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Autor: Nozawa, Shinichiro / Kaneko, Shizuo / Matsuda, Takaji

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Rehabilitation Work for the Upheaval Disaster at Underground Station Caused by the Groundwater Rising

Shinichiro NOZAWA
Manager of Civil Engineering Div.,
Tokyo Construction Office,
East Japan Railway Company
Tokyo, Japan

Shizuo KANEKO
Sub Master of a Section of Technical Dept.,
JR East Japan Consultants
Tokyo, Japan

Takaji MATSUDA
Deputy Manager of Civil Engineering Div.,
Tokyo Construction Office,
East Japan Railway Company
Tokyo, Japan

Toshihiko NIIBORI
Assistant Manager of Civil Engineering Div.,
Tokyo Construction Office,
East Japan Railway Company
Tokyo Japan

Summary

An abnormal rise in the groundwater level due to a spell of heavy rain resulted in ground upheaval of up to 1.34m over a 100m section at an underground station operated by the East Japan Railway Co. in the early morning of 12th October, 1991. The underground station is located in a cutting protected by U-shaped retaining walls, which are approximately 12m high and weigh approximately 120tf/m.
The railway line passes through tunnels on either side of the station.

In the restoration work, the elevation of the station structure was reduced by approximately 50cm by lowering the groundwater level. The space underneath the retaining walls was filled with low-strength non-segregating underwater mortar to ensure the stability of the lowered structure. As the station structure itself was undamaged, it was decided that it should be retained in use. In sections where sufficient lowering could not be achieved, the slabs were demolished and the ground was excavated for installation of new U-shaped slabs. As a measure against buoyancy the structure was fixed down with ground anchors to provide it with an adequate resistance even if the water level were to rise as far as the ground surface in future.

1. Introduction

During the night of 11th October 1991, damage was caused by the rain due to Typhoon No. 21 at Shin-Kodaira station on the Musashino line, which is located in a cutting (U-shaped retaining walls) with tunnels on either side. Rainfall had been almost continuous up to this point since 6th October, and the precipitation during this period was 227mm. Prior to this, a cumulative rainfall of 724mm had been observed between 1st August and 30th September, which was more than twice the average rainfall for these two months in the Musashino area. As a result of this rain, the groundwater level showed a sharp rise (to approx. 2.5m below the ground level), and the uplift (buoyancy) due to the groundwater caused upheaval (max. 1.34m) of the U-shaped retaining walls over a distance of 100m. The upheaval, in turn, resulted in a maximum opening of 70cm at the tops of the expansion joints in the retaining walls.

Large quantities of groundwater and earth flowed into the station through these openings, inundating the railway tracks. A major disruption of the passenger and freight traffic resulted during the subsequent two months until the restoration work was completed (Figure 1, Photos 1 and 2).
The report below is concerned with the causes of estimated disaster and design / execution method for the restoration work.

