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HONG KONG MASS TRANSIT RAILWAY TSUEN WAN EXTENSION AN EXTRACT OF "CONSTRUCTION OF PRINCE EDWARD STATION"

香港地下鉄路公団チュンワン延長線工事(第2期工事)
プリンスエドワード駅の建設(抜粋)

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Synopsis

Prince Edward Station, locating in one of the busiest urban area of Hong Kong, is the most important interchange station between existing line and future line.

This station was constructed in two stages and the previously constructed half part was under usual operation of train during second stage construction.

Under Contract 301 the remainder part of the station was constructed and integrated as one structure. On construction the following points were raised, i.e.

1. Soil strata consisted of weathered granite with boulders.

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2. How to avoid possible influence for the existing station and possible settlements of adjacent structures.
3. Construction programme was extremely tight.
4. Etc.

Considering these problems, PIP-W as temporary cofferdam and a top-down construction method were adopted.

On designing PIP-W and temporary structures, some of members were found not enough refering to British Standard, then detailed analyses by elastoprastic theory using computer were made, by which the safety of the temporary structure were confirmed.

Through construction, actual forces, stresses, deflections and settlements were monitored obtaining very similar results as calculated.

§1. Introduction

Prince Edward Station of the Mass Transit Railway of Hong Kong is one of the most important interchange stations at which the existing line connects with a new line. The station was constructed in two stages, i.e. one longitudinal half of the station was constructed in a previous contract and carried operating trains for the public during construction of the remainder of the station.

The completion of Prince Edward Station involved the construction of the second half of the structure in a deep excavation adjoining the existing railway and the integrating of the two parts.

The un-precedented underpinning construction procedure and some of the monitoring data are described in this paper.

The characteristics of the construction of this station are as follows:-

1. The previously constructed MIS half station consisted of the east side permanent benoto wall and a temporary benoto pile wall (secant piles) on the station centre line to provide separation between MIS and TWE whilst maintaining the structural stability of the half station box. Hand dug caissons* were adopted for the walls under a crossing flyover where the working room was insufficient for the benoto pile rig.
2. The second TWE half station was constructed by means of a top-down method using the PIP-W wall method, a contiguous piling technique (**Fig. 3**), successfully employed previously in the construction of Jordan and Tsim Sha Tsui stations on the MIS.
3. After completion of the TWE half station box the two independent parts were then united by the progressive removal of the temporary benoto pile wall and the concurrent integration of the floor slabs.
4. During construction normal train operation was to be satisfactorily maintained in the MIS box.

The construction site was located in one of the busiest urban areas of Hong Kong so that many difficult problems were raised in designing and planning of the station in terms of disturbance to the surrounding area, possible settlements of adjacent buildings, maintenance of the MIS train operation below ground and the traffic flow above ground on a main street.

**Hand dug caisson is similar to 'SHINSO' in Japan and is one of the cast in-situ piling technique in Hong Kong.*

Also the control of ground water table was specifically required in order to avoid nuisance problems which might be happened during the works.

Under these circumstances the construction of PRE station was carried out utilising an extraordinary method. The full system of the Mass Transit Railway is shown in **Fig. 1**.

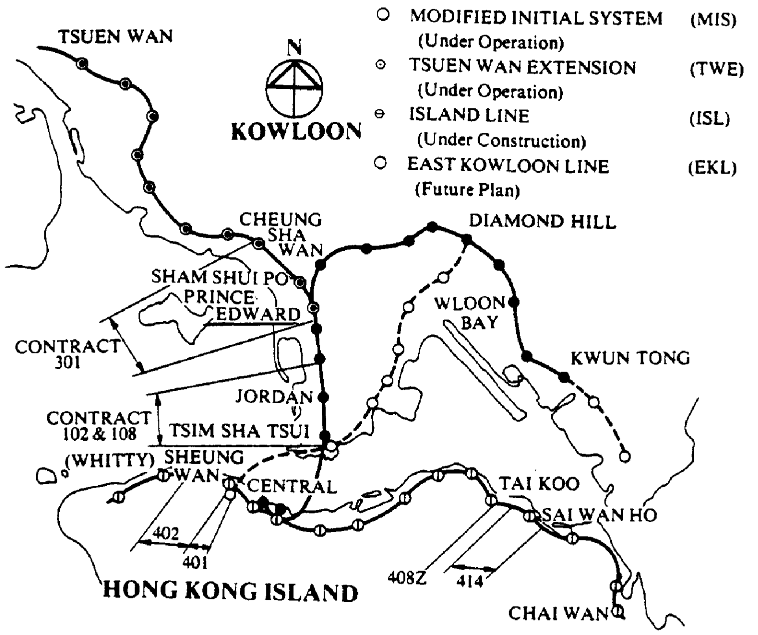


Fig. 1 Mass transit railway-full system

§2. Profile

2-1 Outline of PRE Station

The scope of the works extended over a length of 240m comprised of a 3-storey structure (roof, concourse, upper and lower track slabs) with an excavation depth of approx. 24.5m.

The structural profile of the station in plan are shown in **Fig. 2** and in section before and after integration in **Fig. 3** and **4**.

2-2 Soil Conditions

1. From ground level a 5m deep layer of fill materials covers the whole site.
2. Below the fill an approx. 10m thick marine deposit exists over 2/3 of the station. This stratum consits of coarse sand to silty clays with shells and has SPT values of 2-10.
3. A 1-2m thick alluvium layer is sandwiched between the marine deposit and the CDG in the northern half of the station. It is composed of coarse sand carrying heavy ground water which cause problems several times during bulk excavation by bursting through gaps in the PIP-W wall.
4. Beneath the alluvium is completely decomposed granite (CDG) layered in part with many residual fresh granite boulders. SPT values in the CDG vary from 20 to over 200.
5. Granite bed rock only intruded into the works at the south end, rising to the concourse slab level.
6. Ground water table was generally only 1.0m below the ground surface.

Geological profile is shown in **Fig. 5**.

