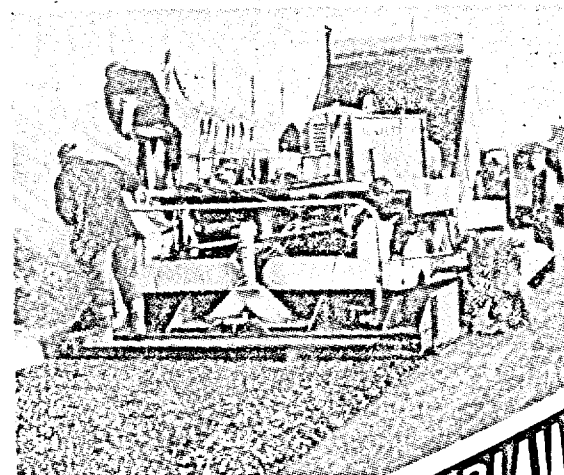


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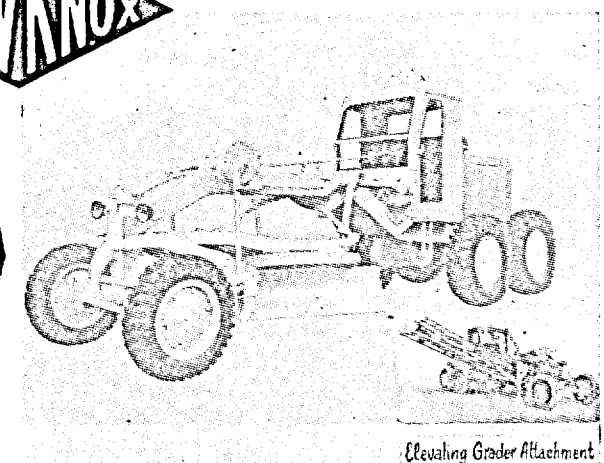
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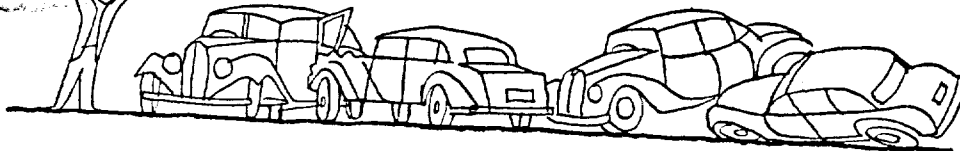
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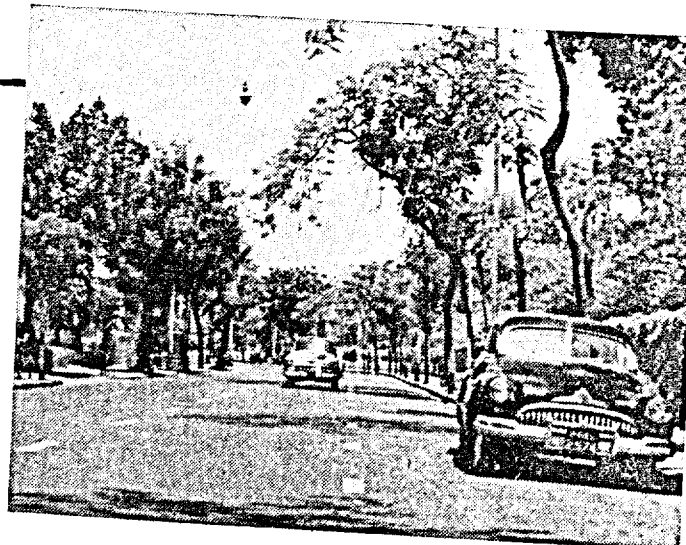
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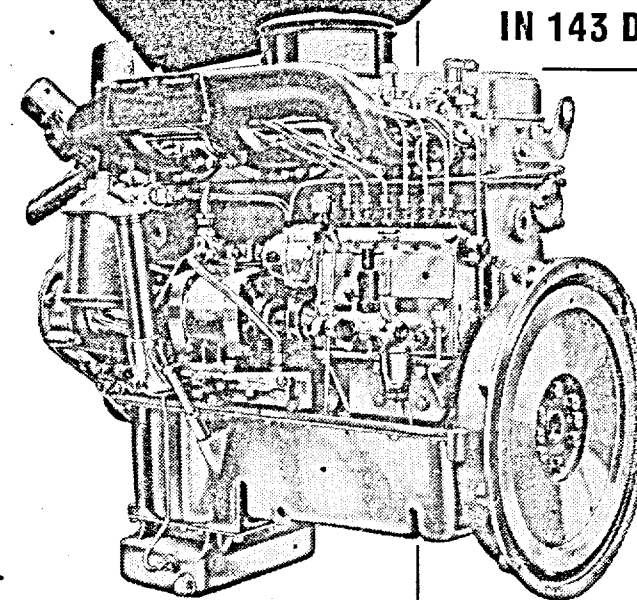
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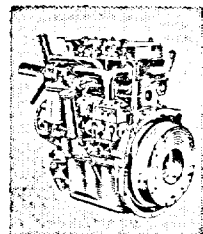
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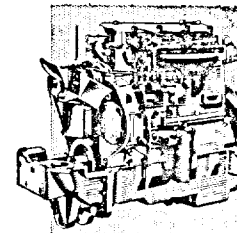
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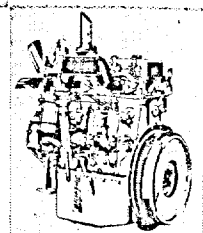
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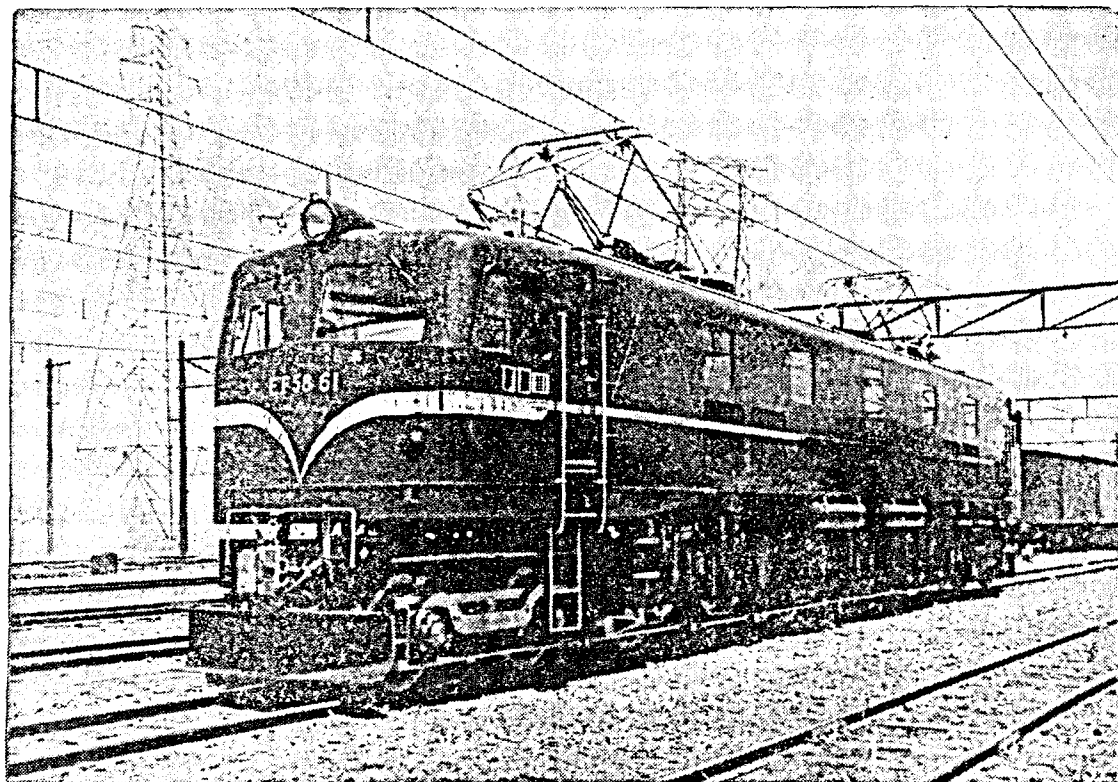
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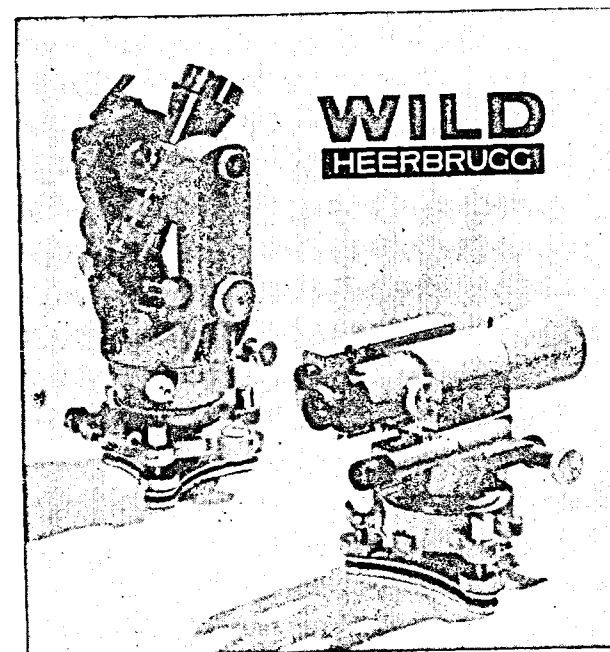
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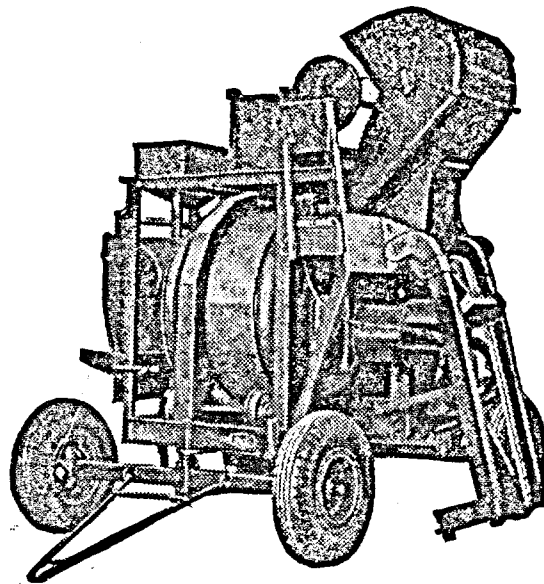
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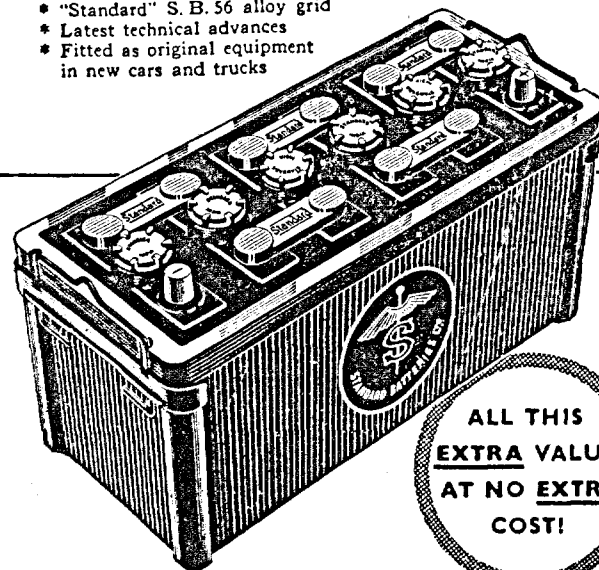
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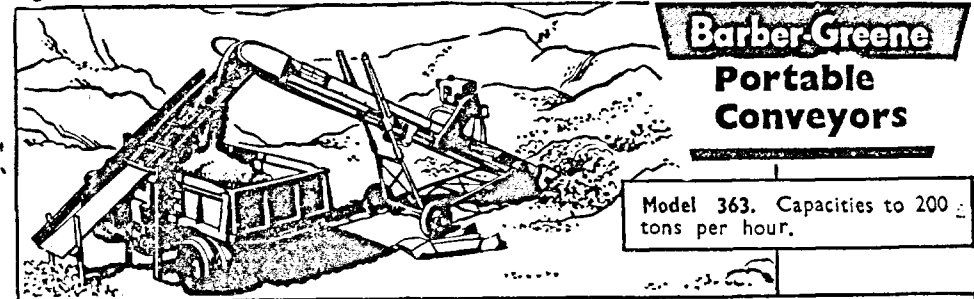


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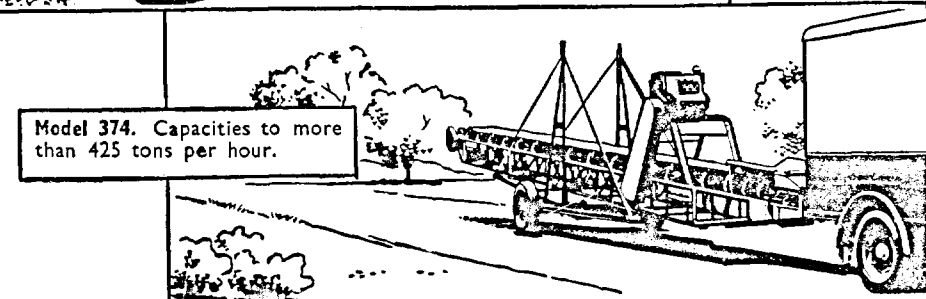
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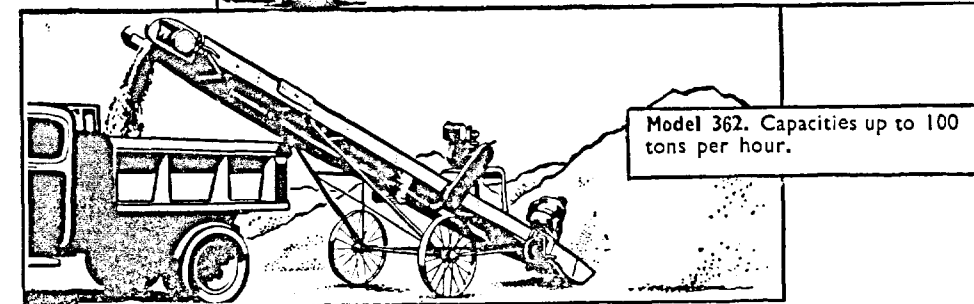
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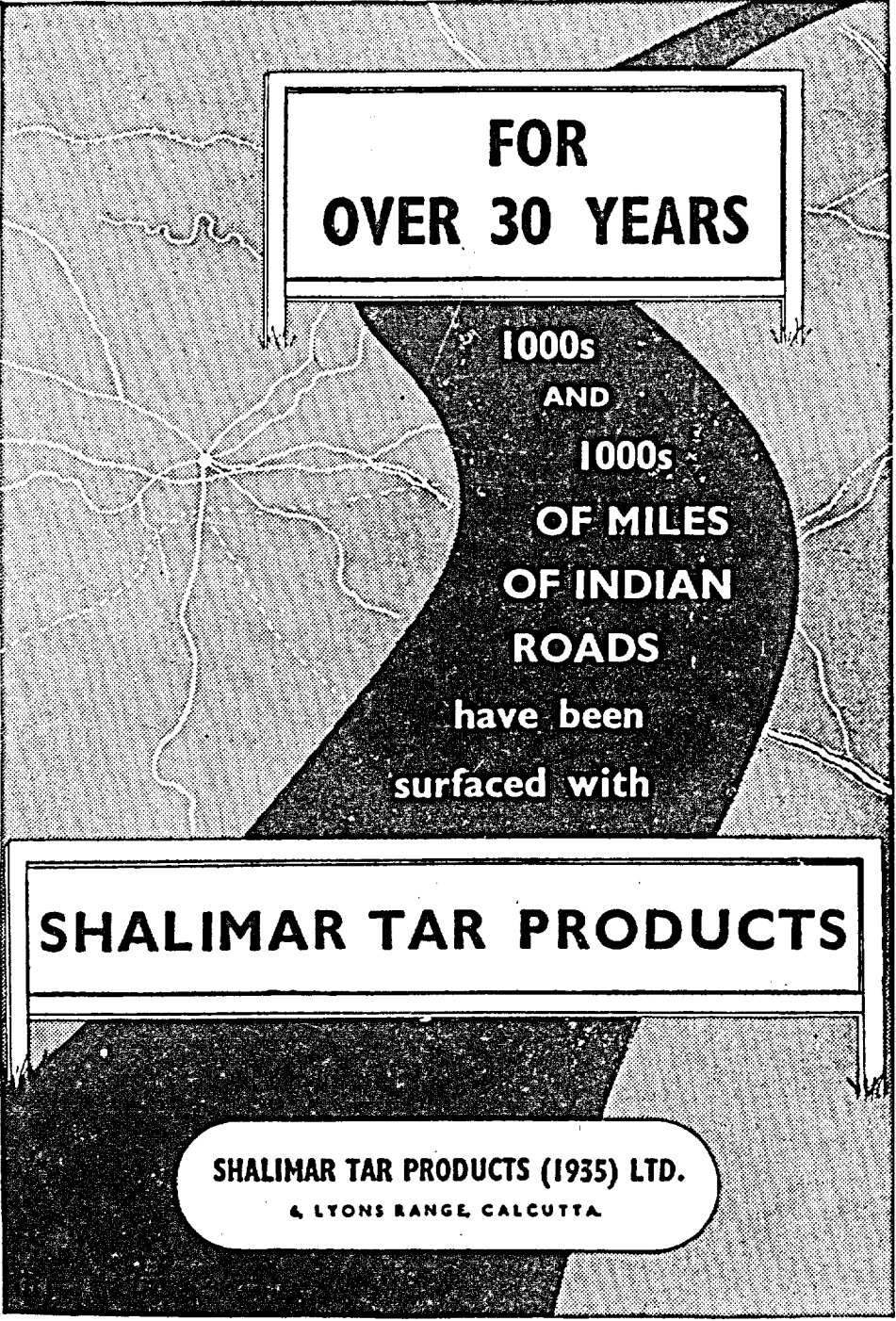
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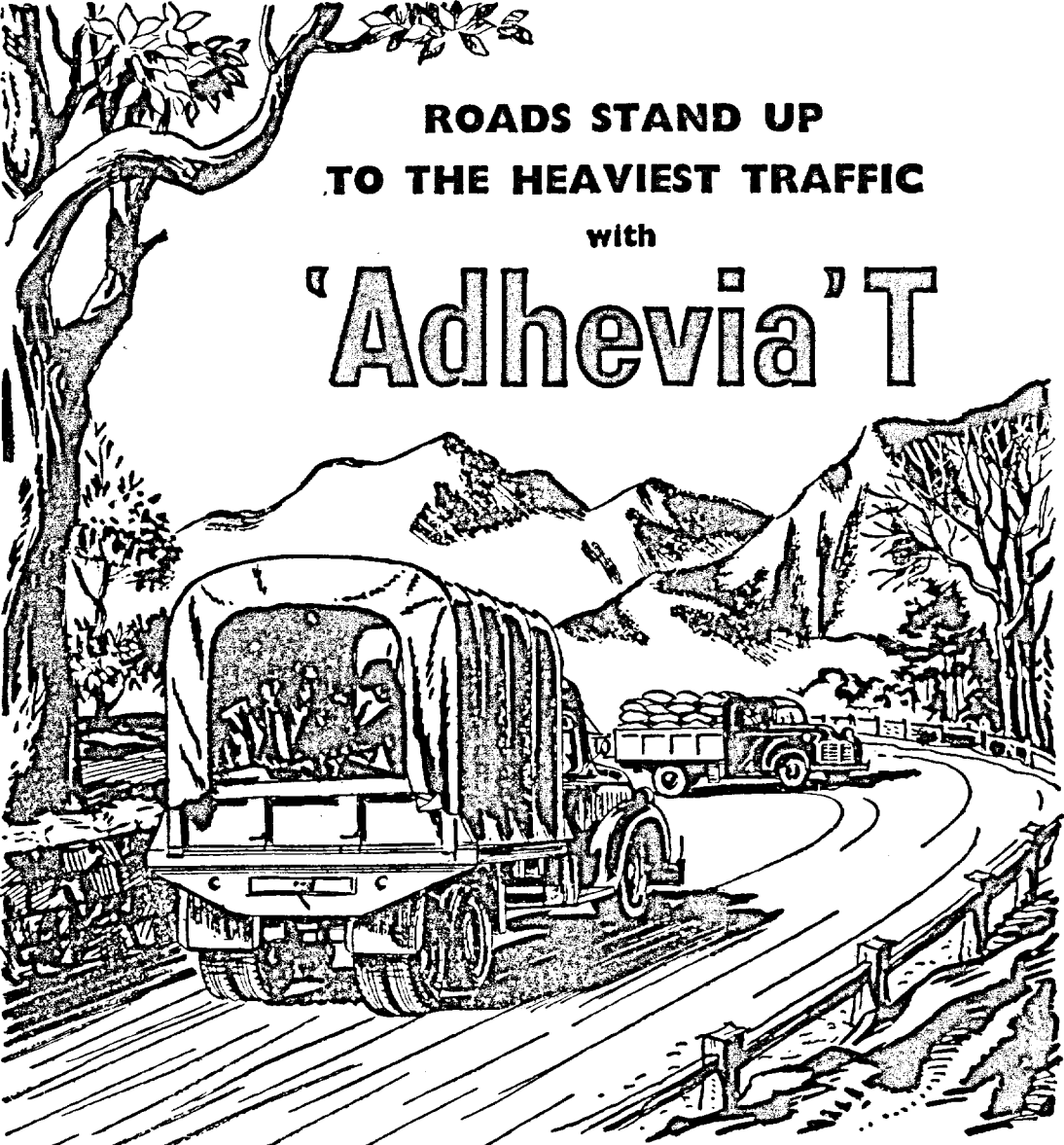
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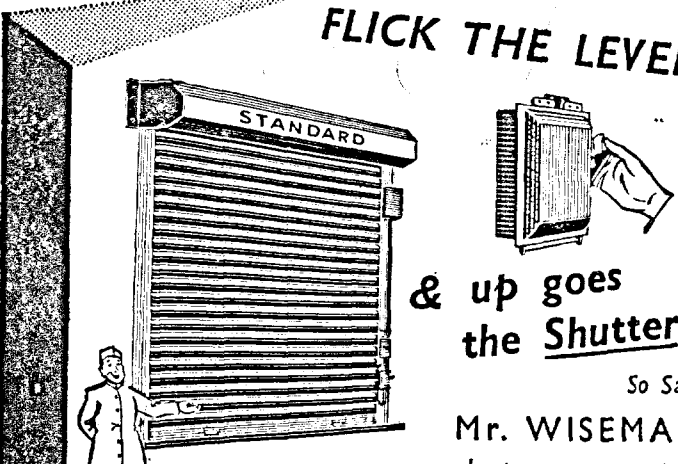
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 **IMPERIAL CHEMICAL INDUSTRIES (INDIA) PRIVATE LIMITED**
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TRANSPORT- COMMUNICATIONS

