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Reinforced earth seawalls at Sydney Airport

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ABSTRACT: The use of Reinforced Earth was critical to the success of the construction of the Parallel Runway Project at Sydney Airport. The project required extensive reclamation of the sea into Botany Bay. The Reinforced Earth system allowed record construction rates and provided a durability which exceeded the client's requirements. This paper outlines the elements of design and construction of the Reinforced Earth system.

1. BACKGROUND

Sydney's Kingsford-Smith Airport is Australia's main international gateway, handling more than half of all international air traffic. It is also the major domestic airport and acts as the principal hub of Australia's aviation network.

As forecasts indicated that total passenger traffic at Sydney Airport will be more than doubled by 2010, a third runway at Sydney Airport was considered the only practical way to avoid the development of extensive delays (Kinhill Engineers 1990).

The location and length of the new runway was selected to achieve a balance between the airport's operational needs and environmental and cost factors.

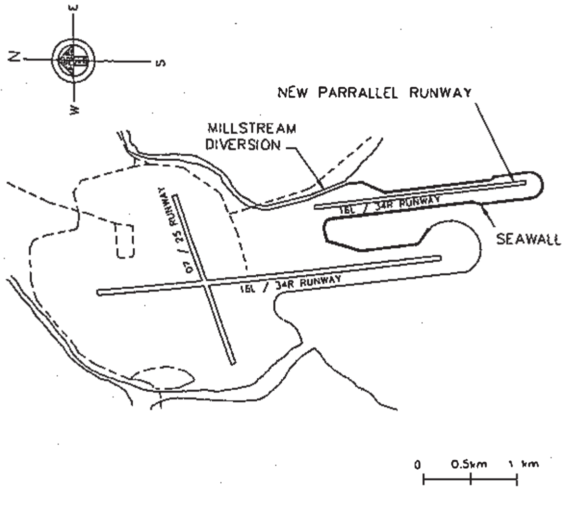

Figure 1. Layout of the Project

The runway is 2,400 metres long and parallel to the main runway at a distance of 1,037 metres (Figure 1).

The principal elements of the project were:

1. Reclamation of 138ha of land from Botany Bay using 15 million m^3 of dredged sand fill
2. Diversion of 1 km of the Mill Stream channel
3. Construction of a 7 km seawall
4. Construction of the 2,400 metre runway and associated taxiways, two bridges and aprons

The seawall was designed as a vertical wall to avoid the creation of a habitat for wading or roosting birds, thereby reducing the potential for bird strike.

The original seawall design incorporated steel sheet piling to provide the vertical seawall and to contain the reclamation.

2. PROJECT DESCRIPTION

2.1 *BHDRC's design alternative*

The contract was won by the Baulderstone Hornibrook Dredeco Runway Consortium (BHDRC) - a consortium of Baulderstone Hornibrook Engineering and Dredeco. The BHDRC design was based on standard construction of dredged fill 'bunds', dewatered for the construction of the seawalls and contained in an efficient silt screen to meet the environmental limits for turbidity control. This was essentially the same technique which was adopted on the original runway extension but with more elaborate environmental protection.

The selection of Reinforced Earth for the seawalls by BHDRC was made after detailed evaluation of a number of alternatives, including steel sheet piling, precast concrete wall units, diaphragm walls and Reinforced Earth. Both wet and dry construction techniques were considered. Ultimately, Reinforced

