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Hong Kong Mass Transit Railway Modified Initial System: design and construction of the driven tunnels and the immersed tube

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The first part of this Paper describes the design requirements and construction methods adopted for the construction of the driven tunnels for the Modified Initial System of the Mass Transit Railway. The ground conditions and methods adopted for driving the tunnels through a wide range of soil and rock types are reviewed. Particular problems encountered on the seven contracts involved are described, together with the solutions evolved by the individual contractors. The second part of the Paper reviews the design principles and construction methods for the immersed tube across the harbour. The governing local conditions and other criteria are outlined and a description is given of the design and its special features. The construction method and sequence of construction are also described.

Design and construction of the driven tunnels

Information issued to tenderers

In each of the seven contracts which involved driven tunnels tenderers were provided with borehole information together with details of the assumptions made by the Engineer in preparing his outline design. This included assumed positions of soft ground/rock interfaces, the approximate locations where marine soils would be encountered and the need for compressed air and/or ground treatment.

2. The tenderers were not at liberty to vary the alignment, which provided

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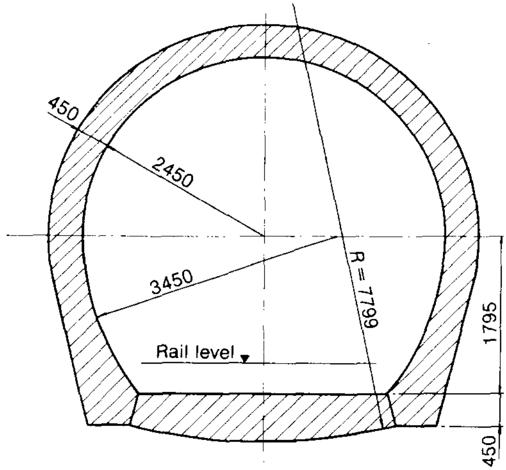

Fig. 1. In situ concrete lining, 4·9 m i.d. (dimensions in millimetres)

ground cover of 8–25 m for soft ground tunnels and up to 50 m for tunnels through rock. It had been established in driving trial tunnels[1] that the stability of the tunnel faces in soft ground or weathered rock had to be controlled by ground treatment, groundwater lowering or compressed air; in the unweathered rock sections water inflow was likely to be minimal.

3. Lining types were selected in accordance with ground conditions, and the Engineer's proposals comprised three types for the running tunnels with special sections for enlargements.

4. Plain concrete lining, cast in situ, was proposed for the rock sections and segmental lining in soft ground. The segmental lining was precast concrete in homogeneous ground and spheroidal graphite iron (SGI) in mixed conditions or where tunnels were in close vertical and/or horizontal proximity. It was envisaged that segmental linings would be caulked with asbestos cement or lead to achieve the specified degree of watertightness.

5. Construction methods envisaged by the Engineer were directly related to the expected ground conditions and hence lining type. All drives were relatively short and ground conditions variable, hence full face mechanical excavation was not considered viable. In rock, drill and blast would be the normal method; in soft ground, hand methods or manual shields were indicated.

Design and construction methods

6. Contractors' designs and construction methods generally followed those envisaged by the Engineer. Hong Kong's geology is such that at tunnel level, variable conditions of rock and soft ground, and mixed soft ground conditions, were expected throughout the route. For this reason methods adopted by the contractors differed at some locations from those originally proposed by the Engineer. The major difference was the use of in situ lining in soft ground and of large rectangular access shafts; the contractor's country of origin and his usual method also affected choice.

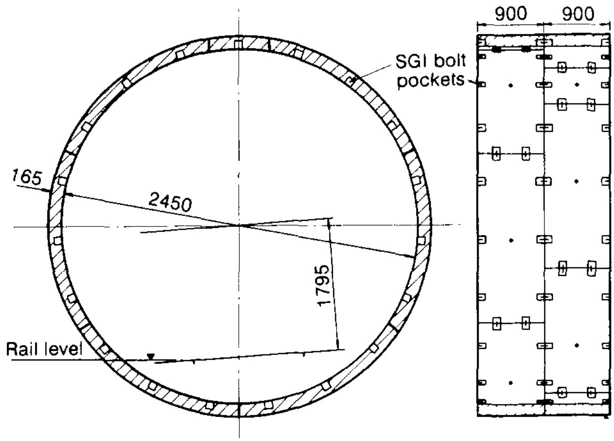

Fig. 2. Precast concrete segmental lining, 4·9 m i.d. (dimensions in millimetres)

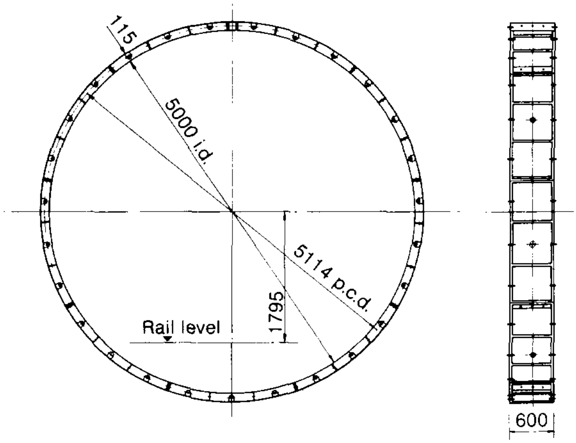

Fig. 3. SGI segmental lining, 5·0 m i.d., width 600 mm (dimensions in millimetres)

7. Lining designs for rock tunnels were similar to those proposed by the Engineer, being 450 mm thick plain in situ concrete of horseshoe section (Fig. 1). The design of in situ lining in soft ground was similar. Where short lengths of rock occurred within a soft ground drive the segmental lining was carried through and where a shield was installed this was taken through the rock section on skids after excavation.

8. Precast concrete segmental linings were used on three contracts. The segments were all of the smooth bore type but there were considerable variations in details. SGI bolt pockets were used on contract 109 (Fig. 2) and curved long bolts on contract 201, while on contract 107 no circumferential bolts were used for rings

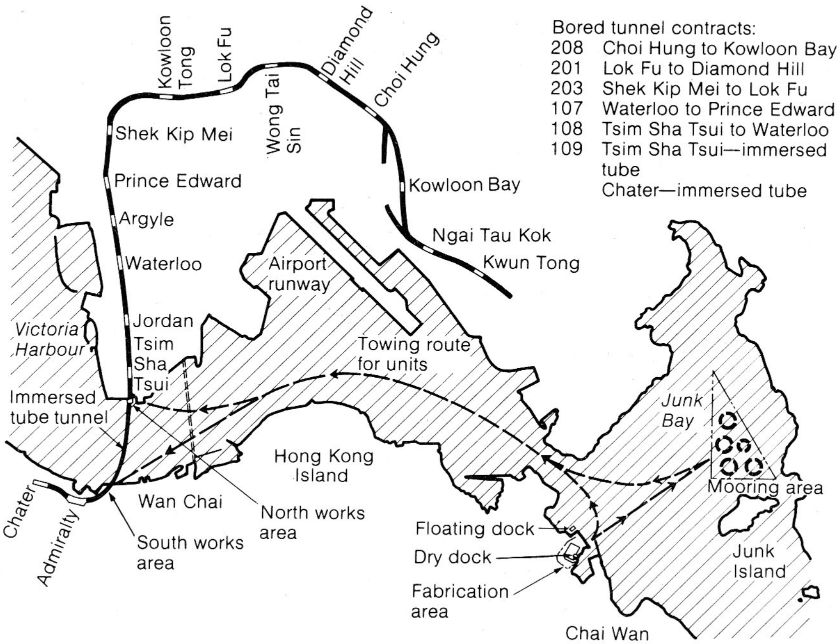

Fig. 4. Location of contracts and site areas

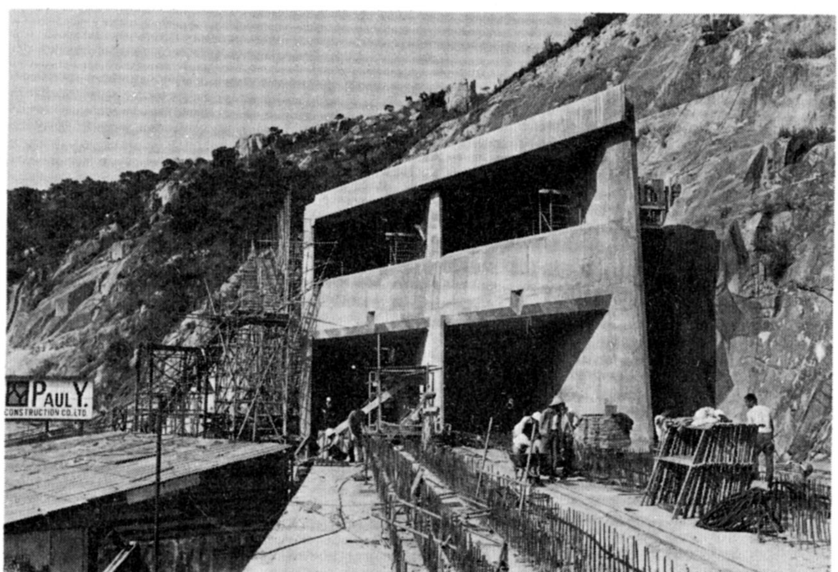

Fig. 5. Kwun Tong portal nearing completion

erected behind shields but longitudinal tie bolts were installed where the lining was hand-built. Segment widths varied from 600 mm to 1000 mm and thicknesses from 165 mm to 320 mm. Progress rates were variable but this was more related to ground conditions and access than lining type. In homogeneous conditions of weathered granite, maximum rates were achieved. The presence of core stones or marine deposits was generally a retarding factor.