MONTHLY REVIEW

MAY 1957—No. 121

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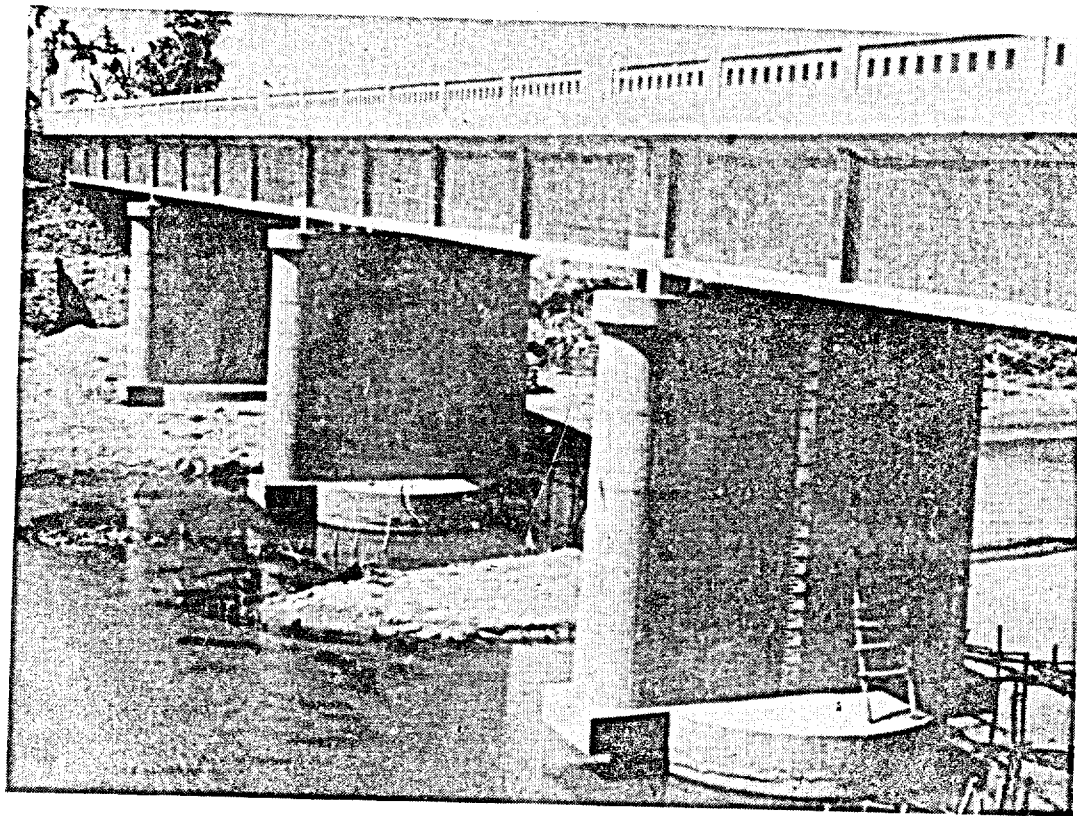
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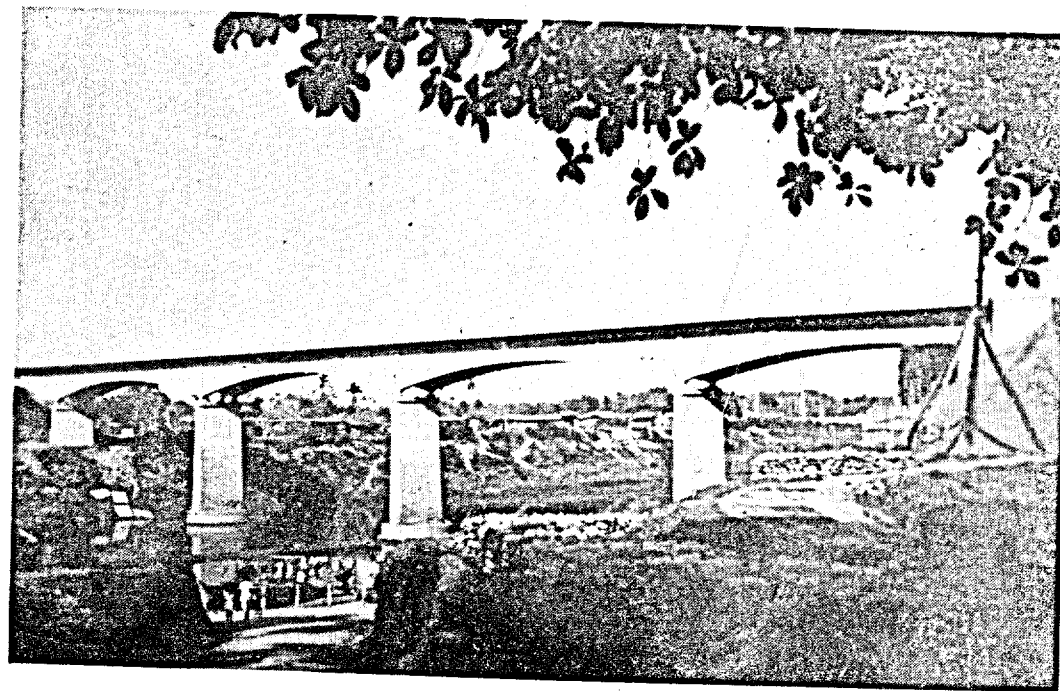
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A REVIEW OF
ROAD ENGINEERING, DEVELOPMENT & TRANSPORT

MAY 1957

BRIDGING INDIA'S RIVERS

THE RANGANADI BRIDGE

THE Ranganadi has its source in Daffla hills of the Balipara Frontier Tract. After flowing to the south, it joins the Subansiri river. The road from North Lakhimpur to Tezpur, an important route in Assam, crosses the Ranganadi at a distance of 4 miles to the west of North Lakhimpur town.

In the year 1936, a plate-girder bridge of the deck type, consisting of 26 spans of 40 ft each, was constructed at this crossing. Within a couple of years of construction, the bridge was heavily damaged by an earthquake. The piles of the bridge got bent and shifted laterally giving the bridge the appearance of a bow. Although the bridge was damaged it could be used by traffic. So, as a temporary measure, it was being maintained to afford connection across the river. During the earthquake of 1950, the bridge was again severely damaged and the entire bridge collapsed.

For reconstruction, various sites were considered and an alignment about 2,450 ft downstream of the collapsed bridge was selected. The location of the bridge at the old site would have involved difficulties in constructing the foundations because of the presence of piles which served as foundations for the old bridge.

The approaches to the old bridge were also not well aligned.

General Description

The bridge as reconstructed consists of seven spans of 100 ft each with prestressed concrete longitudinal girders and R.C. deck. The girders rest on cellular piers and abutments on well foundations.

Foundations

Borings revealed that the bed of the river contains sand, sand mixed with shingle, soft clay, and clay mixed with sand. Well foundations were considered suitable for the type of soil met with. For each pier there are two twin wells, each 16 ft 6 in. in diameter, and spaced at 20 ft 6 in. centres. The steining of the wells consists of 1 : 3 concrete composed of boulders 4 to 8 in. in size, and is 2 ft 3 in. in thickness. Twelve mild steel rods, 7/8 in. in diameter, placed centrally in the steining extend from the well kerb to the pier cap. These reinforcing rods are tied circumferentially at intervals of 10 ft with 1/2-in. diameter rods. Wherever necessary, the rods of the steining have been extended by using bottle-nut jointing. The kerbs of the wells are of reinforced concrete 1 : 2 : 4, fitted with a cutting edge of mild steel angle 3 in. by 3 in. by 1/2 in. and a steel plate 6 in. by

$\frac{3}{8}$ in., Fig. 3, Plate II. The kerbs have been cast in situ and the colcrete steining built over it in heights of 3 ft 9 inches.

Shuttering consisting of inner and outer mild steel plates curved to the correct shape and stiffened with angle irons, has been used for casting the steining. The plates have been bolted together by bolts passing through distance pieces 2 ft 3 in. thick and 2 in. square. Boulders 4 in. to 8 in. are then hand-packed in the shuttering with 2-in. size shingle between them to reduce the voids. Flues are left in the boulder packing at intervals and colcrete grout of 1 of cement and 3 of sand mixed in colcrete mixer is fed into the flues through a flexible pipe. After the colcrete had set, the shuttering is removed.

After sinking to the specified depth, the wells are plugged at bottom with colcrete concrete of 1:3 proportion to a height of 3 ft. The well is filled with sand. At every 11 ft 3 in. height a mat of colcrete (1:3) thick 1 ft 6 in. is provided and the top is sealed with a reinforced concrete slab, 2 ft thick. Fig. 1, Plate II.

Well Sinking

During sinking of the wells, tree trunks buried under the sand were met with in six of the wells. Wherever the tree trunks were small, they were removed by cutting them to pieces. In one well, a very big tree trunk was met with at a depth of 13 ft 6 in. and its shape was as shown in Fig. 5, Plate II. It had a number of branches. The branches were cut and the main portion of the trunk on which the well kerb was resting was pulled out of the well by a jack and chain block. The portion of the tree trunk on which the well kerb was still resting could not be pulled out completely because of the risk involved in the well suddenly slipping down over the cut face of the tree trunk and injuring the diver. It was hoped that after cutting away portions of the trunk, the well might slip down with the assistance of kentledge. But this did not happen. Attempts were made to remove the earth by means of grabs outside the well in the hope that the remains of the trunk would be pushed out by the weight of the well.

This also was not successful because sand was frequently blown into the well. Finally, a charge of explosive of about 4 ounces of gelignite was put in a hole bored in the trunk and the blast was set off by an electric detonator. A small kentledge was applied at the top of the well. The blast successfully removed the tree trunk. It took four months to remove this tree and cost Rs 9,250.

Substructure

The piers and abutments are of cellular type reinforced concrete. The piers are 28 ft long including semi-circular cutwaters and 7 ft wide. The thickness of the walls of the piers is 1 ft and is reinforced with $\frac{3}{8}$ in. diameter mild steel rods both vertically and horizontally at 12 in. centres. The two longitudinal walls of the piers are connected at four places with R.C.C. diaphragms 6 in. thick. The top of the piers is closed with a slab of 1 ft 3 in. thickness Fig. 1, Plate II. The abutments are of box type with reinforced concrete walls.

Pier Cap

It consists of an R.C.C. slab with reinforcements of the pier bent in and tied to the pier cap reinforcements. The roller bearings of the beams rest on one side of the pier cap and the rocker bearings on the other side. The pier cap is 2 ft thick at the rocker side and slopes down to 1 ft 3 in. at the roller bearing. The bottom plates of the bearing are anchored to the pier cap by means of rag bolts and the top plates fixed to the bottom of beams, Fig. 2, Plate II.

Centring Arrangements

Country wood piles, 16 to 18 ft long and 6 to 8 in. in diameter, were driven at 3 ft 6 in. intervals along the beams and cut off at low water level. Wooden planks, 8 in. by 2 in., were put over these piles at right angles to the beams. The frames to support the shuttering of the beams rested on these cross planks. In places where the water was deep, spurs were provided on the upstream, Photo 5. These spurs caused silting up of the deep portions of the river.

As the site of the bridge was not far from the hills, flow of water in the river

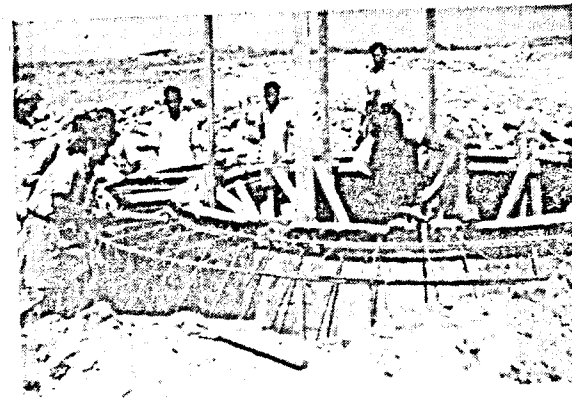


Photo 1.
Reinforcement for the kerb.

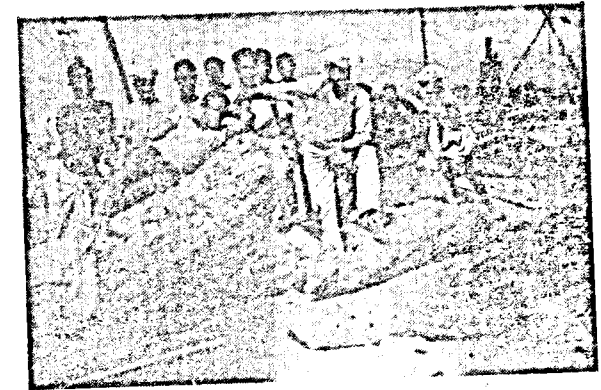


Photo 2.
Tree trunk removed from one of the wells.

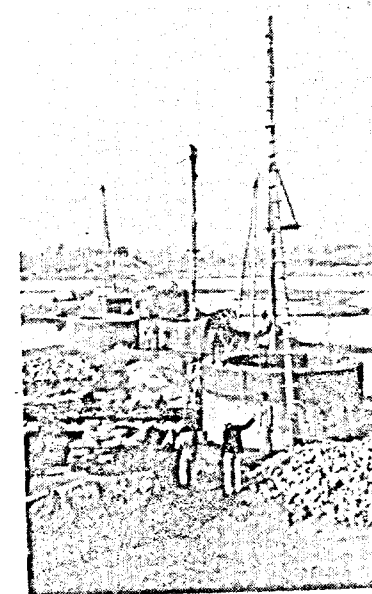


Photo 3.
Well sinking.

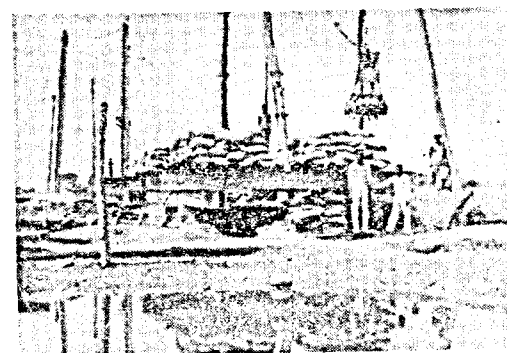


Photo 4.
Kentledge for sinking the wells.

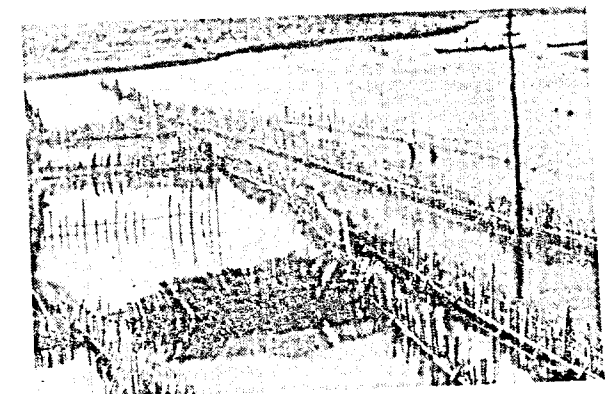


Photo 5.
Spurs which caused silting of the deep portion of the river.

