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President of the Republic of India

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By Command of the President of the Republic of India

Military Secretary's Office,	*B. Chatterjee, Colonel,*
The 5th March 1951.	*Military Secretary to the President*

219

INDIAN ROADS CONGRESS

NOTICES AND ANNOUNCEMENTS

NEW ADMISSIONS

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1469	1613	Suram Singh Datta,	Assistant Engineer, P. W. D., B. & R., Sadulgarh, Gangapur District, Bikaner Division (Rajasthan).
1470	1614	Mangat Rai Singhal,	Supervisor (Bridges), Western Railway, Bulsar (Bombay).

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Paper No. 160.

"Design of Road Beds."

By

G. SRIRANGACHARY, B. E., M. I. E.

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SYNOPSIS

The Paper suggests that unbalanced design of the road structure is the cause of most of the ills that afflict Indian roads. A road should be designed as a structure with the "traffic width", i.e. the width needed by traffic, forming the starting point of the design. Earth shoulders, turfed edges, and slopes protected with grass should be provided with the express function of retaining and maintaining the "traffic width". Unnecessarily wide "standard" formations consisting of narrow central strips of macadam with wide earth shoulders should be deprecated.

The Paper suggests a method of designing the road structure based on experience and observation of construction and maintenance methods practised in Madras.

1. DESIGN OF THE ROAD BED STRUCTURE

1.1. Introduction

1.1.1. For designing a road successfully, like any other structure, all the factors affecting the design have to be taken into consideration. Whereas the days of hit-and-miss methods in road construction are gone, never to return, stage by stage construction is still in vogue in many countries, India being one of them. To build a road for the immediate requirements, and upgrade it as changing circumstances demand, is a usual practice in this country, just as a man of moderate means builds the ground floor of his house first and adds other storeys as his requirements increase and when he can afford to make additions. But such storeys can be added only if the foundations are initially designed for the heavier loads. So also a road can be upgraded to stand heavier traffic only if the foundation, that is the road bed, has been well and truly laid.

1.1.2. It is the purpose of this Paper to suggest a method of designing road beds in this country.

1.2. The Principal Variables

1.2.1. An examination of the available design methods and published literature shows that there are FOUR PRINCIPAL FACTORS generally recognised as important for the rational design of the road structure. These four factors are [1]* :—

(1) The natural soil under the road-bed ;

(2) The volume and character of traffic expected to use the road ;

(3) The moisture conditions which would exist in the road-bed and the subgrade after the construction of the road structure ;

(5) The climatic conditions, e.g., rainfall, temperature range, etc.

1.2.2. To arrive at a reasonable method of handling the variables involved, they may be evaluated on a numerical basis, or else they may be expressed by the use of adjectives like "fair", "good" etc.; or a combination of the two methods may be adopted.

* The references are to the bibliography at page 351.

Evaluating on a numerical basis requires research and simple test procedures. The "adjective" method involves personal judgement based on experience.

1.2.3. Subgrade Soil

There are several methods and procedures for testing soils, e.g. the C. B. R. method, the 30-in. plate bearing test, the North Dakota Cone method, and the Triaxial-shear test method. The results of these methods are correlated with road behaviour records, and numerical values evolved for use in design.

1.2.4. Traffic

Traffic censuses can be taken and the figures obtained used in designing a road structure, allowance being made for the expected growth of traffic.

Adjectives like "heavy", "medium", and "light" can be used to denote specified ranges of traffic density.

1.2.5. Moisture Conditions

This is a factor which is difficult to determine with accuracy. It is generally evaluated only on the "adjective" basis, e.g.—

(1) Arid or high land not subject to standing water ;

(2) Subgrade subject to occasional standing water ;

(3) Subgrade subject to saturation for short periods ; and

(4) Subgrade subject to saturation for long periods.

1.2.6. Climatic Conditions

Variations in weather from one area to another are well understood and may be evaluated correctly. Sometimes the evaluation may be on a rainfall basis, e.g.

Wet,	average annual	rainfall	over 50 inches
Medium,	-do-	-do-	25 to 50 ,,
Dry,	-do-	-do-	15 to 25 ,,
Very Dry,	-do-	-do-	under 15 ,,

Such classification may help to evaluate erosion of shoulders and embankments, probable duration of soaked conditions, and wear and tear of the wearing surface.

1.2.7. Breakdown of the Principal Variables Affecting the Design of Road Beds

Design of pavement and foundation to offer adequate resistance to deformation for the required standard of riding surface and width.—

- —Type of Subgrade Soil
  - —Granulometric Composition
  - —Plastic limit and Plastic index
  - —Colloid content
  - —Presence of detrimental salts
  - —Degree of compaction
  - —Bearing Power
- —Moisture Condition
  - —Availability of water
  - —Ground water
  - —Precipitation
  - —Condensation
- —Traffic
  - —Speed
  - —Total Wheel loads
  - —Repetition
- —Climate
  - —Rainfall causing erosion
  - —Rainfall softening the shoulders, and/or the wearing surface
  - —Variation in temperature
  - —Maximum temperature affecting pavement.

1.3. Breakdown of Design Problem into Four Steps

1.3.1. The design of road-beds may be broken down into four important steps :

The **first step** is to determine the component members which the road-structure should consist of, for the given conditions, i.e. for the values of the four principal factors. As the given conditions may vary along the length of the road, the road having dissimilar prevailing conditions should be considered separately, and separate designs worked out if necessary.

Accurate data should be obtained regarding the principal factors for each section of the road, if they vary from section to section.

1.3.2. The **second step** is to fix the criteria or standards pertaining to the service required of the road, and the strength and other properties necessary for each component member. Materials suitable for each member, for the desired economy, durability, and safety should then be selected.

1.3.3. The **third step** is to find out the strength and other properties of the chosen materials or mixtures of the materials, and work out the thickness and width necessary for ensuring that the road will provide the service it is designed for.

1.3.4. The **fourth step** is to work out complete designs for each change in any of the variables, draw up specifications, and work out the limits and other details regarding field controls required for ensuring satisfactory construction.

1.4. Structural Requirements

1.4.1. In the design of all structures, including road-beds, the first step is the determination of the structural requirements : in other words, to determine the type and magnitude of the loads and other forces which the component members of the structure may be called upon to withstand.

1.4.2. The different component members of road-beds are shown in Fig. 1(a) and (b). Certain component members, (e.g., sub-base, base, edge protection, slope protection) may be omitted under certain conditions, if the design considerations support such omission. The design procedure should lay down that reasons for omitting any of the component members of a road-structure should be placed on record.

1.4.3. For determining the type of each component member, a careful consideration of the principal factors is required.

1.4.4. The nature and intensity of traffic largely determines the type of the wearing surface, for economy, safety, and durability. Climatic factors also may require consideration under certain conditions, e.g., steep hill roads, dry and waterless areas, etc.

1.4.5. The type of subgrade soil and moisture conditions determine the type and strength of foundation layers[2].

1.4.6. For a given soil, climate, and rainfall indicate the nature of the edge and slope protection required (members v and vi of Fig. 1). Slope protection may sometimes be obtained after a lapse of time by a natural growth of grass or shrubs. If the protection is not provided as a part of the design, serious erosion and damage may be caused before the natural growth makes its appearance. Grass cultivation, sodding, or turfing used for slope protection have to be carefully designed and planned.

1.4.7. Edge protection (v of Fig. 1) will be necessary for most types of soils. If properly designed and constructed, it should also protect the soil shoulders from erosion.

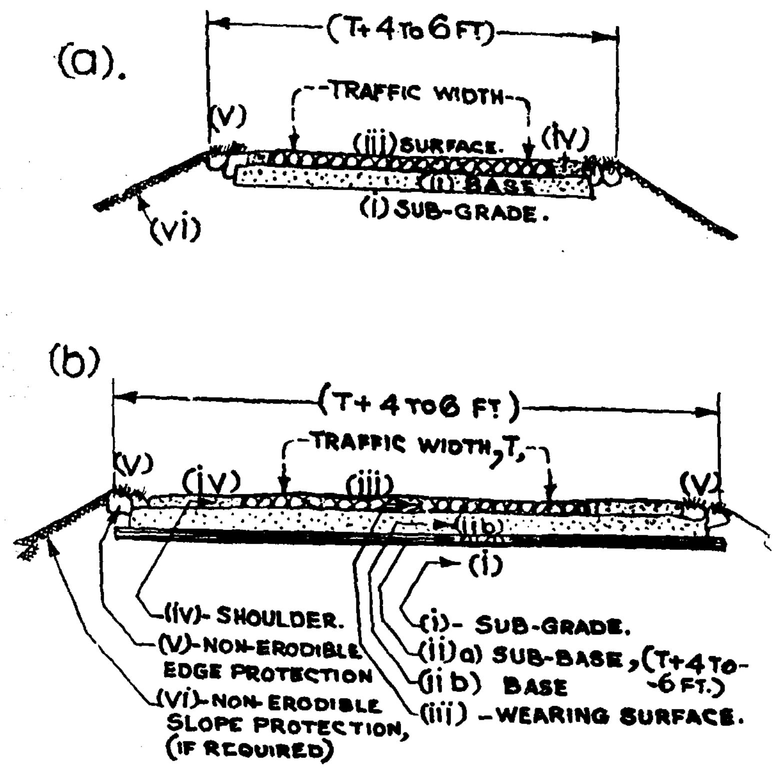


FIG. 1. BASIC REQUIREMENTS OF ROAD-STRUCTURE FOR CONDITIONS IN INDIA

1.5. Criteria Applicable to Design

1.5.1. It is usual to have different general standards for judging the performance of different road-structures, depending on the service desired. The standards relate to

(1) the width available for traffic ;

(2) the distortion permissible in the cross-section ;

(3) the riding surface ;

(4) the speed of traffic using the road ;

(5) safety ; and

(6) durability of the road structure.

1.5.2. There is no one simple criterion to judge the performance or adequacy of a road bed, but the overall problem is one of economics. "What is the minimum width and thickness which will be most economical for a given situation ? "What is the type of pavement and base most suitable ? " These questions have to be answered with an eye on the overall economy.

1.5.3. Many good roads have been successfully built in the past before these questions were fully answered as it was always possible to ADD adequate thickness and strength to cover uncertainties in the design concept[3].

Even now there may be situations where, on grounds of economy, the width or thickness of the wearing surface may have to be so designed that a part of it is to be ADDED at a later date, provided the base and subgrade are adequately designed for that purpose.

1.5.4. It is important to remember that there is no one particular thickness below which a pavement would utterly fail, and above which it would serve without any signs of failure. Serious and widespread failures occur with small thicknesses of pavement. As the thickness is increased, the rate of development of failures and the area that fails decreases with increasing thickness, rapidly at first and then more and more slowly [4].

1.5.5. As a general rule, the sub-base and base MUST be designed of adequate width, strength, and durability even if the wearing surface is to be constructed in "stages". It is often extremely costly to rectify a foundation failure, whereas a pavement failure may not be so serious.

1.6. Balance in the Design of Road Beds

1.6.1. Up to very recent times road construction has been of the nature of almost a practical trade. The specifications were often based on previous experience or merely on rule-of-thumb methods.

1.6.2. Ask a competent road engineer the following questions —

"Why was the road from A to Z built to a specification of 6 in. of soling with $4\frac{1}{2}$ in. of metal on top ? "[5]

"Why was the metalled surface 12 ft wide and the soling 13 ft wide ? "

The answer will probably be supported by reference to departmental specification or to some practice in vogue.

It is quite unlikely that the answer will be based on calculations or experimental evidence.

1.6.3. Take, for instance, the widely prevalent practice of making the foundation layer of the road structure from 6 in. to 12 in. wider than the wearing surface. This practice is very much like that of dimensioning the footings of buildings. The inference is perhaps admissible that the practice was originated by building engineers who had also to look after roads, as in "Buildings and Roads" departments of State governments.

1.6.4. Design concepts applicable to building foundations may not be valid for the design of road foundations. The absence in the past of any reference in departmental specifications to this important aspect of design was due to the lack of criteria or standards regarding the TRAFFIC WIDTH which should be available for safe use by traffic.

1.6.5. Speed and safety of travel and reasonable safety without a serious loss of speed while crossing, are expected by the road user.

1.6.6. Road bed design should, therefore, start off with the above-mentioned requirements as the chief criteria. The "TRAFFIC WIDTH", or the width of the road surface which should be available for the safe passage and crossing of traffic, is a function of the present traffic and its probable future development. It is a simple matter to fix reasonable standards in this respect.

1.6.7. The type of the wearing surface is of no interest to the road user so long as he is assured, within limits, of the required service all through the year. Whether the traffic width is made up of one or more types of wearing surfaces should be governed solely by economics and not by traditional rules and standards.

1.6.8. If a one-lane traffic-width could be conceived of as sufficient, i.e. if no vehicles would ever pass or overtake each other, there would be no justification for flanking the lane with wide earth shoulders. Such roads would, however, be very few. On most roads in India the traffic uses mainly the centre of the road for the greater part of the travelled length, yet there is need for a greater traffic width for the safety of passing and overtaking traffic. If, on grounds of economy, the extra width is to be provided by means of a cheaper surface flanking a central strip

of a superior type, the design should take into account the detrimental effects of the junction between dissimilar surfaces.

1.6.9. Regardless of the nature of the strips which comprise the wearing surface, the whole of the "Traffic Width" should at all times be assured of adequate lateral support. One way of achieving this is by providing the required width for the foundation[6]. Limiting the width of foundations so as to support only the central wearing strip results in exposing the edges of the central strip to water-softening under the zone of loading[6]. Such a condition, if allowed to persist, may cause a permanent deformation of the central wearing strip. This is usually accompanied by holes and other irregularities at the edges of the central strip, resulting in an appreciable diminution of the available main-traffic width.

1.6.10. The importance of traffic width is pertinent not only to the wearing surface and the foundation, but also to other component members of the road-structure. Given the "traffic width",—the starting point of the design—the other component members should be determined so as to suit that "traffic width". In other words, **the design of the component members should be such that the resulting road bed is structurally well balanced** [7].

1.6.11. It is important, however, that drainage crossings, the positioning of avenues, the land-width to be acquired, and other details take into account the future increase in the traffic width.

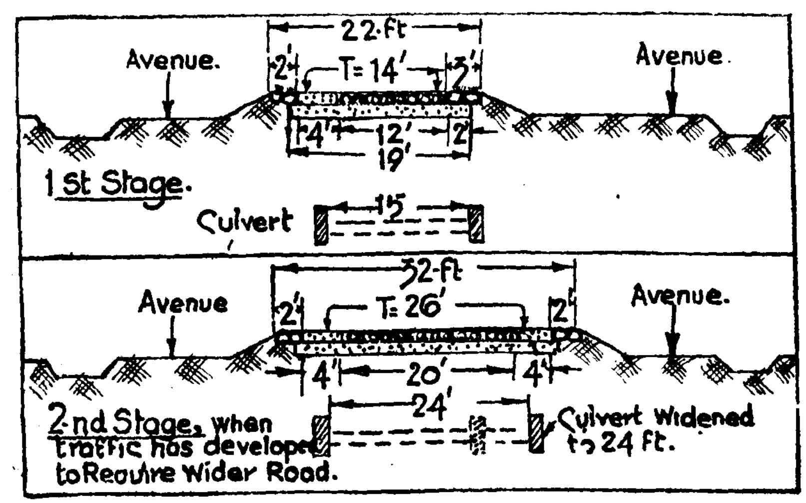


FIG 2. BALANCED DESIGN FOR STAGE CONSTRUCTION.

1.6.12. Fig. 2 illustrates a design in which the traffic width (T), initially 14 ft, is required to be increased to 26 ft at some future date. When the road is new and traffic light, the formation need be only 22 ft wide at the top. The foundation, which is 19 ft wide, supports the entire traffic width (14 ft). The wearing surface is 16 ft wide and consists of 12 ft wide macadam and only one shoulder 4 ft wide. (The useful traffic width is reckoned as two feet less than the wearing surface leaving out one foot on each side to allow for a possible weakness of the edges). Edge protection with grass sodding should be of sufficient thickness and strength effectively to prevent erosion of the shoulder.

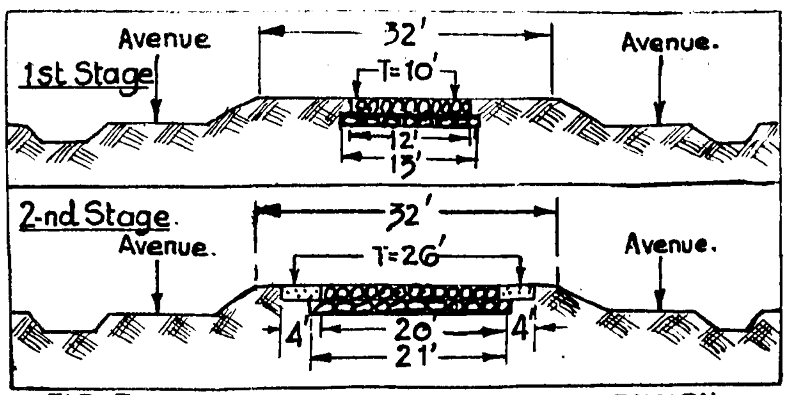


FIG. 3 . EXAMPLE OF AN UN-BALANCED DESIGN.

1.6.13. Fig. 3 illustrates an **unbalanced** design as conceived by the Author, the subgrade initially formed to the final width of 32 ft whereas the effective traffic width is only 10 feet. The soil shoulders, each 10 ft wide, are not of much use for traffic as they are liable to get eroded seriously in every heavy shower of rain. (See photo 1.)

1.6.14. In the design shown in Fig. 2 the road width over drainage structures may preferably be the final width desired. If, on grounds of economy, the larger structures are to be of minimum cost compatible with safety, the traffic width over such structures may be as low as 15 ft.

Fig. 2 shows a suggested alignment of drainage crossings with respect to the road centre line. It may often be advantageous to construct the substructure of culverts to the required final width to avoid construction difficulties in future widening operations.

Photograph 1.
An unbalanced design
with a 12-ft macadam strip with wide earth shoulders. (Para. 1.6.13.)

1.6.15. The design procedure should indicate the relevant data to be used under different conditions for obtaining a balanced design of the road-bed.

1.7. The Subgrade

1.7.1. The travelled portion of a paved road consists of a pavement structure resting on the subgrade. The pavement structure consist of a top portion serving as a wearing surface, and the lower portion which may consist of one layer, the base course, or two layers, the base course and sub-base.

All design procedures deal with the design of these several layers to suit the bearing capacity of the subgrade and the given traffic loads.

1.7.2. Field and laboratory studies combined with long experience in the evaluation of soil, geological, climatic, and moisture conditions, serve to determine the practice to be followed in design[2].

1.7.3. Most of the design controls regarding soil refer to conditions which can best be determined by sight, feel, and judgment in the field when the soil survey is made[2]. Bearing capacities are usually determined at the expected moisture condition, and maximum density by the Proctor Test. Depending on the nature of the soil and the expected moisture conditions, soils may be "rated" into categories. A suggested "rating" into four categories as adopted in North Carolina (U.S.A.) is shown in Table 1[8].

Table 1—Subgrade Bearing Values[8].

Subgrade Bearing value pounds per sq. inch.	Classification.	Description of soil.
30	Good	Sandy soils, with not more than 35 per cent passing No. 200 sieve—Good drainage.
20	Fair	Clay soils but not heavy clay soils (50 per cent or more clay) — Good drainage.
15	Poor	Heavy clay soil, or Clay soils where drainage is uncertain.
10	Very poor	Heavy clay soils. Poor drainage.

1.7.4. Since the strength of a given subgrade varies inversely as the water content [9], its future value will depend on the amount of water that it can absorb. It is, therefore, most important that—

(i) the subgrade should be well compacted so as to reduce as much as possible the water it can absorb :

(ii) the variation in the moisture content from the centre to the edge should be as low as possible, so as to minimise the variation in the bearing power.

1.7.5. The entrance of water into compacted soil may simply displace the contained air, or in addition it may produce swelling. The latter is not likely to occur if the soil is adequately confined, such as by the overlying layers of a road-bed. Swelling may, however, occur if the soil is unconfined as in earth roads, or in the earth shoulders of paved roads. If swelling occurs during absorption, additional water-holding capacity is created, and by this process practically all the strength of the soil can be destroyed[9].

1.7.6. Fine grained (clayey) soils are most likely to suffer by water absorption. It is, therefore, important to ascertain the wettest condition in a given case and the basis of design should be the strength of the wet soil [6].

1.7.7. Let us consider a subgrade of clay soil, confined at the top by means of a pavement which is porous but compact and well-knit. Let the subgrade be dry to start with and let water percolate through the porous pavement to wet the subgrade (Fig. 4 (a)).

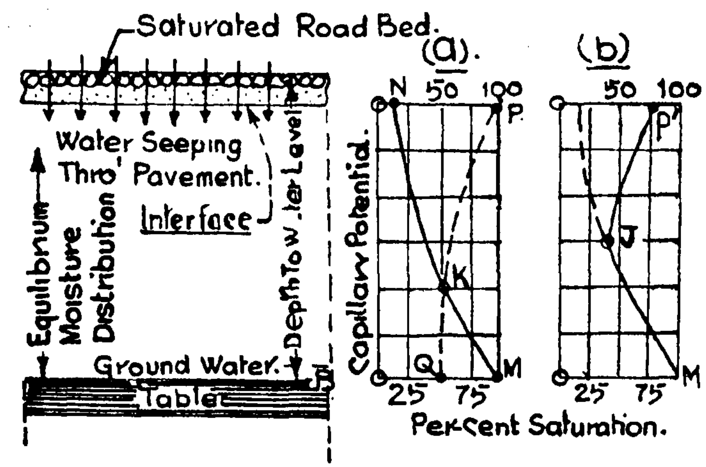


FIG. 4 .. MOISTURE ABSORPTION THROUGH POROUS, PAVEMENT, WITH GROUND WATER BELOW.

The fine-grained soil of the subgrade of high capillarity will quickly absorb the moisture passing down the porous pavement. The rate of capillary flow is a function of the net capillary force at the interface, and the resistance offered by the pavement to the passage of water. If water is available for a sufficient length of time, it will get absorbed by the subgrade and the percentage moisture at the interface will increase. Fig. 4 (a) indicates capillary potential curve MN for equilibrium for the supply of water from groundwater alone [10] and curve PQ for equilibrium for the supply of water percolating from the top only. Consider the case where water is being supplied from both directions. After some time, when both top and bottom get saturated, the line of capillary potential will be MKP.

At some intermediate stage when the top is not yet saturated, the line of capillary potential will be $M'JP'$ in Fig. 4 (b).

If the supply of water from the top is not adequate the moisture content at the interface may not reach saturation but remain at some intermediate value.

It has been observed that this value, in general, may not exceed 75 per cent of saturation moisture for clay soils (see para 1.7.9) except near the edges of the road bed and when the water table is almost at pavement level.

If the groundwater table is at a considerably lower level, the maximum moisture content at the interface will be less than when the groundwater table is at higher levels.

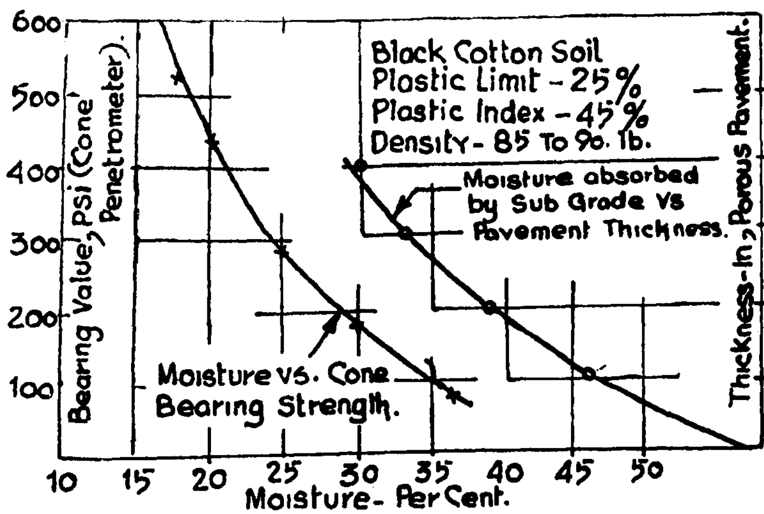


FIG. 5. SUB-GRADE OF BLACK-COTTON SOIL UNDER POROUS PAVEMENT.

