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Recent Diaphragm Wall Technologies and Future Challenges

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ABSTRACT: Recent technologies for diaphragm wall are highlighted and their technical level is reviewed whether they meet the technical requirement and demand. Then their future views and challenges on the following technologies are discussed: slurry fluids, trench stability, concrete, rebar, slime removal, vertical accuracy control, low headroom machine, and machine for the installation of diaphragm wall under-existing structures.

KEYWORDS: Polymer, Trench stability, High grade concrete, Rebar cage movement, Slime removal, Base grouting, Vertical accuracy, Low-headroom excavator, Existing under-structures.

1. INTRODUCTION

The first slurry trench cut-off wall was probably built at the Terminal Island near Long Beach, California in 1948 and another at Santa Marina Dam, Italy, in 1950. The diaphragm wall construction method has been now widely and extensively used to various fields of underground construction based on dedicate contribution to development by many challenges for a long time. The applications of diaphragm walls are numerous as functional structures such as retaining walls, underground stations, underground parking, traffic underpass, cut and cover tunnels, underground industrial facilities (such as oil tanks and LPG tanks), waterfront facilities such as quay wall,water cut-off wall, miscellaneous foundations (such as barrettes) and so forth. Reviewing the present available technologies adopted in the construction of diaphragm wall, it is evident that some technologies have already been developed to a practical engineering level but some have not met the demand sufficiently and effectively.

In this paper, firstly, the recent diaphragm wall technologies are introduced and reviewed from overall engineering fields which are supporting diaphragm wall construction and secondly, the following technologies to be challenged or to be developed more to meet the technical requirement and demand are highlighted: slurry fluids, trench stability, concrete, re-bars, removal of slime, control of vertical accuracy, low headroom machine, and machine for excavation of under-existing structures. The diaphragm wall can be classified into four types in terms of base material and construction method as follows:

Type A: Concrete diaphragm wall is typical one which consists of concrete with re-bars or steel beams.

Type B: Continuous bored pile wall is called SBP (Secant Bored Pile) and CBP (Contiguous Bored Pile).

Type C: Slurry-Cement Wall consist of hardening slurry with cement as base material of the diaphragm wall instead of concrete.

Type D: Soil- Cement Wall consists of mixing soil with cement as base material of the diaphragm wall instead of concrete.

In this paper, technologies for mainly Type A (concrete diaphragm wall) are discussed.

2. SLURRY FLUIDS

2.1 Alternative Slurry Fluids - Polymer Slurry[1)]

The bentonite slurry has been successfully used for over sixty years in many countries. The rising cost of bentonite and the frequently high transportation costs, however, have resulted in the development of new types of colloidal materials, polymer slurry. The polymer slurry has been used in the USA and in Asia but also increasingly in European countries as the alternative to bentonite slurry. Polymers consist of macromolecules whose molecule can be characterized by a repeated sequence of certain structural units. Polymers used for slurry are water soluble macromolecules and can be distinguished according to their manufacturing as follows:

- Biopolymer such as Chitosan, Xanthan gum and so forth.
- Semi-synthetic polymers which are manufactured by conversion of natural macromolecules such as Sodium Carboxyl Methyl Cellulose (CMC).
- Fully synthetic polymers such as PHPA (partially Hydrolyzed Polyacrylamide).

Japan was probably one of the first countries to adopt the polymer fluids technologies for deep foundation works[2)]. It was in 1973s that the biopolymer “Telmarch” and the semi-synthetic polymer “OP-4” that was classified into CMC (Carboxyl Methyl Cellulose) were developed and used for deep foundation and diaphragm wall works in Japan. The Telmarch was a highly successful as a stabilizer at early stage of its development. The use of “Telmarch” gradually declined because of its characteristic of bio-degradation of biopolymer and this product ceased later. The degree of substitution (D.S.) of the early stage of CMC (OP-4) was less than 1.0 and this type of CMC was sensitive to the biodegradation. The original CMC was revised with regard to D.S. and the revised CMC has tolerance of bio-degradation and salt tolerance with 1.3 to 1.5 D.S. In 1983, the CMC polymer slurry was specified by JNR (the Japan National Railroad) for Nagoya Station of Subway Line 6 Project and after that, CMC polymer slurry has been widely applied to diaphragm wall construction as the main stream slurry technology, mainly adopted in Japan and Asian countries.

The full synthetic polymer had been used mainly in the USA as slurry suspensions for deep boring.

The PHAP (Partially Hydrolyzed Polyacrylamides) became available for the foundation projects in 1990s

PHPAs are available as dry granular powders and are more rapidly dispersible liquid emulsions.

Today the use of PHPA polymer fluids for deep foundations and diaphragm walls has spread from first applications in North America to many other countries.

The semi-synthetic polymers and fully synthetic polymers are usually used as the slurry medium for the diaphragm wall construction.

The polymer slurry has various benefits over traditional bentonite slurry: compact separation plant, higher frequency of recycling, reduced environmental impact, lower construction cost taking into account of the final disposal and better foundation performance. However, the application of polymer slurry in deep foundations and diaphragm walls has not been always successful.

Despite proper understanding of the polymer characteristics and suitable usage of the polymer slurry, there are possibilities of unexpected problems arising. If the synthetic polymer such as polyacrylamides is in contacts with fine soil particles in the trench during excavation, it will be adhered to the surface of soil particles and gets transported to the de-sanding plant together with the soil particles. At the de-sanding plant the absorbed polymer is not separated from the soil particles and is discharged from the slurry circulation line.

Consequently the concentration of the polymer in the trench is gradually reduced. Therefore, careful control of polymer concentration is inevitable during excavation. In some cases, if the polymer gets in contacts with the cement particles considerable flocculation occurs suddenly in the trench. It is noted that the polymer slurry fails to function properly if the polymer quality is not controlled carefully during excavation.

In some projects, the actual mix proportion of the slurry consists of a combination of bentonite and polymer to compensate the weakness and disadvantages of each product. Generally, concentration ratio of bentonite/polymer with less than 10 is classified as "polymer slurry" and with more than 40 as "bentonite slurry". The concentration ration of bentonite/polymer with between 10 and 40 is classified as "medium type slurry".

2.2 Friction Resistance from Laboratory Test (Box Shear Test)

A shear box test was carried out to confirm the strength of mud cake formed in bentonite slurry and polymer slurry. Figure 1 shows the test apparatus setup which was remolded after the conventional box shear apparatus and was equipped with 17 acryl ring sheets (0.2mm thick each) put in layers at its shearing plane. This arrangement was designed to obtain the smallest values of shear resistance mobilized between concrete and soil. The test soil consists of the decomposed granite whose shear strength and physical characteristics were as follows:

φ_d=39.9°, c_d=0, unit density 20kN/m^3, e_{max}=0.88, e_{min}=0.59, d_{50}=1.41, d_{60}/d_{50}=5.88.

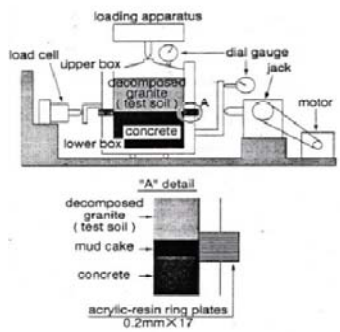


Figure 1 Reformed Box Shear Test Apparatus[3)]

Two types of slurry fluids, bentonite slurry and polymer slurry were used in the test. The mix proportions of these slurries are shown in Table 1.

