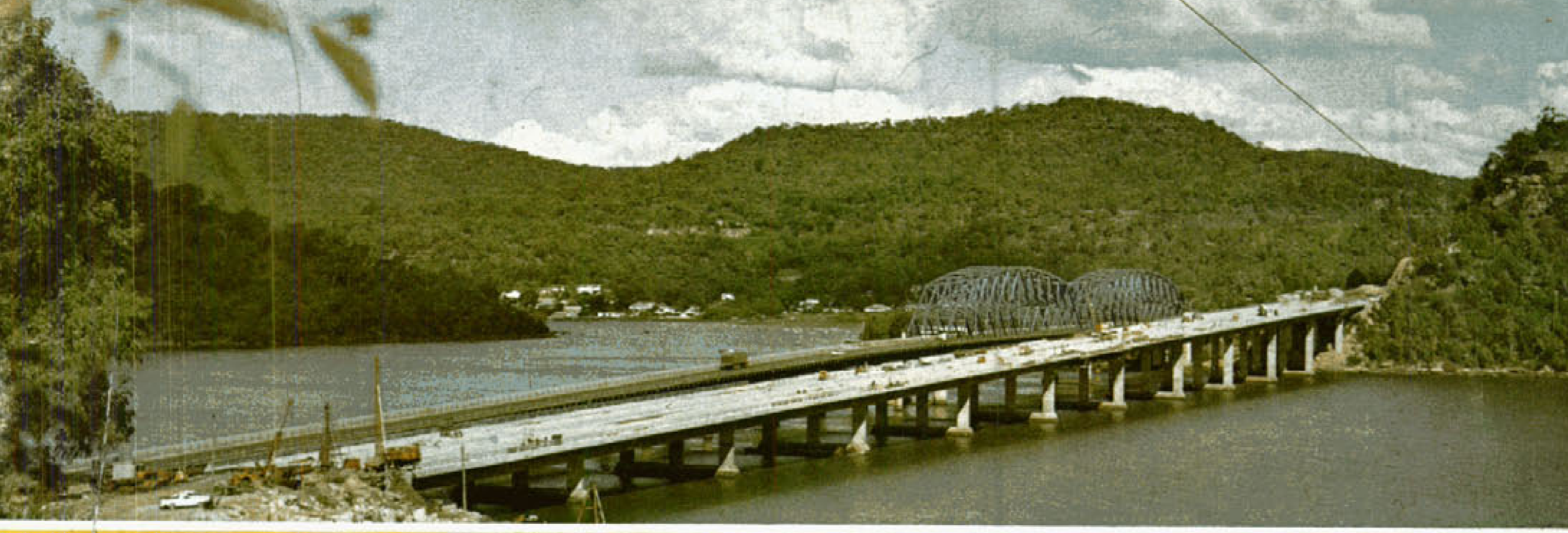
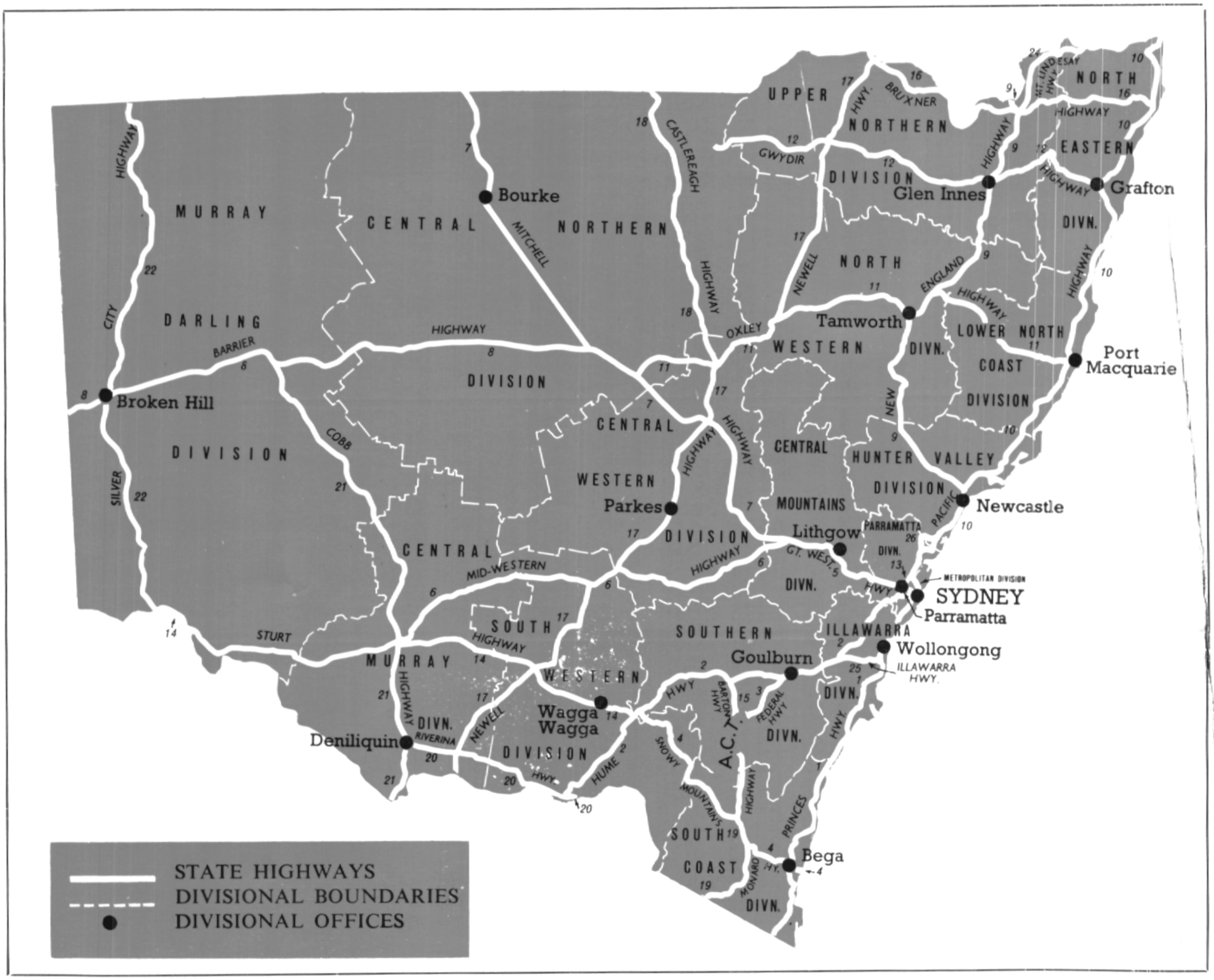


JUNE 1973

MAIN ROADS



Area of New South Wales—309,433 square miles

Length of public roads within New South Wales—129,715 miles

Population of New South Wales at 31st March, 1972—4,671,800 (*estimated*)

Number of vehicles registered in New South Wales at 31st July 1972—2,218,451

ROAD CLASSIFICATIONS AND MILEAGES IN NEW SOUTH WALES

Mileage of Main, Tourist and Developmental Roads, as at 30th June, 1972

Expressways	30
State Highways	6,535
Trunk Roads	4,375
Ordinary Main Roads	11,513
Secondary Roads (County of Cumberland only)	177
Tourist Roads	251
Developmental Roads	2,553
	25,434
Unclassified roads, in western part of State, coming within the provisions of the Main Roads Act	1,490
TOTAL	26,924

MAIN ROADS

JOURNAL OF THE DEPARTMENT OF MAIN ROADS, NEW SOUTH WALES

JUNE, 1973

VOLUME 38 NUMBER 4

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R. J. S. Thomas

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Front cover: (*Top*) *Relocation of New England Highway* (*S.H.* 9) 32 *miles north of Murrurundi*
(*Centre*) *Hawkesbury River Bridge near completion*
(*Bottom*) *At Marrangaroo, a new overbridge crosses the Great Western Highway* (*S.H.* 5)

Back cover: *Reconstruction on the Oxley Highway* (*S.H.* 11) 75 *miles west of Port Macquarie*

BY AND BY

Country towns and cities situated on main roads suffer traffic congestion when local traffic clashes with a high volume of through traffic. The most desirable solution involves the construction of what is commonly known as a "by-pass" designed to separate the two categories of motorists to the mutual benefit of each.

Unfortunately, many people see the word "by-pass" as a step towards the creation of a "Ghost town". The "pass" stays fixed in mind, associated with such meanings as "pass over", "pass by", or even that final end, "pass on". Little consideration is given to the "by" and its overpowering importance to the meaning of the word and the scheme behind it.

The practical meaning of "by" is—"at the edge of, alongside or by the side of". In a by-pass scheme, the new route becomes just that—alongside the town but part of the total town planning. It is for this reason that the Department refers to them as "Traffic Relief Routes", for that is what they are. They provide the town with relief from congestion so that it can better achieve its primary purpose of serving the local community, and allow long distance traffic to pass unhindered through the district. These routes are not intended to withdraw all out-of-town traffic away from the commercial interest of the town. Their design incorporates ample provision for through traffic to gain convenient access to the commercial centre.

Traffic congestion is not the only reason for construction of a route around a town. At Camden, where the old Camden Bridge needed urgent replacement, a flood free route forming part of the Hume Highway and incorporating the new Macarthur Bridge is being constructed around the main commercial area of Camden. Similarly, a deviation of the Pacific Highway passes around Port Macquarie. At Gundagai, a new bridge and deviation is under construction to form a new highway route west of the town. Here again ample provision is being made to protect local interests and suitable access to the town will be available.

The research and planning of a traffic relief route are considerable. This is evident from the article relating to the route around Orange detailed on page 121 of this issue.

Orange is growing and will continue to grow. The Mitchell Highway traffic is growing and will continue to grow.

The proposed Traffic Relief Route will ultimately separate the two to the benefit of both ●

In June, 1971, *earthworks had begun on a reconstructed section of the highway, near Rules Point . . .*

IMPROVEMENTS TO THE SNOWY MOUNTAINS HIGHWAY

At the same location looking towards Rules Point work was almost completed by May, 1973

The Snowy Mountains Highway (State Highway No. 4) extends for 225 miles in a northwesterly direction from Tathra on the far south coast of New South Wales to its junction with the Hume Highway (State Highway No. 2) 18 miles south of Gundagai.

The Snowy Mountains Highway traverses country ranging from rolling grazing country near Bega, mountainous country over Brown Mountain, a rolling plateau through Nimmitabel, Cooma, Adaminaby and Kiandra to Rules Point; much of this country is above the snow line. From Rules Point to Inspiration Point the country is heavily timbered and mountainous. The highway then descends steeply into the Tumut River Valley near Talbingo and adjoins the shore of Blowering Dam and on to Tumut. It then traverses undulating grazing country to its end at the Hume Highway. In the length from Adaminaby to Tumut the Highway passes through the Kosciusko National Park.

Reconstruction on this highway is in progress along a section between Kiandra and Inspiration Point and two Divisions are involved in the project. The South Coast Division is responsible for the 13 miles from Kiandra to the westernmost junction with Main Road No. 324 (Yarrangobilly Caves Road). From this junction the South Western Division is reconstructing approximately 16 miles to Inspiration Point.

Special Considerations

Although the work of each Division is detailed later in this article, there are several factors common to the reconstruction as a whole.

The entire 29-mile reconstruction occurs in the Kosciusko National Park. The new road is also in an altitude which is subject to snow falls for approximately four months of the year. Location and design of the new highway were therefore discussed with officers of the National Parks and Wildlife Service before final plans were approved.

The design, therefore, was undertaken with the purpose of minimising interference with the natural features of the area and the effect of heavy snowfalls. These two objects often conflicted and final design necessarily had to be accepted as a compromise. The cost of reducing the effects of snow to an absolute minimum would also have been excessively high and some limitations had to be accepted here.

The basic purpose of design was to present a smooth profile to the prevailing winds to avoid the eddying which causes deposition of wind-borne snow in drifts.

A major problem in the reconstruction of this section of the Snowy Mountains Highway was the short working season. Between May and late October of each year, the weather usually made work impossible. Construction was continued over the Christmas period and carefully programmed to ensure that the maximum amount of work was completed in the limited time available.

Working within a National Park also created other practical problems, for example it was necessary to build and maintain silt traps on streams to avoid material being washed into the limestone caves at Yarrangobilly.

To further minimise disruption to the National Park's environment, the Department engaged the Soil Conservation Service in treatment of all batters and embankment slopes to accelerate re-vegetation and merge the elements of the road formation with the terrain.

In addition, specialised techniques developed by that Service have been employed in treating stormwater run off paths to combat erosion.

SOUTH COAST DIVISION

The 13-mile section from Kiandra to the westernmost turnoff of the Yarrangobilly Caves Road (Main Road No. 324) was completed and opened to traffic over its full length during May 1973.

Over the first seven miles from Kiandra the road is located in gently undulating country and earthworks were moderate at approximately 34,000 cubic yards per mile. Beyond this length however, as the country becomes more rugged, earthworks were heavier and the final mile required cuttings totalling over 290,000 cubic yards. Cuttings over this final section were up to 40 feet in depth and fills up to 86 feet high. Grades on this section are up to 8 per cent and climbing lanes are provided.

Wherever possible, box cuttings with steep batters were avoided since they tend to encourage the formation of snow drifts on the formation. Batters were flattened when this could be done economically, and sometimes, in sidelong country, cuttings were widened so that the formation level extended out to meet the natural surface on the lower side. These objectives became increasingly difficult to achieve towards the northern end of the work where the terrain becomes hilly.

Pavement

A sealed pavement width of 20 feet is provided over the earlier sections of the work and 22 feet on the more recently completed sections (6 miles to 13 miles). The pavement is designed to withstand the low winter temperatures in the area and the operation of snow-clearing equipment. It provides for a sub-base layer seven inches thick underlying a layer of free draining basecourse nine inches thick. The second free draining basecourse layer is provided to prevent frost heave caused by the build-up of ice lenses at the boundary between frozen and unfrozen soils. In this case, the free draining material is a river gravel up to 2½ inches maximum size obtained from the Murrumbidgee River

Two views of earthworks on Snowy Mountains Highway (SH 4)—from crest of the hill near Mills Creek.

North of Mills Creek looking towards Inspiration Point.

Construction of approaches to Mills Creek looking south.

IMPROVEMENTS TO THE SNOWY MOUNTAINS HIGHWAY

near Adaminaby. After priming, a two-coat $\frac{3}{4}$ inch and $\frac{3}{8}$ inch seal with separate applications of bitumen has been provided as a temporary running surface.

The full length is later to be sheeted with asphaltic concrete two inches thick as the normal flush seal has a very limited life affected by the operation of snow-clearing plant such as blades, tracks, etc., in addition to the effects of chains on tyres.

On the high side of curves the subgrade and basecourse layers are given a negative crossfall from the edge of the pavement to the outside of the shoulder to minimise the risk of water from melting snow penetrating beneath the pavement. It also reduces that amount of water from melting snow which flows across the pavement.

Structures

The largest structure required was a bridge of four 35 feet spans over Bullock Head Creek just beyond Kiandra. Larger waterways on the first six miles of undulating country were provided for by R.C. box culverts, the largest being a 4-cell 12 feet x 8 feet structure. All concrete placed incorporated 4–6 per cent entrained air to resist freeze thaw action.

At the latter end of the work under the higher fills the use of R.C. box culverts was not economical and the larger catchments were accommodated by using corrugated metal pipes. Three culverts of this form measuring 9 feet, 11 feet and $13\frac{1}{2}$ feet diameter, were installed. Tests of adjacent soil however, gave pH values of between four and five and it was necessary to provide additional protection at the outside of the pipes with a tar epoxy coating.

The beds of the streams where the metal pipe culverts were installed are steep and, in view of the location within a National Park, special provision of inlet and outlet structures was made to ensure that erosion would not result during peak flows. A gabion weir is provided approximately 100 feet upstream of each culvert to collect any larger boulders that may be washed down. Concrete wingwalls and apron are provided at the culvert entry and the length between the apron and the gabion weir protected with heavy stone pitching. At the outlet a concrete apron is provided with "dragons teeth" energy dissipators. Both sides and the downstream edge of the apron are protected with heavy rock spalls set at 45° and faced with Reno mattresses.

