ADVANCES IN DIAPHRAGM WALL CONSTRUCTION

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ABSTRACT

Diaphragm walls are reinforced concrete retaining wall structures that are constructed by excavating the ground using slurry supported trenches. Although diaphragm walls have numerous benefits, and are a favourite option of engineers who intend to retain the ground without massive preparatory excavations or who desire to create dry excavated work places, and even though this type of wall has been in use for approximately 40 years in Australia it is the experience of the authors that this technology has not received sufficient exposure, its benefits and its construction process still remains mysterious to the typical geotechnical engineer. In this paper the history of diaphragm walls and the benefits of this wall system will be briefly presented, which will then be followed by a discussion on the how diaphragm walls are constructed and how technology has evolved and advanced throughout the past several decades. Reference to projects will also be presented to further demonstrate local Australian and international achievements.

1. HISTORY

Diaphragm walls are reinforced concrete retaining wall structures that are constructed by excavating the ground using slurry supported trenches; however the process that we refer to by this term today is the outcome of approximately one hundred years of geotechnical construction practices. Hajnal et al. (1984) relate the history of diaphragm wall construction to the first appearance of rotary drilling in 1901 and the application of a clay suspension as a drilling mud with borehole stabilising properties. Freundlich was among the first to investigate the thixotropic properties of bentonite as a colloid suspension, and analysed the stability of boreholes filled with slurry. By 1929, suspensions were used to stabilise boreholes (Hajnal et al., 1984, Soletanche Bachy, 2009).

The concept of diaphragm walls can be conceived as a development from bored piles to interlocking piles and further advancing to trenching under slurry. Veder is said to have designed the principle of a diaphragm wall in a slurry trench in 1938 (Soletanche Bachy, 2009). In 1934 a diaphragm wall consisting of 30 m deep interlocking piles was constructed at Nice in France for a gas project. In 1950 concrete for the piles of a power plant at Vitry in France was poured under slurry without the use of a steel casing, and in the same year Veder obtained a relatively smooth diaphragm wall by making vertical borings at each end of the panel, and excavating the remaining interlocking volume by a grab (Hajnal, 1984).

Marconi was the first to develop a method for making long panels using percussion drilling with reverse flushing by a rig which could be operated continuously parallel to a trench in 1953. Since then trenching equipment has continued to advance under the leadership of Europe in general and France, Germany and Italy in particular. Soletanche received its patents for a new generation of excavation equipment, the *hydrofraise,* in 1971 that has since become the generic term for the technology. 1984 saw the introduction of an innovation to cold joints with the appearance of Bachy's CWS joints. A year later, in 1985, Soletanche introduced the first generation of on board controls to measure the depth and inclination of the hydrofraise at in point in time. This technology assisted the hydrofraise to reach a world record depth of 122.5 m during the construction of Mud Mountain Dam in the United States in 1988 (Davidson et al., 1992), but the record only lasted until 1993 when barrettes of Petronas Twin Towers in Kualu Lumpur, Malaysia pushed to world record slightly deeper to 125 m. The first decade of the 21st century was the period of fine tuning. 2001 was the year that the steerable KS2 hydraulic grabs were introduced, and by 2006 the hydrofraise that was by then equipped with a new generation of on board guiding systems achieved an impressive tolerance record of 0.1% during the construction of the 65 m deep Carrousel circular diaphragm wall for the A86 road tunnel in Versailles, France (Soletanche Bachy, 2009).

The diaphragm wall does not require footing and can be either self-supporting by cantilever behaviour or can be laterally supported by anchors; hence it can be constructed before excavation. Other benefits of the diaphragm wall include the ability to be installed to any practical depth, to have recesses and couplers for connecting to other structural elements, and to incorporate soft eyes for facilitated passage through the wall during drilling and tunnelling. Its number of construction joints compared to other types of footing-less walls is the least, it is watertight, and can have water stop strips installed at the construction joints, and it has the most acceptable finished surface among concrete walls that are cast in-situ without formwork. Diaphragm walls are well known for their use as planar surfaces; however they can also be built as horizontally curved forms, which can at times bring huge financial and time savings to a project by eliminating or minimising the requirement for lateral supports.

