untreated gravel was constructed in 1942 at Gawler, South Australia; it is satisfactorily providing standby service for wheel loads in excess of 30,000 lb. In this instance, in spite of the absence of any seal coat, there is not the slightest evidence of deformation nor has there been any change of shape.

(*f*) The use of soil-cement in the U.S.A. has however been very widespread, dating back to 1935, even for larger projects. It has ranged from soil-cement using about 2 to 3 per cent of cement for improving the soil to structural soil cement wherein the minimum requirements of strength at 28 days are stipulated at 1,000 lb. per sq. in. Surveys of the condition of old soil-cement roads in service show that more than 15 years of service can be easily expected, the replacement of the bituminous seal coat or surface painting alone being necessary during this period. Experience has proved that even these surface repairs on soil cement bases are at the barest minimum compared to those on other flexible or stabilized bases.

Since both in the U.S.A. and Australia traffic consists of only pneumatic tyred vehicles, experiments there have been with light bituminous topping and no attempt has been made to provide a light duty concrete finish for withstanding wear due to steel tyred traffic. A well designed and properly constructed soil cement layer hardens and remains a non-yielding structural material under all conditions. The bonding of a thin 2 in. concrete topping to the soil cement course so that the two may together act as an integral whole should not therefore present any difficulty, especially when laid before the complete hardening of the soil cement mass. Such a pavement will withstand even the heaviest axle loads so long as their frequency is not high and will be ideal for medium traffic roads carrying up to about 1,000 tons a day.

Other fields suitable for soil-cement construction are streets in residential areas of cities, estate roads, side strips of heavy traffic roads, foundations of concrete and bituminous roads in new construction, and aprons of airports., etc.

1.5. Basic reasons for adopting soil-cement

After reviewing the cases of the use of soil-cement in pavements a study of its advantages brings out the following salient points :

(a) Since soil-cement is made by mixing cement with the soil existing at site, hauling costs are minimised, expensive imported materials are avoided and where suitable granular materials are not readily available and their costs are relatively high substantial savings are realized.

(b) The process of construction is sufficiently simple to be carried out with available light equipment except when very speedy construction is attempted. There are not several stages of construction as in other alternative base course construction methods. The construction is also relatively much speedier.

(c) There is minimum interruption to traffic during the construction period, and in most instances the hardened soil-cement can be put under traffic before any surfacing is provided.

In this system, the soils are classified into different groups according to their capacity to support wheel loads. This capacity varies with the reactions of the soil both to loads and climatic changes. The reactions depend upon five basic properties of soil, namely cohesion, internal friction, compressibility, elasticity and capillarity. Among the physical characteristics or soil constants which are used as aids in identifying these important highway sub-grade properties are the Liquid-limit, the Plastic limit and the Plasticity index all based on the plasticity of soils.

2.1.3. **Plasticity of soils**

Plasticity is a major characteristic of all 'cohesive' soils, that is, soils containing an appreciable proportion of clay particles. It is the property through which rapid deformation takes place in a material without noticeable elastic recovery and without cracking or crumbling.

Of the many methods, mainly arbitrary, which exist for determining the plasticity of soils, that of the Swedish soil scientist, Atterberg, is widely used in soil engineering. In this method the plasticity of a soil is measured by the range of water content over which the soil can be considered to remain in a plastic state. A number of consistencies for plastic soils, varying with water content, with their limits are specified as follows:

Consistency	Limits
Liquid stage in which particles are held in suspension	The upper limit of viscous flow at which a mixture of clay and water flows almost as a liquid.
Semi-liquid stage in which the soil has the nature of a viscous fluid	The lower limit of viscous flow defined as the **liquid limit.**
Plastic stage in which the soil exhibits plasticity.	The lower limit of the plastic stage defined as the **plastic limit**.
Semi-solid stage	The lower limit of volume change or the **shrinkage limit** at which further loss in moisture does not cause a loss in volume.
Solid state	

The limits are all expressed as the percentage water content of the soil at that particular stage.

2.1.4. **Soil constants**

The liquid limit of a soil is the minimum moisture content that brings it to the liquid state and is measured by the amount of moisture at which the soil is sufficiently fluid to flow a specified amount when lightly jarred 25 times in a standard apparatus. The plastic limit gives the moisture content at which the soil just becomes plastic and is measured by the moisture content at which the soil can be rolled into threads 1/8 in. in diameter without breaking.

The numerical difference between the liquid and plastic limits is termed the **Plasticity index** and indicates the range of the moisture content over which the soil is in a plastic condition.

2.1.5. **Significance of soil constants**

The liquid and plastic limits are both dependent on the amount and type of clay in a soil, but the plasticity index is generally dependent only on the amount of clay present. It follows that information regarding the type of clay in the soil may be obtained by considering the plasticity index in relation to the liquid limit, while the amount of clay present may be deduced, within broad limits, from the value of the plasticity index. These facts form the basis for the soil classification systems for cohesive soils based on the plasticity tests.

At the plastic limit the soil particles are lubricated sufficiently to slide over each other, but they still possess cohesion. At the liquid limit the soil particles are separated by water to such an extent that there is no cohesion. The plasticity index is a measure of the cohesion of the soil under damp conditions. The higher the plasticity index, the higher the clay content of the soil. Cohesive soils may be classified by their plasticity index as hereunder.

Soil type	**Plasticity Index**
Heavy clay	Greater than 20 (Upto 45 or more)
Silty clay	10-20
Sandy clay and clayey sand	Less than 10

The soil constants also enable variations in the five following properties to be ascertained—

Properties.	Soils of equal LL with increasing PI	Soils of equal PI with increasing LL
Compressibility	About the same	Increases
Permeability	Decreases	Increases
Rate of volume change	Increases	...
Toughness near PL	Increases	Decreases
Dry strength	Increases	Decreases

Although it is not possible for any mathematical limits to be determined with a combination of the gradation and physical test constants for the case of pulverisation, for type and weight of rollers for compaction, and for quantity of hygroscopic moisture in soil, certain general deductions can be made.

A low liquid limit (say 35 per cent and under) and a low plasticity index (say 15 or under) are indicative of easy pulverisation.

Likewise the gradation of soils provides information on which to base the choice and weight of rollers. For example, soils with less than 10 per cent silt and clay are difficult to compact under a sheepsfoot roller; on such soils pneumatic tyred rolling equipment may be necessary to supplement the sheepsfoot roller to obtain proper density.

In soils with combined silt and clay contents of over 50 per cent, a predominance of clay leads to an increase in the plasticity index to over 20 and the liquid limit to over about 45, which necessitates special attention for pulverisation. On the other hand in the case of soils with combined silt and clay contents of over 50 per cent, and with a predominance of silt content, pulverisation would be simple because of natural friability of silt particles.

2.1.6. Particle size

In addition to these soil constants, the particle or grain size of the soil as determined by mechanical analysis is of considerable importance.

The chief value of the soil constants and particle size as a means of identifying soils lies in the relation existing between them rather than in the individual test results considered separately. This method of classification provides for certain overlapping of soils of different groups but by itself does not provide a rigid measure of soil behaviour. Thus some soils of a particular group may have some of the characteristics of other groups. The suitability of a particular soil for construction purposes and any drawbacks can be judged on the basis of its various physical characteristics and the relation between these characteristics rather than on the fact that the soil falls into a particular group.

2.1.7. Method of determining soil constants

The method of determining the above mentioned soil constants is based on the standardised tests of the American Society

for Testing Materials indicated below:

Preparing soil samples	—	A.S.T.M.	Designation No.	D 421–39
Mechanical analysis	—	,,	,,	D 422–39
Liquid limit	—	,,	,,	D 423–39
Plastic limit	—	,,	,,	D 424–39
Plasticity index	—	,,	,,	,,

2.1.8. U. S. P. R. A. Classification

Based on the relationship of gradation, liquid limit and plastic limit, soils have been classified by the U. S. Public Roads Administration into groups ranging from A–1–a to A–7–6 determined on their performance when used in highway work. Descriptive details of these groups and sub-groups are given in Table 1. The characteristics of these groups and sub-groups are given in Table 2. Chart 1 shows (a) the relation between liquid limit and plasticity index which determines the Group Index; (b) the relations between particle size and liquid limit and (c) particle size and plasticity index respectively. An empirical formula to determine what is known as Group Index has been devised to evaluate approximately the variations in each group.

The formula for the Group index is $0.2a+0.005ac+0.01\,bd$ in which

a = that portion of percentage passing No. 200 sieve, greater than 35 and not exceeding 75, expressed as a positive whole number (1 to 40)

b = that portion of percentage passing No. 200 sieve, greater than 15 per cent and not exceeding 55 per cent, expressed as a positive whole number (1 to 40)

c = that portion of the numerical liquid limit greater than 40, not exceeding 60, expressed as a positive whole number (1 to 20)

d = that portion of the numerical plasticity index greater than 10 and not exceeding 30, expressed as a positive whole number (1 to 20).

The method of using these tables and the chart is explained by the following example.

2.1.9. Example

During investigations a certain soil was found to have the following physical characteristics:

(i)	Liquid limit	43.7
(ii)	Plastic limit	21.7
(iii)	Plasticity index	22
(iv)	Per cent passing No. 200 sieve	47

Proceeding from column 2 to column 12 of Table 1 and comparing the limits given in the columns for the various constants with the values for the particular soil in question, it is found that

(1) the liquid limit of the soil (43.7) is greater than 41;

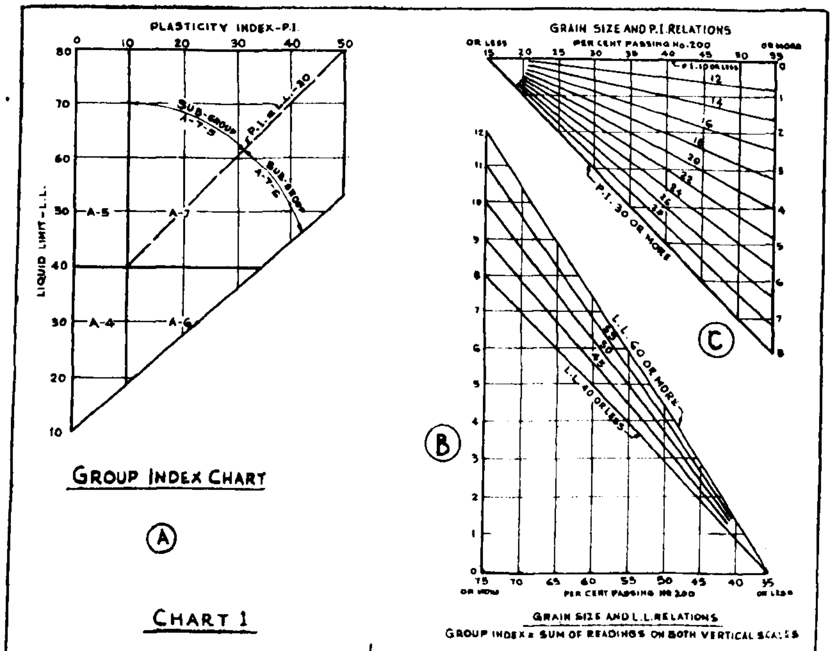
(2) the plasticity index of the soil (22) is greater than 11;

(3) the percentage passing No. 200 sieve is greater than 36.

Now these are the minimum values set for soils of group A-7.

If the plasticity index is equal to or less than liquid limit minus 30, the soil belongs to sub-group A–7–5; otherwise to sub-group A–7–6 (vide note *a* at bottom of Table 1). In this instance liquid limit minus $30 = 43.7 - 30 = 13.7$ and the plasticity index of the soil is 22 which is greater than liquid limit minus 30.

Therefore the soil belongs to group A–7–6.



GROUP INDEX CHART

CHART 1

GROUP INDEX = SUM OF READINGS ON BOTH VERTICAL SCALES

LIQUID LIMIT AND PLASTICITY INDEX RANGES FOR THE A-4, A-5, A-6 AND A-7 SUBGRADE GROUPS.

To determine the group index:

Reading on the vertical scale of chart No. 1 (B), corresponding to liquid limit $= 43.7$ and percentage passing No. 200 sieve *i.e.* $47 = 2.75$.

Reading on the vertical scale of chart No. 1 (C), corresponding to plasticity index $= 22$ and percentage passing No. 200 sieve *i.e.* $47 = 3.80$

Therefore Group index = Sum of the two readings = 6.55 or 7.

The Group index of the soil is shown in parenthesis after the group symbol thus: A–7–6 (7).

2.1.10. **Textural classification**

The other classification system which may be used with coarse-grained soils is the textural classification of soils devised by the U. S. Bureau of Chemistry and Soils. Textural classification is based exclusively on the particle size distribution of soils. The usual method is to classify soils on a triangular diagram vide Chart 2 which relates the percentages of the sand, silt, and clay fractions in the soil. In this method soils are primarily divided into three groups on the basis of clay content and subdivided as follows:

1. Soils with less than 20 per cent clay

1-a	Sands and gravels	Less than 15% silt and clay combined
1-b	Loamy sands	15 to 20% ,, ,,
1-c	Sandy loams	20 to 50% ,, ,,

2. Soils with 20% to 30% clay

2-a	Gravelly clay loam	25% or more of fine and coarse gravel.
2-b	Sandy clay loam	50% or more of sand and gravel combined.
2-c	Clay loams	Less than 50% silt and less than 50% sand and gravel.
2-d	Silty clay loams	More than 50% silt

3. Soils containing 30% or more of clay

3-a	Gravelly clay	25% or more fine and coarse gravel
3-b	Sandy clay	50% or more sand and gravel combined

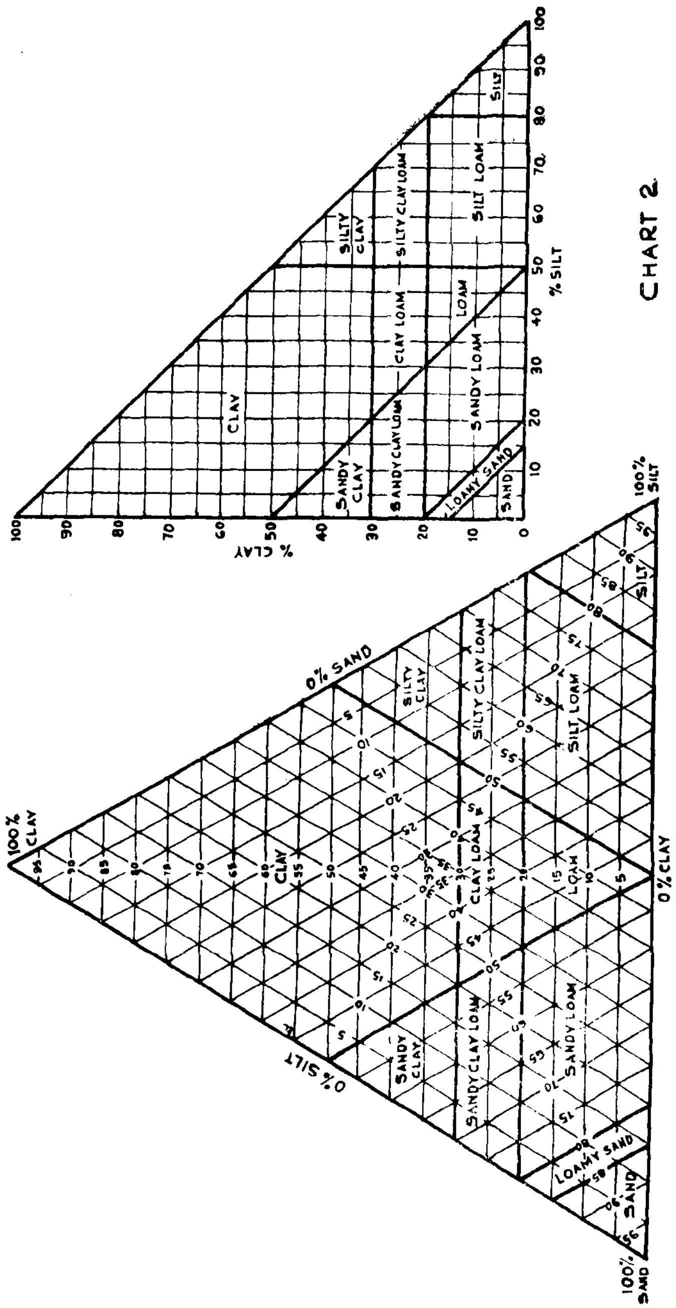


CHART 2

TWO TYPES OF CHARTS INDICATING TEXTURAL CLASSIFICATION OF SOILS.

3-c	Clay	Less than 50% silt and less than 50% sand and gravel combined.
3-d	Silty clay	50% or more of silt

The nomenclature given below refers only to particle sizes and does not take into account the mineral compositions.

Coarse gravel	3 in. to	No. 4	sieve	76.2 mm to 4.76 mm.
Fine gravel	No. 4 to	No. 10	,,	4.76 mm to 2 mm.
Coarse sand	No. 10 to	No. 60	,,	2 mm to 0.25 mm.
Fine Sand	No. 60 to	No. 270	,,	0.25 mm to 0.05 mm.
Silt	...	...	,,	0.05 mm to 0.005 mm.
Clay	...	...	,,	0.005 mm to 00 mm.
Colloids	...	...	,,	.001 mm to 00 mm. also included in clay fractions.

After determining the soil classification the various tests indicated in para 2.2 are carried out to determine the quantity of cement and of water required for stabilisation.

2.1.11. Hygroscopic moisture

The water taken up by the particles of a soil from the surrounding atmosphere is called hygroscopic moisture.

The amount of hygroscopic moisture varies for a given soil with the temperature and relative humidity of the air. In general it increases with decreasing grain size. For sands it is negligible. For silty soils, it is very small, yet sufficient to induce bulking, due to the extreme fineness of the particles. In clay it may amount to more than 5 per cent of the dry weight. In soil-cement work, therefore, it is important to take into account the hygroscopic moisture content of the soil, when estimating the total water requirements. This is easily done by weighing a quantity of soil, drying it in an oven at 105° C, and weighing again. The difference in the weights is the hygroscopic moisture.

2.2. Effect of cement admixture

2.2.1 Characteristics :

As already mentioned, the addition of cement to the soil, results in a mixture the behaviour of which under various stages of construction is totally different from that of either of the constituent materials. Hence, for successful results a study of the characteristics of the mixture is as essential as a study of those of the soil. The physical characteristics of the soil at best indicate certain trends, but these cannot be reduced into a simple formula to obtain a positive answer for a particular soil.

Tests have to be carried out after admixture with cement, because soils with identical gradation and physical characteristics may have altogether different chemical composition and contain varying amounts of organic impurities. It is probable that sulphates are the most harmful ingradients in soil. The effect on the setting of cement due to presence of sulphates may not be as harmful as the subsequent disruption of soil-cement caused by crystallization of highly hydrated salts in the pores. These effects have been observed in the laboratory but the extent to which they occur in the field is not known.

Again, it has been observed that the presence of organic materials, particularly in sand, prevent normal cement hydration and crystallization in concrete. Obviously similar results should be expected in the case of soil-cement mixtures, especially those in which top soils, which would contain vegetation in various stages of decomposition, are used. Considerable research data show that there is a definite tendency for compressive strength and durability of soil-cement mixtures to decrease, as the organic content of the soil increases. A practical safe limit of 2 per cent organic matter is normally recommended, although in some cases soils with 3 to 4 per cent organic impurities have been successfully stabilised. The standard calorimetric test used for determination of organic impurities in sand can be used for estimating the extent of organic impurities.

High percentages of organic impurities may indicate that an agent detrimental to cement is present, particularly in sandy soils. In any case, the durability tests described later will indicate whether the soil will harden satisfactorily with cement or not. Blending of borrowed soils or use of about ½ to 1 per cent calcium chloride has sometimes made it possible to stabilise sandy soils.

With regard to the physical characteristics of soil suitable for soil-cement work, while it may be stated that almost any soil which can be pulverised can be stabilised with cement, practical and economic considerations render certain soils such as heavy clays unsuitable for stabilization. As a result of extensive studies in various laboratories and in the field, the U. S. Highway Research Board suggest that, as a rough guide the following limits may be applied to particle size and to soil constants when the soil is used in soil-cement construction.

Particle Size:		Maximum size 3 in.
Passing No.	4 sieve	Minimum of 50 per cent
,,	40 ,,	15 to 100 per cent
,,	200 ,,	not more than 50 per cent

Physical Characteristics:

Liquid limit: Not more than 40.

Plasticity index: Not more than 18.

If in-situ soils do not fall within the above range, it may be economical to import soils from nearby sources which will enable less cement to be used and can be processed with greater ease. Also, certain soils, which may appear difficult to be pulverised can be made mellow and capable of being rapidly pulverised by prior absorption of moisture through wetting.

2.2.2. **Shrinkage**: A characteristic of soil-cement is that during the hydration and hardening of cement, shrinkage takes place which is greater in volume than any expansion that subsequently takes place with the usual seasonal exposures to moisture, heat, and cold. In soil-cement mixes with very high granular content however, volume change characteristics follow the pattern of those of concrete. Normally soil-cement begins to develop shrinkage cracks shortly after construction. The appearance of these cracks is an indication of the proper hardening of soil-cement work and, conjointly with the results of field tests of moisture and density, assures satisfactory soil-cement work. The pattern of these cracks varies from widely dispersed cracks with granular materials to close spaced cracks with the heavier clays. Field experience and laboratory experiments have confirmed that water creeping down these cracks does not soften or materially weaken the soil cement itself. Protection is however necessary to prevent the water entering and weakening the subgrade.

2.2.3. Moisture - density relationship

The second important factor affecting the value of soil-cement as a constructional material is the relationship demonstrated by R. R. Proctor that, with a certain moisture content, a particular soil will compact to a certain fixed density with a fixed compaction force. Also, with a fixed compaction force, there is an optimum moisture content that will give a particular soil the maximum compacted density and also the greatest load bearing capacity. Laboratory tests have confirmed that this relationship applies to soil-cement as well.

Under a constant force of compaction the density of a soil-cement mixture varies as the moisture content of the mixture changes. If the moisture contents are plotted against the corresponding dry densities, the points will usually form a parabolic or regular type curve, the peak of which will indicate the optimum moisture content and the maximum dry density. A typical, regular moisture-density curve is shown in chart 3.

The shape of the moisture-density curve can be explained as follows:

When soil-cement is relatively dry, there is considerable resistance to the soil particles sliding over one another and the

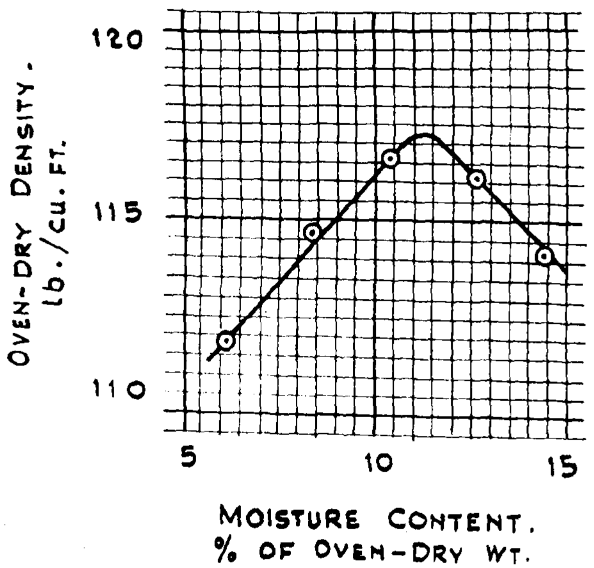


TABLE OF MOISTURE-DENSITY TESTS

TRIAL	DRY DENSITY Lb./Cu.Ft.	MOISTURE CONTENT.%
1	111·9	6·1
2	114·6	8·3
3	116·6	10·5
4	116·2	12·6
5	114·1	14·4

CHART 3

particles cannot arrange themselves to a maximum state of density. As the moisture content of the mass is increased, the moisture films about the soil grains increase in size, and, under compaction, act as a lubricant, assisting the particles to slide over one another with relative ease, resulting in greater number of particles being collected within a constant volume. Further water increments continue to increase the thickness of the moisture films around the soil particles, thereby increasing their lubricating qualities (or lessening their cohesive qualities), so that a greater density is obtained with standard compaction. After the maximum density is reached additional water increases the thickness of the film about each soil grain to such an extent that the grains remain apart even under the standard force of compaction resulting in a decrease in density. It is apparent that the moisture films about the soil grains at this stage still possess considerable stability and cannot be displaced with the compacting force being used.

With some heavy silt soils and some clay soils the dry density does not increase with the successive increments of water, but decreases or remains about the same. This density decrease is explained by assuming that the initial moisture in these cases either enters into the clay structure and expands the volume of the clay particles, or it acts as a practically solid film about the particles. In either case the moisture does not act as a lubricant but only increases the apparent size of the particles. There is, therefore, a decrease in density with initial increases in moisture. As the density continues to decrease with increases in moisture, a point is usually reached when the forces increasing the apparent size of the clay particles are equalised and subsequent additions of water act as a lubricant. At this stage the oven-dry density will increase or remain about the same with successive increments of water. The peak of this secondary bump or the levelling off place in the moisture density curve indicates the optimum moisture content for these soils.

The moisture-density relations of soil-cement are of the utmost importance, because (1) **in the laboratory** they make it possible to mould duplicate specimens with a fixed degree of compaction at any one moisture content and in each case the density of the specimens will be practically the same; and (2) **in the field** the moisture density relations are used for moisture and density controls to ensure satisfactory construction. In soil-cement work only the peak of the regular or parabolic type curve is used, since this gives the greatest density.

It has been found through experiments that the highest compressive strength is obtained when slightly lower moisture contents than optimum are used during compaction. For durability a moisture content slightly above the optimum yields better results. In practice compaction should be carried out at the optimum moisture content or at a slightly higher moisture content. The effect of moisture content in soil-cement cannot be compared with the water-cement ratio of concrete, as the moisture content has a negligible effect on the quality of soil-cement except in so far as it affects the compacted dry density.

2.2.4. **Cement content**. The third important factor is the influence of cement content. If cement is adequately hydrated in the mixture, an increase in the cement content increases the strength. The rate of increase of strength varies with the nature of soil, sandy soils showing a higher increase than clay soils. The presence of coarse aggregate, leads to a low compressive strength with low cement content but to a relatively high compressive strength with richer mixes. Another interesting feature is that over a fairly wide range of cement content, the initial rate of increase of strength is the same so that specimens tested within a day or two after preparation may give the same strength with say, 12 per cent cement content as with 8 per cent with increased period. In the majority of cases mixtures with higher cement content eventually give a higher strength. Hence compressive strength at more than seven days should be used for determining the quality of soil-cement mixture.

For any required strength a higher cement content is necessary with an increase in the silt and clay content except in the case of soils which lack in grading on account of being devoid of silt.

2.2.5. **Physical characteristics of soil-cement**

The physical characteristics of soil-cement may be illustrated by the following typical test data :

Soil used	Percentage of Cement	Seven-day strength lb per sq. in.		Coefficient of thermal expansion
		Compressive	Flexural	
Sandy	8	539	122	.0000069 per ° F
Silty	14	279	80	.0000062 per ° F
Clayey	12	402	95	.0000057 per ° F

Modulus of elasticity	100,000 to 1,000,000 lb. per sq. in.
Absorption	10 to 30 per cent over dry weight
Maximum density	85 to 135 lb per cu. ft.
Voids	25 to 40 per cent
Optimum moisture	8 to 35 per cent

2.3. **Performance tests**

2.3.1 A study of the various aspects affecting the life and performance of soil-cement is necessary to establish its suitability as a construction material. The principal requirement of a hardened soil-cement mixture is that it should be sufficiently durable to withstand exposure to the elements. Strength is also a principal requirement, but most soil-cement mixtures possessing adequate resistance to elements also possess the required strength. A third requirement of importance is that of economy in use.

