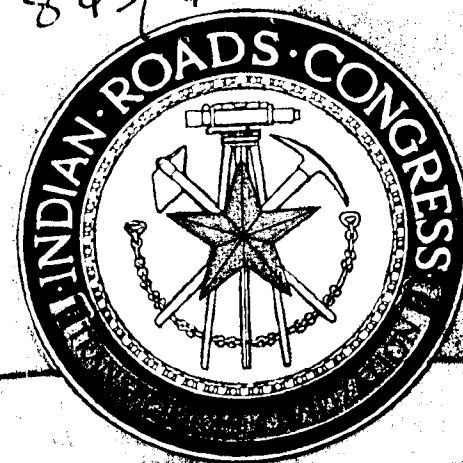


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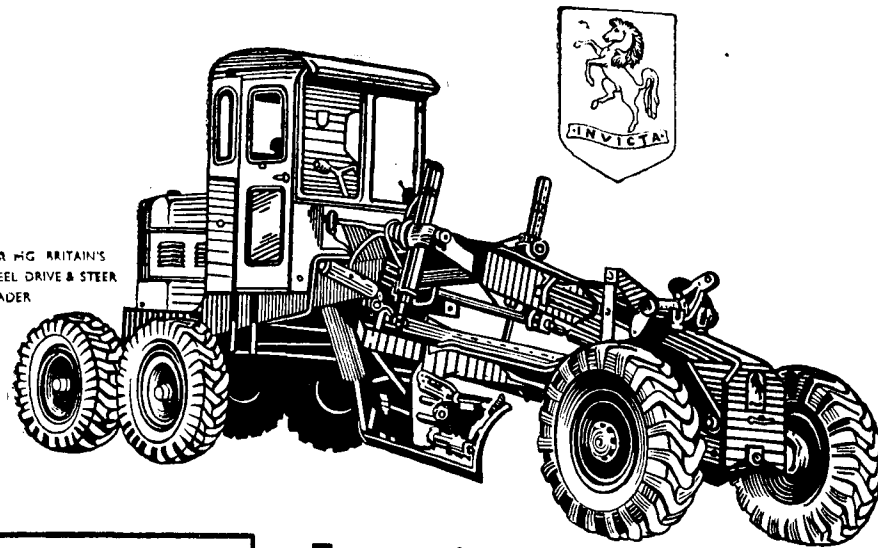
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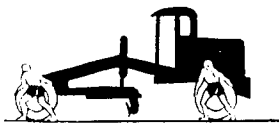
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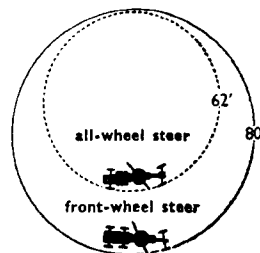


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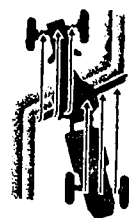
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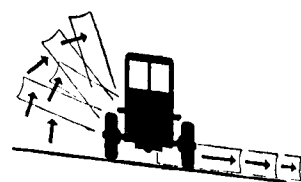
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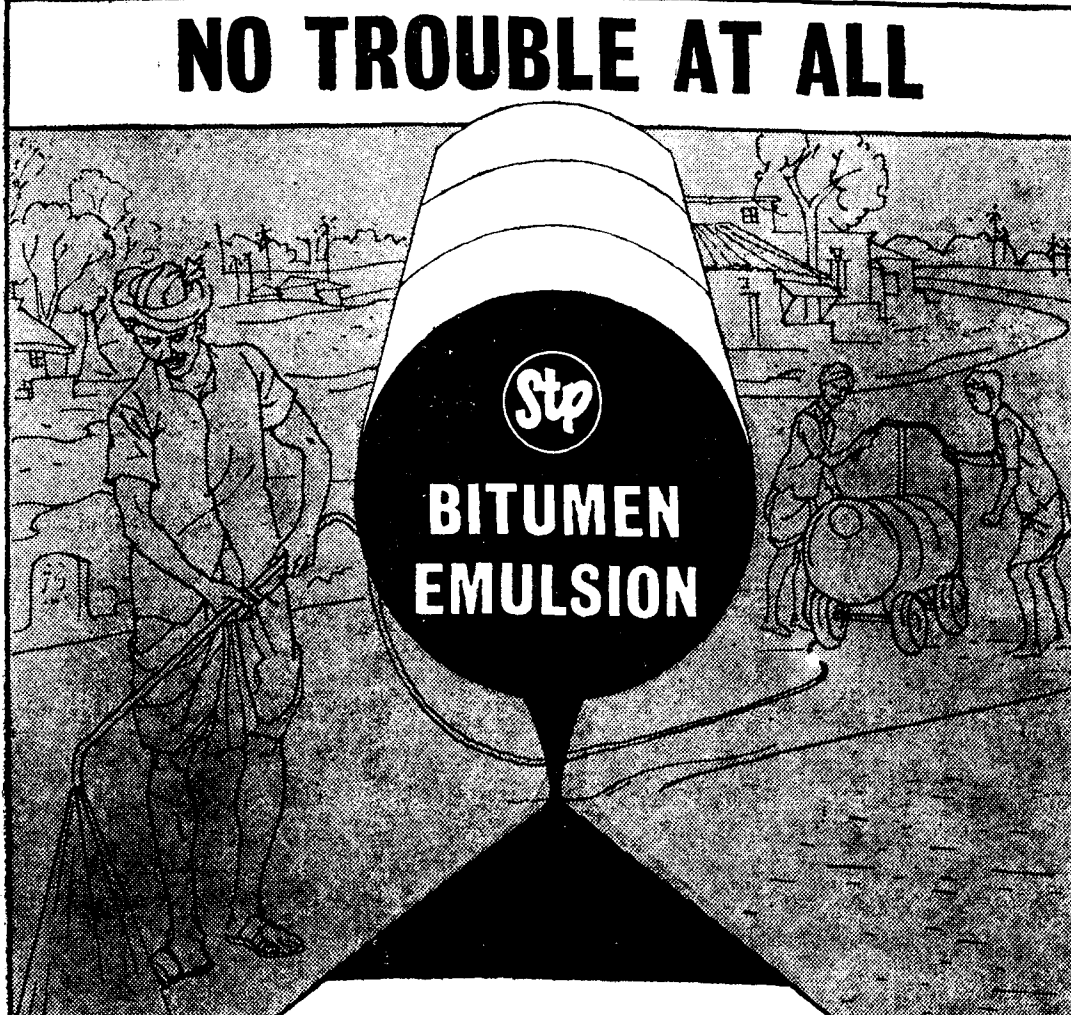
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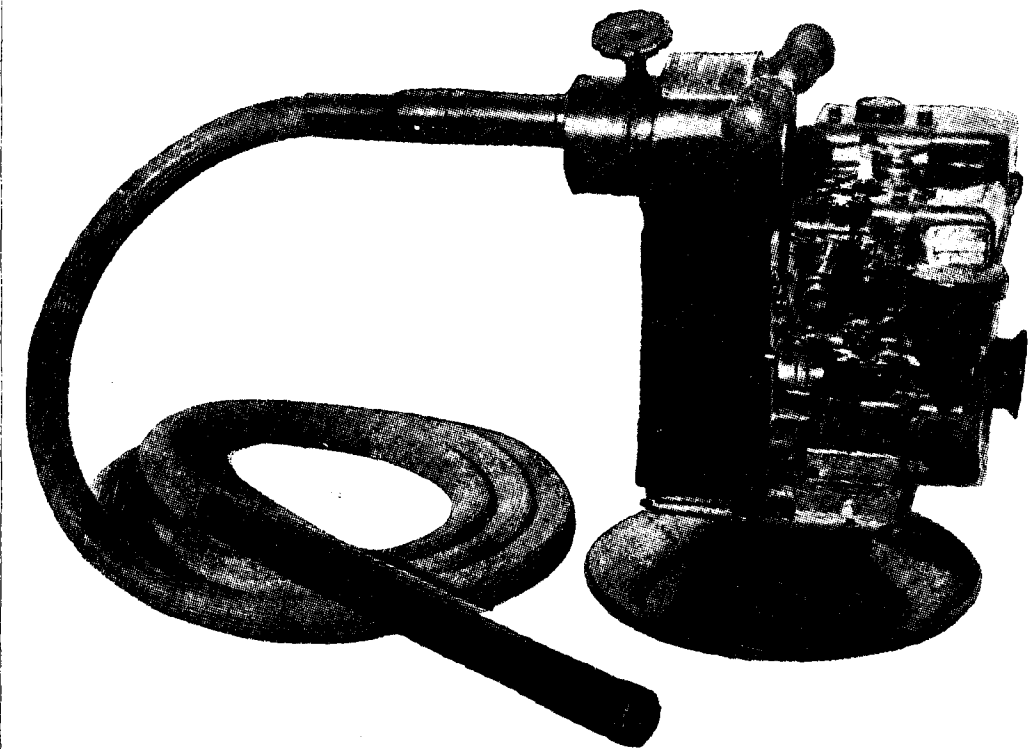
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MONTHLY REVIEW

DECEMBER 1961—No. 176

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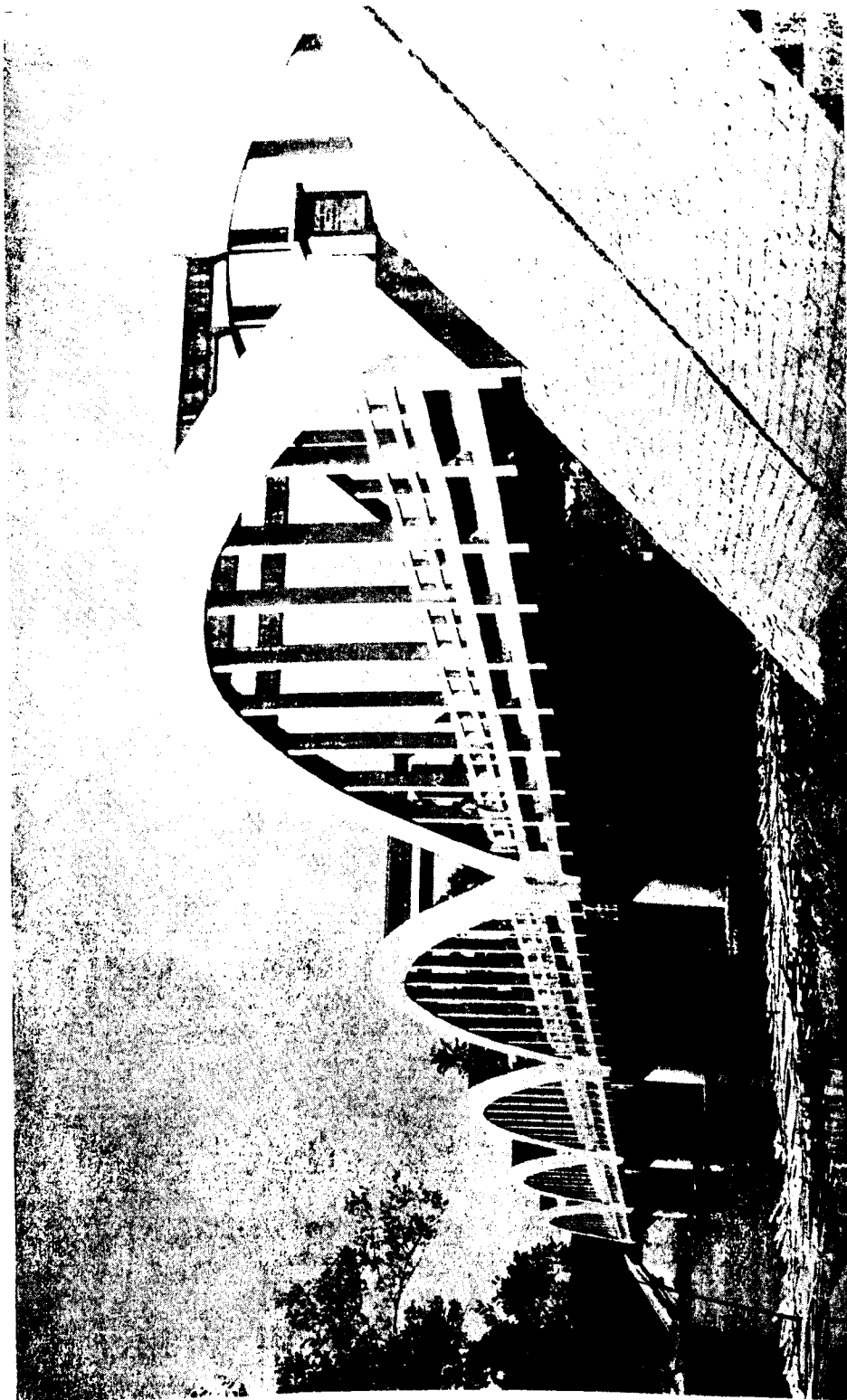
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TRANSPORT-COMMUNICATIONS MONTHLY REVIEW

A REVIEW OF
ROAD ENGINEERING, DEVELOPMENT & TRANSPORT

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DECEMBER 1961

PREVENTION OF RIBBON DEVELOPMENT

Improvement of communications is one of the primary requisites of any scheme aiming at the planned development of an area. This implies widening and upgrading of existing roads, providing bridges over water-ways and the construction of new roads. More important than all these aspects, the improvement of communications includes yet another vital aspect, namely, proper control of road usage and prevention of haphazard building development in parallel strips along roads particularly there "having arterial" character. This results in what is called "Ribbon Development" which throttles the free flow of traffic. Without proper control of this tendency, much of the utility of our efforts towards the expansion of our road network will be considerably defeated. In the context of the phenomenal growth in the population of our country and the rapid trend towards urbanisation, this problem has already assumed serious proportions and is pregnant with grave consequences if not checked in time.

Ribbon development is not only a serious obstacle to the free movement of traffic, which itself is a great loss to the nation by way of reduction in the transport capacity due to loss of time resulting from obstructions, but is also a grave source of traffic hazards. Day in and day out, reports are received of the loss of precious human lives in road accidents. Not a few of these tragic incidents are due to this evil of ribbon development which is fast growing in and around our urban areas.

Various solutions have been suggested to tackle this problem. Acquisition of sufficient land keeping in view the future development is, of course, one solution which suggests itself to one, at the outset. This is, however, circumscribed by financial difficulties and has its own limitations.

Formulation of a comprehensive highway legislation is yet another solution suggested to tackle this problem. The main point made out in making this suggestion is to consolidate at one place and to empower one single authority, viz., the Highway Authority, to exercise the various measures necessary to tackle this problem, viz., fixation of building and control lines, restrictions on buildings between highway boundary and building lines and between building and control lines, payment of compensation for the imposition of restrictions on the erection of buildings, prevention of encroachment and unauthorised occupation on the road, etc. Already some progress has been made in this direction. But the pace is rather slow, especially as framing of a legislation is essentially a time-consuming process and is bound to take time. Nonetheless, this is a very important matter and needs to be given high priority by all concerned.

Yet another solution suggested, is to educate the public opinion through the media of poster publicity, films, radio, etc. It needs hardly to be emphasised that unless we educate the public opinion about the great evil that lies behind haphazard building development alongside roads, no amount of legal action is going to provide any permanent solution to the problem.

Group Development is another solution which can ensure a safe pattern especially in newly developed areas. Steps should, therefore, be taken to encourage this pattern having self satisfactory road circulation system by planning in advance the development of self contained units sufficiently away from the roads. This applies not only to residential areas but also applies in greater measure to factories and workshops where concretion of traffic is likely to be more intense. Control over access to roads especially the arterial roads is also important and needs to be given sufficient attention.

It is true that there is already a growing awareness to the gravity of this problem, but the pace at which it is being tackled is not satisfactory and needs to be stepped up considerably if any tangible results are to be achieved commensurate with the magnitude of the evil. It is, therefore, high time that the entire problem is reviewed in detail and steps are taken to gear up the whole machinery to ensure expeditious action on all fronts.

"I consider the existence of good roads as a vital factor in determining whether our society can become an equalitarian homogeneous one or will continue merely as a conglomeration of social groups leading their own separate lives and maintaining different levels of social and economic development."

—DR. RAJENDAA PRASAD

GUJERAT STATE ACHIEVEMENTS IN CONSTRUCTION OF ROADS AND BRIDGES IN THE SECOND FIVE-YEAR PLAN AND ASPECTS OF THE THIRD FIVE-YEAR PLAN

"The dominant economic fact of our own age the development not of the manufacturing, but of the transport industries".

Marshall



U. J. BHATT

Chief Engineer, Public Works
Department, B & R, Gujarat
Ahmedabad

Introduction

THE recently constituted State of Gujarat has population of 16.25 millions and area of 72,137 sq. miles, which includes a number of erstwhile Princely States. Most of these Princely States neither individually nor collectively had either the vision or the resources for launching a comprehensive road development plan of this region; as such, this State has to-day a very low road mileage in spite of the efforts made by the ex-Saurashtra and Bombay States.

The data relating to mileage of new roads constructed and expenditure incurred in this State, during the First 5-Year Plan, Gujarat State will show 15,000 miles of roads against the Nagpur Plan target of about 24,000 miles. Excluding sub-standard roads, the mileage of roads constructed would be 13,202 miles.

Achievements of Second 5-Year Plan

During the Second 5-Year Plan, completion of National Highways No. 8

and 8B and most of the State Highways are important landmarks on the road development of this State.

Utility of these roads would have been incomplete but for the construction of various bridges, including the prestressed concrete bridges across river Rangholi on Rajkot-Bhavnagar road (S.H.), river Mahi at Vasad on N.H. 8, river Purna on N.H. 8, river Tapti at Mandvi; balanced cantilever bridges across rivers Khari, Rupen, Pushpavati, and Saraswati on Ahmedabad-Mehsana-Abu road (S.H.); R.C.C. solid slab type submersible bridges on rivers Khari and Rupen on Sami-Panchasar road (S.H.); steel truss bridge on river Uben on Vanthali-Manavadar road, R.S. joist, R.C.C. decking, R.C.C. piers and pile foundation bridge across river Khari at Raipur on Ahmedabad-Nandol-Dehgam road, R.C.C. hollow box girder type bridge across river Karjan on Ankleshwar-Rajpipla road. Construction of these bridges is example of wide variety of design and advance construction methods and technique employed by the State P.W.D. in bridge construction.

Another great achievement of the Second 5-Year Plan is the setting up of Engineering Research Institute at Baroda. This Institute has undertaken Statewide project of collecting and testing aggregates and making systematic study of their physical and chemical properties for their economic use in road construction.

Spillover Works from the Second 5-Year Plan

Works likely to cost Rs 1,200 lakhs are expected to spill over to the Third 5-year Plan. Also there will be spillover of a major portion of 235 miles long Ahmedabad-Kandla National Highway No. 8A. This National Highway on completion will connect Ahmedabad, the biggest industrial city and capital of the State, to Kandla, a fast developing major port of Western India, and also many important towns and cities of Saurashtra. This work will include construction of one mile long road-cum-bridge (the biggest of its kind in Western India) over the little Rann of Kutch likely to cost Rs 150 lakhs.

The construction of bridges over rivers Par, Damanganga, Kolak Auranga and Kampari on National Highway No. 8, which connects the 320 miles length of N.H. 8 in Gujarat State with Delhi and Bombay via Ahmedabad will spillover to the Third Plan, completed in the first year of the III Five-Year Plan. Out of the proposed total allocation of Rs 1,650 lakhs, Rs 1,200 lakhs have been earmarked for the above described works and Rs 200 lakhs for village approach roads. The nett balance of Rs 250 will be available for new works.

New Works

The proposals for the Third 5-Year Plan for Rs 600 lakhs have been formulated.

Criteria for New Works

The policy governing new schemes in the Third 5-Year Plan is for providing

all-weather roads connecting taluka headquarters with district headquarters, providing drainage works and missing culverts on important State routes, constructing missing bridges on State Highways and Major District Roads, construction of new roads such as missing links, construction of village approach roads, and arboriculture along the roads.

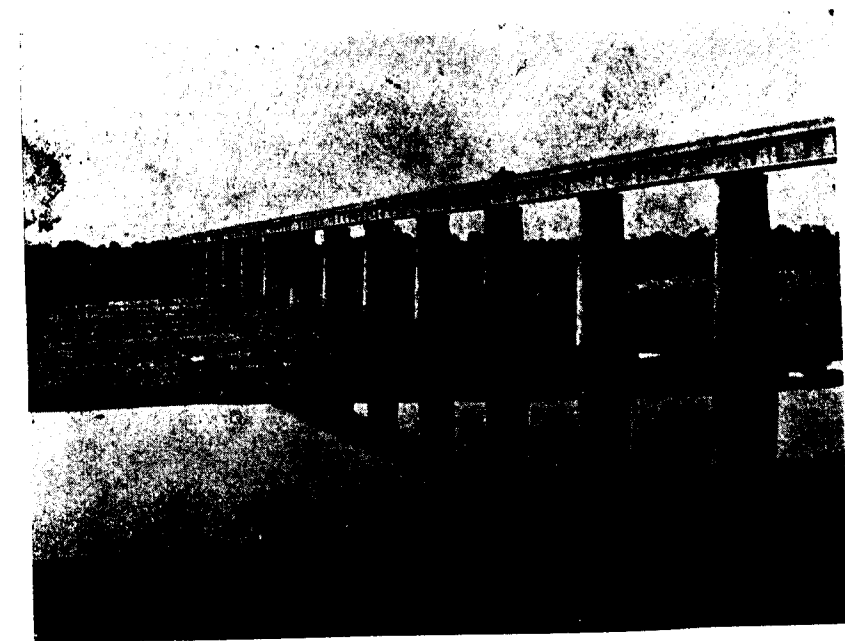
Details of Third 5-Year Plan

Construction of 63 miles of two-lane State Highway Improvement of 63 miles of roads connecting taluka places with district headquarters, cross drainage works on important routes to make them all-weather roads are proposed. This programme will have to be carried over to the Fourth Five-Years Plan. It is proposed to complete 92 bridges under construction in the Second 5-Year Plan, to start the construction of 136 new bridges during the Third Five-Year Plan, and to complete missing links so as to establish inter-district communications. Construction of 1,755 miles started in the Second 5-Year Plan shall be completed during the Third 5-Year Plan in addition to completion of improvement to 723 miles of roads also started in the Second Plan. The new works of constructing 316 miles of missing links, and improvement of 75 miles will be taken up during III Five-Year Plan. In order to properly landscape the roads in the State, extensive and planned arboriculture programme will be launched on 340 miles of the roads during the Third 5-Year Plan. In addition, 1,000 miles of village approach roads with moorum surface connecting villages to the nearby network of roads are provided for.

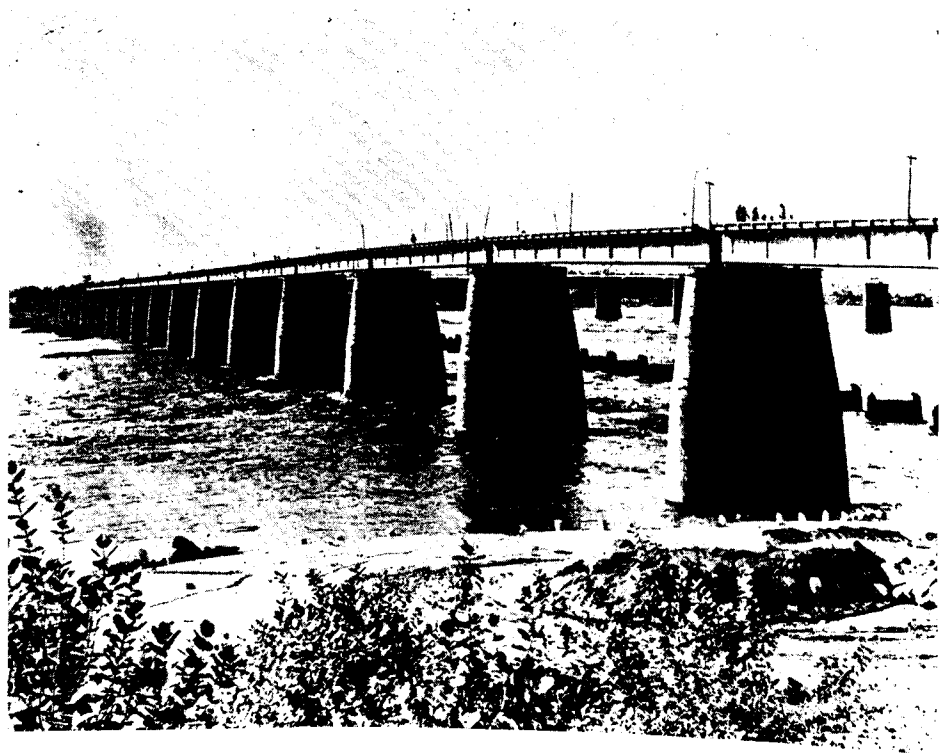
Gujarat has reasonable agriculture economy due to its fertile alluvial soil and has also got very good power potential for industrial development. Recent discovery of oil and other minerals in the State together with the programme of irrigation and power schemes, has immensely multiplied the probable rate



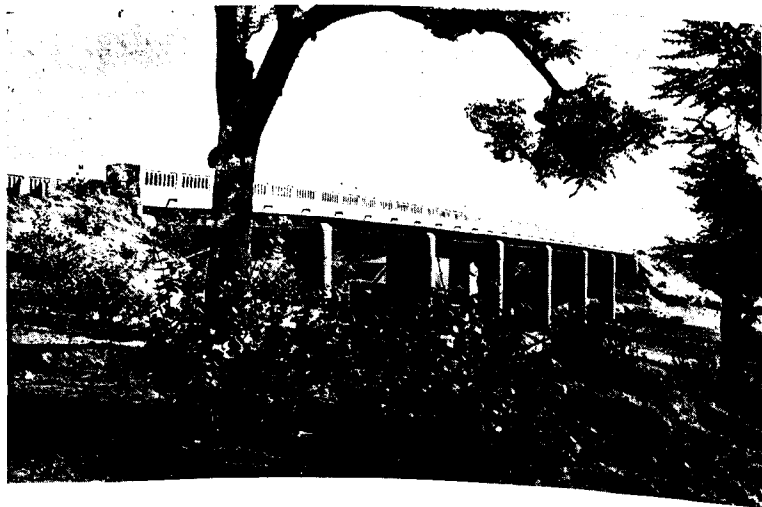
Photograph of experimental asphalt road over soil cement base in Mile No. 46/0 to 47/0 on section Jagudan Mehsana of Ahmedabad-Kalol-Mehsana-Sidhpur Road (S.H.)



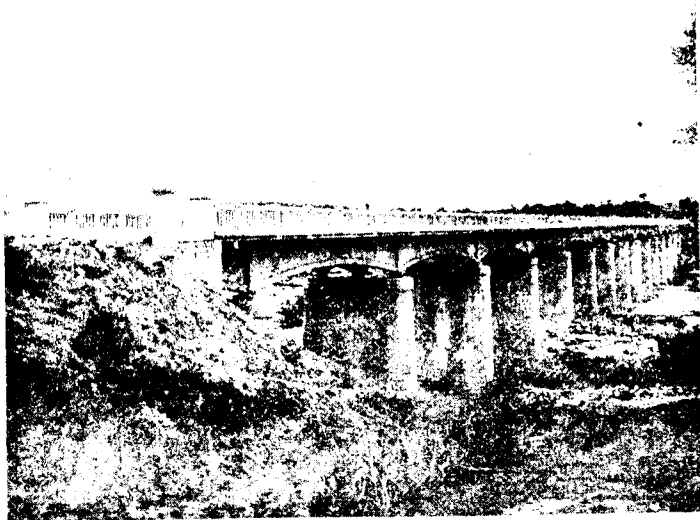
Mahi River Bridge near Vasad on N.H. No. 8



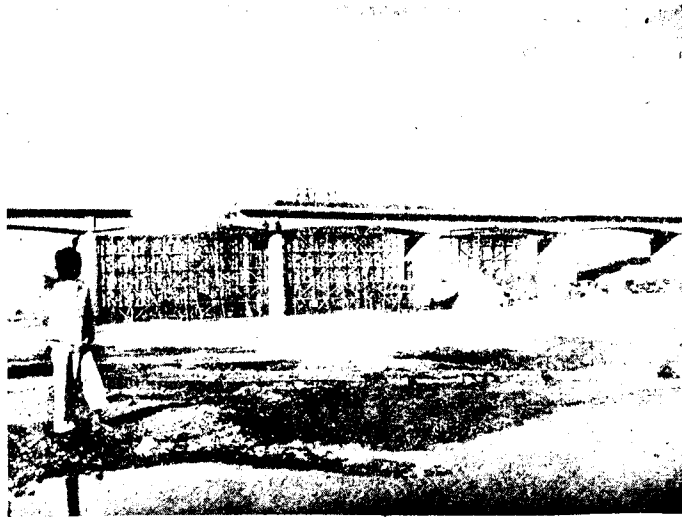
Bridge over Mahi River at Sevalia (Dist. Kaira), Bombay State showing 19 prestressed Concrete Deck spans completed on 29th June, 1956



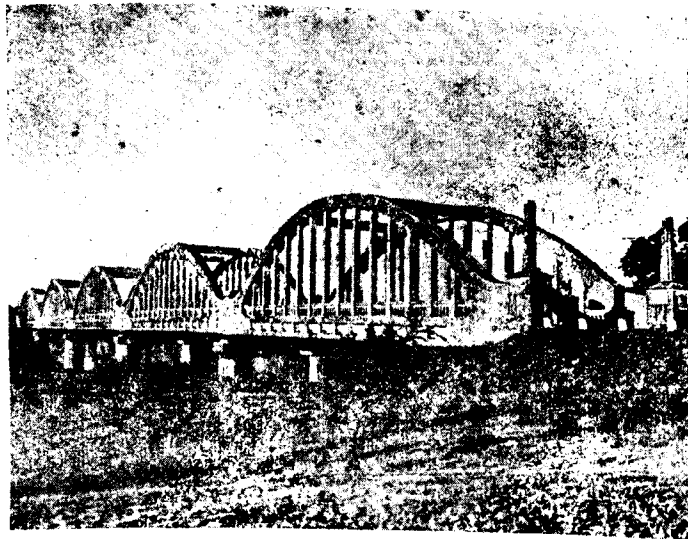
Bridge over Khari River at Raipur



R. C. C. solid T. Beam & Slab 3 girder system balanced cantilever type Panam Bridge near Lunawada completed in 1957



R. C. C. 3 T Beam and Slab Balanced Cantilever Bridge over river Panam at Sant Road completed in 1959



Ambika Bridge near Matwad (Navsari) on Bombay-Ahmedabad Road. N. H. No. 8

of development. Keeping these in view, there is a very sound and reasonably justifiable case for this State to have even greater mileage of roads than that envisaged in the Nagpur Plan, which must be carried out to avoid strangulation of industry and agriculture.

With the proposed allocation of Rs 1,650 lakhs as against estimated expenditure of Rs 2,100 lakhs during the Second 5-Year Plan the mileage achieved at the end of the Third Five-Year Plan will be about 15,900 miles against the target of 13,203 miles for the Second 5-Year Plan. It is very important to note that while most of the States of the Indian Union would be exceeding the Nagpur Plan targets, Gujarat will be deficient to the extent of 45 per cent if cart-tracks and sub-standard mileage are not accounted for. Even at the end of the Third 5-Year Plan, the length of the roads in this State will be short by,

35 per cent of the mileage envisaged in the Nagpur Plan targets while many of the States of Indian Union would have achieved 25 per cent targets of the 20 Year 1961-81 Plan in addition to the Nagpur Plan targets.

Gujarat has suffered since long as far road communications are concerned as a result thereof, development of industries and agriculture and consequent economic standard of the people has been adversely affected to a great extent. The State is rich in natural resources, has got a big potentiality of being a modern industrial State, for want of communications it is not possible to fully exploit it.

If Nagpur Plan targets are aimed to be achieved at the end of the Third 5-Year Plan, the State shall need about Rs 7,000 lakhs, unfortunately only Rs 1,650 lakhs have been provided for.

ROAD DEVELOPMENT IN UTTAR PRADESH ENDING MARCH 1961

180



JAGDISH PRASAD
Chief Engineer,
P.W.D., B & R., U.P., Luck

UTTAR Pradesh is one of the most thickly populated State in India. It has a population of 73.75 million according to 1961 census against 63.21 million in 1951.

Majority of the people live in rural areas, they are handicapped socially and economically for want of good road system and find it difficult to provide good schooling, medical facilities, carry their agricultural produce economically direct to the markets to get better price, etc. Lack of roads also account for the migration of the rural population to urban areas leading to growing congestion in the cities. The only answer to this problem is to develop the road system in the rural areas as quickly as possible.

Since the era of Independence, this State embarked on the development of communications first under the Post-War Development Plan and subsequently through the First and the Second Five-Year Plans and even then has only about 14,500 miles of roads in all, this falls short of the Nagpur Plan Targets by about 2,000 miles. The road mileage per hundred sq. miles area comes to 12.83, and per thousand of population 0.23 based on 1951 Census against 32.26 and 0.51 respectively of Madras State.

Though progress made so far in the development of road system in the State of Uttar Pradesh is very meagre, the Chief Engineers in their Report on Road Development Plan in India (1961-81) have provided a target of 48,700 miles of roads in Uttar Pradesh by the end of March 1961. This has awakened the public opinion to the imperative necessity of providing roads which precede all other developments.

In the First Five-Year Road Development Plan of this State, the ceiling of expenditure was limited to Rs 500 lakhs, yet both the financial and physical targets were exceeded, and resulting in to an increase of about 2,150 miles of roads bringing the total mileage to 11,534. In the Second Five-Year Plan for roads and bridges, expenditure ceiling was Rs 16.14 crores, this amount did not include schemes financed from Central Road Fund allocations or those sponsored by Government of India, or those financed from other sources, such as construction of Cement Concrete Tracks, surface treating roads to sugar factories sponsored by the Cane Development Department at a cost of Rs 247 lakhs to encourage development of sugar-cane industry. During the Second Five-Year Plan ending 31st March 1961,

an expenditure amounting to Rs 144.126 lakh was incurred to build 190 miles of roads under the scheme of cane development department. The remaining work is proposed to be carried out during the 3rd Five-Year Plan period.

Special scheme for development of border roads to cost Rs 54.0 lakh in the 3rd 5-Year of the II Five-Year Plan was sanctioned by the State Government. Road Construction Programme of the Second Five-Year Plan was also supplemented by the Backward Area Scheme and Additional Backward Area Scheme sanctioned during the 4th and 5th years of the II Five-Year Plan at a cost of Rs 107.46 lakh and Rs 131.42 lakh respectively. Expenditure amounting to Rs 28.32 lakh was incurred by 31st March, 1961 on Special Border Area Scheme, Rs 73.932 lakh on Backward Area and Additional Backward Area Schemes. Government also sanctioned scheme for roads for rehabilitation of displaced persons in connection with reclamation of land at a cost of Rs 43.637 lakh against which 31 miles of roads were constructed at an expenditure of Rs 35.95 lakh.

Under the Second Five-Year Plan against expenditure ceiling of Rs 1,614 lakh, expenditure of Rs 1,414.85 lakh was incurred with the implementation of all these schemes the mileage of roads in Uttar Pradesh reached a figure of 14,500.

Bridges

Bridging the rivers is as essential as the construction of roads. The problem of bridges in this State is very important and ticklish due to the very large number of rivers, having alluvial beds, very flat bed slopes and large scale meanderings.

Out of the huge task of bridging small and big rivers, 26 major bridges at a total cost of Rs 95 lakh were completed and opened to traffic during the 1st Five-Year Plan period.

In the 2nd Five-Year Plan, provision for constructing 118 high level bridges and 30 pontoon bridges was made, against this, 57 bridges were completed and work on 39 bridges was in progress. The bridges constructed cover a wide variety, viz., steel, masonry, prestressed, reinforced cement-concrete, and cement concrete bridges. A few of the major high level bridges completed are given below:

Sl. No.	District	Description	Cost (Rs in lakh)
1.	Shahjahanpur	Bridge over Khanaut in mile 104 of Lucknow-Bareilly road N.H. Route 24	...
2.	Muzaffarnagar	Bridge over Hindan river in mile 19 of Muzaffarnagar-Budhana road	8.11
3.	Meerut	Bridge over Ganga at Garhmukteswar on N.H. Route 24	8.09
4.	Almora	Bridge over Ramganga at Thal on Bageshwar-Askot Road.	77.97
Some of the important bridges under construction are given below:			...
1.	Azamgarh	Tons bridge on N.H. Route 29	5.00
2.	Bareilly	Ramganga bridge on Bareilly-Mathura Road	12.489
3.	Faizabad	Sarju bridge at Ayodhya on N.H. Route 28	58.86
4.	Gorakhpur	Bridge over Rapti on N.H. Route 28	182.04
5.	Jhansi	Bridge over Betwa at Naughat	41.43
6.	Mathura	Bridge over Jamuna on Bareilly-Mathura Road	24.00
7.	Muzaffarnagar	Bridge over Jamuna at Kairana	28.78
			56.01

National Highways

All works connected with the construction and maintenance of the National Highways within the State boundary are financed by the Government of India and are executed by the State Public Works Department. There were 1,502 miles of the National Highways in the State, recently another 22 miles length from Agra to Fatehpur Sikri have been added, increasing the total to 1,524 miles. Many miles of the National Highways have been improved

by widening, improving the surface, constructing by-passes and over-bridges, etc. The most important bridge over Ganga on National Highway No. 28 has been completed and the work of the bridge over River Sarju at Ayodhya is in hand. On Agra-Delhi road, which also has the tourist traffic, widening of the carriageway and cement concreting work is in hand.

A statement showing milages of roads with various types of surfaces on March 31, 1961, is given below:

Sl. No.	Type of Highways	Length of various types of surfaces (in miles).				
		Cement concrete	Black top	Water-bound	Cement concrete tracks	Total
1.	National Highways	275	1,110	28	—	1,413
2.	State Highways	224	4,980	4,998	—	10,202
3.	Major Distt. Roads					
4.	Other Distt. Roads					
5.	Hill Carts Roads	—	—	702	—	702
6.	Roads to sugar factories	—	—	—	401	401
Grand total ...		499	6,090	5,728	401	12,718

The mileage of metalled roads with local bodies are not included in the above statement.

Length of roads completed in Uttar Pradesh costing Rs 5.0 lakh or more during the year 1960-61 are given below:

Roads

Sl. No.	District	Name of road	Length Mile Fur.	Estimated cost (Rs in lakh)
1.	Muzaffarnagar	Budhana-Khatauli	15-2	8.000
2.	Garhwal	Karanprayag-Lobha-Dwarhat	15-0	9.870
3.	-do-	Mohan Domain Baijrao	29-6	12.000
4.	Moradabad	Sambhal Hasanpur	21-4	7.475
5.	Banda	Nariani Kalinjer	15-1	8.245
6.	Kheri	Pallii-Tirkullia-Khajuria	21-4	18.001



P. N. WANCHOO,
Chief Engineer, R. and B., Kashmir.

ROAD DEVELOPMENT OF KASHMIR VALLEY

IN the absence of Railways, as an alternative means of transport, development of roads forms a vital item of economic development in the State of Jammu and Kashmir. The road development, therefore, received due attention during the First and Second Five-Year Plans.

The total length of roads in the Kashmir Province with the P.W.D. before the First Five-Year Plan period was 623 miles. Marked progress was made in road development during the First Five-year Plan and the length of

roads rose to 998 miles by the end of First Five-Year Plan costing approximately Rs 102.00 lakhs.

During the Second Five-Year Plan, the road length in the State increased to 1,294 miles. In addition, surface of 510 miles was improved. The total expenditure on the construction of roads and major bridges in the Valley during the Second Plan was Rs 207.719 lakhs.

The targets of road development in the Valley and achievements during the Second Five-Year Plan are given below:

	Targets miles	Achievement miles
(a) Upgrading of Roads		
1. From metalled to black topped	151	159
2. From shingled to metalled	172	147
3. From fair-weather to shingled	213	204
Total ...	536	510
(b) New Construction of Roads		
1. Jeepable to fair-weather	247	272
2. New construction	232	275
Total ...	479	547

In addition, out of the major bridges Sopore Bridge with 510 ft waterway and Rajbagh Bridge with 490 ft waterway costing Rs 7.064 and Rs 7.462 lakhs respectively were completed during the Second Five-Year Plan period.

In spite of considerable progress made, most of the roads are narrow, have sub-standard surface, inadequate thickness of road crust and have many nallahs without bridges.

According to the Chief Engineers' Report on the Road Development Plan for India (1961-81), the State should have the following length of roads by the year 1981, in Kashmir province alone.

	Miles
1. National Highways	200
2. State Highways	400
3. Major District Roads	600
4. Other District Roads	1,000
5. Village Roads	1,600
Total : ...	3,800

Against this, we have at the end of the Second Five-Year Plan 1,294 miles of roads. Thus, additional 2,500 miles of roads shall have to be constructed during the next twenty years.

The above increase in the mileage will be in addition to the upgrading and improving of existing roads.

For the preparation of the Third Five-Year Plan, the following points have therefore been kept in view :

- Existing roads have been classified on all-India basis.
- The first priority has been given for completion of the spillover works of the Second Five-Year Plan under each classification.
- Suitable upgrading of roads has been provided for roads of various classes.
- Construction of additional roads.
- Proposals have been formulated for construction of new bridges and strengthening of old bridges for higher loading according to priority.

During the Third Five-Year Plan, it is proposed to carry out the following works estimated to cost Rs 423.60 lakhs.

	Miles
1. Road mileage to be black topped	179.80
2. Road mileage to be converted from shingled to metalled	237.10
3. Road mileage to be converted from fair-weather to shingled	433.75
4. Jeepable to fair-weather	449.05
5. New Construction	403.75

"I have come to the conclusion that one thing to which we must give topmost priority is roads—roads of all kinds, not only very up-to-date bitumenized or cement roads, but roads of any kind, to open vast areas of this country which are closed up today and which you cannot reach unless you walk or ride.....I give roads first priority."

—SHRI JAWAHARLAL NEHRU

WEST COAST ROAD IN KERALA

By

C. S. PADMANABHA IYER*

Introduction

WITHOUT properly developed and maintained roads, the industry and agriculture cannot develop and prosper. The West Coast Road in Kerala was one of many roads taken up for improvement during the post-war period. This road is 198 miles long, runs mostly along the coast in the erst-while Malabar area. Beyond Kerala, it extends in north up to Bombay, and in south up to Kanyakumari.

History

Due to financial stringency, West Coast road in Kerala State was excluded from the list of National Highways given in the Nagpur Plan. Due to the Madras Government's justified and continued advocating to include the West Coast road in the system of National Highways, the Government of India in 1953 agreed to its development on a 50-50 expense basis. However, in 1955, the Government of India on reconsideration agreed to meet the full cost of the construction of this road as a single-lane fully bridged black topped road and also let the State Government provide wider carriageway if they so desire at their own cost. Additional funds could not be provided out of the State Plan, as such only a single lane carriageway is being provided.