9. Two types of SGI lining were used for 5·0 m dia. running tunnels. Rings of width 600 mm (Fig. 3) were used on contract 107 and 900 mm wide rings on contract 109. The latter, with a skin thickness of 12 mm, had marginally less material weight than the Engineer's proposed lining. The designs included a 3 mm allowance for corrosion on the skin thickness.

10. Methods of ground treatment fell into two categories: the sleeved tube system, which allowed multi-stage grouting; and the single-stage nozzle injector system, where the grout, normally a rapidly setting sodium silicate gel, was injected through a rotating nozzle within a casing as the equipment was removed from the drill hole. The latter system generally resulted in ground fracturing, whereas fairly uniform penetration was achieved by the sleeved tube method.

11. Ground treatment was carried out at a considerable number of locations, including station connections in soft ground, structures sensitive to differential settlement, and where soft ground excavation in free air was necessary.

12. Compressed air working was extensive. The Hong Kong compressed air regulations incorporate the recommendations of CIRIA report 44 and were established following driving of the trial tunnels in 1975.[1] Contractors were required to provide sufficient compressed air capacity to deliver 5 m^3/min of air per square metre of face area, with an additional 50% of this capacity as standby using an alternative power source. Some contracts included a number of drives and this enabled contractors, when experiencing retarded progress rates on difficult mixed faces, to open other drives. Extra resources in the form of increased labour and plant were provided to do this.

13. Shaft sinking methods were numerous, ranging from hand-dug caisson walls to segmental circular underpinning in treated ground.

14. Hong Kong had no pool of trained tunnel miners, and contractors had to rely on overseas labour to train the local labour.

15. Figure 4 shows the location of the seven bored tunnel contracts. The descriptions below are not exhaustive but attempt to highlight some of the problems and the methods by which they were overcome.

Contract 208—Kwun Tong Road Portal to Choi Hung

16. The tunnels of contract 208 were driven through fresh granite which varied to slightly weathered in some areas. Adjacent to Choi Hung station a short section was driven through completely weathered granite (CWG). The rock tunnels were excavated by drill and blast methods and the soft ground section was excavated by hand. The CWG was harder than expected and some explosives were used. SGI segmental lining was erected at this location, whereas an in situ lining was used for the tunnels in rock.

17. The drives at Kwun Tong Road entered directly into rock through a skew portal (Fig. 5). The rock face was stabilized with rock bolts, shotcrete and mesh. The portal structure, which houses a ventilation building and forms the abutment to the adjoining overhead section, was constructed of reinforced concrete on completion of tunnelling.

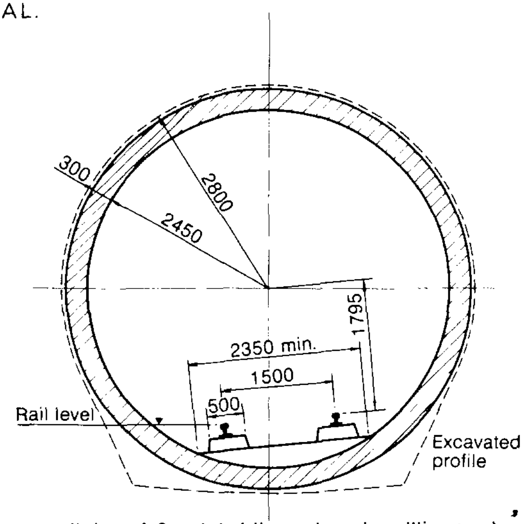

Fig. 6. In situ concrete lining, 4·9 m i.d. (dimensions in millimetres)

Contract 201—Wong Tai Sin to Diamond Hill

18. Tunnelling between Wong Tai Sin and Diamond Hill stations, a distance of 560 m, was carried out in free air using groundwater lowering methods. Wells were sunk at 10 m centres in three rows alongside and between the tunnels to draw the water table down by about 21 m. Tunnels through weathered granite were then excavated using arch ribs, mesh and shotcrete as temporary support. A reinforced circular concrete lining 300 mm thick (Fig. 6) was then cast in situ. A recently completed flyover, not open to traffic, crossed the line of the tunnels. This two-span structure was founded on rock at the centre pier and at one abutment, and on friction piles at the other abutment (Fig. 7). Differential settlement was expected due to groundwater lowering. Ground treatment was carried out to reduce the effect of settlement at the friction piles. In the event, settlement did occur at this abutment and also at the pier and abutment assumed to have been founded on rock. The pier resting on the rock pillar between the tunnels was safeguarded by installation of rock anchors between the tunnels. The bridge deck was finally jacked and the bearings were reseated to counter the differential settlement and ensure that the bearings were evenly loaded.

Contract 201—Lok Fu to Wong Tai Sin

19. Twin tunnels in the section from Lok Fu to Wong Tai Sin were driven consecutively by a Bade shield (Fig. 8). This was an entirely soft ground section through fill material, colluvium and CWG. The initial drive of 86 m was in free air with the aid of groundwater lowering by deep wells. Locks were then installed and the remainder of the 615 m drive was completed under compressed air with a maximum pressure of 1·8 bar.

20. A segmental precast concrete lining of bolted design, 5·1 m i.d. and 320 mm thick, with neoprene sealing strips, was erected behind the shield and grouted using a bentonite–cement–sand mix. Very little caulking was required to produce the specified degree of watertightness.

21. Cement–bentonite ground treatment was carried out under the most

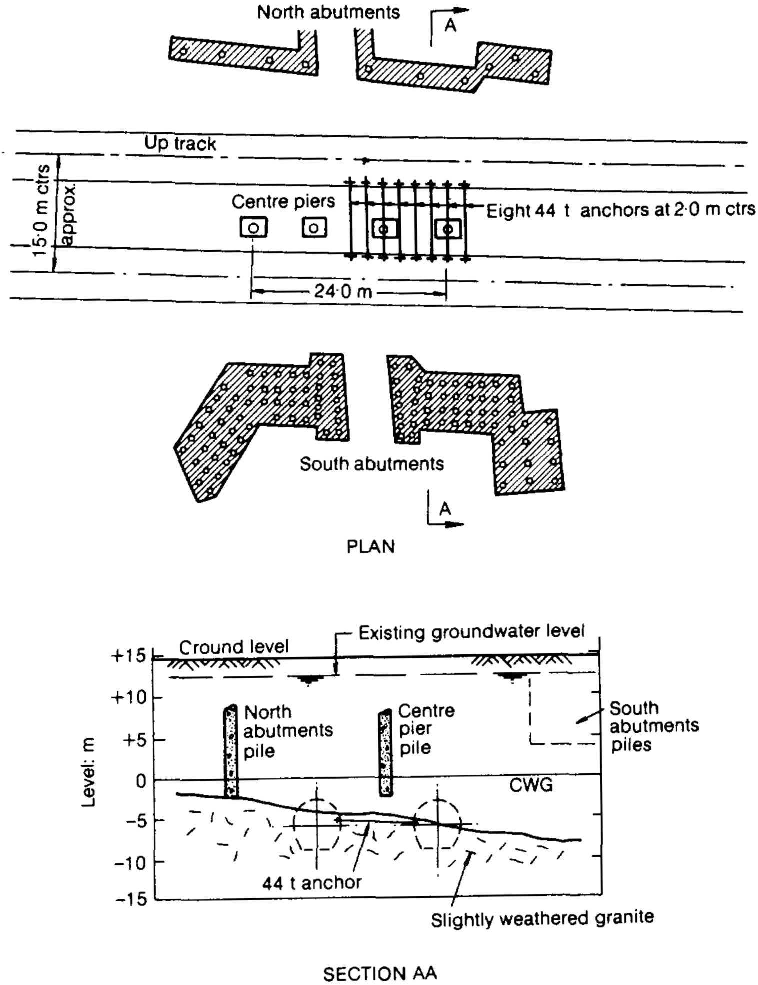

Fig. 7. Flyover and tunnel layout at Po Kong interchange

sensitive structures. However, settlement did occur, ranging from 117 mm at a swimming pool to 63 mm at a resettlement housing block. At another resettlement block, where no treatment was used, settlement was 129 mm. Angular distortion, however, remained within acceptable limits.

Contract 203—Kowloon Tong to Lok Fu

22. The tunnels to Lok Fu were in rock and were driven without incident. An access shaft was sunk in open ground approximately midway between the stations.