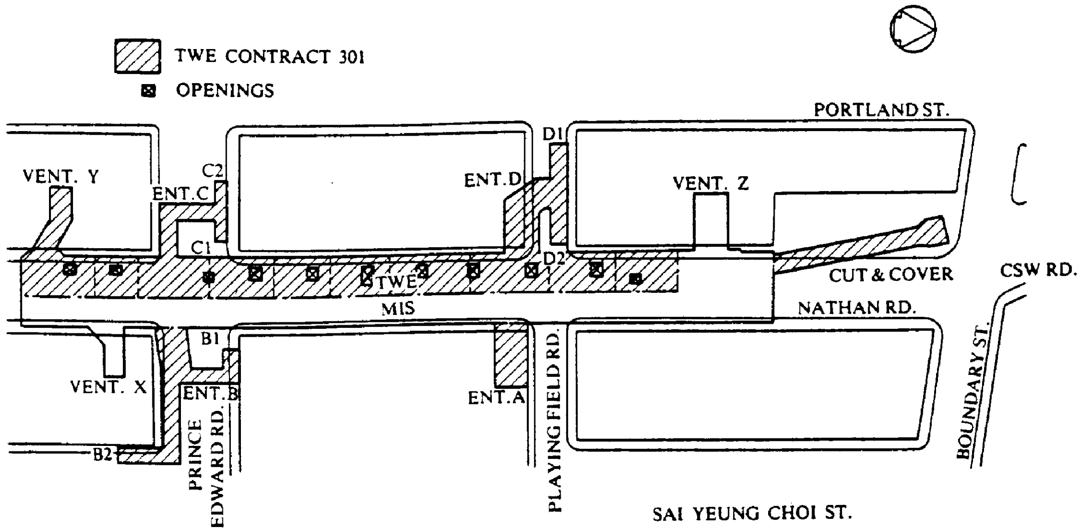


Fig. 2 Plan of pre-station

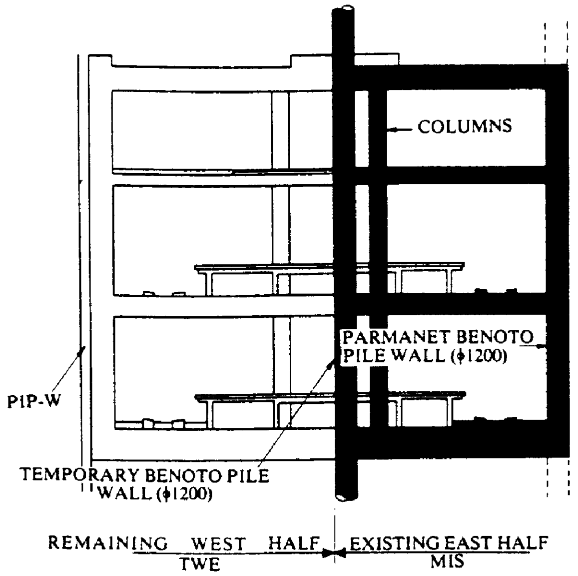


Fig. 3 Typical cross section before integration

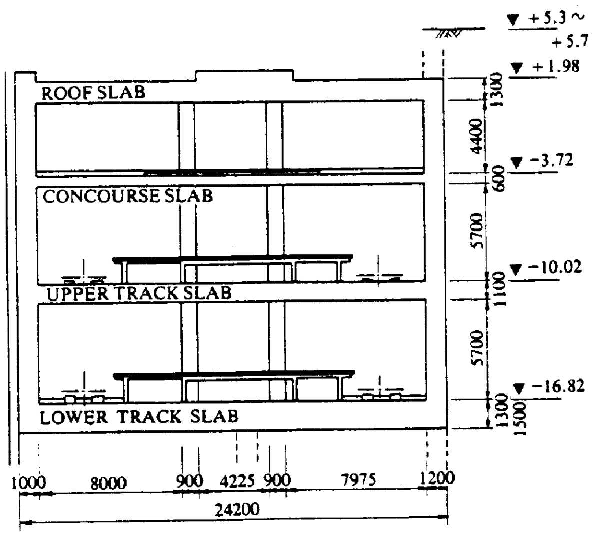
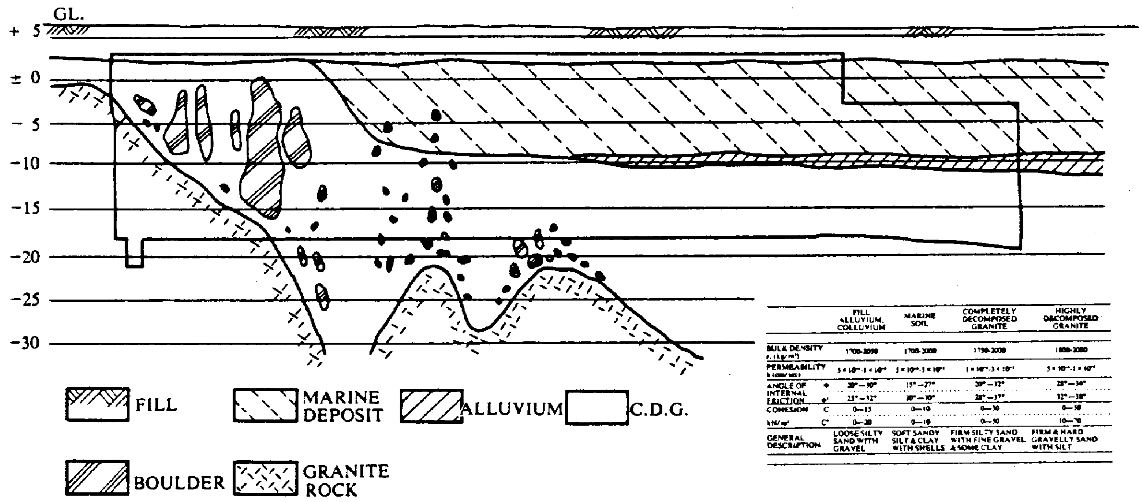


Fig. 4 Typical cross section after integration



		FILL ALLUVIUM, COLLUVIUM	MARINE SOIL	COMPLETELY DECOMPOSED GRANITE	HIGHLY DECOMPOSED GRANITE
BULK DENSITY r. (kg/m³)		1700-2090	1700-2000	1750-2000	1800-2000
PERMEABILITY k (cm/sec)		[illegible]	[illegible]	[illegible]	[illegible]
ANGLE OF INTERNAL FRICTION	φ	20°—30°	15°—27°	30°—32°	28°—34°
	φ'	25°—32°	30°—30°	28°—37°	32°—38°
COHESION kN/m²	C	0—15	0—10	0—30	0—50
	C'	0—20	0—10	0—50	10—70
GENERAL DESCRIPTION		LOOSE SILTY SAND WITH GRAVEL	SOFT SANDY SILT & CLAY WITH SHELLS	FIRM SILTY SAND WITH FINE GRAVEL & SOME CLAY	FIRM & HARD GRAVELLY SAND WITH SILT

Fig. 5 Geological profile

§3. Construction Method

3-1 Methodology

(1) Alternative Construction Method.