1.7.8. Fig. 5 represents moisture condition in a typical black cotton soil subgrade in Madras State, with NO groundwater table, after the equilibrium condition is reached for different thicknesses of the porous pavement. It is seen that for a 9-in. thickness of pavement the subgrade moisture is 33 per cent, which is in excess of the plastic limit of the subgrade soil by 8 per cent. Fig. 5 shows also the bearing power by the cone penetrometer, at the different moisture conditions of the subgrade.

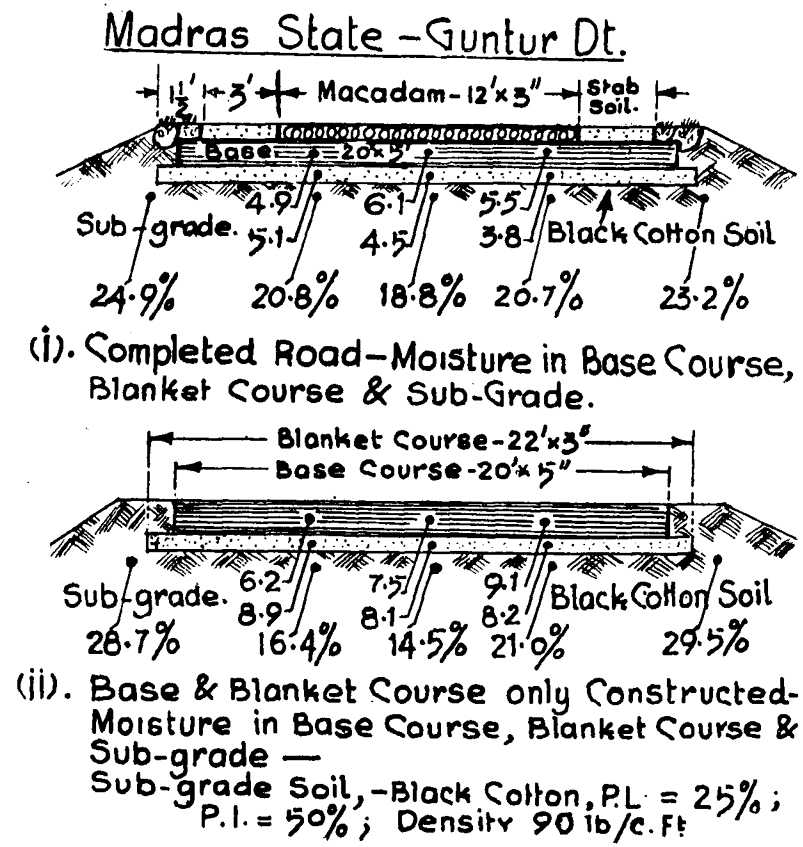


FIG. 6.

1.7.9. Fig. 6 represents a typical equilibrium condition of compacted subgrade of black cotton soil under a road bed in Madras State. Fig. 7 shows the final moisture condition after full saturation of the subgrade, sub-base and base-course at the Test Track at Alipore, 1949-Series Tests.

After a severe summer, a few inches of rain over the macadam pavement may often be adequate for the subgrade to attain equilibrium moisture condition. The strength of the subgrade in this condition may be taken to represent the worst condition for the purpose of design, as will be explained in greater detail in paragraph 2.3.

(W·Bengal)-1949 Series Test at Alipore Test Track.

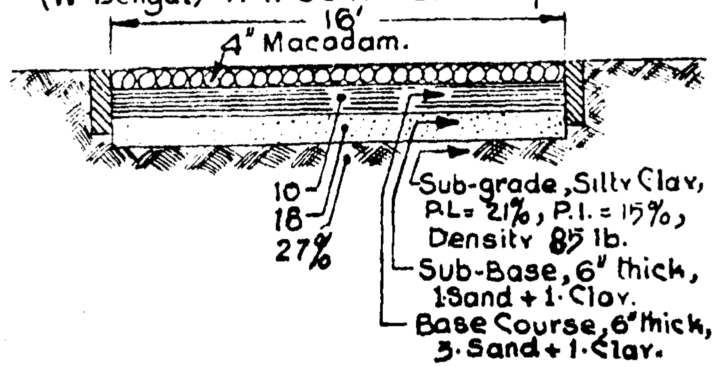


FIG. 7. MOISTURE IN THE ROAD-BED AND SUB-GRADE AFTER THE FIRST RAINS AFTER SUMMER.

Fig. 8 represents an analysis of the moisture conditions at 80 test points on a six-mile length of road where the wearing surface, base-course, blanket course, and subgrade were the same as those shown in Fig. 6. The analysis shows that for the greater part of the road

(i) the moisture content in the subgrade beyond the width of the foundation layers was within 85 per cent of full saturation, or 120 per cent of its plastic limit;

(ii) the moisture in the subgrade just below the foundation was for the most part within 80 per cent of its plastic limit, or 59 per cent of saturation moisture;

(iii) the moisture in the stabilised soil base-course was for the most part within 75 per cent of its theoretical water absorbing capacity.

The soil mix of the base-course was designed so that its theoretical water capacity was within 70 per cent of its plastic limit, i.e. 70 per cent of 15 per cent, or 10.5 per cent. Hence the maximum moisture content of the base-course was within 75 per cent of 70 per cent, i.e., 52.5 per cent of its plastic limit.

1.7.10. An experienced soil engineer should be able to identify and to estimate quite reliably the consistency and bearing strength of natural clay soils and compacted clay soils.

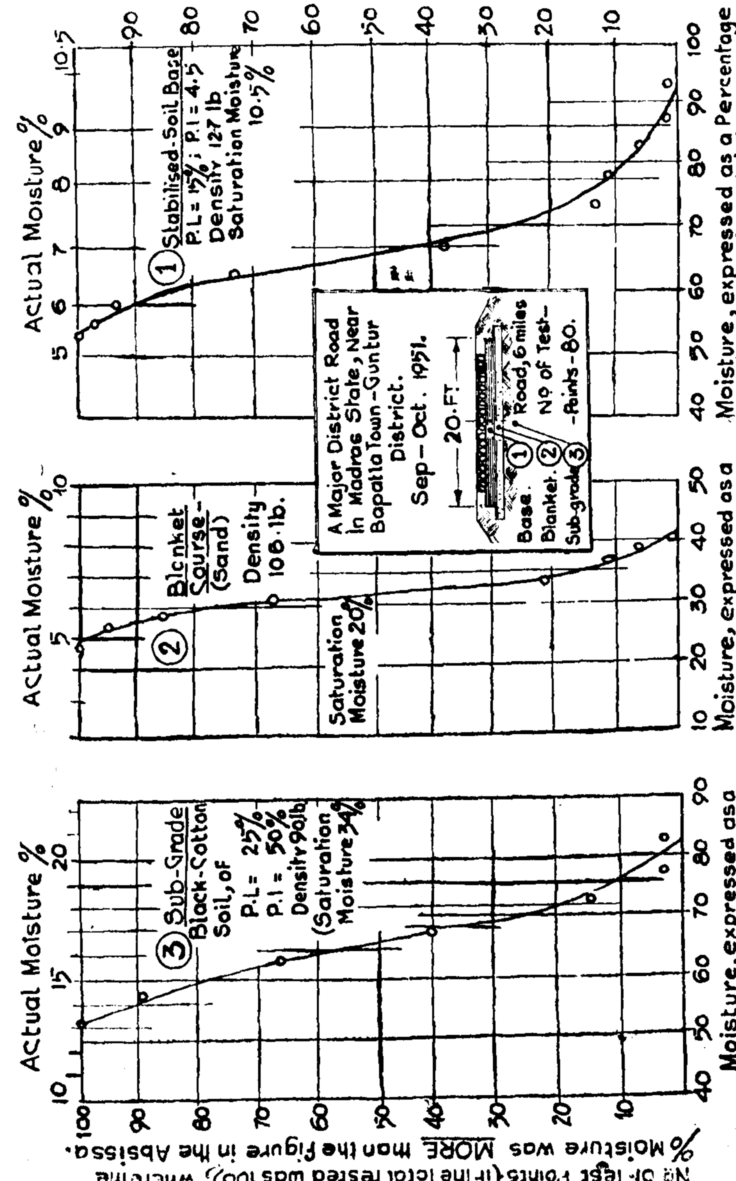


FIG. 8. ANALYSIS OF 80-TEST POINTS, MOISTURE DUE TO SOAKING BY RAIN.

Burmister has suggested tentative criteria for rating clay soils with regard to relative supporting value on the basis of relative shearing strength [11] as shown in Table 2.

Table 2—Rating of Clay Soil.

	Maximum shearing strength lb per sq. in.	Relative supporting value
Clay soil at Liquid limit	0.5	None
Clay soil, soft	0.8 to 1.5	Practically none
Clay soil, medium soft	1.5 to 8	Poor to Fair
Clay soil, moisture at "ball" moisture	8	Fair
Clay soil, firm or stiff	8 to 16	Fair to Good
Clay soil, medium hard	16 to 30	Good to Very good
Clay soil, hard	30 to 65	Very good

1.7.11. In the case of sandy soils, the detrimental effect of moisture is much less than in clay soils. As the subgrade resistance is largely dependent on shearing strength [12], and as the shearing strength of sand depends on its density[13], it is important to compact sandy subgrades to the maximum density possible.

1.8. The Base Course

1.8.1. Under this heading will be considered the principles governing the design of the base course, which may consist of one layer or two layers, the base course and the sub-base. The design should aim at the highest quality economically attainable under the given conditions. The use of a sub-base is indicated when the emphasis is on economy, since a large variety of combinations of materials may have been successfully used. **Specifications should not be too rigid**, but should be designed to take full advantage of locally available materials [6].

1.8.2. Almost every construction material, whether it be stone, concrete, lumber, or metal, consists of some kind of particles bound together by some sort of cementing agent. The particles may be sand grains, pieces of metal, crystals, etc.; the cementing agent is normally composed of smaller particles, liquid, plastic, or solid [14].

The strength and durability depend on the type of the particle, the type of the cementing agent and the method of compounding or processing. Tests on untreated soil materials of known behaviour indicate that they should have a cohesion value of at least 12 pounds per sq. inch and an angle of internal friction in excess of 30 degrees, for successful use in the base course[15]. Fairly well graded sand can be used as a sub-base to within 6 in. of the road surface [16] if the sand, by suitable compaction, can develop "apparent cohesion" of about 5 pounds per sq. inch. The angle of internal friction may then be over 35 degrees.

1.8.3. The importance of a sufficient base width on the outside to provide enough lateral support to develop the full effect of the base thickness cannot be over-emphasised. This is particularly important in widening rigid-type pavements, if the widening is limited and receives only a one-wheel assembly, the other wheel being supported by the older part of the road[6].

1.8.4. If an average can be indicated, base courses vary from 6 in. to 8 in. and sub-bases from nothing to 12 in., and the trend is towards an adjustment of the overall thickness in the sub-base as a matter of general practice [17].

1.8.5. The strength-imparting power of base courses has been examined by several investigators. This power varies with the inherent properties of the subgrade, and for a contact area of 890 sq. in. it may be 0.8 to 4.0 lb per sq. inch per inch of the thickness[18] of the base course. The effect of the size of the contact area, in other words the perimeter-area ratio, on the load-bearing capacity is important when considering the design of road beds to carry heavy pneumatic tyred loads. The perimeter-area ratio of these heavy wheel loads may be much lower than the wheel loads usually assumed and the bearing capacity may consequently be lower. This may require a thicker base than what is required for the lighter load. By knowing the rate of strength increase, as indicated in Fig. 9, the extra thickness required of the base can be calculated[19].

1.8.6. For the design of rigid pavements, the stability of the subgrade is, within reasonable limits, more important than the thickness of the pavement [20]. Reinforcing the subgrade with an additional underlayer of selected material, granular in type, is recommended in all cases of subgrades of low bearing value.

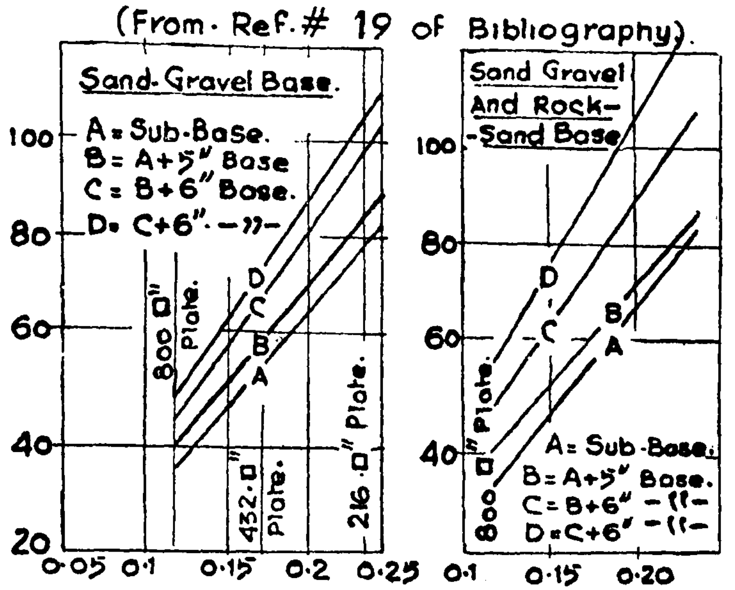


FIG. 9. STRENGTH-IMPARTING POWER OF SUPERIMPOSED LAYERS—Aro·25" Deformn.

1.8.7. In the design of flexible pavements, the most important requirement is to prevent shear deformation. If the base or subgrade should fail by shear deformation a major repair job may be necessary. The second requirement is to avoid excessive compaction by traffic, which may be more important than was assumed in the past[21].

1.8.8. The latter requirement is met by considering the strength of the road bed under repeated loading. Investigations made in other countries show that the **permanent deformation after** 10,000 repetitions may be taken as equal to the **total deformation** produced by one application of the load.

1.8.9. If the base is designed as suggested above, the wearing surface should be capable of withstanding the deformation produced. The use of thin bituminous pavements, therefore, requires a base of very high quality and strength, such as macadam or sand-clay-gravel of high density and stability[16].

1.9. Wearing Surface

1.9.1. In the design of the road-structure two sets of criteria are used, one for selecting the type of wearing surface and another for determining the foundation design[2].

1.9.2. The volume and type of traffic largely control wearing-course design. While it is usual to refer to the "30-year Cost" basis for selecting the type of the wearing surface, inability to meet the heavy initial cost may, in some countries, be a determining factor in constructing a large milage of low-cost roads[22].

1.9.3. Stage by stage construction of the wearing surface, as widely practised in India, may be adopted where conditions are favourable to the durability of lower type pavements. A macadam surface is frequently used for this purpose. A superior surface may be laid over the macadam surface subsequently depending on traffic, economic, and other considerations.

1.9.4. There is no fixed procedure governing the use of the traffic factor in designing wearing surfaces. It is usual to keep in mind some arbitrary limits of traffic, by count or by total tonnage, when deciding upon pavement type.

1.9.5. As the deforming of the subgrade and base tends to bend the surface upward, it is necessary to make the surface of a material capable of resisting tensile deformation. Therefore the wearing surface must possess enough cohesive strength to resist tensile stresses[3].

1.9.6. A striking illustration of the effectiveness of tensile reinforcing was the discovery during World War II that a strong waterproof membrane like P. B. S. (pre-bituminised sheeting) placed under a metal network would automatically seal the openings in the network. The upward flow of soil under load was thus completely blocked. The P. B. S. made an effective seal even in waterlogged clay soil, as the plastic soil pressed on the P. B. S. and held it firmly against the metal network. By this method a large number of all-weather runways were built within a matter of days, instead of months[23].

1.9.7. If a macadam wearing surface is built with fragments of a refractory stone like granite or quartz, and a soil binder, the surface is likely to get **torn** up due to suction by fast-moving pneumatic tyred wheels. The wearing surface may often be completely torn up within a matter of days or weeks. The superior wearing quality of hard stone has little chance to develop, and the surface might have to be re-built at frequent intervals.

1.9.8. A possible explanation is to be found in the negligible tensile strength of the surface owing to the poor cohesion of ordinary soil used as binder. The use of a highly cohesive soil as binder is hardly possible on account of its tendency to stick to the roller during compaction. "Rotten granite" soil or disintegrated granite soil taken from below the layer of the top soil has been found to serve more effectively than ordinary soil binders in Hongkong,[24] and in Madras State[25]. Analysis of typical "rotten granite" soils is given in Table 3.

Table 3—Engineering Properties of "Rotten Granite", known also as "Disintegrated Rock Gravel" in Madras.

Source	Atterberg Limits			Mechanical Analysis % passing sieve No.					Maximum Density (Proctor)lb	Cementation value*
	L.L.	P.L.	P.I.	1"	10	40	60	200		
1. M. 23/1 of Southern Trunk Road (Madras)	36.3	18.3	18.0	94.8	64.8	36.4	26.7	19.6	113	70 to 80
2. M. 25/3 of Southern Trunk Road (Madras)	39.8	21.9	17.9	100	96.1	71.1	66.0	46.3	111	-do-
3. M. 27/8 of Southern Trunk Road (Madras)	34.1	17.9	16.2	100	97.8	65.3	53.6	27.8	109	-do-

* Cementation Value of Macadam Stones:

Granite — 2 to 6

Lmie Stone — 20 to 27

Laterite — 25 to 40

1.9.9. The interlocking of hard stones by itself is ineffective, as the interlock exists only in two dimensions on the road surface. As there is hardly any interlock in a vertical direction, the use of a binder of the required cohesion is of considerable importance in localities where a macadam surface is still the ruling specification.

1.9.10. The foregoing discussion leads to the inference that the true function of the binder is to prevent "**un-locking**" of the macadam stone fragments by the suction and the un-ravelling forces caused by pneumatic tyred traffic. These forces act in a vertical plane. A thin tenacious film of asphalt-sand mixture, if effectively **keyed** to the "interlocked" macadam surface, (Fig. 10) has been found, from recent experience in Madras, much more effective than the best "rotten granite" soil binder. (Also see Section 1.10.)

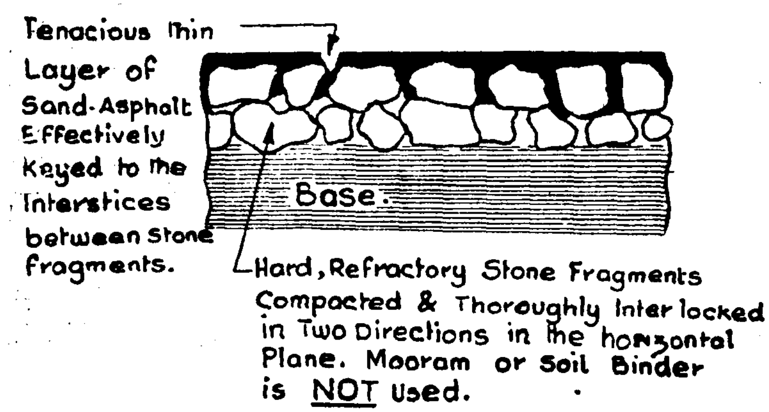


FIG. 10 · TENSILE STRENGTH REQUIRED FOR MACADAM SURFACE.

1.9.11. Whereas there is no fixed procedure for calculating the thickness of the macadam wearing course, a weaker base would obviously require a greater thickness of macadam than a stronger base. In actual construction, an extra thickness of 1 inch to 2 in. has to be added for possible wear before the next renewal of the surface.

1.9.12. It has been suggested that the pressure distribution through a macadam surface follows a curved line, and from

Boussenesq's equation and pressrue bulb conception the following formula for the depth of road metal has been derived[26] :—

$$d = r \times \sqrt{\frac{P}{p} - 1}$$

where d = metal depth in inches

r = radius of equivalent circle of tyre contact area

P = tyre pressure, lb per sq. inch

p = base or subgrade bearing valve, lb per sq. inch.

Another method of determining metal thickness is described in paragraph 2.6.

1.9.13. Asphaltic concrete which has high tensile strength requires less thickness than macadam for the wearing surface. It has been reported from tests made in the Asphalt Institute (U. S. A.) [27] that the load-supporting power of asphaltic concrete may be more than twice that of crushed rock of equal thickness.

1.9.14. The design of cement concrete pavements is based on the modulus of subgrade reaction. The limiting deflection is taken as 0.05 in. in Westergaard's formula which, with some modifications where necessary, has been widely used in all countries.

Where a cement concrete pavement is constructed over an old macadam surface of considerable thickness (over 10 in.) the cement concrete may be only 3 in. to 4 in. thick,—a specification widely used in India. As the cement concrete surface is then very thin, great care is taken to obtain the strongest possible concrete with the given materials.

1.9.15. Protecting the edges of the pavement from water absorption is of great importance. If the shoulders are pervious, successive rains may cause waves of wetness to travel downward and also towards the centre of the pavement where the subgrade is relatively dry [10]. Increasing the thickness of the pavement at the shoulders is a remedy, but it may be insufficient. In some cases protection of the shoulders by an impervious blanket may also be necessary to prevent the entrance of water and consequent loss in subgrade bearing value at the edges.

1.10. Braking Stresses

1.10.1. In addition to vertical pressure, the stopping of vehicles causes shear stress, s, on the surface in the direction of

travel. The magnitude of s depends on the rate of deceleration and is limited by the product of the vertical pressure and the coefficient of friction between the tyre and the surface.

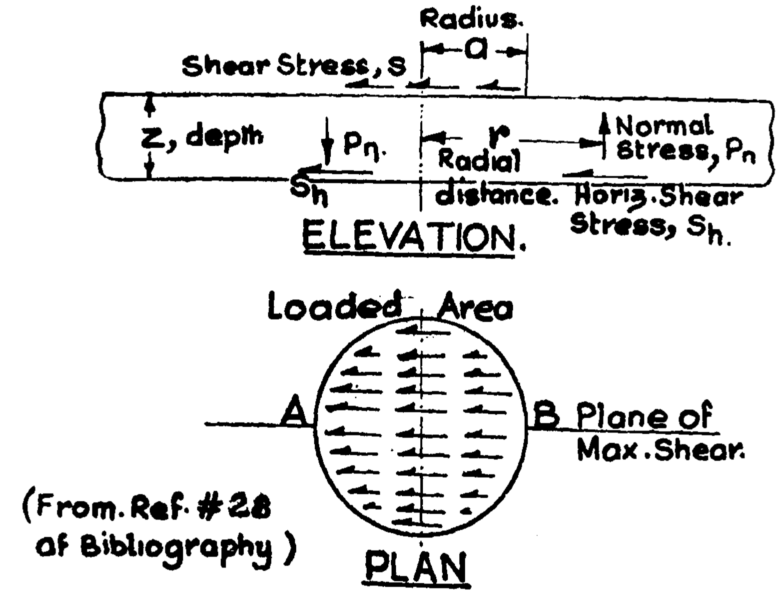


FIG. 11. STRESS FROM APPLIED SHEAR STRESS.

1.10.2. The horizontal shear stress s_h near the contact of a surface course may be a factor determining the thickness of thin bituminous surfacings. The horizontal normal stress developed by the braking action, (Fig. 11), is compressive in front of the area of contact, but tensile behind the centre of the area. The maximum tensile stress is at the edges A and B of the contact area, and theoretically is of infinite value[28].