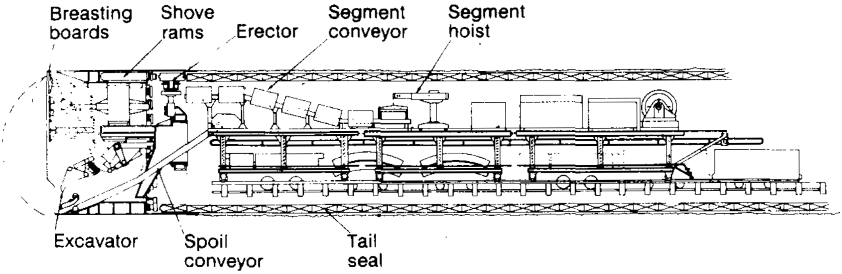

Fig. 8. Bade shield (5·92 m) and shield train

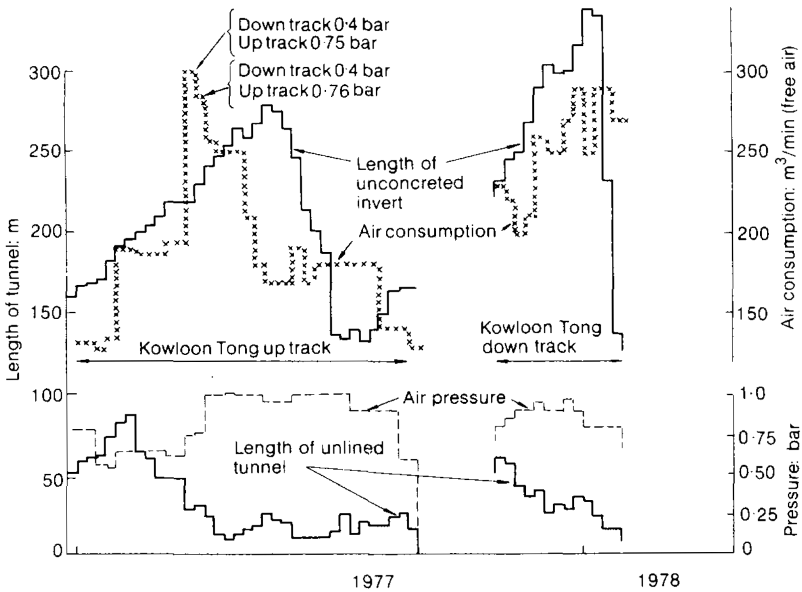

Fig. 9. Kowloon Tong to Lok Fu: comparison of air consumption with air pressure and exposed tunnel lengths

The 450 m long tunnels to Kowloon Tong were driven under compressed air. Steel ribs and steel lagging were used as temporary support prior to the casting of a 420 mm thick plain in situ concrete lining, which was the form of permanent lining used throughout this contract. This method led to a large area of exposed ground compared with the Engineer's assumption that shields and segmental lining would be used. Fairly high air consumption resulted (Fig. 9).

23. During the initial stages of the up track drive it was not possible to raise the air pressure above 0·65 bar as the 150 mm air supply pipe had reached its maximum capacity of 197 m^3/min. At this stage balancing pressure was occurring at 1 m below

axis, resulting in slurry conditions in the invert and consequent settlement of the arch ribs. To proceed further it was decided to divert a second 150 mm air supply pipe from the adjacent down track tunnel which had not yet been pressurized. The air consumption rose to 300 m^3/min and it was decided to halt the down track tunnelling operations. The clearance of some 9 m between the up and down tunnels allowed air to escape from one tunnel to the other. The arch lining and invert concrete were then brought to within 40 m of the face in the down track tunnel, and in the up track the arch lining reached within 25 m of the face, while the invert concrete remained 215 m behind. On resumption of tunnelling in the up track alone, a pressure of 1·0 bar was maintained, with air consumption being reduced initially to 255 m^3/min, and subsequently to 180 m^3/min.

24. Settlements directly above the tunnels ranged from 19 mm to 190 mm. Chemical treatment was again carried out in sensitive areas, which made a contribution towards reducing total settlement. The largest settlement resulted from dewatering due to excessive air losses.

Contract 203—Shek Kip Mei to Kowloon Tong

25. Tunnelling between Shek Kip Mei and Kowloon Tong was carried out from a shaft sunk in open ground approximately midway between the stations. Towards Shek Kip Mei, ground conditions were better than anticipated and the compressed air and ground treatment envisaged were not required.

26. Adjacent to Kowloon Tong, alluvium appeared in the top half of the tunnel, which consisted of irregular layers of clays, gravels, sandy silts, sands and organic matter. Groundwater lowering was carried out in this area. Where the tunnels passed under the Kowloon Canton Railway with only 8 m ground cover, chemical ground treatment was used. The tracks subsided by 150 mm but they were packed up continually to ensure uninterrupted running of the railway, albeit with a speed restriction.

Contract 107—North Nathan Road tunnels

27. Contract 107 comprised all the tunnelling work between the three stations constructed under contract 101. It was divided into two sections, Waterloo to Argyle and Argyle to Prince Edward.

28. Between Waterloo and Argyle, there are four running tunnels and two crossover tunnels, both at the lower level, while from Argyle to Prince Edward there are four running tunnels, reducing to three north of the high level crossover tunnel; all within a 25 m width.

29. Although the running tunnels generally are arranged as one pair above another this arrangement is constantly changing to provide interchange between the two routes, which bifurcate north of Prince Edward (Figs 10 and 11).

30. Some statistics relating to the work are listed in Tables 1 and 2.

31. The ground encountered varied considerably, with the predominant material being CWG containing a varying and occasionally high proportion of fresh granite core stones. Other materials were marine deposits and from moderately weathered to fresh granite. The water table was generally 2 m below ground level.

32. All the tunnels were driven in compressed air except for short lengths at the northern end of the contract where fresh granite was met.

33. The running tunnels in the Waterloo to Argyle section were mainly shield-driven. Progress was particularly slow in mixed face conditions where core stones

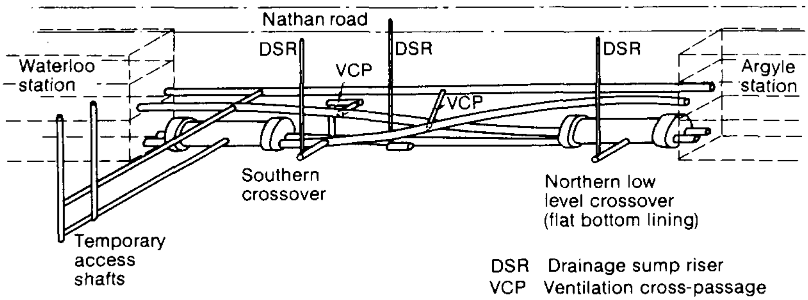

Fig. 10. North Nathan Road tunnels: Waterloo to Argyle

Table 1. Contract 107: general statistics

Total excavation	83 000 m^3
Total length of 4·9/5·0 m dia. running tunnels	2850 m
Total length of 10·0/11·6 m crossover tunnels	240 m
Maximum depth to invert	31 m
Minimum cover	8 m
Total tunnel linings (SGI)	13 405 t
Explosives used	14 t
Tunnels driven by hand	58%
Tunnels driven by shield (three)	42%
Maximum number of faces worked concurrently	16
Labour:	
Local labour (at peak)	1000
Expatriate labour	91

Table 2. Contract 107: compressed air statistics and construction dates

	Waterloo to Argyle	Argyle to Prince Edward
Compressed air statistics (low pressure)		
Installed capacity, m^3/min	842	365
Installed horsepower	4290	2180
Standby diesel generator capacity, kVA	1875	625
Working pressure, bar	0·5–2·5	0·5–2·5
Tunnel temperatures, °C:		
average	27	27
maximum	32	32
Construction dates		
Commenced sinking access shafts	August 1976	December 1976
Last tunnel ring built	July 1979	November 1978
Substantial completion	December 1979	July 1979

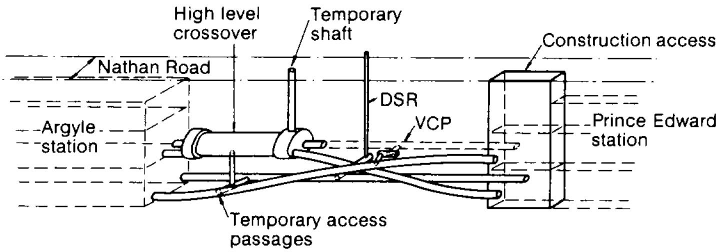

Fig. 11. North Nathan Road tunnels: Argyle to Prince Edward

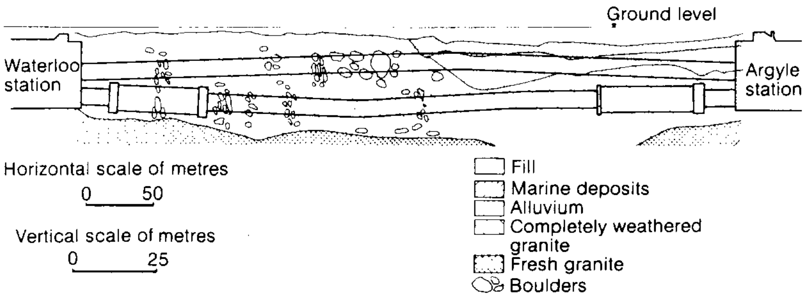

Fig. 12. North Nathan Road: geological section

were frequent, and achieved a mean rate of 11 rings per week compared to 35 rings per week in a full face of CWG. Fig. 12 shows the longitudinal ground profile in this section.