2.3.2. One of the most common questions raised regarding soil-cement is whether all soils can be used. Available data indicate that about 85 per cent of all soils which may be used for soil-cement can be adequately hardened by the addition of cement upto a maximum of 14 per cent by volume. To determine whether or not a soil falls into this 85 per cent category does not require much testing. It is interesting to note that over 50 per cent of all soils so far tested for soil cement by the Portland Cement Association have been found to require only 10 per cent or less cement for adequate hardening.

2.3.3. The problem of the engineer is to determine the least quantity of cement, which, after hydration, will retain the soil-cement mass at maximum density to which it has been compacted and prevent any deterioration in its properties when exposed to the elements. The following three laboratory tests have been developed to determine the quantity of cement and water and the durability of soil-cement.

(1) Test for Moisture-density Relations of Soil-Cement Mixtures (D-558-44)

(2) Wetting and drying test of compacted Soil-Cement Mixtures (D-559-44)

(3) Freezing and Thawing test of compacted Soil-Cement Mixtures (D-560-44)

Details of these tests issued by the American Society of Testing Materials are given in the Appendices 1, 2 & 3.

2.3.4. The moisture density test helps to establish the correct quantity of water and the proper density to which the mixture should be compacted. Wet-dry and the freeze-thaw tests give an indication as to the durability of the material, namely its capacity to withstand the cycles of nature and still maintain its original stability of mass. The wet-dry test indicates whether the cement has been beneficial in materially reducing the abnormal volume changes with changes in moisture content, whereas the freeze-thaw test determines the resistance to large expansive forces produced by freezing water. It may be possible to omit this last test if very low temperatures are not encountered in the area concerned.

2.3.5. Moisture-density test

The moisture-density test determines the moisture content at which maximum density is obtained, when the soil is compacted in a standard manner. The degree of compaction obtainable in the test is designed to be comparable with thorough compaction under field conditions. The test consists in dropping a standard rammer a given number of times from a specified height on to a specimen prepared in a standard mould with varying moisture contents, computing the dry-densities for these moisture-contents and then plotting dry density against moisture content. From the graph the maximum dry-density, with the corresponding optimum moisture content, is determined. On practical considerations only the portion of the soil passing No. 4 sieve is used for testing. If larger material is present and only tests on the fines are available, it is necessary to compute the values for the material from the experimental data obtained for the fines. This is possible provided that sufficient fines are present to fill the voids in the coarse material.

2.3.6. Exhaustive tests show that

(1) the maximum density of soil-cement mixtures with a sandy soil is 2 to 4 lb per cubic foot greater than the maximum raw soil density and with a silty clay soil equal to the maximum raw soil density, and that

(2) the optimum moisture content for soil-cement mixture, is one or two per cent above or below that for the raw soil.

2.3.7. Wet-dry and freeze-thaw tests

In the wet-dry and the freeze-thaw tests (for cold climates), the exfoliated material obtained by brushing after each drying

FLOW DIAGRAM
OF SOIL-CEMENT TESTS

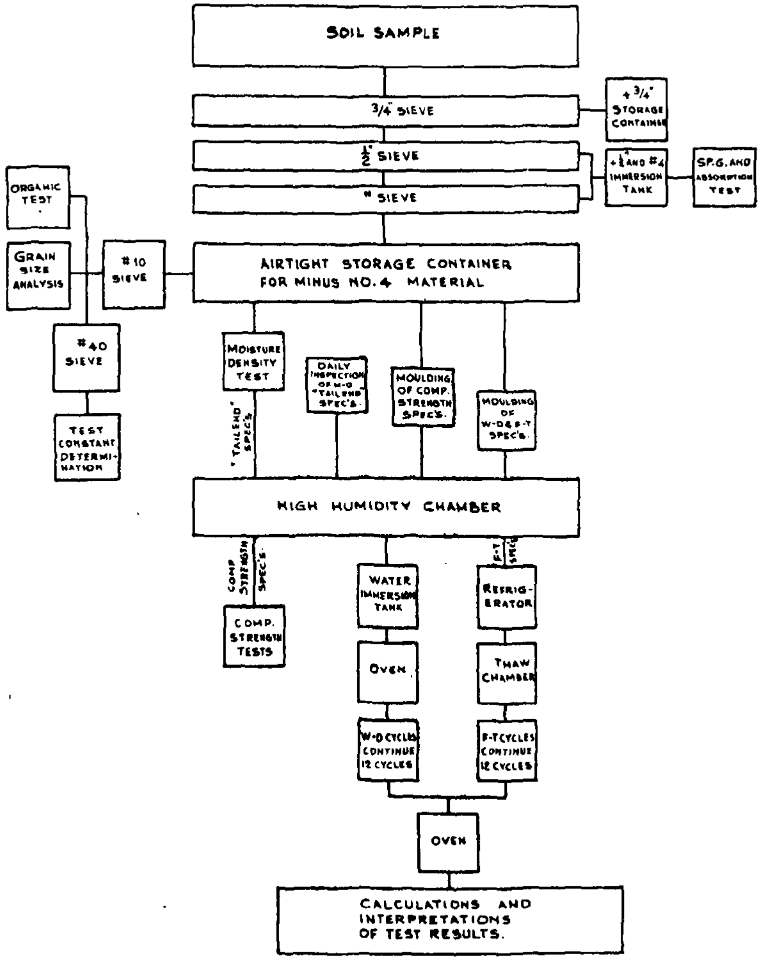


FIG. 2

SEQUENCE OF TESTING PROCEDURES
FOR SOIL AND SOIL-CEMENT MIXTURES

and thawing period respectively, over 12 alternate cycles in each test, is taken as a measure of the soil loss. Since some granular material also separates on brushing, a slightly different limit of soil loss for satisfactory performance is fixed on the basis of soil groups, wherein greater soil losses are permitted for sandy soils than for clayey soils.

2.3.8. The following criteria, based upon test results are set by the Highway Research Board for determining cement content.

(i) Losses during 12 cycles of either the wet-dry or the freeze-thaw test should conform with the following limits:

P.R.A.	classification groups	A-2 and A-3	not over 14 per cent.
,,	,, ,,	A-4 ,, A-5	not over 10 per cent.
,,	,, ,,	A-6 ,, A-7	not over 7 per cent.

(ii) Maximum volume at any time during either wet-dry or freeze-thaw test should not exceed the volume at the time of moulding by more than 2 per cent.

(iii) Maximum moisture content at any time during either wet-dry or freeze-thaw test should not exceed that quantity which will completely fill the voids of the specimen at the time of moulding.

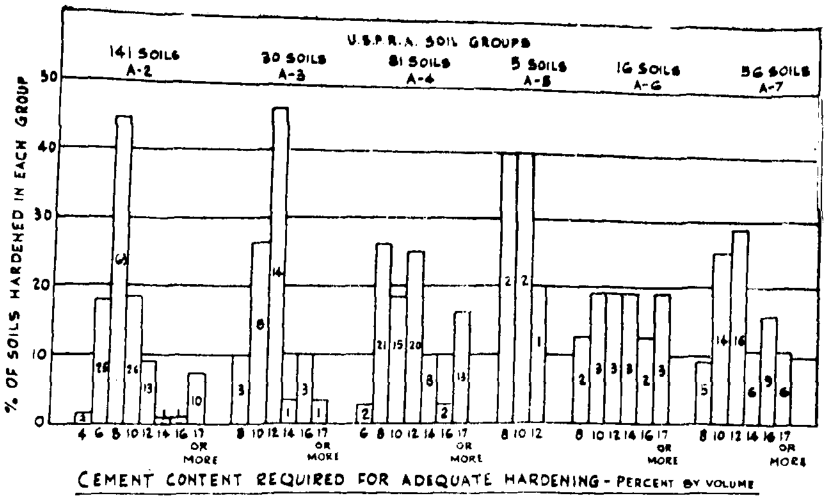


CHART 4

(iv) The compressive strengths of specimens, tested after soaking in water for 1 to 4 hours, should increase with age and cement content, for cement contents which produce results that meet the requirements given in items (i), (ii) and (iii) above. These criteria for judging cement contents that produce satisfactory results should not be considered as rigid recommendations, but as the satisfactory data in the present state of knowledge.

2.3.9. The general physical relations of soils and compacted soil-cement mixtures to attain the hardness and durability standards set by the Highway Research Board are brought out clearly in Table No. 3 and Chart No. 4. These are based on a study of 329 samples of soil in the U.S.A. The Table and Chart show that although certain cement contents predominate in each group there is a wide range of cement contents required to harden satisfactorily the entire range of soils in each group. However, in a particular locality or area the range will be smaller. To determine the suitable cement content three different percentages are normally tried in all standard testing procedures. With experience of particular soils, it is possible to estimate the cement content fairly accurately.

2.4. Field control factors

2.4.1. Usually the cement content of the specimen which satisfies the wet dry test and freeze-thaw test in cold climates can be considered satisfactory. The lowest cement content of the usual three specimens should not be accepted without a further test with a cement content two per cent less, as the further test may indicate that a lower percentage of cement would be satisfactory and this would result in economy.

2.4.2. Since most laboratory tests are conducted on minus No. 4 material, the specified cement content should cover definite requirements in case of the presence of and variations in plus* No. 4 aggregate content. The minimum cement content in minus No. 4 material is not relatively so important in well-reacting sandy soils. Recommendations usually provide that during construction, the cement content of the whole soil-cement mixture shall be increased so that, due to the presence of plus No. 4 material the percentage of cement in the minus No. 4 material does not decrease by more than one per cent from that determined by laboratory tests.

* Plus No. 4 material is gravel, stone, or rock in the soil that is retained on a No. 4 sieve and minus No. 4 soil material is fine gravel, silt, and clay in the soil that passes this sieve.

2.4.3. The laboratory moisture-density data are not directly applicable to field control, but they are accurate enough for estimating equipment needs. Moisture-density tests carried out towards the end of damp mixing indicate the optimum moisture content of the mixture and the density to which the mix should be packed. This procedure enables small variations of soil type and variations in optimum moisture and maximum density resulting from prolonged mixing operations to be accounted for.

A check on the compacted soil-cement is the field density test described in Appendix 4. This helps in establishing whether adequate compaction has been obtained with the procedure and equipment used.

2.5. Rough and ready tests for soil-cement

2.5.1. Procedure

Whenever soil-cement work has to be undertaken in a hurry, or estimates for cement content have to be prepared before actual testing is possible, the following method which was used to advantage in the Second World War in U.S.A. may be adopted. In general, these estimates are more conservative than the values obtained from tests.

The procedure consists in first obtaining the physical constants and gradation of the soil and identifying its proper grouping in both the P.R.A. and the textural classifications. With this information reference is made to chart No. 4 which gives the percentage of cement required for hardening for the various classifications of soil based on a large number of tests. The predominant percentage namely the percentage found satisfactory for the largest number of soils for satisfactory hardening in the particular group is read. Moisture-density specimens are then moulded with three different cement contents with 2 per cent variation on either side of the predominent percentage and tested. The last specimen or one half specimen which is obtained during the usual procedure of cutting a specimen vertically for taking a representative sample for moisture content determination is set aside to observe its hardening characteristics when placed in an atmosphere of high humidity.

2.5.2. "Pick and click" tests

A daily examination of these specimens with a relatively sharp-pointed instrument such as a dull ice-pick and also by clicking them against a sound specimen helps in the determination of the relative rate of hardening from day to day. Specimens that are hardening satisfactorily will resist the penetration of the pick while one which is not hardening satisfactorily will have

poor resistance. Specimens not over 7 days old which resist the penetration of the pick under a considerable force to a depth not greater than 1/8 to 1/4 in. can be considered to be of satisfactory hardness and to indicate that their wet-dry and freeze-thaw tests would prove satisfactory. When clicked, specimens that are hardening satisfactorily give a ringing note. A dull thud or a 'punky' sound is obtained with a poorly hardened specimen. The "pick and click" tests are used jointly. The necessity of both the tests is apparent from the following illustration. Specimens containing a slow hardening sand, in some cases develop a hard surface about 1 8 in. thick without hardening of the core. The hard shell resists the picking but when clicked against a hard specimen it produces a dull thud, characteristic of poorly hardened specimens. The fine textured soils have a less ringing sound than coarse textured and granular soils. Generally two to four-days old specimens which stand up well to these tests are satisfactory. In a few cases, it may be necessary to wait 7 days or more.

2.5.3. Other guides

Another guide in preliminary investigations for soil-cement is the compressive strength at various ages. Since there is considerable variation in compressive strength with the same cement content the strength test cannot be taken as conclusive. It may however be employed for determining cement contents for laboratory tests and estimating purposes. A soil-cement mixture having a compressive strength of approximately 450 lb per sq. in. or more at 7 days will pass the durability tests, provided there is an increase in strength with age and with cement content. Information on these lines, can be obtained within about 10 days of receiving the sample.

Another method of obtaining rough and ready information is by reference to soil-cement losses after a few cycles of wet-dry or freeze-thaw tests. This would, however, presume that typical curves on cumulative losses at every cycle of the freeze-thaw and wet-dry tests are available.

These short-cut methods are essentially intended to estimate cement requirements before the results of regular tests are available and are particularly useful in an emergency.

3. CONSTRUCTION PROCEDURE AND EQUIPMENT

3.1. Methods of construction

3.1.1. The fundamental requirement in soil-cement construction is to obtain an intimate mixture of soil and cement with the correct moisture content and to compact this mixture to its maximum density. There are three methods of construction practised in soil-cement road work namely the "pre-mix" method, the "mix-in-place" method and the "travelling plant" method.

3.1.2. **Pre-mix method**. In principle the pre-mix method differs little from concrete road construction. After the necessary tests, the soil from the site, or borrow soil, if local soil is not suitable, is mixed in a suitable mixing plant with the necessary cement and water. The mixing may be done on the site or at the source of supply in the case of imported soil. The material is then laid on the prepared sub-grade and compacted to the required density by accessory equipment. The advantages of this method are, that it allows an accurate proportioning of the mixture with easy control of the depth of treatment. Concrete mixers can be used and moisture losses during mixing operations and transport minimised. The method is very suitable for placing the mix between side forms. The process, however, is expensive particularly if soil in-situ is processed. Moreover as the material has to be compacted as delivered, there are possibilities of slight variations in compaction from section to section. Generally the process cannot be adopted for large-scale works. The daily output on pre-mix construction can seldom exceed 2,000 sq. yd.

3.1.3. **Mix-in-place method**. In the mix-in-place method the ground is first scarified to within half an inch of the depth to which it is to be stabilised and the soil, either in-situ or borrowed, is pulverised in place until at least 80 per cent (exclusive of stones) passes the No. 4 sieve. Pulverising should be carried out from the sides of the road working towards the centre and is best done with a moisture content within two per cent of the optimum.

The next step is to spread the cement. The area to be covered by a bag of cement is calculated and the bags are distributed accordingly. They are then slit open and the cement spread reasonably uniformly over the surface. This can be done by spike harrows or a blade grader. The cement and the soil are then intimately mixed. Experience has shown that it is desirable to mix at a moisture content slightly higher than the optimum except when heavy rain is expected. If more water is required during the mixing operation, it should be added carefully and well mixed with the soil to its full depth.

After mixing the surface is graded and compacted; compaction is done as in the pre-mix process and is continued until uniform density has been obtained over the whole area.

No forms are used but the road is built slightly wider than necessary and cut back to the desired width when the stabilised construction is completed.

The main **advantages** of this method are that any equipment from simple agricultural tools to the most elaborate plant can be employed depending on the magnitude of the work. There is a greater degree of uniformity in compaction as the entire cross-section is processed at the same time. Also in view of the considerable time required for operations, removal of excess moisture by evaporation is easier, particularly in wet climates.

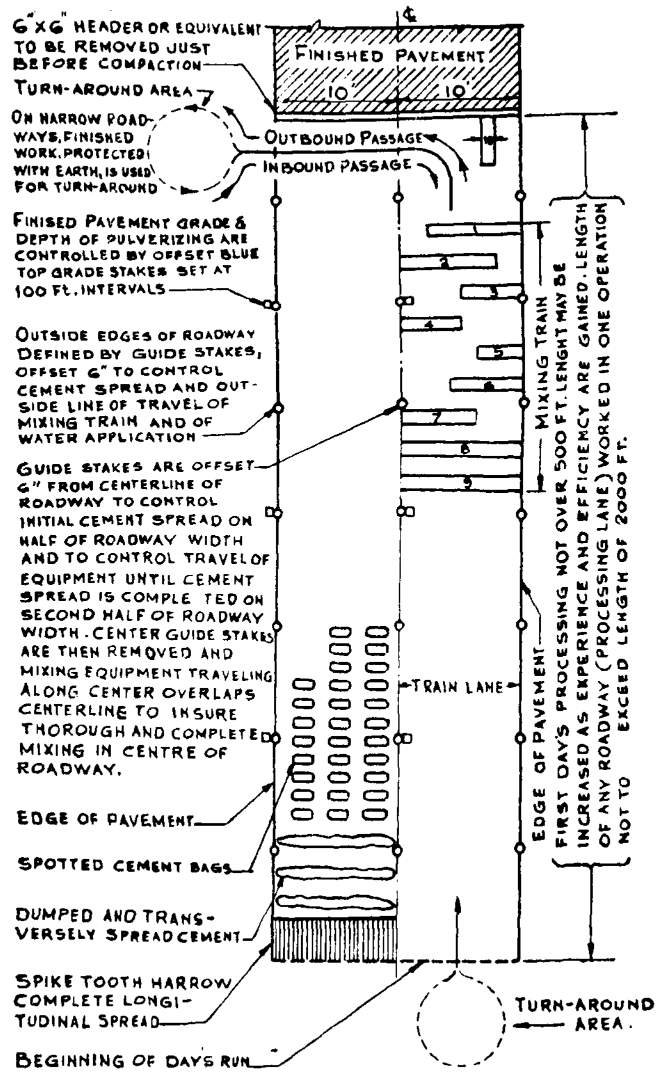
The **disadvantages** are that it is not easy to obtain a uniform thickness of treated material, because the depth of the furrow may vary slightly. The mixing may also not be as uniform as in pre-mix work. The work is susceptible to changes of weather and, once the cement has been spread, the work cannot be suspended in inclement weather. Rain during construction is likely to spoil the portion under construction. In addition, in dry climates, water lost by evaporation requires replacement.

3.1.4. **Travelling plant method**. A relatively new development in the technique of construction, is the travelling plant method. In this method, the soil after pulverization and proper placement of cement over it, is lifted up by an elevator to a mixer mounted on a travelling platform where the necessary water is added. The mixture is then discharged on to the subgrade where it is spread by blade graders and compacted as in the mix-in-place method. This method combines the merits of both the premix as well as mix-in place methods by combining high output with thorough mixing.

3.2. Types of equipment

3.2.1. The choice of equipment depends on the method selected for construction. Equipment is required for pulverising, mixing, and compacting either separately or in combination. In the early stages of development simple equipment was used for pulverising and mixing and smooth wheel rollers for compacting. With increasing knowledge of working the soil, a wide range of equipment to meet varying needs has been developed.

3.2.2. In the simplest form of mix-in-place construction, the equipment used are gang plough, rake, disc harrow, nail drag, blade grader, a mounted tank for water distribution, a smooth-wheel roller, a sheepsfoot roller in the case of clayey soils, and a pneumatic tyred roller in the case of silty soils. In premix construction, either hand mixing or a concrete mixer takes the place of disc harrow, nail drag, and water distributor, other operations being similar to mix-in place method. One or two tractor



NOTE:- FOR 18 TO 22 FOOT ROADWAYS USE SAME TRAIN ARRANGEMENT, BEING SURE ON 22' WIDTH TO OBTAIN COVERAGE WITH MIXING EQUIPMENT. FOR 24 FOOT ROADWAYS DIVIDE INTO THREE (3) TRAIN LANES AND PROCEED AS INDICATED

SOIL-CEMENT "TRAIN" PROCESSING
FOR 20' ROADWAY
TWO TRAIN LANES, EACH 10' WIDE

FIG. 3

INDEX TO
MIXING TRAIN UNITS IN FIG. 3

UNIT NO.	NO. REQ'D	WIDTH	DESCRIPTION OF UNIT	MOTIVE POWER
1(2)	1	8'	CULTIVATOR, HEAVY DUTY, SPRING TOOTH, 12" SPACING. FITTED WITH 4" DOUBLE POINTED CHISELS.	TRACTOR, CRAWLER TYPE. MIN. 35 D.B.H.P.
2	1	8'	CULTIVATOR, HEAVY DUTY, SPRING TOOTH, 12" SPACING. FITTED WITH 4" DOUBLE POINTED CHISEL.	TRACTOR, CRAWLER TYPE. MIN. 35 D.B.H.P.
3	1	4'-8"	PLOW, TRACTOR, HEAVY DUTY 4-14 INCH BOTTOMS (OMIT COULTERS)	TRACTOR, CRAWLER TYPE. MIN. 35 D.B.H.P.
4	1	4'-8"	PLOW, TRACTOR, HEAVY DUTY, 4-14 INCH BOTTOMS (OMIT COULTERS)	TRACTOR. CRAWLER TYPE. MIN. 35 D.B.H.P.
5(1)	1	12'	GRADER, ROAD MOTORIZED 12' MOLDBOARD.	SELF-POWERED
6(2)	1	6'	ROTARY SPEED MIXER, SELF-POWERED TRAILER MOUNTED.	TRACTOR, CRAWLER TYPE. MIN. 35 D.B.H.P.
7	1	6'	ROTORY SPEED MIXER, SELF-POWERED TRAILER MOUNTED.	TRACTOR, CRAWLER TYPE. MIN. 35 D.B.H.P.
8(3)	2	10'	DISTRIBUTOR, WATER PRESSURE 1000 GALLONS	SELF-POWERED
9	1	10'	HARROW, SPIKE TOOTH 2 SECTIONS	TRACTOR, RUBBER TIRED. MIN. 23 D.B.H.P.
(2) 10			CULTIVATOR-UNIT NO. 1 OF MIXER, ROTARY TILLER - UNIT NO. 6	

FIG. 3 (Continued)

units are used for various operations. For increased output it is the practice to use the units in trains. The sequence of operations and train of units as recommended by Highway Research Board for a 20 feet wide paving job are given in figure No. 3.

3.2.3. The main features of developments in plant for soil cement construction are higher speeds of construction, uniformity in scarified depth, more thorough mixing operation, and uniform compacted density. The modern rotary hoes enable processing of coarse grained soils with relatively high percentage of stones ;

the pulvimixer is a combination machine with pulverising and mixing operations both carried out by a series of spikes on a rotor which can be lowered to requisite depth with a high powered hydraulic ram and is most suitable for fine grained soils. The P & H single-pass stabilizer, which is the latest of these fast operating machines, consists of 4 contra rotating rotors and is driven by a 260 H. P. unit. In one passage of the machine, the soil is cut, pulverised, and mixed with the requisite quantity of cement and water. The machine operates on a width of 10 feet with speeds upto 30 ft. per min. and as much as 8000 sq. yds. have been completed in a day with this equipment.

3.2.4. Another development is the travelling mixing plant which picks up windrowed soil material and mixes it with cement and water. The mixture is spread on the subgrade with an attached grader.

3.2.5. Compacting equipment is necessary with all the above plant and consists of smooth rollers of the tandem and conventional three wheel types, sheepsfoot rollers of either the taper-foot or the club-foot types, and pneumatic tyred rollers ranging from laden trucks to trailers with multiple wheel axles. Sometimes vibrating equipment used for concrete road finishing is also employed. The process of compaction is dependent on factors like particle size, moisture content, and thickness to be compacted. A preliminary trial is desirable in case of large works to determine the weight, number of passes and suitability of the various rollers.

3.2.6. Water distributors range from ordinary tanks mounted on trucks with distributing pipes to pressure water distributors which can accurately regulate the amount of water in each unit of area.

3.3. Suggestions for adapting simple farm equipment

3.3.1. In the selection of equipment for soil-cement construction, the similarity in process in the pulverising and mixing stages to farming permits use of farming equipment on soil-cement road work. The requirements are determined by the scale of operations, by the speed of each process and the relative economy in comparison with special types of equipment. There is however a wide latitude because, before the addition of cement, there is no limit to the available time except as required by the scheduled progress. A period of about six hours is available for completing the operation after the addition of cement. The degree of control necessary in the pulverising stage is also not so strict as in the subsequent processes. Hence, it is not always necessary to await special equipment before undertaking soil-cement road construction and considerable adaptation can be effected with simple equipment. If higher rates of progress are desired, tractors can be used with advantage for towing the various units of farming equipment.

3.3.2. A simple farming plough, spike-harrow or nail-drag can be used for the removal of roots and vegetation and also for scarifying to the required depth. The dry mixing of cement can be done easily by first spreading the cement on the pulverised soil surface uniformly by means of a nail-drag and then using a gang plough or offset disc harrow for mixing at site. Water is sprinkled by means of steel tanks fitted with ordinary perforated pipe distributors, the quantity being controlled according to the rate of travel by a control valve. Wet mixing can be carried out with the same equipment as dry mixing, till a uniform mix is obtained. A conventional road roller can be used for soil-cement work, or alternatively hand tampers or vibratory tampers can be adopted for final compaction. This would require the use of side forms. A sheepsfoot roller or pneumatic-tyred roller becomes necessary if the tendency of damp soil is to roll in layers. In the case of clayey soil better compaction is obtained with a sheepsfoot roller having tapered feet. In the absence of a sheepsfoot roller, sheepsfoot hand tampers (Fig. 4) may be substituted and sufficient labour for this operation employed to keep pace with other processes. Where pneumatic tyred rolling is necessary, an ordinary truck can be used with sufficient loading and properly guided movement.

3.3.3. In pre-mixing operations a concrete mixer of the usual type can be used. There is therefore no process in soil-cement construction which cannot be managed with simple equipment for works of moderate size.

3.4. Special provisions

3.4.1. As the finished surface is to serve only as a base course, surface treatment or a wearing coat is necessary to withstand the abrasive action of traffic. Different treatments are necessary, depending on the type of wearing coat desired. Where only pneumatic-tyred traffic is to be catered for, a soil-cement topping or a bituminous surfacing is adequate. Where bullock-cart traffic is to be allowed for in addition to pneumatic-tyred traffic, a concrete topping, the thickness depending on the intensity of traffic and on the wheel load is the best surface. Heavy duty bituminous surface may also be adopted.

3.4.2. The three essentially different wearing coats, each with a number of variations are :

(a) Soil-cement topping with a richer cement content than the base course ; suitable for driveways, estate roads, etc. used exclusively by pneumatic tyred wheels.

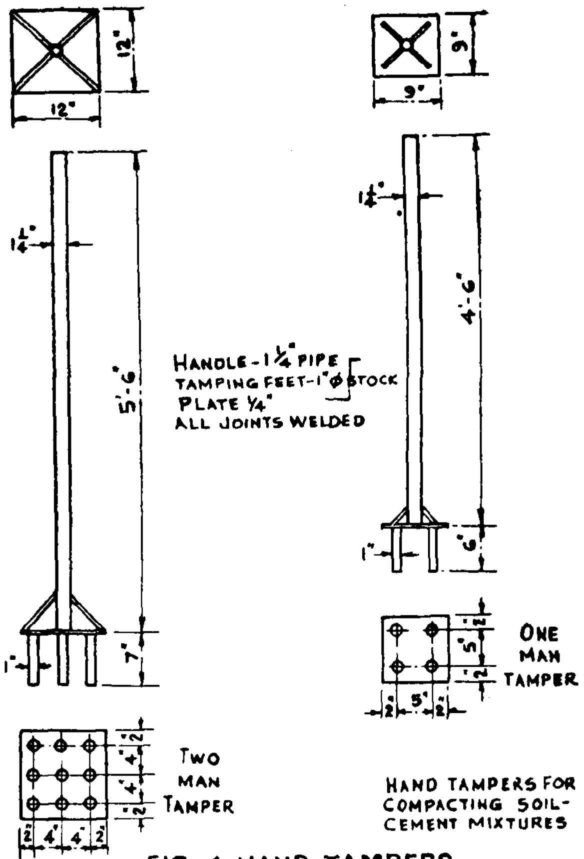


FIG. 4 HAND TAMPERS

(b) Concrete topping; suitable for medium and heavy duty road surfaces and particularly for mixed traffic, namely bullock-cart and pneumatic-tyred.

