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PAPER 9638 ROAD PANEL

Elevated section of the West Gate Freeway, South Melbourne, Australia. Part 2: design and construction of substructure and superstructures

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The Paper describes the principal features of the design and construction of the substructure and match cast segmental superstructures of the elevated section of the West Gate Freeway. The freeway structures comprise two independent structures—the north or city-bound carriageway and the south or westbound carriageway; they are 1·85 km long and cross numerous roads, railway lines, tramways, railway yards and container handling yards close to the Central Business District of Melbourne. The design and construction of the foundations of the freeway structures are described in Paper 1. The complex geometry of the superstructures called for high precision geometric control both in the precasting factory and on site. Design innovations included the use of high strength cast steel corbels stressed to precast concrete in special expansion joint segments, the largest elastomeric bearings ever used in Australia and the most sophisticated launching girders yet used in precast segmental construction. A feature of the project was the quality of the precasting: 2070 segments were cast, none rejected and no geometric corrections were necessary during segment erection. The city-bound (north) carriageway was opened to traffic on 18 December 1987, and the south carriageway was opened on 29 September 1988.

Introduction

The selection of the method of construction and type of superstructure has been discussed by Jellie[1,2] and Valentine.[3] Piers and deck structures were to be as unobtrusive visually as possible, and to be constructed with minimal disruption to traffic. Community evaluation and opinion towards the new freeway were actively canvassed from the earliest planning stages. The project received full support from local councils and no protests were lodged, although it should be noted that very little residential property was affected by the project.

Selecting type of construction for superstructures

2. Prestressed concrete structures were favoured over steel, mainly because of the high initial cost of steel fabrication and the continuing maintenance that would be necessary in an environment of appreciable corrosion hazard. In addition, there

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Written discussion closes 15 October 1991; for further details see p. ii.

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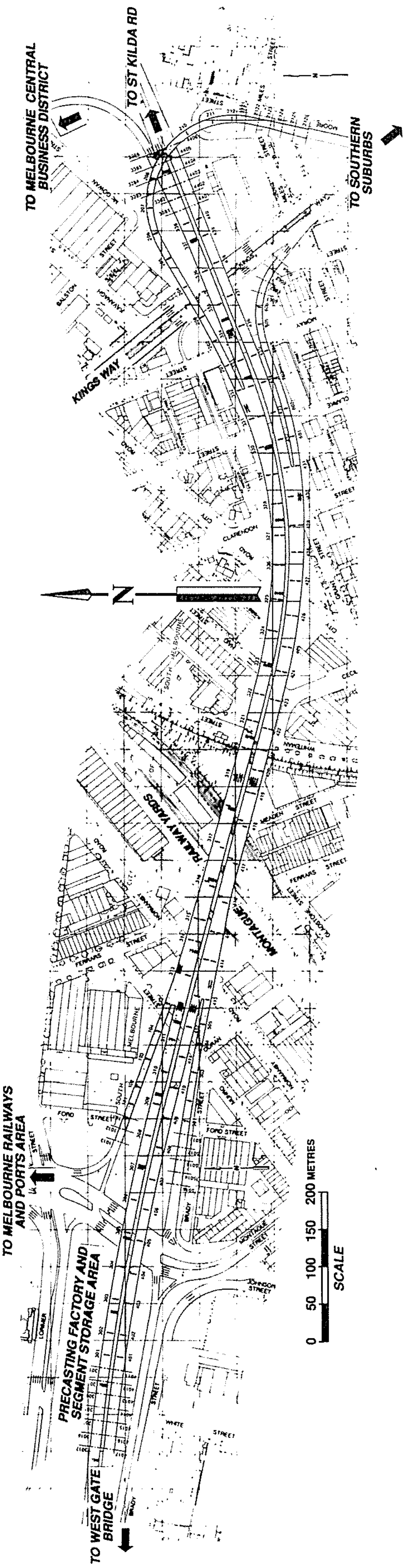


Fig. 1 Layout plan

was a poor history of steel delivery for previous projects, as well as risks associated with the industrial climate that had developed on West Gate Bridge following its collapse during construction and its subsequent reconstruction.

3. In the preliminary design phase, three alternative prestressed concrete structures were studied[2]

(*a*) cast-in-place box girder
(*b*) precast U-beams with composite deck
(*c*) match cast segments erected by balanced cantilever.

4. The match cast segmental box girders were finally adopted for the following reasons.

(*a*) In an area where the ground conditions are so poor, no reliance was necessary on support from the ground. Span layouts could be achieved with balanced cantilevers (or one segment out of balance) at almost every pier over the full length of the project. The only exceptions were at the abutments, where three or four segments were erected initially, and at two other locations where temporary supports were required and had to be piled to achieve the carrying capacity required.
(*b*) Because no falsework was necessary, the bridges could be built to minimum vertical clearances over roads and railways, thus reducing grades and ensuring the shortest possible overall length of structures.
(*c*) The rapid construction rates achieved by the use of precast segments would minimize the disruption to traffic and public transport, especially over major arterial roads. No work was to be carried out over operating traffic, and during temporary operations—such as advancement of the launching girders—traffic was to be stopped for a few minutes until the way was clear. Work over major roads and public transport routes was to be scheduled to avoid peak periods.
(*d*) The repetitive, assembly-line production of segments created efficiencies in labour.
(*e*) Because segment manufacture was carried out in a factory building, the effects of wet weather delays were greatly reduced.
(*f*) The precasting of segments proceeded in parallel with other construction activities, thus minimizing the overall construction period. However, this benefit was significantly reduced as a result of a demarcation dispute between two unions which delayed the construction of the precasting factory, and hence the precasting of segments, by about 12 months.
(*g*) The effects of long-term deformation due to shrinkage and creep were reduced because of the maturity of the concrete at the time of placing segments in the structure, and at the time of application of prestress.

General description of bridges

5. One of the main problems encountered in the layout of the bridges was the control on the location of piers. This arose because of the irregular street pattern, the railway lines and the presence of major underground services in the area, as can be seen in the layout plan in Fig. 1. Despite these limitations, the concept of relative uniformity of spans, and therefore of a constant depth of superstructure, was pursued with the aim of achieving a simple, pleasing appearance and economy

of construction. Many underground services were relocated to assist in the balancing of span lengths.

6. The maximum span length is 55·1 m, but most spans are in the vicinity of 40 m. The bridges are on curvilinear alignments with variable crossfall. Entry and exit ramps are connected to the elevated freeway as part of the structures. Each carriageway incorporates 3 m × 3·5 m wide traffic lanes, and a fourth lane is provided between the bifurcations for the merging and weaving manoeuvres of traffic. The entry and exit ramps carry two traffic lanes. A breakdown lane of 2·4 m is provided on the left-hand shoulder, except on the ramps where 1·2 m is provided. The right-hand shoulder is 1·2 m wide in all locations (Fig. 2).

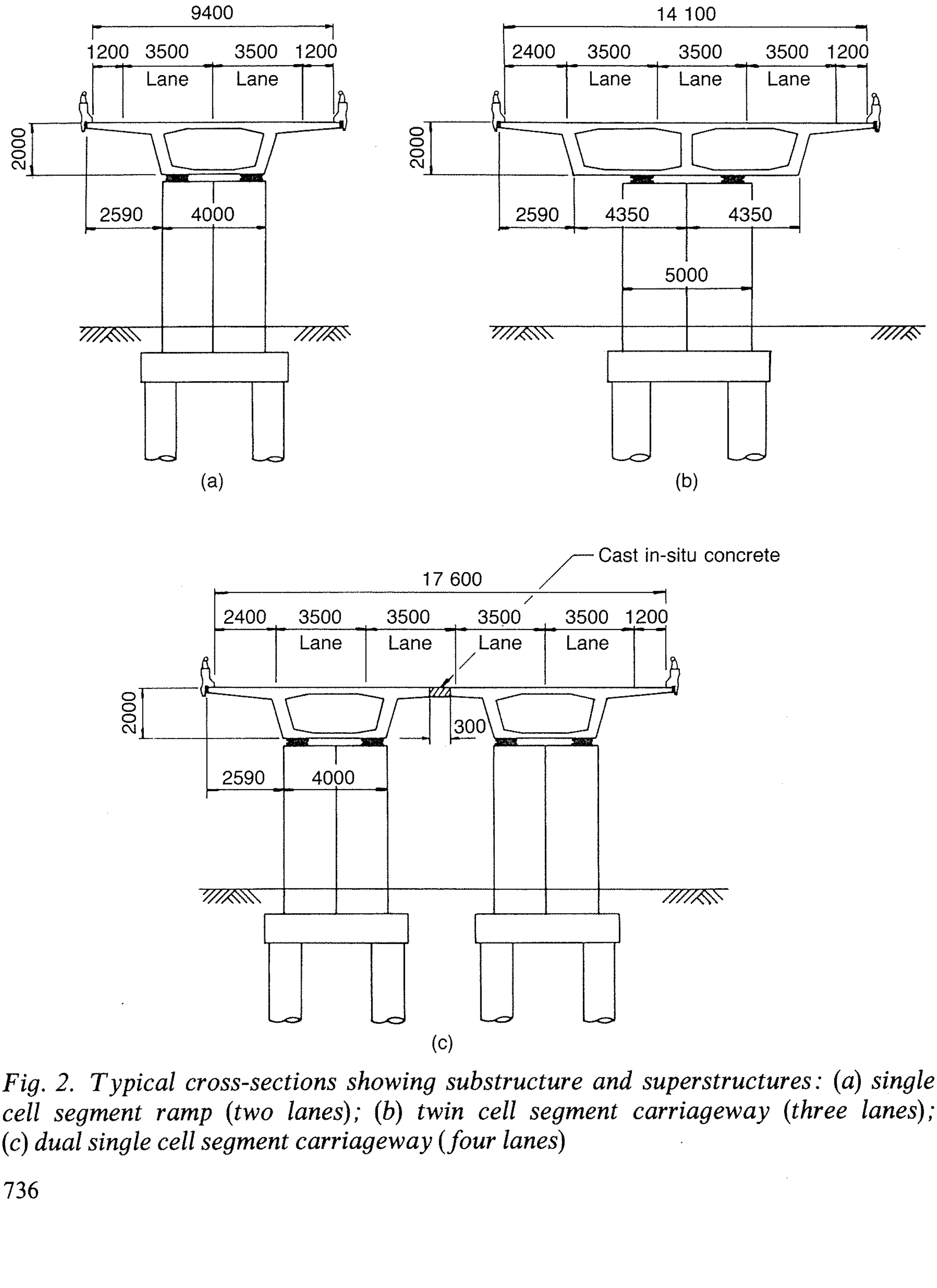


Fig. 2. Typical cross-sections showing substructure and superstructures: (a) single cell segment ramp (two lanes); (b) twin cell segment carriageway (three lanes); (c) dual single cell segment carriageway (four lanes)

7. The box girder superstructure has a constant depth of 2·0 m and there are two basic segment types, namely single cell and twin cell. By using these two segment types, it was possible to construct all the variations in deck width (Fig. 2). The entry and exit ramps were constructed using single cell segments to accommodate two lanes of traffic. Single cell segments were also used to form the four-lane superstructures between the bifurcations. The two spines of single cell segments are joined by in-situ concrete to form these four-lane carriageways. Variations in deck width are accommodated by varying the width of the infill section.

8. The superstructure between the bifurcations and each end of the elevated structures carries the three traffic lanes on twin cell segments. Short lengths of 17·5 m span beam and slab approach structures provide a transition between the box girders and the road embankments.

9. Segment deck slabs are all planar or slightly warped. Crossfalls were achieved by rotation of the box girders or, in some areas, by varying the depth of asphaltic roadway surfacing to provide crossfall from a central crown. Crossfall variation within spans was achieved by warping individual segments in the precasting process.

10. The vertical alignment was governed by the need to provide minimum clearance of 5·6 m over major roads and 8·0 m over the Montague Railway Yards to allow for handling and stacking of containers underneath the bridges.

11. All services carried on the bridges are accommodated inside the box girders. Connections to ground are concealed within the piers or within the approach structures at each end of the bridges. The services include deck drainage, water mains, electrical conduits for freeway and understructure lighting, emergency telephones and traffic detection wiring. Fire hydrants are incorporated within the bridge parapets at intervals of around 180 m (Fig. 3).

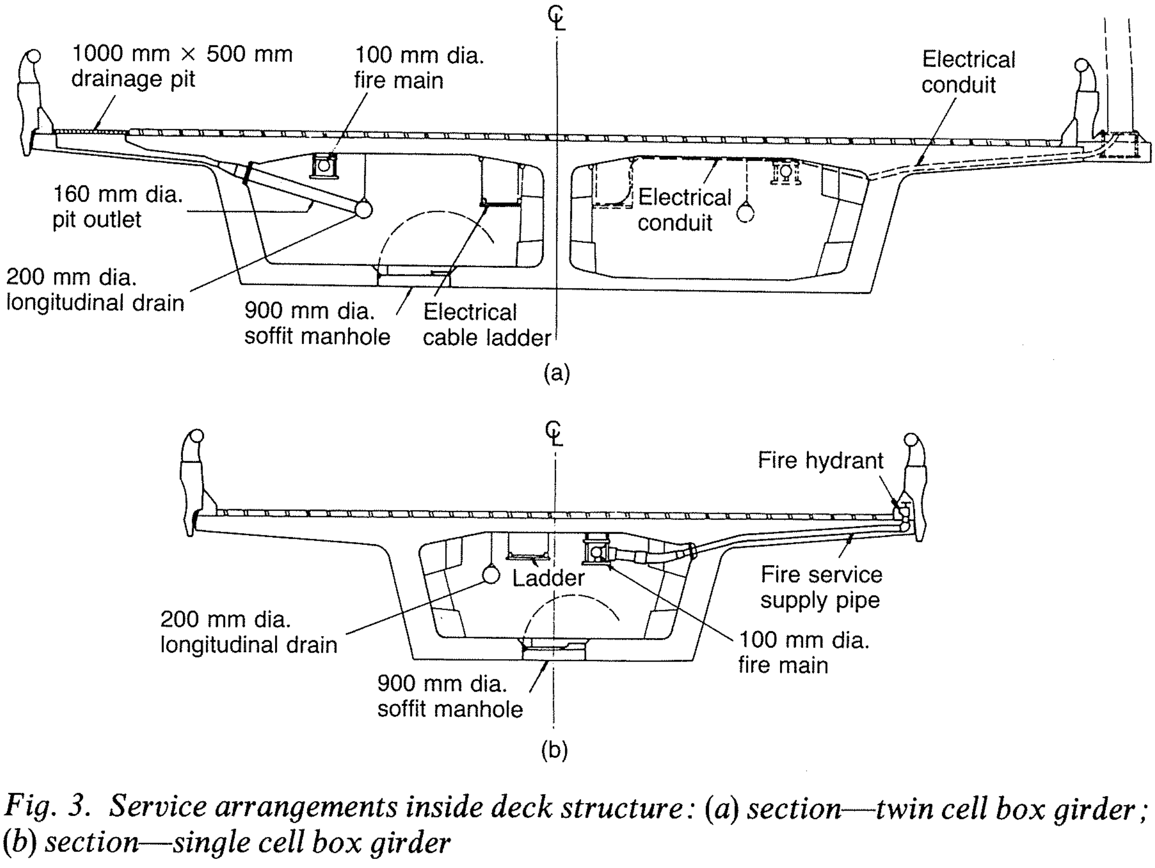


Fig. 3. Service arrangements inside deck structure: (a) section—twin cell box girder; (b) section—single cell box girder

12. Construction began with the north carriageway. Government approval to proceed with the south carriageway was not given until the north carriageway was almost completed. Fig. 4 shows an aerial view of the project, with the northern carriageway nearing completion. The precasting factory and segment storage area can be seen at the west end of the structure, and the West Gate Bridge is visible in

Fig. 4. Overall view of project looking west, early in 1987, with the north carriageway nearing completion

the background. The Central Business District of Melbourne is just out of sight to the right, north of the Yarra River.

13. In a radical departure from usual practice, all the major items of construction equipment—casting cells, pier brackets and launching girders—were designed and documented as part of the design for tender. This decision followed directly from the Quality Assurance philosophy that guided preparation of the project, the construction equipment being intimately related to many aspects of superstructure design loadings and segment detailing.

Construction and testing of prototype segments

14. The selection of precast segmental construction required a very detailed understanding of the physical properties of the concrete used in the precast segments. Concrete deformation characteristics have a critical influence on the behaviour of segments during erection. In the long term, creep and shrinkage are major factors in bridge performance.

15. Laboratory tests for modulus of elasticity, creep and shrinkage are usually performed on standard test specimens under controlled conditions of environment and loading. These values are then used to predict performance of the structure. However, the relationships used in design codes are imprecise, and it was considered desirable for a project of this magnitude, which required 2070 segments, to examine practical design aspects and concrete properties in two full-size single cell prototype segments.

16. The specific aims of the prototype testing were

(*a*) to determine creep, shrinkage and shortening responses under prestress in a full-size pair of segments
(*b*) to confirm practical working clearances for post-tensioning operations
(*c*) to observe casting operations with a view to improvements to concrete mix design, reinforcement details, formwork details and concrete dimensional details
(*d*) to test the adequacy of proposed rubber gaskets to seal the post-tensioning tendon ducts at the joints between segments
(*e*) to determine practical controls on the design, mixing and application of epoxy adhesives on segment joints
(*f*) to study resultant stress distributions across the section for comparison with computed values
(*g*) to measure strain distributions in the vicinity of post-tensioning anchorage zones.

17. The prototype segments confirmed the practical aspects of clearances for prestressing jacks and the efficacy of the sealing of tendon ducts. Potential difficulties were identified with reinforcement congestion, the location of tendon ducts and anchorages, and the casting of concrete. Modifications in design were made to web and blister reinforcement details, reinforcement bending tolerances were reviewed and a curved reentrant corner was introduced into the bottom of the web, and concrete mix designs were finalized.

18. All instrumentation was connected to a logger which scanned and recorded 100 channels of data. From these records and the response of unstressed slabs of the same concrete over the same period, creep and shrinkage effects were determined. Fig. 5 shows the strain–time records of three embedded strain gauges.