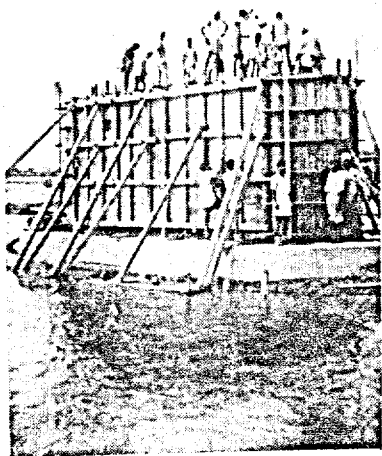


Photo 6.
Shuttering for pier.

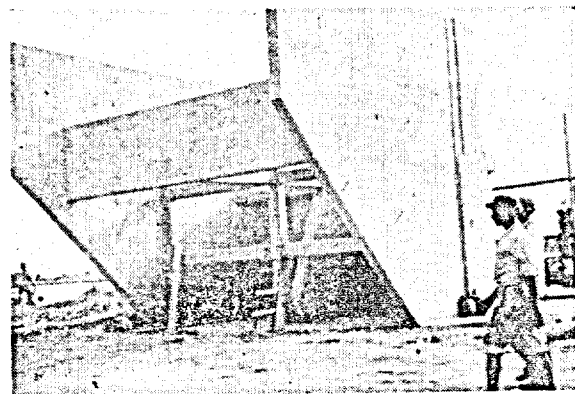


Photo 7.
Abutment.

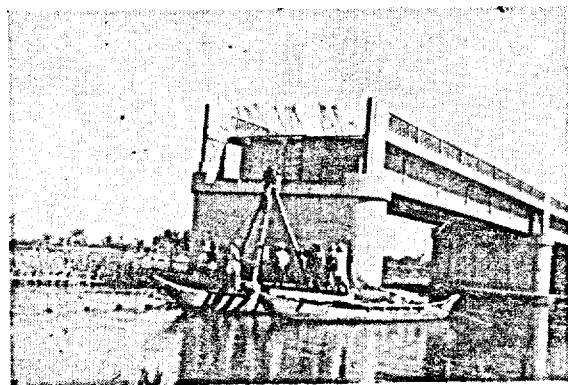


Photo 8.
Driving wooden piles for staging.

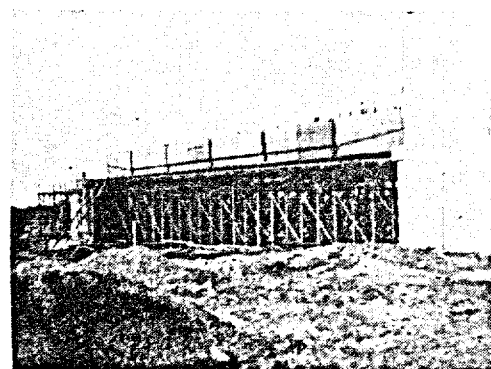


Photo 9.
Staging and sides' shuttering for beams.

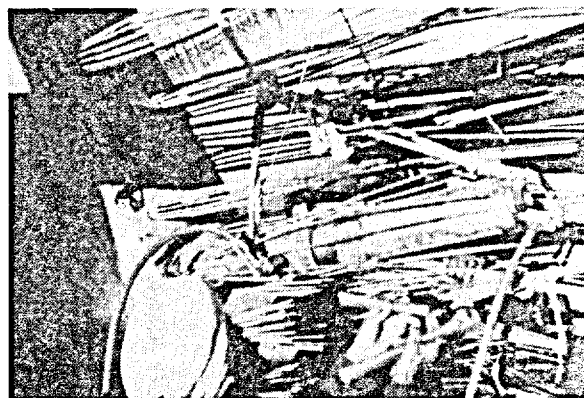
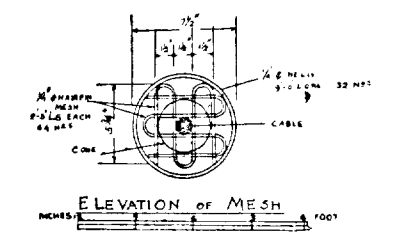
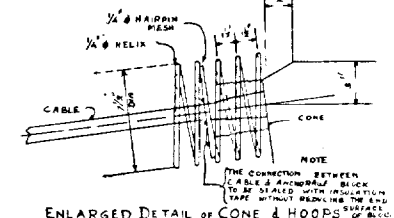
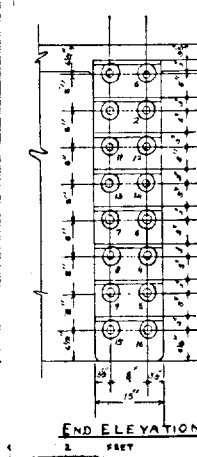
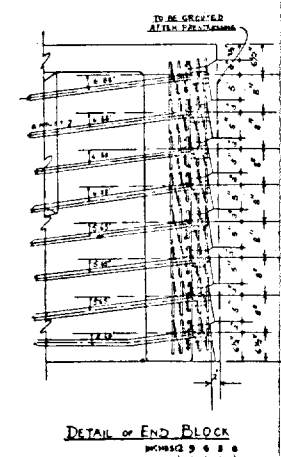
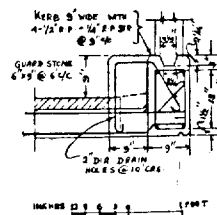
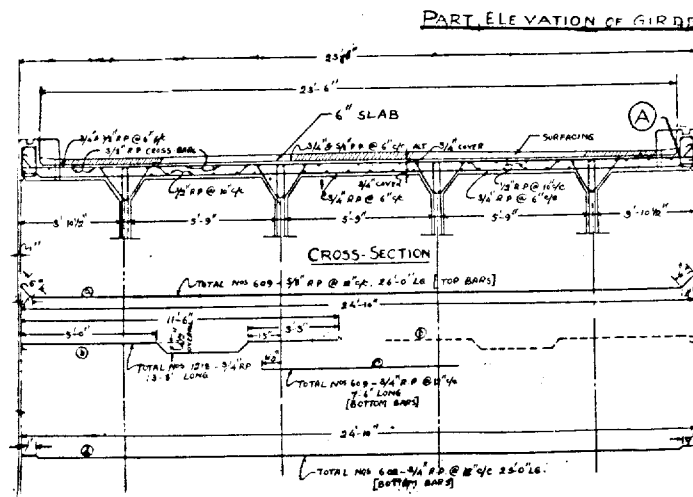
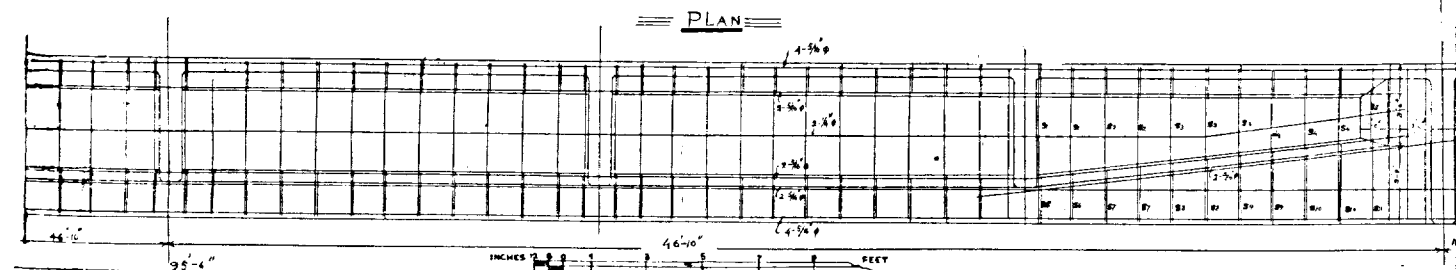
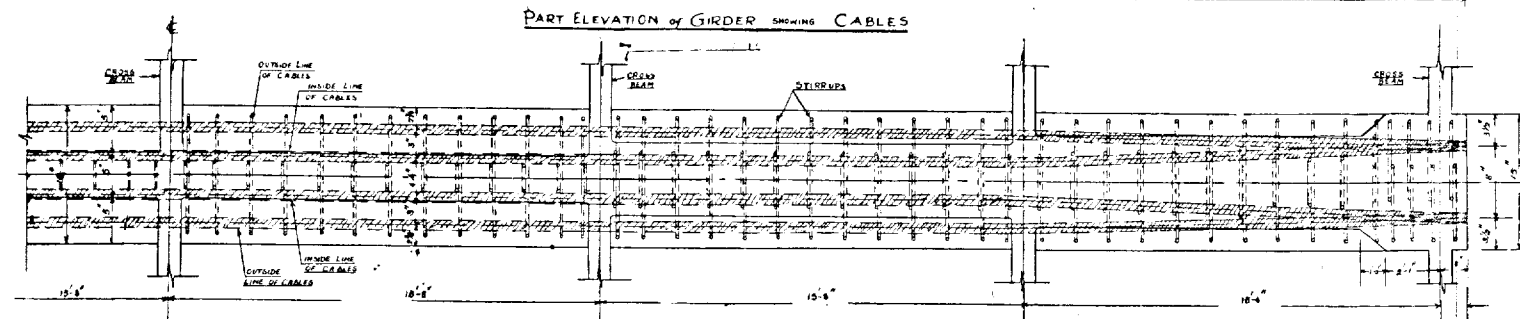
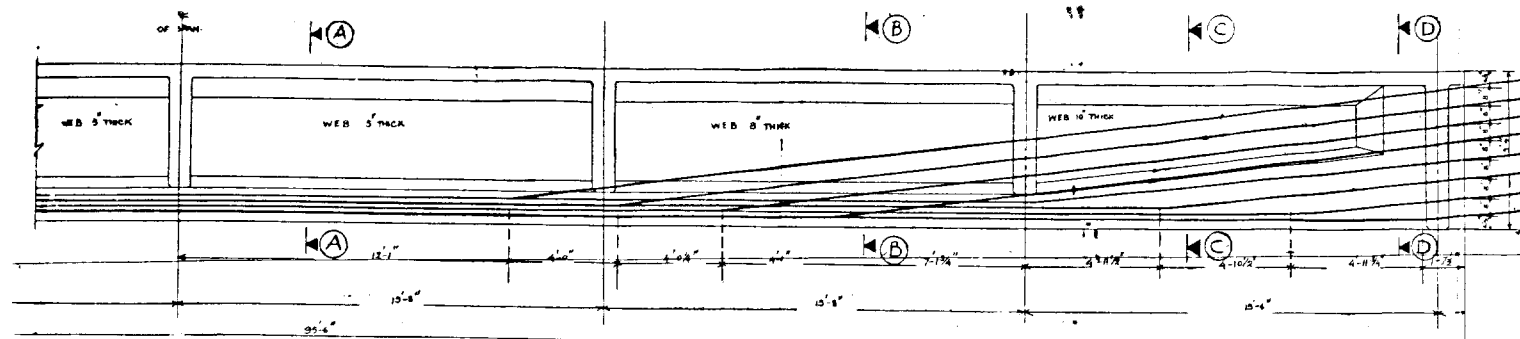
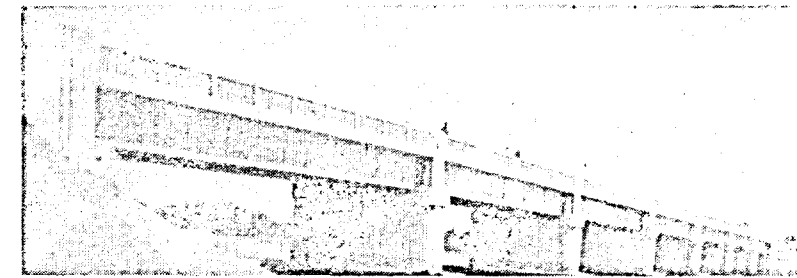


Photo 10.
Hydraulic Jack butting against the outer cone.



DETAILS OF PRESTRESSED
CONCRETE SUPERSTRUCTURE
RANGANADI BRIDGE



Ranganadi Bridge

was seldom constant. So this method of supporting the formwork was adopted to withstand unexpected increase in the flow of the river which scoured the bed. Besides, as the staging had to remain in position for over a month after concreting, extra precaution was necessary.

Superstructure

The superstructure consists of four longitudinally prestressed beams and the deck which act as a composite unit. Prestressing was done according to the Freyssinet system. The deck is also stiffened transversely by R.C.C. diaphragms spaced about 16 ft centres. The slab panels are designed for bending stresses in both directions using Pigaud's method. The deck, the main beams and the cross diaphragms are cast in situ and the cables are tensioned after 21 days. The prestressing cables consist of 12 No. of 5 mm high tensile steel wires with a breaking stress of not less than 90 tons per sq. in. The working stress after allowing for creep and shrinkage, etc., does not exceed 55 tons per sq. in. The following working stresses have been adopted in the design:

Concrete in compression	750 lb per sq. in.
Steel in tension	18,000 lb per sq. in.
Modular ratio	15

There are 16 cables in each of the beams. Details are given in Plate I.

For the concrete of the beam, river shingle 1 in. in size and river sand, were used. A slump of $\frac{1}{2}$ in. to 1 in. was specified. For concrete at bottom of the beams a

slump greater than 1 in. was allowed until the cables had been covered.

For ramming the concrete, bamboo poles were preferred since the steel rods for rammers might damage sheathing of the cables. The shuttering was constantly hammered to give compactness and a good finish. However, steel rammers were used for consolidating the deck concrete.

It was observed that when 50 per cent of the cables were prestressed, the beams lifted themselves up slightly from the shuttering. After the cables were stressed and locked, they were grouted to prevent rusting. Cement, fine sand and water were mixed in a colcrete mixer in the proportion of 1 : 1 : 1 by volume and the grout was poured into the pressure cylinder till it was three-quarters full. Before applying the grout, the cables were first flushed with water and grout was applied with the help of a compressor through the hole in the inner cone. The grout was pumped until some grout oozed out at the other end.

Wearing Coat

The wearing coat consisted of cement concrete 1 : 2 : 4. Two layers of $\frac{1}{4}$ in. diameter hoops were provided at longitudinal intervals of 2 ft 9 $\frac{1}{2}$ in. and transverse intervals of 1 ft 6 in. to prevent cracking. The concrete of the wearing coat was 3 $\frac{1}{2}$ in. thick at the centre and 2 $\frac{1}{2}$ in. thick at the edges. In the first three spans, the wearing coat was laid in alternate bays for the full width of the bridge, before the cables were stressed. It was found, while stressing the cables, that the gentle movement of the deck units caused by the prestressing set up

(Continued on page 12)

ROAD DEVELOPMENT IN ASSAM

(Contributed)

ASSAM is the easternmost State of the Indian Union and is connected to the rest of the country only through narrow corridor. The Himalayan range on the north and the toes of Khasi and Jaintia hills, the Garo hills, etc., on the south form the boundary of the State. In the State there are two important valleys between the hills. The larger of the two valleys, popularly known as the Assam valley is between the Himalayas and the Southern hills. It is approximately 40 miles in width and runs from East to West. The Brahmaputra traverses the entire valley and has a large number of tributaries having their sources in the Himalayan ranges on the north and the hills in the south.

The extreme Southern hills are known as the Mizo (Lushai) hills and are intercepted by the valley known as Surma valley during pre-partition days. Barak is the largest of all the rivers flowing through this valley.

Due to the hilly nature of the country, the State is not served well by railways. Road communications are, therefore, of great importance to the State.

The main road system of the State consists of the National Highway running from west-east along the Brahmaputra, connecting Buxirhat in Cooch-Bihar (West Bengal) border and Saikhowaghat (towards North-Eastern Frontier Agency) in the east. There are two other National Highways, one connecting Gauhati and Tamabil (Dawki) on the East Pakistan border via Shillong, the capital, and the other branching from the east-west National Highway connecting Dimapur, Kohima in the Naga Hills, Imphal in Manipur and Moreh in Burma border.

Post-War Programme

According to Grid and Star formula evolved at the Chief Engineer's conference at Nagpur in 1943, the target road mileage for the State was 13,200 miles. After independence a programme was

drawn up under Post-war Plan, but it had to be dropped before it got through even halfway, for financial reasons. The development of communication was practically at a stand still, till the First Five Year Plan took shape.

The bridges constructed during the pre-independence period were mostly in steel and timber and had load carrying capacities varying from class 5 to 12.

At the time of independence there were 2,586 miles of motorable roads in the State.

FIRST FIVE YEAR PLAN

The first 5-year Plan provided for 1,373 miles of roads and 4,300 running feet of major bridges at an estimated cost of Rs 432 lakhs. The target of expenditure fixed was 265 lakhs, within the plan period. The target achieved was however 262 lakhs. The balance was carried over to the Second Plan.

In addition, in the Autonomous hill Districts, 625 miles of roads at an estimated cost of Rs 342 lakhs were included under this programme. The target of expenditure was 256 lakhs within the plan period. The target achieved was however 184 lakhs. Due to various bottlenecks, the main being the shortage of labour and non-availability of bridging materials the target could not be achieved in full. In tribal area of the plains-82 miles of roads were included under this programme at an estimated cost of Rs 26.8 lakhs. Rs 16.8 lakhs were spent during the plan period and the balance carried over to the next plan as spill over.

Important Non-Plan Works, after Independence and Within the Plan Period

The construction of Passi-Badarpur Road connecting Tripura with Assam, 82 miles in length, was taken up at an estimated cost of 110 lakhs.

Photo 1.

Gourang Bridge
R. S. Joist with
timber deck
10½'-6"
carriageway.

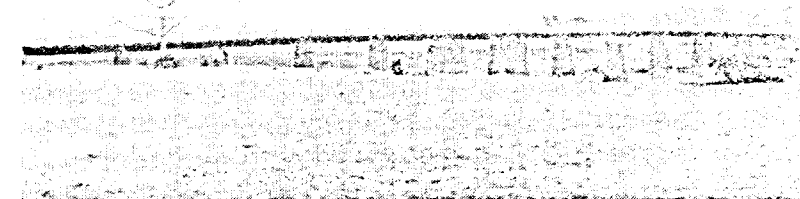


Photo 2.

A 200 ft long sal timber bridge
over Nonai River.

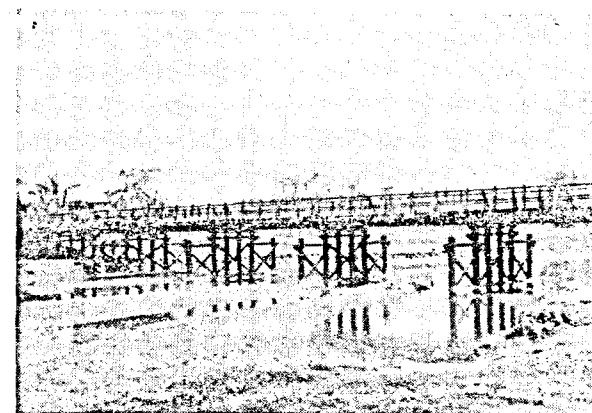
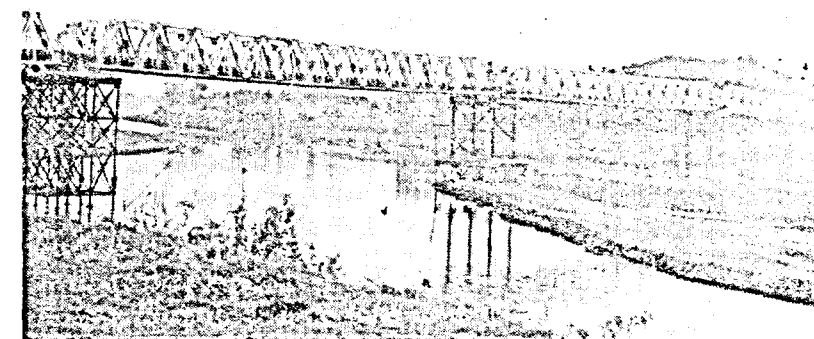


Photo 3.