(c) Bituminous light and heavy duty surfacings.

3.4.3. Soil-cement topping.

Soil-cement topping with a higher cement content than the soil cement base course is laid on the base course just after the compaction of the top one inch mulch has been completed. After lightly scarifying the surface, the topping material, which is prepared by mixing suitable soil with adequate cement and water to ensure a crushing strength when compacted, of 1000 lb per

sq. in. at 28 days, is laid and compacted. The compacted depth of topping should not be less than 2 in. and the field controls on cement content, compacted density, optimum moisture content, and protection have to conform with requirements similar to those of the base course.

3.4.4. Concrete topping.

For medium and heavy duty a concrete surface, the thickness of which depends upon the magnitude of the traffic, and on maximum wheel loads is laid on the soil-cement base. No treatment of surface is necessary but the topping should be laid only after the base is adequately cured. The surface of the soil-cement base should however be cleaned of all loose materials with stiff brooms. The width of the base course shall extend at least 6 in. beyond the edge of the concrete pavement.

For a light-duty concrete pavement of a thickness below 3 in. prior treatment of the surface of the soil-cement base is necessary to provide adequate bond between the layers. It is suggested that the bond may be provided by grooving the finished soil-cement surface transversely with grooves about 3/4 in. deep at 3 in. centres with a suitable raking tool before the soil-cement surface has hardened. A coat of cement sand grout is applied to the soil-cement surface before placing the concrete surface mix. The concrete surface is laid preferably within two or three days of the compaction of the soil-cement base.

3.4.5. Bituminous topping.

Before placing the bituminous topping the soil-cement surface is treated with a primer. The primer may be applied just after finishing the soil-cement base or just before laying the bituminous surface. As certain primers adversely affect the hardening of soil-cement, excessive penetration of primer is prevented by moistening the surface of the base to close the surface pores before applying the primer.

Light duty surface dressings are laid preferably after the soil-cement base course has been in use to enable any defective spots in the base course to be rectified. On the other hand, heavy duty bituminous toppings are laid before opening the road to traffic.

4. RECOMMENDED CODE OF PRACTICE

4.1. In order to simplify the application of soil-cement construction to roads, an attempt has been made to draft a code. This code is perforce tentative and may need modification in the light of local experience.

4.2. **Scope**. The requirements of this code meet the needs of soil-cement pavement construction using either soil in-situ or borrowed soil or blended soil, premixed or mix-in-place construction, suitable generally for base construction. Surface treatment either for weathering and wear resistance or for additional pavement thickness with other materials is also indicated.

4.3. **Materials**

4.3.1. **Portland Cement**. Portland cement shall be of one of the brands manufactured in India, complying with requirements of I.S.S. 269-1951. Any cement which has become wet, damaged, or contaminated in transit, handling, or storage shall not be used.

4.3.2. **Water**. The water used shall be clean, clear, free from injurious salts, acids, alkalies, organic matter, or other deleterious materials and be of a quality fit for drinking purposes.

4.3.3. **Soil**. The soil for the construction shall consist of material existing in the area to be paved, selected soil or mixtures of both which shall be approved by the engineer. The soil shall not contain more than 5 per cent gravel or stone retained on 3 in. sieve, nor more than 45 per cent retained on No. 4 sieve.

4.3.4. **Admixtures:** No admixture shall be used unless approved by the engineer. Admixture for soil-cement work shall be calcium chloride* of approved quality when required to be used.

4.4. **Equipment**

4.4.1. Any equipment, machine, or combination thereof meeting the requirements of soil pulverisation, cement mixing and compaction may be used for producing the soil-cement base course and for providing an adequate surface cover as given in the following clauses.

4.4.2. Use of all equipment and machines shall be subject to prior approval by the engineer-in-charge.

4.5. **Construction methods**

4.5.1. **Preparation**: The area to be paved shall prior to other operations, be graded and shaped as required to grades, alignment, thickness, and typical cross-sections shown on the plans. Any additional soil needed shall be placed as directed by the engineer. All unsuitable soil or material shall be removed and replaced with acceptable material.

The subgrade shall be firm and able to support without displacement, the construction and compaction equipment hereinafter specified. Any soft or yielding subgrade shall be corrected and made stable before construction proceeds.

4.5.2. **Pulverising:** The soil to be processed shall be scarified and pulverised before adding cement. Pulverising shall continue even during mixing operations if necessary until all the soil passes a 1 in. sieve and a minimum of 80 per cent of the soil passes a No. 4 sieve excluding gravel and stones retained on the sieves. In premixed construction, partial pulverisation may be permitted in the mixing plant subject to test on soil quality indicating thorough mixing. Pre-wetting of soil for pulverisation may be permitted in special cases; in such cases the amount of water shall not exceed the optimum needed for mixing and compacting. In general, pulverisation should be planned so that only that quantity of soil which can be processed in two days is pulverised ahead of construction.

4.5.3. **Addition of cement**. The specified quantity of cement shall be applied uniformly on the soil in a manner satisfactory to the engineer or in specified quantities for each batch when mixed in a mixer.

At the time of adding cement, the percentage of moisture in the soil shall not exceed the quantity which will permit a uniform and intimate mixture of soil and cement during mixing operations.

All operations from the time of adding cement to the finishing stage shall be completed within 6 hours.

4.5.4. **Mixing**. After the cement has been added, it shall be continuously mixed with the soil till all the cement has been sufficiently blended with it, to prevent the formation of cement-water nodules or balls when water is applied.

Any mixture of soil and cement that has not been compacted and finished shall not remain undisturbed for more than 30 minutes.

4.5.5. **Application of water and damp mixing**: Immediately after the soil and cement have been mixed, water shall be applied uniformly to the mix. The water supply and distributing equipment should have adequate capacity to ensure the applica-

* *Note:* Admixture of calcium chloride is a method of improving soil-cement mixtures especially for poorly reacting soils. Its action is to increase the rate of hydration so that high strengths are obtained at early ages. It also tends to improve permeability and durability. It is added in solution as part of the mixing water.

tion of the necessary quantity of water in the section in three hours in mix-in-place construction. Water should be added in as large increments as the equipment and soil will permit. (One gallon per sq. yd. per application is found satisfactory for most soils). As the moisture content approaches the optimum, the application should be reduced to 1/2 gallon per sq. yd. or less.

After all the water has been applied, mixing shall continue until a thorough uniform and intimate mixture of soil, cement, and water has been obtained. At the conclusion of the damp mixing operations moisture-density tests (A.S.T.M. designation D 558-44) shall be carried out in the field on representative samples of soil-cement mixtures. The percentage of moisture in the mixture and in unpulverised soil lumps based on oven-dry weights, shall not be below nor more than one-fifth above the specified optimum moisture content. It shall however be less than that quantity which will cause the base course to become unstable during compaction and finishing.

When mixing in concrete mixers, the water shall be added in suitable increments depending on the type of mixer. It is desirable to add a little excess water to make up for evaporation losses in handling and transit. The mixed materials shall be spread on the desired area uniformly with suitable distributing equipment.

4.5.6. **Compaction**. Prior to the beginning of compaction, the mixture shall be in a loose condition for its full depth. Compaction shall be started with sheepsfoot rollers beginning at the edges and working towards the centre.

After about 2/3 of the depth is thus compacted, the remaining mulch should be uniformly distributed with a grader and rolling continued until only one inch of loose mulch remains. Any loose spot shall be corrected by addition of extra material and re-rolling. In very sandy soils which offer little resistance to compaction, pneumatic tyred rollers shall be used for compacting from the start of the operations. Also sufficient water should be added to keep the moisture content at optimum or slightly above it during the entire operation. The number of passes and the speed of operation of rollers shall be sufficient to compact uniformly the section being processed to the required density within two hours.

The specified density shall be determined in the field by the moisture density test (A.S.T.M. designation D 558—44) on representative samples of soil-cement mixture obtained from the work.

4.5.7. **Finishing**. After the mixture has been compacted, the surface of the compacted soil-cement shall be lightly scarified to loosen any imprints and remove compaction planes left by compacting or shaping equipment. The resulting surface shall then be thoroughly rolled with smooth wheel or pneumatic-tyred rollers or both, and of the type and size specified by the engineer. Rolling shall be supplemented by brushing if required by the engineer. The moisture content of the surface material shall be maintained at not less than its specified optimum moisture content during all finishing operations. Surface compaction and finishing shall be done in such a manner as to produce in not more than two hours, a smooth dense surface, free from surface compaction planes, cracks, ridges or loose material. The completed base course shall conform to the grades, lines and cross-sections shown on the plans.

Any portion of the work which has a density of 5 lb or more below that specified by the engineer shall be corrected or removed and replaced to meet the specifications.

4.5.8. **Protection and curing**. After the soil-cement base course has been completed as specified herein, it shall be protected against drying for fourteen days by applying a 2 in. cover of earth or not less than 4 lb (dry weight) of straw or hay per sq. yd. which shall be moistened initially and subsequently as may be necessary, or by applying other cover materials approved by the engineer. The protection shall be applied as soon as possible after the completion of the finishing operations. The finished work shall be kept continuously moist until the cover material (earth or straw) is placed. After the curing period, cover shall be removed.

Finished work adjacent to portions under construction shall be protected from travelling equipment in such a manner as to prevent damage to the completed work.

4.5.9. **Construction joints**. At the end of each day's work a straight transverse construction joint shall be formed by cutting back into the completed work to form a true vertical face, and by installing a temporary wooden header, if required by the engineer.

The full width of the base courses for roads, streets, or similar areas shall be processed and finished each day without longitudinal construction joints.

Base courses for runways, and other large areas shall be built in a series of parallel lanes of convenient length and width approved by the engineer. Straight longitudinal joints shall be formed at the edge of each day's work by cutting back into the completed work to form a true vertical face, free of loose or shattered material. For laying to correct grades, temporary wooden forms, or their equivalent, shall be set firmly to line and grade if so directed by the engineer.

4.6. Traffic.

4.6.1. Local traffic and construction equipment shall be permitted to use completed portions after the base course has hardened sufficiently to prevent marring or distorting of the surface, provided the protection and cover are not impaired.

4.6.2. After the 14-day curing period, all traffic shall be permitted to use completed portion provided the base course has hardened sufficiently to prevent marring or distorting of the surface by traffic.

4.7. Maintenance.

4.7.1. The contractor shall be required, within the limits of his contract, to maintain the entire base course in good condition to the satisfaction of the engineer from the time he starts work until all work has been completed and accepted. Maintenance shall include immediate repairs of any defects that may occur either before or after the cement is applied and such work shall be done by the contractor at his own expense, and repeated as often as may be necessary to keep the area intact. Repairs are to be made in a manner to insure restoration of a uniform surface and durability of the part repaired. Faulty work shall be replaced for the full depth of treatment. Any low areas shall be remedied by replacing the material for the full depth of treatment and not by adding a thin layer of soil-cement to the completed work.

4.8. Wearing coat

4.8.1. A wearing coat shall be applied to the base course as specified by the engineer. The wearing coat may consist of :

(a) Soil-cement topping with a higher cement content;

(b) Premixed concrete topping ; or

(c) Bituminous surfacing.

5. CONCLUSIONS

5.1. Soil-cement is a method of stabilised soil construction which has become increasingly popular during recent years. The process, as already explained in detail, involves the admixture of cement with pulverised soil after which it is watered and compacted. The effect of this admixture is to produce a material possessing appreciable hardness and resistance to weathering, and

having properties that differ greatly from those of the soil from which it is made. As distinct from other low-cost materials, the technique of soil-cement construction and testing is scientifically controlled. The findings based on scientific methods enable correct estimates being made of the performance of the material with regard to its structural properties and its permanence.

5.2. Soil-cement construction employs the maximum of cheap local material and the minimum of the more expensive products, either manufactured (i.e., cement) or conveyed over varying distances (i.e., sand and stone aggregates). As the soil which constitutes ninety to ninety-five per cent of soil-cement is wholly indigenous and as the processing can be carried out with great ease, using only unskilled labour, on account of similarity to farming methods, it is by far the most economical of reliable, low-cost materials.

5.3. Soil-cement can be used for an unlimited variety of purposes, such as :

(a) surfacing for low-cost roads, for areas of ground desired to be water-proofed, for example, winnowing and stacking grain in villages, for car-parking places, store yards, etc.

(b) a sub-base under all types of road surfacings and under footpaths, cycle tracks, school playgrounds, car-parks, aerodrome runways, floors of factories and store-yards.

(c) for filling in pipe trenches under roads where it is necessary to obviate settlement and consequent damage to the road surfacing.

(d) for filling in behind retaining walls, over arches and culverts and in limited spaces where it is not practicable to obtain adequate compaction by rolling. Composite construction of small span bridges in which precast reinforced concrete units are combined with soil-cement to sustain loads has already been tried out. Maximum benefits can however be derived only if greater enthusiasm is evinced in giving this new material a fair trial and more and more experimental stretches of soil-cement construction are attempted. Experience in other countries, particularly the U.S.A., indicates that soil-cement combines excellent performance with low cost. The largest field for the application for soil-cement in India lies in road construction. The conditions appear to be ideal for soil-cement construction. A very large mileage of roads has to be built and the funds for building the roads are limited. Optimum results can be achieved with soil-cement.

5.4. A proper understanding of the soil, which is the major constituent of soil cement is necessary before any progress can be achieved in soil-cement. Addition of cement to soil alters its structure and the properties of soil-cement are not predictable from the physical constants of either the soil or the soil-cement mixture. Additional laboratory tests for durability and strength are required to give correct indications. The minimum cement content is determined by durability requirements whilst economy determines the maximum cement content. Compaction for maximum density under optimum moisture conditions is an important factor in soil-cement construction. The U. S. Portland Cement Association is of the view that for practical reasons no cement contents less than 7 per cent can be recommended for field use. Due to the impracticability of precise measurements, increments or decrements of cement content are usually made by two percentage points.

5.5. Development in construction equipment has been from the most simple farming equipment to comprehensive and efficient machinery, the main objects of elaborate machinery being speed of construction, proper control of the compacted depth, proper mixing, and adequate compacted density. The wide range of possibilities permits soil-cement construction to be carried out even with the minimum of equipment.

5.6. Field control allows reasonable tolerances both in respect of time and factors such as composition, moisture content, and compacting effort and permits easy manipulation of the scale of operations. Soil-cement is equally suitable for a structurally sound road base and for a pavement of good quality and strength with appropriate surface coatings.

6. ACKNOWLEDGMENT

The Author wishes to acknowledge gratefully the assistance rendered by Messrs. C. L. N Iyengar and T. M. Menon, engineers of the Concrete Association of India in the compilation of this Paper.

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Appendix 1.

Standard Method of Test for Moisture-Density Relations of Soil-Cement Mixtures.

A. S. T. M. DESIGNATION: D 558—44

Scope

1. This method of test is intended for determining the relationship between the moisture content of soil-cement mixtures and resulting densities (oven-dry weight per cubic foot) when the soil-cement mixture is compacted in the laboratory, before cement hydration, as specified herein.

Apparatus

2. The apparatus shall consist of the following:

(a) *Mould.*—A cylindrical metal mould having a capacity of $\frac{1}{30}$ cu. ft with an internal diameter of 4.0 in. and a height of approximately 4.6 in., which has a detachable collar assembly approximately $2\frac{1}{2}$ in. in height, to permit preparing compacted specimens of soil-cement mixtures 4.0 in. in diameter and approximately 4.6 in. in height which have a volume of $\frac{1}{30}$ cu.ft. The mould and collar shall be fastened to a detachable base (See Fig. 5).

(b) *Rammer.*—A metal rammer having a 2-in. diameter circular face and weighing 5.5 lb. The rammer shall be equipped with a suitable arrangement to control the specified drop (see Fig. 6).

(c) *Sleeve.*—Closed cylindrical sleeve slightly less than 4.0 in. in diameter or similar device for removing compacted specimens from the mould.

(d) *Balances.*—A balance or scale of 25-lb. capacity sensitive to 0.01 lb.; and a 100-g. capacity balance sensitive to 0.1 g.

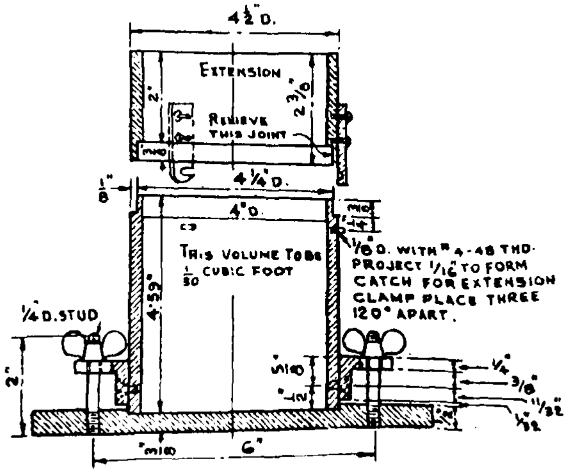
(e) *Drying Oven*—A thermostatically controlled drying oven capable of maintaining temperatures of about 110° C. (230° F.) for drying moisture samples.

Procedure

3. (a) The air-dry soil first shall be pulverized to pass a No. 4 (4760-micron) sieve so as to separate the soil particles without reducing the particle size.

(b) The required cement shall be added to the pulverized soil. The test shall be performed only on that portion of the soil-cement mixture which will pass a No. 4 (4760-micron) sieve. A quantity of water sufficient to produce slight cohesion shall then be thoroughly mixed with the soil-cement sample to permit ready compaction. The thoroughly mixed soil-cement sample shall be immediately compacted in the mould in three equal layers, to give a total compacted depth of about 5 in.; each layer being compacted by 25 blows of the rammer dropping free from a height of 12 in. above the elevation of the soil when a sleeve-type rammer is used or 12 in. above the approximate elevation of each finally compacted layer when a stationary mounted type rammer is used. During compaction, the mould shall rest on a uniform, rigid foundation weighing 200 lb. or its equivalent. The blows shall be uniformly distributed over the surface of the layer being compacted. After compacting, the collar shall be removed and the top carefully trimmed to the exact height of the mould with a steel straightedge to produce a specimen approximately 4.6 in. in height and having a volume of 1/30 cu. ft.

(c) The weight of the compacted soil-cement mixture shall be determined, the material removed from the cylinder, sliced vertically in the centre and a 100-g. sample taken from the centre, weighed immediately, dried in



CYLINDRICAL MOULD FOR MOISTURE DENSITY TEST

FIG. 5

an oven at 110° C. (230° F.) for at least 12 hrs. or to constant weight. This procedure establishes the moisture-density relation of the initial soil-cement mixture.

(d) The soil-cement mixture shall be again finely pulverized, so as to separate the soil particles without reducing the particle size, to pass a No. 4 (4760-micron) sieve and a small increment of moisture carefully added and thoroughly mixed to insure uniform distribution. Then the moistened and pulverized material shall be compacted in the mould in three layers as described in Paragraph (b), weighed, and a moisture determination made to establish the moisture-density relation for a slightly moistened soil-cement mixture. The soil-cement mixture again shall be finely pulverized, so as to separate the soil particles without reducing the particle size, to pass a No. 4 sieve and additional increments of water added and the same procedure followed of mixing, compacting in the mould, weighing, and making moisture determinations until the moisture content of the soil-cement mixture reaches a condition where it is difficult to pulverize the moistened materials, or it has a moisture content near the liquid limit. This procedure will establish the moisture-density relations of a soil-cement mixture for the moisture contents practicable to handle in the field.

Calculation

4. The moisture content and oven-dry weight of the mixture as compacted shall be calculated by means of the following formulas:

Moisture per cent =

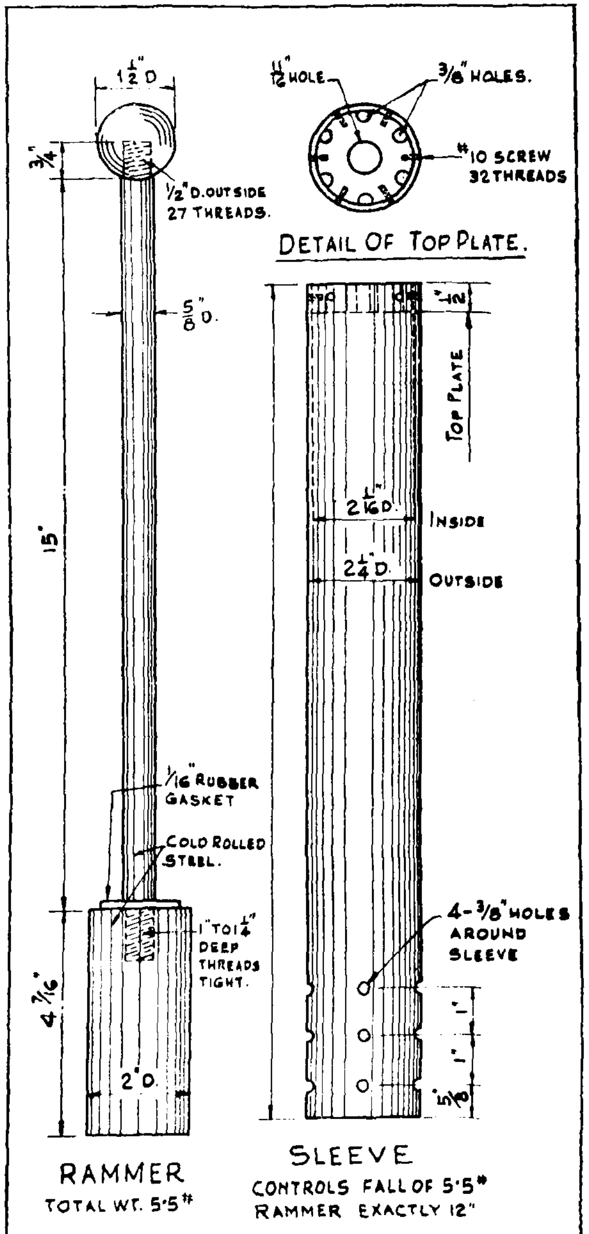
$$\frac{\text{wt. of wet soil-cement} - \text{wt. of oven-dried soil-cement}}{\text{wt. of oven-dried soil-cement}} \times 100$$

Dry weight (per cu. ft of soil cement as compacted)

$$= \frac{\text{wet wt. in lb. per cu. ft}}{\text{percentage of moisture plus 100}} \times 100$$

Moisture-density relationship

5. (a) The calculations in section 4 are made to determine the moisture content and corresponding compacted oven-dry weight (density) of the compacted soil-cement



MOISTURE-DENSITY COMPACTION RAMMER.

FIG. 6

for each test made on the mixture. The oven-dry weights per cubic foot (density) of the soil shall be plotted as ordinates and the corresponding moisture contents as abscissae.

(b) *Optimum Moisture Content*—When the moisture-density relations have been determined for a soil-cement mixture and the results plotted as indicated in Paragraph (a), it will be found that by connecting the plotted points with a smooth line a curve is produced which is, generally, parabolic in form. The moisture content producing the peak of the curve shall be termed the "optimum moisture content" of the soil-cement mixture under the above compaction.

(c) *Maximum Density*—The oven-dry weight per cubic foot of moist soil-cement mixture at "optimum moisture content" shall be termed "maximum density" under the above compaction.

(d) When the curve resulting from connecting plotted points does not produce a parabolic-type curve with a definite peak, the "optimum moisture" and "maximum density" for such mixtures shall be determined from the results of additional tests conducted in conformity with the Standard Method of Wetting-and-Drying Test of Compacted Soil-Cement Mixtures (A. S. T. M. Designation : D 559) and the standard method of freezing and Thawing Test of compacted Soil-cement Mixtures (A. S. T. M. Designation : D 560) on compacted soil-cement specimens. These specimens shall be molded with moisture contents approximately equal to the plastic limit of the soil and 1, 2, and 3 per cent above and below this governing moisture content. The densities of these specimens shall be those obtained with the prescribed moisture contents in the moisture-density test for the mixture. The cement content of these specimens shall be such that the soil-cement losses on the specimen giving the best results in these tests shall not exceed 10 per cent of the dry weight of the specimen. The moisture content and density corresponding to the specimen giving the best results shall be used as the "optimum moisture content" and "maximum density" of soil-cement mixtures which do not produce a parabolic type curve.

—o—

Appendix 2.

Standard Method of Wetting-and-Drying Test of Compacted Soil-Cement Mixtures

A. S. T. M. DESIGNATION : D 559—44

Scope

1. This method of test is intended for determining the soil-cement losses, moisture changes, and volume changes (swell and shrinkage) produced by repeated wetting and drying of compacted specimens of soil-cement mixtures of known composition and of known uniform density and moisture content.

Apparatus

2. The apparatus shall consist of the following :

(a) *Mould.*—A cylindrical metal mould having a capacity of $\frac{1}{30}$ cu. ft. with an internal diameter of 4.0 in. and a height of approximately 4.6 in., which has a detachable collar assembly approximately $2\frac{1}{2}$ in. in height, to permit preparing compacted specimens of soil-cement mixtures 4.0 in. in diameter and approximately 4.6 in. in height which have a volume of $\frac{1}{30}$ cu. ft. The mould and collar shall be fastened to a detachable base (See Fig. 5).

(b) *Rammer.*—A metal rammer having a 2-in. diameter circular face and weighing 5.5 lb. The rammer shall be equipped with a suitable arrangement to control the specified drop. (See Fig. 6.)

(c) *Sleeve.*—Closed cylindrical sleeve slightly less than 4.0 in. in diameter or similar device for removing compacted specimens from the mould.

(d) *Balances.*—A balance or scale of 25-lb capacity sensitive to 0.01 lb.: and a 100-g. capacity balance sensitive to 0.1 g.

(e) *Drying Ovens.*—A thermostatically controlled drying oven capable of maintaining temperatures of about 110°C. (230°F.) for drying moisture samples.

A thermostatically controlled drying oven capable of maintaining temperatures of about 71° C. (160°F.) for drying compacted soil-cement specimens.

(f) *Moist Room.*—A moist room capable of maintaining a temperature of 21 ± 1.7° C. (70 ± 3° F.) and a relative humidity of not less than 90 per cent for seven-day storage of compacted specimens.

(g) *Water Bath.*—Suitable tank for submerging compacted specimens in water at about 21°C. (70°F.)

(h) *Wire Scratch Brush.*—A wire scratch brush made of 2-in. by $\frac{1}{16}$ in. flat No. 26 gauge wire bristles assembled in 50 groups of 10 bristles each and mounted to form five longitudinal rows and ten transverse rows of bristles on a $7\frac{1}{2}$ in. by $2\frac{1}{2}$ in. hardwood block.

Preparation of material for moulding specimens

3. This test shall be performed on soil-cement mixtures compacted in the mould, before cement hydration, to maximum density at optimum moisture content and in the same manner of compaction as specified in the Standard Method of Test for Moisture-Density Relations of Soil-Cement Mixtures (A.S.T.M. Designation : D 558) of the American Society for Testing Materials. Representative roadway soil shall be used except as modified in the following paragraph (b).

(a) For Soil-Cement Mixtures containing no material retained on the No. 4 Sieve.—A sufficient quantity of each soil-cement mixture to be investigated shall be prepared at optimum moisture content to provide two compacted specimens approximately 4.6 in. in height and 4.0 in in diameter.

