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A study on the method for design and construction management considering strain level of ground during excavation

Etude sur la méthode pour la gestion de conception et de construction prenant en considération le niveau de contrainte du terrain pendant l'excavation

K. Takahashi
Japan Railway Construction, Transport and Technology Agency, Yokohama, Japan
Y. Okochi
NOM Co. Ltd. Tokyo, Japan

ABSTRACT
Rinkai Oimachi station was constructed by excavating and connecting between shaft and shield tunnel underground. 2D and 3D FEM were carried to evaluate the deformation and member forces during construction. As a result, following conclusions were obtained. 1) Linear elastic FEM using the deformation modulus of PS/2 showed acceptable result. 2) Nonlinear elasto-plastic FEM did not show better result than linear elastic case. 3) Practical procedure to improve the accuracy of linear elastic analysis considering ground strain level was proposed.

RÉSUMÉ
La gare de Rinkai Oimachi a été construite par l'excavation et le raccordement entre le puits et le boucllier du tunnel souterain. Les méthodes des éléments finis bidimentionnelle et tridimentionnelle ont été exécutées pour évaluer les forces de déformation et celles excerçant sur les éléments pendant la construction. En conséquence, les conclusions suivantes ont été obtenues. 1) La méthode des éléments finis d'élasticité linéaire utilisant le module de déformation de PS/2 a indiqué un résultat acceptable. 2) La méthode des éléments finis élastoplastiques non-linéaires n'a pas indiqué de résultat meilleur que le cas d'élasticité linéaire. 3) Une procédure pratique d'améliorer la précision de l'analyse d'élastique linéaire prenant en considération le niveau de contrainte du terrain a été proposée.

1 INTRODUCTION

Rinkai Oimachi station was constructed by removing part of segments of shield tunnel and excavating the soil between tunnel and the adjacent shaft because of the restriction of construction site. The diameter of the shield tunnel was enlarged from 7.2m to 10.3m for this purpose. Also the construction work had to be done very carefully because of the neighbouring construction that both the distance of the tunnles from the central shaft and the elongation between two tunnels were 1m. As the depth of excavation exceeded 40m, we didn’t have much experience of similar situation of construction.

As the pore pressure of Tokyo gravel was xpected more than 300kN/m^2, a counter mesures for the water pressure was regarded most important for this construction. In addition, though the final force of the members were estimated in original design, the behavior during construction was not investigated. So, the indecies for the construction management utilising fileld mesurement were not clalified. In order to configure the indecies for the safety construction, 2D and 3D FEM were carried out. Eventually, the construction work completed safely. This paper shows the comparison between observed data during construction and analized data by FEM before and after construction.

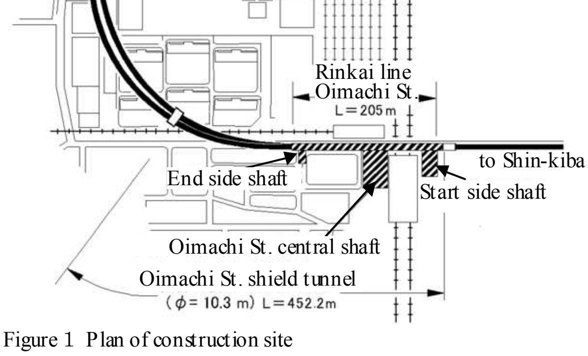

Figure 1 Plan of construction site

2 OUTLINE OF THE CONSTRUCTION WORK OF RINKAI OIMACHI STATION

The construction site of Oimachi station tunnel is 452.2m long as shown in Figure 1. Usually, station is constructed by open cut. However, surface of Oimachi station is heavy traffic road and its width is only 15m. In addition, there are commercial buildings at both sides of the road. Because of this situation, constructing station was planned as follows. Only platforms and railways were located under the road and the concourse and other passenger facilities were planned to locate under the existing station square by constructing shaft first, then they were connected by underground excavation.

There were three underground connections in this site. This paper describes the construction at the central shaft. Soil profile and the location of tunnels are illustrated in Figure 2 and listed in Table 1. Almost of the significant layers are diluvium, Musahino gravel, Tokyo clay, Tokyo gravel, Edogawa sand were underlain below Loam. Beneath the diluvium layer, Pleistocene of Kazusa layer is underlying. The outbound tunnel is located in Tokyo clay which N-value is around 10 and stable layer.