SOUTH WESTERN DIVISION

This Division's section of reconstruction, adjoining that of the South Coast Division and continuing for 18 miles to Inspiration Point, is nearing completion and expected to be open by June 1973. Some bituminous surfacing towards the junction with the Yarrangobilly Caves Road may not have been undertaken because such surfacing is not possible in the severe weather conditions usually experienced in the region from about May onwards. Final surfacing will be carried out in the spring of 1973.

Reconstruction of this section was commenced at Inspiration Point above Talbingo and has proceeded towards the Yarrangobilly Caves Road. The locality is densely timbered and it was necessary, in clearing operations to remove some trees up to 100 feet in height, to obtain the required alignment, but this was done selectively to avoid any unnecessary disturbance to the local ecology.

Excavation and Filling

The largest cutting has a depth below actual surface of 53 feet while the largest embankment has a height of 74 feet. In the total of 1,427,000 cubic yards of material which had to be excavated, rock and soil types encountered included shales, siltstone, limestone, rhyolite, tuff and feldspar porphyry. To undertake such earthworks a fleet of tractor-dozers, motorised scrapers, loaders, graders and various types of rollers was engaged.

Drainage

Prior to the commencement of embankment construction drainage structures ranging from conventional reinforced concrete pipes to 8 feet diameter corrugated steel pipes were installed. At certain locations downstream from culverts, silt traps were constructed to prevent siltation of waterways.

Alignment

The alignment of the completed road in this mountainous terrain has a minimum radii of curvature of 1,600 feet and a maximum radius of curvature of 12,000 feet. The minimum radius will permit speeds up to 60 mph. The most severe grade on this length of highway will be 8 per cent over a distance of $\frac{3}{4}$ mile.

To process rock for the pavement layers, a crushing plant was set up near the Yarrangobilly River. Rock was crushed by the plant and reduced from a dimension of up to 24 inches to $1\frac{1}{2}$ inches.

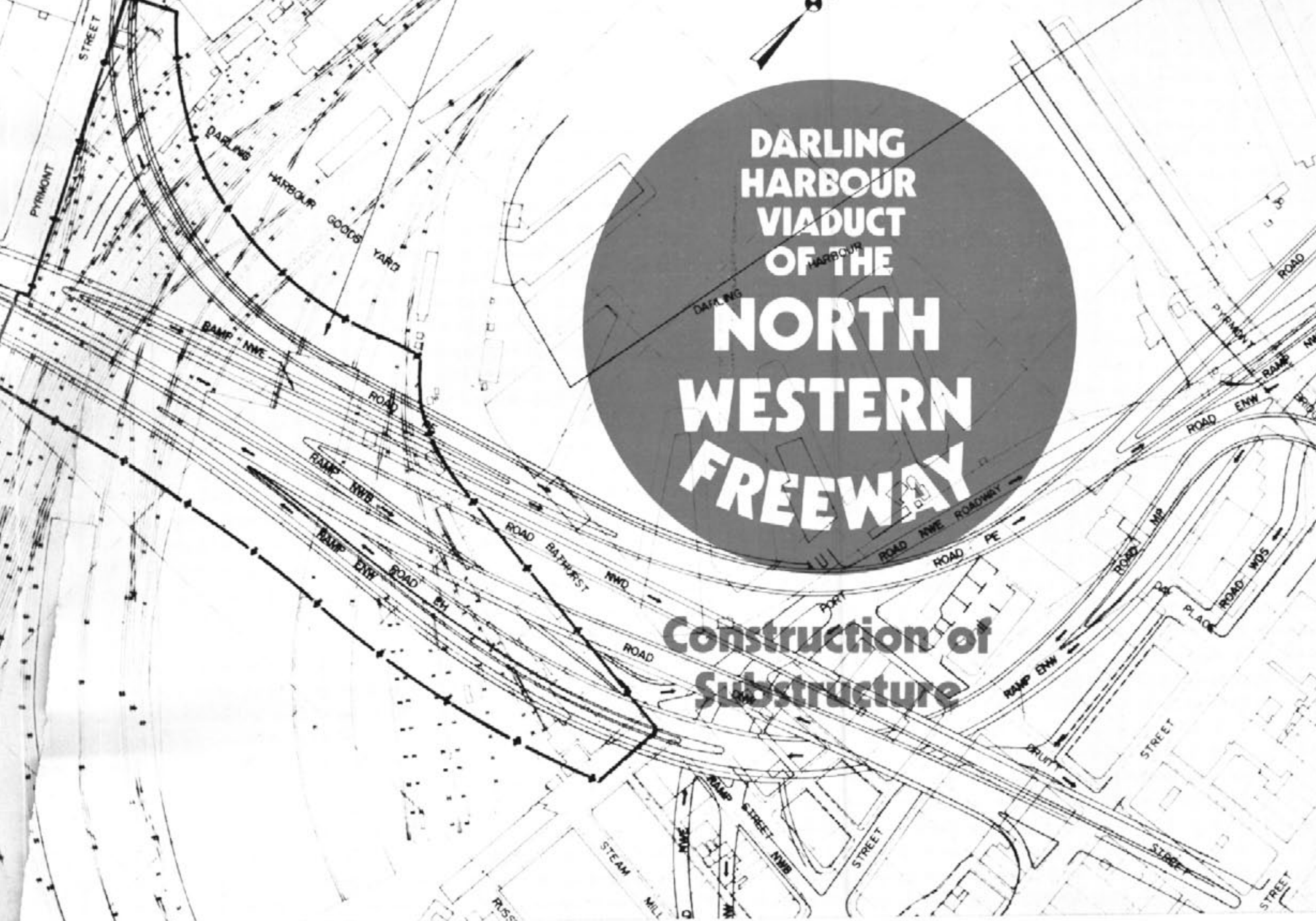
Total Cost of Reconstruction

The total cost of the work from 0M. to 13M. north of Kiandra is of the order of \$3·5 million with asphaltic concrete surface estimated to cost an additional \$500,000.

The total cost of the adjoining 16-mile section is expected to be approximately \$3·3 million bringing the overall cost of reconstruction for this 29-mile section of Snowy Mountains Highway to more than \$7 million●

DARLING HARBOUR VIADUCT OF THE NORTH WESTERN FREEWAY

Construction of Substructure



The North Western Freeway is an extremely complex project designed to form part of a freeway system connecting Sydney and Newcastle. One section comprising the Gladesville, Tarban Creek and Fig Tree bridges, the Huntley Point overpass and associated roadworks, was completed several years ago.

A 70 feet high viaduct crossing the Darling Harbour Railway Goods Yard will form the first part of the City to Glebe section of the North Western Freeway. Construction of the substructure of the viaduct is under way and this article describes the procedures adopted for this work and some of the difficulties encountered.

Because of the problems associated with the operations of the Goods Yard, the Department is undertaking all substructure work using its own forces. The work is under the direction of the Divisional Engineer, Inner Freeway Construction Division. The site staff consists of a resident engineer with one assistant engineer. An engineer is also in charge of the Benoto Organisation which carries out the necessary pile work. Substructure work was commenced in late 1971 and is expected to be completed in 1975. The Department will arrange for superstructure and decking to be carried out by contract.

The Substructure

The viaducts are to be founded on 46 in diameter reinforced concrete cast-in-place piles to bedrock, the depth of which varies up to 80 feet. The majority of the piles are raked at 6° to the vertical and the rest are vertical. On groups of two, three or four piles, pilecaps and column pedestals are cast. Prestressed concrete columns up to 70 feet in height are then to be constructed from hollow segments, precast in contact and joined in place with epoxy resin. Cast-in-situ headstocks which will be post-tensioned "horizontally" will complete the substructure.

Later, by contract, plate-web steel girders will be launched across the headstocks from the western abutment near Harris Street.

Constructure Procedure for Piles

Darling Harbour Goods Yard is situated on a reclaimed part of the harbour so that the piles pass through a mixture of clay and broken sandstone fill, followed by relatively little natural clay and sand, overlaying solid sandstone. In some locations old building rubble and timber piles from a buried wharf were encountered. Because of the very poor nature of the fill material, permanent steel casing was used for the piles.

Benoto Operation—The casings were sunk to rock, and at least one foot into the sandstone, with the Department's Benoto (Super EDF) Drilling Machine. Excavation within the casing, using the Benoto's hammer grab, proceeded as the casing was sunk. The Benoto machine was then withdrawn.

Calweld Operation—The rock below the casing was tested by coring in about one third of the pile excavations. The cores were inspected and where necessary tested for strength. The depth of the rock socket was then calculated from the loading on the pile and the strength of the sandstone. These depths varied from 4 feet to 10 feet. A Calweld rotary drilling machine was then set accurately over the excavation and the socket was drilled using a rock bucket bit and telescopic kelly (Stem).

Cleaning and Inspection—Being close to the Harbour the water table was only 6–7 feet below the surface so that each pile excavation was practically full of water. After completion of excavation a few feet of mud (mainly pulverized sandstone) remained in the pile. This was cleaned out initially by air lift but, because of the large quantity of rock drillings to be removed and because of the large diameter and rake of the piles, the air lift alone was not completely effective. The air lift was eccentrically mounted in a guide ring just smaller in diameter than the pile, and was fitted with a scraper blade and auxiliary inlets branching off the main pipe, to improve its performance. Eventually the use of a diver was found to be more efficient.

This was especially so in the deeper piles which had to be inspected by diver in any case to ensure that there were no leaks of clay into the uncased socket from seams in the rock. The deeper piles could not be de-watered for inspection because of the likely collapse of the $\frac{3}{8}$ in thick steel casing under hydrostatic pressure. The diver directed the end of a simple air lift pipe by hand to clean out the sockets, and plugged any soil leaks with "Perma Cement". Cleaning and inspection by diver was found to be economical even in relatively shallow piles, say 40 to 50 feet in depth.

It was found worthwhile to remove as much mud as possible with a grab (Benoto type) operated by crane, before airlifting because the airlift discharge had to pass through a settlement tank to gully pit.

Concreting by Tremie

The reinforcing steel cages for the piles were fabricated in sections at the Department's Central Workshop and joined as they were lowered into the holes. Sealing the holes completely to water leaks would have been extremely difficult and de-watering of the deeper holes was not safe. Consequently, all concrete was poured underwater by the tremie process.

This is a special concrete mix that is placed under water using tremie pipes. The mix has a slump of 6 in to 8 in and a low percentage of coarse aggregate, ensuring a mix of high workability. It is also desirable and in some cases necessary to use a retarder in the mix to allow more time for placing in case of delays. The retarder also tended to reduce the amount of water required in the mix.

Specified strength was 3,000 psi but test cylinders averaged 5,000 psi and cones tested above 4,000 psi.

The tremie pipe was constructed in such a way that all joints were waterproof and such that it could be lifted without danger of getting caught in the reinforcing cage. Guides on the bottom length of pipe were necessary to keep the pipe central in the pile. The tremie pipe had an internal diameter of 10 in to ensure that no blockages occurred during pouring.

The tremie system is basically a simple U-tube with the tremie pipe as one arm and the pile as the other. In equilibrium the pressure of fluid in one arm must balance the pressure of fluid in the other. In the tremie the two fluids are water and concrete. Concrete is twice as heavy as water so that, neglecting friction, equilibrium is maintained by concrete in the tremie only to half the height of the water outside. However, friction does exist in the system and this causes the concrete to rise in the pipe as the immersion depth of the pipe in the placed concrete increases; this case is shown in diagrams 6, 8 and 10 below.

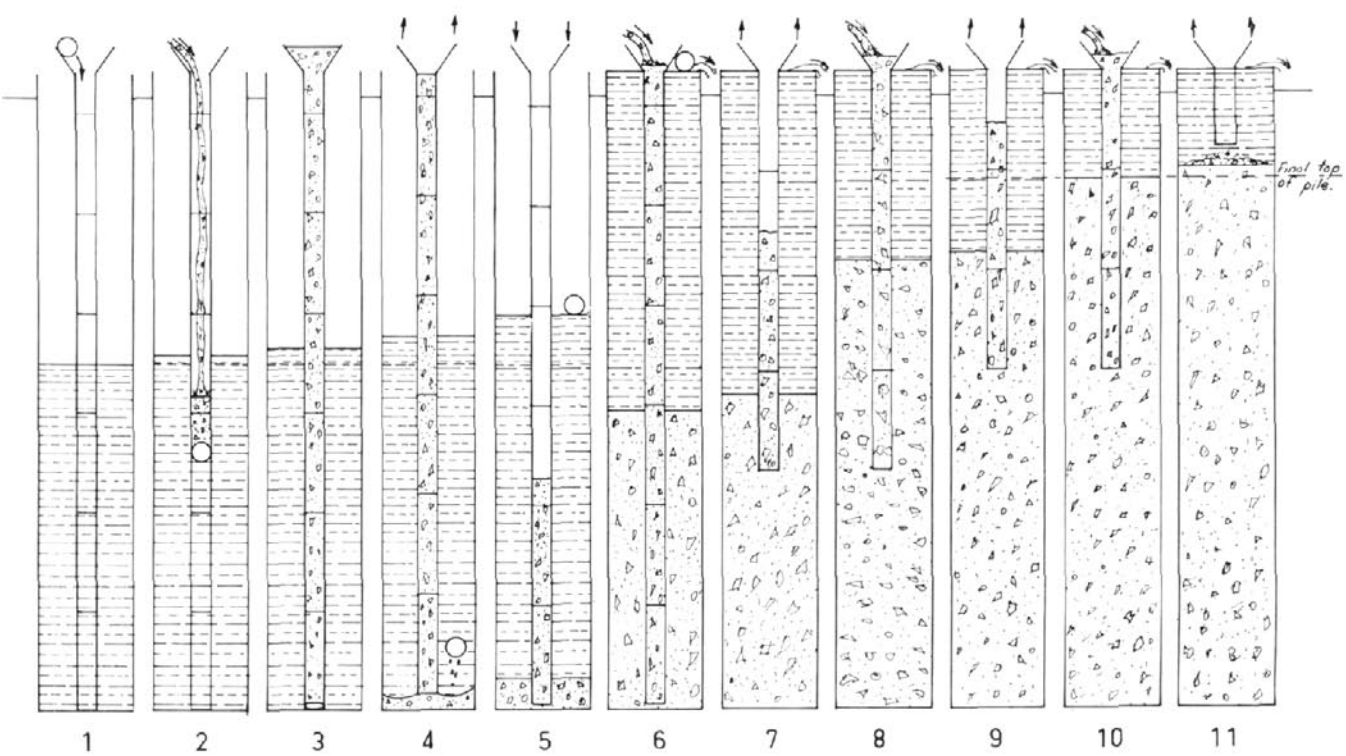
The adopted method of placing tremie concrete is illustrated in the diagrams below (Nos 1 to 11). The details of the various stages are:

1. The tremie pipe is placed in the pile with the end of the pipe on the bottom of the pile. An inflatable rubber ball is placed into the tremie pipe. The ball should be a neat fit.
2. Concrete is discharged into the pipe on top of the ball which is driven down the pipe displacing the water.