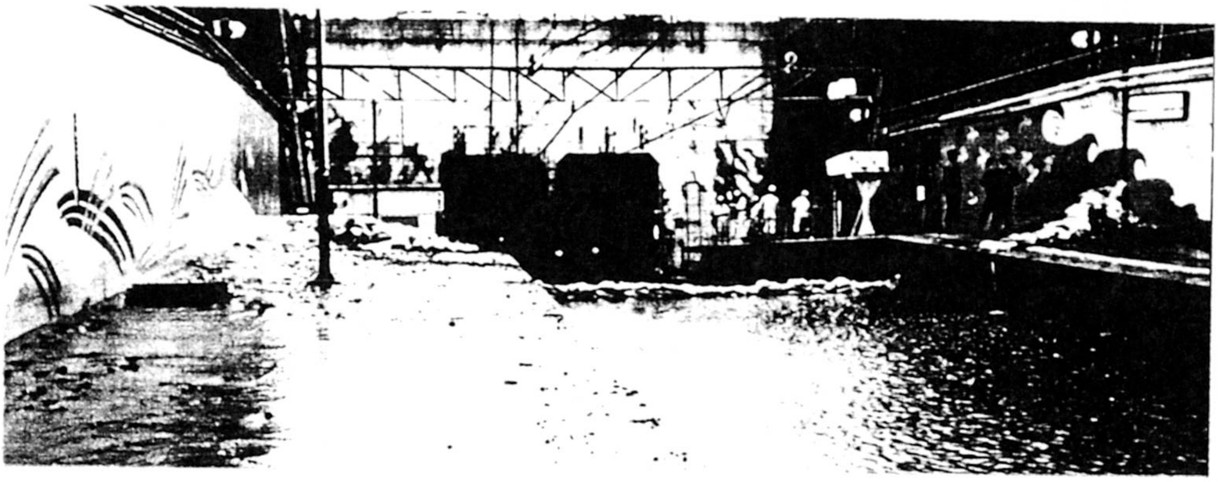


Photo 1 Conditions of Damage

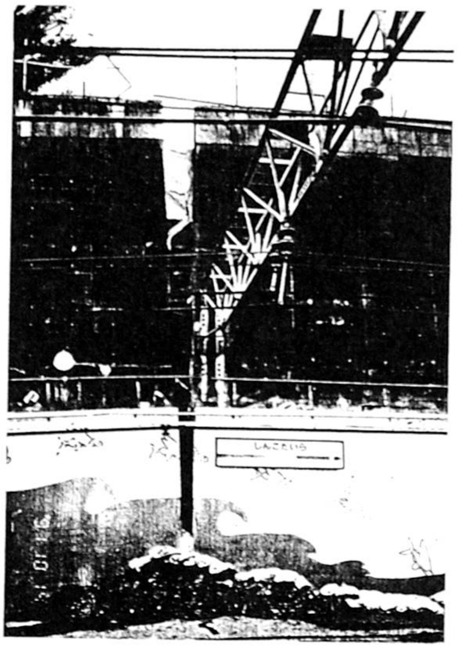


Photo 2 Conditions of Damage

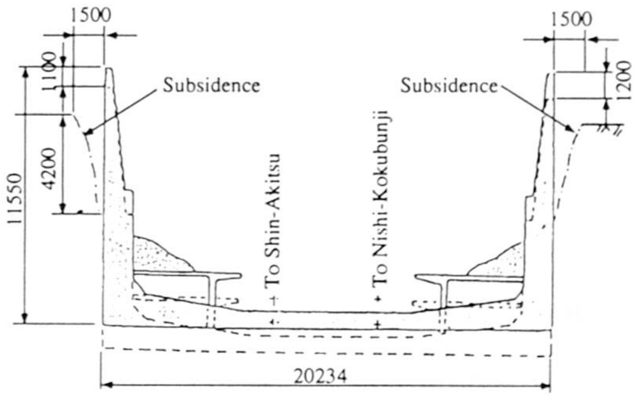


Fig. 1 Conditions of Damage

2. Shin-Kodaira Station

The Musashino line was constructed with the aims of improving the means of transportation by rail to central Tokyo and the connection between the satellite cities , as well as of promoting the development of the nearby areas. It was planned as a new circular line along a 20 to 30km radius from the center of Tokyo to supplement the existing circular line (Yamanote line) and runs from Nishi Funabashi to Fuchu-Honmachi. The construction work, implemented by the Japan Railway Construction Public Corporation, was begun in November 1966, and took approximately eleven years to complete before the opening of the last section in March 1976.
Shin-Kodaira station, opened on 1st April, 1973, is located in a cutting sandwiched between the Kodaira Tunnel (2,563m) on the Nishi-Kokubunji side, and the Higashi-Murayama Tunnel (4,380m) on the Shin-Akitsu side. It was constructed by the cut and cover method, using temporary earth retaining walls with H-shaped soldier piles. There are two side platforms on either side of the two railway tracks, each 6.0m wide and 140m long. The station is used by approximately 15,000 passengers per day (1990).

3. Geological Conditions

The Musashino Plateau, the diluvial plateau on which Shin-Kodaira station is located, is known as the largest plateau in Japan. Around Shin-Kodaira station, the Musashino gravel layer has a depth of 20m, below which are found alternating layers of clay containing shells and of sandy gravel. The Musashino gravel layer is covered with around 5m of Kanto loam soil. Groundwater is found under rather different conditions in the Musashino gravel layer and the sand layers further down. The Musashino gravel layer has a high void ratio and permeability and contains large quantities of groundwater. The groundwater in this area flows south-westwards towards the edge of the Musashino Plateau, crossing below the station (Figure 2).

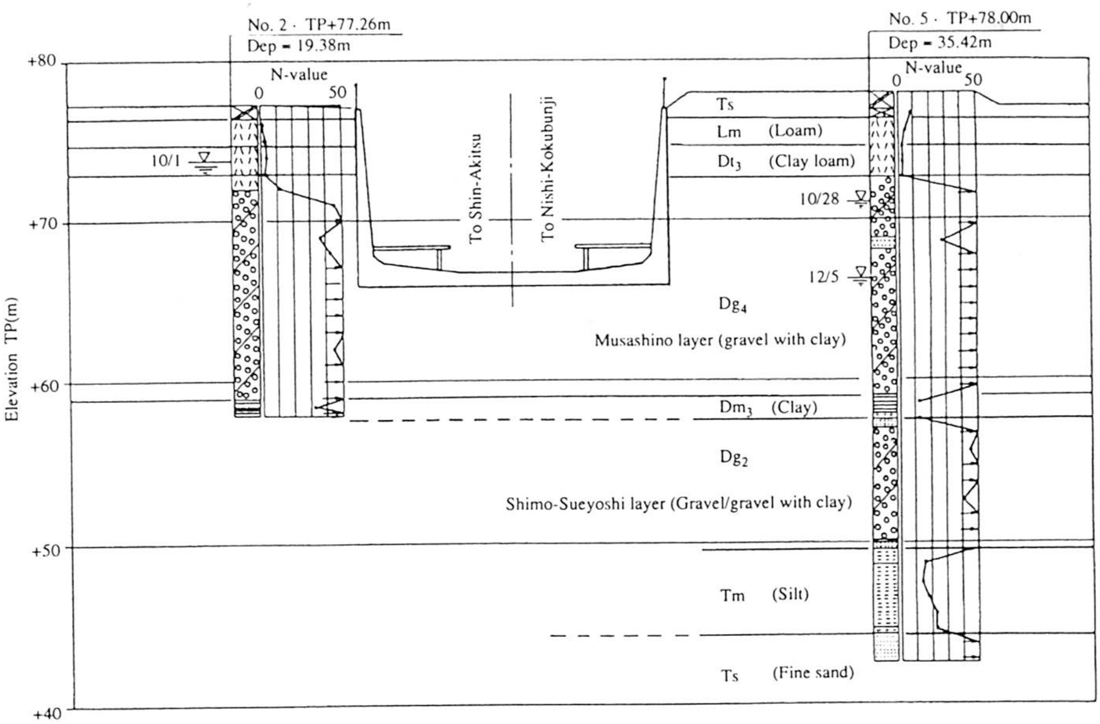


Fig. 2 Geological Profile

4 Restoration Work (outline)

4.1 Original Design

The U-shaped retaining walls that underwent upheaval were 11 to 12m high and were approximately 30m wide in the section supporting the bridge with the station building and 20m wide in the ordinary section. The ordinary section was divided into eight blocks with expansion joints at intervals of 10 to 15m.
As the original calculation documents were not available, calculations were made back from such factors as the weight of the structure (w=117.21 t/m) for the original U-shaped retaining walls approximately 20m in width and 12m in height, and it was estimated that the weight of structure would balance with the buoyancy when the groundwater level was 6m above the bottom of the slabs (5m below the ground surface). A slight margin would have to be allowed in this figure in view of such factors as the friction between the side wall concrete and the soil.

4.2 Estimated Causes of the Disaster (Buoyancy due to an Abnormal Rise in Groundwater Level)

Water quality analysis of the water entering the U-shaped slabs containing the track and platforms showed that it contained no organic components and was of a clearly different composition from that of the water in the nearby river. Therefore the water was judged to be groundwater and not sewage or river water.
Figure 3 illustrates data on groundwater levels gathered by Hosono of the General Center for Fire Prevention Science in Kodaira-Nakamachi, located approximately 1.1km east of Shin-Kodaira station. These observations continued for 24 years between 1968 and 1991. They show an annual cycle with the annual low water level around April rising through summer and autumn to reach the maximum. The highest groundwater level in the 23 years up to the preceding year was approximately 4m below ground level in 1974, but on 13th October 1991 the groundwater level exceeded this record by a further 1.5m, reaching 2.48m below ground level.