Dhansiri Bridge
at Golaghat.



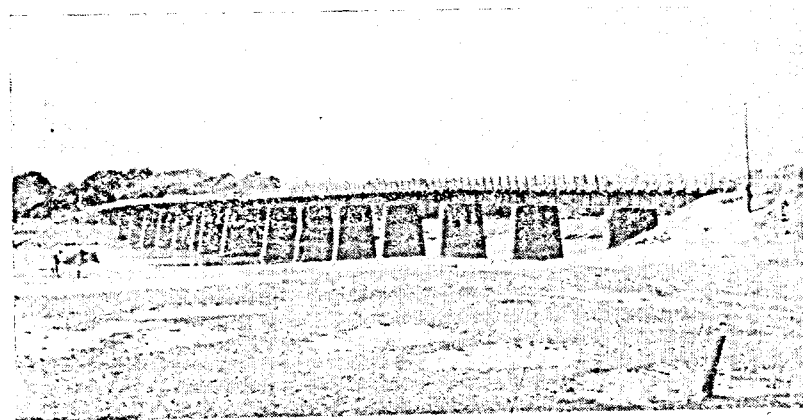


Photo 4.
Tipkal Bridge.

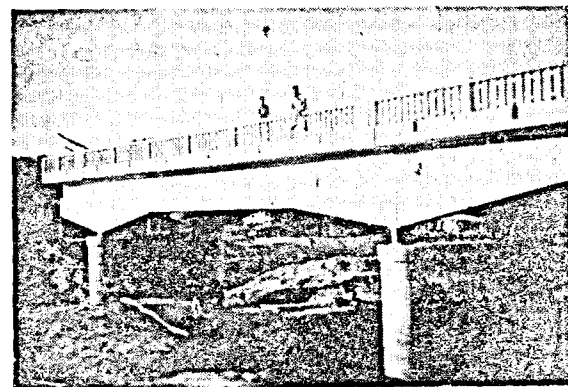


Photo 5.
Kullong Bridge at Nowgong.

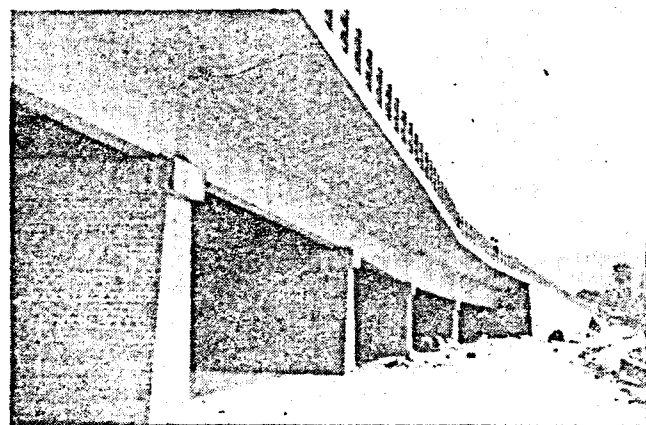


Photo 6.
Bridge over railway line
at Gauhati.

Before this road was taken up, construction of a road from Shillong to Silchar via Jowai-Garampani and Haflong, was taken up in 1947, immediately after partition, when Surma Valley was cut off from the rest of Assam, as a result of partition. The road is a fair-weather one. 2/3 of the cost of this road was borne by the Government of India.

The length of roads suitable for vehicular traffic, constructed after Independence under different programme is about 4,017 miles. Thus, a mileage of more than 150 per cent over what was in existence at the time of independence, has been added within 10 years. This is really a great achievement.

Grant-in-Aid Works During the Plan Period

Besides the above mentioned works, various other works have been taken up during the first five year plan period. These are mentioned below under grant-in-aid programme.

(a) Entire cost to be borne by the Government of India Rs 113 lakhs. This programme includes construction of Jia-Bhorali Bridge in Darrang District, at an estimated cost of 60 lakhs and improvement of North Trunk road from Benki to Sankosh—about 80 miles, at an estimated cost of 53 lakhs.

Trial boring works for Jia-Bhorali Bridge is in progress and works on the improvement of North Trunk Road are in full swing. Jia-Bhorali Bridge is likely to be the longest bridge in Assam, so far built, the proposed length being about 2,000 ft.

(b) 90 per cent cost to be borne by the Government of India 24.4 lakhs.

This includes metalling and surfacing of two roads in the Cachar District, which are of inter-State importance. The works are in progress.

(c) 50 per cent cost to be borne by the Government of India 62.94 lakhs.

The following works are included in this programme.

(i) Construction of Buroi Bridge on North Trunk Road (East)—1,060 ft in length Rs 19.69 lakhs.

(ii) Construction of Dikrong Bridge on North Trunk Road (East)—725 ft in length Rs 10.4 lakhs.

(iii) Reconstruction of Ranganadi bridge on North Trunk Road (East) damaged by the great earthquake of 1950-700 ft.....Rs 15.0 lakhs.

(iv) Construction of a railway over-bridge at Gauhati level crossing—1,143 ft.....Rs 7.37 lakhs.

(v) Approach road to Dikrong Bridge—18 miles.....Rs 10.50 lakhs.

Of these, re-construction of Ranganadi Bridge has been completed. Construction of over-bridge at Gauhati has also been completed; works on Dikrong Bridge and its approaches are in progress and tenders for Buori Bridge have been settled.

(d) 1/3 cost of improving about 165 miles of Local Board roads at an estimated cost of 9 lakhs to be borne by the Govt. of India, 1/3 by the State Government and 1/3 by the Local Bodies.

National Highway Works taken up During the Plan Period

Works amounting to Rs 99.13 lakhs have been undertaken for improvement of national highways during the First Five Year Plan period. This includes re-construction of five bridges, totalling 1,687 feet in length at an estimated cost of about 40.6 lakhs.

Bridges

In the State there are 321 major bridges totalling about 58,500 running feet. Yet, there are more than 105 boat ferries. These ferries present a serious bottlenecks to the free movement of road traffic. Priority was therefore been attached to the constuction of bridges during the First Five Year Plan period.

The Position of Bridging Activity (Major Bridges).

Particulars.	No.	Total Length in ft.	Cost in Lakhs.
Bridges completed since 1947.	8	2969 ft 6 in.	33.529
Bridges under construction.	14	6767 ft	113.768
Bridges to be started shortly.	5	1146 ft 3 in.	24.342

SECOND FIVE YEAR PLAN--STATE SECTOR

The following is the approved programme of road development during the Second Five Year Plan period.

(A) General State.**Total Allocation in Lakhs.**

1. Construction of roads (Village roads) and major bridges—spill-over works.	161. 00
2. Construction of roads (Village roads-new works.)	180. 27
3. Improvement of existing roads, black topping.	117. 41
4. Improvement of existing roads gravelling.	73. 66
5. Improvement of existing roads inspection bungalows, etc.	5. 16
6. Re-construction/improvement of existing bridges and culverts.	74. 12
7. Construction of major bridges—new.	38. 32
8. Purchase of steam vessels for crossing over Brahmaputra at Gauhati and Kamalabari	30. 00
	679. 94
9. Rural road development schemes (Aid to Local Boards and District Councils).	20. 00

TOTAL699. 94 or
700. 00**(B) Article 275 Schemes (Hill Areas)**

1. Spill over from First Five Year Plan	158. 00
2. New road schemes.	242. 00
	400. 00

TOTAL

400. 00

(C) Article 275 (Tribal Areas, Plains).

1. Spill-over from the First Five Year Plan	10. 00
2. New road schemes.	75. 84

TOTAL

85. 84

GRAND TOTAL

1185. 84

On completion of the Second Plan, a length of 1,872 miles of new roads will be added to the existing system bringing the total to 8,499 miles. In the Second Plan, stress has been laid on improving the existing roads where traffic is developing considerably, rather than on adding the length.

An additional programme of Rs 38.0 lakhs is being framed for new road works in hill areas. This will bring the total road length to about 8,668 miles, by the end of Second Five Year Plan.

Bridges in the Second Five Year Plan

The design and estimate for 16 new bridges estimated to cost Rs 157.55 lakhs are under preparation or about to be sanctioned. Besides these, collection of hydraulic data for 23 major bridges has been taken up.

A self anchored type stiffened steel suspension bridge, the first one of its type to be constructed in India is under construction across the river Barak, near Silchar Town in the southern part of the State. The cables are anchored to the stiffening truss instead of being anchored to separate firm foundations, so that the vertical component of the tension of the cable only comes into play as an external reaction, the horizontal component becoming a part of the internal forces of the whole system. The solution of this statically indeterminate structure has been arrived at by the elastic theory. The span being small the application of this

theory is considered to give sufficiently accurate results.

Second Five Year Plan—Central Sector

(a) **National Highways:**—A programme for Rs 138.76 lakhs has been approved for improvement of national highways during the Second Five Year Plan.

(b) **Other than National Highways:** A grant of Rs 6.50 lakhs have so far been made by the Govt. of India for improvement of communication in Assam, during the Second Five Year Plan.

Road traffic in Assam, is developing considerably and steadily.

The following expenditure figure on maintenance will give an idea.

Expenditure in Lakhs

Year	National Highway	Other State Roads
1951-52	33.50	64.00
1952-53	32.00	79.50
1953-54	33.00	91.50
1954-55	38.00	114.54
1955-56	38.08	157.50
1956-57 (budget)	32.46 (Final)	121.40

In the State floods occur regularly—particularly after the great earthquake of 1950 considerable amount is being spent for restoration of flood damages annually. For 1954 flood, the amount of estimate went upto 49 lakhs for State roads alone.

DEFINITIONS OF HIGHWAY ENGINEERING TERMS**CAUTIONARY SIGN**

A sign indicating to traffic, usually by means of appropriate symbols, of their approach to a place where caution is required.

DIRECTION SIGN

A sign indicating the direction to a particular locality.

ROAD SYSTEM IN WEST AUSTRALIA

AFTER the end of the First World War, in the early twenties, there was rapid development in land settlement in Western Australia, particularly in the wheat growing area in the south-west corner of the State. During the post-war years, to provide for the transport of products of the area, a railway construction programme was put on hand. As a matter of Government policy, the railways were being located 25 miles apart. Half of this distance was considered to be about the maximum for the transport of wheat by the horse wagon! Since 1918, 973 miles of new railways have been built in Western Australia.

In 1921, there was one motor vehicle to every 80 persons; the total number of vehicles was 4,181. In 1931, there was one motor vehicle to every 11 persons and the total number of vehicles was 38,621. In 1956, there was one motor vehicle for every 4 persons.

Considering the rapid increase in the number of motor vehicles throughout Australia, the Commonwealth Government, under the Commonwealth Main Roads Development Act of 1923 provided money for road construction to the various States on a 50:50 basis. In Western Australia, the Main Roads Act was passed in 1925. Under this Act, a Main Roads Board consisting of three members was formed. This Board was "required to make surveys and investigations to ascertain what roads shall be main roads, the resources of the State in road making materials, and the most effective methods of road construction and maintenance". The Board was also required to supervise the construction, improvement and maintenance of the main and developmental roads.

The Board in the first planning dealt only with the south-west portion of the country. The plan prepared envisaged the construction of 856 miles of developmental roads and 1,953 miles of trunk roads. The mileage of trunk and developmental roads at present is 3,457. During the last few years, however, a secondary

road plan has been prepared. In addition to the 3,457 miles, there is, at present a mileage of 6,929 secondary roads classed as important, Fig. 1.

Roads in Pastoral Areas of the State

During the early thirties, in the southern half of the State, roads were non-existent. There were tracks which were not even surveyed. Before any work could be done on these routes, it was necessary to map the alignments of the tracks. The unorthodox method described below was used as an expedient for mapping of the routes. "The tracks were traversed by speedometer and compass, preferably using two vehicles—one proceeding as far as it could go within sight of the other, the compass reading taken forwards by one driver and backwards by the other, and the speedometer distance recorded. Refinements were adopted to increase the accuracy of the speedometer readings. After plotting the traverses it was found that generally they fitted reasonably closely over lengths of 100 miles or more to known points at either end". This survey showed that these beaten tracks were formed in the days of the horse and camel wagons and that it was possible to connect places by more direct routes. As funds became available and traffic justified, roads in the pastoral areas were improved. The crossings of rivers on these tracks were unsuitable and, in many cases, investigations as far as 10 miles on either side of existing crossing, had to be done for locating the crossing on the direct route.

Low Cost Roads

The Department of Main Roads had made investigations, during the years of depression, on building low cost roads and conditions favouring the formation of corrugations. These investigations revealed that corrugations could be reduced by stabilization of the road pavement material, i.e. by mixing imported soil to obtain a suitable grading of the particles. In large areas of the sandy portions of

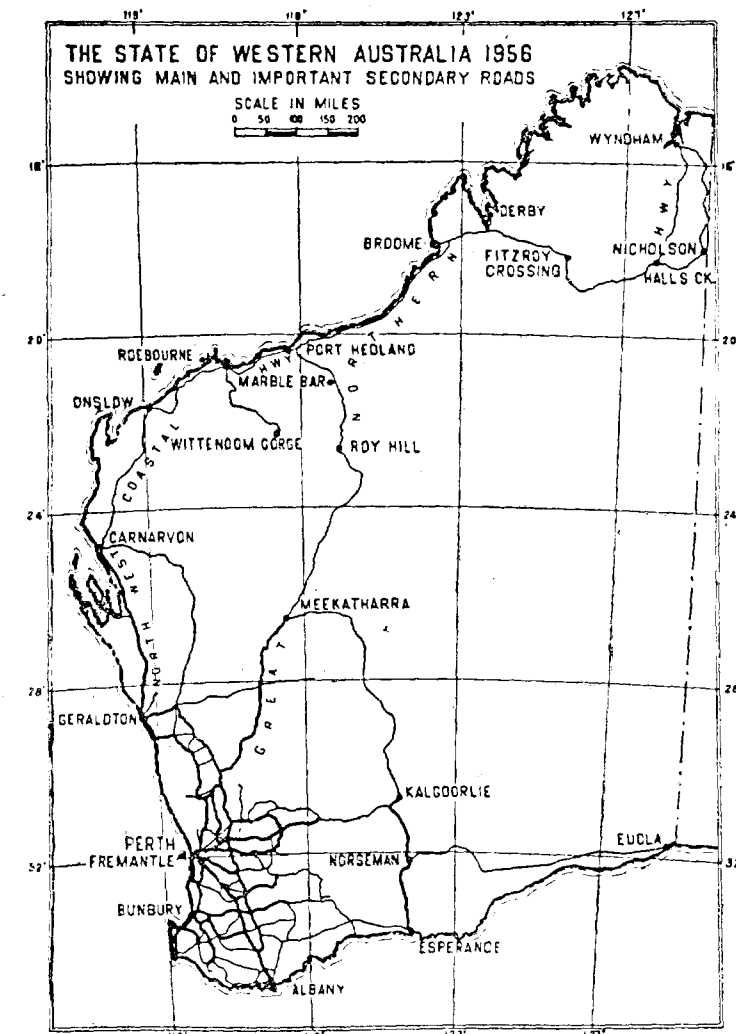


Fig. 1. Map of the State Main Roads and Important Secondary Roads.
Secondary roads are shown thin.

the country the soil consisted of particles of one dimension. By filling in the voids in the top 8 in. of the soil by the application of 2 to 3 in. of imported soil, remarkable results were achieved. The plasticity index of the imported soil ranged from 0 to a maximum of 9. It was also found that non-plastic mixtures with a dry compressive strength of as low as 100 lb per sq. in. gave very satisfactory results under light traffic. During the last 2 or 3 years stabilization of sandy soil in the area west of the roadway between Perth and Geraldton has produced pavements that are not prone to corrugate, Photos 1 and 2. In these stabilized surfaces, the

wind blown dressing of sand often gave protection to the pavement from the abrasion of traffic.

A low cost expedient used for traversing sandy areas is described below: Four logs of hard wood into which 1½ in. diameter steel bars protruding about 2½ in. were driven in a screw pattern, were attached to the rear of a tractor by swivel bolts. As the tractor moved forward this implement known as rut drag produced two parallel ruts, tearing the old surface, roots, etc. Photo 3. It revolved slowly and left a good running surface at the bottom of the rut. Motor



Photo 1. Showing nature of sandy country.

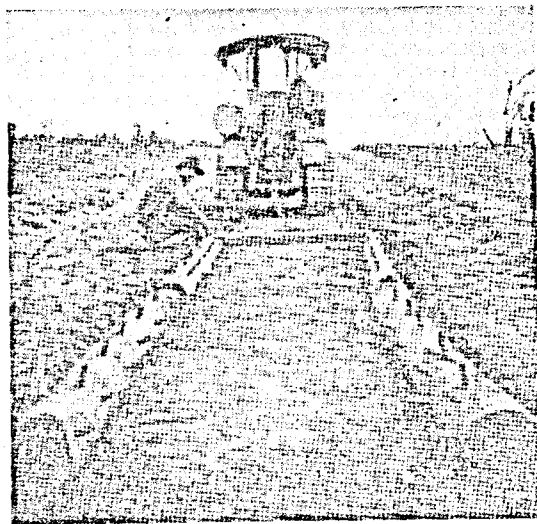


Photo 3. A rut drag hauled by crawler tractor.

Table 1

Mechanical Analysis of Materials Used in Road construction in The Carnarvon Delta

Size	Gascoyne River sand	Silty clay	silt	Dune sand
	Per cent passing			
3/8 in.	100			
3/16 in.	97			
No. 7	85	—	—	100
14	53	—	—	—
25	19	—	100	—
36	9	100	99	88
52	3	90	98	—
100	—	61	81	—
200	—	52	24	40
Dust Ratio	—	52	25	45

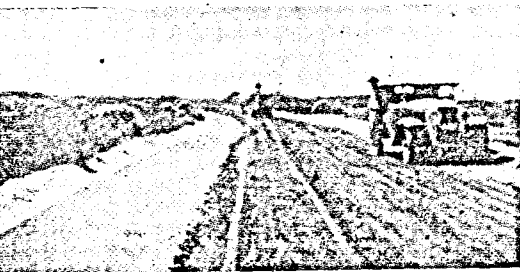


Photo 2. Stabilisation in progress.

traffic could use the rut so formed at about 40 miles an hour.

Stabilized Soil Specification

In the silty areas of the State in the Carnarvon area 600 miles north of Perth the tracks were dusty, corrugation prone and much erosion was caused by strong prevailing winds. Mechanical stabilization was successful in the area. But there was excessive wear of the pavement. Therefore, it was eventually decided that a bituminous protective seal would be an economical proposition. The grading of the materials found suitable for the purpose are given in Table 1.