2. SLURRY

As could be understood from the history of diaphragm walls, the concept of this technology is based on constructing a wall by trenching, and this would not be possible without reliable slurry, which provides trench stability from commencement of excavation to completion of concrete filling.

The most frequently used slurry for the construction of diaphragm walls is a mix with the main ingredients being bentonite and water. The main types of bentonite for industrial purposes are natural sodium, natural calcium and sodium-activated bentonites. Sodium bentonite expands when wet, absorbing as much as several times its dry mass in water. The property of swelling also makes sodium bentonite useful as a sealant, since it provides a self-sealing, low permeability barrier. Calcium bentonite has a much lower swelling ability and liquid limit, and a much higher filter or fluid loss than sodium bentonite and is hence not the choice for stabilising open trenches. Sodium-activated bentonite is the product of adding sodium carbonate, also known as soda, to calcium bentonite which makes the bentonite display the properties of natural sodium bentonite.

Stability of slurry-supported trenches has been attributed to the hydrostatic lateral pressure from the slurry, plastering of the slurry on the trench wall to prevent movement of individual grains of the native soil, a structural membrane effect created by the bentonite filter cake on the trench walls, gelling of slurry that has penetrated into the native soil, electro-osmotic forces, passive resistance of the slurry in the trench, and three-dimensional effects (Fliz et al., 2004). Filz et al. also note that plastering of slurry on the trench wall produces a filter cake that helps to stabilize individual soil grains, but the filter cake itself is not strong enough to produce a structural membrane effect that would make any significant contribution to the global stability of the trench.

Common values of bentonite slurry unit weight are from 10.3 kN/m^3 to 11 kN/m^3. Although increasing the slurry unit weight will increase its pressure and thus increase the trench's stability, the viscosity of the slurry should not be too high in order to protect the equipment, to ensure the quality of the concrete, and to avoid the undue spoiling of the slurry. Hanjal et al. (1984) report that prior to 1969 relatively high slurry unit weights (larger than 11 kN/m^3) were quoted; however, in projects such as Mallemort Dam with slurry unit weight of 11.5 kN/m^3 this caused defects in the concrete of four panels. There is no mixing between two fluids of different viscosities if the one with the higher viscosity penetrates into the fluid with the lower viscosity, but mixing can occur in the reverse case. If the shear strength of the slurry gel is too high, the concrete might cut through the gelatine like matter and slurry lenses might be enclosed, reducing the quality of the concrete and its adhesion to the reinforcement. At the same time the workability of concrete requires a relatively low shear value, but a high shear value is needed to prevent the mixing of concrete and slurry. Considering the fact that shear strength of slurry depends on the time on a logarithmic scale and on the concentration on an exponential scale, low slurry unit weights in addition to reducing the technological periods should be used. Experiences show that this requirement is satisfied if the unit weight of the slurry does not exceed 11kN/m^3, or in exceptional circumstances 11.2 kNm^3 (Hanjal et al., 1984). On the other hand, some viscosity is necessary to grease and cool the equipment, and in the case of reverse circulation excavation, to transport the muck to the surface.

Natural bio polymer slurries are sometimes used in lieu of bentonite slurries. Although these bio polymers cost more than bentonite, and their slurry mixes are more complex than the straight-forward bentonite slurry, they can have advantages that make them attractive to some projects. The most common polymer slurry is biodegradable guar gum. It is a naturally occurring carbohydrate polymer that is derived from guar beans. Guar gum slurry is broken down by naturally occurring micro-organisms or by introducing enzyme compounds and bleach, and do not require to be disposed off-site at special landfills or tips. This feature makes biodegradable polymer slurry an attractive alternative to bentonite slurry when environmental issues are of concern. Residual by-products (prior to consumption by soil micro-organisms) are simple sugars and water. Guar gum slurry is relatively instable on its own, but the active life of the slurry can be extended by about a week with additives. While bio polymer slurry is resistant to most contaminants, hot weather and concentrated micro-organisms can create a situation in which stability is much more difficult to control (Day et al., 1999).