Table1 Proportion of Slurry used in Test (per 100cc of water)

Type of Slurry	Water(cc)	Bentonite(g)	Polymer (CMC)(g)	Distribution Agent(g)
Bentonite Slurry	100	6.0	0.1	0.2
Polymer Slurry	100	2.0	0.4	0.0

Figure 2 shows test results of shear strength of the mud cake formed in bentonite and polymer slurry with time (in days).
The box shear test revealed the followings:

a) The friction resistance of the mud cake formed in bentonite slurry and polymer slurry is able to be expressed by effective stress parameters, cohesion c' and internal friction angle Ø′
b) The friction resistance of the mud cake formed in the polymer is larger than that in bentonite slurry
c) The cohesion c' of mud cake formed in bentonite slurry tends to increases considerably with age (time) but Ø′ of mud cake in bentonite slurry tends to deceases gradually with age (time)
d) The cohesion c' of mud cake in polymer slurry increases at early age and after that keep constant value regardless of the age(time) and friction angle Ø′ of mud cake in polymer slurry is almost constant regardless of the age (time)

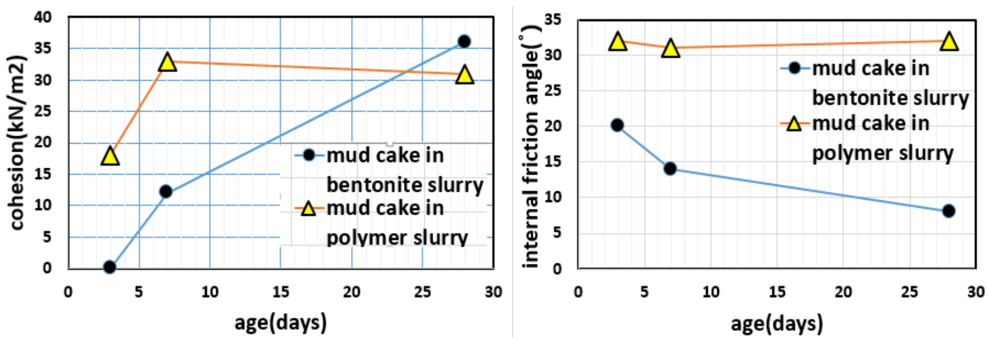


Figure 2 Cohesion and Friction Angle of Mud Cake in Bentonite and Polymer Slurry with Time from Laboratory Test[3)]

The box shear test was carried out with maximum vertical stress 1.5kg/cm^2 (150kN/m^2) and the mud cake consisted of pure cake without sand content, Therefore comparing to the actual friction valued obtained from the field load test the friction tests results would be slightly smaller than the field test values.

2.3 Thickness of Adhered Mud Cake in Slurry Fluids

Figure 3 shows the thickness of the mud cake in the slurry with time obtained from the laboratory test done by Kita [4]

It is clear from Figure 3 that

a) Thickness of adhered mud cake in polymer slurry is much thinner than that in bentonite slurry

b) Thickness of adhered mud cake in the slurry with excavated

c) soil (10%) is slightly thicker than that in the slurry without excavated soil

d) Thickness of adhered mud cake in bentonite slurry increases drastically after immersion time 24 hours.

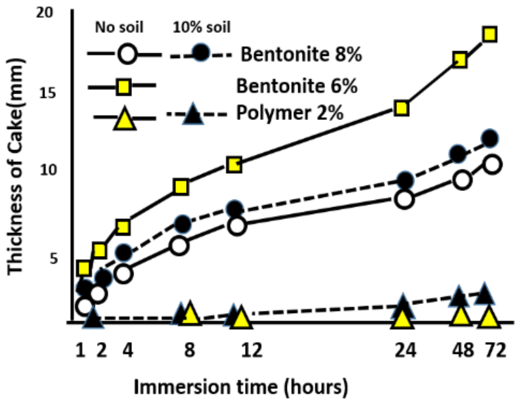


Figure 3 Thickness of Adhered Mud Cake in Slurry after Kita [4]

2.4 Shaft Resistance by Field Load Test

Figure 4 presents pile load test results which were carried out for 1m diameter bored pile in bentonite slurry and polymer slurry in OA layer in Singapore. The construction time for these tests were carried out approximately within 24 hours from commencement of trench excavation to completion of casting concrete.

Comparing the shaft resistance mobilized in bentonite slurry (circle with black mark) with that in polymer slurry (triangle with yellow mark) the difference could not be distinguished.

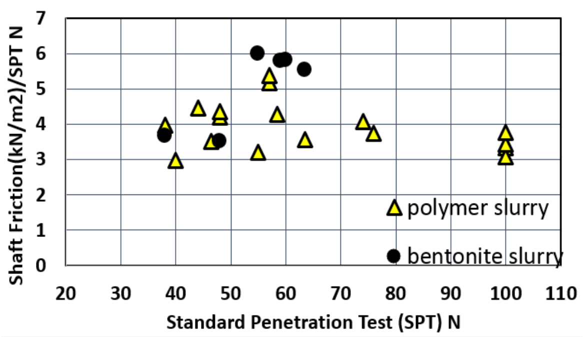


Figure 4 Shaft Friction in Bentonite and Polymer Slurry by field load tests (Ø1.0m bored pile) in Singapore

Figure 5 shows the comparison between ratio of actual to predicted shaft resistance after pile load tests (project carried out in Bangkok and Singapore). In Figure 5, Polmer-1 and bentonite-1 are obtained from bored pile (diameter 1.5m) load tests in Bangkok[2)] and polymer-2 and bentonite-2 are obtained from bored pile (diameter 1.0m) load tests in OA layer in Singapore.

Shaft resistance mobilized in bentonite slurry is expected to be slightly smaller than that in polymer slurry approximately within 24 hours (one day) of construction time (from commence of excavation to completion of casting concrete). However, the shaft resistance in bentonite slurry shows drastically decreased with construction time.

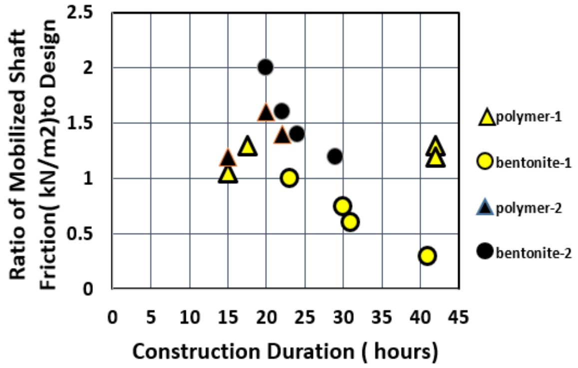


Figure 5 Ratio of Mobilized to Design Shaft Resistance vs Construction Duration

It can be found from these field load tests and the laboratory tests that:

a) The shaft resistance mobilized in bentonite slurry is almost the same as that in polymer slurry within 24 hours construction time (from commencement of excavation to completion of casting concrete). Possible reasons of that are, thickness of mud cake would not be thick and the shear strength c' of the bentonite mud cake would increase to the almost same strength of the polymer mud cake at the time of pile load test judging from Figure 2 and Figure 3.

b) The shaft resistance mobilized in bentonite slurry is drastically decreased with construction time. On the other hand, the shaft resistance mobilized in polymer slurry has not changes with construction time. One of the reasons of decrease in the shaft resistance in bentonite slurry would be the increase in the thickness of the bentonite mud cake with construction time judging from Figure 3.

3. TRENCH STABILITY

3.1 Outline of Trench Stability

There are two types of the trench stability to be considered as shown in Figure 6:

- Overall trench stability
- Local trench stability

Many methods for overall trench stability have been proposed based on numerical analysis, scale model-tests, full-size tests and monitoring at the sites. On the other hand, the method of local trench stability has not studied sufficiently so far.

According to the past records there are four types of localized collapses as shown in Figure 6.

The shallow sliding (collapse) occurs at shallow portion just below the guide wall. The lack of slurry pressure causes this type collapse.

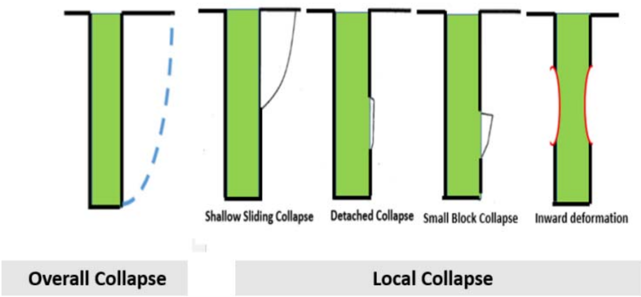


Figure 6 Overall and Local Trench Stability (added to 5))

The detachment of soil surface around the trench wall is usually caused by water penetration into potential fissure of hard soil due to stress release.