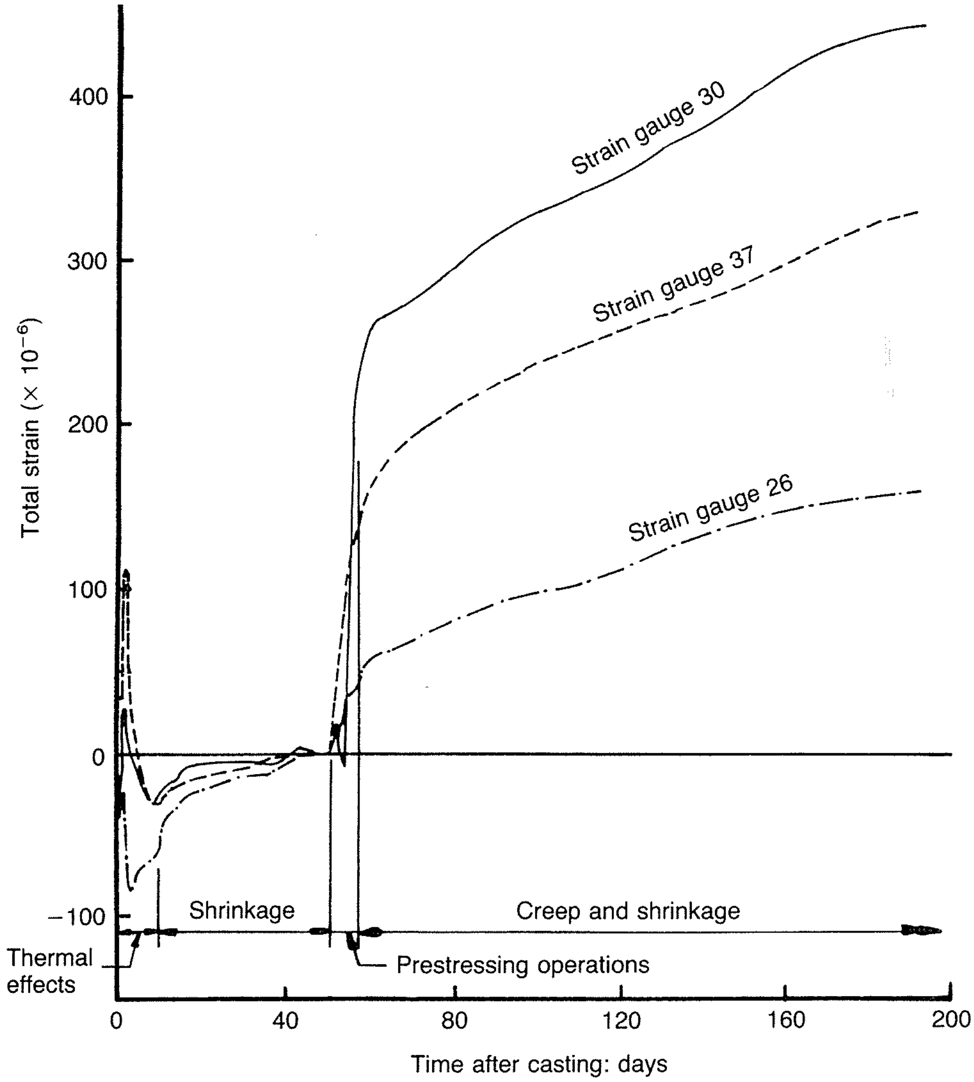


Fig. 5. Strain–time records from three strain gauges in prototype segments

The measured strain distribution across the deck section proved to be very similar to the stress distribution calculated using finite element analysis.

19. The prototype segment testing was an important step in the overall Quality Assurance programme, as it led directly to the successful detailing and precasting of the superstructure segments.

Design

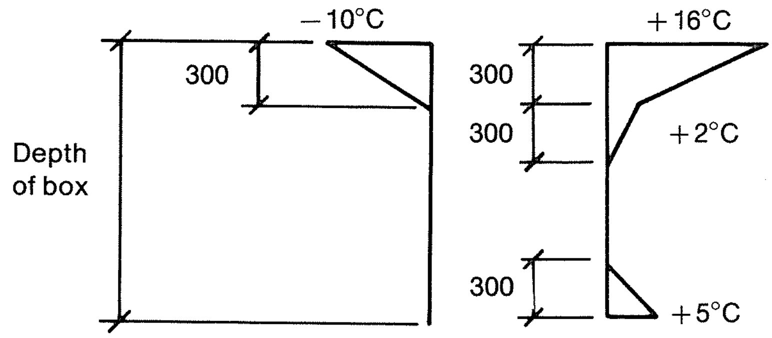
Design standards and loadings

20. The principal design standard for the elevated structures was the 1976 NAASRA *Bridge design specification.*[4] In addition to NAASRA, the Australian Standard AS 1170, *SAA loading code, part 2: wind forces,*[5] was used for wind load evaluations. The main design loads and related parameters are summarized in Table 1.

21. The design load combinations were not quite the same as the NAASRA combinations, but identical notation was used. Table 2 gives the principal combinations which differed from NAASRA.

Table 1. Design loads and related parameters

Standard vehicle designation and total weight	T44–44 t
Abnormal vehicle designation and loading description	SAV = standard abnormal vehicle Load per wheel: 60 kN Load per axle: 240 kN Total vehicle weight: 196 t
Temperature effects linear gradients	 From −5°C to +50°C Cold top Hot top



Deck surfacing thicknesses	40 mm min.–100 mm max.
Differential settlements	Between piers along structure after span continuity = 15 mm Between adjacent piles beneath common pile cap prior to span continuity = 5 mm Between adjacent piles beneath common pile cap after span continuity (short term) = 3 mm As above, but creep and relaxation effects acting to reduce bending moments and torsional moments (long term) = 6 mm

Table 2. Design load combinations

Group	Loading combination*	% of allowable stress
I	$D + PS + L + I + 0{\cdot}7\ T$	100
IV	Group I but with T	125
X	$D + PS + AL + 0{\cdot}7\ T$	140

* *D* dead load; *PS* prestress; *L* live load; *I* impact; *T* temperature load; *AL* abnormal load.

22. The most important allowable stress criteria for the segmental concrete box girder design were the minimum residual compressive stresses required at joints between segments. These stress levels (see Table 3) were set specifically for this project.

Table 3. Minimum residual compressive stresses at segment joints

Group	Top fibre: N/mm^2	300 mm below top fibre: N/mm^2	Bottom fibre and 400 mm above: N/mm^2
I	1·5	2·0	1·5
II to IX	1·0	1·0	1·0
X	1·0	1·0	1·0

Superstructure and substructure materials

23. The principal material grades and properties are given in Table 4. Quality Assurance requirements were detailed in the specification to ensure that material quality was maintained at a uniformly high level.

Loads during construction

24. Loads applied during construction, notably from the 200 t launching girders, were of particular importance in the design of the superstructure. For example, loads applied at the stabilizing arm attachment points required special reinforcing in the segments to diffuse the effect of the concentrated loads of ± 65 t in the deck slab haunches close to the webs. Another unusual effect was from rotation in plan of the launching girders, that created torsional moments in a horizontal plane. All these effects had to be evaluated and checked against the structural capacity of the superstructures and substructures.

Detailing

25. Refinements in segment detailing and construction procedures were introduced to eliminate at the source the known problems associated with precast segmental construction. Particular problem areas were: geometry control; post-tensioning losses; tendon duct sealing at segment joints; standardization of expansion joint segments.

Table 4. Principal material grades and properties

Material	Properties
Concrete (using high density basaltic aggregate)	Density = 2720 kg/m (including reinforcement)
	Strength (cylinder crushing strength at 28 days on 150 mm dia. cylinders)
	Superstructure $F'_c = 45$ N/mm^2 & 50 N/mm^2
	Substructure $F'_c = 30$ N/mm^2
Reinforcing steel	Strength: grade 410C $F_y = 410$ N/mm^2
Prestressing steel	12·7 mm low relaxation strand
	Minimum breaking load = 184 kN
	Nominal area = 100 mm^2
	Relaxation at 1000 hours at 80% UTS = 3·5%
	Modulus of elasticity = 195 000 N/mm^2
	38 mm prestressing bars
	Minimum breaking load: 1230 kN
	Relaxation at 1000 hours at 70% UTS = 4%

26. In addition to the usual bridge design and analysis software, a number of programs were developed specifically for this project.

(*a*) A post-processing program was developed which transformed the output from a number of design and analysis programs into formats required for input to other programs or into a convenient form for review. It also provided stress groupings from applied loads and prestress, and produced plots of moment, stress and shear envelopes.

(*b*) A program was developed to set out the reinforcement pattern in the segments. Taking account of specified cover requirements and three-dimensional duct profiles, the program calculated a range of acceptable bar shapes that could be used at each reinforcement layer in a segment.

(*c*) A program was developed which calculated segment stresses during the cantilever erection phase and which was used to check amendments to construction sequences as they occurred during construction.

Span distribution

27. From the layout plan (Fig. 1), it can be seen how the number of roads, railways and tram routes prevented selection of completely uniform spans. Span lengths range from 24·1 m adjacent to the abutments, to 55·1 m over the Montague Railway Yards. The greatest number of spans was close to 40·0 m in length, and only the twin cell segment spans were consistently in excess of 40·0 , ranging from 42·6 m to 50·2 m.

Segment sizes and layout

28. Standardization of segment length was complicated by the variation in span lengths. It was possible, however, to minimize variations by limiting non-standard lengths to one segment per cantilever. Fig. 6 shows a typical twin cell segment span layout with standard segment lengths.

29. The single cell segments were typically between 2·44 m and 3·23 m in length and weighed up to 50 t, whereas the twin cell segments were either 2·56 m or 2·70 m in length and weighed up to 68 t. Segments over piers were 1·6 m in length for the single cell box girders, and 1·4 m in length for the twin cell. The maximum segment weight was 76 t for three of the twin cell expansion joint segments.

30. The distance between expansion joints in the superstructures varies from 270 m to 330 m, and each section is longitudinally restrained at an anchor pier close to the elastic centre of the length concerned.

31. Expansion joints are located within spans at points between $\frac{1}{3}$ and $\frac{1}{4}$ of the

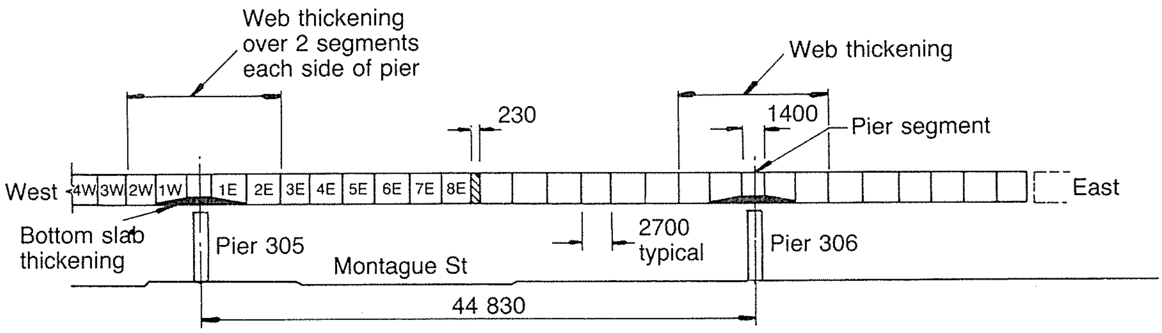


Fig. 6. Elevation on segment layout for typical twin cell segment span

span length. Special expansion joint segments were used to accommodate the bearings and road joints. To guarantee ease of access for inspection and maintenance, all expansion joints are located away from roads, railway lines, tramways or other trafficked areas.

Substructure

Piers and pile caps

32. There are 137 piers in the substructure, 71 for the north carriageway and 66 for the south carriageway, with heights varying from 3·50 m to 10·70 m.

33. The shape and surface finish of the piers were the result of architectural and construction assessments. Fluted surfaces and other finishes that required special form liners were rejected in favour of a dark concrete finish obtained by the addition of Bayer black iron oxide No. 318 at a dosage rate of 25 kg/m^3 of concrete. A plane surface finish was preferred to fluting, as the latter would have created difficulties for the bearing of temporary pier brackets against the pier concrete. On completion of the entire project the piers were lightly sandblasted and painted with an anti-graffiti coating.

34. Transverse slope on the top of a pier corresponds to the deck crossfall at that location. Vertical holding down bar anchors embedded in the piers were used to stabilize the first two or three segments during cantilever construction.

35. Anchor piers—i.e. the piers providing longitudinal restraint for the superstructure—are located approximately midway between expansion joints, and they have slightly increased thicknesses in the longitudinal direction—i.e. 150 mm or 100 mm greater than the standard dimensions. The anchor piers are supported on four piles set out on a 4·0 m or 4·5 m square grid in place of the two piles at typical piers. The frame action of the four piles at the anchor piers provides longitudinal fixity.

Elastomeric bearings

36. Elastomeric bearings were selected in preference to other types of bearing because of their multi-dimensional flexibility and ability to continue functioning even after material failure. Mechanical sliding bearings are used only at abutments and at expansion joints.

37. The largest laminated elastomeric bearings ever used in Australia were designed for this project. There were two bearings per pier, and bearing sizes range from 800 mm × 550 mm × 231 mm to 1050 mm × 750 mm × 322 mm. Elastomer layer and steel shim plate thicknesses are given in Table 5.

Table 5. Elastomeric bearings: elastomer layer and shim plate dimensions

Description	1050 mm bearings: mm	All others: mm
Thickness of internal elastomer layers	18	16
Thickness of external elastomer layers	9	8
Side cover of elastomer to steel shim plates	10	10
Shim plate thickness	5 (Steel grade 350)	5 (Steel grade 250)

Table 6. Elastomeric bearings: physical properties

Property	Value
Durometer hardness	55
Young's modulus	3·9 N/mm^2
Shear modulus	0·77 N/mm^2
Bulk modulus	1000 N/mm^2
Elastomer constant	0·65
Minimum ultimate tensile strain	575%

38. The bearings were designed according to the Australian NAASRA *Bridge design specification*,[4] with the exception of the two following parameters

(*a*) an average compression stress limit of 16·5 N/mm^2 was placed on the elastomer
(*b*) the British Standard, BS 5400, was used for limiting the tension in the elastomer in place of the 'no tension' provision in the NAASRA specification.

39. The use of unusually thick elastomeric bearings was a key design factor in assisting the superstructure to 'absorb' the torsional effects of differential settlement of the foundations.

40. Properties of the elastomer are critical to the performance of elastomeric bearings, and a number of prototype bearings were manufactured, sampled and tested during validation of the design. Table 6 summarizes the principal physical properties of the elastomer.

41. All of the 274 bearings manufactured for the project were checked and proof tested for conformity with the specification. The checking and testing programme included verification of dimensions, hardness of rubber, cover-to-steel shim plates, general appearance, lateral stability under load, and stiffness in compression and shear. Compression load testing was carried out to 1·5 times the maximum design load for each bearing. Design loads in service varied from 4950 kN for the smallest bearings up to 11 400 kN for the largest bearings.

42. Design values for compressive stiffness ranged from 690 kN/mm to 1430 kN/mm. Theoretical shear stiffness varied from 1·33 kN/mm to 2·92 kN/mm. Rejection of bearings occurred in the early stages of the project until consistent bearing quality and test results were attained.

43. A special bearing test machine was designed and manufactured by the Roads Corporation for this project. The test machine could apply a maximum compressive load of 20 000 kN and a shear load of 200 kN.

44. The elastomeric bearings have now (August 1991) been in service for at least 34 months and no deviations from predicted behaviour have been observed.

Superstructure

Description of segment types

45. Figure 7 shows a front elevation of a typical single cell segment. Webs and bottom slabs increase in thickness adjacent to piers, the web dimension of 420 mm (400 mm actual thickness) varying up to a maximum of 600 mm (580 mm actual thickness). Increments are 60 mm over the length of a segment. Bottom slab

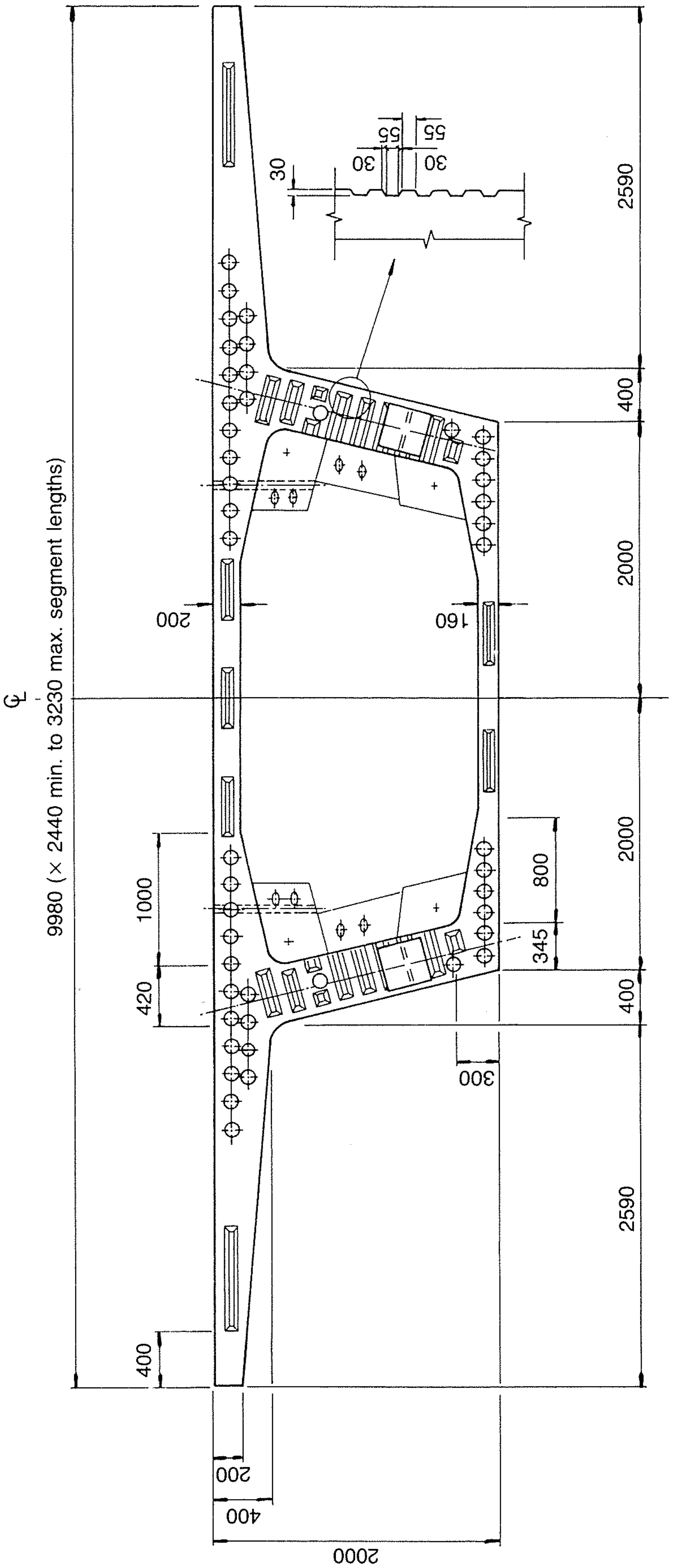


Fig. 7. Typical single cell segment (420 mm web dimensions)

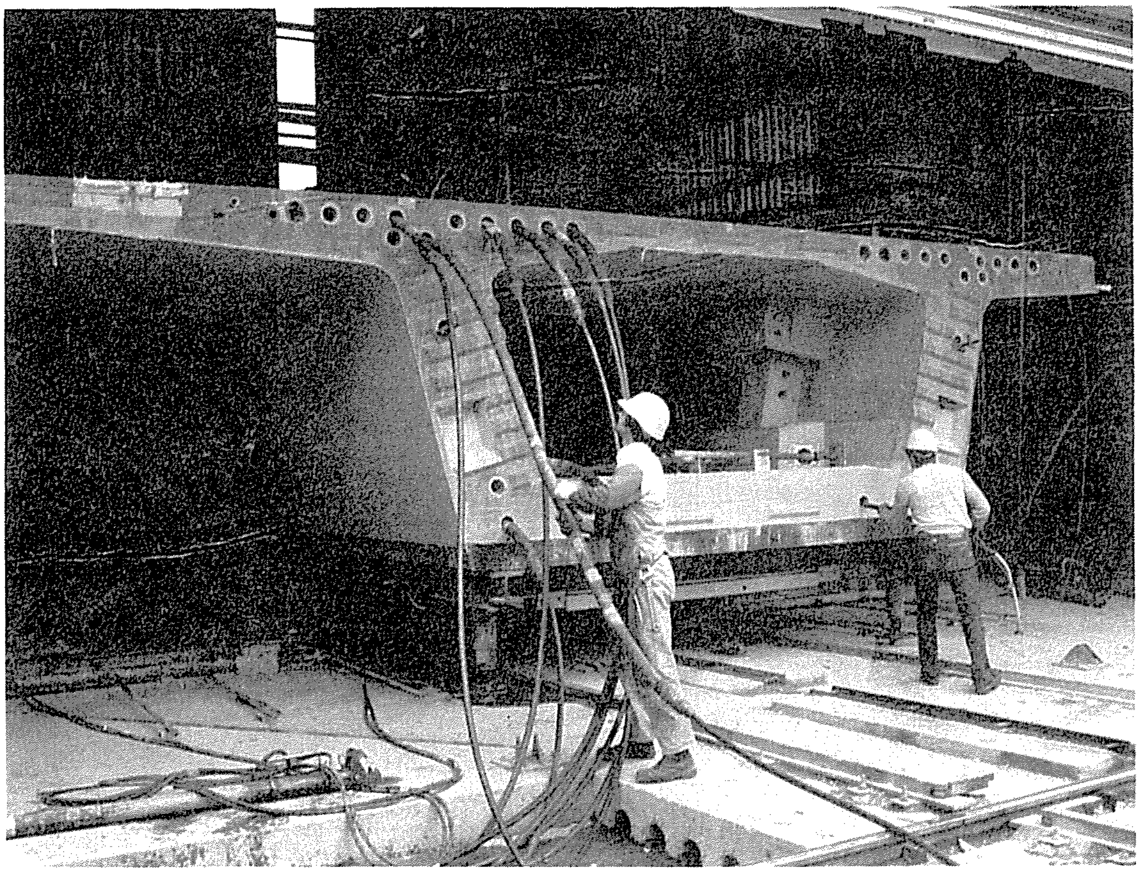


Fig. 8. Single cell segment with 600 mm webs and 400 mm bottom slab in casting factory

thicknesses increase to 400 mm in three stages: 160 mm to 300 mm; 300 mm to 350 mm; then to 400 mm. Fig. 8 shows a single cell segment with 600 mm webs and 400 mm bottom slab in the casting factory.