Synthetic bio polymer slurries exist in the form of a synthetic low molecular weight cellulosic polymer formulation combined with a colloidal mineral suspension. After penetrating porous formations, this fluid forms a thin low permeability filter cake that is capable of maintaining a substantial head over groundwater level. Despite the cake formation capability, the trenching fluid has little or no gel strength so that all suspended solids finally segregate and is thus self-cleaning. Also, the polymer slurry contains an additive preventing the swelling of clayey soils. This reduces the weakening of clayey trench walls, and limits the amount of clay dispersing in the slurry which reduces the potential for gel formation. In addition, the metallic surfaces, such as the shells of the grab, remain free from sticky clay (Tallard, 1992).

Synthetic bio polymers easily disperse in fresh water at a rate of 0.1 to 0.2% by weight of water. By comparison, a natural gum bio-polymer requires 0.5% to 0.8% to produce the same viscosity, and bentonite requires 4 to 6% by weight of water (Tallard, 1992). Viscosification of bio polymer slurry occurs within minutes with gentle agitation; however bentonite slurries require at least 24 hours for hydration.

3. EXCAVATING TOOLS

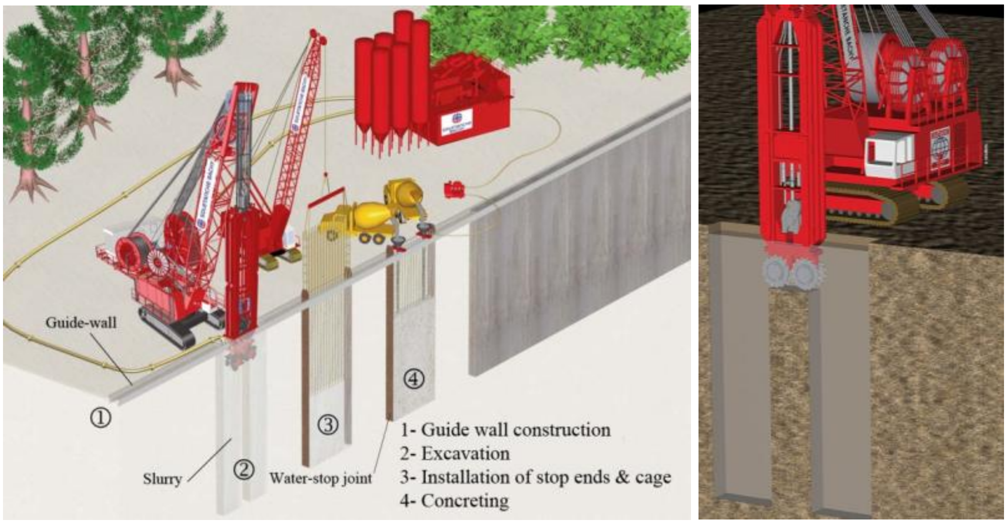
The most common trenching tool for typical diaphragm walls is the grab or clamshell. Grabs can be either mechanical or hydraulic, and they can also be either suspended or connected to a kelly bar although the former is more common. Grabs come in different conventional widths of as narrow as 0.50 m to as wide as 1.50 m. Grab lengths have been standardised by diaphragm wall specialist contractors, for example Soletanche Bachy’s KL mechanical and KS hydraulic type grabs are 2.80 m, which is also in the range with other manufacturers. However, grabs with other lengths may also be manufactured for a specific project. For example, a 0.8 m by 5 m grab was specifically made for taking large 5 m bites at Ho Chi Minh City’s Thu Thiem Tunnel. Grab heights are from as low as approximately 5 m to as long as 10 m. These large sized equipments with considerable lengths use assistance from their weight during the biting process and are able to maintain verticality in the first several metres of excavation while they are in the still in guide wall height. Grabs are designed in a manner that during the biting process the grab readjusts its elevation so that its jaws remain at the same level throughout the biting process. This ensures that the bottom of each bit and consequently the toe of each panel remain planar rather than curved.

Grab excavation is relatively efficient at shallower depths, but as the depth of excavation increases, the distance that the grab has to travel up and down the trench for each bite increases until the grab will be utilising most of the work time in travel. Experience shows that for depths beyond approximately 40 m it is beneficial to use an alternative technology that implements a reverse circulation method for removing the excavation cuttings and spoil to the ground surface. The *hydrofraise*, sometimes translated to English as hydromill, was developed and patented by Soletanche in 1971. The hydrofraise has hydraulic motors to drive rotary drum cutters.

