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TRANSPORT-COMMUNICATIONS

MONTHLY REVIEW (TCMR)

MAY 1961—No. 169

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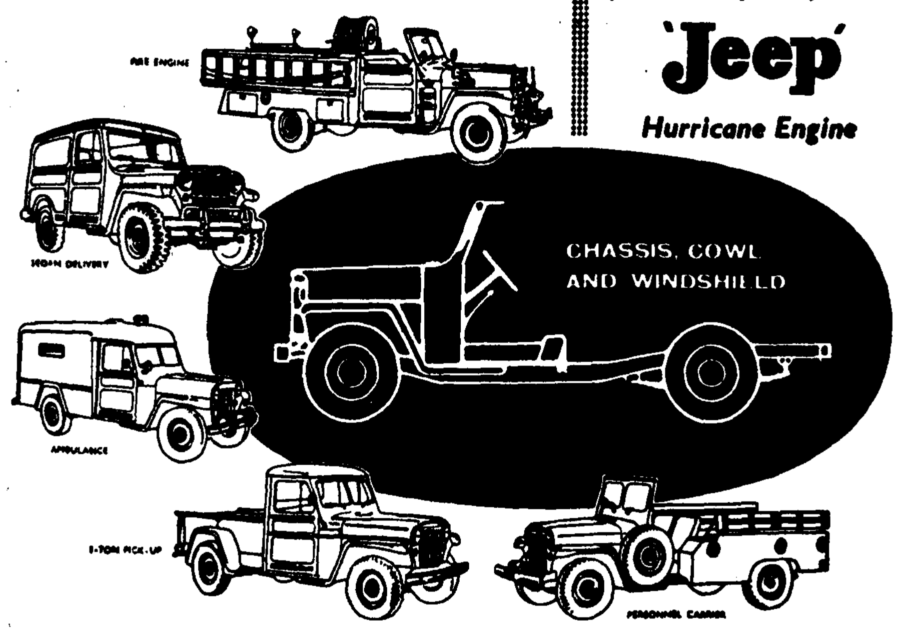

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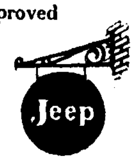

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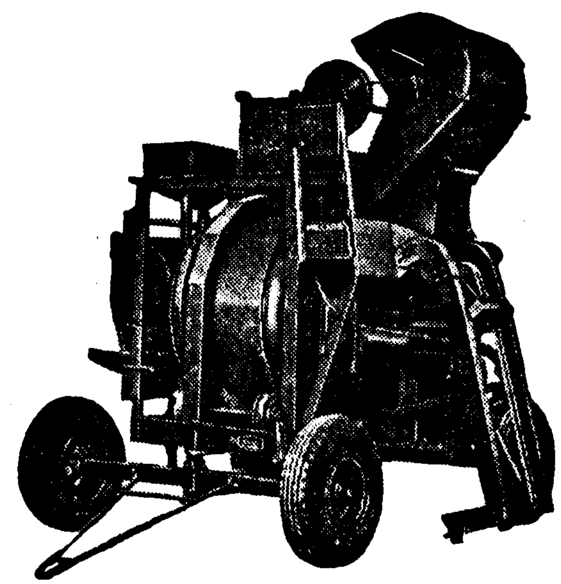

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TRANSPORT-COMMUNICATIONS MONTHLY REVIEW

MAY 1961—No. 169

LIST OF CONTENTS

	Page
GENERAL	
Public Works and Accounts	1
The Sukhtel Bridge—Bridging India's Rivers	2
The Influence of the Traffic Engineer on road design in Natal	10
TECHNICAL	
Standard Bridge Beams in Prestressed Concrete	20
Improving soil by Chemical means	26
Preflex Beams	31
Surface Dressing	34
HERE AND THERE	
1. Soil Compaction and Proof Rolling of Subgrades	37
2. Recording defects in Highway Surfaces by Photography	37
3. To survey Turkey to Iran Highway	38
4. Facts about Rourkela	39
CURRENT LITERATURE	
List of Publications added to the I. R. C. Library	40
List of Approved works on National Highways	42

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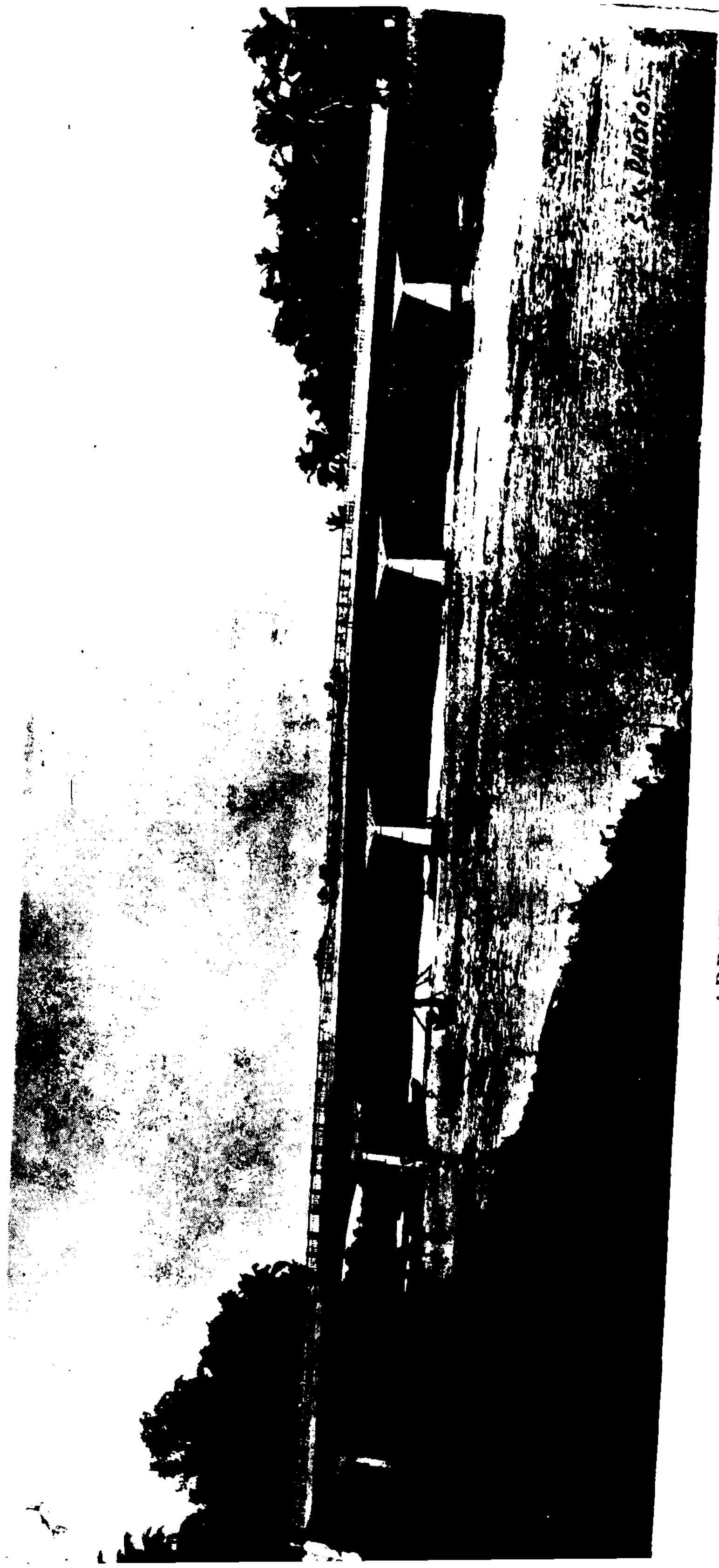


ARRATTUKADAVU BRIDGE
ACROSS THE ACHENCOIL RIVER AT MAVELIKARA IN MILE 11 OF MAVELIKARA-TIRUVELLA ROAD

TRANSPORT-COMMUNICATIONS MONTHLY REVIEW

A REVIEW OF
ROAD ENGINEERING, DEVELOPMENT & TRANSPORT

MAY 1961

PUBLIC WORKS AND ACCOUNTS

SINCE independence India has made rapid strides in planned development. In particular the record in the field of public works has been strikingly impressive. Yet the Engineers in this country do feel that this could be improved if certain forms and formalities which they have to conform to in their day-to-day work are simplified. It has been evident during the last decade that quite a good deal of time, labour and routine work have come up against quick decisions and rapid progress. In the method of investigations, designs and plans, considerable rationalisation has taken place recently, and these can stand comparison with the most up-to-date method followed in the western countries. But the same could not be said of the accounts procedures followed at present. This is a legacy handed down from the past and it has not undergone much change so far.

It should be remembered that the P.W.D. Accounts Code was framed at a time when the major bulk of public works was maintenance of existing structures while original work was only a small proportion of the annual budget. Today, the position is exactly the opposite. By far, a large proportion of the annual allotment under public works whether it be irrigation, highways or buildings, is under maintenance, original works forming only a small percentage. It is in this context that the existing code rules have proved inadequate to meet the demands of the time. However, when both the Government and the people are anxious and impatient to get through developmental works as rapidly as possible, the handicaps which face the officers responsible for such execution have not been fully recognised. It is high time that a serious attempt is made to review the existing code rules and amended so that the programme of action which the Government envisages in the succeeding plan periods may be fulfilled with all speed and efficiency.

For want of simple methods of accounting there has been considerable delays and lowering of efficiency and the usual red tape associated with administrative work has hampered the detection of mistakes in time. Though financial control is essentially required, the achievement of best results with minimum of expenditure and friction should be the ultimate goal. A system, therefore, should be evolved with simple forms of accounts and procedures, wherein the executive and audit can achieve the results with minimum labour and maximum efficiency. It is, therefore, suggested that a high level committee concerned with these aspects be constituted to go into this question and revise the existing rules with a view to prescribe simple and effective code so that the implementation of the plans from now onwards may be made more easy and efficient.

(From 'Irrigation and Power', February, 1961)

THE SUKHTEL BRIDGE

(A PRESTRESSED CONCRETE BRIDGE IN ORISSA)

By

A. C. De*

The Site

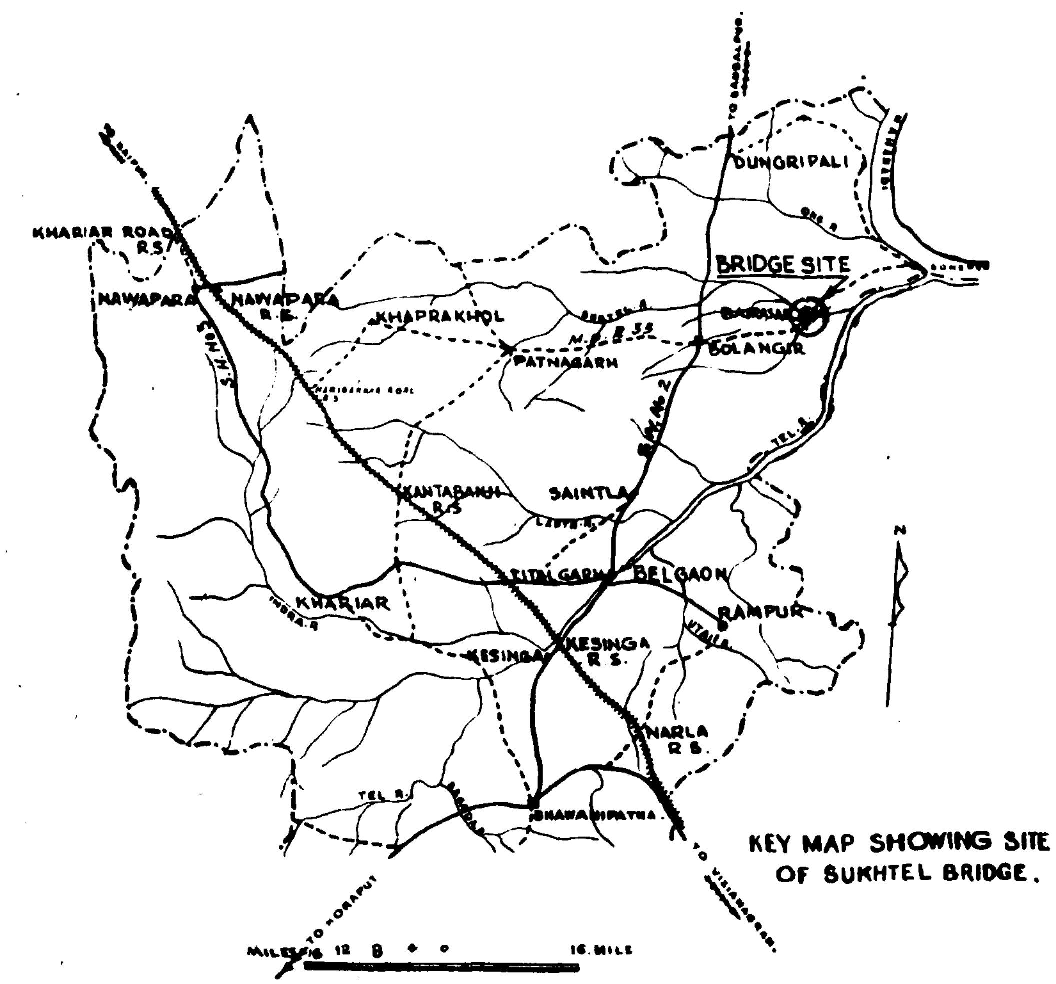
THE bridge across river Sukhtel in mile 16 of Bolangir-Sonepur road removed a long-felt necessity of the area.

Hydraulic Particulars and Waterway

At the bridge site the river has 744 sq. miles catchment area; calculating discharge by various empirical formula and comparing it with proportionate discharge of the river in case of another bridge on Bolangir-Sambalpur road the final design discharge arrived at was 70,000 cusecs for which linear waterway of 700 ft was worked out. The Sukhtel bridge is 735 ft wide between abutment faces.

General Features of Design

The bridge has 7 spans each 105 ft centre to centre. The superstructure of each span consists of 4 Nos. T-shaped precast and



KEY MAP SHOWING SITE OF SUKHTEL BRIDGE.

* Executive Engineer, Construction Division, Cuttack.

Sinking of well

End block with shuttering on one side

Launching of girder

prestressed girders, fixed at one end, and free at the other designed to carry two lanes of I.R.C. Class A loading or one lane of I.R.C. Class AA loading whichever created higher stresses. 3-in. wearing coat with light reinforcement has been provided. Road width between wheel guards is 24 ft.

The piers are made of mass concrete 1 : 3 : 6 with nominal reinforcement. Pier cap is heavily reinforced.

The foundation for each pier consists of two circular wells each 8 ft 6 in. internal diameter, 1 ft 6 in. thick reinforced steining, placed 4 ft 6 in. apart.

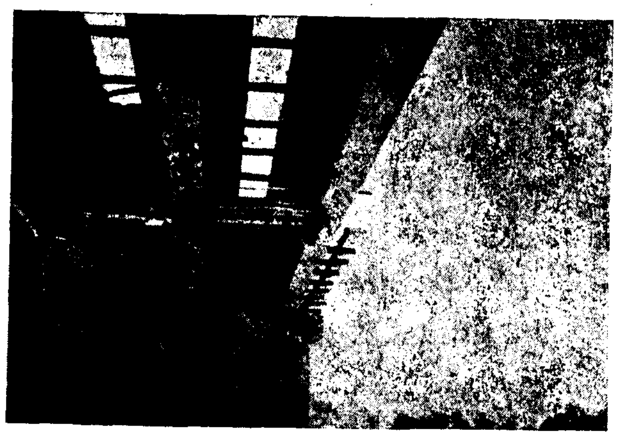
Well Curbs

The curbs are of 1 : 2 : 4 R.C, 1 ft 7½ in. thick at top tapering down to 4 in. in 2 ft sharp edged and bolted together was used to loosen the clay. Kentledge upto 150 tons was applied wherever necessary.