34. Coarse marine sand of uniform particle size and up to 2 m thick was encountered in the crown of the Tsuen Wan up drive near Argyle station. This resulted in doubled air consumption and very difficult tunnelling conditions. A 50 m length was chemically treated from the surface through stuffing boxes because of the compressed air. A cross-section through the northern crossover is shown in Fig. 13.

35. The Prince Edward to Argyle tunnels were driven without shields through fresh granite and CWG with isolated boulders.

36. Chemical ground treatment was carried out at the high-level crossover location where there was inadequate ground cover for normal compressed air working. The treatment carried out from the surface provided a thick annulus of consolidated and virtually impermeable ground around the crossover position; the area within the annulus was also treated but at a reduced injection rate.

37. The success of the ground treatment was such that the 5 m dia. pilot was driven in free air. Compressed air would not have been needed for the enlargement

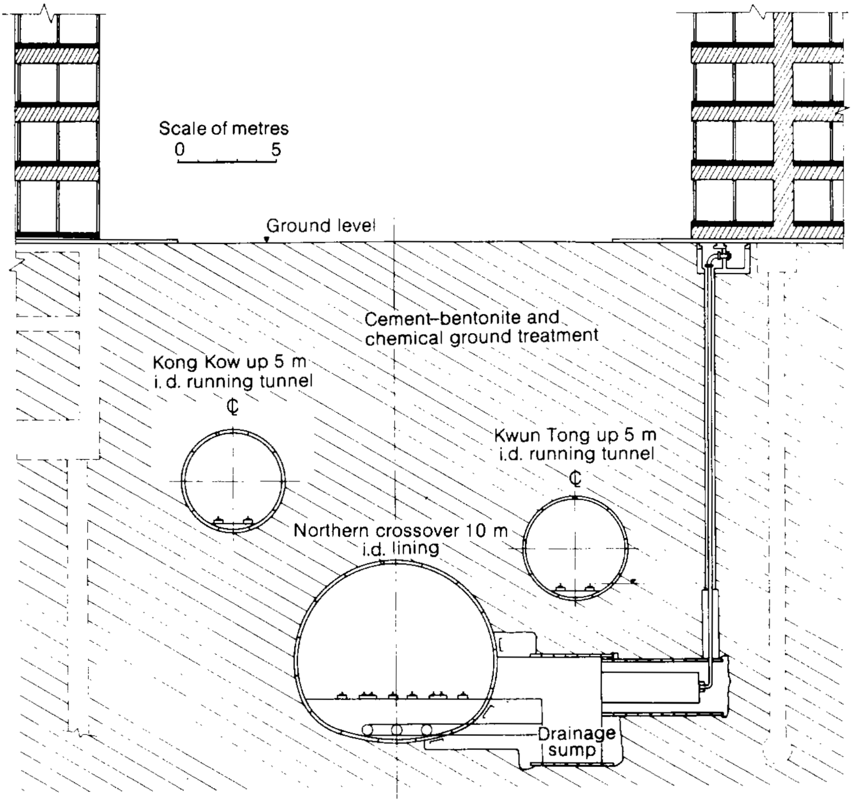

Fig. 13. Nathan Road: cross-section through northern low-level crossover

except that the running tunnels beneath were pressurized to over 2 bars at that time and it was considered prudent to use a nominal pressure of 0·7 bar in the enlargement.

38. Two compressed air installations were required, one at Hamilton Street for Waterloo to Argyle and one at Playing Field Road for Argyle to Prince Edward. Provision was made for additional compressors or interchanges of machines if necessary.

39. The atmospheric conditions in Hong Kong are of moderately high temperatures and high to very high relative humidities. In addition to the normal equipment installed it was necessary to introduce chiller-dryers to cool and dry the low pressure compressed air to maintain a tunnel wet bulb temperature not exceeding 27°C.

40. During the first half of the tunnel construction the temperature was satisfactory, but as the volume of air supplied became a smaller proportion of the whole tunnel volume, it was necessary to introduce spot coolers at certain tunnel

Fig. 14. Completed crossover tunnel showing elliptical lining

faces. These were supplied with chilled water from additional coolers installed at the surface.

41. It was planned to work two running tunnel faces and one crossover face in each section concurrently; the high-level crossover to be in free air. Serious delays arose due to the difficult ground conditions and the need to train labour on site. It therefore became necessary to adopt accelerative measures requiring more gangs.

42. At one time 16 faces were being worked concurrently with more than 700 men working in compressed air. The shifts had to be staggered so that three full decompressions could be run in the man locks in each 8 h shift or cycle.

43. Particular attention was paid to the training and supervision of lock attendants and the provision of full site medical facilities. The success of this is illustrated in Tables 3 and 4, which show the incidence of decompression sickness at all working pressures above 1 bar.

44. All linings were manufactured in the UK. The design intention was that, with the exception of one high-level running tunnel where precast concrete was to be

Table 3. Contract 107: incidence of decompression sickness

Pressure, lbf/sq. in.*	Pressure, bar	European			Chinese			Total		
		Exposures	Bends	Bends rate, %	Exposures	Bends	Bends rate, %	Exposures	Bends	Bends rate, %
14–16	0·95–1·09	3 244	1	0·031	20 697	3	0·014	23 941	4	0·017
16–18	1·09–1·22	2 859	1	0·035	15 081	2	0·013	17 940	3	0·017
18–20	1·22–1·36	812	5	0·616	6 209	6	0·097	7 021	11	0·157
20–22	1·36–1·50	1 121	4	0·357	7 722	21	0·272	8 843	15	0·283
22–24	1·50–1·63	1 546	6	0·388	9 933	24	0·242	11 479	30	0·261
24–26	1·63–1·76	1 470	12	0·816	6 999	27	0·386	8 469	39	0·460
26–28	1·76–1·90	1 275	15	1·176	9 856	74	0·751	11 131	89	0·800
28–30	1·90–2·04	3 695	31	0·839	25 327	211	0·833	29 022	242	0 834
30–32	2·04–2·18	10 097	174	1·723	79 289	673	0·849	89 386	847	0·948
32–34	2·18–2·31	1 516	24	1·583	13 888	178	1·282	15 404	202	1·311
34–36	2·31–2·45	279	–	–	1 121	19	1·695	1 400	19	1·357
36–38	2·45–2·58	390	1	0·256	2 128	26	1·222	2 518	27	1·072
14–38	0·95–2·58	28 304	274	0·968	198 250	1264	0·638	226 554	1538	0·679

* Tables in the Hong Kong regulations are in imperial units. Contract 107 used gauges in imperial units. All other contracts used metric pressure gauges.

Table 4. Contract 109: incidence of decompression sickness; pressure exceeding 1 bar but not exceeding 2 bar

	Exposures	Bends	Bends rate, %
European	3 300	3	0·090
Chinese	24 422	164	0·671
Japanese	9 956	82	0·823
Korean	41 970	68	0·162
Total	79 674	317	0·398

used, all permanent linings were to be of SGI. All were designed to carry full overburden including water, building and traffic surcharges and earthquake loading.

45. The contractor was concerned at the proposed use of full-circular-profile linings for the 10·0 m and 11·6 m dia. crossover tunnels and at time of tender had proposed the use of a shallow invert or flat-bottomed profile. The reasons for this concern were the height of the face to be supported plus the high excess air pressure at the top of the face; the high air pressure required when that pressure has to balance the hydrostatic head at the invert; the high air consumption with such a large face area; and the limited cover over the high-level crossover. Further consideration of these factors after the award of contract led the contractor to choose a near-elliptical lining for the high-level and south low-level crossovers, which did not interfere with any permanent installations. The final design is shown in Fig. 14. It was based on the Muir Wood method[2] of predicting deflexions of tunnel linings in elastic ground.

46. Alternate rings were arranged so as to stagger the longitudinal joints. One design requirement was that the crown deflexion should not exceed 34 mm, and the maximum recorded was 12 mm with an average of about 6 mm.