At the time of Tender a construction method of a bottom-up with PIP-W wall was proposed and accepted as a part of Tender.

The construction method of PRE Station was, however, reconsidered taking into account two aspects namely;

1. Stability of MIS Structure
2. Programme considered as very tight.

   To ensure the stability of the MIS structure throughout TWE construction, a PIP-W wall and top-down construction method was recommended by both design and project staff with the following comments:

1. Continuous diaphragm wall construction was considered extremely difficult judging from the extent of boulders and the previous construction results of the benoto pile wall, and might cause programme delays, PIP-W wall method was preferred as a steady method well proven in similar conditions.
2. Time was likely to be saved because excavation and structure work would be parallel activities.
3. As working room would be spread not only horizontally but also vertically, many activities could be simultaneously carried out.
4. A scheduling of labourers might be easier than a bottom-up method.
5. No propping and scaffolding materials were required for slab construction.
6. Strutting and waling were reduced.
7. No surface water entered below roof slab.
8. Further consideration would be required to the timing of both column construction and slab integration.

The top-down with PIP-W wall method shown on **Fig. 6** was proposed to MTRC and accepted in principal. In this method, roof and concourse slabs were of simple beam between T-brackets on PIP-W wall and temporary benoto pile wall of which I-beams were partially exposed before cast in. Upper track slab was simply supported by T-brackets and I-brackets on temporary benoto pile wall, hence its surrounding concrete was to be remained as the stiffeness was required to prevent buckling of I-beams inside temporary benoto pile wall.

As for roof slab it was found that the steel bar area of existing slab was not sufficient against the transmitted bending moment due to the flooding load on the simple beam slab between PIP-W wall and existing RC columns after integration of the slab (**Fig. 7**).

Then partial slab thickening with drill-in bars was suggested as a better solution and concreted together with roof slab. It was named CAMELS BACK from its shape.

As described above, after discussions a top-down method was decided, but, of course, there were anticipated many disadvantageous points.

They were, mainly, that whole construction sequence became more complicated and those complicated sequences had to be dealt with under more narrow working space.

Transportation of materials, mucking-out debris and all other activities were done through limited temporary openings on each slabs.

They were all disadvantageous matters, but not critical because the site of PRE station was under the main street and a complete temporary decking covering whole site was required by the conditions.

These problems were considered not to be reduced even if a bottom-up method was adopted and ultimately, the construction requirement was to overcome these matters for its characteristics.

As a result it would be said that the selection of method has been correct and the achievement of the jobs has been made by a completely intimate co-operation of all parties.

(2) Difference of Construction Method

The definitive difference in structural conditions are as follow:-

1. The assumed procedure was that diaphragm wall would be initially constructed and simultaneously temporary steel stanchions (2 H-sections) would be installed in permanent column positions. Then each slab was to be supported by the temporary benoto pile wall, stanchions and diaphragm wall. These were two spaning continuous beam slabs which would be cast by means of a top-down method.
2. The proposed procedure was to install a temporary PIP-W wall with which the existing temporary benoto pile wall simply supported the permanent slabs being constructed by a top-down method. The permanent wall and columns were then constructed by a bottom-up.

3-2 Construction Problems

(1) PIP-W Construction

PIP-W construction, of which experience for construction of more than 2,500 Nos through the previous contract had been obtained, raised no major problems except the following cases.

1. How to construct through rock and/or boulders.
   This problem was solved by the combinations of Big-man (N-50, 100) and PIP rigs.
   There were some time-lag between the operation of two machines which caused delay of programme, then it was very important to control the timing of change of the machines.
2. How to construct under the balconies of existing buildings of which head-room being less than 6.0 m.
   For the construction of these area, PIP-D technique (lap method) was applied, which have special short-rig similar to BH method.
3. How to deal with areas of which water tightness being not enough.
   It was impossible to form the water tight walls in the areas where Big-man and PIP-D were used, because of those equipments having no facilities of forming "jet-fin" between piles.
   As for those part, JGP method was adopted after the construction of piles in order to obtain the water-tightness between piles.

Through this construction, it was considered that PIP walls as temporary cofferdam for such a deep excavation with various strata were near the limit of its characteristics, because of its difficulties of maintaining verticality, maintaining enough strength and of controlling construction programme.

(2) Stress and deflection of PIP-W wall

According to the analysis method for temporary structure based on the British Standard (BS), PIP-W wall might be over stressed during the excavation in certain area. However the calculation based on the design concept described in Section 4 was finally accepted by the Engineer (MTRC) and some monitoring data proved that the actual condition was very similar to those anticipated (Section 5).

(3) South End Stability

The construction method for the South end was to be different from other sections because the temporary benoto pile wall was founded on high bedrock and its support would automatically be removed during excavation.

As a solution, 4 nos of temporary steel stanchions (double H-beams) were installed in the holes made by hand dug caissons prior to the excavation.

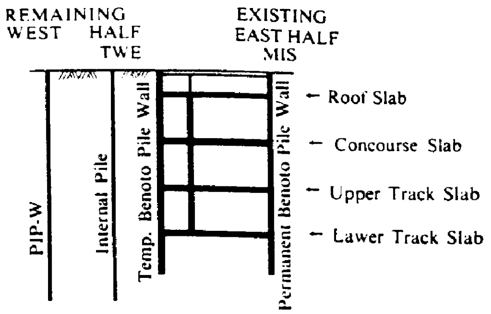
While the vertical stanchions would support the new slabs, the existing track slabs were to be supported horizontally against ground water pressure from the eastside permanent wall before the new slabs were cast in.