Table 1 Soil profile of the site

Geological time	Stratum	Symbol
	Surface	F
Diluvium	Tachikawa・ Musashino loam	Lm
	Loam	Lc
	Musashino gravel	Mg
	Tokyo clay	Dc3
	Tokyo gravel	Dg2
	Edogawa sand	Ds2
Pleistocene	Kazusa	Ka

However, as the inbound tunnel is in Tokyo gravel that contains boulders which diameters are around 250~350mm and shows high water pressure, excavation in this layer had to be cautiously done.

3 PRIOR ANALYSES BY FEM

Original design method of the underground excavation was to check the member force by using beam-spring model applying total earth pressure. Therefore, only the final forces of the members were available.

For the simpler case, indices for construction management are often specified multiplying some ratio against the final force. However, in this case, as the installing procedure of the members were complicated, it was regarded very important to obtain the overview of the member force during construction.

For this purpose, step analyses simulating the construction process by using FEM were carried out. Considering the excavated part was in diluvium layer which strength and deformation modulus seemed high, small ground strain was expected. For this reason, linear elastic model was used. Mainly, 2D FEM was used and 3D analyses were carried out to investigate longitudinal effect of the tunnel. The cross section of excavation part of central shaft is illustrated in Figure 3.

The segments, horizontal braces and inner columns were modeled by beam elements in 2D case and only eight-node brick element was used for 3D. As plane-strain was used in 2D, central part of the shaft was modeled. EI and EA of the beam elements were converted to equivalent values of unit thickness. As all members of support were modeled by the solid element

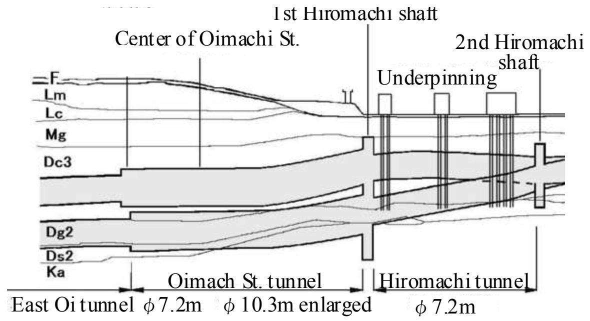

Figure 2 Soil profile and tunnel location

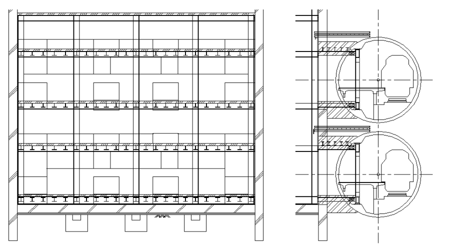

Figure3 Cross section of central shaft and tunnels

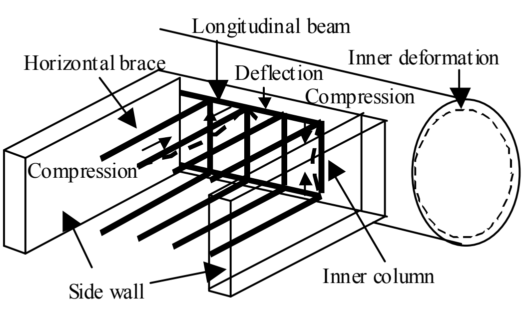

Figure 4 Deformation pattern during excavation

Table 2 Soil properties

Soil	Thickness m	N: SPT	ρ t g/cm^3	E kN/m^2		ν
Loam	10.7	5	1.36	3500*2	90000*4	0.48*4
Musashino gravel	4.1	50	1.90	35000*2	480000*4	0.3
Tokyo clay	17.1	10	1.87	28000*3	370000*4	0.48*4
Tokyo gravel Edogawasand	28.1	50	1.89	140000*3	1470000*4	0.3
Grouted part*1	-	-	1.89	-	3000000*4	0.3

Note *1:SPT data unavailable *2:E=700N(kN/m^2) *3:E=2800N(kN/m^2) *4:PS logging

in 3D, it was impossible to set both EI and EA equal at the same time in the case of the members were hybrid or hollow structure. EI of the members were set the same as real in the case that flexural rigidity was significant such as horizontal and longitudinal beam, on the other hand, EA was set equivalent for the compressive or extensive members. The deformation pattern of the members is illustrated in Figure 4.

Deformation modulus for soil is more significant than structural members. Though an empirical relationship such as E=700N is widely used, following approach was adopted considering dominant layers were diluvium and assumed strain would be small.