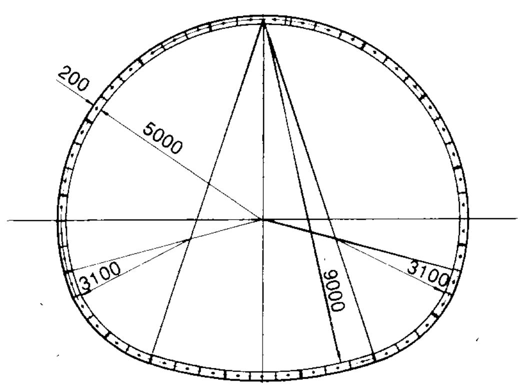

Fig. 15. 10·0 m flat-bottomed SGI lining (dimensions in millimetres)

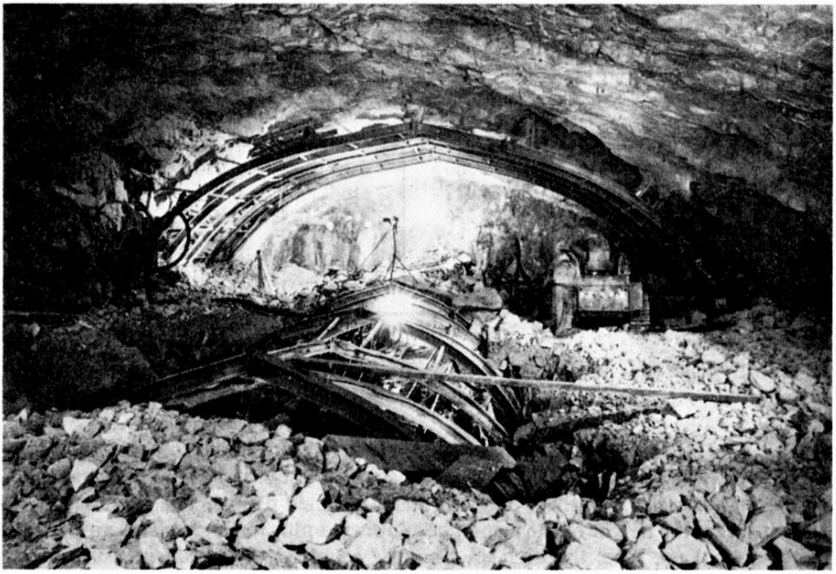

Fig. 16. Crown heading in Jordan/Tsim Sha Tsui crossover

Fig. 17. Crossover tunnel

47. At the location of the north low-level crossover the ground was found to have a very high voids ratio and consequently low soil modulus of elasticity. The lining deflexions would have been excessive and a more conventional flat-bottomed lining was designed for this section (Fig. 15).

Contract 108

48. Contract 108 consisted of twin running tunnels with a connecting crossover tunnel between Jordan and Tsim Sha Tsui, and twin running tunnels between Jordan and Waterloo stations. Also in the contract were twin siding tunnels running south from Waterloo station at a lower level.

49. More than half the tunnelling was in rock. The contractor elected to construct all the tunnels using rock tunnelling techniques.

50. Compressed air tunnelling was carried out adjacent to the stations where soft ground occurred. In all there were six compressed air working locations with two faces working at any one time.

51. The crossover tunnel, 90 m long and between 10 m and 11·5 m wide, was constructed by initially excavating a 4 m horseshoe pilot in the invert, which was used to transport spoil from the crown (Fig. 16). The rock was competent and it was possible to complete the whole of the upper excavation before concreting the arched roof. The lower excavation followed and support for the roof excavation was maintained while the walls were concreted in short lengths. The completed crossover is shown in Fig. 17.

52. Progress in the running tunnels was variable according to the situations that prevailed. The best weekly progress in a rock tunnel was 33 m, with 106 m in one month. In compressed air, which was always less than 1·0 bar, the best weekly and monthly advances were 24 m and 71 m respectively.

53. Ground treatment was carried out at eight locations over a total of length 333 m of running tunnel where mixed faces occurred. All the treatment consisted of sodium silicate with a hardener giving a setting time of less than 1 min.

54. The contractor relied heavily on experienced miners from Japan, a policy which proved to be successful. Face gangs consisted of experienced men supplemented by local operatives. During the busiest period over 300 men were employed on the contract.

Contract 109

55. Contract 109 comprised twin running tunnels from Tsim Sha Tsui station on the north side and Admiralty station on the south to the immersed tube and four tunnels between Admiralty and Chater stations. Two of these latter were running tunnels and two led to sidings below platform level at Chater station (Fig. 18).

56. The tunnels north of the immersed tube were driven for much of their length in a mixed face of soft ground and rock. Colluvium and marine deposits were evident in the tunnels for a short length. Fill and marine deposits were more frequent between Admiralty station and the immersed tube and a large section of the tunnelling was through decomposed granite with core stones. Similar mixed conditions were encountered on the Admiralty to Chater section (Fig. 19).

57. All tunnels were shield-driven in compressed air and/or with the aid of ground treatment and were started from three on-line shafts. The shaft in Kowloon was constructed using the PIP system and the one at Admiralty East used sheet piles. The shaft at Admiralty West (Fig. 20) used both of these techniques plus hand-dug caissons. All shafts were braced.

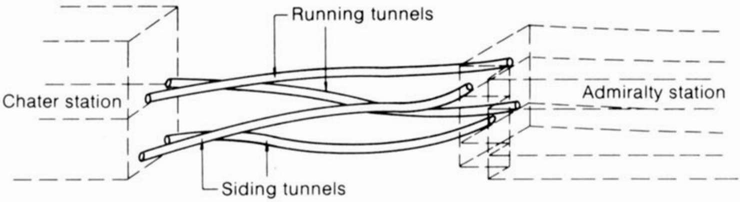

Fig. 18. Chater to Admiralty tunnels

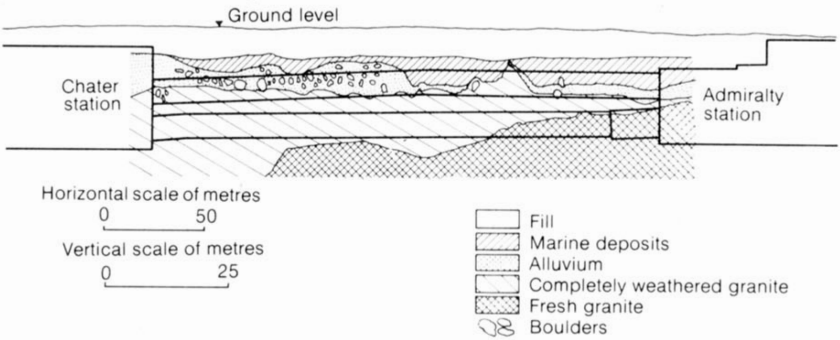

Fig. 19. Chater to Admiralty: geological section

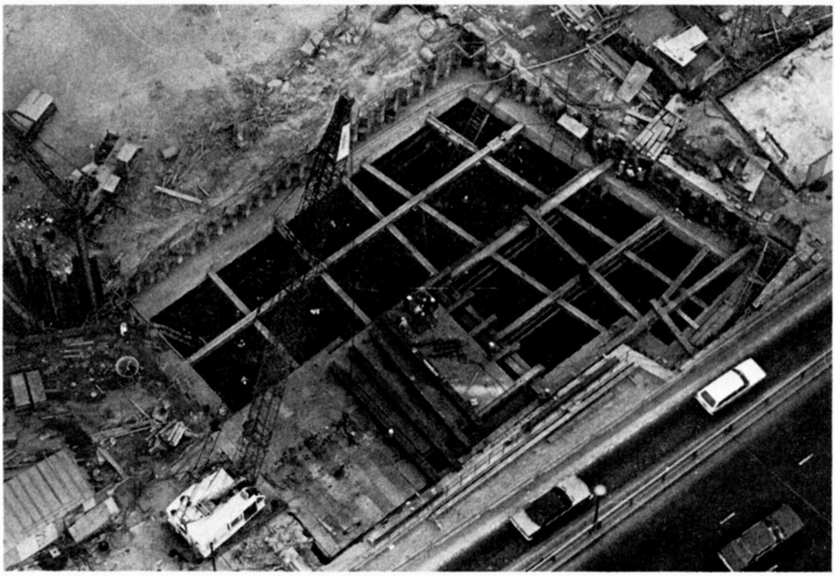

Fig. 20. Access shaft at Admiralty

58. Sixteen sets of compressed air plant were installed. All shield skins were abandoned at the end of their respective drives. The original intention was to provide six sets of equipment for the shields; however, due to slow progress early in the contract, a further two sets of equipment were ordered and used.

59. The contractor followed the Engineer's design implicitly in deciding where SGI and precast concrete linings were to be constructed.

60. Progress varied from 3·6 m/week in mixed rock/soft conditions to 28 m/week in a full face of decomposed granite.

61. Ground treatment using the sleeved tube method was carried out under the Cotton Tree Drive flyover, adjacent to the shaft west of Admiralty station, and at both connections to the immersed tube. Bentonite–cement followed by sodium silicate was injected at these locations. Special backfill consisting of bentonite–cement of strength 5 N/mm^2 had previously been placed within the dredged area around the ends of the immersed tube. The interfaces of the special backfill with the virgin ground were treated. Elsewhere and on a smaller scale ground treatment using only sodium silicate with a rapid-set gel was used. The characteristics of the ground using the second method were improved but not to the same extent as with the first.

62. Tunnel face gangs were predominantly Korean, along with some Chinese labour. During the period when six faces were advancing 360 men were employed on the contract.

Summary

63. This review serves to illustrate the range and variety of the contracts involved in constructing the bored tunnel sections of the railway. All the contracts were completed on or ahead of schedule. Delays against the programme which occurred during the construction period were made up by rephasing the work or increasing resources and by deciding upon and implementing alternative methods in the shortest possible time.

Design and construction of the immersed tube tunnel

General description

64. The railway tunnels where they cross Victoria Harbour from Wanchai on Hong Kong Island to Tsim Sha Tsui in Kowloon comprise 14 twin track precast concrete immersed tube units each 100 m long and weighing approximately 7800 t.