The small block collapse occurs due to lack of the slurry pressure for local weak layer or excess pore water pressure induced by movement of excavator.

The inward deformation of the trench wall is mainly caused by the stress release in the under-consolidated soft clay layer.

In this section, the following examples are introduced as possible causes of the local collapse and excess deformation of the trench:

a) Induced excess pore water pressure in the sandy layer due to excavator movement.
b) Stress change after trench excavation in the under-consolidation clay ground

For the overall trench stability, case study is carried out based on the actual soil conditions.

3.2 Local Trench Stability

3.2.1 Excess pore water pressure by excavator in sandy layer

The excavation machine for the diaphragm wall construction can be classified into two types, grab bucket type and trench cutter type. The mechanism induced the excess pore water pressure in the ground due to the movement grab bucket type is different from that due to cutting by trench cutter type.

In order to confirm the relationship between grab bucket motion and the induced pore water pressure in the sand layer during excavation, a field test was carried out. A relief hole was installed near the trench to mitigate the increase in pore water pressure which is induced in the sand layer due to grab bucket motion as shown in Figure 7.

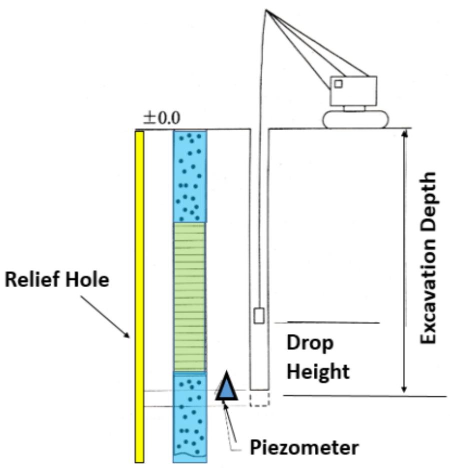


Figure 7 Grab Bucket Drop Test

The relief hole consists of PVC tube with a diameter of 115mm with slits and small gravel less than 10mm filled around the PVC (drilling hole diameter 150mm). In the relief hole a packer was prepared to allow the adjustment of the function of the relief hole.

Figure 8 shows the test results of the grab bucket drop test. It can be found from the test results that:

a) Induced pore water pressure increases with drop height of the grab bucket
b) The relief hole installed near the trench is effective in reducing excess pore water pressure.

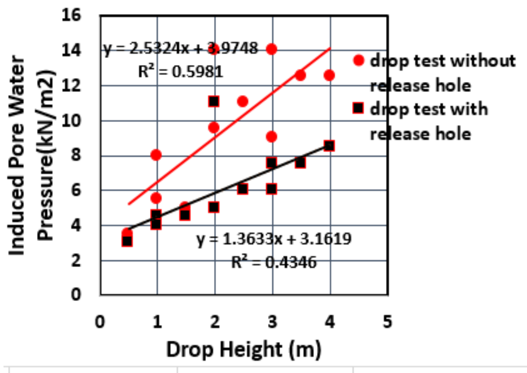


Figure 8 Test Results of Grab Drop Test [6)]

The excess pore water pressure induces in the ground near the trench during excavation could also occur by the trench cutter type excavator.

According to the field excavation test by using the trench cutter [6)] the monitoring results of the piezometer installed at the distance 1.3m from the trench wall shows the excess pore water pressure is equivalent to approximately 10% of effective overburden stress was induced in the ground as shown in Figure 9 a).

The induced excess pore water pressure ΔU (tf/m2) is expressed as follows:

$$\Delta U = 0.102\ \sigma' - 0.204 \text{ for A, B gut,} \quad (1)$$

$$\Delta U = 0.086\sigma' - 0.474 \text{ for C gut} \quad (2)$$

Where,
σ': effective overburden pressure (tf/m^2)

Figure 9 b) presents also the test results of the induced excess pore water pressure in the ground during excavation by the trench cutter with regards of cutter torque (T) and the distance between the cutter and the monitoring points (L). From the test the excess pore water pressure (ΔU) can be expressed as following equation:

$$\Delta U = 2.11(L/T2) + 1.41 \quad (3)$$

Where
ΔU: induced excess pore water pressure (tf/m^2)
L: distance between center of cutter and monitoring point (m)
T: cutter torque (t-m) for trench cutter 4 drums

It is clear from these field tests using grab bucket type exactor and trench cutter type excavator that the excess pore water pressure during excavation is surely induced in the sandy ground near the trench. There is possibility of leading to local collapse by the induced excess pore water pressure during excavation in the sandy layer. The excess pore water pressure due to the excavator, therefore, should be considered together with overall trench stability.

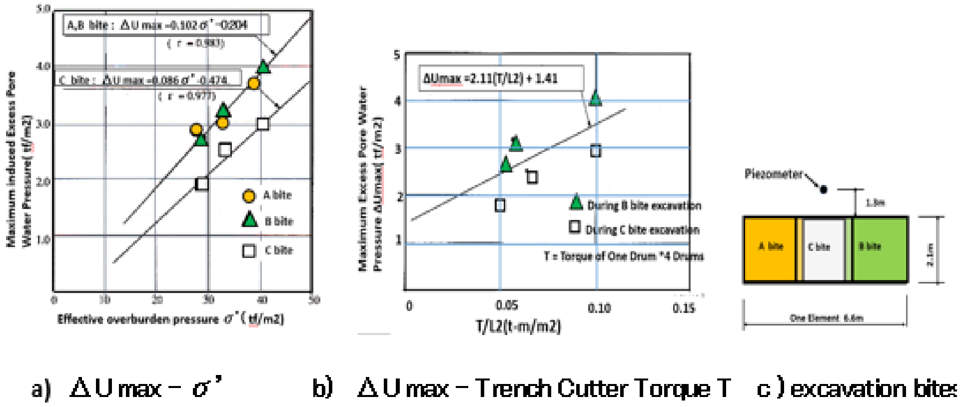


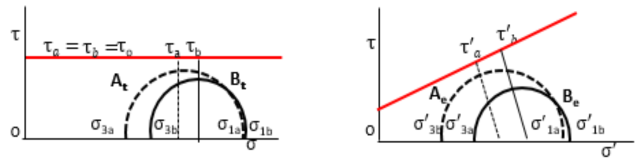
Figure 9 Induced Excess Pore Water Pressure due to Trench Cutter Excavator[7)]

It is noted that in case of the low safety factor of overall trench stability, the careful excavation method for the sensitive sandy layer should be considered to minimize the induced excess pore water pressure, that is, for the grab bucket excavation lower drop height of the grab bucket should be carried out and for the trench cutter excavation, smooth excavation with small thrust to minimize the cutter torque should be achieved.

3.2.2 Inward movement of soft clay

There have been various records of inward movement of the trench wall. Possible causes would be excess pore water pressure in under-consolidation ground, decrease in effective stress due to negative dilatancy of soft clay, decrease in slurry density due to replacement of used slurry with new slurry before casting concrete and so forth. The safety factor of trench stability is described using Mohr's stress circle below.