Investigation of the relation between the groundwater level and rainfall over these 23 years confirmed that the amount of rainfall over the preceding several months combined with the rainfall over the immediately preceding days produced a rapid rise in groundwater level. A relationship between cumulative rainfall over the preceding 60 and 90 days and the rise in groundwater level was also confirmed. Using 116 years of the precipitation data from the Central Weather Agency in Otemachi, Tokyo (27km away) the period of recurrence of the cumulative rainfall seen in the 60 days and 90 days surrounding the disaster was studied. In either case the recurrence period was 100 years (Tables 1 and 2). Calculations based on actual water level data put the recurrence period at 66 years.

The balance point between the self-weight of the structure of Shin-Kodaira station and buoyancy due to groundwater (neglecting the effects of friction with the soil) is reached when the groundwater level rises to approximately 5m below ground level. Thus it can be inferred that the continuing rain from mid-August to early October raised the groundwater to a level unprecedented in the life of the station and caused buoyant upheaval of the U-shaped retaining walls.

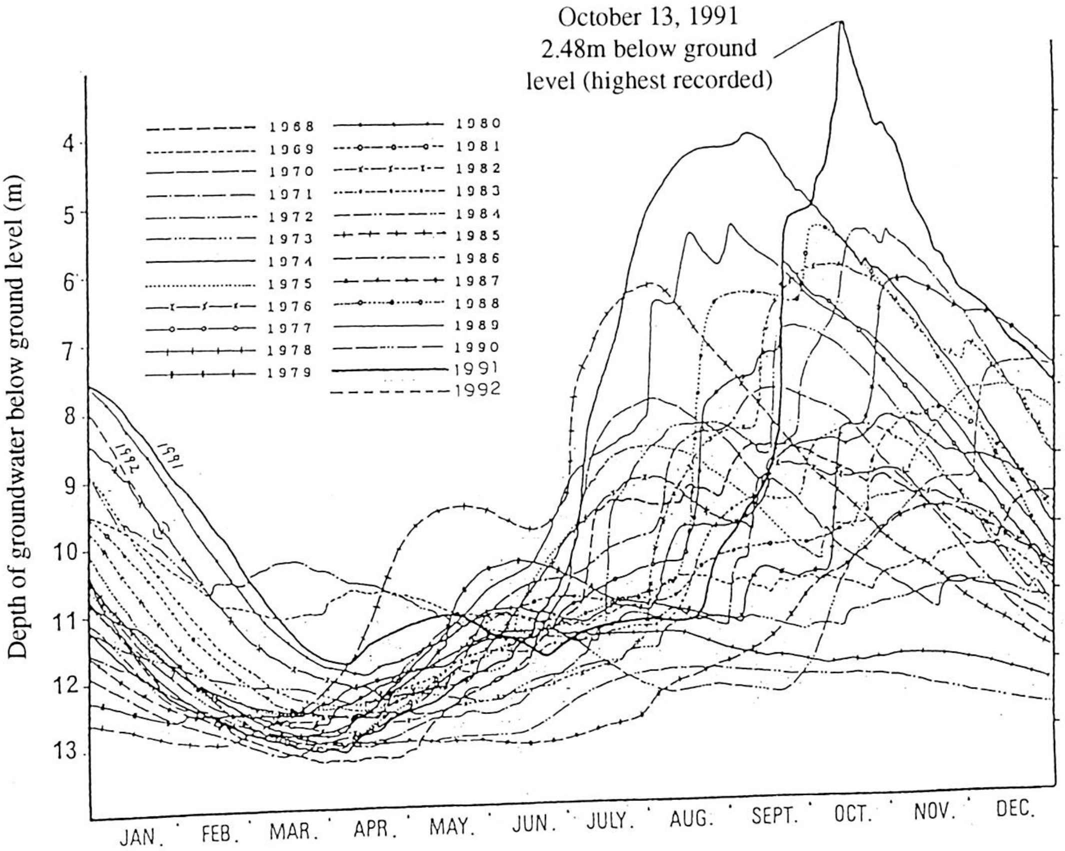


Fig. 3 *Changes in Water Level Over Time in a Well at Kodaira-Nakamachi (produced by Hosono committee member)*

Order	Year	Month	Day	Rainfall (mm)	Exceeding probability (%)	Recurrence period (years)
①	1991	10	17	1036.5	0.60	166.5
2	1938	7	15	958.0	1.25	79.8
3	1958	11	9	957.0	1.26	79.1
4	1941	8	3	940.9	1.47	68.0
5	1911	8	10	894.9	2.26	44.3
6	1929	11	2	867.6	2.91	34.4
7	1925	10	1	842.8	3.66	27.3
8	1921	10	12	785.5	6.16	16.2
9	1966	7	9	785.0	6.19	16.1
10	1924	10	23	740.9	9.17	10.9

Table 1 *The years With the Ten Highest 60-day Rainfall Records, and their Recurrence Periods*

Order	Year	Month	Day	Rainfall (mm)	Exceeding probability (%)	Recurrence period (years)
1	1938	9	3	1281.0	0.45	221.8
②	1991	11	8	1217.0	0.79	127.3
3	1941	8	14	1153.4	1.36	73.7
4	1929	12	1	1073.7	2.66	37.6
5	1958	11	18	1070.0	2.74	36.5
6	1921	10	20	1047.6	3.30	30.3
7	1911	8	21	1032.8	3.73	26.8
8	1925	10	18	978.3	5.81	17.2
9	1910	10	21	956.9	6.89	14.5
10	1920	11	1	938.5	7.96	12.6

Table 2 The years With the Ten Highest 90-day Rainfall Records, and their Recurrence Periods

4.3 Restoration design

Although there was little damage to the U-shaped retaining walls themselves, the gradient of the railway line was altered from 8 per thousand to 21 to 35 per thousand by the upheaval. This gradient posed difficulties in the operation of freight trains and had to be corrected. It was decided to lower the existing displaced U-shaped retaining walls as far as possible, in order to restore the station as quickly as possible and secure adequate platform space.