The West Coast road in Kerala State comprises of portions of a number of existing State Highways, the important among these are given below:

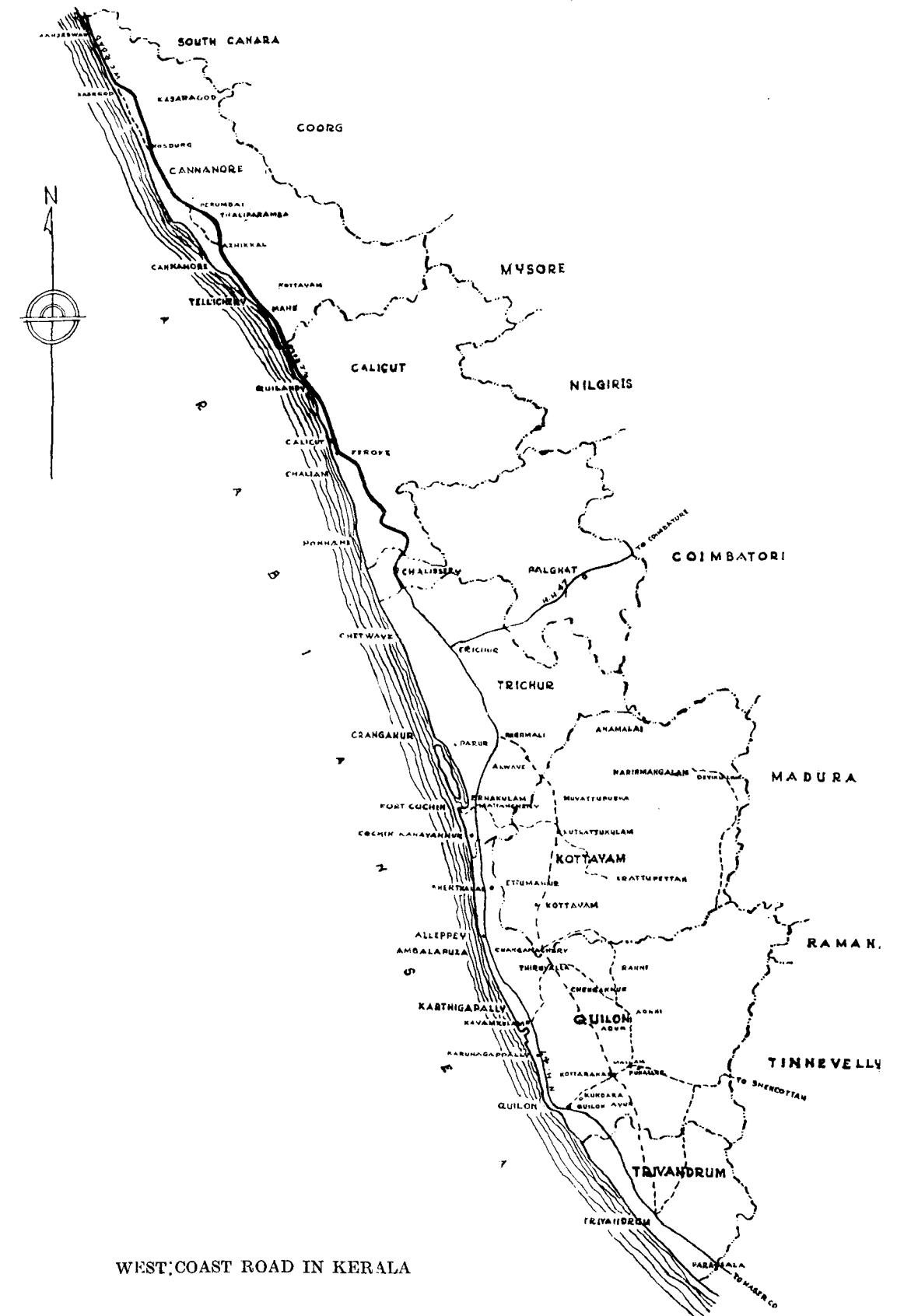
1. Cochin Frontier to Edapal Road.

2. Edapal-Kuttipuram road
3. Kuttipuram-Valenchery road.
4. Ramanattukara - Thirurangadi road.
5. Madras-Calicut road M. 435/3 to 445/2.
6. Calicut-Cannanore road 0/0 to 60/0.
7. Baliapatam-Taliparamba road.
8. Mangalore-Cheruvathur Coast road M. 45/6 to 57/2.
9. Kalladakka-Kanjanagad road.
10. Kasargode-Jaloor road and
11. Mangalore-Cheruvathur Coast road M. 7/7 to 28/4.

West Coast road in Kerala when constructed will be maintained by the State Government till such time this road is declared a National Highway.

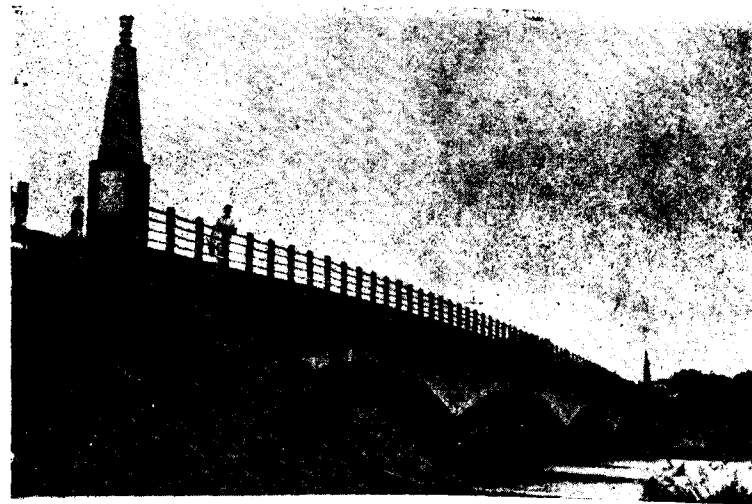
Standards

Due to the Kerala State being very densely populated and the agricultural land highly developed, it was very difficult to get the required width of land for the road. The Government of India relaxed the standard for the width of road land in the case of existing alignment to 40 to 60 ft width in built-up areas, 60 to 100 in open agricultural areas and 150 ft for by-passes of major towns and in the case of new alignment formation width is 32 ft in plains and 24 ft in hilly sections. The minimum radius of curves will be 800 ft in plains,



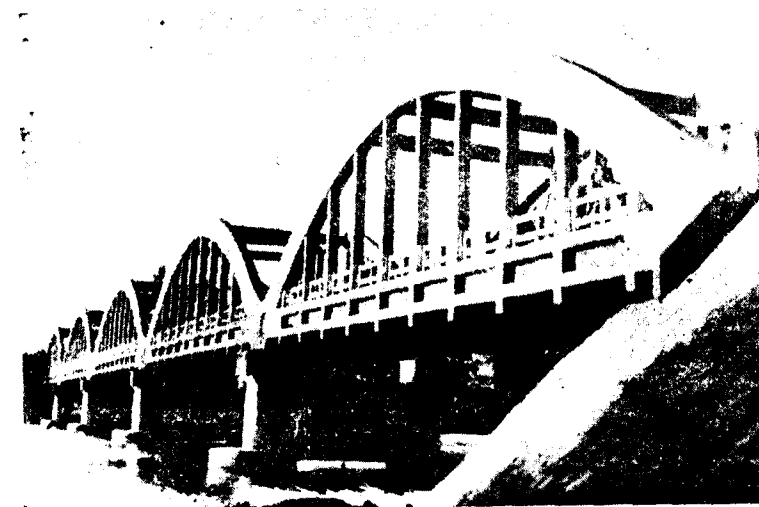
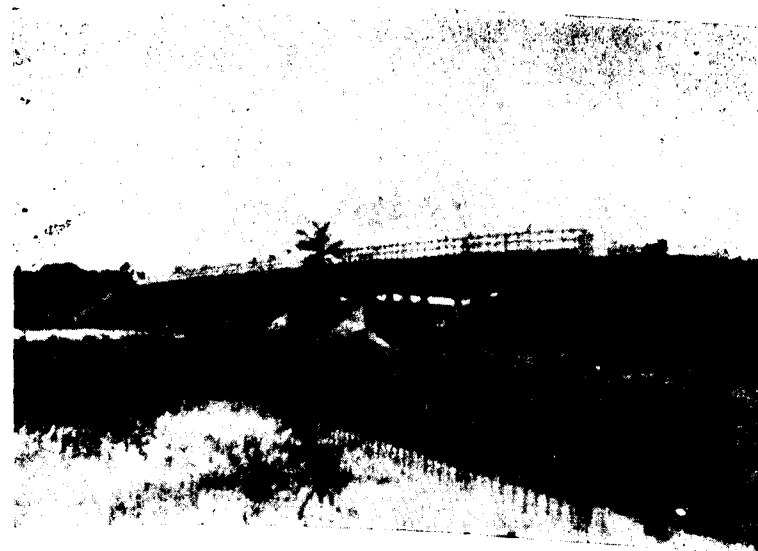
WEST COAST ROAD IN KERALA

* Chief Engineer, P.W.D. (B. & R.), Kerala.



Nileswar Bridge
in West Coast
Road

Perumbai Bridge
in West Coast
Road



Kuppum Bridge in
West Coast Road

500 ft in rolling countries and 300 ft to 100 ft in ghat or mountainous sections. The gradients are provided as per I.R.C. standard for single lane roads (given in the I.R.C. Paper No. 156). In the first stage of development, bridges and culverts with less than 18 ft roadway and those not safe to carry a 12-ton road roller will be reconstructed to carry 2 lanes of I.R.C. Class A loading or 1 lane I.R.C. Class AA loading whichever produces maximum stress. Culverts will have 32 ft width (including the parapets) and the bridges will have 24 ft clear roadway.

Administrative Facilities

For speedy construction, the Government of India increased the powers of the State Government to sanction estimate for road works from Rs 5 lakhs to

Rs 25 lakhs and for bridge works from Rs 3 lakhs to Rs 5 lakhs.

Progress

Out of the 198 miles of West Coast road, work on 40 miles length has been completed and the remaining 158 miles are under completion. Out of the 27 bridges to be constructed or reconstructed, 4 have been completed and 4 are in progress.

General

The West Coast road running along the Coastal regions of Kerala when completed will provide better means of road communications essential for the development of the Industries, transportation of agricultural and other produce of the coastal areas, viz., fish coir, mineral sands, etc., which is sure to add to the prosperity of the region.



K. BARUA
Chief Engineer & Secy.,
A.D., B. & R., Assam, Shillong

ROAD AND BRIDGE PROGRAMME IN ASSAM IN THE THIRD FIVE-YEAR PLAN

AT the end of Second Five-Year Plan period, the State of Assam completed about 9,000 miles of motorable road consisting of 1273 miles black topped, 141.5 miles metalled, 4,425.5 miles gravelled, 2160 miles earthen and 1,000 miles under local bodies. Ten major bridges out of 23 nos. undertaken during the 1st and 2nd Five-Year-Plan period were completed. In addition 10,240 rft. of bridges and culverts were reconstructed.

According to the report of Chief Engineers on Road Development Plan for India, 1961-81, State of Assam should have 22,000 miles of motorable roads by the end of 1981. This means construction of at an average of 3,250 miles of roads in each of the next Four Plan periods. In view of difficult financial position it is proposed to

construct only about 1,600 miles of motorable roads. (of these, 600 miles are expected to be financed out of the proceeds of the Central Fund). In addition it is proposed to metal and black top 274 miles of roads, upgrade nearly 500 miles of existing low standard roads and to complete 13 major bridges which were under construction at the end of Second Five-Year Period. Besides this, it is proposed to undertake construction of 5 more major bridges, and reconstruct and strengthen 21,000 rft. of existing weak and temporary bridges during the Third Five-Year Plan Period.

It is proposed to take in hand works estimated to cost Rs 1250 lakh during the Third Five-Year Plan period and limit the expenditure to Rs 900. lakh. Break-up of these works is given below :

Name of Schemes	Aims	Amount allocated (Rs in lakh)
1. Spillover Schemes from 2nd Five-Year Plan	180 miles of roads, construction of 13 major bridges, construction of 750 rft. of bridges and culverts, reconstruction of 11 weak bridges, liquidation of liabilities of land acquisition costs, etc.	265.00

Name of Schemes	Aims	Amount allocated (Rs in lakh)
2. Metalling and black topping of important P.W.D. Roads (State level)	134 miles	30.00
3. Metalling and black topping of roads (district level)	140 miles	100.00
4. Improvement of existing low standard P.W.D. roads	500 miles	80.00
5. Reconstruction of weak bridges and culverts and construction of timber bridges	21,000 rft.	115.00
6. Construction of permanent major bridges	5 nos.	85.00
7. Improvement of new roads	1,000 miles	575.00
8. Construction of roads for the Industries	219 miles	—
Total ...		Rs 1250.00 lakh

ALWAYS ON THE ROAD

A healthy man once went to a doctor; the latter asked him the secret of his good health.

The man replied : " Doctor, when I married I entered into a pact with my wife that when I become angry she was to laugh and when she became angry, I had to leave the house. So, doctor, I am always on the road and that is the reason of my good health. "

MOTORWAYS TO STAND STRESS OF MODERN TRAFFIC

DETAILED REVIEW OF ROAD IMPROVEMENTS IN BRITAIN

By
N. W. JENSON*

THE five years to 1961 has been a period of considerable activity in the construction of major traffic routes in Britain, with a motorway system of about 1,000 miles currently envisaged. About 150 miles of motorway have been completed and it is possible to draw conclusions from the experience gained. The Institution of Highway Engineers devoted four of its meetings during the 1960-61 session to studying and discussing the lessons learned in the hope that these might be applied to future works.

This article is not intended to be a learned engineering treatise, neither can it include statistics or any measure of detail in the space allotted, but much publicity has already been given to the overall figures and specifications. It is intended to give a broad picture of how British engineers are tackling the designing and building of roads and bridges to carry the modern volume of traffic swiftly from place to place. No attempt is made to deal with design or construction for urban traffic, which presents entirely different problems and requirements.

Preliminary Work

While works of the magnitude of those under consideration stem from similar basic procedures, control is achieved in various ways. With trunk roads, the county authority is almost invariably the nominated agent for the Ministry of Transport under the various Acts of Parliament dealing with this

subject. The county authorities collaborate with the Ministry to design the work which is usually put out to contractors for construction. Frequently, the county authority employs consultants for the design of bridge-works, although the roadworks, and in many cases the bridges, are usually designed by the county. Motor-road work is entirely different, and the Ministry is unrestricted by any overall agency agreements with the county authorities. Generally speaking, the Ministry of Transport considers that motor-roads crossing several counties should be designed by consultants to maintain continuity of design, but about half the current motorway programme is being handled by county councils as the agents of the Minister of Transport.

It has to be borne in mind that trunk-road motorways, as distinct from trunk and county roads, are a part of the national grid for road transport; trunk-road motorways are financed from Central Government funds and are not limited by county boundaries. Furthermore, motorways are being constructed for a limited section of the population and not for the public at large.

They are segregated from the existing highways, necessitating a considerable number of bridges and viaduct structures. Junctions are only made at reasonably long intervals with certain pre-selected main traffic routes serving major centres of population.

Before actual construction can begin, a lengthy programme of preliminaries has to be carried out, broadly as follows:

Preparation of a draft scheme by the county authority or consultant and its submission to the Ministry of Transport.

Discussions of the draft scheme, settlement of variations and ultimate acceptance by the Ministry of Transport, culminating in the making of an official order under the Special Roads or Trunk Roads Act, following, if necessary, a public inquiry into the scheme.

The working up of the complete scheme and preparation of contract documents.

Issue of invitations to tender and placing of contracts.

The acquisition of the necessary land is a major item in a geographically

small country such as Britain, where almost every piece of land has its earning value to the owner. Arrangements have to be made for the local hearing of objections and their settlement, often necessitating changes in route, before all necessary land is acquired and the final order issued. It is, therefore, obvious that the art of public relations plays a considerable part in the early stages.

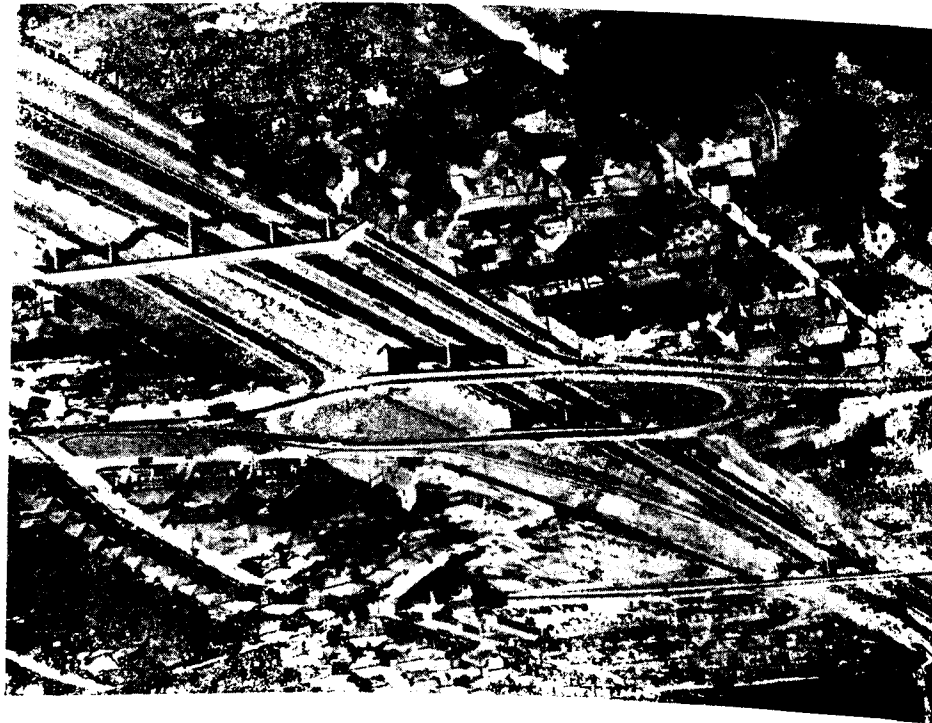
General Design and Layout

Motorways construction has so far adhered strictly to the motorways layout prescribed by the Ministry of Transport. This has been arrived at first by experience in other countries where motorways are established, and with the economy of land use in mind, and second from experience of the first British motorways which led to a variation in the width of hard shoulders and verges.



A scene during the building of the Birmingham to Preston motorway, M. 6, in Britain. This picture shows the massive 80 feet high reinforced concrete portal frame piers. They will carry the 28-ton approach span girders of the superstructure for the one and a quarter mile long Thelwall Viaduct. This viaduct will carry the motorway over the Manchester Ship Canal and the River Mersey

* Secretary of the Institution of Highway Engineers, U. K.



A view of part of the 15-miles Doncaster (Yorkshire) motorway in the North of England. It cost 6,000,000 pounds and has more than 30 bridges. Subsidence has been a difficulty here because of coal workings and both the road and the bridges have been made flexible, the bridges with readily accessible jacking points. Main contractors were Cubitts-Fitzpatrick-Shand, 455 Old Ford Road, London, E. 3

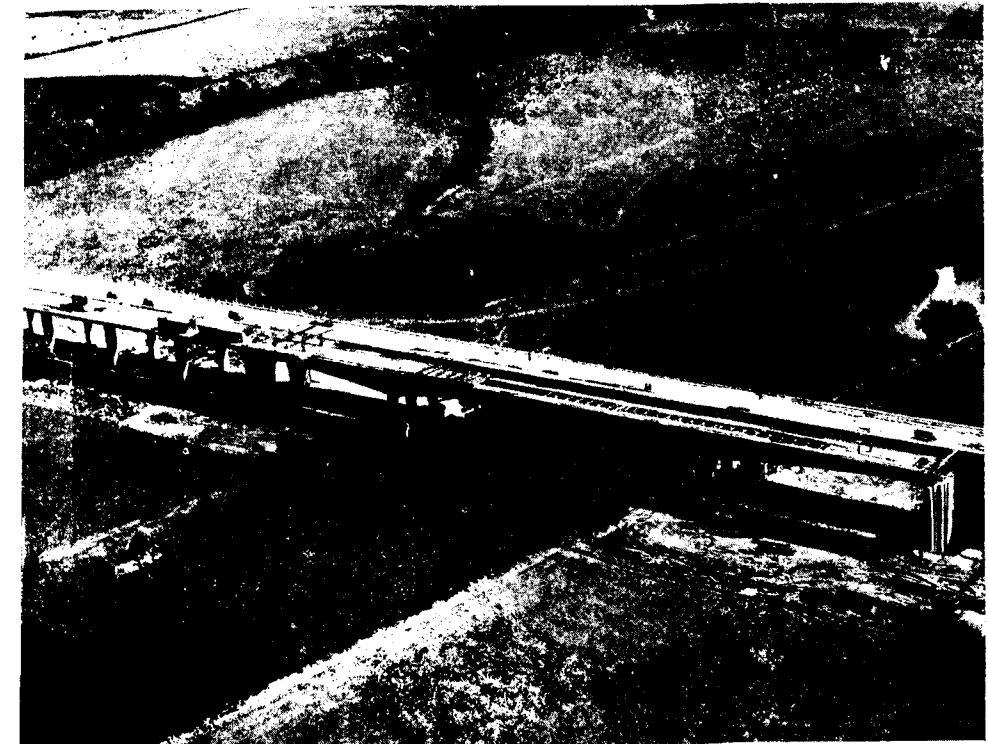
Originally the standard dual two-lane carriageway had a net overall width of 88 feet, comprising dual 24 feet wide carriageways separated by a 13 feet wide median strip plus a twelve inches wide marginal strip; eight feet wide hard shoulders were separated from the carriageway by a twelve inches wide marginal strip and on the outside of the hard shoulders a three and a half feet wide verge was provided. The dual three-lane carriageway had a similar layout with twelve feet added to each carriageway.

Through experience it has been decided to increase the width of hard shoulders from eight feet to ten feet and the verge width from three and a half feet to ten feet, with the result that the net overall width of a dual two-lane

motorway is now 105 feet and dual three-lane motorway 129 feet.

There has been criticism of the provision of dual two-lane motorways. There is a measure of opinion that a dual two-lane carriageway can only be adequate for its full design of speed of 70 miles an hour under conditions of low traffic density, and that a third lane is essential if speed is to be maintained safely, the third lane to be used by the slower, heavy vehicles.

The Ministry governs the layout on assessment of estimated traffic volumes. It does not authorise three-lane dual carriageways where it is estimated two lanes will suffice, but it has been suggested that a sufficient overall width be allowed in the land acquired to permit subsequent widening, in cases



Construction work on the Ross Motorway to link the Welsh industrial areas with Liverpool. The illustration shows Queenhill Bridge with the south carriageway completed and the north carriageway under construction

where future traffic volume is likely to make this necessary. In the current programme provision has been made on schemes amounting to 115 miles which, although built or planned to dual two-lane standards initially, have provision for a third lane in each carriageway.

Making the Motorway Attractive

Hard shoulders are a feature of motorway construction to accommodate disabled vehicles and so on and to keep the carriageway clear. Earlier hard shoulders had a width of eight feet only and were grass finished, but nine feet, and subsequently ten feet, have been authorised. Generally speaking, hard shoulders are now built to the same thickness as the carriageways, the fundamental difference is that the surfacing consists of two inches dense

bituminous surfacing, surface dressed to provide contrast in texture and colour with the carriageway.

The location of interchanges on a motorway is usually governed by the presence of existing trunk and other important roads, but they are kept to a minimum. Siting is important, and whenever possible interchanges are located in cutting, thus giving a gradient which works for, and not against, vehicles entering and leaving the motorway.

One of the cardinal rules for design appears to be the co-ordination of vertical and horizontal curves, though achievement of this can prove costly in excavation. It does seem, from experience, to be more important to avoid a great extent of even gradient, which

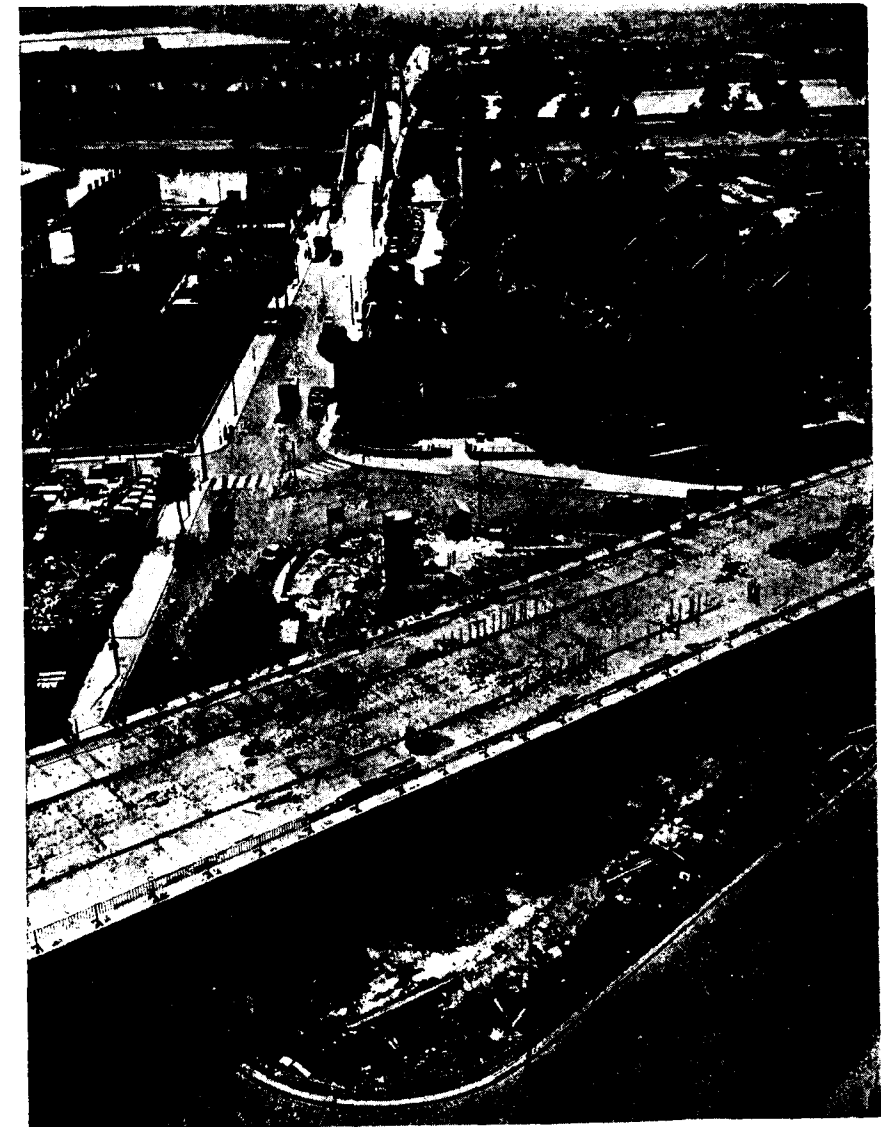
seems to look more unnatural than do straight lengths of road in plan.

Every effort is made to make the road as attractive as possible and to merge it into the landscape. The Minister of Transport has appointed a Landscape Advisory Committee consisting of eminent authorities in this field. Their advice is obtained before the line of the motorway is fixed, and on occasions

alterations have been made to the alignment to implement the advice given. In addition the Committee also gives advice on planting schemes for the new road. Thus the responsible authority preparing the contract details has expert aesthetic opinion on the alignment proposals before details are worked out. Attraction in appearance can sometimes be achieved by incorporating otherwise useless small, severed



A view of Queenhill Bridge and viaduct, with Bushly Viaduct in the background, part of the work on the Ross Motorway, which will link the industrial areas of South Wales and the English Midlands



The erection of the main superstructure of the Hammersmith, London, flyover, completed after eight and a half months. There are 16 spans totalling 2,043 feet and they were put up on an average of one span every 17 days. The flyover is part of the London section of the Great West Road improvement. The London section alone is costing about 4,300,000 pounds

areas of land into the motorway, and using them for tree-planting. Additional means of providing amenities are by widening the central reservation to preserve existing trees or by separating the levels of the carriageways on side-long ground.

Standard Pattern of Bridges

A standard pattern of bridge design appears to have emerged on motorways so far constructed, primarily to provide full headroom over a wide formation, coupled with economy in design. The

trend is for the structures to be of the light open type, with open parapets, giving the minimum of interference with the sight line.

Overbridges are generally four-span designs using bank seats where the motorway is in cutting or skeleton abutments where the side road is on filling. Piers are at the minimum thickness which will accommodate the beams which have to rest on them. Underbridges are either three-span or single-span, with a tendency towards the latter.

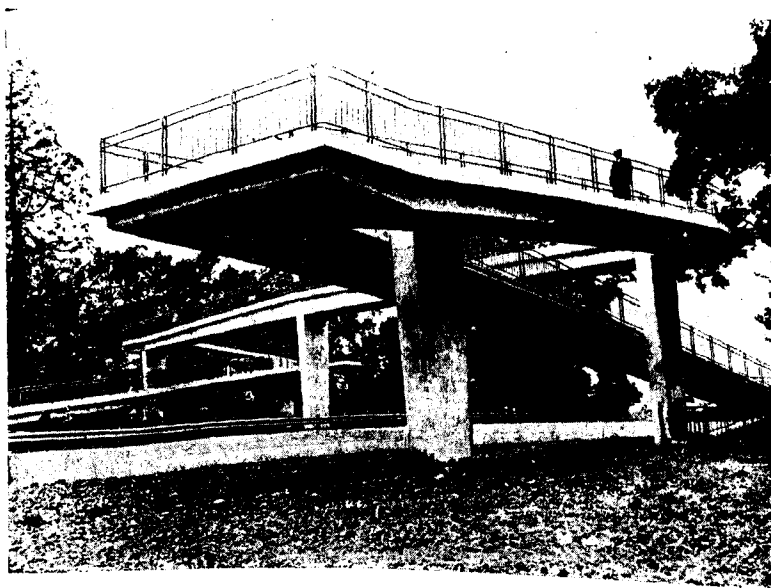
From the contractor's angle, the adoption of some form of universal bridge design, at least on each major section of a motorway, is advantageous as was shown on the first motorway, M.1., where there are only four main designs of bridges. Obviously there has to be individual variation according to loads to be carried and span, but the standardisation of steelwork alone results in economy of materials and labour. While there may thus be a certain tendency towards basic standardisation, including universal concrete construc-

tion, variety in appearance is achieved by numerous individual surface finishes, angles of skew, curvatures of span, depths of profile of parapets and other variants.

Improving the Piers

So far as detailed design is concerned, substructures have generally employed spread footings, the piers being hinged at the feet with the deck expansion transmitting to one abutment, though the variation has been adopted of fixing the centre pier and transmitting the expansion equally in each direction. Where the skew becomes large and the motorway is wide, the pier design has been split into halves, partly because the design of a pier of more than four legs is tedious, but also in one district particularly — to counteract mining subsidence.

Another method of obtaining a slightly lighter appearance to the pier has been to cantilever the lintel beyond the outer leg, though care is needed to avoid the necessarily rather large lintel ends



A new footbridge in Britain, built with ramps instead of steps to help cyclists and people with perambulators. This is on the Maidenhead Motorway near London

breaking up the line of the bridge soffit in elevation. The development of rubber bearings has helped the problem of the bearing of beams on narrow piers.

The substructures of single-span underbridges are normally in the form of counterfort walls, although reinforced cantilever abutments have also been used. Another method employed on one motorway is the use of wing walls slung from the back of the abutment. With regard to superstructures, pre-stressed concrete is almost universally used for motorway bridge decks, except for small underpasses and culverts, and its economy and minimum depth span ratio are telling points for road over road bridges, in addition to the achievement of lightness.

Central Piers

There has been considerable development in the pre-casting industry in Britain, which has influenced the development of pre-stressed concrete on rather different lines from those on the European Continent. Using pre-cast members, the standardisation is of moulds rather than of beams, because it is possible to alter spans slightly by means of different cable arrangements within the same beam section.

Most motorway overbridges have embodied a central pier, though there are exceptions. These are fine-looking structures and, with the clean lines which the absence of a central pier gives, there is a greater degree of lightness and better visibility. The presence of a pier in the central reservation is a potential hazard to the fast lane traffic and it is now standard practice to provide it with some form of safety fence or kerb. The latest practice is to eliminate the central pier, but on skew spans it is not always possible.

Choice of Materials

The early motorway construction called for the execution of the works in a 19-months' period. Experience has

shown that although this period can be achieved, bad weather would severely handicap the contractors and later contracts have been far from 21 to 30 months.

A 19-month construction period represents an overall time of some 21 months from the order being given for work to start. This allows for preparatory planning, including ordering and delivery of initial materials and plant, marshalling of labour and arranging the layout of work. Ideally in Britain the contract should be awarded in the first three months of the calendar year, which allows a start to be made immediately the bad winter weather is over and completion during the fine weather well before the difficult winter months two years later.

Leading contractors, however, consider rapid construction, with as great a mileage of highway as practicable in service as quickly as possible, to be in the interest of national economy.

It is not possible to deal in any detail with the subject of excavation, which is so dependent upon local conditions and these vary almost from yard to yard in Britain.

With regard to the construction of embankments, care is required in the selection of materials to be placed and compacted, otherwise difficulties are likely to occur in the wet season. Some authorities have suggested that layers of imported granular filling material should be considered under very wet conditions. With the speed of progress required, the construction of embankments during very wet conditions presents serious problems and to allow contractors facilities for moving on tops of embankments immediately below sub-base levels revised specifications call for light quality fill at that level.

Foundations and Drainage

On the question of depth, some doubt has been raised as to whether the practice

of providing a standard foundation depth over the full width is right. The California Bearing Ratio method of design depends on the frequency of the heaviest loads. Heavy vehicles are invariably on the inside lane, and it might be more economical to provide extra depth in the slow lane and reduce depth gradually across the width of the road to a minimum at the central reservation. This would increase the crossfall of the formation and assist drainage during the construction period.

It has been found that allowance for shrinkage of fill can be underestimated, for various reasons—the use of big plant for stripping top soil results in a slightly greater depth being removed than is specified; the passage of the plant itself gives more compaction of the existing subsoil, the compaction obtained when rubber-tyred scrapers are used being very good. It is therefore advisable to allow for an excess of fill. A substantial allowance needs to be made for soft material which needs replacing and sufficient land taken for normal earth cutting. It is easier to hand back surplus land than to acquire additional land to allow for a variation in slope from that predicted.

Construction drainage is essential and has, on occasion, been linked with the permanent drainage by installing the drainage when the earthworks have been completed to the first stage, that is, to road level. It is becoming general practice to lay a sub-base over the whole formation, so that there is a greater width of smooth tract for plant to run on and less chance of breaking up the material, while similar extension of base material over the whole width of the motorway, including the central reservation, considerably assists drainage.

Where heavy marginal haunches have been used, or a raised central reserve laid, these have been found to trap water, which discharges on to the carriageway during construction, and the raised

central reserve will continue to do so after the motorway is in use. The latest specifications have deleted the use of the heavy marginal haunch, increased the amount of subsoil drainage and adopted dished central reserves with drainage facilities. If the base material is carried over the whole width, drains can be provided at the outside of the hard shoulders.

Sub-base materials, being low grade, are susceptible to moisture changes and it is worth sealing by means of surface dressing, material suitable for such treatment, if it has to stand for a time.

Base Material and Surfaces

Base material has, in most cases, been put down in two distinct lays to provide adequate compaction with the machinery currently in use, but experiments are being made to see whether a satisfactory single-pass laying can be achieved, as obviously more economical. Wet mix, using local aggregates, has been used on many works, with base course asphalt laid in short sections behind, to give it immediate protection.

The type of surfacing on motorways in general has been flexible, though a concrete finish has proved equally successful. The application of the final hot laid surface at once, or the use of an initial temporary surface of fine cold asphalt, has been subject for debate. The latter does give more facility for adjustment of small settlements which are likely to occur, but the application of the hot laid surface (which has considerable strength in itself) gives the maximum strength to the whole road when it most needs it and the Ministry of Transport at present favours this course.

Precast Concrete Bridge Units

The construction of motorway bridges has fallen largely into two parts—the substructure which is invariably in site work and the superstructure, which is

(Continued on page 37)

THE PROBLEM OF MOTORWAYS IN FRANCE



A. RUMPLER
President, Permanent International
Association of Road Congresses,

FRANCE has an exceptionally close network of roads as, in addition to 82,000 kilometres of National Highways, it includes 1,350,000 kilometres of roads of different administrative classifications, namely, "Departmental" (i.e., State Highways), "Communal" (i.e., Local Body Roads), "Rural" (or Village Roads) and "Urban Roads". This network is a heritage which has come down to us progressively in the course of many centuries.

It can thus be seen that the problem of building motorways in France has arisen in quite a different way than in the case of other countries, and that the comparison between the length of motorways existing in France with that in the neighbouring countries is not as unfavourable as would appear from a superficial examination of the question.

Again it is necessary to add, as the data of the problem, the great diversity in the geographical, demographical, economic and political conditions in France and the neighbouring countries.

When towards the end of 1935, the development of automobile traffic led the Highway Engineers to re-examine the road network, the conclusion was

that, except in certain very exceptional cases—e.g., bye-passes at big towns or cities—the best use of the highway funds would be to improve the characteristics of the existing roads, particularly on some important routes selected in view of their importance.

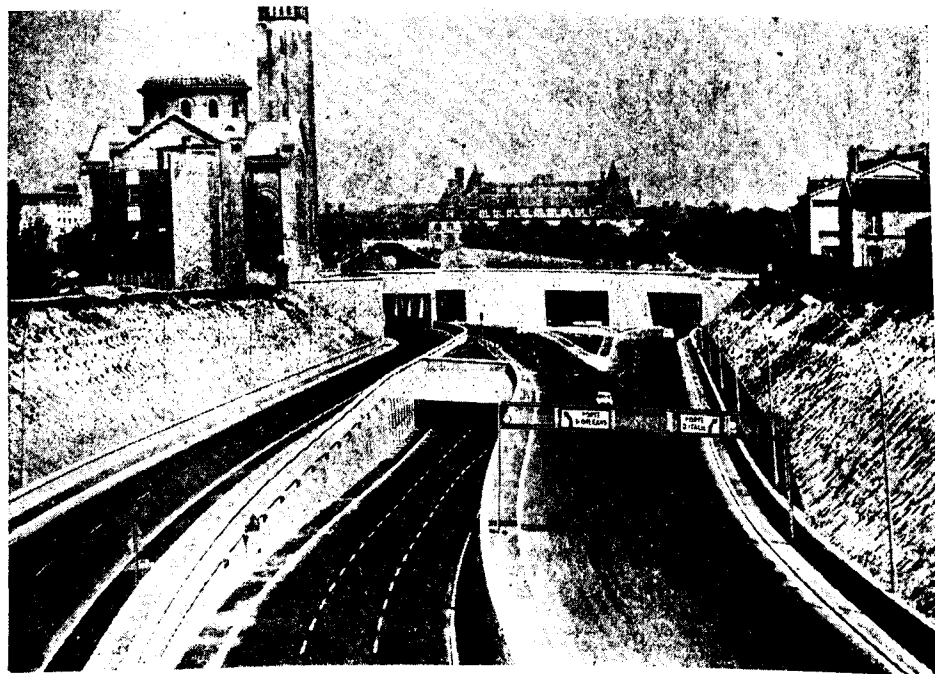
And yet it was at that very time that the construction of the Western Motorway at Paris, and of the motorway links in the City of Lyon was taken up and the construction of a motorway connecting Paris with the North of France and Belgium was projected.

After the Second World War automobile traffic increased by leaps and bounds, and as a result of the creation of the Highway Investment Fund, the programme was extended to the construction of four motorways meant to bypass the great urban centres of Paris, Lille, Marseilles, and Lyon.

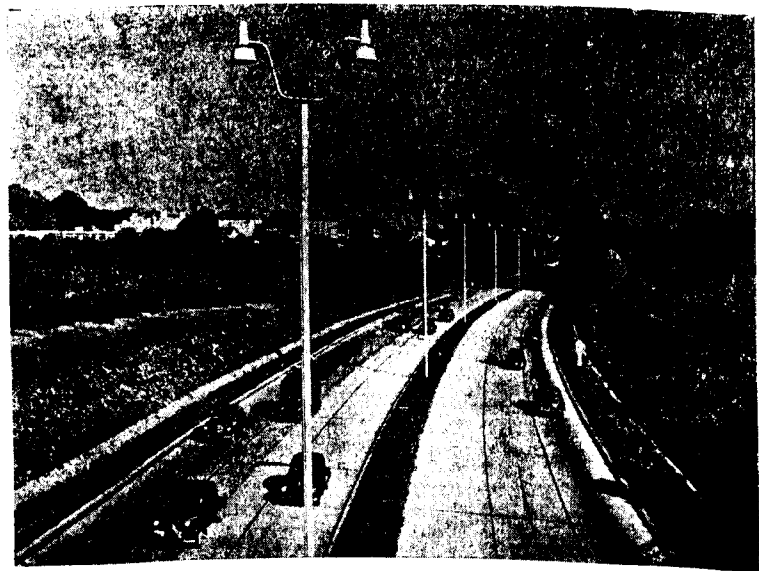
Then, in 1955, a Five Year Plan comprising 640 kilometres of motorways connecting cities was drawn up.

As a result of the implementation of this Plan, and those preceding it, 217 kilometres of motorways have up to now been opened to traffic.

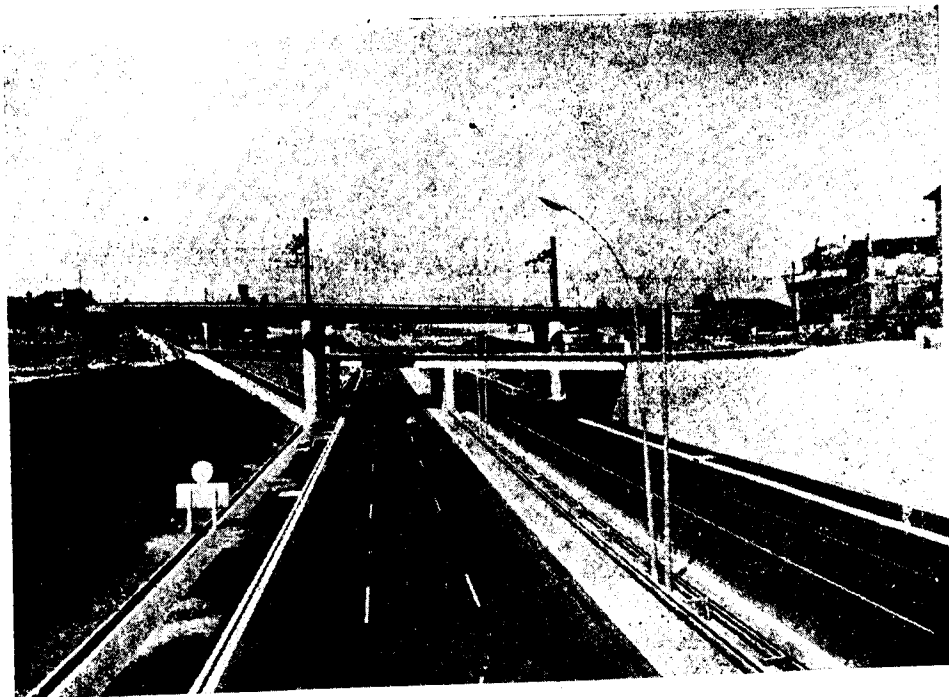
This programme was extended by a decision taken in 1960 concerning 3,306 kilometres of motorways, the comple-



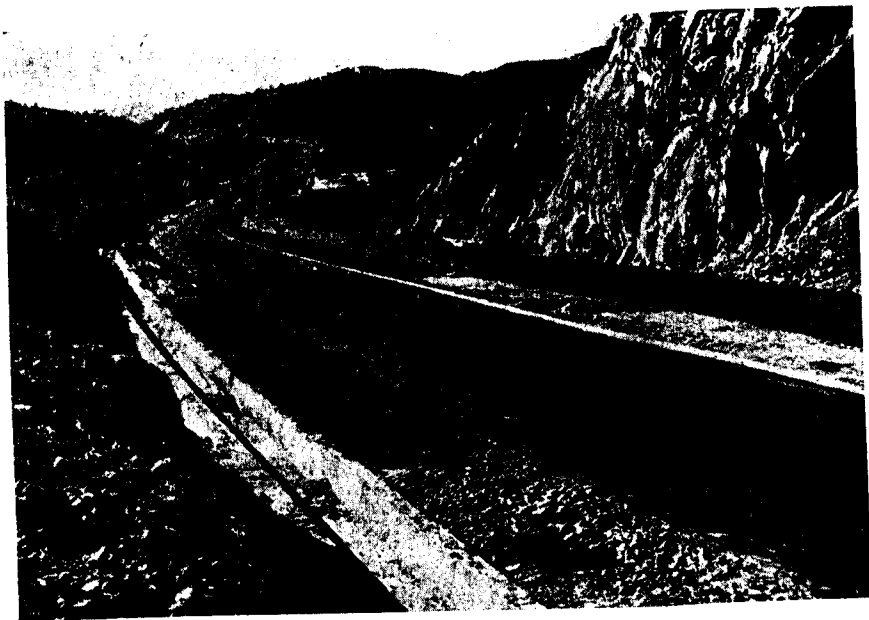
Southern Motorway—Paris
Connection with the Ring Road of Paris
View taken from the North



Western Motorway—Paris



Southern Motorway—Paris
Three-level structure



Esterel-coat D'Azur Motorway



Esterel-Cote D'Azur Motorway—Toll post at Mandelieu (Aerial View)

Western Motorway—Paris
Access Tunnel

tion of which is expected during the period 1960—1975. The programme has three degrees of priority :

- 1st priority ... 1,828 kilometres
- 2nd priority ... 571 kilometres
- 3rd priority ... 907 kilometres

If to this, the length of motorways in use or under construction is added, in 1960 we get the total length of 3,560 kilometres.