1) The case adopting conventional method using empirical equation for SPT blow count N.
2) The case using deformation modulus obtained by PS logging

The strain level of the modulus using empirical equation was regarded as 10^{-0}~10^{-1} (JGS, 1999), experience shows that resulting deformation was often greater than observed. In order to obtain the range of the deformation, the latter case, which strain level was 10^{-6}, was carried out. The input parameters of the ground are listed in table 2.

The analysis procedure was as follows. Note that process 4) and 7) were unable to be simulated by 2D analysis.

1) Self weight analysis.
2) Install the shaft and excavate.
3) Install the segments of the tunnels and excavate inside.
4) Install inner columns and longitudinal beams.
5) Excavate first part, remove segments and install part of lower horizontal braces for the excavated part.
6) Excavate second part, remove segments, install upper horizontal braces, refill.
7) Install sidewall of outbound tunnel.
8) Excavate third part and remove segments.
9) Install rest of the lower beams and refill.

The two-dimensional mesh is shown in Figure 5. The number of elements is 1150 and the number of nodes is 2371.

Three-dimensional mesh is also shown in Figure 6. The number of elements is 27475 and the number of nodes is 29666. The tunnel part is enlarged in Figure 7.

Comparison between maximum values of member force of analysis results by FEM and those of original beam-spring model is listed in Table 3. Almost of the maximum values of FEM were obtained from the case of 2D analysis using empirical equation. In 3D case, the values were 15~95% of those by 2D that meant 2D gave the safer side results. Considering the results, criteria for execution management and planning of observed construction were determined.

The main section for field observation including the axial

Table 3 Comparison of loads between designed and analyzed

	Designed	2D	3D
Axial force of column (Outbound)	21000	8800	7000
Axial force of column (inbound)	27000	11700	11200
Axial force of horizontal brace (Outbound)	9000	2200	1200
Axial force of horizontal brace (Inbound)	12000	2500	400

Unit kN

force of horizontal brace, ground feformation, earth pressure and so on was located in the middle of central shaft because maximum deformation would be expected. The items easily mesured such as deformation of segments and axial force of inner colums were measured in several sections.

In addition, a simple ground strain meter was developed (Takahashi et. al., 2003). It is very important to choose appropriate stiffness of backfill material when installing this kind of strain meter and earth pressure gauge into borehole. If the stiffness of the backfill is too high compared to the ground, a concentration of stress might take place or on the contrary, only small strain might be mobilized in the backfill. On the other hand, if it is too low, the backfill might not be able to transfer the change of stress, the sensors would measure too small strain. Considering above situation, the stiffness of the backfill was determined as follows.

1) Cement-bentonite was adopted and deformation modulus was measured by uniaxial loading test.
2) In order to evaluate the value of small strain level, LDT (local displacement transducer) (JGS, 2000) was utilized.
3) The target stiffness was about half of measured value by PS logging.

As a result, the stiffness measured by LDT at small strain level was about 10 times grater than E_{50}. Also, it was confirmed that E_{50} was about same level as the empirical equation of N-value in this site (Matsumoto et. al., 2003).

The measured data by the ground strain meter was shown in Figure 8. The crown of the outbound tunnel was about 23m deep and the invert of the inbound tunnel was about 45m. As observed in this figure, order of the measured strain was smaller than $2X10^{-3}$. This means the strain level could be regarded as the intermediate between empirical equation for N and PS logging. This fact also suggested the possibility of better deformation modulus for the analysis.

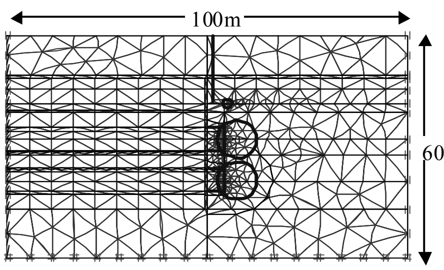

Figure 5 Two-dimensional mesh

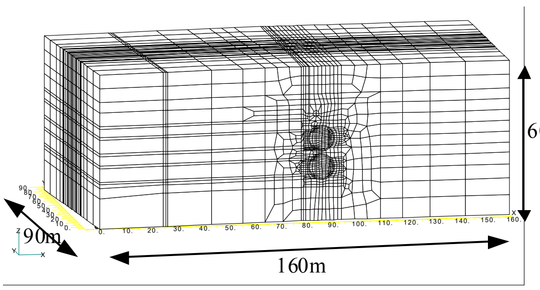

Figure 6 Three-dimensional mesh

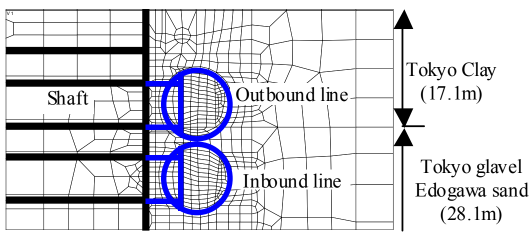

Figure 7 Three-dimensional mesh (Tunnel part)