When the bituminous seal was first introduced, coarse river sand was used as aggregate. Later on, it was found that a granular horn-shell 3/4 in. long and 1 1/4 in. base provided a good affinity for the bitumen, better resistance to abrasion, and being white in colour had a lower road temperature, giving a longer life to the binder. The base course speci-

fications for use with thin bituminous seals are given in Table 2.

Table 2.

Natural River Sand Silt Clay Mixture

	Depth 1 ft 6 in.— 2 ft 6 in.	Depth 2 ft 6 in.— 3 ft 6 in.

Grading:

per cent passing		
3/8 in.		100
3/16 in.	100	99
No. 7	99	96
No. 14	91	89
No. 25	73	73
No. 36	57	57
No. 52	42	39
No. 100	24	16
No. 200	21	13
0.0135 mm.	18	11
Dust ratio	37	23
Liquid limit	19.3	N. P.
Plastic index	13.7	
Linear Shrink.	5.6	

Cheap Enrichment Coat for improving Life of a Black Top

In Western Australia, laterite gravels are found throughout the country and are extensively used for road making. These gravels are relatively soft and are crushed even under moderately heavy wheel loads. As a result, the pavements with these aggregates wear very quickly. Light bituminous surface treatments have been given to prevent the excessive wear of the laterite gravel roads. Some of these light surface treatments have given good service for 10 to 15 years. It has been

found that the life of the bituminous treatment can be prolonged as they age by the application, from time to time, of a relatively cheap enrichment coat with aggregate cover. This method consists of applying a bituminous emulsion with the addition of water of 1 volume to 4 of emulsion to the old black-top surface. This dilution conditions the surface tension of the mixture so that it penetrates into the fine cracks of the old seal rather than bridging them.

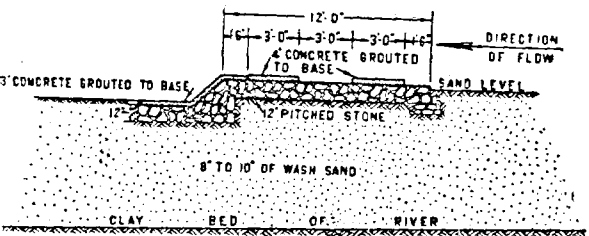


Fig. 2. Typical Sandy Creek Crossing on the North-West Coastal Highway

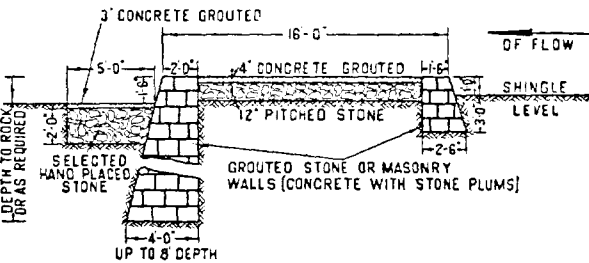


Fig. 3. Typical Crossing over the Aehn River and Forest River on the Wyndham-Nicholson Road.

Typical sandy creek crossings adopted in Western Australia are shown in Fig. 2 and 3.

From "Journal of the Institution of Engineers, Australia, December, 1956.

REAL SERVICE

Real service must be a cardinal element in true professional practice. Without positive decision or dedication on the part of the individual service as a profession would not exist.

E. R. NEEDLES.

SOME REMARKS ON BITUMINOUS SURFACE DRESSINGS*

By

C. G. SWAMINATHAN

Factors Influencing the Success of a Surface Dressing

The various factors that influence the quality and the life of a surface dressing can be broadly divided into the following:

- (1) Binder
- (2) Aggregates
- (3) Construction

These are dealt with in detail below:

Binder

The following four variables can be considered:

- (a) quality,
- (b) viscosity,
- (c) quantity,
- (d) application.

Quality of the Binder

The success of a surface dressing depends on the proper wetting of the road surface and embedment of the aggregate in the fluid binder. In order that it may perform these functions it should be capable of flowing at the temperature of application and at the same time set quickly to develop the necessary bond to hold the aggregate. To withstand the disturbing effect of traffic, it should have reasonable resilience or ductility as well.

The various binders that can be used for the purpose are tars, asphaltic bitumens, cut-backs and emulsions. Tars containing high proportions of lower boiling oils, reasonably high penetration grades of asphaltic bitumens and cut-backs, which cure rapidly but leave a highly viscous residue, are suitable.

Quick setting high viscous emulsions can also be used with advantage.

Tars are liable to weathering and oxidation as a result of which they may show signs of brittleness earlier than asphaltic bitumens. But when two course surface dressings are applied simultaneously, the effect of oxidation in the first course can be very much reduced. Some have a notion that softer asphalts, since they are not over-refined, have a longer resistance to oxidation. But, because of the accurately controlled and scientific methods of manufacture, the harder grades of asphaltic bitumen are not 'burnt', so to speak, and hence both hard and soft grades of asphalt have more or less similar oxidation characteristics. It has also been proved^{1, 2} that the source of the crude, nature, production and the method of manufacture of various binders do not have any pronounced effect on the general behaviour, condition or the life of surface dressings. And it can safely be said that so long as the binders employed satisfy the general requirements, such as given in Table I, page 25, all types of high temperature tars and asphaltic bitumens prepared by different methods from various crudes can be used to yield good results.

Here, it is interesting to note that whereas in the U. S. A. and in the U. K. there are at least six to ten different grades in each category of binders such as tar, asphaltic bitumen and cut-backs, in India hardly one or two grades in each type are available. Compared to the U. K., the country is very large and a very wide range of climatic and other conditions exist. It is but reasonable to expect that a greater number of grades in each of the various types of binders to cater to all the needs in surface dressing operations should be specified and made available in India.

Viscosity of the binder

To wet and coat the aggregate, the binder chosen should be such that its viscosity would be low initially. When cooling takes place, it should increase rapidly in order that a proper bond may be achieved. Since a compromise between the two apparently conflicting requirements is necessary, it is obvious that the viscosity of the binder chosen should depend on factors such as the road temperature at the time of construction, weather, traffic, etc.

A high viscous binder would be preferable during hot weather, for use with smooth and round aggregates, large size stones, and for surfaces called upon to carry a heavy traffic, whereas a low viscous binder is required during wet weather for use with crushed aggregates, softer stones, and for low traffic loads. Also, a lower viscous binder is suggested if the binder is supplied in barrels instead of in bulk.

The use of a soft asphalt does not lead to bleeding. Irrespective of the fact whether a hard or soft grade is used, bleeding can occur as a result of unsuitable aggregates, use of insufficient quantities of aggregates, or an improper ratio of the quantity of binder to the quantity of aggregates used and lack of proper control during construction. Bleeding is caused by weather and traffic conditions. A harder grade only causes a short delay in the process of bleeding.^{2, 3}

Use of tars of too high a viscosity results in difficulty in spraying and this prevents adhesion to the chippings. Viscous tars may also become too hard on the road during the winter and lead to loss of chippings. Too low a viscosity permits disturbance of chippings when freshly laid.⁴ If the viscosity of the binder is too low (especially for tar) in the initial stages of application, some disturbance of the aggregate may take place initially but after some time with the loss of volatile oils, viscosity would be increased and the surface would be satisfactory. For the same reasons, binders should not be overheated at the time of construction.

Asphaltic bitumens having the same penetration do not necessarily have the

same fluidity at the identical temperature of application. So, viscosity should be the main criterion rather than the penetration, though, generally, a lower penetration might mean a more viscous material.

Hank and Brown² found it difficult to wet and embed aggregates with a harder grade of bitumen. They also found that harder grades have ultimately an advantage in that their stronger bonds tend to prevent displacement of aggregates once they have been embedded, and that higher percentage of embedment is permitted by the softer grades initially.

On a comparison of the relative efficiencies of straight run bitumen, cut-backs and emulsion, Hank and Brown² have found that the retention of aggregates is better with the heaviest cut-back used than that obtained with the highest grade of asphaltic bitumen. For a given rate of application of binder, too much of fines in the aggregates hinders the proper wetting and embedment of the larger aggregates more with straight run bitumen than with emulsions or cut-backs. Also, liquid asphalts yielding a viscous residue compensate more readily any differences in the nature of aggregate than straight run asphaltic bitumens, since, for example, retention of siliceous gravel and a crushed rock was of the same order when the binder was a cut-back while with a straight run bitumen the retention of the crushed rock was better than that of the siliceous gravel. All this goes to indicate that the viscosity of the binder both at the time of application and after wetting influences the performance of the dressing very much. The temperature of application should be such that the viscosity falls within a certain range so that a good embedment could be obtained initially. The critical temperature for the purpose differs with different binders. The lower this temperature the better. In cut-backs and emulsions it is low, because of the presence of carrier medium, namely, solvent for the former and water for the latter. But the final strength depends on the final viscosity attained by the binder after setting. In cut-backs, the rate at which this is developed and the duration for development depends on the volatile oils. The sooner the cut-back

* Continued from page 9 of April 1957 issue of T. C. M. R.

cures the better and from this point of view Hank and Brown² suggest a curing index of 20 or below as quite a satisfactory figure. Nevitt⁴ suggests that the solvent shall be a naphtha cut with a boiling point between 300° and 400° F. An analysis of the binders available in India shows that the cut-backs have a residue of penetration more than 100 and straight run bitumen of 80/100 have been used very extensively. The above mentioned requirements of the binder, namely, adequate fluidity initially and a high viscosity when it sets, suggest the need for a cut-back grade with 80/100 or even harder grades such as 60/70 penetration, that is a rapid curing high viscous cut-back. Similarly, the need for a high viscous quick setting emulsion is also indicated. Perhaps in the colder regions in the north of India, harder grades might become brittle and crack in winter with a resultant tendency for the embedded aggregate to be lost. This, it is felt, has yet to be proved by actual observations or factual determinations. If the construction is done properly in hot weather with a hard binder, cracks may not develop in winter.

Quantity of the binder

The quantity of the binder employed should meet the following requirements:

- It should flatten to provide a uniform layer.
- It must maintain the intended thickness rather than run off.
- It must provide the required embedment of aggregates.
- It must penetrate into the voids of the old surface.
- It should work up in the aggregate sufficiently during the curing period to provide aggregate embedment.
- And at the same time it must not creep up after the penetration and filling of voids in the aggregate layer are over.

Lee¹ has shown that for surface dressings especially with tar, the rate of spreading of the binder is the most important factor governing the life and general behaviour. The correct

rate of binder will depend upon the size and the shape of the chippings used and to a less extent on the traffic intensity and the nature of the old surface⁵. Choosing the rate of binder spread is largely a matter of experience and the local road and traffic conditions should be taken into consideration.

Rounded gravel aggregates require a greater quantity of binder than crushed aggregates because of the greater void volume between particles. Flaky aggregates require less binder because of their thinness of embedment. The lesser rates of binder are applicable to roads rich in binder and higher quantities of binder are required for roads which are dry and starved or porous such as a water-bound macadam. A greater quantity of binder is required for lightly trafficked roads such as roads carrying traffic of less than 2,000 tons per day, whereas, a thinner rate is suggested for heavy trafficked ones, since heavy traffic secures proper embedment and compaction.

Theoretically, although it may be said that the quantity of binder employed should be sufficient to give a bitumen level of half to two-third the individual aggregate height, the exact requirement is difficult to state, since some binder would penetrate into the old mat depending on its porosity. Also, a certain loss due to evaporation of oils might take place. However, when the aggregates have attained a shoulder to shoulder packing under the action of traffic, the binder is required to form a film between each stone and the road and between stone and stone. The difference between excesses and deficiency of the binder is represented by only a relative small change in the quantity of binder used. A deviation of 20 per cent from the correct quantity could give rise to unsatisfactory results, bleeding if in excess and scabbing if deficient⁶. The binder quantity should not be too low to effect negligible embedment nor should it be too high to completely submerge the aggregates.

Let Qa = quantity of aggregate in cu. ft. per 100 sq. ft.

Qb = quantity of binder in lb per 100 sq. ft.

t = thickness of mat in inches.

Assuming that the specific gravity of binder to be equal to 1, that aggregates are all of one size, and that the depth of binder after embedment equal to d inches = e times thickness of mat in inches.

- X = depth of original layer of binder in inches

$$X = \frac{12}{6,250} Qb$$

$$t = \text{one stone layer} = \frac{12 Qa}{100}$$

$$\frac{Qb}{62.5} = \frac{100 et}{12} = eQa$$

$$\text{So, } e = \frac{1}{62.5} \cdot \frac{Qb}{Qa}$$

For a value of $Qa = 5$ and $Qb = 45$, it is seen that the depth of original layer = .0864" and the depth of embedment is about 1/7th of the mat thickness.

Ritter and Paquette⁸ suggest the approximate quantity of binder required in gallons per sq. yd. = 1.1 of the aggregate in lb per sq. yd. and the quantity should be sufficient to fill up 50 to 70 per cent of the voids in the aggregates.

Nevitt⁷ has worked out another formula on the assumption that the level of the bitumen is at the centre line of the aggregate. The bitumen obviously occupies the unit area to half the particle height and utilises the corresponding volume less half the retained aggregate volume. So if—

B = bitumen residue volume for embedment as specified per unit of road surface area,

D = parameter dimension of the aggregate particle taken on the screen opening,

K = shape constant of the aggregate,

V = net volumes of the aggregate particles retained on surface per unit of road area,

N = number of particles per unit area,

K_n = parameter constant of the aggregate so that particle height = $K_n D$,

K_h = parameter constant of the aggregate so that particle volume = $K_h D^3$

Now $V = N \cdot K_h D^3$

and B = (allowing ten per cent allowance)

$$= 1.10 \left(\frac{K_h D}{2} - \frac{V}{2} \right)$$

So $B = 0.55 (K_h D - N K_h D^3)$

This clearly indicates that for any given size of aggregate, such as ' D ', the binder quantity required depends on the height and volume constants of the aggregate. This equation indicates fairly the reason for the change in the quantity of aggregates required for flaky or rounded aggregates. The equation does not hold good for graded aggregates where the voids will be filled both by bitumen and the smaller particles.

The above equation also shows, in a way, that as the size of the aggregate used decreases the quantity of binder decreases as evinced by the A.S.T.M. Tables 2 and 3 pages 25, 26 indicate the quantities of binder that are usually specified in most of the States in India. Table 4, page 27 indicates an American practice. Tables 5 and 6, page 26 & 27 indicate a British practice (for tar mainly). For cut-backs, slightly higher quantities, 5 to 10 per cent more, should be used. For emulsions, a suggestion³ is to use 1/100th of a gallon per sq. yd. per pound of retained aggregates allowing an addition of 1/10th gallon per sq. yd. for coating the pavement, on the assumption that aggregates weigh about 95 lb per cu. ft. and about 80 to 90 per cent of applied aggregates are retained.

While comparing the quantities of binder employed, for the corresponding sizes, abroad (vide Tables 2, 3, 4 and 5), it can be seen that, in India, there is a tendency to use a higher rate of binder. It is because of this that most of the surfaces have a smooth black top and also appear richer as compared to the surfaces in the west, where the aggregate faces in the surface dressing are quite noticeable on the surface. Repeated applications of a higher quantity of binder would cause corrugation on the road and progressive failure takes place. Comparison with practices abroad indicates the need for a slight reduction in the rate of application of the binder. As mentioned earlier, slight difference in the quantity of binder could make all the

difference between a good and a bad surface.

Method of Application of the Binder

Application of the correct quantity of binder required for unit area alone would not yield a good surface. The quantity should be spread uniformly over the entire surface. The success of a surfacing depends on the efficiency with which uniformity of application in both the transverse and longitudinal directions of the road is achieved. In places where the binder has been less, the aggregates have a tendency to come out early and in the other places, where the binder is too much, the surface is fat with the result that potholes are formed in some places and bleeding occurs in others. Non-uniformity of the binder layer would

cause undulations which would accentuate subsequent formation of corrugations.

It is equally obvious that the binder should be applied at the proper temperature in relation to the weather and temperature of the road. For, application at a higher or lower temperature not only impairs the adhesion to the original surface but also to the chippings.

As stated by Lee¹, the only reliable means of achieving a specified rate of spread of binder is by the use of good mechanical sprayers fitted with necessary indicators and which would spray the binder at the required rate under uniform pressure and at the same speed over the entire width and length of the road.

(To be continued)

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Cutback Bitumens	—	217-1951
Digboi Cutback	—	454-1953
Road Tar	—	215-1951

Table 1
Binder for Surface Dressings*

Nature	Grades
Bitumen	Penetration, 40/50; 60/70; 80/100.
Tar	Road Tars No. 1, 2, 3.
Cutbacks	RC-1, RC-2, RC-3, RC-4, RC-5

*For details refer to the Indian Standard Specifications — Reference 10.

Table 2
Single Coat Surface Dressing Practice in various States of India

Agency	Binder	Rate of Binder lbs 100 sq. per ft.	Size of Aggregate inches	Rate of Aggregate cu. ft per 100 sq. ft.	Remarks
C.P.W.D.	Tar No. 3 Bitumen	45	$\frac{1}{2}$	$5\frac{1}{2}$	Chippings— 100 per cent passing $\frac{3}{4}$ in. 90-100 per cent passing $\frac{1}{2}$ in. and 0-15 per cent passing $\frac{1}{4}$ in.
Bombay P.W.D.	Tar No. 2 Bitumen 80/100	27 37	$\frac{3}{4}$ - $1\frac{1}{8}$ $\frac{5}{8}$ - $\frac{3}{8}$	3 4-5	
U.P. P.W.D.	Tar	41-45	$\frac{3}{4}$ - $1\frac{1}{4}$	$4\frac{1}{2}$	
Madras P.W.D.	Bitumen 80/100	53	$\frac{3}{4}$ - $\frac{3}{8}$	6	
Rajasthan P.W.D.	Bitumen Tar	45 36	1 - $1\frac{1}{4}$ 2mm - .2mm	$3\frac{1}{2}$ $3\frac{1}{2}$	
Assam P.W.D.	Tar no. 1 & 2	27	60 per cent-1 40 per cent- $\frac{3}{4}$ in. size	— —	Passing $\frac{3}{8}$ in. should be rejected
Roads Wing Ministry of Transport	Bitumen	45	$\frac{3}{4}$ - $\frac{3}{8}$	5	

Table 3

Two Coat Surface Dressings—Practice in the various States of India :

No.	Agency	First Coat				Second Coat				Remarks
		Binder	Rate of Binder lbs/100 sq. ft.	Size of Aggregate Inches	Rate of Aggregate cu/100 sq. ft.	Binder	Rate of Binder lbs per 100 sq. ft.	Size of Aggregate Inches	Rate of Aggre- gate cu. ft per 100 sq. ft.	
1.	C. P. W. D.	Bitumen Tar no. 3	40 40	$\frac{1}{2}$ $\frac{1}{2}$	5 5½	Bitumen Tar	25 25	3/8 3/8	3¼ 3¼	For ½ in. chippings 100 per cent passing ¼ in., 90-100 passing ½ in. and 0-15 per cent passing ¾ in. Similarly for ¾ in. and 0-20 per cent passing ½ in. 90-100 per cent passing ¾ in.
2.	Bombay P. W. D.	Bitumen 80/100	37	5/8-3/8	4-5	Bitumen 80/100	28	3/8-1/4	2-3	
		Tar no. 2	27	3/4-1/8	3	Tar 2	14	3/8-1/8	1½	
3.	U. P. P. W. D.	Bitumen Tar	41-45 41-45	3/4-1/4 3/4-1/4	4½ 4½	Bitumen Tar No. 2	23-29 22-28	1/2-1/4 1/2-1/4	2¼ 2¼	Not less than 3 months 2nd coat.
4.	Madras P. W. D.	Bitumen 80/100	25	3/4-3/8	6	Bitumen 80/100	45	3/8-No.8	3-4	
5.	Rajasthan P. W. D.	Bitumen Tar	45 36	1-1/4 1-1/4	3½ 3½	Bitumen Tar No. 2	25 18	1/2-1/8" small grit 1/2-1/8"	2¼ 2¼	Passing ¾" to be rejected. 2nd coat to be laid at least 2 months after first coat.
6.	Assam P. W. D.	Tar no. 1 & 2	25	60 per cent 1 40 per cent ¾	—	Tar 1 & 2	25	60% 3/4" 40% 1/2"	—	
7.	Roads Wing Ministry of Trans- port	Bitumen	40	3/4-3/8	5	Bitumen	25	3/8-1/8	3	

Table 6

A Typical British Practice for quantity of Binder and Aggregate
(Reference 5)

Normal Size Inches	Aggregate in Sq. Yds. per Ton	Binder (Tar) Gallons per Sq. Yd. for angular chippings
3/4	79—80	.30—.22
1/2	90—100	.22—.17
3/8	100—120	.18—.14
1/4	140—170	.15—.12

BUSINESS LIKE

Motorist "I ran over your cat and want to replace him."