Both grabs and hydrofraises that are currently manufactured are equipped with real time control and reporting systems to assist in maintaining the walls coordinates. The features include control of deviation in the excavation plane vertical to cutting tool penetration and rotation. The ventilation and emergency access shaft for the A86 Road Tunnel, which was built as part of the Socatop project in 2006 in France was 65 m deep and its verticality tolerance was 0.3%. Comparison of this figure with the 4% that Australian Standard (2009) allows demonstrates the stringentness of the criterion; however an unbelievable record of 0.1% vertical deviation was achieved using the state of the art Hydrofraise Evolution 2.3 equipped with Enpafraise 3 guidance technology.

While the typical hydrofraise is about 10 m long, and sometimes extended to become as long as 24 m (Davidson et al., 1992), there are many cases where the machine has to be compact enough to fit in confined spaces. The frequency of such needs in congested metropolitan areas has proven to be so great to justify the manufacturing of compact excavtion tools and rigs. Figure 1 shows the operation of a modified crane and grab under a roof and in a confined space for the construction of National Art Gallery in Singapore.

Figure 1: Specially designed low headroom excavation rig at work in a confined space of Singapore National Arts Gallery



(a) (b)

Figure 2: The process of (a) diaphragm wall construction and (b) excavating a panel

4. DIAPHRAGM WALL INSTALLATION

4.1. PREPARATORY WORKS

As the name implies, a guide wall, shown in Figure 2(a), is a short set of walls that is built on the two sides of the diaphragm wall during preparatory works, and serves mainly as a means to guide the trenching tool into the wall location. Other functions of the guide wall includes controlling the trenching path, serving as a base line for controlling levels, protecting the edges of the trench from the trenching tool and slurry waves during trenching tool penetration and withdrawal, and for providing stability in the uppermost excavation levels.

Bentonite is usually mixed with water in a high shear mixer to ensure that most amount of individual bentonite grains have well mixed with water, and additives such as sodium carbonate may be added to the mix to activate or enhance it. The newly mixed slurry can be used immediately after mixing if it is able to meet the specifications; however it is usually stored in a pond or tank, and in consideration of its thixotropic properties kept in circulation, for a period of at least 12 hours and up to 24 hours to hydrate sufficiently.

British Standard Institute (2010) specifies limits for density, viscosity (measured using the Marsh cone), fluid loss, pH, sand content and filter cake thickness for the re-use of slurry and for slurry before concreting stages; Federation of Piling Specialists (2006) has almost identical specifications as the British Standard and both specifications are commonly used in Australia.

4.2. TRENCH EXCAVATION AND CAGE PLACEMENT

Perhaps, the first thought for constructing a diaphragm wall would be to start excavating from one end and to construct one panel after the other until the end of the wall is reached. In fact, this is what is basically done for slurry type walls, such as soil bentonite or cement bentonite walls. However, this approach does not work when concrete and steel cages are involved, and excavation cannot proceed continuously without major delays due to the minimum required setting times of the already poured concrete.

Consequently, diaphragm wall excavation is done sequentially as primary and secondary panels, with secondary panels being installed only after the primary panels on the two sides have been constructed, and have reached the minimum time for reaching sufficient setting.

Although theoretically a panel length can be any figure, in practice it is usually a function of the excavating tool, lift capacity and concreting capacity. Common panel lengths in Australia are usually from 6.2 to 7 m. For excavating a panel initially the two sides of the panel are dug to depth while slurry fills the trench and stabilises it. As a last step the central column of ground is excavated to the required depth. This process is shown in Figure 2(b).

The reinforcing steel bars of each panel are fabricated into the shape of a cage, and lifted and placed into the excavated panel as shown in Figure 2(a). The length of the steel cage will depend on availability of space, lift capacity, and the required reinforcing depth that may be shorter than the diaphragm wall depth itself.

In addition to the structural steel that forms the cage, each cage can also incorporate special components to make the diaphragm wall a versatile wall subject to numerous options. Couplers that are installed in the cage by screwing them from one end to steel bars in the cage allow simple and highly reliable connection points with the future beams and slabs in the wall. All that has to be done at later stages in to screw the beam steel bars to the couplers. Furthermore, recesses can be formed for mechanical transfer of beam and slab loads by installing *block-outs* made of polystyrene sheets. *Trumpets* can also be placed in the cage for the future ground anchors. Rather than placing steel bar everywhere in the cage, it is possible to place glass fibre bars at locations where holes have to be drilled through the wall to improve drilling time and cost. Figure 3 shows the cage with glass fibre bars at Binningup Desalination Plant, Western Australia.