Figure 10 illustrates change of major and minor principal stress in the ground near the trench wall before excavation (original) and after excavation in terms of total stress and effective stress. B_t and A_t show Mohr's total stress circles before and after trench excavation respectively. B_e and A_e present Mohr's effective stress circles before and after trench excavation respectively.



a) Total Stress **b) Effective Stress**

Figure 10 Change of Stress after Excavation of Trench

In the normal consolidated ground, relationship between total stress and effective stress are shown in the Equations (4) assuming pore water pressure is equal to hydrostatic pressure.

$$\sigma'_{1a}=\sigma_{1a} - U, \quad \sigma'_{3a}=\sigma_{3a} - U, \text{ for after excavation} \tag{4}$$

Where,

σ_{1a}, σ'_{1a} : Major principal stress in total stress and effective stress after excavation respectively

σ_{3a}, σ'_{3a} : Minor principal stress in total stress and effective stress after excavation respectively

U: Pore water pressure (Hydrostatic water pressure)

The safety factor (SF) of trench stability after excavation can be expressed in the Equations (5) and (6)

SF for total stress $SF_t = \tau_o/\{(\sigma_{1a}-\sigma_{3a})/2\}$ (5)

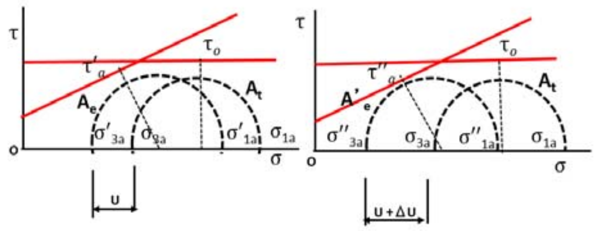
SF for effective stress $SF_e = \tau'_a/\{(\sigma'_{1a}-\sigma'_{3a})/2\}$ (6)

In case that the ground is under-consolidation and excess pore water pressure ΔU remains in the ground, Equation (7) can be given as follows:

$$\sigma'_{1a}=\sigma_{1a} - (U+\Delta U), \quad \sigma'_{3a}=\sigma_{3a} - (U+\Delta U) \tag{7}$$

Figure 11 presents comparison of the stress in the normal consolidation ground after trench excavation with that in the under-consolidation ground.

The Mohr stress circle (A'_e) shifts by ΔU to point O direction from the Mohr's stress circle (A_e). Where, A_e and A'_e are Mohr's effective stress circle after excavation in the normal ground and the under-consolidation ground respectively.



a) Normal Consolidation Ground b) Under-Consolidation Ground

Figure 11 Stress Condition near Trench Wall in Normal Consolidation and Under- Consolidation Ground

It can be found from the stress conditions of A_e and A'_e that safety factor for effective stress in under-consolidation ground is smaller than that in normal ground as illustrated in Figure11.

Safety factor for total stress in under-consolidation ground, however, is the same as that in the normal consolidation ground. Consequently in some cases, excess deformation of trench walls occurs with time due to decease in the safety factor. It would be, however, difficult to predict the inward movement concretely before excavation of the trench because inward movement is affected by many factors: soil conditions, excavator movement, panel dimensions, slurry conditions and so forth.

3.3 Overall Trench Stability

Numerical methods of overall trench stability are classified into three categories as follows:

a) Equilibrium of forces between slurry pressure and hydraulic pressure/ earth pressure considering arching effect (hereafter referred as "arching effect method").
b) Limit equilibrium method with regards of sliding wedge (hereinafter referred as "equilibrium wedge method")
c) 2D and 3D FEM analysis

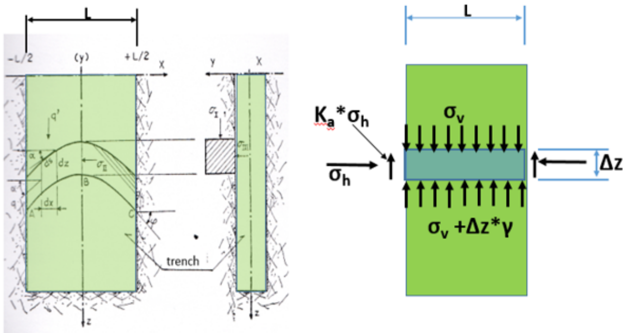
3.3.1 Arching effect method[8)]

Figure 12 shows the arching effect method. The arching effect of short panels has been investigated by Schneebeli (1964)[9)]. This analysis is based on the theory of lateral earth stress for silos (Caquet and Kerisel, 1956). Schneebeli has derived the following expression for the horizontal stress σ_z at depth z

$$\sigma_z = (\gamma L/N_\emptyset \sin 2\emptyset)\ \{1\text{-exp}\ (1\text{-nsin}\ 2\emptyset)\} \tag{8}$$

Where,
n=z/L, L = panel length, $N_\emptyset = (1-\sin\emptyset)/(1+\sin\emptyset)$
Huder (1972)[10)] has proposed reduction factor A for active earth pressure $K_a\gamma z$ acted on the trench wall based on Terzaghi theory (1943) as follows:

$$A = \{1 - exp(-2nK_a tan\emptyset)\}/(2nK_a \tan\emptyset) \tag{9}$$



a) Schneebeli Method(1964) b) Huder Method (1972)

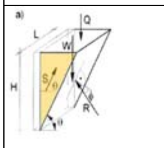
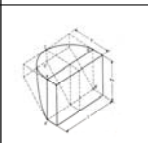
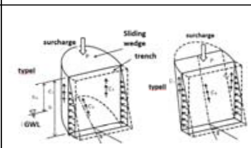
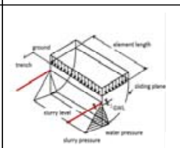
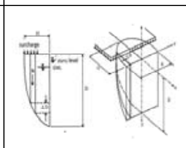
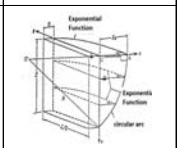
Figure 12 Arching Effect Method

3.3.2 Equilibrium wedge method

The equilibrium wedge methods have been proposed by many researchers with different wedge shapes and theories. The definition of the safety factor of trench stability is also based on the different concept. Table 2 presents some of the equilibrium wedge methods that are applied to some projects at present.

Method A has been applied to DIN 4126 [11)] and the commercial software of this method named GGU-Trench[12)] can be used for trench stability. Method C, D, E and F are usually used for the diaphragm wall projects in Japan.

Table 2 Example of Equilibrium Wedge Method

No Method	A	B	C	D	E	F
Shape of Sliding Wedge	Triangular Prsim	Vertical Parabolic	Half Vertical Cylinder	Horizontal Cylinder	Vertical Parabolic based on Protodyakonov Theory	Exponential Function
						
Persons who proposed	Morgennstern and Amir Tahmasseb[15)]	Piaskowski and Kowalewski[16)]	Y.Kanatani and N. Akino [17)]	H.Uchida and H.Mizutani[18)]	Protodyakonov [19)] Y.Ohtsuka [20)]	Y Yoshida [21)]
Proposed Equation for Safety Factor	Safety Factor = (Ps-Pw)/(Wh-Rh)	Safety Factor = (Ps-Pw)/(Wh -Rh)	Safety Factor ={(Ps-Pw)+Rh}/(Wh)	Safety Factor =(M_E + M_S)/(M_W+ M_{pw})	Safety Factor =(Rh + Ps)/(Wh-Rh+Pw)	Safety Factor =ultimate shear strength/shear stess
Recommendation Minimum Safety Factor	1.1 or 1.3 (if loads from structures act in critical area	1.2	2.0	1.2	1.0	1.2
Remarks	Adopted to DIN 4126 GGU-Trench soft ware is avalable	Applied tunnel load proposed by Protodyakonov theory to Wedge sliding theory	Y.Kanayani and A.Noriyuki proposed " Half Vertical Cylinder Wedge based on a series of model tests and Full scaled Model Test. The Wedge Sliding shape has been found to be similat to that by A.Piaskoeski et al	H.Uchida and H.Mizutani proposed Horizontal Cylider Wedge based on Model tests	Y.Ohtsuka applied loosening shape at tunnel crown by Protodyakonov to trench wall. Y. Ohtsuka proposed the sliding wedge with the same shape as Protodyakonov based on two field tests	Y.Yoshida proposed sliding wedge with horizontal section formed by exponential function based on model test. Y. Yoshida applied this method to sliding example of Rierre Dam and Dacum Dam and confirmed well fitting

3.3.3 FEM method

Generally the trench shape of panel length, trench width and trench depth are 3m to 6m, 0.6 to 1.5m and 10m to 50m respectively. Therefore, 3D FEM analysis is required to analyze trench stability.

"Plaxis 3D foundation"[13],"SSR-FEM"[14] and others have been developed as 3D FEM analysis for the trench stability.

3D FEM would be the most reliable analysis method for the trench stability. In the diaphragm wall projects, however, arching effect methods and equilibrium wedge methods have been usually used because of the effective analysis in running time and cost.