The Department's Benoto (Super EDF) Drilling Machine in operation at the site

3. When the ball reaches the bottom, the pipe fills to the top with concrete. Another method of filling the tremie pipe with concrete without it mixing with the water is to tape a plate across the bottom of the pipe before placing the pipes into the pile. This displaces the water as the pipe is placed and permits inspection of all joints for leaks. When the pipe is on the bottom it is filled with concrete.
4. The tremie pipe is raised 6 in to 10 in to allow the ball to be forced from the pipe. As soon as the ball is forced from the pipe the concrete rapidly moves down the pipe out into the pile. This first concrete discharged mixes to a degree with the water but subsequent concrete flows underneath this layer and displaces it upwards. If the level of the concrete in the pipe does not drop when the tremie is raised, the tremie must be lowered and raised again as jerkily as possible until concrete starts to flow.
5. As soon as the concrete in the pipe starts to move down, the pipe is lowered to the bottom then raised approximately 2 in from the bottom to minimise laitance and promote better cleaning of the rock bottom by the discharge. At this stage the level of the concrete inside the tremie is lower than the water outside so the joints can be inspected for leaks. Subsequent concrete placed with the pipe in this position will scour the toe of the pile clean.
6. Concrete is now discharged into the tremie. The concrete flows into the pile displacing the concrete initially placed and also displacing the water. The concrete is poured continuously until the level of concrete rises in the tube and the flow is substantially reduced or when one batch of concrete is finished. As water overflows from the pile it may be necessary to remove it with a pump to prevent flooding of the working area around the top of the pile.
7. Sections of tremie tube are removed as the tremie is lifted ensuring that adequate immersion of the end pipe in the concrete is maintained, this would be a minimum of 5 feet to 10 feet under normal circumstances. It is usual to remove pipes between the delivery of each load of concrete rather than wait until the pipe fills up as shown in Figure 6.
8. and 9. Repeat steps 6 and 7.
10. Pouring is continued until the level of concrete reaches the final top of the pile, then an excess of 9 in to 12 in is poured to allow for laitance on top of the pile which is to be removed later.
11. The tremie pipes are then removed. The level of concrete in the pile will rise a little more as concrete flows out of the pipe. When the pipe is extracted a small amount of concrete will mix with the water on top of the pile which can be removed later.

At all times it is most important to know the immersion depth of the tremie pipe and to ensure no leaks occur in the pipe joints.



Method of placing Tremie Concrete (see details in text)

Leaks in the Tremie Tube Joints

Experience showed that water in the tremie tube, no matter how little, was a serious hazard and had to be removed before continuing to pour concrete. A water level in excess of one or two inches in depth is significant.

The result of pouring concrete into a tremie tube containing several feet of water above concrete previously poured is that in a significant amount of concrete, the grout is separated, leaving raw aggregate. This aggregate forms a rigid plug in the tremie, stopping the flow of concrete.

If the flow is restarted by an additional head of fresh concrete, the rigid plug of aggregate is forced down through the concrete already placed to a considerable depth. It remains at this depth roughly retaining the shape it held as a plug in the tremie. Concrete placed subsequently flows out of the tremie and up to a higher level leaving the aggregate plug lower in the pile.

Water leaking into the tremie above the static level of the concrete will accumulate during a break in the pouring. Before continuing the pour, the tremie was always inspected for water with the aid of a torch or sunlight reflected down the tremie by a mirror.

On this job, water problems were experienced in three piles.

(a) In one pile 60-feet deep, the problem arose when less than 10 feet of concrete had been placed. The flow could not be restarted, after the aggregate plug had formed, until the tremie was extracted from the concrete. Thus the charge was lost, the concrete in the tremie falling into the water.

A test core showed regions of loose aggregate distributed throughout the concrete, indicating that the final discharge had disturbed the underlying concrete for almost its whole depth.

This concrete was removed by jackpick before pouring concrete in the pile again. (A point worth watching is that the earlier it is removed, the easier it is to remove.)

(b) In another 60-foot pile, three feet of water entered the tremie through a joint when 20 feet of concrete had been placed. This produced an aggregate plug which was later forced down into the concrete already placed when flow was restarted, in this case without losing the charge. Pouring continued to the top of the pile.

Subsequent investigation consisted of a 2 in core approximately on the axis of the pile (actually 4 in off centre). This showed that the length of the aggregate plug was about 8 feet and was situated partly in and partly above the rock socket. Some knowledge of the diameter of the aggregate plug was required, especially in the region below the steel pile casing which provides a considerable reserve of strength at depths below the levels where corrosion occurs.

Several investigating methods were considered to determine the diameter of the 8 feet long plug. These include:

(1) Radioactive probe down the core hole, measurement depending on back-scatter.

(2) Electrical probe.

(3) Ultrasonic probe.

(4) Measuring the volume of water removed from the damaged zone to give some measure of the cavity volume.

(5) Water loss test over the damaged zone to determine if the cavity extended to the rock at any point.

It was concluded that none of these methods would be more accurate or useful than additional cores. Therefore a second core was removed from 11 in off centre diametrically opposed to the first core. This passed along the outer surface of the aggregate plug suggesting that the diameter of the plug was less than 24 in, leaving an annulus of sound concrete of 11 in minimum thickness. This condition would have been structurally adequate, but further measures to completely determine the size of the cavity were not pursued. Rather, remedial action was taken by removing the loose aggregate and grouting the cavity.

The aggregate was removed by air-lifting, followed by the use of a wire rope flail attached to drilling stems to dislodge any remaining stones, and then airlifted again. The cavity was then examined visually by the use of a borehole T.V. camera and was found to be well cleaned.

(c) In the third pile, 80 feet deep, a pocket of loose aggregate approximately 2 feet in depth, resulting from similar causes to those in (b), was revealed by coring at a depth of 60 feet. The pile is cased at this level. The cavity was cleaned and grouted as above.

Cause of Leaks

In two of the above cases the leak in the tremie resulted from an unmatched joint. It was found that the rubber seal in the joint requires small tolerances in the joint manufacture and that not all combinations of the tremie tube sections were suitable. To prevent a recurrence of this problem all joints were match marked and coupled accordingly.

In the third case, the leak resulted from a joint being loosened during the removal of a section from the top.

On one other occasion a leak occurred in a joint which had been strained by heavy lifting required to free the tremie when caught on the reinforcing cage. In this case the tremie was extracted, after filling it with water to just the same level as the water outside. This procedure obviates a turbulent discharge of concrete when the tremie is extracted as all concrete is pushed out by the water in the tremie. The strained joint was sealed and the tremie was also sealed with a plate on the bottom and was reinserted and recharged. Subsequent coring showed that a small pocket of laitance had been pushed down by the tremie into the previously poured concrete and had remained at that level. Thus re-charging of the tremie appears to be not generally advisable.

One further point is worth noting. On this job the tremie was generally placed in the water with the bottom open and an inflated ball was used to separate the concrete from the water as the tremie was charged. This method was found to be more convenient in operation than the alternative of sealing the bottom of the tremie with a plate, and does not result in a plate remaining at the bearing end of the pile.

However, when a ball is used there is no visual check on the water-tightness of the joints before pouring commences.

GENERAL DATA (PILING)

Benoto

Benoto used to excavate rock:

Pile type: 46½ in O.D., 6° rake permanently cased, mild steel cutting edges.

No. of piles: 178.

No. of piles completed: 93.

Average depths of piles completed, not including sockets (measured from G.L.): (max. 79 ft, min. 10 ft): 44 ft.

Total depth drilled: 4,113 ft.

Average drilling rate (drilling only): Approx. 8·0 ft/hr.

Average drilling rate (overall): Approx. 2·0 ft/hr.

Calweld

Used to drill rock socket in piles after Benoto excavates to rock:

(*Left*): *Pitching sheet piling*

(*Right*): *Timber piles from an old wharf were encountered during excavations*

(*Below*): *The completed pedestal*

Average depths of rock drilled (per socket): 6·1 ft.
Total depths drilled (hard sandstone 90 per cent, medium sandstone 10 per cent): 563 ft.
Average drilling rate (drilling only): 2·0 ft/hr.

Concrete Placing

All concrete placed with 10 in I.D. Tremie pipe:
Total quantity placed: 1,690 cu yds.
Average pouring rate: Approx. 26 cu yd/hr.

COSTS (Direct Costs)

Benoto and Calweld drilling costs (drilling only): $27/ft.
Cost of finished pile: $86/ft.

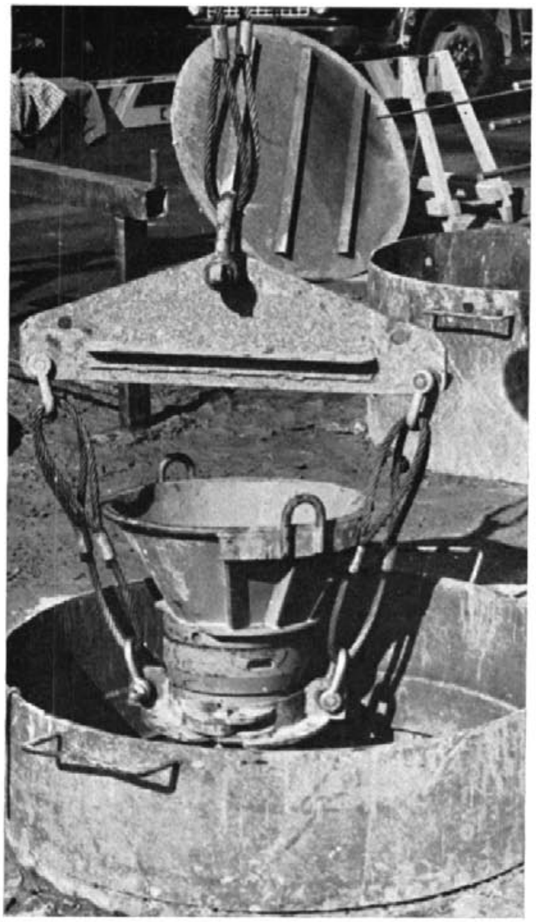
Construction Procedure for Pilecaps

The pilecaps were constructed below ground level so that the piers would obstruct the Goods Yard as little as possible. As the water table was above the bottom of pilecap level, cofferdams of relatively light gauge steel sheet piling were driven for the excavations which were 8–9 feet deep. The steel sheet piling was driven 3 in outside the design profile of the pilecap to allow for deflections in the sheets during driving, the sheets being used to form the pilecap. After the steel sheet piling was driven the material inside the dam was excavated.

Steel sets were lowered into the dam to support the steel sheeting under combined earth pressure and railway loadings. A concrete seal was then poured. The piles were cut back to their correct level and the steel reinforcement placed. At this stage, the anchor plates and starter ducts for all Macalloy bars for the columns were placed. Plastic sheeting was placed against the steel sheet piling to prevent bond between the sheet piling and the concrete. The cap was then concreted and a day later the steel sheeting was extracted simply by a crane. The steel sheeting was then used in the next pilecap.

The pedestal was formed using timber forms. The top of the pedestal had to be finished to a tolerance of $\pm\frac{1}{32}$ in. This was achieved by forming the top surface. It was necessary to drill holes at close intervals in this top form to minimise air-bubble cavities in the top surface●

A previous article on the North Western Freeway appeared in "Main Roads", Vol. 37, No. 1, September 1971, *pp.* 2–3.



Hopper and tremie tube

SUMMARY OF POINTS TO WATCH IN TREMIE CONCRETING

(1) The tremie tube should be constructed in such a way that it will not foul the reinforcing cage, particularly at laps.

(2) Use a retarder in the concrete mix to reduce the hazard of delays. The slump should be 7 in. Never vibrate tremie concrete.

(3) Take every precaution possible against leaking joints in the tube. Inspect for leaks between each exchange of trucks and immediately after charging the tremie when a ball is used.

(4) An accurate check should be kept on the level of concrete in the pile and the level of the bottom of the tremie pipe to ensure that there is a sufficient immersion depth at all times.

(5) If leaks do occur, stop the concreting and determine the magnitude of the leak and how much water is in the tube. One of the following procedures should then be adopted:

(a) If only a little concrete has been placed and a significant leak was present during the placing of the concrete, stop work altogether and remove the concrete already placed before it gains strength.

(b) If the leak occurred after the concrete was placed (i.e. during removal of tubes) and the leak can be rectified, rectify the leak, remove water from the tube and continue the pour. A leak can sometimes be eliminated by lifting the tremie further.

(c) If a lot of concrete has been placed and it is suspected that a significant leak was present during placing of the concrete, fill the tremie tube with water to the level of water outside the tremie and withdraw the tremie. Do not recharge. If there is enough time the top few feet of concrete should be removed by airlifting as this is much easier than removing with a jackpick later. A test core should then be taken as soon as possible to determine the quality of the concrete placed.

(d) If a lot of concrete has been placed and a leak which cannot be rectified occurs after the concrete is placed, fill the tremie with water to the level of water outside the tremie and withdraw the tremie. Complete concreting the pile "in the dry" after having removed the top two feet of placed concrete. The tremie should only be recharged if it is known that a small pocket of laitance remaining buried in the pile is of no structural consequence.

(e) If the leak is small, say less than 1 foot/$\frac{1}{4}$ hour, its effect on the concrete during periods of continuous flow is probably not significant. However, if any water accumulates during periods between concrete discharges, this water must be removed before continuing with the pour.

(6) Pour more concrete than necessary so that all laitance is removed when the excess concrete is removed●

Science on the roads

The Department of Main Roads adopts modern technologies for its work wherever worthwhile benefits can be gained from their use. This article features three such procedures—Pavement Evaluation by Benkelman Beam, Seismic Refraction Surveys and the Carbon Monoxide Detection System for the Cahill Expressway Tunnel.

PAVEMENT EVALUATION BY BENKELMAN BEAM

The Benkelman Beam is a device for measuring the deflection of a pavement under a dual-tyre wheel load. It has gained widespread recognition as a non-destructive test yielding a reliable indication of the strength of existing pavement structures. Its principal uses are in the testing of existing pavements and as a check on strength variations in construction work.

For existing pavements, the Beam is generally used in two circumstances. Firstly, if a length of old road is being considered for inclusion within a section of new construction, then Beam testing will indicate whether the old road has adequate strength for inclusion without reconstruction. Secondly, on existing flush sealed pavements, the Beam is used to obtain deflections for the design of asphaltic concrete overlays.

Equipment

(a) Benkelman Beam apparatus.

(b) Truck:

(i) Load. The vehicle must have an 18,000 lb rear axle load equally distributed by each of the dual wheels.

(ii) Tyres. Size: 10 x 20, 12 ply.
Pressure: 80 psi.
Spacing: The tyres should be 12¾ in apart measured centre to centre of the dual wheels.

(c) Tyre gauge.

(d) Tape, note books, etc.
At times thermometer, pick, camera, etc. may be required.

Procedure

(a) Lateral location.
For each lane, readings may be recorded and calculations carried out separately for both the outer and inner wheel tracks. The outer wheel track deflections, which are normally used for evaluation, are generally greater than the inner wheel track deflections which are recorded for check purposes.
Deflection test points should be located at specified distances from the edge of the lane. These distances will vary with the width and number of lanes. As a guide Table A1 may be adopted for the location of the outer wheels.

TABLE A1

Location of Outer Wheel Track

Lane width feet	Distance from edge feet
12	3
11	2½
10	2
9 or less	1½

(b) Longitudinal location.
Test sites should be located at a constant spacing of 50, 100 and 200 feet depending upon the length of the section, possible changes in construction and/or subgrade, drainage conditions and the importance of the particular road.

(c) Vehicle position.
The dual tyres of the outer wheels of the truck should be centered at approximately 4½ feet from the test point.

(d) Deflection measurement.

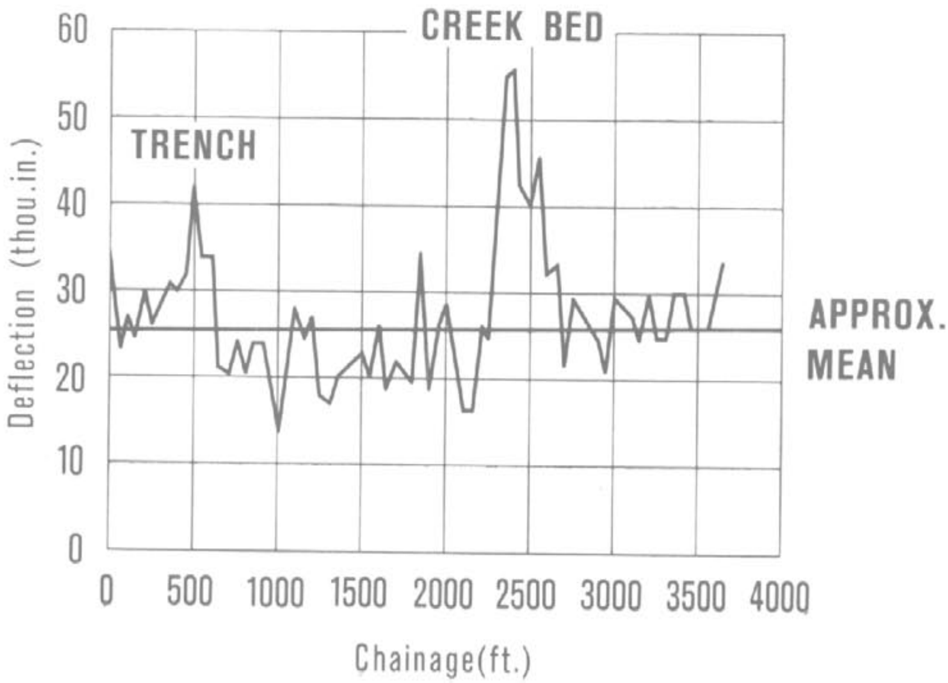
(i) Insert the probe arm between the dual tyres of the loaded test vehicle to a distance of about 4½ feet, lining up the arm by eye so that the beam will not be disturbed by forward movement of the truck.