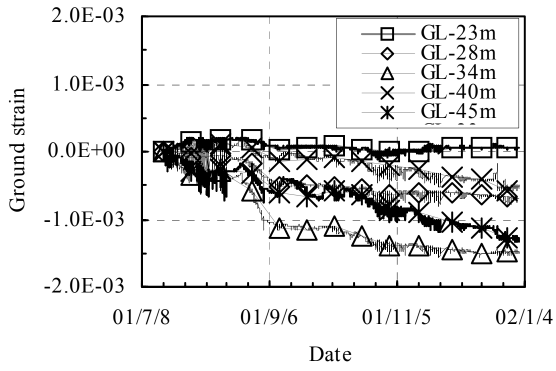

Figure 8 Measured data of ground strain meter

4 COMPARISON BETWEEN ANALYSIS AND OBSERVED DATA

Though observed data during construction were within the expected range, for the purpose of improve the predictive accuracy, post analyses were carried out by two-dimensional FEM changing deformation modulus and adopting advanced constitutive model of the soil as following. The mesh used was the same as shown in Figure 5.

1) Specifying method for the deformation modulus of linear elastic analysis.
2) Using nonlinear constitutive model that includes the effect of the strain and stress dependency of the deformation modulus.

1) intended to improve the result of simple linear elastic analysis because from the practical point of view, simpler model is often preferable to complicated constitutive model. Considering the observed data of ground strain were around 10^{-3}~10^{-4}, half of the values obtained by PS logging were used as the deformation modulus.

For the nonlinear analysis, hardening soil model (refer HS afterwards), which was proposed by Shanz (PLAXIS B. V., 2002), was used. This model is like a combination of Mohl-Coulomb and Duncan-Chang. Virgin loading curve is defined by the hyperbolic function and stress dependency of modulus is defined as the power function of the ratio between current and reference stress. The soil parameters used for HS are listed in Table 4. Only the shaft was modeled by linear elastic. As E_{50} is defined as half of E_0 by this model, E_{50} was set to the half of the values obtained from PS logging. The stress strain curve for Tokyo clay was fitted to those results of triaxal test. The parameter m for specifying stress dependency of the modulus was set to 0.5 which is commonly used (Jamiolokowski et. al., 1995), because there were no test results.

Comparison of the axial forces of inner columns is illustrated in Figure 9. Apparently, the force is overestimated in the N-value case. Comparison of the horizontal displacement of the tunnels is shown in Figure 10. Except at the crown of the outbound tunnel, analyzed results are too small, which means the results were in danger side. It is supposed that the reason was derived the actual constructing process such as setting the

Table 4 Soil parameters for hardening soil model

Soil	Thickness m	N: SPT	γ_t kN/m^3	E_{50} kN/m^2	E_{oed} kN/m^2	C kN/m^2	ϕ °	ν
Loam	10.7	5	13.57	50000*1	37000*1	80*1	17*1	0.48
Musashino gravel	4.1	50	19	240000*2	144000	0.1*3	42*3	0.3
Tokyo clay	17.1	10	18.65	100000*1	59000*1	130*1	17*1	0.48
Tokyo gravel Edogawa sand	28.1	50	18.87	735000*2	441000	0.1*3	42*3	0.3
Grouted part	—	—	18.87	1500000*2	900000	0.1*3	42*3	0.3

Note:*1:Triaxal test, *2:1/2PS logging, *3:Empirical equation for SPT horizontal braces could not be done as ideal as analysis. Especially, HS shows smaller deformation at the center of the tunnel than PS case. Comparison of vertical displacement of the tunnels is illustrated in Figure 11. Analysis results show better coincidence than horizontal displacement. At the crown of inbound tunnel, analysis showed danger side results, among those, HS model showed best result.

From those results, following conclusion are obtained.