Figure 3: Implementation of glass fibre bars at Binngingup Deslaination Plant in Western Australia

4.3. CREATING WATERTIGHT JOINTS

Contrary to other installed reinforced concrete wall systems such as secant or contiguous piled walls that have cold joints, it is possible to make diaphragm wall panels water tight by incorporating PVC water bars at the panel ends. Prior to the early

1980s, construction stop end forms, usually having a cylinder shape, were placed at the two ends of primary panels to create an uncontaminated clean interface between panels. The stop ends pulled out vertically before the setting of the primary concrete panel. This procedure required considerable knowledge for extracting the stop ends too early would have caused the concrete to collapse into the void, and pulling out the stop ends too late would have required considerable amounts of vertical pull out force.

Although tubular stop ends are still being used by some diaphragm wall contractors they should be deemed as a technology of the past. In 1984, Bachy was awarded the innovation prize by the National Public Works Federation of France for inventing its patented CWS joint (Soletanche Bachy, 2009). This new stop end installation process was not only able to eliminate the need for the immediate removal of stop ends during the concreting phase, but was also able to incorporate the installation of water stops between adjacent wall panels (Depeuble, 1985; and Vanel, 1992).

The CWS stop end is flat on the side facing the secondary panel and projected with a groove on the side facing the primary panel. A water-stop strip, similar to what is used in other water tight structures, is placed into the groove, and as shown in Figure 4(a), the stop end is placed into the trench.

The stop end's flat side acts as a guide for excavating the secondary panel, and it is only removed after placing of the steel reinforcing cage and concreting the primary panel and excavating the secondary panel. Then, the stop end is pulled off laterally, leaving behind the water-stop. This water tight joint is shown in Figure 4(b) after excavation of the secondary panel. It can be seen that the extended joint width has a tongue and groove shape that is also able to provide mechanical continuity between the panels.

(a) (b)

Figure 4: (a) CWS water stop installed in a primary panel and (b) watertight joint visible after excavation of the secondary panel

4.4. TRENCH CLEANING AND CONCRETING

Due to the slurry viscosity, soil particles will contaminate the fresh slurry during excavation, and will remain either suspended in the slurry or will slowly descend to the bottom of the excavated trench. Depending on the slurry viscosity and types of soil encountered, the contamination may be up to 20% by volume (Tallard, 1992). Contaminated slurry can create defects, and hence the slurry must be de-contaminated by desanding. Also, the presence of loose or soft sediment can induce uncalculated and excessive settlements; thus any such sediment must be removed by an appropriate means such as air lifting.

Slurry desanding and bottom cleaning is achieved by extracting slurry from the trench, with sand removed by a desanding unit. Slurry extraction is either by submersible pumps or airlifts. An airlift consists of a tremie pipe assembly that extends to the base of the excavation, fitted with an external air line for delivery of compressed air. At the airlift head the airline is returned 180 degrees to propel slurry up the tremie pipes to the surface. The velocity at which the slurry moves up the pipe creates a suction effect at the head, allowing successful removal of rock, gravel or fill size particles up to that of cobbles.

For both pumping and airlifting, best practice to prevent blockages requires commencement of extraction above the panel toe when suspended matter within the slurry is at its maximum.

Once out of the trench, the slurry is passed through a mechanical separation unit called a desander, which usually consists of vibrating screens and a centrifuge, also referred to as a hydrocyclone, to separate the solid particles from the slurry. The cleaned slurry is then returned back to the excavation.

British Standard Institute (2010) does not have a limit for sand content for reuse of slurry, but limits the percentage of sand content in volume to less than 4% before concreting. An upper limit value of 4 to 6% can be used in special cases; e.g. for the unreinforced sections of the wall).

Concreting of the trench should start as soon as possible after airlifting and desanding the trench to avoid build-up of filter cake on the sides of the excavation, degradation and increase of slurry viscosity, and re-sedimentation of soil particles onto the bottom of the excavation.