3.3.4 Case studies of trench stability

The case study has been done using the arching effect methods and the equilibrium methods to compare the safety factor of the trench stability.

The arching effect methods (Schneebeli method and Huder method) and Method A (DIN 4126), Method C and Method E as the equilibrium methods are selected, which have been used in diaphragm wall projects in Singapore and in Japan.

Table 3 shows soil parameters (effective analysis) in the case studies.

Table 3 Soil Parameters using Case Studies of Safety Factor

Layer	Bottom Level from GL	γ (kN/m3)	γ'(kN/m3)	Φ(°)	c' (kN/m2)
Fill	-9	19.5	9.5	30	0
Marine Clay	-23	16	6	24	0
F2	-27	19	9	25	0
OA (C)	-35	20	10	35	10
OA (B)	-50	20	10	35	15
OA (A)	-70	20	10	35	20

Other conditions in the case studies are as follows:

panel length L=3m, 4m and 5m, trench width B=1.5m, trench depth H= 50m, GWL=GL-1.5m, Slurry level=GL-0.3m, slurry density=10.5kN/m3, surcharge =25kpa.

These trench excavations have been completed without collapse at all in the actual project.

Figure 13 presents results of safety factors obtained from the case studies. It has been found that:

1) Method A (DIN 4126) presents reasonable safety factors.
2) Shneebeli method presents greater safety factors than that of Huder method.
3) Method E presents almost similar safety factor as Method A
4) Method C presents almost similar safety factor as Huder method

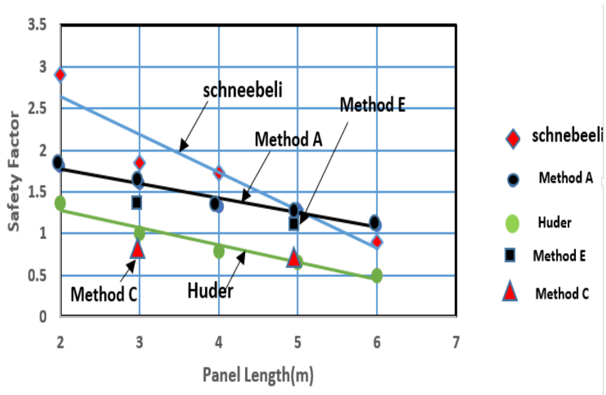


Figure13 Comparison of Safety Factor of Trench Stability

The case studies have been carried out using soil parameters of the limited site conditions in Singapore.

Furthermore, the definition and concept of safety factor of trench stability are different from each method. Therefore, it would be difficult to conclude the above absolutely.

3D FEM method would be the most accurate method of analyzing trench stability but it takes longer time to calculate the safety factor and also is also costly.

Apart from FEM, Method A would be the simplest and easily understandable method out of the various equilibrium methods from geotechnical point of view and also shows a reasonable safety factor judging from the actual trench excavation.

In the arching effect method proposed by Schneebeli and Huder, the safety factor of the trench is expressed as the balance between slurry pressure from trench side and pore water pressure in the ground and earth pressure from the ground side. This concept is very straightforward and corresponds well to geotechnical theory. If further research and studies are made on the arching effect for the earth pressure based on the actual collapse records, the development of this arching effect theory would be the most suitable and simplest method for analyzing the trench stability.

4. CONRETE AND REBARS

As the need for development of underground space utilization, the following requirement for concrete in diaphragm wall construction has been intensified.

a) High fluidity-concrete under high pressure condition and high density of re-bars
b) Higher strength for great depth concrete and permanent structures
c) Lower heat for thicker mass concrete

Table 4 presents examples of the diaphragm wall construction with great depth.

The depth of diaphragm wall applied to No.3 shaft for Flood Control Tunnel surrounded in outer Tokyo Metropolitan block is 140m which is the deepest diaphragm walls constructed so far. Excavation depth inside circular shaft was 74m and to prevent water ingress below base slab the toe of the diaphragm wall was extended to the impermeable layer with the diaphragm wall depth reached 140m. For the trial test of the diaphragm wall for this project, the depth reached 150m.

Table 5 shows application of high strength concrete to the diaphragm wall structures.

All these examples were applied to underground circular LPG Tanks. For underground circular tank earth pressure and water pressure acts on the diaphragm wall and diaphragm wall thickness is governed by axial force instead of the bending moment. The high strength concrete has great advantage to reduce the thickness. In order to achieve high strength concrete, water cement ratio should be reduced to approximately 30 or less than 30. The high-fluidity in the high strength concrete should be kept.

High performance AE reducing water agent or reducing water agent is used to keep the high fluidity of the high strength concrete.

Accompanying greater depth of the diaphragm wall construction, downward and upward movement of re-bar cage during concrete casting requires greater control and confirmation.

Figure 14 shows one of the examples in which re-bar cage of the diaphragm wall (total depth 100m, thickness 1.2m) was dragged down during concrete casting. This re-bar cage arrangement did not reached to the bottom of trench because lower part of the diaphragm wall (20.47m) was designed as water stop wall without structure re-bars. According to the monitoring results huge force (approximately 50tons) was induced on the re-bar cages.

Table 4 Example of Great Depth Diaphragm Wall [22)]

Project Name	Clients	Costuction Year ' (starting)	Thickness of Dwall(m)	Depth of Dwall(m)	Excavation Depth inside (m)	Design Strength of Concrete (N/mm2)	Joints
No1 to No4 Shafts for Flood Controll Tunnels surrounded in Outer Tokyo Metorpolitan Block	Ministry of Construction , Japan	1991 to 1995	2.1 (No1,2,3) 1.7(No4)	122 to 140	69 to 74	24	Concrete Cutting
Shaft of Tokyo Bay Crossing Tunnels	Tokyo Bay Crossing	1991	2.8	119	75	36	Concrete Cutting
Chita LNG Undergroung Tank	Chita LNG	1993	1.6	118	53	37	Concrete Cutting

Table 5 Example of High Strength Concrete (more than 50 N/mm2)[23]

Project Name	Clients	Thickness of Dwall(m)	Depth of Dwall(m)	Design Strength of Concrete (N/mm2)	Joints
Tokyo Gas Sodegaura C-4 T1 Underground Tank	Tokyo Gas	1.2	100	51	Concrete Cutting
Oogishima TP-1 LNG Underground Tank	Tokyo Gas	1	71	60	Concrete Cutting
Oogishima TL-11,21 LNG Underground Tank	Tokyo Gas	1.5	69	51	Concrete Cutting

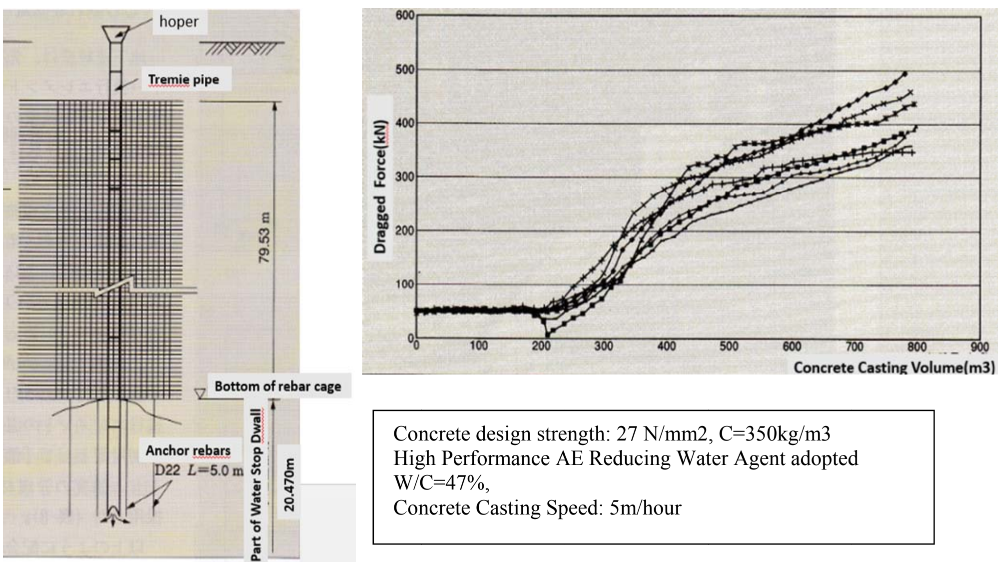


Figure 14 Re-bars Cage and Dragged Force during Casting Concrete [24)]

Figure 15 shows the monitoring setup of the rebar cage movement due to concrete casting of the diaphragm wall that reaches the depth and thickness of 53m and 1.5m respectively.