(b) For Soil-Cement Mixtures containing Material Retained on the No. 4 Sieve—When the soil-cement mixtures contain material retained on the No. 4 (4760-micron) sieve, the following procedures shall be followed which vary from that described in Paragraph (a); Since the specimens are 4 in. in diameter, it is necessary to prescribe the maximum size material to be used in the specimen and this size shall be set at 3/4 in. Compensation shall be made for any material larger than 3/4 in. by replacing it with an equal dry weight of No. 4 sieve to 3/4 in. material of the same gradation as that contained in the original soil sample. A sufficient quantity of minus No. 4 sieve soil-cement material shall be prepared at the optimum moisture content to provide two compacted specimens 4.6 in. in height and 4.0 in. in diameter after the requisite amount of saturated, surface-moist, plus No. 4 sieve material, as defined above, has been added to the damp, minus No. 4 material.

Moulding specimens

4. (*a*) The prepared material shall be compacted in the mould in the same manner as that specified in the Standard Method of Test for Moisture-Density Relations of Soil-Cement Mixtures (A.S.T.M. Designation : D 558) of the American Society for Testing Materials, with the further provision that before placing and compacting the next layer, smooth compaction planes produced during compaction of each layer shall be scarified to form grooves, at right angles to each other, approximately 1/8 in. in width and 1/8 in. in depth and approximately 1/4 in. apart. During compaction, a representative soil-cement mixture moisture sample shall be taken from the batch, weighed, dried in an oven at 110° C. (230° F.) for at least 12 hr. or to constant weight. The compacted specimens shall be removed from the mould weighed, and the density determined ; the volume shall be determined by (1) averaging the diameters measured to the nearest 0.02 in. (0.05 cm.) at right angles to each other at the centre and the quarter points of the specimen and averaging the heights measured to the nearest 0.02 in. (0.05 cm.) at the quarter points on the circumference of the specimen, or (2) by immersion and displacement methods in a non-absorbent liquid such as mercury.

(*b*) One specimen shall be identified on a metal tag (or other suitable device) as No. 1, together with other needed identification marks and shall be used to obtain data on moisture and volume changes during the test. The other specimen shall be identified as No. 2, together with other needed identification marks and shall be used to obtain data on soil-cement losses during this test.

(*c*) After the required data have been obtained, the specimens shall be placed on suitable carriers in the moist room and protected from free water for a period of seven days. The No. 1 specimens shall be weighed and measured daily to give data for calculating the moisture content and volume of the specimens each day.

Procedure

5. (*a*) At the end of the storage in the moist room, the specimens shall be submerged in tap water at room temperature for a period of 5 hr. and removed. The No. 1 specimen (volume and moisture change specimen) shall be weighed and measured.

(*b*) Both specimens shall be placed in an oven at about 71° C. (160° F.) for 42 hr. and removed. Then, both specimens shall be weighed and No. 1 specimen measured. Specimen No. 2 (soil-cement loss specimen) shall then be given two firm strokes on all areas with a wire scratch brush to remove all material loosened during wetting and drying. The specimen shall again be weighed after brushing. The moisture content of the material brushed from the specimen shall be determined directly, but if this is not possible, it may be assumed to be equal to the moisture content of the No. 1 specimen. The oven-dry (110° C., 230° F.) weight of material brushed from the specimen shall be calculated.

(*c*) The procedures described in Paragraphs (a) and (b) constitute one cycle (48 hr.) of wetting and drying. The specimens shall then be again submerged in water and the procedure continued. The data collected will permit calculations of volume and moisture changes of specimen No. 1 and the soil-cement losses of specimen No. 2.

(*d*) The wetting and drying of the No. 1 specimens shall be continued for 12 cycles or discontinued prior to 12 cycles should the measurements become inaccurate due to distortion or soil-cement loss of the specimens. The wetting and drying of the No. 2 specimens shall be continued for 12 cycles, if possible.

Calculation

6. The volume and moisture changes and the soil-cement losses of the specimens shall be calculated as follows :

(*a*) The difference between the volume of specimen No. 1 at the time of moulding and subsequent volumes shall be calculated as a percentage of the original volume.

(*b*) The moisture content of specimen No. 1 at the time of moulding and subsequent moisture contents shall be calculated as a percentage of the original oven-dry weight of the specimen.

(*c*) The soil-cement loss of specimen No. 2 shall be calculated as a percentage of the original oven-dry weight of the specimen.

Appendix 3.

Standard Method of Freezing-and Thawing-Test of Compacted Soil-Cement Mixtures

A.S.T.M. DESIGNATION: D 560 - 44.

Scope

1. This method of test is intended for determining the soil-cement losses, moisture changes, and volume changes (swell and shrinkage) produced by repeated freezing and thawing of compacted specimens of soil-cement mixtures of known composition and of known uniform density and moisture content.

Apparatus

2. The apparatus shall consist of the following:

(a) *Mould.*—A cylindrical metal mould having a capacity of 1/30 cu. ft with an internal diameter of 4.0 in. and a height of approximately 4.6 in., which has a detachable collar assembly approximately $2\frac{1}{2}$ in. in height, to permit preparing compacted specimens of soil-cement mixtures 4.0 in. in diameter and approximately 4.6 in. in height which have a volume of $\frac{1}{30}$ cu. ft. The mould and collar shall be fastened to a detachable base (See Fig. 5).

(b) *Rammer.*—A metal rammer having a 2-in. diameter circular face and weighing 5.5 lb. The rammer shall be equipped with a suitable arrangement to control the specified drop. (See Fig. 6).

(c) *Sleeve.*—Closed cylindrical sleeve slightly less than 4.0 in. in diameter or similar device for removing compacted specimens from the mould.

(d) *Balances.*—A balance or scale of 25 lb. capacity sensitive to 0.01 lb.: and a 100-g. capacity balance sensitive to 0.1 g.

(e) *Drying Oven.*—A thermostatically controlled drying-oven capable of maintaining temperatures of about 110° C. (230° F.) for drying moisture samples.

(f) *Freezing Cabinet.*—A freezing cabinet capable of maintaining temperatures of minus 23° C. (minus 10° F.) or lower.

(g) *Moist Room.*—A moist room capable of maintaining a temperature of 21 ± 1.7°C. (70 ± 3°F.) and a relative humidity of not less than 90 per cent for seven-day storage of compacted specimens and for thawing frozen specimens.

(h) *Wire scratch Brush.*—A wire scratch brush made of 2-in. by $\frac{1}{16}$ in. flat No. 22 gauge wire bristles assembled in 50 groups of 10 bristles each and mounted to form five longitudinal rows and ten transverse rows of bristles on a $7\frac{1}{2}$ in. by $2\frac{1}{2}$ in. hardwood block.

Preparation of material for moulding specimens

3. This test shall be performed on soil-cement mixtures compacted in the mould, before cement hydration, to maximum density at optimum moisture content and in the same manner of compaction as specified in the Standard Method of Test for Moisture-Density Relations of Soil-Cement Mixtures (A. S. T. M. Designation: D 558) of the American Society for Testing Materials. Representative roadway soil shall be used except as modified in the following paragraph (b).

(a) For Soil-Cement Mixtures containing No Material Retained on the No. 4 Sieve.—A sufficient quantity of each soil-cement mixture to be investigated shall be prepared at optimum moisture content to provide two compacted specimens approximately 4.6 in. in height and 4.0 in. in diameter.

(b) For Soil-Cement Mixtures containing Material Retained on the No. 4 Sieve.—When the soil-cement mixtures contain material retained on the No. 4 (4760-micron) sieve, the following procedures shall be followed which vary from that described in Paragraph (a): Since the specimens are 4 in. in diameter, it is necessary to prescribe the maximum size material to be used in the specimens and this size shall be set at 3/4 in. Compensation shall be made for any material larger than 3/4 in. by replacing it with an equal dry weight of No. 4 Sieve to 3/4 in. material of the same gradation as that contained in the original soil sample. A sufficient quantity of minus No. 4 sieve soil-cement material shall be prepared at optimum moisture content to provide two compacted specimens 4.6 in. in height and 4.0 in. in diameter after the requisite amount of saturated surface-moist, plus No. 4 sieve material, as defined above, has been added to the damp, minus No. 4 sieve material.

Moulding specimens

4. (a) The prepared material shall be compacted in the mould in the same manner as that specified in the Standard Method of Test for Moisture-Density Relations of Soil-Cement Mixtures (A.S.T.M Designation: D 558) of the American Society for Testing Materials with the further provision that before placing and compacting the next layer, smooth compaction planes produced during compaction of each layer shall be scarified to form grooves, at right angles to each other, approximately 1/8 in. in width and 1/8 in. in depth and approximately 1/4 in. apart. During compaction, a representative soil-cement mixture moisture sample shall be taken from the batch, weighed, dried in an oven at 110°C (230°F.) for at least 12 hr. or to constant weight. The compacted specimens shall be removed from the mould, weighed, and density determined: the volume shall be determined by (1) averaging the diameters measured to the nearest 0.02 in. (0.05 cm.) at right angles to each other at the centre and the quarter points of the specimen and averaging the heights measured to the nearest 0.02 in. (0.05 cm.) at the quarter points on the circumference of the specimen, or (2) by immersion and displacement methods in a non-absorbent liquid such as mercury.

(b) One specimen shall be identified on a metal tag (or other suitable device) as No. 1, together with other needed identification marks and shall be used to obtain data on moisture and volume changes during the test. The other specimen shall be identified as No. 2, together with other needed identification marks and shall be used to obtain data on soil-cement losses during this test.

(c) After the required data have been obtained, the specimens shall be placed on suitable carriers in the moist room and protected from free water for a period of seven days. The No. 1 specimens shall be weighed and measured daily to give data for calculating the moisture content and volume of the specimens each day.

Procedure

5. (a) At the end of the storage in the moist room, water-saturated felt pads, blotters or similar absorptive material shall be placed between the specimens and the carriers, and the assembly placed in a refrigerator having a constant temperature not warmer than minus 23°C. (minus 10°F.) for 22 hr. and removed. The No. 1 specimen (volume and moisture change specimen) shall be weighed and measured and both specimens placed in the moist room. Free water shall be made available to the absorbent pads under the specimens to permit absorption of water by the specimens by capillarity.

(b) After 22 hr. in the moist room, both specimens shall be weighed and the No. 1 specimen measured. Specimen No. 2 (soil-cement loss specimen) shall then be given two firm strokes on all areas with a wire scratch brush to remove all material loosened during freezing and thawing. The specimen shall be again weighed after brushing. The moisture content of the material brushed from the specimen shall be determined directly, but if this is not possible it may be assumed to be equal to the moisture content of the No. 1 specimen. The oven-dry (110°C, 230°F.) weight of material brushed from the specimen shall be calculated.

(c) The procedure described in Paragraphs (a) and (b) constitute one cycle (48 hr.) of freezing and thawing. The specimens shall then be replaced in the refrigerator and the procedure continued. The data collected will permit calculations of volume and moisture changes of specimen No. 1 and the soil-cement losses of specimen No. 2.

(d) The freezing and thawing of the No. 1 specimens shall be continued for 12 cycles or discontinued prior to 12 cycles should the measurements become inaccurate due to distortion or soil-cement loss of the specimen. The freezing and thawing of the No. 2 specimens shall be continued for 12 cycles, if possible.

Calculation

6. The volume and moisture changes and the soil-cement losses of the specimens shall be calculated as follows:

(a) The difference between the volume of specimen No. 1 at time of moulding and subsequent volumes shall be calculated as a percentage of the original volume,

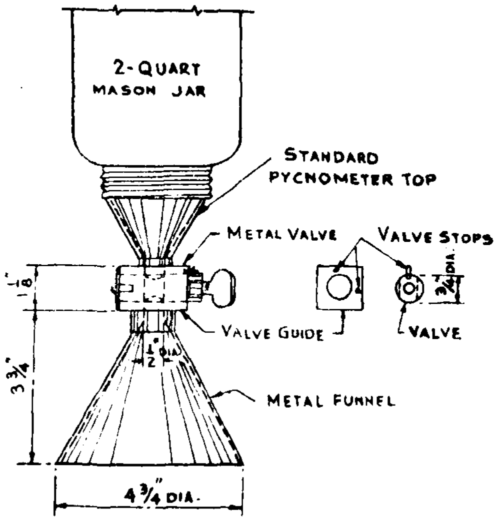
(b) The moisture content of specimen No. 1 at the time of moulding and subsequent moisture contents shall be calculated as a percentage of the original oven-dry weight of the specimen.

(c) The soil-cement loss of specimen No. 2 shall be calculated as a percentage of the original oven-dry weight of the specimen.

Appendix 4.

Use of sand funnel density apparatus for determination of field densities of soils and soil mixtures in place.

The sand-funnel density apparatus shown in Fig. 7 consists of a 2-qt. mason jar having the following pyknometer and funnel arrangement: The base of the density apparatus consists of a metal funnel with openings of $4\frac{3}{4}$ in. and $\frac{1}{2}$ in. A valve having a $\frac{1}{2}$ in. diameter hold is attached to the small end of the funnel. To the opposite opening in the valve, the standard metal pyknometer top is attached. The 2-qt. mason jar screws into the pyknometer top, completing the apparatus. (See Fig. 7).



FIELD DENSITY DETERMINATION APPARATUS

FIG. 7

In using the apparatus, a hole approximately 4 in. in diameter is excavated into the compacted soil or soil mixture to the proper depth. (When appreciable quantities of rock or stone are present in the soil or soil mixture, a larger diameter funnel and hole than those mentioned here should be used). The material removed from the hole is carefully saved and subsequently is dried and weighed.

The sand-funnel density apparatus full of sand of known bulk density is then placed over the hole and sand permitted to fill the test hole and the funnel portion of the apparatus. The total weight of sand used is determined by subtracting the final weight of the density apparatus plus the remaining sand in the jar from the original weight of apparatus full of sand. The weight of sand in the hole is equal to the total weight of sand used minus the weight of sand to fill the funnel portion of the apparatus. The volume of the hole equals the weight of sand in grams to fill the hole divided by the bulk density of the sand in grams per cubic centimeter. The density of the compacted soil or soil mixture in grams per cubic centimeter is equal to the dry weight in grams of material excavated from the hole, divided by the volume of the hole in cubic centimeters. The density of the compacted material in pounds per cubic foot equals the density in grams per cubic centimeter multiplied by 62.4.

TABLE 1

U.S.P.R.A. System of Soil Classification

Revised U.S. Public Roads Administration (P.R.) System (1945): Classification into Sub-Groups

General classification	Granular materials (35% or less passing No. 200 sieve*)							Silt-clay materials (more than 35% passing No. 200 sieve) *			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
1	2	3	4	5	6	7	8	9	10	11	12
Sieve analysis, percentage passing:											
No. 10 sieve * ...	50 max.	...	...	...	...	...	...	...	...	...	...
No. 40 sieve * ...	30 max.	50 max.	51 min.	..	...	...	...	...	...	...	...
No. 200 sieve * ...	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40 sieve *											
Liquid limit (%) ...	...		N.P.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index (%) ...	6 max.		...	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min [a]
Group index [b] ...	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.
Usual types of significant constituent materials...	Stone fragments, gravel and sand.		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as a sub-grade ...	Excellent to good							Fair to poor			

CLASSIFICATION PROCEDURE: With required test data available, proceed from left to right on above chart and correct group will be found by process of elimination. The first group from the left into which the test data will fit is the correct classification.

[a] Plasticity index of A-7-5 sub-group is equal to or less than liquid limit minus 30. Plasticity index of A-7-6 sub-group is greater than liquid limit minus 30.

[b] See group index formula Para. 2.1.8. for method of calculation. Group index should be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-6(12), A-7-5 (17), etc.

* Sieve sizes are American. Approximate British equivalents as follows.

United States A.S.T.M. E-11/26	Great Britain B.S. 410
10	8
40	36
200	200

TABLE 3

Characteristics of soil and soil-cement and their relation to cement content required for adequate hardness and serviceability

		TREATMENT GROUP								
		I			II			III		
Cement content by volume per cent	4 and 6	8			10 and 12			14 and 16		
U.S.P.R.A. soil group	A–2	A–2 and 3	A–4 and 5	A–6 and 7	A–2 and 3	A–4 and 5	A–6 and 7	A–2 and 3	A–4 and 5	A–6 and 7
	Range in test values									
Soil or soil-cement characteristic :										
Liquid limit of the raw soil ...	10–32	11–40	(1) 19–39	(2) 35–52	(3) 14–30	25–50	34–72	9–33	25–40	41–66
Plasticity index of the raw soil ...	0–19	0–22	3–20	17–37	0–26	1–23	14–43	0–11	7–17	18–40
Silt content of the raw soil per cent ...	6–39	3–36	29–66	12–45	0–26	16–72	7–66	0–16	51–79	15–64
Clay content of the raw soil per cent...	4–31	3–30	5–42	13–49	0–25	5–28	15–60	0–15	12–26	23–61
Organic content of the raw soil parts per million ...	tr*–4,000	tr–30,000	tr–3,500	tr–620	tr–20,000	tr–7,600	tr–6,000	2,500–36,000	tr–6,000	tr–32,000
Solids of soil cement, per cent ...	62–76	60–75	(4) 59–70	52–69	61–75	(5) 58–72	49–67	61–73	(6) 58–62	52–63
Density of soil-cement, lb. per cu. ft. oven-dry wt ...	99–129	110–127	(7) 103–122	88–117	104–125	(8) 96–113	86–112	102–121	(9) 98–103	(10) 84–103
Optimum moisture of soil-cement per cent ...	7–19	8–16	(11) 12–17	13–30	7–19	12–23	15–30	8–20	17–21	18–31
Compressive strength 10 per cent cement specimens, lb. per sq. in.										
Age 7 days ...	443–1,002	247–894	326–668	382–800	(12) 144–613	150–621	202–655	(13) 35–148	(14) 304–489	166–388
Age 28 days ...	(15) 726–1,078	373–1,261	464–920	(16) 460–710	(17) 326–920	207–890	248–939	(18) 212–515	(19) 327–570	223–600
Number of soils included in group ...	(20) 27	66	(21) 25	7	61	38	36	6	10	20

(1) One at 51. (2) One at 70. (3) Two soils with L. L. of 9, one at 10, one at 36 and one at 45. (4) One at 50. (5) One at 51. (6) One at 65. (7) One at 89, (8) One at 90 and one at 118. (9) One at 109. (10) One at 109. (11) One at 19 and one at 28. (12) One at 110. (13) Only two strengths. (14) One at 201. (15) Four other soils with strengths of 1,360, 1,460, 1,725, and 1,790. (16) One at 1,142. (17) Two soils with strengths 238 and 239. (18) Only two strengths. (19) Two other strengths 234 and 288. (20) Only two soils requiring 4 per cent cement. (21) Two soils require 6 per cent cement.

*tr — trace

A Policy on
Road-Side Advertisement

By the

SPECIFICATIONS AND STANDARDS COMMITTEE OF THE
INDIAN ROADS CONGRESS

1. INTRODUCTORY.

1.1. Indiscriminate erection of hoardings or display of advertisement panels (including wall panels on road-side structures) leads not only to the creation of traffic hazards but often also causes serious detriment to the local amenities and the general aesthetic characteristics of a neighbourhood.

1.2. At present the desired measure of control is either not exercised or is only partly exercised probably because the rules regulating the matter are designed primarily to ensure revenue collections in relation to the size of the structure or panel and the commercial value of the site.

1.3. Advertisements can often effectively distract the attention of drivers of motor vehicles and in that case are a public hazard or nuisance. They may also obstruct the view of the drivers of a fast moving vehicles and are then a public danger.

1.4. When they become an eyesore or otherwise are an " injury to amenity " they interfere with public rights and can be logically prohibited on that ground.

2. ADVERTISEMENT CONTROL.

2.1. " **Advertisement** ", in the following paragraphs, means any word, letter, model, signs, placard, board, notice, device, or representation, whether illuminated or not, in the nature of, and employed wholly or in part for, the purpose of advertisement, announcement or direction, (excluding any such thing employed wholly as a memorial or railway signal).

2.2. " Advertisement " includes any hoarding or similar structure used or adapted for the display of advertisements.

Note :—Any sign-post, bus sign, name plate, warning sign etc., not being on a vehicle, will be included under the head " Advertisement ", the dictionary meaning of the word being " public announcement ".

2.3. In the **Model Highway Bill** powers are given to control advertisements by regulation. It is also laid down that for all new advertisements, (except for those which are "deemed as consented") prior consent must be obtained from the road authority.

2.4. The following paragraphs discuss the principles which it is proposed should be applied before consent is given to the erection of advertisements on road lands or structures adjoining roads. They also cover the case where advertisements on private land adjoining a road may destroy amenities or otherwise may be a public nuisance.

3. PRINCIPLES OF ADVERTISEMENT CONTROL.

3.1. What may be considered as objectionable

3.1.1. In general, advertisements should not be permitted ;

(1) at or within 350 ft of any road junction or crossing ;

[*Note* :—The safe stopping distance for a vehicle travelling at a speed of 30 m.p.h. is 200 ft. This should be the "uninfluenced distance" for a driver approaching a junction. Assuming that 3 seconds is the time during which the influence of an advertisement board persists the distance travelled in this time will be 132 ft. The sign should, therefore, be more than 332 ft away from the junction. Hence 350 ft suggested.]

(2) at places where the line of vision of the driver of a motor vehicle is obstructed, thereby creating a hazard, as on curves ;

(3) within 30 ft of the edge of a carriageway ;

[*Note* :—A distance of 30 ft may be taken as the normal minimum set-back from the edge of the carriageway, the maximum area of the advertisement being one square foot for every foot set-back.]

(4) within 150 ft along the road, of any sign board erected for the regulation of traffic under the orders of a Public Authority such as a Traffic Authority, a Public Transport Authority, or a Local Authority ;

[*Note* :—This is intended to keep traffic signs and signals clear from all advertisement boards. As traffic signs are generally planted 6 to 8 ft from the edge of the carriageway, the set-back of advertisement boards should be such as not to obstruct these sign boards.]

(5) in such a form as will obscure or render difficult of interpretation the signs erected by the Public Authorities. For example the advertisement should not correspond in colour representation to the standard legal traffic signs or use such words as STOP, WARNING, etc. in such a size as to correspond with the same words as used on traffic signs ;

(6) on boards or placards hung across a road as they distract the attention of the driver and are, therefore, hazardous.

3.2. What may be considered as Affecting Amenities

3.2.1. From aesthetic considerations the display of commercial advertisements should be strictly controlled where either the general characteristics of the locality are predominantly those of a residential neighbourhood or where natural scenery or public improvements are thereby likely to be depreciated.

3.2.2. The frontage of buildings of a dignified and subdued character such as hospitals, educational institutions, public offices, memorials of national importance, etc. should be protected from the vandalism of commercial advertisements.

3.3. Relaxations

3.3.1. In the public interest advertisements are permissible in the following circumstances :

(1) Official warning signs, traffic directions, sign posting, and notices or advertisements posted or displayed by or under the direction of any public or court officer in the performance of his official or directed duties. For example notices of the sale of Improvement Trust Lands, direction signs to petrol filling station, repair shops, and garages etc.

(2) Signs relating solely to any city, town, village, or historic place, shrine, places of tourist interest, subject to the proviso that they do not obscure any traffic warning signs and conform reasonably with the requirements of para 3.1.1.

(3) Signs, notices, etc. for the information of members of the armed forces or the public erected by the Defence Departments.

Table 1

Class of Advertisement.	Maximum Area	Height to top of board	Illumination	Number	Size of Symbol	Remarks
1. Functional. (Public Notices)						
BUS STATION — Local Authorities P.W.D. etc.	..........NO LIMIT					
2. Advertisements on Buildings.						
2.1 Identification, Warning, etc.						
X-RAY; MIND THE STEP	2 sq. ft.	Not more than 12ft	Only when night service exists	As required	1 ft	
2.2. Personal Business.						
RAM LAL AND COMPANY	3 sq. ft.	-do-	-do-	One on each frontage	-do-	
2.3. Institutions, Residences etc.						
COLLEGE OF ENGINEERING	12 sq. ft.	-do-	-do-	-do-	-do-	Must have entrances where displayed.
3. Temporary Advertisements						
3.1. Temporary Notices.						
HOUSE FOR SALE; TO LET	20 sq. ft. but projection from buildings not more than 3 ft	-do-	Nil	One	-do-	
3.2. Sales.						
SALE THIS WEEK	12 sq. ft.	-do-	Nil	One	-do-	

(4) Signs or notices, 2 sq. ft. in area or less, placed so as to show direction to a residence or place of business, and planted sufficiently away from the carriageway.

4. DIMENSIONS OF ADVERTISEMENTS.

4.1. **General Rule**. The general rule for the maximum permissible area of advertisements should be one square foot area for every foot of set-back from the edge of the carriageway.

4.2. Table 1 gives permissible dimensions in particular cases.

5. CONDITIONS ATTACHED TO PERMITS.

5.1. The following conditions might be attached to permits :

(1) The sign must be maintained in a clean, tidy, and safe condition ;

(2) If the Road Authority requires the removal of the sign, it must be removed forthwith ;

(3) The sign must not be put in such a position as to obscure or hinder the ready interpretation of any road traffic sign, railway signal, or any other public notices ;

(4) The grant of permission is not to be taken as operating to discharge any obligation or liability imposed or incurred by any other enactment in force in relation to any operation or other matter in connexion with building bye-laws or any other laws or bye-laws concerning roads, buildings etc.

The Standards & Specifications Committee of the Indian Roads Congress recommended that a provision for controlling advertisements on private property abutting roads should be included in the Model Highway Bill and that the conditions enumerated in para. 5 should be followed until such a provision is made.

"**Roads in New Zealand**"

By

B. G. Naik, B. E., A.M.I.E.

1. INTRODUCTORY.

1.1. Under the Technical Assistance Scheme of the Colombo Plan the Author had an opportunity of studying the Road System of New Zealand last year. In the following paragraphs an attempt is made to cover briefly the interesting points seen from a Road Engineer's point of view.

1.2. New Zealand is a small country consisting of two large and several small islands lying in the South Pacific Ocean, 1200 miles south of Australia. The two main islands are known as North Island and South Island with an area of about one lakh square miles. In the South Island the central portion is mostly mountainous and on the east coast there are alluvial plains. The west coast gets very heavy rainfall, varying from 100 in. to 200 in. On the eastern side the rainfall is much less and it varies from 12 in. to 25 in.

1.3. North Island is volcanic with some active volcanoes in the central portion. The annual rainfall in North Island is much better distributed and varies from 25 in. to 50 in. As the rains are spread all over the year, the whole country appears to be covered with a green carpet all the time. The scenery is grand and romantic. The abundant rainfall results in luxuriant growth of vegetation of all types. Most of the trees are of the evergreen sort. Hills with green slopes and streams of crystal clear water are special features of this country.

1.4. The population of the country is about 19 lakhs and the traffic consists mostly of trucks and cars. The registered number of vehicles in the country is about four lakhs. No trace of iron-tyred traffic is seen anywhere. The islands have a temperate climate and the temperature generally varies between 40 deg. F and 65 deg. F. Two-thirds of the population of the country live in North Island which has generally a warmer climate than South Island. The population is spread throughout the country, with big sheep and cattle farms and small towns served by small-sized dairy, fruit, and wheat farms. The main cities with a population of more than one lakh are Auckland, Wellington, and Christ Church.

2. THE ROAD SYSTEM

2.1. The roads in the country are divided into the following main categories :—

(1) State Highways (Financed wholly by the State).

(2) Main Highways.

(3) Public Roads.

2.2. State Highways are constructed and financed by the Ministry of Works while the Main Highways are maintained by the Counties and Boroughs and the State gives a subsidy in the proportion of 3 to 1. Public Roads are totally financed by the City, Borough, or County Authorities from their own revenues. For extra-ordinary repairs of damage done by storms or floods, the State gives special assistance to the rural or local authorities by way of some subsidy or grant.