Housewife "Well, get busy. There's a mouse in the pantry right now"

Table 4

A Typical U.S. Practice
(From reference)

Binder in gallons per sq. yard and aggregate in lb. per sq. yard						
	1	2	3	4	5	6
First Layer :						
Quantity of Binder	0.30	0.22	0.25	0.15	0.30	0.20
Size of Aggregate	1/2"(down)	1/2"(down)	3/4"(down)	1"(down)	1"(down)	1 1/2"(down)
Quantity of Aggregate	25	25	35	40	50	70
Second Layer :						
Quantity of Binder	—	0.13	0.25	0.30	0.35	0.40
Size of Aggregate	—	3/8"(down)	3/8"(down)	1/2"(down)	1/2"(down)	3/4"(down)
Quantity of Aggregate	—	10	15	12	20	20
Third Layer :						
Quantity of Binder	—	—	—	.15	—	.20
Size of Aggregate	—	—	—	3/8"(down)	—	3/8"(down)
Quantity of Aggregate	—	—	—	8	—	12
Fourth Layer :						
Quantity of Binder	—	—	—	—	—	.20
Size of Aggregate	—	—	—	—	—	3/8"(down)
Quantity of Aggregate	—	—	—	—	—	8
Total :						
Quantity of Binder	0.30	0.35	0.50	0.60	0.65	1.00
Quantity of Aggregate	25	35	50	60	70	110

Table 5

A Typical British Practice for Quantity of Binder and Aggregates
(From Reference)

Rate	First Coat	Second Coat	Third Coat	Total Thickness inches
Size or gauge	1/2"-3/4"	1/8"-3/8"	—	1 1/2
Aggregate per ton	120 Sq. Yd.	150 Sq. Yd.	—	
Binder per gallon	4-8 Sq. Yd.	4 Sq. Yd.	—	
Size or Gauge	3/4"-1"	1/4"-3/8"	1/8"-1/4"	1
Aggregate per ton	30-40 Sq. Yd.	150-180 Sq. Yd.	180 Sq. Yd.	
Binder per gallon	4-8 Sq. Yd.	3 Sq. Yd.	4 Sq. Yd.	
Size or Gauge	3/4-1 1/2"	1/4"-3/8"	1/8-1/4"	1 1/2
Aggregate per ton	20-30 Sq. Yd.	150-180 Sq. Yd.	180 Sq. Yd.	
Binder per gallon	4-8 Sq. Yd.	2 1/2 Sq. Yd.	4 Sq. Yd.	

HOW INDIA MAY USE PHOTOGRAMMETRIC AND ELECTRONIC DATA-PROCESSING DEVICES

By

OLIN W. MINTZER*

Synopsis

With an enabling act in prospect, American highway engineers decided that despite the man-power shortage a faster means of getting the highway improvements into being was necessary. The "bottleneck" seemed to be in the construction-plan preparation stage. Many organizations had already started with effective techniques, some of which are enumerated. However, Ohio seems to have the best answer, i.e. the "marriage" of photogrammetry and electronic computers. The techniques of this combination of recently developed tools is described in detail. A suggestion of how the American techniques may be applied in India is made.

Introduction

The purposes of this article are to outline some of the techniques by which Highway Engineering Departments in America are meeting the challenge of modernizing American roads, and to suggest how some of these techniques may be applied in India. Recently, last July, the American Government authorized the expenditure of billions of dollars for better roads. When this legislation was being passed considerable attention was given to the man-power requirements involved. For the past few years America has been troubled with a shortage of professional engineers. With the expansion of industrial as well as governmental operations the technical man-power shortage was keenly felt. Engineering organizations, therefore, were looking about for more effective means than were currently in practice. After due study it was decided that to get highway projects under construction as soon as the money became available, the plans for these projects must be made ready quickly; that is, to make sure that the new road programme would get underway without delay, construction plans for Highway improvements would need to be prepared at a much faster rate. It was seen that the current usage of engineering man-power could not produce the plans fast enough. Thus new ideas or combinations of old ideas were created out of this great need.

After due study of the problem it was seen that many routine tasks performed by professional engineers could be performed by sub-professional personnel; project planning and design work could be given on contract basis to private consulting engineers; much of the survey data taken by field crews could be obtained by means of aerial surveys and photogrammetric techniques; electronic data-processing machines could be utilized to compute earthwork quantities, pay rolls, and tabulate a host of statistical data; drafting and tabulation of quantities in plan preparation could be simplified and improved; and a combination of the above techniques could be made to reduce the great need for professional engineers, thus releasing them for more creative thinking and design tasks. The following paragraphs describe briefly what some of the States are doing; a detailed discussion of the techniques now being tried in Ohio is given and finally how the techniques may be applied in India is suggested. Many organizations both public and private are using such techniques as enumerated below to make possible the more effective use of professional man-power. Some of the procedures as are applied in Arizona, California, Delaware, Louisiana, Ohio and Oregon have enabled the States to cut costs and save months of precious time.

When the expansion of the U. S. highway program became effective with the

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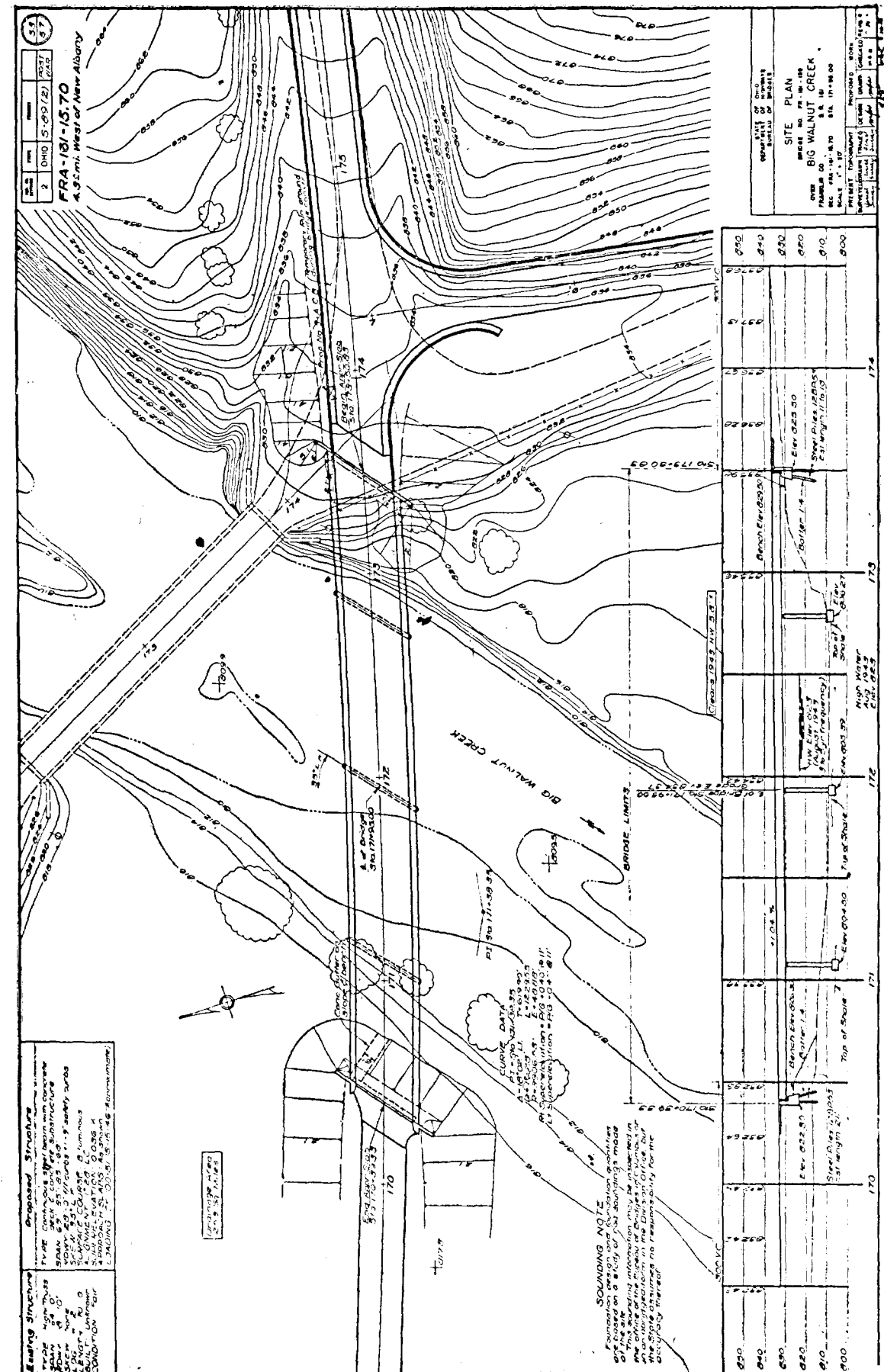


Fig. 1. A Bridge Site plan prepared utilizing photogrammetric techniques (Courtesy, Ohio Department of Highways)

"Highway Act of 1956", highway organizations in the States, had already looked around for new methods, new techniques, or tools or both. Delaware was speeding up construction plan-preparation by simplifying the plans and eliminating non-essential elements of the plan such as those for roadway widening and relocation projects. In addition, the department was employing engineering consultants to do a great deal of the design and plan preparation.

Arizona was making effective use of the Univac 120 Electronic Computer in routine financial and statistical reports in handling pay rolls and inventory control; to reduce the time-consuming computations of checking of longitudinal-section data and for computing areas and volumes of earthwork quantities.

Louisiana was making use of automation equipment in much the same manner as Arizona. The data, from survey notes taken in the field, were calculated by means of the Univac 120 to determine the earthwork quantities.

Ohio was also keen to make improvements to enable the faster production of construction plans despite the man-power shortage. Several steps were being followed: (1) improving the department organization to make more effective use of the professional engineer, (2) improving employment practices, (3) using private consulting engineers to prepare designs and construction plans, (4) improving and simplifying drafting procedures and developing typical designs more applicable throughout Ohio, and (5) using aerial surveys and photogrammetric techniques to prepare topographic maps, to prepare soil and drainage surveys, to determine longitudinal-section and cross-section data, and to prepare bridge-site data. It was found that the cost of preparing bridge-site data photogrammetrically, was, on the average, less than 500 rupees, whereas ordinary ground methods cost at least ten times that amount and required weeks of survey time. (See Fig. 1.

The Oregon Highway Department had freed professional engineers from many of the tedious, time-consuming computations involved with earthwork and construction pay quantities by using the high-

speed electronic computers to reduce the hard-to-get engineering man-hours by some 5,000 per year. The Oregon Department spent only about rupees 125/- per mile to do the computing that formerly cost many times as much.

Combining Photogrammetry and Electronic Computers

After some research and study of current tools and techniques, Ohio was not satisfied that the steps being taken, already enumerated above, would be sufficient to offset the man-power shortage and produce the construction plans as fast as necessary. Hence, it was decided that the combination of photogrammetry and electronic computers should be brought into the construction-plan preparation. Photogrammetry may be defined as the art and science of using aerial photography to determine the distances between, directions to, and elevations of all objects of interest in a given area on which the highway improvement is to be constructed. There are three basic operations in photogrammetry applied to highway engineering (1) flying and processing the photography (2) procurement of the ground control, (3) studying of the photography for quantitative and qualitative measurements.

Ohio engineers have found an instrument called the **Kelsh Plotter** is most suitable for the quantitative measurements. This instrument has been adopted for use with the combined use of photogrammetry and electronic data processing devices. See Fig. 2.

The **Kelsh Plotter** is an instrument normally used in the examination of stereopairs of diapositives at negative scale of aerial photographs in the process of producing large-scale topographic maps. The function of the Plotter is to produce a three-dimensional image of the ground, which is formed by the stereoscopic model from the fusion of the images of two overlapping diapositives. One blue image and one red image each is projected to form a spatial model. The spatial view seen through complementary colored glasses which match the projected images may then be measured and delineated. The projection of the images

occurs at a magnification of four times the diapositive image. In this step of the process the Plotter is used primarily to obtain the cross-section data at right

order, and after processing gives the answer on a card form. The flow pattern for processing data in the simplest terms, begins with input data. Once the data is

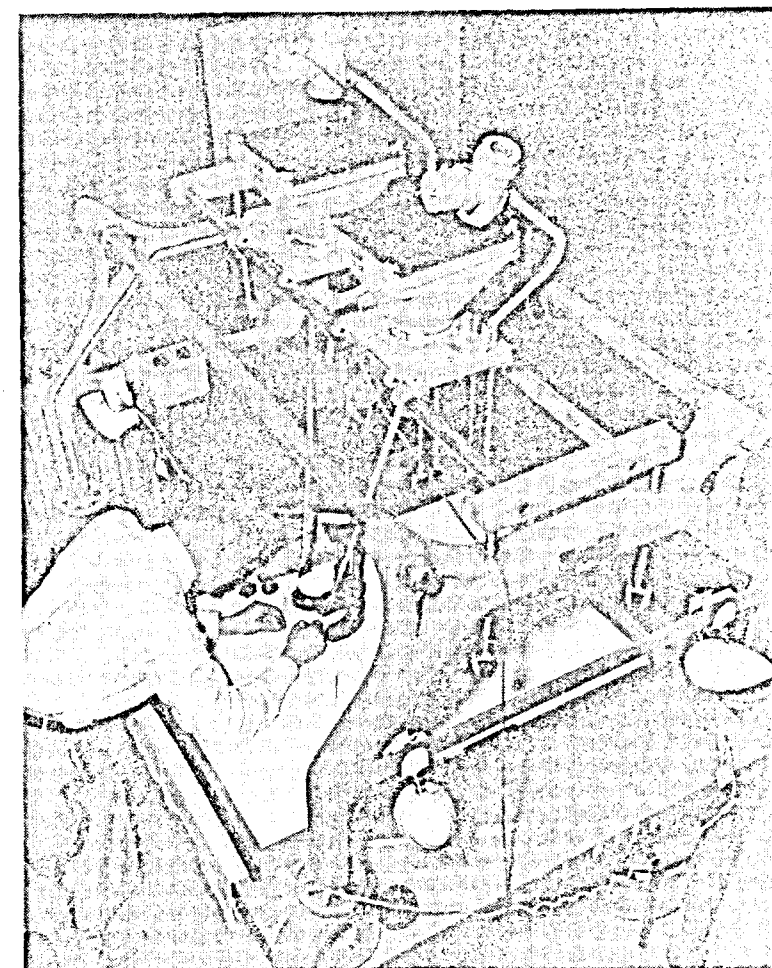


Fig 2. The Kelsh Plotter in Operation (Courtesy. Ohio Department of Highways)

angles to centerline, i.e., road elevation and offset distance from the centerline of survey. These data are recorded on cards in the form of ground coordinates.

The electronic computer used is known as the 650 Magnetic Drum Data Processing Machine manufactured by the International Business Machine Corporation. There are three units of the 650; the Read-Punch, the Console, and the Power Unit. The entire assemblage handles the data in a card form, in any

coded it passes to the arithmetical unit, then to the magnetic drum. After the computing effort the information becomes output data. The machine operates exactly in accordance with instructions given to it on coded cards. When calculated results will be needed in other steps these are stored on the magnetic drum until needed. There is one limitation; the data, including the coded instructions, may not require more than 20,000 digits, the storage capacity of the magnetic drum. All data being processed by the IBM 650

circulate on and off of the drum as per instruction. The arithmetical operations performed are addition and subtraction. Therefore, if operations of multiplication or division are required, the 650 processes such data by repetitive addition or subtraction. See Fig 3.

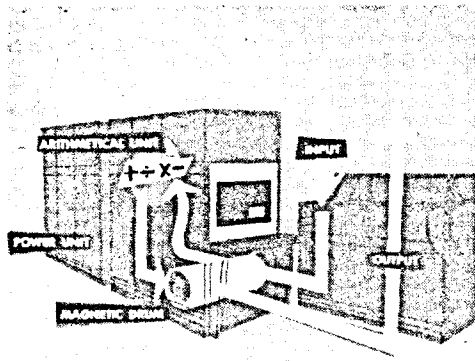


Fig. 3 Magnetic Drum data processing machine type 650 (Courtesy International Business Machines.)

After the aerial photographs have been processed, field surveys of the centerline for profile data, and the alignment design have been completed and processed, the images on diapositive plates are projected as in aerial mapping and examined by the Kelsh Plotter operator for data of the cross-section of the terrain at right angles to the centerline at frequent intervals. These data are recorded in the form of coordinates which describe the position of ground elevation above some datum and the offset distance left or right of centerline. The plotter operator by means of a terrain data recording machine automatically codes the terrain data on cards. Then the initial proposed roadway section data and the terrain data, are read and collated by the IBM 650 datum data processing machine. From the latter step the output data, coded-cards are read into a machine which produces a facsimile of the ground-roadway section; then an automatic exposure is made on microfilm. This microfilm is viewed as a miniature portrayal of the cross-section. A microfilm viewer then affords an enlargement of the cross-section to about the scale seen in the construction plans. The design engineer observes the microfilm view of

the roadway cross-section, at which time, he modifies the design and returns the data for coding and preparation of a new proposed roadway section. Any new design features are incorporated, collated, and processed as described above. The new design is then observed on the microfilm viewer; modifications again may be necessary; the data are fed back, and once more the sequence of events is set in motion. This feed-back procedure is repeated until the design engineer is satisfied with the terrain-roadway design conditions. Enlargements of the microfilm views of a few special sections are generally made of the final design for inclusion in the plans. All the coded terrain data, proposed section data, and the machine instruction, at this stage, are on cards. These cards are read by the IBM 650 for computation and tabulation of pertinent quantities to be utilized for estimation of roadway costs. Finally, the output data from the IBM 650 and associated printing devices are automatically reproduced for incorporation into the construction plans.