(ii) Set the dial gauge to zero. Turn on the buzzer, thus causing vibration in the reference beam and preventing sticking of the stem activating the dial gauge.

(iii) Move the vehicle forward at creep speed. The operator should ensure that the tyre walls do not touch the probe arm. Record the maximum dial reading.

(Above): Benkelman Beam in operation

(Below): Typical plot of deflections showing causes of extreme deviations



Seismic survey explosion to generate shock wave

(iv) Move the vehicle at least 10 feet beyond the probe arm and record the final reading after 2 minutes, or sooner if the dial hand has come to rest.

Maximum Deflection

The maximum deflection is determined from the maximum dial gauge reading multiplied by the appropriate probe arm pivot length ratio. Typical deflections are shown in the figure on page 108.

Residual Deflection

The residual deflection is recorded 2 minutes after removal of the load or sooner if movement of the dial gauge has stopped. It is determined from the final gauge reading multiplied by the appropriate probe arm pivot length ratio. Permanent downward set is treated as positive deformation and is recorded with a positive sign.

Generally a downward set occurs with a magnitude of 0·000 in to 0·006 in. A rise in pavement level indicates the possibility of shoving. The rebound deflection values obtained on pavement where significant residuals (greater than 0·006 in) occur are of doubtful use and reasons for the occurrence of high residuals should be investigated. Isolated high residuals may be caused by movement of a surface stone.

Rebound Deflection

The rebound deflection is calculated by subtracting the residual deflection from the maximum deflection. Rebound deflections are used when considering the tolerable deflection. However, although the pavement may test within tolerable rebound deflection limits, it could still have excessive residual deflection.

Tolerable Pavement Deflections

The tolerable pavement deflections referred to below have been adopted by the National Association of Australian State Road Authorities.

The deflection of a pavement can be used as an indication of its overall strength. The following is a guide to tolerable deflections of a pavement and is based on measurements made by a Benkelman Beam placed between the tyres of a 9,000 lb dual wheel load.

The values are based on present knowledge and may be altered in the light of further experience.

For Department's work these data should be applied to REBOUND DEFLECTIONS.

For multi-lane roads the traffic density should not be assumed to be divided equally between lanes.

The values for tolerable deflections of a pavement are based on:

(a) Consideration of recommended values of tolerable deflections from economic performance of pavements reported from extensive deflection surveys in Europe and North America.

(b) Limited surveys conducted by the ARRB in Victoria and Tasmania, correlated with later performance.

(c) Experience of State Road Authorities with deflection surveys.

TABLE B1. Tolerable Deflections

Present traffic volumes (*commercial vehicles per lane per day)	**Tolerable deflections (ins) flexible pavement
Up to 25	0·070
25–75	0·055
75–225	0·040
225–750	0·030
750–2,250	0·025
Over 2,250	0·020

* Commercial vehicles greater than 3 tons.

** Mean plus twice the standard deviation.

The table should be taken as a guide but consideration must also be given to other factors for which quantitative adjustments cannot be made.

SEISMIC REFRACTION SURVEYS

The two main applications of seismic refraction surveys in the Department are:

1. To assess the depth to, and rippability of, various rock layers in road cuttings.
2. To determine the depth to bedrock at bridge sites.

The seismic refraction technique involves generating a shock wave at a certain known point close to the ground surface and measuring the arrival of the shock wave at a number of other known points using sensitive detectors called geophones. The time taken for the shock wave to travel to the geophones is dependent upon the elastic properties of the underlying materials. Knowing the position of the various detectors and the arrival times of the elastic wave at each of these detectors the operator can detect various layers through which the wave has travelled with a particular velocity, and can calculate depths to these layers. The calculations involve a number of assumptions about the underlying material and the way in which the elastic waves are propagated through the surface.

Depending on the size of the job and its position with regard to power lines, houses or heavily trafficked roads, the survey is either carried out with a 12 geophone system in which the shock wave is generated by explosives, or with a single geophone system in which the shock wave is generated by a 10 lb hammer striking a metal plate.

Because the shock generated by the hammer is relatively small the seismograph used to record the arrival of the shock wave operates on a signal stacking principle which allows signals from repeated hammer blows to be stacked (or added). Random background noise, generated by traffic for example, tends to cancel out during the stacking process which thus selectively reinforces the signals arriving from the hammer source.

Reference was made above to certain assumptions which are made about the way the elastic waves are propogated through the subsurface. Because these assumptions are rarely fully justified in practice, the depths determined by the seismic refraction method are approximate depths. Generally the accuracy of prediction is around $\pm$ 10%. If the seismic refraction technique is properly used however, this lack of accuracy is not critical.

For example, consider a bridge site. In the very early stages of investigation a number of sites may be under consideration. The seismic technique used at this stage will give a rapid, cheap, and quite accurate ($\pm$ 10%) idea of depths to bedrock. This information should be useful in selecting a final position. With a final position chosen and detailed design undertaken more accurate information is required. This is supplied by drilling. However even at this stage the preliminary seismic data is useful because the Department can give more precise details to the drillers as to likely rock levels and the driller is thus better able to select the appropriate drilling equipment to use. Further, both the designer and the driller can have more confidence that rock levels found in drilling are true bedrock levels, and not floaters, if the seismic survey has indicated bedrock at around the same depth. The continuous bedrock profile across the site, given by the seismic survey, will also enable extra-polation between boreholes to be made with greater confidence.

To conclude, the seismic refraction method is a cheap, rapid, reconnaisance technique which should be used at the planning stage of a project in order that full use can be made of the information it supplies. At the detailed design stage the seismic refraction data is still useful, both in planning the slower and costlier drilling programme and also in helping to assess and extend the drilling data.

CARBON MONOXIDE DETECTION SYSTEM FOR THE CAHILL EXPRESSWAY TUNNEL

Since February this year, the Department of Main Roads has operated an improved type of carbon monoxide detection system in the Cahill Expressway Tunnel.

During a severe traffic delay, when vehicles are likely to be moving very slowly through the tunnel, exhaust fumes will raise the normal level of carbon monoxide. At certain levels, depending on length of exposure and various other factors, humans will be affected by carbon monoxide poisoning and show symptoms commencing with a headache and progressing to nausea and drowsiness.

For this reason an effective carbon monoxide warning system has been installed in this tunnel and linked to a powerful automatic ventilating system.

The one other tunnel in the Main Roads network with a similar type of warning and ventilating system is on General Holmes Drive (Main Road No. 194) under the extended runway of Kingsford Smith Airport. The small circular tunnel leading on to the Bradfield Highway at the southern end of Sydney Harbour Bridge has no carbon monoxide detection equipment but does contain a ventilating system which clears the air by working on a fixed programme. All other tunnels are short enough to have effective natural ventilation. When completed, the Kings Cross Road Tunnel will become the third tunnel in New South Wales to contain a full detection and ventilation system.

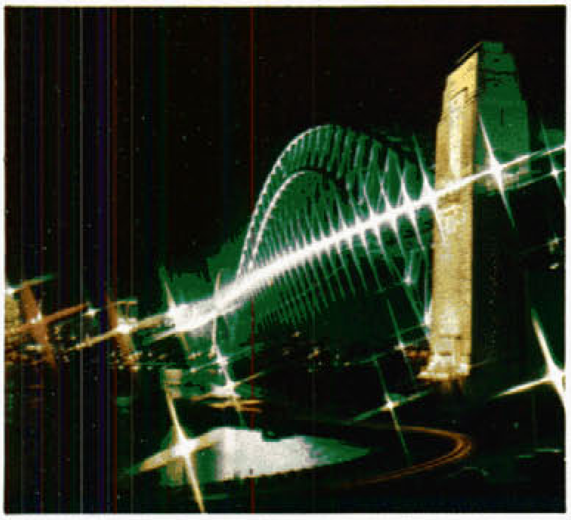
The System

The Cahill Expressway Tunnel has been divided into two equal sections, each with four sampling points evenly spaced along the ceiling of the tunnel, directly above the median strip.

At the eight sampling points, a diaphragm pump draws gas samples from the atmosphere inside the tunnel. These samples pass through filters which remove all dust, moisture and other components likely to effect the accuracy of the analyser. When the filtered air reaches the analyser it is heated to a constant temperature and a catalyst causes the carbon monoxide and oxygen to react. This reaction results in a change of temperature which in turn is transformed into an electrical signal to drive a meter and a continuous chart recorder.

At set levels of carbon monoxide concentration, a relay system is automatically activated to switch the fans on and off. When the carbon monoxide content reaches 100 ppm (parts per million) the fans begin to operate. They stay on until the level drops to 50 ppm. However, should the level continue to rise regardless of fans, and exceed 400 ppm for more than 3 minutes, then the traffic lights at both ends of the tunnel switch on. The lights switch off when the level drops below 400 ppm. To date, these traffic lights have only needed to operate during routine testing.

When maintenance staff are working in the tunnel, for example on cleaning work or linemarking, the fans are switched on to ensure that the carbon monoxide content is maintained below the 50 ppm level●



DUSK TO DAWN

Top: A fascinating pattern of light trails cross Stockton Bridge, Newcastle at sunset.

Above: By day or night, Sydney Harbour Bridge is always a major attraction, but night lights give it a special beauty.

Right: Emerging from the dawn light, the outline of Gladesville Bridge appears in silhouette against Sydney's skyline.

1

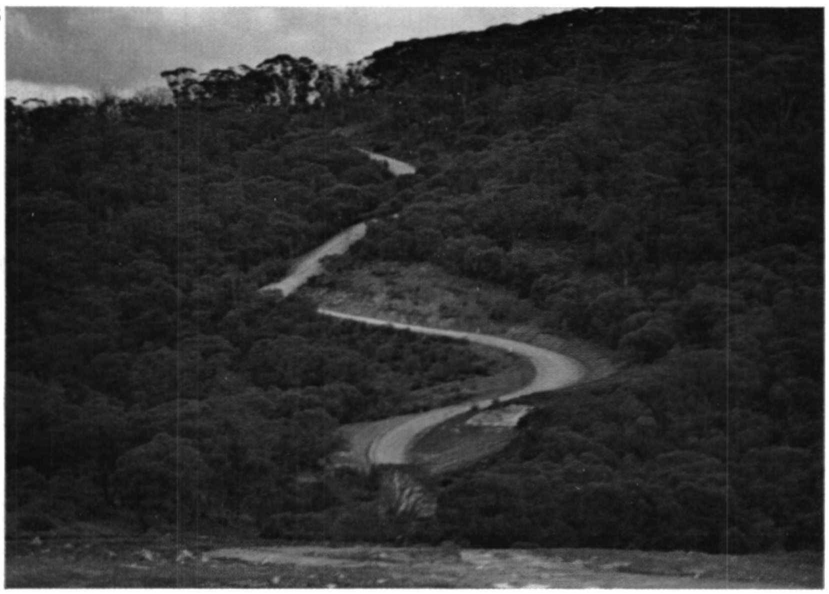

'Then fast the horsemen followed, where the gorges deep and
black,
Resounded to the thunder of their tread,
And the stockwhips woke the echoes and they fiercely
answered back
From cliffs and crags that beetled overhead.
And upward, ever upward, the wild horses held their way,
Where mountain ash and kurrajong grew wide;
And the old man muttered fiercely, "We may bid the mob
good day,
No *man can hold them down the other side".'*

The land that inspired A.B. 'Banjo' Paterson's stirring ballad, "The Man From Snowy River", becomes a reality when travelling along the Snowy Mountains Highway (State Highway No. 4), in the Kosciusko National Park.


2

The Snowy Mou

4


3


1. *Looking south from near Ash Creek.*
2. *Basecourse being stock piled prior to spreading on Fiery Ridge section between Mills Creek and Ash Creek.*
3. *The highway at present in use, near Rules Point, presents a contrast to the smooth curves of the new route.*
4. *Blowering Dam is one of the scenic highlights along the route of the Snowy Mountains Highway.*
5. *Pipe culvert at Mills Creek with new construction proceeding above it.*
6. *Fenced section of reconstruction near Rules Point.*
7. *Looking towards Inspiration Point from a hill above Mills Creek.*
8. *Section near Rules Point under construction and old section still in use (on right).*


5


6

ntains Highway

The Kosciusko National Park, formerly the Kosciusko State Park, is administered by the National Parks and Wildlife Service. Centre of Australia's foremost alpine sports, there are also many other activities for tourists in the Park area including walking, horse riding, trout fishing and camping. The well known highlights such as Yarrangobilly Caves and the Snowy Mountains Hydro-Electric Scheme attract thousands of visitors every year.

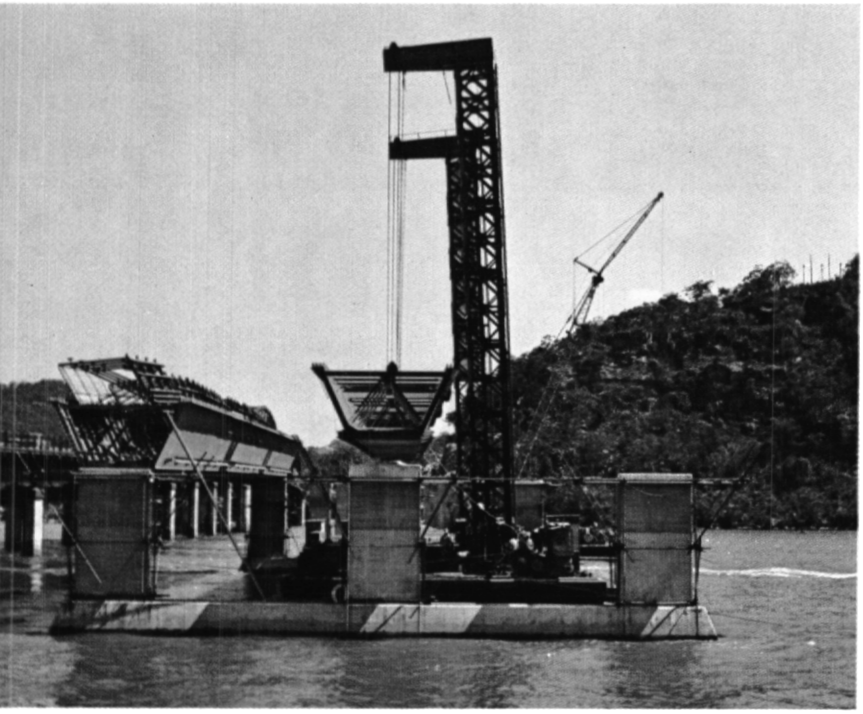
Between Kiandra and Inspiration Point, a 31-mile section of the Snowy Mountains Highway is being reconstructed. This section of the highway is located entirely within the boundaries of the Kosciusko National Park, and the new reconstruction will permit improved access to the main tourist areas of the Park for travellers.