2.3. There is a **Highway Board** to advise on road finance and policy. The Board consists of six members. Three of them are appointed by Government, two being officers of the Public Works Department. Two representatives on the Board are from the County Councils, and one from the Automobile Association. The Board has full powers of taking decisions in the matter of construction, reconstruction, improvement, maintenance, repairs, and control of all State and Main Highways. No new construction, however, is actually commenced without the prior consent of the Minister.

2.4. For the purpose of road maintenance the country is divided into twenty **Highway Districts**, each District being composed of several counties grouped together according to their geographic situation and community of interests. For each Highway District there is an advisory body known as the District Highways Council which is constituted to include an engineer of the Works Department and one representative of each constituent county. The principal function of these councils is to recommend to the Board each year as to which roads within the District should be declared Main Highways and which special repair works should be undertaken.

2.5. Government assists in the construction of roads and bridges in counties and Highway Districts wherever necessary in the national interest after taking into consideration the recommendation of the Board. Wherever possible the County Councils arrange to finance their road construction from their own revenue but as a rule the construction of roads and bridges necessitates the raising of loans. Such borrowing is subjected to the approval of the Government Loans Board.

3. FINANCE.

3.1. Only State Highways are totally financed by Government. For Main Highways the Government pays a subsidy at the rate of £3 to £1. The revenue accrues mainly from road users, from the motor spirit tax, the mileage tax on non-petrolconsuming vehicles, motor vehicle licensing, and the tyre and tube tax. Fees levied on drivers' licenses and heavy traffic (over two tons) are retained by the local authorities. The law requires that an amount at least equal to the above-mentioned revenues will be provided in the budget for expenditure on highways in any one year. This amount normally comes to a high percentage, as much as six per cent, of the total budgeted expenditure of the Government. Again, this expenditure is incurred on about 15 per cent only of the total mileage of roads as Government finances only State and Main Highways—the latter only partly. The balance 85 per cent of the roads are financed wholly by County Councils who collect their revenues by rates on property. Thus, a major portion of the expenditure on roads comes from the general tax payer.

3.2. The Government also pays subsidies to cities and counties for the construction of footpaths, pedestrian crossings, centre line marking, and other improvements wherever necessary.

4. ROAD MILEAGE

4.1. Table 1 sets out the mileage of roads and highways and the type of surface as on the 31st March 1950:

Table 1—**Milage of Roads in New Zealand**

Surface	State	Counties*	Cities and Boroughs	Total
(a) Bituminous Low Cost (b) do High Cost	2903	2593	2467	7963
(c) Concrete	35	13	—	48
(d) Brick & Block	—	—	6	6
(e) Other all-weather surfaced	2290	34396	1467	38153
Total mileage surfaced	5228	37002	3940	46170
Mileage graded and drained	—	7289	244	7533
Total mileage improved	5228	44291	4184	53703
Total mileage unimproved	—	22665	349	23014
Grand Total	5228	66956	4533	76717

* These figures include the following Main Highway mileage.

(a) and (b) 1385
(c) 13
(d) 5898 Total 7296.

5. MOTORWAYS

5.1. Wherever the traffic intensity has exceeded 2000 vehicles per hour, the Government have planned the construction of motorways on the lines of turnpikes or freeways of the U.S.A.

5.2. In addition to providing the most efficient and economic transport service, the main characteristics of a motorway are the control of access and a total elimination of ribbon development, with a resultant improvement in road safety.

5.3. In many respects a motorway resembles a railway ; it provides no property frontage, and access to it is limited to stations located at intervals to serve townships and settlements.

5.4. Motorways are generally four-lane routes, with a central hedge or a wide plot separating the two up-lanes from the two down-lanes.

5.5. Pedestrians, cyclists, horse-drawn vehicles, and cattle are prohibited by legislation from entering the motorway.

5.6. A six-mile section of the Wellington-Foxton Motorway has been completed and, during the twelve months it has been in service, it has been free from accidents.

5.7. Further construction of motorways is in progress on the following sections :—

(1) Auckland to Kumon
(2) Auckland to Hamilton
(3) Dunedin to Waitati

6. PAVEMENT DESIGNS

6.1. A striking feature of pavement design in New Zealand is the elimination of the boulder soling and the macadam type of top course. Instead flexible gravel base and top courses with an admixture of clay matrix are provided. This is found more economical.

6.2. Apart from the high cost of macadam construction it is found that for bitumen surfacing it is essential to have a smooth base which it is not possible to obtain with the roller-compacted macadam. In the opinion of Brigadier Hanson, the Deputy Commissioner of Works in New Zealand, a surface compacted with a smooth-wheel roller, no matter how smooth it is immediately after construction, will not retain its smoothness. The smooth-wheel roller has a mutilating effect on the aggregates and causes greater compaction at the surface than at the bottom of the metal crust. A roller also rides on slightly high spots and leaves the rest of the area with much less density.

Again, the macadam type construction requires hard stone of high quality to stand the crushing effect of the roller. A rough road surface (which a roller-compacted road ultimately turns out to be) requires a greater thickness and a better subgrade to distribute the high impact stresses caused by heavy traffic.

6.3. Further, the development of low-viscosity bituminous primers has done away with the necessity of a hard well-rolled macadam surface as a base for an asphalt wearing surface, which was supposed to be very essential at one time. For all these reasons preference is given in New Zealand to gravel base and top courses compacted with pneumatic-tyred rollers.

6.4. **Base Course.** This generally consists of a dense, graded, mineral aggregate-mix including cohesive clay, laid on the subgrade, bladed, sprinkled with water, and compacted with pneumatic-tyred rollers to a true finish. The mineral aggregate consists of either natural pit gravel or an artificial mixture of river shingle, crushed stone, sand and an approved clay binder.

6.5. The thickness of base course generally varies from 5 in. to 7 in., depending upon the subgrade and on whether the aggregate is well graded or not. When the depth of base course is 5 in. or more, the aggregate has to satisfy the following sieve test :

			Per cent
Passing	$2\frac{1}{2}$ in.	(circular screen)	100
,,	$1\frac{1}{2}$ in.	(do)	75–85
,,	$\frac{3}{4}$ in.	(do)	55–65
,,	$\frac{1}{4}$ in.	(do)	30–40
,,	10 mesh	(B. S. Series)	20–30
,,	50 mesh	(do)	10-20
,,	200 mesh	(do)	5–10

6.6. Besides the sieve test it is necessary that the aggregate retained on 10-mesh sieve, should be of stone having a French coefficient of wear of not less than 3.5. So also the tension of the binder fraction passing 50-mesh sieve should not be less than 10 lb per sq. inch and the lineal shrinkage should not be more than 5 per cent.

6.7. **Top Course.** This is similar to the base course except that the size of aggregate is $\frac{3}{4}$ in. and less. The thickness of top course is generally $1\frac{1}{2}$ in. to 2 in. and the aggregate has to meet the following requirements.

			Per cent
Passing	$\frac{3}{4}$ in.	(Circular screen)	100
,,	$\frac{1}{4}$ in.	do	60–80
,,	No. 10 mesh	(B. S. Series)	40–60
,,	No. 50 mesh	do	25–35
,,	No. 200 mesh	do	10–20

The portion of aggregate retained on the 10-mesh sieve should be of stone or rock with a French co-efficient of wear of not less than five. The quality of binder fraction passing 50-mesh is the same as that required for base course.

6.8. The approximate top course mixture of stone screenings, sand, and clay is generally :

3 parts of metal screenings graded from $\frac{3}{4}$ in. (circular) down to 10-mesh—i.e., graded screenings without sand, as is often the practice for producing screenings for concrete work.

Plus 2 parts of graded sand, i. e., coarse sand from 10-mesh down to fine sand.

Plus 1 part of clay, i. e., plastic material mostly passing the 200-mesh sieve.

In short, this stabilised gravel mixture may be considered as a 1 : 2 : 3 mix.

6.9. From the foregoing description it will be clear that the pavement design is based on the principle of correct gradation of aggregate and the strength of the clay binder as opposed to the mechanical interlocking of the macadam type of construction.

6.10. **Wearing Courses or Surface Treatments.** Roads which have a daily traffic of more than 200 vehicles are mostly sealed by surface dressing. The sealing coat consists of the application of an asphaltic road-oil covered with stone chips after priming the gravel surface with tar primer.

6.11. A special feature of the surface dressing system is the use of a one-size aggregate conforming to the following specifications :

Passing 1 in. circular screen, not less than 45 per cent.

Retained on $\frac{3}{4}$ in. circular screen, not less than 95 per cent.

6.12. The bitumen comes up approximately to half the height of the single layer of chips, thus leaving a rough surface with open interstices. This system gives a saving in bitumen, since only half the surface area of each chip is coated with bitumen instead of the whole area coated in the normally close graded surface dressing used in India.

6.13. The large sized metal (i.e., up to 1 in.) used in New Zealand gives a greater thickness of wearing surface with a life of up to 10 years. The rough surface ensures good grip between the road and the tyres and the risk of skidding in winter is much reduced. It, however, makes driving noisy and increases the wear and tear of tyres.

7. BRIDGES

7.1. Due to the very hilly and undulating nature of the country, it is difficult to adopt straight alignments. This has necessitated the construction of curved and skew bridges which are very common. In all recent constructions no effort has been made to obtain right-angled or straight crossings. The bridges are laid according to the natural crossing whether on curve or skew. The curved bridges are also adequately superelevated for the design speed of the road.

7.2. Old bridges in the country are mostly single-lane wooden truss bridges constructed out of Australian hard wood. Many such bridges are still serving the needs of traffic, though on the Main and State Highways, they have been replaced by modern R. C. and steel girder bridges.

Where swift streams, flowing in deep valleys and carrying much drift and ice, have to be crossed, single lane suspension bridges are constructed.

7.3. In general the trend of modern bridge construction in the country shows that

(1) the existing single lane wooden bridges are gradually being replaced with two lane bridges;

(2) steel I-beams and plate-girders are being used in diverse ways in combination with concrete flooring for spans up to 100 ft;

(3) R. C. girder bridges are used for spans up to 80 ft;

(4) Concrete flooring is replacing timber flooring;

(5) alignments are being improved and proper grades, sight distances, width of roadway, and superelevation are being provided to cater for speeds of the order of 60 miles per hour.

Photo: 1
A view of a state Highway.

Photo: 2
Top course being dressed and compacted by a grader.

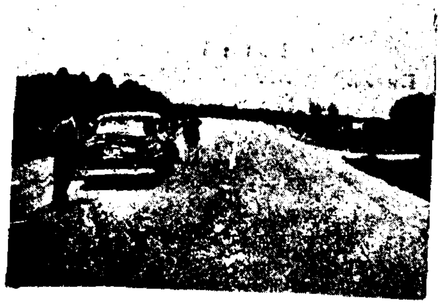


Photo: 3
A Main Highway sealed with one-size aggregate.

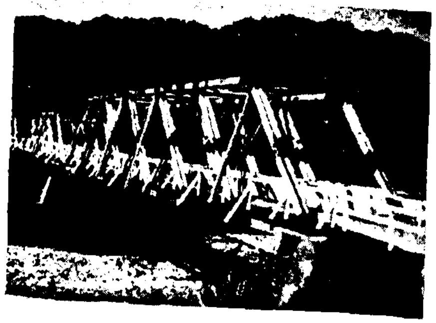


Photo: 4
An old wooden truss 80 ft span bridge

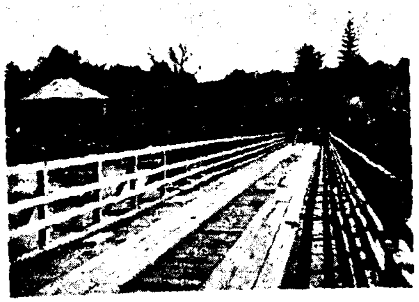


Photo: 5
A typical wooden stringer bridge

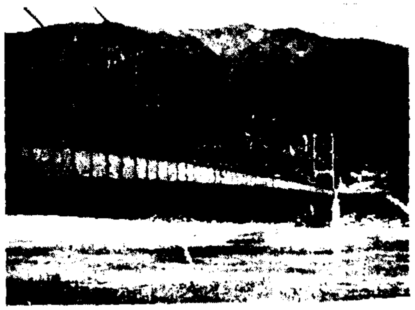


Photo: 6
Suspension bridge on the Karang River, South Island (420 ft)

Photo: 7
Karangna River bridge—View of the roadway.

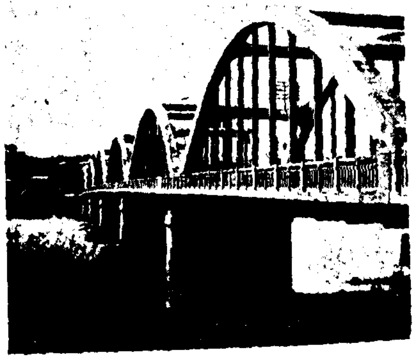


Photo: 8
R. C. Bowstring girder bridge on the Balchan River in South Island—Six spans of 120 ft.

7.4. **Materials used in bridge construction.** Principal materials used are timber, concrete and steel. Stone masonry is almost extinct as, due to high labour charges, its cost is prohibitive. Timber is extensively used for foundation piles, floors, and railings. Recent bridges are constructed mostly of reinforced concrete or steel.

7.5. **Bridge Design.** As regards loadings for bridge design, the practice is to follow the A.A.S.H.O. Specifications of 1941, H 20 - S 16. Provision is also made for earthquake stresses as North Island lies in a seismic area.

7.6. **Foundations.**—When open foundations are not feasible, the common practice is to drive piles. Precast reinforced concrete piles, varying from 20 ft to 60 ft in length, are used and are driven with a hammer operated by gravity, steam, or air.

7.7. **Use of Bailey Bridges.**—To meet emergencies when bridges are damaged, the Department has a large stock of Bailey bridges. The Author saw many Bailey bridges functioning in place of old damaged bridges, while new permanent bridges were being built.

8. WORKSHOPS AND PLANT ORGANIZATION

8.1. This forms a very important part of the organization as, on account of a great scarcity of labour, almost all works are done with the aid of modern mechanical equipment, mostly imported. There are three main **Zone Workshops** situated at Auckland, Wellington, and Christ Church respectively, which look after the maintenance, overhauling, and supply of machinery to the Districts. Besides these zone Workshops there are **District Workshops** under the District Engineers to look after the routine maintenance of machines.

8.2. The equipment used on different types of road work is listed out in Table 2.

Table 2.

Type of Work	Equipment used
1. Ordinary Earth Roads or Fair Weather Roads	
(1) Shaping of road.	Tractor-drawn and motorised graders.
(2) Cutting down shoulders and disposal of surplus material.	Tractor-drawn and motorised graders. Trucks and scrapers are used for disposing of surplus material.

Type of Work	Equipment used
2. Gravel-Surface Type	
(1) Blinding or topping cover.	Dump trucks with chained tail boards or graders.
(2) Removal of iron scrap and waste from the road surface.	Truck-mounted units of lifting electro-magnet outfits are used, energised by generators of up to 5 k.w. capacities.
(3) Stock piling, transporting, loading, and unloading of materials.	Belt conveyors are used on stock piles, and cranes with chain buckets are used for loading and unloading work.
3. Treated Surfaces	
(1) Spreading of aggregates.	Chip spreaders of various types.
(2) Surface cleaning.	Rotary power brooms.
(3) Application of bitumen to the road surface.	Distributors mounted on trucks with capacities of 800 to 1,200 gallons, attached with power spray are used.
(4) Mixing of bituminous materials and aggregates.	Mostly portable asphalt plants are used. On small jobs, tilting drum concrete mixers are also used.
(5) Aggregate handling and distributing.	Belt Conveyors, bucket conveyors, and truck cranes are used.
4. Major Excavation and Hauling	
(1) Shallow cuts and side borrow pit construction.	Towed or self-propelled graders, tractors, and angle dozers.
(2) Heavy cuttings.	Elevating graders, elevating loaders, and tractor-drawn scrapers.

Photo: 9
A modern R. C. girder super-elevated bridge on a curve.

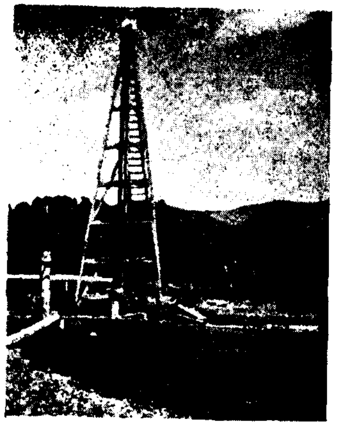


Photo: 10
Pile driving in New Zealand

Photo: 11
A Bailey Bridge

Photo: 12
Center line marker

Photo: 13
A veiw of a construction camp

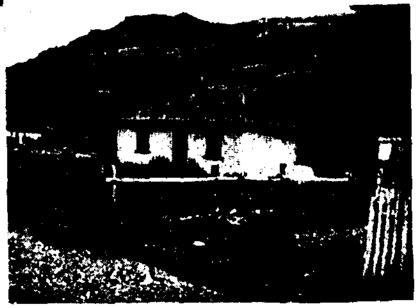


Photo: 14
A married road man's quarter

Photo: 15
Single workers' quarters

Photo: 16
Temporary work-site quarters

Type of Work	Equipment used
(3) Excavation on side hills (ghats) where the material is to be hauled laterally.	Power shovels of tractors with bulldozer or angle dozer attachment.
(4) Rock excavation.	Power shovels and bull dozers.
(5) Hauling excavation material.	Scrapers and trailers pulled by Crawler-type tractors.
(6) Excavating trenches.	Rotary ditchers.
5. General	
(1) Cutting and disposing of weeds and grass on berms.	Motor drawn powers.
(2) Field garages for maintenance of equipment.	Wrenches, chain hoist, air compressors and usual kit of shop tools.
(3) Rolling.	Tandem steel wheel rollers; tractor-drawn sheepsfoot rollers, and pneumatic tyre rollers.
(4) Shifting heavy machinery.	(a) Heavy duty trailers pulled by trucks.
	(b) 5 ton, 6-wheel truck which can transport any machine up to 25 ton load.
(5) Centre line marking on roads.	Centre line markers with 60-gallons spraying tanks and side-wheel attachment mounted on 2-ton trucks.

9. LABOUR CONDITIONS

9.1. The country is very short of labour. The labour regulations require a 40-hour week with two holidays. Overtime is paid at a "time-and-a-half" on working days and all work is paid at twice the rate on holidays. Employment conditions for labourers are very liberal. Most of the workers are members of New Zealand Workers Industrial Unions.

9.2. The prevalent rates and wages for certain classes of workers are given below :—

(1)	Labourer	Rs 2/2/-	per hour
(2)	Skilled labourer	Rs 2/5/-	,, ,,
(3)	Mason, carpenter, black-smith, electrician, plumber, etc.	Rs 2/8/-	,, ,,
(4)	Road gangmen	Rs 2/7/-	,, ,,
(5)	Truck driver	Rs 100/-/-	,, week
(6)	Steam road roller or Tractor driver	Rs 110/-/-	,, ,,

9.3. When working under special conditions, the workers are entitled to various types of allowances such as, Dirty Work Allowance, Cement Allowance, Coal Allowance, Wet Weather Allowance, Gumboot and Protective Clothing Allowances, etc.

9.4. On construction works, accommodation is provided for married and single workers separately. Facilities for recreation are also provided by the Department.

9.5. **Married Workers Accommodation Scale.**—Minimum three rooms, 2 x 10 ft x 11 ft, and 1 x 10 x 12 ft, with 7 ft sheds, shelter parts and a suitable combined wash house and bathroom.

9.6. **Single Workers' Accommodation.**—Huts or tents are provided for single workers. The minimum size is 10 ft x 8 ft width at least 6 ft shed lined with timber and equipped with canvas bunk, small table, stool, shelves, and fire place.

Other facilities include, hot and cold water supply, washing platform, drying places, and canteen, etc.

10. CONCLUSION.

The observations made above relate to only a few of the impressions obtained and are by no means a detailed or complete account of roads in New Zealand. The colonisation of New Zealand was started in 1840 only and, therefore, few would expect to see much of development in this country. But it has to be admitted here that the progress made by this small country in a short span of hundred years is amazing. The credit goes to the New Zealand engineers, both past and present, who, facing many adverse conditions, have built up this beautiful country.

—o—

RESEARCH SECTION

THE COMPACTION OF SOME ALLUVIAL SOILS OF INDIA

CENTRAL ROAD RESEARCH INSTITUTE

OKHLA, DELHI.

FOREWORD

In 1950 the Road Research Laboratory, Harmondsworth, London, published a particularly useful study in its Road Research Technical Paper No. 17, "The Compaction of Soil. A Study of the Performance of Plant." The main object of the British investigation was to study the compaction effects of different kinds of rollers and of other compaction plant on different soils. Their research extended over four years.

In India similar problems to those of the United Kingdom have arisen in the compaction of alluvial soils encountered in engineering construction. Accordingly, the Central Road Research Institute here has investigated the problems on similar lines.

The initiative taken by the Engineer-in-Chief's Branch of the Indian Army in regard to this investigation, as well as the supply of plant and personnel, are acknowledged.

The studies were conducted as a part of the research programme of the Central Road Research Institute and were executed collectively by the staff of the Institute. The Director wishes also to acknowledge the help given by Mr. J. M. Trehan, Honorary Secretary, Indian Roads Congress, in editing this note.

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THE COMPACTION OF SOME ALLUVIAL SOILS OF INDIA*

1. INTRODUCTION

1.1. For the proper use of soil in highway engineering construction it is very essential that the soil should fulfil a number of primary conditions expected of a road material. A very important condition is that it should be capable of being properly compacted. During the process of compaction the soil particles are forced to draw close to each other and this results in a lower percentage of voids and a higher density. It is well known that soil improves in its all-round engineering properties as the degree of compaction increases.

1.2. Compaction is generally brought about by mechanical means and therefore it is desirable to study the effect of various types of compacting units to find out their suitability for particular types of soil.

The objects of this study are :

(1) To investigate the performance of the types of plant available in Delhi in compacting some of the alluvial soils met with in India ;

(2) To find out the number of passes of the plant required for a desired compaction of the soil ;

(3) To find out the effect of moisture content of the soil on its compaction ;

(4) To compare the results of some laboratory compaction tests with those of plant compaction in the field ; and

(5) To determine the output of each plant in terms of the volume of soil compacted per hour.

1.3. In order to obtain a comparative study of the effect of compacting plant on Indian and British soils, the present

*In comparison with a study of the performance of plant entitled: "The Compaction of Soil. A Study of the Performance of Plant." Road Research Technical Paper No. 17, Road Research Laboratory, Harmondsworth, England.

The content and sequence of this study follow the British study.

investigation has been carried out on lines similar to those described in Road Research Technical Paper No. 17 of the Road Research Laboratory, Harmondsworth, London, on 'The Compaction of Soil.' The Figures and Tables in this study generally follow those in the Road Research Paper and it will therefore be advantageous to the reader if he compares the results of the two studies side by side.

1.4. In the British investigation the degree of compaction has been measured in terms of dry density, which means the amount of dry soil per unit volume. This can be expressed either as kilograms of dry soil per cubic metre or pounds of dry soil per cubic foot. In the present study the relationship between density and moisture content has been expressed in both the systems.

2. FIELD EXPERIMENTS

2.1. Soils used in the investigation

2.1.1. It may be seen by comparison that the soils used for the study are, in their grading properties, between sand and silty clay (vide Fig. 1). The reason for selecting only these soils is that the main soils met in the northern parts of India belong to these categories. It was not considered necessary to collect more soils with a well distributed grading to cover a similar range as in the British Paper. Therefore, the present study has been purposely confined to soils covering the most interesting range.

2.1.2. The four soils used in the investigation were a clayey soil, a silty soil, a sandy soil, and a sand. Information concerning their particle size distribution is given in Fig. 1, and their physical and chemical characteristics and plasticity in Table 1.

2.1.3. Besides the sand content (i.e., particles bigger than 0·05 mm) given in Table 1, the four soils had the following contents of clay and silt :

Name of soil	Sand 2 mm to 0·05 mm %	Silt 0·05 mm to 0·005 mm %	Clay Less than 0·005 %
Clayey soil	32	48	20
Silty soil	52	30	18
Sandy soil	62	24	14
Sand	100	—	—

The names given to the soils in the study do not comply fully with the usual textural classification. They have been selected to indicate the main properties of the soils in comparison with each other.

2.2. Compaction Equipment used

2.2.1. The performance of the following types of plant was studied :

1. 7-ton smooth-wheel roller (Photo 1) ; the leading dimensions and loadings are given in Table 2.
2. Pneumatic-tyred roller (Photo 2); the leading dimensions and loadings are given in Table 3.
3. Taper-foot Sheepsfoot roller (referred to simply as "Sheepsfoot roller" in this study), (Photo 3); the leading dimensions and loadings are given in Table 4.

2.2.2. The smooth-wheel roller was self-driven and the other two machines were towed by a tractor (D-4) of 43 H. P.

2.3. Layout of the road track for compaction trials

2.3.1. A straight length of about 400 ft was prepared by compacting the subgrade with a 7-ton smooth-wheel roller. The width of the track was kept at 10 ft and the sides were lined with brick-on-end, giving a finished height of 6 inches. Since a 9-in. layer of loose soil was taken for compaction at a time, planks about 9 to 10 ft in length, with a width of 9 in., were used to support the soil on the sides. The planks themselves were supported by the brick-on-end edging (see Photo 7).

After the layer of 9 in. loose soil was levelled on the surface with a straight edge, the side planks were removed and used for subsequent lengths.

2.3.2. A cross section of the track is shown in Fig. 2.

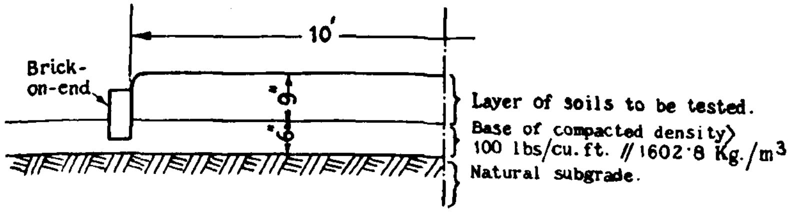

FIG. 2–CROSS SECTION OF LAYOUT FOR SOIL COMPACTION TEST.

2.3.3. The working speed of the smooth-wheel roller was kept at about two miles an hour and that of the pneumatic-tyred and sheepsfoot rollers between three and four miles an hour. The effect of the speed of the plant on the compaction of soils has been studied by Maclean, Bailey and Hunt[1] and it has been shown that a variation of speed between two and five miles an hour makes no appreciable difference to the resulting density of the compacted mass.

2.3.4. The soil required for the experiments was stacked close to the track to minimise the possible evaporation in handling the soil. The track was kept uncovered so as to correspond to actual field conditions.

A general view of the entire layout is shown in Photo 5, and the preparation and laying of the soil in Photos 6, 7, and 8.

2.3.5. The moisture content desired in each case was added in all tests before compaction began. As some evaporation was expected to take place during compaction the initial percentage of the moisture added was kept a little higher than the moisture content nominally under test. The extra moisture to be added was calculated from the experience of the previous day, i.e., from the amount of moisture which had evaporated during compaction on the previous day. As the weather conditions were expected to be uniform from day to day, this computation was easily done. In spite of these precautions some unforeseen variation took place but was found to be within tolerable limits. The moisture content was always determined before starting compaction.

2.3.6. The track differed widely from the one used for similar tests at the Road Research Laboratory, Harmondsworth, London. The test track in London was circular in form and protected against rain by being covered. In addition, the roof reduced evaporation. The tests could be carried out under the same conditions because all the plant could be made to run exactly on the same circular path, being driven by a machine fixed outside the track, at the centre of the circle.