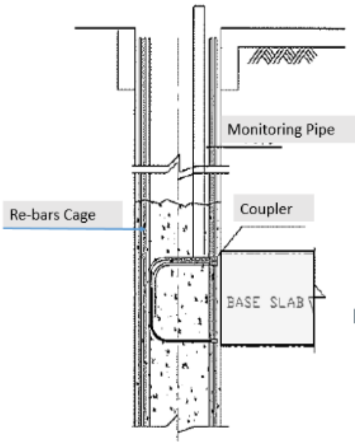


Figure 15 Monitoring of Re-bar Cage Movement in Singapore Project [25)]

The diaphragm wall was designed as a permanent structure for the MRT station in Singapore and couplers were attached in the re-bar cages for connection to the diaphragm wall at the base slab, concourse slab and top slab of the station.

In order to confirm the variating change of the coupler's level resulting from the re-bars cage movement, this monitor regime was carried out to decide the setting levels of couplers of each slab. Table 6 shows the monitoring results of the change in level of the couplers at the level of base slab. According to Table 6 the couplers were settled by 40mm after concrete casting. Taking into account this monitoring result the couplers were attached in the re-bar cages.

Table 6 Monitoring Results of Coupler Level Change

Coupler Level Before Intalllation of RE-bar Cage (A)	Coupler LevelAfter Intallation of Re-bar Cage	Coupler Level After Casting Concrete (B)	Difference =A-B (mm)
83.098	83.097	83.058	40

The possible causes of movement of the re-bar cage in concrete casting operation could be as follows: For upward movement of re-bar cage upward flow of the fresh concrete would push up the re-bar cage. For the downward movement the consolidation of the concrete would drag down the re-bar cage. The precise causes in the movement of re-bar cage, however, have not been clearly defined and also it is not been clear whether induced stress of re-bars during concrete casting has adverse effect to the permanent structure.

5. REMOVAL OF SLIME

Fine soil particles suspended in the slurry trench during excavation will be eventually settled and deposited with time at the bottom of the trench and accumulated.

The sedimentations consist of soil particles, cutting concrete particles in the slurry and collapsed soils during installation of re-bar cage. These sediments accumulated at the bottom of the trench are referred as "Slime".

If the concrete casting operation is carried out without removal of the accumulated slime at the bottom of the trench, these accumulated slime will float up in the slurry again and is rolled in with the fresh concrete that results in the deterioration of the concrete grade. The careful removal of the slime shall be carried out as follows:

1) Primary removal of the slime
After excavation of the trench the removal of slime accumulated at the bottom of the trench is carried out using excavation machine, taking into account of the duration for sedimentation of the slim. Usually the sedimentation speed of the slime in the bentonite slurry and the polymer slurry are approximately 4m/hour to 5m/hour and 8m/hour to 10m/hour respectively.
2) Secondary removal of the slime
Just before the installation of re-bar cages the secondary removal of the slime using air lift system, sand pump system or suction pump system with the same de-sanding slurry system as shown in Figure 16.

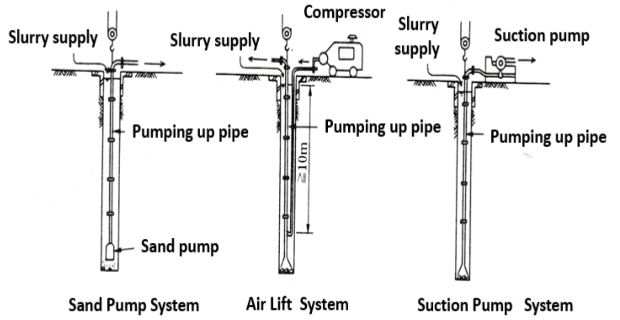


Figure 16 Slime Removal System[25)]

Recently alternative method in which the secondary removal of slime is shortened has been applied to some projects. In this method the primary removal of slime is implemented using excavation machine supplying new slurry and immediately concrete is cast. After concrete casting, the "Base Grouting" is carried out to compensate the removal of the slime. Figure 17 shows schematic of base grouting system.

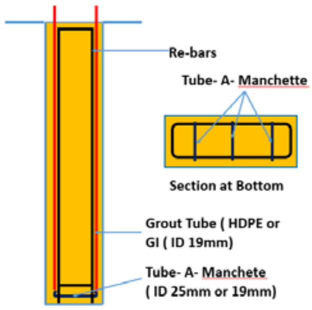


Figure 17 Example of Base Grouting System

6. CONTROL OF VERTICAL ACCURACY

A few years ago, the required depth of diaphragm wall was approximately 20m to 30m range and now, the required depth reaches 40 to 60m range. The required control of vertical accuracy, however, is still lacking in some projects. In the near future, required depth of diaphragm walls might reach 100 to 150m depth.

Figure 18 presents the deviation in plan for each vertical accuracy 1/200, 1/500 and 1/1000 and each depth of the diaphragm wall. In the instance of the depth at 100m and vertical accuracy 1/200, 1/500 and 1/1000, the deviation in plan is 500mm, 200mm and 100mm respectively. Even if the vertical accuracy is improved to 1/500mm, the deviation in plan is still 200mm. If the cutting method for element joint is applied design cutting size of end of the element is more than 300mm and even if the vertical accuracy is 1/500 it would be unacceptable justification for cost and construction duration.

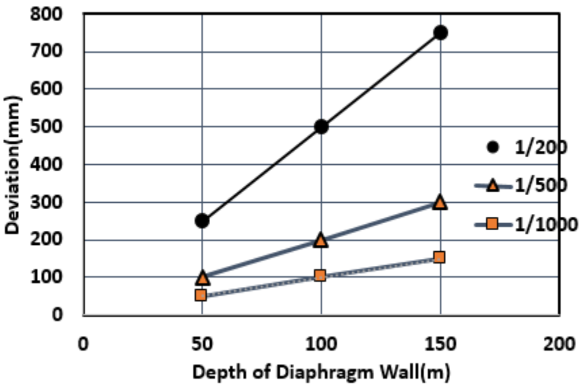


Figure 18 Vertical Deviation of Diaphragm Wall

Corresponding to the increase in depth of the diaphragm walls excellent control system for the vertical accuracy is essential. Recently the control systems have been developed to meet the demand of deep diaphragm wall as illustrated in Figure 19. The control systems proposed consist of two wires connected with trench cutter and monitoring instrument such as inclinometer, displacement gauge, magnetic sensor and so forth. Using these systems, the vertical accuracy of 50mm has been achieved regardless of the depth. However, there is a limitation to the depth of the diaphragm wall to which vertical control system using wire system can be applied effectively because of vibration and swing of the wires in the trench. It is desirable that further improvement to the vertical control system will be developed.

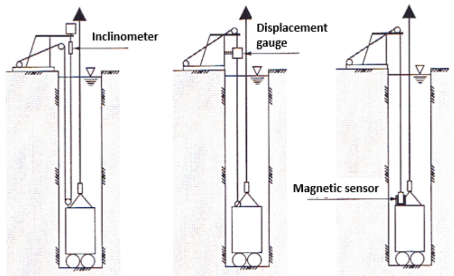


Figure 19 Control System for Vertical Accuracy of Diaphragm Wall

7. LOW HEADROOM EXCAVATION MACHINE

Low headroom machines for diaphragm wall installation have been developed to facilitate the requirement of working in very restricted conditions: extremely low headroom and narrow working space. Table 7 shows the low headroom machines which are commonly used.