4.3.1 Investigations on Restoration Method

The alternatives for the restoration methods included reconstruction, lowering and rail-level alteration. Since an early reopening of the Musashino-line was desired in view of the importance of the line as a major freight and commuter route, the lowering plan was adopted, and it was decided that where adequate lowering could not be achieved, the track sections of the lower slabs should be demolished and removed for construction of new shallow U-shaped slabs. As a measure against future rises in the groundwater level, the structure was to be fixed down with ground anchors to raise its safety.

4.3.2 Lowering of Structural Frame

Reduction of the buoyancy through lowering of the water level was chosen as the method for lowering the structural frame, as this was judged to be the quickest and most efficient method. Deep wells were arranged to lower the water level, seven inside the U-shaped retaining walls (ℓ =7m, Ø=650mm, lifting pipes: 100A, high lift underwater pumps: 15kw) and nineteen surrounding the outside of the retaining walls (ℓ =20m, Ø=700mm, lifting pipes: 100A, high lift underwater pumps: 15kw). Pumping began gradually on 22nd October and continued until 30th November, after the completion of the new slabs. There had been no past cases, however, where complete lowering of the structural frame was achieved after an upheaval as large as 1.3m and there were concerns as to whether adequate lowering of the water level could be achieved through use of deep wells. (Figure 4).

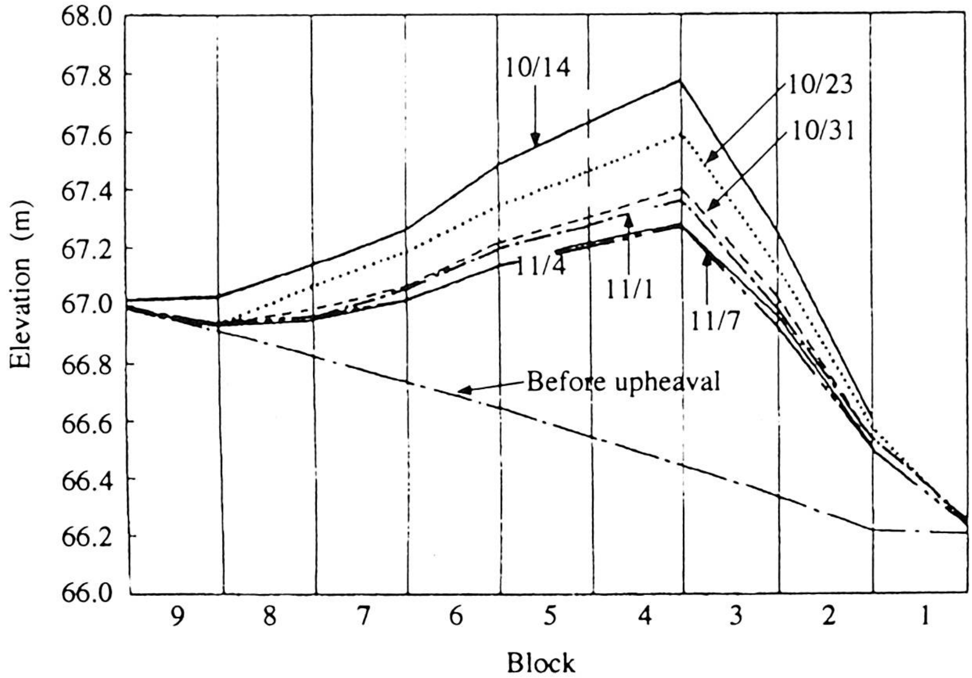


Fig. 4 Elevation of Lower Slabs (Slab Crests) : Measurement Results

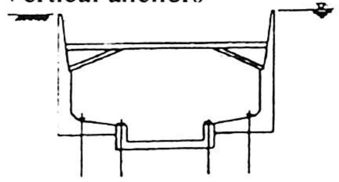
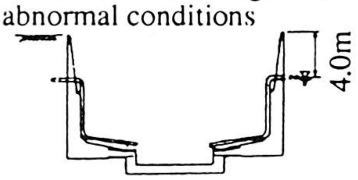
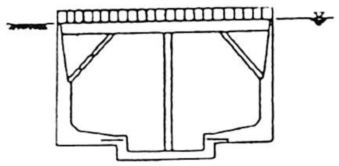
	Measure	Remarks	Evaluation
Alternative 1	Vertical anchors 	• Need for construction of reliable vertical anchor • Short execution period	○
Alternative 2	Water level lowering under abnormal conditions 	• Maintenance of drainage equipment required to retain its durability • Piping required within the station structure	△
Alternative 3	Additional load 	• Reinforcement of side walls required for placement of additional load on U-shaped retaining walls • Stable appearance • Long execution period	△

Table 3 Comparison of Measures Against Buoyancy

4.3.3 Design of Structural Frame

For the final form of the structure after the restoration work, the following three alternatives (Table 3) were considered as methods for ensuring stability, to prevent repeated upheaval of the restored structure in the event of future rises in the groundwater level up as far as the ground surface level.

1) Fixing to position with earth anchors
2) Prevention of buoyancy under abnormal conditions through drainage
3) Provision of additional load on structure

The use of earth anchors was selected out of the three methods as the most reliable and the fastest to construct.

The analysis model of the structural frame is shown in Figure 5. In the analysis, four rows of vertical anchors were installed to provide resistance to buoyancy and struts were installed on the upper parts of the structural frame. Through a comparison of the construction periods required, roller bearings were selected for use in the connections between the old slabs and sections where excavation was to be conducted below the slabs.

The analysis results are given in Table 4. It was confirmed that the stress generated in the lower slabs would not exceed the resistance stress. At the connections between the old and new slabs, taking into account the fact that complete roller bearings and pin bearings cannot be created in concrete structures, reinforcement was provided with steel rebars and anchors to prevent opening and dislocation. The details are given in Figure 6. To ensure early generation of strength, the concrete strength was set at $\sigma 28 = 360 kgf/cm^2$ in the mix design.