2.3.7. The track at the Central Road Research Institute in India was kept in the open because, during the time of the year in which the tests were carried out, there are usually no rains. As the track was exposed to the direct sun, the tests were carried out in the early morning hours. It was further considered advantageous to have the test track in a straight length and the site permitted this. Since each unit of the compacting plant was either towed or self-driven, it was not possible for it to move every time exactly on the same path during the compaction. As a result each compacting unit made a trail wider than its rolling width by about 1 ft on each side.

2.3.8. It would, however, appear that the conditions mentioned above are not disadvantageous. They are very similar to those prevailing in actual practice and, therefore, the experiments are expected to give a more realistic picture of a large scale field test than those done in a covered place.

2.4. Measurement of Compaction

2.4.1. The degree of compaction of a soil has been expressed in terms of its dry density. It is calculated according to the following formula:

$$D_d = D_w\left(\frac{10}{100 + m}\right)$$

where D_d = dry density, lb per cu. ft. or kilograms per cubic metre (weight of dry soil particles in a unit volume)

D_w = bulk density, lb per cub. ft. or kilograms per cubic metre (weight of one unit of soil including solids, air, and water)

m = moisture content (per cent) (i.e., loss of weight of the soil when dried to constant weight at 105 deg. C., expressed as a percentage of the final weight.)

2.4.2. The volume of the hole in the track, out of which the soil samples to be tested were removed, was determined by the sand replacement method as described in the 'Procedure for testing soils[2]'.

Two density tests for each investigation were made and the arithmetical average of the two is given in Table 5.

2.5. Field Test Procedure

2.5.1. Since the final dry density of a soil as a result of compaction depends on the initial looseness of the soil, every attempt was made to attain the same degree of pulverisation in each soil before compacting with different plant. The initial densities were also determined.

2.5.2. To enable the results to be compared with the mentioned British study, a 9-in. thickness of loose soil, as in that study, was adopted. The reasons for selecting a thickness of 9-in., as discussed by Maclean, Bailey and Hunt,[1] were that the dry density of a compacted layer of soil decreases with the depth from the surface and that this decrease in density becomes excessive when the **compacted** thickness is more than 6 inches.

2.6. Investigations

2.6.1. A study of the effect of moisture content on the dry density of the soils when compacted with each type of compacting plant.

Each soil was laid in a number of equal lengths in which the moisture content varied from 5 to 16 per cent. These lengths were then compacted with the plant under test. The dry density was determined after 64 passes. A series of curves (Figures 3, 5, 7, 9) were then drawn to find out the optimum moisture content of each soil in the field with a particular plant.

2.6.2. A study of the effect of increasing number of passes on the dry density of soil at different moisture contents.

The four soils were compacted at different moisture contents with increasing number of passes of the three types of plant under test. The effect of the increasing number of passes on the dry density was studied. For the 7-ton smooth-wheel roller and the pneumatic-tyred roller, the depth of density holes was 6 inches. In the case of the sheepsfoot roller, a loose mulch was left at the top and the density was obtained after removing the loose material. In regard to the values of density at different depths, when the loose layer was removed, attention is invited to the study mentioned in para 2.6.4. below.

2.6.3. A study of the effect of increasing the number of passes on the compaction of soils at optimum moisture.

Each soil was given the optimum moisture content for the plant under test as determined from study (see para 2.6.1 above). It was then compacted with an increasing number of passes of each plant and the densities were determined (Figs. 11 to 14.)

2.6.4. A comparison of the dry densities in the upper and lower halves of the compacted layer of soil.

In carrying out this study, the procedure given in the mentioned British Paper No. 17 was modified. The layers of all the soils (except sand) compacted with the three types of plant were considered. In the case of the smooth-wheel roller and the pneumatic tyred roller, the whole compacted crust was taken out and the dry densities of the upper and lower halves were determined. The crust was cut horizontally into two parts of nearly equal thickness. In the case of the sheepsfoot roller, the loose layer

(about 2 to 2.5 inches) was removed and the whole compacted crust (about 5 to 5.5 inches) treated like the crusts relating to the two other types of plant.

This method differs from that chosen in the British study, where out of the determined densities of the top 4 in. and the entire 6 in. depth of the crust, the densities of the bottom 2 in. were calculated.

3. LABORATORY COMPACTION TESTS

3.1. The dry density/moisture content relationships obtained in the field were compared with those obtained from three laboratory tests, using the same soils as in the field.

3.2. The B. S. Compaction Test (B. S. 1377 : 1948, test No. 9)-(also known as the Proctor Test)[3]

About 4 lb oven-dried soil passing through $\frac{3}{4}$ in. B. S. sieve was mixed carefully with water and compacted at a low moisture content in a mould 4 in. in diameter and 1/30 cu. ft. in volume. The compaction was carried out in three equal layers with a rammer $5\frac{1}{2}$ lb in weight. The striking face of the rammer has a diameter of 2 inches, and falls freely, along the side of the mould, through a height of 12 inches on to the soil surface. The mould containing the soil is turned round gradually so that the whole soil surface is covered by the rammer blows. Each layer receives 25 blows during compaction, and the soil compacted over and above the height of the mould is trimmed off after removing the extension collar.

The dry density of the soil was determined and the experiment repeated with increasing percentages of moisture.

3.3. Modified AASHO Compaction Test[4]

The test is similar to the B. S. compaction test except that the soil is compacted in five equal layers instead of three, with 25 blows on each layer from a 10 lb rammer (instead of $5\frac{1}{2}$ lb) falling through a height of 18 inches instead of 12 inches. The rammer has the same area of striking face as the B. S. compaction rammer. The dry density/moisture content curve is obtained as in the B. S. compaction test.

3.4. Test with the Abbott Cylinder

3.4.1. This is a test well-known in India, developed and introduced by an Indian Army officer, named Abbott.

The test itself represents none of the international standardised tests; therefore, the number of blows are not limited. In this study the number of blows are always 10, 20, 30, and 40, so that the resultant compaction can be adjusted to the compaction obtained with different plant It has been considered worth while to add this test to the other two because it allows further comparisons to be made and conclusions drawn.

3.4.2. Against the two methods described above, where a constant volume of wet compacted mass is used for the determination of the density of the mass, the test with the Abbott cylinder uses a constant weight of dry soil to find out the change in the volume of a compacted mass brought about by the addition of known moistures. The original cylinder used by Abbott was slightly modified in so far as the diameter was changed from 2 in. to 2.5 in. The change in the diameter was made in order to adopt the standardised size of test blocks of some other testing apparatus. A brief description of the method, which remains unchanged in principle, is given below.

3.4.3. Two hundred grams of oven-dried soil sample passing through No. 10 U. S. Standard ASTM sieve are used. The soil is mixed with a known quantity of water and compacted with a rammer $5\frac{1}{2}$ lb in weight, in a brass cylinder, 2.5 in. in diameter, fitted on a detachable base plate. The resultant volume is read off from the scale provided on the sliding rod after compaction. The process is repeated with increasing percentages of moisture.

4. EXPERIMENTAL RESULTS

4.1. Relationship between dry density and moisture content of soils fully compacted by the plant and in the laboratory.

4.1.1. The moisture/density curves of the four soils used in the study, namely clayey, silty, and sandy soils and sand with the three types of plant, are shown in Figures 3, 5, 7 and 9. The moisture/density relation curves obtained with the three laboratory tests for all the four soils are given in Figures 4, 6, 8 and 10. The figures for maximum dry density and the optimum moisture content obtained from these curves are recorded in Table 5.

4.1.2. It was found (Fig. 9), as at the Road Research Laboratory, Harmondsworth, that the sheepsfoot roller could not effectively compact the sand. Further study on the performance of this roller on sand was not made.

4.2. Relationship between dry density of soil and number of passes at different moisture contents

4.2.1. Experiments with increasing number of passes of all the three types of plant for different moisture contents of all the four soils were carried out with the exception of the sheepsfoot roller on sand. The results were similar to those obtained in the British study and are therefore not recorded here.

4.3. Relationship between dry density of soil compacted at optimum moisture content for the plant and the number of passes of the plant.

4.3.1. The curves showing the effect of compaction at the optimum moisture content and increasing number of passes of the three types of plant are given in Figures 11—14 for the clayey soil, silty soil, sandy soil, and sand respectively. The densities were taken at the end of 4, 8, 16, 32 and 64 passes.

4.3.2. The values of densities obtained in these investigations (shown in Figures 11—14) are sometimes considerably lower than those obtained in the study of relationship between dry density and moisture content of soils, fully compacted by the plant in the field (Figures 3, 5, 7 and 9). This is due to the fact that the same moisture content could not be easily maintained during the investigations.

4.3.3. A further deviation in moisture content occurred on account of the longer time which was needed to take densities repeatedly at different passes which led to evaporation and resulted in values of lower densities. Moreover, during this time evaporation of moisture took place to a varying degree for each soil.

4.3.4. The curves in Figs. 11—14 show an almost horizontal tendency after about 6 to 16 passes of the plant. This may have been due to the deviations mentioned above. As this is an important fact it may be discussed here briefly, including the possible correction.

4.3.5. The most notable individual cause of deviation was the evaporation of the moisture content during compaction. As the soil became drier and drier it was impossible to compact it to the maximum density (as given in Figs 3, 5, 7, 9) even by more passes of the plant.

4.3.6. Before compaction started the soil was deliberately given some extra moisture so that it might have the exact nominal moisture content after the first few runs (say 8 to 12 runs) of the plant. The effect of this precaution could vary, as well as the density obtained after 64 passes, according to the speed of the plant. The loss from evaporation would, thus, be greater with the smooth-wheel roller than with the two other plant which moved faster.

4.3.7. The actually obtained curves (full lines in Figs. 11—14) had therefore to be corrected as suggested below :

The part of the curve up to about 8 to 12 passes can be taken to be correct and unaffected by evaporation. Beyond that the curves show much too flat a progress. From Figs. 3, 5, 7, and 9 the approximate real positions of the last points of the curves in Figs. 11—14 are known, namely the maximum density expected after 64 passes. With these three elements known viz., the position of the initial portion of the curve, its general character, and the last point on it—the correction could be made. The corrections are shown as dotted lines in Figs. 11-14.

4.3.8. This deviation was foreseen and could be corrected as suggested because the data for carrying out the correction were available.

4.4. Comparison of dry densities obtained in the upper and lower halves of the 6-in. compacted crust obtained after compacting 9-in. loose soil with the 7-ton smooth-wheel roller and the sheepsfoot roller.

4.4.1. Samples of the crusts of different soils compacted at optimum moisture with the smooth-wheel roller and the sheepsfoot roller were taken. These were cut horizontally into two equal parts and the densities of the two parts along with the density over the total depth are given in Table 6.

4.4.2. To permit of ready comparison of the dry densities when taken separately from the upper and lower parts of the 6-in. crust, compacted with the smooth-wheel roller and the sheepsfoot roller at various moisture contents, the results are reproduced in Figs. 15 and 16.

5. DERIVED RESULTS

5.1. Comparison of performance of plant

There is no simple basis for comparing the performance of the different plant owing to the many factors involved. It is, however, hoped that the behaviour of each type of plant in compacting soils can be derived from the following three values :

5.1.1. Number of passes required at the optimum moisture content to obtain values of relative compaction of 80, 90, and 100 per cent

The relative compaction of a soil is defined as :

$$\frac{\text{(Field Dry Density)}}{\text{Maximum dry density in laboratory compaction test}} \times 100 \text{ per cent}$$

The laboratory compaction test used in this formula was the B. S. Compaction Test, and the densities were expresssed in lb per cu. ft. and in kg. per cubic meter. The figures are given in Table 7. It may happen that the densities obtained in the field are sometimes higher than the densities indicated by the laboratory test. This resulted in a relative density of more than 100 per cent ; it can vary between 95 to 105 per cent. These variations depend on the type of laboratory test apparatus chosen.

It is therefore seen that a value of 100 per cent does not necessarily represent full compaction under field conditions.

Moreover the relative compaction of the soils in a completely loose state was about 80 per cent, as against 70 per cent for the soils used in England, probably owing to their nature as well as to the method of laying the soil on the track, which might have resulted in a higher relative compaction before the plant was brought on.

The influence of the initial moisture content on the density before compaction started in the case of sand is given below :

Initial % of moisture	Density of sand lb/cu.ft.	kg/m³
5	84.00	1346
8	87.50	1402
11	89.75	1438
13	92.75	1486
15	94.00	1506
16	95.62	1532

Table 7 shows the number of passes required to obtain certain values of relative compaction at the optimum moisture content for the plant.

5.1.2. **Efficiency of compaction obtained after different number of passes for soil compacted at the optimum moisture content of the plant**

The efficiency of compaction is defined as:

$$\frac{\text{(Compacted dry density)}-\text{(Loose dry density)}}{\text{(Maximum dry density in B. S. Compaction Test)}-\text{(Loose dry density)}} \times 100 \text{ per cent}$$

This gives the scale for the amount by which a soil is compacted ranging from zero compaction to full compaction which is approximately about 110 per cent. Like relative compaction, the efficiency of compaction is partially dependent on the type of soil (Table 8).

5.1.3. **Output of plant in terms of soil compacted per hour**

The output has been expressed as the volume of loose soil (when laid in a 9-in. loose layer) compacted to a relative compaction of 80, 90, and 100 per cent at optimum moisture content for the plant (Table 9). The speed of the plant in each case has been assumed as 2 miles per hour; and the number of passes required to obtain the various relative densities was obtained from Figures 11–14. In the case of the sheepsfoot roller, an average density of the compacted mass only has been taken into consideration for purposes of calculation.

6. DISCUSSION OF EXPERIMENTAL RESULTS

6.1. Effect of moisture content on the compaction of soil

General: As in the corresponding British study, it is observed that moisture content is the main factor affecting the compaction of soil. There is a definite relationship between moisture content and dry density, and it is further observed that compaction is easier at higher than at lower moisture contents.

6.2. Comparison of maximum dry density and optimum moisture content as obtained with the plant and in the laboratory

6.2.1. From Table 5 it may be seen that in most cases field-compaction plant units do not produce as high densities as are obtained by laboratory compaction tests. Laboratory tests should, therefore, be taken only as a guide for compaction in the field. It should be noticed that each plant-unit has its own conditions for attaining optimum results.

6.2.2. An approximate relationship is, however, observed in the following cases.

6.2.3. The optimum moisture content, for example, corresponding to the laboratory test with the B. S. compaction test and Abbott's apparatus at 40 blows approximately equals the optimum moisture content for the sheepsfoot roller. This is true with the sandy soil and the clayey soil, vide Figs. 3–4 and 7–8, where the results of these tests correspond to those given by the sheepsfoot roller. However, there is only a rough agreement of dry densities.

Similar relationship may be observed in the case of other plant, though completely satisfactory agreement is always lacking, e.g., with silty soil when the sheepsfoot roller corresponds in density and optimum moisture content to the results of Abbott's test with 30 blows. This shows that by varying the number of blows laboratory tests can be modified so as to give results comparable to those yielded by any plant.

6.3. Effect of moisture content on dry density of soil

6.3.1. The moisture-density curves (Figs. 3, 5, 7, 9) follow generally the parabolic shape. There are exceptions, however, due to slight variations in the moisture added.

6.3.2. The smooth-wheel roller gives the maximum compaction with all the soils. The sheepsfoot roller gives almost equal results, as far as the compacted crust alone is considered.

6.3.3. The sheepsfoot roller is a poor compacting unit on sand (Fig. 9). Because of the lack of cohesion between the sand particles, the feet of the roller have a tendency to throw up the material once compacted.

6.4. Effect of moisture content on the number of passes required to compact soil.

6.4.1. During the initial few passes of the plant the density increased rapidly, then at a decreasing rate, till finally further passes of the plant brought about practically no change in density.

It may be generally observed that there is a particular moisture content, sometimes above and sometimes below the optimum moisture content, at which the plant under test gives the requisite compaction with the minimum number of passes. It may, therefore, sometimes be economical to work on the wet or dry side of the optimum moisture content as the case may be.

6.4.2. With the smooth-wheel and the pneumatic-tyred rollers almost maximum compaction was obtained with about 16 passes of the roller. The results, however, were inconclusive with the smooth-wheel roller on the clayey soil.

6.4.3. The sheepsfoot roller gave the maximum density with about 8 to 16 passes on the clayey soil and the sandy soil. However, over the silty soil about 32 passes of the roller were required.

6.5. Effect of moisture content on the effective depth of compaction

6.5.1. The curves in Fig. 15 represent the densities of the top and bottom halves of the compacted crust in relation to moisture content. For this the compacted crust was cut horizontally into halves, whose densities were determined. With the smooth-wheel roller the densities of the two layers tend to equality beyond a moisture content of 12 per cent in the case of silty and clayey soils.

6.5.2. Similar tests were carried out with the sheepsfoot roller (Fig. 16). The results obtained showed that the effect of compaction was reversed.

6.6. Relationship between dry density and number of passes of the plant for soil compacted at optimum moisture content.

6.6.1. The curves relating to the number of passes and dry density at optimum moisture content are given in the Figs. 11-14. For the sheepsfoot roller, the densities shown are the averages of the compacted crust and the loose mulch. Therefore they appear very low here.

6.6.2. With the smooth-wheel roller, full compaction was obtained with all the soils (except the sand) with about eight passes of the roller and any further passes did not affect the density materially. In the case of the sand, about 32 passes were required.

6.6.3. The pneumatic-tyred roller gave full compaction after about 16 passes. Beyond this, even when there was a tendency for an increase in density, the additional density obtained could not be economically justified.

6.6.4. The sheepsfoot roller gave full compaction with 16 passes. Any further passes did not affect the density materially.

6.7. Effect of the thickness of the compacted layer on compaction of soils.

6.7.1. Tests were carried out with the different plant by compacting loose layers of 9 in. at optimum moisture content

6.7.2. The compaction of the sand was not possible with the sheepsfoot roller. Similarly, sandy soil creates difficulty in compaction, but to a lesser extent. With all the soils a loose mulch of 3 in. was left behind.

6.7.3. The effect of factors like soil type, moisture content, and type of plant are discussed below.

(a) **Soil type** :—There was a reduction in density with depth in the case of all the plant, except the sheepsfoot roller. The difference in density was greater with the clayey or silty soil than with the sandy soil.

(b) **Moisture content** :—Fig. 15 shows that the change in density with depth was greater when compaction was carried out at moisture contents on the dry side of the optimum. On the wet side, there was a tendency towards uniform density throughout the depth.

(c) **Type of Roller** :—From Figs. 15 and 16 it may be seen that with the smooth-wheel roller there was a decrease in density with depth. But with the sheepsfoot roller, the density increased with depth on soils on which the plant could effectively work. This was the case with the clayey and the silty soils, but not with the sandy soil.

The latter result is in keeping with the generally expressed opinion that the sheepsfoot rollers compact from bottom upwards. Though these results are contrary to the findings in the corresponding British Paper, they indicate that the generally expressed opinion may be correct.

6.7.4. With the smooth-wheel and sheepsfoot rollers the differences in density were the largest with the silty soil and the clayey soil. In the case of the sandy soil the degree of compaction was more uniform.

6.8. Performance of Plant

6.8.1. **Smooth-wheel Roller** :—Tables 7, 8, and 9 indicate that the smooth-wheel roller was better suited than the other plant tested for compacting the silty soil and the sandy soil because of the small number of passes required to attain satisfactory compaction. Its higher efficiency for compacting can be seen from Figs. 11—14. When the moisture content is a little more than optimum, the smooth-wheel roller will compact more easily.

6.8.2. **Pneumatic-tyred Roller** :—Table 9 shows that the pneumatic-tyred roller is best suited for sandy soils. Though the compaction obtained is not as high as that with the smooth-wheel roller, its ease of negotiating such soils is largely in favour of its use.

6.8.3. **Sheepsfoot Roller** :—The sheepsfoot roller is not effective with sandy soils but is a useful compacting unit on clayey soils at low moisture contents (Fig. 16). It is, therefore, specially useful for compacting for example puddle cores and embankments laid with suitable soils in relatively thin layers. The top layer has then to be finished with a smooth-wheel roller. The main disadvantage with a sheepsfoot roller is the large number of passes necessary for complete compaction. This is to some extent offset by the speed at which the roller can be operated.

6.8.4. A comparative study of the different plant is given in Table 9.

6.8.5. Table 10 gives recommendations on the plant best suited for particular soils to give sufficient compaction with the minimum passes. Owing to the fact that the sheepsfoot roller compacts more densely to a certain depth than any other plant (compare Fig. 16), especially in the clayey soil, a combination as shown in Table 10 may prove advantageous.

7. SUMMARY

7.1. A study of the compaction of three alluvial soils and a sand of kinds generally available in north-west India has been carried out with three kinds of compaction plant—a 7-ton smooth-wheel roller, a pneumatic-tyred roller, and a taperfoot sheepsfoot roller. The investigation was carried out on a straight open test track. In all tests a 9 in. layer of loose soil was used.

7.2. The object of the investigation was to compare the performance of three types of compaction plant on the different soils and to determine the effect of moisture content and the number of passes of the plant on the resultant dry density of the soils. The optimum moisture contents and the corresponding dry densities in the field tests were compared with the results of laboratory tests, viz., the B. S. compaction test, the modified AASHO test and the test with the Abbott cylinder.

7.3. The **7-ton smooth-wheel roller** was the most effective in compacting the different soils, even with relatively fewer

passes. The resultant density varied in a descending order of magnitude from the top to the bottom of the crust. Compaction at a moisture content a little higher than the optimum moisture content of the soil for the plant leads to economy, but results in lower densities, and consequently smaller bearing capacities.

7.4. The **pneumatic-tyred roller** was found to be best suited for the compaction of sandy soils at all moisture contents because of the ease with which it negotiated such soils. However, the resultant compaction is less than that obtained with the other plant. It is useful when high densities are not required.

7.5. The **sheepsfoot roller** was found to be ineffective in compacting sand. In the case of the other soils, the results were satisfactory and the density of the crust increased with the clay content. The density in the lower layers of the compacted crust was higher than in the upper layers. The sheepsfoot roller leaves a loose mulch of about 2 in. with all soils, which calls for the use of a smooth-wheel roller for the final compaction.

7.6. With all the plant, there was a rapid rise in density during the first few passes and further passes did not affect the density materially. With the smooth-wheel roller, almost maximum density was obtained with about 16 passes as compared to 32 - 64 passes and 16 - 32 passes with the sheepsfoot and pneumatic-tyred rollers respectively.

7.7. Working on the wet side of the optimum moisture content is to be recommended where high density can be sacrificed for economy. This brings about compaction with comparatively fewer passes and results in a more uniform density throughout the compacted mass.

7.8. No single laboratory test can reproduce conditions obtained in field compaction tests, but it is possible to develop one or more methods in the laboratory to obtain results comparable with those yielded by each plant. In general, the laboratory methods are, therefore, suitable only as a guidance for field work.

7.9. With the exception of the effect of the sheepsfoot roller, these results comply with and confirm, in general, the results obtained at the Road Research Laboratory, Harmondsworth. They show that the conclusions drawn at Harmondsworth are also applicable to some of the typical alluvial soils met with in the northern parts of India.

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3. British Standards Institution. British Standards Specification No. 1377 : 1948. Methods of test for soil classification and compaction : Test 10.

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Table 1.

Physical and Chemical Characteristics of Soils Selected for Study

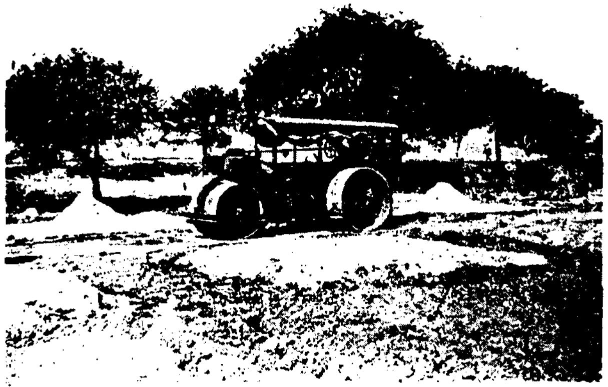
Soil No.	Nature of soil	Physical			Chemical						
							Soluble Salts		Exchangeable		
		Liquid limit %	Plasticity Index	Sand Content (Above 0.05 mm) %	Sulphate %	Total solids %	Sulphates %	Chlorides %	Ca (m. e./100 gm soil)	Na & K (m. e./100 gm soil)	$CaCO_3$ %
I	Clayey soil	35.12	15.75	32.0	Nil	0.074	0.0173	0.017	12.98	0.5	8.0
II	Silty soil	30.00	10.80	52.0	Nil	0.075	0.0156	0.016	7.40	0.9	1.6
III	Sandy soil	25.56	6.16	62.0	Nil	0.077	0.0132	0.012	3.90	0.8	0.4
IV	Sand	—	Non-plastic	100.0	Nil	0.025	0.0115	0.007	0.60	0.3	0.2

Table 2

Characteristics of the 7-ton Smooth-wheel Roller

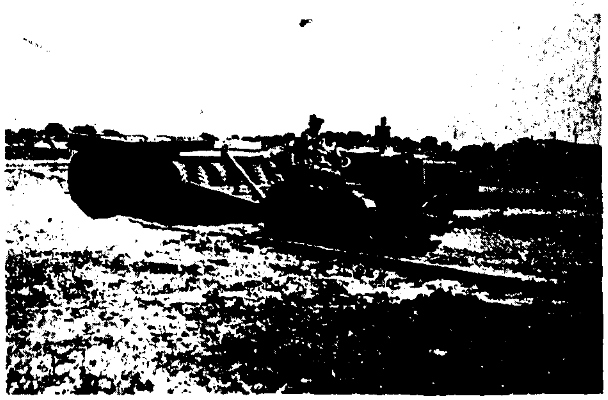
	Metric Units	English Units
Gross Weight	7.112 t	tons
Weight on front roll	1.778 t	1.75 tons
Weight on rear rolls	5.334 t	5.25 tons
Diameter of front roll	100.7 cm	44 in.
Width of front roll	(2 ×) 50.8 cm	(2 ×) 20 in.
Diameter of rear rolls	140.9 cm	55½ in.
Width of rear rolls	50.8 cm	20 in.
Rolling width	182.8 cm	6 ft
Wheel base	289.6 cm	9 ft 6 in.
Load: Front roll	35.8 kg/cm. width	98 lb/in. width
Rear rolls	105.0 kg/cm. width	294 lb in. width
$\frac{\text{Load on roll}}{\text{Width of roll} \times \text{Dia. of roll}}$		
Front roll	0.355 kg/cm²	2.2 lb/sq. in.
Rear Rolls	0.743 kg/cm²	5.3 lb/sq. in.

Photo 1.



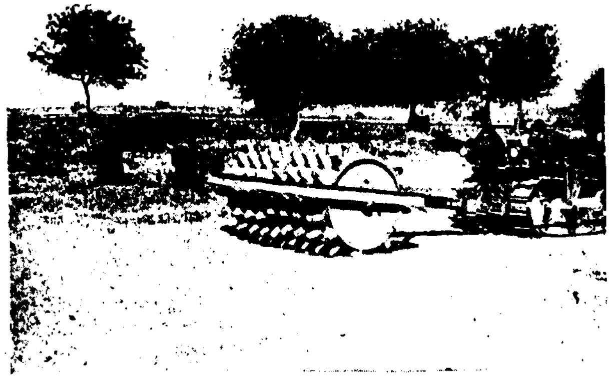
Smooth-wheel roller, 7-ton.

Photo 2.



Pneumatic-tyred roller.

Photo 3.



Sheepsfoot roller.

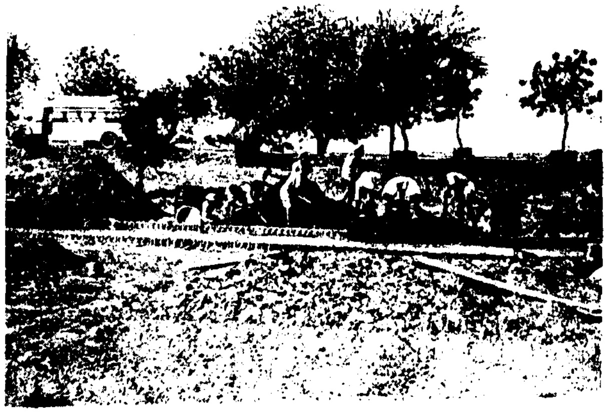
Photo 4

Ballasting the pneumatic - tyred roller with bricks.