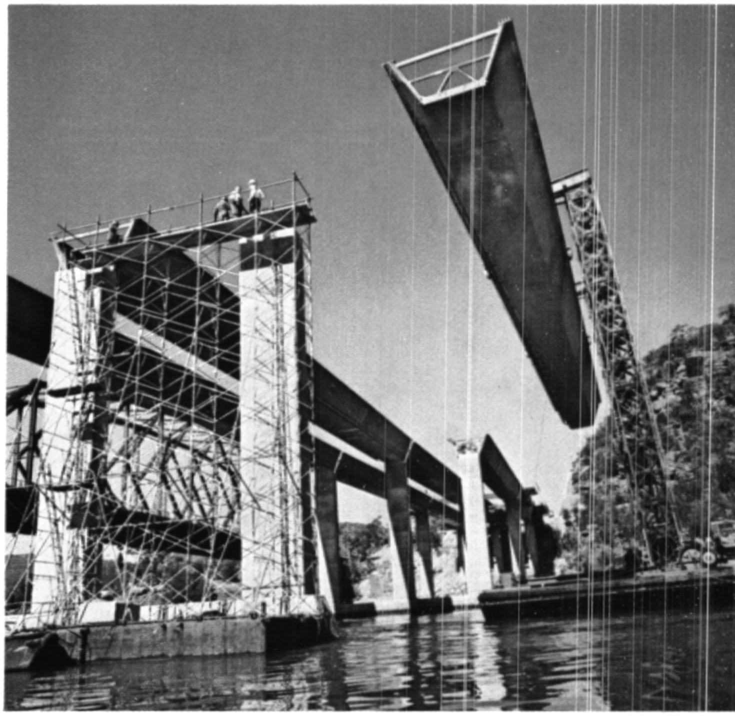
7


8


Motorists will soon be using the new 2,020 feet long bridge over the Hawkesbury River which forms part of the Sydney-Newcastle Freeway linking the two sections of Tollway already in use.

NEW BRIDGE





Top: November 1971 and the superstructure begins to grow.

Centre: Two views of the erection of the steel girders – February 1972 on the left and August 1972 on the right.

Far left: Vibrating concrete on the deck, with the pontoon towers used for positioning the girders clearly visible in the background.

Left: October 1972 and work on the deck is well advanced.

The 84 feet wide deck will have six lanes for traffic. There will be three lanes in each direction separated by a median barrier two feet wide and 2 feet 8 inches high, the first of its kind to be constructed on a bridge in New South Wales. The bridge has a constant downhill grade of 2½ per cent from the South. Pier positions of the new bridge have, where possible, been matched with those of the existing highway bridge which is situated 100 feet downstream. Minimum interference with river traffic is thus assured.

OVER THE HAWKESBURY RIVER

Towards the end of 1970, as the contract for the piling work on the new Freeway Bridge neared completion, tenders were called for the construction of the superstructure of the bridge. The successful tenderer was the Hornibrook Group and the contract was signed on 28th January, 1971.

The superstructure contract required the construction of:

- 30 pier columns over water, ranging in height from 14 feet to 54 feet;
- the north and south abutments;
- the spread footing and three 57 feet columns for pier 1;
- 36 open trough steel girders, three of 90 feet, nine of 160 feet, and twenty-four of 180 feet length;
- the bridge deck in prestressed and reinforced concrete;
- kerbs, median and safety railing.

In the design, prepared by Departmental engineers, the girders were not planned as closed "boxes" at the time of erection but were to be erected as open trough trapezoidal girders. Concrete boxing for the deck is supported from the girders after erection. The top 18 in x 2 in flanges of the girders have welded to them a pattern of stud shear connectors.

The casting of the concrete deck on to these studded flanges completes the "box" shape of the girders and develops composite action between the steel flanges of the girders and the concrete of the bridge deck.

CONCRETE MIXING AND PLACING

Concrete required for the contract, amounting to 7,050 cubic yards, was batched at the Pioneer Concrete Plant at Thornleigh, 20 miles south of the bridge. Two classes of concrete were specified, viz.: a 4,500 psi class for the pier columns (except the top sections) and for the bridge deck and a 7,000 psi class reserved for the last pour in each column and for the column crosshead girder. Because of the long haul from Thornleigh, a minimum of one hour of continuous mixing of concrete in the trucks took place before discharge. An agent to delay set in the concrete was therefore used in all batches. No adverse effects from the prolonged mixing have been observed. It was not considered advisable however to prolong the mixing of the 7,000 psi concrete due to the high cement content. Cement for this concrete was therefore stored in a silo and batched into the trucks at the bridge site. Placing of the concrete would then proceed promptly. 580 cubic yards of concrete was batched by this method.

The deck slab comprises both reinforced and prestressed concrete. The sequence of pouring was laid down in the plans. In this sequence, a 70-feet length of deck, formed centrally over each pier was cast first. The pour also included the column crosshead girder (see fig. 2 on page 118). This section of deck was completed by the stressing of the vertical column-to-girder Macalloy bars, longitudinal stressing of the 70-feet deck section and transverse stressing of the crosshead girder. The reinforced concrete deck work between these sections, also placed to a sequence, then proceeded.

Good use was made of the proximity of the existing bridge for placing concrete in the new structure. Concrete for the 33 pier columns was delivered by agitator trucks to the old bridge and placed by concrete skips swung from a barge mounted crane. Concrete for a number of deck pours was also pumped from this position across a light steel bridge. As the work progressed concrete deliveries were made over, and pumped from, the completed sections of the deck. A Torket twin piston concrete pump with a 5-inch delivery line was used for this purpose and was able to maintain a steady delivery rate of approximately 30 cubic yards per hour. The largest single continuous pour was 260 cubic yards. A team of twelve concrete placers and finishers handled the deck concrete. The concrete was compacted by internal vibrators, trimmed by single hand screed boards, sealed by use of a long-handled double-action steel trowel and finally broomed to roughen the surface for better bond with the bituminous road surfacing. This will consist of a minimum of 2 in of bituminous concrete placed by the Department's own labour and specialised equipment.

STEEL GIRDER FABRICATION, SPLICING AND ERECTION

Fabrication

The 2,809 tons of steel required for the 36 "box" girders was produced by the B.H.P. Company at their Whyalla, Port Kembla and Newcastle mills and transported by sea and by rail to Newcastle. Inspection of the steel at the mills was undertaken for the Department by the New South Wales Public Transport Commission railway steel inspectors.

Two classes of steel were specified: AS A135 1965 to grade ND1 Class B for box girder bottom flange plates and AS A149 1965 for all other steel except rolled sections.

The manufacture of the girder segments (there being three to each girder) was sub-contracted by the Hornibrook Group to John Lysaght Aust. Ltd and carried out by that Company's Engineering Division at Newcastle. Difficulties to be faced in fabricating such large girders (14 feet wide top width x 7 feet high x 8 feet wide bottom width) having web plates of $\frac{3}{8}$ in thickness, were recognised from the outset. Mockup panels were prepared to test weld sequences, electrode sizes and types, and heat shrinkage effects on the flatness of typical web plates. The fabrication of the segments required them to be turned a number of times and for this purpose special strong frames were made. Although the girders have a system of internal ribs and stiffeners, both temporary and permanent, they are nevertheless somewhat flexible until construction of the deck slab is complete.

METHOD OF FABRICATING SEGMENTS

Work commenced with the fabrication of the internal cross ribs. Accurate jigging assured uniformity of members. Two 30 ton gantry cranes spanning 80 feet were used throughout to handle and position all elements of the girders.

The segments were assembled in an upside down position in order to complete as much of the welding as possible in the down-hand position, for ease and convenience. The heavy 2 in thick top flange plates were laid down first and wedged to line stops in the jigs. Ribs were then erected, spaced and fixed by welding to these flanges. The $\frac{3}{8}$ in web plates were next placed against the ribs, straightened, tacked, then welded by backstep sequence to the flange plates and to the ribs. The web plates had been previously prepared with bevelled edges by machine flame cutting and finish grinding to fit snugly at the correct side slope on to the flange plates in readiness for inside and outside fillet welds.

The heavy Tee section longitudinal stiffeners to the bottom flanges were next welded into position. The bottom flange plate (8 feet wide x $1\frac{1}{4}$ inch thick) required butt welding to make up the full 60 feet segment length. These butt welds were checked radiographically. After lifting the plate into position a heavy weight, in the form of a roller, was used to flatten the plate in readiness for tack welding to the ribs and to the webs. The partly completed girder segment was then lifted and turned into the upright position. Welding of the web plates to the bottom flange then proceeded in the down-hand manner. Shop matching, trimming and bevel edge treatment of the inner ends of the segments completed this stage of the work.

FINAL DETAILS

A total of 73,500 shear studs (max. size 9 in x $\frac{7}{8}$ in diam.) were welded to the girder flanges and ends by a semi-automatic welding gun while in the Newcastle fabrication shop.

Protective treatment of the steel commenced with abrasive grit blasting of all surfaces. Hot zinc spraying to a minimum thickness of six one thousandths of an inch then followed. Areas to be field welded at the bridge site were masked. Final shop treatment comprised an acid etch coat followed by a spray coat of zinc chromate. Surfaces to be cast into concrete were given a cement spray coat. The sections were treated in the upside down position in a paint shop adjacent to the fabrication shop to which they were moved by rail track.

Final paint treatment at the bridge site consisted of two coats of gloss paint; forest green on the outside of the girders and white on the inside and on the kerb crash rails. All paint and protective treatment work has been carried out under subcontract by Newcastle Protective Coatings Pty Ltd.

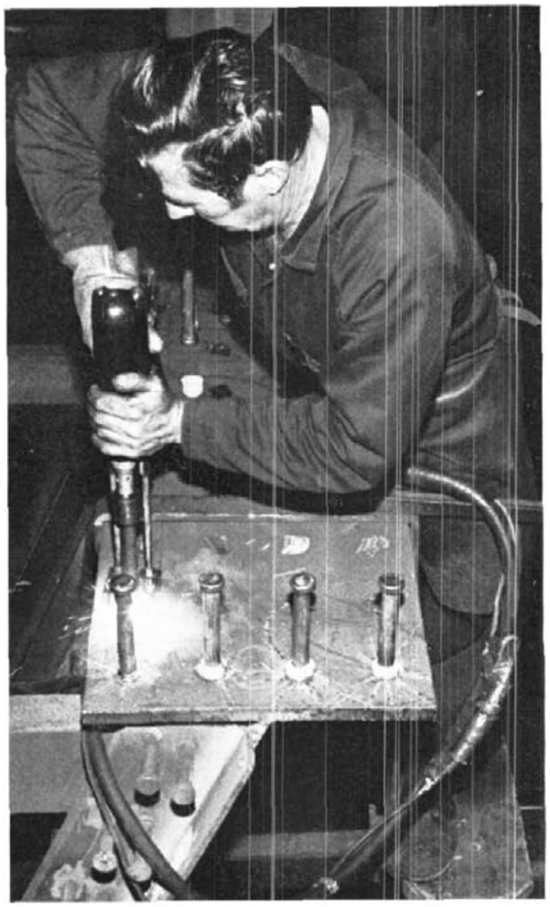
At the peak of production, up to ninety men were employed on girder fabrication. Three shifts were worked and, for a period, on a seven-day-week basis.

The girder segments were transported by road to the Hawkesbury River site, a distance of 77 miles, using a prime mover and jinker. The segments were mostly 60 feet in length. In Span 1 however, the three girders are 90 feet long and these were delivered full length to the site for launching from the southern bank.

ON-SITE SPLICING OF GIRDER SEGMENTS

Handling and splicing preparation

A 40-ton gantry crane spanning 52 feet, and running on rail tracks, was erected to lift the girder segments from the trucks and position them in readiness for welding. Six sets of steel trestles were set up at a height of 5 feet to support the three segments in each girder and thus provide a comfortable height for the manual welding work on the underside of the 8 feet wide bottom flange. The completed girders were moved from the welding bay to the launching jetty on rail tracks provided for this purpose.

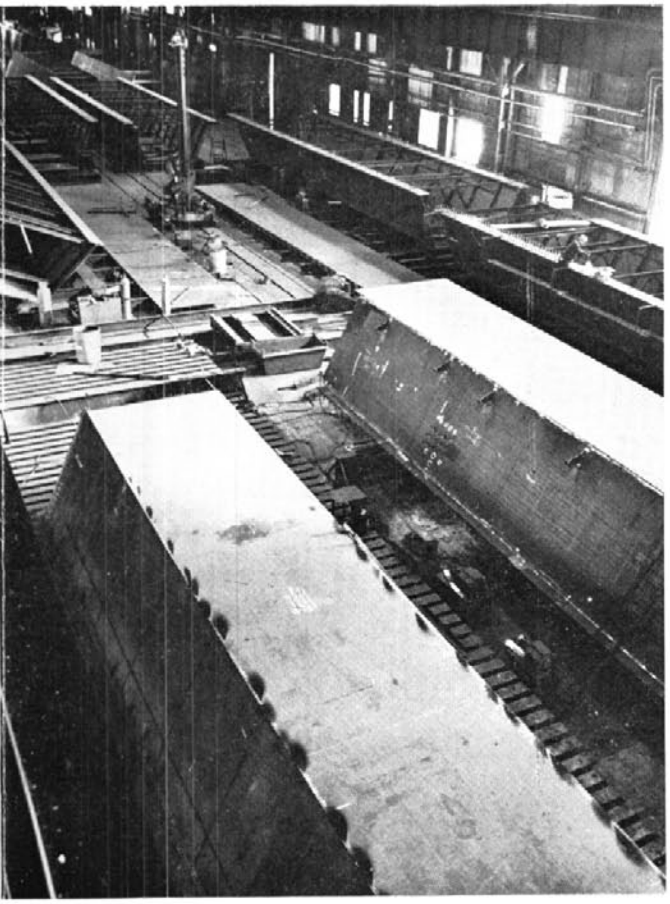


One of the 73,500 sheer studs being welded to a girder flange.

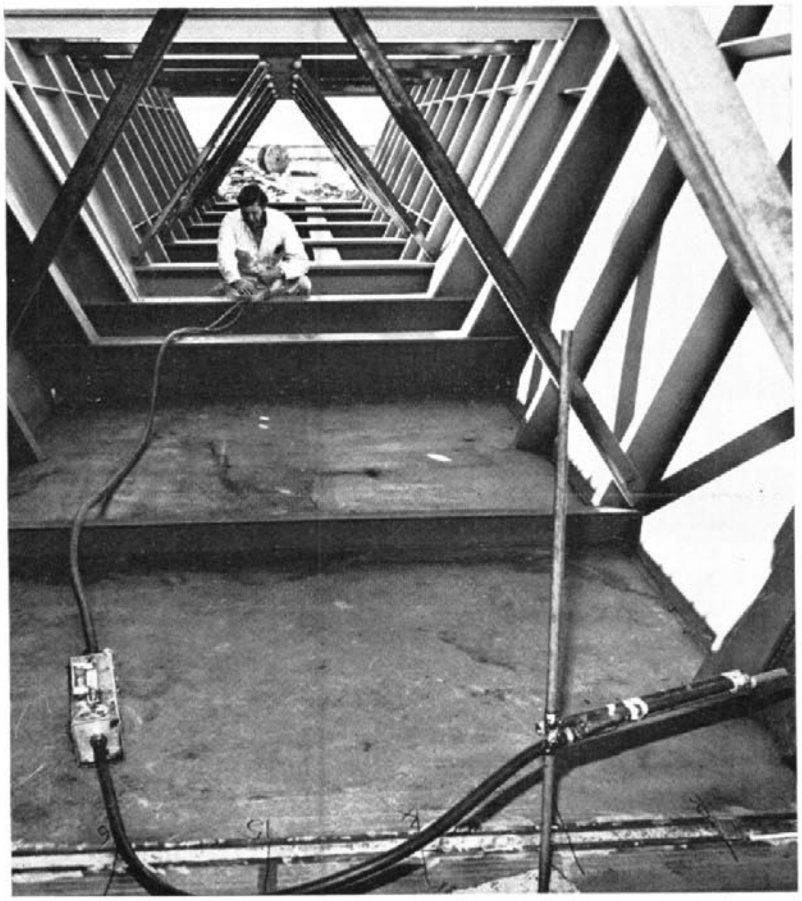
The girder segments were butt welded together and being of such large cross sectional dimensions and trapezoidal shape, this presented a challenge. To facilitate lining up the units, a number of welded "dogs" were permitted on the interior side of the girder web plates and top flanges. The segments were brought into line by use of a single taut piano wire. The girder camber was then set and checked with a dumpy level. After final checking for root gap clearance between the segments the abutting edges were tack welded.

Welding

Specification requirements for the butt welding were demanding. Full radiographic inspection of all bottom flange welds was called for and, in addition, lengths of welds in other positions were



Interior of the workshop where girders are being fabricated.