After final sinking of wells 4 ft thick bottom plug of 1 : 2 : 4 cement concrete was laid in water with steel skip measuring 3 ft × 3 ft × 3 ft 9 in. lowered by a steam crane, 14 cu. ft. concrete was released on each operation.

After laying the bottom plug, sand was filled up to 1 ft below the top of steining. The top 1 ft was filled with 1 : 3 : 6 cement concrete.

The wells were capped with 27 ft 6 in. by 11 ft 6 in. 1 : 2 : 4 high grade concrete slab heavily reinforced with 7/8 in. rods at 5 in. centres at the top and bottom in longitudinal direction and 7/8 in. rods at 4 in.



Underside view of girders

6 in. depth. The cutting edge consists of 4 in. × 4 in. × ½ in. angle iron with 8 in. × ½ in. M.S. plate welded to it. 5/8 in. bond rods have been provided right from angle iron of the curbs up to top of well cap.

Steining of Well

The steining consists of 1 : 3 : 6 cement concrete with 1½ in. to ½ in. graded aggregate and 32 Nos. 5/8 in. dia. main vertical bond rods and ½ in. dia. distribution rods at 12 in. centres.

Form work for steining consisted of salwood planks faced with steel sheets, each form being 4 ft high.

Well Sinking, Plugging & Well Cap

Wells were sunk by open dredging. In case of sticky clay stratas, a cluster of rails centres across at the top and bottom and shaped to cover outer periphery of wells.

Piers and Abutments

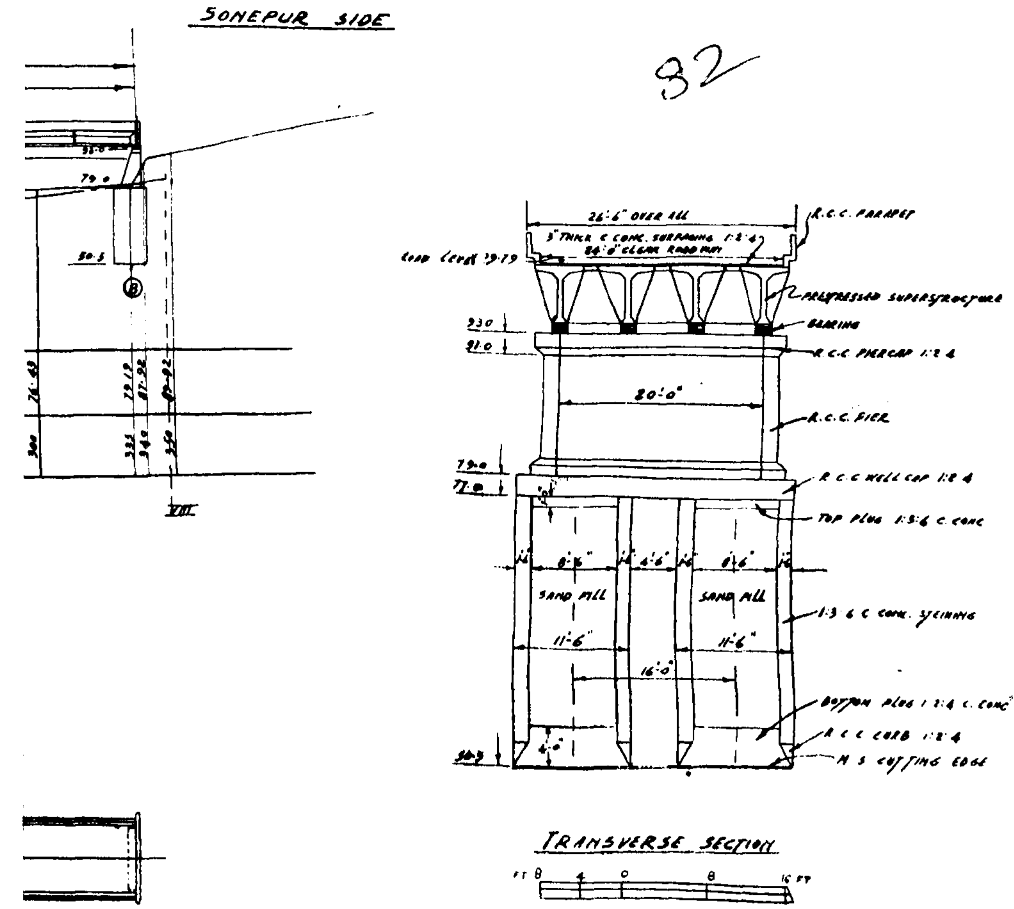
The piers were of mass concrete 1 : 3 : 6 nominally reinforced. Cutwaters both upstream and downstream were semi-circular.

The abutments were 8 ft wide at bottom (sloping faces 1 in 3 on riverside) made of 1 : 3 : 6 cement concrete with 15 per cent plums of 6 in. size, were founded over wells as for the piers and were designed as retaining walls. Masonry wing walls were provided.

Superstructure

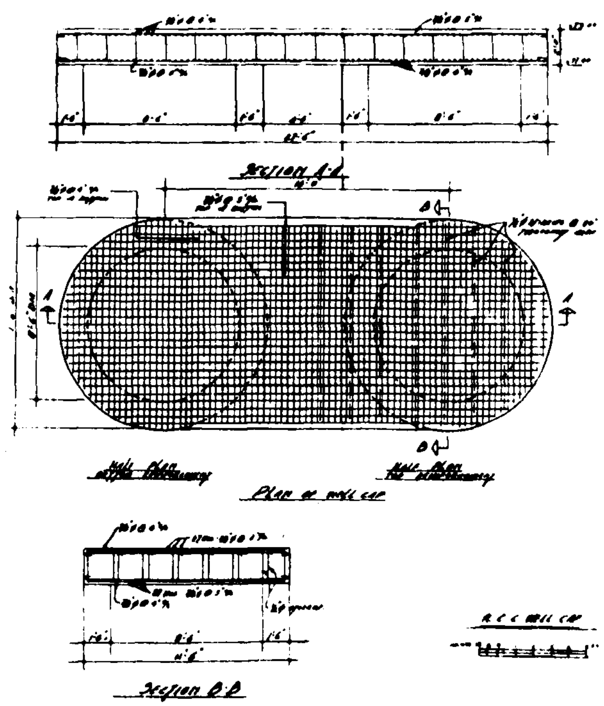
The superstructure consists of 4 Nos. prestressed T beams with 6 in. thick × 60 in. wide flange and 60 in. wide × 6 in. thick web.