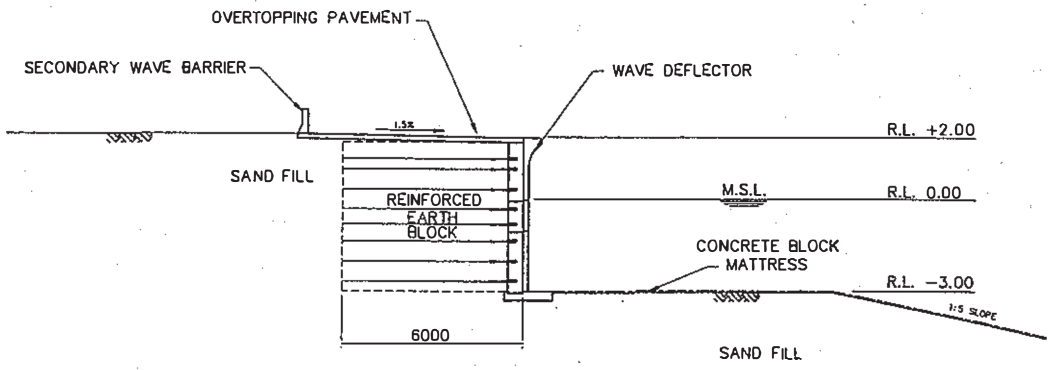

Figure 2. Typical Seawall Section

Earth, constructed in the dry, was selected on the basis of risk, constructability, durability and cost assessments.

Reduced risk was assured from experience. In addition the proven constructability of the Reinforced Earth solution allowed the contractor to optimise his construction method to minimise cost and time. The enhanced durability of the Reinforced Earth solution further ensured that the design criteria could be exceeded with significant cost savings

A typical seawall section is shown in Figure 2 .

Both two and three dimensional model tests were carried out to refine the wave deflector profile and the scour protection system, by Australian Water & Coastal Studies Pty Ltd (Gomes and Cox, 1995). Foundation analyses and global stability studies were carried out by Douglas Partners (Coull and Wiesner, 1995)

2.2 *Site Conditions*

Botany Bay is a semi-enclosed tidal estuary for both the Cook and Georges Rivers. Water depths are generally less than 8m with the seabed mainly fine to medium grained sands with some bands of organic clay. The sands varied from very loose to very dense.

Three wave conditions were quantified for the location of the runway - frequent, maximum and extreme. Frequent conditions (20 knot wind) produce a significant wave height of 0.4 metres. Maximum operational wind condition (40 knot wind) produce a significant wave height of between 0.8 metres and 1.0 metre. This relates to a 1 in 5 year event. The extreme wave conditions, relating to a 1 in 100 year event, produce a significant wave height of 1.7 metres with a mean period of 4.5 seconds from the South West.

Mean sea level is approximately RL 0.0 metres (AHD). Mean tidal range is from RL +0.64 metres to RL -0.68 metres. The top of the seawall is at RL +2.00 metres.

2.3 *Design criteria*

The seawalls are required to support the runway embankment and to allow airport operations up to the maximum operational wind condition of 40 knots.

The original sheet pile walls were designed for a design life of 50 years. The durability and cost effectiveness of the Reinforced Earth seawall design allowed for the increase in the seawall design life of the seawalls to 100 years.

The Reinforced Earth structures are designed for a general live load surcharge of 5 kPa and a specific live load surcharge of 20 kPa over the access roads and parking areas

3. REINFORCED EARTH DESIGN

3.1 *Background*

The Reinforced Earth system has been used on many marine projects around the world. Standard Reinforced Earth systems have been adapted to marine environments in projects such as La Reunion (France) and Knabb Rock (UK). Modified facing systems have been incorporated in the seawalls along the Gaspe Peninsula (Canada). Underwater construction techniques have been developed for projects in the Solomon Islands and Canada.

The early experience of the Canadian seawalls is described by Gagnon (1979) and the general experience of seawalls in Canada and the United Kingdom by Wu and Smith (1990). The development of underwater construction techniques is described by Weinreb and Boyd (1992).

The Reinforced Earth solution is based on a proven technology with over 15,000 projects completed around the world. The combination of the earth and reinforcements with an articulated facing forms a massive yet flexible structure which can support large loads and accommodate differential settlements due to foundation variability without loss of structure integrity. Furthermore, in the marine condition, permeability of the Reinforced Earth structure internally and externally, allows it to efficiently accommodate large variations in water level.

3.2 *Wall geometry and configurations*

The geometry and configurations of the Millstream Diversion Channel Walls and the Seawalls are summarised in Table 1.