The choice between the construction of a new motorway and the improvement in the characteristics of an existing National Highway rests essentially on the volume of estimated traffic. A motorway is the only reasonable remedy applicable to the main routes, called upon, in the near or distant future, to carry heavy traffic, that is to say, going upto 10,000 vehicles per day.

Moreover safety of traffic under such conditions implies the characteristics of a motorway :

Control of access—suppression of crossings, separation of traffic in opposite directions. Experience gained so

far shows that, for equal traffic volumes, the motorway leads to 60 to 70 per cent reduction in the cost of accidents as compared to a road of normal design characteristics.

The general characteristics of motorways are clearly defined in France by legislation :

“ Highway routes with special destinations, without crossings, accessible only at points meant for the purpose, essentially reserved for mechanically propelled vehicles”, and on which “the abutting properties do not enjoy any right of access”.

Statutory provisions impose certain restrictions on traffic of motorways, notably :

prohibition of parking on the carriageways or on the berms, except under circumstances beyond one's control, and in that case, obligation to do everything possible to clear the roadway quickly ;

prohibition of the use of openings in the central verge meant for needs of maintenance, and making 'U' turns through them.

What are the geometric characteristics of motorways.

As regards the cross-section, two carriageways of 7 metres each are provided, being increased to 10.50 metres each on sections common to two motorway routes. The carriageways are separated by a central verge of 5 metres width and are flanked by berms whose width is determined by topography and environments. The formation width, accordingly varies between 25 metres and 27.50 metres for a motorway with two 7-metre carriageways, and between 30 and 33 metres for a motorway with two 10.50 metre carriageways.

The carriageways have a uniform slope of 1.5 per cent to 2 per cent towards the outside, carried through curves on 1,000 to 1,500 metres radius but, in that case, towards the interior of the curves.

The radii of horizontal and vertical curves are fixed as a function of the design speed, which again depends on topography and site conditions.

Wherever possible every effort is made to impart to the Plan as to the longitudinal section characteristics corresponding to a high design speed but as the chief merit of a motorway is essentially traffic capacity, a high capacity is preferred to a high design speed.

For instance, the design speed was fixed at 160 kilometres per hour for the motorway in North of France (plain country) and 80 km per hour for the Esterel-Cote d'Azur motorway built in a very markedly mountainous region.

As regards the choice of the type of pavement—bituminous carpet or cement concrete—it depends, in each particular case, only on the initial cost of construction and the present value of the

subsequent cost of maintenance and renewals.

The entries and exits from the motorways as well as intermediate accesses, consist only of lateral disengagements without any cutting across, and are limited in number, as each access is a point of conflict creating disturbance in traffic. At intermediate access points acceleration and deceleration lanes of 200-metre lengths are provided.

Motorways comprise certain supplementary works:

On straight alignments the central verge is planted with shrubs, arranged in groups, to prevent headlight glare in sections where street lighting is not provided;

A tele-communication table is laid to serve telephonic call boxes for assistance regularly spaced to enable contact with one or more buildings where services in charge of enforcement are located;

And, of course, in the case of toll roads toll collection posts are situated at access points.

As a matter of fact, it is admitted that on the one hand, motorways which serve as bye-passes in the neighbourhood of big cities should be free from tolls because it is important that they should attain the maximum traffic capacity and because the return from tolls is very small on the short lengths of these motorways. On the other hand, it has been decided that motorways connecting big cities should be toll roads.

The principle of this levy is, indeed, justified, although it involves certain drawbacks: Extra cost of construction and some evasion by traffic;

The toll is not a fiscal measure but is a remuneration for service rendered;

It prevents the development of disparities between different means of transport;

And above all it lightens the burden on the national budget and thus brings nearer the time when the construction of a motorway can be taken in hand.

To bring tolls into effect, the Government has constituted societies with mixed economy in which public bodies as well as private concerns are associated. The works are then financed by capital contributions or by raising loans, the interest and amortisation of which

are guaranteed by the income from tolls, the loans being further underwritten by the public or private bodies concerned.

The first toll motorway built in France is the Esterel-Cote d'Azur Autoroute which runs along the Mediterranean coast for 50 km and was opened to traffic on July, 1961.

(The above article was very kindly translated from French to English by Shri J. M. Trehan, Deputy Standards Officer, Government of India, Ministry of Transport & Communications, Department of Transport, Roads Wing, New Delhi.)

(Continued from page 30)

largely precast. The quality of precast units has reached a high level and the finish is, for the most part, excellent. The small compact anchorages and smaller stressing gear currently used have made easier the secondary prestressing and transverse-stressing in restricted spaces.

Much experience has been gained in high-grade concrete and this can now be produced with consistently high strengths. By early striking of shuttering, rubbing down with a wooden float, and the filling of blow holes with a silica sand, grout can effectively be done. Bush hammering is frequently used as a surface finish and also point tooling, though the latter is more expensive. Plastic linings to shutter panels are being used to give a large and rugged block finish. It is essential, however, whatever the finish, for good shuttering and good concrete to be used.

As far as decks are concerned, a plain line seems desirable; ornamentation of the deck face does not enhance the appearance of the bridge.

Further research seems necessary to overcome difficulties arising from settlement at the point of junction between bridge abutments and main banks. It is possible the use of a wedge-shaped piece of weak concrete might allow compacting plant to be used right up to the abutment.

Among the many bridge works so far completed, the Don Viaduct, 760 feet long, has an interesting design, to cope with anticipated extensive mining subsidence; Barton High Level Bridge, 2,455 feet long, carries the road over the Manchester Ship Canal at a height of 100 feet; Creswell Viaduct, half a mile long, has three bridges connected by embankments which are built on reinforced concrete rafts, bearing on piles carrying the load down through peat to the strata below.

Acknowledgements

Acknowledgements are made to H. BOWDLER, M.I.C.E., M.I. MUN.E., F. INST.H.E., R. F. EARLEY B.S.C., M.I.C.E., M.INST. H.E., and W. KIRBY LAING, M.A., M.I.C.E., M.INST.H.E., whose papers have provided the basis for this article.

THE BASIS OF THE IMPROVEMENTS TO THE BELGIAN ROAD SYSTEM (NETWORK)

By
H. HONDERMARC *

BELGIUM is a small country situated in the Western part of Europe between France and Germany and Holland and opposite to England. Its area is 30,507 sq. km. its population has reached 9 millions and has increased on average by a million, every 25 years. Its density of population is one of the highest in Europe.

The road network is also very dense, surfaced roads, inclusive of the urban thoroughfares, total to nearly 53,000 km. Of this total, important routes of communications happen to account for

about 23,000 km. Out of this 10,000 km are in the charge of the Government. The length of free ways or motor-roads, (auto routes) so far in use is about 200 km. it is envisaged that their length would rise to 550 km by 1965 and, ultimately reach 1,528 km. Simultaneously, the standardisation of the most important routes will be proceeded with.

The problem of improvements to the Belgian road network is at present dominated by the growth of number of automobiles. Its development since the end of the war is given in Table 1.

TABLE 1
Development of Belgian Automobile

Year	Motor cars	Auto bus	Lorry	Total
1944	12,957	141	28,266	
1945	46,120	261	60,234	41,364
1946	86,001	790	91,039	106,615
1947	129,329	1,439	110,732	177,830
1948	177,288	1,942	125,739	241,500
1949	226,961	2,176	130,811	304,969
1950	273,599	2,325	142,241	359,948
1951	304,044	2,495	151,480	418,165
1952	343,000	3,069	147,306	458,019
1953	368,057	3,363	159,026	493,375
1954	439,623	3,352	132,052	530,446
1955				575,025
1956	537,131	4,389	144,146	685,666
1957				
1958	633,349	5,058	155,915	794,322
1959				
1960	782,111	5,500	170,282	957,893

*Director General of Bridges and Roads, Belgium

COMPARATIVE INDICES OF THE GROWTH OF VEHICLES AND OF TRAFFIC
1949 = 100

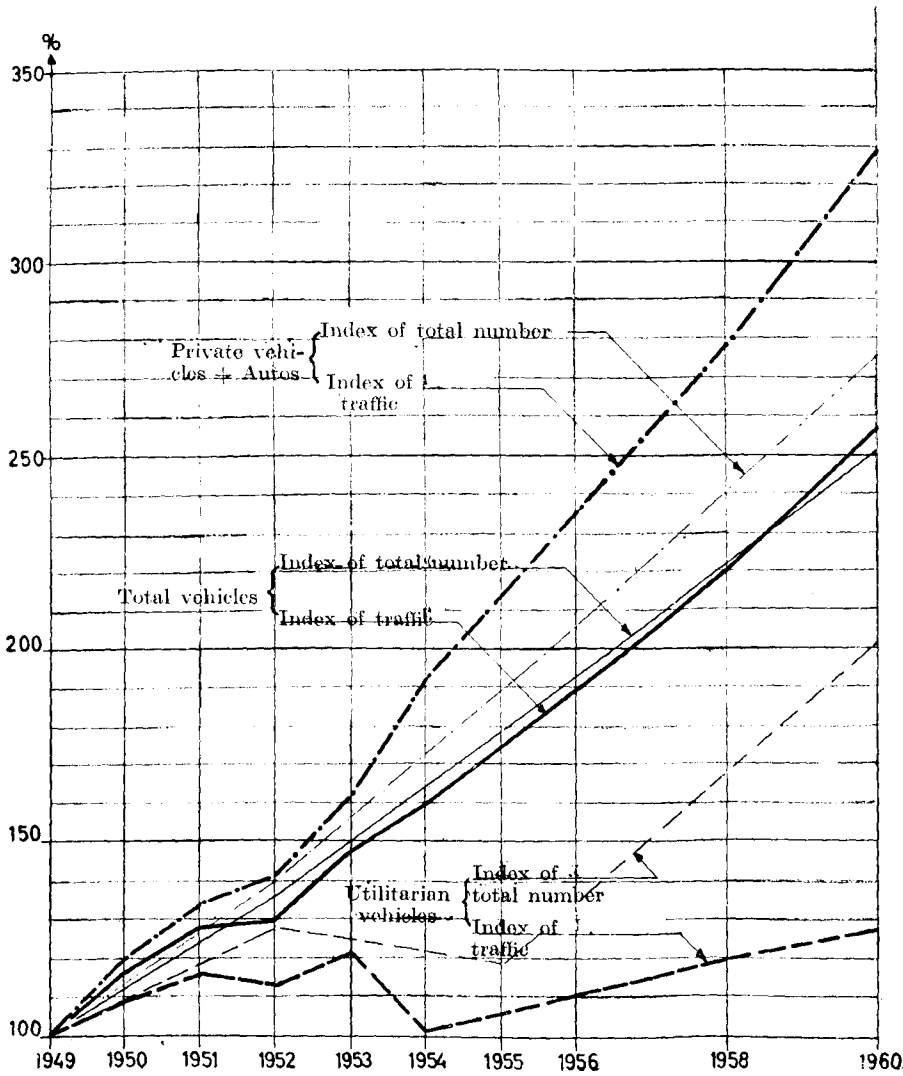


Fig. 1

It can be seen that before long we would reach a figure of one million vehicles. The largest portion consists of private vehicles above 80 per cent, and small proportion of commercial vehicles viz. about 20 per cent which has increased less rapidly than the private vehicles. None-the-less from the point of

view of traffic, reverse is the case. Attached diagram, fig. 1 shows that since 1949, the index of traffic of utilitarian vehicles have increased more rapidly than that of total number of vehicles in circulation where-as the index of traffic of private vehicles, on the contrary, has remained constantly below the index for the total number of private vehicles.

For the entire traffic, it can be said that it has increased in the same proportions as the total number of vehicles. But this is not true on the whole: on certain routes, particularly important ones, the traffic has increased more rapidly than the increase in total number of vehicles. Also notwithstanding the density of the network, most of the roads have attained or exceeded the admissible level of saturation.

The estimates of the growth of total number of vehicular automobiles and the traffic in the ensuing years is no less impressive. The investigations made on many sides make one to think that in about 20 years, the number of vehicles would reach or exceed a figure of 3 million vehicles and that it would continue to increase upto about a limit of 400 to 500 vehicles per 1,000 inhabitants. It could be expected that at that time the actual number of vehicles and the traffic generated will be five times higher.

On account of the rapid increase of the traffic, unfortunately it is very frequent that the improvements of roads carried out do not correspond to the future needs of the traffic and only cater to the traffic level which has already been exceeded since many years. In that case, the realisation of the urgent improvements necessitate measures which are, generally more costly than those initially needed for the requirements of traffic.

The Belgian Roads Administration has thus been led to conceive of a combined plan, taking into account the maximum anticipated needs of the traffic and those which would bring about the integration of the measures aimed at the progressive adoption of road network to intermediary levels of traffic. Of course, the system of the execution of works is influenced by factors other than the traffic and in particular the economic contingencies. But what is essentially of importance is that the improvements realised progressively should be consistent with the ultimate solutions and safeguard the possibilities of their realisation.

How to face the economic situations, finances available and the increase in demands of traffic?

The study of this question undertaken for some years in Belgium, has led to conclude that it is possible to find a satisfactory solution of this problem.

It can be stated that, in fact, the road traffic is very unequally distributed on the arteries of the road system (network) in as much as that about 70 per cent of the total traffic is concentrated on 30 per cent only of the length of the national highways. The greatest part of the traffic is thus found to be concentrated on some main roads or axis which connect the centres of economic activity of the country. By limiting the construction of new roads of very high capacity to these axis it is possible to restrict the expenditure for the rest of the network to only more or less improvements without construction of new roads.

Considering the very high capacity that new arteries should have, it is indispensable that they should be conceived in the form of freeways (autoroutes) because the traffic capacity of each lane of such roads is $2\frac{1}{2}$ times of that of ordinary roads.

The map, given below, Fig. 2, represents what the Roads Administration has estimated necessary for the ensuing period of 20 years. The freeways repre-

than 5,000 population. The network thus constituted, would in their turn be completed by a network of secondary roads, not represented here

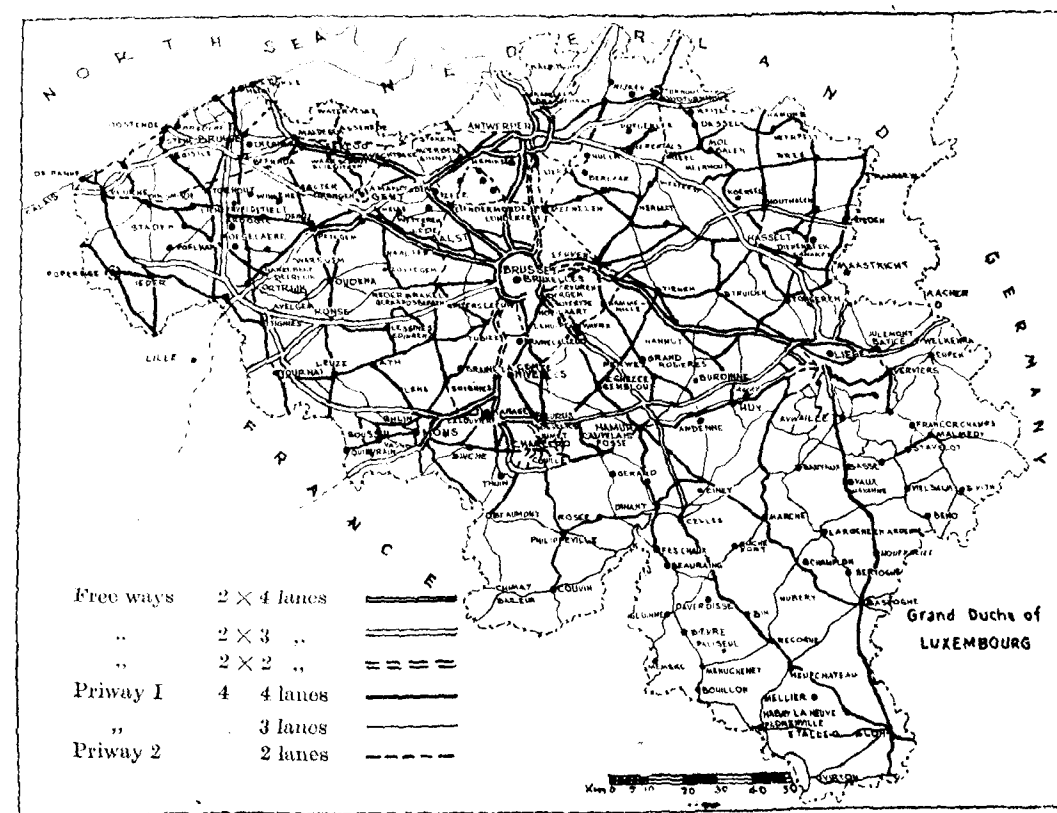


Fig. 2. Advance plan of road system 1960

sented by a double line are laid out new in site, to a formation of sufficient width to permit their ultimate widening. Their length totals to 528 km.

The primary network, represented by a single line, does not differ in fundamentals from the present network improved by widening and standardisation. The new roads only totaling to 423 km. This network divided into two categories of roads, called primary I or primary II, according to their importance, serve all localities with more

and having a total length of about 3,000 km.

On the whole, 1,307 km of freeways and 423 km of new roads must, still, be constructed while 2,187 km of roads must be converted from 2 to 3 lanes, 156 km 3 to 4 lanes and 327 km of 2 to 4 lanes.

The manner of improvement of the Belgian road network would be integrated harmoniously with important European itineraries defined by the Organisation of the United Nations (Fig. 3).

Standards have been drawn up for the construction of the new roads or the improvement of the existing. The section would comprise, at least, two lanes of traffic with each 3 meters width.

The roads which form a part of the international network have been dealt with in a manner to take head of all the international standards promulgated at Geneva.

Fig. 4 gives the number and the width

of the lanes and Fig. 5, the other geometrical characteristics of the five categories of standards adopted in Belgium. The standards have been drawn up with reference to the safe speed and chosen in relation to the importance of the traffic and the relief, that determine the classes.

The corresponding standards for capacity are given in Fig. 6 (practical capacity outside built-up areas). They correspond appreciably to American Standards.

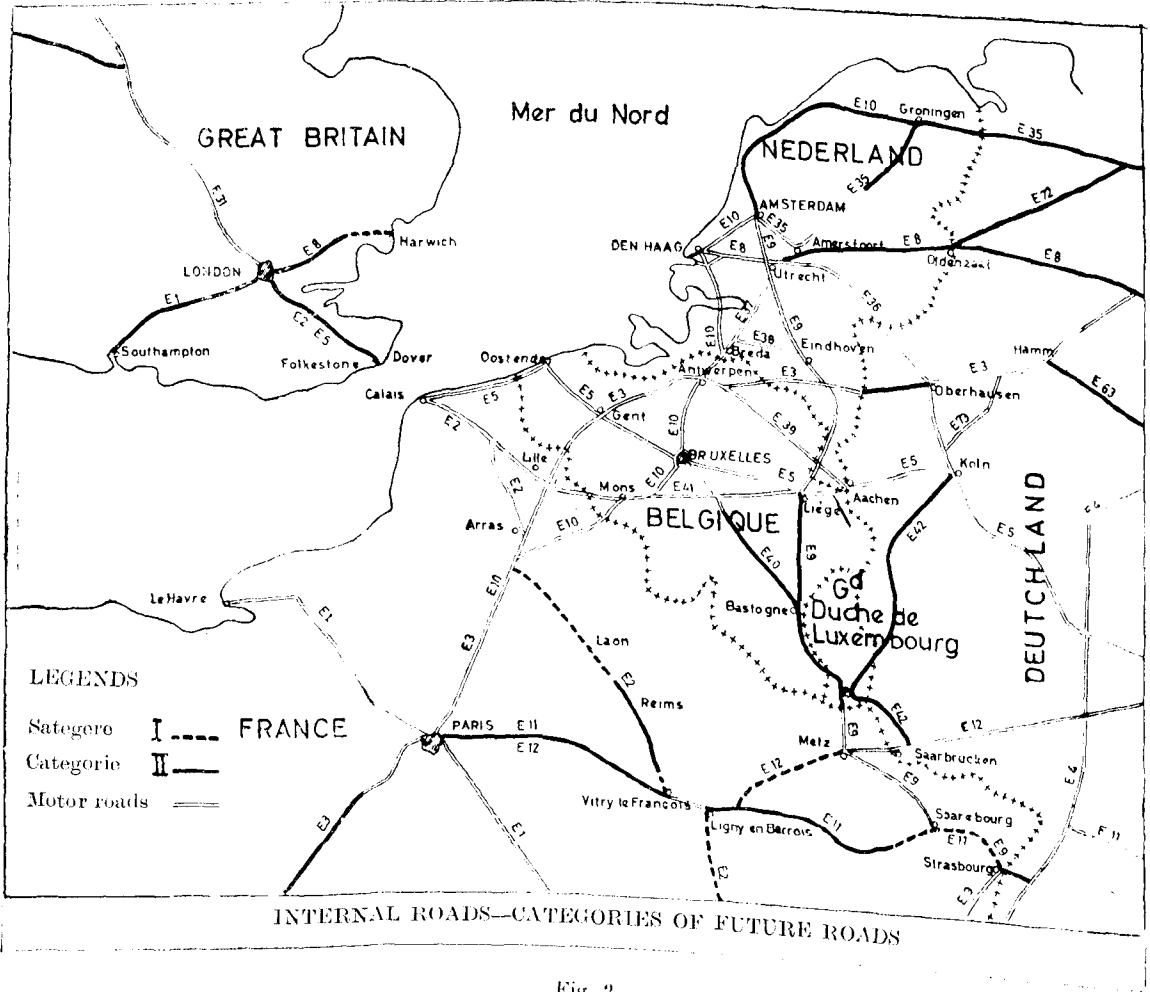


Fig. 3

Category of the road	Number of Traffic lanes	Width of lane	Standard		
			Class I	Class II	Class III
Freeways	4 and more	3.75	A	B	C
Primary System I	3 or 4	3.50	B	C	D
Primary System II	2	3 or 3.50	C	D	E
Secondary System	2	3	D	E	E

Fig. 4

Belgian Standards

Standard	Safe Speed in Km/h	In plain Minimum Radius in m	Longitudinal Section			
			Straight Alignment	Final max. Gradient	Maximum gradients in per cent	Cross Sections Max. in per cent
			R. Convex minimum in. m	Visibility min. in. m		
A	120	770	19,500	300	4	3
B	100	470	9,200	200	5	4
C	80	260	3,700	130	6	5
D	60	130	1,200	70	8	6
E	40	50	300	35	10	8

Fig. 5

Practical Capacities

Category	Number of lanes	Volume		Absolute index of capacity	Index of capacity per lane
		Hourly	Per day		
Freeway	2 x 4	7,000	60,000	10	2.5
"	2 x 3	5,500	45,000	7.5	2.5
"	2 x 2	3,500	30,000	5	2.5
Ordinary Road	4	2,500	20,000	3.3	1.66
"	3	1,200	10,000	1.7	1.11
"	2	750	6,000	1	1

Fig. 6

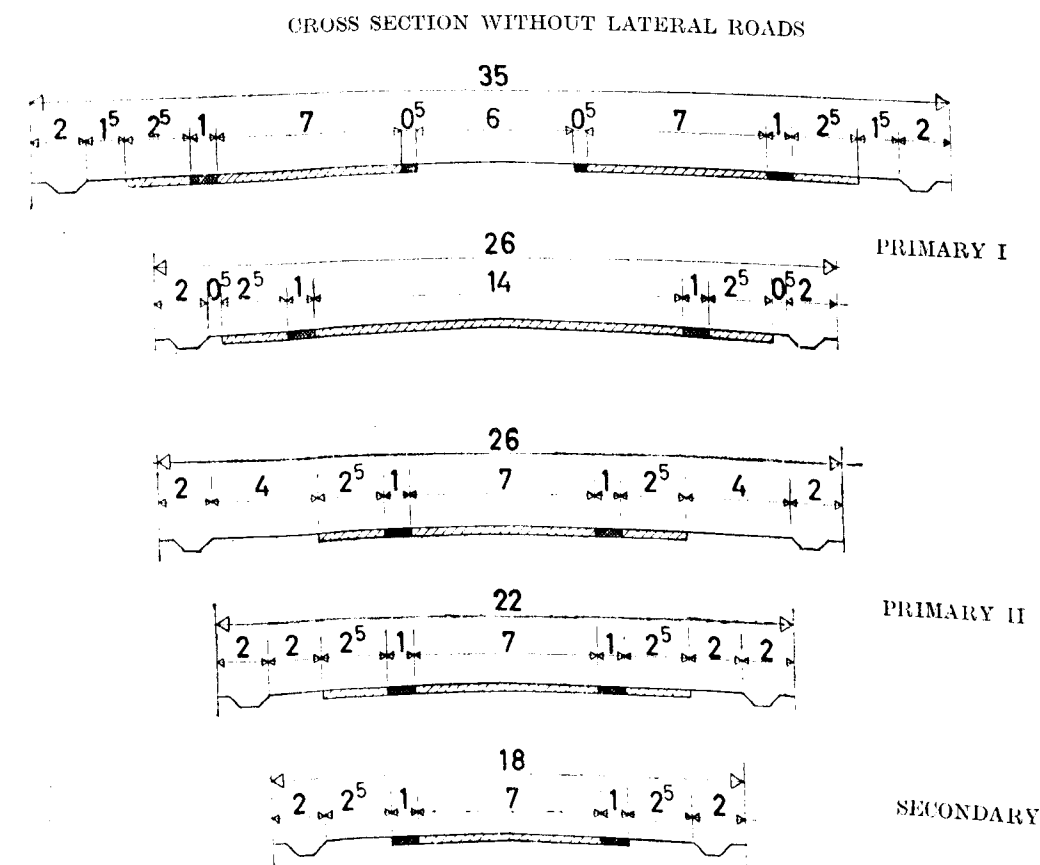
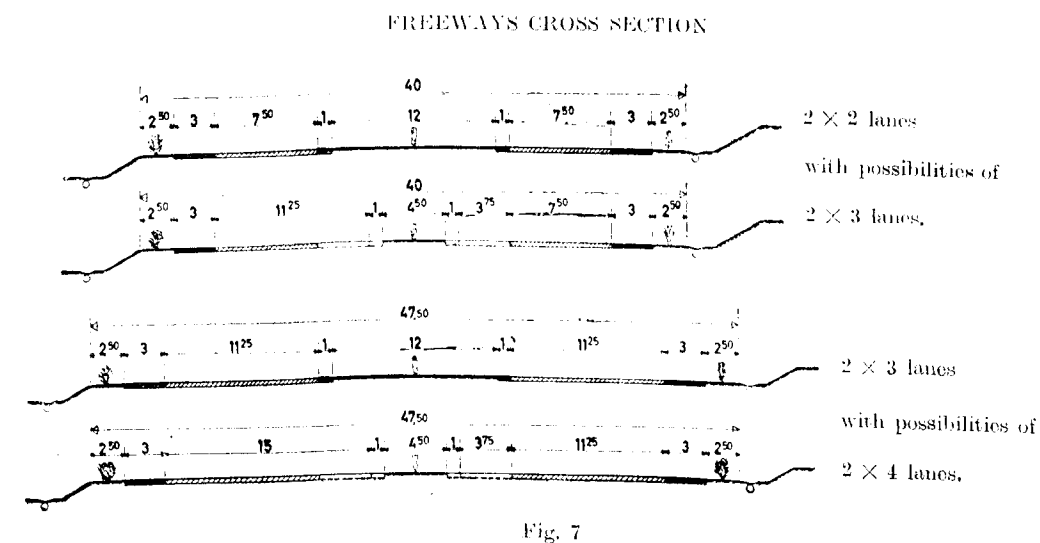


Fig. 8

The international standards recommend routes with more than two traffic lanes for a traffic flow of more than 600 vehicles per hour with 30 per cent of heavy vehicles or 900 vehicles per hour without heavy vehicles. It is further recommended that every time it is possible, divided highway with a central marginal strip should be provided.

This layout has been adopted for all the Belgian Freeways and Expressways. Yet, in the modernisation of the existing roads, simple 3-lane roads have equally been provided for 30th peak hour volume corresponding to 750 to 1,200 vehicles per hour.

3-lane roads have, sometimes, been represented as excessively dangerous. In reality, for a volume of traffic between 5,000 to 10,000 vehicles per day, their accident rate is lower than those with two-lanes. On the contrary, they are well adapted to a traffic volume unequally distributed in both directions, which is generally the case of peak volumes. And they permit accordingly an improvement of the itineraries when, as it is the case in Belgium with high density of population, the widening of formation for the construction of a divided highway would lead to mass scale destruction and expenses of expropriation outrightly prohibitive.

When a divided highway is imperative, the economic solution in our country is generally the construction of a new road outside the built-up areas.

Figures 7 and 8 represent the typical cross-sections of different categories of roads constructed with reference to the above standards.

The width of the traffic lanes are to be increased in curves. Cycle paths have been provided with for a daily average volume beyond 500 cycles per day.

The distance between the alignments normally permits the provision of a parking or safety zone of 2.50 m. to 3.00 m. between the carriageway and the cycle path.

When the normal width between alignments is not realisable, one sacrifices successively the berms, the ditches and finally the parking strip.

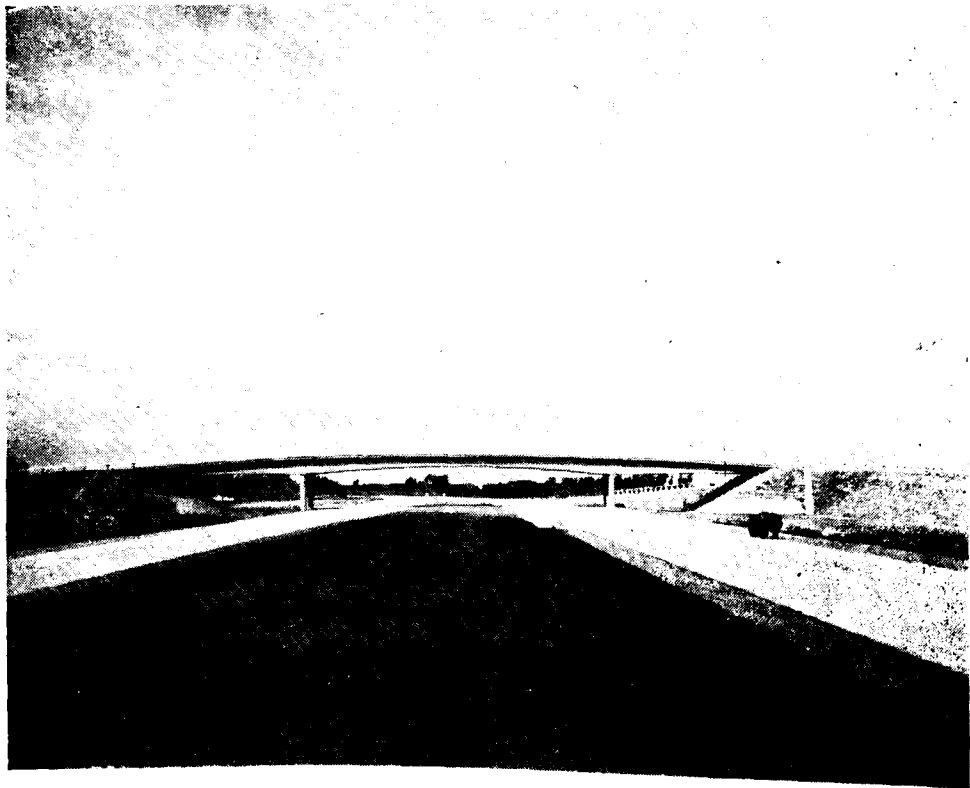
The photograph given below shows an example of recent constructions of a freeway, of roads with 4, 3 and 2 lanes of traffic of a primary network and also one road belonging to the secondary network.

There is still to say a word about financing the works provided in the road programme.

In principle, the financing of road works is the responsibility of the State. Nevertheless, the necessities of the financial policy of the Government often hinder the grant of as much credit to road works as would be necessary for permitting continuous realisation of a rational programme. That is why the Government created in 1955 a special organization for financing road works, called "Autonomous funds for roads". This organization enjoys a legal individuality distinct from that of the state and can by itself issue public loans upto total amount fixed for a period of 15 years.

The financial responsibility of these loans is guaranteed by the State, which reserves to itself the right of the financial administration of the funds.

This organization has given vigorous impulse to the road works in Belgium. During the first 5 years of its existence, it has to its credit of making 1,829 allocations totalling to 13.043.000.900FB having permitted the construction of 187 Km. of freeways, improvement of



Motor Anvers-Liege-Wommelgem Autobahn of 2 x 2 lanes with central Strip of 12 m

1294 Km. of ordinary roads and the construction of 201 bridges.

Belgians wish that this organization could achieve the work that it has well begun, in union with other European contries for the revival of the greatness of their dear country.

[The above has been very kindly translated from French to English by Shri S. K. Rajagopalan, Divisional Engineer (Consultant), Government of India, Ministry of Transport & Communications, Department of Transport, Roads Wing, New Delhi.]

“ Among the social services, I put communications first since I do not see how it is possible to effect any great improvement in health and education in the village of India unless they can be reached surely and quickly at all times.

—LORD WAVELL

ROAD CONSTRUCTION IN EGYPT

By
Y. EZZELDINE

Introduction

MOTOR vehicles are influencing in an ever increasing extent, the popular economy of all countries. If roads are to be useful they must give the shortest connection between all parts of the country and must be so built that they should remain smooth and serviceable with a minimum expenditure for maintenance.

Since the volume of traffic and the number of motor vehicles is conti-

nuously increasing (in 1959: 1,03,000 vehicles), the road construction has become an all-important problem of life in Egypt.

The real development of the net-work of roads in Egypt began only since 1952; when the revolutionary government instituted studies on industrialisa-tion and construction, special attention was also devoted to road construction projects.

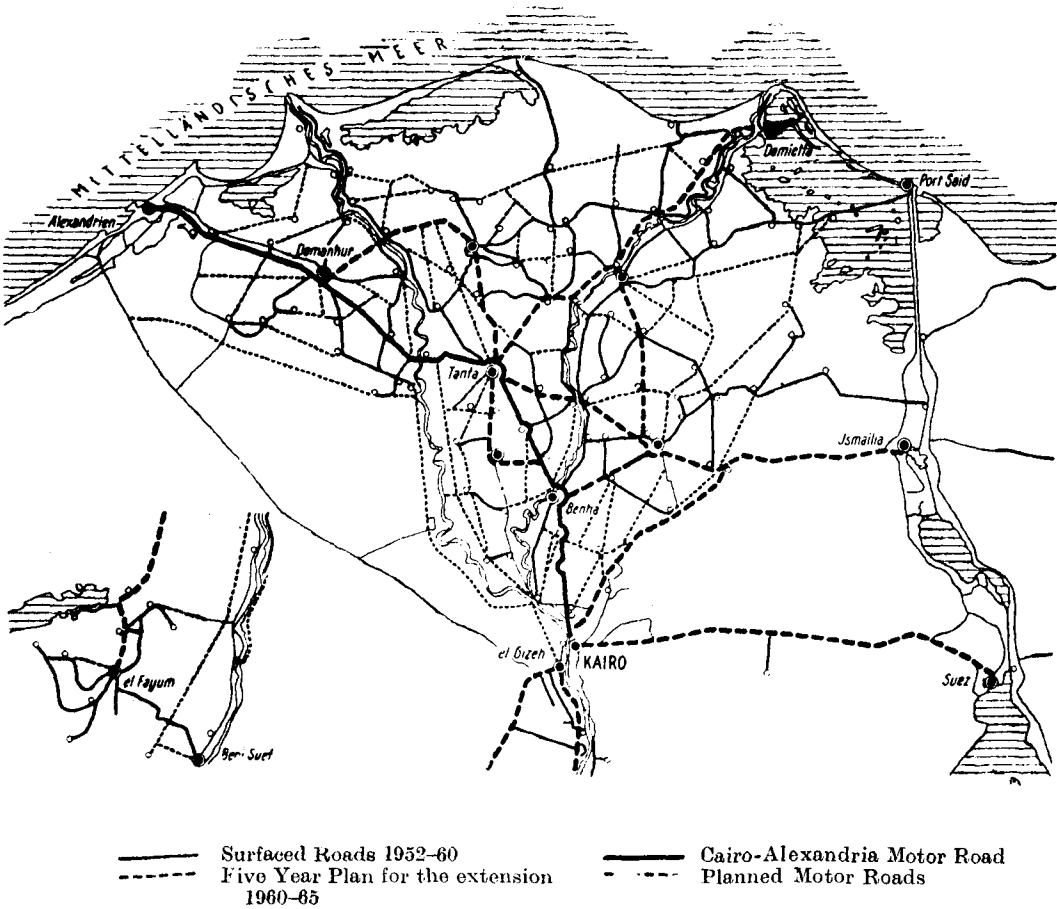


Fig. 1 Extension Scheme for the roads in the Nile Delta upto 1965.
(a) Surfaced roads 1952-1960. (b) Five year plan for the extension 1960-65.
(c) Cairo-Alexandria Motor road. (d) Planned Motor Roads.

Geographically, Egypt possesses a remarkable position. It lies at the intersection of the principal routes of three continents, Africa, Europe and Asia. The significance of this position for international traffic both on land and sea as also in the air cannot be sufficiently emphasised. It is, therefore, important that we must devote the greatest attention not only to the planning of our internal road system but also to their construction in accordance with the necessary standards.

History of Roads in Egypt

The history of road construction in Egypt is lost in the darkness of the past centuries till 2,700 before the period of our era, when mankind constructed the first connecting way between the stone quarries in the Eastern Nilufer and the construction sites of the pyramids. The connecting way or road—about 10 km, long—was plastered with lime-stone and bore for centuries the burden of traffic, even without the help of bitumen which

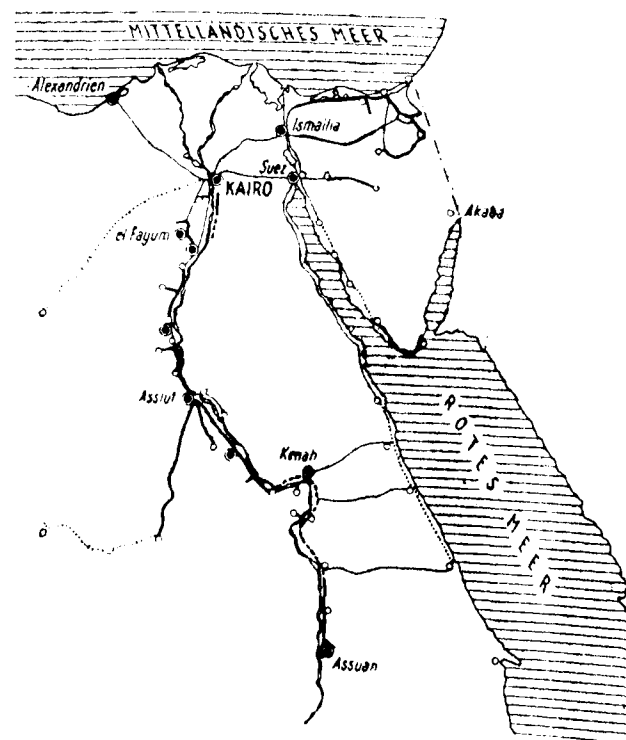


Fig. 2. Extension Scheme for roads in upper Egypt till 1965.

- (a) Road already reinforced before 1952. (b) Roads reinforced in the years 1952-1960.
(c) 5 year plan for the extension 1960-65. (d) Reinforced roads for industrial purposes.

It is obvious that the level of road construction in our country must be so maintained that the constructed roads can not only take up the increasing traffic resulting from our own development, but our roads must also be suitable for the task within the network of international connecting roads between the countries of the Middle and Near East.

The Development of Roads in Egypt

If we now turn to modern times we find that road construction in Egypt was begun at the commencement of the twentieth century under the direction of a small department in the Ministry of

Public Works, which in those times was predominantly concerned with irrigation problems and roads were used only as facilities for the maintenance of irrigation canals. In 1912, this small department was separated from the Ministry of Public Works and the Department of Roads and Bridges—now called the Roads and Inland Traffic Department—was founded and was concerned with the maintenance of the non-metalled roads along the canal dams, in order to connect the different parts of the country with each other. In the period between 1912 to 1936, the maintenance of the non-metalled roads was its chief task, and only 400 km of roads were actually metalled—an average of only 17 km per year.

In 1936 the activity of this department increased, and on an average, up to 1952, about 130 km of roads per year were improved. During the 7 years after that, the annual average of newly strengthened roads increased to 300 km. Today there exist over 5,700 km of surfaced roads in Egypt and more than 2,000 km are under construction, though at the same time we find still 12,000 km more of non-metalled roads as in Figs. 1 and 2.

Problems of Road Construction

The Government has clearly recognised the importance of establishing the greatest possible lengths of roads in the shortest time, since roads are in demand for the national security and defence as well as for the national wealth and economy.

Besides the administration costs of the Highways Department and the costs for maintaining the non-metalled roads, the yearly budget for the actual road construction amounted to not more than $\frac{1}{2}$ million Egyptian pound. From 1952 till to-day the yearly budget for road construction has increased step by step up to 6 million Egyptian pound. At the same time the collections from taxes on motor vehicles, petrol, tyres, etc., are esti-

mated at about 14 million Egyptian Pound for 1958. We can, therefore, say that less than $\frac{1}{2}$ of the money collected from the road users is spent on Road Construction.

1. THE EGYPTIAN POTENTIAL FOR ROAD CONSTRUCTION

(a) Favourable conditions for road construction

There are many conditions in Egypt which are favourable for road construction and for the possible development of a suitable road system.

Weather and Topography

The weather as well as the terrain in Egypt are favourable for road construction and maintenance. Since heavy rainfall and frost with its damaging effects for the road surface do not occur in the greatest part of the country, the necessary expenses for adequate roads are very much reduced. The terrain is generally flat and well drained and these conditions also considerably reduce the average costs of new roads.

Materials

Egypt is in the lucky position of possessing more than sufficient quantities of sand, gravel, rock, bitumen and cement required for road construction. There are numerous quarries for these material in the deserts and hills on both sides of Nile valley.

The Refineries in Suez can produce yearly 1,60,000 tons bitumen. The estimated consumption of this product generally do not exceed 1,20,000 tons.

The three cement factories existing in Egypt can produce yearly 1,20,000 tons of cement. The requirement of cement for road construction do not, however, exceed probably 5% of this quantity.

Public Support

Today there is in Egypt a genuine public interest in better roads which supports all road programmes.

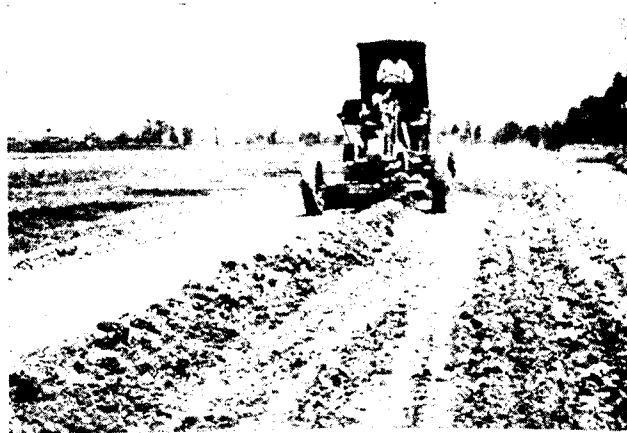


Fig. 3
Use of levelling Caterpillar tractor for making the substructure.