All machines shown in Table 7 are "Drum-Cutter type with reverse circulation system which is usually referred as "Trench-Cutter "and the machines have been specially designed with compacted height and width.

Figure 20 and Figure 21 show EMX-240 and MBC30 respectively.

Figure 22 presents the photo of MPD-TMX which is the lowest headroom machine as shown in Table7.This machine has been developed for diaphragm wall construction in extremely congested and restricted sites at which existing Railway running on the ground was converted to underground (Odakyu Electric Railway in Japan).

The excavation speed of the trench-cutter based on the field test is given by

$$V = k\,(Tm*N)/\,(A*S) \qquad (10)$$ [28]

Where,
V: excavation speed
k: constant coefficient
Tm: thrust force
N: rotation speed of cutter drum
A: excavation area of trench cutter
S: unconfined compressive strength of the ground

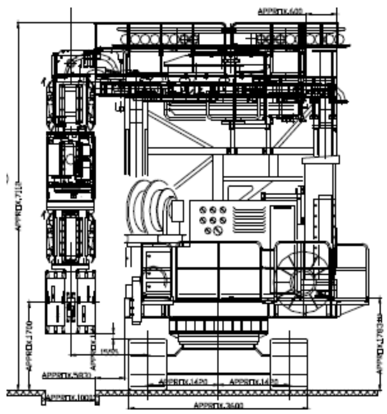


Figure 20 EMX240 [23]

Figure 21 MBC30[26]

Figure 22 MPD-TMX[27]

Table 7 Main Low Headroom Machines

MACHINE NAME	MANUFACTURER (MAKER)	MACHINE THICKNESS (mm)	MACHINE WIDTH (mm)	MAX EXCAVATION DEPTH (m)	MACHINE HEIGHT (m)	MACHINE WEGHT (tons)	CUTTER TORQUE	MAX ROTATION SPEED(rpm)	TOTAL HEIGHT(m)
EMX-150	TONE	650-1500	3200	70	4.7-5.3	29	71*2-85*2	18-22	4.7-6.05(Rail) 5.0-6.35(Crawler))
EMX-240	TONE	1200-2400	2400	70	4.9-5.1	29			
HFA-4RCII	SOLETANCHE OBAYASHI	630-1200	2400	50	4.1	15	40	22	5.0(Crawler)
MBC-30	BAUER	640-1500	2800	50	4.4	17-21	2*70	30	4.9(Rail) 6.4(Crawler)
MPD-TMX	TONE	650-1000	2400	70	3.537	7	47.5	20	3.873(Crawler)
FDC 28/70	CASAGRANDE	800-1800	2800	70		22-32	86*2	30	5.0(Crawler)

Generally, 10 to 30% of the self-weight of the trench cutter machine Wm are applied to the ground as the thrust force ($Tm = \alpha Wm$, $\alpha = 0.1$ to 0.3) to improve vertical accuracy using the pendulum principal. The weight of low headroom machine is lighter than that of ordinary trench cutter machine as shown in Figure 23. In order to improve the excavation speed with low headroom machine α should be increased which result in the increase in thrust force (Tm). Consequently vertical accuracy becomes lower due to larger α and smaller vertical length of the low headroom machine.

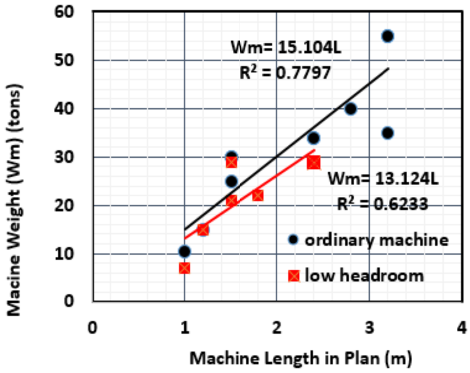


Figure 23 Weight of Ordinary machines and Low headroom machines

In order to negotiate around and eliminate this disadvantage there are two possible measures, that is to increase in the self-weight and/or to increase in vertical length under the low headroom conditions. The visible measure would be to develop attachable tools to the excavator body after lowering down the excavator in the trench to reach the required level. To actualize this concept and put to practical use is going to be a great challenge.

8. UNDER STRUCTURE EXCAVATION

The underground of the urban area holds various life lines such as water supply and sewerage, city gas pipe lines, communications infrastructures, electric cables and so forth.

As the developments of underground space are increasingly demanded, the diaphragm wall installation works need the attention to overcome the presence of such utilities.

In such case of diaphragm wall construction obstructed/ crossing utility lines, it does not allow the installation work to be carried out from the ground. Usually, the utilities are diverted to unaffected areas to allow the diaphragm wall construction.

However, as illustrated in Figure 24, various methods are available to construct the diaphragms without the necessity of diverting the utilities.

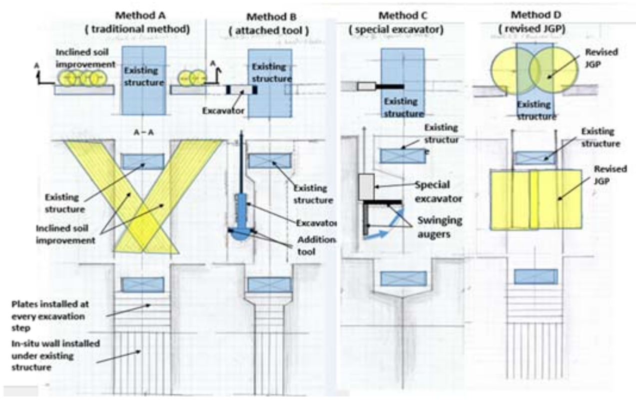


Figure 24 Diaphragm Wall Construction Method under Existing Structure

In Method A, the diaphragm walls are installed with gap to avoid the existing utilities with soil improvement and the diaphragm walls for the gap part are constructed under the existing utilities after excavation to required headroom of excavation machine.

For the Method B additional tool is attached with grab bucket, however, the maximum overcutting from one side is approximately 1.0m Both Method A and Method B may cause difficulty in construction and safety control and consequently, the extended construction duration to complete the works.

Recently new technologies have been developed to mitigate these difficulties and disadvantage: one of the technologies is the application of special excavation machine under existing structures (Method C) and another is the "revised jet grouting" method with large diameter to be able to cover large gap and excellent control of pressure (Method D).

For the Method C, the special augers which can be swung to 90 degrees with 2 axes are attached to excavation machine. The maximum overcutting length is 5m from one side (10m from two sides). Figure 25 shows the machine for Method C [29]

For Method D, jet grouting capable of producing the diameter of approximately 4m to 5m is applied to close the diaphragm wall gap. The diameter of traditional jet grouting is approximately 2m and control of the jetting pressure is difficult. Consequently ground movement due to jetting pressure is generally unavoidable.

Figure 25 Excavation Machine under Existing Structure (SATT method) [29)]

In order to improve the formed diameter and control of jetting pressure, the revised jetting method has been developed in Singapore based on the original basic Jet Grouting Theory. Special designed "Nozzle" has been developed to enable to a further penetration without increasing the pressure instead a high jet flow rate is used for eroding soil and it is therefore important to provide a path for cutting slurry return. The short casing is installed before jetting and annulus space is easy for pressured slurry to flow out as pressurized fluids choose flow path of least resistance. In the trial test carried out in MRT project in Singapore, the formed diameter of jet grouting has been achieved to be 4m to 5m and ground movement just above jet grouting area has been controlled to be negligible[30)]. Method B or Method D would be the best solution for excavation works under existing structures in the near future.

For Method B, the ordinary excavation machine can be used attaching the special developed tools with the ordinary machine therefore, additional preparation or mobilization of the special machine .for excavation under existing structures is not necessary. For Method D, if the formed diameter can be increased around to 6m to 8m this method would be the most preferable method.