Concrete is placed into the bottom of the trench using the tremie method. The bottom of the tremie pipe should be maintained a minimum of 3 m below the top of the concrete at all times during the placement. Sufficient number of tremie pipes should be used during concreting for the concrete to be able to rise uniformly and to adequately displace the slurry from the bottom upwards. A rule of thumb is to use one tremie per 4.6 m of panel (Millet et al., 1992) which is in loose agreement with the author's experience of using two tremie pipes for a typical wall panel of up to about 7 m length to displace slurry vertically rather than horizontally.

For diaphragm walls, the concrete must have good flow characteristics with a slump of approximately 180 to 230 mm to assist scour at the sides of the excavation, ensure concrete spread and maximise slurry displacement. Vibration of the concrete is not required due to the high slump of the concrete mix creating a self-compacting mix.

5. DIAPHRAGM WALL SHAPES

While it may be a misconception that diaphragm walls must be built as planar walls following a straight line path, this is far from the truth. Figure 5 shows the numerous types of panels that can be constructed (Tamaro, 1992) by constructing the guide walls as needed , and taking bites in appropriate directions while excavating. Implementation of these panel types can realise almost any shape with practical dimensions.

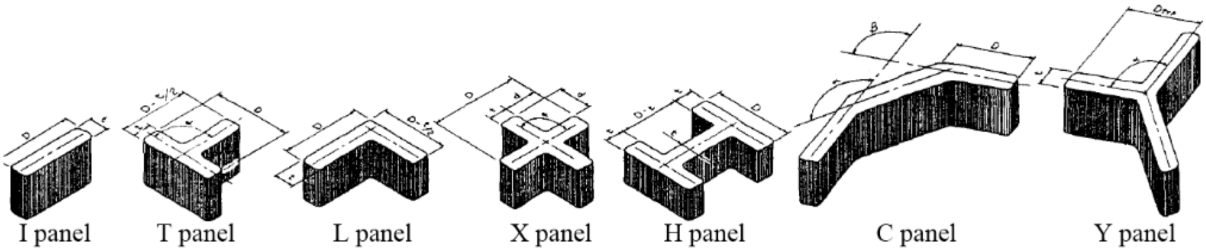


Figure 5: Panel shapes (Tamaro, 1992)

The versatility of diaphragm wall panel shapes has made this wall type a favourable option for constructing, among other structures, numerous water storage reservoirs and tunnel shafts. The circular shape of these structures is efficiently able to implement the ring effect to withstand horizontal loads without ground anchors or internal strutting. This can lead to considerable reduction of construction time and cost.

(a) (b)

Figure 6: Construction of circular shafts at: A86 Tunnel in France, and (b) Lee Tunnel in England

(a) (b)

Figure 7: Application of peanut shaped diaphragm walls at (a) The Sail project in Singapore and (b) Lievin stormwater basin in France

Two recently deep circular diaphragm walls that have been recently constructed are the ventilation and emergency access shaft of A86 Road Tunnel and Lee Tunnel shafts, built in London in 2011. A86 Road Tunnel was constructed as a bored tunnel at depths of up to 80 m between Jouy-en-Josas and Rueil-Malmaison in France. As shown in Figure 6(a), the diaphragm wall that formed the shaft had an internal diameter of 7.80 m. Lee Tunnel had four shafts with internal diameters of 23 m, 28 m, and 40 m. The depths of these shafts were 72 m, 79 m, 81 m, and 82.5 m.

When the project's geometry does not allow the utilisation of a circular shape, it is possible to implement curved walls to minimise the wall's required horizontal resistance. The implementation of a number of such arcs can be used to construct retained excavations with one dimension being considerably larger than the other. Figure 7(a) shows the triple celled *peanut* shaped The Sail at Marina Bay in Singapore. This diaphragm wall did not have any internal struts and horizontal forces were resisted at two superficial points of the intersecting arcs. Similarly and as shown in Figure 7(b), Lievin stormwater basin utilised a six-celled peanut shaped diaphragm wall to store 10,000 m^3 of water.