1.10.3. If c is half the compressive strength of a cohesive surface, the thickness, Z, of the surface is given by[28]

$$Z = \frac{2}{3} a. \frac{(s - s_b)}{c}$$

where a = radius of loaded area

s = shear stress induced at the contact surface

s_b = shearing resistance between the wearing surface and base-course

$c = \frac{1}{2}$ the compressive strength of the surface.

For the worst condition, $s_b = 0$ and $s = c$, $Z = 2\ 3\ a$; which, for a 5,000 lb load and $a = 3\frac{1}{2}$ in., gives $Z = 2\frac{1}{2}$ in..

For c greater than s, and with an appreciable value of s_b, the thickness Z will be less. If $c = 2s$, and $s_b = \frac{1}{4}s$, Z equals $\frac{1}{8}a$, which, for a 5,000 lb load and $a = 3\frac{1}{2}$ in., gives $Z = 0.9$ inch.

2. A SUGGESTED METHOD OF ROAD-BED DESIGN

2.1. Operational Research*

2.1.1. Experience of road construction and maintenance in Madras State focussed attention on the need for a rational assessment of the current specifications and methods, on the basis of economy, durability, and satisfactory performance from the user's point of view.

2.1.2. Investigation into the problems met with in construction and maintenance was followed by the devising of solutions compatible with given technical-economic considerations Factors met with in the actual working of functioning organisations (contractors, departments, etc.) were considered for assessing the effect of differences between actual practice and written specifications.

2.2. The Problem Expressed as Three Issues

2.2.1. The result of these studies in Madras has been expressed in the form of three issues as described hereunder :—

(1) (a) The lack of standards required to ensure a minimum "**traffic width**" for different classes of service.

(b) The lack of standards, except for superior type pavements, required to ensure a satisfactory riding surface.

* "Operational Research" is a term used since the last war to describe the application of scientific methods of thought to the analysis of observation and experience.

(2) The width and the thickness of base course according to the current practice are rarely ever designed for the actual soil and drainage conditions. (In most specifications the soil and drainage conditions are hardly ever mentioned.)

(3) In designing foundations for a road in "unfavourable" areas the occurrence of a prolonged saturated condition of the subgrade and base course may be frequent. The foundations should therefore be designed for such conditions.

Issue (1): Traffic Width and Riding Surface

2.2.2. Current specifications prescribe a minimum width of 20 to 32 feet for the subgrade. The wearing surface, however, is limited to a narrow width, 8 ft to 12 ft at the centre of the formation. The surface type on either side of the central wearing strip is rarely ever specified. In consequence it is usual to fill it with soil from nearby borrow pits, forming in effect two narrow earth roads called "berms" or "shoulders."

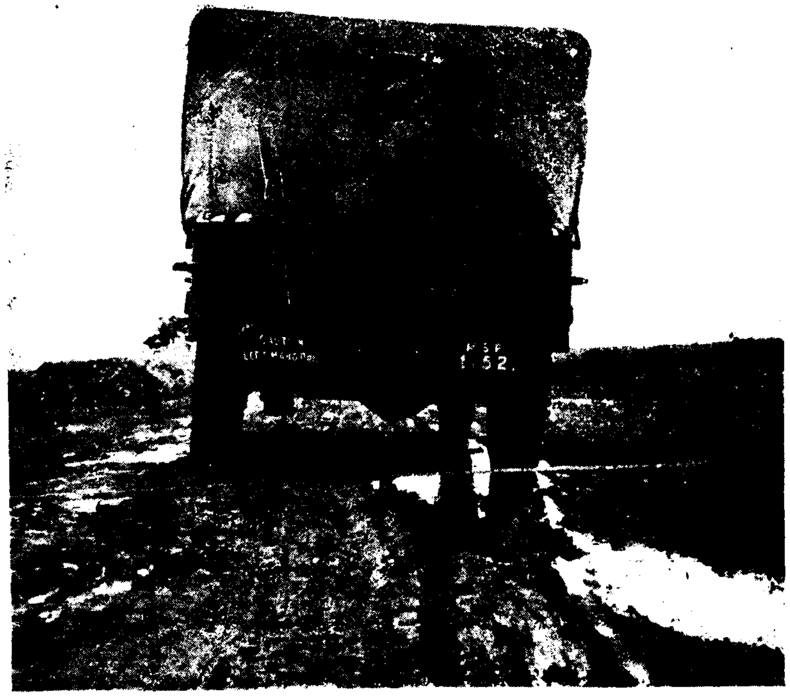
2.2.3. Inadequate width of the central pavement results in a frequent use of the shoulders by fast traffic. The shoulders, however, are rarely ever in good shape. They are either "gullied," or have had a recent addition of fresh soil. Rains saturate the fresh soil and often make the shoulders useless for fast traffic.

2.2.4. The unsatisfactory condition of earthen shoulders[29] is often a recurring feature. A by-product of this condition is the undermining of the edges of the paved surface. Improvement of the "traffic width" is hardly ever possible and the road remains unsatisfactory for several months each year. (See Photo 2.)

2.2.5. A satisfactory solution of this problem obviously rests on the principles of structural design, as will be described in section 2.5. Results of experiments in Madras clearly indicate the all-the-year-round satisfactory service that can be ensured by specifying the "traffic width" as the **chief criterion** of design. In this specification the base width and subgrade width are dependent on the "traffic width" instead of being independent according to traditional specifications.

Issue (2): Width and Thickness of Foundations

2.2.6. The current practice in Madras and some other States in India is to make the base course 6 in. to 12 in. wider than the pavement. In other States it is just equal to the width of the pavement.



Photograph 2.
Undermining of the edges of the pavement. (Para. 2.2.4.)

The thickness of the base course under flexible pavements is usually a fixed quantity irrespective of soil and drainage conditions. This thickness is 6 in. in Madras, and 8 to 9 in. in Madhya Pradesh.

2.2.7. The need for a scientific basis for determining the thickness of the base course is indicated by:

(a) the unsatisfactory performance or early failure of the road structure under unfavourable conditions of soil and drainage;

(b) the avoidable waste due to over-design, i.e. providing a thicker base course than is necessary. (If the soil and drainage conditions are favourable, a total thickness of as low as 6 in. including the wearing courses may be adequate in most cases.)

2.2.8. For a given soil, the subgrade moisture content is the most important factor in determining whether or not failure occurred at any given thickness. Tests in Minnesota State (U.S.A) showed that at 20 test points where the subgrade moisture was below 70 per cent of the plastic limit, only one failure had occurred where the combined thickness of the base course and the wearing course was not less than 6 inches. At 17 test points where the subgrade moisture was 95 to 105 per cent of their plastic limit, the surface was good only at three points where the combined thickness was less than 10 inches[16].

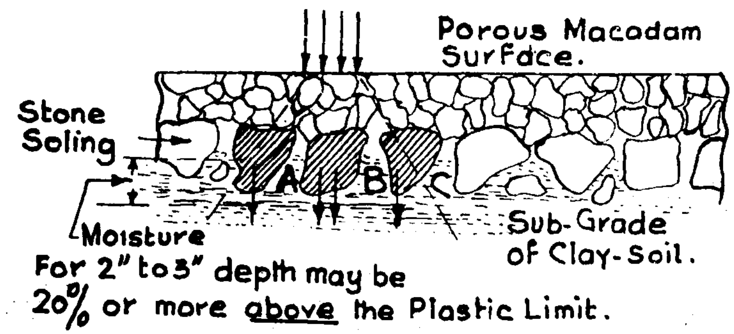


FIG. 12. FAILURE OF STONE-BASE OVER CLAY SUB-GRADES IN WET CONDITION.

2.2.9. In subgrades of high-plasticity soil, and where the wearing course and the base course are permeable, observations in Madras show that the moisture may rise up to 20 per cent above the plastic limit for a few inches at the top of the subgrade. Where stone soling is used as a base, this condition may result in the wet soil going up into the space or cavities A, B, C (Fig. 12) between the stone pieces. As plastic deformation in all wet bodies of earth including fragile and tough clays depends on the moisture content and not on the magnitude of the load,[30] even light traffic during successive wet periods may cause progressive deformation of the road-bed.

2.2.10. It has been recognised that a sandy sub-base (or sand blanket) is effective as a capillary cut-off if soil and drainage conditions are unfavourable[31]. For such conditions the use of stabilised soil has distinct advantages. Experience in America[32] and in Madras State indicates that stabilised road-beds are required for long life and satisfactory performance.

Issue (3): Design should take Account of Actual Soil and Moisture Conditions

2.2.11. Current specifications generally do not take account fo the soil and drainage conditions actually existing. Subgrades are usually allowed to get compacted by rains or by "ponding" with water, a practice quite harmful for fine grained soils. If, however, the subgrade is used by traffic for some time as an earth road, the top (12 to 18 inches) is likely to get compacted to 90 or 95 per cent of the maximum density of the subgrade soil. In the absence of traffic compaction, the subgrade should be compacted for at least one foot depth[7], to 85 to 90 per cent of the maximum density[33].

2.2.12. It is important to know how much water the subgrade is likely to absorb. The strength of a given subgrade varies inversely with the water content[9], and its future value will depend on the amount of water it can absorb. It is, therefore, of utmost importance to ascertain the **wettest** condition in a given case, and the basis of design should be the strength of the subgrade in that condition[6].

It is recognised, however, that ascertaining the data in each case as stated above may involve laborious field and laboratory work. Classifying the prevailing soils and the drainage conditions into a few categories may be sufficient to cover different conditions in a given area.

2.3. Suggested solution

2.3.1. Most roads in Madras State are provided with a wearing surface of macadam, which is more or less pervious depending on the cementing value of the stone fragments. Continuous rains may cause sufficient water to percolate through the macadam to saturate the subgrade.

2.3.2. Standing water on one or both sides of the subgrade may keep the subgrade in a saturated or nearly saturated condition for prolonged periods. After the wet season, evaporation of the subgrade moisture through the macadam layer may cause serious variations in moisture content between the centre and the edges of the subgrade.

2.3.3. Experience in Madras State indicates that it may be adequate to provide for three categories of wetness of the subgrade as follows :—

(i) Good — Subgrade not likely to get saturated.

(ii) Poor — Subgrade likely to get saturated and remain so for short periods.

(iii) Bad — Subgrade likely to get saturated and remain so for prolonged periods.

No data are available regarding the subgrade moisture conditions existing just after construction It has therefore, been thought desirable to use as a basis the conditions which might be expected to exist after a period of service[16].

2.3.4. On this basis, experience in Madras shows that fine-textured soils with plastic limits above 20, may reach 85 to 90 per cent saturation. Sandy loams with plastic limits of 15 or less may reach up to 70 or 75 per cent saturation.

2.3.5. The moisture content of subgrades of black cotton or other clay soils may rise to 30 to 35 per cent, if the density of the subgrade is 85 to 90 per cent of the maximum. The actual strength of clay soils in that condition should be measured for ascertaining the lowest bearing value.

Subgrades of loamy soils may be expected to be compacted to a density of 105 to 110 lb per cu. ft., and moisture may not exceed 15 to 20 per cent.

2.3.6. It has been suggested that probably two, and certainly not more than three, values may be assigned to soils ranging from good to poor[34]. Experience in Madras using the North Dakota Cone Penetrometer suggests that a bearing value of subgrades from 100 to 200 lb per sq. in. may be taken to represent the prevalent conditions.

2.3.7. **Design Load.** The prevalent loads in Madras are :

(1) Bullock-cart wheel load 1500 to 1800 lb, and

(2) Pneumatic-tyred dual wheel, 5000 lb.

It is the latter load that has the greatest effect on the subgrade. The future load standard in India, which is to be 9,000 lb will only affect the subgrade at lower depths where it may generally be stronger having less moisture than the top of the subgrade.

2.3.8. Designing at the present moment for a **single-tyre load of 5,000 lb** would cover also the future 9,000 lb wheel load which will consist of two tyres with 4,500 lb loading on each. The amount of load transferred from one tyre of a dual wheel to the point of maximum subgrade pressure beneath the other tyre is not in excess of 10 per cent. The maximum intensity under dual 4,500-lb wheels may then be equivalent to the maximum intensity under a single tyre of 4,950 lb load[34]. Hence to design for a single tyre of 5,000 lb load seems to be adequate.

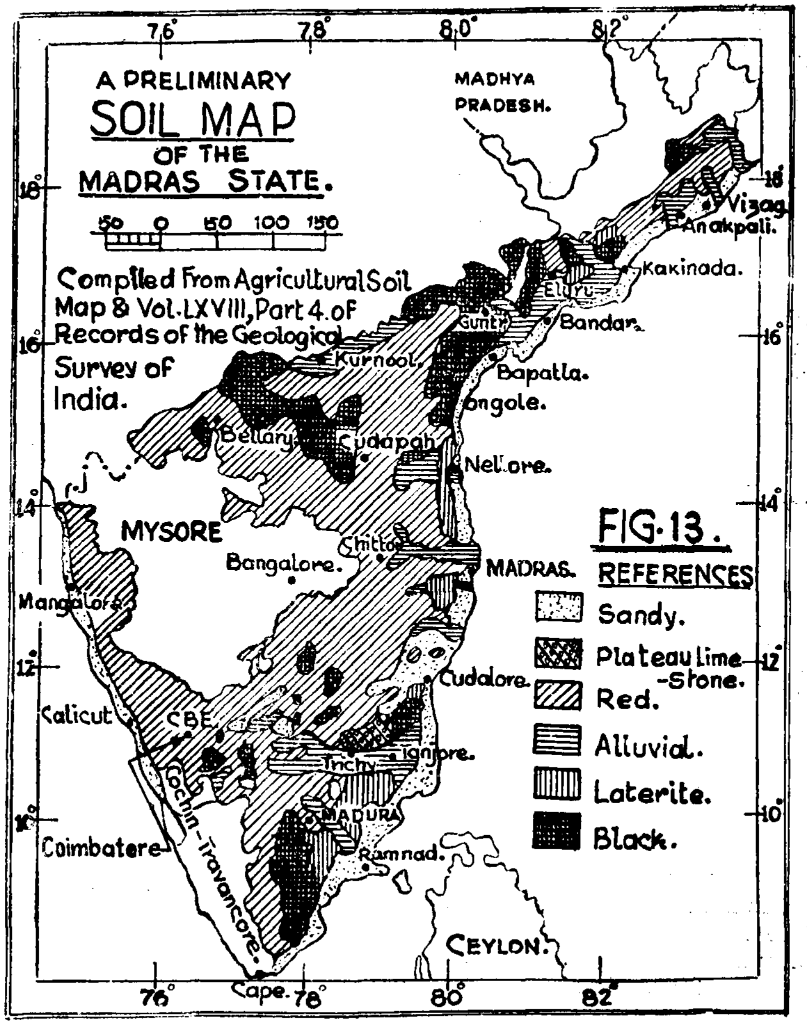


FIG. 13.

2.4. Typical Soils of Madras State

2.4.1. The map (Fig. 13) indicates broadly the chief soil groups of Madras State. It has been drawn by the addition of certain particulars found in the 'Sketch Map of the Geological Foundations of the Chief Soil-groups of India',[35] to the map of the agricultural soils of the State. It shows the four main soil groups, as well as the areas of lateritic soils and plateau lime-soils.

2.4.2. A perusal of the map shows that soil and drainage problems are severer on the coastal regions than in the interior. The black-cotton soil areas and the deltaic areas present special problems which are described in the paragraphs which follow.

2.5. A Suggested Method of Road-bed Design

2.5.1. As a result of the experimental work carried out in Madras State, a new design method has been developed. The principles underlying this method may be used for reviewing the specifications and practices relating to road construction and maintenance in Madras State and possibly in other parts of India. The criteria applicable to this method, as well as the actual design procedure, are, however, much at variance with the current specifications.

2.5.2. Based on the principles of structural design, the Madras method starts with the **'Required Traffic Width'** as the chief criterion.

Table 4 shows tentative standards which may be suggested for meeting different conditions of soil, drainage, and traffic in Madras State. The availability of the full "traffic width" for safe use during all seasons of the year is assured irrespective of the composition of the wearing surface. Gullies caused by erosion are completely eliminated by providing grass sodding of the required width shown in Table 4.

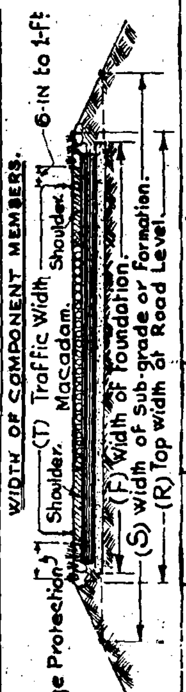
Photographs 3 and 4 show views of a road built according to this design, after having been under traffic for about two years. Photographs 5 and 6 illustrate the edge protection provided by grass-edging.

2.5.3. This design was initially worked out for the coastal and deltaic regions of Madras State, where the soil and moisture conditions are generally unfavourable. In these areas the cost of stone is very high. Wide foundations with stone soling have been found to be prohibitively costly. Stabilised soil under these conditions is very economical. In several cases it is even cheaper than the narrow width of stone soling of the old specifications.

2.5.4. As already stated in Section 2.3, drainage conditions are divided into three categories.

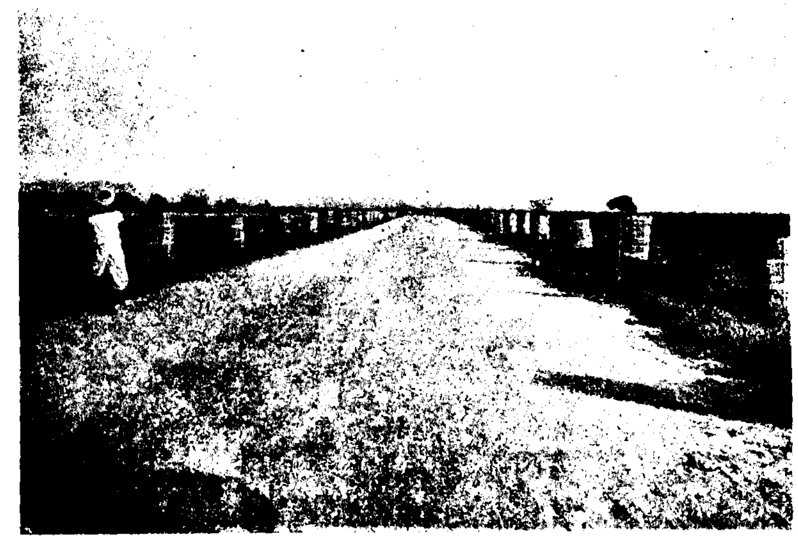
TABLE 4. SUGGESTED STANDARDS FOR ROAD-BEDS IN INDIA.

WIDTH OF COMPONENT MEMBERS.



SRI RANGACHARY. 1-JUL-1951.

Class of Service.	(T). Traffic Width	(F) Width of foundation F	(R) Top width at Road Level. Macadam.	Two Shoulders	Edge Protection Grass Sods	Total (R)	(S) Width of formation Sub-grade.
	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
A. For Dry Areas. 1. National and Provincial Road	28 / 18	32 / 22	18 / 12	12 / 8	6 / 4	36 / 24	40 / 28
2. Major District Road.	18	22	12	8	4	24	28
3. Other District Road.	14	18	10	6	4	20	24
4. Village Road.	12	16	8	6	4	18	22
B. For Wet Areas. 1. National and Provincial Road	22 / 16	28 / 22	16 / 18	8 / NIL.	4 / 4	28 / 22	32 / 26
2. Major District Road.	18 / 14	24 / 20	12 / 16	8 / NIL.	4 / 4	24 / 20	28 / 24
3. Other District Road.	14 / 14	18 / 18	15 / 9	NIL. / 6	4 / 4	18 / 18	22 / 22
4. Village Road.	11 / 9	15 / 13	12 / 10	NIL. / NIL.	3 / 3	15 / 13	18 / 16



Photograph 3 (above) and 4 (below):

Two views of a Major District Road—2 years old—designed according to the Madras Method; Subgrade Black Cotton Soil. (Para. 2.5.2.)

Photograph 5 (above) and 6 (below) :

Edge Protection by Grass Sods. Same Road as in Photographs 3 and 4. Gullies have formed on the slope only. Traffic width is maintained right up to the edge. (Para. 2.5.2.)

2.5.5. Traffic is divided into only two categories :—

(1) Under 500 tons mixed traffic per day ; and

(2) Over 500 tons mixed traffic per day.

At the present moment the average wheel load of heavy vehicles is about, 5,000 lb and it is probable this may remain so for many years to come. Heavier vehicles of the future may be confined to small areas around cities and industrial centres. This traffic can be provided for at a later date by suitable modifications in the design data.

2.5.6. As the design procedure was developed in connection with the use of stabilised soil for the road foundation, it may be pertinent to describe briefly the factors on which the design is based.

(1) The lowest bearing power of a given soil layer (base course) is dependent on its theoretical water-absorbing capacity. If water is available for sufficient time the water absorbed may approach or be equal to the theoretical capacity[32].

(2) An appreciable part of the strength of a soil is due to the cohesion of the clay binder. If the maximum moisture of the soil is not in excess of the plastic limit of the clay, the cohesive strength is still appreciable. Restricting the maximum moisture content to 70 per cent of the plastic limit is reported to provide an adequate factor of safety against excessive deformation[32].

(3) Testiug the compacted soil for bearing strength **at 100 per cent saturation** is essential for areas subject to water logging.

For the uniformity of test procedure. the same test may be applied for other areas also even though the maximum moisture may not amount to full saturation. A suitable weightage is then to be allowed for assessing approximately the bearing power at the expected moisture condition.

(4) The test procedure and the apparatus to be used for determining bearing power should be simple and not costly. The North Dakota Cone bearing test was selected as the most suitable both for use in the laboratory and for field control.

(5) For the present and expected future conditions of traffic in Madras State, two categories, i.e. light traffic and heavy traffic, appear to be adequate for design purposes.

(6) The variable due to drainage and maximum moisture may be reduced to three categories as already described in section 2.3.

Given the required traffic width, the design procedure is quite a simple affair as will be described by some examples in section 2.6.

2.6. Design Procedure : Examples

2.6.1. The average accuracy of the Cone Penetrometer is 80 per cent which is about equal to that of the C.B.R. method. Usually three to four readings are taken and any reading which varies by more than about 20 per cent from the majority is discarded, and the average taken of the others.

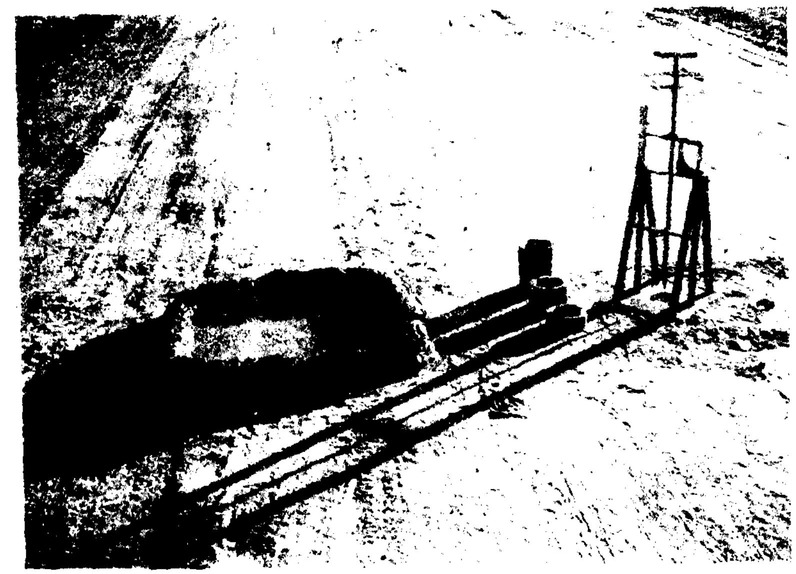
The presence of small pebbles (e.g. when moorum is used for the soil mix) within the volume of the soil may cause erratic results. A larger number of tests, 5 or 6 or more, may be necessary in such cases, from the results of which the average of the **lower** values is worked out.