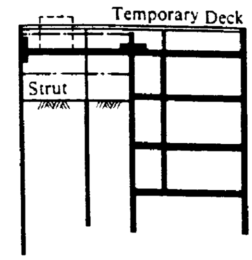
Numbers of rock anchors were installed in the east side permanent wall to prevent any horizontal movement since passenger carrying trains were running on the slabs.

(4) Slab Integration

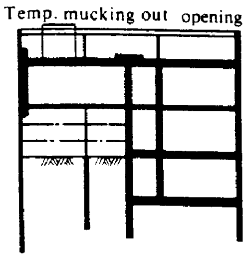
As a result of late handover of the works area and difficulties in PIP-W wall construction, MTRC requested acceleration from the commencement of bulk excavation to recover the original programms. A further reconsideration of the construction sequence was undertaken to establish methods to speed progress. This reconsideration addressed to problem of how to demolish 6,500 m^3 of concrete temporary benoto pile wall and when to integrate floor slabs. Stage 1 and stage 2 integration methods were adopted in April 1981. The staged integration required the localised breaking out of alternate 3-5m wide bays of the benoto pile wall, integrating the MIS and TWE slabs in each bay, and then repeating the exercise in secondary bays. Temporary benoto pile wall between slabs was removed after integration was completed in order to avoid elastic shortening in benoto pile wall. This procedure was required to maintain structural continuity both in the vertical and horizontal directions, at all times bearing in mind the operation of passenger carrying trains supported on the MIS-side slabs. In addition much consideration was given to the deflections of slabs arising from elastic shortening in columns as the temporary benoto wall loads were progressively transfered to the columns.

Apart from the above a number of problems were raised in design and construction aspects during the works. Although some of them were anticipated, the majority were not reasonably foreseeable when the alternative method was proposed. The method, therefore, involved considerable expenditure to at least satisfy the design requirement and to proceed the works without any delay. Notwithstanding the matters the method was technically successful, the works were completed on time and no adverse effects were observed in MIS structure.

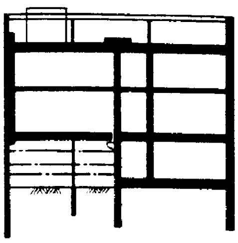




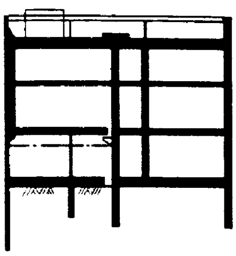
(1) Construct PIP-W and internal piles.
(2) Install temporary deck.
(3) Excavate to underside of roof slab installing 1st row struts.
(4) Construct roof slab.
(5) Excavate to underside of concourse slab installing 3rd row struts.



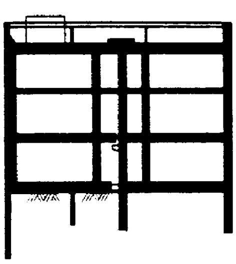
(6) Construct concourse slab.
(7) Excavate underside upper track slab installing 4th and 5th rows struts.
(8) Construct concourse wall.



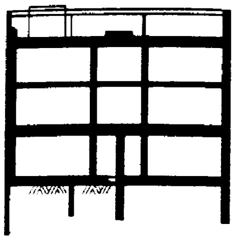
(9) Construct upper track slab.
(10) Excavate to underside of lower track slab installing 6th, 7th and 8th rows struts.
(11) Construct upper track wall.



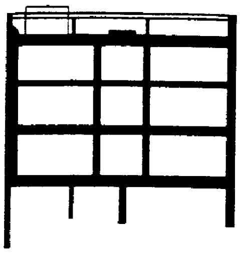
(12) Construct lower track slab.



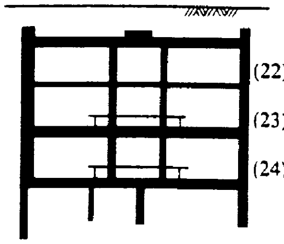
(13) Construct columns.
(14) Construct 1st lift of lower track wall.
(15) Construct 2nd lift of lower track wall.



(16) Remove temporary benoto wall down to concourse level.
(17) Integrate concourse slab.
(18) Remove temporary benoto wall down to upper track slab level.
(19) Integrate upper track slab.



(20) Remove temporary benoto wall down to lower track slab level.
(21) Integrate lower track slab.



(22) Construct platforms at both upper and lower track levels.
(23) Install waterproofing, backfill and remove temporary decking.
(24) Reinstate road slab.

Fig. 6 Typical construction sequence in cross section

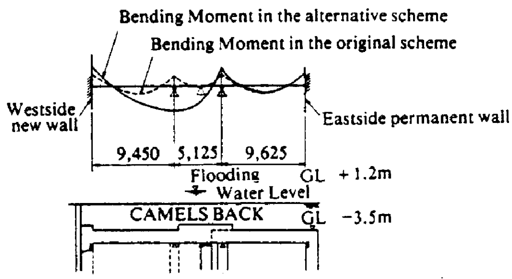


Fig. 7 Structural difference in roof slab

§4. Temporary Works Design

4-1 Stress of Temporary Cofferdam

The typical structure model is illustrated in **Fig. 8** and the calculation for temporary cofferdam was carried out for each stage of excavation.

(1) Earth pressure

Terzaghi's pressure diagram for dense sand was adopted. Passive pressure considerations were based on Terzaghi and Pecks "Soil Mechanics in Engineering Practice" taking into account cohesion in Coulomb's formula and also friction angle, i.e.

$$p_p = K_p \cdot h + 2c$$

The ground water level inside the cofferdam was always kept 2m below the excavation level in order to develop cohesion and no cohesion was considered for calculation of active earth pressure.

(2) Water pressure

The total water pressure was considered as reducting to 50% of the maximum value at the underside of the wall taking into account seepage affect.

(3) Amendment procedure of passive resistance range

Since soil was assumed to be an elastic material, the passive resistance was amended by the following:-

First, the initial elastic horizontal reaction pressure, line B was calculated by computer using the typical structure model shown in **Fig. 9**.