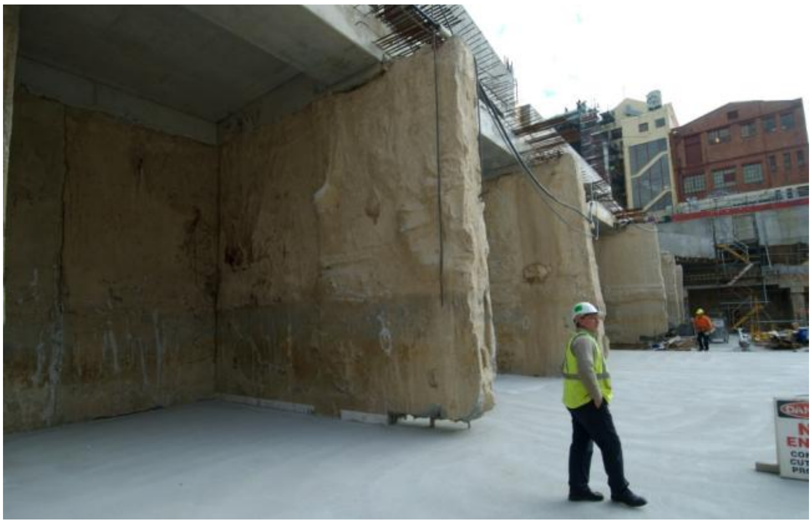


Figure 8: Application of temporary internal T panel buttresses at Perth's 100 St Georges Terrace tower

There are also instances when for some reason neither circular or arched diaphragm walls, ground anchors nor internal struts can be used. In such cases it is possible to use T shaped panels to create buttresses. If space allows, the buttresses can be built behind the wall and remain in the ground permanently. Otherwise, as was the case for 24 story 100 St Georges Terrace building in Perth, diaphragm wall buttresses were installed temporarily inside the excavation and construction zone, but were gradually demolished and replaced by the floor slabs. The buttresses in this building are shown in Figure 8.

6. CASE STUDY: BINNINGUP DESALINATION PLANT

Following conception by the government of Western Australia, a 20 ha site located approximately 1 km north of the town of Binningup and 120 km south of Perth was secured by Water Corporation. At full capacity of phase 1, the desalination plant will provide 50 giga litres of water per annum to Perth's current supply, with the potential to expand to 100 giga litres per year. When combined with the current contribution from the Kwinana Desalination plant and other smaller plants, 30% of Western Australia's water supply will be supplied by the desalination process. This project included the construction of diaphragm walls for the Intake Pump Station (IPS); the subterranean structure receiving sea water from bored tunnels for on-pumping to the processing sections of the plant (Webley, 2010).

The final design concept for the IPS diaphragm walls consisted of 6,250m^2 of panel construction, forming 5 separate chambers with common walls. Two intake chambers had the ancillary function as Tunnel Boring Machine (TBM) launch shafts for pipe jacked tunnels, the brine chamber (outfall) had the ancillary function as TBM launch shaft for pipe jacked tunnels, and there were also two pumping chambers.

To accelerate the construction programme and eliminate requirement for ground anchors and tiebacks, top down construction methods were selected to form each of the shafts. Following roof slab construction, panels were designed to cantilever approximately 10 m during bulk excavation to base slab level.

The preliminary geotechnical investigations indicated that voids in the underlying Tamala Limestone presented risks associated with piping failure and base slab heave following decommissioning of dewatering. To counter these risks panels were extended to depths of 25 m, achieving a socket into the underlying Leederville formation.

Further challenges for diaphragm wall reinforcement design were posed by the IPS' 100 year design life in an aggressive salt water environment and stringent crack control requirements, complex loading scenarios given the staged excavation and flooding process, and reaction loading during pipe jacked tunnel construction.

Typical ground conditions in the area of the IPS consisted of 4 m of medium dense sand, 3 m of sandy clay (lagoonal deposits, designated as acid sulphate soils), 6 m of dense sand followed by Tamala Limestone down to the depth of 25 m. In some areas high strength sandstone with an unconfined compressive strength of more than 100 MPa was encountered from the depths of 22.5 to 24.5m.