65. Clearances set by the Hong Kong Government for the harbour crossing required that the top of backfill should not exceed −12·5 m PD (Principal Datum), except within 120 m of the existing sea walls, where existing depths should not be reduced. This governed the vertical alignment of the tunnel. The alignment is curved in plan with a radius of 2800 m and the inverts at the two ends are at −19·6 m PD and −21·6 m PD, with the deepest point at −24·2 m PD. The horizontal curve is achieved by constructing each unit with the same curvature (Fig. 21), while the vertical curve is accommodated by making each unit straight but with the ends bevelled at a slight angle to the normal.

66. Twelve units are of a standard design and construction, whereas the two end units accommodate the ventilation buildings at each side of the harbour and the connections with the bored tunnels. The two ventilation buildings are similar in design and construction and are just over 40 m high.

67. The two tracks in the immersed tube are contained in separate compart-

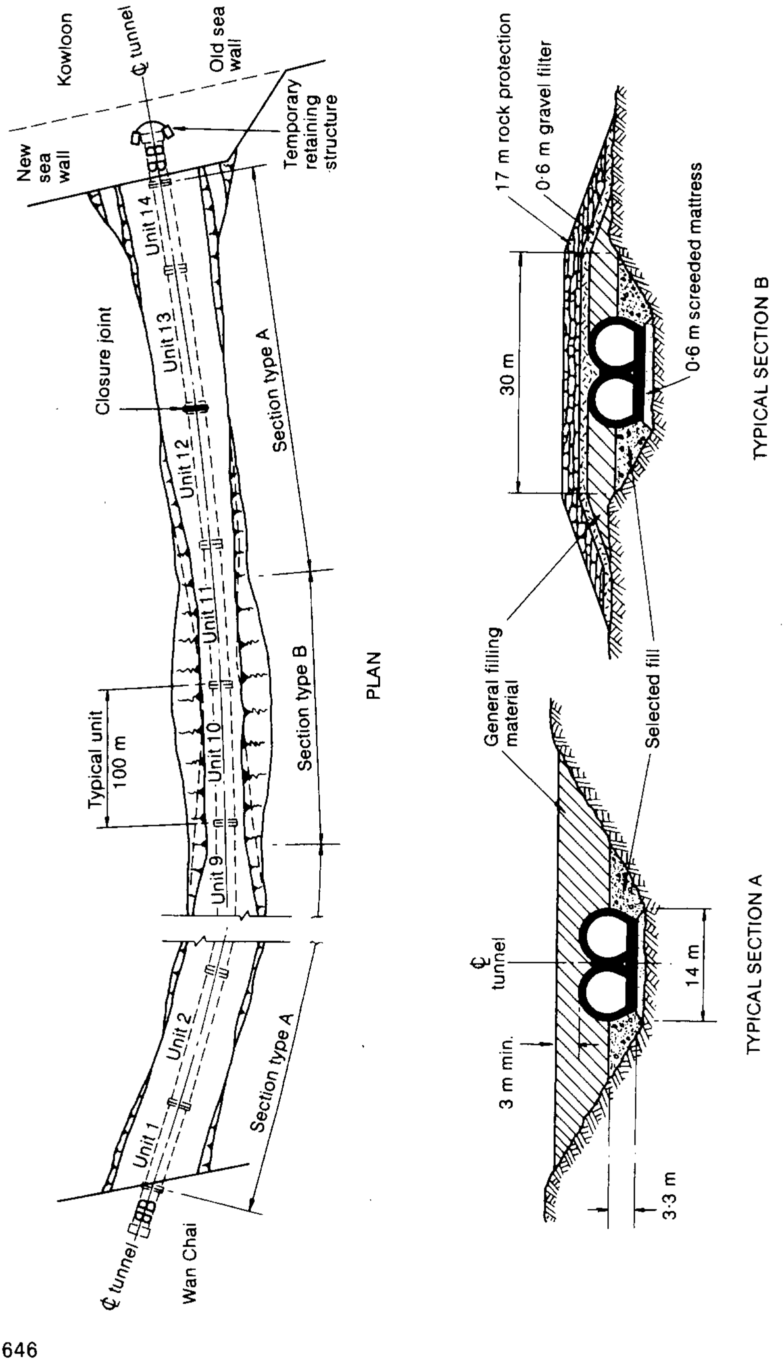

Fig. 21. Immersed tube tunnel: general arrangement

ments. In addition to the normal space required to accommodate the structure gauge and electrical and mechanical plant, an emergency walkway, 750 mm wide at car floor height, is provided in each compartment adjacent to the dividing wall to permit detraining in case of emergency. Adits fitted with doors are provided at 18 m intervals to allow emergency access between compartments.

68. Ventilation structures within reclamations at each end of the crossing provide vertical shafts for air intake and exhaust to relieve the piston effect of trains and for ventilation. Floodgates are provided in the ventilation buildings to permit isolation of the immersed tube from the rest of the system. The buildings also house other major items of electrical and mechanical plant.

Design

69. The principal design parameters are summarized in Table 5 and the factors of safety used are given in Table 6.

70. Besides the tolerances given in Table 7, a further tolerance of 150 mm around the canted structure gauge was required to allow for possible misalignment of the prefabricated tunnel units both in construction and in placing. To allow for tolerances in thickenss of backfill and to allow for erosion or siltation, the assumed thickness of backfill for design was 3 m $\pm$ 3 m.

71. The analysis took account of the following:

(*a*) the effect of a sunken ship;
(*b*) complete flooding of the immersed tube;
(*c*) earthquake (a horizontal acceleration of 0·15 *g* allowing a 33% overstress was assumed);
(*d*) loss of support due to irregularity of the screeded mattress;
(*e*) differential settlement due to weak spots in the subsoil;
(*f*) frictional restraint of movement caused by temperature, shrinkage and creep.

72. All tunnel units are reinforced in the transverse direction and are longitudinally post-tensioned. Construction joints were formed at about 9 m intervals. Continuous welded steel bottom plates, 6 mm thick, extending 1 m up the outside walls, cover the joints between the invert and arch pours and act both as a part of the waterproof membrane and as permanent bottom formwork (Fig. 22).

73. Durability of the structure was considered as important as strength in view of the unusually corrosive water and soil conditions of the harbour. The waterproof membrane is the first line of defence for watertightness, durability and prevention of corrosion. The concrete of the structure was designed and constructed as an independently watertight structure as a second line of defence. The external steel bottom plate has a protective coating. All reinforcement and prestressing cables are electrically bonded with provision for cathodic protection if found to be necessary.

74. There is a significant change in the loading conditions on the end units part-way along their length due to the ventilation buildings and the much greater depth of backfill. This was expected to give rise to differential settlements and, to counter these, the foundation in the area of higher loading was grouted through holes specially left in the units for this purpose. Also a special joint was provided at the point of changed loading to accommodate movement. This joint was designed to transmit shear and a temporary prestress of 29 MN was applied to keep the unit rigid during floating, towing and sinking; the prestress was released after these units were partially backfilled.

Table 5. Design parameters

Current velocity	1·5 m/s
Low water level	0·0 PD
High water level	4·3 PD
Maximum water level	5·0 PD
Wave height	1·0 m
Wave length	20 m
Wave period	3–5 s
Annual variation of concrete temperature:	
mean	12°C
max.	16°C
Temperature differential across concrete:	
mean	2°C
max.	6°C
Sea water density:	
min.	1010 kg/m^3
max.	1025 kg/m^3
mean	1020 kg/m^3
Concrete density:	
min.	2290 kg/m^3
max.	2370 kg/m^3
mean	2340 kg/m^3
Reinforced concrete density:	
min.	2360 kg/m^3
max.	2460 kg/m^3
mean	2400 kg/m^3
Saturated soil density	1840 kg/m^3
Granular backfill density	1960 kg/m^3
Armour protection density	1960 kg/m^3
Backfill material internal angle of friction	30°
Frictional coefficient between screeded mattress and bottom steel plate	0·57
Equivalent sunken ship uniform load	50 kN/m^2
Surcharge on land areas	20 kN/m^2
Subsoil modulus of elasticity	30–60 MN/m^2
Mattress modulus of elasticity	2–6 MN/m^2
Fire resistance	4 h
Concrete cover:	
buried/immersed	60 mm
interior of immersed	40 mm
above HWL	30 mm
Maximum crack width:	
without differential settlement (no tension class 1)	0·0 mm
with differential settlement (class 3)	0·1 mm
Characteristic concrete strength	30 Pa
Characteristic reinforcement strength	410 Pa
Characteristic post-tensioning wire strength	1720 Pa
Characteristic structural steel strength	Grade 43A (BS 4360)

Table 6. Factors of safety

Factors of safety:	
ballasted unit during sinking	1·03
ballasted unit after sinking	1·05
completed unit with minimum 1·5 m backfill	1·20
Metacentric height while floating	0·1 m
Positive buoyancy while floating	1%
Structural ultimate load partial safety factors	CP110 clause 2.3.3.1