Secondly, earth pressure for the Coulomb passive stage was calculated; line A in **Fig. 9**..

Thirdly, the point was determined where the two curves for the initially computed horizontal reaction pressure and the Coulomb passive earth pressure intersected. (Above this point, the theorectical elastic horizontal reaction exceeded the theoretical passive resistance of soil).

Fourthly, the level of the intersectional point was taken as an assumed excavation depth so that the horizontal reaction pressure could be recalculated; line C (for this particular calculation, the Coulomb passive pressure acts on D1 length instead of no spring).

The final reaction pressure used in calculating stresses in PIP-W wall was thus defined by line ODE and line C passing through point F. The error involved in including area DEF was small and could be neglected.

4-2 Deflection of Temporary Cofferdam

The behavior of the cofferdam wall for a braced excavation is shown in **Fig. 10** i.e. the second struts were installed under the conditions of deflected cofferdam wall due to the stage 1 excavation being residual.

Continuously, stage 2 excavation was carried out and, as a result the cofferdam would be further deflected due to the forces which would increase during the excavation procedure. The third struts were installed in this condition. Thus, the above procedures were repeated until the final excavation stage.

Accordingly, the calculation procedures were as follows:-

1. Calculate the increased forces which occur during two excavation stages, i.e. between the former stage and proceeding stage.

2. Calculate the increased section forces and deflections caused by the increased forces.
3. Calculate the section forces and deflections at a certain excavation stage by accumulating the increasing section forces and deflections by turns from the first stage.

4-3 Typical Design Section

Fig. 11 indicated a typical design section and soil parameters which were determined from the results of the site investigation. The coefficients of the subgrade reaction were determined from the pressure-meter tests (**Fig. 12**).

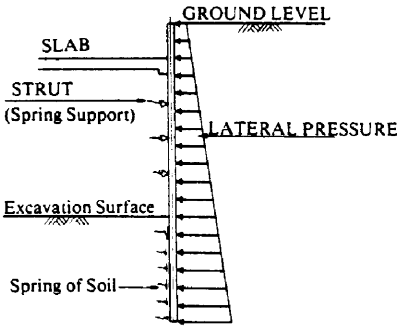


Fig. 8 Typical structure model

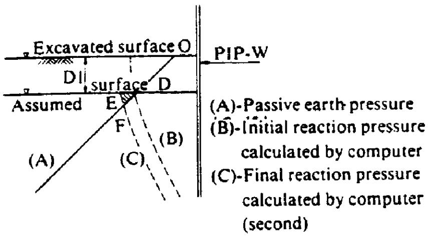


Fig. 9 Amendment procedures of passive range

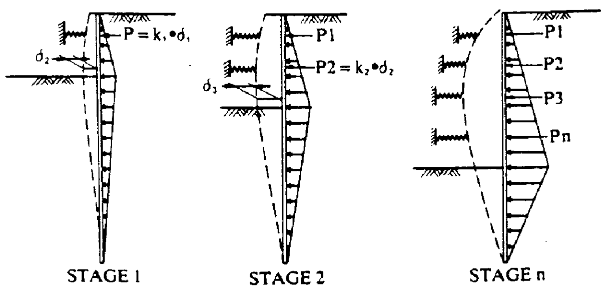
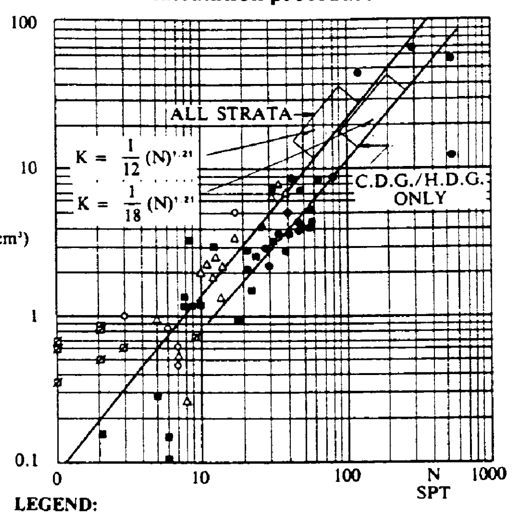


Fig. 10 Deflection calculation procedure



LEGEND:

- ○ —Fill
- ▫ —Marine Deposits — ▪ —Sand; ▪ —Clay, Sandy Clay.
- △ —Alluvium
- • —C.C.G./H.D.G.

Ro = 30 cm

α 0.33 for fill, sandy marine deposits alluvium & C.D.G./H.D.G.
= 0.50 for clayey marine deposits.

R = 100 cm (continous wall)

E_M = is the pressiometric modulus at each test location (kgf/cm²)

$$K = 1.5 \times \left(\frac{1.3}{3 \times E_M} R_o \left(\frac{R}{R_o} \times 2.65 \right)^{\alpha} + \frac{\alpha}{3 \times E_M} \times R \right)^{-1}$$

Fig. 12 SPT & subgrade reaction profile

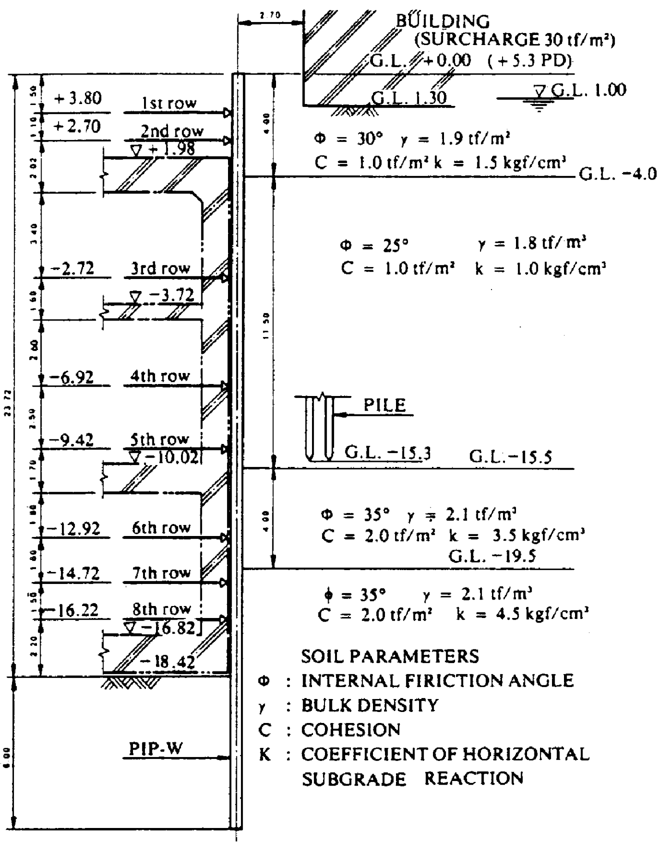


Fig. 11 Typical design section

§5. Monitoring of the Works

5-1 Instrumentation

During construction of PRE Station, many monitoring devices were installed in order to measure and control settlement of adjacent buildings, deflection of PIP-W wall, behavior of piezometric pressure and movements of surrounding ground.