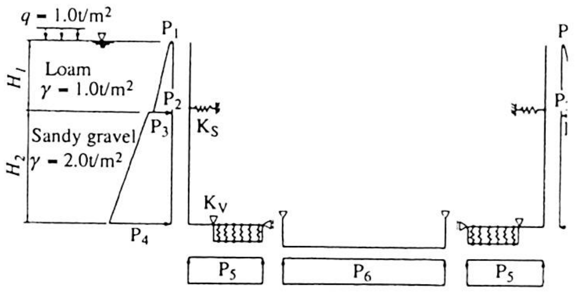


Fig. 5 Analysis Model

Block	2		3		4	5	6	7
	Resistance stress	Stress generated	Resistance stress	Stress generated				
Moment (t·m)	9.4 1.7 29.9 51.4 173.7 234.8 9.2 263.5 558.2 421.3	a = 6.01 b = -6.81 c = 72.85 d = 76.65	10.6 2.2 44.6 5.0 118.9 256.6 14.3 437.1 325.2	a = 6.48 b = -14.14 c = 49.51 d = 52.32	6.94 -12.90 49.66 52.00	7.91 -11.57 51.06 53.35	3.51 -14.34 78.86 82.69	3.82 -14.34 81.05 84.38
Shear force (t)	14.1 28.8 49.8 61.9	a = -11.46 b = 25.88 c = -35.46	13.5 19.5 21.4 49.6 60.8	a = -11.46 b = 25.88 c = -35.46	-15.87 23.66 -14.54	-15.87 22.75 -21.36	-11.22 29.63 -37.29	-11.63 29.29 -37.89

Table 4 Structural Stress

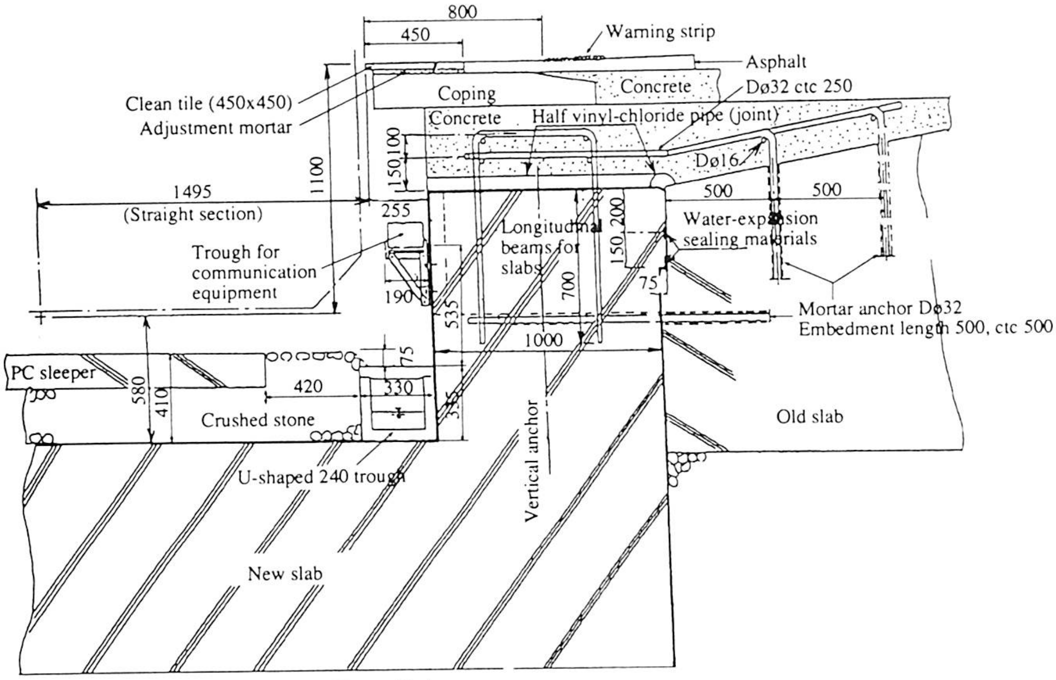


Fig. 6 Connection (Details)

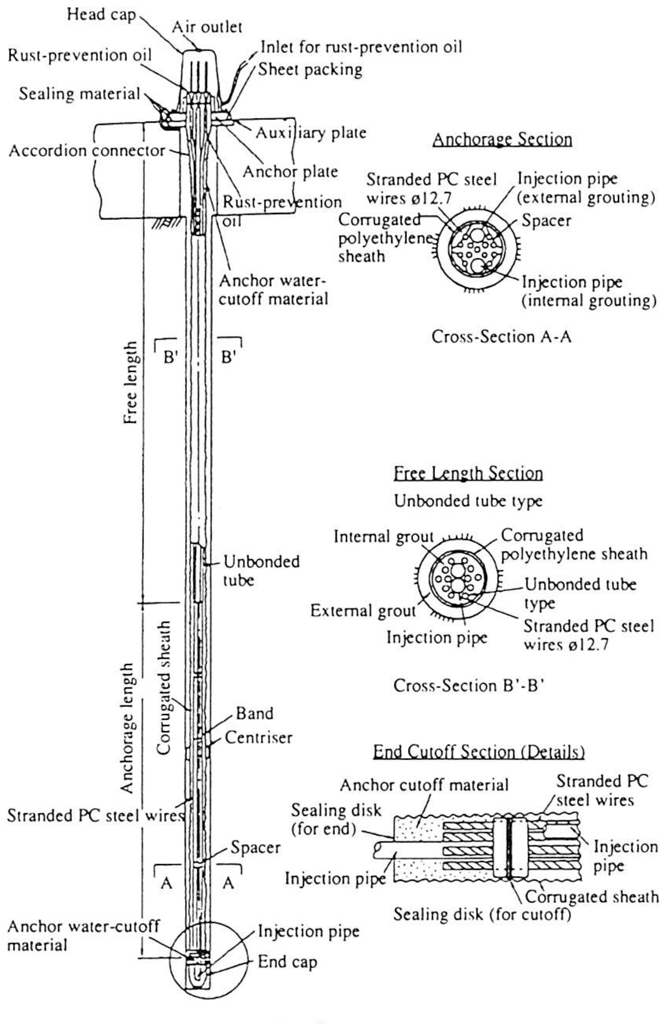


Fig. 8 Anchor Tendon

4.3.4 Design of Vertical Anchors

Permanent anchors aimed at prevention of uplift have been used in construction of buildings and, in recent years, have also been applied to railway facilities.