Panel excavation was carried out by two rigs, each equipped with 2.8 m long KL mechanical grabs digging under a bentonite slurry. To facilitate excavation in rock, each rig was also equipped with star, cross and box chisels. Upon encountering hard ground, a chisel is attached to the rig, and allowed to freefall onto the underlying rock. With each chisel's weight exceeding 10 tons, successive raising and dropping is relied upon to break up the rock, for subsequent grab removal. Excavation rates exceeded 10 m^2/hr upper 12m. Excavation in the next 13 m, with the sandstone in particular, was more challenging. Initially, it was required that the diaphragm wall penetrate 2.5 m into the sandstone. Total excavation time for this socket totalled 75 hours, with an average penetration of 71 mm per hour for each pass. Maximum rock socket requirements were revised to 0.5m thereafter.

The Tamala limestone at the location of IPS proved to be highly variable in strength and permeability due to voids encountered within the rock mass. High losses of bentonite were experienced in this formation as a result. Over the course of the project 1,750m^3 of support fluid were mixed for supply to each rig during excavation, with 80 m^3 disposed of upon completion. This deficit equates to an average loss of 44 m^3 per panel approaching 30% of theoretical volume. Increasing bentonite viscosity and density was considered to limit losses; however the detrimental effects on desanding and concreting negated this action, and oversupply with close monitoring of bentonite levels within the excavation enacted.

The challenges discussed earlier were manifested in the structural design of panel reinforcing cages. As constructed cage weights exceeded 360 kg/m^3, with a total of 10 layers of reinforcing bar required to account for the complex bending and crack control requirements. This amount of steel, which was designed for specific water tank requirements, was about double of what would be encountered on a typical diaphragm wall project.

To maintain programme a fabrication crew of 35 persons was required to produce an average of three completed cages per week. Cages were constructed flat on ground, and lifted to vertical for installation into the open excavation by a 150 ton crawler crane. The unit weight outlined above required the specialist contractor to conduct detailed lift assessments, in order to prove that crane capacity would not be exceeded and to ensure safety of lift personnel. The assessments found that any cage larger than 3.1m in width could be lifted as a single 25m long unit, with a maximum cage weight of 33 tons. Any cage wider than 3.1m required fabrication in two separate sections that would be installed separately and spliced over the excavation. Maximum cage weights lifted from horizontal in this case was 42 tons, with a maximum spliced lift of 72 tons. Prior to placement of the reinforcing cage the excavation bases and walls were freed of bentonite cake and settled material by cleaning with the grab. The panels were then desanded to remove the suspended sand from the bentonite slurry.

Concrete was placed by the tremie method. Immediately prior to placement of concrete loose vermiculite is poured onto the bentonite surface within the tremie annulus to form a plug between it and the concrete when discharged. Given the heavy, closely spaced reinforcing a high flow self compacting concrete was specified for use in the diaphragm walls. To further ensure concrete encasement of the bars, the maximum aggregate size was limited to 10 mm. In consideration of the aggressive environment, hydrophobic admixtures, which fill pores within the concrete and reduce permeability, were also included in the mix design. Some interesting project data are summarised in Table 1.

Table 1: Summary of project data (Webley, 2010)

Description	**Comment**
Works type	Construction of diaphragm walls and barrettes
Volume of slurry used	1,750 m^3
Volume of leftover slurry	80 m^3
Percentage of slurry loss	Approximately 30% of theoretical excavation volume
Deepest excavation	25 m
Heaviest weight of cage lifted	42 tons
Heaviest cage after splicing	72 tons
Weight of steel per cubic metre of diaphragm walls	360 kg/m^3*
Total concrete volume in diaphragm walls and barrettes	6,250 m^3
Total steel weight in diaphragm walls and barrettes	1,900 tons
Concrete permeability	3×10^{-12} m/s

** Weight of steel/m^3 of diaphragm wall was very high in this project due to special requirements for cracking control.*

7. CONCLUSION

Diaphragm walls came into being as the result of slurry application in drilling and as an off-shoot of piled walls. Today, this technology is deemed as a superior wall solution and it implemented in thousands of projects that require watertightness, deep excavation with stringent verticality requirements, and when space limitations does not allow for the installation of ground anchors or internal strutting.

The equipment used in this process has also evolved and the introduction of superior mechanical components with state of the art electronics and software allows works to be well recorded, controlled and corrected to reach maximum compliance with design objectives. Examples of projects that have reached such achievements have been given in this paper.

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