Fig. 4
Making the foundation with motor grader.

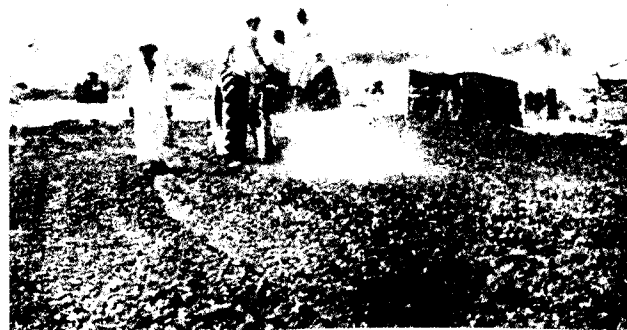
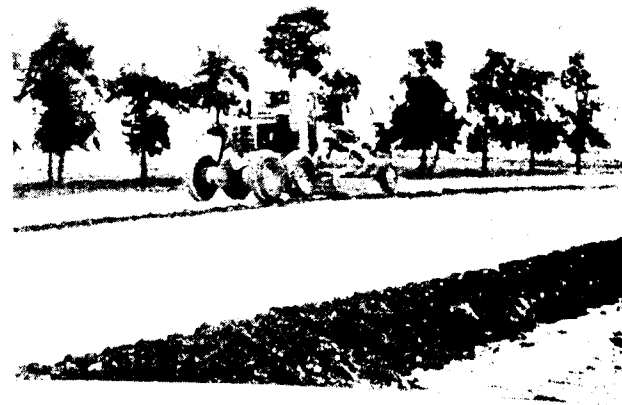


Fig. 5
Crushing of Clay soil with pulvi-mixer.



Fig. 6.
First class road after completion.

Technical Personnel

Finally the success of every road construction programme depends, to a great part, on technical personnel which is responsible for carrying out road structures. A centre for training technical hands has already been established.

(b) Unfavourable conditions for road construction

Unsurfaced Roads

The large number of unsurfaced roads of several kilometres, which probably most be taken into account in construction programmes, represent a basic problem which cannot be avoided, but the problem of classification and urgency and even the removal of some roads from the system of the country can be gone into and solved by sound technical planning.

Transport of Material

Although there are several good quarries for aggregate material in Egypt and the deserts provide material for the building of local roads, there is a perceptible want in aggregate material in large regions, especially in the delta. This problem of transport of aggregate material was very serious in the past and had increased the costs of surface-building considerably in many regions.

Soil Problems

The type of soil, occurring in the delta, often presents serious problems

for the production of sub-grade and earthwork. In many regions, the land is very costly and the roads must be logically built on embankments of the canals. In addition to the stated characteristics of the delta soil, the problem is made still more difficult by the fact that every soil which is disturbed in its natural structure no matter since what time it may have been filled up, requires very careful and costly methods of construction in order to give a reliable foundation for surface covering. High ground-water level below the road surface is like-wise a frequent problem.

Problem of Machinery

We have the problem of insufficient deliveries of road building machines and spare parts, since the countries which already possess a large number of improved roads, according to all appearances, employ essentially more implements during the construction and for maintenance of these roads. The specifications of road building require the employment of modern machines which every building contractor must possess.

2. ROAD CONSTRUCTION IN EGYPT

Road construction practice before 1950

The building of first-class road surfaces in Egypt was limited only to a few general types; a large part of this surfacing has a substructure built by heavy stones with various bituminous

basis. Other types include also concrete surfaces.

The use of the stronger hand-built basic layers for the substructure is expensive, even when relatively cheaper hand-labour may be available. Although till 1950, the single layer method of construction was employed in road-building, this method of construction has been abandoned.

The bituminous method of construction for high class road surfaces includes impregnated macadam, asphalt concrete mixtures and soil cement stabilisation.

Road construction practice after 1950

The most important development in road building in the past 10 years was the revolutionary step in the use of road-making machinery. This increased the output considerably and improved the quality of work. The soil could be better compacted, the substructure can be laid down with greater accuracy and consolidated, and the surface can be laid with incomparably better wearing qualities.

(a) Substructure and earthwork

Till now practically all the roads were built on canal embankments and consisted of the silt excavated during the construction of the canal. This silt soil forms a very weak substructure for the roads. Egypt has carried out researches for long years for improving this soil. No doubt in the past years the knowledge of the soil and its treatment has developed more quickly than the knowledge of any other facts in road building.

The part which the water content plays in the behaviour of the soil is of fundamental importance. Due to its property of binding together the soil particles, a limited quantity of water contributes very much to the bearing capacity of clay soil, while on the contrary, too high a content of water reduces very much the bearing capacity.

Now-a-days before planning any new road in Egypt very thorough and detailed soil investigations are made.

For road and embankments, the systematic consolidation of the soil is recommended. The soil must be consolidated in limited thicknesses of layer of 15 to 25 cm, whereby the degree of consolidation attainable depends on the water content. Modern earth moving apparatus like the 'probe bucket', have a good consolidation, but very often special consolidation apparatus like rubber wheel rollers, smooth wheel rollers and vibration rollers are used in special situations systematically for consolidation. Tests are carried out on the construction site in order to find out the degree of consolidation and to ascertain the necessary quality of the construction work.

(b) Surface construction

The practice of fixing the surface thickness according to the soil properties and the traffic load has been introduced only in the last 10 years. The basis for the surface construction is formed by the experiences of our Engineers on the results of the concerned roads and the study of factors which lead to damages. One of the best known methods which is applied by our department is the California Bearing Ratio (C.B.R.) test.

(c) Construction of the foundation of roads

A fundamental change in the choice of materials for the substructure has taken place in Egypt. The necessity for mechanisation had led to the giving up, almost entirely, of the traditional method of employing hand-set basic layers and instead of these broken materials are employed. The following types of foundation are now used:

- (a) Ordinary concrete substructure 15 cm thick.
- (b) Broken limestone 20—25 cm thick.
- (c) Pit gravel 15—25 cm thick.

Till then they were not used for the substructures for road building. This kind of substructure of sufficient strength contributes, however, very much to a solid quality of surface and is very much more economical, thereby costly foundation material can be replaced.

(d) soil-cement stabilisation

The soil-cement stabilisation is a development which has perhaps contributed more to economy in road building than any other process. In Egypt especially in the north of the delta where substructure material cannot be made, the soil-cement stabilisation is successfully used to a great extent.

Various kinds of additions are made for the stabilisation. Very good results were obtained by addition of cement and granular substances, whereas experiments with bitumen or oil were unsuccessful.

Approximately 200 km of road were built with stabilised substructure.

It is necessary to carry on further experiments with the Egyptian soil before bigger programmes could be carried out by using the soil stabilisation method. The Egyptian soil has qualities which must be carefully studied among which also the problem of a high ground-water level particularly in the agricultural regions.

(e) Road surfacing

In the construction of road surfaces, enormous progress has been made in Egypt. All types of surfacing were laid, beginning from the surface-treatment in a thickness of 1 in. upto asphalt-concrete of 10 cm thickness and concrete-surfacing of 20 cm thickness. The principal types of surface commonly used in Egypt are:

1. Impregnated macadam 5 cm thick
2. Double surface treatment 2 cm thick

3. Sand-Asphalt mixtures 5—10 cm thick.
4. Cold Asphalt mixtures 5—7 cm thick.
5. Asphalt concrete 4—10 cm thick.
6. Concrete 10—15 cm thick.

Modern machines and implements were employed in surface-construction in Egypt. The specifications for the construction of qualitatively high grade surfaces are very strict.

The Engineers in Egypt as in every other country are faced with the problem of making the right choice with regard to the asphalt mixtures, the aggregate substances in the mixtures and the laying of the surface.

It requires still further patient investigation to find out the best mixtures in relation to the bearing capacity and investigations of material to find out the best aggregates for the surfacing.

The motorable qualities of the roads are now-a-days essentially better than before.

3. IMPORTANT ROAD STRUCTURES IN EGYPT

(a) Cairo-Alexandria Motor Road

The most important road-building carried out in Egypt was the Cairo-Alexandria Motor Road.

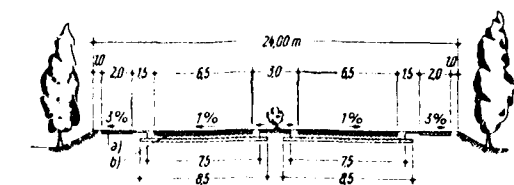


Fig. 7 Cross section of the Cairo-Alexandria Motor Highway

- (a) Track for local traffic 10 cm. gravel base, 2, 5 cm. bitumen covering.
- (b) 10 cm. asphalt concrete, 25 cm. gravel, 15 cm. stabilised sand.

This road was planned as part of the connecting ways which connects Cairo (Capital City) with Alexandria (the principal port). It has a length of 200 km and cost more than a 6½ million Egyptian pounds. It consists of a four

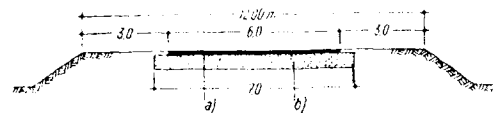


Fig. 8. Tanka-El-Mahalla (1940) Length 25 km.
(a) 5 cm. impregnated macadam, 30 cm. crushed lime stone.



Fig. 9. Assiout-Sahag Road (1956/1957) length 95 km.
(a) 5 cm. asphalt concrete.
(b) 20 cm. gravel.
(c) 15 cm. consolidated soil.

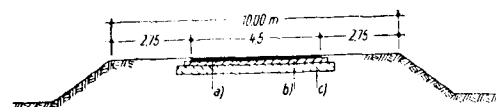


Fig. 10. Disuq Fuwa Mutubis (1956-57) length 55 km.
(a) 5 cm. sand asphalt.
(b) 5 cm. sand asphalt.
(c) 15 cm. consolidated soil.

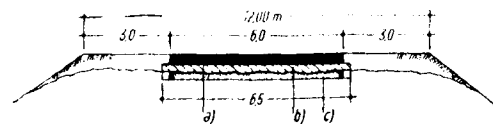


Fig. 11. Kantara-El-Arish (1956-57) length 102 km.
(a) 25 cm. sand asphalt.
(b) 7.5 cm. Reinforcing layer of sand asphalt.
(c) Old Gravel sand road.

lane road with a central strip 2 to 4 m wide.

The main characteristics of the cross-section are:

1. The substructure was formed of pit gravel for the first time in Egypt.
2. The giving of the old surface-covering which was used only

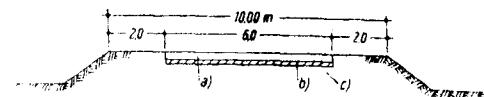


Fig. 12. El-Mahalla-Kair Kl Shikh (1956-57) length 26 km.
(a) 18 cm. concrete.
(b) 2 cm. sand.
(c) 15 cm. consolidated soil.

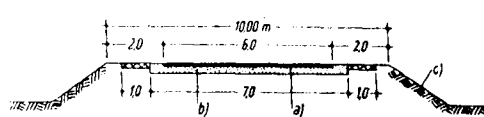


Fig. 13. Dammanhur-Shubrakhit (1957-58) length 24 km.
(a) 4 cm. asphalt concrete.
(b) 20 cm. gravel.
(c) Embankment of consolidated soil in 15 cm. layers.

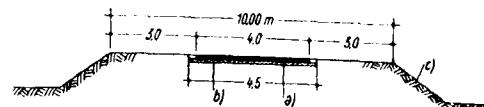


Fig. 14. Assiout - Elkharga Oasis (1957-58) length 160 km.
(a) 4 cm. sand asphalt with filler.
(b) 6 cm. bitumenised sand gravel layer.
(c) embankment in 15 cm. layers with consolidated soil.

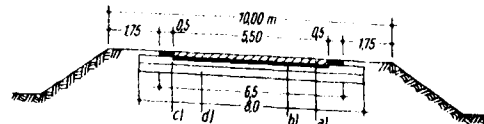


Fig. 15. Damanhour-Elmahmodia Road (1957-58) length 18.5 km.
(a) 4 cm. asphalt concrete.
(b) 2.5 cm. surface treatment.
(c) 15 cm. sand basis consolidated with clay.
(d) 15 cm. clay layer reinforced with sand.

as subgrade in sections where the motor road was covered with the old road covering.

3. The use of hot-mix for road covering.
4. The building of over bridges at the intersections with canals, roads and railways.
5. The large quantities of material, which were used in building

and indeed more than 4.5 million cbm. soil working, 350,000 cbm. sand, one million cbm. pit gravel, 200,000 cbm. broken aggregate substances, and for surfacing, 32,000 tons bitumen and 30,000 tons. filling material.

This road was open to traffic last year and carries traffic of more than 6,000 vehicles perday.

On the basis of the success of this road, the department is studying plans for a system of motor roads which connect the principal towns of the provinces with each other.

The total length of the system of motor road amounts to about 1910 km and 3 stages of construction have been provided.

1. Section	700 km	28 mil. Egyptian pounds
2. „	670 km	27 mil. „
3. „	540 km	22 „ „

For every section, a building time of 8 to 10 years was calculated according to the means available. The financing of such a system of roads must be considered by the Highest Authorities and must be defrayed from the provisions of the Government Budget.

(b) Road in the Nile valley

In the last seven years more than 1,700 km of surfaced roads were built in the Nile valley which corresponds to a doubling of the existing surfaced road construction according to different methods. It is interesting to know that more than 20 different types of roads were built simultaneously. No doubt we had many roads which were a success but of course failures also were present.

The substructures of the road were built of limestones, coarse gravel, impregnated macadam and of stabilised soil with bitumen sand clay, and sand asphalt. The surfaces were laid with surface treatment, cold mixed substances, sand asphalt mixtures, asphalt concrete, asphalt spilt and concrete.

During this time the department gained great experience which helped to find out the best method for economical road construction.

The following remarks might give an idea of the problems of construction which faced the department and which had to be solved.

1. Planning problems because of want of information on the density of traffic load and frequency. The want of accurate information and experiences of investigations on the availability and quality of road-building material in Egypt.
2. During the construction of the different types of road many problems cropped up. The sand-clay stabilisation was not successful on account of the bad quality of the Egyptian clay soil. Clay with L.L. 80 and P.I. = 30 can only be pulverised with great difficulty, and mixed with sand. Soil with 10 per cent cement proved not suitable under the above circumstances although better results were attained in other regions where the soil conditions were different. The soil stabilisation with bitumen was a failure on account of the lack of adhesion between the clay particles and bitumen.

We had also a few defects in road surfacing which were for the most part traceable to the problem of material. Lime mixtures proved unsatisfactory in many regions. We had also difficulties in the use of surface treatment on account of the fact that material and implements were not satisfactory.

On the other hand, we have very successfully built roads according to the following methods:

- a. Broken rocks
- b. Coarse gravel

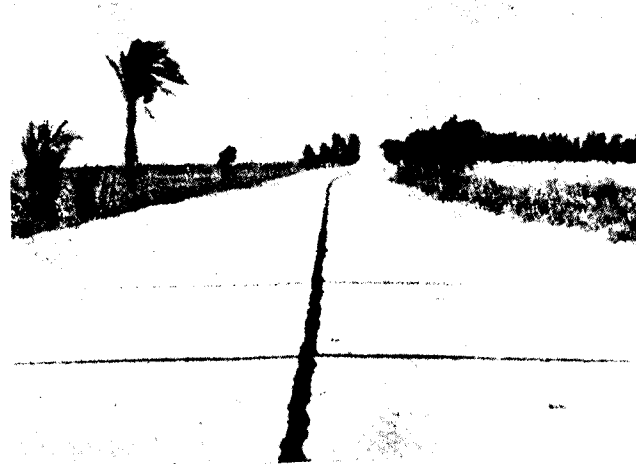


Fig. 16
Concrete road in Egypt.

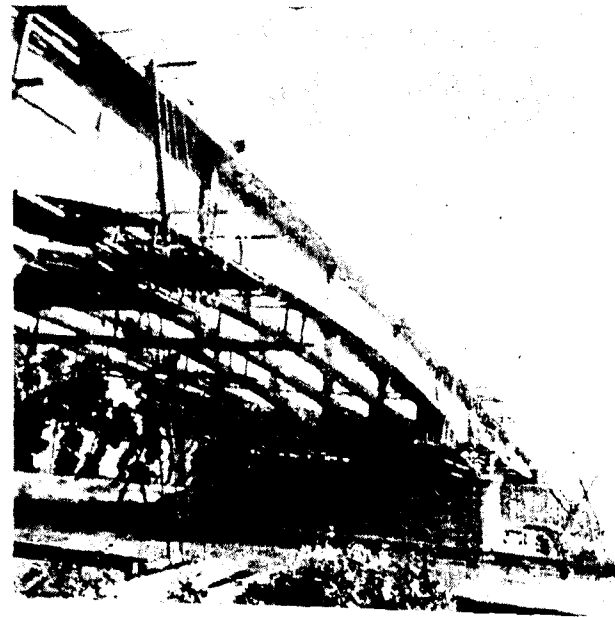


Fig. 17
Road Bridge in
Reinforced concrete.

- c. Asphalt impregnated macadam
- d. Sand asphalt mixtures
- e. Asphalt concrete
- f. Cement concrete

There are 5,700 km. of black top roads and 400 km of concrete roads. The concrete construction is used to a great extent in Air ports.

(c) *Desert Roads*

We possess already over 2,500 km of

surfaced roads and further 2,000 km are in the course of building.

Actually we have no problem in the construction of roads in the desert regions. All building materials are available and there does not exist any subsoil problem.

The roads now under construction are built with a substructure of hot sand mix, on which asphalt concrete is

applied as a wear-resisting layer for attaining sufficient firmness.

Most of these roads in the desert were built to suit the needs of industrialisation. Further investigations were carried out corresponding to the necessities of mining.

4. FUTURE PLANS FOR ROAD CONSTRUCTION IN EGYPT

The plans for the future road construction in Egypt distributed over the next five years contain the following:

a. Surfacing the road sections of the road system of the first order which were till now neglected (see map), which connect the provincial towns with each other, and the second step, the surfacing of the roads of second order which connect the district towns.

Their length amounts to some 1,200 km and the costs of construction are estimated at 7.5 million Egyptian pounds.

b. The metalling of 1,000 km roads of the third order which connect villages with the system of surfaced roads. The cost for this is about 2.5 million Egyptian pounds.

c. Improvement of old roads and maintenance of roads. We have approximately 1,800 km roads which must be strengthened by new wear-resisting coating; the cost of this and for the maintenance of the roads amount approximately to 6 million Egyptian pounds.

d. Tourist roads of 100 km length for which the cost proposed were approximately a half million Egyptian pound.

e. Four bridges over the Nile and bridge-crossings which cost about 6 million Egyptian pounds.

f. The building of the Port Said Mataria road through the Manzal sea for a length of 30 km leads to expenses of an estimated 3.5 million Egyptian pounds. The total cost of the road construction within the Five-Year Plan is estimated to be about 26 million Egyptian pounds.

This means that more than five million Egyptian pounds must be provided yearly for road-building in the departmental budget. In addition there is also the money which is required for the construction of leading roads to the mines which need about 10 million Egyptian pounds in the next few years.

To an increased extent in the next five years, will also be built roads from the villages to the consumer markets whose total length amounts to 1,000 km and for which a cost of about 2 million Egyptian pounds will be incurred and which must be provided by the local authorities.

The conclusion from all this is that we are facing in the next few years in Egypt a powerful swing in favour of road construction since President Gamal Abd El Nasser and his Traffic Minister Dr. Mostafa Khalil are for it. More roads will be built which denotes a better life for all.



P. D. HILLS, M.L.A.,
Minister for Local Government and
Minister for Highways

HIGHWAY DEVELOPMENT IN NEW SOUTH WALES

ROADS give access to land, minerals, forest, work places and homes, and serve to transport men and goods from the places where they are to the places where they are required to be. Without road access, the lands and other natural assets of a country are largely inaccessible and unavailable for use. It is means of transport, especially roads which give value to potential resources. Industry and commerce are equally dependent on the existence of roads, and the better the roads, the cheaper is the resulting cost of transport.

As well as serving an economic function, roads are also the first essential of social life, to recreation, and all cultural and leisure activities.

Governments are dependent on roads to aid in making possible the many public services needed by the modern community, such as education, police protection, ambulance, medical and hospital services, water supply, electricity, gas, sewerage, drainage, telephone, and fire-fighting. Defence also depends considerably on the availability of good roads.

In March, 1960, there were 2,802,914 motor vehicles, including motor cycles, in Australia corresponding to 3.7 per-

sons per vehicle, 959,296 of these vehicles were in New South Wales.

The roads of New South Wales have a total mileage of 127,095 miles. As at 30th June, 1957, the roads were constructed as follows:—

Dustless surfaces, mainly bitumen	... 11.24 per cent
Gravel	... 34.60 per cent
Earth surfaces	... 54.16 per cent

The corresponding percentages for the United States are as follows:—31.43 per cent, 37.24 per cent, and 31.33 per cent.

In both countries about 20 per cent of the recorded road mileage carries negligible traffic, and may be disregarded.

A map showing all the roads of New South Wales would show a confusing pattern of lines which it would be difficult to comprehend at first glance. However, roads do not all serve the same function, and if the network is studied it will soon become evident that the roads may be "ranked" or put in "classes", in order of significance from a State point of view. I will first describe the position in the country areas of the State outside the Country

of Cumberland which comprises metropolitan Sydney and its immediate rural environs.

Classes of Roads

Roads in the first class are those which link this State with other States, or connect the vast interior with the coast. These are the most important of our Main Roads, and are called State Highways. The State, through the Commissioner for Main Roads, meets the full cost of constructing and maintaining the State Highways. Most of them are named, e.g., Hume Highway, Oxley Highway, and so on. The State Highways have a total mileage of only 6,504 miles, and thus represent only 5.11 per cent of the total recorded road mileage of New South Wales, but they carry about 45 per cent of the total vehicle-miles of travel in the State.

The next roads in order of importance are those Main Roads which, together with the State Highways, link the main regions of the State. These are known as "Trunk Roads", and by reason of their importance, the State, through the Commissioner for Main Roads, meets three-quarters of the cost of their construction and maintenance, and the Council in whose area they are situated meets the balance of the cost.

Then, and only a little less in importance, are the remaining (or "ordinary") Main Roads, which, together with State Highways and Trunk Roads, form a network linking all districts and all centres of population throughout the State. The State, through the Commissioner for Main Roads, meets two-thirds of the cost of the construction and maintenance of "ordinary" Main Roads, and the Councils through whose areas they run meet the balance. All Main Roads together, i.e., State Highways, Trunk Roads and ordinary Main Roads, total 22,423 miles in length, i.e., approximately 17.63 per cent of the total recorded road mileage in New South Wales. The Main Roads, how-

ever, carry at least 75 per cent of the total vehicle-miles of travel in the State.

The remainder of the roads of the State, comprising 82 per cent of the total recorded mileage, but carrying not more than 25 per cent of the total traffic, are the roads of local interest, i.e., side roads giving access to properties, and their upkeep is a local responsibility of the Municipal or Shire Council, although Councils receive some financial aid as described later.

In the County of Cumberland, which includes metropolitan Sydney, the pattern of roads and road classification is very similar to that elsewhere in the State. However, in this case, the Commissioner for Main Roads is fully responsible for all works on Main Roads, whether State Highways or otherwise. The Councils are responsible for all other roads in the County of Cumberland, but receive financial assistance from the Commissioner for Main Roads on certain "Secondary" roads which are accepted as serving to relieve traffic on Main Roads.

Who carries out the work

The task of the Commissioner for Main Roads is to carry out the provisions of the Main Roads Act, and in particular to see that the Main Roads of New South Wales are maintained and constructed to meet the needs of road users to the extent that available funds will permit. The actual work of maintenance and construction on the Trunk and "ordinary" Main Roads is carried out by the Municipal and Shire Councils, who also do all the work on local side or access roads. In the case of the Country State Highways, however, and the Main Roads of the County of Cumberland, which are the direct responsibility of the Commissioner for Main Roads, the greater part of the construction and maintenance work is done by the Department of Main Roads,

the balance being done by Councils by arrangement with the Commissioner.

Where the Funds come from

The total annual expenditure on roads in New South Wales is about 45,000,000 pounds. The funds expended by the Department of Main Roads total about 20,000,000 pounds per annum, and are drawn mainly from State taxation on owners of motor vehicles, and from grants from the Federal Government known as "Commonwealth Aid". (The Federal Government has exclusive rights in Australia to impose a tax on motor fuel.)

The amount which Councils themselves contribute to Main Roads works is drawn mainly from rating on land and to a lesser extent from loans. Work by Councils on local side roads is financed partly from these sources, and partly from a special proportion of "Commonwealth Aid" which the Federal Government requires shall be spent on rural roads which are not Main Roads. In addition, funds may be provided by the Commissioner for Main Roads towards the construction of "Developmental Roads" i.e., roads which have not been previously constructed and which if now built will bring about rural development.

Maintenance and Construction

Nearly half the money spent each year on the Main Roads of New South Wales is spent on maintenance, i.e., repairing the ravages of traffic and weather, so that the roads and bridges will at all times give as satisfactory a standard of service as they are capable of without rebuilding.

The balance of the money is spent on construction, which means providing facilities superior to those which exist, for example constructing new bridges in place of old structures, or at sites where no bridge has ever existed; rebuilding country Main Roads to modern standards—sometimes involv-

ing extensive deviations from existing routes; rebuilding old road pavements that have worn out; constructing bituminous surfaces in place of gravel surfaces in the country; widening metropolitan roads and constructing higher-class pavements; and so on.

Road improvement trying to catch up with Traffic Growth

The Department of Main Roads was established in 1925, and since then tremendous progress has been made by Councils and the Department in improving the Main Roads of New South Wales. For example, we now have about 8,000 miles of bitumen-surfaced Main Roads, whereas in 1925 there were scarcely any dustless surfaces outside towns, and we can now travel at high speeds over long distances throughout the State. Since 1925 1,700 new bridges have also been built on Main Roads. This progress has been obtained in face of serious difficulties, especially, in turn, the economic depression of the 'thirties', then World War II and finally the inflation of post-war years. Simultaneously with the growth in population and in motor vehicle ownership. A most notable feature of the post-war traffic has been the big relative increase in the use of heavy commercial motor vehicles. Although post-war road revenues soared with the growth of motor vehicle usage, the value of the money has increased 200 per cent in the post-war years, the effective work value of the total available motor revenue is little more than 50 per cent. In excess of that available in pre-war years. Thus traffic needs have advanced far more rapidly than can be met from revenues available, and many road improvements urgently needed in both city and country cannot yet be undertaken. Nevertheless, big programmes of work are coming to fruition and others are being instituted.

Current Works

On the average, each year, at present, the Department of Main Roads and

Councils on Main Roads are converting about 500 miles of gravel road to bitumen-surfaced road, and are rebuilding and strengthening about 100 miles each year of heavily trafficked State Highways built many years ago in order to adapt them to modern traffic. In addition, about 60 to 70 new bridges are built each year on Main Roads.

In the maintenance and construction of Main Roads the Councils and the Department employ some 7,400 men and large numbers of all types of modern road plant, especially motor graders, dozers, scrapers, loaders and trucks.

Some very large road works involving heavy earth works which are in hand on Main Roads in the country include the construction of a new route for the Gwydir Highway from Grafton to Glen Innes through rough mountainous areas, a length of 74 miles of new road; the reconstruction of the Trunk Road between Canberra and the coast, also through much mountainous country, a length of 77 miles; the deviation of the Pacific Highway between a point near Raymond Terrace and Taree (79 miles of new road); and road construction on the Prince's and Snowy Mountains Highways on the south coast. Progress in bitumen surfacing on State Highways is making especially rapid progress at present in plains. Large bridges under construction include a steel bridge over the Hastings River near port Macquarie; a steel and concrete bridge over the Lane Cove River at Figtree; a steel bridge over the Richmond River at Lismore; a pre-stressed concrete bridge over the Murray River at Albury; a pre-stressed concrete bridge over the Parramatta River at Silverwater and a long span concrete arch bridge over Parramatta River at Gladesville.

Future Works

Plans in preparation for large new bridges include a steel bridge over the south arm of the Hunter River at

Newcastle; a bridge over the Richmond River near Ballina; a bridge over the Clarence River near Maclean; a bridge over the Hunter River at West Maitland; and a bridge over George's River between Rocky Point and Taren Point. At least a hundred new bridges of all sizes require to be built on Main Roads each year.

In the country, immediate objectives additional to bridges, are the extensions as rapidly as possible of bitumen surfaces on all Main Roads where this is warranted, and the improvement of the Main Roads serving Newcastle and Wollongong. Considerable improvement will also be undertaken on the principal inter State routes.

In Sydney, Newcastle and Wollongong the Department of Main Roads has planned the future development of the Main Roads systems including the provision of modern expressways serving the metropolitan area and beyond. This planning has been done in close association with other aspects of the town planning of these cities.

In the metropolitan area, the main immediate objectives of the Department of Main Roads, additional to provision of bridges, are first to widen the principal arterial roads serving industrial areas and the new residential areas. In addition the Department has put in hand surveys and designs for work on part of the proposed Warringah Expressway, which is planned to lead north from the Harbour Bridge and serve all north shore suburbs, and also for that part of the Western Distributor which will link the city markets area with Ultimo and beyond. On account of inability to obtain sufficient additional experienced staff locally, the surveys and plans for these latter works are being done by a firm of experienced American engineering consultants.

The most heavily-trafficked rural road in New South Wales is the Sydney to

Newcastle portion of the Pacific Highway. This road is mainly only of two-lane width, and much of it passes either through steep rocky terrain, or through close-settled areas. The provision of a four-lane road of expressway type is highly desirable, but funds cannot yet be provided for it. Proposals have therefore been invited for the construction and operation of such a road by private enterprise, on a toll base.

Other expressways have been planned by the Department of Main Roads extending from Sydney, south as far as Kiama, south-west as far as Mittagong, and west the Blue Mountains. These, together with metropolitan expressways, will be among the most important works to be undertaken in the future, simultaneously with the further improvement of the Main Roads system as exists today, together with such additions to the system as time and traffic may show to be necessary.

Highway Engineering A Challenge

Growth in weight and volume of traffic on city streets and on country roads is one of the outstanding features of the age in which we live, and it is the task of the Highway Engineer to develop ways and means of economically meeting the needs of this traffic for safe, speedy and convenient movement. New problems are constantly appearing, and research plays an increasingly important part in the search for solutions. The Department of Main Roads in New South Wales has large laboratories for testing materials and for day-to-day research, and co-operates in this work with similar bodies in other States through an inter-state organisation known as the National Association of Australian State Road Authorities. Recently the Association has established the Australian Road Research Board further to widen the field of road research in Australia and to co-ordinate all road

research activities. The Board will hold periodical conferences, and publish the results of road research work, as well as granting funds to universities and other bodies for undertaking specific pieces of road research.

The field of road research covers not only road materials, road making methods, and the characteristics of road traffic, but also research into finance and administration, and into future road needs. An extensive study into future road needs in New South Wales has recently been carried out by the Department of Main Roads.

New South Wales is fortunate in having at the University of New South Wales Chairs of both Highway Engineering and Traffic Engineering. In addition, the University authorities, in conjunction with the Department of Main Roads, are planning for the establishment at the University of important road research facilities. Thus young engineers in this State are especially favoured to equip themselves as highway engineers.

Highway Engineering comprises the application of science and engineering practice to the new and particularly pressing problems resulting from modern traffic. Highway Engineering is therefore a challenge to young engineers, involving early responsibility in management, self-reliance, often relative isolation, control of large bodies of workmen and equipment, or it may be special technical assignments in design and research. Australia has need for many highway engineers because its road systems are still only in the initial stage of development for the motor age.

I commend highway engineering to you as a field of great and increasing technical and administrative interest, and a field where satisfaction can be derived from visible accomplishment.

(From "Main Roads", March, 1961.)

REPORT ON ROAD AND TRAFFIC PLANNING IN SWEDEN

By

ERIK HASSELQUIST

IN November, 1957 a specially appointed Committee, which included road and traffic experts and specialists on economic and industrial geography and social planning, and representatives of industry and the motoring organizations, put forward proposals for a road network and investment plan for Sweden based upon investigations on the development and localization of the population, on industrial and economic trends in different parts of the country and on traffic developments in general. The Swedish Government has adopted in principle this 20-Year Road Plan and, since 1958, investment in roads has kept close to the plan: regional plans (common summary plans for a number of communities), master plans (Plans for one community) and town planning are all based upon it.

The programme for the development of the road network was not intended to be definitive: rather was it the intention to produce a skeleton plan which could be translated into an active programme by the working out of yearly plans, but could be successively revised in accordance with technical and economic developments. A continuous revision of the plan would thus be carried out in the light of new results and experiences in the relevant fields. In this way, it would also be possible to make use of, *inter alia*, the results of current investigations abroad, and it would be easy to adapt the plan to developments in other social spheres.

In the year 1955 the population of Sweden was about 7.3 millions of whom

5.1 millions, that is 70 per cent, were living in built-up areas. An assessment based upon, among other things, developments to date, and upon certain assumptions concerning the continued flight from agriculture, the structural changes in the timber industry, the growth of the urban trades and industries, and the general building development in settled areas indicated that by 1975, 80 per cent to 85 per cent of the population, about 7.8 millions, would be living in built-up areas.

Investigations carried out with respect to industrial and economic development show, *inter alia*, that a continued mechanization and specialization in both stationary industry, and in building activities and agriculture, must be expected, entailing increased or new transport needs. With current transport tendencies in mind, it is most probable that these will be met by road transport. The present and continuing development with respect to surface transport leads to the prognosis—especially in view of the increased call for door-to-door service—that a continued switch to road goods vehicles for the shorter distances is likely; at the same time new transport needs will be met by this transport form. The closing down of smaller railway lines may also be expected to contribute to the growth of truck traffic.

The future increase of production in stationary industry has been estimated to amount to about 3 per cent per annum. In view of the fact that the domestic movement of goods seems to be increasing more than production, a

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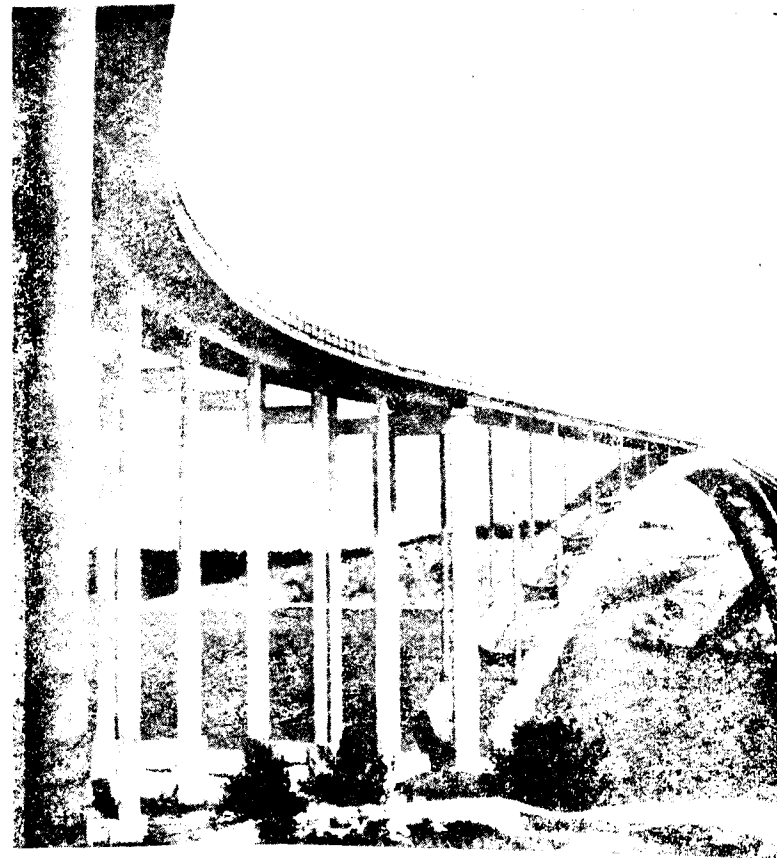


Fig. 1. Asheröfjörds Bridge, part of a new major road project connecting the islands Tjörn and Orust, with the mainland. The bridge has a 912-ft. tubular steel arch with a clearance of 135 ft. (Photo. by the Contractors, Skanska Cementgjuteriet)

future annual growth of transport on the railways, by road vehicles and by water, is estimated at 4 per cent to 5 per cent. Allowing for the trend to road transport, a future annual increase in road goods transport of 7 per cent to 10 per cent has been assumed, and a four to six times increase by 1975 over 1955. Results of transport support this assumption. If the above mentioned level for domestic goods traffic is reached, by 1975 one-half to two-thirds of the domestic goods transport in Sweden will be by road as against scarcely one-third today. By far the greater part of the assumed increase in passenger traffic in Sweden of a minimum of 3 per cent per annum is estimated to be in personal transport, primarily the motor car.

Growth in Personal Transport

An important feature of a modern industrial community is the marked increase in the daily journeys to and from work, and the purchase of motor vehicles for this purpose is facilitated in some industries by loans on favourable conditions.

As far as can be judged from developments in Sweden, no marked relative increase in personal transport over long distances need be anticipated. Expansion of domestic flying and express train traffic underlines this. Thus, as regards road passenger and road goods transport, one is probably justified in estimating that the main increase is likely to be in relatively short distance transport.

Among European countries, Sweden has gone farthest in regard to motorization. Whereas in 1950 there was one motor vehicle (motor cars, lorries and buses) every 20 inhabitants, today there is one to every six inhabitants. (If motor bicycles and motorized bicycles are included, however, there is one motorized vehicle to every 3.5 inhabitants.) According to an actuarial forecast, by 1965 there will be one motor vehicle for every 4.5 inhabitants and by 1975 one to every three inhabitants (Fig. 2).

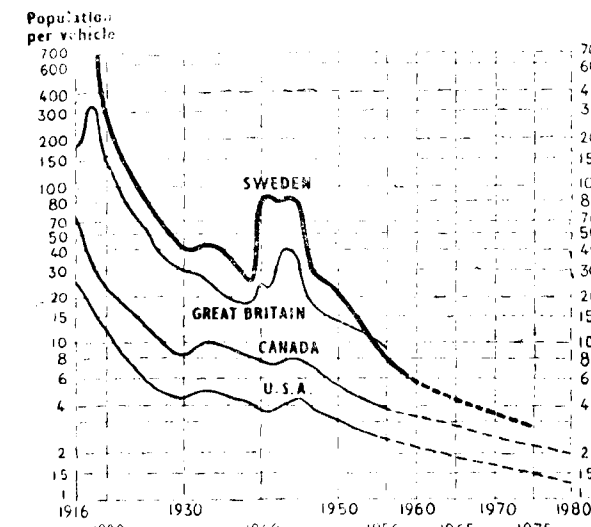


Fig. 2. Predicted growth in motorization

If one assumes that in future the motor car is used on an average to the same extent as now, it will be responsible by 1965 for 75 per cent, and in 1975, 80 per cent to 85 per cent, of the total number of vehicle/kilometres of public and private transport, compared with 55 per cent for 1957.

On the whole, from available data concerning the number of vehicles and traffic flow, it would appear that traffic growth follows, roughly, development on the vehicle front. This emerges from a comparison of the factors affecting the traffic growth with the corresponding factors as regards vehicles. Investigations carried out with reference to traffic

growth on different types of roads show that traffic—both the annual average daily traffic and the summer average daily traffic, June-August—tends to increase at a quicker rate of busier roads than on others. Thus, a comparison of the results of the 1936 and 1953 nationwide traffic counts shows that on the majority of the national main roads, and the more important county roads, traffic during the period 1936 to 1953 increased by from four to five times as much, and traffic on the more local road network of the country districts by two to two and a half times that for 1936. On an average, traffic on all roads tripled during these years. The corresponding tendency is seen as regards the volume of summer traffic in relation to the annual average for daily traffic. The marked increase in recorded traffic during the summer months is also more pronounced on the national main roads than on the cross-country roads. The same applies to the latter in relation to other county roads.

Determination of Road Standards

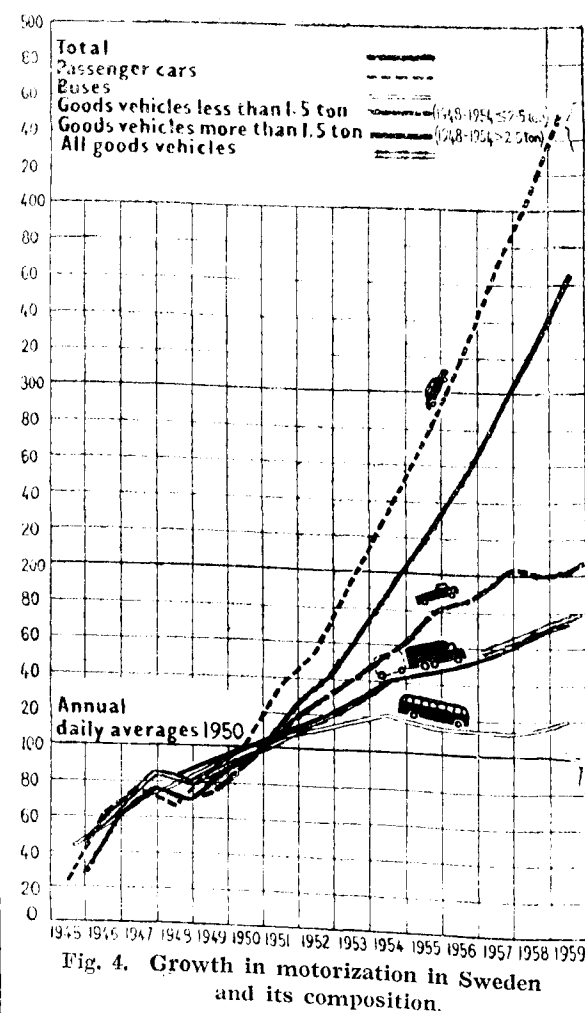
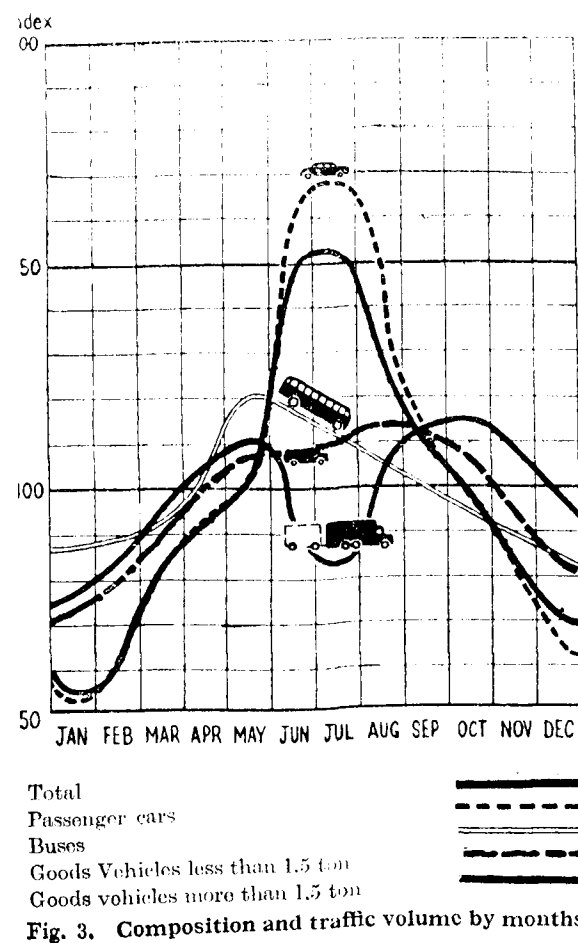
The norms, proposed as a basis for future road standards, are determined with reference to the summer average daily traffic estimated for 1957. In certain cases where estimated traffic volume is so great that dual carriageways may be needed, variation in hourly traffic throughout the year is also taken into account. This is to preclude the possibility that the number of hours during the year when troublesome queue formation or traffic blocks occur may be more than can reasonably be tolerated when set against the great expense of the subsequent conversion of a two-lane road to a dual carriageway.