They were;

1. Building monitoring points for settlement of buildings (**Fig. 13**)
2. Piezometers and standpipes for monitoring piezometric reduction and ground water movement (**Fig. 13**)
3. Inclinometer tubes inside PIP-W wall for monitoring deflections (**Fig. 13**)
4. Load cells in strut for monitoring actual axial force in strut (**Fig. 13**).

5-2 Observation Results

(1) Building settlement

In the course of construction of the station, the adjacent buildings settled and the reasons can be classified as follows:-

1. Consolidated settlement due to the effective stress coming from lowering of ground water and piezometric pressure.
2. Settlement caused by lateral movements (deflection) of cofferdam wall associated with excavation.
3. Settlement due to relaxation of the soil around cofferdam wall during construction.

Fig. 14 show the relationship between construction progress, water level, piezometer level, PIP-W wall deflection, and settlement of adjacent building. From this it was reasonably supposed that no particular relaxation of the surrounding soil occurred during PIP-W wall construction. It could be said that the drill holes of PIP-W wall were kept in quite stable condition, owing to the arch action and mortar head.

The foundation of a building as shown on **Fig. 14** is much deeper than station excavation level and it was hardly affected by excavation and dewatering. The total settlement of the building was 14mm and differential settlement was 1/3300 as angle. In contrast the foundations of another building just reached to the level of upper track slab level and are founded in CDG, therefore there was considerable effect on the building by excavation. The total settlement value was 50mm and differential settlement was 1/600 as angle.

Consequently there were observed no excessive building settlement during construction, of which reasons are assumed as follows:-

1. Deflections of PIP-W wall were similar to the figure calculated (refer to **Fig. 16**)
2. Pre-loading into each strut was effective.
3. Each slab was acted as rigid concrete strut.
4. The strata had already been consolidated during the previous construction.

(2) Load cell

Fig. 15 shows the relationship between axial force imposed on struts and construction progress. Judging from the reading results, the actual reading figures were less than approx. 60% of the designed figures.

(3) Inclinometer results

Fig. 16 show the design deflection and the actual monitoring results at each excavation stage of inclinometer readings from the tubes installed inside PIP-W wall. The actual deflection curves were almost similar to the design values.