PLATE I

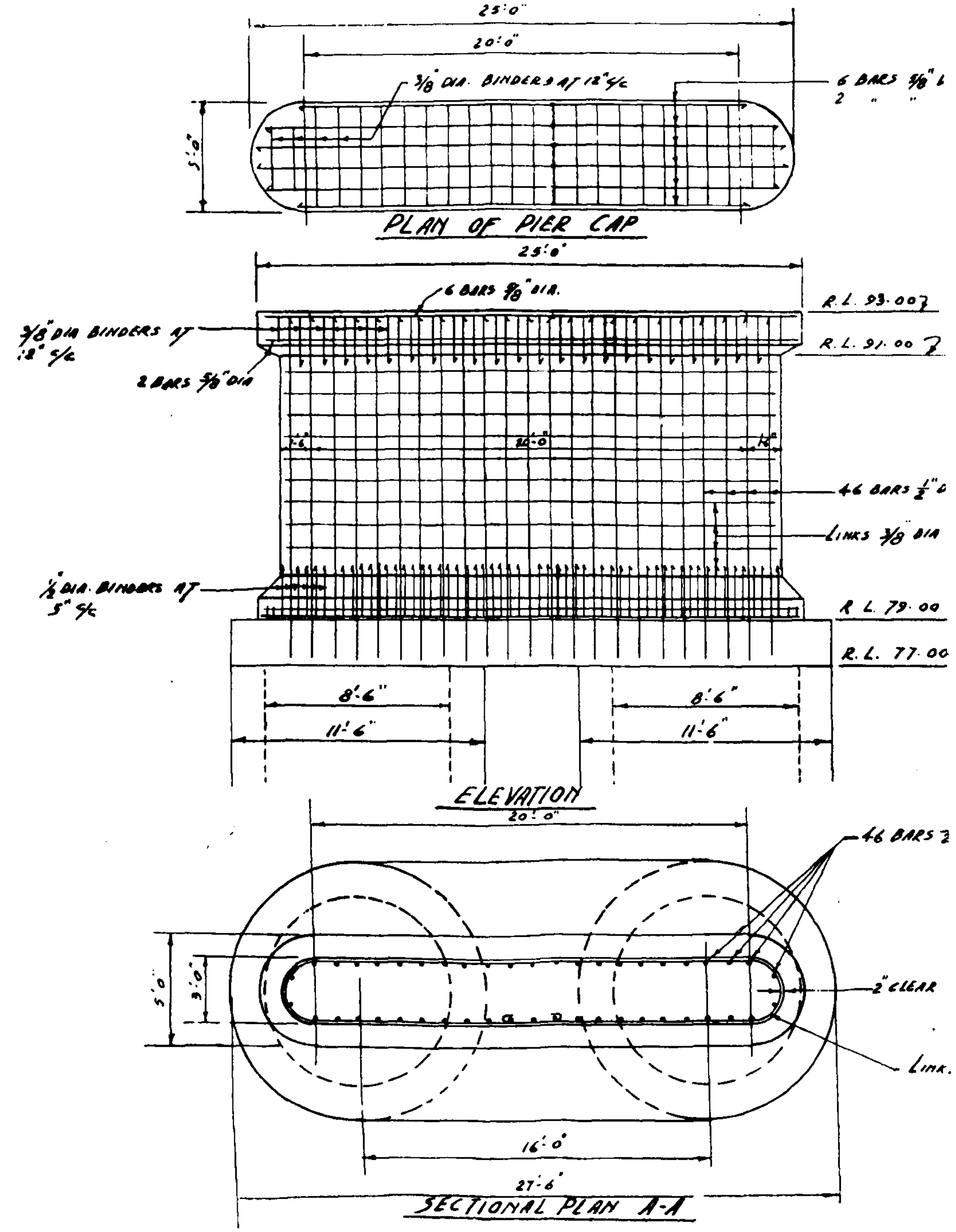


GENERAL PLAN

PLATE II

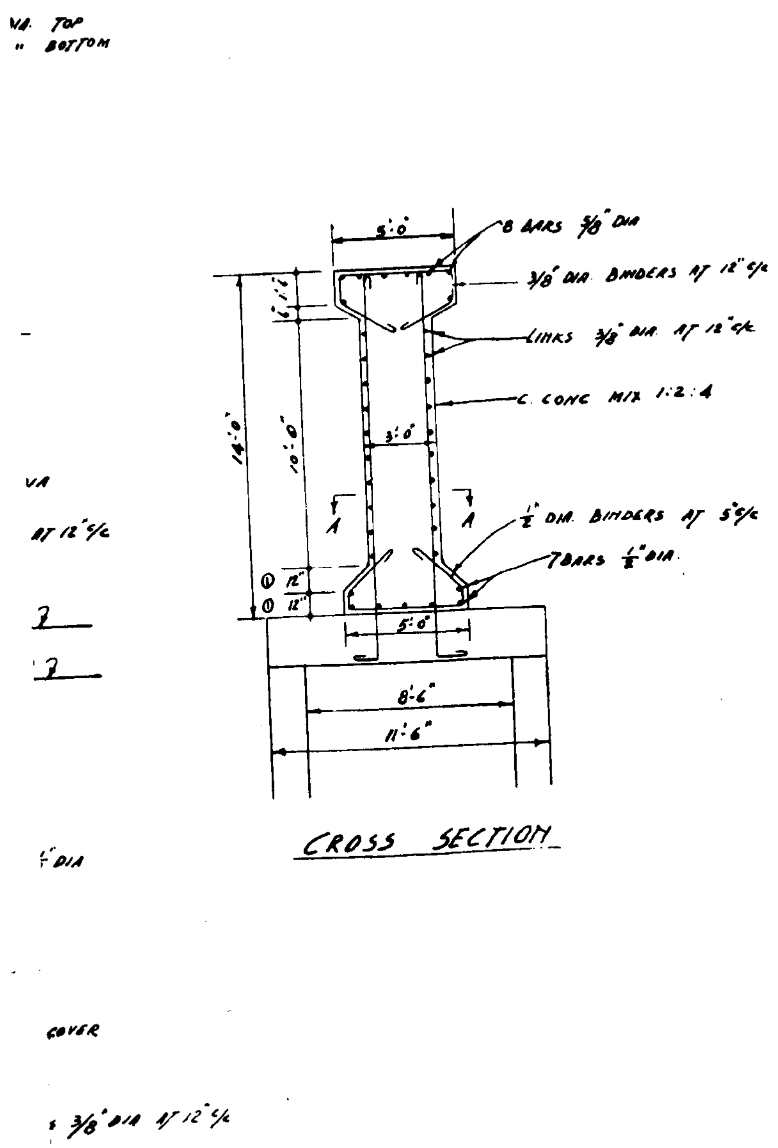


Section A-A
Plan of well cap
Section B-B



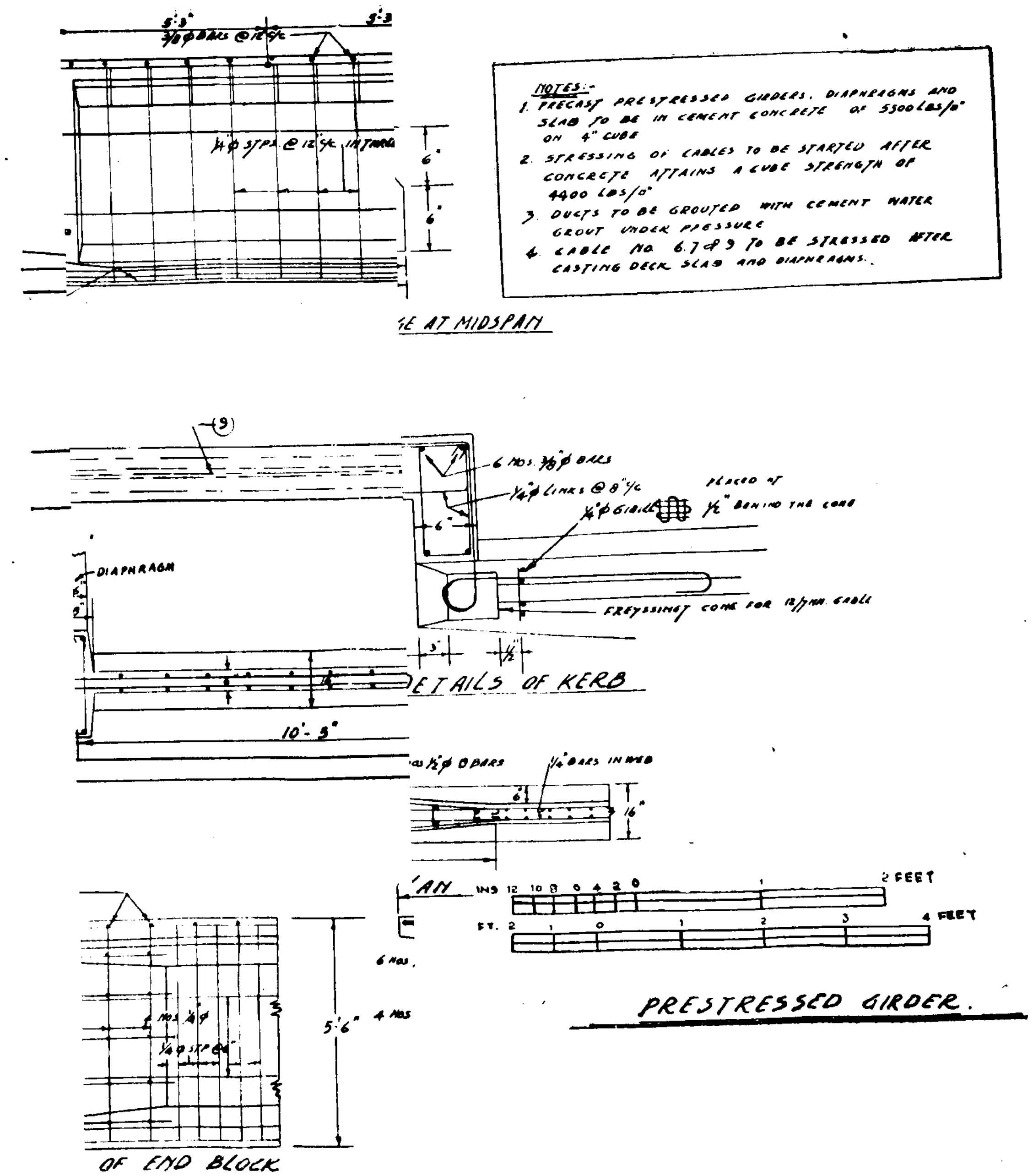
25'-0"
20'-0"
3/8" DIA. BINDERS AT 12" c/c
6 BARS 5/8"
PLAN OF PIER CAP
5'-0"
25'-0"
6 BARS 5/8" DIA.
3/8" DIA BINDERS AT 12" c/c
2 BARS 5/8" DIA
R.L. 93.00
R.L. 91.00
46 BARS 1/2"
LINKS 3/8 DIA
1/2" DIA. BINDERS AT 5" c/c
R.L. 79.00
R.L. 77.00
8'-6"
11'-6"
8'-6"
11'-6"
ELEVATION
20'-0"
46 BARS
5'-0"
3'-0"
2" CLEAR
LINK.
16'-0"
27'-6"
SECTIONAL PLAN A-A

PLATE III



CROSS SECTION

PLATE IV



PRESTRESSED GIRDER.

Precast and Prestressed Beams

Each Freyssinet cable used consisted of 12 Nos. of 0.28 in. dia. wires. Each cable was kept 36 in. longer than actual length required for facilitating prestressing operation.

Concreting

The end blocks were 1 ft $11\frac{1}{2}$ in. long and heavily reinforced.

After completing absolutely watertight shuttering, and placing the cables in position, beams were concreted with 1 : 1·2 : 3 cement concrete (materials being measured by weight) and water cement ratio of 0.4. Both external and internal vibrators were used.

Prestressing

Prestressing was done 7 days after concreting of the beams. For prestressing operations, two Freyssinet jacks—one at each end, two hydraulic Clemang pumps, two steel scales and two wooden struts were used.

In the first instance only 6 out of the 9 cables were prestressed, the remaining 3 cables were prestressed after casting of diaphragm and the slabs.

The cable ducts were grouted with 2 parts cement, one part water after cables were prestressed.

Launching of Girders

The following main operations were adopted for launching of girder on to the span:

(a) Laying of trackline in the river bed on both sides of pier.

(b) Lifting of girder in the casting yard on to the trolley.

(c) Transfering of girder from trolley to gantry.

(d) Dragging girder to span, lowering and placing the same.

By means of 50 tons jacks and launching beams, girder was lifted and placed over trollies—one in the front and one in the rear.

The girder was then taken forward and transferred to the gantries, which were dragged over the track line by 2 No. 5 tons crab winches. From gantry, beam was lowered on the supports. The beams were placed from far end, i.e., from opposite side of casting yard—inner beams were placed first and then outer.

Diaphragm and Deck Slab

After placing of 4 beams in position, the diaphragm and deck slabs were cast in site. The deck slab and diaphragm were later on prestressed. The balance cables of the main Girders were then prestressed and the whole operation of prestressing was over. The span was ready for laying of wearing coat and railing.

Wearing coat and Railing

3 in. wearing coat of 1 : 2 : 4 cement concrete, reinforced with 1/4 in. dia. rings—1 ft-$10\frac{1}{2}$ in. dia. was provided.

Expansion joint consisting of 2 in. × 2 in. × $\frac{1}{4}$ in. angles fixed to edges of wearing coat and welding one 4 in. × $\frac{1}{4}$ in. flat to one angle so that the flat moved over leg of opposite angle.

Load Test

Load test for one span was carried out. The loading was done by means of sand bags over a platform made of sleepers and rails. Sleepers were so arranged that when loaded, the deadload conformed to I.R.C. Class AA loading.

Total load was 70 tons + 10 per cent for impact, i.e., 77 tons. About 1,920 bags of sand were used.

Deflections were noted by levelling instruments. It took 16 hours to complete loading and recording levels of load test. The load was kept on the span for 3 days after which unloading was done and levels noted again.

THE INFLUENCE OF THE TRAFFIC ENGINEER ON ROAD DESIGN IN NATAL

By

L. W. BRADFORD, B.A., B.A.I., A.M.I.C.E, A.M.INST.H.E.*

NATAL, the smallest province of the Union of South Africa, is renowned for its scenery, its holiday resorts, both on the Indian Ocean coast and among the Drakensburg mountain range with peaks rising to 11,000 ft and for its roads.

Although the smallest province, it is about 35,000 square miles in area, which is equivalent to the English counties south of the line from Liverpool to the Wash, excluding Wales.

Broadly speaking, the topography can be divided into three belts, the coast strip, which is mainly given over to sugarcane plantations, a central plateau of about 3,000 ft above sea level, which is mostly a mixed agricultural area, and the mountainous escarpment on the west as the natural division between the province and the Transvaal and the Cape.

The capital city of the province is Pietermaritzburg which lies 50 miles inland from Durban, the principal port for the eastern coast of the Union.

The province spends some 5,000,000 pounds annually on its 7,720 miles of road—2,000,000 pounds on national road construction, 1,000,000 pounds on provincial road construction and 2,000,000 pounds on maintenance and betterment of the entire road system. The construction of new roads is simplified to a great extent by two Government Ordinances.

Government Ordinances

Firstly, the Government incorporated in the title deeds of land sold for farming a clause giving them power to expropriate a 100 ft strip of land for road or railway construction, with further power to compulsorily purchase land in widths upto 250 ft. No compensation is therefore paid to the land owner for strips of land less than 100 ft wide, but he is compensated for loss of improvements such as crops, buildings, irrigation, drainage, etc., which are destroyed through the construction of a road. As farms are usually between 500 and 3,000 acres in extent the farmer is prepared to give up a narrow strip of his land in order to gain access to an improved road, except in the case of expressways and, indeed, the value of his property increases when served by an all-weather road.

Access to main roads is also controlled by ordinance in that no access may be made to a main road without the permission of the Chief Roads Engineer and the Chief Engineer may alter any existing access, at his cost, to a more suitable position. In the case of expressways, access is denied altogether.

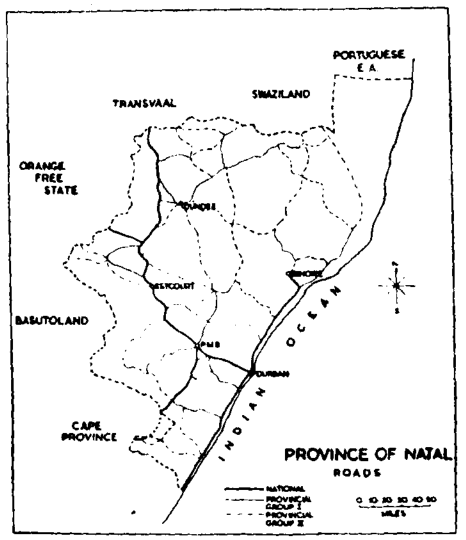
The effect of these ordinances is to speed new construction, since, once the road design has been drawn up, the compensation to the farmer is fixed by mutual agreement between the Government's Chief Roads Engineer and the land-owner. The question of public inquiry never occurs. In the event of disagreement over compensation the case is brought before a Statutory Board for arbitration.

Secondly, the maximum axle load is fixed by law at 18,000 lb, hence the design of the pavement is based on a wheel load of 9,000 lb leading to lighter and, therefore, cheaper construction than is the case in Britain, where the pavement design is often based on wheel loads of 20,000 lb. This law is not difficult to enforce and the transportation companies overcome the difficulty by modifying their vehicles to incorporate an extra axle if the laden weight is excessive. Occasionally this is not possible when heavy industrial goods require transportation and, in such cases, special transport permits are issued similar to those of the Ministry of Transport here. This load limitation is only

* Roads and Traffic Engineer; Brandt and O'Dell Consulting Engineers, Natal.

applied to pavement design. Bridges are designed to full British Ministry of Transport loading, as a bridge failure under excessive heavy loading is much more serious than isolated pavement failures.

The pride of the people of Natal in their roads is very notable and they are prepared to accept these ordinances for the benefit of the community and are happy to see the



road programme developing, with the result that the cases of petty bickering are few and far between.

National and Provincial Roads

Broadly speaking, the roads in the province are divided into two groups: (a) National and (b) Provincial. (a) National roads are the inter-city and inter-province main roads such as the Durban-Johannesburg, the Durban-Capetown and the Durban-Bloemfontein roads, which are also termed the Natal-Transvaal, the Natal-cape and the Natal Orange Free State routes. There are approximately 550 miles of National Roads within the province which are constructed and maintained by the province to a specification laid down by the National Transport Commission, and are also financed by that body. The roads are usually 24 ft wide blacktop carriageways with 8 ft shoulders and are designed to a minimum 50 m.p.h. design speed with a maximum gradient of 5 per cent. The design is naturally based on traffic studies, and dual carriageways are provided in sections of high traffic densities. In general, provision is made in the design for future improvement of all National Roads to dual carriageway limited access (expressway) standard. The bridges are normally designed to allow for doubling the deck width and, in some instances, the abutments to bridges are constructed to allow for this future requirement. Flyover structures and rail-over roadgrade separations are often built incorporating two spans required for their future development.

Typical example of Group II provincial road

(b) Provincial Roads which are financed by the Provincial Council can be sub-divided into two groups.

In Group I are the inter-town roads, built to the same standards as National Roads and with some sections of dual-carriageway where traffic densities are high. There are 470 miles of roads in this group.

In Group II, totalling 6,700 miles, are the minor roads or farm-to-market roads, usually of gravel surface construction. These roads are being improved, in a priority depending on traffic counts, in horizontal and vertical alignment to 22 ft wide blacktop standard with elimination of all railway level-crossings and to a design speed of 50 m.p.h. with a maximum gradient of 7 per cent.

At present, roads in this group with an A.A.D.T. over 100 vehicles per day (A.A.D.T.—Annual Average Daily Traffic) are in line for improvement.

Roads Department Organisation

In Natal, roads are mainly constructed by direct labour, though about 10 per cent of road construction work is let to contract each year. All road and bridge design is carried out by the Chief Engineer for the province, with the exception of one or two structures which are designed by local consultants. The Continental system of calling for tenders to design and construct bridges has also been used and found to be very satisfactory. Bridges costing more than 20,000 pounds are usually let to contract, the remainder of the yearly bridge programme totalling 3/4 million pounds per year being undertaken by direct labour. The staff organisation of the Roads Department is outlined in Appendix A.

For the purpose of road maintenance and minor improvement work the province is divided into five districts controlled by District Engineers, each with a complete staff organisation and a definite annual financial allocation under their direction. The entire organisation is under the Chief Roads Engineer who directs the work of the department in accordance with a Five-Year Road Development Plan. The allocation of finances voted by the Provincial Council for Roads is apportioned by him to the various divisions of the department. The financial allocation is based on a budget prepared by the Chief Roads Engineer and

Example of bridge design by chief engineer's department, in reinforced concrete

Example of prestressed bridge designed by chief engineer's department

is met in the following ways. For National Road construction and maintenance the allocation is derived from fuel tax ; in the case of Provincial Road construction it is met from Capital funds, with a 20-year redemption period for roads and a 40-year period for bridges. Maintenance of roads is met from revenue, derived mainly from an annual vehicle licence fee based on the weight of the vehicle.

This brief resume of Natal and the organisation of its Roads Department serves as a background to its Traffic Engineering Section, which contributes greatly to the work of the department in all stages of the road programme.

Traffic Studies and Surveys

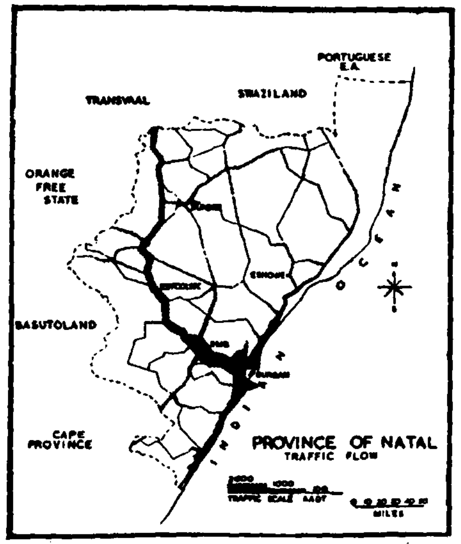
The Traffic Engineer organises and records the traffic statistics for the province. He undertakes traffic studies and loadometer surveys, is advised of all road accidents by the police, and controls the sign-posting and traffic-marking of roads. From his records and statistics he advises the Chief Engineer and other sections on pre-planning, priorities and design standards.

Traffic censuses are carried out on all roads in the province each year, there being 700 odd counting points, with the duration of the counts varying from 12 hours in any one day to continuous 24-hour counts taken for every day in the year.

The latter permanent counting stations are used to give the traffic trends in particular areas and the day, week or month duration counts within the areas are adjusted to give the A.A.D.T. for the particular road on which the short duration counts are carried out. The method of counting used is the pneumatic tube mechanical recorder, which stamps the time and the traffic volume on a recording tape. The results of these counts are analysed annually and adjusted to A.A.D.T., for which a traffic volume map is produced for the province.

Growth of traffic is then estimated, using the information from the above map to calculate the vehicle-miles, which is then plotted on a semi-log graph against time. The number of vehicles registered in the province are plotted in a similar manner and also the sales of fuel. The latter method gives a good indication of traffic growth, because the miles per gallon run by cars has not changed appreciably in the past 10 years and is likely to remain fairly constant. Consideration of the trends in each of these items enables a reasonable prediction of future traffic growth to be made.

From these graphs it is interesting to note that, between the World Wars, the growth of traffic was 5 per cent compound per year. The decrease in traffic during the second World War was afterwards rapidly brought



back to this 5 per cent line and remained constant at 5 per cent for a number of years. Over the last few years there has again been a marked increase in road traffic, probably due to the growth of population and the economic position of the country, with the result that, since 1956, traffic has been increasing at 7 per cent per annum on the major routes and 6 per cent per annum on all roads, and these figures have now been accepted for design in Natal.

Traffic surveys of various types are carried out by the section for specific design problems, the most important being origin and destination surveys, intersection counts and loadometer surveys.