46. Figure 9 shows a twin cell segment which, except for the central web, is similar to the single cell segment. Adjacent to piers, the web dimensions increase from 420 mm (400 mm) to 540 mm (520 mm) in 60 mm increments, and the bottom slab thickness increases from 160 to 300 mm.

47. Segments over piers, or pier segments as they are usually designated, were of much shorter lengths than the standard segments because of the necessity to respect the lifting weight limits of the construction equipment. Single cell pier segments were 1·60 m and twin cell pier segments 1·40 m in length. Important features of the pier segments at the anchor piers and at some piers in curved deck zones were the restrainer keys between deck and piers. At anchor piers, the keys prevent the superstructure from moving relative to the substructure, whereas in the curved deck zones, the keys act as lateral guides by way of stainless steel and teflon sliding surfaces. Figs 10 and 11 show the single cell and twin cell pier segments.

48. Transverse prestressing was necessary in the pier segments. Single cell pier segments were lightly prestressed by four 4 × 12·7 mm strand tendons tensioned during segment storage to a maximum jacking force of 552 kN per tendon. Twin cell pier segments were heavily prestressed, using eight 27 × 12·7 mm strand tendons tensioned to 3700 kN per tendon. The twin cell pier segments cantilevered laterally beyond the elastomeric bearings and therefore required substantial pre-

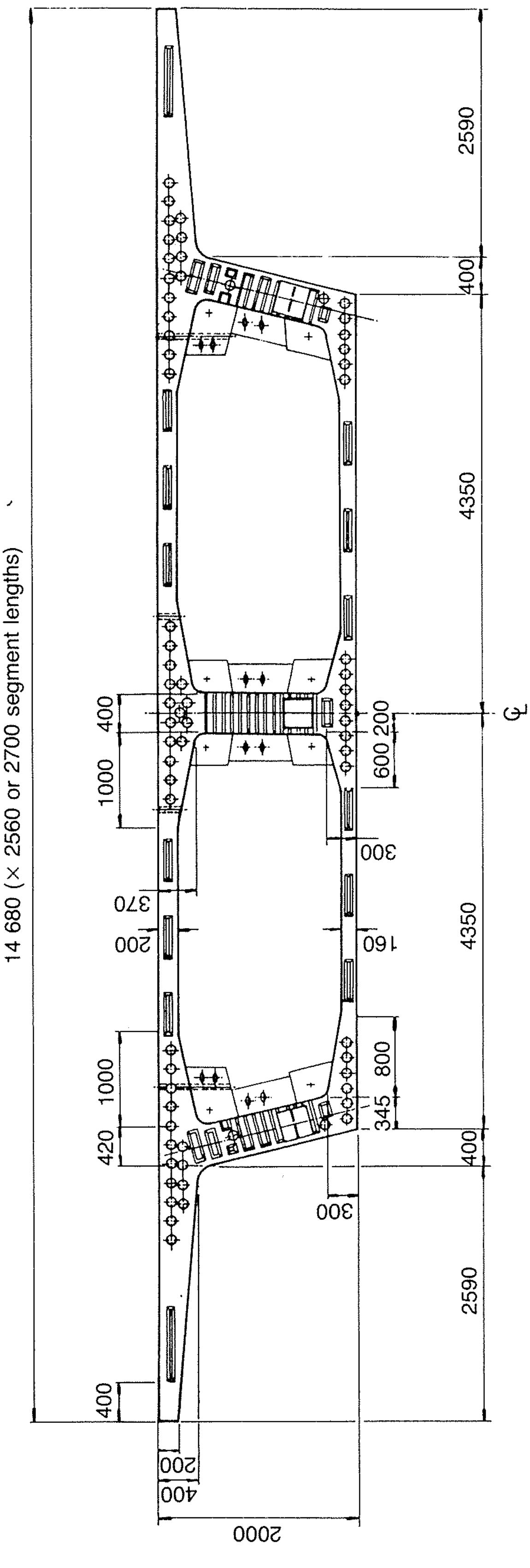


Fig. 9. Typical twin cell segment (420 mm/400 mm web dimensions)

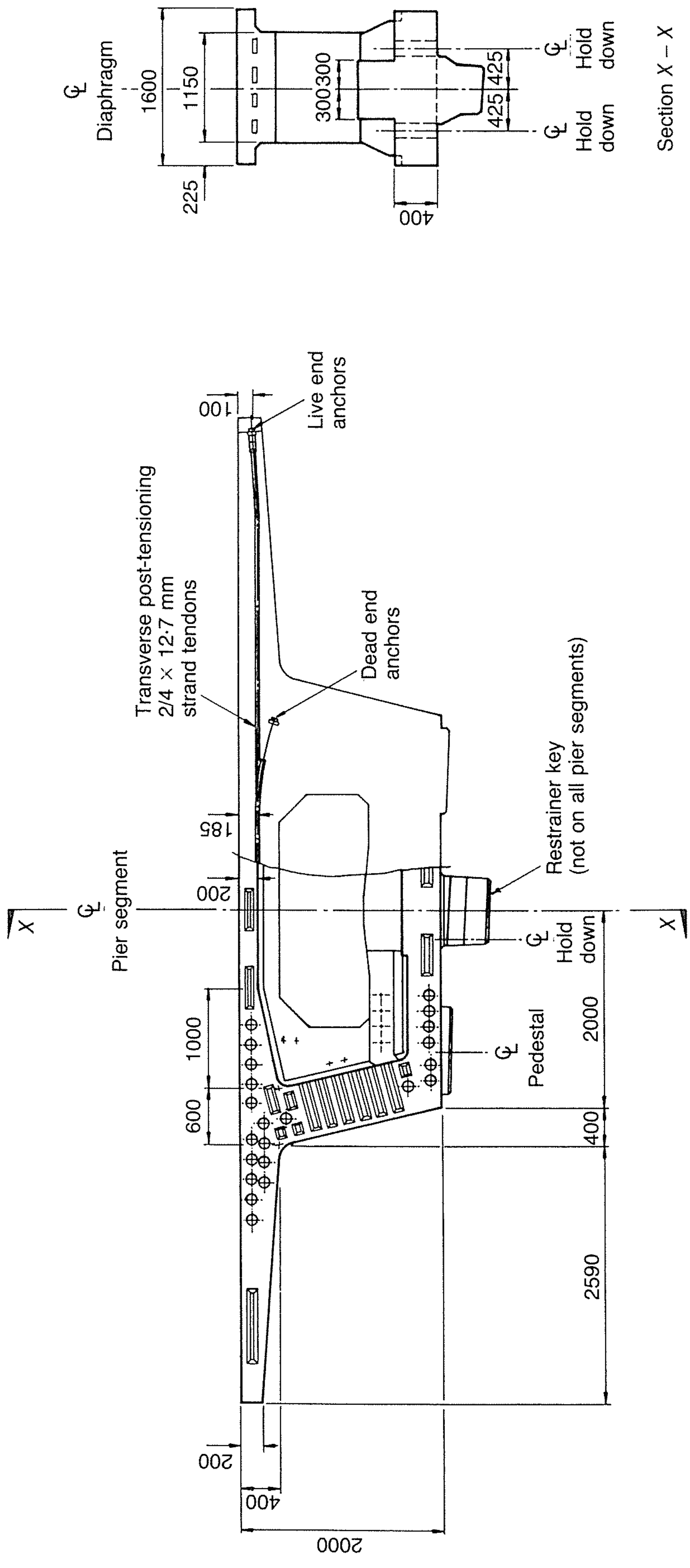


Fig. 10. Single cell pier segment 600 mm web dimension)

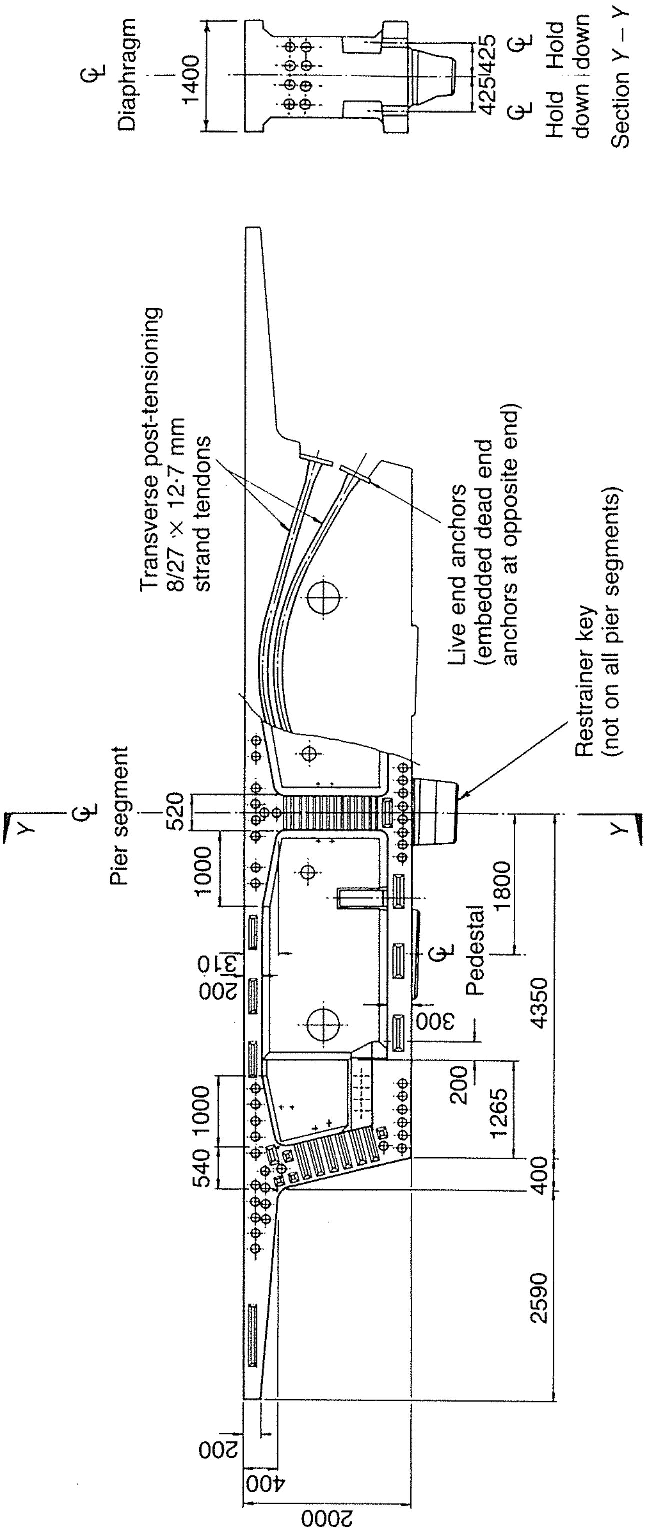


Fig. 11. Twin cell pier segment (540 mm/520 mm web dimensions)

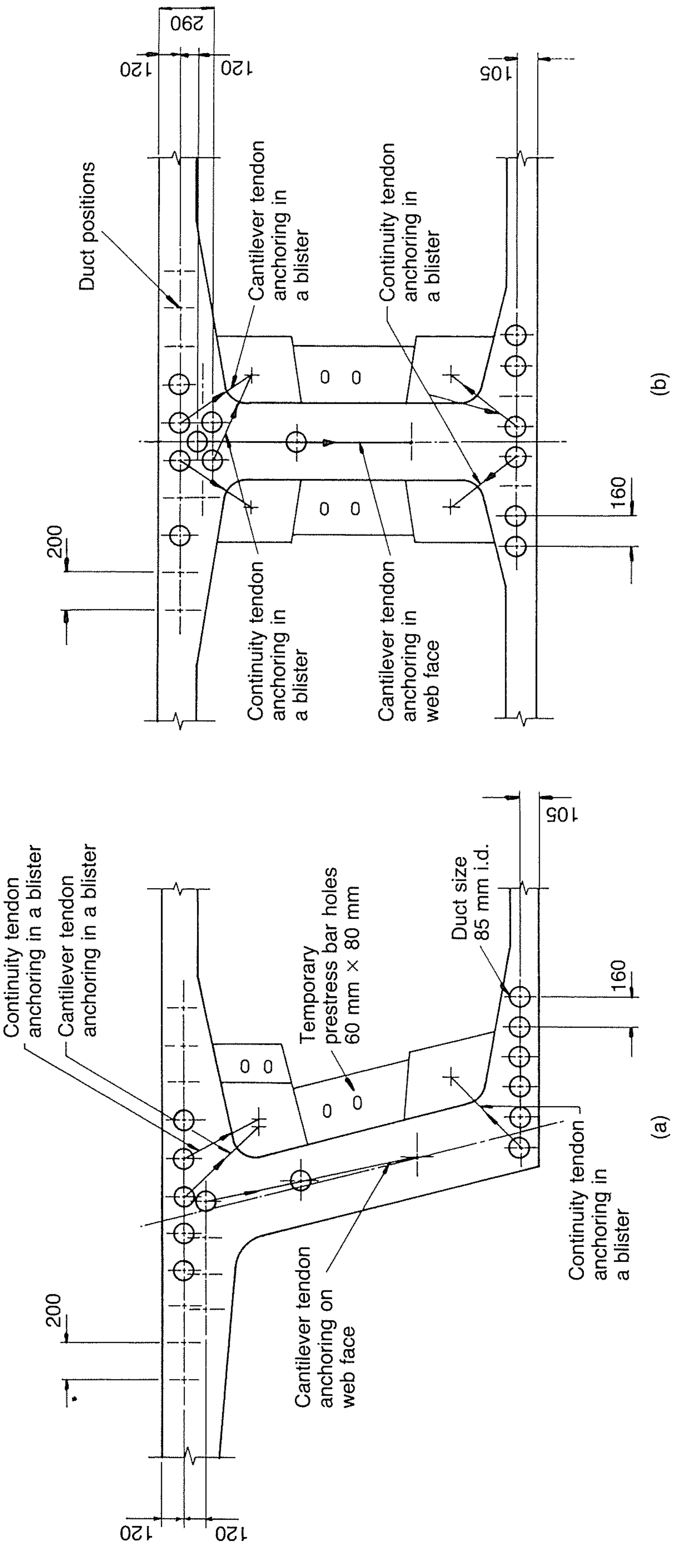


Fig. 12. Cross-sections showing arrangement of post-tensioning tendon ducts in segments and paths to anchorages: (a) single cell segments and outer webs of twin cell segments; (b) centre web of twin cell segments

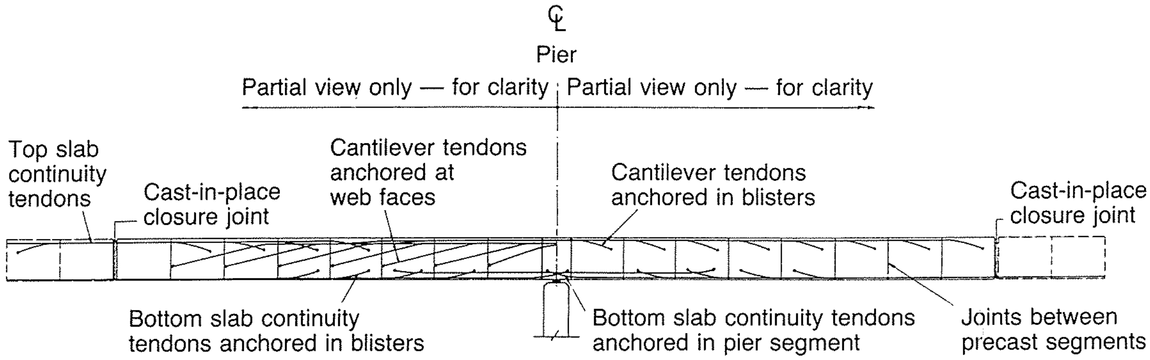


Fig. 13. Elevation on typical layout of post-tensioning tendons

stress. Figs 10 and 11 show the tendon profiles in the single cell and twin cell pier segments. The large transverse tendons in the twin cell pier segments were installed before casting, and six of the eight tendons were not stressed until the segments were placed in the structure. Because of the long delay, special precautions were taken to prevent corrosion during storage by sealing the ducts and keeping them full of dry air under pressure.

Permanent longitudinal prestressing

49. Longitudinal prestress in the superstructure was provided by 19 × 12·7 mm strand tendons, tensioned to a maximum jacking force of 2970 kN per tendon. The longitudinal tendons were anchored either in the internal blisters or at the end faces of the segment webs. Fig. 12 shows the standard tendon duct layouts in the segment cross-sections and the paths to the anchorages. Spirally wound galvanized steel (0.3 mm wall thickness; 85 mm i.d.) was used for the tendon ducts.