It may be mentioned that the total road transport for 1955 amounted to approximately 12,000 million vehicle kilometres. The multiplier in relation to the traffic for the year 1953, used in this connection for the year 1965 normally between 2.5 and 4.0, and for the year 1975 between 4.5 and 6.0. In

certain cases, higher multipliers than those given above have been used for the calculation.

Extensive traffic counts and origin and destination surveys play a most important part in the preparatory work on which Sweden's Road Plan is based. Road traffic intensity is calculated from systematic traffic counts and the last manual count covering the entire country took place in 1936. Since then manual counts have been made at 130 points seven times a year from Friday to Sunday in order to obtain information on the variations in traffic and its composition (Fig. 3) and on the increase from year to year (Fig. 4).

Since 1951, these counts have been supplemented with mechanical counts at



control stations and cover stations. The former provide material for the annual variation curves for main roads and provincial roads within different districts, and results are published as printed traffic flow maps giving the average daily traffic volume for the year and for the summer months (June-August) for almost every public road (about 60,000 miles). These maps are based on counts of about 13,000 and 15,000 traffic streams and are available for the years 1953 and 1958 (Fig. 5).

The O-D surveys were made at special points spread over the whole country, and some 250,000 roadside interviews were made to obtain information con-



Fig. 5. Traffic flow map for the outer area of Stockholm showing annual average daily traffic (black band) and average daily traffic during summer time. Maps of this type are made up for a number of more densely populated areas

cerning the origin, destination and purpose of the trips. The surveys were mostly restricted to one week-day during the spring or autumn and one week-day during the summer. In some cases, week-end traffic was studied also. The purpose of the surveys was primarily to reveal the origin-destination pattern of the traffic, and the relation between traffic to and through the principal urban areas along the present road network. The structure of long distance traffic, and the distribution of travel distances was studied at local observation points, as well as along the whole of a certain stretch of the road. The distribution of transport mileage between various travel distances and types of road was analysed.

Furthermore, analyses were made of traffic field structure around urban areas and along various types of roads.

To carry through, within reasonable time and costs, field work analyses of the data obtained, a sampling method was used. The method decided upon was to study during a certain period of the day—in most cases from 6 a.m. to 10 p.m.—the traffic flow at special points along the road, which were generally selected immediately outside urban areas. This made it possible to register both long distance through traffic and the more intensive local traffic to and from the area. The sampling method enabled each interviewed group to cover within one period of 16 hours two or

three different stations outside the built-up area. All drivers passing the station in one direction within a quarter of an hour were interviewed. After a pause of five minutes, the interviews were resumed on the other side of the road for another 15 minutes, to cover vehicles travelling in the opposite direction. The interviewer were then given 20 minutes to move to another location where the same procedure was repeated. During the day, the personnel thus moved from place to place according to a fixed schedule. Each interviews usually took 20 to 30 seconds. The answers were recorded on a tape recorder and registered on punch cards. To facilitate the computation of travel distances with modern electronic computers and the co-ordination of the data obtained at different stations, the places of origin and destination of each trip, as well as certain points on the road, were given in co-ordinates relative to the Greenwich Meridian and the Equator.

For road planning it is essential to know which roads are the most important to commercial traffic. For that reason, special kinds of surveys have been made to obtain figures of the annual number of ton/kilometres on roads of special importance for goods transport. For instance, traffic flow maps have been prepared, showing the importance of different roads used for transport by the timber industry.

In addition, the National Swedish Road Board has tried a method of weighing the traffic, which is now considered to be sufficiently tested to enable a programme to be instituted covering 210 weighing stations. The country is divided into five districts, and each district is served by a patrol performing weighings at three adjacent stations at the same time. After weighing for a week, the patrol moves to another group of three stations. The machines work automatically and are only inspected twice a day. The weighing

plate, which is boxed in the carriageway, is dimensional and placed in such a way as to permit weighings of only one wheel on each axle. Recording is made on a paper strip as the vehicles pass over the plate. When weighings are finished, the weighing plate is removed and replaced by a wood plate and the instruments placed in a transportable instrument box.

On roads with heavy traffic flow, one weighing machine is placed in each lane of the carriageway. On smaller roads, there is only one weighing plate is one of the lanes; the other lane is shut off and the direction of every vehicle passing over the weighing plate is automatically recorded. A speed limit of 20 km/hr (12.5 miles/hr) is necessary for accurate recordings. The weighing machine also serves as a traffic counter, and it is possible to separate the registered vehicles into various classes according to the number, size and gaps of the axle load registrations. The weighing equipment used for these surveys has been developed at the Swedish Road Research Institute.

A closer study has shown that the present road network is ill-equipped to meet future traffic development. Bad traffic blocks and queue formation may be expected to be general, not only in the larger towns and at their approaches, but also on the many low standard roads which do not afford sufficient space for future development. Roads are becoming more congested, and the transport both of persons and goods is subject to increasing delay. At the same time, the accident curve is tending, despite many measures taken to regulate traffic, to rise in proportion to the growth in traffic.

For the road inventory two cars are used and their equipment includes radio transmitters and receivers, trip odometers, inclinometers, superometers, gyrocompasses and wire recorders. All data relevant to road standards are recorded: road section, type of surface,

curvature, gradients, super-elevation, intersection, bridges and underpasses, roadside obstacles, off-street parking spaces and visibility, both on the road itself and at intersections. Data has been obtained for all national main roads—about 9,000 miles.

The priority classification is based on such data. Within scale of 100 points, technical standards (condition of road surface, bearing capacity), driving conditions (width of carriageway, alignment and passing possibilities) and road safety (width of carriageway and shoulders, stopping distance, unity of alignment and accidents hazard), have been given 25, 30 and 45 points. The sum of points received for each part of the road is then corrected in relation to the traffic volume. For these purposes a special formula is used. The standard of 100 points is the standard that fulfils the design standards now in use. For each road a diagram is drawn up which shows the sum of points for each section along the road.

Classification and Standards

The road plan includes the following classes of road, all of which are to have a carrying capacity of 10 tons axle-load and 18 tons bogie-load:—

- (a) Divided highways, median strip and wide bearing shoulders. Width of pavement 7.0 m. (23½ ft), width of median strip normally at least 7.0 m. and width of shoulders 3.0 m. (10 ft). Paved, 0.5 m. (1½ ft) wide border strips directly adjacent to the pavements, constituting part of the median strip and the shoulder. This type of road is constructed either as a freeway with grade separated intersections or as a four-lane road with at-grade intersections where the terrain and the traffic conditions do not call for grade separated intersections. Design speed for freeways is 120 km/hr. (75 miles/hr) and for other roads 100 (62 miles/hr.) to 120 km/hr.

- (b) (i) Road with 7.0 m wide pavement and 3.0 m wide bearing shoulders. In the case of larger volumes of traffic, 0.5 m wide paved border strips are constructed on the shoulders directly adjacent to the pavement. Special shoulder parking facilities are to be provided where this is considered necessary. Design speed is normally 100 km/hr.
- (ii) Road with 7.0 m wide pavement and 1.0 m (3½ ft) wide bearing shoulders. Regular shoulder parking facilities provided along the road. Design speed normally minimum 80 km/hr. (50 miles/hr).
- (iii) Road with 6.0 m. (20 ft) wide pavement and 1.0 m. wide bearing shoulders. Shoulder parking facilities are provided where considered suitable in the light of the traffic conditions obtaining. Design speed is normally minimum 80 km/hr.

For two-lane roads in hilly country special lane should be provided on steep upgrades for the slower truck traffic. Divided roads with two three-lane pavements are deemed necessary in certain cases. It has not been thought necessary to deal specially with such roads, as it may be assumed that they will be found only locally and will then be given special consideration. This classification for the national main roads may also be applied to county roads. As a rule, however, the requirements for design speed should be made less stringent because in the main they will serve local traffic only.

Since in addition, a large number of county roads will, according to calculations, be carrying a relatively inconsiderable amount of traffic, the Committee thought it possible to reckon also with smaller road widths than the standard proposed for national main roads, with 6.0 m wide pavement

and narrow shoulders. Types of road that have been considered are those with 6.0 m pavement without shoulders, and one-lane roads in this connection with 3.5 m (11½ ft) pavement and passing facilities. The former type is presumed to be the normal type for county roads, and it is proposed that the latter type should only be considered for roads carrying very little traffic, e.g., roads in the less developed areas.

The volumes of traffic for the time being proposed as a basis for the design of the network of national main roads are:

Type of road	Future summer average daily traffic
A	9,000
B : 1	3,000—9,000
B : 2	1,500—3,000
B : 3	1,500

For county roads a 6 m pavement and narrow shoulders are proposed to be used for design traffic volume between 1,000 and 2,000 and 6 m pavement without shoulders for volumes between 150 and 1,000. For lesser volumes than 150 vehicles, a 3.5 m pavement with passing facilities is considered sufficient.

The road plan, in contradistinction to earlier official road building programmes in Sweden, is based upon a detailed inventory of actual need, and is regarded as a solid basis for an investment plan.

For the roads in the county districts, the development has been estimated to cost a total of 1,120 million pounds and it is proposed to make at least 55 per cent of the investment in national main roads and extremely important county roads during the 10-year period, 1958-

1967 is considered reasonable. Such an allocation has been deemed fully justified because of the urgent need to improve the more heavily used and industrially important roads. Thus, during years 1958 to 1967 nearly 40 per cent of the total expenditure in the county districts will be invested. About half, or 220 million pounds, will be for national main roads.

Financing the Programme

As to State assistance, the towns obtain 95 per cent of the cost of road investment and maintenance (roads and streets of importance for through traffic), and it has been estimated that, including both national main roads and local roads, this will amount to 290 million pounds. In allotting investments to the two periods, it has been assumed that 65 per cent of the investment requirement for the national main roads in the towns would be met during the years 1958-1967. The traffic difficulties that may be expected on the approaches to the towns thus seem to justify a greater immediate contribution in the towns than in the country districts. For local town roads it has been assumed that about 30 per cent of the investment would be made during the first period and approximately 45 per cent of the investment need would be met during the periods 1958—1967. According to the investment plan, investment in roads in the county districts will increase from 7.3 million pounds to 20.6 million pounds. According to the calculations, these costs, as well as those for maintenance, will be covered by the funds that, on the basis of current taxation, will accrue from motor vehicle traffic.

ROAD STANDARDS

ROUND THE WORLD

The Indian Roads Congress issued enquiry relating to Geometric Standards in the form of a questionnaire to various countries.

The following countries have very kindly sent the replies:—

U.S.A. (Bureau of Public Roads, Washington)

Burma

Canada (Department of Highways, Ontario)

West Germany

Great Britain

Iran

Israel

New Zealand (National Roads Board)

The ensuing tables have been prepared from the information supplied by the above countries and from the following sources:—

(I) Technical report on the Phillipines Highways, September, 1958.

(II) Report on the first Session of Expert Working Groups on International Highways for zones 1, 2 and 3 in ECAFE region:—Proposed Geometric Standards for roads outside built up areas were prepared for study by Ad hoc Zonal Working Committee and will be presented in final form to the Governments concerned for approval and eventual adoption.

(III) BULLETIN DE L'ASSOCIATION INTERNATIONALE PERMANENTE DES CONGRES DE LA ROUTE:—Geometric Standards for International Roads in Europe.

(IV) Proceedings of the Indian Roads Congress:—Geometric Standards for roads discussed in the technical papers in the Annual Sessions from time to time.

The Indian Roads Congress is very grateful to the above mentioned countries for the information supplied.

UNITED STATES OF AMERICA (Bureau of Public Roads, Washington)							
Classes of Highways	Inter-State (Freeway)			Typical State Roadways			
	Rural	Suburban	Urban	Primary (Rural)		Secondary (R)	
				Multi-lane	2-lane		
1. Design capacity in vehicles per hour per lane	1,000 per lane	1,200 per lane	1,500 per lane	1,000 per lane	900 Two ways	900 Two ways	
2. Number of traffic lanes for each class of highway	Multi-lane	Multi-lane	Multi-lane	Multi-lane	2	2	
3. Design speed per hour							
(a) Flat topography	M.P.H. 70	M.P.H. 60	M.P.H. 50	M.P.H. 70	M.P.H. 60	M.P.H. 50	
(b) Rolling "	60	60	50	60	50	40	
(c) Mountainous "	Desirable 50	50	50	50	40	30	
4. Pavement width per traffic lane							
	Feet	Feet	Feet	Feet	Feet	Feet	
Desirable	12	12	12	12	12	12	
Minimum	12	12	12	11	11	10	
5. Min. width of shoulders							
(a) Flat topography	Feet Left Right 6 10	Feet L R 6 10	Feet L R 6 10	Feet R 8	Feet R 8	Feet R 6	
(b) Rolling "	6 10	6 10	6 10	6	6	4	
(c) Mountainous "	6	6	6	4	4	4	
6. Width of roadway							
(a) Flat topography	Varies	Varies	Varies	Varies	Feet 38	Feet 32	
(b) Rolling "					34	28	
(c) Mountainous "					30	28	

BURMA					
	@ 15 V	50 V	300 V	1,500 V	
* F.M.V	20—40	20—40	20—50	30—60	ventional standards
—	15	50	300	1,500	v.p.h.
* Farm to market road for vehicular traffic					ways both directions ... 700
a Road for a probable maximum hourly traffic volume of 15 self-propelled vehicles travelling at speeds ranging from 20 to 40 m.p.h.					ays average per lane— ... 800 ... 1,200
1	1	1	2	Dual 2 lanes	
M.P.H.	M.P.H.	M.P.H.	M.P.H.	M.P.H. Desir- able Mini- mum	limit for Phillipine Highways
40	40	40	40	60 50	enger vehicles kmph 70
30	30	30	30	50 40	ks and buses 50
20	20	20	20	40 30	
Feet	Feet	Feet	Feet	Feet	aily Width of pavement in metres
12	12	12	22 (2 lanes)	Dual 22 ft lanes	Minimum Desirable 4.0 5.5 6.7 7.3
Feet	Feet	Feet	Feet	Feet	aily Width of shoulders (M)
					Minimum Desirable
}	2	5	5	9	1.5 1.5
					1.5 1.5
					2.5 2.5
					3.0 3.0
Feet	Feet	Feet	Feet	Feet	
24	32	32—40	64—80	128—160	
20—22	24—30	24—30	48—60	96—120	
20—24	24—32	24—32	48—64	96—128	

UNITED STATES OF AMERICA (Bureau of Public Roads—Washington)							
Classes of Highways	Inter-State (Free-ways)			Typical State Roadways			
	Rural	Suburban	Urban	Primary (Rural) Multi-lane	2-lane	Secondary (R)	
7. Side slopes of road embankment and cutting	Height of cut or fill Side slopes Flat topography Rolling Mountainous	Feet 0—4 4:1 4:1 4:1	Feet 4—10 4:1 3:1 2:1	Feet 10—15 3:1 2½:1 2:1	Feet 15—20 2:1 2:1 1½:1	Feet Over 20 1½:1 1½:1	
For Inter-State Projects, side slopes should be 4:1 or flatter where feasible and not steeper than 2:1 except in rock excavation or other special conditions.							
8. Minimum width of right-of-way		Feet 150	Feet 150	Feet 150	Varies 100	Feet 60	
9. Road camber or cross fall		Per cent	Per cent	Per cent	Per cent	Per cent	
(a) Water-bound macadam road		—	—	—	2.5	2.5	
(b) Black top		2	2	2	2	2	
(c) Concrete		1.5	1.5	1.5	1.5	1.5	
(d) Shoulders		3.1	3.1	3.1	4	8	
10. Gradient maximum		Per cent	Per cent	Per cent	Per cent	Per cent	
(a) Flat topography		3	4	5	5	6	
(b) Rolling „		4	4	5	6	7	
(c) Mountainous topography		5	5	5	6	8	
11. Max. length of road permissible with grades		Feet	Feet	Feet	Feet	Feet	
(a) 3 to 4 per cent		1,100	1,100	1,100	1,100	1,100	
(b) More than 4 per cent		800	800	800	800	800	

BURMA					
	F.M.V.	15 V 20—40	50 V 20—40	300 V 20—50	1500 V 30—60
Slopes for					
		Fills		Cuts	
	Under 4 ft high	2 : 1		2 : 1	
	Over 4 ft high	1½ : 1		1 : 1	
		Rock ¼ : 1			
	Feet	Feet	Feet	Feet	Feet
Minimum	50	150	150	250	250
Desirable	66	250	250	—	—
(a) 1 : 60 (b) 1 : 80 (c) Not specified (d) 1 : 15 or 1 : 20					
	Per cent	Per cent Desir- able	Per cent Max.	Per cent	Per cent
	4	5	8	3	5
	5	12	—	7	6
	6	10	15	10	7
Grade per cent	4 to 6	8 to 15	3 to 10	5—7	5—7
Length in ft	1,000	1,000	← Not specified →		

ventional standards			
f surfacing	Right-of-way width		
d crushed stone	20 metre		
tone with bituminous			
eatment	20	,,	
e treatment	20	,,	
lam	30	,,	
aded mix bituminous			
spphalt	60	,,	
te rock asphalt	60	,,	
oncrete	60	,,	
%			
ual average daily traffic volume			
Below 100		Over 2,000	
ini-	Desir-	Mini-	Desir-
um	able	imum	able
cent	Per cent	Per cent	Per cent
4	3	3	3
6	5	4	4
6	6	5	5

UNITED STATES OF AMERICA (Bureau of Public Roads—Washington)											
Classes of Highways	Inter-State (Freeways)				Typical State Roads						
	Rural	Suburban	Urban		Primary (Rural)				Secondary		
					Multi-lane	Two-lane					
12. Minimum distance between brake points of grades	No design control										
13. Sight distances (stopping and overtaking)	Speed in m.p.h.	Stopping distance			Overtaking sight distance						
	70	600 ft									
	60	475 "			2,000 ft						
	50	350 "			1,700 "						
	40	275 "			1,300 "						
	30	200 "			800 "						
	In the case of multi-lane highways passing sight distance is not controlling and is not specified.										
	Speed in M.P.H.	70		60		50		40		30	
	Grade per cent	3	6	3	6	3	6	3	6	3	6
	Increase in ft	50	110	30	80	20	50	10	30	10	20
Decrease on upgrade	Grade per cent	3	6	3	6	3	6	3	6	3	6
	Decrease in ft	40	70	30	50	20	30	10	20	20	10
14. Minimum radius of horizontal curves	Design speed in m.p.h.	70		60		50		40		30	
	Minimum radius in ft	1,490		1,490		700		430		230	
15. Minimum length of transition curve	Design speed in m.p.h.	Minimum curvature			Minimum length of transition						
	30	3' 30'			ft						
	40	2' 15'			100						
	50	1' 45'			125						
	60	1' 15'			150						
	70	1' 15'			175						
16. Extra width of carriageway on curves	No widening required for 12 ft travel lane and maximum of 8 degrees of curvature, for sharper curves—see below:										
					Widening on 2-lane 22 feet wide pavement						
	Degree of curve				Design speed in m.p.h.						
	9—10				30		40				
	11—13				1.0		1.5				
	14—17				1.5		2.0				
17. Super-elevation	Speed in m.p.h.	70		60		50		40		30	
	(a) When necessary	Radius in ft less than	22,900	17,200	11,450	7,600	3,800				
	(b) Rate of super-elevation	Max. Super elevation	0.10	0.08	0.06	0.10	0.10				

BURMA									
F.M.V.	15 V 20—40		50 V 20—40		300 V 20—50		1,500 V 30—60		
Not specified									
Speed in m.p.h.		Stopping distance			Overtaking sight distance ft				
60		550 ft			} Not specified				
50		400 ..							
40		250 ..							
30		150 ..							
In the case of 1,500 V 30 60 the stopping sight distance be as below :									
Speed in m.p.h.		60/50		50/40		40/30			
Stopping distance in ft		550		350		250			
Overtaking sight distance not specified.									
No variation due to grades.									
Degree of curvature									
Topography	Desirable	Maximum	D	M	D	M	D	M	
Flat	11	14	7	11	6	9	5	7	
Rolling	18	25	11	18	9	14	7	11	
Mount	36	56	18	36	14	25	11	20	
D=desirable M=maximum									
Curves less than 5,000 ft radius to be transitioned. If length of curve is small and it is not possible to introduce transition length, whole of the curve to be made transitional.									
If radius less than ft	1,500		1,000		500				
Extra width per traffic lane	1 ft		1½ ft		2 ft				
Added width should be attained by uniform widening along the length of transition									
$e = \frac{3 V^2}{80 R}$									
V = ½ designed speed in m.p.h. R = Radius of curve in ft.									
e = Rate of superelevation									
Rate of superelevation not to exceed 1 : 15 where preponderance of animal-drawn vehicles. If road used by motor vehicles only 1 : 10									

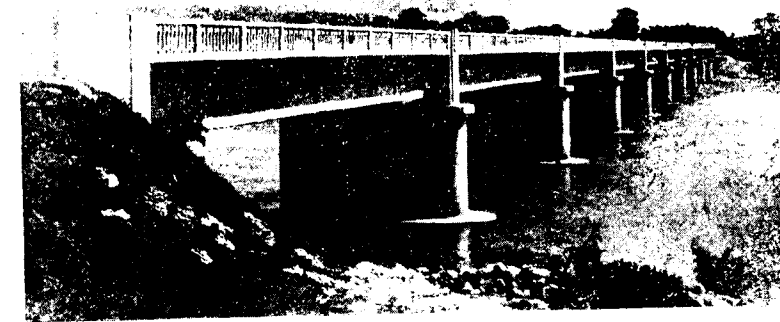
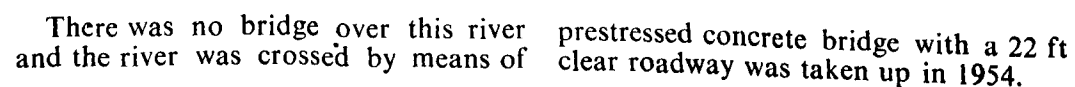
Conventional standards			
Non-passing sight distance			
Annual average daily traffic volume			
Below—100		Over 2,000	
M	D	M	D
60	80	90	150
50	60	80	110
40	50	60	80
Minimum radius in M.			
Annual average daily traffic volume			
Below 100		Over 200	
Min.	Desirable	Min.	Desirable
80	120	170	360
50	80	120	220
50	50	80	120
Transition curve.			
maximum			
er of roadway			
p.h.			
urve in M			

Classes of highway	UNITED STATES OF AMERICA (Bureau of Public Roads - Washington)					
	Inter-State Freeway			Typical States Roads		
	Urban	Suburban	Rural	Primary	Rural	Secondary
18. Minimum distance between two serpentine curves.	Design speed in m.p.h. 400			70 350	60 300	50 250
19. Minimum radius of vertical curves: (a) Summit or convex curves (b) Valley or concave or sag curves	Not applicable to parabolic curves					
20. Design standards for hairpin bends: (a) Maximum radius of basic curves (b) Minimum crossfall (c) Minimum length of transition curve (d) Extra width of carriageway. (e) Maximum grade (f) Maximum speed	Due to its high design standards for the Inter-State roads, hairpin curves are not contemplated.			There are no typical design standards for hairpin curves, as such are designed as a special case		
21. Minimum clearance: (a) Horizontal (b) Vertical	(a) Left $\leftarrow$ 4.5 ft $\rightarrow$ 4.5 ft if divided Right $\leftarrow$ 8 ft $\rightarrow$ 6 ft (b) $\leftarrow$ 16 ft $\rightarrow$ 14 ft			6 ft 6 ft 6 ft 14 ft 14 ft		
22. Bridges: (a) Loading (b) Minimum width between wheel guards	H20 — S16 — 44 Varies depending on approach width and structure length			H—45		
24. (a) Axle load (b) Max. permitted wheel load	(a) 18,000 lb (b) 9,000 lb					

BURMA					Conventional standards
F.M.V.	15 V 20—40	50 V 20—40	300 V 20—50	1,500 V 30—60	
A minimum of 100 ft					
Summit curves } Radius 4,000 ft Valley curve }					
(a) 30 ft (b) 1 : 6 ft (c) Depending upon site conditions (d) Depending on site conditions (e) 1 in 6 (f) M.P.H. 20					
Note:—Only F.M.V. and 15 V 20—40 can have hairpin bends					
12 ft 14 ft					
daily traffic volume below 100 to					
7 B.S.U. $\leftarrow$ 15 B.S.U. $\rightarrow$ 12 ft 22 22 22 Road pavement					
(a) Not specified (b) 7,000 (Max. 50 commercial vehicles per day) 9,000 (50—300 commercial vehicles per day) 12,000 (More than 300 commercial vehicles per day)					(a) 18,000 lb (b) 9,000 lb

BUROI BRIDGE

History : The bridge has been constructed over river Buroi at 68th mile of the North Trunk Road (East) between Tezpur and North Lakhimpur in Darrang District in Assam.



Buroi Bridge

Hydraulic Data: The maximum calculated discharge of 77,830 cusecs has been adopted for design purpose. The linear waterway provided is 1,000 ft. Since the river carries drift timbers, etc., vertical clearance of 9 ft between the H.F.L. and the underside of the girder has been provided. Other particulars are given below:—

- (i) Calculated velocity ... 4.73 ft/sec
- (ii) Calculated scour depth ... 17.70 ft
- (iii) Design depth of scour below H.F.L ... 35.40 ft

- | | | |
|-------|-------------------|------------------|
| (iv) | Catchment
area | ... 254. sq.mile |
| (v) | H.F.L. | ... R.L. 97.00 |
| (vi) | L.W.L. | ... R.L. 84.00 |
| (vii) | L.B.L. | ... R.L. 82.00 |

Design Load and Forces

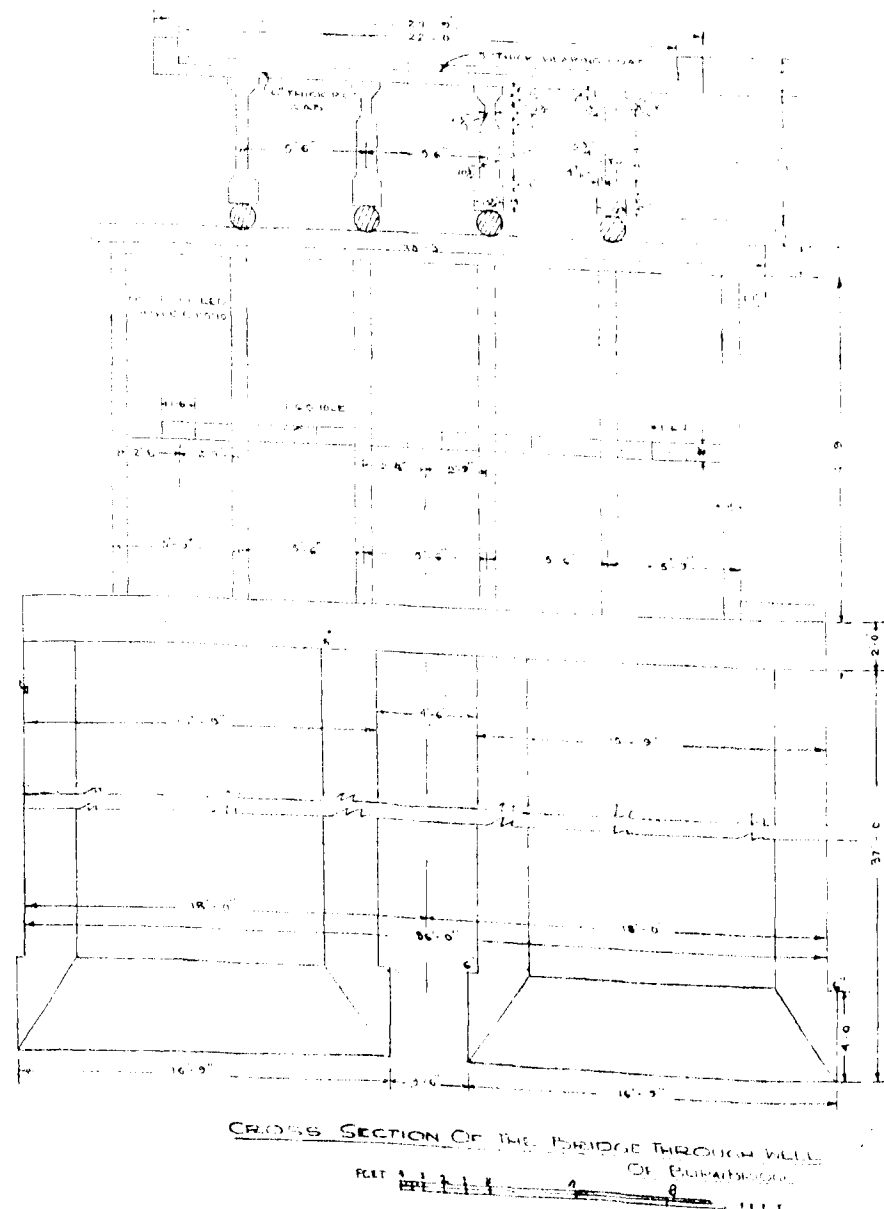
(ii) Horizontal loads due to wind, water current, longitudinal forces, buoyancy effects, deformation, secondary, and erection stresses have been considered.

(iii) Allowance of G/10 was made to withstand effects of horizontal seismic forces.

(iv) Temperature variation considered for $\pm 30^{\circ}\text{F}$.

Working Stress : Permissible stress for concrete for flexural and compression stresses was 750 psi and for reinforcement steel in tension was 18,000 psi with the modular ratio of 15 in R.C.C. works.

Each prestressing cable consists of 12 Nos. 0.2 in. dia. high tensile steel wires with a breaking stress of 100—110 tons

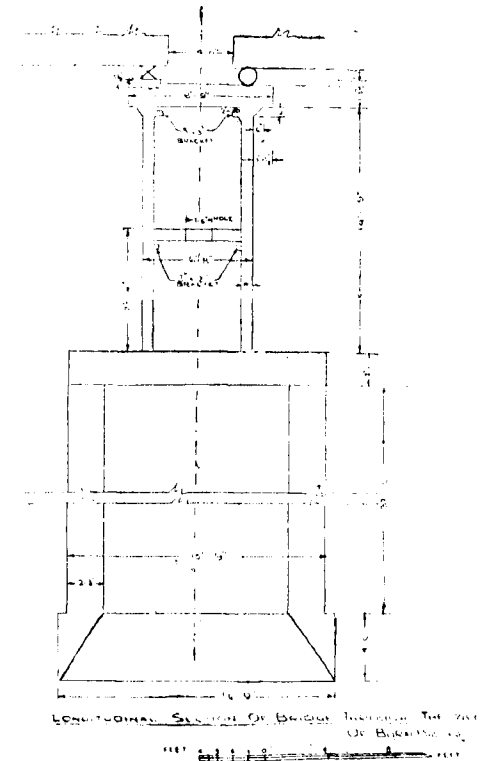


per sq. in. and a minimum 0.1% proof stress of 80 tons per sq. in. After allowance for total shrinkage creep of concrete plus relaxation of steel stress, the stress in the cables assumed to be 50 tons/sq. in.

General Arrangement: The bridge consists of eight intermediate spans of 106 ft between centre of piers and two end spans of 104 ft between centres of end piers and end of decking; the

total length of decking provided being 1,056 ft.

Substructures: Each pier and abutment was founded on 2 Nos. circular R.C.C. wells placed at about 20 ft centres. The outside diameter of the wells is 15 ft 9 in. except for the foundation of pier No. 6 (where two rockers are placed,) and abutments. Each well under foundation of pier No. 6 and abutments is 17 ft 3 in. dia. The



thickness of steining is 2 ft 3 in. The wells have been taken down to R.L. 47.00. Reinforced concrete well cap 17 ft 3 in x 36 ft x 2 ft thick was provided under each pier.

The R.C.C. piers cellular type with semi-circular ends are 14 ft 9 in. high, they are stiffened transversely by four cross walls each 6 in. thick running throughout the height of the pier and located directly under the prestressed concrete beams.

The R.C.C. box type end abutments with front and side wall in beam and

slab construction rest on strong R.C.C. well cap.

Bearings: Cast Steel Rocker and Roller bearings are provided alternately over the piers under each of the four beams.

Superstructure: Each span is a complete unit designed on the Freyssinet system of prestressed concrete and supported on piers and abutments and consists of 6 in. thick deck slab over four main beams placed at 5 ft 0 in. centres. The decking is stiffened transversely by R.C.C. cross diaphragms spaced at 16 ft 6 in. centres. The deck slab is supported by four prestressed concrete beams and is designed for bending in one direction only with due allowance for continuity. Each prestressing cables consists of 12 Nos. 0.2 in. dia. High tensile steel wires placed around a 16-gauge wire helix of about $\frac{3}{4}$ in. pitch and covered by a protective sheath of 32 gauge iron sheet.

The road surface consists of an average 3 in. thick layer of 1:2:4 cement concrete reinforced with two layers of $\frac{1}{4}$ in. diameter hoops at about 3 ft centres to prevent cracking. The railing is in reinforced concrete and consists of pre-cast hand rail and open web units joined together by cast-*in situ* R.C.C. post and pillasters.

Cost of the Bridge:—The total cost of the bridge is Rs 19,04,300.00 which includes Rs 32,500.00 for approach road and Rs 4,36,500.00 for guide bund pitching.

The construction of the bridge was started in 1957 and completed in 1960.

YAMUNA BRIDGE

By

A. K. CHATTERJEE*

General

RIVER Yamuna crosses Mathura-Raya State Highway in mile 1 of the Road, just outside Mathura Town in Uttar Pradesh. Road traffic used to cross the river over a Road-cum-Rail Bridge built in 1883. During the recent past the rail traffic increased manifold, resulting in more oftener detention of the road traffic causing very great inconvenience. The Railways wanted to regirden this bridge to carry their heavier present-day loading. All this led to considering either to provide a separate Road bridge or to provide double deck girders, one deck for the Railways and the other for the Road which would have involved construction of very costly high approaches in congested built up areas involving greater expenditure than that of providing a separate Road bridge. Therefore, it was decided to build a separate Road bridge.

Site

The Road bridge was located approximately 1,900 ft downstream side of the existing Railway bridge. The site was nearer to the town, permitted construction of good approaches without involving demolition of large number of buildings, and complied to the Defence Ministry's requirements not to have two major bridges within 1,200 ft of each other.

Hydraulic Data and Waterway

The maximum discharge at the proposed bridge site worked out to 200,000 cusecs, for which according to the Lacey's formula 1,200 ft of water-

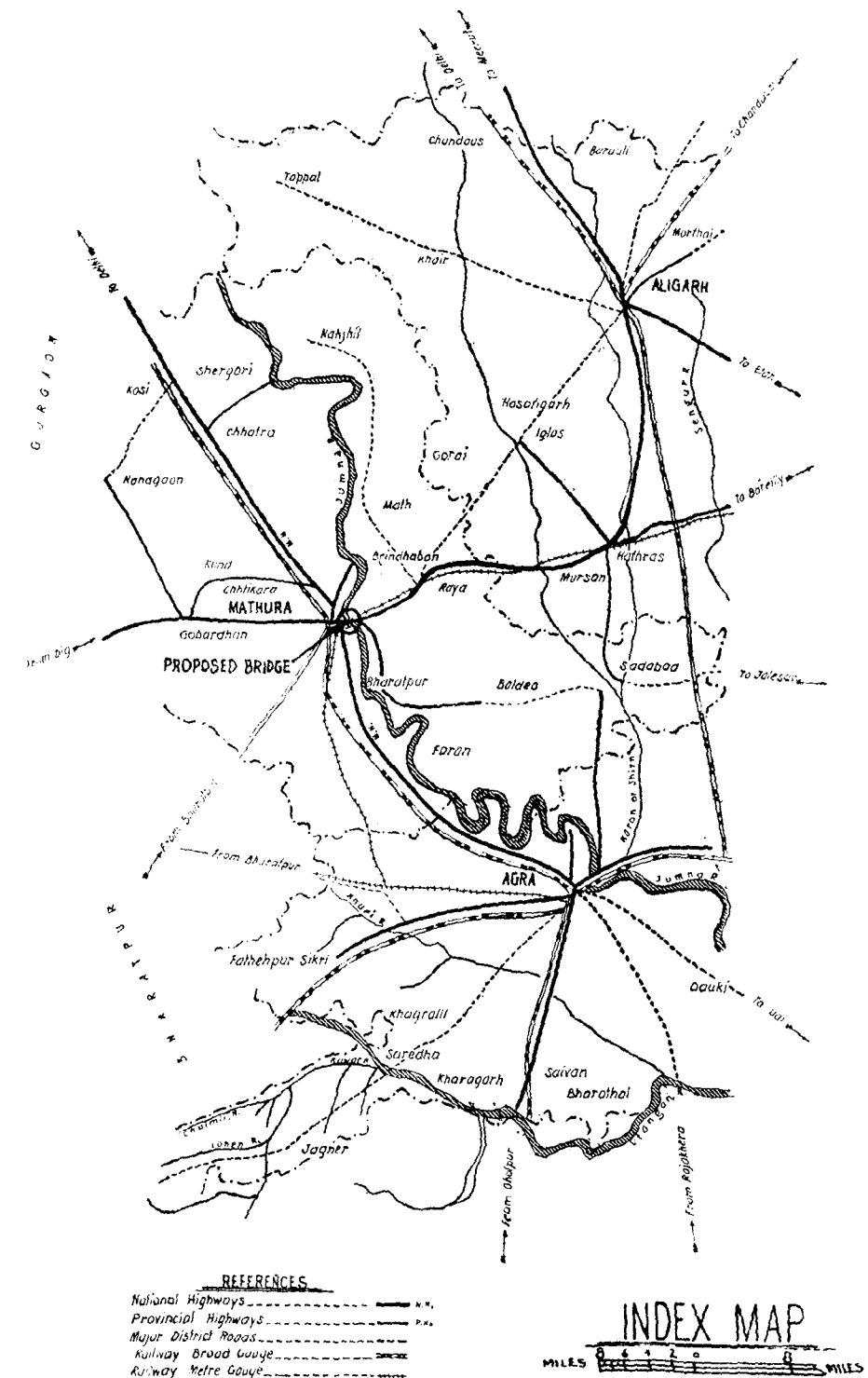
way was required. The existing Railway bridge had 1,050 ft waterway, viz., 7 spans of 164 ft 6 in. centre to centre. This waterway had been found satisfactory in the past. Therefore, 1,000 ft waterway was provided for the Road bridge.

General features of Design

(i) *Superstructure*:—Balanced cantilever box type reinforced concrete girder with 7 intermediate spans each 140 ft and two-end spans of 55 ft including a floating span of 10 ft were provided. The bridge has 22 ft wide carriageway between kerbs and 5 ft wide footpaths on each side. The bridge was designed to carry two lanes of I.R.C. Class A loading or single lane of Class AA loading whichever created severer conditions. Each box girder has 3 ribs each 9 in. thick 11 ft 6 in. high at the support and 5 ft 6 in. high at the centre. The ribs are stiffened by cross beams spaced 17 ft 6 in. centre to centre. The soffit slab is 6 in. thick and the deck slab 7 in. The working stress allowed for concrete and steel were 950 lb per sq. in. and 18,000 lb per sq. in. respectively. The beams rest on cast steel rocker and roller bearings.

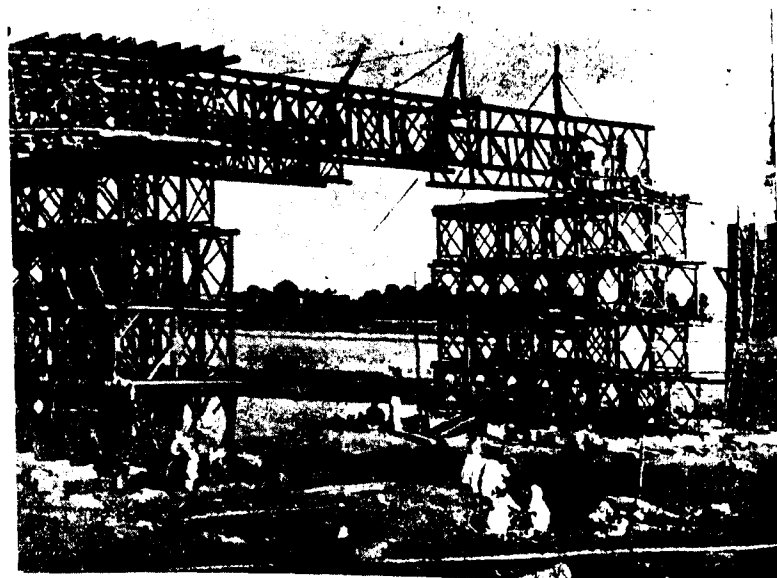
(ii) *Substructure and Foundations*: Each pier rests on a dumbbell shaped twinwell 31 ft long, 16 ft wide, 3 ft 9 in. thick steining and 8 ft 6 in. dia. dredge holes. Foundation wells were sunk down to 75 ft below L.W.L. R.C. Curb with steel cutting edge was provided.

Executive Engineer, Temporary Division, P.W.D., Mathura.





Concreting of main beams & articulations. (Note the hopper which is being used for pouring concrete)



Erection of staging of steel frames at Yamuna Bridge

Trial borings revealed existence of hard clay with kankar at depths of 16 ft and 65 ft below the L.W.L.

Construction

Out of the total of 8 foundation wells, 3 were sunk to full depth of 75 ft below low water level, the remaining 5 wells were left at the depths ranging between 63 to 73 ft, due to the difficulties encountered in well sinking. The first 25 ft of sinking was easy and was done by dredging with heavy grabs without much kentledge. Some wells could be completely dewatered and excavation below the cutting edge upto 40 ft depth was possible by manual labour. For sinking wells below 25 ft depth kentledge upto 100 tons per well had to be provided.

At about 40 ft depth porous water bearing strata consisting of kankar was encountered which made dewatering of wells very difficult. As such, sinking through hard clay had to be done under water. As the grab could not be effective in hard clay, a heavy chisel worked by a diesel engine operated winch and crane was resorted to and found very effective. Blasting with gelignite was also done.

In spite of all efforts well No. 3 could not be sunk for a month even when loaded with 8 cwt per sq. ft. kentledge. Therefore, repeated blasting was done and at the same time the sump in the well deepened. One day suddenly the well went down 22 ft without either damage to the steining or any tilt or shift.

Staging and Centering

The staging comprised of Bailey Bridge components. Intermediate supports were necessary for these trusses to carry the dead load of 140 ft span. These supports were also made out of Bailey Bridge components which rested on grillage foundation of sleepers on the bed of the river. The side shuttering of the beams was of steel plates bolted together and held in position by timber

battens and brackets. The leakage of mortar through joints was prevented by the use of rubber packing between the joints.

Reinforcement and Jointing of the bars

The main reinforcement consisted of $1\frac{1}{2}$ in. dia. bars joined by threading the ends of the bars and providing a coupling box. The coupling box was 7 in. long with 3 in. threads at either ends. $\frac{3}{8}$ in. round holes were left on each side of couplings to facilitate inspections and tightening of the coupling. The end of the couplings were fillet welded alround to compensate the loss in the sections of the steel due to threading.

Construction joints and sequence of Concreting

To start with, the soffit slab was concreted in its entire length upto the top level of the bottom fillets. Concreting proceeded from each pier towards the articulation point on the cantilever side and towards centre in the main span, the two parties moving out from piers to the centre of the main span meeting at the centre at the end of the day's work, without necessitating any construction joint. It was ensured before laying the soffit slab that the reinforcement and side shuttering of the main beams were erected to such a stage that it was possible to concrete all the 3 beams within 7 days of laying the soffit slab.