Three types of O. & D. survey have been used in Natal and are undertaken on lengths of road entailing major improvements, such as complete re-location or by pass roads to towns—in the latter case 'speed and delay' studies are carried out at the same time. The direct interview system is the most common, wherein the area is cordoned off and drivers stopped for questioning by interviewers, supervised by qualified engineering staff and assisted by the police. The driver, after the interview, is handed a card for courtesy, explaining the purpose of the survey and apologising for any delay. A high percentage of the traffic is interviewed as the maximum A.A.D.T. in Natal is only between 3,000 and 4,000 with the exception of the Durban-Johannesburg road where the A.A.D.T. has now reached 14,000 at the Durbanborough boundary.

Other methods of O. & D. survey used have been by recording registration numbers or by the issue of coloured cards at control points. As these methods make the analysis of the results rather more tedious and give less information than the direct interview system, they are rarely used.

Intersection counts are performed by unqualified staff, using manual counters to record the turning movements on a predesigned form. The number of observers used depends on the expected volume of traffic using the intersection and usually 2 to 4 are necessary for a 12-hr. period.

Loadometer surveys are carried out with police assistance at selected points where, in some cases, permanent weigh-bridges have been constructed, or on lay-bys where portable loadometers are used. The police do not enforce the laws previously mentioned, while surveys are taking place, to avoid any tendency for drivers to bypass the check-point and so distort the result. However, they carry out their own checks for law enforcement purposes.

The traffic section analyses the results to set wheel-load standards for design: although the maximum wheel-loads are fixed at 9,000 lb in some areas, this limit is not reached by traffic, in others it is considered that 9,000 lb is not sufficient, depending on the type of agriculture or industry served by the road, and the design wheel-load in such cases is either reduced to 7,000 or increased to 12,000 lb. The soils design is based on the repetition of the damaging wheel-load causing ultimate failure, therefore a wheel-load in excess of the design figure will not necessarily cause surface failure.

Road signs now come under the direct control of the Traffic Engineer, whereas previously the district engineers decided the type of sign required at any substandard point or intersection, with a resulting variation in the use of signs, similar to the variation one finds between

English counties in the use of 'sharp corner' and 'double bend' signs. It was therefore considered necessary in the interest of uniformity to place the decision on signs with one authority and, with his knowledge of traffic, accidents and design, the natural choice was the Traffic Engineer.

Accident records are sent to the Traffic section by the police and are analysed and plotted on strip maps of the roads. These maps are a graphical representation of the roads features, with the road shown as a straight line and the alignment featuring the intersections bridges, etc. In this way the 'black spots' become apparent and are subsequently investigated by the Traffic Engineer, who advises the District Engineer on steps to be taken to effect an improvement in road safety.

These may be improvements in sight distances, road surface improvement either by re-surfacing or increase in super-elevation, or the erection of advance warning signs.

Advising on pre-planning and design

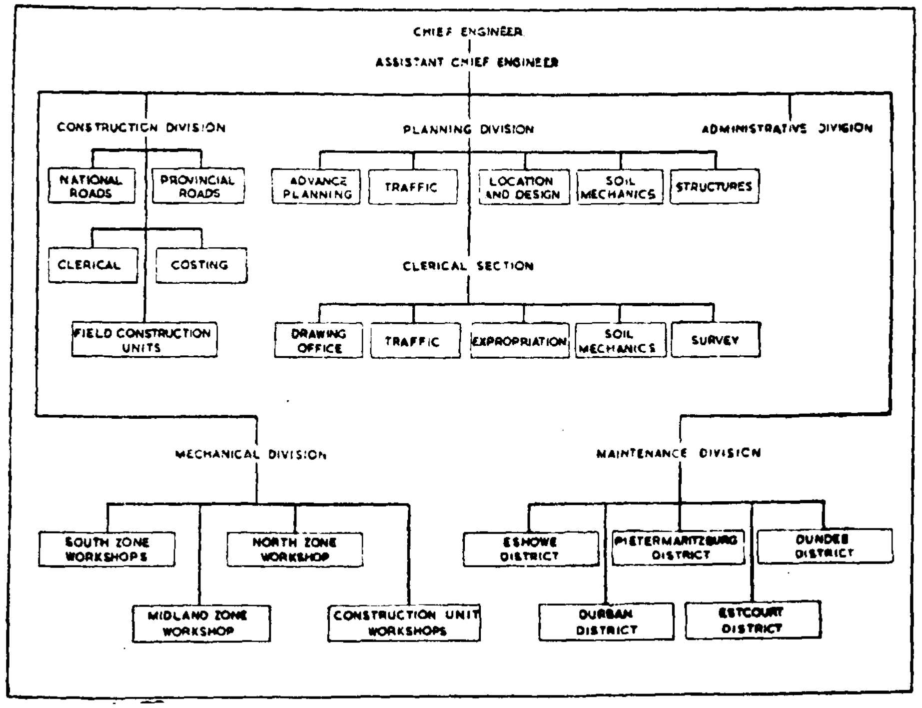
As well as acting as an independent section dealing with the duties outlined above, the Traffic Engineer advises the chief Engineer and the other sections of the department on pre-planning and design standards.

The Chief Engineer requires facts on traffic to substantiate his reports to the Provincial Council on road expenditure and in planning his road programme. Road improvement schemes are therefore prepared on the basis of factual and scientific information which ensures that the work undertaken will be of real benefit to the community and no recommendation to the Council would be considered unless based on traffic figures. A similar procedure is insisted upon by the National Transport Commission (comparable to the Ministry of Transport) when it comes to the allocation of finances from the Central Government for roads and for the design standard of roads.

The location and survey section consults the Traffic Engineer when difficulty is experienced in deciding on the location of a new road. In such cases the Traffic Engineer undertakes the economic analysis of the alternative routes to determine the

Typical example of dual carriageway construction

APPENDIX A



location giving the best 'road-user cost benefit ratio,' and the final survey may then proceed.

It is interesting to note that analyses of this type are used only to compare alternative routes between common points, and not to compare different roads. This type of analysis has also been used to determine when a project will become economic to construct, and in this way has been used to assess relative priorities. The complete analysis is performed by one individual, to obviate any difference which may arise in the selection of the constants used in the calculation. An example of an analysis is given in Appendix B.

Geometric design is based on traffic studies, either on counts alone or on O. & D. studies, whichever the case may demand. Roads are designed for a 20-year economic life, therefore the traffic volume is calculated on the present traffic expanded at a compound rate of 7 per cent or 6 per cent, whichever may apply, over 20 years and the average volume at the end of this period is taken as the design volume. Furthermore, the composition of traffic is also considered, that is, roads with truck traffic (3-ton weight or over) in excess of 20 per cent are termed heavy traffic roads, and below 20 per cent truck traffic are termed light traffic roads. The design standards are therefore as shown in Table I.

TABLE I.

A.A.D.T. 20-year anticipation (at 7% compound)	A.A.D.T. present volume approx.	Surface type	Road width pavement plus hard shoulders		Paved or gravel width	
			Heavy	Light	Heavy	Light
Over 5,000	Over 1,300	Blacktop	4 lane divided			
5,000–1,000	1,300–250	Blacktop	40	38	24	24
1,000–400	250–100	Blacktop	34	32	24	22
400–100	100–50	Blacktop	30	28	22	20
Under 100	Under 50	Gravel	24	24	20	20

Sometimes the full design is not built immediately, for example, two lanes of four required may be built and doubled at a later date. In the case of four-lane dual carriageways, the central reservation is generally 40 ft wide. This standard has been recently introduced to allow for future development, previously the width being stipulated as 16 ft minimum. Where guardrails are to be provided, the shoulder width is increased by 2 ft and the width of the fill by a similar amount.

This is the broad picture of design standards of roadway widths and each case is considered by the traffic engineer, who may recommend higher standards. The 20-year anticipation of traffic is determined by him taking into account development of the area served by the road, which may affect traffic over the design period.

Capacity of roads are based on the thirtieth highest hourly volume which, on the average, is 15.6 per cent of the A.A.D.T. For instance, a practical capacity of 900 vehicles per hour is equivalent to an A.A.D.T. of 5,750.

Pavement Design

Pavement design in Natal is based on consideration of four methods: (1) the C.B.R., (2) the Hveem Stabilometer, (3) the U.S. Highway Group Index and (4) Resistance values from Group Index. For method (1) the design wheel-load is required, and methods (2), (3) and (4) are based on the Hveem design chart as given in the Engineering News Record of July 1948. The chart incorporates a traffic index of the number of passes of a 5,000 lb wheel-load equivalent to the traffic volume using the road over its design life of 20 years.

For this purpose the following information is given to the Soil Mechanics Section by the Traffic Engineer:

(1) The present A.A.D.T.

(2) The A.A.D.T. in 20 years.

(3) The percentage of the A.A.D.T. which is commercial traffic.

(4) The design wheel-load.

(5) The number of equivalent 5,000 lb. wheel-loads, together with a report on any variation in these standards over the length of road under design. A chart showing the relationship of these standards to resistance value and cover thickness is given in Appendix C.

The functions of the Traffic Engineer in connection with maintenance and construction sections have been mentioned previously and relate to prevention of accidents and sign-posting, road-marking, etc.

It is hoped that this article shows to some degree the importance of a Traffic Engineering section in any road design and maintenance organisation, together with its functions and the benefits to be derived.

It poses the question whether we, in this country, are making full use of traffic engineering. Are we looking far enough ahead in our design and design standards? Do we base financial allocation to road improvements on detailed traffic studies? Are we doing enough to find the accident 'black spots' and taking action accordingly?

Acknowledgment.—The author wishes to express his appreciation to his employers, Messrs Brandt and O'Dell, for their assistance in the preparation of the article; to the Chief Engineer, Natal Roads Department and to Mr. Hindle, Traffic Engineer of that Department, for permission to publish and for the wealth of information they were good enough to supply. Photographs by kind permission of the South African Tourist Corporation.

APPENDIX "B": EXAMPLE OF ECONOMIC ANALYSIS

NATIONAL ROAD: *Durban-Johannesburg*: Section 3/2.

Umgeni river to Mooi river.

Two routes were located and the benefit to the road-user of these routes over the existing road is to be found. The common points of all routes are the crossing of the Umgeni and Mooi rivers. The existing road and Route 1 cross the Lions River twice, whereas Route 2 does not cross the river but follows the valley. The construction costs for Routes 1 and 2 are based on a roadway width of 24 ft with two 8 ft shoulders of which 6 ft is surfaced in a contrasting colour.

Existing road : The distance between the common points is 30.5 miles, with 32 m.p.h. average running speed over gradients between 0 and 3 per cent. The horizontal alignment is particularly poor.

Route 1 : This follows the existing road, with improvements to the horizontal alignment, to Nottingham Road where it turns north-east to join Route 2 at a point 23,200 ft from Mooi river. The distance is 27.7 miles, consisting of 22.6 miles at a gradient between 0 and 3 per cent and 5.1 miles between 3 and 5 per cent the average running speed has been taken as 40 m.p.h.

Route 2 : Turns north-east at the Umgeni river and follows the Lions River valley and the old Curries Post road, thence to Mooi river. This route necessitates the construction of a link road 4.4 miles long serving Nottingham Road as well as the maintenance of the existing road to carry local traffic from O. & D. survey the traffic volume on the existing road will be 200 A.A.D.T. The length of this route is 24.7 miles, with 14.9 miles at a gradient between 0 and 3 per cent and 9.8 miles between 3 and 5 per cent; the average running speed is assumed to be 40 m.p.h.

Cost to the road-user in pence for vehicle-miles, based on passenger car units, assuming two-lane highways under free operating conditions with pavements in good condition, are as follows :

Speed	Gradient	Fuel	Oil	Tyres	Repairs	Depreciation	Time
40	0–3	2.03	0.13	0.29	1.00	1.28	2.71
	3–5	2.13	0.13	0.34	1.00	1.28	2.71
32	0–3	1.88	0.11	0.19	1.00	1.28	3.37

The time cost is only applicable to 50 per cent of the traffic

	Road-user costs per annum £	Annual maintenance costs £
Route 1	693,600	10,400
Route 2	656,300	14,400
Existing road	772,000	9,200

Construction costs :

Route 1 : 953,800 Pounds annual repayment 57,500 Pounds.

Route 2 : 1,008,300 Pounds annual repayment 59,300 Pounds.

$$\text{Benefit Ratio } \frac{\text{Road-user Cost (Existing—New route)}}{\text{Annual Cost (New route—existing)}}$$

Benefit ratio to existing road of :

$$\text{Route 1}: \frac{772{,}000 - 6{,}98{,}600}{57{,}500 + 10{,}400 - 9{,}200} = \frac{78{,}400}{58{,}700}$$

$$= 1.335$$

$$\text{Route 2}: \frac{772{,}000 - 656{,}300}{59{,}300 + 14{,}400 - 9{,}200} = \frac{1{,}15{,}700}{64{,}500}$$

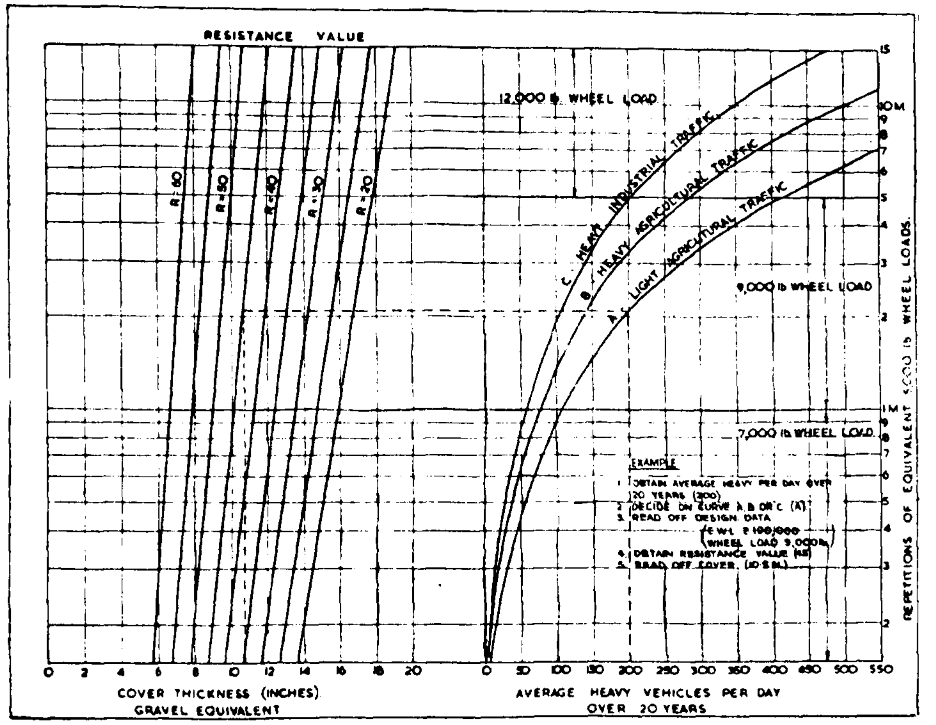
$$= 1.795$$

Conclusion ;

Route 2, though more expensive to construct and maintain, is of greater benefit to the road-user and final design on this location is therefore recommended.

Note.—A project is economic to construct when the benefit ratio exceeds 1.0.

APPENDIX C



(*From 'The Surveyor & Municipal & County Engineer', 12th November, 1960.*)

A salesman was making his pitch on an electric dishwasher to a housewife :

" Madam, if you buy this dishwasher it will help you to save the cost of a maid. You will be saving money every month."

The housewife replied : " Well, I am not so sure we can buy it. We bought an automobile to save bus fare. Then we bought a television set to save movie and entertainment expense. Last week we bought an automatic washer to save laundry bills. You know, mister, I think we are saving about as much now as we can afford."