The VSL method was selected as the method for construction of the permanent anchors, which were made of twelve stranded PC steel wires, Ø12.7mm in diameter. The anchors were given an anchorage length of 10m in the gravel layer and a free length of 5m. The design anchoring force (per anchor) was 134tf for the side wall anchors (pitch: 4.0m) and 92tf for the anchors for new slabs in the middle (pitch: 2.0m). The details of the anchor tendons are given in Figure 7.

4.3.5 Design of Struts

After the completion of the restoration work, the U-shaped retaining walls would comprise a composite structure consisting of the original L-shaped sections and the newly constructed U-shaped sections. It was decided that struts should be provided in the L-shaped sections which were liable to destabilization through overturning at times of reses in the water level.

The struts were to be approximately 19m long. As a measure relating to the buckling stress intensity, angle braces with hinge metals at both ends were installed at points approximately 4.4m away from the two ends of the struts to reduce the buckling length. The struts were to be connected to each other with longitudinal beams to prevent sideways movement due to vibration and to ensure structural stability. The members used for the struts and angle braces were H 400x400x13x21 H-shaped SS 400 steels galvanized by hot-dipping.

4.4 Execution

The cross section after restoration and the restoration procedure are shown in Figures 8 and 9, respectively.

4.4.1 Lowering of Groundwater Level

Deep wells were used to lower the groundwater level. In this method, the permeability coefficient provides the basis for the design. Valid values for the permeability coefficients were not available at the initial stage of the design, but a value of $k=7.69\times10^{-3}$ (m/sec) was obtained by a deep well pumping test and used in the planning of the water level lowering.

The required reduction in the water level was calculated by the Theis' method and used for the determination of the number and pumping capacity required of the deep wells. Nineteen deep wells were created on the periphery of the U-shaped retaining walls and seven inside the retaining walls to lower the water level. During the water level lowering, inflow of earth was expected in the parts below the ends of the old slabs, which had been raised by the buoyancy of water. It was found that as buoyancy reduced, the old slabs would be supported from their ends by this earth, in which case they would be ruptured by the combined action of the weights of the old slabs themselves and the lateral earth pressure from the side walls. For this reason, temporary supports were constructed inside the U-shaped retaining walls to provide reinforcement (Photo 3).

The permeability coefficient calculated after the lowering of the water level from the reduction in the water level and the number of days required was $k=7.0\sim 8.0\times10^{-3}$ (m/sec), indicating that the value obtained through measurements in the deep well was close to the actual value.

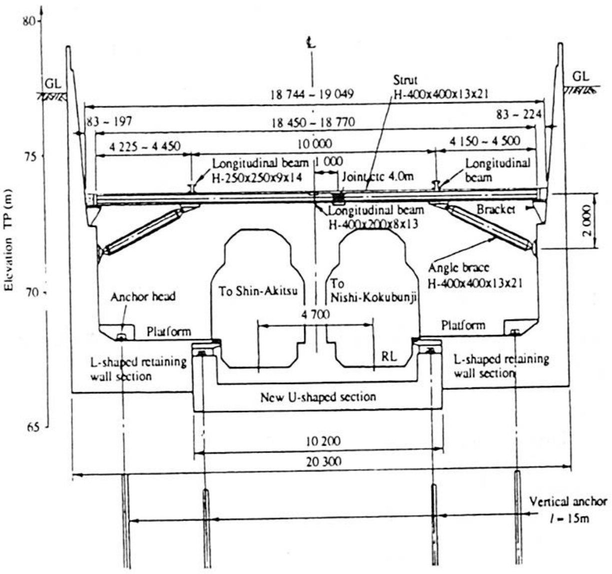


Fig. 8 Restoration Work: Typical Section

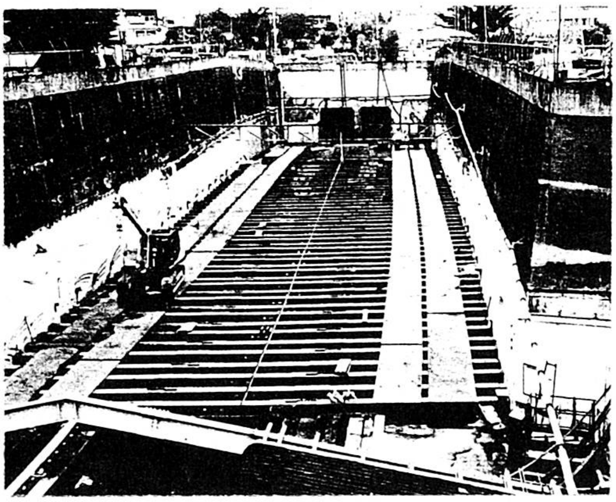
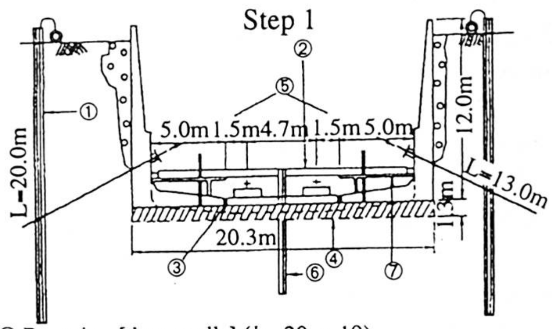
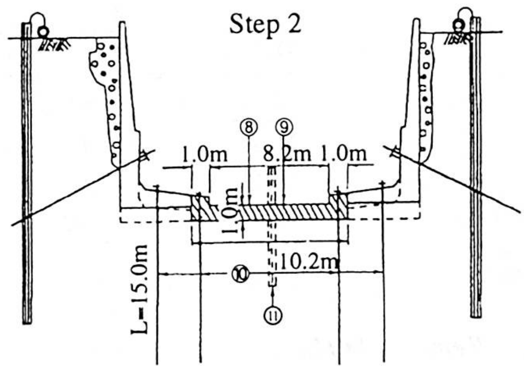


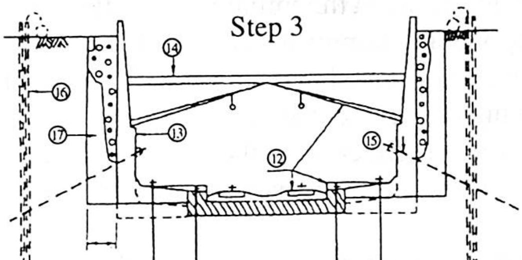
Photo3 Temporary supports



① Pumping [deep wells] (l = 20, x 19)
② Temporary supports (Blocks 2 to 8)
③ Coring of lower slabs for drainage (ø 50 mm)
④ Concrete grouting below slabs (coring ø 150 mm, x 24)
⑤ Removable horizontal anchors (ctc = 2.5 m, x 70)
⑥ Pumping at centre of slabs (l= 5 m, x 7)
⑦ Removal of supports and platforms