2.6.2. The "soaking test," as developed by the Author, is conducted by constructing a prototype of the proposed base course of a thickness of 5 in. to 8 in. and measuring about 3 ft square. The density should be plus or minus 2 per cent of the Proctor standard density of the soil-mix.

This prototype is laid over a good, well compacted subgrade in the laboratory grounds or near to the field laboratory and is called a "Test Block."

After the "test block" is laid and the density checked, a small bund of clay is formed alround, 4 in. to 5 in. high. Burnt bricks or a perforated plank of wood is placed over the test block, and water is filled to a depth of 3 to 4 inches (Photograph 7).

This creates a small pond over the test block and permits complete saturation to be attained in a few hours. However, the time prescribed for complete saturation is three days, during which period the 'pond' is kept full making up from time to time the shortage caused by the seepage and evaporation of water.



Photograph 7.

Ponding for the Soaking Test in the Madras Method of Road-Bed Design. On the right is a patch already ponded and soaked, and a Cone Penetrometer set up for Bearing Test. (Para. 2.6.2.)

After the three-day soaking, the 'pond' and the bricks are removed. Usually a top layer of loose soil ¼ to ½ in. thick may be noticed due to the slaking action resulting from ineffective confinement at the top. This loose skin is carefully removed by a thin trowel and the bearing power of the soaked soil determined by the Cone-Penetrometer. Samples of the saturated soil are then taken near the test points and the **total moisture** determined by oven drying at 105 to 110 deg. C.

2.6.3. Table 5 shows typical results obtained in the laboratory during the design stage, as well as the results measured actually on the road under waterlogged conditions. One of the tests shown in Table 5 relates to the 1949-Series Tests at the Alipore Test Track. The other examples shown are typical of the results observed in Madras State. The soaking test is used in the laboratory as a design test and on the field as a control test to check the bearing power after construction.

TABLE·5. SOAKING TEST– SOME EXAMPLES.

Showing Typical Cases of Laboratory Results at The Design Stage, And Actual Moisture Contents on The Road Subjected to Prolonged Water-logging.

Location of Road	Type of Soil-Mix And Use on the Road	Laboratory Results. Spec. Gr. of The Particle (Sg)	Plastic Limit of Binder Soil	Max. Proctor Density.	Pore Space (V%), b. Vol.	Theoretical Water Capacity ⊗ By Wt.	Moisture in Laboratory Soaking Test.	On The Road. Moisture Observed on the Road after prolonged Water-logged Conditions
			%	lb.	%	%	%	%
Ramnad-Mandapam	Sand-Clay Sub-Base	2·60	17	118	28	14·9	11·8-12·8	10·9 – 12·9
T.P.P. Road Madras.	Sand-Clay Base.	2·65	15	128	23	11·2	9·5–11·5	8·0 – 9·0
Southern Trunk Road (Madras)	Moorum Sand-Mix Sub-Base for C.C. Road.	2·60	14·5	133·5	20	9·6	9·0-9·5	8·5
Bapatla Road. (Madras).	Moorum - Sand Base.	2·60	16	123	24	12·1	11·5-12·5	8·0
Majherat Test-Track	Sand-Clay Base	2·65	21	127	23·5	11·6	10·9	10·0
(W-Bengal).	Sand-Clay Sub-Base.	2·65	21	121	26·5	13·6	18·0	12·0 to 18·0

⊗ Theoretical Water Capacity $= \dfrac{V \times 100}{(100-V)S_G}$ %

2.6.4. As the governing factor is the bearing strength after full soaking, any soil which may utterly fail under this test is to be discarded or it should be improved by compaction or by a suitable admixture. Even if the location is a dry area and water-logging may not be serious, the soil mix and the subgrade must be tested after full soaking, the only exception to this being sub-grades of sandy soil.

A suitable weightage, up to 100 per cent, may be allowed only if the soil is good enough to show at least a small bearing value after full saturation. This specification provides for the possible condition that might occur due to sudden cloud-bursts even in dry areas: and water percolating through the porous macadam surface may saturate the base and the subgrade.

2.6.5. Samples of available soils are obtained and alternative design mixes worked out within the prescribed limits of

plasticity index. The mix which shows the maximum strength is usually selected. However, economic considerations may sometimes indicate the selection of a mix other than the best as regards strength.

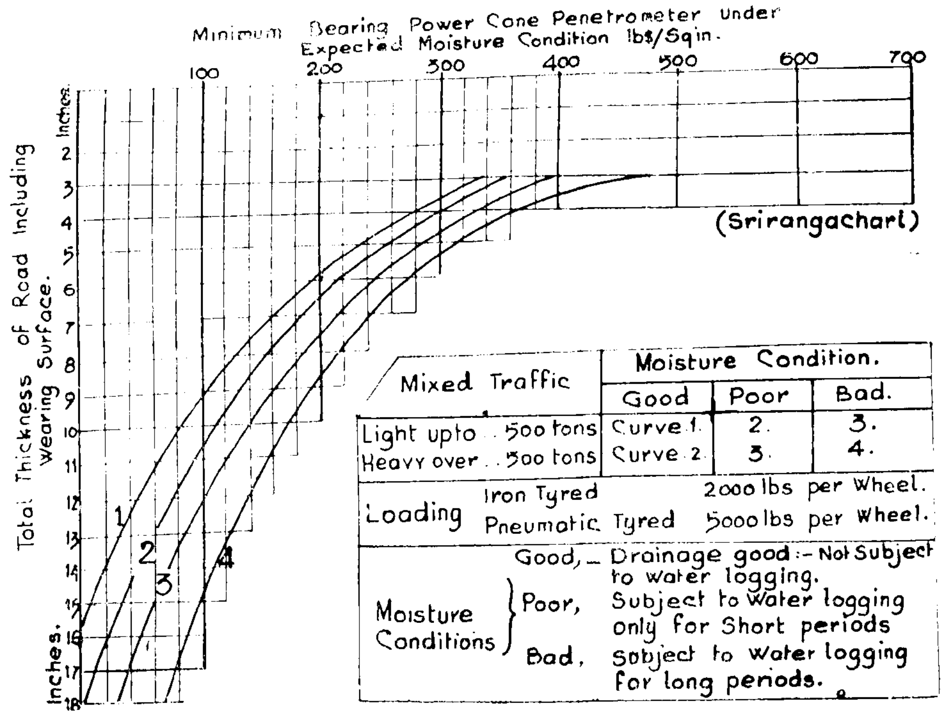


FIG. 14.

TENTATIVE DESIGN CURVES FOR FLEXIBLE PAVEMENTS

The thickness of the several layers is worked out by reference to the design curves given in Fig. 14. The following examples are taken from actual problems in the Madras and Bengal States and will serve to illustrate the general procedure for design and for fixing up specifications and controls.

Example (1):

(a) **Problem**.

2.6.6. A major District Road recently formed in black cotton soil area required to be surfaced to have a total traffic width of 16 feet. Traffic was 'light' and was expected to consist of about 100 tons per day of bus and car traffic and 300 tons of bullock-cart traffic.

The subgrade being newly formed had not yet attained full compaction: the density at the top 12 in., was only 75 lb per cu. ft. Moisture conditions "bad", subject to waterlogging for long periods.

Stone was very costly for soling. Locally available materials were moorum and sand. Clay-soil suitable for pulverising was not available.

(b) **Proposed Solution**:

2.6.7. The formation width required for a traffic width of 16 ft was 26 ft. The subgrade was to be made up to this width wherever deficient. The subgrade was then to be compacted to at least 80 lb per cu. ft. density for the top 12 inches.

The bearing power of the subgrade, compacted to 80 lb density and containing moisture 10 per cent above its plastic limit, was 120 lb per sq. inch by the Cone Penetrometer.

A reference to the design chart (Fig. 14) showed that the design curve to be used was curve No. 3, which gave a total thickness of road-bed of 10½ inches.

The best stabilised soil mixture was 1 part of moorum to 3 parts of sand. P. I. = 4.5 per cent; maximum density = 128 lb per cu.ft.; bearing power in fully saturated condition was 300 lb per sq. inch.

Another reference to curve No. 3 against bearing power 300 lb per sq. in. showed that 4½ in. was the thickness of the macadam layer needed over the base course after compaction.

The base course, therefore, worked out to (10½−4½) = 6 inches. This might consist of a 2-in. thickness of sand-blanket compacted to about 112 lb per cu.ft. as a sub-base; and a 4-in. thickness of moorum-sand base course of 128 lb density.

(c) **Economics**

2.6.8. The total cost of the design described above was considered a little on the high side and it was necessary to reduce the total cost slightly by reducing the macadam to 3 in. compacted thickness. This requires a base course of 400 lb per sq. in. bearing strength.

Experience had shown that a moorum-sand base course, if allowed to be ironed out by pneumatic-tyred traffic for a short time after construction (during which period it should be kept more or less moist) and then cured by drying, had a final bearing strength of 400 to 450 lb per sq. in. in a soaked condition.

A reference to design curve No. 3 against 400 lb per sq. in. showed that a 3-in. thickness of macadam was adequate. The base course then might be 4½ in. thick and the sand-blanket of 3 in. compacted thickness.

The final design is shown in Fig. 15.

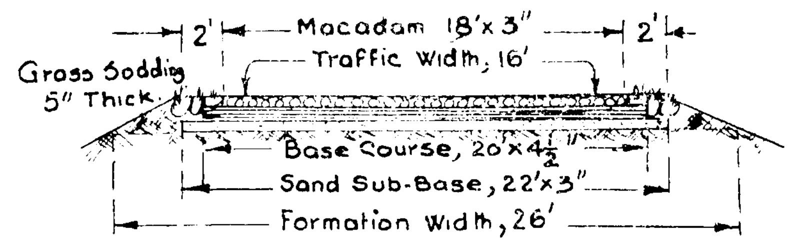


FIG. 15. DESIGN FOR MAJOR DT. ROAD.
In Black Cotton Soil Area - Traffic light; Moisture Condition 'Bad'.

(d) **Specification and Controls**:

2.6.9. The specification was then drafted in detail embodying the factors discussed above, particularly the requirement regarding traffic compaction and curing by subsequent drying out to acquire the maximum strength of the base course.

The controls should specify the limits within which the finished work may vary for each item. Detailed procedure should be worked out for the field controls, reporting of results, and action to be taken if the required standard was not attained.

Example (2):

(a) **Problem**:

2.6.10. The 1949 Series Tests at the Majherat (Alipore) Test Track were intended to test the value of stabilised soil as a foundation in a road-structure. The subgrade was to be newly formed 2 to 3 ft thick and "ponded" with water and, on drying, rolled with a power roller. The subgrade soil was silty-clay, a typical Bengal soil, with a P.I. of about 15 per cent, a plastic limit of about 21 per cent, and the maximum Proctor density of about 101 lb per cu.ft.

The stabilised soil was to consist of a mixture of the silty-clay soil and coarse sand available at 30 miles from Calcutta. The problem was to design the road-bed for heavy loading of the test, namely, a 2000-lb bullock-cart wheel load followed by a 6000 lb pneumatic-tyred wheel load. The test was to be made with the subgrade fully saturated and with water standing at 12 in. below road level on the sides of the Test Track.

(b) **Proposed Solution:**

2.6.11. The subgrade soil when compacted to its maximum density (101 lb per cu. ft.) and soaked for three days, was found to have **no** bearing strength at all, and the total moisture was 24 per cent which was three per cent above its plastic limit.

With a surcharge of about 12 in. above the subgrade the bearing power was 100 lb per sq. inch. The conditions of the test required design curve No. 4 to be used. A reference to curve No. 4 (Fig. 14) against bearing power 100 lb per sq. in. showed the required total thickness of road-bed as 15 inches.

Laboratory tests of bearing power (after soaking) of different mixes of the sand and the silty-clay yielded results shown in Fig. 16.

The foundation was obviously to consist of a base or top layer of 1 sand to 3 clay; and a sub-base less strong than the base. The sub-base could be of 1 sand to 1 clay which had a bearing strength of 180 lb per sq. in. after soaking. A reference to design curve No. 4 against 180 lb per sq.in. showed the thickness required above the sub-base as 9 inches.

The base course which had a bearing strength of 340 lb per sq. in. after soaking required a surface layer of $4\frac{1}{2}$ in. thickness. From this the thickness of the base course was found to be $(9 - 4\frac{1}{2}) = 4\frac{1}{2}$ inches. (See Fig. 17.)

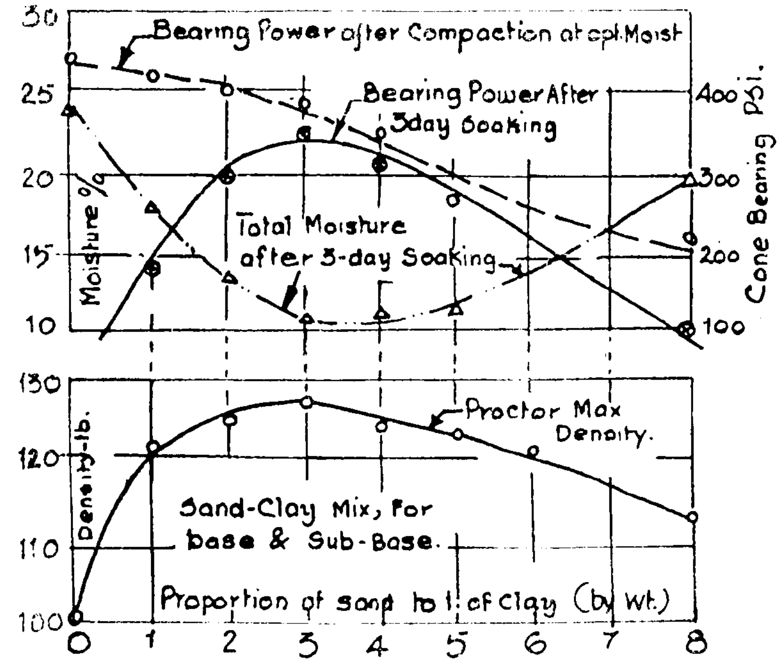


FIG.16. DESIGN CURVES FOR 1949-SERIES TEST AT ALIPORE TEST TRACK-(W. BENGAL)

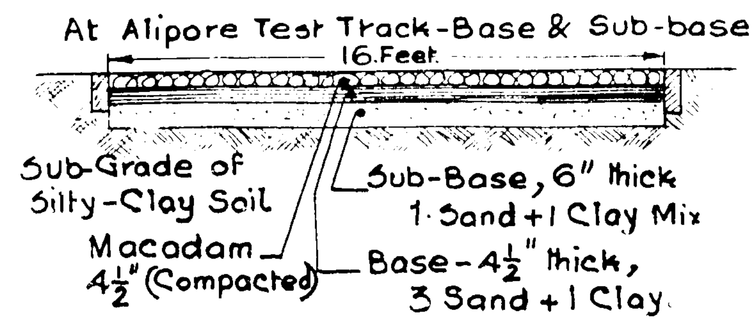


FIG. 17. DESIGN FOR '1949 SERIES TESTS'

(c) **Economics:**

2.6.12. As silty-clay soil was the prevailing soil in most parts of Bengal, the cost of the foundation layers was determined largely by the cost of transporting the required sand. For localities where sand was costly, the foundation could be in three layers as follows :-

Lower layer of sub-base of 1 sand to 2 clay, bearing strength 150 lb per sq.in. after soaking. This required $10\frac{1}{2}$ in. thickness over it (from curve No. 4). Therefore, this layer might be $(15 - 10\frac{1}{2}) = 4\frac{1}{2}$ in. thick.

Upper layer of sub-base of 1 sand to 1 clay as before, required above it a total thickness of 9 inches. If the macadam was $4\frac{1}{2}$ in. thick, the base would be $4\frac{1}{2}$ in. thick. That left only $1\frac{1}{2}$ in. for the sub-base.

If the sub-base of 1 sand to 1 clay was cured by drying before the base course was laid, its strength increased to over 200 lb per sq.in. after soaking. This permitted a base course 3 in. thick and the sub-base would then be 3 in. thick. The different layers would then be :

Lower layer of sub-base	– $4\frac{1}{2}$ in. thick of 1 sand to 2 clay
Upper layer of sub-base	– 3 in. thick of 1 sand to 1 clay
Base-course	– 3 in. thick of 3 sand to 1 clay
Macadam	– $4\frac{1}{2}$ in. thick
Total	15 in.

This design (Fig. 18) resulted in a saving of about 18 per cent in the total quantity of sand required as compared with the previous design solution.

(d) **Specifications and Controls:**

2.6.13. The specifications and field controls should be drafted with allowable tolerance to narrower limits than usual in view of the rapid decrease in bearing power with decrease in density of the silty clay soil. Particular attention was to be paid to the necessity for curing (by drying) of the sub-base before applying the base course. Pneumatic-tyred construction-vehicle might be encouraged to use the completed sub-base and the base course to improve their strength.

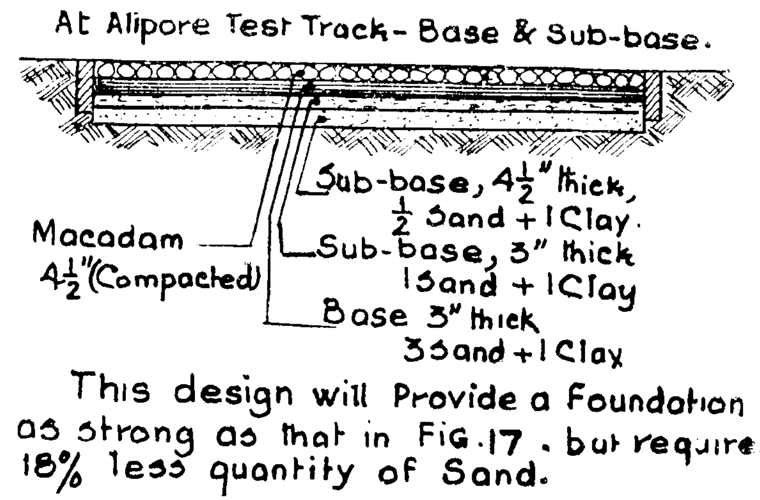


FIG. 18. ALTERNATIVE DESIGN, '1949 TESTS'

Example (3):

2.6.14. This example pertained to the case of a subgrade consisting of sandy soil or sand.

(a) **Problem** :

An "Other District Road" formed in fine sandy soil required to be surfaced to carry a total traffic of about 300 tons per day, including 50 tons bus and car traffic. The traffic width was 14 feet.

The subgrade had been under local traffic for over one year and had a density of about 95 pounds per cu. ft. The drainage was generally poor being subject to occasional waterlogging.

Medium sand and clay soil were available nearby. The clay soil having a P. I. of 20 per cent was easily pulverisable.

(b) **Proposed Solution:**

2.6.15. The formation width required was 24 feet. The sandy subgrade was to be scarified and compacted for about 6 in. depth to 105 lb per cu. ft., so that the bearing power by the Cone Penetrometer immediately after compaction was 150 to 180 lb per sq. inch. (The "soaking test" was not required for subgrades of sandy or loamy soil.)

The best stabilised soil mixture consisting of medium sand, fine sand (subgrade soil) and powdered clay had a density of 123 lb per cu. ft., and a bearing power of 250 lb per sq. in. after soaking for three days.

The design curve to be used was No. 2. Reading against bearing power 180 lb per sq. in. the total thickness required was 7 inches.

As the base course was subject only to occasional waterlogging, its bearing power under the expected worst conditions could be calculated from the soaked bearing power by allowing a weightage of 50 per cent, i.e. as 375 lb per sq. in. A reference to design curve No. 2 against 375 lb per sq. in. showed the required thickness of macadam as 2½ inches.

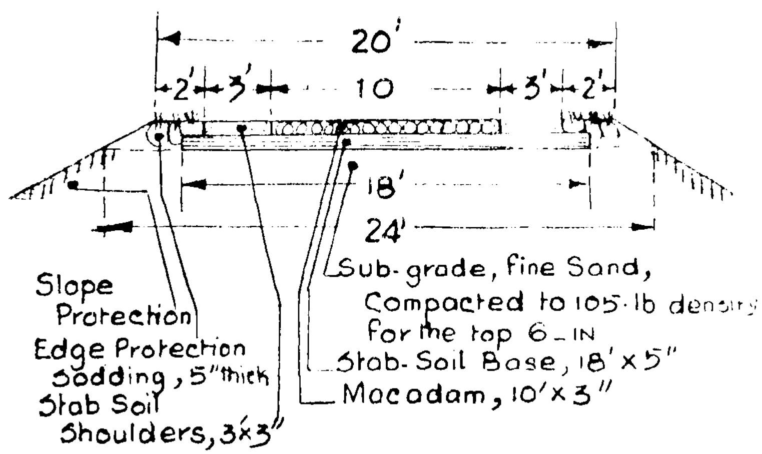


FIG. 19. DESIGN FOR A ROAD OVER SANDY SOIL

'Other District Road', Light Traffic, Drainage Poor

Therefore the base course had to be (7 − 2½) = 4½ or 5 inches. The final design could be macadam 10 ft × 3 in.; base course 18 ft × 5 in., and shoulders 3 ft × 3 in. (Fig. 19).

(c) **Economics:**

2.6.16. If the original cost of construction was required to be as low as possible, the thickness of the macadam could be 2½ in. provided the stone used was **not** of a refractory nature, e.g. granite, quartz, etc. The shoulders itself could consist of fine sandy soil (subgrade soil) well compacted at optimum moisture.

(d) **Specifications and Controls:**

2.6.17. The specifications should draw particular attention to the necessity for a renewal coat of macadam at the end of one or two years if the "economy design" is adopted as suggested above. The behaviour of mere fine sandy soil for the shoulders should be carefully watched under traffic; and if there should be any signs of distress at the junction with the macadam surface, prompt steps should have to be taken for re-doing the shoulders with stabilised soil of adequate strength.

CLASSIFIED SUBJECT INDEX

(The numbers refer to the bibliography)

General Information:
3, 10, 32

The Principal Variables:
1, 4, 32

Performance of Road-bed: Stability: Failure:
3, 7, 14, 20

Load: Repetition of Load:
2, 7, 19, 24, 34

Deformation:
7, 19, 30

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3, 11, 12, 15

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Shoulders:
3, 6, 29

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RESEARCH PUBLICATION

"CAUSEWAYS OR SUBMERSIBLE BRIDGES.*"

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SYNOPSIS

A "Causeway" is a submersible road bridge across a stream with the double function of allowing the normal dry weather flow to pass through the vents below the roadway, and the occasional floods both through the vents and over the roadway, thus entailing a temporary cessation of traffic. On account of their dual function causeways present problems peculiar to themselves and unlike other hydraulic structures like spillways, sluices, barrages, lifting weirs, and road bridges. This article deals with some hydraulic investigations in connection with causeways.

1. DEFINITION OF "CAUSEWAYS"

The word "Causeway", at least in India, denotes a submersible road bridge across a stream, which is designed and built in such a way that the normal dry weather flow of the river passes entirely through the vents below the roadway and the occasional floods pass both through the vents and over the roadway, thus entailing temporary cessation of traffic.

2. TYPES OF CAUSEWAYS

These causeways may have the roadway just a few feet above the river bed, say about the order of 5 ft when they are sometimes called LOW LEVEL CAUSEWAYS. Or, the roadway may be very high, up to say 25 or 30 ft above the bed, when they are called HIGH LEVEL CAUSEWAYS.

Secondly, the vents may consist of pipes or short-span culverts. Or, they may be constructed with arched vents of big spans, say of 25 or 30 feet.