How the techniques may be applied in India.

This article has reviewed some of the important developments in America to prepare road construction plans more rapidly despite the professional engineer shortage. India is not similar to America in that India has not yet developed a roads program to the extent of the United States, although such developments are possible in time. Yet this author has observed that there are some similarities, in that there is a shortage of professional engineers. (This is especially true in the Public Works Department Buildings and Roads Branch.) In addition there is a definite lack of large-scale topographic maps where industrial and public work developments are planned. With the expansion of governmental and industrial operations now taking place under the second five year plan Indian engineers may want to make use of some of the techniques described above. These techniques may be summarized as follows:—

1. Transfer routine tasks from professional to sub-professional engineers.

2. Transfer on a contract basis the design and plan preparation for roads projects to private consulting engineering firms.
3. Prepare large-scale topographic maps for use in highway planning and design using photogrammetric techniques.
4. Prepare soil and drainage surveys for engineering and planning purposes by utilizing aerial photography.
5. Prepare bridge-site maps by using photogrammetric techniques.
6. Use electronic data processing devices to tabulate statistical data.
7. Modernize the drafting and tabulation of quantities in plan preparation.
8. Compute earthwork quantities by means of electronic data processing devices.
9. Wherever necessary make combined use of photogrammetry and electronic drum data processing machines.

Summary

This paper has been written to give its readers in India a chance to review the techniques American highway engineers are using to speed up the preparation of construction plans despite the shortage of professional engineers. Since there are also large industrial and public works expansion programs in India some of the techniques described may be equally useful in India. The procedures described above apply to certain stages in the highway improvement program leading up to the plan preparation. However, it is to be noted that the combined use of photogrammetry and electronic computing is also applied during the preparation of the preliminary engineering studies, and in the final surveys and tabulation of the the contractor pay measurements.

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HERE AND THERE

1. ADVANTAGES OF COMMITTEES

● **ORIGINALLY**, the committee system was created out of the idea that two heads are wiser than one. That proposition may possibly be true; but there is no evidence at all to suggest that ten or twenty heads are even better still—rather the reverse. The lively go-ahead man of ideas more often finds so many heads obstructive. The real purpose, excellently served, of a committee is to scotch opposition at birth. For it is a curious point in the psychology of most men that they object less to a committee decision with which they disagree than to a decision, even though obviously the right one, taken without any hearing being given to their views! Thus a good chairman, even when it is obvious to him what a decision ought to be, even when he knows that those few members of a committee

who have bothered to think about it beforehand agree with him, will still insist upon a discussion of alternatives. For he knows there is nothing more likely to arouse ire and opposition amongst the rest than a suspicion that a proposal has been bulldozed through the committee. Necessarily this business of reaching agreement is time consuming. Yet when, at the expense of giving a hearing even to stupid and misconceived comment, an agreement is reached, it usually proves lasting and acceptable amongst all those people or organisations represented on the committee. No body cares, once the decision has been taken, to act in direct opposition to it.

(From 'The Engineer', January 11, 1957).

2. THE VALUE OF RECORD KEEPING

● **THE WORK** of any investigator is not completed merely by the discovery and interpretation of facts. These facts and interpretations must be embodied in some concrete and tangible form, be it a drawing, model or a report, in order that they may be introduced into the common fund of knowledge and to permit their use as a basis for possible future action.

Quite often the effectiveness with which the results of an investigation can be applied is dependent as much on the clarity and completeness of the presentation as on the accuracy of the data and the acumen with which they are interpreted.

It is obvious in the particular case of road works that no one observer can possibly be in touch with every job that is being carried out, so that the importance of records being kept is particularly apparent here in order that details of very many jobs, all of the same type, may be later studied, and any common factors which seem to contribute to success or failure extracted, with a view to testing whether this is really so or not.

To do this questions must be asked and answers obtained. Individual minds react differently to the same set of circumstances, and it is therefore desirable to reduce the possibilities of variability in both observing and recording data to a minimum. This is where a standardised form becomes the most suitable means, so that all observers enter the same data under the same heading, so facilitating the removal of whatever information is required for purposes of comparison.

It is desirable that all standard forms of record, such as have been considered, be reviewed at fairly long intervals (four to five years) to see whether they require modification. The usefulness of the data being collected can then be assessed and a decision reached upon whether to add to, or subtract from in future.

All records should be adequately classified and be immediately accessible when required.

(From 'Road Tar', London, March, 1957).

3. PRECAST CONCRETE BRIDGES IN THE U. S. S. R.

● **THE** following is an interesting review of railway and road bridges, constructed in precast concrete.

Railway Bridges

The transport of precast concrete bridge units from the production centre to the site is much simplified in the case of railways. The maximum overall dimensions of precast members are limited only by traffic regulations and the weight of a single unit can reach 100 tons or more. The great load carrying capacity of railway cranes permits the use of heavy members. Short and medium span bridges of the beam type have been developed with a maximum span of 75 ft (23m) in the case of prestressed concrete. A typical section of a single track railway bridge is shown in Fig. 1.

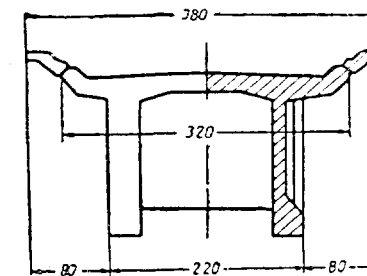


Fig. 1

The unit is designed to span up to 54 ft (16.5m) and weighs about 76 tons. An interesting feature of the design is a hinged connection between the ballast guard and the deck. Ballast guards are bent inside during transportation in order to bring the width of precast unit within the limits of the required railway clearances. The main difficulty in assembling these units lies in the provision of a firm support at four points. Cracks in diaphragms and deck slab are bound to occur if the supports are not correctly placed in one plane. Attempts to introduce precast concrete piers and abutments have proved to be unsuccessful both for economic and structural reasons.

Road Bridges

With the introduction of prestressing, precast concrete bridges of 137 ft (42m) span have been constructed.

Precast Concrete Arch Bridges

When foundation conditions are favourable, precast concrete arch bridges have considerable economic advantages over beam type bridges. An open spandrel three-hinged arch is usually adopted for precast concrete bridge designs. The method of assembling the precast members depends on the length of span and on local conditions. The design department of the Ministry of Transport has developed an arch bridge of 492 ft (150m) span. The arch consists of precast concrete box-shaped segments which are assembled on standard steel trusses. In this case, the arches will be precast in two segments and will be floated into position on barges. The weight of one segment is about 2,000 tons. A similar method of erection will be adopted in the construction of a number of bridges with arches spanning from 131 to 328 ft (40 to 100 m). Another method of assembling the arches is shown in Fig. 2 where a railway crane with counter-balance is used.

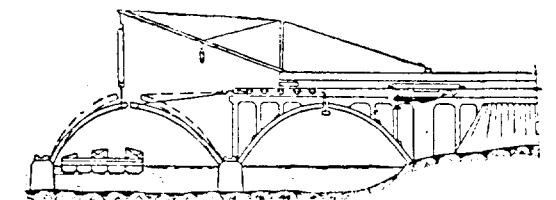


Fig. 2

This method of erection was used in the construction of a railway bridge and proved to be highly successful. The bridge was designed as a three-hinged arch, spanning 92 ft (28m) and consisting of two precast segments, each weighing 50 tons. The labour required for erection was 74 man-days per bay. The superstructure for the bridge, which was 787 ft (240 m) long, was assembled in 68 days. In railway bridges the deck is supported on rigid precast concrete frames, which are placed transversely, while in the case of road bridges, precast concrete posts and beams are used.

(From 'Civil Engineering and Public Works Review', Vol. 50, July, 1955).

4. AERIAL PHOTOGRAPHY IN AMERICA

● AERIAL photography is being used extensively for highway location in the United States of America. The selection of alignment for a highway with the aid of aerial surveys may be said to consist of four stages.

The first stage consists of examining vertical photographs to a scale of 1:30,000. Generally, for this stage work there is need to make separate aerial surveys. Photographs available with other departments like the Geological, Irrigation and Land Reclamation are used. The equipment necessary for the examination is very simple and consists of only micro-stereoscope and scales. Expensive equipment like the multiplex and Kelsh Plotter are not required. By this preliminary examination, all the feasible alternative routes that are worthy of consideration are obtained.

In stage 2, the alternative routes considered feasible are examined either by

reconnoitering on the ground or by examining large-scale aerial photographs by means of stereoscope and checking on the ground for critical points.

The third stage consists of detailed survey of the selected route either on the ground or by means of photogrammetry for preparing a topographical map of the alignment. The fourth stage consists of marking of the route on the ground.

In the U.S.A., many firms have quoted lesser amounts for photographic surveys than for preparing maps by the conventional methods. Measurements on the ground at selected points have shown that the survey by photographic methods is as accurate as ground surveying, and several State Highway Departments have photogrammetric equipment for the purpose of surveys.

(From a report by Shri N. S. Surya on his training in the U. S. A.)

5. JOINT SPACINGS IN CONCRETE PAVEMENTS

IN 1940-41, several experimental concrete pavements were constructed in six States of the United States of America for studying the spacing of joints. The experimental sections consisted of plain and reinforced portions. The plain concrete sections had expansion joints at 120, 400, 800, and 5,280 ft with contraction joints at 15 to 25 ft. The reinforced sections had expansion joints at 120 ft with one intermediate contraction joint. Load transfer devices were used at all expansion joints.

Since the construction of these measurements, observations have been made on the following:

- (1) Daily and seasonal variations in temperatures;
- (2) Daily, seasonal and progressive or permanent changes in the widths of contraction and expansion joints;

- (3) Measurements of faulting at joints, if any; and
- (4) General condition of pavements.

After observations covering over a period of nearly 10 years, the data obtained in the different States have been analysed. The Highway Research Board Report 17-B "Joint Spacing in Concrete Pavements" contains reports from the six States, viz., California, Kentucky, Michigan, Minnesota, Missouri and Oregon, as also an analysis of data from the State reports.

These observations showed that in pavements with expansion joints spaced at intervals considered to be suitable with contraction joints at close intervals, there was a tendency for the expansion joints to close up progressively. The contraction joints had a tendency to progressively open with time. These movements were rapid in the early life of the pavements.

Within a few years, the movements were sufficient to destroy the aggregate interlock at the contraction joints. Wherever expansion joints were eliminated or spaced at large intervals, there had been no tendency for the contraction joints to open progressively.

On the basis of 5-years reports of these experimental pavements, practice in many

of the States has changed. Practically every State in America has now eliminated expansion joints in non-reinforced concrete pavements. This has resulted in a greater resistance to pumping and faulting because of the aggregate interlock in the contraction joints.

(From 'Highway Research Board Research Report 17-B' 1956.)

6. TESTING THE CONSISTENCY OF CONCRETE—A FIELD APPARATUS

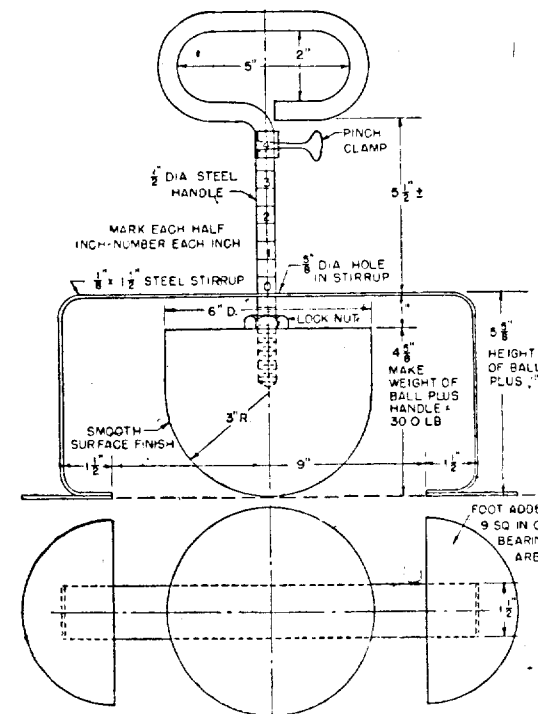
● A TEST known as the Kelly Ball has been developed by Professors Kelly and Polivka of the University of California, for the purpose of finding out the workability of concrete in the field. The apparatus originally developed has been modified by the Bureau of Public Roads to make it more adaptable for field conditions. The modified apparatus is shown in the figure below.

Description of the Apparatus

The apparatus consists of a solid right cylinder 6 in. in diameter, one end of which is machined into a hemisphere. The height of the cylinder and the hemisphere is $4\frac{1}{2}$ in. It is fitted with a graduated vertical rod $\frac{1}{2}$ in. in diameter and which serves as a measuring scale as well as a handle. The vertical portion of the handle is graduated. The ball is guided by a stirrup or a frame $1\frac{1}{2}$ in. in width and is made of $\frac{1}{8}$ in. steel sheet. There are two feet, 9 sq. in. in area each, fitted to the bottom of the stirrup. The stirrup is $5\frac{1}{2}$ in. in height, i.e. one inch more than the height of the ball. The zero of the graduations in the handle coincides with the top of the stirrup when the bottom of the ball rests on a level rigid surface. The weight of the ball and the handle is 30 pounds. To facilitate reading of the depth of penetration of the ball, a vertically movable pinch clamp is attached to the vertical handle.

Use

The surface of the concrete to be tested is smoothened and levelled by means of wooden float. The ball is held vertical by the handle with very light contact with the levelled concrete surface and with the zero in the rod coincident with the top of the frame. The handle is then released and the depth of penetration of the ball into the concrete is noted to the nearest tenth of an inch. A minimum of three readings are generally taken. Tests made with this Kelly ball apparatus show that the penetration readings obtained with the apparatus have a correlation to the slump values. The ratio of penetra-



Ball-penetration apparatus for consistency of concrete, Bureau of Public Roads modification.

tion to slump in one series of test was 1 : 1.5, whereas in two other projects it was 1 : 1.3. Experience has also indicated that the minimum depth of concrete when this test is employed should be at least three times the maximum size of the coarse aggregate used. The following advantages are claimed for this method of test:

1. The concrete is tested at site and, therefore, the selection and preparation of a sample are eliminated.
2. The time required to make this test is less than that required for a slump test.
3. Whereas the slump test is not practi-

cal for use when testing concrete with aggregate more than 2 in. in size, the Kelly ball could be employed if sufficient volume is available to provide adequate depth.

Experience has indicated that while testing with this Kelly ball, the spot selected should at least be at a distance of 9 in. from the face of the forms to avoid boundary effects.

(Digested from an article on "Use of the Kelly Ball for Field Measurement of Concrete Consistency" by William E. Grib and Robert A. Marr, Jr.—Highway Research Board, Bulletin 132 on Concrete Control and Construction.)

7. AVOIDING FAILURES OF EXCAVATIONS, SLIP-OUTS AND SLIDES

● SOIL is a complex material which the engineer has to deal with. Principles of soil mechanics developed during recent years have, to a great extent, provided the necessary knowledge for the engineer to understand the behaviour of the soils.

Failures of excavation, slip-outs and slides are caused when the stresses in the soil exceed its strength. Anything which increases the stresses or reduces the strength of the soil could cause failure. Often, water is the contributing cause. The following are some of the principal conditions leading to increased soil stresses;

1. An increase in the cut depth. This increases the load on the soil in the bottom face of the cut, increases the soil tension at the top and increases the unbalance in the bottom.
2. Saturation of soil by water which increases its weight.
3. Water pressure in cracks in the soil which tends to open them up and increase the horizontal stresses.
4. Weight of excavated soil and machinery on the ground surface adjacent to the cut.
5. Shock and vibration due to heavy machinery and blasting.

Decreases in soil strength may result from the following:

1. Prolonged stress, particularly tension. The soil slowly deforms plastically until it finally breaks.
2. Excess water pressure or upward flow in sandy soils.
3. The swelling of some clays due to water.
4. The drying out of damp cohesionless soils such as fine sands, which destroys their capillary forces and cohesion.
5. Breakdown of loose, unstable arrangements of soil grains.

The first step in reducing failures is to determine the characteristics of the soils by sampling and testing it for physical characteristics. The second step would be to pay particular attention to the drainage of the area. Before a slip occurs, generally the adjacent ground subsides. Tension cracks develop at the upper surface of the cut. Oftentimes, small spalls or stone pieces come out of the face of the cut indicating that the soil is being plastered.

(From "Roads and Streets", January 1957.)

8. ACCIDENTS AND ROAD LAYOUT

● PERCENTAGE reduction in accidents observed in the U. K. after improvements to roads is given below:

(i) Junctions:

- | | |
|----------------------------|--|
| (a) Staggering of junction | 85 per cent |
| (b) Roundabout | 60 per cent |
| (c) Improved visibility | 30 per cent |
| (d) Traffic lights | 40 per cent |
| (e) Halt signs | 85 per cent (vehicles from out of side roads). |

(ii) Improvement of slip-perry surface

80 per cent reduction in skidding accidents but there may be a 20 per cent increase in other types.

(iii) Improved alignment in bends

70 per cent

(iv) Dual-carriageways 30 per cent

(v) Introduction of street lighting 35 per cent reduction in accidents at night.

(vi) Zebra crossings 70 per cent reduction in pedestrian accidents.

(vii) Replacement of trams by buses 60 per cent reduction in P. S. : V. accidents (London).

Pedestrian guard rails, introduction of speed limit, one-way systems, would tend to reduce accidents. Also, a reconstruction of a road on the same line would tend to increase accidents.

(From "British Road Federation, London, March 1, 1957").

9. CREOSOTED WOOD TO PROTECT CONCRETE PILES IN SEA WATER

● THE DETERIORATION of precast concreted piles in the tide waters in and adjacent for periods ranging up to 33 years by means of the placement of pressure creosoted wood jackets in the tide zone. In contrast to this record, in a structure comprising about 1,000 unprotected piles in the same tide waters, over 970 required extensive and costly repairs.

The lumber for the jackets was pressure creosoted for two basic reasons: (1) to protect it against decay in the above-water and in the alternately wet-and-dry area in the tide zone; and (2) to protect it against the activities of marine wood borers which infest these waters.

One of the structures investigated was

the Cross-Bay Boulevard Viaduct built in the Borough of Queens in 1923. It contained some 3,500 precast reinforced concrete piles, all jacketed with double layers of 2-in. yellow pine plank about 10 ft long, pressure treated by the full-cell process to a retention of 16 lb of creosote per cu. ft.

Records reveal that these piles, 24-in. square with chamfered corners, were made of 1 : 1 : 2 concrete, and were cured 60 days before driving. The piles were painted with a bituminous paint in the tidal section before the timber jackets were installed. The jackets extend from a point 3 ft above high water through a 5 ft tidal zone to a point 2 ft below low water, and are fastened to the piles by

three metal straps of $\frac{1}{2} \times 2$ -in. galvanized wrought-iron secured by $\frac{1}{2}$ -in. galvanized bolts and nuts.