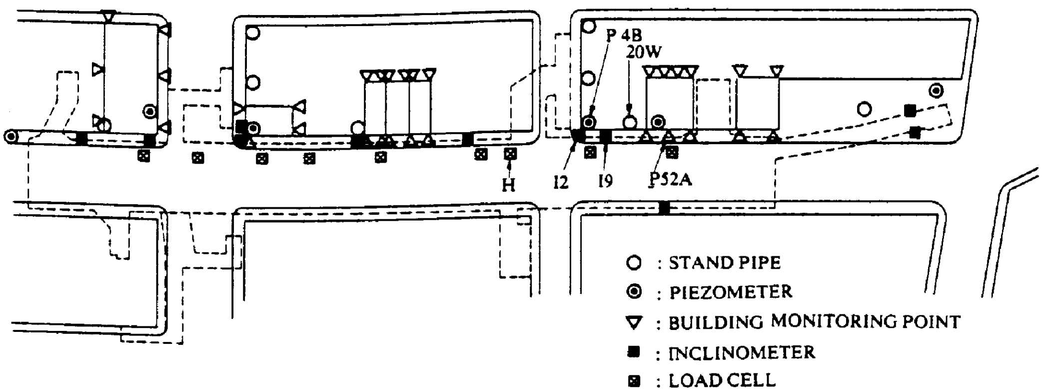


Fig. 13 Location of monitor

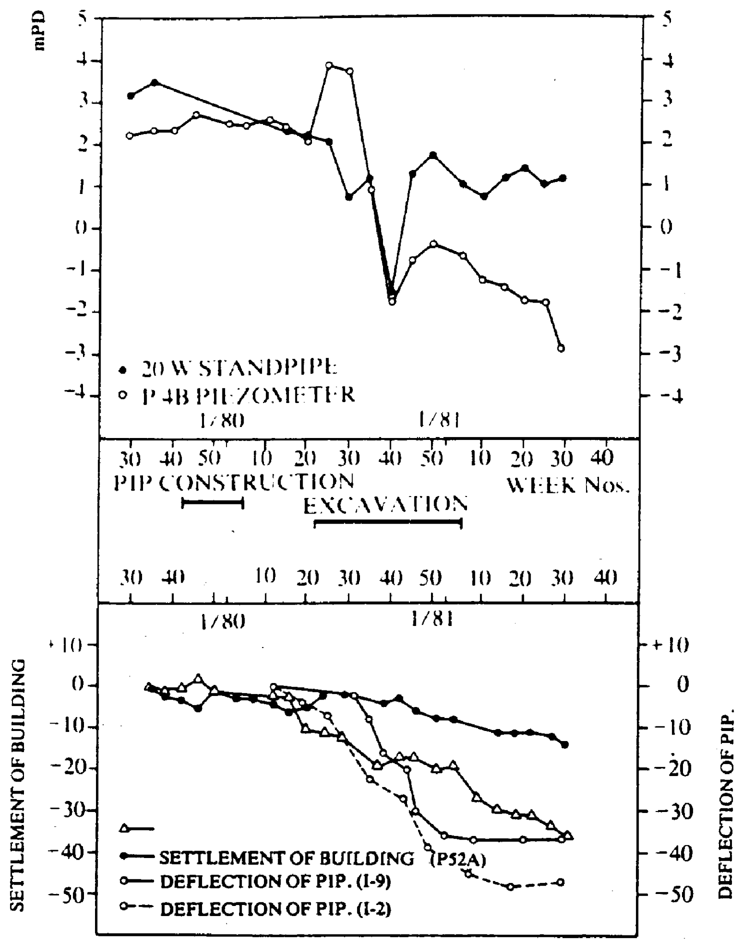
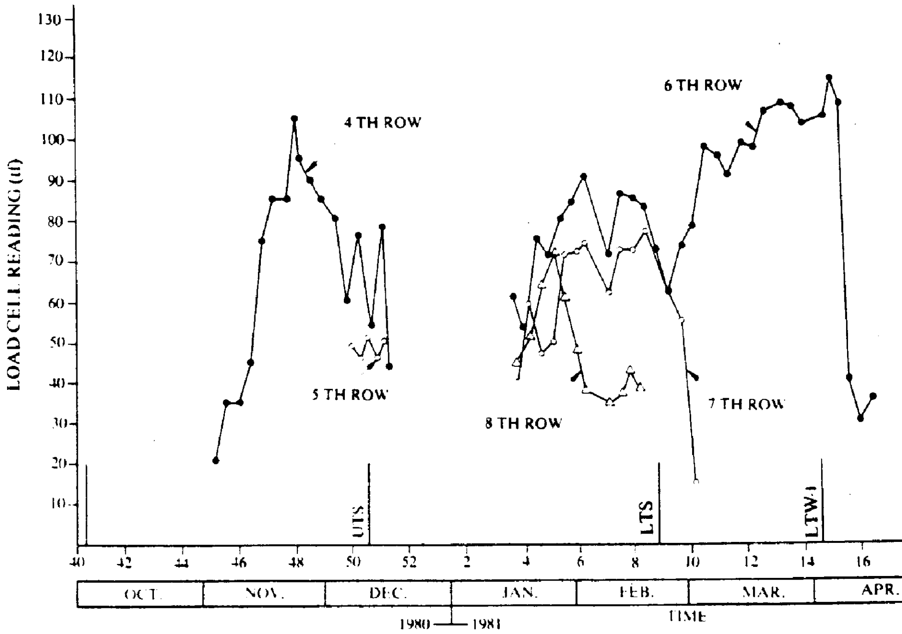


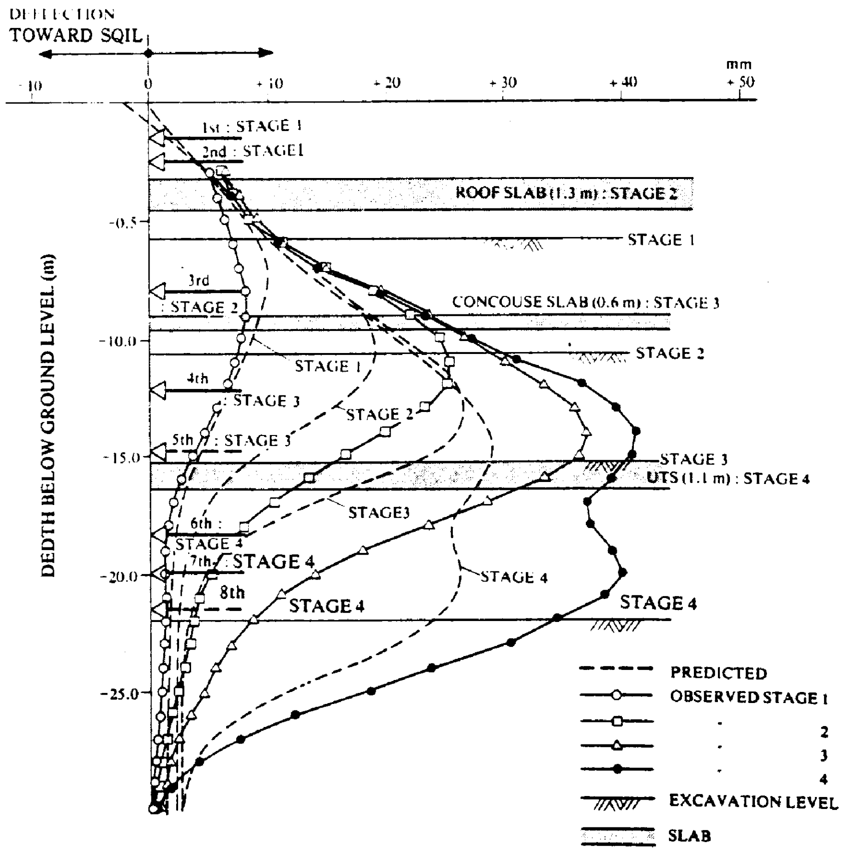
Fig. 14 Construction progress vs water & piezometer level, PIP deflection, building settlement



STRUT No.	RM.	RDN.	RM./RDN
4TH	105	196.5	0.53
5TH	51	196.5	0.26
6TH	114	196.5	0.58
7TH	78	222.6	0.35
8TH	72	222.6	0.32

NOTE:
RM.: MEASURED VALUE (tf)
RDN: DESIGN VALUE (tf)

Fig. 15 Load cell reading diagram block H



EXCAVATION LEVEL

STAGE 1 : EXCAVATED UP TO ROOF SLAB CONSTRUCTION LEVEL
STAGE 2 : 〃 CONCOUSE SLAB
STAGE 3 : 〃 5th ROW STRUT INSTALLATION LEVEL
STAGE 4 : 〃 8th

Fig. 16 Deflection curve for each excavation stage

5-4 Progress Report

(1) Construction result of PIP-W

Except some parts of which water-tightness were not obtained due to boulders and/or weak marine soil stratum, generally it was cosidered PIP-W being constructed satisfactorily.

The general construction rate of PIP-W were approx. 0.5 Nos./day for rock-zone (Bigman day/night shift and PIP rig day shift) and approx. 2 Nos./day for another zone (PIP rig day shift).