⑧ Demolition of reinforced concrete slabs (width: 10.2 m, depth: 0.8 to 1.0 m)
⑨ Construction of new reinforced concrete slabs
⑩ Vertical anchors (x 150)
⑪ Removal of pumping works at centre of slabs



⑫ Restoration of tracks, platforms and station building
⑬ Repair of retaining walls and tunnels
⑭ Construction of supports (Blocks 2 to 7)
⑮ Removal of horizontal anchors
⑯ Removal of pumping works
⑰ Grouting behind retaining walls

Fig. 9 Procedure for Restoration Work

4.4.2 Concrete Grouting Underneath Slabs

The progress of the water level lowering and lowering of the structural frame was monitored, and concrete was injected into the space underneath the slabs once a drop was observed in the rate of lowering. Utmost care was taken in the determination of the timing of the concrete grouting underneath the slabs, as this had a major effect on the subsequent full-scale restoration work. The grouting was injected through Tremie pipes installed on the slabs. The uniaxial compressive strength of the concrete was set at σ=80kg/cm^2, bearing in mind that a part of the concrete would need to be removed at a later stage. The water level was still above the level of the slabs at the time of the concrete grouting., which as a result took place under pressurized conditions.

To ensure satisfactory filling performance and resistance to segregation under water, non segregating under water concrete was used in the grouting, with the mix proportions determined through tests. The mix composition is given in Table 5.

Cement	W/C	S/a	Unit contents			
	(%)	(%)	W	C	S	G
Ordinary	83.3	45	250	300	720	902

Chemical admixtures (kg/m^3)			Slump flow (cm)
Non-segregation agent	Fluidiser	Air-entraining agent	
3.0	10	0.6	60 x 60

Table 5 Grouting under Slabs: Concrete Mix

4.4.3 Removable Horizontal Anchors and Demolition of Lower Slabs

Before demolishing the lower slabs, seventy removable horizontal anchors 13.0m in length (Ø= 12.7mm, nine-stranded PC steel wires) were first installed at 2.5m pitches to stabilize the L-shaped sections. The design anchoring force was 100t.

The lower slabs were demolished after the removal of the obstructions such as the temporary supports and platforms. As the old slabs below the platforms were to be retained as they were, they were isolated from the demolition section by concrete coring along the boundary. Concrete coring was selected as it was possible to use a large number of coring machines at one time, allowing a reduction in the time required. After isolation, the concrete in the demolition section was crushed with giant breakers and removed from the site.

4.4.4 Slab Concrete and Vertical Anchors

After creating a shallow sump on the Nishi-Kokubunji side of the station, slab concrete was placed and vertical anchors were installed according to the predetermined sequence. The mix for the slab concrete is given in Table 6.

Nominal strength (kg/cm^2)	Cement	W/C (%)	S/a (%)	Slump (cm)	Unit contents(kg/m^3) W	C	S	G	Chemical admixture (kg/m^3)
360	Ordinary	42.1	42.0	8±2.5	157	373	749	1,060	3.79

Table 6 Slabs: Concrete Mix

Besides the 150 anchors used in the main anchoring work, four were installed for measurement control and two for pullout tests, adding up to a total of 156 anchors. Thorough care was taken in the quality control of the anchors, with standard tests (pullout tests) carried out on three anchors and strength confirmation tests on five of the 150 anchors. Checks were also made on the rest of the anchors, the results of which were found to be satisfactory.
Stronghold anchors were used in blocks 1, 8, 9 and 10, while VSL anchors were used in blocks 2 to 7. Water-expansion sealing materials with high cutoff performance were used at the connections between the old and new structural frames to guard against a return of high groundwater (Photo 4 and Figure 10).