Photo 5.

General view of the layout.

Photo 6.



Preparation of the soil beside the track.

Photo 7.

Laying the soil on the prepared subgrade.

Photo 8.

Laying the soil on the prepared subgrade.

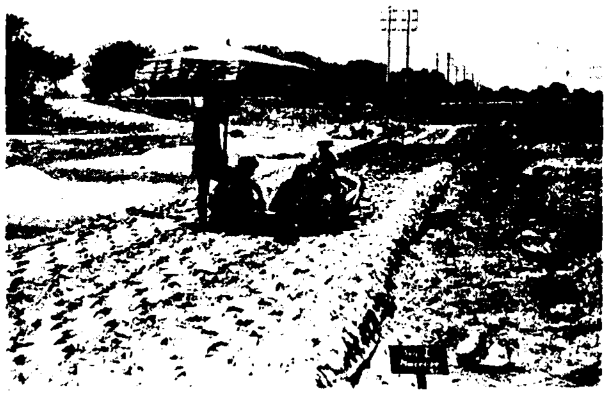
Photo 9.

Taking samples for determining the dry bulk density.

Photo 10.

Taking samples for determining the dry bulk density.

Photo 11.



View after 8 passes of the sheepsfoot roller.

Photo 12.

View after 16 passes of the pneumatic-tyred roller.

Table 3

Characteristics of the Pneumatic-tyred Roller

	Metric Units	English Units
Tare weight	3556 kg	3½ tons (7840 lb)
Laden weight	8428 kg	18,565 lb
Number of wheels	13 (6 front, 7 rear)	13 (6 front, 7 rear)
Size of tyres	26.7 cm × 50.8 cm	10.5 in. × 20 in.
Spacing between wheels	12.7 - 20.3 cm	5 in. to 8 in.
Rolling width : Front	213 cm	7 ft
Rear	258 cm	8 ft 6 in.
Tyre inflation pressure	1.756 kg/cm²	25 lb per sq. in.
Load per tyre	646.5 kg.	1424 lb (average)
Wheel mounting	Pairs of wheels fixed to oscillating axles	

Table 4

Characteristics of the Sheepsfoot Roller

	Metric Units	English Units
Weight (empty)	3600 kg	7940 lb
Weight (water ballasted)	4600 kg	10,100 lb
Number of drums	2	2
Frame	Floating	Floating
No. of feet on drum	64	64
No. of feet in row per drum	4	4
Distance between centre	30.5 cm	12 in.
Size of feet	5.7 × 5.7 cm	2¼ in. × 2¼ in.
Foot pressure (empty)	13.57 kg/cm	193 lb per sq. in.
Foot pressure (water ballasted)	17.5 kg/cm	249.4 lb per sq. in.
Diameter of drum	106.7 cm	42 in.
Length of feet	17.8 cm	7 in.
Overall length	391.4 cm	12 ft 10 in.
Overall width	250 cm	8 ft 3 in.
Width of drums	122 cm	48 in.
Space between drums	7.6 cm	3 in.

Table 5

Comparison of maximum dry densities and optimum moisture contents obtained for the four soils in laboratory tests and with various types of compaction equipment

Soil type.	Clayey Soil			Silty Soil			Sandy Soil			Sand		
	Maximum dry density.		Optimum moisture content.%	Maximum dry density.		Optimum moisture content.%	Maximum dry density.		Optimum moisture content.%	Maximum dry density.		Optimum moisture content.%
	lb/ft^3	Kg/m^3		lb/ft^3	Kg/m^3		lb/ft^3	Kg/m^3		lb/ft^3	Kg/m^3	
B.S. compaction test.	124	1986	10	121.0	1936	11.4	120.5	1928	11	101	1616	15.5
Modified A.A.S.H.O. compaction test	128.5	2056	9	127.0	2032	9.3	130.5	2088	9	106	1696	12.5
Test with the Abbott cylinder (40 blows)	125	2002	12	125.5	2008	11.5	119.5	1912	12	—	—	—
7-ton smooth-wheel roller	121	1936	11	122.5	1962	9	120.5	1928	8	106	1696	12
Pneumatic-tyred roller	110.5	1768	12	116.5	1864	8.3	112.5	1800	11	100.5	1608	11
Sheepsfoot roller crust [a]	122	1954	11	123	1968	12	119	1904	11	—	—	—
Sheepsfoot roller average [b]	103	1645	11	inconclusive		11	106	1696	11	—	—	—

(a) These figures comply with those of the upper part of the compacted crust (vide Fig. 6).
(b) Average of loose mulch and compacted crust.

Table 8.

Efficiency of compaction (per cent) of plant after various numbers of passes on the four soils compacted at the optimum moisture content for the plant

Soil Type :	Clayey Soil				Silty Soil				Sandy Soil				Sand			
	Efficiency of compaction (per cent) after the stated number of passes :															
Passes :	4	8	16	32	4	8	16	32	4	8	16	32	4	8	16	32
Smooth-wheel roller 7-ton	70	76	84	84	63	80	93	100	55	72	83	92	45	81	111	141
Pneumatic-tyred roller	33	48	57	57	25	42	61	77	30	43	60	70	50	68	87	97
Sheepsfoot roller	17	31	49	54	9	16	29	34	14	26	43	52	—	—	—	—

Table 9

Output per hour of the plant to obtain 80, 90, 100 per cent relative compaction at the optimum moisture content

Rolling speed	2 m.p.h.	=	3.2 km/h.
Thickness of loose layer	9 in.	=	23 cm.

Soil Type	Clayey Soil			Silty Soil			Sandy Soil			Sand		
	Output in cu. yds. per hour of loose material in 9-in. loose layers of relative compaction of											
	80%	90%	100%	80%	90%	100%	80%	90%	100%	80%	90%	100%
Smooth-wheel roller 7 ton	1760	440	(a)	880	440	63	1760	352	29	-	1760	126
Pneumatic-tyred roller	1247	(a)	(a)	1247	208	(a)	831	113	(a)	-	623	-
Sheepsfoot roller	345	(a)	(a)	345	(a)	(a)	302	318	(a)	-	-	-

(a) Compaction obtained in the field was lower than that obtained in the laboratory by B. S. compaction test.

Table 10

Recommended plant for particular soils and number of passes for sufficient compaction

Plant or combination of plant	Clayey soil	Silty soil	Sandy soil	Sand
Smooth-wheel roller, 7-ton	—	8—16	8—16	8—16
Pneumatic-tyred roller	—	—	8—16*	8—16*
Sheepsfoot roller	—	—	—	—
Sheepsfoot roller	8—16	—	—	—
Smooth-wheel roller	4— 8			

* If material is fairly wet higher moisture content than optimum.

Note: 1. In the case of the sheepsfoot roller, another 4—8 passes of the smooth-wheel roller will be required to finish off the mulch.

2. In selecting the plant the speed of operation of the different plant has to be taken into consideration.

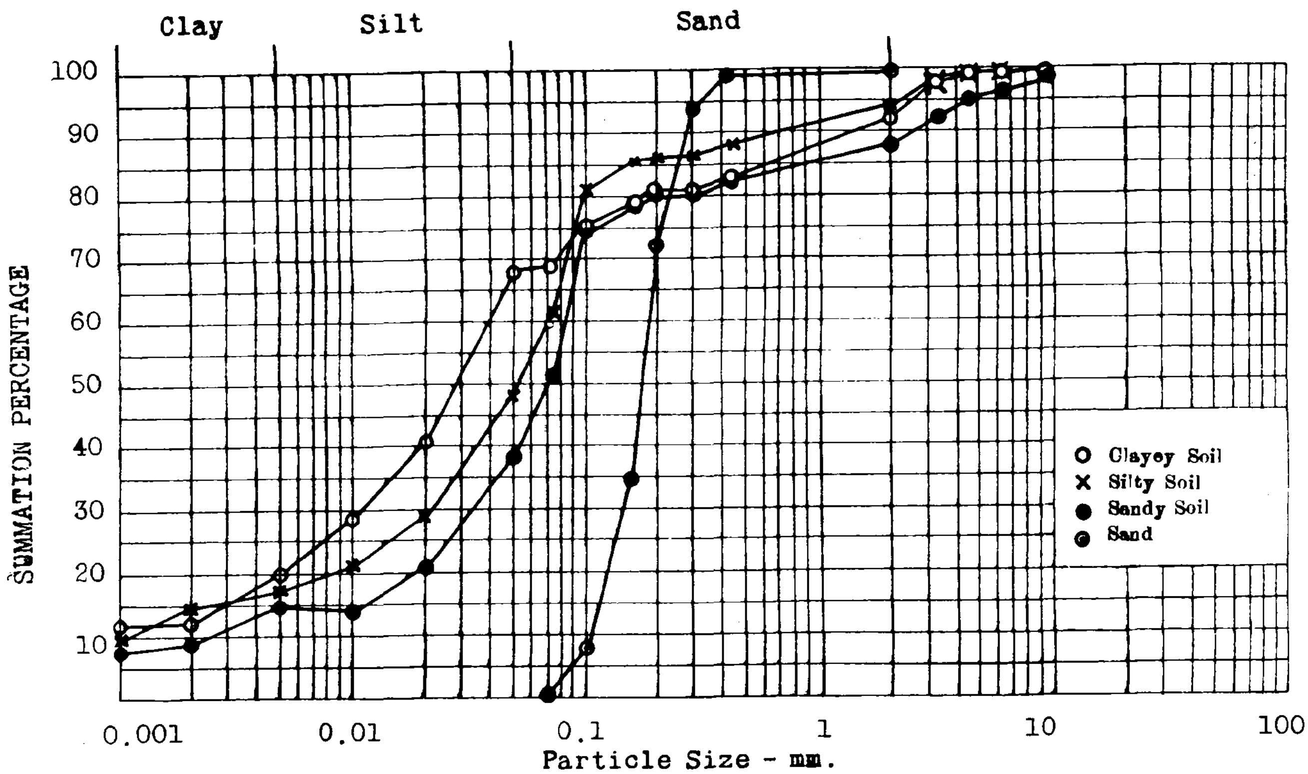


FIG. 1 - PARTICLE SIZE DISTRIBUTIONS OF SOILS USED IN THE INVESTIGATION.
Fig. 2 - On page 299.

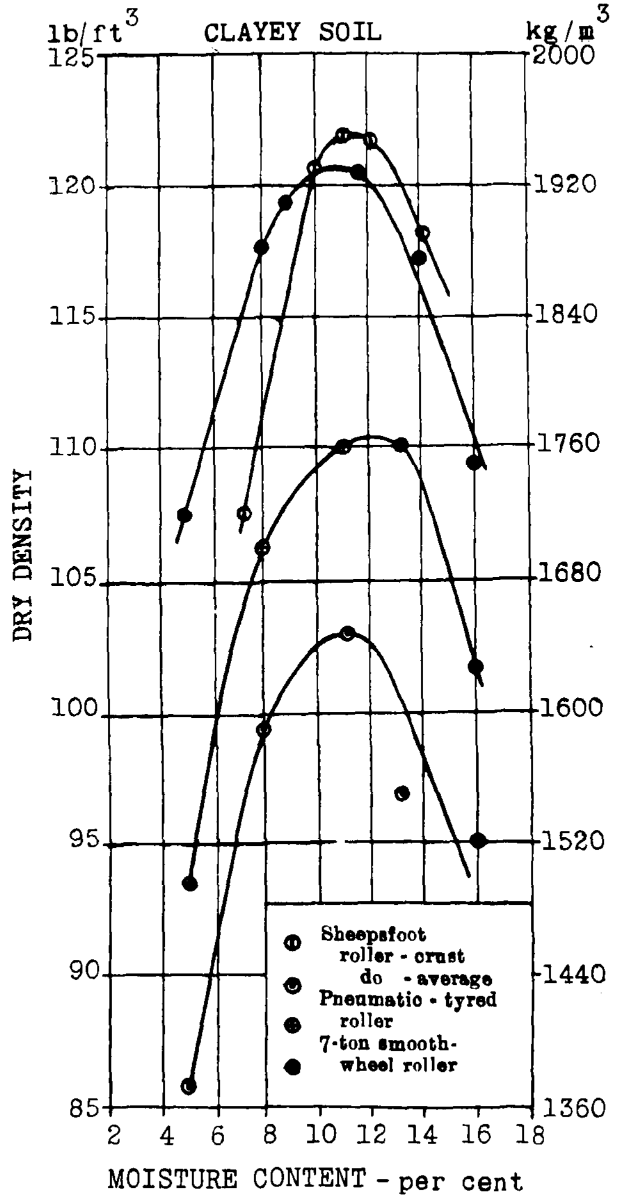


FIG. 3 - EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE CLAYEY SOIL FULLY COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF DIFFERENT TYPES OF PLANT.

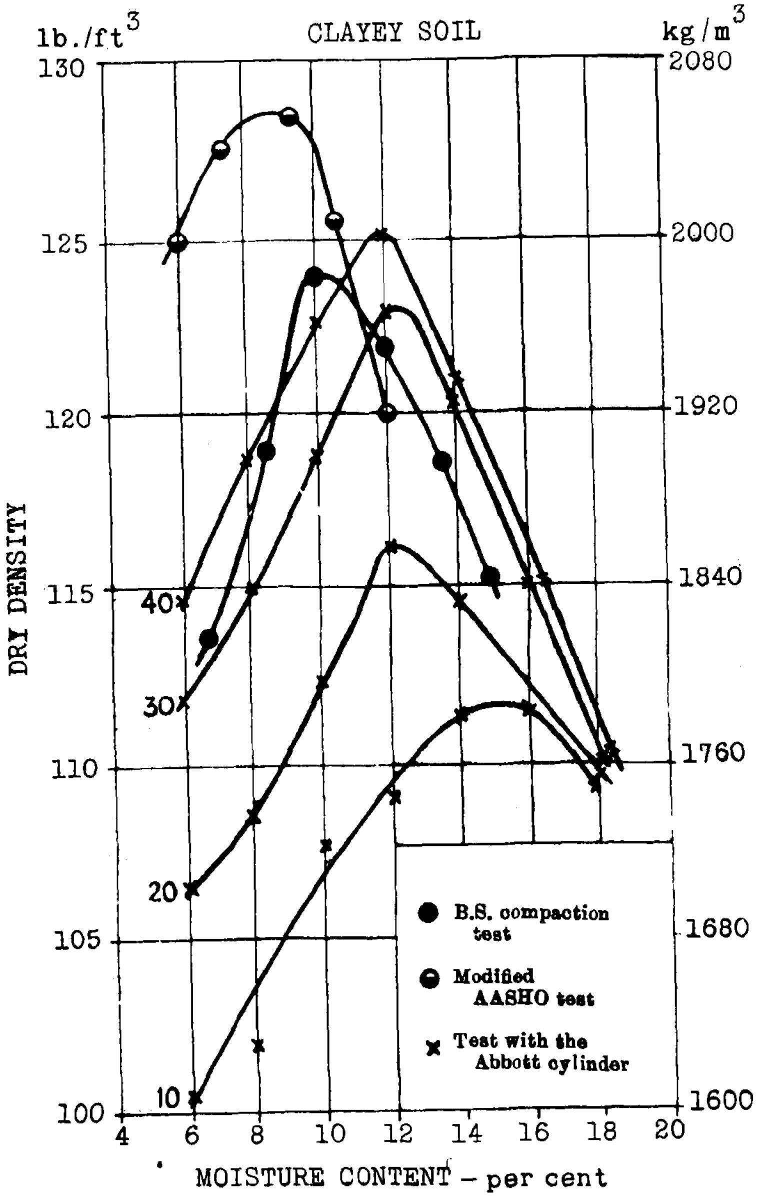


FIG. 4 - EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE CLAYEY SOIL COMPACTED ACCORDING TO THREE DIFFERENT LABORATORY TEST PROCEDURES.

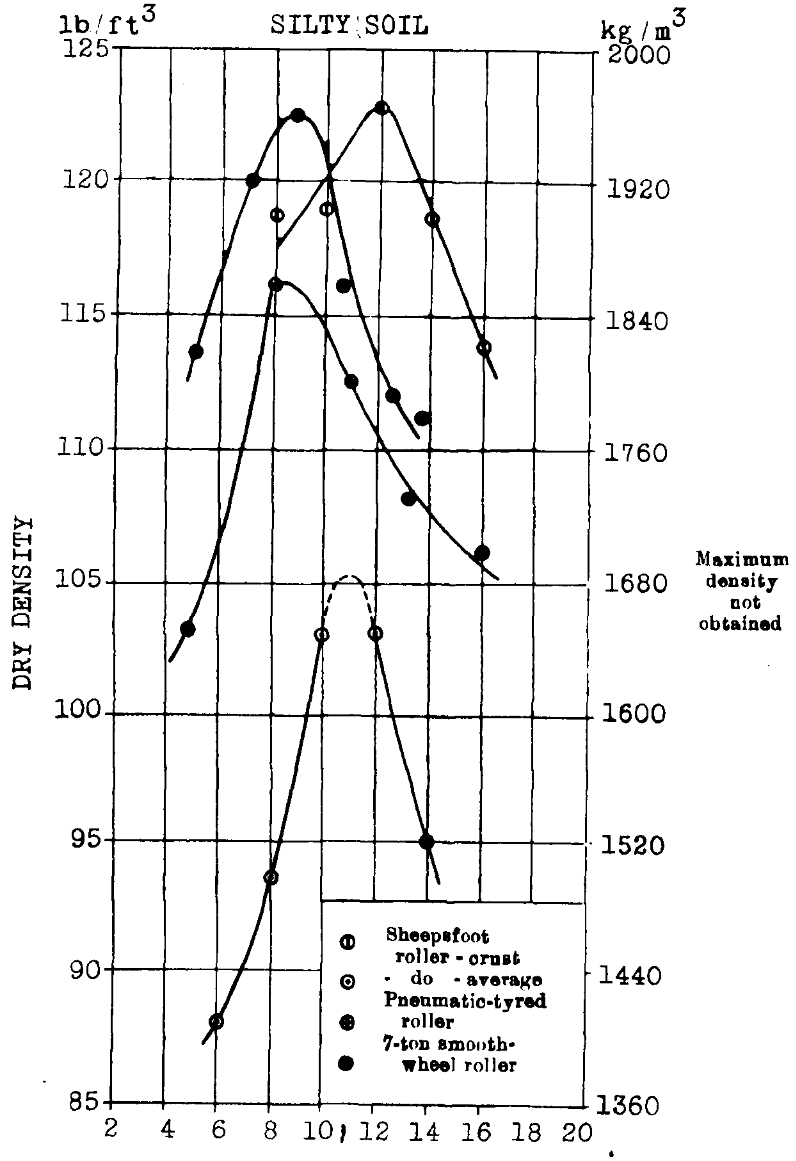


FIG. 5 - EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE SILTY SOIL FULLY COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF DIFFERENT TYPES OF PLANT.

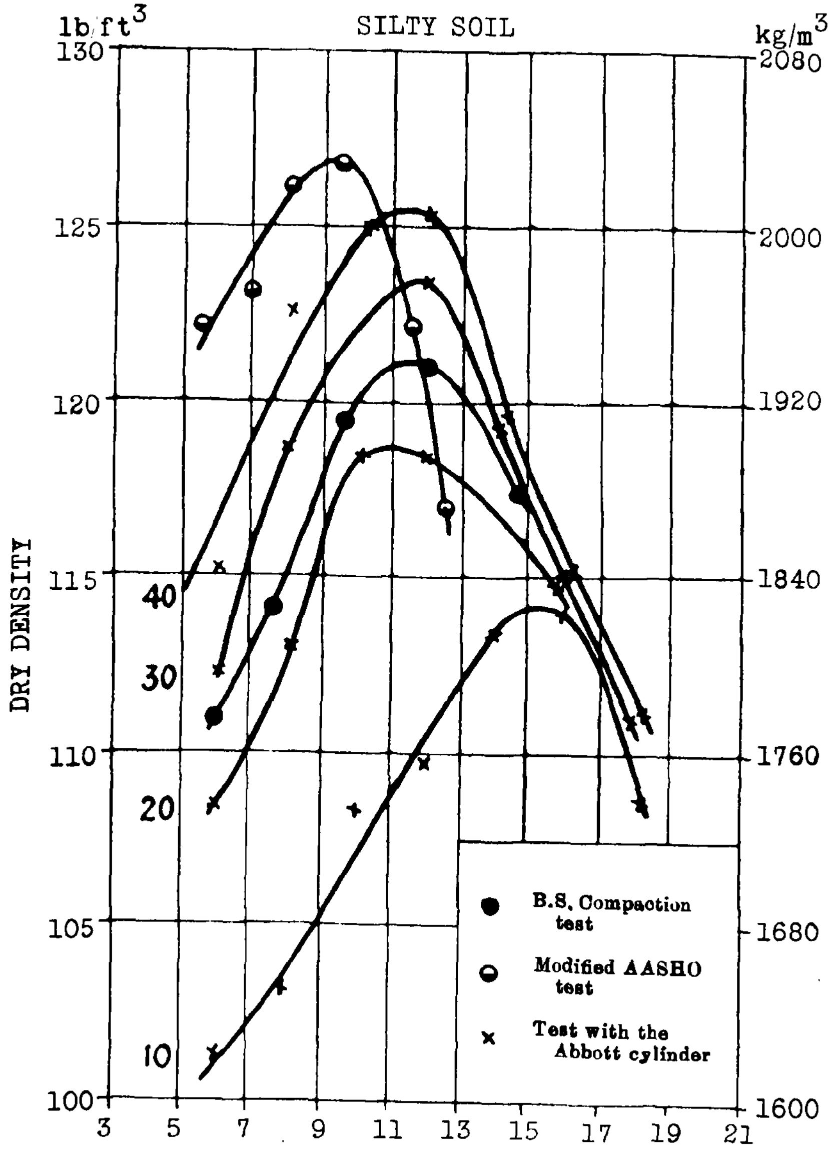


FIG. 6 - EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE SILTY SOIL COMPACTED ACCORDING TO THREE DIFFERENT LABORATORY TEST PROCEDURES.

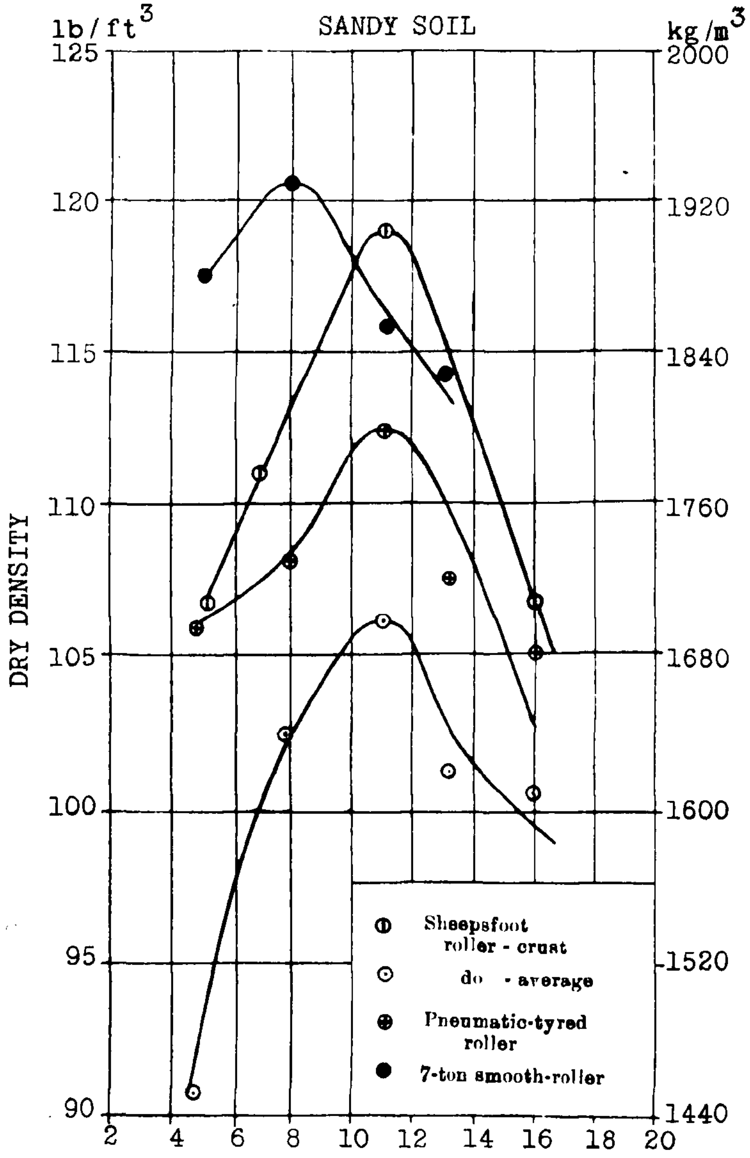


FIG. 7 - EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE SANDY SOIL FULLY COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF DIFFERENT TYPES OF PLANT.

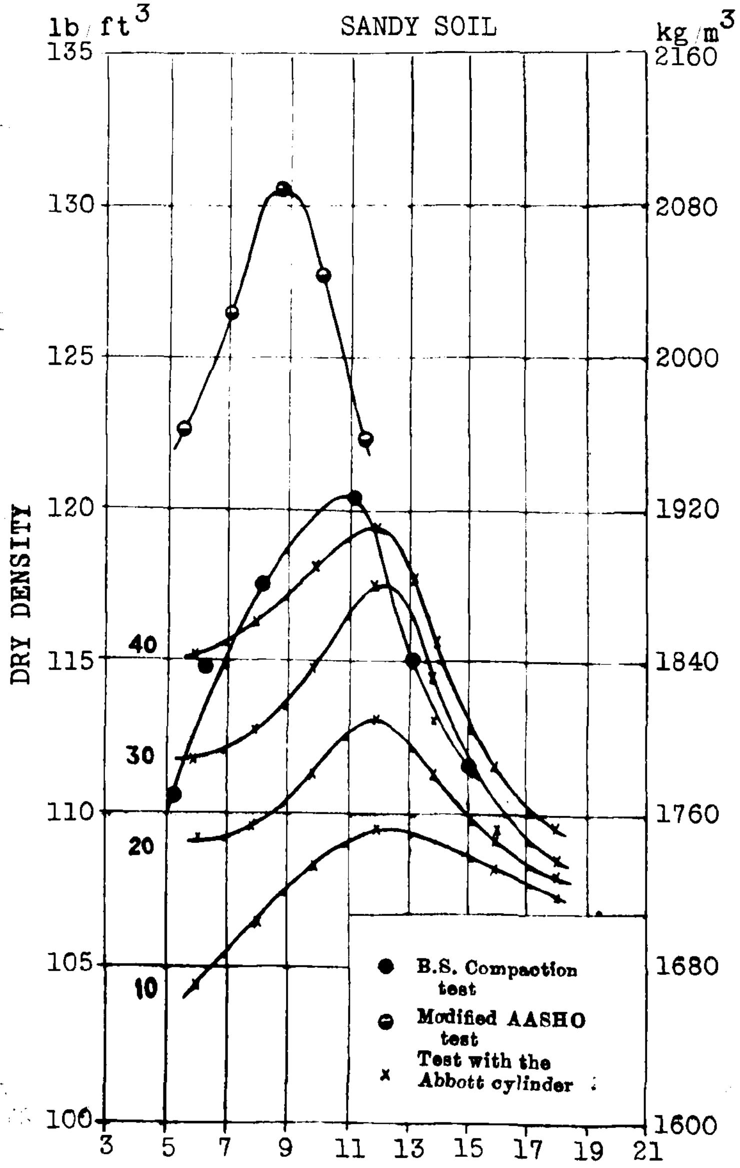


FIG. 8 - EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE SANDY SOIL COMPACTED ACCORDING TO THREE DIFFERENT LABORATORY TEST PROCEDURES.