50. Tendon geometry was standardized and duct holders were pre-set in the reinforcing cages to follow the required profiles. In particular, the radius of curvature was set uniformly at 3·0 m. This is a relatively small radius but, in practice, it was not found to cause any observable problems. Threading of tendons was carried out by machine pushing of individual strands. Fig. 13 shows an elevation on the tendon layout for a typical span.

51. Post-tensioning operations proceeded without any of the friction loss difficulties sometimes associated with segmental construction. Measured tendon extensions were consistently greater than design values. Table 7 sets out twelve sets of stressing results from tendons in three different spans. The friction coefficients used in the design were: $\mu = 0{\cdot}20$ (basic coefficient); $\beta = 0{\cdot}012$ (unintended angular deviation in radians/metre).

52. The measured extensions allowed for a 5 mm extension inside the jack (calculated) and a measured draw-in of the wedges, usually 6 mm. Stressing was carried out from one end only, lift-off tests being specified at the dead ends of selected tendons. Lift-off tests invariably confirmed the satisfactory stressing of the tendons.

53. The exceptionally good results for the post-tensioning operations were not fortuitous. Attention to detail in the design of duct profiles and the fixing systems used in the casting cells meant that the post-tensioning tendons respected very closely the theoretical layouts. The positive attachment of ducts at the casting cell bulkheads (see Fig. 14) plus the use of inflatable duct stiffeners and precisely

Table 7. Prestressing: typical stressing results (extensions)

Pier/Span	Tendon	Length: m	Measured extension: mm	Calculated (design) extension: mm
102–103	B2 North	23·94	155	150
	B2 South	23·94	155	150
	T3 North	23·83	161	159
	T3 South	23·83	161	159
103–104	B1 North	17·53	124	121
	B1 South	17·53	126	121
	T4 North	30·23	196	193
	T4 South	30·23	198	193
312 N	E3 North	20·86	155	136
	E3 South	20·87	157	136
	C3 North	19·03	117	118*
	C3 South	19·04	119	119

* One of the rare cases where tendon extension was less than calculated (by less than 1%).

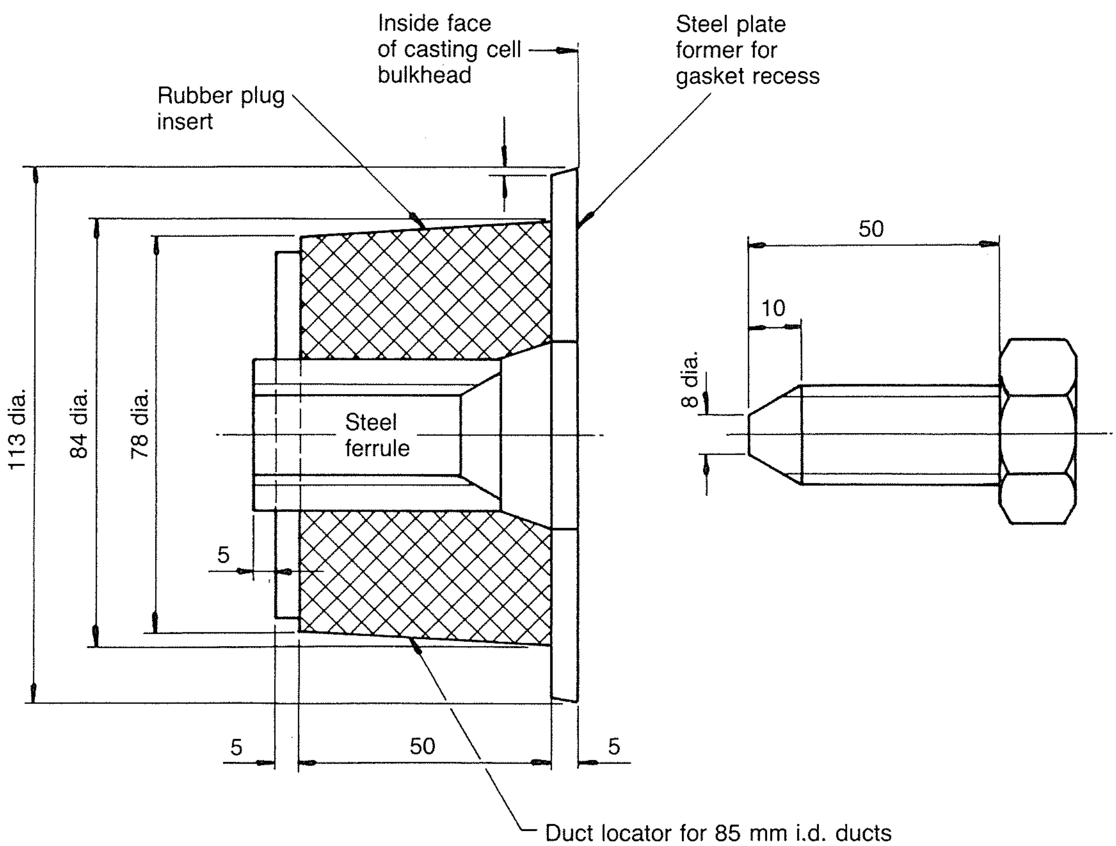


Fig. 14. Post-tensioning duct locator detail on casting cell bulkheads

positioned intermediate duct supports all contributed to trouble-free prestressing. Also, a slight belling of the inflatable duct stiffeners adjacent to segment joints together with the use of the rubber gaskets combined to eliminate a common friction trouble spot in segmental construction. Inflatable duct stiffening tubes are shown being installed in Fig. 8.

54. Grouting of tendons was programmed to lag at least three spans behind segment erection; this eliminated any possibility that grout would migrate into unoccupied tendon ducts. The grout was a conventional mix, using type A Portland cement, a water/cement ratio of 0·45 and a Methocel K15MS additive dosed at 80 grams per 40 kg bag of cement.

55. Vent tubes were provided at high points and low points, with intermediate vents at not greater than 10 m intervals. This provided readily available additional vents if required for flushing or inspection. Because of excellent sealing at segment joints, it was generally possible to grout individual tendons without interconnection. Grouting was carried out in symmetrical tendon groups containing from 2 to 6 tendons.

Bifurcations—transitions at exit and entry ramps

56. At the exit and entry ramp transitions with each carriageway, there are the geometrically complex areas which form bifurcations in the plan layout of the structures.

57. The locations of some expansion joints were chosen to coincide with the bifurcations in order to facilitate the detailing of the transition. At each expansion joint there is a diaphragm which assists the change of cross-section where two single cell box girders (four–lane carriageway) change to one single cell (ramp) and one twin cell (three–lane carriageway). The transverse diaphragms were cast in situ over the central zone between the precast box girder spines. Transverse prestress was provided by 38 mm dia. bars which crossed the full width of the precast and cast in-situ diaphragm sections. Tight geometric control of longitudinal positioning of the box girders permitted threading of the prestressing bars. Fig. 15 gives a view from underneath a completed bifurcation.

Expansion joint segments

58. The original design concept consisted of a prestressed concrete stepped joint with horizontal and vertical prestressing using bar tendons. This concept allowed no room for anchoring longitudinal prestressing tendons in the expansion joint segments. Longitudinal tendons were to be carried through the joint and anchored in segments beyond the expansion joint. When construction was advanced far enough to establish stability of the structure, the tendons were then to be cut at the joint and the forces transferred to the concrete by bond. Transfer of these forces had to occur over the length of the expansion joint segment to maintain the required compression at the joint with the next segment. Initial results from testing indicated that transfer could be achieved in the required length, but the transfer length increased with time, thus rendering the original design concept unsatisfactory.

59. To provide adequate room for anchoring longitudinal tendons in the segments, the area taken up by the corbels had to be reduced. This was achieved by the use of the cast steel corbels shown in Figs 16 and 17. The cast steel is grade L1A, with a yield stress of 350 N/mm^2, normalized and tempered to AS 2074—1982. The corbels had a unit weight of 2·4 t.

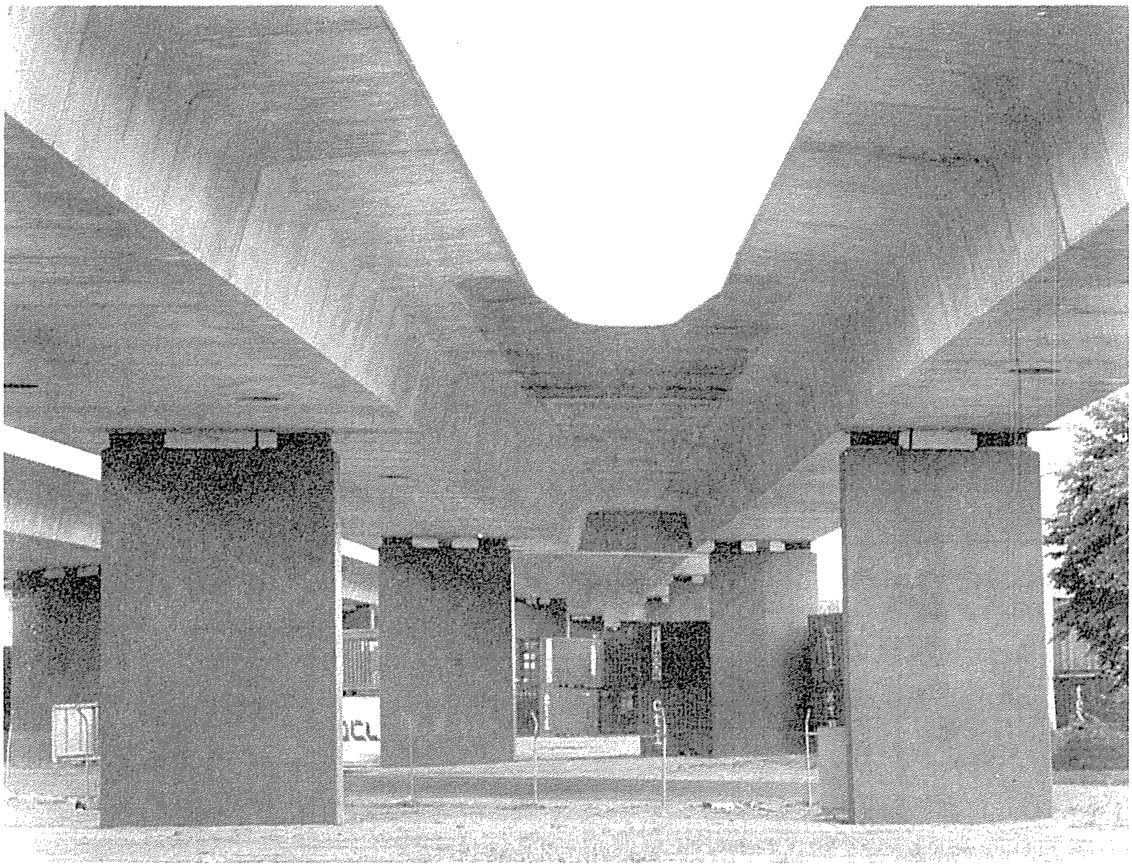


Fig. 15. View looking east into the Montague Street exit ramp bifurcation

60. Each corbel was designed for a maximum in-service vertical load of 2540 kN applied through the bearing and a maximum uplift of 150 kN. Proof loading of the first two production corbels was performed at 1·2 times the maximum in-service load, with flat jacks simulating the eccentric bearing positions and applying the loads. Movements were in the range 0·1–0·5 mm. Non-destructive evaluation after testing confirmed the integrity of the corbels.

61. The revised design of the segments gave an arrangement that was fully standardized, not as complex nor as congested as the original design and considerably lighter. Fig. 16 shows a sectional elevation of the final design layout.

62. During the cantilever erection of segments, each joint was locked by temporary prestressing bars which transferred force across the temporary packing concrete in the joint. After construction had advanced beyond the next anchor pier, the temporary prestressing bars were released and the packing concrete removed, thus allowing the joint to function.

63. The bearings are Wabo-Fyfe high load structural bearings models B1-600 and B2-600, the principal Bonafy (elastomeric) element being protected from shear loads by a shear pin to accommodate the load and to permit rotation of the structure. The majority of bearings are multi-directional except that one unidirectional bearing was specified at each joint to retain longitudinal alignment.

64. Uplift restrainers were specified for all corbels, as uplift forces could occur under some load cases. A 35 mm dia. prestressing bar tensioned to 175 kN was placed between the two matching corbels, the lower end being fixed onto the corbel and the upper end clamped on to a sliding plate and guide bracket placed

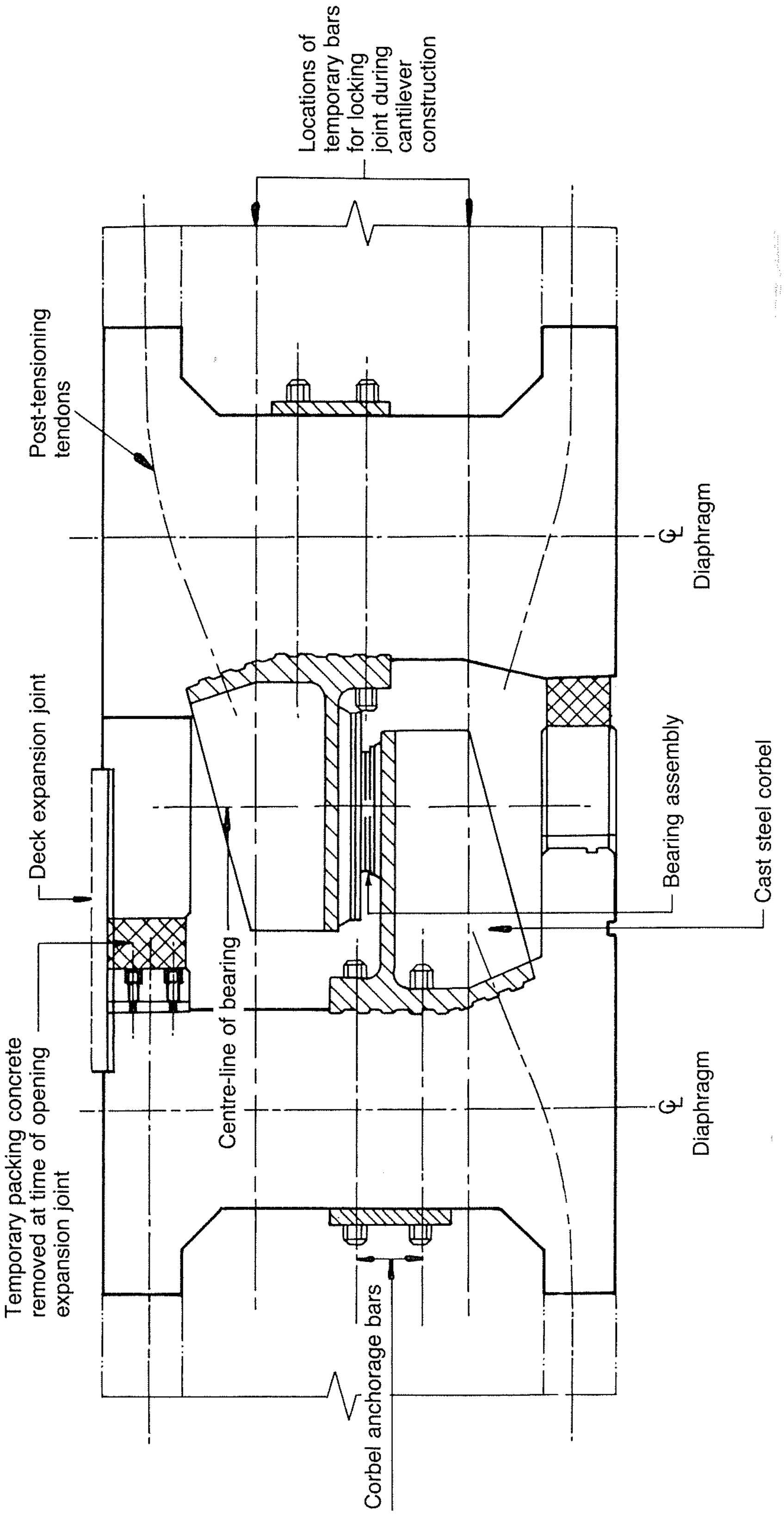


Fig. 16. Section through typical expansion joint

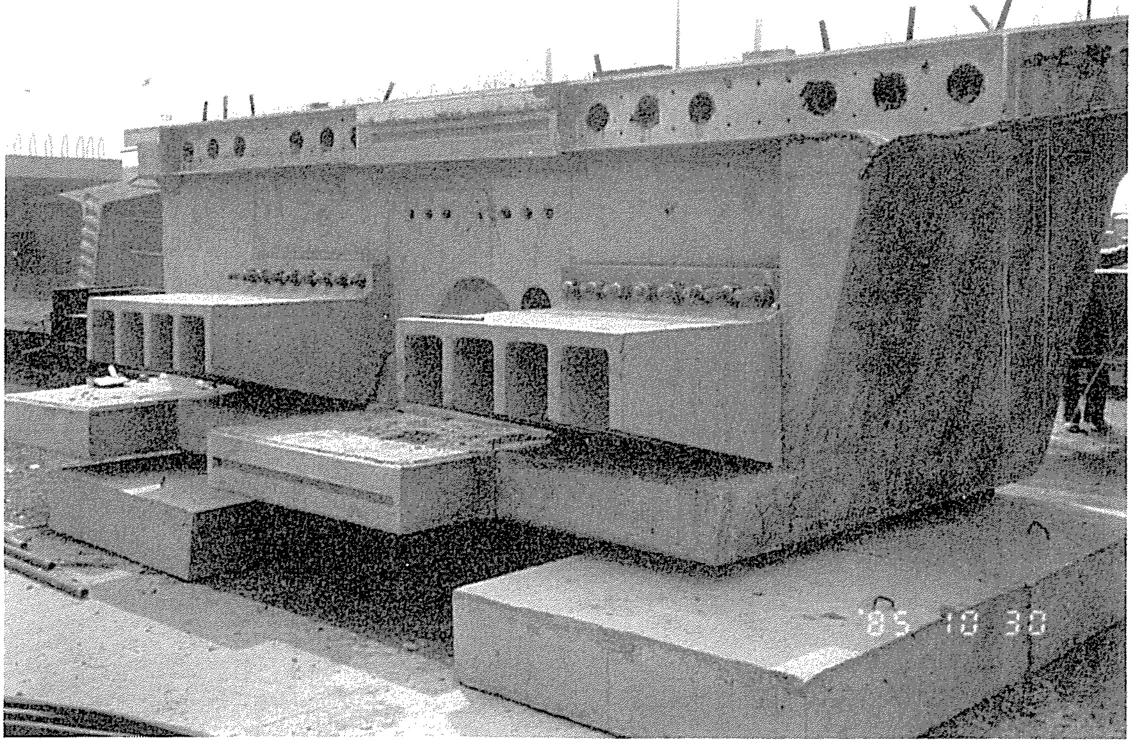


Fig. 17. Single cell expansion joint segment

between the two outer ribs at the upper corbel. Dust cover plates protect the assemblies and access is possible to inspect the condition of the upper connection or to replace bearings if necessary. Fig. 17 shows a view of a completed, single cell expansion joint segment with two corbels.