STANDARD BRIDGE BEAMS IN PRESTRESSED CONCRETE

Savings in cost and design time for short span highway bridges

SOME 45 precast concrete manufacturers with works all over Britain are now ready to supply the new standard bridge beams in prestressed concrete which have been approved by the Ministry of Transport. The Ministry's Chief Bridge Engineer, Mr. H. C. Adams, M.C., M.A., M.I.C.E., stated this week that the Ministry has been in touch with all local highway authorities about the use of standard prestressed beams and were behind the Prestressed Concrete Development Group in the splendid work they had done in preparing the design information.

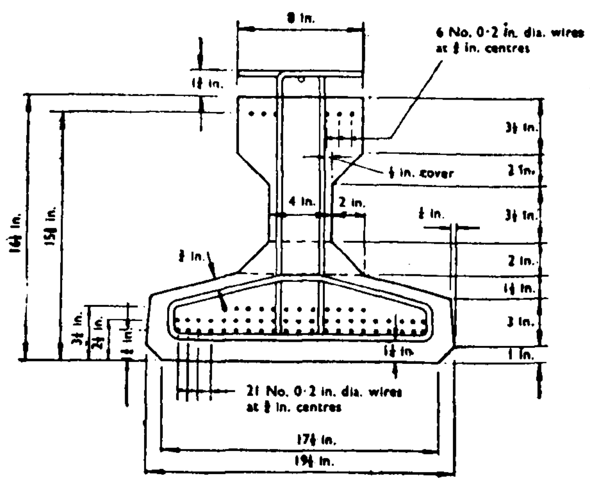
Within the range of spans from 25 ft to 55 ft two types of beam are recommended for consideration; these are indicated in Fig. 1. Both types of beam employ the same top and bottom flange shapes and web thickness, with different web depths. For spans between 25 ft and 35 ft the smaller section is used; for spans between 35 ft and 55 ft the larger section is used with varying top flange thicknesses. Hence, it is possible to standardise the side shutters for the entire range of spans with provision for two web depths, the varying top flange depths being obtained by screeding to different levels inside the standard shutters.

All the beams are designed to withstand the load of the *in situ* concrete and construction traffic and to act monolithically with the *in situ* concrete to carry live-loads. A minimum thickness of 3 in. of concrete is placed over the beams; the precise thickness is determined to meet the design requirements.

It will be noted that the chief items of standardisation are as follows:

1. The width of the beams is 19½ in. so providing three beams for each 5 ft of width assuming a ½ in. joint between units.

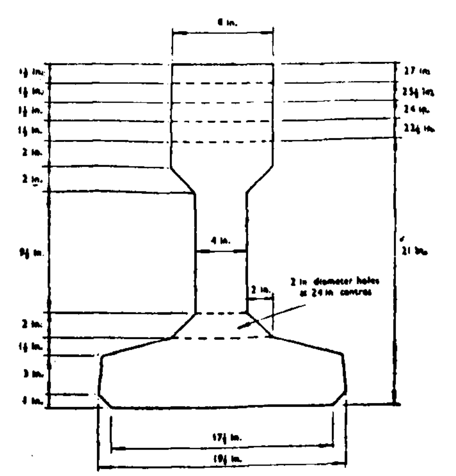
2. The thickness of the web of the beam 4 in.



TRANSVERSE SECTION

Precast section (i) and (ii)

TYPE 1



Precast section (iii) (iv) (v) (vi) and (vii)

TYPE 2

Fig. 1: **Top surface of beam to be roughened**

3. The variation in depth is provided wholly in the top flange for spans between 35 ft and 55 ft.

4. Only the web depth is decreased to cover the range of spans from 25 ft to 35 ft there being provision for variation in the top flange.

5. The shape of the bottom flange is constant with a 2-in. deep fillet with web; this ensures that the holes for the transverse reinforcement occur only within the depth of the fillet and may be inclined to the longitudinal axis of the beam if necessary to meet the requirements of skew bridges.

6. Holes will be provided to accommodate transverse reinforcement equivalent to 0.50 in.²/ft. length; these holes are of 2 in. dia. at 24 in. centres with their centres 6½ in. from the soffit of the beam. It is recommended that the use of holes of greater diameter or at closer spacing should be avoided. The reinforcement may consist of a single bar, or smaller diameter bars of equivalent total area spaced peripherally.

In some cases it will be possible to prop the concrete beams while the *in situ* concrete is placed and this can produce a substantial economy. Account is taken of this in the following calculations: (a) indicating design data for unpropped conditions, and (b) design data when propping is provided. On long spans it will be necessary when propping to support at the quarter points as well as at mid-span.

It should also be noted that if propping is not possible, some economy can be obtained by casting a portion of the *in situ* concrete at mid-span, and allowing this to harden before casting the remainder of the *in situ* concrete. This will enable some of the effects of the *in situ* concrete to be sustained by a monolithic slab.

Details are given of the design assumptions, which have been agreed with the Ministry of Transport, and the section properties, followed by a general summary of the suggested prestressing for the range of spans considered. This prestressing can be provided using 0.2 in. or 0.276 in. dia. high tensile steel or prestressing strand. In the general summary it has been assumed the 0.2 in. wires are to be used but for the largest spans the use of 0.276 in. dia. wires or strand will reduce the congestion of steel. In the case of strand special attention should be given to bond and ultimate load conditions.

It should also be noted that the various sections produced from the standard moulds overlap as regards the range of spans for which they may be used; this ensures that the maximum economy can be obtained by the designer by virtue of having available a number of suitable sections for any given span.

Some examples are given of the design of bridges within the span range considered using the suggested precast sections; these examples are provided as a guide to the engineer concerned as it is anticipated that even when these sections are used the engineer will still prepare general details and calculations for each bridge problem involved.

The Prestressed Concrete Development Group hopes that a general use of these recommendations for small span highway bridges will result in a further economy in construction costs when using prestressed concrete.

Design Assumptions

Precast units:

Stresses at transfer of prestress are less than	$\frac{u_t}{2}$	or 3,000 lb./in.² whichever is the lesser in compression; u_t is the cube strength at transfer 150 lb./in.² tension.
Working stress ...	$\frac{u_w}{3}$	in compression, u_w is the 28 day cube strength zero tension.
Initial steel stress ...	70	per cent of the ultimate tensile strength of wires or of ultimate tensile load of strand.

Section properties

Precast section number	Depth in.	Area in.²	Centre of gravity from top in.	Section moduli (in.³) Top fibre	Bottom fibre	Self weight lb/ft.	Standard beam type
(i) ...	15	150.5	9.53	312	544	157	1
(ii) ...	16½	162.5	10.27	403	665	169	1
(iii) ...	21	174.5	13.40	565	1,002	182	2
(iv) ...	22½	186.5	14.00	703	1,158	194	2
(v) ...	24	198.5	14.60	843	1,310	207	2
(vi) ...	25½	210.5	15.22	983	1,458	219	2
(vii) ...	27	222.5	15.86	1,125	1,605	232	2

PRECAST SECTION (i)

Span ft	Depth of Slab in.	Depth/Span Ratio	M.O.T. STANDARD LOADING Prestress lb.	No. of 0.2 in. ϕwires	Eccentr. in.	Precast Concrete Strength (lb/in.²) At transfer	At 28 days	M.O.T. ABNORMAL LOADING Prestress lb	No. of 0.2 in. ϕwires	Eccentr. in.	Precast Concrete Strength (lb/in.²) At transfer	At 28 days
25 (a)	18	1 : 16.7	161,400	33	2.32	4,500	6,000	190,200	39	2.34	4,500	6,000
(b)	18	1 : 16.7	135,500	28	2.44	4,500	6,000	166,400	34	2.36	4,500	6,000
30 (a)	18	1 : 20	224,800	46	2.29	4,650	6,300	251,500	51	2.27	5,200	6,750
(b)	18	1 : 20	190,500	39	2.33	4,500	6,000	216,500	44	2.31	4,500	6,000
35 (a)	19.5	1 : 21.5	292,000	60	2.24	6,000	8,350	304,500	62	2.26	6,250	8,550
(b)	19.5	1 : 21.5	230,000	47	2.28	4,750	6,000	244,500	50	2.28	5,050	6,000

PRECAST SECTION (ii)

Span ft	Depth of Slab in.	Depth/Span Ratio	Prestress lb.	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days	Prestress lb	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days
30 (a)	19.5	1 : 18.5	205,000	42	2.77	4,500	6,000	228,500	47	2.77	4,500	6,000
(b)	19.5	1 : 18.5	174,000	36	2.86	4,500	6,000	198,000	41	2.86	4,500	6,000
35 (a)	19.5	1 : 21.5	273,500	56	2.72	5,350	7,250	289,000	59	2.70	5,600	7,500
(b)	19.5	1 : 21.5	231,000	47	2.77	4,600	6,000	247,000	51	2.73	4,850	6,000

PRECAST SECTION (iii)

Span ft	Depth of Slab in.	Depth/Span Ratio	Prestress lb.	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days	Prestress lb	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days
35 (a)	24	1 : 17.5	223,000	46	3.63	4,500	6,000	234,000	48	3.62	4,500	6,200
(b)	24	1 : 17.5	181,200	36	3.72	4,500	6,000	192,200	39	3.69	4,500	6,000
40 (a)	24	1 : 20	288,000	59	3.55	5,100	7,850					
(b)	24	1 : 20	233,500	48	3.61	4,500	6,000					
45 (a)	25.5	1 : 21.2	358,500	73	3.49	6,300	9,900					
(b)	25.5	1 : 21.2	273,500	56	3.57	4,850	6,000					

PRECAST SECTION (iv)

Span ft	Depth of Slab in.	Depth/Span Ratio	Prestress lb.	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days
40 (a)	25.5	1 : 18.8	267,000	54	4.24	4,500	6,550
(b)	25.5	1 : 18.8	221,000	45	4.31	4,500	6,000
45 (a)	25.5	1 : 21.2	337,000	69	4.13	5,760	8,500
(b)	25.5	1 : 21.2	277,000	57	4.21	4,750	6,000

PRECAST SECTION (v)

Span ft	Depth of Slab in.	Depth/Span Ratio	Prestress lb.	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days
40 (a)	27	1 : 17.8	257,000	53	4.74	4,500	6,250
(b)	27	1 : 17.8	208,500	43	4.85	4,500	6,000
45 (a)	27	1 : 20	323,000	66	4.66	5,270	7,550
(b)	27	1 : 20	261,300	53	4.76	4,500	6,000
50 (a)	28.5	1 : 21	395,000	81	4.59	6,400	9,300
(b)	28.5	1 : 21	303,500	62	4.67	5,000	6,000

PRECAST SECTION (vi)

Span ft	Depth of Slab in.	Depth/Span Ratio	Prestress lb.	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days
45 (a)	28.5	1 : 18.9	310,000	63	5.17	4,900	6,900
(b)	28.5	1 : 18.9	252,000	52	5.3	4,500	6,000
50 (a)	28.5	1 : 21	382,000	78	5.08	5,950	8,450
(b)	28.5	1 : 21	310,500	63	5.17	4,900	6,000

PRECAST SECTION (vii)

Span ft	Depth of Slab in.	Depth/Span Ratio	Prestress lb.	No. of 0.2 in. ϕwires	Eccentr. in.	At transfer	At 28 days
50 (a)	30	1 : 20	368,000	74	5.46	5,500	7,650
(b)	30	1 : 20	297,000	61	5.57	4,500	6,000
55 (a)	31.5	1 : 20.9	448,000	91	5.48	6,750	9,050
(b)	31.5	1 : 20.9	342,000	70	5.50	5,200	6,000

Notes:—(a) For sections (iv) to (vii) the M.O.T. Standard Loading is more critical than the abnormal loading and hence the relative values for the abnormal loading are omitted from the tables.

(b) The highest values of concrete strength shown in the tables are not recommended, but are shown for purposes of interpolation only. It will frequently be more economical to choose a deeper section.

(c) The tensile stresses in the top of the beams should generally be checked when interpolation is used.

Loss of prestress ...	25 per cent, of which 5 per cent occurs immediately as a result of elastic contraction of unit.
Cover to mild steel stirrups Cover to high tensile steel	$1\frac{1}{4}$ in. minimum on exposed surfaces, i.e., on soffit of unit.

It should be noted that the precise losses of prestress should be determined in each case in the manner set out in the Code of Practice for Prestressed Concrete; the figure of 25 per cent has been assumed to simplify the calculations which follow.

***In situ* concrete:**

Minimum cube strength at 28 days. ... 6,000 lb/in.^{2}

Working stresses:

Compression ... ... 2,000 lb/in.^{2}

Tension:

(a) Where abnormal loading is critical, this may be regarded as of an infrequent nature ... ... 675 lb/in.^{2}

(b) Where standard loading applies ... 600 lb/in.^{2}

The above stress of 675 lb/in.^{2} is that given in C.P. 115 for rarely occurring maximum loads; the value of 600 lb/in.^{2} has been taken as being a suitable value, intermediate between those given in the Code for often occurring and rarely occurring maximum loads, on the basis that where the standard loading moment on a slab bridge is greater than the abnormal loading moment, it is reasonable to assume that the maximum loading is then somewhat of the nature of a rarely occurring load.

Since the *in situ* concrete and the precast concrete are virtually of the same quality, it may be assumed that their moduli of elasticity are not significantly different. The effect of differential shrinkage has been ignored in the calculations; this seemed a warranted assumption as recent tests [1] have shown.

Mild steel ... ... 18,000 lb/in.^{2}

Loading

The live loading was taken as either the Ministry of Transport's abnormal or standard loading. For this type of bridge, and over the range of span considered, the M.O.T. Standard Loading is identical with the standard H.A. Loading of British Standard 153, Part 3A, 1954. In dealing with the abnormal loading, use was made of the suggestion by Holland [2] that an increased standard loading in the span range 20 ft to 40 ft caused equivalent stresses; the increase is 50 per cent at 20 ft span and decreases to zero at 40 ft span. This has been shown [3] to be adequate for 45 units of abnormal loading. Hence, in the span range, 20 ft to 40 ft the abnormal loading is the design criterion; in the span range 40 ft to 55 ft the standard loading is the design criterion.

In addition to the self-weight of the precast units and the *in situ* concrete there will be some form of wearing surface on a bridge. In the calculations, this effect has been considered as an additional uniformly distributed live-load of 30 lb/ft^2. In practice, a camber to the road surface will probably be required; this is best obtained by cambering the structural *in situ* concrete and providing a wearing surface of uniform thickness over the entire carriageway. The effect of cambering the *in situ* concrete on the self-weight stresses given in the paper will, generally, be insignificant since a greater structural depth will be available where the increased self-weight is effective.

General

The span given in the tables always refers to the effective span and may be taken as the lesser of the distances between centre of supports or the clear distance between supports plus the depth of the precast unit.

The transverse design of the bridge can be considered satisfactory for all types of loading since 0.5 in.^{2}/ft of mild steel reinforcement is provided; this amount of steel has been shown to be fully adequate [1] by a series of tests on slab bridges of this type. This steel should, of course, be placed as low as possible in the slab: the position of this steel and its spacing is governed by the holes in the precast units. The centres of the 2-in. dia. holes will be $6\frac{1}{2}$ in. from the soffit of the beam and at 24-in. centres. For bridges of large skew it may be necessary to skew the holes in the precast units.

To sustain the small transverse hogging moments and to ensure that any transverse shrinkage above the precast units is controlled, it is suggested that some transverse steel be provided in the *in situ* concrete with a cover of $1\frac{1}{4}$ in. The recommended steel is 3/8 in. dia. bars at 12-in. centres or 0.11 in.^{2}/ft.