Thirdly, the depth of water designed to flow over the road may be of the order of just a few feet or as high as 20 feet.

*Reprinted from the Annual Report of the Hyderabad Engineering Research Laboratories for 1950.

3. PECULIARITIES OF CAUSEWAYS

Among the hydraulic structures built across rivers, causeways have a peculiarity of their own. Unlike irrigation dams, spillways, barrages, anicuts, (or lifting weirs) and road bridges, causeways have a double function of passing normal discharges through the vents below the roadway and the flood discharges both through the vents and over the roadway itself.

On account of this dual function causeways present hydraulic problems which are peculiar to this kind of structures It is not known to what extent such submersible bridges are used in other parts of the world. At any rate, it has not been possible to get any information.

4. CAUSEWAYS INVESTIGATED AND THEIR HYDRAULIC FEATURES

The general problem of causeways was first brought to the notice of the Author in 1936, while he was Professor of Hydraulics in the Engineering College of the Osmania University. The Hyderabad Public Works Department had built several causeways in the State and during the rains of that year some failed in ways, the reasons for which could not be easily explained. The Chief Engineer for Roads and Buildings referred them to the Author for investigation in his Laboratory.

The first case taken up was the Aler Causeway typical of many, consisting of 2 ft diameter hume pipes concentrated in the centre of the river and built on sand foundation. The roadway was 4 ft above the bed and the depth of flood water above the causeway about 4½ ft. The next causeway that came up for investigation was the proposed Tintiny submersible bridge across the river Krishna, with a maximum discharge of about 6,00,000 cusecs. This was proposed to consist of 56 arched vents of 30 feet span. The roadway was 30 feet above the bed and the depth of flood over the roadway was to be 20 feet.

In 1947 there were some failures of causeways which were investigated in the Hyderabad Engineering Research Laboratories. The Peddavagu Causeway had 25 vents of 8 ft span, the height of roadway above the bed being 8.26 ft and the flood 11.55 ft above the roadway. The other one so far investigated is the Shahbazpalli Causeway with vents of 4 ft span, the height of roadway above the bed about 4 ft and the depth of flood above the causeway over 8.58 feet. This was placed in a very acute bend of the river.

The following is a statement of the Causeways investigated and their **hydraulic features:**

Features	Causeways			
	Aler	Tintiny	Peddavagu	Shahbazpalli
1	2	3	4	5
Maximum discharge in cusecs	50,000	6,00,000	1,00,000	68,000
Nature of vents	Hume pipes 2 ft dia.	Arched vents 30 ft span	Arched culverts 8 ft span	Culverts of 4 ft span
Location of vents	Middle	All along	All along	Towards the side
Height of roadway above the bed in feet	4	30	8.26	4.00
Depth of flood water above roadway in ft	4.56	20	11.55	8.58
Total flood water in ft	8.56	50	19.81	12.58
Year of investigation	1937–38	1938–39	1948	1949

5. NATURE OF CAUSEWAY PROBLEMS

The experience gained from these investigations has indicated that the three factors that contribute towards the causation of problems in causeways are :—

(i) the design of the causeway,

(ii) the configuration of the river including bed and banks,

(iii) the conditions of flood flow.

Experience also indicates that the causeway problems for purpose of study may be classified as follows :—

(1) **Effects of the design of the causeway**:

(a) on the river upstream of the causeway ;

(b) on the river downstream of the causeway ;

(c) on the river bed below the causeway ;

(d) on the flow of water inside the vents ;

(e) on the flow of water over the roadway ;

(2) **Effects of the conditions of flood flow:**

(a) when the flood is normal and rate of rise of flood is slow ;

(b) when the flood is abnormal and rate of rise of flood is rapid ;

(c) when the flood brings jungle trees and blocks the vents.

6. INVESTIGATIONS CARRIED OUT SO FAR

A resume of the investigations carried out so far will first be given before discussing the findings with respect to the problems as analysed above.

6.1. Investigations on Aler Causeway

6.1.1. **Problems.**—This causeway was built across the Aler river on the Hyderabad-Hanamkonda Road. The level portion of the causeway is 500 ft long and 4 ft above the bed. The openings for the dry weather flow consist of 20 hume pipes of 2 ft diameter, placed 2 ft 9 in. apart from centre to centre and with their bottoms at the bed level, so that the clear space between the top of the pipe and the road surface was 2 feet. The pipes occupying a total length of 61 ft were placed 169 ft from the right end, and 270 ft from the left end of the level portion.

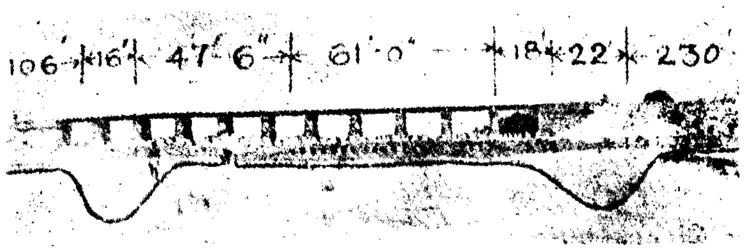
The bed consisted of pure sand, and the foundations for the retaining walls were taken to a depth of 4 ft on the upstream side and 6 ft on the downstream side.

In April and September of 1936 severe floods occurred, which according to observations rose to 8.56 ft upstream of the causeway and the flood level 500 ft downstream was 5.98 ft showing an afflux of 2.58 feet.

The significant features of the effects of the flood were :—

(1) On the upstream side of the causeway there was practically no scour.

(2) On the downstream side, while there was scour all along the causeway, it was significantly small immediately in front of the vents amounting to only 2 ft deep compared to 10 ft at some other places.



Photograph 1.
Aler Causeway—showing the approximate line of scour in black line.

(3) On either side of the vented portion the scours were more severe extending from 6 to 10 ft depth near the wall. Longitudinally they went up to about 100 ft on the right side and about 75 ft on the left. The deepest scours of 10 ft depth were spread over lengths of 16 and 22 ft on either side, and their distances from the end vents were 47 ft on the right and 18 feet on the left, that is to say, for 16 and 22 ft lengths the scour went down 4 ft below the foundation and the wall was overhanging in those places without any support (Photo 1).

6.1.2. **Analysis of the probable causes of the scour**:

The scours must have been caused either:

(1) by the subsoil flow, or

(2) by the surface flow, or

(3) by both.

The subsoil flow in permeable foundations may do damage to the structure in two ways: (1) by uplift, and (2) by the undermining of the soil.

Regarding uplift, no rift or raising of the floor has been observed either in Aler Causeway or in any other, so that this possibility may be left out of consideration. Regarding undermining the subsoil in the case of Aler Causeway is apparently homogeneous, and the structure in the foundations was symmetrical all along the level portion. It is, therefore, to be expected that the scour, if due to piping, must be uniform all along the downstream end of the causeway. But this was not the case, not only in this but also in other causeways of similar conditions. Severe scours did not occur all along, but only at the sides of the vented portion, the vented portion itself being significantly free from any high degree of scour.

It was, therefore, to be concluded that the scour was caused, not by the subsoil flow, but by the surface flow. This was confirmed by the preliminary investigations in which it was possible to reproduce in a model the conditions of scour corresponding to those in the prototype by means of surface flow only.

6.1.3. **Investigations.**—Investigations were made on a partial model of a causeway with three vents in the middle placed in a glass flume with sand up to the level of the bottom of the vents. Two rectangular vents were provided at the sides in such a way that the glass of the flume itself formed one of their sides and flow inside could be observed. These could be closed or opened as desired.

The water was made to flow first through the three vents only, the level upstream being just to the surface of the roadway. As shown in Fig. 1 (a), the experiment indicated that the scour at the sides was more than in front of the vents due to vortices formed by a high velocity jet from the vent flowing adjacent to region of still water at the side as shown in Fig. 1 (c).

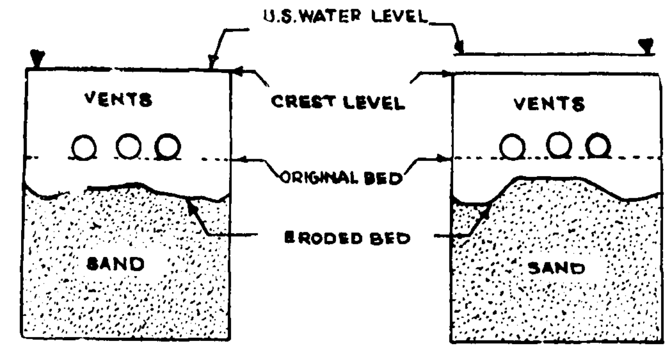
When the water was made to flow through the vents and over the model, the character of the scour remained the same but only intensified in degree as shown in Fig. 1 (b). It will be noticed that this characteristic of the the scour bears a strong resemblance to the actual method of scour in the prototype as shown in Photo No. 1. These experiments showed that the greatest scour at the sides was due to dead-water region.

In the next experiment, the dead-water region was removed by opening one of the side vents and making water flow up to the level of the causeway. This showed that on the side where the vent was opened there was no scour, Fig. 2 (a). When the water was made to flow above the causeway also, the same trend continued, Fig. 2 (b). When next both the side vents were opened, the scour at the sides was removed entirely, Fig. 2 (c). These investigations have shown that the peculiar characteristic of scour in the Aler Causeway was due to the concentration of vents in the middle, the moral being, of course, that in the design of the causeways the vents should be distributed throughout the length of the causeways.

Further studies were made to investigate conditions of flow when the level of the water was gradually increased over the causeway. When the water began to flow over in the addition to passing through the vent, the flow became rather complicated. To begin with at low head, when the nappe clings to the downstream side of the causeway, there is a little burrowing into the sand bed immediately next to the wall. But as soon as the nappe springs clear and falls at an angle on the sand through the water, the impact breaks the nappe into two vortices of roller as shown in Fig. 2 (d).

(a) Water Flowing through the Vents only.

(b) Water Flowing through the Vents and over the Model.



(c) Water Flowing through the Vents showing the formation of Vortices.

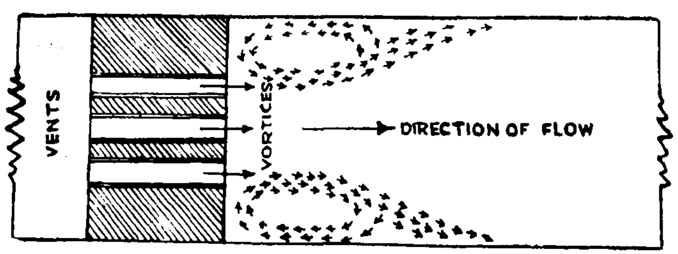
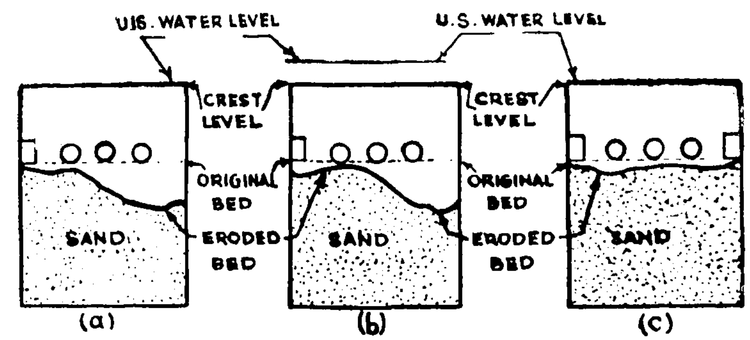


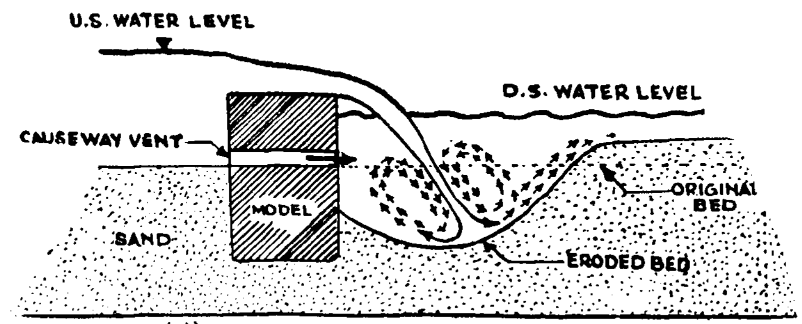
FIG. 1 – MODEL OF CAUSEWAY– *Side Vent Closed*

The forward roller lifts the sand at the line of impact and carries it forward, while the back roller tends to keep the sand up. As the sand is depleted from the zone of impact by the forward roller, the sand in the rear begins to fall down but is maintained in position for a while by the back roller moving upward till the sand surface become steeper and steeper and even gets beyond its angle of repose. There is a peculiar pulsation observed insuch movements, which seems to be periodic. There are move-

ments at intervals at which the back roller seems to stop functioning immediately the sand slips down and is carried forward by the forward roller. Thus, there is a continual sinking of the sand level next to the causeway. It was observed that for a flow of 3 in. above the surface of the model the scour went deep down to 7 in. and exposed the foundation in about 20 minutes and thereafter remained stable even after running the water for 45 minutes. This simulates the scour going below the foundations in the prototype.

CONDITIONS OF SCOUR WITH DIFFERENT OPENINGS UNDER VARIED CONDITIONS OF FLOW





(d) ROLLERS ON OVERFLOW

FIG. 2 – MODEL OF CAUSEWAY–*Side Vent Open.*

6.2. Investigations for reducing the period of Submergence of the Causeway:

In the rapid development of road construction in India the causeway or the submersible bridge is drawing increasingly greater attention of road engineers on account of its possibilities of providing a cheaper cross-drainage work than an insubmersible bridge. The period of submergence and the period of cessation of traffic over the bridge may not be a matter of great importance in out of the way places where time is not of much consequence. But on important roads some delay of traffic may be warranted by the savings effected by the construction of a submersible bridge but it behoves the road engineer to reduce this period as far as possible by suitable methods in the design of a causeway.

To ensure this what is desired to see that the expected flood passes through and over the causeway with as low an afflux as possible, leaving the road surface dry as quickly as possible for the passage of the traffic. In other words, it is necessary to devise methods by means of which the discharging capacity of the causeway for a given afflux is increased as much as possible. From this point of view the vent of the causeway may be looked upon as a short tube under submerged conditions and the top of the causeway or the road surface as a weir. On this basis, experiments were carried out to find methods of increasing the discharging capacity or the coefficient of discharge, both of the vent and the road surface.

6.2.1. **Increasing the coefficient of the vent.**— The coefficient of discharge of a vent of the kind used in causeways is of the order of about ·78 to ·80. The experiments have indicated that a provision of a bell-mouth on the upstream side increases the co-efficient to about ·88. In addition to this when a suitable divergence is given at the end of the pipe the coefficient rises to about 1·24.

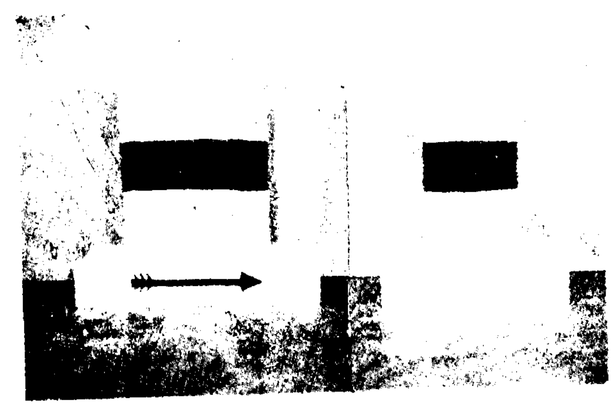
6.2.2. **Increasing the coefficient of discharge of the roadway.**— A broad crested weir with a sharp upstream edge corresponding to the top of the causeway has a coefficient of discharge of the order of about 2.94. Experiments in the Laboratories as well as elsewhere indicate that when upstream edge is rounded or bevelled the coefficient may be increased up to about 3.23.

6.2.3. **Streamlined Guardstone.**— At present guardstones that are used on causeways are rectangular in plan with sides vertical, the top portion being curved towards innerside of the causeway. This shape naturally offers a considerable resistance to flow and increases the afflux. This can be reduced by making guardstones streamlined (as indicated in Photo 2.)

6.2.4. **One-way camber of the causeway towards downstream.**— A road generally is given a camber on both sides of the centre. Experiments have indicated that one-way camber towards the downstream have two advantages as shown in Fig. 3.

(a) The afflux is considerably reduced and there is no chance of a standing wave being formed, as in the case of a two-way camber.

(b) As this arrangement raises the level of the upstream edge, this contributes towards keeping the roadway dry up to a higher depth of causeway upstream.



Photograph 2.
" Streamlined " Guardstone.

6.3. Investigations on Tintiny Causeway

Studies on the model of the Tintiny causeway proved very revealing regarding the effects of slow and rapid raise of flood on the flow inside the vents.

The model was made to a scale of 1 : 36. In order to facilitate the observations of the character of flow inside the vents, the model was made to represent only half the pier, the vent side being fitted against a glass wall.

Manometric arrangements were made for the measurements of pressure at three sections, near the upstream, downstream and middle of the model.

For simulating normal floods with slow rise and abnormal floods with rapid rise the screw valve of the supplying pipe was given different number of turns to suit the flow determined after some trials.

To adjust the downstream levels accurately a special streamlined shutter was designed with vertical streamlined vanes with arrangements for adjusting them as in the case of guide blades in an inward flow turbine.

Observations of the rate of rise and for flow under both conditions are given in Figs. 4 and 5 respectively. The pressures obtained for different flows are shown in Fig. 6.

While model discharges of 0·93, 1·25, and 1·40 cusecs could be adjusted to give the same depth of water on the upstream measuring section, the discharge of 1·83 cusecs however could not be adjusted to the same level on account of want of sufficient height in the forebay. The results showed :

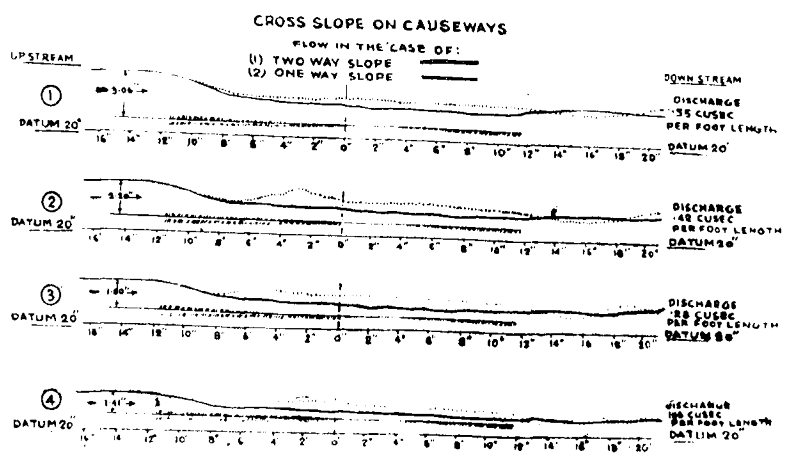


FIG. 3 - FLOW OVER CAUSEWAYS WITH ONE WAY & TWO WAY CAMBERS.

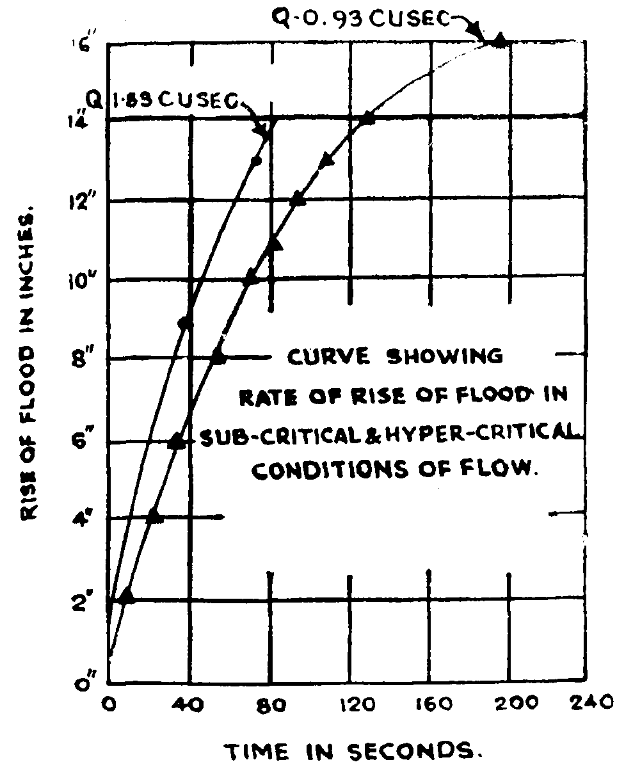


FIG. 4 - SUBMERSIBLE BRIDGE

(1) When the flood is normal and the rate of rise is slow the pressures inside the vents follow the hydrostatic pressure law. The velocities are below the critical.

(2) When the flood is heavy and the rate of rise is rapid, a standing wave is formed inside the vent. The velocities go above the critical and the pressures indicate that they do not follow the hydrostatic pressure law, but are considerably below. At very high floods, they go even below atmospheric pressure.

The consequence of the results of these investigations is, that for normal floods the arch has an upward hydrostatic thrust to compensate for the downward hydrostatic thrust above it. But in the hyper-critical stage the downward thrust may be far greater, in which case allowance has to be made in the design of

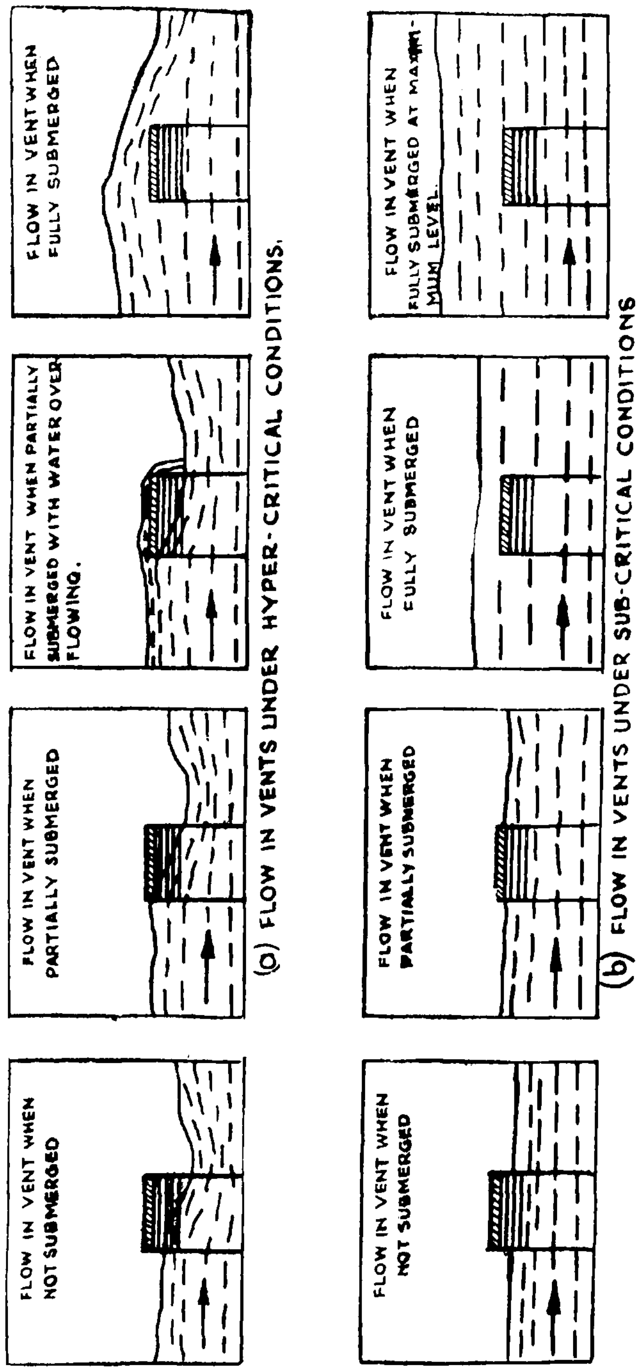


FIG. 5 - SUBMERSIBLE BRIDGE - *Flow under sub-critical & hyper-critical conditions.*

the arch and the piers. Each case may have to be determined by model experiments.