65. One construction problem arose which demonstrated the need for vigilance in construction supervision. The cast steel corbels are stressed to the precast concrete segment, using 12 × 38 mm dia. prestressing bars tensioned to 925 kN per bar (jacking force). To confirm the bar tension, lift-off tests were performed on completion of stressing.

66. During lift-off testing for one of the expansion joint segments, it was observed that significant and inexplicable losses of tension were occurring. Further investigation revealed that the problem arose because the threads of the nuts were being cut to the extremes of allowable tolerances. Stripping of threads was creating the loss of prestress. The offending nuts were replaced with nut sets taken to closer tolerances and no further difficulties were encountered.

Precasting of segments

General

67. To produce the 2070 superstructure segments, a complete precasting facility was established at the western end of the site.[6] This comprised

(*a*) a factory building, approximately 180 m × 18 m, containing reinforcement cage fabrication and storage areas, and five segment casting cells (three single cell and two twin cell), all serviced by three 10 tonne SWL overhead cranes

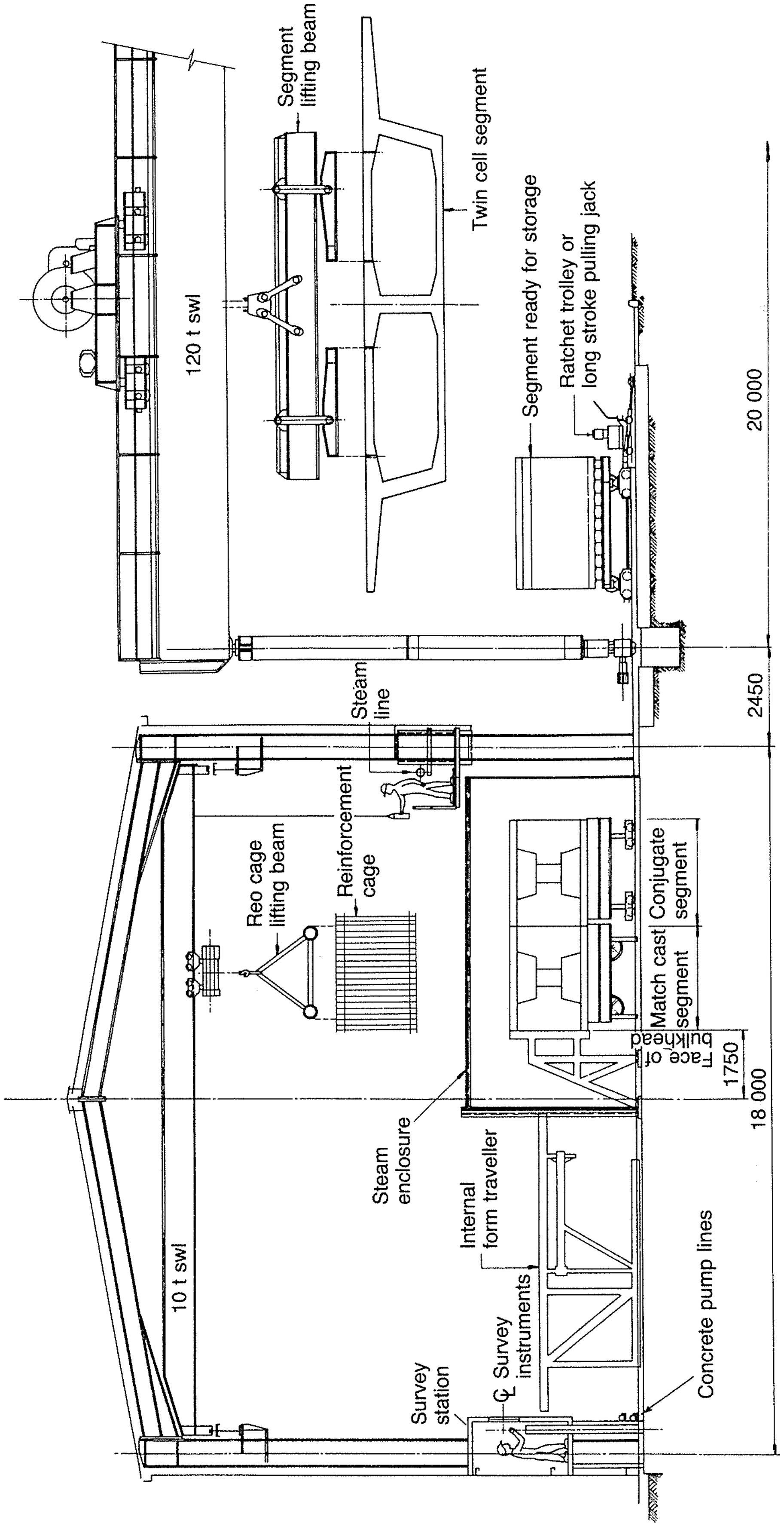


Fig. 18. Cross-section through precasting factory and segment storage area

(*b*) a plant room for steam generation, air and water supply
(*c*) a concrete batch plant and pumps (1·5 m^3 Fejmert pan mixer, with pumped concrete delivery by way of 125 mm dia. slick lines at a maximum rate of 40 m^3/h)
(*d*) a segment storage area, approximately 450 m × 20 m, serviced full length by a 120 t SWL gantry crane; a sixth casting cell for the casting of expansion joint segments plus sand blasting and inspection areas for segments were also located within the storage area.

Fig. 18 shows a typical cross-section of the precasting factory and segment storage area, and Fig. 19 gives an aerial view of the facilities.

68. The factory building was split equally into reinforcement cage assembly and segment casting areas. One casting cell of each type was partially dedicated to pier segments. Because of the poor ground conditions the foundations for each casting cell consisted of a 1·2 m thick octagonal slab supported on multiple piles. Aligned with each casting cell was a survey station with the instruments situated on top of a pile sleeved through the floor slab.

69. Computer aided drafting was used to prepare two product drawings for each segment: one for concrete details and one for all items associated with the reinforcing cage. A total of 4076 product drawings was produced, not including the expansion joint segment drawings which were prepared manually.

Production of segments

70. In order to maximize productivity and quality control, the assembly of reinforcement cages for typical segments was arranged on a production line basis. Cages were totally assembled before being placed in the forms. Four cage jigs were fabricated, two for pier segments and two for typical segments. These jigs matched the profiles of the casting cell forms less concrete cover. The cages for pier segments were assembled bar by bar. The reinforcement for typical segments was sub-assembled into vertical layers before assembly of the whole cage. Two special layer tables were designed and fabricated to enable the sub-assembly of reinforcement layers.

71. Short line match casting of the segments began with the pier segment which was cast between steel bulkheads. Each remaining segment of a cantilever was then match cast sequentially between a steel bulkhead and the segment adjacent to it in the superstructure, as can be seen in Fig. 20.

72. The casting cells in the factory operated on a daily cycle, with the aim of casting one typical segment per day in each cell. Pier segments generally took 2–3 days each to manufacture. Concrete was delivered by pumping to each casting cell, and was internally vibrated. No external vibration was used following initial unsatisfactory trials. Crew sizes were three men per casting cell, plus a single six-man team which carried out the concrete pours for all casting cells.

Steam curing of concrete

73. Steam curing ensured that the forms could be stripped the next morning without damage to the segments. The steam curing cycle was adjusted over the first few months of segment production in order to achieve an optimum cycle for achieving the necessary concrete strength of 25 N/mm^2 specified for demoulding.

74. The final cycle is given in Fig. 21. Steaming began after an initial maturity period of two hours and the rate of temperature rise was specified as not greater

Fig. 19. Casting factory and segment storage yard

than 24°C/h. The maximum temperature in the steam enclosure was 50°C, which was maintained for 3–4 hours before the temperature was allowed to drop. The steam covers could be opened as soon as the enclosure temperature was less than 30°C above the ambient temperature. Total duration of the cycle—from the completion of concreting to the opening of the steam tents—normally varied between 8 and 11 hours. In a typical production run, steaming would begin at about 20.00

hours and the steam covers would be opened at 04.00 hours the following morning.

75. The steam enclosures enveloped both the segment that was being cast and the conjugate segment. This is essential if differential deformation between the two segments is to be avoided. The geometry control checks confirmed that the small relative temperature differences did not create any adverse geometric effects.

76. The strength grade of the concrete was either 45 N/mm^2 or 50 N/mm^2, and it was not uncommon to achieve 28 day cylinder results of 65 N/mm^2 from the concrete which was batched on site.

77. Actual production achieved over the course of the project averaged 3·5 segments per day, with a weekly maximum of 27 segments produced in a six-day week.

78. After removal from the casting cell, each segment was stored for a minimum curing period of 40 days in order to respect the design assumptions for creep and shrinkage behaviour. The area adjacent to the factory building provided storage for up to 280 segments. Before transportation out to the erection site, each segment face was lightly sand blasted and any additional minor repairs were carried out.

Control of concrete quality

79. Because of the large volume of concrete involved (36 700 m^3) in the manufacture of the precast segments, special attention was given to the design of a testing scheme for both compliance testing and the early assessment of concrete quality. The scheme involved the taking of twelve test cylinders from each segment plus concrete for slump tests.

80. The Roads Corporation also decided to conduct a programme of accelerated strength testing based on 24 hour results. This programme had no contractual significance but was intended to provide information for the contractor to assist in making an early decision on likely concrete acceptance. Accelerated strength testing, at 24 hours, using the hot water method in British Standard BS 1881, was carried out from the start of precasting in May 1985 until July 1986.

81. The 24 hour testing was discontinued when it was shown that quality control procedures were producing concrete of consistent quality. A three-day test on normal cured cylinders was retained as providing a useful early confirmation of strength development. The procedures and results are described in detail by Weinberg.[7] Table 8 summarizes typical strength test results for the period September to November 1986.

Table 8. Typical concrete cylinder strengths, September to November, 1986

Specified concrete cylinder strength grade	45 N/mm^2: N/mm^2	50 N/mm^2: N/mm^2
28 day test		
(normal curing) mean	66·6	66·5
standard deviation	4·2	4·5
28 day test		
(steam curing) Mean	61·6	61·8
standard deviation	4·2	3·9
3 day test		
(normal curing) mean	47·7	48·6
standard deviation	3·4	2·9

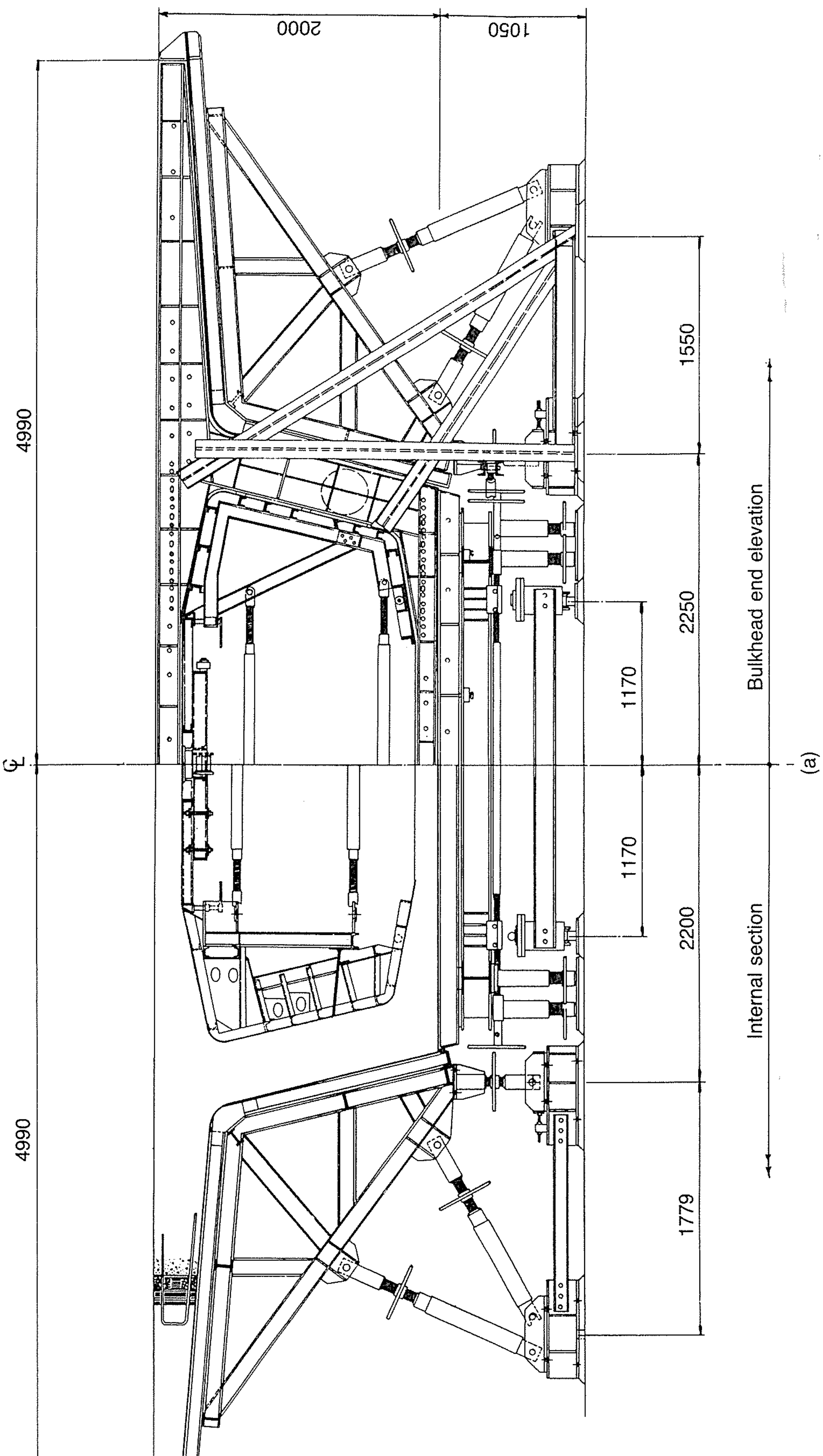


Fig. 20 (above and below). Short line casting cell for single cell segments: (a) views looking through casting cell; (b) longitudinal section through casting cell

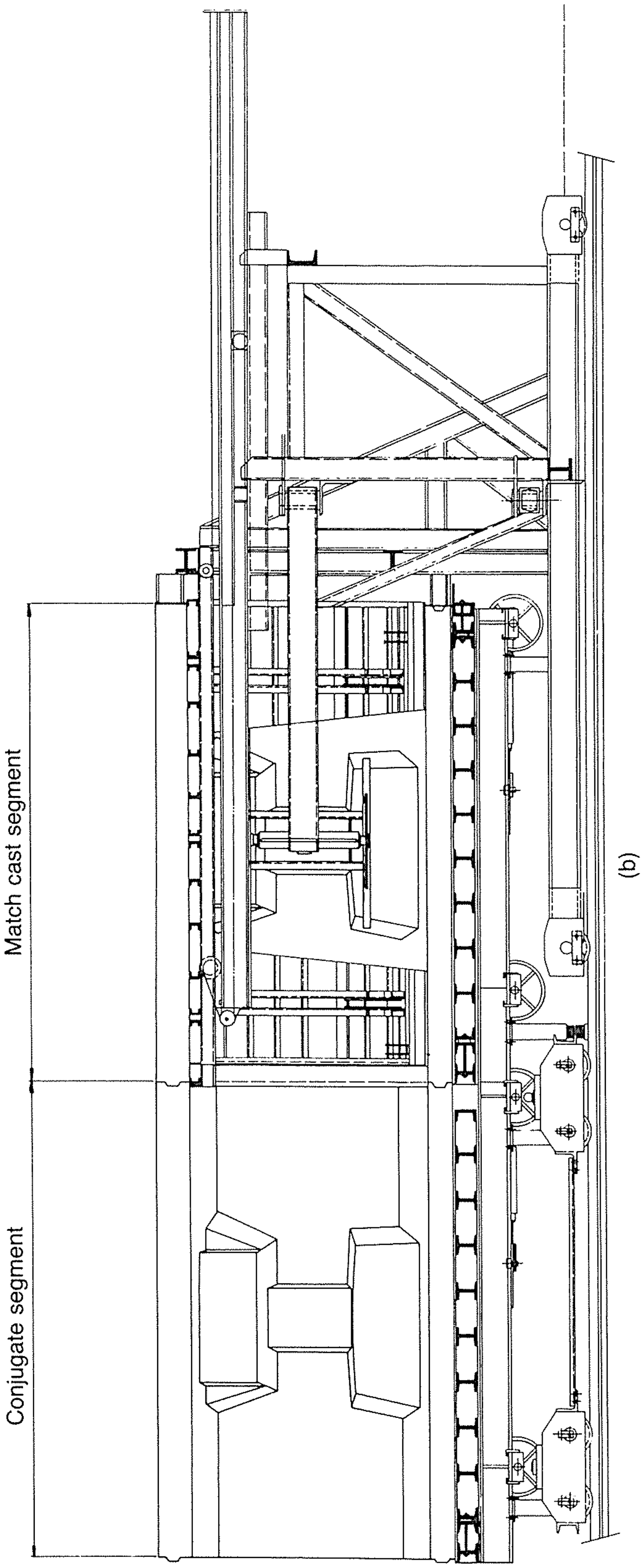


Fig. 20—continued

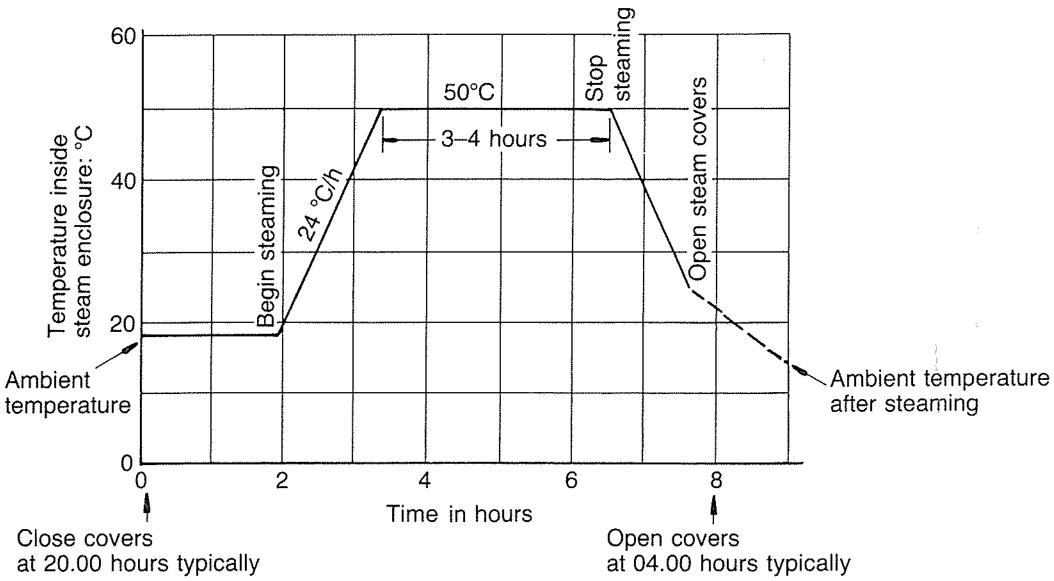


Fig. 21. Steam curing cycle for segments

82. The similarity of the average strengths of the two grades of concrete in Table 8 can be attributed to the differing dose rates of superplasticizer used during the period examined. The average dose rate of superplasticizer for the 45 N/mm^2 concrete was 745 ml/100 kg cement compared with 665 ml/100 kg cement for the 50 N/mm^2.