The three ribs were then concreted one by one and upto the bottom of the top fillets, simultaneously with the cross beams. No construction joint was given in the main ribs but construction joints in the cross beams were left at $\frac{1}{3}$ and $\frac{2}{3}$ of their spans in alternate beams, to stagger the construction joints. Special care was taken near the articulation where richer mix (1 : $1\frac{1}{2}$: 3) was also used.

After concreting all the 3 ribs the staging for the deck slab was erected and

reinforcement placed. Care was taken to expedite the work so that concreting in the deck slab could be placed within 7 days of the laying of the first rib.

Wearing Coat

Wearing coat with slightly stiffer mix was laid simultaneously, with the deck slab.

The River Training Works

The river flows in a more or less straight reach between existing Railway bridge and the new bridge site. The right bank is made of stiff clay and therefore a short guide bund with pitching on the slopes of the approach road only was needed. The left bank is however gently sloping. To train the river to flow through the new bridge during floods it was considered necessary to provide guide bunds 430 ft long on the upstream side and 70 ft on the downstream side. One foot thick boulder pitching and 6 in. thick inverted filter was provided on the slopes of guide bund. Flexible apron was provided at the noses and

shank of the guide bund to cater for the maximum anticipated scour.

General and Costs

The work was started in December, 1959, sinking of wells and construction of piers was completed by March, 1961. Winter floods during February, 1961, caused a great setback in the work. The water suddenly rose up by about 10 ft above the low water level and completely disturbed the staging work of three spans. Fortunately, by that time the span which had been concreted had attained sufficient strength and no structural damage was caused. As a result of this setback only 3 main spans and one suspended span could be concreted before 1961 monsoon. The remaining main span and 2 suspended spans are expected to be completed by the end of March 1962 and bridge opened to traffic before the rains of 1962.

The total cost of the project is likely to be Rs 28.78 lakh including cost of approaches and river training works.

DAMODAR BRIDGE AT BAGNAN

On

NATIONAL HIGHWAY No. 6
(CALCUTTA-BOMBAY)



S. N. GUPTA,
Chief Engineer, Special Road
Development Deptt.,
West Bengal.

Introduction

THIS bridge forms a part of Calcutta-Bombay road route, now designated as the National Highway No. 6. Beyond Howrah Municipal area, this Highway runs almost west until it reaches the State border. On completion of developments of different sections, contemplated under the 3rd Plan

programme, the route mileage will be about 105. In West Bengal, there is no separate existence of Calcutta-Madras Highway, which starts a short distance off from the Subarnarekha river bridge in Bihar.

Between Calcutta and West Bengal border, there used to be 4 unbridged rivers on this route, making communi-



DAMODAR BRIDGE

cations by road practically impossible. Of these, the first bridge, completed in 1957 over the river Dulong near Bihar border, serves local needs at present in view of the remaining gaps.

The Damodar Bridge is the second bridge completed in this route. It was opened by Shri Raj Bahadur, Minister of State for Transport and Communications, Government of India, on the 17th June, 1961. The 3rd bridge, over the Kangsabati, is in hand and is expected to be completed by the end of 1962.

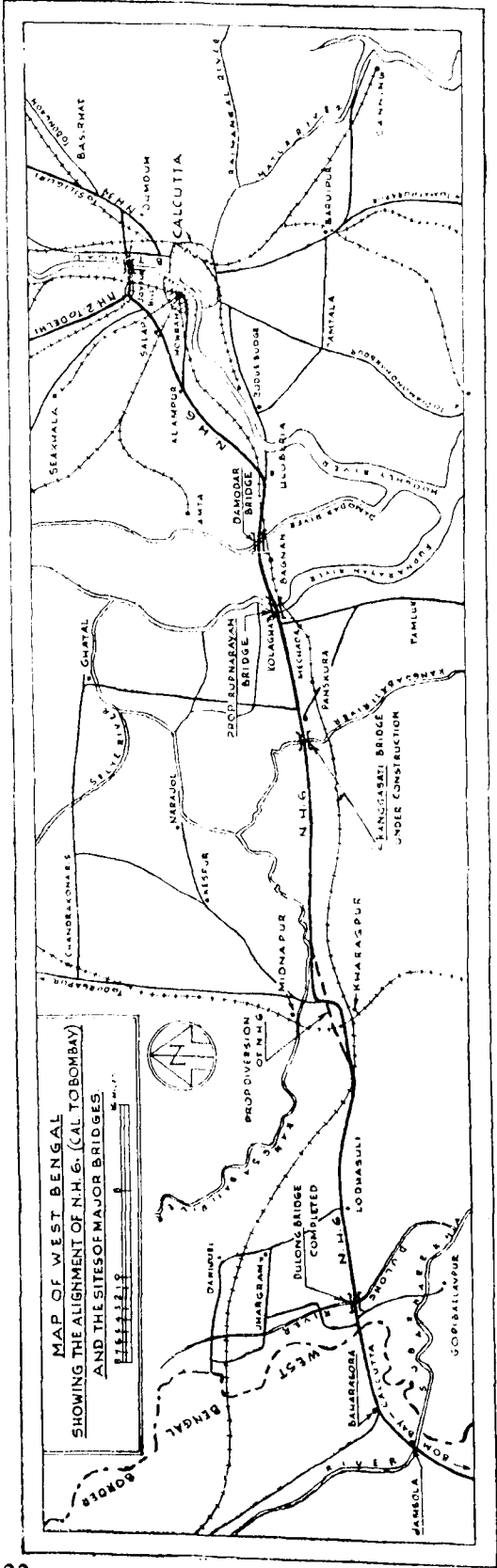
Thus, there is only one unbridged river, viz., the Rupnarayan, over which no work has been possible so far. After trying for several years, it has just been possible to select a bridge contractor. The actual construction works may begin after this monsoon.

Though the distances are short, the Calcutta and industrial areas have remained cut off, so far as road transports are concerned, from Midnapore district and part of Howrah district, by these unbridged rivers so long. When fully bridged, this road route may serve the needs of the State of West Bengal no less than those of the whole country. People all over India are eagerly waiting for the day when the Rupnarayan bridge, the last and the greatest obstacle in this State, will be ready for use.

It may be mentioned here that for ferrying public vehicles across the Rupnarayan, some special arrangements are being made and it is hoped that an unrestricted service may be opened to public after the rains and after a lessee can be appointed for the purpose by Civil Authorities.

River Damodar

The Damodar has her origin in the Choto-Nagpur hills of Bihar near Hazaribagh and flows for a considerable distance till she merges with the Barakar river at the State border. She then flows through the Durgapur Barrage and after some distance, divides and discharges into two channels. The Western channel carrying the major portion is known



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is the Mundeswari and falls into the river Rupnarayan. The other course, now dwindling into a comparatively small stream, retains the name Damodar and discharges into the river Bhagirathi. The Damodar so long known as the "River of sorrows and destructions" for people of Burdwan and Hooghly districts is not so destructive now, due to the dams built in the upper reaches by the D.V.C. The maximum discharge has been controlled to remain within 4,00,000 cusecs. The lower reach of the Damodar carries a maximum flow of about 30,000 cusecs. The river is tidal in the lower reach and is navigable during high tides. A headroom of about 12 ft above H.F.L. has been provided under the bridge for boat traffic.

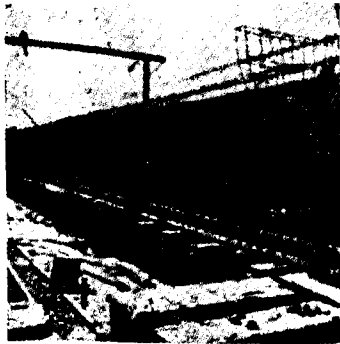
Location

The bridge site has been selected quite near the crossing of the existing high-

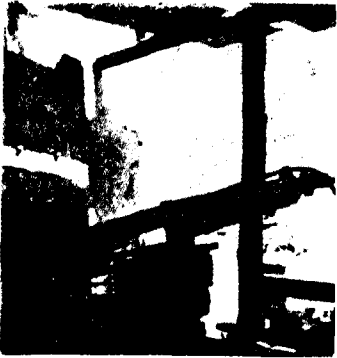
way, which historically is well known as the Orissa Trunk Road. The road bridge stands at about 2,000 ft upstream of the Railway bridge. The present location was found to be quite suitable and



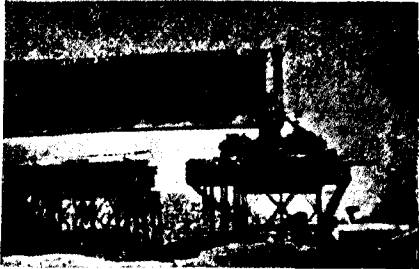
Girders in position



Prestressing cables in position



Cables are being tensioned



Beam being launched on bed trollies

the alignments of the road approaches have also improved in consequence. To maintain proper grades, constructions of about 3/4th mile of new approaches have become necessary.

Preparation and Execution of the job

After detailed investigations, the designs, estimate and drawings were prepared by the State Engineers and were forwarded to the Government of India for sanction in 1953. The scheme was approved by the Government of India in March, 1954, at an estimated cost of Rs 12.29 lakh. After completing

preliminaries, such as the selection of the contractor, preparation of detailed drawings, land acquisitions, etc., the work was started in 1954 and proceeded satisfactorily for a year or so, when well sinking operations received a setback in meeting with a deep strata of soft mud at a depth of about 30 ft below bed. In the deep slime were many buried logs or bits of wooden boats obstructing the sinking of the wells. The nature and extent of the soil type could not be fully appreciated during preliminary investigations. Means for proper explorations were not available at that time. However, fresh investigations were carried out into lower depths till suitable soil strata were detected at about 75 ft below the river bed. This soft strata had extended well below the designed depth of 45 ft for the wells. After necessary tests, it was decided to found all wells in sandy strata, at a depth of about 80 ft below the river bed. This was a tremendous change and the construction arrangements had to be much modified. The sinking of wells through the slush, meeting occasional buried obstructions was really a challenge and the difficult problems were dealt with all available resources and determination. But these operations took much time and delayed the completion of the bridge by more than 2 years. This bridge which was scheduled to be completed in 1958, was practically so by the end of 1960. The problems did not disappear with the sinking of all wells. Due to unavoidable tilts and shifts of the wells, the design of the entire superstructure had to be changed from the conventional type of R.C.C. hollow-box girders to that of prestressed concrete beams and decks.

Design of the Bridge

(a) *Waterway*—The existing Rly. bridge, built in the last century has a clear waterway of 670 ft. Due to deteriorations in the lower region and construction of Dams in the upper reach, it was decided in consultation with the

Irrigation and the D.V.C. authorities to allow for a lower volume of flood discharge. Accordingly, the waterway has been fixed at 500 ft clear and 545 ft between faces of abutments.

(b) *Foundations*—All piers are founded on wells, about 80 ft in depth, resting on coarse sand. The abutments are built on open excavations, the bed being compacted by sal piles. The double D-shaped wells have two dredge holes each and are built in brick masonry, stiffened by R.C. Columns and bands at intervals and have a 3 ft thick R.C.C. Cap.

(c) *Piers*—R. C. C. Cellular type piers, stiffened vertically and horizontally with cross walls and diaphragms, are adopted to reduce loads on the foundations. The average height of the piers is 23 ft.

(d) *Abutments*—R.C.C. open type abutments have been used. The height of the abutment is 28 ft.

Embankments behind the abutments being very high, two approach spans have been added and the earth slopes have been allowed to spill freely around the abutment structures and have been protected by brick pitchings.

(e) *Superstructure*—The superstructure consists of simply supported spans of varying lengths, each span is composed of 2 numbers of prestressed concrete main beams with cross-girders and deck slab which also are prestressed. All prestressing works were done according to Magnel-Blaton system. The main girders having a depth of 8'-7½" were precast on the casting bed made up on an approach road embankment and were prestressed by means of 4 cables, three of which were composed of 48 numbers of 7 mm. wires and the fourth cable had 64 wires of same size. After initial prestressing of the 3 cables, the girder was moved sidewise on rollers to the shore trolley.

(Continued on page 150)

PRESTRESSED CONCRETE SUSPENSION BRIDGES AND THE INFLUENCE OF TIME EFFECTS ON THE MAGNITUDE OF THE PRESTRESSING FORCE

By

DANIEL C. C. VANDEPITTE*

Summary

[The Author outlines the general ideas underlying the concept of the prestressed concrete suspension bridge and briefly discusses the range of application of that type of structure. After giving some information on actual construction of such bridges, he shows how the decrease of the prestressing force, due to shrinkage and creep of the concrete and relaxation of the main cables and suspenders, can be predicted from the properties of the materials. Knowledge of the relationship between the initial and the final magnitude of the prestressing force enables the designer of a prestressed concrete suspension bridge to dimension it properly. That relationship is not as simple as for conventional prestressed concrete beams.]

Relationship between the Prestressed Concrete Suspension Bridge and other Structures

IN a number of publications, some of which are listed at the end of the present paper (1 to 5), the writer has expounded in some detail the general ideas underlying the concept of the prestressed concrete suspension bridge. He has also described the existing bridges of this type. He will not repeat here all he wrote elsewhere, but will summarize the main points developed at length in those previous papers.

A prestressed concrete suspension bridge is, in fact, a self-anchored suspension bridge, differing from conventional self-anchored suspension bridges in that the steel stiffening girders, which resist thrust and bending, are replaced by concrete stiffening beams (Fig. 1). Whereas the steel stiffening members of an ordinary self-anchored suspension bridge

require a considerable amount of additional steel in order to be able to resist the thrust as well as the flexure, the concrete stiffening beams of a prestressed concrete suspension bridge are prestressed by the horizontal component of the pull in the main cables and therefore they need very little reinforcement, provided, of course, that they be adequately designed.

A prestressed concrete suspension bridge may also be regarded as a conventional prestressed concrete bridge, whose main girders are prestressed by cables that have large eccentricities, much larger in fact than the depth of the stiffening beams. Since large eccentricities in prestressed concrete are in order when the spans grow long, it follows that the prestressed concrete suspension bridge is suitable only for crossings of sufficiently long span. The live load produces a non-negligible increase of the pull in the cables of a concrete suspension

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Fig. 1

bridge, which thus carry directly a portion of the live load. The stiffening beams are correspondingly relieved and they have to resist smaller live load moments than the main beams of an equivalent prestressed concrete bridge. This effect also becomes more important as the span length increases.

There is, finally, a close relationship between a prestressed concrete suspended beam and a bowstring girder with a very flexible arch. In the former, the tensile member, the main cable, is curved and has no stiffness, and the compression member, the stiffening beam, is straight or practically straight. In the latter, the compression member, the arch, is curved and has little stiffness, while the tensile member, the beam, is straight and possesses rigidity. Both structural systems behave very similarly. Prestressed concrete suspended beams, like bowstring girders, may be used as the main carrying members of roof structures of very long span as well as of bridges.

Comparison between the Prestressed Concrete Suspension Bridge and Alternate Structures

Comparison of a prestressed concrete suspension bridge with a true or a self-anchored steel suspension bridge yields the following points:

- (1) The former structure is heavier and needs larger foundations to carry the vertical loads.
- (2) It requires no external anchorages having to resist large horizontal forces, as a true suspension bridge does.

- (3) In the former type of bridge a much cheaper concrete superstructure replaces the steel superstructure.
- (4) In a prestressed concrete suspension bridge steel is required only for the main cables and for the hangers; the total amount of steel thus is very much smaller than the amount required for a true steel suspension bridge, and the difference is even greater in the case of a self-anchored steel suspension bridge.
- (5) A prestressed concrete suspension bridge needs very little maintenance.
- (6) A prestressed concrete suspension bridge is much more rigid than an equivalent steel structure, whether a true or self-anchored suspended structure.

Provided that the spans are long enough, the stiffening beams of a prestressed concrete suspension bridge are much shallower and contain far less concrete than the main beams of its conventional prestressed concrete counterpart. Moreover, since the cables of the former have larger eccentricities, the requisite prestressing force is correspondingly smaller and the amount of prestressing steel is considerably reduced.

When a prestressed suspended beam and a bowstring girder with a flexible arch are compared, it is seen to be easier and cheaper to install a cable than to cast a concrete arch high in the air. Lateral buckling is much more of a problem for such an arch than for a

prestressed suspended beam, which in a bridge is normally part of a relatively wide, monolithic superstructure comprising two such beams.

Range of Application of Prestressed Concrete Suspension Bridges

The considerations above show that the prestressed suspended beam has some definite advantages over alternate structural systems and that it therefore provides an economical solution to many bridge problems. Sweeping generalizations are, of course, not warranted. The author is of the opinion that prestressed concrete suspension highway bridges are technically and economically feasible in the range of spans between, say, 175 ft and about 1,700 ft.

The limitation does not reflect any technical impossibility to exceed the limits indicated but derives from economic considerations. It is not worthwhile to design highway suspension bridges for spans of less than 175 ft length, conventional prestressed concrete bridges being the more suitable solution. For very long spans exceeding about 1,700 ft, a conventional true steel suspension bridge, the mechanical properties of the foundation soil permitting, would probably be cheaper than a prestressed concrete suspension design.

The two figures given are intended only to indicate the order of magnitude of the limiting span lengths. Obviously, no single figure can represent the true economic maximum or minimum in all possible cases since both depend on many factors, such as the ratio of width to span length (for the lower limit), the quality of the soil, and the relative cost of labour and building materials. Only a comparative study of alternate designs can, in any given case, decide whether one or the other is the cheapest.

Construction of Prestressed Concrete Suspension bridges

The concept of the prestressed concrete suspension bridge has left the

realm of speculation and entered that of actual construction. Two specimens have been built and a third is under construction in the vicinity of Gent, Belgium. The two completed structures are described in more or less detail in previous papers (3, 4 and 5).

Figures 2, 3, 4 and 5 show the elevation, the longitudinal section, the cross-section and a portal of the bridge which is now being built at Merelbeke. It spans the access channels to twin locks and to a weir. Part of the wall between the flow channel and one of the navigation channels provides support for the bridge portal on the right-hand side in fig. 3.

Although the natural range of spans for the type of bridge discussed extends far beyond 330 ft, none of the spans yet built or a building exceeds a length of 330 ft. They have proved to be very economical structures. Opportunities to build longer spans in Belgium are almost non-existent.

The concrete for all three suspended structures was or will be cast in shuttering supported by scaffolding. That procedure was and is the most convenient, for the bridges span a canal not yet dug at the time of construction. It is, however, quite possible to build a prestressed concrete suspension bridge without making use of any scaffolding.

Prestressing of all three bridges was or will be achieved in the following manner. When the floor structure and the two portals are concreted, the latter in a position several feet (depending mainly on the span length) below their design elevation, the main cables are strung without any tension and all their connexions with the suspended structure are installed. The two portals are then alternately jacked up with respect to the piers by means of hydraulic presses until they reach their design elevation, and steel hinges are inserted subsequently under the tower legs. The elevation of the roadway above the piers is not altered by the prestressing

operation, for each portal is entirely independent of the roadway and of its connexions with the pier.

Again prestressing can be carried out in other ways, but the procedure described appeared to be the most efficient in the cases at hand.

Figures 6, 7 and 8 show the elevation, the cross-section and the portal of a prestressed concrete suspension bridge with a centre span of 720 ft and proposed for a river crossing in Venezuela. The suspended structure is made up of precast sections.

Properties of materials and other parameters involved

We now come to the main purpose of this paper which is to discuss the loss of prestressing force likely to occur in a prestressed concrete suspension bridge due to time effects. The problem differs from the corresponding and extensively investigated problem in conventional prestressed concrete construction and it therefore requires special treatment.

We assume that the stiffening beams are prismatic, symmetrical with respect to the centre of the bridge, and continuous over their interior supports, and that their axis is parabolic in elevation (the straight line being a special case) in each span taken individually. We also suppose that the shape of the main cables is, in each span, a polygon inscribed in a parabola.

We use the following notations (Fig. 9):

$I = Ar_s$ the moment of inertia and A the cross-sectional area of one concrete stiffening beam

A_c = the cross-sectional area of one main cable

A_s and $A's$ = the cross-sectional area of one suspender in the centre span and in a side span respectively

L and $l = \lambda L$ the length of the centre span and of one side span

f_c and f_s = the camber of the stiffening beam in the centre and the side spans respectively

f_{c1} and f_{s1} = the sag of the main cable in the centre and the side spans respectively

$$f = f_c + f_s$$

$$f_1 = f_{c1} + f_{s1}$$

$$S = \frac{f_1}{\lambda + 1}$$

γ, β' and β'' the slope of the cable at three different points

a the downward eccentricity of the cable above the end supports

$$m = \frac{2f + 2\lambda f_1 + \lambda a}{3 + 2\lambda}$$

the moment produced above each intermediate support by two forces $P=1$ acting on the ends of the supposedly cut cable (fig. 9), these forces being the only load on the elastic system

E, E_c and E_s the moduli of elasticity for loading of very short duration respectively of the concrete, the main cable and the suspenders; E is supposed to be measured at the time of the prestressing operation and not to increase appreciably thereafter.

$\sum l_s$ and $\sum l's$ the sum of the lengths of all the suspenders that are tied to one stiffening beam respectively in the main span and in one side span.

δ = the relative displacement (overlap) of both ends of the severed cable instantaneously produced by two forces, P , acting on those ends (Fig. 9)

w the total permanent load per unit length of one stiffening beam; the weight of the main cable and

of the suspenders is included in w

$$P_0 = \frac{wL^2}{8f}$$

$P_0(1 + \alpha)$ the initial magnitude of the prestressing force in one stiffening beam

$P_0(1 + \epsilon)$ the final magnitude of the prestressing force, $P(a - \epsilon)$ denoting the loss of prestressing force due to time effects.

$$\alpha' = s \alpha = (1 - s)$$

$$\epsilon' = s \epsilon = (1 - s)$$

ϵ_s = the unit shrinkage of the concrete of the stiffening beams occurring after prestressing is carried out; if the concrete is several months old when it is prestressed, ϵ_s may be about 0.0001 in a moderate climate

E the final value of the modulus of elasticity of the concrete for loads of very long duration applied after the prestressing operation; in other words: the strain instantaneously produced by a constant compressive stress σ is $\frac{\sigma}{E}$, but it increases with time due to creep, and its final value is $\frac{k\sigma}{E}$; k is supposed to be independent of σ ; k may be of the order of 2.25 or 2.75 for good concrete, depending on its age at prestressing time

p the unit creep of the main cable under the average permanent working stress σ_c it sustains in the structure; in other words: the magnitude of the strain produced by the

constant stress σ_c is $\frac{\sigma_c}{E_c}$ initially,

but it gradually increases slightly and its final value is $\frac{\sigma_c}{E_c}(1 + p)$; p

is quite small, about 0.01 or 0.02, for cables made up of parallel galvanized wires of good quality with a working stress equal to or

slightly exceeding half of their ultimate strength.

E_s the final value of the modulus of elasticity of the suspenders under the average permanent working stress σ_s they sustain in the structure; in other words: the magnitude of the strain produced by constant stress

σ_s is $\frac{\sigma_s}{E_s}$ initially, but it gradually

increases due to creep, and its final

value is $\frac{n \sigma_s}{E_s}$; $n = 1$ for suspen-

ders consisting of solid steel bars, but perhaps $n = 1.3$ for strands

$\eta = \frac{a - \epsilon}{1 + \alpha}$ the relative decrease of the prestressing force.

Deformability of the Superstructure

By virtue of Castigliano's theorem, δ is equal to the derivative, with respect to P , of the internal energy W stored in the superstructure (Fig. 9):

$$W = \frac{1}{2} \sum \int \frac{M^2}{EI} ds + \frac{1}{2} \sum \int \frac{N^2}{EA} ds$$

and

$$\delta = \frac{dW}{dP} = \sum \frac{1}{EI} \int M \frac{dM}{dP} ds + \sum \frac{1}{EA} \int N \frac{dN}{dP} ds$$

all integrations being carried out over all parts of the superstructure.

Working out the latter expression, one finds:

$$\delta = \frac{PL}{EI} K = \frac{PL}{EI} (K_{bM} + K_{bN} + K_c + K_s) \dots (1)$$

$$K_{bM} = \frac{8}{15} (f - m)^2 - \frac{m}{15} (4f - 7m) + \frac{2}{3} \lambda \left[a^2 - m(-m + 2f_1 + a) + 2f_1(a + \frac{4}{5}f_1) \right] \dots (2)$$

$$K_{bN} = (1 + 2\lambda) r^2 \dots (3)$$

$$K_c = \frac{E}{E_c} \cdot \frac{I}{A_c} \left\{ \frac{F(\gamma)}{8f_g \gamma} + \frac{\gamma^2 L}{32 f_g} \right. \\ \left. \left[F(\beta') - F(\beta'') \right] \right\} \dots (4)$$

$$K_s = 64 \frac{Ed^2}{E_s L^2} \left(\frac{1}{A_s} \cdot \frac{f_s^2}{L^2} + \frac{\sum l_s}{L^2} \right. \\ \left. + 2 \frac{I}{A_s^2} \cdot \frac{f_s^2}{L^2} + \frac{\sum l_s^2}{L^2} \right) \dots (5)$$

with $F(a)$ denoting the function

$$F(a) = \operatorname{tg} a (5 + 2 \operatorname{tg}^2 a) \sec a + 3 \lg$$

(see $a + \operatorname{tg} a$)

K_{bM} , K_{bN} , K_c and K_s respectively denote the effect, in terms of $\frac{PL}{EI}$, of the curvature of the beam, the contraction of the beam, the elongation of the cable and the elongation of the suspenders, on the elastic discrepancy δ due to the two forces P . K_{bM} and K_c are normally small with respect to K_{bM} and K_c ; this translates the fact that the deformability of the superstructure is mainly due to the flexibility of the stiffening beam and to the elongation of the main cable.

The larger K_{bM} with respect to K_c , the more rigid the cable with respect to the beam and the more the live load moments in the beam decrease due to interaction between beam and cable. The ratio $\frac{K_{bM}}{K_c}$ increases with increasing span lengths.

One could add another term to the four terms appearing in the above expression for δ . The fifth term would reflect the influence on δ of the elastic shortening of the towers. It is, however, found to be entirely negligible with respect to the others.

When two constant forces P act during a very long time on the cut ends of the cable, the overlap δ increases and in the long run its value becomes:

$$\delta = \frac{PL}{EI} \left[kK_{bM} + kK_{bN} + (1+p) \right. \\ \left. K_c + nK_s \right].$$

The value of δ for loads of short and long duration is useful in the analysis of the effects of live load and of a temperature differential between the cable and the beam.

Magnitude of the Prestressing Force

Efficient design of a prestressed concrete suspension bridge requires the prestressing force to be such that the upward forces exerted on the suspended structure by the suspenders and through the camber of the beam exceed, on the whole, the self weight of that structure. Under these conditions, the stiffening beam deflects elastically as shown in Fig. 10 in the absence of live load, and the permanent bending stress in the upper edge of the beam is compressive above the interior supports and tensile at the centre of the main span. Since the live load gives rise to curvatures and bending stresses of opposite sign, the deflected shape pictured in Fig. 10 is indeed desirable, as in the case of a conventional continuous prestressed concrete beam.

A force equal to $P_0 = \frac{wL^2}{8f}$ in the cable develops forces in the suspenders of the centre span and upward forces due to the hogging of the beam, which together are exactly in equilibrium with the self weight of the suspended structure in the centre span. It is worth noting that the f appearing in the denominator of the expression for P_0 is not the sag f_s of the cable, but the sum of that sag and of the camber f_b of the beam, for the combination of thrust in and hogging of the beam produces arch action and contributes to carrying the load.

A prestressing force exceeding P_0 thus is necessary to generate curvatures of the sign shown in Fig. 10. Let $P_0(1+a)$ denote that force, or, more precisely, the initial value of that force.

The prestressing force decreases with time due to shrinkage, creep and relaxation. It is desirable that the curvature

of the beam under permanent load alone not change sign while these phenomena run their course, hence that ε in our notation $P_0(1+\varepsilon)$ for the final value of the prestressing force, although smaller than a , still be positive. ε evidently depends on a .

The problem now is to find the relationship between ε and a .

Relative Displacements of the ends of the severed cable due to time effects

We suppose that the main cable is cut immediately after the prestressing operation is performed, and we apply on the cut ends of the cable two forces which are equal, at any given moment to the tensile force existing at the same moment in the cable of actual structure. The two forces consequently are made to vary from $(1+a)P_0$ initially to $(1+\varepsilon)P_0$ finally.

We now study the relative displacement of the cut ends of the cable which would gradually occur due to each time effect taken separately.

Shrinkage

Shrinkage alone obviously produces an overlap given by

$$\Delta_1 = (1+2\lambda)L\varepsilon_s$$

Relaxation of the main cable

Since the elongation of the main cable under two equal forces P instantaneously produces a total discrepancy $K_c \frac{PL}{EI}$, the differential discrepancy studied here is

$$\Delta_2 = K_c (1+p)(1+\varepsilon) \frac{P_0 L}{EI} - K_c$$

$$(1+a) \frac{P_0 L}{EI} = K_c \left[p + (1+p)\varepsilon \right. \\ \left. - a \right] \frac{P_0 L}{EI}.$$

Creep Contraction of the Axis of the Stiffening Beam

The beam finally shortens a total amount $\frac{(1+\varepsilon)P_0 k}{AE} \cdot L(1+2\lambda)$, while

the immediate contraction generated by the prestressing operation itself is $\frac{(1+a)P_0}{AE} \cdot L(1+2\lambda)$. The overlap

$$\Delta_3 = (1+2\lambda)(k + k\varepsilon - 1 - a) \frac{I}{A} \frac{P_0 L}{EI} = (1+2\lambda)(k - 1 + k\varepsilon - a)r^2 \frac{LP_0}{EI}.$$

Creep Flexure of the Stiffening Beam

The forces acting on the beam considered as a free body immediately after prestressing and in the absence of live load are indicated in Fig. 11. They are:

- the permanent load w (including the weight of the cable and of the suspenders)
- the uniformly distributed upward load $\frac{(1+a)P_0 \cdot 8f}{L^2} = (1+a)w$ in the centre span
- the uniformly distributed upward load $\frac{(1+a)P_0 \cdot 8f_1}{l^2} = (1+a) \frac{f_1}{\lambda^2 f} \frac{8P_0 f}{L^2} = (1+a)sw$ in both side spans.
- the thrust $(1+a)P_0$ at both ends
- the end moments $(1+a)aP_0$ due to the eccentricity a of the cable above the end supports.

We have taken the camber of the actual beam into account by drawing its axis as a straight line on the one hand and by including the effect of arch action in the upward forces on the other hand.

The net loading initially acting on the beam is shown in Fig. 11-b, with

$$a^1 w = (1 + a)sw - w = w[s - (1 - s)]$$

The net loading finally acting on the beam is obtained simply by replacing everywhere

a by ε and a^1 by $\varepsilon' = \varepsilon\varepsilon - (1 - \varepsilon)$.

The initial bending moment M_0 over the interior supports is easily found to be given by

$$(3 + 2\lambda)M_0 = P_0[2a' + 2\lambda^2 a + \lambda a(1 + a)].$$

The initial moment in the left-hand side span at a distance $x = tL = t\lambda L$ from the left hand end support is

$$M = -(1 - t)(1 + a)aP_0 + tM_0 - \frac{a^1 w l^2}{2}$$

$$t(1 - t) = -(1 + a)aP_0 + t\left[M_0 - \frac{a^1 w l^2}{2} + (1 + a)aP_0\right] + t^2 \frac{a^1 w l^2}{2}$$

The moment in the centre span at a distance xL from the left-hand interior support is

$$M = M_0 - \frac{awL^2}{2} t(1 - t) = M_0 - t.$$

$$\frac{awL^2}{2} + t^2 \frac{aw^2\lambda}{2}.$$

Initial bending of the beam would produce a total overlap Δ_1' given by

$$\Delta_1' = \frac{1}{EI} \int M \frac{dM'}{dP} dx, \text{ in which } \frac{dM'}{dP}$$

represents the derivative, with respect to P , of the moment M' induced by two equal forces P pulling on the cut ends of the cable. It is easily found that

$$\frac{dM'}{dP} = -a + t(m + a - 4f_1) + t^2 4f_1$$

in the left-hand side span and

$$\frac{dM'}{dP} = +m - t4f + t^2 4f \text{ in the centre span.}$$

Performing the integration, one finds, after some manipulation of the symbols:

$$\Delta_1' = \frac{5\lambda a(1 + a)(2a - m + 2f_1) + 2f_1[a^1\lambda^2(5a - 5m + 8f_1) + a(4f - 5m)]}{P_0 L} \\ \frac{15EI}$$

The total discrepancy Δ_1'' brought about in the long run by flexure of the beam is found by replacing a by ε, a' by

$$\varepsilon' = \varepsilon \text{ by } \frac{E}{K} \text{ in the expression for } \Delta_1' :$$

$$\Delta_1'' = \frac{5\lambda a(1 + \varepsilon)(2a - m + 2f_1) + 2f_1[\varepsilon^1\lambda^2(5a - 5m + 8f_1) + \varepsilon(4f - 5m)]}{K P_0 L} \\ \frac{15EI}$$

The discrepancy due to creep flexure is

$$\Delta_1 = \Delta_1'' - \Delta_1'.$$

If a' and ε' are replaced by their expressions $\frac{f_1}{\lambda^2 f}(1 + a) - 1$ and $\frac{f_1}{\lambda^2 f}(1 + \varepsilon) - 1$, it is finally found that

$$\Delta_1 = \frac{(k - 1)\lambda[5a(2a - m + 2f_1) - 2(\lambda^2 f - f_1)(5a - 5m + 8f_1)]}{\lambda^2 f}$$

$$+ \frac{k\varepsilon - a}{3 + 2\lambda} [5\lambda a(3\lambda a + 4\lambda f_1 + 6a + 12f_1) + 4f(f + 4\lambda f - 10\lambda f_1 - 5\lambda a) + 12\lambda f_1^2(1 + \lambda)] \frac{P_0 L}{15EI}$$

$$= (A + k B\varepsilon - Ba) \frac{P_0 L}{EI}$$

$$\text{with } A = \frac{1}{15}(k - 1)\lambda[5a(2a - m + 2f_1) - 2(\lambda^2 f - f_1)(5a - 5m + 8f_1)] \quad \dots (6)$$

$$\text{and } B = \frac{1}{15(3 + 2\lambda)} [5\lambda a(3\lambda a + 4\lambda f_1 + 6a + 12f_1) + 4f(f + 4\lambda f - 10\lambda f_1 - 5\lambda a) + 12(4 + \lambda)\lambda f_1^2] \quad \dots (7)$$

Relaxation of the suspenders

The initial elongation of the suspenders would produce a total overlap

$$\Delta_s' = K_s(1 + a) \frac{P_0 L}{EI}, \text{ which increases}$$

gradually and finally becomes $\Delta_s'' = K_s(1 + \varepsilon) \frac{nP_0 L}{EI}$. The overlap Δ_s due to relaxation of the suspenders hence is

$$\Delta_s = \Delta_s'' - \Delta_s' = K_s(n - 1 + n\varepsilon - a) \frac{P_0 L}{EI}.$$

This expression slightly overestimates the discrepancy, for we calculated K_s on the assumption that the full force P in the cable induces tensile forces in the suspenders, whereas part of the actual cable pull $(1 + a)P_0$ or $(1 + \varepsilon)P_0$ serves to carry the own weight of the cable and does not stress the suspenders. The error is quite small unless the spans are very long, and can be easily corrected if the need arises.

Relation between ε and a

Actually there would be no discrepancy in the real structure if the cable were severed, neither immediately after prestressing, nor at any time afterwards, for the prestressing force varies in such a way that the cut ends permanently remain in contact. The final value $(1 -)P_0$ of the prestressing force is governed by precisely that condition.

We may write therefore:

$$\Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 + \Delta_s = 0, \text{ or}$$

$$(1 + 2\lambda)L\varepsilon s + \frac{P_0 L}{EI} \{pKc + (1 + 2\lambda)$$

$$(k - 1)r^2 + A + (n - 1)K_s + \varepsilon[(1 + p)Kc + (1 + 2\lambda)kr^2 + kB + nK_s] - a[Kc + (1 + 2\lambda)r^2 + B + K_s]\} = 0$$

$$\text{or } \varepsilon[(1 + p)Kc + (1 + 2\lambda)(kr^2 + kB + nK_s)] = a[Kc + (1 + 2\lambda)r^2 + B + K_s] - [pKc + (1 + 2\lambda)(k - 1)r^2 + A + (n - 1)K_s + (1 + 2\lambda)\varepsilon s \frac{EI}{P_0}]$$

We thus find a linear relation of the form $\varepsilon = Ca - D$ (9), with $C < 1$ and $D > 0$. The numerical values of C

and D are easily computed from formulae (4), (5), (6), (7) and (8), provided, of course, that the properties of the materials, notably $E, E_c, E_s, \varepsilon_s, k, p$ and n , are known or assumed, as well as all the geometrical characteristics of the structure. When a value is selected for a , hence the magnitude of the initial prestressing force $(1 + a)P_0$ is decided upon, formula (9) yields ε , and the final value $(1 + \varepsilon)P_0$ of the prestressing force, the loss $(a - \varepsilon)P_0$ and percentage loss

$$\eta = \frac{a - \varepsilon}{1 + a} \text{ of prestressing force are}$$

known immediately. Knowing ε enables the designer to calculate the prestress distribution prevailing after shrinkage, creep and relaxation have taken place.

Conversely relation (9) enables the designer to select an adequate value for a . He can write the initial prestress in two or three critical cross-sections as a function of a , and the final prestress in the same sections as a function of ε , hence also a . The condition that the prestress combined with the live load stress may not exceed the stresses allowed by the governing specifications is translated into a number of inequalities in which a appears as the only variable, and by means of these the designer can decide upon the most suitable value for a .

$a = 0$ yields $\varepsilon < 0$. Consequently, if the initial prestressing force is such that the forces acting upwards on the stiffening beam in the main span are exactly equal to the permanent load, the beam will, after a while, sag below its unstressed shape in that span under permanent load alone. This merely confirms what could be expected intuitively.

In the case of the Merelbeke bridge shown in figures 2, 3, 4 and 5, equation (9) becomes $\varepsilon = 0.72a - 0.055$. The most satisfactory value for a turned out to be 0.19, and, consequently

$$\varepsilon = 0.082 \text{ and } \eta = \frac{a - \varepsilon}{1 + a} = 0.091.$$

Decrease of the prestressing force in a conventional post-tensioned beam—comparison with a prestressed concrete suspended beam.

We assume that the concrete beam is prismatic and that the prestressing cable coincides everywhere with the axis of the prism.

Let P now denote the initial and $(1-\eta)P$ the final magnitude of the prestressing force.

If the tendon is detached from one end of the concrete beam, the time effects considered above for the suspension bridge cause the following discrepancies between the end of the concrete prism and the loosened end of the cable:

— shrinkage of the concrete prism:

$$\Delta_1 = (1 + 2\lambda) L \varepsilon_s$$

— relaxation of the tendon: $\Delta_2 = (1$

$$+ 2\lambda) L \left[(1 + p)(1 - \eta) - 1 \right] \frac{P}{A_c \varepsilon_c}$$

$$= (1 + 2\lambda) L \left[p - (1 + p)\eta \right] \frac{f_c}{E_s}$$

if $f_s = \frac{P}{A_c}$ denotes the allowable steel stress, which is supposed to be reached in the tendon during prestressing

— creep contraction of the beam:

$$\Delta_3 = (1 + 2\lambda)(k - 1 - k\eta) \frac{PL}{AE}$$

Flexure of the prism does not cause any discrepancy and there are, of course, no suspenders. We may write again $\Delta_1 + \Delta_2 + \Delta_3 = 0$

$$\text{or } \eta \left[(1 + p) \frac{E}{E_c} \cdot \frac{A f_s}{P} + k \right]$$

$$= \varepsilon_s \cdot \frac{AE}{P} + p \cdot \frac{E}{E_c} \cdot \frac{A f_s}{P} + k - 1, \dots (10)$$

This equation immediately gives the percentage loss of prestress η , when the average prestress $\frac{P}{A}$ in the cross-section of the concrete beam is known.

The working stress f_s in the tendons of a conventional post-tensioned beam is normally a larger fraction of the ultimate steel strength than the working stress in the main cables of a prestressed concrete suspension bridge. Consequently p is also higher for the former than for the latter, at least when the suspension cables are made up of parallel wires.

But even when identical values of ε_s , k and p are introduced into the equations (8) and (10), the percentage loss of prestress η turns out to be smaller for a prestressed concrete suspended beam than for a conventional prestressed concrete beam. This is due to the fact that the former system is less rigid than the latter as far as relative displacements caused by tensile forces in the cable and corresponding thrusts in the concrete prism are concerned, and, in consequence, any discrepancy induced by shrinkage, creep or relaxation is cancelled out by a smaller variation of the cable pull in the former structure than in the latter.

The percentage loss of prestressing force η in a prestressed concrete suspension bridge decreases with increasing span length.

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- (1) Self-anchored suspension bridges with prestressed concrete stiffening girders—International Congress on Prestressed Concrete—Gent 1951
- (2) Le pont suspendu a' poutres de rigidité en béton précontraintes par les câbles porteurs—Annales des Travaux Publics de Belgique—Octobre 1955
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- (4) Prestressed concrete suspension bridges—Proceedings of the International Conference on Prestressed Concrete—San Francisco—1957
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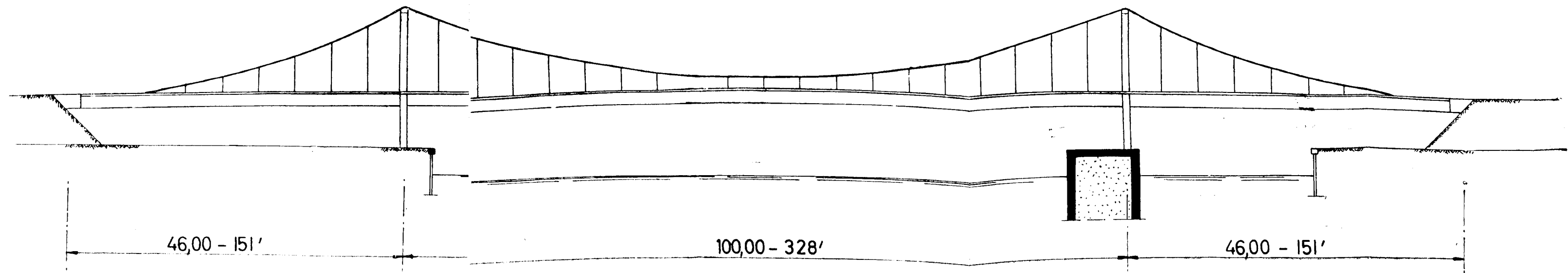


Fig. 2. W12 Merelbeke Aanzicht Elevation

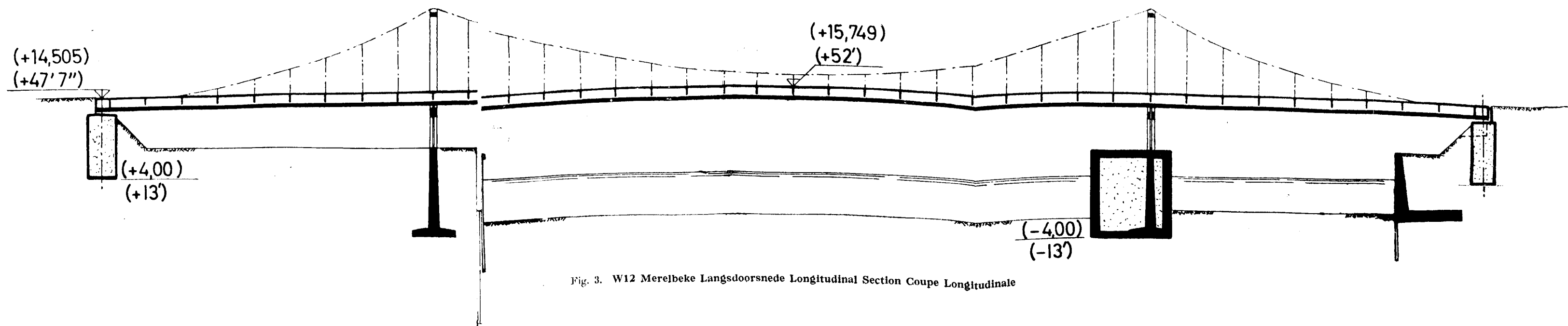


Fig. 3. W12 Merelbeke Langsdoorsnede Longitudinal Section Coupe Longitudinale

LARGE PRESTRESSED CONCRETE BRIDGE COMPLETED

OVER MANN RIVER AT JACKADGERY

(By Department of Main Roads, New South Wales, Australia)

THE construction of a large prestressed concrete bridge across the Mann River at Jackadgery has recently been completed by the Department of Main Roads. It is on the new route of the Gwydir Highway between Grafton and Glen Innes. (The construction of this 73-mile road link through mountainous country was described in the March, 1961 number of "Main Roads".)