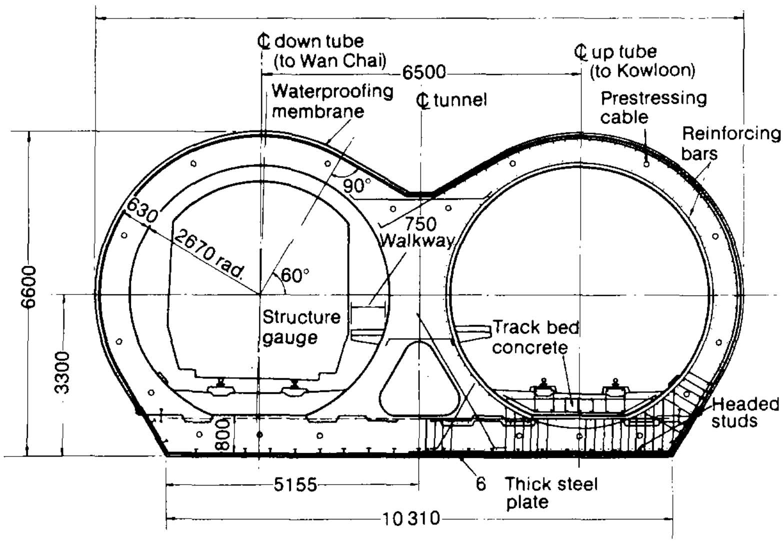

Fig. 22. Immersed tube tunnel: typical unit cross-section (dimensions in millimetres)

Table 7. Tolerances

Slab and wall thickness	±6mm
Average slab and wall thickness	±3 mm
Concrete volume	±0·5%
Elevation of screeded mattress	±50 mm
Variation in 50 m of mattress	±25 mm
Elevation of rock protection	+800 mm −300 mm
Volume of rock protection over 400 m^2 area	−0%

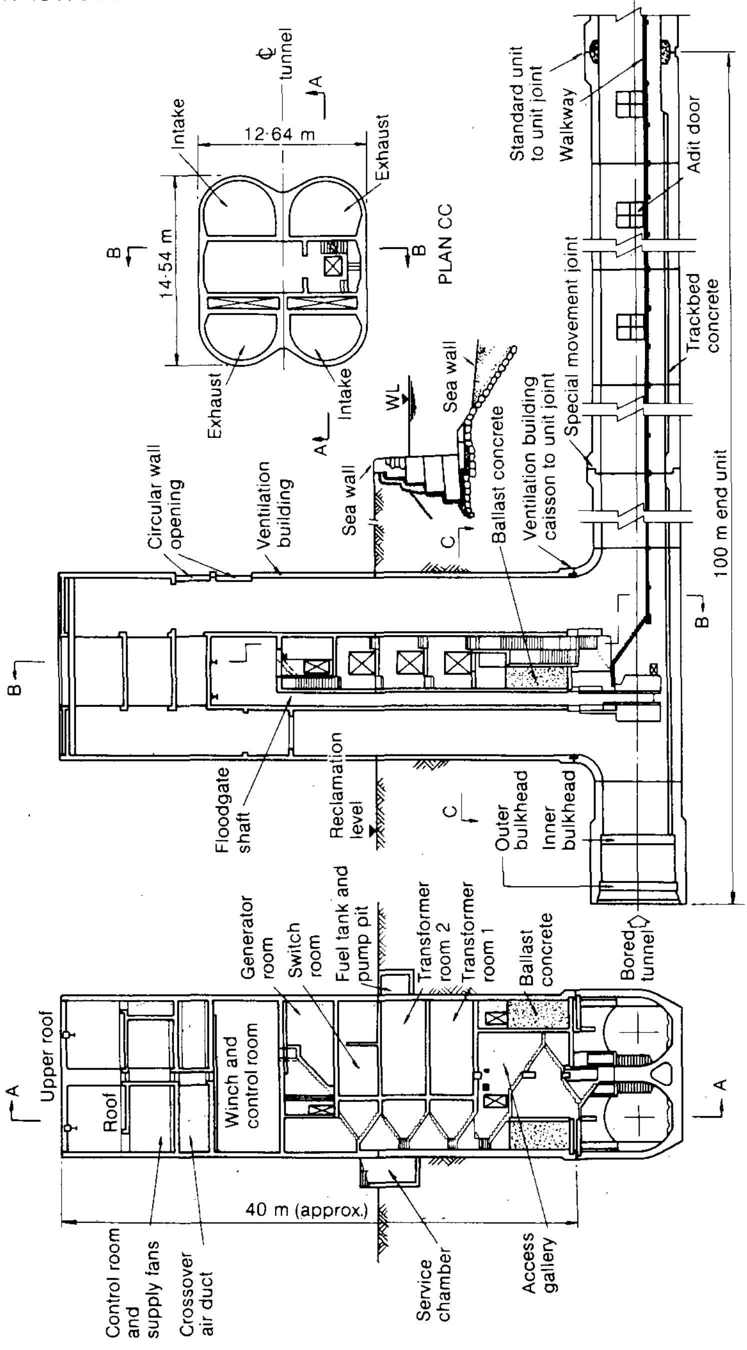

Fig. 23. Immersed tube tunnel: end unit and ventilation building

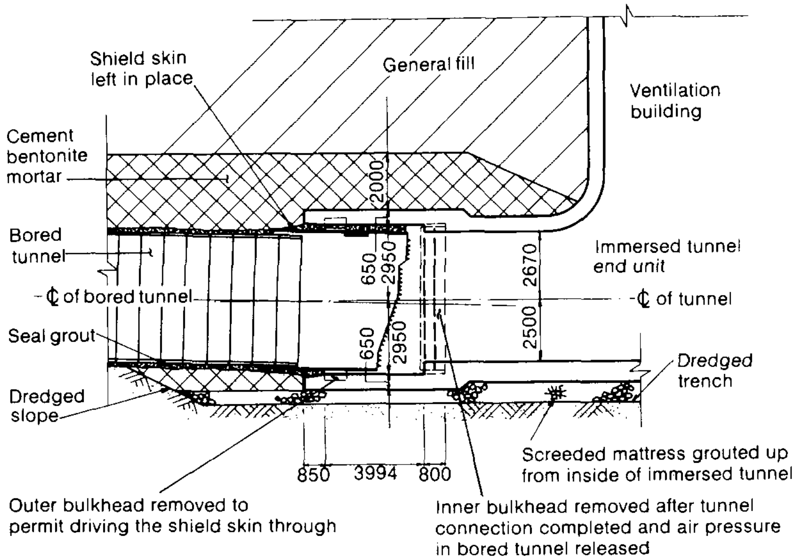

Fig. 24. Connection of immersed tube to bored tunnel (dimensions in millimetres)

75. The ventilation buildings were designed to be constructed as floating caissons to be lowered on to rubber seals on top of the end units (Fig. 23).

76. Vertical joints between units are watertight and capable of transmitting the full shear load as well as the expected rotations and deflexions.

77. The end of the tube was enlarged to facilitate connection with the bored tunnels by driving the shields into the precast unit (Fig. 24).

Construction

78. A dry dock for the prefabrication of the 14 immersed tube units was constructed near Chai Wan at the north-east corner of Hong Kong Island just outside the harbour area. The level of the dock floor was chosen to ensure an adequate water depth with a reasonable margin of time at high water for units to lift off the dock floor and be towed out to deeper water (Figs 25 and 26).

79. A layer of overconsolidated marine clay about 10 m thick about 9 m below the dry dock floor was found to be so weak that, although construction within the dock would not overstress this layer, the proposed tied double-wall cofferdam facing the sea would have developed instability when dewatered. Therefore an alternative dock gate design had to be adopted.

80. The side slopes to the basin had to be so flat to ensure adequate stability that only four tunnel unit casting beds could be accommodated. The 14 tunnel units were therefore made in four batches of four, four, four and two units, leaving spare capacity in the last batch as a contingency measure.

81. The settlement of the dock bottom, calculated on the basis of results of a load test, would have caused distress to the invert concrete if the units had been

constructed as originally specified. The arch concrete was therefore reprogrammed to be at least 20 m behind the invert.

82. The reinforced concrete gate caisson for the cooling water intakes and the two ventilation building caissons were constructed in a floating dock moored at Chai Wan.