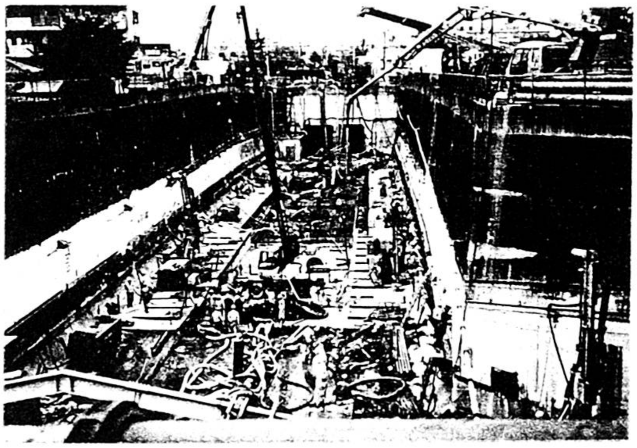


Photo 4 Restoration Work

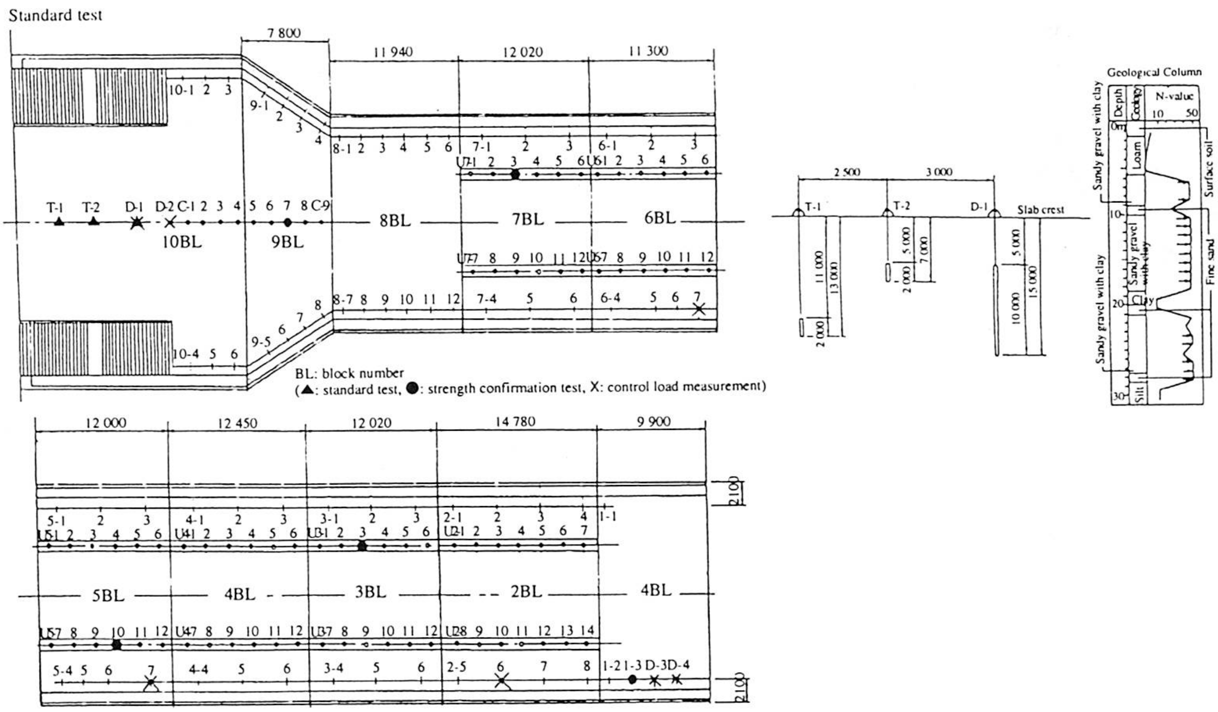


Fig. 10 Vertical Anchor Test Sites

4.4.5 Control Load Measurement on Vertical Anchors

To allow control of the tension in the vertical anchors, load measurement equipment has been installed at seven sites and measurement has been taken on these since the completion of the restoration work. Some of the measurement results are shown in Figure 11.
Forces larger than the design anchoring force were applied as the tension in the vertical anchors to make allowances for such factors as creep. The tension applied initially varies according to positions of the anchors. It was 144tf per anchor in the ordinary side wall section and 106tf per anchor in the central section.

The initial values given above are those at the time of the completion of the restoration work. A slight lowering can be observed in the tension both in the side wall section and the central section after the passage of one year.

The design anchoring force was set at a value that would provide sufficient resistance when the groundwater level reached the ground surface level. The groundwater level being lower, the tension is smaller than the design anchoring data at present.

4.4.6 Full Scale RestorAtion Work and Strut Construction

Full scale restoration of the platforms, rail tracks and station building was implemented along with the restoration of power and communication equipment. At the same time, liquid containing suspended water glass (LW) was injected into the ground behind the U-shaped retaining walls to fill the gaps left by the outflow of earth and for cutoff of water flowing towards the station structure. Struts were also installed at 4.0m pitches as reinforcement for the retaining walls in preparation for future abnormal rises in the groundwater level, thus completing the restoration work (Photo 5).

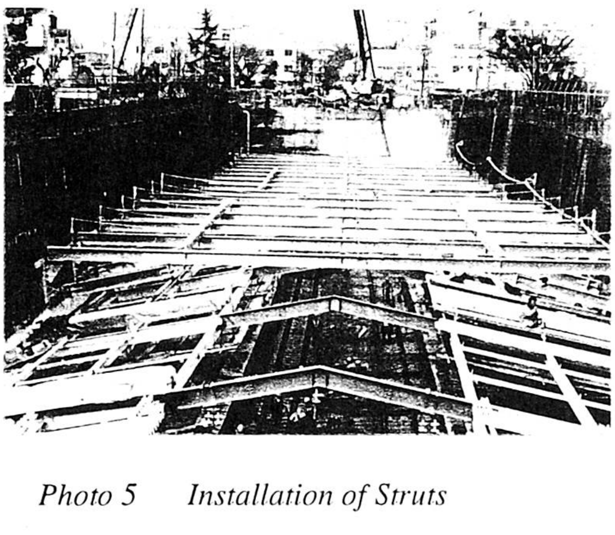


Photo 5 Installation of Struts

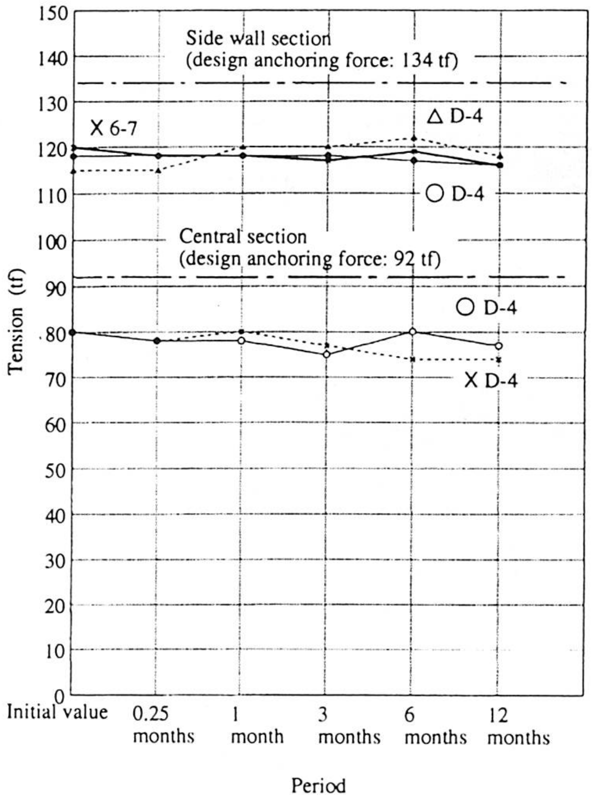


Fig. 14 Vertical Anchor Load: Measurement Results

5. Conclusion

The damage due to ground upheaval caused by Typhoon No. 21 led one to recognize anew the dangers of the natural force of buoyancy. The total damage on the Musashino Line, a major artery for commuter and freight traffic, is estimated at ¥8,600 million. The line was reopened after two months of round-the-clock restoration work, with the co-operation of the agencies concerned and the local residents.

Finally, the authors would like to take this opportunity to express their gratitude to all those who helped in one way or another. We hope this report will help to some degree in preventing a recurrence of this disaster.

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