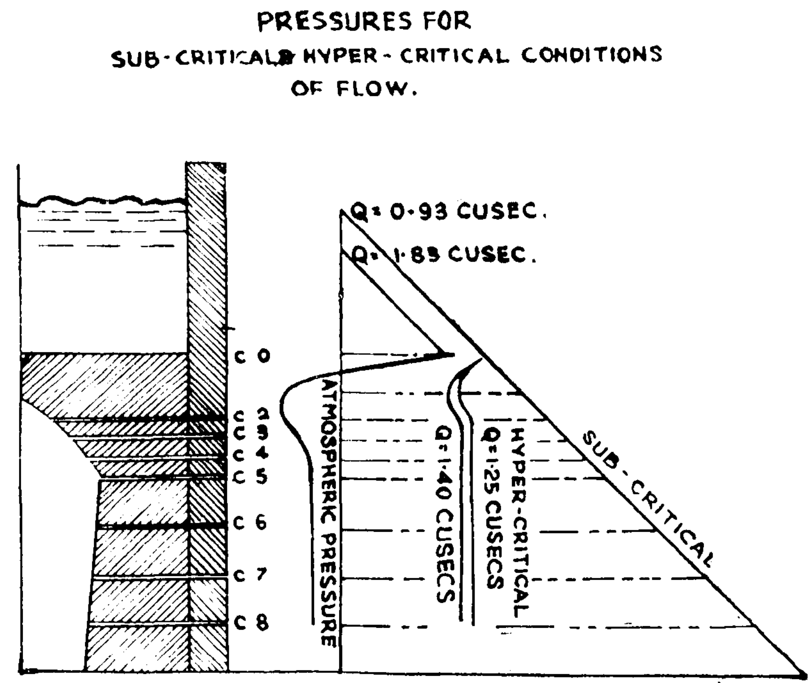


FIG. 6 - SUMERSIBLE BRIDGE - *Pressures for different flows.*

6.4. **Investigations on Shahbazpalli Causeway**

This causeway on the Karimnagar-Kamareddy Road in Hyderabad State had failed three times, since its construction about 40 years back and these failures are primarily due to the incorrect location of the causeway below an S-bend in the river.

The investigations of Mr. R. K. V. Narasimham show the importance of the effect of a bend on the upstream side of a causeway.

During heavy floods the high velocity jets would cling to the left concave bank due to centrifugal action and cause continuous erosion of the bank at the left approach ramp of the causeway. The presence of the causeway itself at this position is responsible for augmenting this erosive action. When the causeway failed in this manner in 1946 it was found that the river had cut into the bank to a distance of nearly 50 feet upstream of the causeway. The result of this erosion is that during heavy floods the foundations of the causeway and approach ramp get undermined and the causeways fail.

To safeguard this causeway from further failure, spurs had to be provided at the bend to deflect the flow. The location, length, and height of the spur were determined by investigations on scale models of the river and causeway. In the severe floods of September 1950 the spur has been effective in saving the causeway from further damage.

6.5. Investigations on Peddavagu Causeway

The Peddavagu Causeway on the Ramayanapet-Siddipet Road in the Hyderabad State is a case of failure of some of the central vents by getting blocked with trees and brushwood brought by the heavy floods.

This had an apron on the downstream side and none on the upstream side. The extreme vents on one side were intact while some on the other side were partially damaged. But the central vents had failed by deep scours on the upstream side resulting in the sinking of the upstream retaining walls. Investigations of Mr. R. K. V. Narasimham have indicated that this may be due to a transverse flow along its upstream side of the blocked vents, creating a deep scour along its upstream side of the blocked vents, creating a deep trench extending at places below the foundation of the retaining walls.

This indicates that for the emergency of the choking of vents by brushwood, etc., an apron on the upstream side is also to be recommended.

7. CONCLUSIONS

The above investigations have led to the following conclusions about the effects of causeways on the regime of rivers and the effects of the conditions of flood flow on the causeways :—

(1) For sub-critical conditions of flow during normal flood and slow rate of rise, the pressures in the vents follow hydrostatic law. As this produces an upward thrust on the arch, there is no special precaution needed in the design of the arch.

(2) For hyper-critical conditions of flow during heavy floods and rapid rate of rise, the pressures in the vents are less than hydrostatic and in extreme cases could

be even sub-atmospheric. The consequent change in the resultant thrust over the bridge must be taken into account in the design.

(3) The blocking of vents may produce transverse flow along the upstream side and cause heavy scour. For protection against trees and brushwood blocking the vents, an upstream apron will be necessary.

(4) If vents are concentrated in the centre, there will be heavy scour at the sides due to high velocity jets reacting on still water regions. The vents should therefore be distributed all along the causeways. Also the jets through the vents act as cushions against the energy of the nappe of water flowing over the causeway and thereby reduce the scour. If soil is loose, an apron is needed.

(5) When the causeway is placed immediately below a bend in the river, the erosive action on the concave bank is intensified and the approach ramp of the causeway attacked, while the silting on the other side is increased.

(6) On the downstream side the effect of the causeway on the regime of the river is more or less local extending to a distance depending upon the local phenomena of hydraulic drop and hydraulic jump.

(7) The period of submergence may be reduced by decreasing the hydraulic resistance and thereby lowering the afflux by the following methods :-

(a) increasing the coefficient of discharge of the vent by bell-mouth on the upstream side and divergence on the downstream side.

(b) increasing the coefficient of discharge of the roadway by rounding or bevelling the upstream edge of the causeway,

c) giving one way slope to the road surface,

(d) making the guardstones streamlined.

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A REPORT ON BRIDGES & BRIDGE ENGINEERING IN CANADA

By

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1. INTRODUCTION

1.1. The writer toured Canada for about 2 months (September and October 1951) as a member of the Indian Technical Mission on Highways & Bridges. (Photo 1). The tour was conducted by the Canadian Government in pursuance of the Colombo Plan for technical assistance to the Commonwealth countries. The tour programme was specially laid out to suit both the Indian Technical Mission and other similar mission from Pakistan. The two missions consisted of 8 engineers in all, drawn from various levels of service, but interested in highways generally.

1.2. The report is a brief record of the writer's impressions about bridges and bridge engineering in Canada, formed in the course of that tour, and is not the outcome of a special study of bridges in that country.

2. NORMAL ADMINISTRATIVE SET-UP OF THE BRIDGES BRANCH AS A PART OF THE HIGHWAY DEPARTMENTS

2.1. The organization of the provincial highways department varies somewhat from province to province, but generally the highways department in each province has a separate chief engineer for bridges, in addition to the chief engineer or chief engineers for construction and maintenance of roads. The work of the different chief engineers is co-ordinated and controlled generally by a deputy minister, which post is usually filled by promoting the seniormost chief engineer.

2.2. For instance, the present deputy minister in British Columbia was formerly a chief engineer for road construction, while his predecessor was formerly a chief bridge engineer.

2.3. Co-ordination and liaison between the bridges branch and the road construction branch was said to be excellent everywhere and no difficulty, whatsoever, was reported to have arisen so far due to the chief engineer looking after the alignment of a highway being different from the chief engineer looking after the

Photograph No. 1

THE INDIAN MISSION IN CANADA

A Four-member technical mission representing India's Ministry of Transport arrived in Montreal on the 28th August 1951 aboard a Trans-Canada Air Lines North Star. They were enroute to Edmonton to attend Colombo Plan conferences and to study construction, maintenance, and snow clearance operations on Canada's roads and bridges. Seen here from left to right are: G. H. KHAN, S. L. BAZAZ, M. N. BHUYAN and GOVERDHAN LAL. (Leader of the mission).

—*T.C.A. Photo.*

location of bridge sites or construction of bridges on the same highway.

2.4. The chief bridge engineer is generally assisted by an assistant bridge engineer (called in some provinces assistant chief bridge engineer), who again has under him a number of engineers of different grades – resident engineer, assistant engineer and bridge inspector.

2.5. The total number of engineers in the bridges branch is, however, much less than that in the road construction or road maintenance branches.

3. BRIDGES BRANCH AS A PART OF THE DEPARTMENT OF PUBLIC WORKS

3.1. In some provinces, the department of Highways is entrusted with the work of roads and culverts only while the bridges are a responsibility of the department of Public Works. This arrangement cannot be expected to work efficiently as it fails to recognise the basic fact that bridges are a part and parcel of the highway system. Lack of proper co-ordination between the two departments can lead to wasteful expenditure, and, if there is any waiting because of lack of proper co-ordination, (which in itself is annoying to the department more keen for the work) the result can easily be a slowing down of progress as indeed was noticed in one case by the Technical Mission.

4. RESPONSIBILITIES OF THE BRIDGES BRANCH

4.1. The bridges branch generally looks after the following types of structures :—

(a) Bridges having a span or total ventway greater than 16 ft.

(b) Culverts having an opening greater than 60 in.

(c) Grade separations.

(d) Railway overpasses and underpasses.

(e) River training and protective works and other works of river and flood control.

(f) Cattlepasses and cattlegates.

(g) Combined railway and highway bridges.

(h) Highway bridges over irrigation channels.

(i) Car ferries.

4.2. The bridges branch makes necessary inspections, surveys, site investigations and foundation explorations for the collection of hydraulic and other data for the design of bridges.

4.3. It also maintains the necessary equipment required for the construction and maintenance of bridges.

5. PROCEDURE FOR DESIGN AND CONSTRUCTION OF BRIDGES

5.1. The responsibility for the selection of a suitable bridge site rests upon the chief bridge engineer who makes necessary investigations for the purpose. He, however, takes the final decision in consultation with the chief engineer, highways. Outline designs for all bridges are invariably prepared by the bridges branch but if the bridge is going to be built in reinforced concrete, the design is usually worked out in complete detail by that branch.

5.2. In some cases, notably in the province of Alberta, fully detailed designs are worked out even for steel structures and the construction is also carried out departmentally. More generally, however, the outline designs are put out to tenders and the successful tenderer is required to prepare working drawings, satisfying the basic requirements incorporated in the outline designs or laid down in the specification. The tenderer is free to offer alternative designs of his own choice also; but whether he submits his own design or works out in detail the departmental design, the design calculations and working drawings have to satisfy the chief bridge engineer.

5.3. It is not unusual for the work of substructures to be let out to one contractor and the work of superstructure to another.

6. MACHINERY FOR CONSULTATION AND ADVICE

6.1. For designs of important bridges, eminent practising engineers are consulted and for very important bridges, *ad hoc* advisory boards of eminent engineers are set up to advise on the selection of site, preparation of plans and specifications, and indeed all matters relating to bridge design. In some cases, e.g. Montreal Harbour Bridge, the advisory board continues to function as an active body during the period of construction also. In such cases, the Board also gives final decisions in cases of disagreement between the contractors and the Engineer-in-charge of the work.

6.2. The constitution of the advisory board for the Montreal-South Shore Bridge may be mentioned as a typical case:

The General Manager of the Montreal Harbour.	—	Chairman.
President of the Dominion Bridge Company, Canada.		Members.
Dean of the Faculty of Engineering, McGill University.		
A Consulting Engineer of Montreal.		
Director of Public Works, City of Montreal.		
Chief Engineer, Department of Public Works of the Province.		

6.3. For outstanding bridges of the class of the Quebec Bridge, which at the time of its construction had the longest and the heaviest single span ever built, the advisory board is constituted with particular care. After the first disaster on this bridge, the board appointed for advising on the revised design consisted of an eminent bridge engineer from London and another from Chicago in addition to a Canadian engineer. Some further changes in the board were considered necessary later on and the board was extended to include two more eminent engineers from New York. After the designs were finalised, a new board was constituted to advise on and supervise the construction work.

7. RESPONSIBILITY OF CONTRACTORS—CONTRACT AGREEMENTS GENERALLY

7.1. Even when a bridge is built to plans and specifications prepared by the department or to plans prepared or approved by an advisory board, the contractor is required to satisfy himself as to the sufficiency and suitability of the design, plans, and specifications, and to undertake entire responsibility not only for the materials and construction of the bridge but also for the design, calculations, plans, and specifications. The contractor has, however, the right to submit alternative proposals when he disagrees with the Engineer-in-charge, but final decision rests with the advisory board where one has been set up, or with the department.

7.2. Agreements are drawn up on somewhat similar lines to contracts drawn up in this country. An important difference, however, is that although there is a time clause stipulating the period for the completion of the work, there is no penalty clause

to be invoked if the contractor fails to complete the work in the stipulated time. Similarly there is no clause for the payment of any bonus to the contractor if he finishes a rush job ahead of the stipulated time. It was said that the contractors are very reliable as a rule and that no difficulty arises in actual working in the absence of the penalty clause.

8. BRIDGE CODE

8.1. Canada does not have a Bridge Code of its own: the American Code prepared by the A.A.S.H.O. for highway bridges is followed generally.

8.2. Canadian bridge engineers were, however, very interested to learn that India has drafted its own Bridge Code with explanatory notes, and several chief engineers asked for copies of this Code to be sent to them. It is suggested that the Indian Roads Congress send some complimentary copies of the Code to the Director, Technical Assistance Service, for distribution to the provincial highway departments.

9. LOADING STANDARDS

9.1. All bridges on the main highways are generally designed for H_{20}–S_{16} loading which is about 20 per cent lighter than the I.R.C. Class A loading for spans up to 30 ft and about 15 per cent lighter in the range of spans from 30 ft to 100 ft. Unimportant bridges on District roads are designed for H_{15} loading which is roughly 15 per cent lighter than the I.R.C. Class B loading.

9.2. For very large bridges, special design loadings are evolved, one of the reasons being that the design is influenced considerably by erection methods and the weight of erection equipment required, which in turn are governed largely by the problems of transporting and loading of materials.

10. GEOMETRIC STANDARDS

10.1. The minimum clear road width on all large bridges on main highway is 24 ft, and on bridges in outlying districts 20 ft. On deck bridges, however, a minimum width of 26 ft is preferred. Over culverts as a rule, and on bridges less than 100 ft overall length generally, the width of roadway is kept equal to the shoulder to shoulder width of the approach roads. It is unusual to provide parapets on culverts, the length of the culverts and the design of wings or head walls being so arranged that no change is noticeable at the culvert in the earth slopes of the approach embankment.

10.2. Gradients on the structure of the bridge proper are quite common and gradients as steep as 4 per cent to 5 per cent are allowed even on very large bridges. On small and medium length bridges, the location and alignment of the bridge axis are governed to a large extent by traffic considerations. Very great care is taken to avoid bad curves in the approaches.

10.3. Where traffic conditions so require, a bridge may be skew and in addition have a gradient and/or a horizontal curve.

11. TRANS-CANADIAN HIGHWAY-STANDARDS FOR STRUCTURES; FINANCIAL RESPONSIBILITY; & CONTROL OF THE CENTRE

11.1. The Federal Government's concept of the Trans-Canadian Highway is not that of a super-highway for high speed traffic from coast to coast: it is rather that of a "first-class low surfaced road, which should follow the shortest practical east-west route across Canada and through each of the provinces, consistent with the needs of the provinces and of Canada as a whole."

11.2. The total length of the highway when completed will be about 5,000 miles. As long lengths of road forming part of this highway already exist and as the various provinces through which the highway runs have in the past independently developed their own standards, which apparently meet their needs adequately, the Federal Government does not attempt to impose upon the provinces higher standards than are necessary for the needs of traffic. At the same time it is realised that the highway has to be built not for today but for the future, and that the standards should not be so low that the highway becomes obsolete in a few years' time.

11.3. The design standards usually suggested include the following :—

Loading standard on all bridges ...	$H_{20}-S_{16}$
Minimum overhead clearance on bridges ...	14 ft
Minimum clear road width over bridges or culverts of 30 ft length or less	Equal to full formation width (paving plus shoulders)
-do- -do- over 30 ft length, up to 100 ft ...	27 ft
-do- -do- over 100 ft length ...	24 ft
Maximum gradient ...	6%
Minimum sight distance, horizontal & vertical	600 ft
Maximum curvature ...	6 degrees.

11.4. The contribution of the Centre towards the cost of completing the highway is regulated by an agreement between the Centre and each Province. Under the terms of this agreement the Federal Government undertakes to pay one-half of the cost of all new work i.e. every mile of new road, every new bridge and culvert, and generally all new work necessary for improving existing roads and drainage structures. The Federal Government will not, however, pay for land and right-of-way costs. The federal contribution is estimated to be about $150 million.

11.5. There are not many large unbridged streams on these highways and the bridge works required are estimated roughly to cost about $15 million only.

11.6. Under the agreement, the form of advertisement for tenders, the tender forms, the specifications, and the plans and estimates for all works on the highway require the approval of the Federal Government. The tenders when received and the specifications proposed are subject to review by the Federal Government who also inspect work on the highway and audit the expenditure. But the highway when completed remains the property and responsibility of the province concerned.

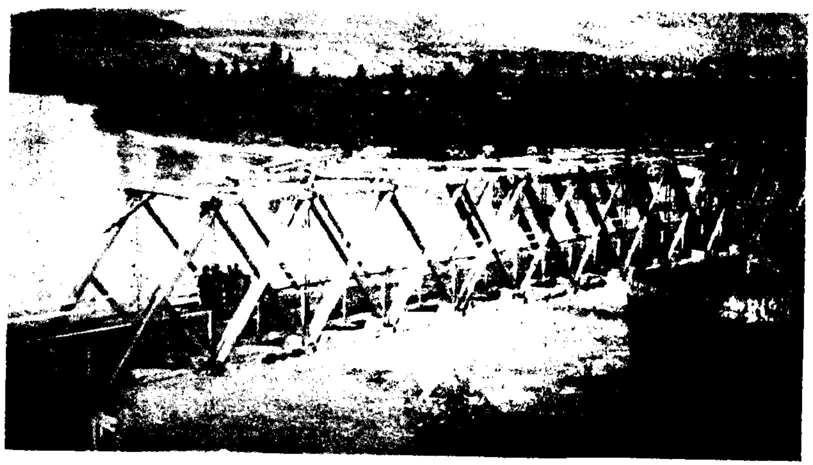
12. TYPES OF STRUCTURES AND STRUCTURAL MATERIALS COMMONLY EMPLOYED

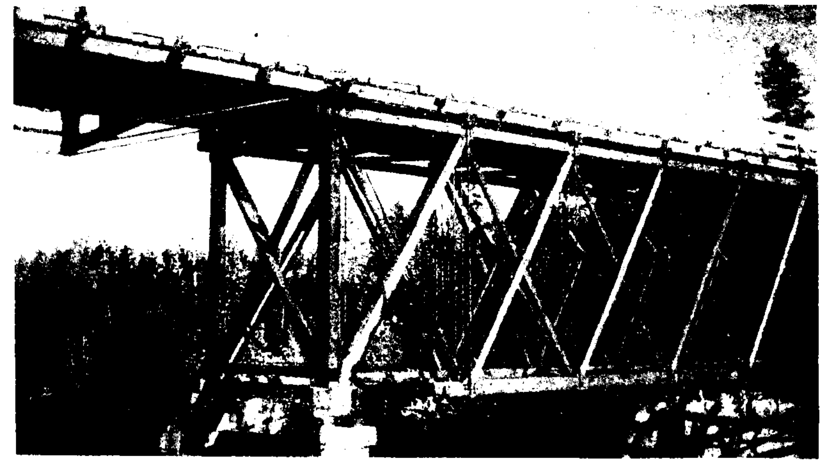
12.1. In the past, timber has been the structural material most commonly used for short and medium spans. Timber pile bridges, culverts and cattle passes have been largely standardised. Timber is used quite economically for trestles, cribs, piers, and retaining walls. Practice varies as regards the use of timber in its natural state, or with surface treatment, or with depth treatment. Generally, however, cedar has commonly been used in the past without any treatment; pine, spruce, hemlock, and Douglas fir with surface treatment: and maple, birch, elm, and oak with depth treatment.

12.2. King rod trusses in timber are in use upto 70 ft span, and Howe trusses upto 160 ft span. They are both of the through type and the deck type. Such large span trusses are, however, not all-timber structures; they employ steel for the tension members and steel castings for joints, the cost of steel used in the completed structure being of the order of 40 per cent of the total cost. (Photographs 2, 3 and 4).

12.3. Large spans are almost exclusively built in steel. Steel structures take the form of the conventional plate girders

Photograph No. 2 (above) and 3 (below):
Two views of the timber Howe Truss Bridge, over the Columbia River, near Canal Flats, British Columbia, (Para. 12.2.)





Photograph No. 4
A deck type Howe Truss Bridge. (Para. 12.2.)

upto 100 ft span (bigger spans have been built with welded construction) and open web girders for spans upto about 400 ft. For still larger spans, cantilever and suspended span bridges or suspension bridges are almost the only types employed, an occasional exception being a steel arch.

12.4. Recently, a very remarkable steel arch of 480 ft span has been constructed over the Chandiere river on the south approach to the Quebec Bridge. The deck is 33 ft wide at an elevation of 140 ft above the river.

12.5. Bailey Bridges have been used at some places as semi-permanent structures but their principal field of use has been as temporary structures to accommodate traffic while a new bridge is under construction or an old one is being replaced.

12.6. Reinforced concrete is coming more and more into use. Simply supported solid slabs are favoured upto about 30 ft span and T-beams and slabs beyond that span upto about 75 ft. Continuous solid slab spans upto 50 ft are in use. Rigid frame structures upto 60 ft span are very common. Here and there a large span structure in reinforced concrete is also visible, an example being the Pont Vieu, having arch ribs of 350 ft span. (Photo 5.)

Photograph No. 5
Pont Vieu, Quebec. (Para. 12.6.)

12.7. A large bridge, employing a central arch span of 290 ft has been built entirely in aluminium. This bridge is described in some detail in paragraph 17 and Appendix II.

13. FERRY CROSSINGS

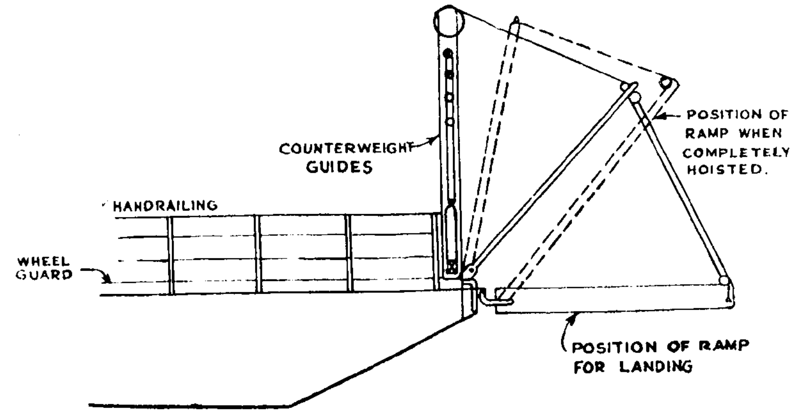
13.1. Major ferries, as a rule, are owned and operated by Government. Ferries of several types and sizes are in use. Examples are:

Arrow Park Ferry:—This consists of a steel barge 70 ft long, beam 24 ft, depth 46 inches, with vehicle ramps 12 ft 9 in. wide and 14 ft 6 in. long overall, hinged as cant frames at each end (as shown in sketch).

Francois Lake Ferry:—Steel barge 116 ft long, 32 ft wide, draught 5 feet, single-ended twin screw, capacity 16 carts and 120 passengers.

Nelson Ferry:—Steel barge 122 ft long, beam 43 ft 8 in., depth 5 ft with power cable machinery. Draught 1 ft 6 in. when empty and 1 ft 8 in. when fully loaded. Capacity 28 cars.