9. FUTURE CHALLENGES ON TECHNOLOGIES OF DIAPHRAGM WALL

Recent fundamental technologies of diaphragm wall such as slurry fluids, trench stability, concrete and re-bars, removal of slime and control system of vertical accuracy, low headroom machine and excavation machine under structures have been highlighted. Some of them have been sufficiently put to practical use for design and construction field but some of them haven't reached the level to meet the demand. At the final of section, future views of diaphragm wall technologies and also challenging diaphragm wall technologies are discussed.

9.1 Slurry Fluids

The polymer slurry has been recently developed and widely used as an alternative to the bentonite slurry. The polymer slurry; semi synthetic polymer (CMC) and full synthetic polymer (PHAP) are commonly used. These polymers have, however, advantages and disadvantages over the traditional bentonite slurry. It is desirable that further development of "revised slurry" which has outstanding features over the bentonite and the polymer which are commonly used should be developed. Recently some "revised slurry fluids" have been developed and applied to some projects such as "Super Slurry"[30),] "Aerated Slurry used in Award Method"[31)], Selective Flocculent Slurry[32)] and so forth. These "revised polymer fluids" have anti-bio-degradation, cement-tolerance, solid tolerance and anti-flocculation features. However, they have not been widely used so far because of unrenowned product names and have not attained the required practical achievement to be applied by users.

The shaft resistance of the diaphragm wall constructed in bentonite slurry would not be different from that in polymer slurry subject to construction duration as discussed in 2.4.

In the use of the bentonite, relationship between mobilized shaft resistance and construction duration should be studied together with quality of the bentonite slurry such as density, viscosity and sand content before casting concrete.

9.2 Trench Stability

For trench stability, the arching effect method, equilibrium Wedge Sliding Method and FEM method for overall trench stability and the mechanism of local collapse have been discussed in 3.2 and 3.3.

For overall trench stability various calculation methods have been proposed. For the local trench stability, however, applicable methods to the site have not been proposed.

A comprehensive trench stability method including the local stability should be developed taking into account local collapse mechanism.

9.3 Concrete Quality and Re-bar

As increasing tendency for great depth of the diaphragm wall better required strength of the concrete will escalate in the near future.

The usual concrete grade used would be G30 to G40 but recently, a much higher grade of concrete at G60 was used in the past project for underground LPG tank. The greatest depth of the diaphragm wall was probably 140m for the shaft in flood control tunnel project.

Super high grade concrete, up to G70 to G100 has been already developed in laboratory base and in future such high grade concrete will be put to practical use. The demand of such super high grade concrete is based on not only inevitable requirement due to the increase in the depth of the diaphragm wall but also, in pursuit of better cost optimization of the diaphragm wall which is applied to circular shaft or tank.

The diaphragm wall applied to the circular shaft and underground tank is governed by the axial force instead of bending moment, therefore, the required thickness of the diaphragm wall depends directly on the strength of the concrete.

The concrete strength quality and fluidity have improved considerably as mentioned above, nevertheless concrete casting method using termite pipe has not changed so much. The special robot for control system of termite pipe position measuring the amount of concrete casting level has been developed but at the site the robot has not been popularized so far.

In future, it is desirable that well balanced development between material and construction method should be carried out.

As mentioned in section 4, the downward and upward movement of re-bar cages has been confirmed in some projects. The theoretical analysis of this movement so far has not been conducted. The requirement of the permanent diaphragm walls connected to the slabs poses a challenge, therefore the re-bar movement should be predicted accurately and also the impact of the induced stress in the re-bars to the permanent structures should be analyzed.

9.4 Removal of Slime and Monitoring Vertical Accuracy

With the increasing depth of the diaphragm wall, removal of slime and control of vertical accuracy become more difficult and challenging.

The removal of the slime at the bottom of the trench is usually carried out using excavator and replacing used slurry suspended fine soil particles with new slurry. Even if replacement method of the used slurry is applied, complete removal of slime would be impossible for deep diaphragm wall.

In case of foundation diaphragm wall such as permanent wall of underground station, Barrette foundations and others, it is better to adopt the Base Grouting method to improve the bearing capacity.

For the monitoring vertical accuracy the two wires system as mentioned in section 6 is applicable up to approximately 150m deep diaphragm wall as judging from past achievement and our experience. Diaphragm wall with greater depth than 150m deep will be required further revised monitoring system to be developed.

9.5 Special Machines

In the near future development of underground space in congested urban area will escalate more and the demand of diaphragm wall construction under the severely restricted conditions; low headroom, narrow space, under-existing structures and the like will also increase as well.

The ordinary diaphragm wall machine is readily available in most countries and use under favorable site conditions. However, the availability of these special machines for use under such severe conditions is limited to most projects. Therefore, it is desirable that ordinary machines can be easily changed/ converted to meet the required conditions by exchanging original equipment with these attaching special tools.

9.6 Other Challenges

Due to constraint of pages, technologies of diaphragm wall are mainly focused on concrete diaphragm wall as mention in section1. The “slurry-cement wall”, the “soil-cement wall”, “steel diaphragm wall” whose base material consist of special steel boxes and concrete or the soil cement have been already developed to mitigate impact to environment and save construction cost.

In the technologies of these alternatives to the concrete diaphragm wall, there are various challenging items such as study of quality of the slurry-cement, long term strength of the soil-cement wall, cost-down of the steel diaphragm wall. However, these alternatives have been applied to various projects instead of the concrete diaphragm wall.

Considering the features and characteristics of overall diaphragm wall types, the selection of the right method in the projects should be carried out from structural, environmental, economic and comprehensive points of views.

10. CONCLUSIONS

In this paper recent technologies on the diaphragm wall have been highlighted and reviewed whether they meet the demand from engineer point of view. Based on the review the technologies to be studied more and to be challenged to improve more to meet the demand in future have been proposed.

1) Slurry
   The polymer slurry has a lot of advantages over traditional bentonite slurry. Careful control, however, is inevitable.
   Development of the revised slurry which has advantage over the traditional bentonite slurry and the already applied polymer slurry is desired.
   The shaft resistance mobilized on the diaphragm wall constructed in the bentonite slurry is almost same as that in the polymer slurry subject to construction duration. In case of long construction duration the shaft resistance mobilized in the bentonite decreases.
2) Trench Stability
   Various overall trench stability methods have been proposed. Their concept however is different and recommended safety factor is also different.
   For the local trench stability methods, the studies of them have not been carried out so far.
   A comprehensive trench stability method considering the local trench stability should be carried out.
3) Concrete
   Maximum grade 60 concrete has been developed and applied to the projects.
   The demand of higher strength of concrete will be increased especially for underground circular structures
   At the present Grade 80 to 100 concrete have been already developed based on laboratory and trial base
   The balance between improved concrete quality and construction method should be considered.
4) Re-bars
   As the depth of the diaphragm wall is increased the downward and upward movement of re-bar cages has been confirmed at the sites
   Theoretical analysis on this movement should be carried out
5) Removal of slime and Control system of Vertical Accuracy
   As the depth of the diaphragm wall is increased, removal of slime and control of vertical accuracy of excavation machine have become more difficult
   Combination of replacement of new slurry and Base Grouting method would be the most suitable solution to compensate the lack of slime removal
   The maximum depth of the diaphragm wall has been probably 150m in the trial excavation with 50mm vertical accuracy using two wires control system. Until 150m depth the two wires system is applicable. However, for more than 150m depth the wire system would be difficult to be adopted because vibration and swing the wires. The control system of the vertical accuracy should be developed for the depth more than 150m
6) Special Excavation Machine
   Under sever restricted site conditions the availability of low head room machines and under-structure excavation with overcut of 5m using special tools and revised JGP is able to provide relieve to the problem.
   In future it is desirable to develop the ordinary machine to allow quick conversion with special attaching tools to meet the demand of under-structure excavation instead of mobilizing special machine.
   For the revised JGP, there would be high possibility to cater for overcut of more than 6m to 8m under existing structures with integrated further studies of jetting method, slurry and control of soil movement.
7) Alternatives to Concrete Diaphragm wall
   In the technologies for Slurry-Cement wall, Soil- Cement Wall and Steel Diaphragm Wall with special steel boxes, there are various challenging technologies. Comprehensive method selection of the diaphragm wall is desirable.

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