1) For linear elastic analysis, the results are highly affected by the deformation modulus. The better results were obtained in the case used half values of the PS logging.
2) For the excavation in diluvium layer and it is supposed that the strain level of the ground is low, adopting nonlinear model, which is able to consider the characteristics of real soil including strain and stress dependency of modulus, is not effective to improve the accuracy of the analysis.

In general, nonlinear model needs incomparably more complicated input parameters than linear elastic and increase computational cost significantly. Therefore, if the expected strain level of the ground is up to around 10^{-3}, a practical approach would be linear elastic analysis with well-chosen deformation modulus.

Though the assumed strain level for the analysis using empirical equation of N-value was 10^{-0}, the result was around 10^{-3}. Meanwhile, the result of PS logging, which strain level was 10^{-6}, was around 10^{-4}. The deformation modulus of half of PS logging was in between. It means that above mentioned procedure for linear elastic analysis has the possibility to improve the accuracy of prediction. The flow-chart of this approach is illustrated in Figure 12.

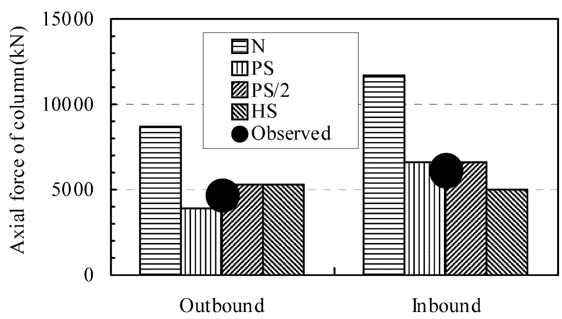

Figure 9 Comparison of axial force of the columns

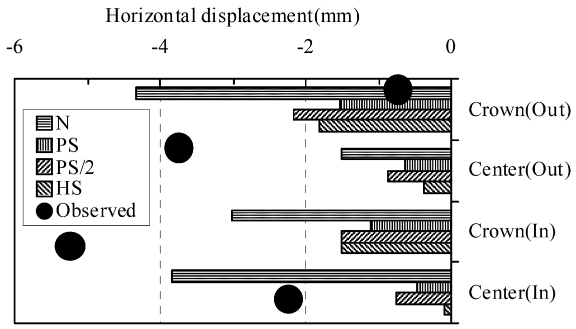

Figure 10 Comparison of horizontal displacement of tunnel

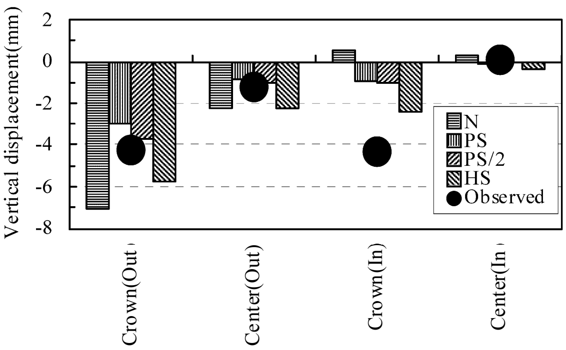

Figure 11 Comparison of Vertical displacement of tunnel

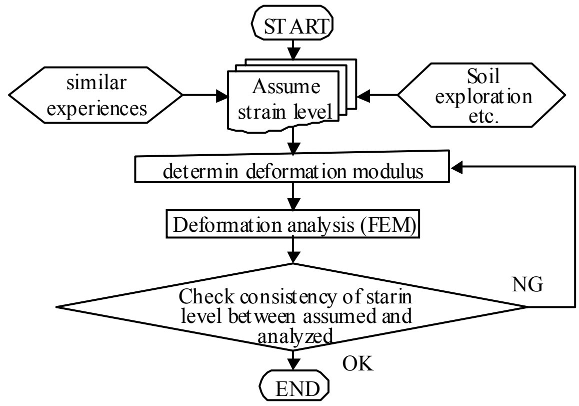

Figure 12 Practical approach for predicting deformation considering strain level

5 CONCLUSION

2D and 3D FEM were applied to evaluate the deformation and member forces during sub-steps of construction, which had not been evaluated in original design, in order to specify the indices for construction management. Also post analyses were carried out to improve the accuracy of simulation. From these research, following conclusions were obtained.

1) Linear elastic FEM using the deformation modulus of PS/2 showed acceptable result.
2) Nonlinear elasto-plastic FEM did not show better result than linear elastic case.
3) Practical procedure to improve the accuracy of linear elastic analysis considering ground strain level was proposed.

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