Checking the welds radiographically. The operator positions himself away from the check point to be well clear of the gamma rays

regularly checked by this method. A high degree of concentration was necessary on the part of the welders to produce fault-free welds to the standard required.

Prior to any welders being employed on this work they were required to undertake a test of the type of welding that would be met on the job. The test weld, which included overhead work, was examined radiographically and only one sixth of the welders nominated for this work passed this test. It was necessary to work on two girders simultaneously to achieve the required production. The work was carried out by a small team of four specialist welders and one iron worker.

For the butt welds, low hydrogen electrodes were used to minimise the chance of dangerous hydrogen embrittlement of the welds. Heating boxes were on hand at all times for the storage of the electrodes at the correct temperatures. To ensure dry and windproof working conditions large removable canopies were set up over each splice zone. The welding sequence was:

(1) butt weld $\frac{3}{8}$ in web plate;

(2) weld underside V of 8 ft wide x $1\frac{1}{4}$ in thick bottom flange;

(3) weld half of topside V of bottom flange;

(4) weld underside V of 2 in thick top flanges;

(5) weld half depth of topside V of top flanges;

(6) complete bottom flange weld;

(7) complete weld of top flanges.

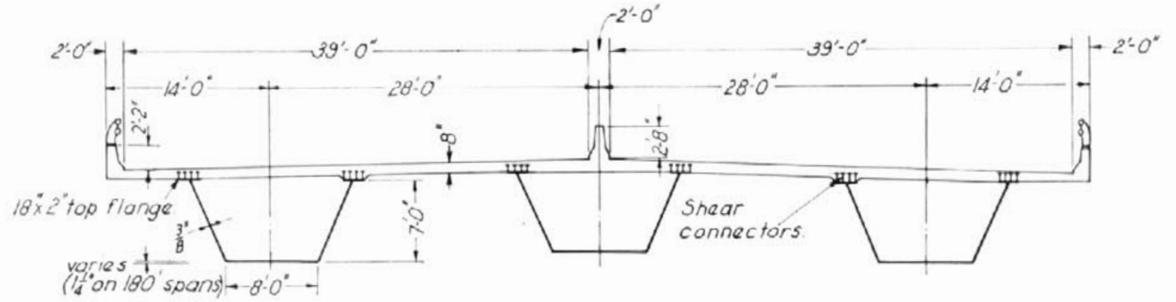
Both manual and semi-automatic welding by the submerged arc process were used, the latter process being confined to stages 3, 5, 6 and 7 above. All plate over 1 in thick was preheated before welding. A number of techniques was devised as work proceeded to minimise the expansion and shrinkage effects on the $\frac{3}{8}$ in plate during the welding operation and the final appearance of the webs has been most satisfactory.

Accidental arc strikes were a source of some concern and strong efforts were made to eliminate these, especially from the bottom flange. Hardboard masks on plate adjacent to the welds proved the most effective device. Back gouging and corrective cut-outs were done by the air-arc method.

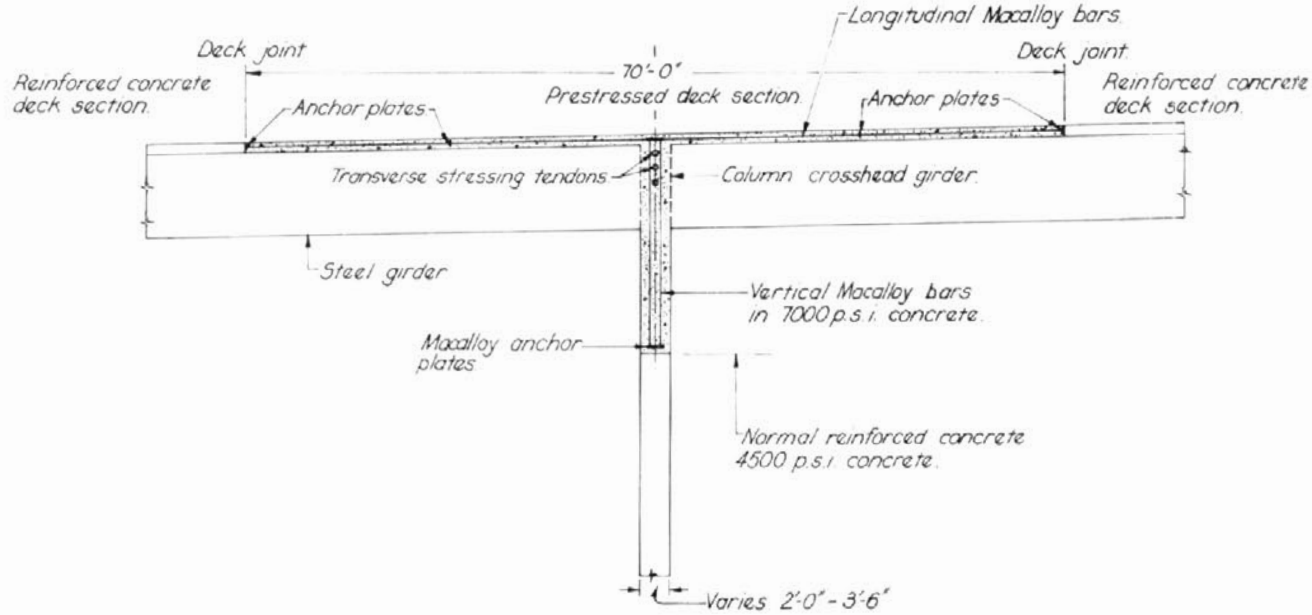
Die penetrant tests were occasionally used to confirm visual examination of the welds. Radiographic examination of completed welds was done using portable gamma ray equipment. This work was undertaken for the Department by X-Ray Inspection Co. Pty Ltd of Marrickville.

The best time recorded for the complete butt welding of a girder, measured from

Welding webplate to bottom flange.



TYPICAL CROSS SECTION OF DECK



GIRDER TO COLUMN JOINT

the time set up of sections was approved until final acceptance of the girder welds in readiness for painting was nine working days. The average time was 13½ working days.

LAUNCHING AND ERECTION OF GIRDERS

Transfer of the 84 ton girders from shore to the lifting barge was facilitated by the construction of two launching jetties, 130 feet apart, extending into the water a distance of 200 feet from the assembly area. Rail tracks were provided on each jetty. The girders (up to 180 feet length) were thus winched side-on to the ends of the jetties and picked up by the lifting equipment on the pontoon positioned between the two jetties. To prevent distortion to the girders special steel lifting beams were bolted to the top flanges of the girders at the two lift points.

The lifting and placing of the 33 girders into position on the pier columns was effected with comparative ease although there were occasional difficulties. The method, planning and execution of the operation reflected considerable credit on the Hornibrook Group's organisation.

The method selected was to set up two 70 feet high towers on a strongly built 100 feet x 40 feet timber pontoon. The pontoon had been suitably strengthened for the work by the provision of a base frame for the towers made up of steel beams bolted to the deck and pontoon ribs. Each tower was assembled from 10 feet Bailey Bridge panels erected seven high and made up of six frames to a panel, in two pairs of three, cross braced. The pontoon was moved to various locations in the river by power winches mounted on the deck and pulling on suitably placed anchors in the river and on shore. The lifting equipment consisted of two winches with a selection of speeds, one winch serving each tower. A headframe cantilevered out 11 ft 6 in at the top of the tower to provide adequate clearance for hoisting the girders. Ten falls of rope were used at each of two lift points. The controlled lowering rate of the girders from the towers was approximately 3 ft per minute.

Lifting operations were done in the mornings and in calm conditions to avoid gusty wind effects. The operation of lifting a girder from the jetty, traversing the river, manoeuvering into position, hoisting and seating the girder on the column tops, took approximately two hours. The midstream current at outgoing tide was about six knots and the tidal range approximately six feet.

FORMWORK FOR DECK CONSTRUCTION

All deck formwork was suspended from, or attached to the girders after erection. As a preliminary to this work the three trough girders were temporarily stiffened across the top flanges by three steel beam "strongbacks" equally spaced along the girders. The "strongbacks" were positioned clear of the flanges to enable adjustment of the flanges into level by a system of clamps and jacks. The relative positions were finally secured by temporary welding ties or struts from the "strongbacks" to the flange tops. In mid-summer conditions, the twisting effect caused by hot sun rays striking one sloping side of the girders, was also restrained by the "strongbacks".

Formwork for the deck slab cantilevered beyond the girders and was supported by triangular steel frames bolted to the outer top flanges and packed against the outer web plate at the bottom. Plastic coated plywood, $\frac{3}{4}$ in thick, was used almost exclusively for formwork sheathing over joists of timber or steel.

To advance the work as rapidly as possible, and to work within the required pouring sequence, sufficient formwork was made up for three 70 ft over-pier positions and for two intermediate mid-span sections of deck.

LAUNCHING OF SPAN 1 GIRDERS

An aluminium truss, 97 ft long, was selected as the means to launch the three 90 ft girders in span 1. The truss had previously been used by the Contractor for the construction of the De Burghs bridge for the Department. One end of the truss was anchored to a girder positioned on bogies and rails on the approach road at the abutment. The other end cantilevered out approximately 90 ft. The truss and girder were moved forward by hand winches. The $2\frac{1}{2}$ per cent downgrade of the bridge assisted this operation. The aluminium truss was then supported on steel support frames (resting on transverse steel beams) at the pier 1 column and at the abutment. Trollies on top of the truss moved freely on power skates. Long hydraulic rams hung from the trollies and were used to lift the trough girder which was then winched forward into position and lowered by the rams. After each girder was placed the launching truss was moved sideways on the steel beams to the next position. Rail tracks to move the girders along the approach road were similarly relocated. The $7\frac{1}{2}$ ton aluminium truss was finally recovered by a large mobile crane; it had proved to be most useful erection tool and one with many similar applications.

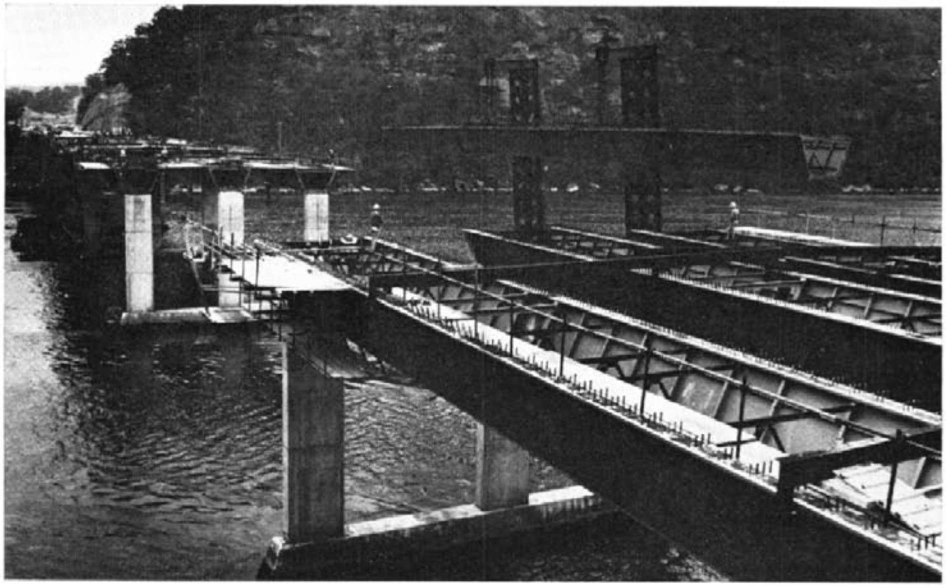
* * *

The new bridge over the Hawkesbury River has been one of the largest bridge building tasks undertaken by the Department of Main Roads in recent years.

At the height of construction work in October, 1972 the Contractor had a labour force of 77 men. A wide variety of occupations were represented including carpenters, labourers, boilermaker welders, ironworkers, riggers, dogmen, crane drivers, scaffolders, boatmen and painters.

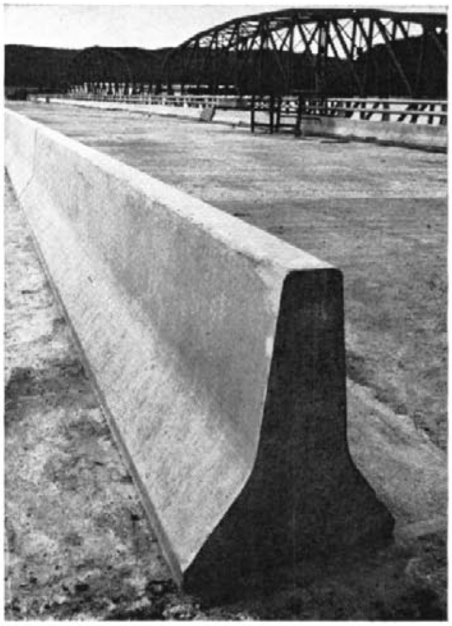
Materials used included 920 tons of reinforcing steel, 185 tons of Macalloy high tensile bars and accessories and over 13 miles of prestressing ducting. Protective treatment was provided to 41,000 square yards of steel surface.

The total cost of the bridge and approach work will be about $5·5 million●



The girder which will seal the link between the shores of the river is about to be set in place

The median strip is of unusual design to prevent "head on" collisions by deflecting vehicles which touch the sloping sides.



Previous articles on the Hawkesbury River Bridge appeared in "Main Roads", Vol. 35, No. 3, March 1970, pp. 66–67 and Vol. 37, No. 2, December 1971, pp. 54–58.

Lawrence Hargrave Drive

THE MAN BEHIND THE NAME

"LAWRENCE HARGRAVE — 1850-1915, whose pioneering research in aeronautics with engines, monoplanes and box kites, much of which was carried out at Stanwell Park, played a vital part in the development of the aeroplane."

For part of its route, Lawrence Hargrave Drive (Main Road No. 185) winds around a spectacular section of coastline in the south coast region where this famous scientist lived and worked. The inscription, above, appears on the Lawrence Hargrave Memorial situated close to the junction of the Drive with Main Road No. 393 at Stanwell Park. At its southern extremity the Drive connects with the Prince's Highway (S.H. 1) at the foot of Bulli Pass.

Born near Greenwich, England in 1850, Lawrence Hargrave arrived in Australia in 1866. His aptitude for mathematics led him to a position in the engineering shops of the Australian Steamship Navigation Co. Later, after a number of adventurous trips to northern Australia and New Guinea, he became Assistant Government Astronomer at Sydney Observatory in 1878.

He held this position for five years and during that time his interest in human flight increased. Relying on a small fixed income, he left paid employment and pursued his scientific experiments. In all, he designed fifty-two models, thirty-three of which were built. Of the engines he designed, his most remarkable was a three-cylinder radial rotary air-screw engine designed in 1889. Lawrence's design was never patented and in 1908 a similar engine was patented overseas. It was this type of engine which later powered most European aircraft in the early years of powered flight.