83. Initial delays in access to part of the fabrication area and further delays caused by the difficult ground conditions resulted in such a late start to dry dock construction that the 9 m/day of tube unit construction achieved could not satisfy

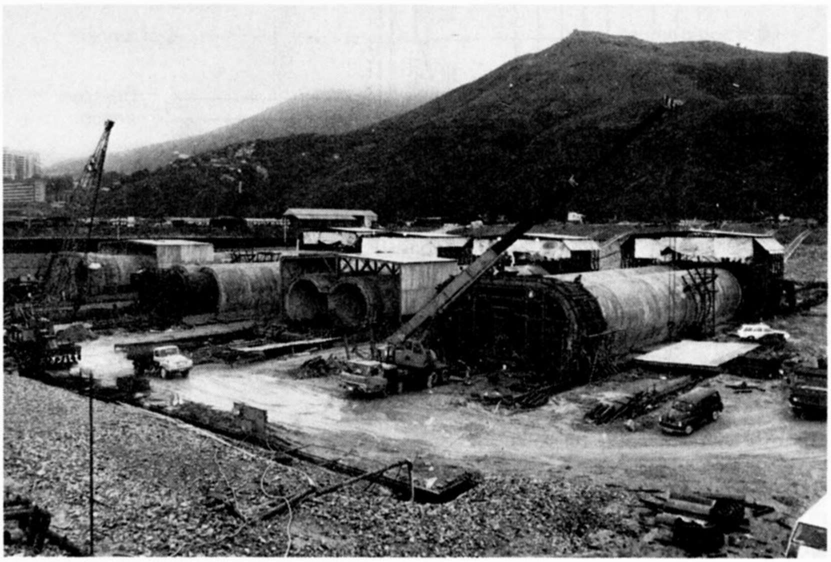

Fig. 25. Immersed tube units under construction

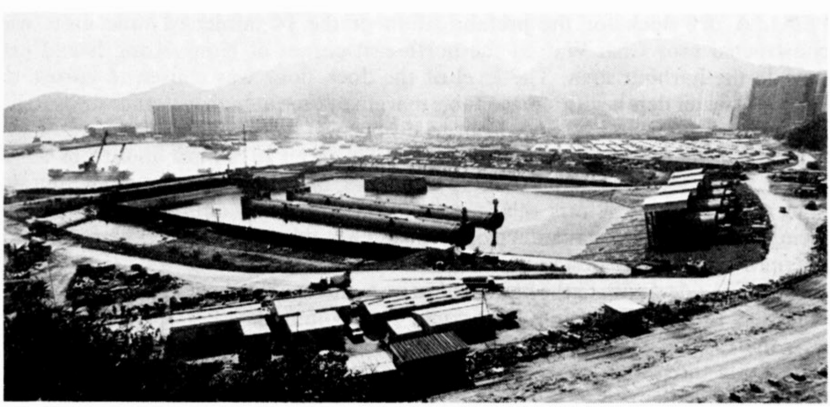

Fig. 26. Dry dock general view

the overall programme and a delay to the bored tunnel connection on the north shore appeared unavoidable. To overcome this the original programme, which required the tunnel units to be placed in sequence from Wan Chai, was rearranged so that units 14 and 13 were installed out of sequence. This required the introduction of a special 1 m wide in situ joint between units 13 and 12.

84. Dredging of the trench across the harbour and fabrication of the tube units were carried out concurrently. Operations in the 200 m wide designated harbour area had to comply at all times with Marine Department requirements for

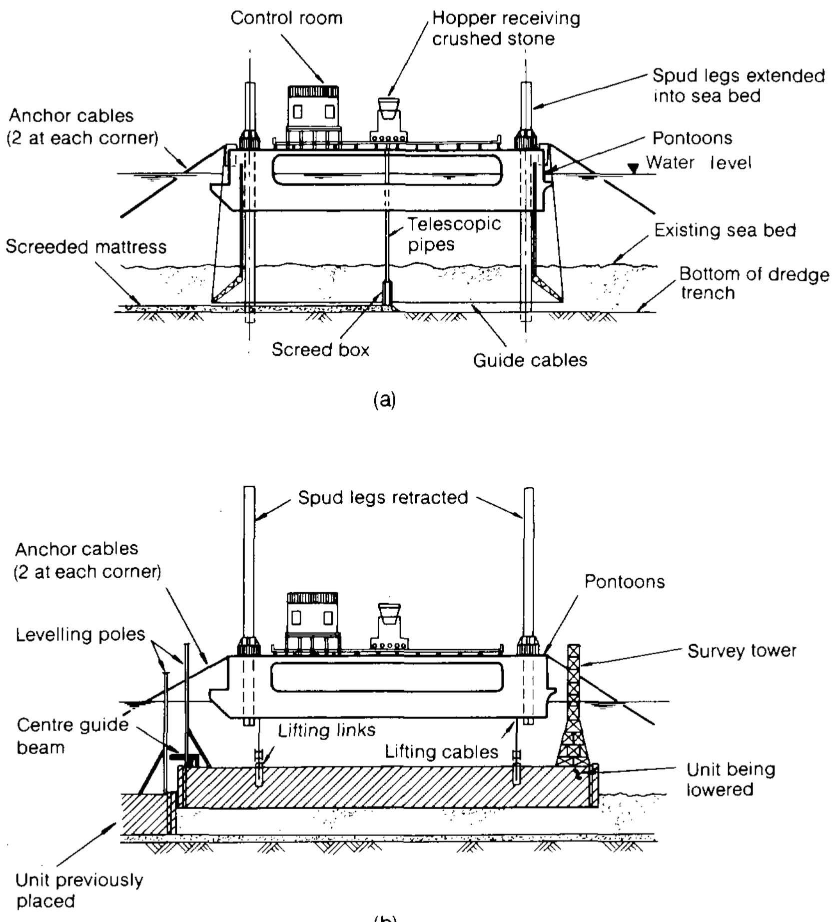

Fig. 27. Operation of special marine plant (SMP): (a) screeding (SMP standing on spud legs with pontoons submerged); (b) placing unit (SMP floating, spud legs retracted)

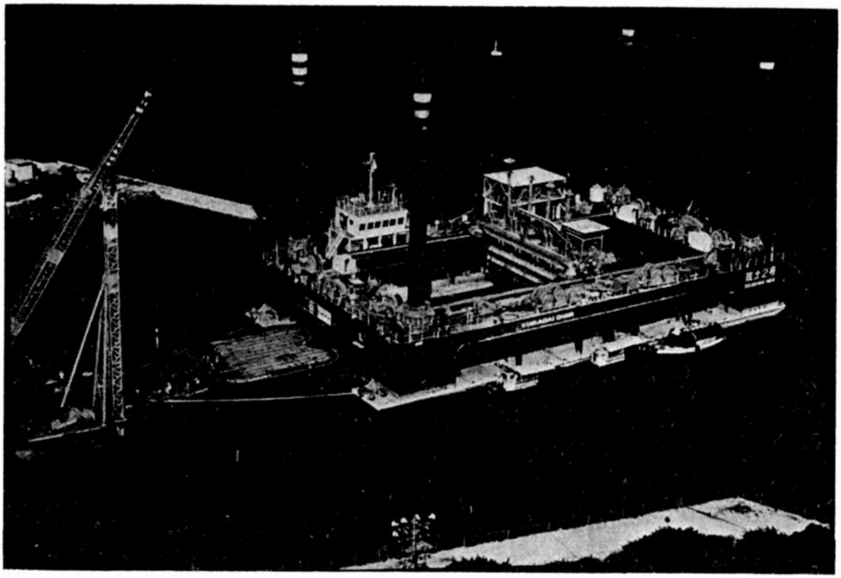

Fig. 28. SMP placing end unit

Fig. 29. Interior of completed tunnel

navigation in the harbour. The dredging and backfilling of the trench was carried out with clamshell dredgers.

85. The two specialized operations of laying the screeded foundation mattress and sinking and placing the tunnel units (Figs 27 and 28) were carried out by a special marine plant (SMP). Laying the gravel bed it worked as a semi-submersible jack-up platform standing firmly on four legs, independent of wave action and tide level. When placing tunnel units it floated as an anchored catamaran straddling the tunnel.

86. Local conditions at the north end of the dredged trench, where public utilities were close behind the existing sea wall, required a temporary retaining structure to enable unit 14 to be placed.

87. This structure was designed as a 1·2 m thick concrete arch dam bearing against two buttress towers which were held in position by post-tensioned rock anchors. During construction unexpected massive boulders were encountered at the crown of the arch and this prevented completion of the bottom portion of these panels, leaving a 10 m × 9 m hole. To prevent movement of the boulders, which were massive enough to transmit the arch thrust, 25 mm dia. high tensile bars were grouted in at 1·25 m centres in three rows.

88. All units were provided with watertight temporary bulkheads to enable them to float and the two end units had facilities for a secondary bulkhead to provide for equalization of air pressure as a safety precaution at the bored tunnel connections.

89. Units were joined by being brought into contact with each other through a soft-nosed continuous rubber gasket bearing on an accurately positioned steel plate. By draining water trapped between the bulkheads into tanks in the previous unit, hydrostatic pressure on the remote end of the next unit being placed held these units tightly together. Secondary water seals and a shear ring, which locks the units in position, were then installed at atmospheric pressure from the inside.

90. The tube units were placed and each was jointed to the previous unit at the rate of one per month. The first element of each batch to be placed was towed straight from the dry dock to the SMP in the harbour, while the other three units were towed across to a specially designated and sheltered area in Junk Bay where they were moored. The tunnel elements were towed from the temporary mooring area to their destination in Victoria Harbour by locally hired tug boats and brought in underneath the SMP aligned with the current and then turned through 90°. The units were winched down to the sea bed under a negative buoyancy of about 300 t, where they were brought within 0·5 m of their final position. Two hydraulic jacks pulled the units together to achieve final positioning and initial sealing of the rubber joint.

91. The good progress maintained following the initial delay, which was aided by the changed sequence of placing the units, ensured that the harbour crossing was completed ahead of programme. The completed tunnel is shown in Fig. 29.

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