The bearing for the precast units should consist of suitable thicknesses of bituminised felt, rubber, neoprene or equivalent material. For simply supported spans upto 30 ft the bearings at either end would be the same; for larger spans and for a series of simply supported spans some form of dowel bars to the supporting member should be provided at one end of each span.

In the design of the precast units special attention should be paid to the ends of the unit in the transmission length. Mild steel reinforcement should be provided within the transmission length to take the appropriate share stresses and to cover the shear lag effect; details of appropriate steel are given in the examples. In addition, vertical stirrups are needed on either side of the holes through the web of the precast unit since stress concentrations exist there: this steel may be nominal.

With the specified spacing of the individual beams the tolerance required on the lateral bowing or distortion of the beams is $\pm \frac{1}{4}$ in. for all spans.

In the following tables the concrete cube strengths are derived from the stresses obtaining in the concrete members except that 4,500 lb/in.^{2} and 6,000 lb/in.^{2} have been taken as the absolute minimum cube strength at release and at 28 days respectively. The requirements of transmission length may in fact imply greater strengths than those given above; such strengths should be determined in each specific case.

Design conditions will usually require the continuation of the deck section for a width of 5 ft beyond the kerb and this will allow a shallow pipe duct (6 in.-8 in. in depth) to be formed under the footpath or verge. Where provision is required for larger services special ducts may be formed between the end of the extended deck and the parapet beam. In some cases it will be preferable to stiffen the extended deck section over a width of 3 ft to 4 ft by increasing the *in situ* concrete cover and so allow extra space facilities between the end of the extended deck and the parapet beam. Another possible solution lies in the formation of holes upto 12 in. dia. by means of cardboard tubes or other forms in the *in situ* concrete. Suggestions incorporating these proposals are shown in the sketches of Fig. 3.

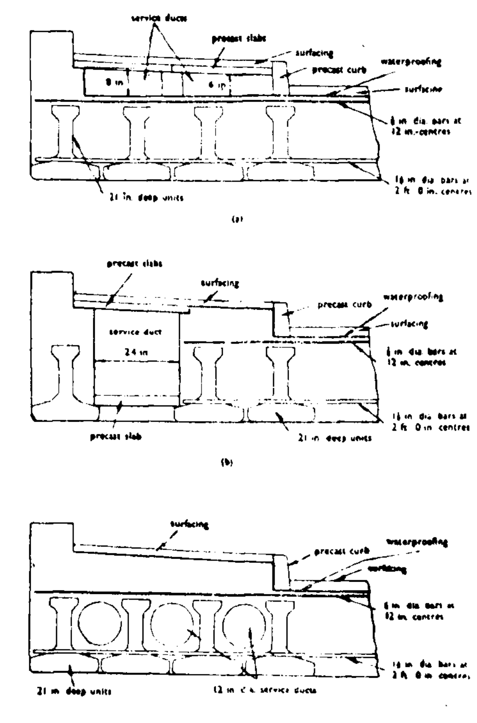


Fig. 3. **Suggested edge treatment. (Precast edge beams may also be used.)**

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IMPROVING SOIL BY CHEMICAL MEANS

LABORATORY INVESTIGATIONS WITH CATIONIC SURFACE ACTIVE AGENTS

By

DR. I. S. UPPAL, M.SC., PH.D., F.R.I.C. *

SYNOPSIS

Effect of addition of small proportions of three commercial cationic chemicals to soil on its resistance to softening effect of water has been studied and it has been observed that while best mechanically stabilised soil cannot withstand the softening effect of water for more than a few hours, the incorporation of only about 0.1% Duomeen-T by weight of soil can make it completely stable to water action for days. Similarly when small proportion of this chemical is incorporated in a bitumen cut-back or a bitumen emulsion, it increases their water proofing property and much lesser quantities of these agents are needed to get desired results. The manner in which these cationic chemicals act has also been indicated.

AT present the most common method of treating soils so that they retain desired properties irrespective of the wetness or dryness of their environment is the mechanical stabilisation consisting of the blending of fine and coarse grained soils in suitable proportions. It has, however, been observed that even the best mechanically stabilised soil generally lacks the required resistance to the softening effect of water as such this method alone is not always suitable for areas where the dampness dominates throughout the year.

In order to overcome this difficulty a large number of chemicals and other admixtures like lime, portland cement, asphalt (in the form of emulsion and cut-back) calcium chloride, sodium chloride, sodium silicate, molasses, gluten, lignin latex, etc., have been tried with varying success. Effect of addition of some anionic and non-ionic surface active agents like neutral soap, sulphonated castor oil, soapnut extract, lissapol-N, etc., on the engineering properties of different alluvial soils of Punjab has been studied[1] and it has been observed that anionic surface active agents like soap and sulphonated castor oil appreciably improve the resistance of soils particularly of those containing higher proportions of coarse grains, to the softening effect of water and also reduce their volumetric shrinkage. This study was now extended to study the effect on the engineering properties of soil caused by addition of some 'Cationic' surface active agents to it.

Three cationic surface active agents sold in the market under the trade names of 'Adhevia-T, 'Cationic Emulsifier' and 'Duomeen-T' were used in this study. These products are used as anti-stripping agents on most types of bitumen or tar roads to prevent failure due to rain. Their use is specially recommended for coating wet stones so that the road works can go on throughout the year irrespective of dryness or wetness of the weather. The action[2,3] of these chemicals is explained below :—

"Each molecule of these 'Cationic Chemicals" has a little 'Head' which attaches itself with incredible force—far, far stronger than a limpet to a rock—to the stone surface, whether there is some moisture on the surface or not. The elongated "body" of the molecule holds on equally strongly to the binder, thus ensuring the solid is coated with an un-interrupted regular layer of binder.

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The important thing to remember is that the 'head' of the molecules cannot stand being occluded in the binder and (just as a droplet of oil will always rise to the surface of the water) by their nature will always dart at terrific speed to the nearest stone surface.

Similarly when a soil is treated with such chemical compounds, the later are more firmly absorbed by polar forces on the solid surfaces than water itself, thus reducing the response of soils to the action of water or electrolytes. When a soil thus treated is dried beyond a certain point, it often also becomes non-wettable by water.

Such treatments also indirectly affect pore water by altering the degree of aggregation of the soil particles and hence the void volume.

Results of some Laboratory investigations on the efficacy of these surface active agents as anti-stripping agents, as well as their probable chemical composition have already been published[4] in the Transport-Communications Monthly Review, May, 1960, issue. In this note some preliminary observations on the effect of these surface active agents on the Engineering Properties of Soil, particularly on its resistance to softening effect of water have been recorded.

A graded soil having a P.I. of about 9% and sand content (fraction coarser than 200 mesh) about 30% was used. As the treatments which modify soil surface properties require extremely small quantities of chemicals and also in view of the economy of the treatment, the proportions of the surface active agents used in this study were only 0.1 to 0.3% by weight of soil. The optimum moisture of the soil was first determined as usual by means of the Modified Abbott's compaction apparatus[5]. The requisite quantity of the surface active agent was dissolved or suspended in the optimum quantity of moisture so found and the solution/suspension thoroughly mixed with the soil and the mixture compacted under the same conditions. The dry density of the resultant blocks was determined, no appreciable difference due to the presence of chemicals was noticed. The presence of these proportions of chemicals also did not show any change in the optimum moisture when tested under similar conditions.

Resistance to softening effect of water

The dry blocks containing varying proportions of cationic chemicals were kept over sand contained in a tray and kept constantly wet, and the moisture absorption through capillarity was determined after 6, 24, 48 and 144 hours. Also some dried blocks were placed (immersed) in water contained in a trough (at 25°C) to see the effect of chemicals on water absorption by the treated soil under these conditions; and the condition of the blocks and the per cent loss (on dry basis) after 5 hours immersion due to the disintegration (if any) was determined. The results of all these tests are presented in Table I. It would be seen therefrom that Duomeen-T treated blocks showed minimum water absorption and maximum stability under both the conditions. The other two chemicals, viz., Adhevia-T and Cationic Emulsifier also gave encouraging results. In the case of Cationic Emulsifier smaller quantities were found to be more effective, while in the case of Adhevia-T the efficacy of the chemical increased with the increase in its quantity from 0.1 to 0.3% by weight of soil.

Effect of Adhevia-T and Cationic Emulsifier was also studied on a silty soil of P.I. 8.7 and sand content less than 20% with similar results as in the case of graded soil. These results are also included in Table I.

The compressive strength of the dry soil blocks containing these chemicals was slightly adversely affected with the increasing quantities of the chemicals. However, the addition of all these chemicals appreciably improved the penetration resistance (as determined in terms of CBR) of the compacted soil-mass in wet state. This improvement was anticipated because of the waterproofing effect of the chemicals on the soil particles. While the ordinary soil without any admixture showed a soaked CBR value of 1.6%, the addition of 0.2% of these chemicals improved it to over 6%. The moisture absorption during 96 hours soaking also decreased from 23% in the case of untreated to about 13% in the case of soil treated with 0.2% cationic chemicals.

TABLE I

Effect of Cationic surface active agents on the resistance to softening action of water and moisture absorption by soil blocks

Sl. No.	Name of Admixture (Cationic)	Percentage used	Per cent moisture absorbed through capillarity after — Hrs.				Time of disintegration when actually immersed in water. Per cent loss after 5 hours' immersion
			6	24	48	144	
I—Graded Soil (P.I. 8.6 per cent ; Sand content 48 per cent)							
1	Blank	—	14.3	16.1	17.1	17.4	Disintegrated within ½ hour
2	Adhevia–T	0.1	4.7	14.6	15.0	15.0	Do. 5 hours
		0.2	4.5	8.4	10.0	11.0	Per cent loss 6.8
		0.3	2.0	4.0	5.5	7.4	Per cent loss 6.1
3	Cationic Emulsifier	0.1	2.5	5.8	7.2	8.2	Per cent loss 7.1
		0.2	4.0	9.3	11.3	13.3	Per cent loss 17.7
		0.3	4.0	7.5	8.7	10.4	Per cent loss 10.0
4	Duomeen–T	0.1	4.5	6.0	6.4	7.5	Stable for more than 1 month without any disintegration when constantly kept immersed in water.
		0.2	1.7	2.7	3.8	4.2	
II—Silty Soil (L.L. 31.8 per cent ; P.I. 8.7 per cent and Sand content 17.7 per cent)							
1	Blank	—	Became very soft and could not be handled				4 minutes
2	Adhevia–T	0.1	20.3	21.0	—	—	76 hours
		0.2	8.0	13.0	—	—	Stable for more than 5 days
		0.3	4.0	7.5	—	—	Do.
3	Cationic Emulsifier	0.1	7.3	16.2	—	—	Do.
		0.2	6.3	11.6	—	—	Do.
		0.3	7.0	11.5	—	—	Do.

Effect of addition of cationic surface active agents to the bitumen on its water-proofing properties when used in the form of cut-back or emulsion

Since these cationic chemicals have proved very good anti-stripping agents (due to their water repelling property) when incorporated in the asphaltic materials used for coating stones, it was considered desirable to try their effect for improving the water proofing properties of asphaltic materials like bitumen emulsions and cut-back when used as soil stabiliser and for water-proofing mud plasters. Accordingly bitumen cut-backs consisting of 80 parts of 80/100 bitumen, 20 parts of kerosene oil and 1 part of chemical were prepared in the laboratory. In the case of bitumen emulsions, varying quantities of each chemical from 0.5 to 2%

(by weight of bitumen) were incorporated at the time of preparation of the emulsion in the laboratory using 2% T.R.O. by weight of bitumen as emulsifying agent. The soil blocks containing 2.5% and 5% of the cut-back and 5% emulsion, containing different chemicals by weight of the dry soil, were prepared as usual by compaction at optimum moisture in modified Abbott's apparatus and tested for absorption of water through capillarity and by actual immersion in water. The results are shown in Tables II and III. It will be seen from these results also that Duomeen-T imparted maximum resistance to water effect in both the cases. The soil blocks prepared from a 5% cut-back containing Duomeen-T absorbed only 2% moisture after 72 hours immersion while the moisture absorbed by blocks prepared from cut-back containing other chemicals was as high as 16 to 17%. In fact 5% cut-back containing no chemical gave better results than those containing the other two surface active agents although

TABLE II

Effect of addition of cationic surface active agents to the bitumen on its water-proofing properties when used in the form of a cut-back or emulsion

(Percentage water absorbed through capillary action)

Sl. No.	Name of Agent used	Percentage by weight of Bitumen	Per cent water absorbed by weight of block containing						
			2.5 per cent cut-back			5 per cent cut-back			
			Duration of test in Hr.			Duration of test in Hr.			
			6	24	48	6	24	48	72
1.	Blank, i.e., no agent in cut-back	—	11.7	15.1	16.3	3.5	8.7	14.8	
2.	Adhevia-T	1	6.3	10.3	13.3	5.6	11.0	16.0	
3.	Cationic Emulsifier	1	6.3	14.0	15.3	9.0	15.6	17.4	
4.	Duomeen-T	1	2.3	5.3	9.3	0.73	1.3	1.9	2.0
	Emulsion								
						5 per cent by weight of soil			
1.	Nil					14.1	18.5	21.5	
2.	Adhevia-T	0.5				11.3	17.4	21.3	22.3
		1.0				14.7	18.5	21.5	22.4
		1.5				13.0	18.0	20.5	22.2
		2.0				10.0	17.5	19.7	21.3
3.	Cationic Emulsifier	0.5				2.0	7.0	12.9	14.2
		1.0				1.7	6.5	12.1	14.4
		1.5				15.3	16.8	17.9	18.1
		2.0				13.4	15.2	15.8	16.3
4.	Duomeen-T	0.5				5.0	15.1	17.1	17.8
		1.0				7.6	16.7	17.3	17.8
		1.5				3.5	9.4	12.0	13.4
		2.0				2.8	9.6	12.0	13.2

TABLE III

Resistance to softening effect of water when actually immersed in water for 24 hours and more

Sl. No.	Name of Chemical	Percentage by weight of Bitumen	Per cent water absorbed					
			2.5 per cent cut-back			5 per cent cut-back		
			Hours of immersion			Hours of immersion		
			6	24	48	6	24	48
1.	Nil	—	Cracks appeared in 5 hours			7.2	15.6	17.5
2.	Adhevia-T	0.5		Do.		—	—	—
		1.0		Do.		12.0	18.3	20.0
3.	Cationic Emulsifier	1.0	—	11.3		—	—	—
4.	*Duomeen-T	1.0	4.3	13.0	15.0	1.3	2.9	4.1

* Blocks prepared from 2.5 per cent cut-backs containing Duomeen-T remained in tact and hard even after 144 hours constant soaking in water. The compressive strength of such blocks in wet condition after 24 hours' soaking was about 150 lb/square inch.

when 2.5% cut-back was used these chemicals appeared to be more effective. The Duomeen-T was, however, really helpful in reducing the moisture absorption both through capillarity as well as under immersed condition irrespective of the proportions of the cut-back used.

When these chemicals were mixed in the emulsion used for soil stabilisation cationic emulsifier in smaller quantities of 0.5 to 1.0% showed definite improvement while Duomeen-T was found to be more effective when used in quantities over 1% by weight of bitumen. The Adhevia-T appeared to have no effect in improving the water resistant quality of such emulsions.

Similar results were obtained when the cut-back and emulsion containing different cationic chemicals were used for the preparation of water-proof and non-erodable mud plasters. Compositions containing these chemicals showed marked improvement over those containing no such chemicals—Duomeen-T on the whole again being better than the other two chemicals.