Geometry control for match casting of segments

83. Proper control of segment geometry is critical for balanced cantilever erection of match cast segments. An essential element of the Quality Assurance procedures for geomety control was independent checking of calculations and measurements at all steps in the process.[8] The steps involved in geometry control of segment casting and erection are summarized in the flow chart of Fig. 22.

84. The contractor was required to carry out an independent deflexion or precamber analysis. The results were compared with the design values, whereupon the precamber values for precasting were determined.

85. The contractor's values were used for all precamber entries. The geometry of the finished box girders confirmed the high degree of precision of the calculations. Final deviations from the theoretical finished deck profile were not more than 10 mm vertically and 5 mm horizontally compared with the specified tolerances of 15 mm.

86. In only one span—the 52·9 m span between piers 417 and 418, comprising single cell segments—was there an inexplicable difference of deflexion behaviour between identical adjacent boxes. Fortunately, the net differences were within allowable limits, and the slope discontinuity on the cast in-situ infill section between the two boxes was compensated within the deck surfacing thickness.

87. The required accuracy of measurement for match casting of balanced cantilever segments is much stricter than for normal concrete work. All survey readings of horizontal offsets and levels were taken to 0·1 mm.

88. Equipment used for the geometry control was high precision equipment which allowed both the contractor and the checking survey teams to work com-

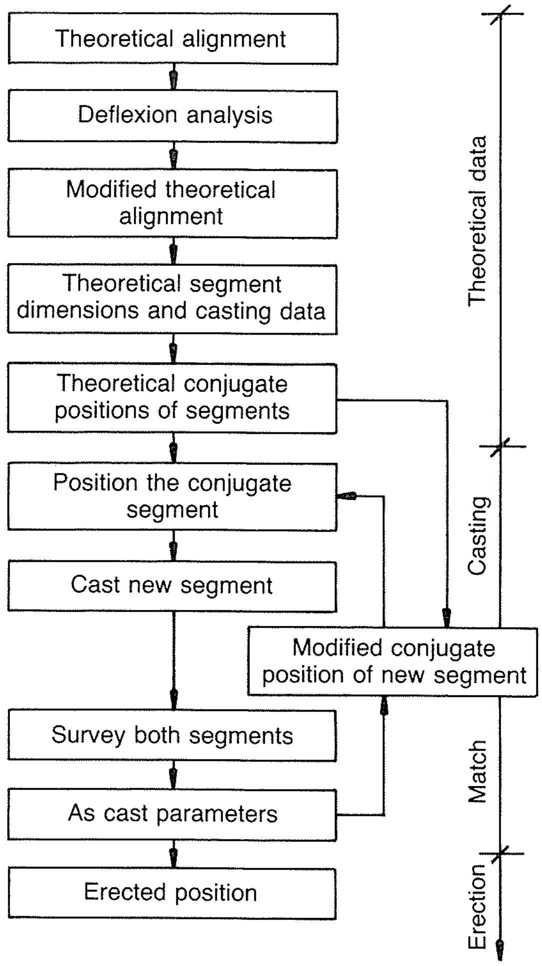


Fig. 22. Flow chart for segment geometry control

fortably within the allowable measurement difference limits of ± 0.2 mm. The main items of survey hardware were

(*a*) theodolite: WILD T2 with translation stage GAD 12
eyepiece × 40 (standard is × 32)
direct reading to 1/000 mm
cross-hairs
(*b*) level: WILD NA2 parallel plate—automatic reading to 1/100 mm
(*c*) staff: WILD GWL 18Z Invar Staff (1·8 m) with stabilizing struts
(*d*) alignment scale: made-up device incorporating steel rule plus screw levelling system.

89. The theodolite and level were mounted on a piled instrument station, independent of the factory floor and foundations. Ten level points and five alignment points were observed in the as-cast survey. Contractor and checking surveyors carried out independent surveys with separate instruments. Agreement between the two surveys was achieved if all readings were within $\pm 0{\cdot}2$ mm. The surveys were repeated for any readings outside this tolerance.

Erection of superstructure

Summary of erection operations

90. The balanced cantilever erection of precast segments used a combination of mobile crane and two launching girders.[6,9] The two box girder spines of each

structure were erected concurrently, beginning with the north carriageway. The launching girders were then transported back to the west end to begin the south carriageway.

91. The first two or three segments on a pier were generally erected with a mobile crane. The segments were then aligned and the bearings grouted. The launching girder was launched into position over the pier segment and the remainder of the cantilever segments erected one at a time alternately on each side of the cantilever. Fig. 23 shows a launching girder working on the 113·5 m radius Kings Way exit ramp.

92. Immediately before the erection of each segment, epoxy glue was spread on one match cast face to a thickness of 2–3 mm. The special rubber gaskets around the post-tensioning duct openings prevented entry of epoxy into the ducts.

93. At the completion of the cantilever erection phase, the short closure joint (200 mm approximately) to the existing continuous structure was formed and poured. It was necessary to carry out this operation at night to ensure that differential thermal effects were kept to a minimum. Steam curing of this section enabled continuity prestressing to be applied across the joint the next morning.

94. The superstructures pass over several electrified tram and train lines. Erection work in these areas was carried out at night, with the overhead power de-energized. Erection over trafficked roads required lanes to be blocked or traffic diverted while work proceeded overhead.

Epoxy jointing of segments

95. The epoxy was mixed on the bridge deck and applied by gloved hand (see Fig. 24). Application of the epoxy was limited to the rear face of the segment that

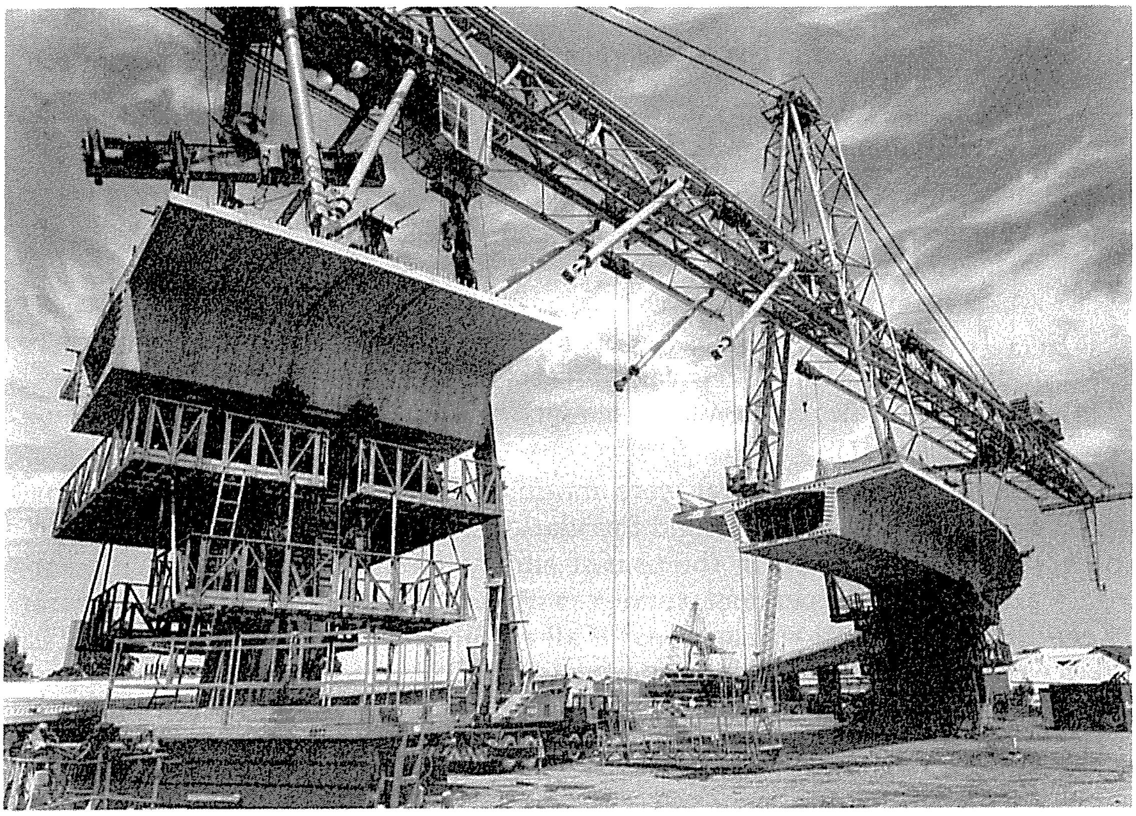


Fig. 23. Launching girder during an advancement phase on 113·5 m radius Kings Way exit ramp (note second launching girder in background)

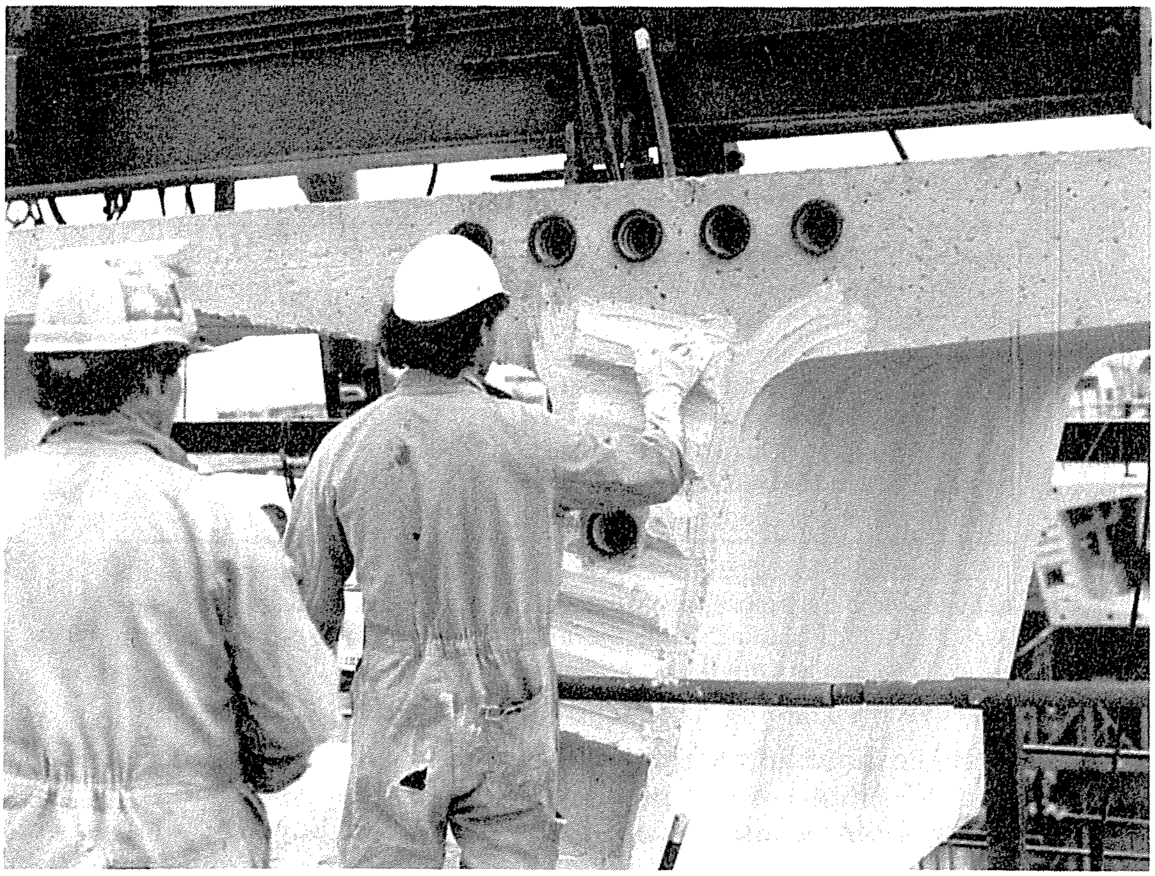


Fig. 24. Application of epoxy to segment joint

was being erected. The entire operation—epoxy application plus segment placing and temporary prestressing—usually took about 25 minutes.

96. Temporary prestress was designed to provide a minimum pressure across the joints of 0·3 N/mm^2. As intended, this gave joint thicknesses of the order of 0·5–1·0 mm. Joint closure thickness monitoring gauge points were installed in the first few spans erected.

97. The epoxy in the joints was designed to perform three principal functions

(*a*) joint sealing to enhance durability of the structure by preventing ingress of water
(*b*) lubrication during segment placing to facilitate the mating of the match cast surfaces
(*c*) filling of any small imperfections or damage to the match cast faces to ensure a full cross-section for stress distribution.

98. Formulations of Fosroc Nonporite Nitobond PCA epoxy were adopted for three temperature ranges: A. 5–10°C; B. 5–20°C; C. 15–30°C. The specified properties for all formulations were generally exceeded by the results from the acceptance tests (see Table 9)

99. Final testing requirements for epoxy adhesive during segment erection were limited to

(*a*) tensile bending tests: one per joint (reduced from three initially specified)
(*b*) contact time: two random tests per cantilever.

There were only six cases of low test results, where subsequent coring at the joint

Table 9. Epoxy joint adhesive: comparison of principal specified properties with typical test results

Property	Specification (for all formulations)	Typical test results
Pot life (minimum)	25 min	35 min
Contact time (minimum)*	90 min	110 min
Minimum compressive strength		
15 mm dia. cylinders at 12 hours	15 N/mm^2	19 N/mm^2
15 mm dia. cylinders at 24 hours	25 N/mm^2	35 N/mm^2
30 mm dia. cylinders at 7 days	50 N/mm^2	62 N/mm^2
Disturbed slant shear strength at 24 hours (disturbance at 12 hours after jointing)	12 N/mm^2	22 N/mm^2

* Contact time is sometimes called 'open time' or 'allowable time' and is defined as the maximum time which may elapse, without detriment to the performance of a joint, from the application of a mixed epoxy resin adhesive to a joint until the closure of that joint.

interfaces demonstrated fully effective bonding/sealing at the joints concerned. The 100 mm dia. cores (three or four per joint) were split along the joint by diametral loading. The split tensile tests showed that the epoxy adhesive had adhered satisfactorily to the concrete in all cases.

100. No overall quality problems were experienced on site with the epoxy. On only one occasion—out of 2070—the liquid epoxy had to be removed from a segment face as a result of extended delays in application of the temporary prestress.

101. Grouting of the post-tensioning tendons demonstrated the excellent sealing properties of the epoxy together with the rubber duct gaskets in the joints. Leakage at segment joints was minimal, and there were very few reported cases of communication or grout migration between post-tensioning ducts at the joints.

Temporary longitudinal prestress

102. As the segments were placed alternately each side of the pier, temporary prestressing bars were installed through internal ribs to stress each segment to the adjacent segments. The magnitude of the temporary prestress force was calculated to maintain a minimum compressive stress across the epoxy jointed face of at least 0·3 N/mm^2.

103. 38 mm dia. Super Grade bars were used for all temporary prestressing. These bars have a minimum ultimate tensile strength (UTS) of 1230 kN, and were generally stressed to only 615 kN—50% UTS to permit re-use. Where segments had to remain supported overnight by temporary prestress alone, the bars were stressed to 925 kN—75% UTS.

104. In some specific locations—e.g. near abutments and adjacent to expansion joints—additional bars were necessary above, and occasionally below, the segments. These bars were fixed to the segments with special brackets that were clamped in position by means of single 60 mm dia. prestressed bars through lifting holes or other construction related holes in segments.

105. Expansion joint segments contained a greater number of temporary pre-

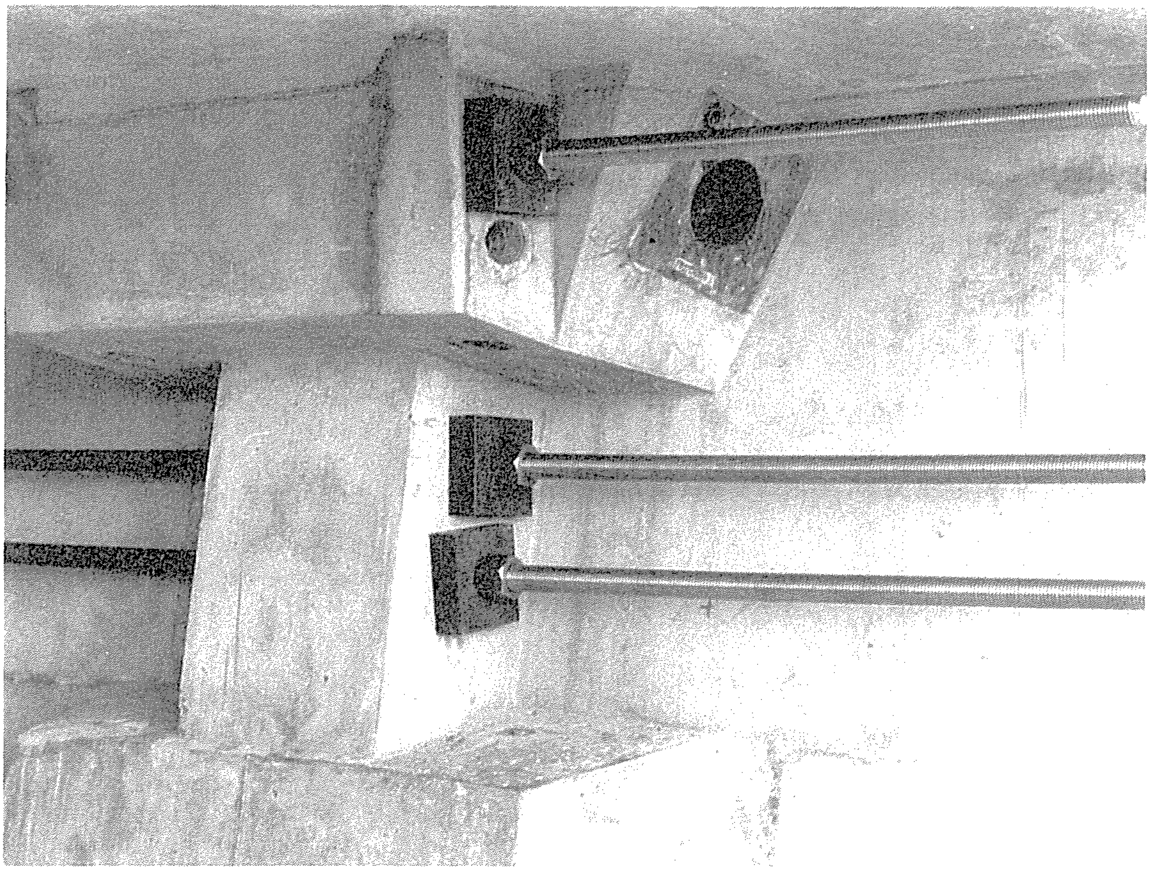


Fig. 25. Temporary prestressing bars anchored inside a segment

stressing bars than typical segments because of the necessity to cantilever up to two segments beyond the expansion joint without the benefit of permanent prestress. Fig. 25 shows prestressing bars anchored inside a typical segment.