Inspection of 1,303 jacketed piles at another viaduct was made during periods of low tide in October, 1956. This inspection revealed that the creosoted lumber

in the jackets after 33 years is still sound, and that the concrete in the jacketed zone is in excellent condition. No jackets were missing although about 30 had slipped down from 2 to 8 ft.

(From 'Engineering News-Record, Feb. 14, 1957').

10. INFRA-RED CURING OF CONCRETE

● AN interesting note on the application of infra-red radiation to the curing of concrete was published in the Russian periodical "Beton i Zhelezobeton" for July 1956. The author, B. V. Spektor, carried out a number of tests in which the concrete specimens were subjected to infra-red radiation. The results show that this new curing technique affects early concrete strength in a much shorter time than in the case of the usual steam treatment. It has been found that the maximum temperature of heat treatment should be 90°C. There is a loss of strength of concrete at temperatures below and above 90°C. Concrete subjected to the isothermal heat treatment for 2 to 4 hours will attain 50 to 70 per cent of its designed strength. An admixture of calcium chloride to the infra-red cured concretes has much the same effect as in the case of steam curing; that is, additional strength.

A rapid initial temperature rise during the infra-red treatment is stated to have no bad effects on the strength of concrete. The explanation of this phenomenon can be traced to the absence of internal stresses which are created during the steam treatment by the opposing currents of steam and air. The application of infra-red curing in the manufacture of concrete products was tried on hollow slabs about 10 ft $\times$ 4 ft $\times$ 5½ in. (302 $\times$ 119 $\times$ 14 cm) in size with nine 4-in. diameter holes. One heater was placed inside the end of each hole and the slab was heated for 4 hours. The temperature was maintained between 86° and 93°C. The strength of this concrete after 4 hours was equivalent to the strength of the steam cured concrete after 12 hours. The amount of electrical energy necessary to heat one panel was 44-kw-hr.

(From 'Highway Research Abstracts, February 1957')

WILL A SECRETARY GO TO HEAVEN

If a Secretary writes a letter, it is too long. If he doesn't send a notice, he is lazy. If he attends a committee meeting, he's bulling in. If he stays away, he is a shirker. If he duns the member for dues, he is insulting. If he fails to collect dues, he is slipping. If he asks for advice, he is incompetent. If he does not he is bullheaded. If he writes his reports complete, they are too long. If he condenses them, they are incomplete. If he talks on a subject, he is trying to run things. If he remains quiet, he has lost interest in the meetings. Ashes to ashes, dust to dust; if the others won't do it, the Secretary must.

CURRENT LITERATURE

ABSTRACTS

FLEXIBLE PAVEMENT DESIGN CORRELATION STUDY—Bulletin 133, Highway Research Board, Washington, D. C.

THIS bulletin contains a report on a correlation study of design methods made by the Chairman of the Committee on Flexible Pavement Design. In it are presented details on how nineteen state highway departments and other agencies, all using the same materials of construction, would design pavements for a given set of conditions as to climate and traffic. It also compares the designs which would be proposed in the event that the nineteen different agencies were designing the pavements still using the same materials, but building the pavements for use within their respective geographic regions.

ROAD ROUGHNESS AND SLIPPERINESS—SOME FACTORS AND TEST METHODS. Bulletin 139—Highway Research Board, Washington, D. C.

THIS Bulletin contains five Papers presented at the HRB 35th Annual Meeting, January 1956.

The first Paper, by Gale Ahlborn and Ralph A. Moyer deals with modifications to the BPR roughness indicator as developed in California, together with some test results using the equipment on actual pavements.

In the second Paper, B. R. Petrok and K. L. Johnson describe modifications developed in Minnesota for the same instrument.

Virginia experience with skid resistant pavements is described by F. P. Nichols, J. H. Dillard and R. L. Alwood in a paper giving several different methods of test, and the results of stopping distance tests

made at several hundred locations in that state. An appendix contains valuable notes on Virginia geology and aggregate sources.

In a paper describing similar work, H. L. Michael and D. L. Grunau discuss the program of developing skid testing in Indiana and the results of tests in that state.

How passenger car speeds are affected by driver reaction to wet and dry pavement conditions was the object of a New York State study reported by Walter Stohner in the fifth paper.

CURRENT PRACTICE QUESTIONNAIRE JOINTING, FILLERS, SEALERS AND EQUIPMENT. A. A. Anderson, Bulletin 138, Highway Research Board, Washington, D. C.

DURING 1955 the Committee on Joint Materials in Concrete Pavement sent out a questionnaire to various highway departments and federal agencies. The questionnaire requested current information concerning jointing practices, types of fillers and sealers as well as equipment used for preparing the joints, heating and applying sealing materials, particular emphasis being placed on sawed contraction joints which are rapidly becoming standard practice.

This report summarizes the replies received from 43 states, the District of Columbia, Hawaii, Department of the Army, Navy and the Civil Aeronautics Administration. However detailed replies which could be tabulated were received from only 38 states. The results are believed to be interesting and informative. The complete tabulation of the replies is contained in this report.

VIBRATIONS AND STRESSES IN GIRDER BRIDGES—Bulletin 124, Highway research Board, Washington, D.C.

THIS publication contains six Papers sponsored by the Committee on Bridges and presented at the 34th Annual Meeting.

The first Paper presents measurements of dynamic deflections in five typical designs of simple-span girder bridges. Amplitudes of vibration at mid-span were 18 to 40 percent of the maximum static deflections, and the principal causes of vibration were found to be the dynamic characteristics of the vehicles and the conditions of bridge approach pavements.

The second Paper describes three kinds of dynamic effects of live loads on highway bridges: (1) impulses due to speed with load rolling smoothly; (2) shock resulting from irregularities in deck or approaches; (3) resonance effects of repetition of loads which may combine with or interfere with the natural frequency vibration of a structure.

In the third Paper, the applications of new electronic strain recording equipment are described and the results obtained in testing two three-span continuous plate girder bridges are presented.

(Continued from page 9)

cracks in the wearing coat parallel to the axis of the bridge and along the central line of the wearing coat. Therefore, in the Spans where the wearing coat was laid subsequently, a longitudinal expansion joint was provided in the wearing coat. It was observed that while stressing the cables, the longitudinal joint opened slightly and there were no cracks.

The hand rails consist of cast in situ R.C.C. posts, 8 in. square, spaced at 10 ft 2 in. centres with precast panels.
Load Test.

The next paper is a report on studies of vibration of a three-span continuous I-beam highway bridge. Data from both static and dynamic loading are presented and curves are developed for use in computing the natural vibration frequency in structures having girders of uniform cross-section.

Paper number five reports on a study of vibrations and deflections in an eight-span bridge which had previously been tested for static load effects. Three spans were of continuous-beam design. The other five were simple spans.

The last paper describes research at the University of Illinois on the highway bridge impact problem.

THE EFFECT OF JET AIRCRAFT ON AIRPORT PAVEMENTS, by G. Mc Fadden, Transactions of the American Society of Civil Engineers, Volume 121, 1956.

THE heat and blast of jet airplanes may damage an airport pavement unless it is properly designed and constructed. Spillage of fuel may also affect adversely an improperly built pavement. This paper describes tests determining the extent of these effects on bituminous concrete and portland cement concrete. In the light of the results, measures to guard against such damage are described.

The bridge was designed for I. R. C. Class A or two lanes of I. R. C. Class AA loading whichever produced higher stresses.

The third span from the North Lakhimpur side was tested. A knife edge load of 1 lakh lb consisting of 1,000 bags, each weighing 100 lb distributed over a width of 20 ft 4 in. and a distributed load was applied. A set of 4 deflection recorders was used to record the deflections, Fig. 4, Plate II. It was found that the deflections were well within the calculated values.

(Contributed)

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X. BRIDGES AND OTHER STRUCTURES

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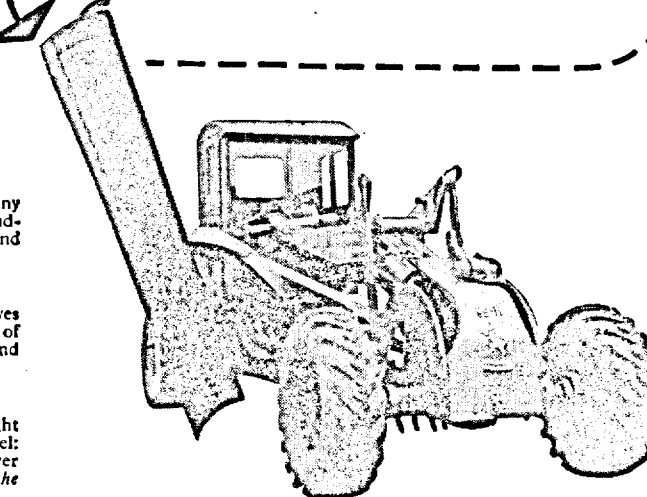
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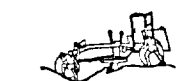




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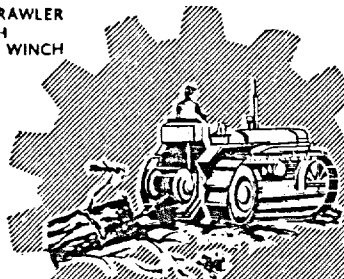
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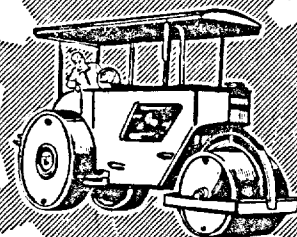
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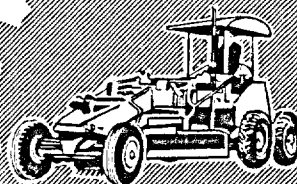
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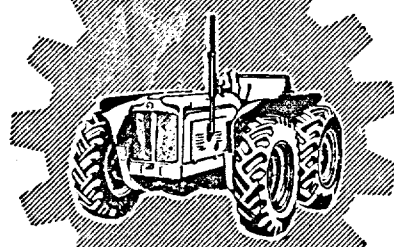
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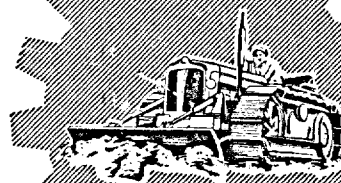
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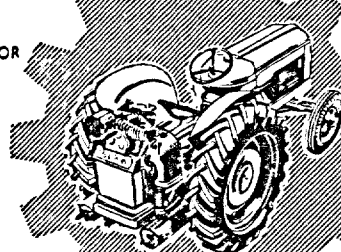
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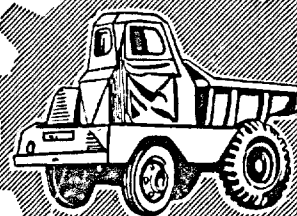
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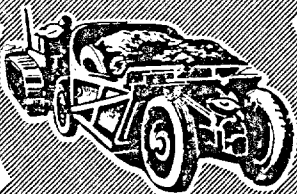
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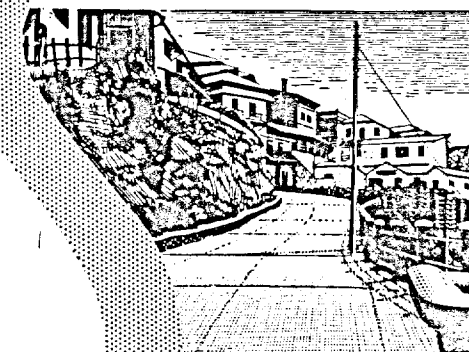


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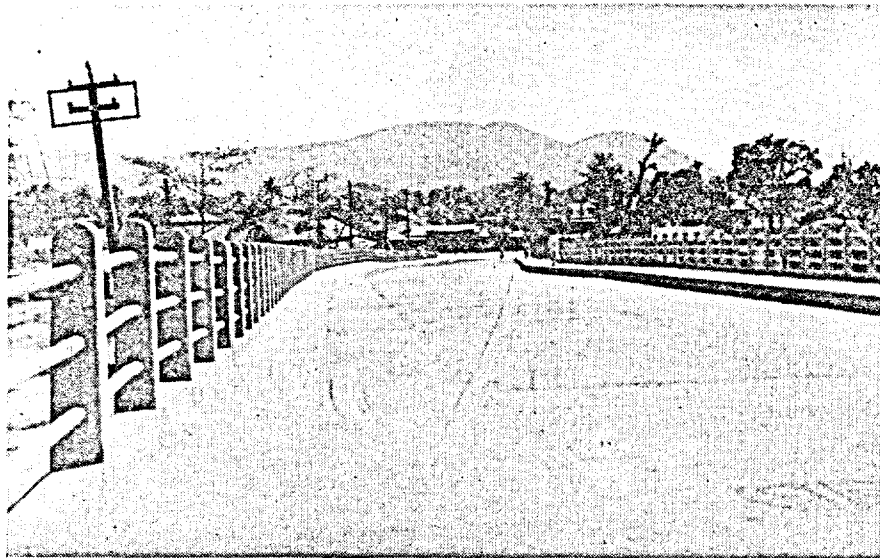
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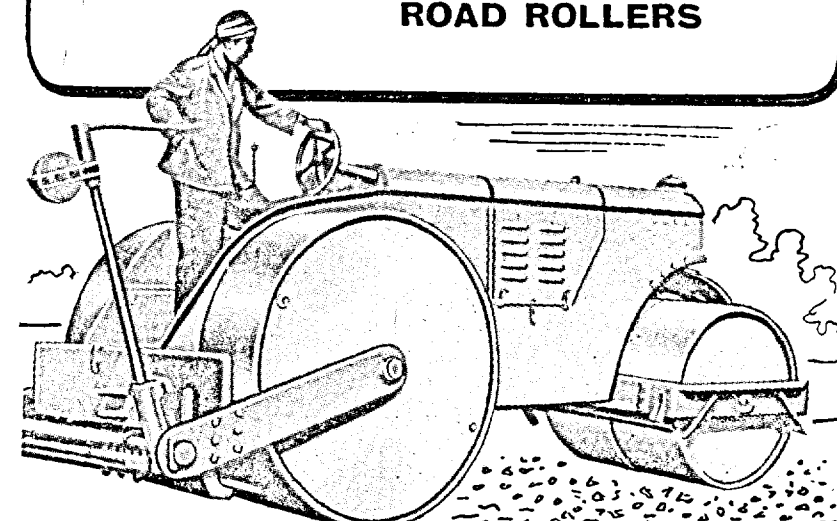
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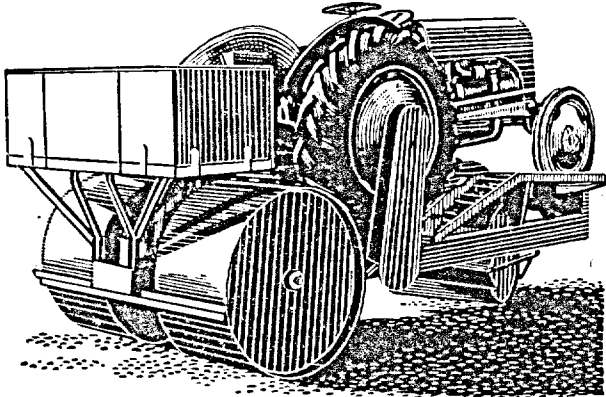
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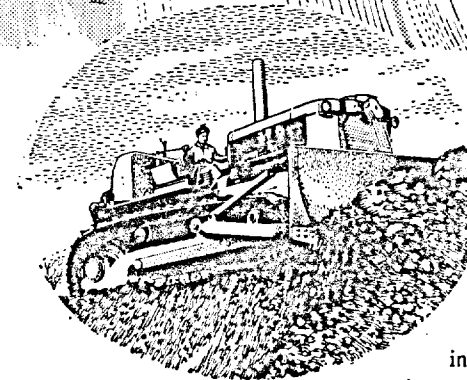
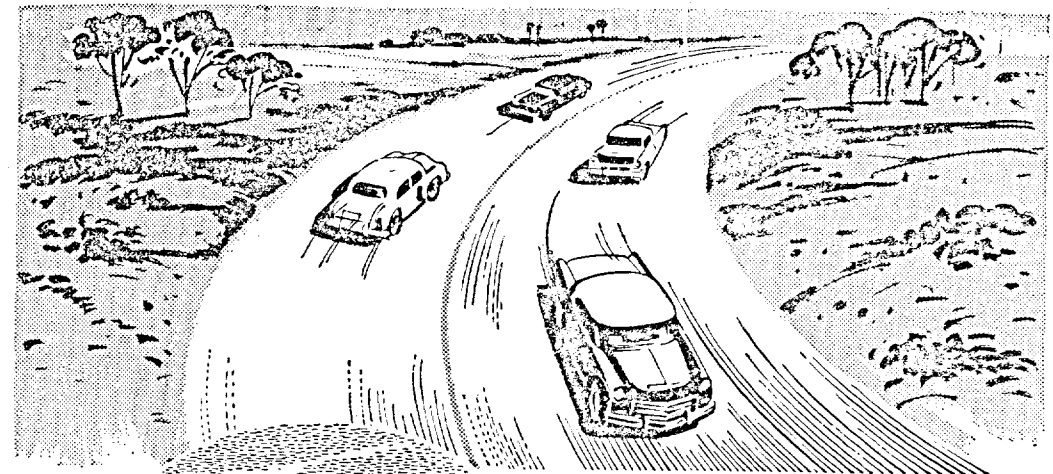
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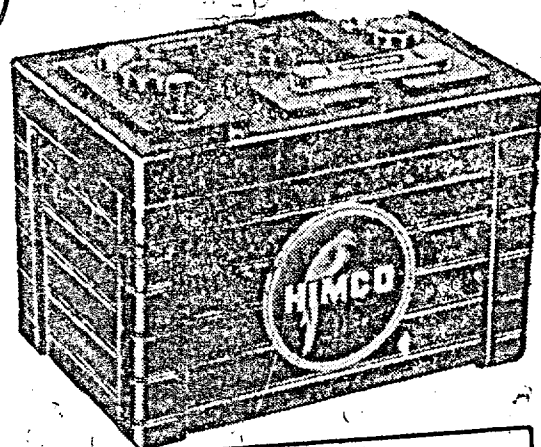
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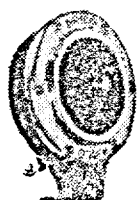


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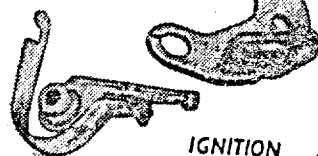
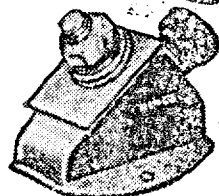
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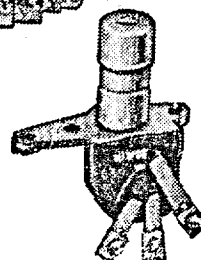
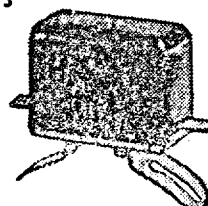
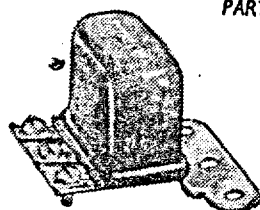
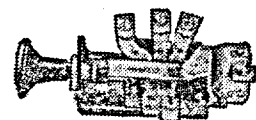
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