Kootenay Lake Ferry:—This ferry is among the largest ferries in use and is described in some detail here.



13.2. Steel barge 182 ft long, 48 ft beam and 11 ft 6 in. in depth. The draught when unloaded is 5 ft and when fully loaded, 6 ft 6 in. The approach pontoon is 41 ft 6 in. × 43 ft and has a small hinged apron which bridges the narrow gap of 2 ft between the pontoon and the end of the ferry.

13.3. The ferry has a capacity for 50 cars at a time and makes 7 trips every day. The distance between the two landings is $5\frac{3}{4}$ miles. The annual range of variation in the water level of the lake is 22 ft, so that the use of an ordinary landing ramp is out of the question.

13.4. Plate I illustrates the details of the landing used at the Balfour end.

[The plan in Plate I does not indicate the position of dolphins, each of which is formed from a cluster of 20 to 30 piles, each 60 ft to 65 ft long. The piles forming the guide bents which keep the pontoon in position are 50 ft to 60 ft long.]

14. CURRENT TRENDS IN THE DESIGN OF STRUCTURES

14.1. **General**. The deck type is preferred to the through type, and not without reason, for the ideal highway bridge is a deck structure having the same width as the highway of which it forms a part and having unrestricted over-head clearance.

14.2. **Steel Structures**. There has been a great development in the design and construction of welded plate girder type bridges and several large spans have already been built, the largest having 180 ft span continuous girders. The factors which have influenced this development are discussed further in paragraph 15.

14.2.1. Where welding is used, shop inspection is very strict. For field splices, both welding and riveting are in use, very careful supervision being exercised over field welds. Complete X-ray outfits are kept for testing of welds and a large number of X-ray photographs of welded joints are taken. In addition, some of the welded pieces are subjected to tensile tests to destruction. The use of supersonic test equipment is also under consideration.

14.2.2. From small spans, employing R. S. Js., to medium spans employing plate web girders, welded or riveted, reinforced concrete roadway decks are favoured.

14.2.3. It is becoming increasingly common to encase the top flanges of the R. S. Js. in the concrete decking, so as to ensure composite action between concrete and steel.

14.2.4. Among shapes and types of structures, going into disfavour, may be mentioned the bow-string girders, for which there was no enthusiasm anywhere. For movable bridges, the bascule type is going out of use and lift type becoming more common.

14.2.5. Ordinary carbon steel is still the most frequently used type of steel, although alloy steels, notably nickel steel and silicon steel, are employed on structures of very large spans, wherever the resulting saving in dead weight justifies their use.

14.3. **Reinforced Concrete Structures**. Prestressed concrete has not yet come into vogue. Rigid frame structures with a very pleasing soffit line are coming more and more into use for spans upto 60 ft. On new highways e.g. the Barrie highway and the Elizabeth Regina highway, rigid frame structures are used to the exclusion of all other types, for grade separations (Photo 6). On the Barrie highway rigid frame spans upto 100 ft have been used, but structures of such large spans are, strictly speaking, not made of reinforced concrete: they are formed of shallow structural steel trusses, encased in concrete.

14.3.1. The continuous girder type with an arched soffit is quite common. In some cases continuity is carried to 6 spans of 100 ft or so. Balanced cantilevers with suspended spans are not used, apparently because subsoil conditions under the foundations

Photograph No. 6
A Grade Separation on the Elizabeth Regina Highway.
(R. C. Rigid Frame Structure) (Para. 14. 3.)

do not call for such designs—Unyielding strata are frequently met with at the bed or at a short depth below the bed.

14.3.2. Thin reinforced concrete webs with wide flanges at the bottom are not used, although on steel structures, webs which would be considered extremely thin by conventional standards, are in common use.

14.3.3. Bowstring girders are disliked for various reasons, the chief being,

(i) they are considered more costly than other types, and

(ii) they do not lend themselves to future widening of the roadway over-bridge.

The second reason was considered important as Canada is a rapidly developing country and has had to widen some of its bridges, built not many years back.

14.3.4. The use of precast slabs is also being developed.

14.4. **Timber Structures**. Timber is perhaps going out of fashion and is now rarely used on new bridges of any importance. Its use on new structures is restricted largely to temporary bridging. Replacements of old and worn out timber bridges are in most cases carried out in reinforced concrete or steel as a matter of long range economy and not under the belief that the timber bridge is an anachronism.

14.4.1. In some cases however a new timber bridge has been built in place of the old one, though with a slightly wider roadway or a somewhat higher loading, if necessary. There are, however, thousands of timber bridges in active use and their replacement by steel or concrete is not thought of. Apart from the enormous cost involved in such replacement, it would indeed be foolish to attempt it.

14.4.2. Where timber is now used, it is generally considered preferable to incur some extra expenditure on treatments with suitable preservatives and thereby increase the life of the structure than to use timber in its natural state, which means low initial cost but more frequent and therefore costly repairs or replacements.

14.4.3. The use of split ring connectors, where several members meet in a timber structure, is becoming common. Timber bridges with a reinforced concrete wearing surface keyed on to a creosoted timber-laminate decking, thus forming a composite timber-concrete structure, are finding much favour in British Columbia.

14.5. **Aluminium Structures**. Although Canada has built the first all-Aluminium arch bridge in the world, (see para. 17), there is very little evidence of the use of aluminium and aluminium alloy on bridge works generally as an alternative to mild steel.

14.5.1. The raw material for the manufacture of aluminium has to be imported and consequently the initial cost of aluminium manufacture is high. While the aluminium is finding its use in other fields, it seems unlikely that its use will extend to normal bridge building programmes. There is only one type of bridge i.e. the movable, whether of the bascule or the lift type, for which aluminium is specially suited, on account of its light weight and it is possible that in future, aluminium is employed on such bridges.

14.5.2. Aluminium does not seem to have come into favour even for light weight bridge deckings, a field of use which has found some favour in the United States of America.

14.6. **Foundation works generally.**

14.6.1 Where foundations have to be laid in deep standing water, and the depth of rock or other suitable strata for foundations is great, special designs are evolved for deep water piers, as

the normal pier design involving pneumatic caisson sinking and cofferdam work with dewatering operations is considered both expensive and hazardous.

14.6.2. The design which is coming into use consists of steel H-piles driven inside a steel sheet pile formwork and " Prepakt " concrete placed round the H-piles, without dewatering the cofferdams. This design has already been employed on the following three bridges recently completed :

(1) **Stave River Bridge** on Lougheed highway in British Columbia (The depth of water averaged 25 to 28 ft).

(2) **Bridge over the Vedder Canal**, British Columbia.

(3) **Bridge over the North-west Miramachi River** in New Brunswick (This bridge is 1660 ft long, with main spans of 287 ft. The deep water pier goes to rock 130 ft below high water level and the maximum depth of water is 34 ft at high tide.)

14.6.3. A special feature of these bridge contracts was the inclusion in the specifications of a definite stipulation that the piers shall be constructed by the " Prepakt " method.

14.6.4. **Prepakt Concrete**. The method essentially consists of the following operations. A steel sheet piling cofferdam is formed around the periphery of the pier base, going down a few feet below the bed to a firm stratum, and sand or other loose material is dredged out. The cofferdam serves as a form for the concrete in the pier base and is left in place. Steel H-piles are driven within the cofferdam going down to hard rock. Selected, graded coarse aggregate is deposited around the H-piles, necessary grout pipes and reinforcement bars, if any, being placed in position before depositing the aggregate.

14.6.5. A special grout consisting of cement, sand, and water with some proprietary admixtres to prevent early hardening of the grout and to hold the solids in suspension, is then pumped under pressure, without dewatering the cofferdam, to form " Prepakt " concrete.

15. WELDED STEEL STRUCTURES

15.1. A large number of welded plate girder bridges have been built of varying lengths and spans. The longest bridge of this type is perhaps that at St. Rose, which is 1,548 ft long, with 2 spans of 79 ft and 8 spans of 108 ft each, while the longest spans employed are those on the Duplessis bridge, which consists of 2 spans of 150 ft and 6 of 180 ft each.

15.2. The basic pattern of most of these bridges is the same, the common features being :-

(1) Each span has two main girders, their spacing being so arranged in relation to the overall road width and the overhang of floor beams, that the positive and negative bending moments in the floor beams are nearly equal.

(2) The main girders are continuous from abutment to abutment, being supported on roller bearings on all piers excepting the central pier where fixed bearings are provided.

(3) The concreted decking is continuous from abutment to abutment and is linked on to the floor beams by means of suitable tension straps and brackets arranged on the floor beams.

(4) Suitably spaced knee braces, welded to the girders and floor beams and forming rigid frame structures with the floor beams, serve as stiffeners.

(5) At the piers and abutments, deep welded sway girders replace the floor beams.

15.3. It is claimed that there is an overall saving of 28 per cent in the girder material, on such spans due to the combination of welding and continuity. In addition, the simplification of design results in further savings on the cost of fabrication and erection. Again, the design enables substantial economies to be made on the design of piers.

15.4. The use of a number of thin flange plates, fillet-welded together, has been completely discarded in favour of a single thick plate. This makes welding easy, reduces stresses in the web-to-flange welding zone, and facilitates inspection of the finished work by radiography.

15.5. On some of these bridges, a very notable departure was made from the conventional practice of limiting the ratio of depth of girders to web thickness to about 170. The corresponding ratio on the Duplessis Bridge (part of which collapsed after about 4 years of service) was 370, a web thickness of only 3/8 in. being used on a girder of 12 ft depth. The Duplessis Bridge is described in paragraph 19.

15.6. On short-span bridges having gradients either on the bridge itself or on the approaches and thus requiring vertical transition curves, the difficulty of providing curved flanges is

easily solved by welding. Welding is similarly useful when curved soffits are provided as on continuous spans. Again, welding is extremely useful on skew bridges which require some awkward connections if the normal riveted design is adopted.

16. NEW BRIDGE CONSTRUCTION

16.1. During the period under report, there was very little new bridge construction in hand, although there was considerable activity in the designs offices. Some very interesting designs were being worked out departmentally by the bridge engineers of Alberta, British Columbia, Quebec and Ontario.

16.2. Apart from some minor bridges and some rigid frame reinforced concrete structures, the only bridge constructions of importance that were going on at that time were :-

(1) **The Rock Creek Bridge**. This bridge lies on the Trail-Penticton highway in British Columbia and provides a very good example of current Canadian practice regarding the erection of steel bridges across deep gorges. The erection of this bridge is described in Appendix I.

(2) **The McKenzie King Bridge** at Ottawa, also known as the bridge over the Rideau canal. This bridge is partly a steel structure and partly reinforced concrete. The decision is conventional.

Some detailed drawings of these bridges supplied kindly by the Department of Highways, Ontario and British Columbia may be seen in the office of the Consulting Engineer (Roads), New Delhi.

17. THE ARVIDA BRIDGE

(All-Aluminium Structure)

17.1. This bridge has formed the subject of several articles in technical journals but a brief note on the interesting features of this bridge would not be out of place in this report and is, therefore, added as Appendix II.

18. CANADIAN PRACTICE ON SOME POINTS OF STRUCTURAL DETAIL

18.1. **Reinforced Concrete Decking Slab.—** Two-way reinforcement top and bottom, is favoured, as the labour involved in bending of bars is costly. Usually 5/8 in. bars are used for stiffness to prevent displacement of bars during pouring of concrete. This design of reinforcement ensures that the slab does not develop any cracks even when used on steel truss spans, where the supports of the decking are relatively more flexible and there is greater vibration than on spans with reinforced concrete main girders.

18.2. **Cement Concrete Wearing Coats**. Practice varies in different provinces as regards laying the wearing coat separately from or integrally with the structural slab. Where the wearing coat is laid integrally with the slab, only 1 in. extra thickness of cement concrete is added, but the design allows for an additional dead load due to an extra thickness of a-2 in. layer of asphaltic concrete which may have to be laid when the 1-in. wearing coat actually shows signs of wear.

18.3. **Light weight floors**. Various proprietary types of open grid floor are in use, e.g. Irving, Trilock and Teegrid.

18.3.1. **The Irving type** has a diamond shaped open mesh and weight only $15\frac{1}{2}$ lb per sft.

18.3.2. **The Trilock type** also provides an open mesh usually of 2 in. × 3 in. also weighing about 15 lb per sft.

18.3.3. **The Teegrid type** consists of 3 in. × 3 in. T irons, placed in contact with each other, with stems upwards, the flanges being welded together. The spaces between the stems are filled with concrete forming an armoured non-skid surface. The complete flooring with concrete filling weighs only 52 lb per sft. as against about 125 lb per sft. for the usual $7\frac{1}{2}$ in. reinforced concrete slab with 3 in. wearing coat. The material of the grid is medium carbon steel with a copper content varying from 0·2 per cent to 0·3 per cent.

18.3.4. Another type of light weight floor is the **Battleship Deck**, which consists of a thin steel plate rigidly fastened over the stringers. A steel grid having a depth of about $1\frac{1}{2}$ in. is welded to the steel plates and the grid is filled with an asphaltic mixture.

18.4. **Asphalt planks**. Asphalt planks consists of preformed mixtures of refined asphalt (or a combination of refined asphalt and flux) of a suitable penetration, organic fibre, and a mineral filler. They are available in various sizes, usually 3 ft and 6 ft in length, 6 in. or 8 in. in width, and $1\frac{1}{2}$ in. in thickness. They are extensively used on timber bridge decks, to which they can be nailed, but are used to some extent on concrete decks also, to which they can be cemented with hot asphalt.

18.4.1. Under favourable conditions, these planks have a very long life. The planks laid on the Main Street Bridge at Winnipeg have not required replacement since 1931, except for some isolated pieces.

18.5. **Drainage of Slab Deckings**. The usual design is to place drainage pipes vertically, extending about 2 in. below the bottom of the slab.

18.6. **Plum Concrete** (Called rubble stone in concrete). The use of plum concrete is permitted in abutments, piers, wing walls, and footings, having a thickness or width of more than 3 ft 6 in., subject to the conditions that,

(1) the stones are placed not less than 6 in. from the formwork or reinforcing bars,

(2) they are placed not less than 8 in. apart, and

(3) the quantity of stones used does not exceed 20 per cent of the volume of the concrete. These conditions are remarkably similar to the conditions laid down, independently in this country, for the use of plum concrete.

18.7. **Joint between Wing Walls and Abutments (concrete)**. The abutments and wing walls are usually poured as a monolith. Construction joints, where necessary, are usually of the tenon and mortise type and continuity of the work is ensured by anchor bars.

18.8. **Bridge Bearings.** With reference to a problem which had arisen in the office of the Consulting Engineer (Roads) as to the desirability of having two in preference to three rollers in a bearing, or an even number of rollers in preference to an odd number, the writer made enquiries about the Canadian practice on this point. The results indicated that the Canadians did not attach any importance to the number of rollers, provided the design was otherwise sound. It was found that an odd number of rollers was actually in use on some very large and important bridges.

19. THE DUPLESSIS BRIDGE

19.1. This bridge was built over the St. Maurice river at "three rivers" in 1947 and has been in service since then. The western arm of the river was bridged, by 8 continuous all-welded spans, 2 of 150 ft each ; and 6 of 180 ft each ; while the eastern arm was bridged by 5 continuous spans, 2 of 110 ft each and 3 of 140 ft each. In the 180 ft spans, the girders were 12 feet deep at the supports and 8 feet deep at midspan with a web thickness of $\frac{3}{8}$ in. The shape of the soffit was parabolic. The clear roadwidth of the bridge was 42 ft with a footpath of 5 ft width on each side. The decking was 8 in. thick reinforced concrete slab.

19.2. On the 31st January 1951, after more than three years of service, 4 out of the 8 spans over the west crossing suddenly collapsed and fell into the river. The remaining 4 spans of this crossing and the entire bridge over the east crossing are intact.

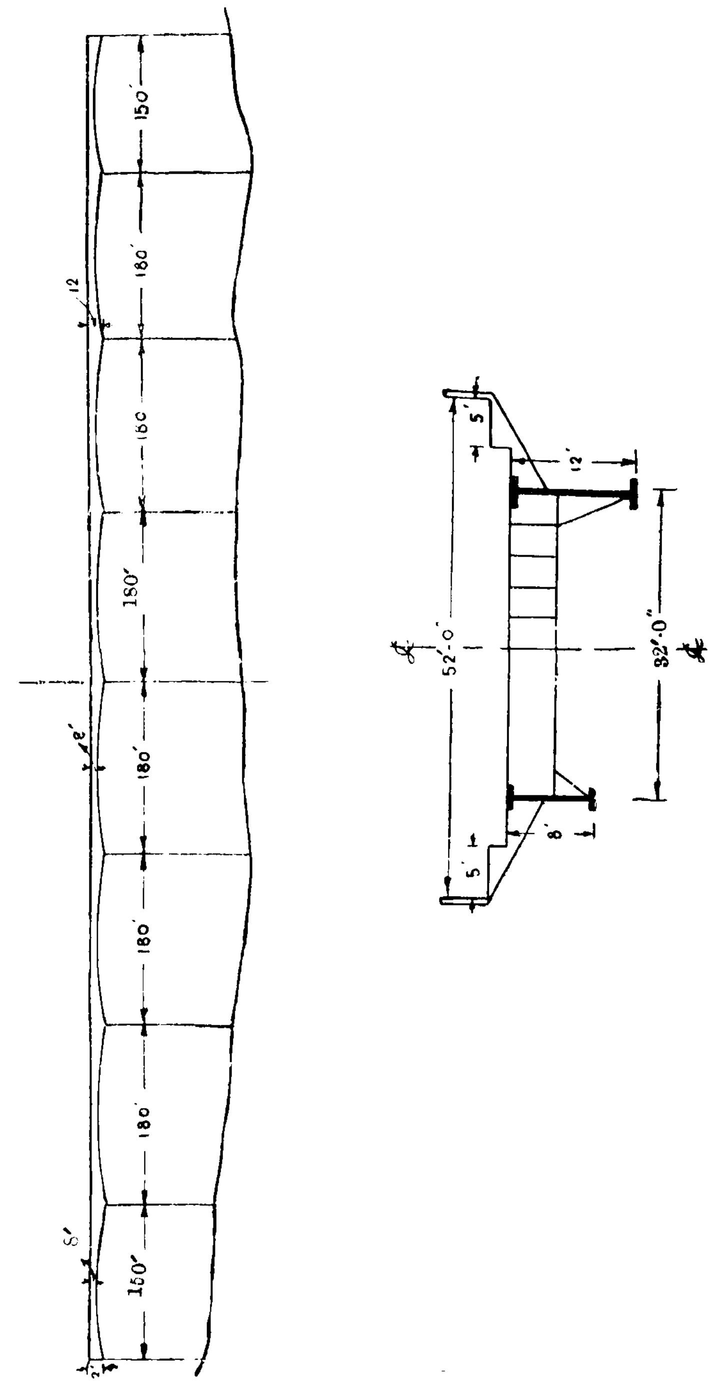
19.3. The bridge has had an interesting history. In February 1950, a fracture was discovered in the downstream girder in the 2nd span of the east crossing, at a distance of about 12 ft from the 1st pier. While this fracture was being repaired, a somewhat similar fracture was noticed at a very similar location in a 180 ft span of the western crossing. In both cases the fractures originated in the top flange and travelled down the web. In the east crossing the web got torn and buckled and the bottom flange was also bent and cracked, but in the western crossing the fracture stopped at about the middle of the depth of the web.

19.4. In the investigation which followed, all joints of these bridges as also of several other similar welded bridges were inspected but no other physical defects were found. Investigation covered physical tests, chemical analysis of the flange, web and weld material, impact tests, hardness tests, microscopic inspection, X-ray examination, checking up of design, actual loadings and impact, examination for pier settlements, temperature and fatigue stresses.

19.5. Many theories were advanced to explain the origination of cracks but the only plausible explanation was that the general quality of the plate material was poor and that internal cracks existed in the plates as rolled before fabrication. It was held that the welding procedure followed on the bridge and the standard of workmanship obtained were of a very high order. The bridge was closed to traffic for 5 days only after which restricted traffic was allowed.

19.6. Subsequently, on the completion of investigations, these restrictions were also withdrawn and the matter was forgotten. But shortly afterwards the engineering profession was startled by the collapse of 4 spans of the west crossing.

19.7. A public enquiry into the failure was demanded, and a commission consisting of a lawyer and a civil engineer was set up to report on the cause of failure.



THE DUPLESSIS BRIDGE.

19.8. As all the four collapsed spans had sunk below the ice and as the spans had to be cut from their heavy concrete deck, salvage work was long delayed. However, the wrecked mass of steel work was salvaged from the river bed in 2 to 3 months and collected in a yard. The various elements of the bridge were then pieced together as far as possible in the positions they occupied in the bridge before collapse. The yard was guarded to prevent any piece of evidence being tampered with in the course of the enquiry, but through the courtesy of the Canadian Govt. the members of the Indian Technical Mission were afforded facilities to inspect the wreck.

19.9. The findings of the court of enquiry had not been published during the period of the writer's stay there but the Dominion Bridge Company, the contractors responsible for the construction of the bridge, were kind enough to give the writer an opportunity of glancing through the results of a series of very exhaustive technical investigations made by the engineers of that firm and by various other eminent engineers.

19.10. It was extremely gratifying to note that the investigations were being conducted in a truly scientific spirit, without fear or prejudice, in keeping with the highest ideals of the engineering profession.

19.11. The design of the bridge was certainly bold and courageous; the web plates employed were perhaps the most slender ever used before on a plate girder bridge, the ratio of the depth of the girder to the thickness of web being 370 as against a limiting ratio of 170 laid down by the A.A.S.H.O., and the span lengths adopted were among the longest for all-welded plate girders continuous over 8 spans.

19.12. As regards the slenderness of the web it is being increasingly recognized that the traditional method of design by the shear flexure theory is not quite in keeping with the demonstrated capacity of buckled webs to carry shear. Test girders have not infrequently borne loads three or four times the calculated critical buckling load. In the field of aeronautical engineering, which indeed has given the present trend to the methods of web analysis, thin web girders are the rule. Investigations have shown that failure of the bridge cannot be ascribed to the slenderness of the web, nor can the failure be charged to any disability of the structure arising from welding, as failure did not originate at a welded joint.

19.13. Various other possible factors which could have contributed to the failure have been investigated e.g. jammed or frozen bearings; frozen expansion joints (finger plates); stresses arising from differential conductivity of steel and concrete; non-thermal increase in the length of concrete, non-adequacy of the repairs of 1950; settlement of piers etc., etc. But none of these actors appear to have been responsible for the failure.

19.14. The failure has certainly marred the record of a number of successful welded structures, but there is no doubt that out of the failure, new ideas will be born, new technique developed, and new means devised for dealing with difficult engineering situations.

20. BRIDGE PROBLEMS ARISING OUT OF THE CLIMATIC CONDITIONS OF CANADA

20.1 The bridge engineer in Canada has to face quite a number of problems arising out of the severe climatic conditions. Some of these are mentioned below.

(1) **Frost heave**—There is a large number of bridges founded on timber piles not going down to rock. Some of these structures heave up regularly every year. The extent of the heave is dependent upon the soil conditions, the depth of piles and the degrees of frost. The usual vertical movement is of the order of 6 inches but in some cases it is as great as 2 feet. Every year the piles have to be driven back so that the bridge decking comes back to the level of the approaches.

The reverse problem of the approaches heaving up, while the bridge remains stationary, is also encountered in some places. The writer was shown an approach road which had heaved up to the level of the top of the kerbs on the bridge decking.

(2) **Ice hammering on pier noses** – Large sheets of floating ice in some of the rivers, cause heavy damage to pier noses. The design features which have been introduced in piers, to meet this trouble are –

(a) a very heavy nose batter is provided, and

(b) angle iron ice cutters are fixed to the pier noses.