Launching girders and special erection equipment

Design of launching girders

106. The West Gate Freeway launching girders represent the third generation in the development of gantry-style segment placing equipment. They were designed to be light, demountable and autonomous, as well as being able to negotiate complex bridge geometry.[9]

107. The conceptual requirements called for two launching girders capable of cantilever placing of segments weighing up to 68 t in spans of between 24 m and 55 m. The bifurcated, twin box girder spines that made up each carriageway made it impractical to use a single launching girder.

108. During cantilevering, it was necessary to provide deck stabilization for the out-of-balance phases when segment numbers and/or weights were not identical on both sides. Consequently, stabilizing arms on the launching girder were used to convert the out-of-balance loads of up to ± 130 t to launching girder support reactions.

109. The principal loading criteria are summarized as follows

(*a*) safe working load = 68 t (maximum standard segment weight)
(*b*) stabilizing arm loads = ± 130 t maximum.

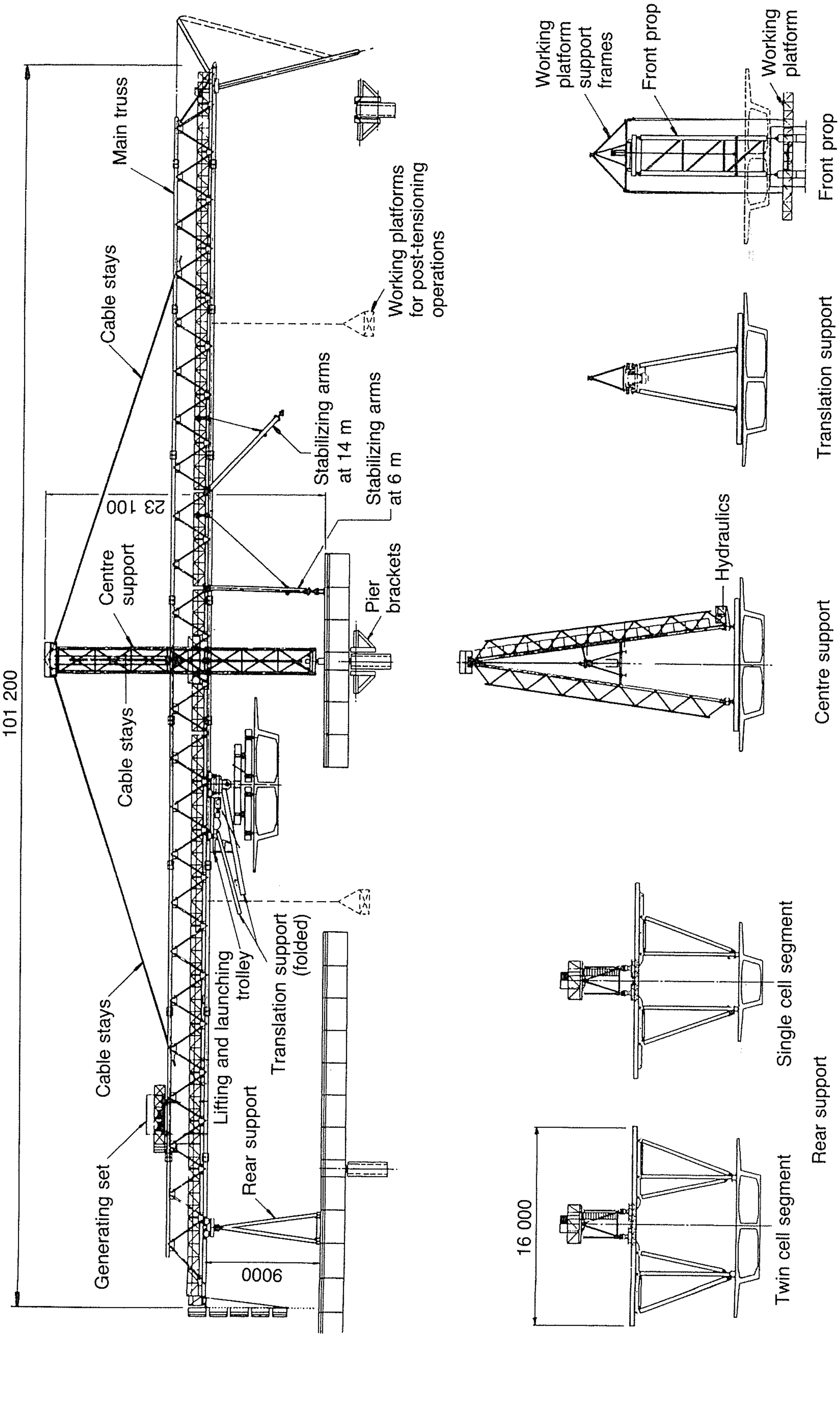


Fig. 26. Principal elements of the West Gate Freeway launching girders

Maximum wind velocity was 20 m/s in-service and 41 m/s for out-of-service or storm wind loading.

110. An important criterion for design was the use of grade 350 L0 steel to minimize the total weight of the equipment. The total fabricated weight of each launching girder, including all deck attachment beams, was of the order of 200 t.

111. A three-phase power supply—415/240 volts at 50 Hz—was provided for each launching girder by a 110 KVA Detroit diesel generator. The nominal circuit voltage for pendant control boxes was 32 volts.

112. The minimum radius of curvature of 113·5 m (except for one end span of radius 103·5 m) required a launching girder of extreme versatility to negotiate the horizontal curvature. Before this project, no launching girder had been used on a radius tighter than 300 m. Fig. 26 shows the principal elements of a launching girder. A key feature was the ability to rotate in plan by $\pm 8.5°$ about the centre support. This facilitated precise positioning of segments and permitted relatively simple movement around curved superstructure zones.

113. The design of the launching girders was carried out by French Consultants, Europe Etudes Gecti under the direction of Mr Jean Muller, in accordance with the requirements of the following Australian design codes: AS 1418, part 1: *SAA crane code;*[10] AS 1250. *SAA steel structures code*;[11] AS 1170, part 2: *wind forces.*[5]

Load testing

114. The launching girders were tested for dynamic and static load conditions in the presence of Department of Labour inspectors. Steel billets were used as the test loads

$$\text{dynamic load test: } 1{\cdot}20 \times 68 = 81{\cdot}6 \text{ t}$$
$$\text{static load test: } 1{\cdot}40 \times 68 = 95{\cdot}2 \text{ t}$$

The load factors specified for testing were in excess of those normally applied for crane testing in Australia: 1·10 and 1·25 for dynamic and static loads respectively. Because of the high load factors used in testing, special permission was given by the Department of Labour to use the launching girders for lifting the three heaviest segments (expansion joint segments) which weighed 76 t each.

Operational phases

115. The launching girders were assembled on the western approach structures—one on the main north carriageway and one on the Montague Street entry ramp. Details of operational phases depend on span lengths and radius of curvature. The standard procedures were common to all cases and are shown in Fig. 27. Computer aided drafting was used to generate over 1000 drawings to cover all movement phases at every working location.

116. Launching girder support attachments and the segment lifting beams were fixed to the segments using prestressed bar connections. Hydraulic systems on the lifting beams oriented the segments to the correct grade and crossfall before placement in the structure.

Performance in service

117. The 2070 precast segments were placed without mishap between September 1985 and June 1988. Fig. 28 shows one of the launching girders and a set of pier brackets in a typical segment placing phase. The maximum number of seg-

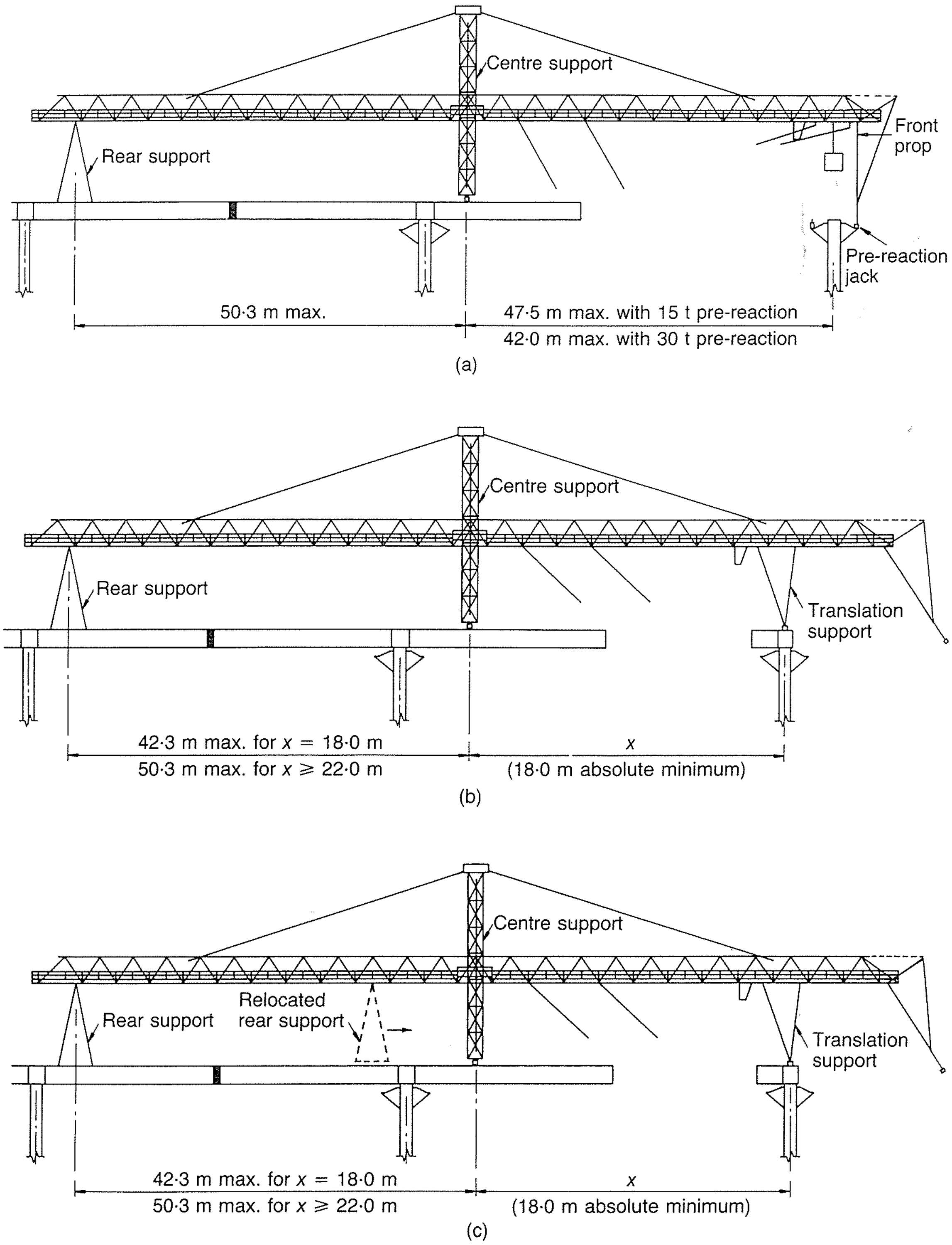
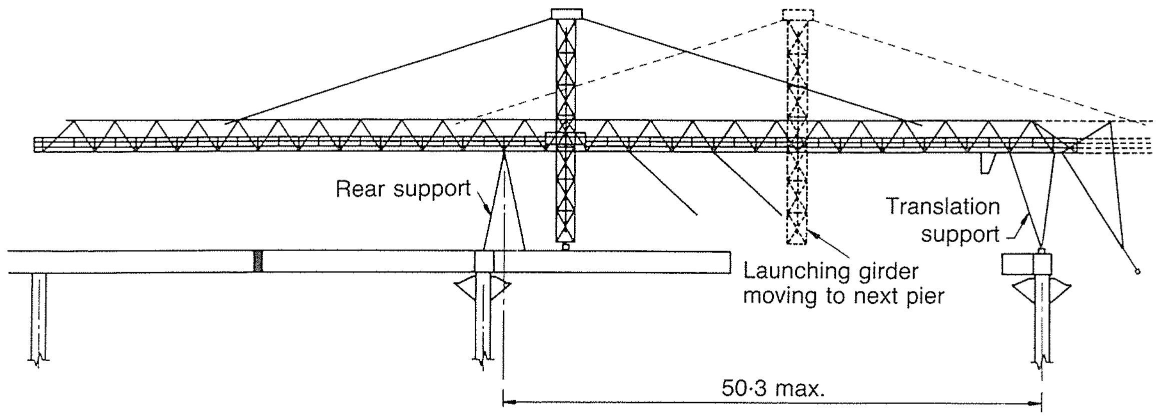
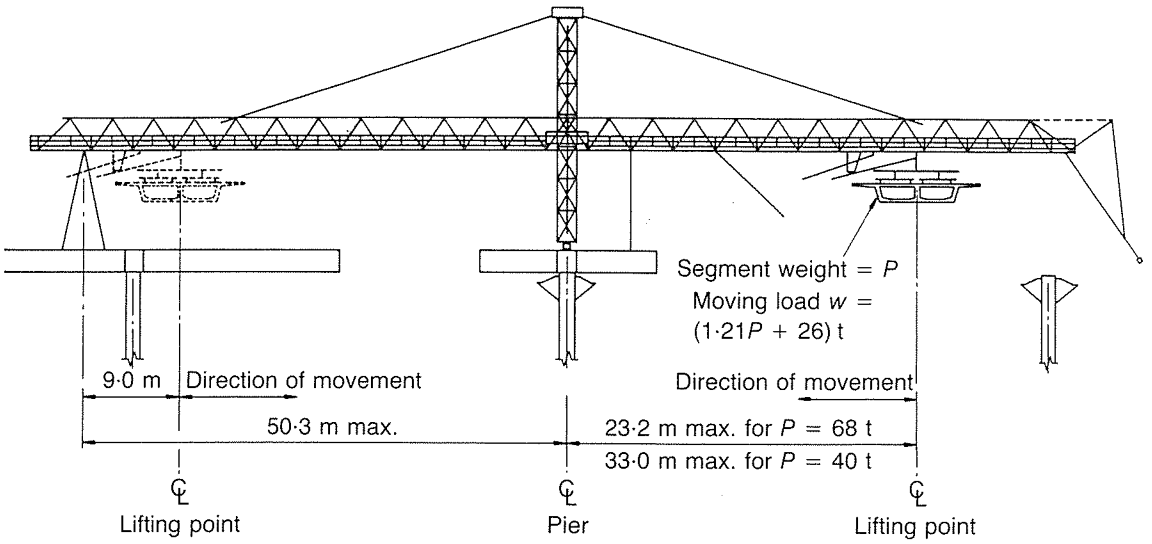


Fig. 27 (above and facing). Operational phases of launching girders: (a) placing of pier segment and one adjacent segment—launching girder on three supports; (b) placing of translation support; (c) relocation of rear support; (d) advancement (translation) of launching girder; (e) operational phases of launching girders



(d)



(e)

Fig. 27—continued

ments placed in a single day was 14 (8 + 6) by both launching girders, and the average number was four per working day taken over the entire superstructure erection period. Each launching girder was operated by a three-man crew. Post-tensioning was conducted by a separate team of seven men for the twin cell segments and four men for the single cell segments.

Transportation of launching girders

118. A critical operation for the construction of the superstructures was the return of the launching girders to the west end of the project to begin segment placing on the southern carriageway. This was achieved without disassembling the girders by loading them on to specially adapted low-loaders and towing them back to the starting point. The actual transportation operation took only about three hours for each launching girder. Fig. 29 shows a launching girder being transported.

Pier brackets

119. Because of the very poor ground conditions, the pier brackets were designed in preference to ground based temporary supports around the piers.