This rate were a little lower in comparison with another stations which excavation depth were shallower approximately by 6.0 m than that of PRE station.

Another two stations did not have rock zone, the average construction rate were approx. 4 Nos./day (PIP rig day shift).

(2) Excavation

The station box was divided into 10 blocks and each block had at least one access opening (cf. **Fig. 2**). A total of 11 Nos. of openings through slabs, including 2 Nos. of E & M openings through which plant and escalators were lowered, were planned. All openings had flood protection walls from roof slab to 1.2m above ground level in order to avoid flooding risk to the operating MIS.

Bulk excavation was carried out through 4 to 5 Nos. of mucking out openings in average. 9 Nos. of openings was used at peak time for excavation.

Soft material was gathered by shovel dozer (D - 30 class) and mucked out by clam shell bucket from crawler crane. Rock was excavated to steel bucket ($3m^3$ class) by shovel dozer (D - 50 class) following with blasting and dumped directly into lorries.

The average incidence of boulders which size varied from a meter to several meters was 4.7% against total excavation volume, but it was more than 10% taking into account the concentration of boulders mainly in the southern area. This figure was quite high even in Hong Kong and caused problems to the whole progress.

As shown in **Fig. 17**, the excavation rate of the boulder zone was approximately $150m^3$/day, while those of the another zones were approximately $220m^3$/day. The excavation of boulders was carried out mainly with blasting.

Bulk excavation progress proceeded as scheduled up to January 1982 (**Fig. 17**). Then, production curves was immediately slowed down when the works came into the boulder zone. Bulk excavation was finally completed 7 weeks later than the programme. Substantial completion of the structure however remained at the required date of July 81.

§6. Conclusion

The first train was run into Tsuen Wan Extension Line on 17th May 1982 which was 6 months earlier than its original programmed opening date.

The early operation of TWE was desired by not only MTRC but also the expanding communities in West Kowloon. It was, however, wholly depending on the completion of PRE Station as interchange with MIS.

The construction of PRE Station was carried out successfully to satisfy both MTRC and public in spite of its difficulty and complication in construction and design. Also no serious

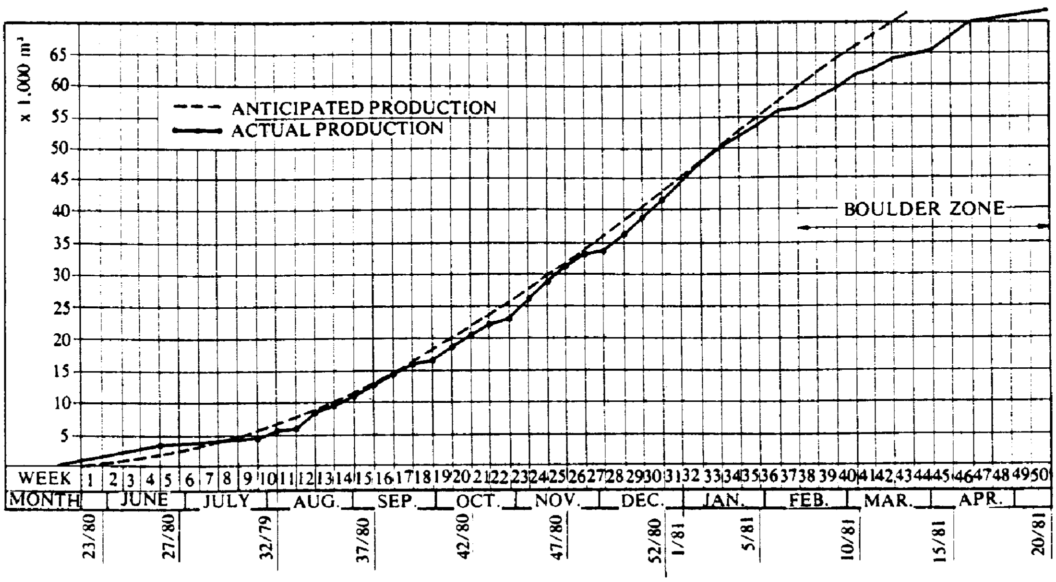


Fig. 17 Progress of bulk excavation

problems were occurred on the MIS structure and adjacent buildings.

Perhaps the most noteworthy matter during construction was divergent engineering approaches to the problems and their resolution exhibited by the parties, MTRC (Client and Construction Supervision), FF&P (Design Consultant) and Nishimatsu (Contractor), and the successful completion of the works notwithstanding these differences.

A lot of records and data have been obtained during the construction. We are intending to study and analysis them in detail hence only some of them are described in this report: Also we would expect that the paper is useful for future design and planning works in Japan as well as in Hong Kong.

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References

— R.V. Davies and D.J. Henkel 'Geotechnical problems associated with construction of Chater Station' May 1980 Mass Transportation in Asia.

— K. Terzaghi and R.B. Peck 'Soil Mechanics in Engineering Practice'

— 'Canadian Manual on Foundation Engineering' issued by Associate Committee of the National Building Code, National Research Council of Canada.

— K. Morton, M.S.M. Leonard and R.W. Cater 'Building settlements and ground movements associated with construction of two stations of the Modified Initial System of the Mass Transit Railway, Hong Kong'.

— M. Murayama 'Construction of Hong Kong Mass Transit Railway' in 'Tunnel and Underground' Vol. 13 No. 5 & 6.

— 'Preliminary Information for Prospective Tenderers' for 'Tsuen Wan Extension' October 1977 by MTRC for 'Island Line' February 1981 by MTRC.

— 'Hong Kong Mass Transit Railway Tsuen Wan Extension Prince Edward Station - Stage 2' November 1977 by Freeman Fox & Partners (Far East)

— D.F. McIntosh, A.J.R. Walker, D.J. Eastwood, M. Imamura and H. Doherty 'Hong Kong Mass Transit Railway Modified Initial System: Design and construction of underground stations and cut-and-cover tunnels' The Institution of Civil Engineers, London, England Vol. 68 November 1980.

— S. Honda and A. Tokuda 'Instrumentation for the underground Railway in Fukuoka' in 'Kisokoh' January 1981.

— 'Guidance Notes on Hand Dug Caissons' by The Hong Kong Institution of Engineers.