(3) **Freezing of concrete** – Alternate freezing and thawing results in the spalling and shattering of concrete whether in piers or in the superstructure. Extensive research is being carried out to evolve a suitable design of a concrete mixture which can stand this annual cycle of freezing and thawing. In general, tests have indicated that the use of an air-entraining agent improves materially the resistance of concrete to such damage. Air-entrained concrete is therefore specified for use on new structures.

20.2. The damage resulting to existing structures is being repaired by guniting the concrete faces. This has to be done every 3rd or 4th year.

21. GENERAL ECONOMICS OF BRIDGE DESIGN

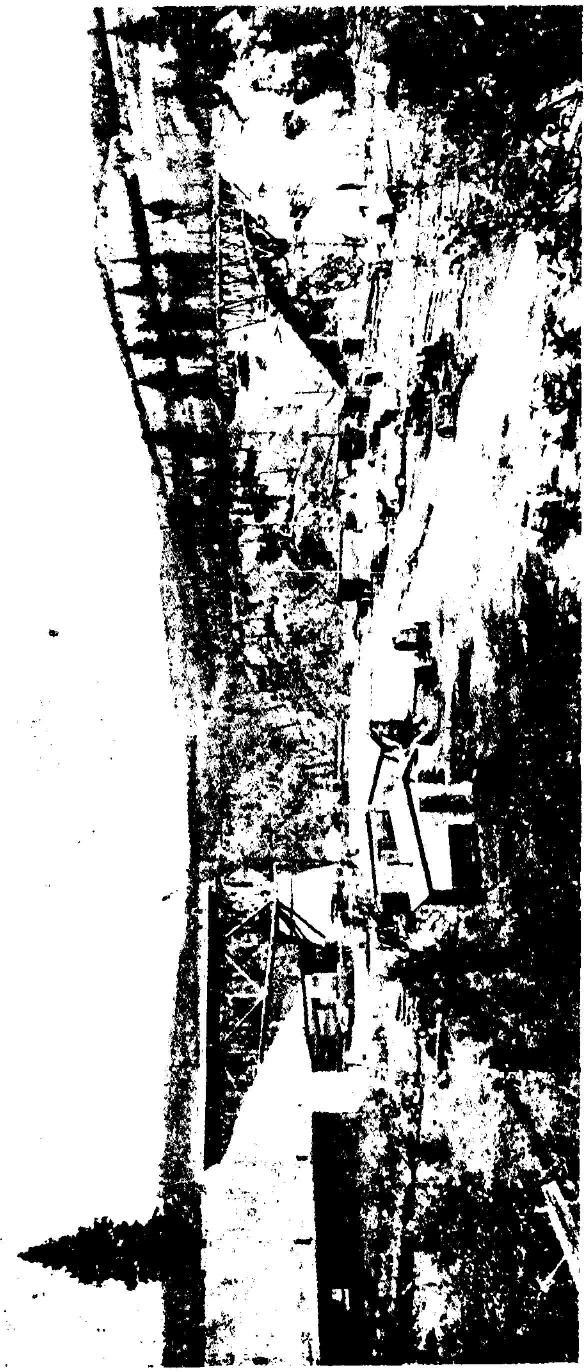
21.1. As compared to conditions in India, labour is extremely scarce in Canada and consequently the cost of labour involved on any bridge work is very high. Attempt is therefore made to get out designs which economise on the use of labour rather than on materials and involve the use of mechanical equipment as much as possible. Steel is relatively cheap, and so is the cost of its fabrication and erection with mechanical equipment. The result is that large spans which would be considered highly uneconomical in this country are frequently adopted. It is common to see a bridge with a pier height of only 50 ft or so but spans as large as 200 to 250 ft.

21.2. The ease of erection, speed of construction and the aesthetic appearance of the bridge are other factors which appear to receive more attention than in this country.

21.3. In examining the relative economy of alternative designs, designs are worked out in fair detail, for different types of structures, employing different methods of construction (e.g. welded or riveted), and employing different qualities of materials (e.g. ordinary mild steel or high tensile steel). Fairly detailed quantities are taken out for each design before taking a final decision.

21.4. In one case, a design with balanced cantilevers and suspended spans was rejected and a design with continuous spans adopted instead, although the foundations could not be relied upon to be unyielding, which is one of the desirable conditions for this design. Calculations showed that provision for extra stresses which would be set up by the maximum probable settlement of a pier, would mean less extra cost than the provision for joints and bearings required by the cantilever design.

21.5. Although, the advantage of continuity beyond 3 or 4 spans, in reducing the design moments, is very nominal, bridges have been built with as many as 8 continuous spans in order to economise on the cost of bearings and to avoid joints in the deck, which are a source of trouble in freezing weather. There are bridges whose length exceeds 1500 ft but are jointless from abutment to abutment, e.g. St. Rose and St. Eustace bridges which are 1,548 ft and 1,520 ft long respectively. On these bridges, provision is made at each abutment for an expansion of over 9 inches.



Photograph No. 7
Rock Creek Bridge, under construction. (See Appendix I.)

APPENDIX I

The Rock Creek Bridge

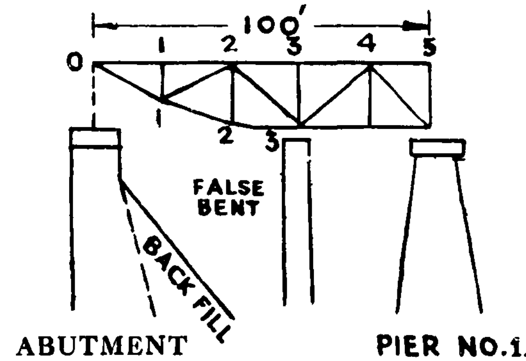
1. This bridge was under construction across a very deep gorge on the Trail-Penticton highway. The overall length of the bridge is 900 ft., with a central span of 300 ft and side spans of 200 ft (2 Nos.) and 100 ft. (2 Nos.)

2. The design and construction procedure adopted was such as to obviate the need of false work in the deep gorge which could have been very costly because of the great height.

3. Designs of ordinary trussed girders and also of steel arches using the conventional method of erection were considered and rejected because they involved the use of very high and costly formwork erected from the bed of the deep gorge.

4. The bridge is essentially a continuous trussed girder supported on tall and slender steel towers having pinned bearings at top and bottom. The bridge is symmetrical about the centre line and erection was started from either end, two 20-ton Lorain cranes with 50 ft booms being used for the purpose.

5. The erection procedure may be described in 7 stages. The joints and members of the upper and lower chords are indicated by their corresponding numbers suffixed to letter U and L respectively.



First Stage.

(1) Erect false timber bent.

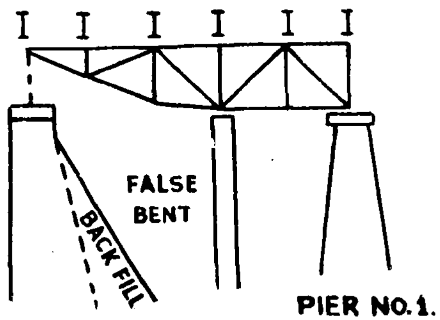
(2) Set bearings on abutment and pier No. 1 and lock the expansion bearing.

(3) Assemble the section consisting of members OU_1, U_1U_2, U_2L_3, L_3L_2, L_2U_2, U_2L_1, L_1L_2, L_1U_1, L_1O on the ground and erect in one piece, placing one end on the abutment and the other on the timber bent.

(4) Fill in lateral system.

(5) Complete erection of 100 ft span upto pier No. 1.

Second Stage.



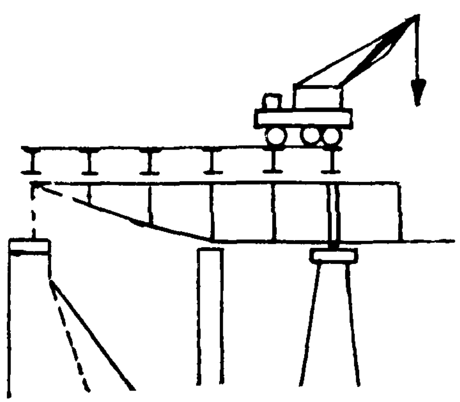
(1) Jack at pier No. 1 to release false bent. Drop false bent.

(2) Unlock expansion bearings on abutment.

(3) Fill in stringers with the Crane.

(4) Assemble 1st section of main span on ground.

Third Stage.

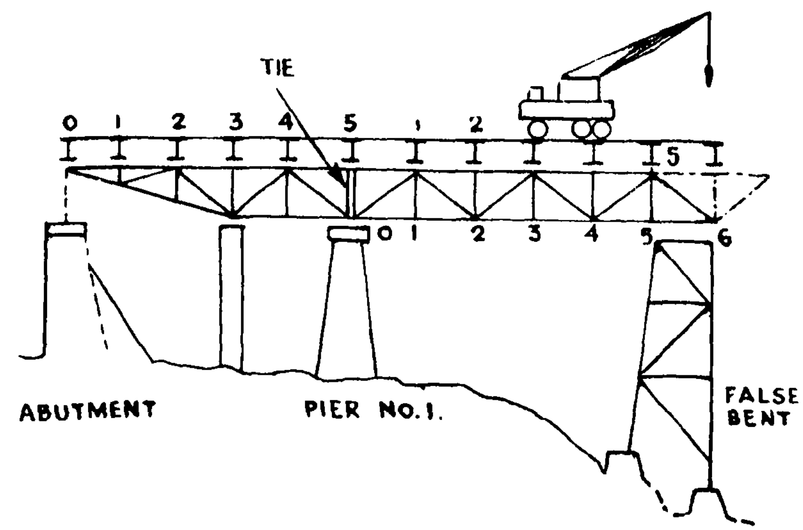


(1) Connect 1st section of main span to the end of the approach span, forming a cantilever arm, connected to the approach span with a tie.

(2) Fill in lateral system.

(3) Continue extending the cantilever arm into the main span till joint L_4 and U_5 is reached.

Fourth Stage. (See photo 8).



(1) Erect false bent in position shown.

(2) Assemble members L_4L_5, L_5L_6, L_6U_5 and L_5U_5 when false bent is in position.

(3) Assemble chain dotted members after closing on false bent.

(4) Free the tie over pier No. 1.

Photograph No. 8

Rock Creek Bridge (4th stage of construction from one abutment). (Appendix I)

Fifth Stage.

(1) Jack on pier No. 1, remove pins from pier members and lower end of main span by 6 in.

(2) Tie down end of main span to bottom chord of approach span.

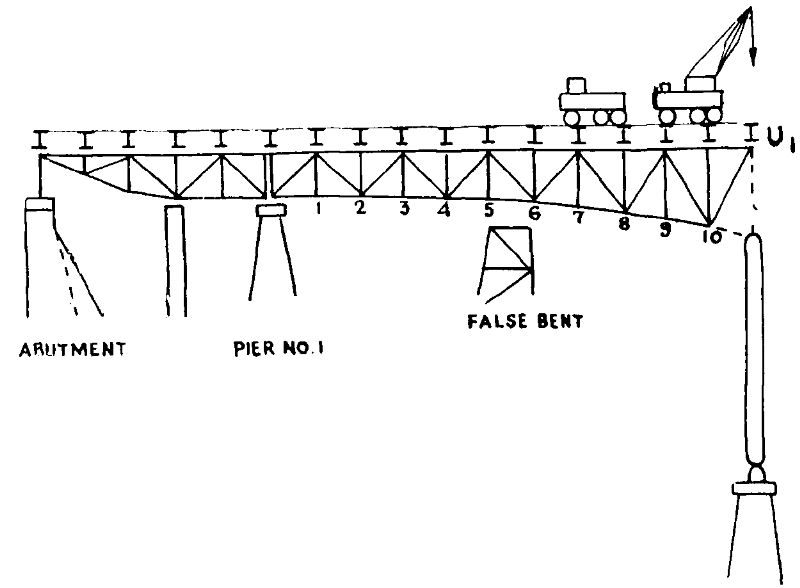
(3) Build out to U_{11}. Crane and truck not to pass positions shown.

(4) Erect permanent steel tower.

(5) Connect dotted members.

(6) Remove crane and truck and jack up on pier No. 1 to close on permanent tower.

(7) Continue jacking to free false bent at L_6.



(8) Jack down again at pier No. 1.

(9) Dismantle false bent.

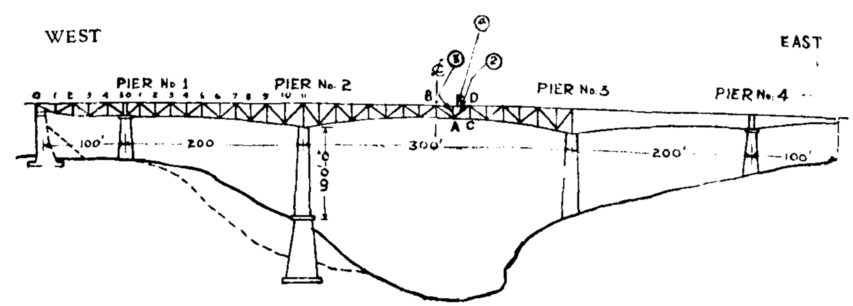
[Stages 1 to 5 carried out simultaneously from the other end also, with a second crane.]

Sixth Stage.

(1) Cantilever central span to centre line and one bay beyond (upto member AB) from one end and upto member CD from the other end.

(2) Take crane and truck off the span.

Seventh Stage.



SIXTH & SEVENTH STAGES

(1) Close bottom chord first at point A

(2) Add members marked (2) and (3) with crane on East Cantilever.

(3) Add top chord marked (4).

(4) Jack up at piers 1 and 4, to close top chord at B.

(5) Jack further to insert main bearing pins.

APPENDIX II

The Arvida, all-Aluminium Bridge

The principal dimensions of this bridge are—

Overall length	—	504 ft.
Main span (arch)	—	290 ft.
Rise of arch	—	47½ ft.
Approach spans	—	5 Nos. of 20 ft each, at each end.
Clear road width	—	24 ft.
Footpaths	—	4 ft wide, one on each side.

Interesting design features.

2. The principal aluminium alloy employed on the bridge was 26S—T Alcan for extruded shapes and also for Alclad plates. A number of other alloys with special properties were, however, also used for various elements such as rivets, hand railings, etc.

3. The outstanding characteristics of this alloy are—

(1) its light weight, about 36 per cent of that of the structural grade steel :

(2) high coefficient of thermal expansion, about twice that of steel ;

(3) low modulus of elasticity, about one third that of steel ;

(4) susceptibility to galvanic corrosion.

4. The total weight of metal used on the bridge is about 380,000 lb as against an estimated weight of 875,000 lb required for a similar structure in steel i.e. the saving in dead weight was about 56 per cent.

5. The heaviest single fabricated piece used on the bridge was an arch section which despite its length of 65 ft weighed only 6½ tons.

6. Thermal expansion of the structure is relatively large as compared with that of a steel structure and is taken care of by specially designed finger plate expansion joints at each end of the structure.

7. A still greater problem, however, which required very careful thought, was the differential expansion and contraction between aluminium and concrete. It was calculated that if the concrete slabs were anchored to the floor members at the arch crown, the relative slip between them at the ends of the bridge would be 1½ inches, which would result in very serious consequences.

8. It was decided to prevent all relative movement between the concrete decking and the floor system by anchoring them to each other. The resulting longitudinal forces both in the floor members and in the concrete slabs were calculated and provided for in the design.

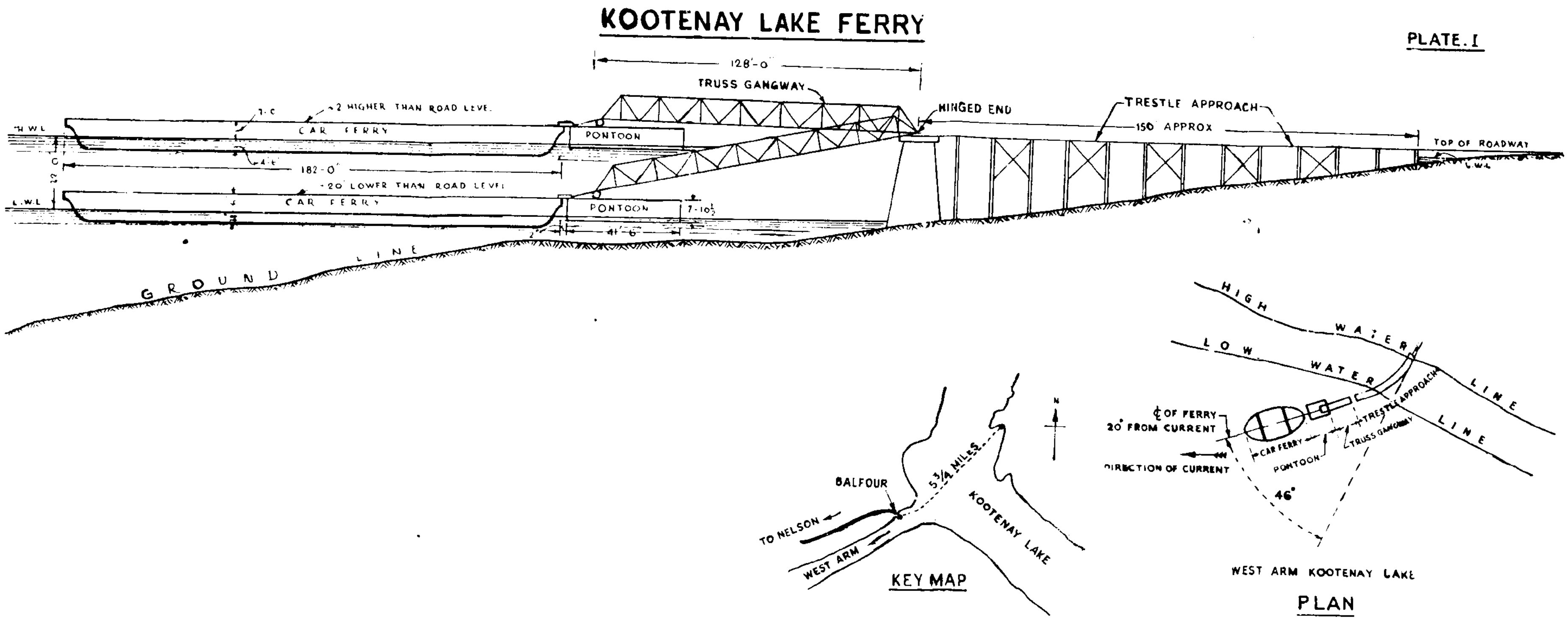
9. The arch ribs are of the closed box type 54 in. deep by 32 in. wide.

[Experience has shown that condensation of water vapour present in the air enclosed in the box takes place. No provision was made originally in the design for draining out the resulting moisture but necessary outlets were being introduced on the day of the writer's visit.]

10. In the interests of safety, the aluminium hand rails are designed to withstand horizontal forces much greater than the forces usually specified. Thus, the individual posts and their connections are designed for a concentrated load of 4,500 lb applied laterally at the top of the post, and the complete hand railing is designed for a total laterally applied, uniformly distributed load of 1,140 lb per rft. or a concentrated load of 5,700 lb.

[The I.R.C. Bridge Code requires the railings to be designed for a lateral horizontal force and vertical force, each of 100 lb per lineal foot applied simultaneously to the top of the railing.]

11. One end of the bridge is 13 ft higher than the other, but the gradient is not uniform from one end to the other. The gradient is 4.125 per cent on one side of the crown of the arch and


KOOTENAY LAKE FERRY
PLATE. I
128'-0"
TRUSS GANGWAY
HINGED END
TRESTLE APPROACH
150 APPROX
TOP OF ROADWAY
CAR FERRY
PONTOON
182'-0"
CAR FERRY
PONTOON
GROUND LINE
HIGH WATER LINE
LOW WATER LINE
OF FERRY
20° FROM CURRENT
DIRECTION OF CURRENT
CAR FERRY
PONTOON
TRESTLE APPROACH
TRUSS GANGWAY
46°
WEST ARM KOOTENAY LAKE
PLAN
BALFOUR
TO NELSON
WEST ARM
5¾ MILES
KOOTENAY LAKE
KEY MAP

1.034 per cent on the other, a vertical curve being introduced at the crown.

12. Roadway consists of a continuous concrete slab 8 in. thick, reinforced in 2 directions, covered by a bituminous surface $2\frac{1}{2}$ in. thick throughout. There is no camber on the road surface and no drainage outlets on the decking.

13. Kerb height is only 6 inches to allow the easy opening of car doors.

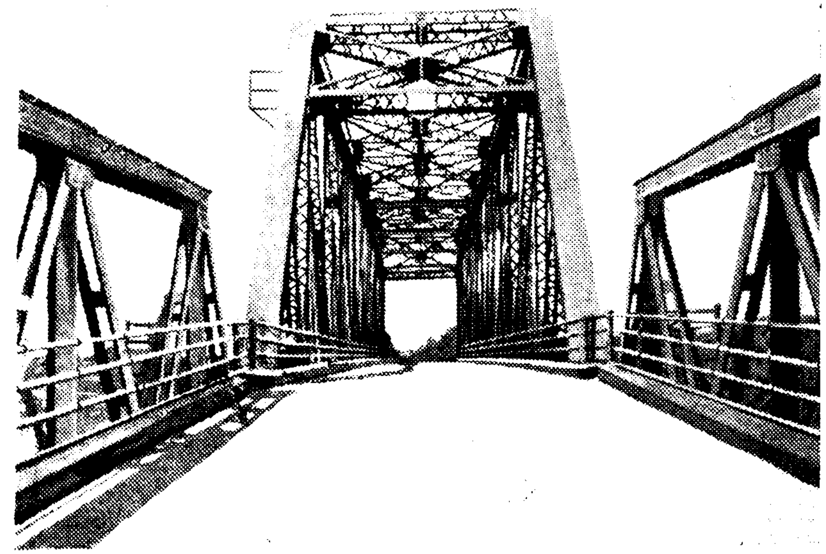
14. **Riveting**. After an extensive programme of experimental rivet driving and testing, a special technique for driving aluminium rivets was developed. The rivets used were all driven cold. Extruded rod was found unsuitable for use as rivet block, and therefore rivets were made from hot-rolled and cold-drawn rods. Where possible a 50-ton power riveting machine was used, but rivets inaccessible to the machine were gun-driven, "annular rivets". A 3/16 in. dia. hole $\frac{3}{8}$ in. deep is made in the rivet end into which fits a projection in the snap of the gun. As the rivet is driven, the gun is rolled and an annular head is formed. To minimise the upsetting of rivet shank driving, the rivets were made only 1/64 in. less in dia. than the rivet hole which was reamed with a 13/16 in. dia. reaming drill.

15. Care was taken in all details to eliminate absolutely the formation of the water pockets. The reason is that aluminium surfaces when in contact with other metals, or even with each other in the presence of moisture, are subject to galvanic action. Even a thin layer of some insulating material like bituminous paint is considered adequate to prevent it. To protect the aluminium from the effect of grout or of the moisture that might sap through the concrete slab, sheets of commercially pure aluminium 1/16 in. thick were interposed between the concrete slabs and the top flanges of all aluminium floor members.

16. **Fabrication in general**. Aluminium and its alloys are not suited for high temperature applications. With increase in temperature, they lose their high strength. Aluminium cannot therefore be flame cut, welded or heated to a temperature higher than 350° F.

17. Sawing is, however, easy and expedient and was used instead of flame cutting and even shearing. Shearing can be used for sections up to $\frac{1}{2}$ in. thick but better edges are obtained with sawing. Punching is satisfactory upto $\frac{1}{2}$ in. thickness if sharp edged punches and neat dies are used.

—o—



Surma Road Bridge at Sylhet, I span of 240-feet, two of 100-feet and 17 of 40-feet on Cylinder Piers.