Further detailed investigations with a view to assessing the economy of such treatment are in progress.

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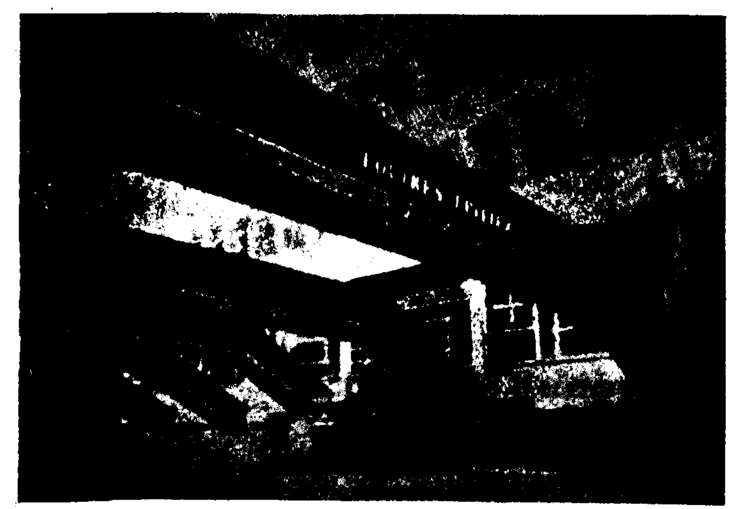
PREFLEX BEAMS

AN important advance for constructional engineering in Great Britain is announced by the Boulton and Paul Group, whose resources for structural steel and prestressed concrete engineering have been combined to manufacture a beam of new design. Known as Preflex, the beam utilises high-tensile steel and high grade factory concrete to provide a high load bearing capacity on a greater range of span to depth ratios, it is claimed, than has previously been economically possible.

Preflex beams combine the best features of high tensile steel and concrete, say Boulton and Paul, It is possible to stiffen a steel joist with an ordinary concrete casing, but when such a beam is loaded to its working stress the concrete around the tension flange becomes tensioned with the

"Hogebrug"—a road bridge over the River Yser at Dixmude. Twelve Preflex beams, each spanning 70 ft, give an overall width of 61 ft, the total construction depth being 2 ft 8 in.

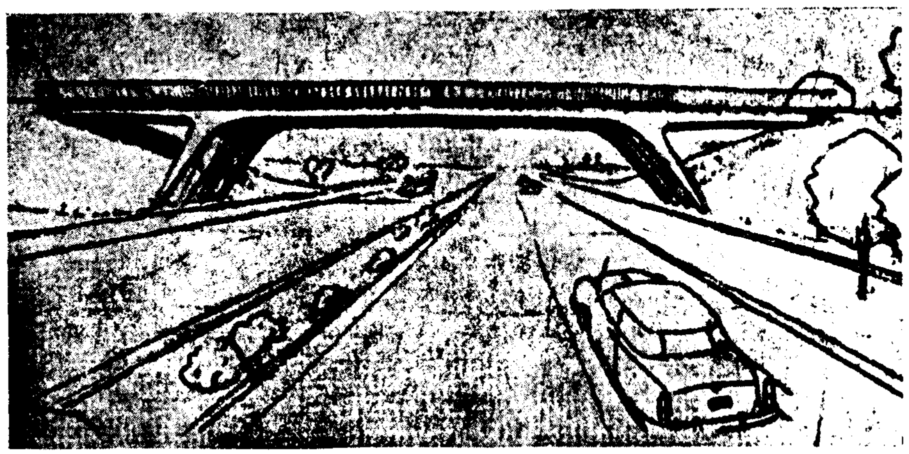


Special use of Preflex beams in a Mansard roof structure steel, loses its stiffening value, and in many cases is subject to severe cracking. These defects are eliminated with Preflex beams.

During manufacture the steel joists are preflexed to take up a similar curvature to that produced by their full working load. Two joists are clamped together at quarter points, one about 18 in. above the other, and the ends are pulled together with hydraulic jacks and sling bolts. Whilst held in this position the joists are placed in a specially designed frame where concrete casings with mild steel reinforcement are cast in moulds around the tension flanges.

Preflexing each steel joist to the maximum design stress, that is to say, ending the steel joist so that the maximum allowable stress is induced in the outer fibres over half the span of the beam, is equivalent to load testing each beam before it leaves the works. The measurement of deflections at all stages of preflexing affords a means of further checking by comparison with calculated values.

The advantages gained by the use of Preflex beams are claimed to be of great importance from the point of view of both economy and planning. Using high-tensile steel to its maximum working stress, less



A proposed motorway bridge of composite design. Preflex beams span between precast concrete columns, with an in situ concrete deck slab

The beams are slowly released when the concreate has hardened and their elastic recovery causes the concrete to become compressed, with a corresponding reduction of stress in the steel joists and a greatly increased stiffness. Thus when the Preflex beam is subjected to working load as part of a structure the concrete casing is decompressed, with no possibility of cracking. Furthermore, the range of stress under load is reduced, giving the beam increased resistance against fatigue. After erection, rigidity is further increased by encasement of the remaining steel profile of the beam. This secondary casing normally forms part of the structure, i.e., the road slab in the instance of a bridge, or floors in a building.

steel is required to support a given loading, and at the same time, less concrete is required in comparison with other systems. Given the span, live-load and allowable deflection, a Preflex beam can provide the solution of any design problem requiring to least depth of construction or alternatively given a construction depth and loading longer spans may be bridged than with any other constructional system.

In buildings minimum floor depths permit reductions in the total height of the incorporation of additional storeys where height is restricted. Freedom of planning may be obtained by the omission of internal columns, and Preflex beams can provide support for secondary members and form-

A pair of 98 ft beams, each 40 in. deep, in position for Preflexing. To induce the required stress into the beams their ends are pulled together by means of the jacking arrangement shown on the exreme left. The beams are then clamped in this position and removed to a specially designed frame where the tension flanges are encased in concrete.

work, allowing clear working space at all floor levels.

For bridges and tunnels reductions in construction depth lead to savings in the cost of approach works. Erection problems can be simplified as special lifting points are not necessary and support for the deck is provided without the need of propping.

Preflex beams can be incorporated into any form of construction, say Boulton and Paul, whether structural steelwork, reinforced or prestressed concrete, since the final beam is composite and connections will be of the normal beam end type for steelwork and monolithic in the case of concrete. Holes can be provided in the webs of the beams, to allow such services as heating, ventilating or air-conditioning ducts to pass within the construction depth of the floor. The elimination of tension cracks in the concrete casing means that complete protection is given to the steel joist against attack by fire and corrosion.

Preflex beams were invented in Belgium, and since 1951 have been used with great success there and in many other continental and overseas countries in the construction of numerous commercial and administrative buildings, garages, schools, hospitals, exhibition halls, factories, tunnels and bridges. Preflex development in Great Britain is a joint enterprise of Boulton and Paul Ltd., Structural Steelwork Engineers, and Anglian Building Products Ltd., Prestressed Concrete Engineers.

A brochure, illustrated by diagrams and photographs describing the Preflex beam, its manufacture and advantages, is available from Boulton and Paul Ltd. of Norwich.

(From ' Highway and Bridges and Engi-Works ', December 14, 1960.)

SURFACE DRESSING

(Comment by P. D. Bell, B.Sc. (Eng.)

In the article on Surface Dressing by Mr. G. P. Jackson published in the T.C.M.R. for June 1960 Mr. Jackson's graph may be expressed mathematically in the form:

$$r = fv \ldots\ldots\ldots\ldots (1)$$

where r = rate of application of binder.

f = fraction of aggregate volume to be filled with binder, to be adjusted according to intensity of traffic and other factors.

v = quantity of aggregate per unit area based on A.L.D. which he assumes to be the "average size of the chippings."

The writer is in full agreement with this basic approach in so far as the determination of the application rate of bitumen is concerned. Any given aggregate when spread on the road surface has a certain percentage of voids. In a seal coat it is desired to fill part of these voids with binder. The quantity of binder may therefore be expressed as a percentage of the total volume of aggregate per unit area. If this percentage be taken as f, the above formula, using our customary units, may be developed as follows:

Let $n = \frac{1}{v}$ = cover ratio of aggregate in sq. yds./cu. yd.

Let r = binder application rate in galls./sq. yd.

Then

$$f = \frac{\frac{r}{6{\cdot}25}\text{ cu. ft./sq. yd.}}{\frac{27}{n}\text{ cu. ft./sq. yd.}} \times 100$$

Whence $r = \frac{1.7\,f}{n}$ galls./sq. yd.

Adding an adjustment, a, for the variation of the existing surface texture, we get:

$$r = \frac{1.7\,f}{n} + a \ldots\ldots\ldots\ldots (2).$$

This is the same form of equation attributed to Hveem published in the African Roads and Transport March/April, 1958, issue.

Let us now examine the components of this formula and compare Mr. Jackson's recommendations with local experience.

Spread Ratio of Aggregate 'N'

The original A.L.D. method of determining the aggregate spread ratio, as expounded by Mr. F. M. Hanson, is undoubtedly sound. It is, however, a little tedious, and this fact has apparently tempted Mr. Jackson to use the average dimension based on the nominal size of aggregate. He then uses a shape factor, only to determine the binder application rate. This seems illogical since it is well known that the shape affects the aggregate spread ratio considerably. For instance, a $-\frac{1}{2}$ in. $+\frac{1}{4}$ in. crushed stone may have a spread ratio anywhere between 90 and 120 sq. yd./cu. yd., depending on the shape of the aggregate.

An alternative method is the use of a tray or board on which the aggregate is spread to simulate conditions on the road. Either the volume of aggregate or the surface area may be varied, keeping the other constant. This method has the merit of being very simple and quick to use. The results agree closely with the A.L.D. method. Irrespective of what method is used to obtain n, the value may be substituted in formula (2) to obtain r.

Volume Percentage 'F'

The effect of continued traffic on a seal coat is to reduce the aggregate voids by compaction. This acts in three ways:

(i) Direct force reducing the thickness of binder film between chippings and road surface, forcing the binder up into the voids.

(ii) Rearrangement of the position of the individual chippings so as to interlock in a more compact manner.

(iii) Attrition and crushing of chippings.

The percentage voids is therefore not a constant but continually decreases under the action of traffic. It may be about 20 per cent initially and may finally reduce to about 10 per cent.

The rate of densification depends on the interrelation between the compacting effort and the resistance offered by the viscosity of the binder. The crushing strength of the aggregate, of course, also has an important bearing on the compaction.

The compactive effort of the traffic can be assessed, at least relatively, by means of a rough traffic count. By comparing the behaviour of seal coats on the Witwatersrand (using crushed quartzite aggregate) after two or three years service, it was concluded that the following percentage bitumen should be used:

Vehicles per day	Bitumen percentage by volume f
Less than 150 ...	13
150—500	11.75
500—1,000 ...	10.75
1,000—2,000 ...	10
More than 2,000	9.5
More than 2,000 with exceptionally heavy industrial traffic ...	9.0

Comparing this table with Mr. Jackson's, it will be seen that the traffic categories are very similar if commercial vehicles are taken to constitute 20 per cent of the total. The examples based on Mr. Jackson's factors also give results similar to those obtained from the above table.

Mr. Jackson indicates that the percentage of voids in rounded gravel will be higher than in crushed aggregate. This is no doubt the case, but the writer cannot speak from experience on this. It is also noted that your earlier author (1958) recommends against the use of this type of aggregate.

The viscosity of the binder under the prevailing day-time temperatures has a very definite effect on the rate of densification of the seal coat. Mr. Jackson has mentioned the varying effect of climatic conditions. It is felt that this should be related to the viscosity of the binder. For instance, a seal coat with 80—100 pen bitumen in a hot climate may behave in exactly the same way as a seal coat with 150—200 pen bitumen in a place where the average temperature is, say, 15°F. lower. The above table of recommended bitumen percentages was based on 150—200 pen. bitumen, and it is probable that the percentage for 80—100 pen bitumen would be higher. On the other hand, a seal coat with a higher penetration bitumen or a cutback having a higher residual penetration would probably require lower percentages. In using cutbacks, of course, one must take into consideration the volume of volatile fluxing oils lost by evaporation. If the amount of fluxing oil is small, however, it is quite possible that the effect of evaporation on the binder application rate may be

Shape of aggregate influences the binder application rate

cancelled out by the effect of the residual bitumen being softer than the usual penetration grade bitumen.

For example, an MC (5) may lose 5 per cent of its original volume by evaporation, thus requiring a correction of + 5 per cent to the standard formula, but the same bitumen may have a residual penetration of 250, thus requiring, perhaps, 5 per cent less bitumen than for the 150—200 pen. grade.

In this connection it is felt that any correction for use of cutbacks should be made to the volume percentage, and not, as Mr. Jackson recommends, as a constant independent of the spread ratio of aggregate.

Finally, the volume percentage may also be varied according to the temperature susceptibility of the binder.

It may also be pointed out that the viscosity of any binder increases with age. The rate of compaction therefore decreases with time until the binder eventually becomes so hard as to resist any further compaction.

Factor "A" for texture of existing surface

Perhaps the most difficult factor to determine is the allowance for deficiency or excess of binder on the existing surface. In the absence of a reliable physical test this factor must be decided entirely on the basis of judgment and experience. From the writer's observations in the Transvaal it appears that most engineers make insufficient correction to their basic application rates for the existing surface texture. It is also frequently over-looked that the effect of traffic and texture on required binder application rates are often cumulative, such as in the case of a dry "hungry" surface which carries little traffic.

General

Mr. Jackson's graph makes an allowance for 10 per cent whip-off. The writer feels that this is too high. If, of course, too much aggregate is applied in the first instance, the whip-off must be proportionately high, but if the application rate is correct and the conditions are suitable, the whip-off should be within 5 per cent. In fact, if a fair amount of crushing occurs under rolling, one could make no allowance for whip-off and still obtain complete coverage.

Reducing the amount of whip-off, however, is more a question of technique which is outside the scope of this discussion.

A point not mentioned by Mr. Jackson is whether or not his recommendations apply to multiple seals. It is well-known that the percentage voids in a graded aggregate is less than in a single size aggregate. Similarly, where a first seal coat is followed by a second seal of smaller aggregate, the percentage voids is smaller than for a single seal coat. Accordingly a different set of binder percentages is required.

Another point worth mention is the use of fine aggregate such as sand or—¼ in. chippings. In South Africa may miles of roads have been surfaced using RC (2) bitumen at about 0·25 gallons per sq. yard and—¼ in. + 1/8 in. chippings, or decomposed dolerite grit. Similarly, in Southern Rhodesia and in the U.S.A. sand seals are quite common. In these seal coats there is not just a single layer of aggregate particles but perhaps two or three layers. The binder is absorbed by the aggregate forming what resembles a premix surface. The shape and grading of the aggregate will have a very large bearing on the proportion of binder to aggregate, but the concept of volume percentage as described above would be useful in the study of these types of surface dressing where the A.L.D. theory is not applicable.

Conclusion

In view of the numerous widely varying conditions encountered in surface dressing, the proportioning of binder and aggregate must necessarily be done by empirical methods. This being the case, the simpler the method the better, provided it is rational and based on field experience. As Mr. Jackson points out, much still depends on one's judgment, but a rational, systematic judgement.

From "(African Roads and Transport)" June, 1960).