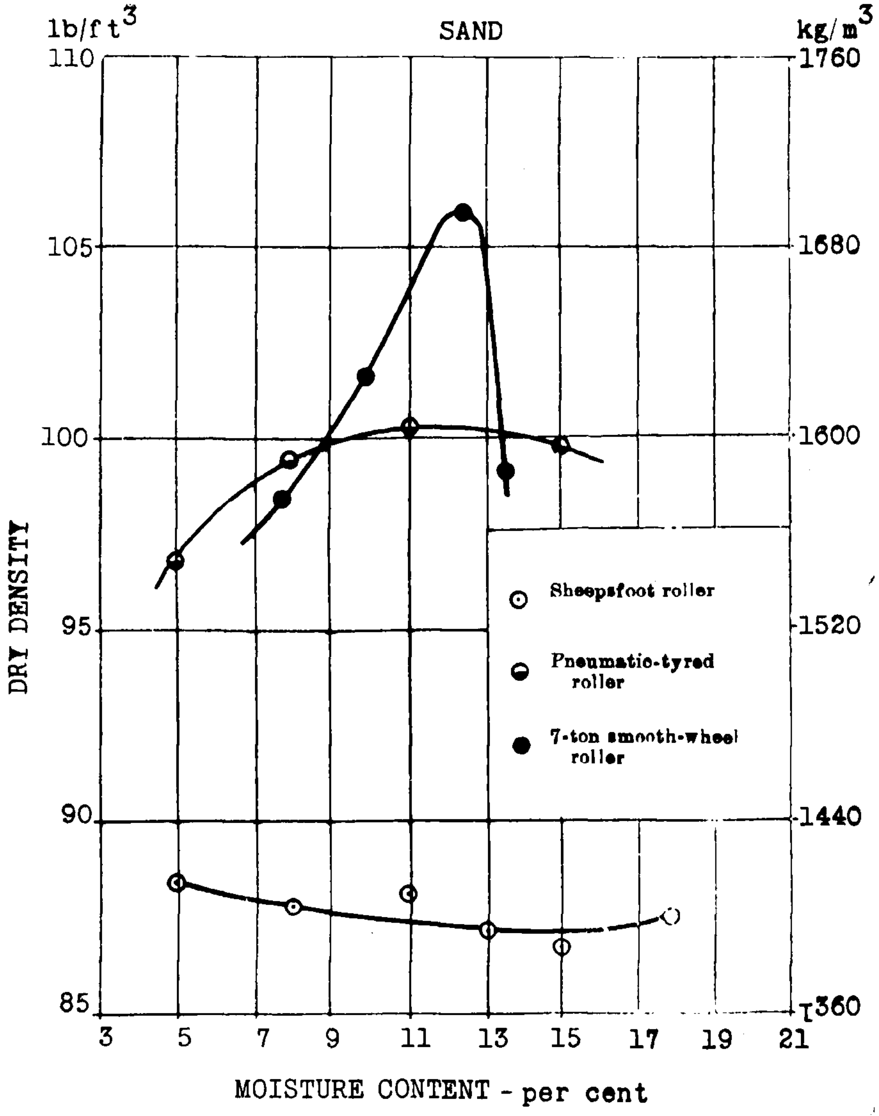


Fig. 9-EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE SAND FULLY COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF DIFFERENT TYPES OF PLANT.

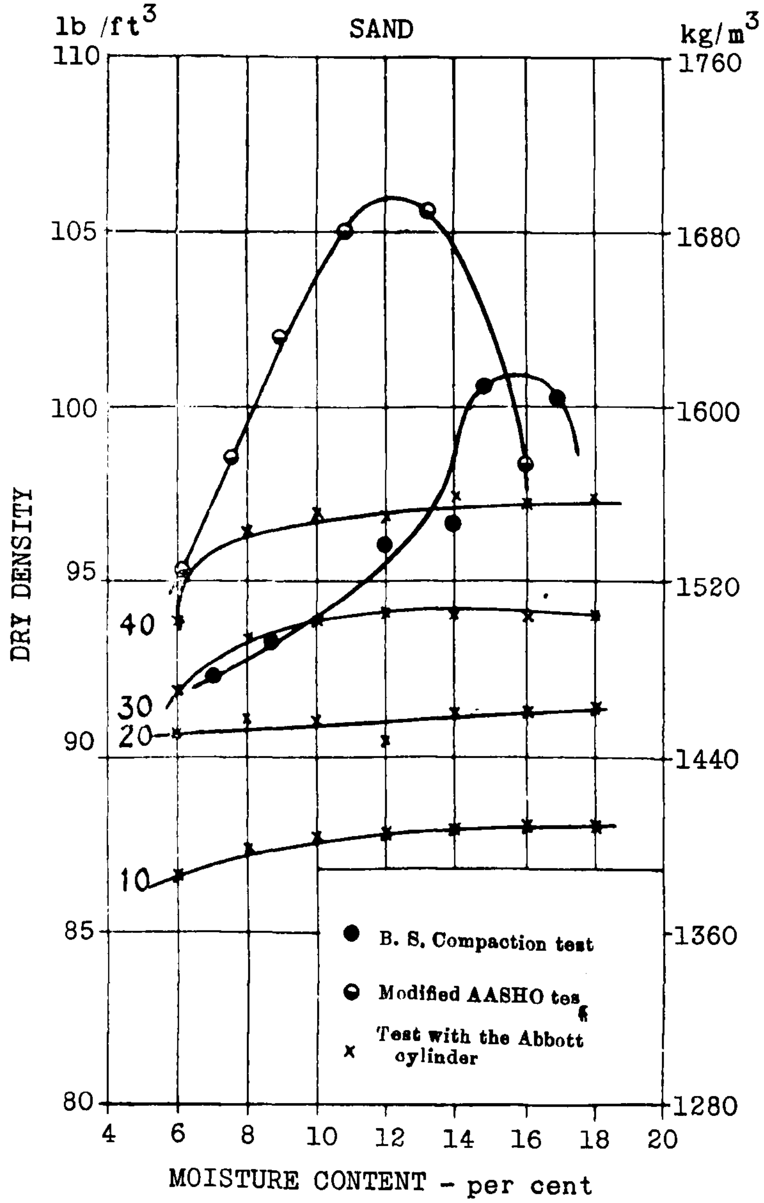


FIG. 10 - EFFECT OF MOISTURE CONTENT ON THE DRY DENSITY OF THE SAND COMPACTED ACCORDING TO THREE DIFFERENT LABORATORY TEST PROCEDURES.

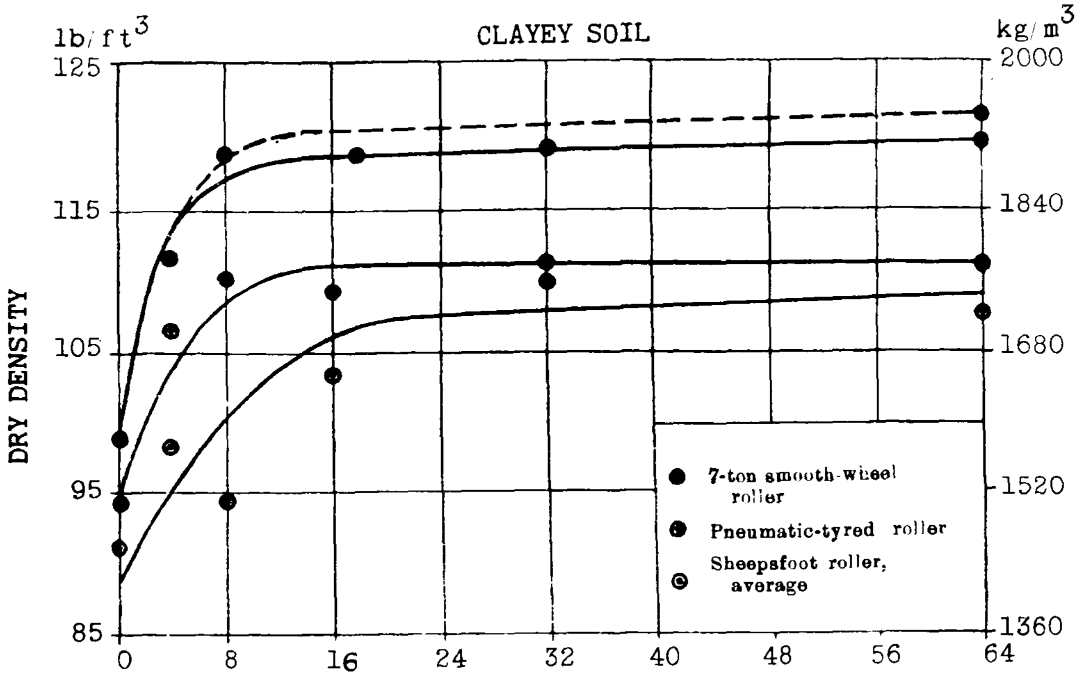


FIG. 11 - EFFECT OF NUMBER OF PASSES OF THE PLANT ON THE DRY DENSITY OF THE CLAYEY SOIL COMPACTED AT THE OPTIMUM MOISTURE CONTENT FOR THE PLANT IN 9-IN. LOOSE LAYERS.

Note: The top curve is for the smooth-wheel roller and the bottom one for the sheepsfoot roller.

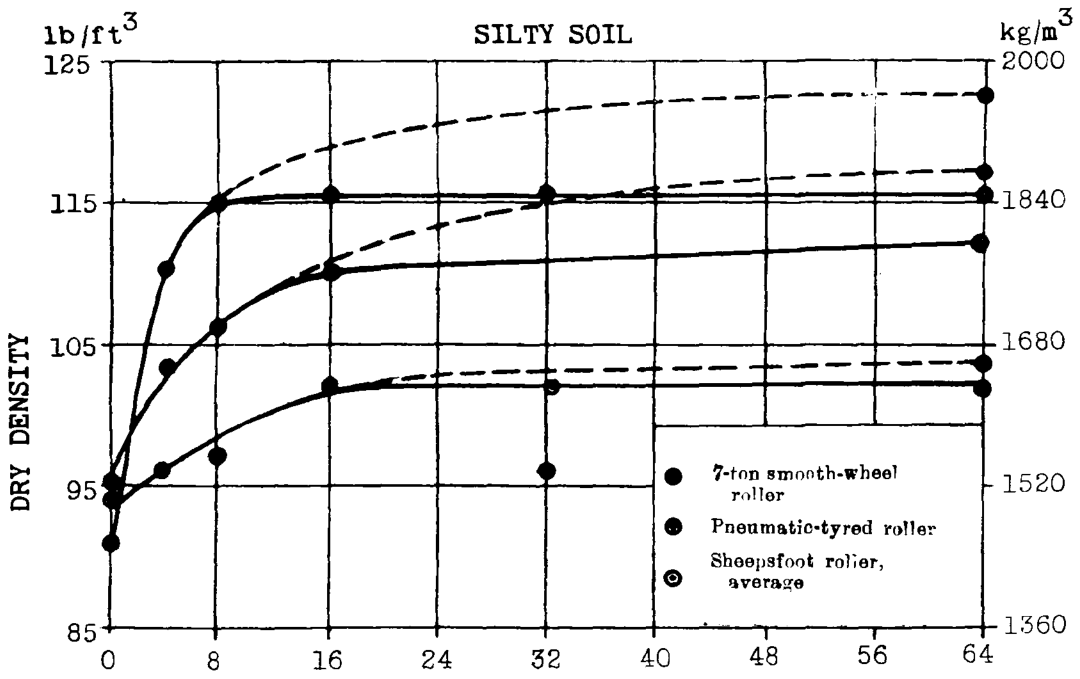


Fig. 12 - EFFECT OF NUMBER OF PASSES OF THE PLANT ON THE DRY DENSITY OF THE SILTY SOIL COMPACTED AT THE OPTIMUM MOISTURE CONTENT FOR THE PLANT IN 9-IN. LOOSE LAYERS.

Note: The top curve is for the smooth-wheel roller and the bottom one for the sheepsfoot roller.

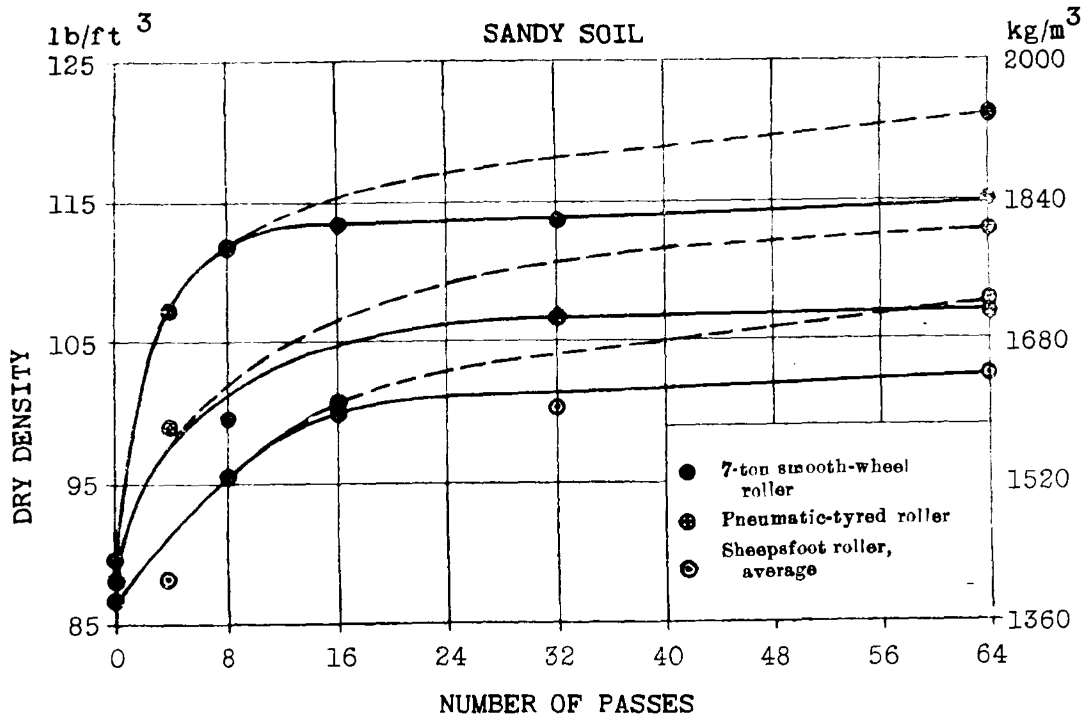


FIG. 13 - EFFECT OF NUMBER OF PASSES OF THE PLANT ON THE DRY DENSITY OF THE SANDY SOIL COMPACTED AT THE OPTIMUM MOISTURE CONTENT FOR THE PLANT IN 9-IN. LOOSE LAYERS.

Note: The top curve is for the smooth-wheel roller and the bottom one for the sheepsfoot roller.

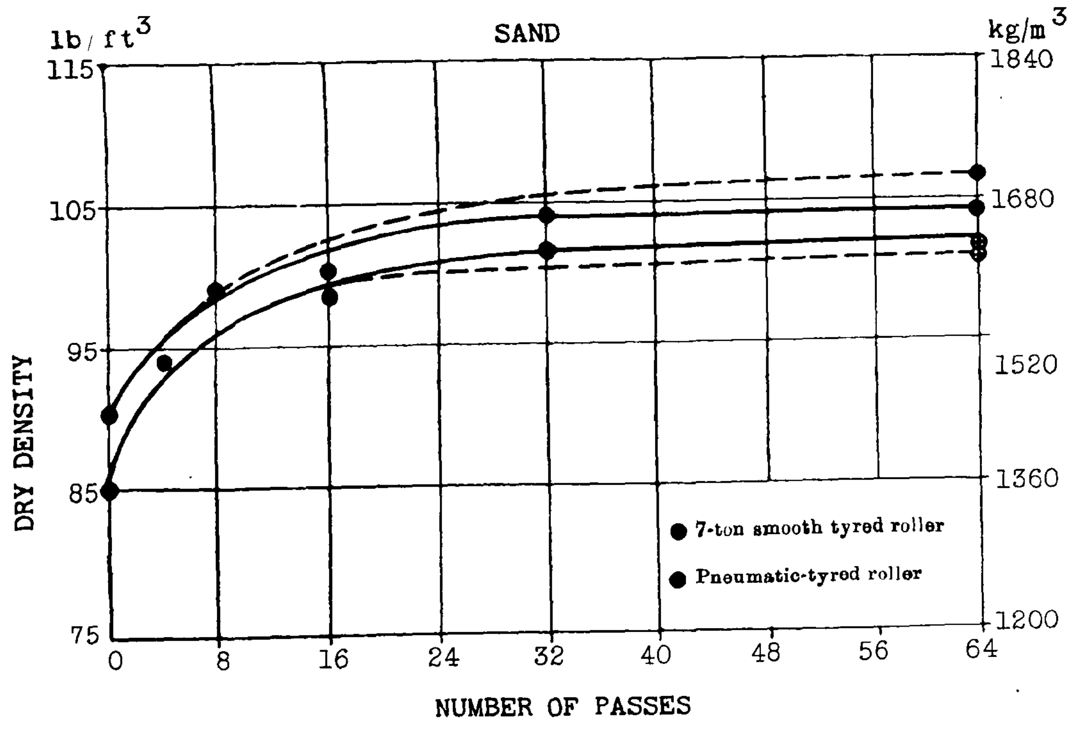


Fig. 14 - EFFECT OF NUMBER OF PASSES OF THE PLANT ON THE DRY DENSITY OF THE SAND COMPACTED AT THE OPTIMUM MOISTURE CONTENT FOR THE PLANT IN 9-IN. LOOSE LAYERS.

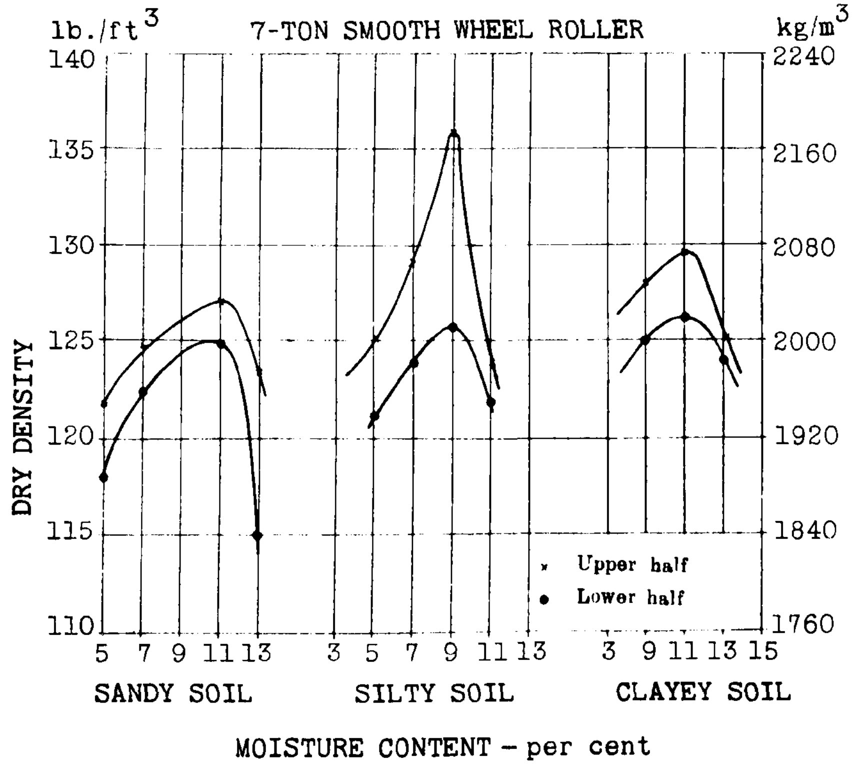


Fig. 15 - COMPARISON OF THE RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR THE THREE SOILS, WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF THE 7-TON SMOOTH-WHEEL ROLLER USING THE TOP AND BOTTOM HALVES.

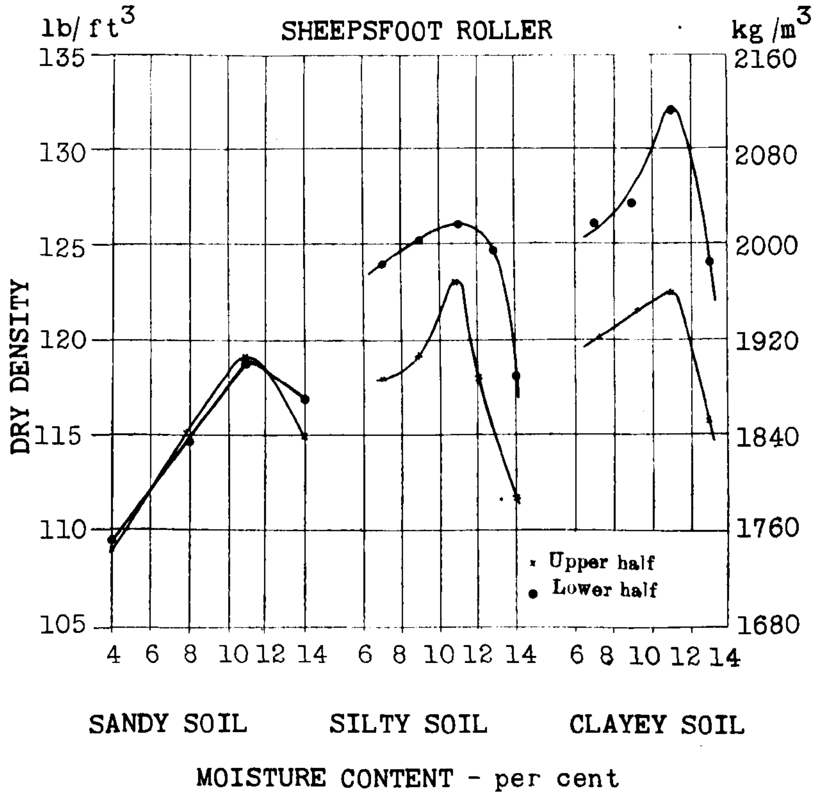


FIG. 16 - COMPARISON OF THE RELATIONSHIP BETWEEN DRY DENSITY AND MOISTURE CONTENT FOR THE THREE SOILS, WHEN COMPACTED IN 9-IN. LOOSE LAYERS BY 64 PASSES OF THE SHEEPSFOOT ROLLER USING TOP AND BOTTOM HALVES.

(d) Other considerations by which the use of soil-cement can be justified are its lasting and trouble-free service with very low maintenance costs.

(e) The use of a cement stabilised sub-base under a concrete surfacing ensures that the sub-grade is water-proofed and protected from damage by traffic during construction.

(f) The thickness of the concrete surfacing and the amount of cement used in its manufacture are considerably reduced.

(g) The infiltration of water through the joints in the concrete surfacing to the sub-grade is prevented, with consequent reduction in the risk of breakage at or near the joints.

(h) The amount of cracking in the concrete slabs is minimized.

(i) The need for steel reinforcement in the concrete is obviated.

(j) The most important and vital argument in favour of soil-cement is that it is an engineering material that eliminates all guess work regarding its quality and performance unlike other low cost pavements. Control tests in the laboratory and the field assure its success in building roads that last. The freezing and thawing tests ensure the performance of soil-cement under the extremely adverse conditions in Himalayan tracts.

The advantages of economy and life of soil-cement pavements are such that this type of construction should be tried in India on a larger scale than hitherto.

In the following discussion, the technology of soil-cement and the technique of its construction are briefly dealt with to elucidate the various aspects of this form of road construction.

2. PROPERTIES OF SOIL-CEMENT

2.1. The classification of soils for highway purposes

2.1.1. Factors affecting quality of soil-cement

The quality of soil-cement as a constructional material depends on several factors including type of soil, cement content,

moisture content, and density. Of these the most important is the soil type. The remaining factors affect the properties of soil-cement made from any given type of soil, but do not provide ways and means of improving a soil that is inherently unsuitable. Fortunately most soils are suitable for soil-cement construction.

2.1.2. Classification of soils

Soils may be classified directly on texture and on physical properties ; but for soil classification to be of practical value, it must be correlated with the engineering behaviour of the soil. This behaviour depends on conditions obtaining at the time of construction and also on the climatic conditions of the locality. These vary widely and make it difficult to devise any soil classification scheme of universal application.

There are six main classification systems as follows :

1. United States Public Roads Administration classification.
2. Textural classification.
3. Casagrande classification system.
4. Civil Aeronautics Administration (C. A. A.) classification system.
5. Compaction classification.
6. Burmister's method of descriptive soil classification.

An important adjunct to soil classification for the success of soil-cement work is the establishment of soil profiles wherein are indicated changes in the nature of soil, depth of various strata, and level of water table. These details help in determining the availability of suitable soil for soil-cement work in the locality and the drainage conditions on the site. They are usually gathered at 300 ft. intervals along the centre line of the proposed construction.

Since most of the published data on soil-cement tests are based on **U. S. Public Roads Administration Classification System,** a brief summary of its salient points is given below.

This system was adopted in 1931 and revised in 1942. It was further revised and simplified by a sub-committee of the U. S. Highway Research Board in 1945. Since then it has been in use with the U. S. Bureau of Public Roads and various State Highway Departments in the U. S. A.

Table 4

Characteristics of the Sheepsfoot Roller

	Metric Units	English Units
Weight (empty)	3600 kg	7940 lb
Weight (water ballasted)	4600 kg	10,100 lb
Number of drums	2	2
Frame	Floating	Floating
No. of feet on drum	64	64
No. of feet in row per drum	4	4
Distance between centre	30.5 cm	12 in.
Size of feet	5.7 × 5.7 cm	2¼ in. × 2¼ in.
Foot pressure (empty)	13.57 kg/cm	193 lb per sq. in.
Foot pressure (water ballasted)	17.5 kg/cm	249.4 lb per sq. in.
Diameter of drum	106.7 cm	42 in.
Length of feet	17.8 cm	7 in.
Overall length	391.4 cm	12 ft 10 in.
Overall width	250 cm	8 ft 3 in.
Width of drums	122 cm	48 in.
Space between drums	7.6 cm	3 in.

Comparison of maximum dry densities and optimum moisture contents obtained for the four soils in laboratory tests and with various types of compaction equipment

Soil type.	Clayey Soil			Silty Soil			Sandy Soil			Sand		
	Maximum dry density.		Optimum moisture content.%	Maximum dry density.		Optimum moisture content.%	Maximum dry density.		Optimum moisture content.%	Maximum dry density.		Optimum moisture content.%
	lb/ft^3	Kg/m^3		lb/ft^3	Kg/m^3		lb/ft^3	Kg/m^3		lb/ft^3	Kg/m^3	
B.S. compaction test.	124	1986	10	121.0	1936	11.4	120.5	1928	11	101	1616	15.5
Modified A.A.S.H.O. compaction test	128.5	2056	9	127.0	2032	9.3	130.5	2088	9	106	1696	12.5
Test with the Abbott cylinder (40 blows)	125	2002	12	125.5	2008	11.5	119.5	1912	12	—	—	—
7-ton smooth-wheel roller	121	1936	11	122.5	1962	9	120.5	1928	8	106	1696	12
Pneumatic-tyred roller	110.5	1768	12	116.5	1864	8.3	112.5	1800	11	100.5	1608	11
Sheepsfoot roller crust [(a)]	122	1954	11	123	1968	12	119	1904	11	—	—	—
Sheepsfoot roller average [(b)]	103	1645	11	inconclusive		11	106	1696	11	—	—	—

(a) These figures comply with those of the upper part of the compacted crust (vide Fig. 6).
(b) Average of loose mulch and compacted crust.

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