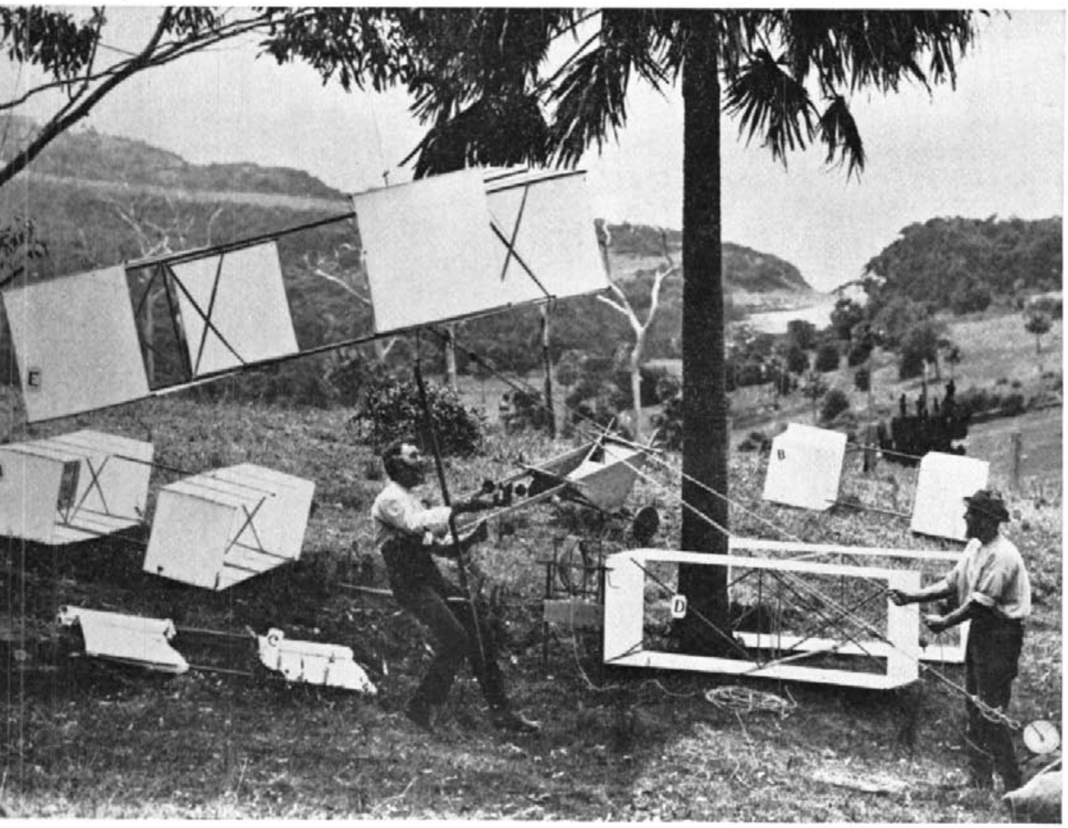
In 1893, Hargrave moved to Stanwell Park, near Wollongong where his family had held land for many years. This move was made to reduce expenses, obtain steady winds and permit experiments to be made without public interruption. It was here that he flew his famous box-kites and conducted many experiments with propellors, engines and curved wing surfaces. He generally flew models but in 1894 flew himself in a set of four box kites at a height of 16 feet.

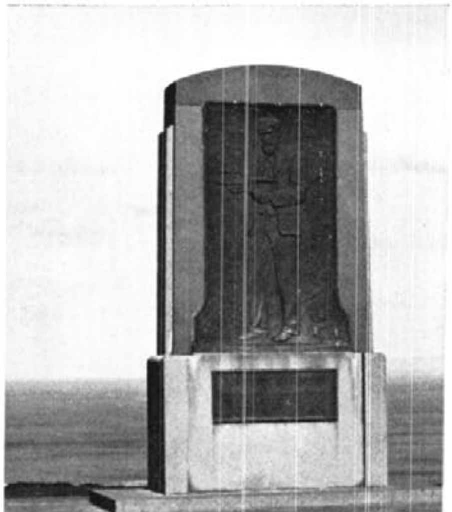
In 1899 the whole family left for England, but as in Australia, he found high costs and lack of opportunity and interest in his work there, so returned to Sydney. The family took up residence eventually in a house on Woollahra Point where flight trials could be made on the waters of Sydney Harbour.

Hargrave's work was still almost ignored in Australia and had he been able to liaise more closely with other aviation pioneers overseas, it is possible that he would have achieved much more. However, his continuing efforts to produce a satisfactory engine were unsuccessful and he suffered further unhappiness and frustrations when ridiculed for his carefully considered theory that the Spaniards had discovered the east coast of Australia in 1595. His son Geoffrey, who showed great promise of following on his father's experiments, was killed in Gallipoli in 1915 and Hargrave died just six weeks later of peritonitis, or perhaps the real cause was simply loss of the will to live●

The assistance of the Museum of Applied Arts and Sciences, Broadway, in the compilation of this article and for permission to reproduce the photograph of the flat-surfaced kites, is acknowledged.

This is the second article on famous people after whom roads have been named. The first, "Who Was General Holmes", appeared in the September 1972 issue (Vol. 38, No. 1, p. 25).





The Lawrence Hargrave Memorial at Stanwell Park

Lawrence Hargrave (figure left) with the four flat-surfaced kites which he flew 16 feet high at Stanwell Park in 1894

A PROGRAMME FOR TRAFFIC RELIEF

ORANGE

(Top of page): Aerial view of main commercial area in Orange

The city of Orange, 163 miles west of Sydney on the Mitchell Highway, is a fairly typical New South Wales country town. Since October 1972 when the New South Wales State Government announced that the State's first planned growth centre would be Bathurst/Orange, more attention than normal has been focused on these cities.

Their selection as a planned growth centre was obviously influenced by factors including location on road communication to many markets (Brisbane to Orange is about the same distance as Brisbane to Sydney and over a highway with a low traffic density; Orange to Melbourne is shorter by nearly 100 miles than Sydney to Melbourne; Adelaide and Perth are obviously closer to the area than to Sydney). Orange and Bathurst are also served by the trans-continental railway route giving direct access to Sydney, Adelaide and Perth. The line running south from Blayney also gives direct access to Melbourne.

A most important point is that Orange and Bathurst are in the "safe" rainfall area of the State with a generally reliable annual rainfall and considerable potential for developing a water supply and sewerage service to the capacity required by a large population.

Orange has a traffic problem shared by many country towns situated on State Highways where the main commercial street is also the highway.

This problem will be accentuated by rapid growth, and extensive investigations were carried out by Consultants and the Department of Main Roads. The Department concluded that a traffic relief route to the north and east of Orange, with suitable access points to the City, is the most desirable solution to the problem.

The purpose of this article is to describe the steps taken to locate the best possible traffic relief route and give a glimpse of the co-operation required in the planning of such a road. Similar investigation methods could be applied to the traffic problems of numerous other rural cities.

The Orange District

The site of Orange was proclaimed a village on 20th November, 1846. Prior to this date the district was being grazed and cropped for wheat, maize and oats and it was not long afterwards that the gold rushes of the 1850's gave it a further boost.

It seems likely that the name of Orange came from Surveyor General Mitchell's association with the Prince of Orange in the Peninsula Wars. Orange was proclaimed a municipality in January 1860, a town in 1885 and a city in 1946. The population has shown a steady growth and as well as the firmly established primary industries of the district, the city has a number of large secondary industries.

A Programme for City Traffic Relief

As can be seen from the map on page 123 the line chosen for the traffic relief route at Orange runs to the north and east of Orange, with suitable access points to the City.

This by-pass route would separate through traffic from local traffic, to the benefit of both, and provide easier, faster, safer access to the City from surrounding areas. The conclusions reached are based on an extensive investigation, carried out by both the Department's staff and by Consultants engaged for the purpose.

Population and Vehicles

In 1971, the population of Orange (city) was 23,143. It has shown consistent growth over recent years and accordingly, the number of road vehicles in the district has also increased. It is no wonder that parking is sometimes difficult or that intersections become congested.

If, into this local traffic, are added the streams of long distance traffic passing through Orange, the reasons for present-day traffic delay and frustration are obvious. It would be better if local commercial and private traffic could go about its business without mingling with long-distance vehicles. Then the through traveller need not be irritated by speed restriction signs and cross traffic, and the local motorist would not have to manoeuvre in, and wait to cross, streams of through traffic.

This problem is not new and is common to all growing cities and towns in the western world. The obvious solution is to plan for a traffic relief route separate from the existing road system and to improve the capacity of the arteries leading into the city from it. There should also be a convenient safe means of travel from the through route into the business and commercial centre.

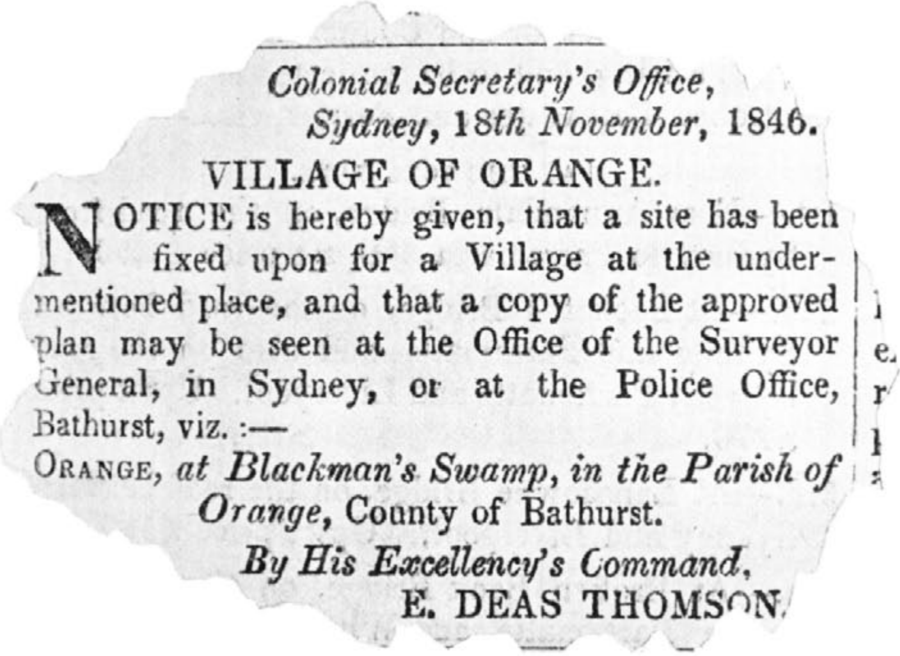
It is impossible to build a railway line or a major road today without disturbing the homes and lives of some people. The objective is to develop transportation facilities that create the least disruption to the smallest number of persons, and the greatest benefit to the greatest number of people.

History of Possible Traffic Relief Routes

As far back as 1955, the Department first adopted a line for relieving traffic congestion in Orange. This line is shown on the map on page 123. Although formally accepted by Orange City Council, it was not long before both Council and the Department realised that the line had unsatisfactory features, particularly relating to development in the vicinity of Woodward Street, making it undesirable for the line to pass along that street.

Preparation of the Orange Town Planning Scheme drew attention to the need for a traffic relief route to be chosen having full regard to town planning, traffic existing and anticipated, and engineering factors.

To assist in the choice of a route an "Origin and Destination" survey conducted by roadside interview was carried out on Tuesday and Wednesday, 5th and 6th June, 1963, for the six major outlets from Orange. The survey was run in conjunction with consultants, Orange City Council, Canobolas Shire Council and the Department of Main Roads.



Colonial Secretary's Office,
Sydney, **18th** *November*, **1846.**

VILLAGE OF ORANGE.

NOTICE is hereby given, that a site has been fixed upon for a Village at the under-mentioned place, and that a copy of the approved plan may be seen at the Office of the Surveyor General, in Sydney, or at the Police Office, Bathurst, viz. :—

ORANGE, *at Blackman's Swamp, in the Parish of Orange*, County of Bathurst.

By His Excellency's Command,
E. DEAS THOMSON.

(Left): Extract from "New South Wales Government Gazette", Friday, 20th November, 1846

The survey's main purposes were to ascertain the origin, destination, volume, percentages and character of medium to long distance traffic outward bound from Orange—to find out the entry points and routes through Orange now taken by this traffic—to ascertain whether, on the days of the survey, a north or south traffic relief route would be the more used, and to assign current volumes to those alternative routes—to provide data for the preliminary design of intersections and interchanges—to assist in determining the most suitable location for a main entrance to the City.

Amongst the survey findings were the following:

1. Over 50 per cent of all vehicles interviewed were travelling to destinations less than 25 miles from Orange.
2. The bulk of through traffic is along the Highway. The Trunk and Main Roads largely carry only local traffic.
3. The predominant traffic flow is to and from the City.

Initial Investigation by the Department

On receipt of the survey's findings, the Department considered the points of comparison between a southern and western route as compared with a northern and eastern route.

Points in favour of the southern and western route were:

1. No possibility of a barrier to further urban expansion.
2. Access well served to the Racecourse, Duntry League and Wentworth Park.
3. More traffic would be served.

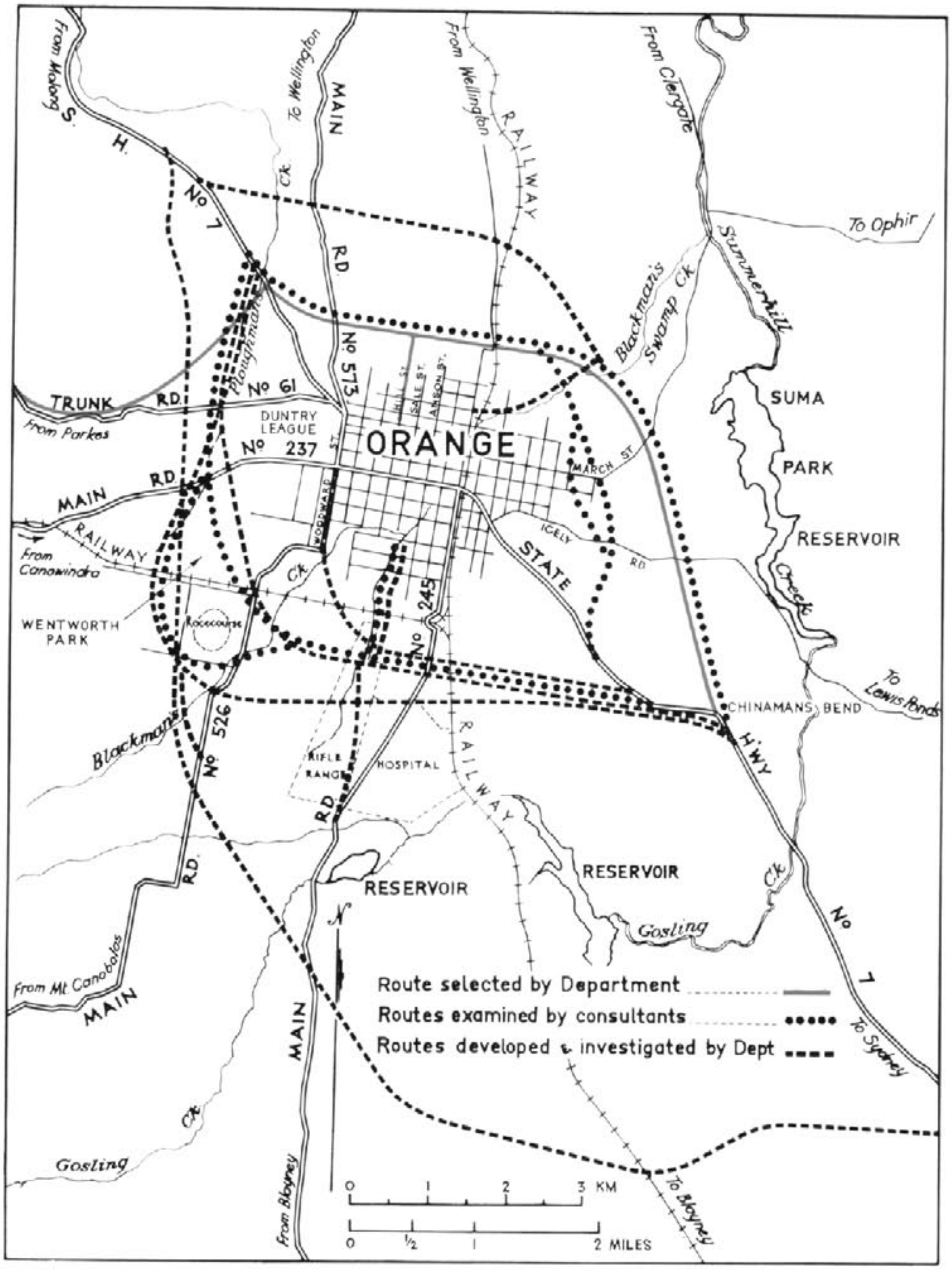
Points in favour of the northern and eastern route were:

1. A shorter length of new construction.
2. Shorter length of journey along the Highway.
3. Less bridging costs.
4. More economical rock excavation.
5. Only one railway crossing (two on the southwest route).
6. Less property acquisition costs.
7. Better road alignment.
8. No difficulties with avoiding Wentworth Park and its golf course.
9. Access well served to the Showground.

Economically, this clearly showed the northern and eastern route in favour. It also was apparent that the north-eastern route would not affect the Orange Town Planning Scheme and would provide the better alignment and grading.

Further Investigations by the Department

To confirm its choice of route, the Department made arrangements for



controlled aerial photography, from which a contour plan to a scale of 50 feet to the inch with contour intervals of two feet, was prepared.