The Mann and Nymboida Rivers, which together drain over 3,000 square miles of the eastern slopes of the Great Dividing Range in northern New South Wales, are major tributaries of the Clarence River. The Mann River, which is joined by the Nymboida River a few miles upstream from Jackadgery, flows into the Clarence River about 45 miles northwest of Grafton. At Jackadgery the river runs between steep high banks over

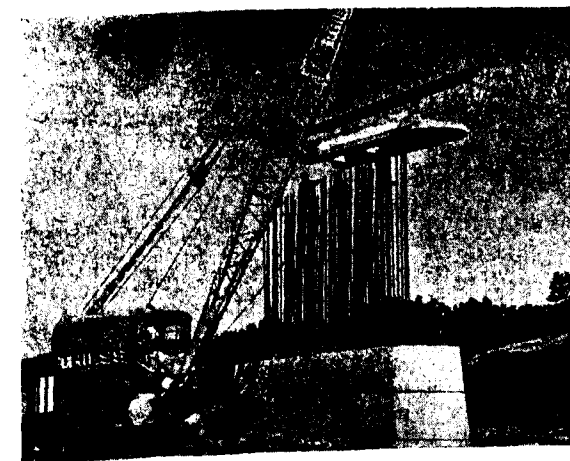
a rocky bed strewn with boulders and shingle. Normally the flow is small, but in flood times the river may run forty feet deep over a width of nine hundred feet with an average velocity of about 16 feet per second. During floods heavy boulders are rolled along the bed while drift timber of all sizes is carried by the river.

At the bridge site a hard sedimentary rock, greywacke, outcrops in the river bed and the banks are relatively stable.

Design

The working drawings of the bridge were prepared for the Department of Main Roads by Messrs Maunsell and Partners, London. The design is for nine spans of simply supported post-tensioned prestressed beams with a reinforced concrete deck acting compositely with the beams and continuous for the full length of the bridge. The girders are prestressed by the Freyssinet system, using cables consisting each of twelve 0.276" dia. wires. The reinforced concrete piers are of the plate type measuring 26 ft x 3 ft for the greater part of their height.

There are seven spans each 105 feet long and two end spans of 95 ft 6 in. The overall length is 938 ft and the distance between abutment faces is 923 ft. Because the western bank is higher than the eastern bank the deck has been built on a grade of 1.5%. At the eastern end of the bridge the undersides of the girders are 1 ft above high flood level, estimated to be 49 ft above normal stream level, while at the western end the clear-



Construction of bridge over the Mann River at Jackadgery—Pier base—showing placing of section of cap

rance is 15 ft. At the centre of the bridge, the deck is approximately 65 ft above normal river level.

The eight piers are founded on rock which forms the stream bed. The base of each pier measures 37 ft x 10 ft and is set a minimum of one foot into sound rock. Pier stability is provided by the dead weight of the structure, but in the three central piers additional stability is given by 1½ in. dia. steel bars extending 8 ft into the rock. The base of each pier is of reinforced concrete cast *in situ* to a suitable common level. Above that level a segmental type of construction was specified using precast concrete shells set on top of one another and progressively filled with heavily reinforced concrete. The shell segments are three feet high, the lower five being tapered, and joints between them are recessed to give horizontal lines for appearance.

The abutments are constructed on steel piling driven to rock at depths of about 50 ft at the eastern abutment and 30 ft at the western abutment.

The superstructure consists of three lines of prestressed concrete beams at 10 ft centres, supporting a reinforced concrete deck. Each beam is 6 ft deep and weighs about 50 tons. The superstructure rests on fixed supports at the eastern abutment and is freely supported on roller bearings at the western abutment and at all piers. The eastern abutment serves as an anchor for the superstructure. Beam ends over each pier are stepped so that the beam in one span rests on the beam from the adjoining span, which in turn is supported by the roller bearing, as shown in Fig. 1. This necessitated progressive construction from the eastern bank.

After the beams were stressed and positioned in adjoining spans, the concrete deck over the central three-quarters of each span was placed. The next concrete placed included the joints between the stepped ends of the beams,

transverse diaphragms between the beams, thus enclosing the joints, and finally the remaining short section of deck over the pier. This method of construction allows the beams in each span to act as simply supported beams in carrying their own weight and much of the weight of the deck, while the deck slab provides continuity for supporting the live load.

The carriageway is 24 ft wide between kerbs 3 ft wide which serve as pedestrian walkways. The handrails are of welded tubular steel. The cross-section of the bridge is shown in Fig. 2.

Specified concrete strengths were 6,000 lb/sq. in. at 28 days for the prestressed concrete and 3,000 lb/sq. in. for all other concrete.

Construction

Construction of the bridge was carried out by contract and supervised by the Department of Main Roads. Basic facilities provided by the contractor were a site batching and mixing plant on the eastern bank with casting beds

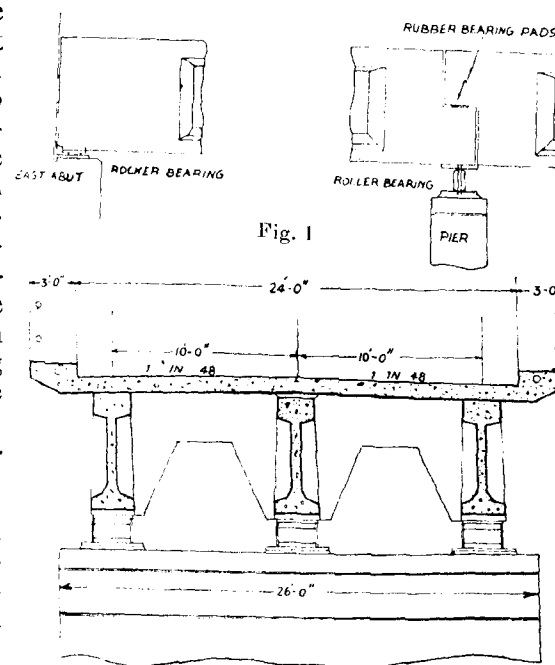


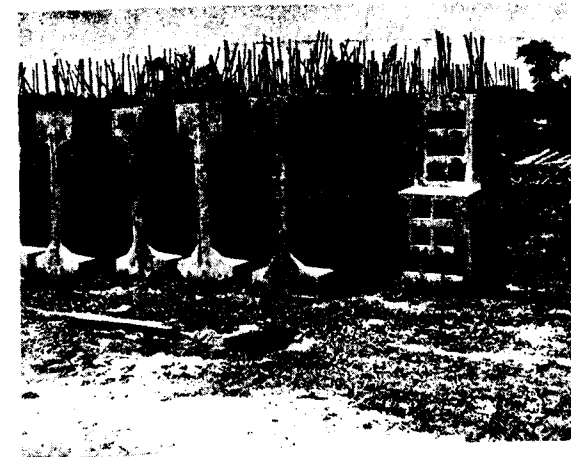
Fig. 2

for precast concrete sections, a steel towered cableway spanning the river, and two steel erection trusses used in placing the beams in position.

Coffer dams of steel sheet piling or of sand bags were used during excavation for the foundations and casting *in situ* of the pier plinths. Precast pier segments were carried from the casting bed to the cableway by mobile crane and the cableway was then used to lower the segments into position on the piers. *In situ* concrete for the piers was also carried in hoppers on the cableway.

Temporary rails were bolted to the edges of completed piers to carry the erection trusses. Each truss was designed to be supported at the top chord on the rails, and sideways motion was achieved using hydraulic jacks. Incorporated in the truss were hydraulic jacks for raising and lowering the completed beam and a screw jack to move the beam longitudinally.

The beams were cast in segments weighing up to 5 tons each. Ducts for the stressing cables, were formed by inflatable rubber tubing. Following curing, the beam segments were transported by cableway and assembled on the steel erection truss placed between



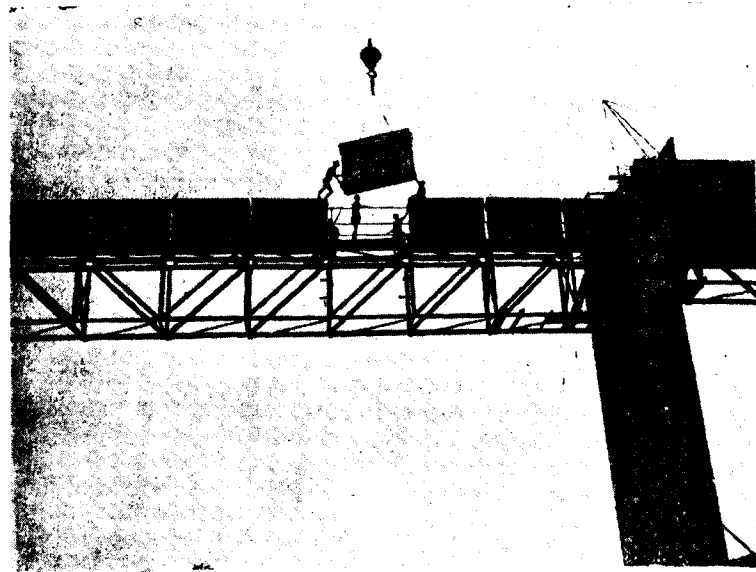
Construction of bridge over the Mann River at Jackadgery—Girder sections and precast endblocks at right

the two piers. Joints approximately 3 in. thick between the segments were poured, again using inflatable tubes to form the cable ducts, and the high tensile steel cables were then drawn through. Concrete samples from the joint concrete were tested after three days and, subject to their strength exceeding the specified minimum of 6,000 lb/sq. in., stressing was authorised.

The number of stressing cables in each 105 ft outer beam was 11 and 13 cables in each inner beam. Specified net extension for each cable in these beams was 6.95". Standard Freyssinet jacks were used with concrete anchorage cones.

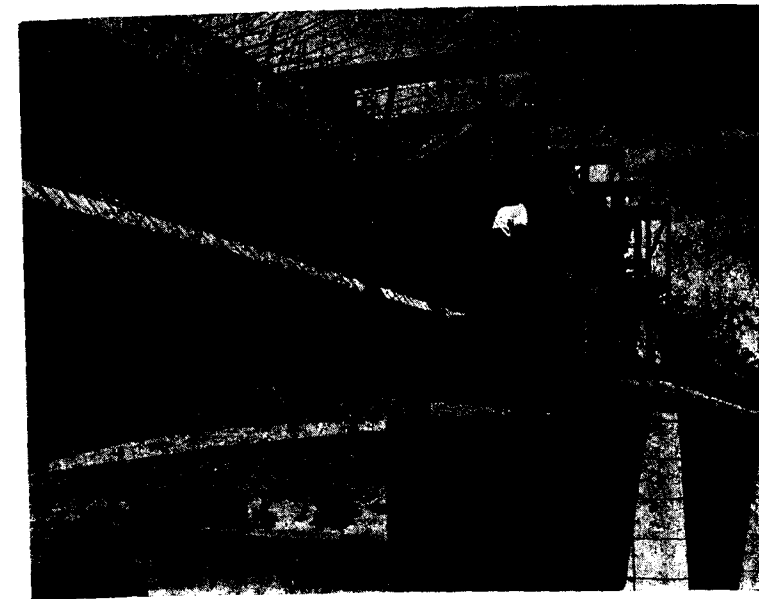
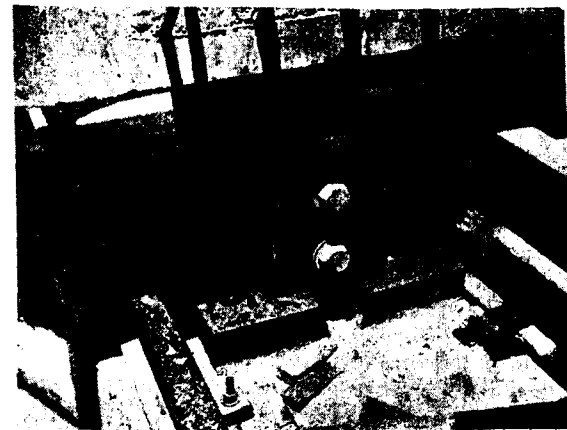
After stressing, the cable ducts were filled with cement grout by pumping from one end, a small quantity of aluminium powder having been added to counteract shrinkage. The end cavities were then filled with concrete and the beam was manoeuvred into position and lowered into its final position, using the inbuilt truss jacks. The erection truss was then disengaged and moved sideways into position for another beam, or, when all three beams were in place, moved to a position between the beams, when the rear end was lifted by the cableway and placed on a skid frame resting on the adjacent completed beams. The leading end of the truss was then lifted and a hand winch used to move the truss forward into the next span. Temporary steel bracing frames were attached to the completed beams pending deck casting.

The designers specified a precise casting order for *in situ* deck concrete. In any two spans the central section of deck 81 ft long and 21 ft wide in each span had to be cast before the intervening length of deck and the transverse diaphragm over the pier. The addition of the cantilevered sections of the deck and the walkway kerbs, beyond the outer beams, was carried out at a lagging distance of almost two spans. A bonding compound was applied to the longitudinal construction joint between the deck and the cantilever immediately



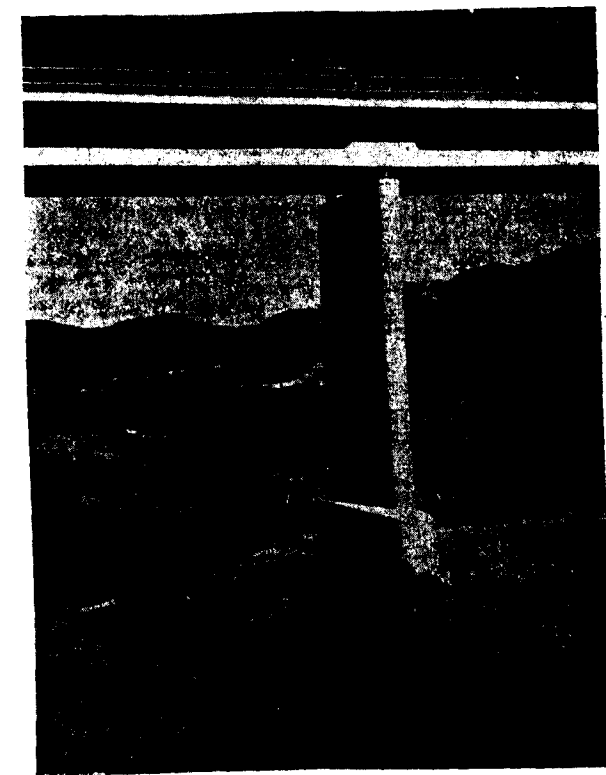
Construction of
the bridge over
the Mann River
at Jackadgery

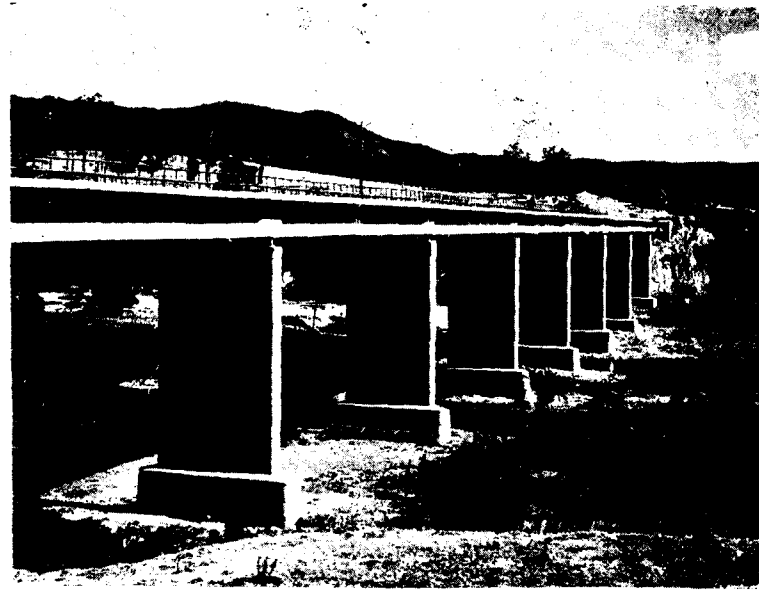
Construction of bridge over the
Mann River at Jackadgery—
Expansion rocker in position
under girder



Construction of the
bridge over the
Mann River at
Jackadgery—Erec-
tion of formwork
for cantilever
section of Deck
slab

Bridge over the Mann river
at Jackadgery





Construction of Bridge over Mann river at Jackadgery

prior to placing the cantilever concrete, to reduce the risk of water penetrating the joint.

The forms used in the work were of steel except for some non-repetitive sections where timber was used. Concrete aggregates were obtained in part from a commercial source at Grafton and partly from local river deposits of sand and gravel, the latter being passed through a small portable crusher.

During construction, concrete consistency was checked using the slump test for the 3,000 lb/sq. in. concrete. For the drier prestressed concrete, Compac-

tion Factor apparatus was used. Field strength tests were carried out at the Department's Divisional Testing Laboratory at Grafton.

The contractor for the construction was Thiess Bros. Pty. Ltd., the amount of the accepted tender being 226,114 pounds. The construction of the bridge occupied slightly less than two years.

Supervision of the work was the responsibility of the Department of Main Roads Divisional Engineer, Grafton, who at the start of the work was Mr. R. J. S. Thomas and later Mr. W. J. C. Orr.

CURRENT TREND OF THE RESEARCH IN THE FIELD OF DESIGN OF CARRIAGEWAY IN FRANCE

By

J. BONITZER *

WHEN the French Engineers build roads, they use in very general manner the empirical methods elaborated by the American before and during the last World War. These empirical methods, viz., C.B.R. and method of group index, furnish results which even if summarily valid, do not permit a precise design of the structure of the roads¹. When both the two different methods are applied simultaneously, there is significant difference in results. Besides, any one of the methods though more or less satisfactory, does not permit taking into account the effects of rigidity that would exist for example in layers of soil-cement or relatively thick layers of bituminous concrete. The inadequacy of these empirical methods are well-known for a long time. In France, as well as in most of the advanced countries, efforts are being made to replace these methods by methods more modern and satisfactory. With regard to these, genuine progress has already been made in a manner to permit the use of new methods of designs for road pavement projects. These significant advances, have upto now been in two directions, viz., (1) mathematical reckonings of mechanics of structures and (2) methods of investigation of these structures with vibrations. In spite of certain partial results of special interest, these recent efforts do not yet permit formulation of

really satisfactory methods. In continuation of the work of JEUFFROY AND BACHELEZ² the French Engineers have for few years made thorough critical investigation aimed at resolving the existing impediments which so far prevented the working out of practically useful methods.

To begin with, we shall give briefly the conclusions of those critical investigations relating to the design, and later on the experimental methods.

Methods of Design

The investigations made upto present have shown very large similarity in essential characteristics and differ only in points of details. These were similar to investigations by BURMISTER in 1941—1943. The main assumptions are: the layers of the pavement are supposed to be constituted of materials that are elastic, homogeneous and isotropic, and the loads are static. The numerical calculations have been limited to cases where the soil (represented by a semi-indefinite mass) supporting a single layer (this case is known under the term "twin layer") or two layers (case of "three layers"). Except FOX, who has made use of finite differences, all the other authors have, for their calculations used the mathematical methods of stress functions and semipolar co-ordinates

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¹ It is to be pointed out that the method of group index is generally not correctly used in France, where the engineers wrongly neglect the findings of Steele which permit taking into consideration to certain degree the rigidity of the upper layers.

² The methodical explanation of the tests and experience of Washo (Revue Generale des Routes-September, and December 1957)

adopted by BURMISTER. Finally by way of details still not unimportant, all the computations have been made for a value of 0.5 (exceptionally 0.25) for the Poisson's co-efficient. Stresses and deformations have been calculated only at the intersection of the axis of load and the interfaces of the layers (only Jeuffroy and Bachelez have made the calculations for the other points, for investigating the case of twin tyres).

These investigations suffer from the following inadequacies:

(1) The hypotheses relating to the conditions of rupture of the pavements are very inadequately defined. JEUFFROY and BACHELEZ, for example, have admitted that the rupture could arise from two causes only: either that the vertical pressure on the soil is excessive or that the tensile stress on a plane axial to the base of pavement is excessive. It appears that this conception is particularly deficient on two points:

(a) Due to fact that the pavement shows visco-elastic-plastic characteristics, its rupture depends on the pace of loading during the time of its application in a manner which is still not clear. It seems doubtful that the proposition by the engineers of Shell Company of substituting the maximum tensile deformation to maximum tensile stress permits satisfactory resolution of the problem and it appears that more research about the rheology of bituminous mixtures is necessary.

(b) Question more important: it is not sure that the rupture of a pavement can be due only to causes that have been envisaged. In particular, observations and reasoning give food to the thought that many pavements could be subject to deterioration with origin in

the upper part of the base layer, which may be subjected to high shearing stresses either outside of the axis of the load or at some distance underneath the pavement—base layer interface, where tensile stresses in a noncoherent medium could be observed. Referring to Fig. 1, it could be said that besides the critical regions I and IV, generally considered as origins of possible ruptures, it would be useful to investigate what happens in the regions II and III. The ruptures of non-coherent base could be at first unsuspected, whereas they lead to intense attrition which can cause alteration of the physical and mechanical qualities of the stratum constituting the base layer. The rupture observed on the pavement could thus be a result of an invisible rupture in the base layer and not initially due to the deterioration of the surface.

These possibilities show that it would be of interest to explore several points which have been omitted upto the present.

(b-i) It is necessary to get an idea of the real values of the Poisson's coefficients. They have, indeed, a considerable influence on the stresses in the base layer, especially in the vicinity of the interface. The assumption of $\nu=0.5$ could lead to grossly erroneous results³. It is necessary to examine if the assumption of a static load is not unduly optimistic does not one hazard to observe, in the case of a dynamic load, concentrations of stress at the vicinity of the interfaces due to the partial reflections of the elastic waves?

(b-ii) It is necessary to examine the stresses outside of the axis of the load (in region III).

On the other hand one could adopt, at least in a first step of analysis, a condi-

³An excess value of the Poisson's coefficient of the base layer leads to high values of shear at the interface, high values of the perpendicular compressive stress on a plane vertical to the axis of the load, and low values of normal tensile stress in the same plane. This consequently leads to conclusions which are too pessimistic in the critical region II and too optimistic in the critical region III. It has only a slight influence in the regions I and IV.

tion of rupture in the base layer deduced according to the theory of Mohr's circle and the intrinsic contour curve.

(2) The methods of computations adopted upto the present refer to a period wherein there was no possibility of automatic computations. As these procedures lend themselves badly to automatic calculations and due to the fact that with hypotheses more complete than those adopted by Burmister (dynamic load presence of visco-elasticity in certain layers of the pavement) still more complicated computation are to be expected, taking in account of such hypotheses has been uptill now postponed. For the same reason the number of layers of the road pavements under investigation had been limited.

We are convinced that the progress in the mathematical methods of resolution, of partial derivative equations linked with use of the big digital computers justifies reconsideration of these questions. A certain number of new methods are being investigated in France with the object of reducing calculating time of each particular case, and for taking into consideration visco-elasticity and dynamic loads, as well as eventually structures which are finite in the horizontal direction, and the case of unsymmetrical loads.

Computations in the case of dynamic loads would be justified for not only because, it is far from being reliable, that static loading computations furnish even proper approximation but also because techniques of non-destructive pavement testing (investigation with vibrations) go more towards utilisation of dynamic methods.

Even if we succeed so far, a very serious difficulty would still exist due to the fact that the problem depends on a large number of parameters which do not permit their explicit consideration. We do not know yet how it would be possible to manage it.

(3) Till now the computations have been essentially in a "direct" sense, i.e., the stresses and deformation in relation to geometrical and mechanical parameters of the problem have been sought after. It is known that in the choice of elastic (or visco-elastic) parameters of the layers it is at the risk of making it difficult, due to the inadequacy of laboratory test methods. It would be of great interest to attempt to deduce these elastic parameters from a testing of actual or experimental pavements. Thus one is led to pose an "inverse" problem in which the question relates to the computation of the elastic parameters in reference to certain deformations. The first attempt in this direction has been made to our knowledge, by Jeuffroy and Bachelez, who deduced values of the elastic moduli of the layers from deflection observed when applying a static load on circular surfaces with varying diameters. These tests, due to the fact that they have been resorted to with static load, do not permit the computation of the characteristics of the varying course. On the other hand, they do not give any information on the Poisson's coefficients, which has very little influence on the deflections. But one could envisage following Jones, of the British Road Research Laboratory proceeding from values of wavelength of Rayleigh waves or those of the mechanical impedance of the pavement, varying when applying sinusoidal load according to the frequency.

This "inverse" problem is not less arduous than the direct one and investigations made up to now imply considerable simplification. We shall seek to deal with the inverse problem only when more advance will have been made about the direct one.

The investigations explained above, alone would permit of verifying, without doubt, upto which point the hypotheses of linearity and isotropicity are sufficient to give an answer to the characteristics of the layers of the pavement.

EXPERIMENTAL METHODS

A. Laboratory Methods

A number of laboratory methods for the investigation of the properties of *bituminous coatings* are in hand. For investigation of their visco-elastic properties, methods of vibrations have to be resorted to enable computation from rheological models. Nevertheless, the use of these methods has certain difficulties which are far from being solved, and which would have to be handled more systematically than in the past. These difficulties relate to the choice of the modes of vibration, fabrication of the test pieces whose composition has to be identical for all vibration tests to be performed, to the role to be played in test involving inevitable anisotropy and the mode of transmission of the stresses, which should avoid the flow of the test piece, to the kind of thermostatic regulation, etc.

For investigation of fatigue of coatings, an apparatus capable of applying sinusoidal flexural stresses on a test piece in the form of a prism with trapezoidal section set up by the Shell Laboratory, appears to be more to the point. We would like nevertheless, to be able to apply on the test pieces repeated stresses separated by a time of rest to permit in a way the relaxation of the material. If it is possible, it would be of interest to investigate the fatigue in the case of various deformations other than flexural.

However, we are now not badly equipped to progress the knowledge and methods of tests on bituminous coatings of the pavements.

It is the same with regard to materials with relatively high cohesion, *such as soil cement*. On the contrary, there are very serious difficulties in the case of non-cohesive materials. These materials, used in bases and sub-bases, have upto now been investigated in connection with the problems of foundation design. From this point of view, the bearing

capacity under static loads alone is of any importance, while for the design of road pavements bearing capacity under rapidly varying loads is most important. Whereas in the design of foundations, the conditions of plastic rupture alone plays a part, for the design of road structures, the elastic-bearing capacity, in the region between upper and lower equilibrium limits must be known. It is necessary to say that from this point of view, at present we are entirely lacking in reliable laboratory tests. The only practical tests are triaxial tests: the materials are subjected to very slow varying stresses, and the results vary considerably according to the test procedure because of the interference of the phenomena of elasticity with phenomena of adaptation. From these tests it is difficult to calculate anything similar to Poisson's coefficient even with little precision.

The investigation of the phenomenon of fatigue of soils (fragility, attrition, physico-chemical decomposition linked with the presence of water, and interaction of all these effects) appears to present less basic difficulties. Nonetheless, it is only beginning and it is difficult to say when it will bear the fruits.

B. Experiments at site

The deficiencies in the means of laboratory investigations, especially, with non-cohesive soils or those with little cohesion gives great importance to the methods of experiment at site.

These require the application of certain loads to the pavements and measurement of the displacement, strain or stresses caused.

(a) Purely static loads are applied. The measurement of deflection is made mostly under the wheels of loaded trucks. The apparatus utilises loaded circular plates. M. CLAEYSEN has used hydraulic pads to get as well as possible, uniform distribution of

the vertical load and eliminate horizontal stresses caused by friction: the pads were used with two types of loaded trailers. The use of trucks or static loading apparatus has the drawback, giving practically no data about the role played by the pavements and generally speaking rather poor information.

One is therefore interested in apparatuses for dynamic loading, which can be ranked into two categories: generators of sinusoidal vibrations and impulse generators. In the field of sinusoidal vibration generators, the main investigations have been made in Holland, by NIJBOER and HEUKELOM (Shell Company) who have set up a heavy vibrator of very low frequency (6 to 60 hertz) and a medium frequency vibrator (upto 10,000 hertz) and in England by JONES (Road Research Laboratory) who has used vibrators with medium and high frequencies (upto 60,000 hertz). We have started using the same means. In particular, it appears to us that the heavy Shell vibrator, whose new prototype is now being set up, is more convenient could offer means of the dynamic testing which could be used on a large scale for routine requirements than theoretical studies.

On the other hand we are now investigating an impulse generator utilising the drop of a ball on a system of springs.

(b) The characteristics measured could be displacements, the strains or stresses.

The measurement of the stresses has been tried for some years by means of rigid cells of rather large dimensions.

We are at present rather skeptical as to the capacity of such cells which could only be utilised rather in a large depth, disturb not only the stress distribution, but also the plastic consolidation of the soils, and permit, only measurement of stresses perpendicular

to a horizontal plane. The hydrostatic cell that has been envisaged seem to come up against similar objections.

We have not attempted to measure strains though the use of strains gauges, flat and not very cumbersome, do escape some of the objections which have been made against the pressure cells.

The only measurements utilised are the displacement ones (or at times in the case of dynamic loads, measurements of speed or accelerations of the displacements).

The most primitive measuring device is the Benkelman beam only applicable to static loading cases, of which there are forty or fifty in France. Another more perfect instrument also very much in general use, is the JOBIN and Yvon-CLAEYSEN, optical deflectometer, used with static as well as slowly moving loads. This apparatus gives the maximum deflection at the centre of the loading or at the points nearby.

One would therefore like to have at his disposal apparatus permitting on one hand, the recording of the vertical displacements under the dynamic loads and on the other hand investigate the surface of the pavement under a more or less extended zone. For this purpose frames to serve as bases of reference, resting on supports not influenced by the loads, have been constructed and these frames support about ten electro-magnetic feelers whose readings are recorded. Such a system has been used with ball deflectometers as well as with trailers for static loading. On the other hand at present, investigations are in hand on an optical apparatus with a movie recorder for recording the vertical displacements of a luminous source placed on the surface of the pavement.

In the case of vibrating loads, one can work mainly in two distinct ways.

On one hand one measures the mechanical impedance of the pavement when the frequency varies. This is what has been done with Shell Machines. One is equally interested on the surface waves and measurement of their wave length. In certain range of frequencies, the accelerometers which have many interesting peculiarities could be utilised. It permits the dispensing with of reference to bases and can record not only vertical accelerations but also horizontal ones.

These measurements are concerned practically with the surface phenomena, although theoretically one can in certain cases explore what happens at depths. Due to the fact that one can make use of supplementary parameters, for example, the frequency of

vibration, when one resorts to sinusoidal vibration generators, and for this reason one can also measure not only a single magnitude but a functional series, the information thus obtained is much more valuable. We cannot still say what exactly would be involved in this information and whether it would permit particularly the still difficult problem of determination of the parameters such as the Poisson's coefficients of the base layers to be solved.

[The above has been very kindly translated from French to English by Shri S. K. Rajagopalan, Divisional Engineer (Consultant), Government of India, Ministry of Transport & Communications, Department of Transport, Roads Wing, New Delhi.]

(Continued from page 124)

It was then pulled towards the river until transferred to the bed trolley and was next moved on rails laid on the river bed upto the opening, over which it is to be placed. Finally it was pushed sideways over the pier caps, till it could be placed exactly over the bearings. The operation of launching heavy girders, the heaviest weighing about 135 tons without any incident, was possible only because of a really good organisation at site.

The bridge deck provides a 24 ft wide roadway, with wheel guards and

railings on either side. It is designed to carry two lanes of Class A or a single lane of Class AA loading of I.R.C.

In designing the superstructure, all the criteria prescribed under Bridge Codes, including those for the Prestressed concrete had to be satisfied. As the aim was to reduce the weight of the structure, it was possible to cut down the quantity of cement used in superstructure by about 30% and of steel materials by about 90%. Of course, the cost of the deck structure, including the imported materials used, could not be made any cheaper in this job in view of the circumstances prevailing.

ESTIMATION OF SUBGRADE MOISTURE DISTRIBUTION

By

K. RUSSAM, B.SC., A. INST. P., A.M. INST. H.E. AND D. CRONEY, B.SC.*

Summary

[Research into soil moisture has been conducted at the Road Research Laboratory in Great Britain for a number of years, and methods have been developed for estimating the ultimate soil moisture distribution under impervious pavements. This paper shows that the main factor controlling the moisture distribution is the position of the water-table when it is close to the surface but that the local climate becomes increasingly important as the depth of the water-table increases.]

Introduction

THE natural stability of soil depends largely on the water present between the particles and on their surface. Local climate is therefore a major factor in determining the engineering problems which any particular soil will present. For this reason soils of predominantly the same type are very differently regarded in different parts of the world. In relation to the design of road pavements the engineer is generally interested in the long-term stability of subgrades; in particular he needs to know whether the natural soil is likely to become wetter when the pavement has been laid and how any such tendency can be taken into account at the design stage.

This is a problem largely in the field of thermodynamics. It is now recognised that the factor which causes water to move in soil is a gradient of hydrostatic pressure (irrespective of whether the actual pressures in the soil water are greater or less than atmospheric pressure). Differences in moisture content, expressed on a weight basis, provide little information on the direction of flow of soil water, since movements may take place from regions of low moisture

content to regions of higher moisture content if such factors as difference of soil type or state of compaction or disturbance are present. It is for this reason that special consideration has been given at the Road Research Laboratory¹ and elsewhere² to the factors which govern the hydrostatic pressures in soil water.

Soil moisture

At any point in a mass of soil the actual measurable pressure of the water in the soil pores (pore water pressure) is determined by two factors, the pressure exerted by the surrounding soil, and the stress-free suction of the soil arising primarily from surface tension and associated absorption effects. Thus in any thin stratum of a semi-infinite mass of soil the suction, s , the pore water pressure, u , and the vertical pressure, P , due to overburden are related by the equation:

$$u = a P + s \quad \dots (1)$$

where a , the compressibility factor, is the fraction of the normal pressure effective in changing the pore water pressure. Since values of s and u are normally expressed from a zero corre-

*Road Research Laboratory, Department of Scientific and Industrial Research, United Kingdom.

sponding to atmospheric pressure, s in equation (1) is always negative and αP positive. Thus if s is numerically greater than αP , u is negative, the condition generally prevailing above a water-table, and if s is numerically less than αP , u is positive, the condition prevailing below the water-table. The value of the suction, s , can be measured very simply from the energy required to extract water from a small unloaded sample of the soil, and α from the effect on this energy of an externally applied pressure. Equipment for making these tests has been developed at the Road Research Laboratory and is described elsewhere.¹

Again, techniques have been developed for studying the relation between suction and moisture content² and

equation 1 can be used for expressing any measured pore water pressure in terms of an equivalent moisture content provided that the appropriate values of α and P are known.

Beneath an infinite impervious surfacing, and in the absence of temperature gradients the ultimate pore water pressure at any point in the soil is determined by the level of the water-table, being less than atmospheric pressure by an amount equal to x feet of water at a point x feet above the water-table, and greater than atmospheric pressure by the same amount at a point x feet below the water-table. In practice, a ground water-table existing close to the surface dominates the soil moisture regime under wide impervious pavements and, in these circumstances, equation (1) can be used

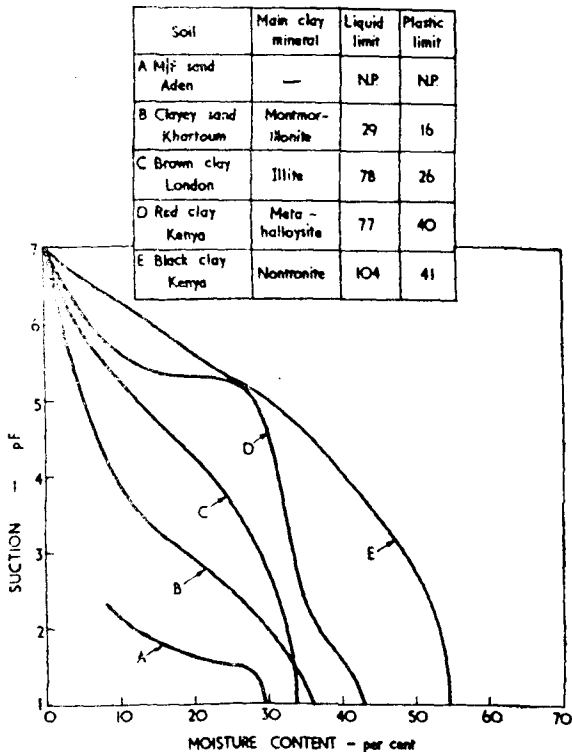


Fig. 1. Relation between suction and moisture content for various soils drying from saturation

to determine the equilibrium distribution of suction and moisture content with depth. If of course the soil is stratified the appropriate relations between suction and moisture content for the different strata must be used in estimating the moisture content distribution. Typical relations between the suction and moisture content for various types of soil are given in Fig. 1. The value of the compressibility factor, α , may vary with the moisture content but a useful guide to its value may be obtained from the plasticity characteristics of the soil as follows:

Plasticity index < 5 ; $\alpha = 0$
Plasticity index > 40 ; $\alpha = 1$
 $5 \leq \text{Plasticity index} \leq 40$; $\alpha = 0.00275 \text{ P.I.} - 0.125$

Although temperature gradients give rise to differences to vapour pressure likely to modify the moisture distribution with depth, investigations suggest that this effect is not large under most practical conditions. In applying the theory to the practical case of the road or airfield pavement a more important factor is the limited size of the impervious area provided by the pavement. Because of this the theoretical moisture distribution based on the position of the water-table is likely to be influenced by 'edge-effects' which will depend largely on the moisture content in the adjacent uncovered soil and hence on climate.

Soil moisture and climate

The moisture condition of exposed soil depends mainly on the relative amounts of rainfall and evapotranspiration. When rainfall exceeds the evapotranspiration, water is available for storage within the root zone of plants and trees and some subsoil drainage and runoff may also occur. The change in the number of inches of water stored in an exposed soil profile is equal to $R - E - D$, where R is the rainfall in inches, E the actual evapotranspiration

in inches, and D the drainage in inches, all occurring in a given time.

When water is not draining through the soil the moisture balance may be referred to the moisture holding or field capacity, C , expressed in inches of water, of the soil profile. Here the field capacity, C , is defined as the amount of water held in the soil after the excess gravitational water has drained away and after the rate of downward movement has materially increased.⁴ Thus, in this case, the number of inches of water stored in the soil profile is $C - E + R$.

The actual evaporation is difficult to estimate. This can in part be overcome by replacing E by E_p , the potential evapotranspiration, defined as that evapotranspiration which would take place in a given climate if the soil and vegetation were provided with an unlimited supply of freely-available water. The amount of freely-available water in the soil at field capacity will depend on the depth and texture of the soil and on the distribution and extension of the root systems. In practice, between 2 and 8 inches of water, depending on site conditions, is available in the soil profile at field capacity but for most soils with normal vegetation the mean appears to be about 4 in. of water.⁵ The amount of available water, M , contained in the soil profile is then given by:

$M = 4 + E_p - R.$

When the readily-available water has been evaporated from the soil profile a soil moisture deficit, d , occurs. Since in dry soils the potential evapotranspiration will exceed the actual evapotranspiration by an unknown amount, d has a qualitative significance only.

Many formulae have been proposed for the calculation of E_p from meteorological data such as those due to Penman⁶, Thornthwaite⁶ etc. The normal weather data can thus be used to give the rainfall and potential evapo-

transpiration each month throughout the year and the soil moisture balance constructed as shown in Fig. 2. A measure of the overall availability of

Calculation of the moisture balance, Madras, India. (inches of water)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.
Potential evapotranspiration	3.4	3.8	5.8	6.7	7.7	7.4	7.2	6.9	6.4	5.9	4.3	3.6
Rainfall	1.4	0.2	0.3	0.6	1.0	1.9	3.6	4.6	4.7	12.0	14.0	5.5
Storage change	2.0	2.0	0	0	0	0	0	0	0	4.0	0	0
* Storage	2.0	0	0	0	0	0	0	0	0	4.0	4.0	4.0
Actual evaporation of available water	3.4	2.4	0.3	0.6	1.0	1.9	3.6	4.6	4.7	5.9	4.3	3.6
Water deficiency	0	1.4	5.5	6.1	6.7	5.5	3.6	2.3	1.7	0	0	0
Water surplus	0	0	0	0	0	0	0	0	0	2.1	9.7	1.9

* Storage is calculated at the end of the month and is continuous from December of the previous year

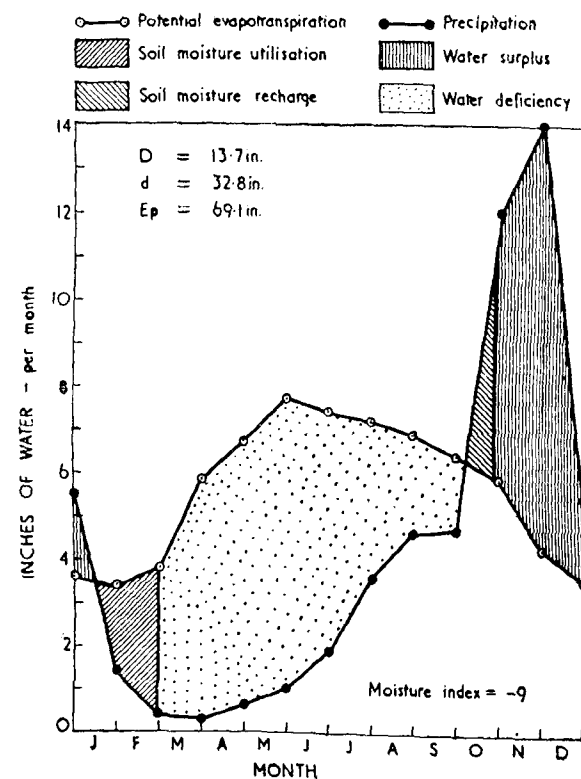


Fig. 2. Calculation of the moisture balance from the monthly rainfall and potential evapotranspiration (after thornthwaite)

moisture throughout the year can conveniently be obtained from the Thornthwaite Moisture Index⁶ defined as:

$$\text{Moisture Index} = \frac{100.D - 60.d}{E_p}$$

where E_p , D , and d have the meanings previously ascribed to them, but taken over an annual period.

Three climatic categories can conveniently be distinguished: Dry ($D=0$ always), Wet ($d=0$ always) and Intermediate ($d=0$ for some months, $D=0$ for other months).