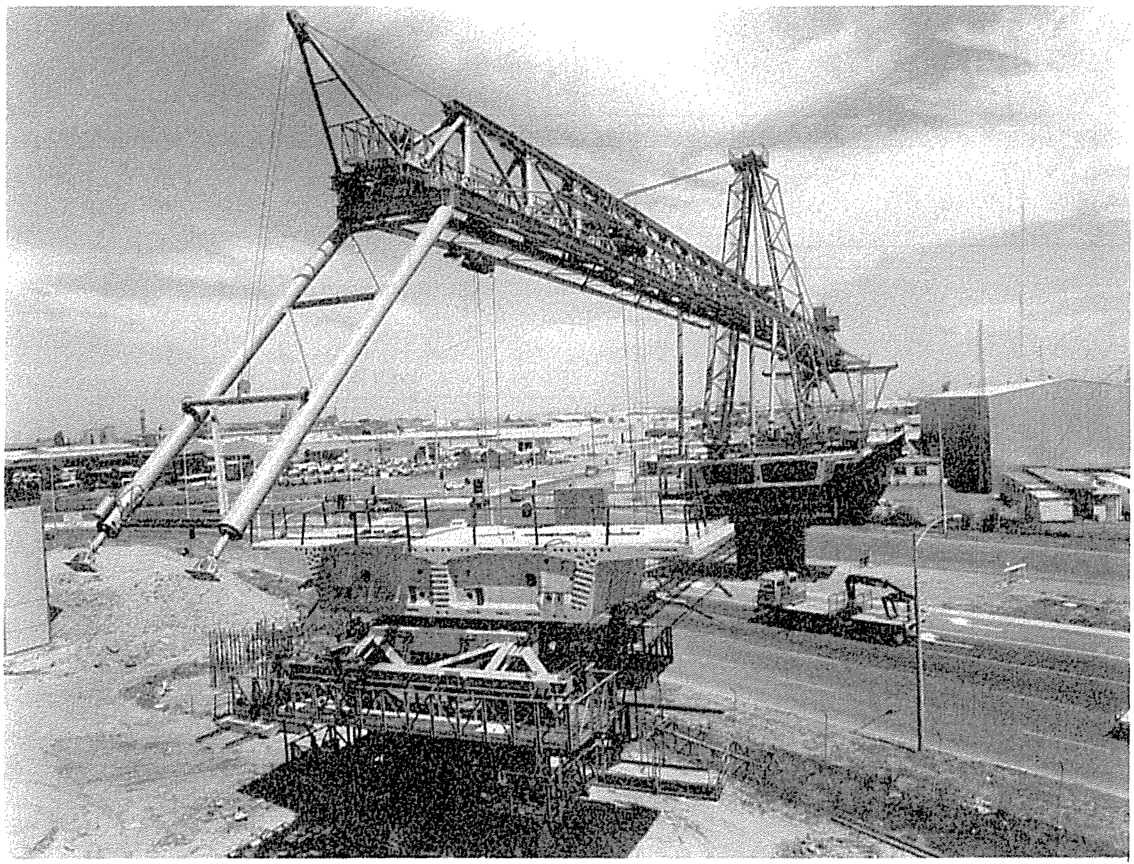


Fig. 28. Launching girder and pier brackets in service

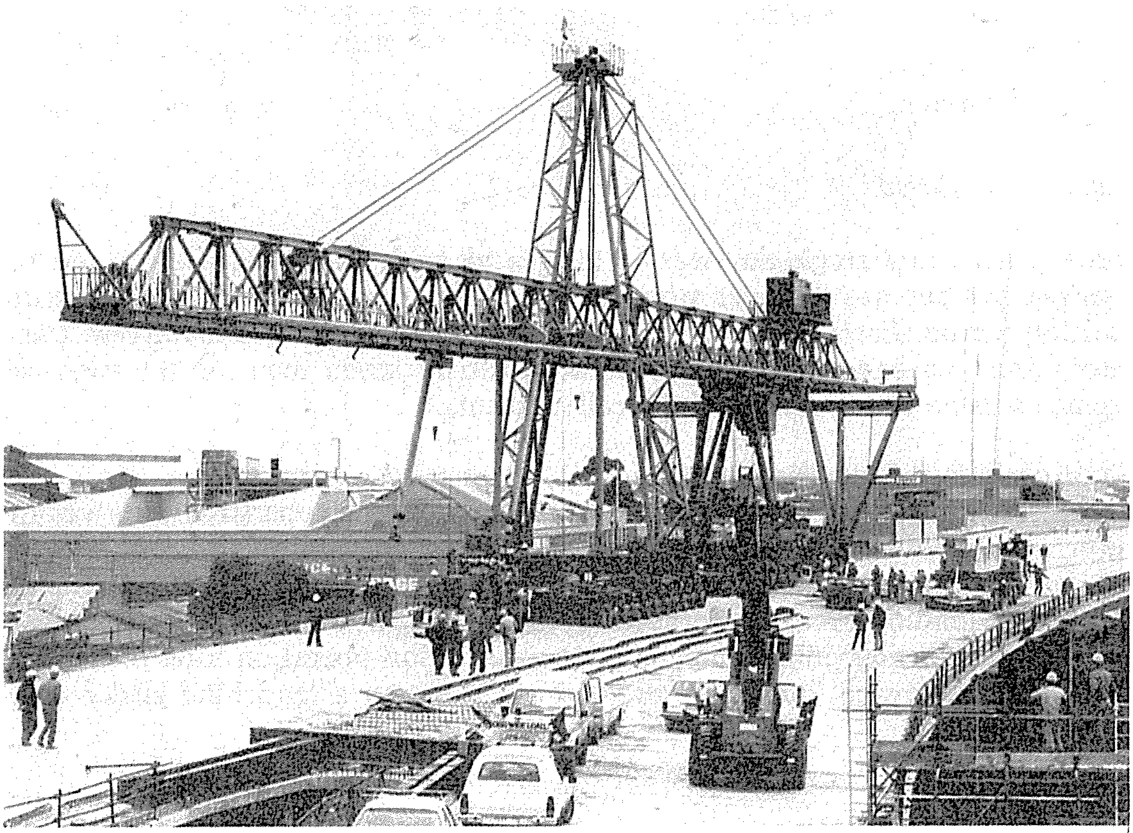


Fig. 29. Launching girder returning to the west end

They performed three main functions

(*a*) to carry the out-of-balance construction loads imposed during the cantilever erection of the segments before the installation of the launching girder stabilizing arms
(*b*) to allow fine adjustment of the vertical and horizontal alignment of the first two or three segments, before the grouting in place of the bearings
(*c*) to provide a support platform for the front prop of the launching girder.

Figures 30 and 31 give schematic views in elevation and plan showing these functional arrangements. The pier brackets were also used to anchor the lateral restraint systems that resisted deck rotation and translation during cantilevering.

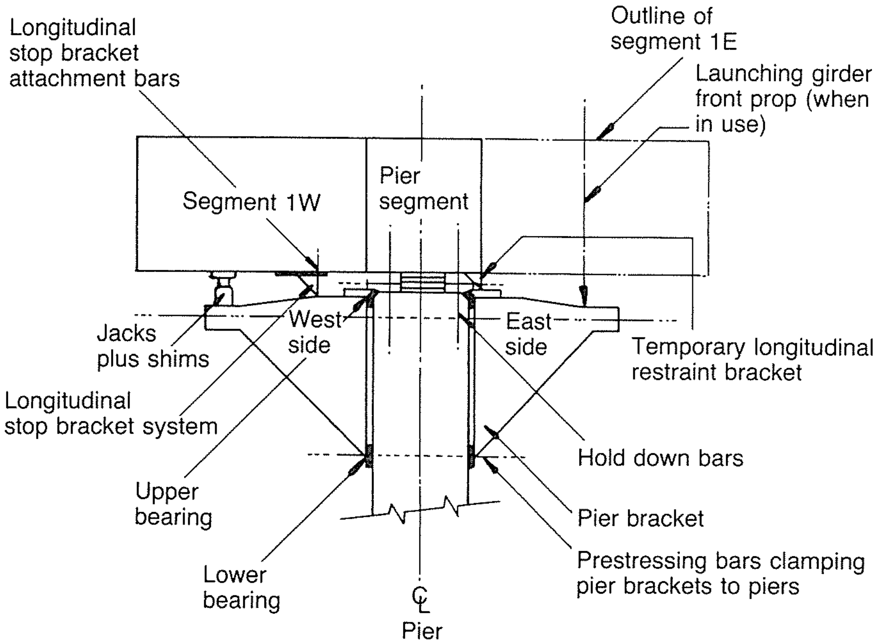


Fig. 30. Schematic elevation on pier brackets

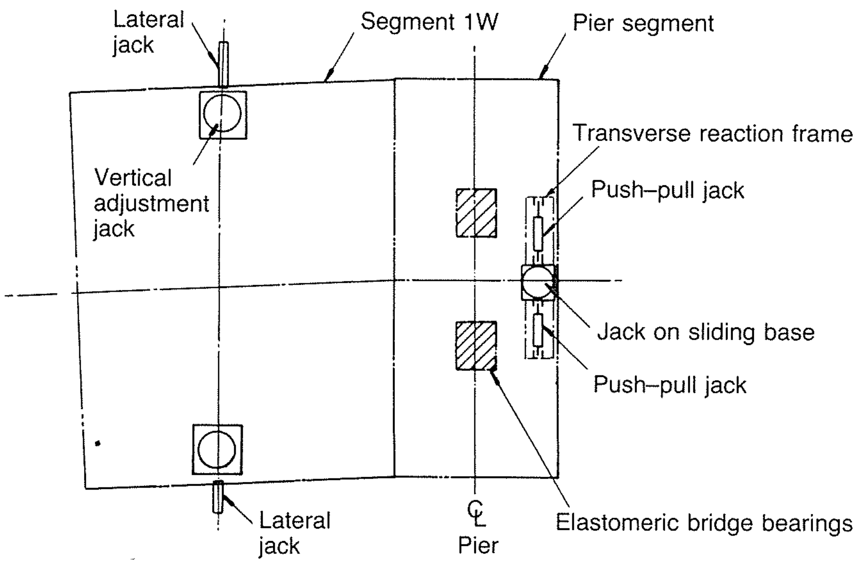


Fig. 31. Schematic plan view showing segment adjustment system on pier brackets

120. To avoid holes or recesses in the concrete, the pier brackets were designed to be clamped to the piers in two halves, using a combination of friction and direct support to resist vertical load. The clamping force was provided by 36 mm dia. prestressing bars (85/105 threadbars) which were stressed to not more than 50% UTS to permit re-use. Bars passed over the top and on both sides of the piers. The steel used in the pier brackets was mostly grade 350, but some grade 250 was also employed. Further information on the pier brackets is given by Shepherd.[9]

121. Total weights of the fully equipped pier brackets (two halves) were: single cell pier brackets = 30 t approximately; twin cell pier brackets = 40 t approximately. Fig. 32 shows a twin cell segment being placed by crane on the pier brackets.

122. In accordance with Quality Assurance criteria, one set of both the single cell segment and twin cell segment pier brackets were test loaded to 1·4 times their design load: Single cell pier brackets = 1600 kN/jacking point (×4); Twin cell pier brackets = 2800 kN/jacking point (×4). The loads were applied at the four jacking points on each set of pier brackets by jacking against ground anchors. Under sustained test load, neither the pier brackets nor the pier concrete showed any sign of distress or any significant deflexion.

Precast parapets

123. The finished line of the superstructure was defined precisely by the use of precast parapet units. As can be seen in Fig. 33, the parapet units overhang the

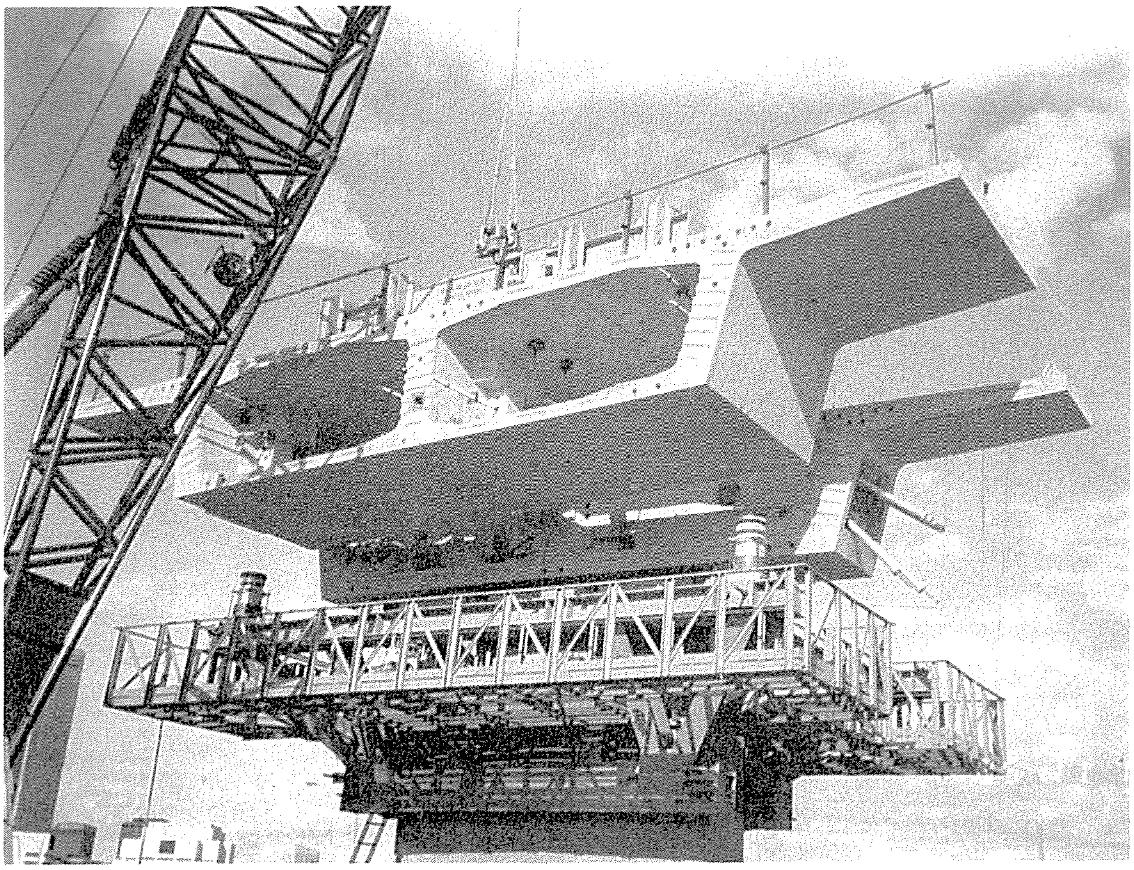


Fig. 32. Crane placing a twin cell segment on pier brackets

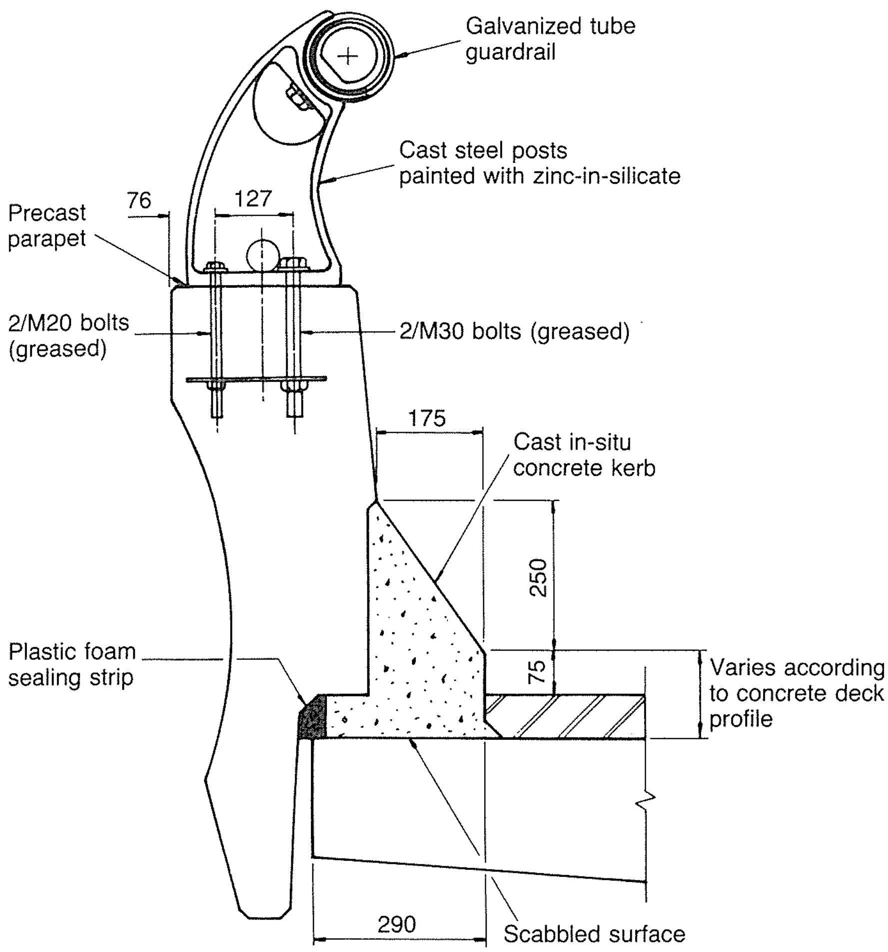


Fig. 33. Precast parapet detail

deck cantilever edges to provide a smooth fascia and drip detail. The parapet line was set to match the theoretical finished surface profile of the asphalt deck surfacing on the bridge.

Deck expansion joints

124. Finger joints of the Freyssinet FT type or similar were chosen as fulfilling all major design criteria as well as providing good riding quality. Joint sizes varied from FT 150 (150 mm maximum movement range) at one abutment to FT 400 (400 mm maximum movement range) at expansion joint segment locations. The joint type selected by the contractor was a VSL finger joint of similar appearance to the standard FT joint. The joints were fixed by means of welded stud bolts to galvanized steel sections cast into the expansion joint segments (Fig. 17).

125. Half of the joints in the north carriageway were installed after the placing of the asphalt deck surfacing, whereas the remainder plus the south carriageway joints were placed before asphalting of the deck. It is interesting to note that the joints placed after asphalting gave a better riding quality than those placed before asphalting.

Construction contract

126. A contract to construct the bridges (excluding the foundations) was awarded to a joint venture of Citra Constructions Limited and its French parent company, Spie Batignolles, in September 1983.

Fig. 34. Completed West Gate Freeway project on south carriageway opening day, 29 September 1988

127. A total of 1078 segments was required for the north structure, and these were cast in exactly 18 calendar months (including one Christmas summer holiday period). These segments were erected in 21 calendar months (including two Christmas summer holiday periods). The north carriageway was opened to traffic in November 1987.

128. Casting of segments for the south carriageway was completed in December 1987. Erection began in April 1987 and was completed on 2 June 1988. The bridge was opened to traffic on 29 September 1988—six months ahead of schedule. Fig. 34 shows the completed structures on the south carriageway opening day in 1988.

Project cost

129. The total value of the contract for the substructure and superstructures was AUS$76 million.

130. The total cost of the bridges, including foundations, was approximately AUS$96 million, which gives a unit rate of AUS$1250/m^2 of deck area.

131. The total cost of the West Gate Freeway project—including land acquisition, at-grade roadworks, a freeway overpass (Ingles Street Bridge) and other associated works—was AUS$175 million.

Project characteristics and material quantities

132. General characteristics of the structures and quantities of principal structural materials are summarized below.

Overall length of elevated structures	1·85 km
Clearances over streets	4·6 m minimum 10·5 m maximum
Maximum crossfall	6%
Maximum longitudinal grade	5·4%
Design speeds:	
Kingsway exit ramp ($R = 113{\cdot}5$ m)	65 km/h
Kingsway entry ramp ($R = 103{\cdot}5$ m)	60 km/h
main structures ($R = 650$ m)	100 km/h
Deck area	77 000 m^2
Concrete quantities:	
foundations	21 000 m^3
substructures	11 000 m^3
superstructures	41 000 m^3
Reinforcing steel	11 000 t
Prestressing steel	2500 t

Conclusions

133. Match cast segmental construction enabled the West Gate Freeway construction to proceed with minimal disturbance to roads, railways and tramways in a heavily trafficked city environment.

134. A Quality Assurance based approach ensured that planning, design and construction objectives were fully respected. The excellent riding quality of the finished roadways demonstrates the high degree of control of geometry and structural behaviour that can be achieved.

135. Refinements and innovations of detail in the application of segmental technology meant that potential difficulties were eliminated before they could occur.

136. An essential conclusion to be drawn from this project concerns not only the technical features but also the human relations side of any major civil engineering works. The success and quality of the West Gate Freeway project was due primarily to the excellent communication which existed between all parties involved.

Credits

137. *Design*

Roads Corporation of Victoria-VIC ROADS (formerly Road Construction Authority of Victoria) Bridge Design Group

Design of launching girders, pier brackets, precasting cells, design for construction and technical advice during construction

Europe Etudes Gecti/Corcoran Shepherd Consultants Pty Ltd.

Proof Engineers

Maunsell & Partners Pty Ltd

Merz and McLellan and Partners

Contractors

Joint venture between Citra Constructions Limited and Spie Batignolles

Major sub-contractors

Prestressing: VSL (Australia) Ltd
Reinforcing steel: Humes ARC Pty Ltd
Concrete supply: PBM Pty Ltd
Manufacture of launching girders: J F Thompson Pty Ltd and R & W Hannaford Pty Ltd
Manufacture of precasting cells: GFC Industries Pty Ltd
Manufacture of pier brackets: LJC Pty Ltd
Supply of prestressing strand: BHP Steel International Group (Wire Products Division)

Client and Supervision

Roads Corporation of Victoria—VIC ROADS (formerly Road Construction Authority of Victoria).

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138. The Authors wish to acknowledge the essential contribution of all technical and design staff, construction personnel and support groups within the Roads Corporation, without whose sustained commitment over many years the project could not have succeeded. The efforts of Mr B. Addis, Mr K. Opie, Mr R. Valentine and Mr J. Wilson are especially noted in steering the project through the difficult concept and development phases, integrating the complex project issues and providing the necessary leadership to all involved. The co-operation, competence and commitment of all contractors working on the project are also acknowledged. Special thanks to Mr B. Houghton and his staff and to Ms E. Fazlic for their perseverance and co-operation with illustrations and text. The Papers are presented with the permission of Mr R. Patterson, Chief Executive of the Roads Corporation of Victoria. The views expressed are those of the Authors and do not necessarily reflect those of the Roads Corporation.

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