With this as a basis, preliminary designs of the various routes were carried out, and preliminary estimates of cost prepared.

From an engineering and economic viewpoint, these investigations clearly favoured the northeast route selected by the Department.

Type of Road

Further developments of plans for the proposed route involved:

(i) location of accesses under or over it;

(ii) provision of a new major access route extending northerly, grade separated where it crosses the future highway;

(iii) the decision to proclaim the road a motorway.

This last factor would proclude development which might interfere with the free and safe movement of motor traffic. It would also make it impossible for commercial premises to establish themselves on the new route, to the detriment of businesses already established on the existing route.

In appearance, the traffic relief route would be a parklike development to enhance the general appearance of any adjacent urban development●

Works in Progress
AROUND THE STATE

Oxley Highway

(*Top*): *Concrete box culvert on the Oxley Highway, 32 miles east of Walcha*

(*Bottom left*): *Fencing, drainage and earthworks on realigned section of the highway, 34 miles east of Walcha*

(*Bottom right*): *Reconstructed section of the Oxley Highway, 17 miles east of Walcha*

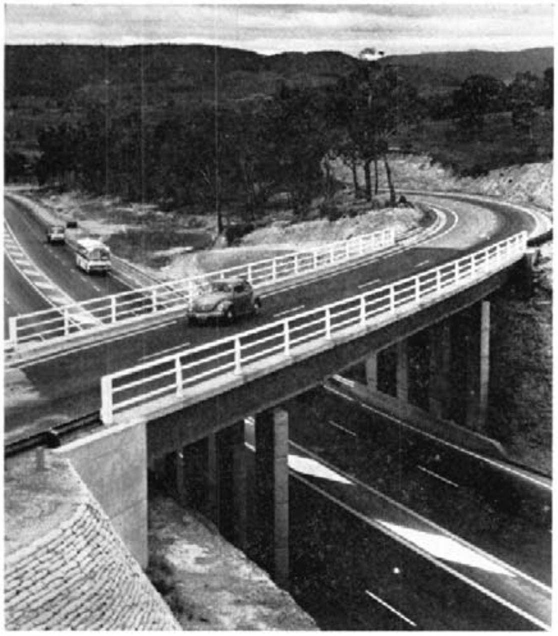
Great Western Highway

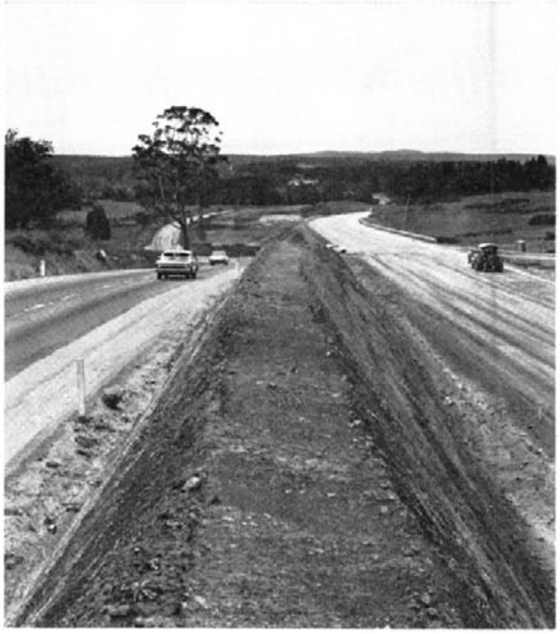
(Top left): A grade separated interchange on the Great Western Highway allows traffic using Trunk Road No. 55 to Mudgee and north to Mullaley to cross the highway without interference to highway traffic

(Top Centre): Reconstruction on the Great Western Highway near Yetholme.

(Top right): Improved alignment and grading between Lithgow and Bathurst near Yetholme

(Bottom): A new bridge on the Great Western Highway over the railway line at Marrangaroo about 5 miles west of Lithgow.





Mitchell Highway

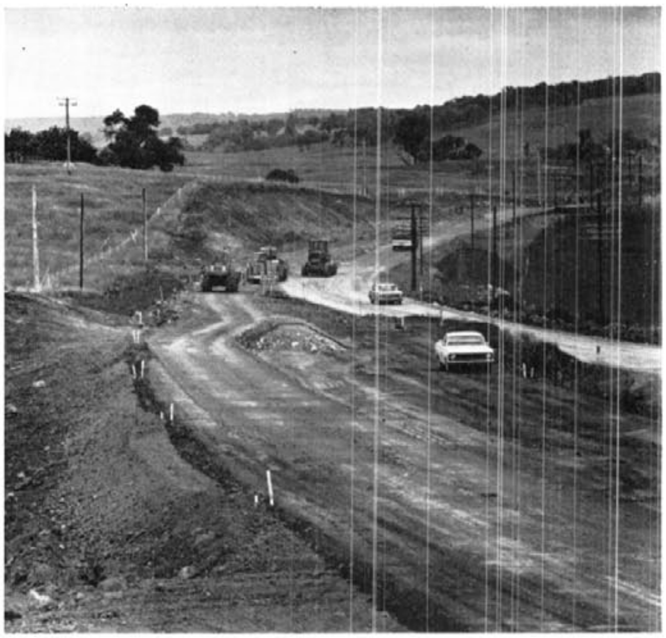
(Top): A concrete bridge being constructed at Three Rivers about 13 miles north of Molong on the Mitchell Highway

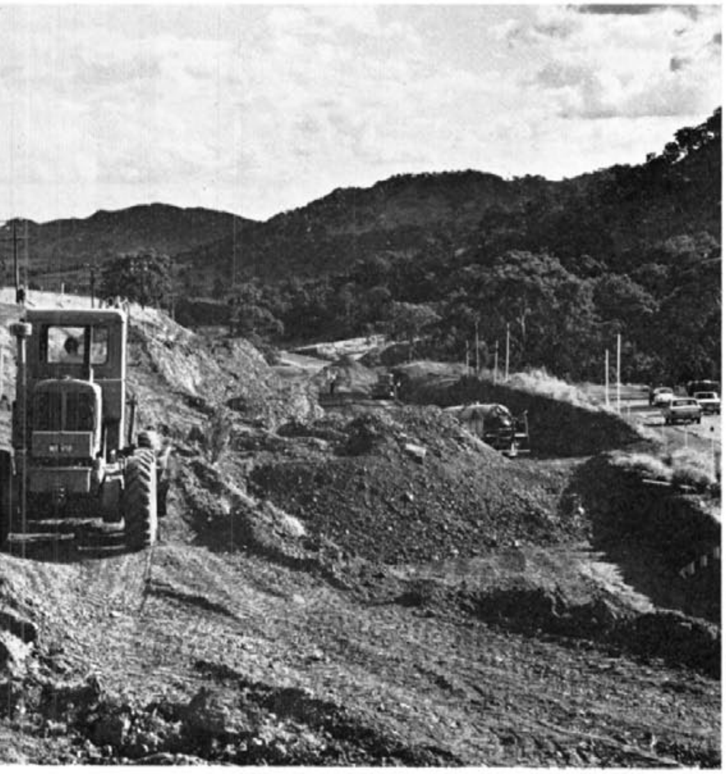
(Bottom): The Mitchell Highway, about five miles north of Molong, being reconstructed to provide a wider pavement

New England Highway

(Top): Reconstruction of the New England Highway through Sugarloaf Mountain south of Tamworth

(Bottom): The reconstruction of this highway through Sugarloaf Mountain will provide a wider pavement and greater sight distance by removal of the crest





TENDERS ACCEPTED BY COUNCILS

The following tenders (in excess of $10,000) for road and bridge works were accepted by Councils for the three months ended 31st March, 1973.

Council	Road No.	Work or Service	Name of Successful Tenderer	Amount
				$
Bellingen	T.R. 76	Bitumen resealing between 3 and 7 miles west of Dorrigo.	Emoleum Ltd	11,107.20
Bellingen	T.R. 76	Construction of 3-cell 12 ft x 12 ft reinforced concrete box culvert over Roses Creek 2·3 miles west of Bellingen.	M. O. & P. J. Kautto	26,312.22
Boree	M.R. 377	Reconstruction and bitumen surfacing from 38 to 45 miles west of Orange.	Allen Bros (Asphalting Contractors)	15,446.90
Cessnock	M.R. 588	Supply and laying of 1,420 tons of tar hot-mix for asphaltic concrete surfacing between 0·5 and 2·2 miles east of Buchanan.	Bituminous Pavements Pty Ltd	24,026.00
Cockburn	D.W. 1032	Construction and realignment from 19·25 to 20·90 miles and 22·4 to 22·98 miles south of Limbri.	Hawkings Earthmoving Pty Ltd	39,650.00
Goobang	T.R. 61	Construction of a 14-span prestressed and reinforced concrete bridge 280 feet long over Gunningbland Creek at Carlachy.	Siebels Concrete Const. Pty Ltd	60,427.90
Guyra	T.R. 73	Construction of 3-cell 12 ft x 12 ft reinforced concrete box culvert over Tienga Creek 5·4 miles north of Bundarra.	D. McNeil	18,915.00
Jemalong	Various	Bitumen sealing and resealing on various roads	Allen Bros (Asphalting Contractors)	18,464.27
Lachlan	T.R. 57	Construction of a 6-span prestressed and reinforced concrete bridge 210 feet long over Bullock Creek 0·5 miles west of Tullamore.	Siebels Concrete Const. Pty Ltd	63,831.00
Talbragar	M.R. 572	Construction of a 5-span prestressed and reinforced concrete bridge 100 feet long over Drillwarrina Creek at Eumungerie.	Herbert Const. Pty Ltd	29,806.10
Uralla	T.R. 73	Construction of a 3-cell 14 ft x 11 ft reinforced concrete box culvert over Two Mile Creek, 31·5 miles north of Uralla.	Enpro Constructions Pty Ltd	20,653.40
Walcha	D.R. 1317	Construction of 3-span prestressed and reinforced concrete bridge 120 feet long over Smith's Creek 24 miles south of Walcha.	J. Parkinson	46,960.00
Weddin	Various	Flush seal and re-seal on various roads	Allen Bros (Asphalting Contractors)	68,738.57
Wellington	D.R. 3214	Construction of a 6-span, low-level prestressed concrete bridge 122·5 feet long over the Talbragar River at O'Leary's Crossing, 3·25 miles northeast of Elong.	A. Cippala & Co. Pty Ltd D. James Constructions	32,254.95
Woodburn	M.R. 145	Construction of a 4-span prestressed and reinforced concrete bridge 200 feet long over Rocky Mouth Creek at Woodburn.	Kennedy Bros	105,755.00
Yallaroi	T.R. 63	Construction of 6-cell 11 ft x 11 ft reinforced concrete box culvert over Flaggy Gully 29·4 miles north of Warialda.	L. J. Rixon	23,797.30

TENDERS ACCEPTED BY THE DEPARTMENT OF MAIN ROADS

The following tenders (in excess of $10,000) for road and bridge works were accepted by the Department for the three months ended 31st March, 1973.

Road No.	Work or Service	Name of Successful Tenderer	Amount
			$
North Western Freeway	City of Sydney. Foundation test bores for Darling Harbour viaduct between Sussex Street, City and Bulwarra Road, Pyrmont.	A.R.D.E.C.	13,000.00 (approx.)
Sydney-Newcastle Tollway	Shire of Gosford. Supply and delivery of expanded polystyrene foam for modification to deck of the Mooney Mooney overpass.	Raymond Industries Pty Ltd	14,760.00

Western Freeway	Municipality of Blacktown. Construction of twin, 4-span prestressed and reinforced concrete bridges, 142 feet long over Eastern Creek.	Lewil Constructions Pty Ltd	181,519.00
Western Freeway	Municipality of Blacktown. Construction of twin, single-span, prestressed and reinforced concrete bridges, 70 feet long over Horsley Road at Eastern Creek.	Central Constructions Pty Ltd	155,020.00
Southern Freeway	City of Wollongong. Construction of concrete, integral kerb and gutter type "A" in the various cuttings on the Freeway between Waterfall and Darkes Forest.	South Coast Paving and Terrazzo Pty Ltd	38,560.50 (approx.)
State Highway No. 2	Hume Highway. Municipality of Camden. Construction of a single-span prestressed and reinforced concrete bridge, 105 feet long over Richardson Road, 2 miles north of Camden.	Simon-Carves (Australia) Pty Ltd	293,302.00
State Highway No. 2	Hume Highway. Shire of Wingecarribee. Construction of twin 3-span, prestressed and reinforced concrete bridges, 225 feet long over Paddy's River 26·5 miles south of Mittagong. The contract price includes the demolition and removal of existing bridge.	Central Constructions Pty Ltd	192,798.00
State Highway No. 5	Great Western Highway. Shire of Turon. Construction of two reinforced concrete box culverts; a 4-cell, 9 feet x 9 feet structure, 103 feet long, and a 3-cell, 7 feet x 6 feet structure, 42·5 feet long, over Diamond Swamp Creek at 21·3 and 21·5 miles west of Lithgow respectively.	Enpro Constructions Pty Ltd	55,918.50
State Highway No. 6	Mid Western Highway. Shire of Lyndhurst. Construction of 3-span reinforced concrete slab bridge, 150 feet long over Eulamore Street, Carcoar.	Enpro Constructions Pty Ltd	96,225.00
State Highway No. 6	Mid Western Highway. Shire of Abercrombie. Construction of 5-span prestressed and reinforced concrete bridge 150 feet long over Evans Plains Creek, 6 miles west of Bathurst.	Herbert Constructions Pty Ltd	55,400.00
State Highway No. 7	Mitchell Highway. Shire of Molong. Construction of 3-span composite steel girder and reinforced concrete bridge, 135 feet long over Molong Creek, 6·4 miles north of Molong.	Enpro Constructions Pty Ltd	64,187.40
State Highway No. 9	New England Highway. City of Maitland. Supply and delivery of up to 43,000 cubic yards of fill material to approaches to bridge over Four Mile Creek.	G. Hawkins and Sons Pty Ltd	14,600.00
State Highway No. 9	New England Highway. Shire of Patrick Plains. Supply and delivery of up to 14,000 cubic yards of base course and surface course gravel to reconstruction of low speed lane from 5 to 6 miles west of Singleton.	W. A. Shearer Pty Ltd	17,500.00
State Highway No. 10	Pacific Highway. Shire of Nambucca. Construction of 5-span prestressed and reinforced concrete bridge, 387 feet long over Newee Creek north of Macksville.	Pearson Bridge (N.S.W.) Pty Ltd	220,952.00
State Highway No. 10	Pacific Highway. Shire of Nambucca. Construction between 0 and 2 miles north of Macksville for deviation of Highway near new bridge over Newee Creek.	Fortescue Motors Pty Ltd	17,250.00
State Highway No. 10	Pacific Highway. Shire of Hastings. Construction of deviation from Smith's Creek to Barry Creek between 10 and 13 miles south of Kempsey.	Newcastle Lime and Cement Co. Ltd	17,250.00
State Highway No. 10	Pacific Highway. Shire of Nambucca. Construction of 2-span prestressed and reinforced concrete bridge, 70 feet long over Teague Creek 2 miles south of Nambucca.	M and E Civil Constructions (Tamworth) Pty Ltd	42,390.00
State Highway No. 19	Monaro Highway. Municipality of Cooma. Manufacture, supply and delivery of sixty, 44 feet long precast, pretensioned concrete bridge planks for bridge over Cooma Creek, Cooma.	Peter Verheul Pty Ltd	27,720.00
State Highway No. 22	Silver City Highway. Shire of Wentworth. Construction of 3 prestressed and reinforced concrete bridges; a 5-span, 100 feet long and a 21-span, 400 feet long bridge, at Four Mile Break, the other a 9-span, 180 feet long structure over Tincha Creek. The bridges are located at 4·1, 5·0 and 5·4 miles north of Wentworth respectively.	Allan Tessier Pty Ltd	230,436.10
Main Road No. 259	Municipality of Camden. Construction of 6-span prestressed and reinforced concrete bridge 210 feet long over the flood plain of the Nepean River near Camden.	Citra Constructions Pty Ltd	103,856.00