Subgrades in regions with a wet climate

The heavy rainfall on sites in this category will keep the soil wet and little change of volume, moisture content or strength, will occur with season. The surplus water draining through the soil will normally form a water-table, the depth of which will depend on the permeability of the soil strata. The moisture distribution under a sealed road or airfield pavement can therefore be estimated using the approach outlined above. Wetter conditions than the estimated distribution might be expected to arise at the edge of the pavement. Monthly measurements at two sites in this category, Tengah Airfield, Singapore (during 1951-55) and Kai Tak Airfield, Hong Kong (during 1952-53), showed, however, that the moisture conditions under the pavement became slightly drier near the pavement edge. The climate at both airfields is tropical monsoon. It would appear therefore that evaporation from the exposed soil together with the normal drainage measures can more than counteract the effect of heavy rainfall and surface run-off from road and airfield pavements. The subgrades at both air fields, at Tengah a silty clay and at Kai Tak a clayey sand, were derived from decomposed granite of very variable properties. Laboratory suction tests carried out on samples of the soils showed that the moisture contents observed under the central part of

the pavement were consistent with the position of the water-table which varied between depths of 1 and 2 ft at Tengah and 2 and 8 ft at Kai Tak.

Subgrades in regions with a dry climate

An arid climate where the monthly rainfall is less than the evapotranspiration gives rise to desert or semi-desert conditions. The conditions at Abu Sueir Airfield, near Ismailia, Egypt, falls within the dry category and monthly measurements of soil moisture content were carried out at this airfield during 1953-54. No watertable was located in the medium-fine sand. The tests showed that the moisture content of the top few feet of the exposed soil and of the subgrade under the pavement was of the order of 3 to 5 per cent. The annual rainfall was less than one inch and was too small to have more than a temporary effect on the exposed surface soil. Laboratory tests on soil samples from Abu Sueir showed that the generally prevailing moisture conditions corresponded to a soil suction of about pF 5.8, which is consistent with the soil moisture content being controlled by the average humidity of the atmosphere. At such high suctions the effect of the overburden pressure on the moisture distribution may be neglected and in these circumstances the moisture content of the subgrade can be taken as the moisture content of a sample of air-dry soil.

Even in desert regions water-tables can exist relatively close to the surface of the soil and these may affect the soil moisture distribution. The water-table in an arid region is usually due to a nearby river or the sea. Underground seepage of water also occurs and at times this water may be within a few feet of the surface in what may appear to be a completely waterless region. In the latter circumstances, the water-table may again become the main source of subgrade moisture under a pavement. Thus at Khormaksar, Aden, where a water-table is maintained in the medium/fine sand at a depth of 2-3 ft by the

adjacent sea, the measured moisture contents were found to be similar to those in a column of the sand which was allowed to reach moisture equilibrium with a water-table. The average of the monthly moisture content determina-

tions carried out at Khormaksar during 1953—55, when the annual rainfall was about 3 inches, is given in Table I and this shows the extent of the drying out in the sand near the edge of the pavement due to the arid climate.

TABLE I
Average of monthly soil moisture contents—Khormaksar Aden
(April, 1953—February, 1955)

Depth	Moisture content under bare soil	Moisture content under pavement		
		3 ft from edge	10 ft from edge	25 ft from edge
6 in.—1 ft 6 in.	14.2	13.9	15.4	15.5
1 ft 6 in.—2 ft 6 in.	20.8	21.7	22.2	21.8
2 ft 6 in.—3 ft 6 in.	27.6	28.0	28.1	27.8

The rate of movement of moisture through unsaturated soil is extremely slow as compared with saturated or nearly saturated soil and as the depth to the water-table is increased it is possible that subgrade moisture may be more freely available from run-off at the edge of the pavement than from the water-table. The availability of this water will affect the rate at which the ultimate moisture distribution is approached. An example of these circumstances was found at Khartoum Airfield, Sudan, where a water-table varied in depth between 23—32 ft. Monthly measurements taken over a period of several years showed that the moisture contents in the top 2-3 ft of exposed clayey sand soil varied between 3 and 7 per cent according to season. It was therefore likely that the moisture content of the top foot of subgrade was within this range when construction of the 50 ft wide taxiway was completed in early 1952. The changes in the moisture distribution which then took place in

the top foot of the subgrade under the taxiway are given in Fig. 3.

Although the annual rainfall at Khartoum is only about 8 inches this water when concentrated, by surface run-off, at the edges of the pavement soon resulted in wet conditions in the subgrade in this position. The rapid wetting of the soil during the rains was followed by a very slow drying of the subgrade during the following dry period, and little subgrade moisture appeared to be lost before the onset of the next rainy period. The resulting moisture distribution six years after construction was such that the subgrade was much wetter than the comparable unpaved soil and became wetter with increasing distance from the edge of the pavement. Although the arid climate produced the drier subgrade near the edge of the pavement, the moisture condition measured under the centre part of the taxiway in 1958

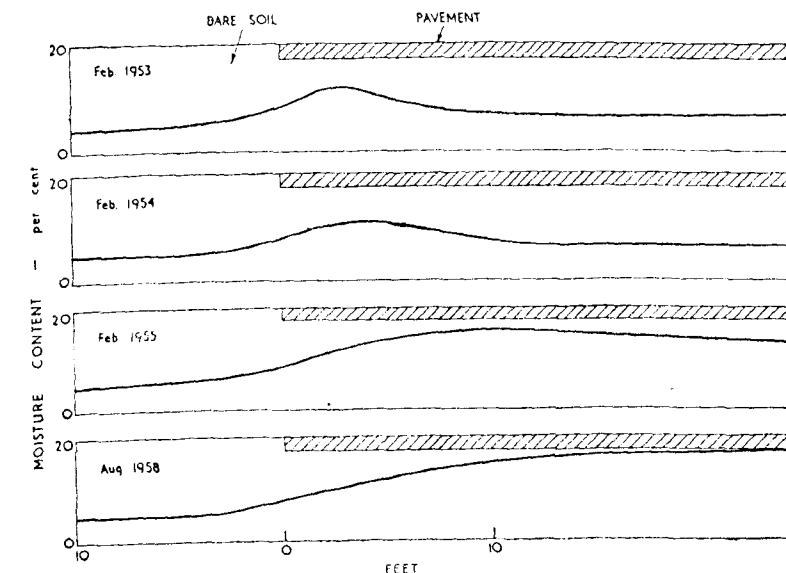


Fig. 3. Moisture in clayey sand subgrade, Khartoum

approximated to the estimated moisture content for the soil in equilibrium with the water-table.

Subgrades in regions with an intermediate climate

A shallow water-table also dominates the climatic factors and has a major influence on the moisture distribution beneath pavements in this climatic category. Where the water-table is deep the factor controlling moisture conditions under a sealed pavement is the moisture available at the edges. Seasonal rainfall may produce large variations in moisture conditions of subgrades within 2-3 ft of the verge. The relatively stable moisture regime existing near the central part of the pavement will then be affected by the comparatively small amounts of water entering and leaving the road subgrade during the wet and dry seasons. Thus the availability of water at the edges of the pavement will not only affect the rate at which the moisture equilibrium is approached but also the general soil moisture regime in the subgrade. A climatic classification of the water balance

between rainfall and evapotranspiration may therefore be related to the moisture conditions beneath road pavements, though other variables will be the shape and type of construction of the road and verge, and the soil permeability and local topography.

To examine this possibility an investigation is now being carried out to determine the moisture conditions under roads in East Africa and Nigeria which have been sealed with a bituminous surfacing for at least five years. The results to date suggest that, in the absence of a shallow water-table, the subgrade is unlikely to become wetter than field capacity (Fig. 4). At field capacity, published figures^{4,7} suggest the suction of the soil is in the range pF 1.6 to pF 3.2, the lower figure corresponding to loose sands and the higher figure to heavy clays. To give rise to wetter conditions in the road subgrade it would appear therefore that a water-table should be within about 2 ft of the top of the subgrade in the case of loose sands and at greater depths for the heavier soil types down to a limiting value of about 50 ft for very heavy clays.

The range of soil moisture conditions found under roads in a given climatic area has emphasized the importance of a knowledge of the unsaturated permeability of soil to water. Laboratory research into the unsaturated permeability of soils together with the method of flow-net construction may provide the means of obtaining an accurate prediction of subgrade moisture distribution in this climatic category; this approach is also being studied at the Road Research Laboratory. At the present, however, the semi-empirical approach outlined above may prove a useful guide to the road and airfield engineer.

laboratories field evidence is scanty and, as mentioned earlier, moisture migration due to thermal effects does not appear to be a general problem to the road engineer. Measurements at three airfields in Rhodesia have provided some data on these phenomena.⁸ The airfields were situated on unsaturated sandy or silty clays and the increase in moisture content of the subgrade took place during the cool season immediately following the rains. The build up of moisture approximated to that found when a temporary water-table was formed during the wet season. It therefore appears that the level of the temporary water-table could be used to

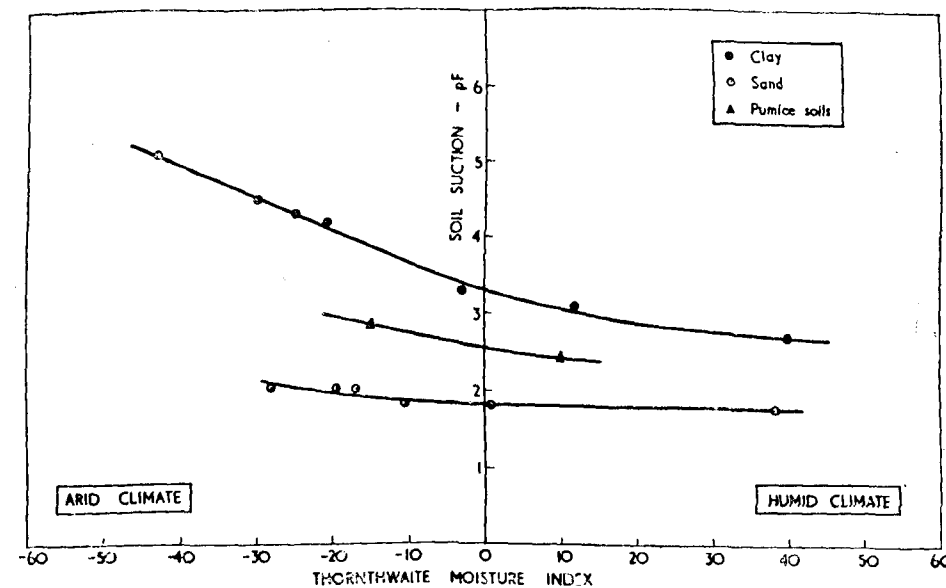


Fig. 4. Variation of suction of soil samples from roads with Thornthwaite moisture Index

Thermal movement of subgrade moisture

Temperature gradients in the soil can cause the migration of water in both the liquid and vapour phases. The movement may be rapid and it is possible that a sudden cooling of the road structure could result in an increase in the soil moisture content at the top of the subgrade. Although these effects have been simulated many times in

estimate the highest subgrade moisture content likely to occur in such circumstances.

Conclusions

Long-term measurements made under roads and airfields in both tropical and sub-tropical countries have shown that under sealed pavements the seasonal variation of soil moisture content is

much reduced but large changes in moisture content may take place over a period of years. The most important change from an engineering point of view usually occurs where the soil has previously been desiccated, the reduction in the rate of evapotranspiration following construction of the pavement permitting the accumulation of moisture in the subgrade. This is clearly a matter which must be taken into account at the design stage of any road or airfield project.

Acknowledgement

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ROAD RESEARCH IN U.P. AND ITS APPLICATION

(By the courtesy of the Chief Engineer, P.W.D., B. & R., U.P., Lucknow)

THE U.P. P.W.D. Research Institute was established in the year 1947 with nucleus staff trained in soil stabilization work and has since been tackling various problems of buildings and roads construction out of which only items of wide application have been chosen for inclusion here.

From the year 1957 onwards, the number of failures of road miles assumed a large proportion and has been increasing in every subsequent year. Investigations were carried out which revealed that in most cases the thickness of road crust was inadequate for the wheel loads passing over the road. This required devising of remedial measures to save the State's roads from major damages. The soil under most of the roads was of A4 and A6 type which had very low resistance to deformation or had very low C.B.R. value under saturated conditions. The thickness of crust required over such subgrades usually worked out from 18 in. to 24 in. whereas the existing thickness of pavement was hardly 9 in. Study of these failures and required thickness of crust brought out the magnitude of the problem and decision was taken to increase the crust thickness to 11 in. This did not solve the problem fully and it became imperative to use some locally available material with higher value of C.B.R. which could economically replace a part of the road crust thickness. The solution lies in lime treatment of soil or providing a sub-base with graded soil. The U.P. chose the latter, it was decided to provide top one foot of embankment of all new roads with graded soil compacted mechanically at optimum moisture content

to give a C.B.R. value exceeding 10. Road crust 11 in. would then suffice over this specially prepared sub-base and the roads are expected to behave well even under worst condition of weather and traffic. The organization and equipment for soil survey work, test and control of 600 miles length of new roads which are to be constructed in third 5-year plan have been arranged.

This decision was backed by experience gained on stabilization of soil on Beni Harauni and Lucknow-Mohan Road. On these roads the base course consisted of 3 in. course stabilized soil with sand content exceeding 50 per cent and P.I. of 5—7. The wearing course consisted of 3 in. thick stabilized soil with a P.I. between 8 to 10 and sand content of more than 30 per cent and mixed with 40 per cent soft aggregate. These were compacted at optimum content to Proctor's density and were topped with 1 in. stone chipping grafting and 2 coats of surface dressing. In some cases kankar and in other brick ballast aggregates of one inch size were used for wearing course. Some control lengths of 6 in. thick water-bound surface have been constructed side by side. These were constructed during last five years and fair amount of experience on these has been gained by now. So far in the miles with light traffic have stood well but lengths where traffic has reached 500 tons or more patches have appeared and signs of distress are exhibited. The reasons for this are being investigated. However, it is manifestly clear that failure by creep occurs in between the grafting and the top course of stabilized soil. The crust gives way at places of exces-

sive creep, a patch is then formed, the soil aggregate mixture of top coat gets pulverised and the patch deepens. Record of the patches occurring and the maintenance cost being incurred is being maintained. The base course in all cases have been found to be more or less intact. Our observations exactly tally with the findings of the I.R.C. on their test-track* which suggest that there should be 3 in. thick coat of waterbound macadam over stabilised soil base course for satisfactory performance of soil stabilised roads.

The problem of finding suitable, cheap and simple specification not requiring too much of technical control of supervision for village roads is another major problem. Low cost roads with stabilized soil require strict control of field laboratory. Village roads do not carry pneumatic tyre traffic. A simple specification lies in providing pavement of layer of flat bricks or brick on edges with high compressive strength bricks, commonly known as engineering bricks over base which may consist of compacted sandy soil. The engineering bricks have compressive strength of 6,000—8,000 lb per sq. in. against 3000 lb in average of 1st class bricks have high flexural

strength, very low water absorption, are highly resistant to abrasion, they do not break under wheel load, and transmit the stresses to the base without being damaged. In due course the base would thus get compacted to a greater extent and develop higher C.B.R. The use of these special engineering bricks has the advantage that crust can be added on to it without dislodging them. The specifications though simple to follow and ideally suited for village roads, involve the problem of production of engineering bricks. Successful experiments on a semi-field scale in Down Draft Kiln at Chinhat, Lucknow, in producing engineering bricks have been made.

Use of Engineering bricks for road construction may not be limited only to rural roads. Layer of Engineering bricks with a premixed black top laid with use of suitable primers have been tried on important provincial highways carrying a traffic of more than 1,500 tons a day with success.

There are several other problems tackled in this State Engineering Laboratory they are still under observations and implementation of findings will have to wait till the results are finalised and advantages are conclusively proved.

* Report published in Indian Roads Congress Research Bulletin No. 7

PRESENT TREND IN THE ART OF BUILDING

HISTORICAL BACKGROUND

By

DR. A. CARBONE*

TODAY there is quite a good amount of research work at the disposal of the Engineer, who practices the Art of Building, which gives him ideas, hypothesis, experimental results in such a rapid evolution that he is often obliged to revalue the real soundness of the assumptions on which is based his project, assumptions chosen by himself or imposed by official specifications, which often are successively contradictory.

Against such a mass of studies the Engineer may ask whether the birth of a new Engineering Static, more simple and exact than the present one is possible in a near future.

We will outline, for clarity, the up-to-date ideas, well-known to all the Technicians, and express some considerations of a general character on the possible results of a more close collaboration between Physicists and Engineers.

It is well-known that the formation of the Theory of Structures (as it is defined the branch which studies the stability of deformable bodies subject to forces that stress them), after the first intuitions of Leonardo da Vinci, the researches of Galileo (first half of the XVII century) and the famous Robert Hooke's Law: "sic tensio utvis" (1676), has taken a development in the last century reaching the solutions of the most complex problems, especially after the studies of the French Engineer de Sant Venant who put the basis of the Mathematical Theory of Elasticity (1821).

The fundamental assumption of the Theory is that the materials are isotropic and perfectly elastic, that is, by hypothesis, they should resume the primitive form whenever the forces which have produced the deformations cease.

With the formulation of the mathematical theory of elasticity the majority of the engineers felt that the last word was said in the theory of structures. But few, enlightened, started raising objections. Nervi wrote "Do building problems, even in their most technical aspects (for instance, stability) permit unique and objective solutions obtainable by the application of mathematical formulas? Or, on the contrary can they be solved correctly only through a superior and purely intuitive re-elaboration of the mathematical results, because of the complexity of the intervening factors, the inherent deficiency of our theoretical knowledge and the wide discrepancy between theoretical premises and physical reality?"

And he goes on: "It would be absurd to deny the usefulness of that body of theorems, mathematical developments and formulas known by the rather inaccurate name of 'Theory of Structures'. But we must also recognise and state unequivocally that these theoretical results are a vague and approximate image of physical reality".

Another weak point of the Theory of Structure is the fact considered—as Nervi says, 'our buildings as being out of time, in a kind of eternal stability and invariability. But the simple and common-

place fact is that all structures decay, and after shorter or longer periods of time become unstable or at least show excessive displacement and amount of damage."

"No soil is perfectly stable nor settles uniformly as time goes by. All building materials flow viscously. The daily and seasonable temperature variations are irregularly distributed in the structure and because of prevented displacement create stresses of unforeseen magnitude and direction. Cement and limes keep hardening for decades and even for centuries. All materials are slowly being transformed. Every building has a physical and chemical inner life, whose existence is clearly shown by the sudden failures of even very old structures."

"In other words, Theory of Structures may be compared to the physiology of perfect organism, which are permanently youthful and untouched by disease or functional deficiencies. This kind of physiology is certainly indispensable in a school of medicine, but such a school would graduate very poor doctors if it did not take into account the difference between this ideal conception and physiological reality."

Every time a new idea, along with a new material or new possibilities is born, it has always found a designer of strong personality who has conceived and realised immediately the big application (les intuitions here uses des pioniers, as Colonnetti says).

The phases, as the new achievement starts, are always the same: A resounding bewilderment in the technical world, after few rare delusions, comes with some delay, often excessive, the plethora of the prudent checkers, in the end the equilibrium is reached, the new idea is accepted.

Eiffel, built the viaduct on the Trujere at Garabit in 1884 with a central arch of 185 meters, few years after the industry had produced the mild rolled steel.

Next year the Firth on Forth bridge reaches a span of 521 meters.

Hennebique in 1909 built the marvelous Risorgimento bridge in Rome using a concrete of 150 Kg/cm² (against the 700 and more reached today).

In 1936 Freyssinet built his famous bridge in prestressed concrete on the Marne with a span of seventy meters.

The Maracaibo bridge, designed by Morandi, which will be shortly opened to traffic, has a central span of four hundred ten meters. This road-cum-railway bridge is nine kilometres long and under it will pass the big ocean liners.

Reinforced Concrete Structures

The artificial Portland cement, whose industrial development goes back to 1880, for its properties of resistance and stability has originated the modern theory and technique of reinforced concrete which has modified rapidly and deeply the Art of Building.

The principles and characteristics of reinforced concrete are too well known and we will not speak about them. In Civil Engineering the freedom of the layout, the rational utilisation of the space in which few columns have substituted the system of internal supporting walls, the possibility of big openings, cantilever balconies and big spans are absolute merits of the reinforced concrete which has determined a period in the history of the Building Art.

This material frees one's structural imagination, but we must note with regret, that except for a few exceptions, there is a strange tendency amongst the Engineers to use all schemes reducing its use to the forms and structural shapes, typical of masonry or steel.

In the field of bridge design we had too, since centuries the arch bridge, always subject to the limitation (as concern the selection of the span, the

*Member, BPT-Copp. Committee on Plan Projects, Planning Commission.

curvature and its dimensions) that the curve of pressures should be contained at every section in the Kernel of inertia (the middle third), to avoid that the masonry, the concrete or the stones, in which the arches were built, should support tensions of traction under different load conditions and by effect of thermic variations.

The addition of few bars of mild steel in a concrete arch, and the possibility of a more complete knowledge of the tensions in a structure, have produced a big jump in the possibilities of application.

Far more than a thousand years, an arch of fifty meters span with a ratio raise/span of $1/4$ to $1/5$ has represented the maximum possibility in the solution of the theme.

Today the arches in reinforced concrete with a span of more than hundred meters cannot be counted. There are many arches of more than 200 meters and the tendency is to go towards 300.

Analogously we have seen especially in Italy the development and the progressive reduction in weight of the arch in reinforced concrete. Hennebique (1909) did build the Risorgimento bridge in Rome (span m. 100.00 raise m. 11.00) adopting an arch with solid spandrels. The resisting structure is formed in this bridge by all the material contained between the intrados of the arch and the road slab.

Krall in 1930 has designed the Africa Bridge in Rome in which the spandrels support only the accidental loads and the tensions due to variations of temperature, while the remaining loads are supported by the arch. Morandi (1946), in the design of the San Niccolo bridge in Florence, has discarded the use of the structural spandrels. The same Morandi (1956) has gone further in his Nuova Repubblica viaduct at Caracas. He has designed and built a shell arch without spandrels of 90 meters span

with a raise of m 6.50 which gives a ratio raise/span of about $1/14$.

Also the beam-bridges have had a great development for economic and stability reasons. Today girder bridges in reinforced concrete of 60, 70 and 80 meters span are found everywhere.

In the multi-span bridges the extension of the idea, generated by the steel structures, to make isostatic a continuous beam with the so-called Gerber joints, has relieved the reinforced concrete bridge designer of the pre-occupations which are derived from possible differential settlement of the supports.

The improved technology of concrete and the use of quality steel has contributed to produce in these last few years bigger and bigger reductions in the dimensions of the resisting sections.

In 1939 a Gerber beam of three span (30.00—40.00—30.00 meters) was described by Gerber himself, who proudly gave the figure of m^3 0.75 of concrete per square meter of deck. Today the common value is m^3 0.45 per square meter of road.

A logical evolution in the process of thinking, which created the reinforced concrete, has lead to the prestress concrete structures. From the idea of absorbing with steel rods the tensions of traction existing in a zone of the bent section of a reinforced concrete structure to the idea of using the same steel rods to produce in the same zone tensions of compression, in order to eliminate the tractions, the step is short.

In a few words the structure is put in a state of co-action in such a way that this state, realised with the known technique of compression, contributes to the resistance of the system.

I will not give here descriptions, known to-day, to all the technical men. I will only note that the technique of stressing concrete, born in France and in Germany around 1930 (not taking

into account early timid attempts) has developed with such series of studies and applications to justify the old definition of Freyssinet: "a revolution in the technique of reinforced concrete".

The application of co-action combined with the modern conceptions of thin supporting elements have produced marvellous achievements.

Shell of 150 to 170 meters span are starting to be fairly common. In Paris, it is possible to see a vault which reaches two hundred meters with a thickness of few centimeters.

In Rome, the Maestoso Cinema, (described in an article of the I. C. J.), designed by Morandi, is one of the most important structures in the world. Its portals, of 40 meters span, support a three storied building. The same Morandi has given an elegant solution for the new underground Hall of the Motorcar Exhibition in Turin. The special portals of seventy meters span support an earth filling which allows the growth of big trees.

I have already spoken of the Maracaibo bridge (Venezuela); at Genova a bridge with beams of two hundred ten meters is under construction on the design of Morandi. Since many years

the Rhine is crossed by beam bridges of 120 meters span.

Parallel to this development of the building technique we have another interesting phenomenon—the pre-eminence of the structure as a fundamental element of stylistic expression. Little by little the structure frees itself from its character of a frame hidden in the mass of masonry, which do not support anything, showing the true architectural form.

The boundaries between Engineering and Architecture are no more clear: "the technocracy has proposed the limiting synthesis in which the Engineer becomes a poet and the Architect becomes a Scientist; it is possible to speak of Architecture only when for Architect we intend a man capable of producing in terms of poetry the requirements of his age".

These new ideas have created in Italy a typical trend: the "Structuralism", which has produced and is producing a noticeable quantity of works of very high standard. But the "Structuralism" may degenerate, as Nervi says, in the false one, which instead of being the natural materialisation of static and constructional requirements is originated by a formal presumption.

Modern Trend

LET us examine how the most recent trends in the researches made to improve the knowledge of the behaviour of the structures in order to design them.

We have already said that all the theory of the Building Art has been based up-to-date on the linear law (Hooke law).

The dimensioning of a section is made according to the calculus of resistance. The maximum tensions which the material is to support, under external loads, are determined and we control that such

tensions, supposed to be contained between linear limits, result sufficiently far from the limiting tensions conventionally indicated as tensions of failure. This scheme seems sufficiently convincing (apart from the hypothesis of the linear proportionality, valid within limits only for some materials) as regards the solid formed by homogeneous and isotropic materials.

In the particular case of reinforced concrete, that is in a structure composed of heterogeneous materials: the steel, coming from the blast furnaces and a rolling process: the cement, coming

from a chemical and milling process; and the aggregates as they are in nature, the scheme has been extended assuming the equivalence of the sections considered homogeneous, that is formed by the compressor concrete and the steel area multiplied by a co-efficient equal to the ratio of the moduli of elasticity, discarding completely the concrete under tensions of traction.

The scheme is very far from representing the true state of tensions for materials which, it is well known, even for light loads, do not follow the linear law. This fact leads to the determination of safety margin very far from the reality, both for the fictitious meaning of the allowable tensions as well as for the consequent assumption of two different co-efficients of safety: one for the concrete and another for the steel.

The interpretation of the idea of safety in a structure is still under discussion.

It is sufficient to consider that in a reinforced concrete structure the dispersion, in the statistic sense, of the resistance due to the defects of fabrication is much more for the steel, and that, especially for what we will say later on, when we will speak of the failure in the plastic field, any section of minimum resistance may modify sensibly the degree of safety of the structure, considered as a whole.

Another dispersion is determined by the difference between the stresses resulting from the calculations and the real ones. The difference is due both to the necessary inaccuracy of the calculations themselves, as well as to the not exact valuation of the external forces assumed in the calculations. The influence of such inaccurate valuation varies enormously from type to type of structure. In effect, admitting that it is easy to determine with sufficient accuracy the

self weight of a structure, the accidental increase of the overload (for instance, hundred per cent) will have much more importance in a roof, in which they determine eighty per cent of the total tensions (due to self weight and overload), than in a big bridge in which they will determine only twenty per cent of the total tensions.

Recently it has been suggested, the adoption of different co-efficients of safety for different types of structures, in consideration too of their economic importance, in relation to the statistic probability that the heaviest condition of load, assumed in the calculations may take place.

These statistical studies have been started by the aeroplane builders and have passed in the field of Civil Engineering.

Some years ago it was proposed, and today is more and more applied, the so called plastic theory (failure in the plastic phase). For the particular case of a section under bending it is assumed a diagram of tensions near the failure point, taking into account the plastic deformations of the materials.

This diagram is no more linear, the co-efficient n has no more meaning and the behaviour at breaking point is influenced by the quantity of reinforcement assumed in the design. The collapse of the structure will depend evidently from which of the two materials: concrete or steel, will first reach the failure limit. The determination therefore of the degree of safety of the section will result one and much more adherent to the reality.

Having entered in this field of ideas and for the usual case of hyperstatic structures it has been observed that, in the field of deformations elasto-plastic, we have the phenomenon of redistribution of the tensions, that is, the

tendency to decrease the tensions where the so called general elastic limit has been exceeded, with consequent increase in the zones in which initially the resistance of the materials is less utilised. This phenomenon obliges to examine the degree of safety of sections which the theory of elasticity does not consider as sections of maximum strain.

This is another step towards the understanding of the behaviour of a structure, a step forward in the determination of its real degree of safety.

For instance we will say that the calculation to failure tell us that it is not correct to put steel in compression at supports of a continuous or encastre beam and that the failure will never happen in correspondence of the sections of maximum moment, if there are noticeable differences amongst the various sections of the beam.

But even the calculus to failure cannot be considered sufficiently exact. It pre-supposes in effect the hypothetical assumption of diagram deformation-tension geometrically defined, (especially for concrete) and an hypothetical force of the diagram of the internal stresses of the bent sections near the phase of failure. This form is very important and many hypothesis have been advanced by different researchers, that try with experiments, often of doubtful value, to prove the greater approximation of their hypothesis in respect of others and also in respect of the reality.

For the calculation of hyperstatic structures some authors have been advised to assume, as a more reliable value. The average between the results of the calculus to failure and the traditional one of the theory of structures (elastic calculus).

At this point it is natural that the Engineer would ask: Is it possible to take into account, during the calculations of a structure, its real elastic and anelastic behaviour, intending as anelastic both the behaviour above the limit of viscosity as well as the anelastic one, in the small deformations as creep, relaxation damping of vibrations and fatigue?

In other words is it possible to describe all the facts with a few parameters practically utilizable?

To answer this question, we must distinguish between the description of the various anelastic phenomena and their correlations, that it is the reduction of the number of the determining parameters.

The description of the phenomena would appear a fact already ascertained by more than a century of experience and experiments on materials. In reality the mechanical variables of the continuous systems, as soon as we go out of the little deformations, become very hard to manage.

Bruno Morandi gives us a clear idea when he writes, "for the components of the tensor of deformation it is necessary to consider also the quadratic terms in the derivatives of the displacements, with the consequence that, for instance components of the same index are no more equal to the unit elongation: for those of the tensor of tensions we must distinguish between the Eulerian and the Lagrangian derivatives with different expressions of the work of deformation"

These problems, known since many years to the Physico-mathematicians have been taken into consideration only recently by a few Engineers, with the result that many of the experimental

tests of the past cannot be compared and therefore are useless.

We have hinted at these questions because they appear to our mind, any time we are dealing with a diagram tension deformation, extended beyond the field of the little deformation, in which it is not specified any definition of the σ and ω , represented on the axis, or which is completely misrepresented by an inadequate definition of the variables. Besides they make very little expressive each value of the failure tensions quoted without any further specifications.

About the problem of reducing to few the parameters which may describe all the numerous and complex phenomenons of the mechanical behaviour of the materials, the most important attempt had been made ten years ago and goes under the name of Rheology. The Rheology is the systematic study of the flow and deformation of the matter in the time.

Even if the aspiration of the first rheologists, to make the little models of the various linear bodies correspond to real phases of the materials, has failed with the necessity of considering continuous distributions of infinite times of relaxation; the fundamental relationships between the different types of deformation has remained. For instance the correlation between the viscous deformation under constant load and the relaxation under constant deformation. The present codes for prestressed concrete are, in definitive, its consequence.

It is known that the attempt to take into consideration the anelastic behaviour, that is the failure calculus, utilises only the most elementary scheme, the one of perfect plasticity and isotropy. There is still a long way before we may answer the problem formulated above.

Let us now outline the phenomenons of which we know very little.

The Fatigue

We have had recently very important researches, but a disheartening impossibility of correlating the results of the different tests. Even to day we do not know, generally, how to calculate from the number of the cycles supported with a given oscillation, at a given average tension, and with a given frequency, the number of cycles which the same material will support at a different frequency, oscillation and average tension.

The Photo-Elasticity

In this field we know only that for few materials, with the variation of the tensions, varies the index of refraction (variation of the optical properties).

The Pieze-Electricity

We know that for some crystals the tensions produce effects of electric polarization.

Such examples induce us to ask another question: Will it be possible to reduce the description of all the phenomenons, from the mechanical behaviour of the materials to the structural and atomic properties of the solids, disclosed by the recent progress of Physics, to few parameters? Will it be possible to express the various parameters, which describe such phenomenons, through the properties of the atoms and the molecules?

That is: which possibilities we have today to derive the mechanical properties of the materials from their intimate structure?

Here too, it is necessary to make a distinction and hint at the three possible approach levels on three different scales: the microcrystal, the molecules or intracrystal (atoms considered as entire) and the intramolecular and atomic level.

The study of the microcrystals and their properties is old. The tensorial and the matrix calculus allow easily the calculation of any type of anisotropy.

At the second level, (molecular or intracrystal) the position and the energy of the atoms in the crystals and their defect have been investigated with very good results, especially with the X-rays diffraction pattern, which allow us to measure, with exceptional accuracy, the patterns and distance of the atoms in the periodical structures.

Another powerful means of research is the measurement of the absorption of the mechanical vibrations, at the various frequencies, with the construction of the *relaxation spectrums* partially analogous to the optical absorption spectrums.

On this scale have origin the plastic and viscous phenomenons. Many theories have tried to interpret them. Their multiplicity show that we have not yet a satisfactory one. We will quote as the most important, that of the dislocations, which attempt to calculate the elastic limits of the crystals from the examination of a particular type of imperfection of the *grid* and from the kinematic study of its diffusion through it.

After all, the elastic properties of the solids are born from the mutual action (interaction) amongst the single atoms. They represent the most intimate mechanical property which explains, why, by example, the moduli of elasticity of the

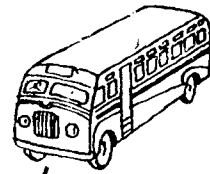
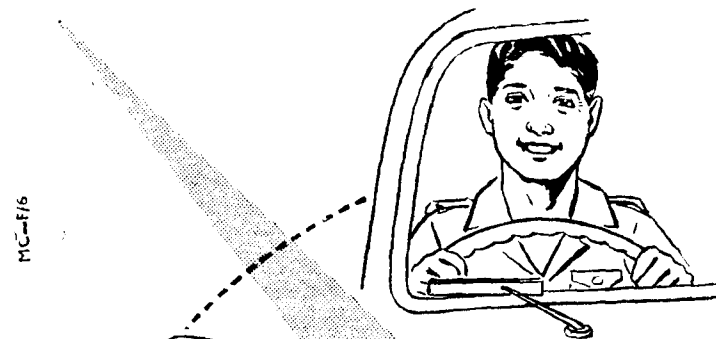
steels depend very little from the metals dissolved in them and from the thermic treatment.

Here the method of approach changes completely for fundamental reasons. In such a small scale the law of the classic mechanic are no more valid and they are replaced by the quantum mechanic, that is we pass completely in the field of theoretic physic.

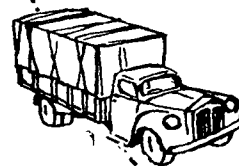
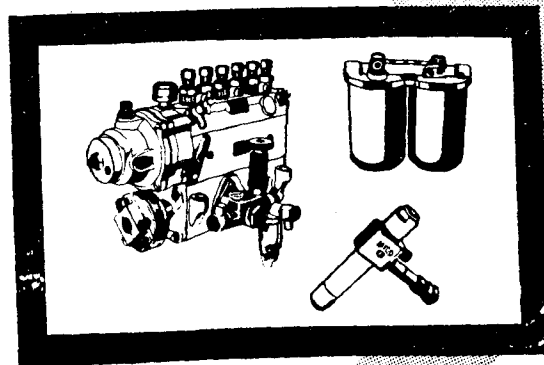
The road to the future development is most probably that of the classic theories modified with quantistic conditions or vice versa through simplified models.

The day in which the complete union between a simplified atomic theory and a clear and deep description of the phenomenons will be realised, a new Engineering Static will be born, more simple than the present one (any approach to the deep reasons produces a simplification) with whom we may perhaps:

- (a) take into account, at one time, the elastic and anelastic phenomenons. The latter are treated today as incidental while instead they are aspect of the same reality.
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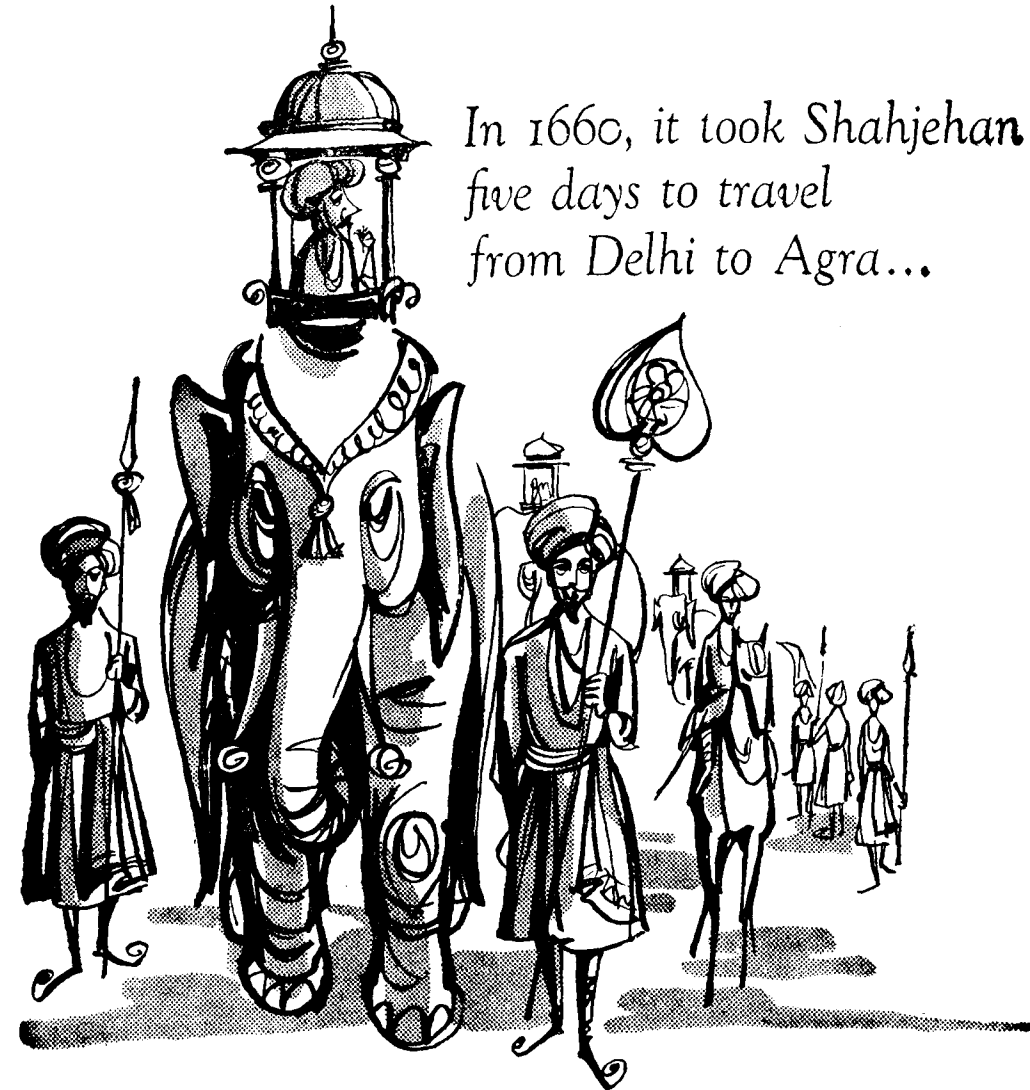
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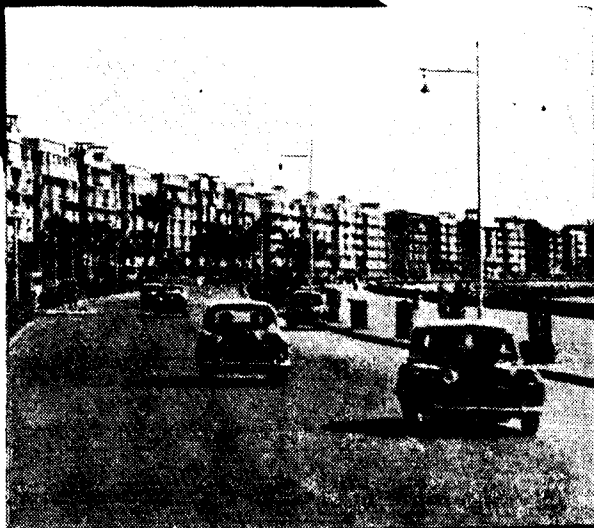
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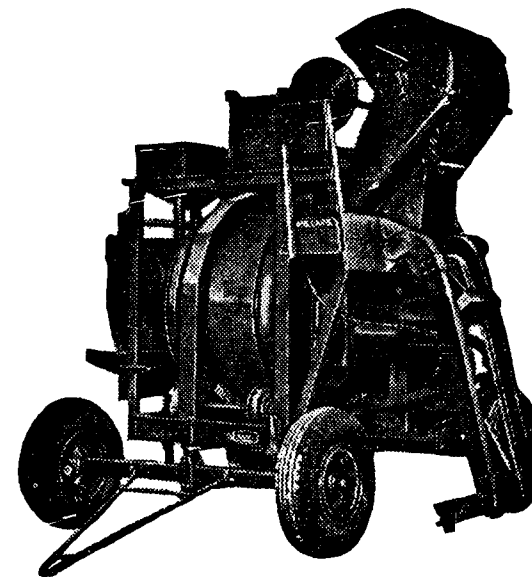
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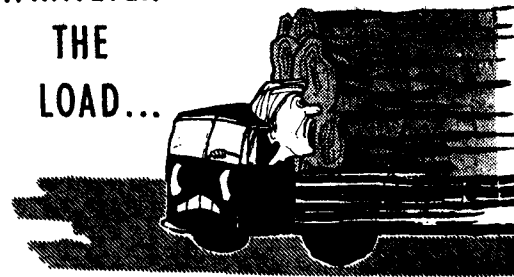
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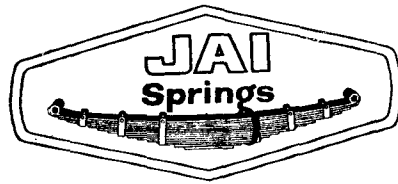
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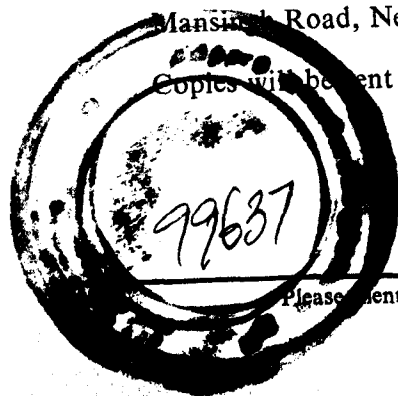
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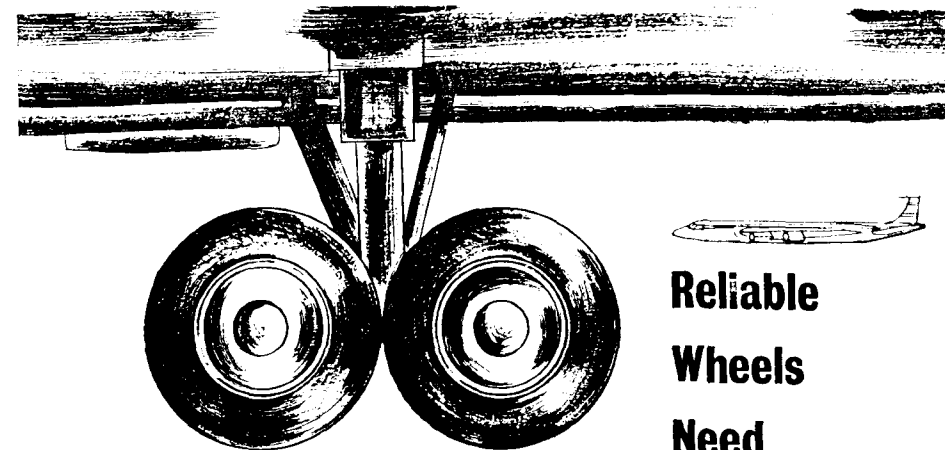
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