HERE AND THERE

1. SOIL COMPACTION AND PROOF-ROLLING OF SUBGRADES

● A NEW 35-page Bulletin (254), issued by the Highway Research Board, contains four papers on the subjects covered, as presented at the 39th Annual Meeting of the Highway Research Board. These are :

"Full-scale Compaction Studies at the British Road Research Laboratory," by W. A. Lewis, which briefly describes the test installation, outlines the techniques used, and summarises the results of the investigation ;

"Proof-Rolling of Sub-grades," by W. J. Turnbull and Charles R. Foster, which describes various cases where proof-rolling will or will not be effective, and gives information usuable to establish desirable roller characteristic and the amount of rolling that should be used for various conditions ;

"Hydraulic Fill Compaction," by Olaf Stokstad and Kent Allemeier, which deals with the use of flowing water, introduced in end dump fill by jetting, as a compacting and stabilizing agent; and

"Rapid Determination of Liquid Limit of Soils by Flow Index Method," by H. Y. Fang, which presents a new approach to liquid limit determinations by a one-point procedure.

(From 'Highways and Bridges and Engineering Works', January 4, 1961)

2. RECORDING DEFECTS IN HIGHWAY SURFACES BY PHOTOGRAPHY

New Night-Time Detection Method

● A NEW low-cost system for recording by photography defects in highway surfaces has been developed by the Aero Service Corporation in Philadelphia. It enables discovery of road defects at a fraction of the cost of walking or driving the roadway and noting or sketching faults by hand. The new system has performed successfully at test areas in Illinois, and a programme for coverage of major test areas in New York State is in progress. Surveys for several other States are being planned.

Styled Sur-Fax, the system includes a vibration-free continuous strip camera, a specially equipped truck, a film projector and a reader-printer, and is the product of more than a year's research and development by engineers of the Corporation.

Picture of 36-ft wide Carriageway

The camera records highway surfaces as wide as 36 ft in a single traverse, on 35 mm. film, at a scale of 1 in. equals, 36 ft. The truck mounts a battery of shielded, high-powered lights. They provide a constant level of light, beamed at a constant angle to obtain the requisite degree of light and shadow. Under this controlled lighting system, such conditions as faulting, scaling and cracking are readily identifiable.

Surveys are made at night at twenty to thirty miles an hour, generally between 1 a.m. and 5 a.m. to avoid heavy traffic and bright daylight. It has been found that sunlight, with its varying intensity and changing angles to the highway surface, does not produce the constant light source needed for success of the surveys. A generator in the rear of the truck provides power for the lights and the camera. The camera is mounted on top of a pole.

The continuous strip film positives may be projected as large as 10 × 10 ft for staff conference studies. They are also useful for presentation to highway maintenance problems to public officials, legislators and civic groups.

With the Sur-Fax projector, the film may be viewed at a rate equivalent to a slow walking speed—about two miles per hour. The adjustable-speed driving mechanism can be stepped up to an equivalent of about 30 m.p.h. or it can be stopped and 'stills' held on the screen for detailed examination.

For detailed analysis, a specially modified 3. M Reader-Printer is used. The 35 mm. film strip is projected on to an 18 in. × 24 in. viewing screen. As pavement sections come into view which require continued study, a touch of the push button stops the film and makes a positive print. This print is delivered within 30 seconds. It may be annotated and used as an actual work sheet by maintenance crews.

Sur-Fax record forms are provided for engineers and technicians to note the data seen on the continuous film strip. For rigid pavements, such facts include cracks of all types, scale, spalling and faulting. For flexible pavements, the recorded conditions include alligator-type or other cracks, revelling, rutting, settlement and bleeding.

(From 'Highways and Bridges and Engineering Works', January 4, 1961.)

3. TO SURVEY TURKEY TO IRAN HIGHWAY

● FAIREY Air Survey Ltd. has been awarded a contract to map 270 miles of road in Iran as part of a major road reconstruction project to link Turkey with Teheran. The road will form part of the route which links the capitals of Europe with India and the Far East.

The Survey has been ordered by an Iranian firm of Consulting Engineers, E.T.C.O., and comprises the mapping at 1/2,000 scale of approximately 269 miles of road between Zendjan and Marand in North East Iran.

The aerial photography will be executed with the collaboration of the National Cartographic Centre of Iran, with whom Fairey Air Surveys was previously closely associated under a consulting and training agreement.

The film will be processed in Iran and then flown back to Fairey Air Surveys' airfield and base at White Waitham, near Maidenhead, Berkshire, for map-making.

In addition to contoured mapping, permanent "bench marks" consisting of concrete blocks, which act as reference points, are being emplaced at 1,000 metre (1,100 yards) intervals along the whole of the route. These have been placed in position prior to photography and will be "heighted" and co-ordinated in a first order photogrammetric plotting machine. This data will be of particular value in sections where a new alignment is being developed through currently undeveloped terrain (as between Mianeh and Siah Tehaman) since it will provide permanent well-surveyed points from which engineers may set out the final centre line by simple survey methods.

(From 'Highways and Bridges and Engineering Works', Dec. 14, 1960.)

4. FACTS ABOUT ROURKELA

● THE plant consists of the following principal units :

Coking Plant	3 Batteries of 70 ovens each : capacity 1.64 million tons of dry coal per year
Blast Furnaces	3, each with a capacity of 1,000 tons per day
Steel Works ...	3, 40-ton LD Converters : Capacity 750,000 tons per year 4, 80-ton Open Hearth Furnaces : Capacity 250,000 tons per year 2 Mixers each holding 1,100 tons
Rolling Mills ...	Blooming and Slabbing Mill, Wide strip Mill, Heavy Plate Mill, Cold Rolling Mill Hot-Dip Tinning Plant
Power Plant ...	75 MW capacity
Auxiliary and Ancillary Plant	Gas Cleaning Plant with a capacity of 17,650,000 cubic feet per hour Oxygen Plant with a capacityof 349, 470 cubic feet per hour Grey Iron Foundry Repair Shop.

Products :

Heavy Plate ...	200,000 tons/year
Medium-Plate and Wide Strip (hot rolled)	300,000 tons/year
Sheet and Strip (cold rolled)	170,000 tons/year
Tinplate	50,000 tons/year

Products include plate for shipbuilding, wagon construction, bridges, cranes, containers, car bodies, etc. Tinplate for cans, apparatus and tanks of all descriptions.

Some Interesting Figures :

The Rourkela integrated iron and steel works has :

Works railway	6,944 miles
Railway bridges	11
Diesel-electric locomotives	29
Cranes with capacities ranging between 5 and 150 tons	75
Water supply piping	15.5 miles
Drinking water supply piping	11.6 miles
Piping of all dimensions for various uses	46.5 miles in the rolling mill alone
Works roads	26.7 miles
Sewage piping	77.5 miles
Roof-covered area	4,304,000 sq. ft. (included in above figure)
Roof-covered rolling mill area	2,259,600 sq. ft.
High and low voltage cables	1,050 miles

(From "All About Steel,"—The Example of Rourkela.)

RECENT ADDITIONS TO THE LIBRARY OF THE INDIAN ROADS CONGRESS

1. **Swiss Standards** (Nos. SNV-40260, 264, 266, 268, 372, 374, 415, 807a, 70005, 008, 510, 710, 720, 71 110a, 111a, 120a, 410a, 420a, 510b, 511a, 520a): Vereinigung Schweizerischer Strassenfach-maenner (VSS) Seefeldstrasse 9, Zurich-8.

2. **B. S. 1707: 1959—"Hot Binder Distributors for Road Surface Dressing"**: British Standards Institution, British Standards House, 2, Park Street, London W. 1.

3. **B. S. 3179: Part I 1959—"Comparison of British & Overseas Standards for Steels—Part 1 Chemical Composition of Wrought Carbon Steels: B. S., S.A.E. and DIN."**: British Standards Institution, British Standards House, 2, Park Street, London W. 1.

4. **"Policy for Geometric Design of Rural Roads"** (Conference of State Road Authorities of Australia), Deptt. of Main Road, New South Wales, Sydney.

5. **Guide to Concrete Road Construction Questions & Answers**: (Second edition) 1958. Cement & Concrete Association, 52, Grosvenor Gardens, London S. W. 1.

6. **Concrete Bridges, 1955**: Concrete Association of India, 1, Queens Road, Bombay.

7. **Third Congress of the Federation, Internationale de la Precontrainte Berlin—1958** Discussion, Cement and Concrete Association, 52, Grosvenor Gardens, London S.W.1.

8. **Transactions of the American Society of Civil Engineers, Vol. 122, 1957**: American Society of Civil Engineers, New York.

9. **Transactions of the American Society of Civil Engineers, Vol. 123, 1958**: American Society of Civil Engineers, New York.

10. **Transactions of the American Society of Civil Engineers, Vol. 124, 1959**: American Society of Civil Engineers, New York.

11. **Highway Research Board Proceedings of the Thirty-Eighth Annual Meeting, January 5-9, 1959**: Highway Research Board, 2101, Constitution Ave. Washington, D.C.

12. **Instruction Generala Sur a Signalisation Routiere, 1955 (Road Signs)**: La Documentation Technique Du Batiment Et Des Trava Publics, 6 Rue Paul Valery 6, Paris-16.

13. **Swiss Standards (Nos. 40300, 40330, 40408, 40417, 40665, 40668, 40670, 40671, 40674, 70120, 70312, 70315, 70317, 70330, 70340, 70345, 71310a, 71320a, 71710a.** Vereinuigeing Schweizerischer Strassen fach-maenne (VSS) Zurich.

14. **" Road Through Kurdistan " The Narrative of an English in Iraq—(1958)**: Faber & Faber Ltd., 24, Russell Sq., London.

15. **Indian Railways one hundred years 1853—1953**: Ministry of Railways (Railway Board), Govt. of India, New Delhi.

16. **First Report of the Stormwater Standards Committee of the Institution on " Australian Rainfall and Run-off (1958)**: Institution of Engineers, Australia, Science House Gloucester and Essex Streets, Sydney N.S.W.

17. **B. S. Code of Practice—CP 143: Part 3: 1960 " Sheet Roof Coverings Part 3: Lead Roof Coverings"**: British Standards Institution, British Standards House, 2, Park Street, London W. 1.

18. **B. S. Code of Practice—CP 143: Part 4: 1960 " Sheet Roof Coverings Part 4: Copper Roof Coverings"**: British Standards Institution, British Standards House, 2, Park Street, London W. 1.

19. **A. C. I.—Standards—1959**: American Concrete Institute, P. O. Box 4754, Redfor Stations, Detroit 19, Mich. (U. S. A.).

20. **Selected Bibliography on Highway Finance (1951)**: Department of Commerce, Bureau of Public Roads, Washington (U.S.A.)

21. **Standard Plans for Highway Bridge Superstructures Revised 1956**: Department of Commerce, Bureau of Public Roads, Washington (U.S.A).

22. **Cassells' Italian-English, English—Italian Dictionary**: Piero Rebora, Cassell & Company Ltd., London.

23. **Swiss Standards (Nos. SNV-40138, 40142a, 40190a, 40191a, 40550, 40776, 71716, 71743, 71752, 71755, 71820**: Vereinigung Schweizeris Cher Strassen-fac-maenner (VSS) Seefeldstasse-9, Zurich-8.

PERSONAL AND MISCELLANEOUS

We record with deep regret the death of Shri V. S. Annaswami, on the 19th March, 1961 due to sudden heart failure. He was a member of the Indian Roads Congress.

It is fine to get up in the world provided it doesn't make you look down on others.

Mother to daughter : " Marriage is a give and take proposition. If he dosen't give you enough, take it."

The desirable quality of a nose is not its length, breath, or curve, but its ability not to be found in other people's business.

LIST OF APPROVED WORKS ON NATIONAL HIGHWAYS

S. No.	Job No.	Name of the work	Amount (Rs in lakhs)
		ANDHRA PRADESH	
1.	2596-AH-5	Constructing a bridge across Mukteswaram canal on N.H. No. 5 (Diversion).	2.06
2.		Two works each costing less than 2 lakhs. ...	0.03
		ASSAM	
3.	2608-AS-37	Construction of a diversion from 4th mile of North Gauhati-Amingaon-Hijo with a link road to proposed Brahmaputra bridge approach.	5.48
4.	2610-AS-37	Metalling and surfacing miles 325/2 F.P. to 331 MP of Assam-Trunk Road east, N.H. No. 37 in Doom Dooma Sub-Division.	7.13
5.	2613-AS-37	Remetalling and surfacing certain portions of miles 179 to 182 of Assam-Trunk Road, N.H. No. 37 in Jorhat West Sub-Division.	3.28
6.		Three works each costing less than 2 lakhs ...	2.01
		DELHI	
7.	2614-DLH-34	Construction of a bridge over River Jamuna behind Humayun's Tomb in Delhi (Main Bridge).	54.4
		KERALA	
8.	2600-KR-47	Blacktopping the portion from 1/0 to 8/5 of Alwaye-Edappally Road, N.H. No. 47.	4.22
8A.	2584-KR-47	Improvement to National Highway No. 47, Trichur-Variampara Road—Miles 4/7 to 8/0.	3.14
9.	2607-KR-47	N.H. No. 47—Blacktopping Alwaye-Ankamali Road—Miles 1/2 to 6/4.	2.23
10.		Three works each costing less than 2 lakhs ...	2.58
		MAHARASHTRA	
11.	2585-MR-4	Widening to two lane traffic (including land acquisition and geometrical improvements) —Miles 110/0 to 127/2 of Poona-Bangalore Road, N.H. No. 4.	3.33
12.	2599-MR-4	Widening of road width to 38 ft along, N.H. No. 4 including land acquisition in miles 148/5 to 157/3 of Poona-Bangalore Road.	2.69

S. No.	Job No.	Name of the work	Amount (Rs in lakhs)
13.		Three works each costing less than 2 lakhs ...	2.52
		MADRAS	
14.	2609-MD-45	Widening B.T. surface to 22′ and formation to 38′ in miles 215/0 to 240/0 (within Pudukkottai Division) on N.H. No. 45.	8.28
		MYSORE	
15.	2587-MY-4	Widening the formation of Poona-Bangalore Road from miles 157/3 to 210/2 in Mysore State, National Highway No. 4.	2.51
16.	2597-MY-4	Widening the narrow tank bund from miles 13/5 to 14/6 of Madras-Cannanore Road, N.H. No. 4 in Mysore State.	2.06
17.	2601-MY-4	Widening the formation width of Poona-Bangalore Road, N.H. No. 4 to 38′ in miles 216/4 to 245/2 of Poona-Bangalore Road.	4.54
18.	2604-MY-4	Widening formation to 38′ of Poona-Bangalore Road, National Highway No. 4, between miles 245/1-385 to 287/0.	2.66
		ORISSA	
19.	2581-RS-6	Improvement such as metalling, blacktopping and regrading Bisoi-Karanjia, Mahadodull section—Miles 32 to 65 of N.H. No. 6.	8.52
19A.	2594-RS-6	Replacing the existing 32 wooden culverts by pucca ones on N.H. No. 6—Reach III-from Deogarh to Barkote.	2.16
20.		Four works each costing less than 2 lakhs ...	2.30
		PUNJAB	
21.		Three works each costing less than 2 lakhs ...	1.28
		RAJASTHAN	
22.	2593-RJ-8	Construction of C.D. Works in Behror-Kotputli section of new alignment of N.H. No. 8 between Gurgaon and Shahpura.	1.06
		WEST BENGAL	
23.	2612-BG-2	Restoration of damages caused by the Sept.-Oct. 1959 floods to the G.T. Road, N.H. No. 2—Reconstruction of damaged culverts.	1.72

Note.—Works estimated to cost less than Rs. one lakh are not included in the above list,

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