Table 1. Wall Dimensions

		Millstream	*Seawall*
Height,max	(m)	6.1	5
Height,min	(m)	4.5	4
Length	(m)	1,700	6,800
Area	(m^2)	10,000	30,000

The walls are vertical and in some areas incorporate a wave deflector profile at the top of the wall.

The earth reinforcement is Reinforced Earth High Adherence Steel (REhas) reinforcing strip of dimension 50 by 10 mm (Millstream) and 55 by 15 mm (Seawall), to allow for the different design life of each wall.

The facing of the Millstream Diversion Channel walls are adapted from the standard Reinforced Earth system using cruciform shaped panels, 180 mm thick and nominally 1.5 metres by 1.5 metres in area. Maximum panel weight is approximately 1 tonne.

The Seawall facing panels are designed as buttressed panels, 180 mm thick on the face and nominally 2 metres by 3 metres in area with 0.5 metre wide buttresses on the rear. Maximum panel weight is 3.5 tonne (without wave deflector) and 5 tonne (with wave deflector). See Figure 3.

The buttressed seawall panel system was first developed by The Reinforced Earth Company in Canada and used on approximately 125,000 square metres of seawall constructed on the Gaspe Peninsula in Quebec. Previously, buttressed panels had also been used successfully for coal slot storage bunkers at nine sites and power stations. The main advantage of these panels was stability during construction, which allowed for more efficient construction of the Reinforced Earth.

Some of the features and benefits of the buttressed panel system are listed in Table 2.

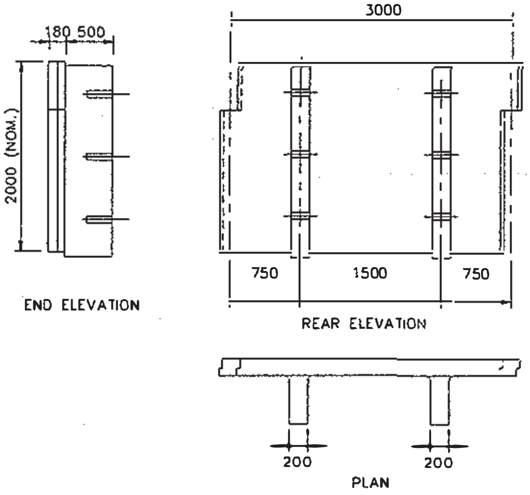

Figure 3. Seawall Panel

Table 2. Features and Benefits of the Seawall Panel System

Feature	*Benefit*
Large panel module	Speed of construction
Buttresses	Construction stability Connection set - back Drainage zone seperation
Integrated wave deflector	Optimum performance
Looped connections	Durability
Articulation	Settlement flexibility Construction tolerance Simple wall curves

While size and stability are important construction features in a project of this size, its ability to integrate the wave deflector profile and in the enhanced durability of the panel/strip connections are significant.

The Seawall is designed to be overtopped in extreme wave conditions. The permeable backfill and the panel joints will allow for free drainage of both overtopping and tidal water movement. In some areas additional drainage material is provided behind the panel joints. The durability of the system is enhanced by removal of the earth reinforcement and its connections from the drainage zone so that the steel components are totally encased in dense sand fill which will inhibit corrosion and ensure the reduction of corrosion rates with time. Furthermore, by using looped tie strips to connect the earth reinforcement

(REhas strip) to the panel buttresses, no steel connections were directly cast into the concrete.

3.3 *Design*

3.3.1 Material Characteristics

The soil properties adopted for the design are summarised in Table 3.

Table 3. Design Soil Properties

Properties	*Reinforced Earth Block*	*General Backfill*	*Foundation*
Friction angle (deg)	36	32	32
Cohesion (kPa)	0	0	0
Density (kN/m^3)	18 (max) 16 (min)	18 (max) 16 (min)	

The dredged sand backfill used in, and adjacent to, the Reinforced Earth structure was a uniform fine sand with general grading characteristics as shown in Figure 4. This material was compacted to a minimum Density Index of 80%.

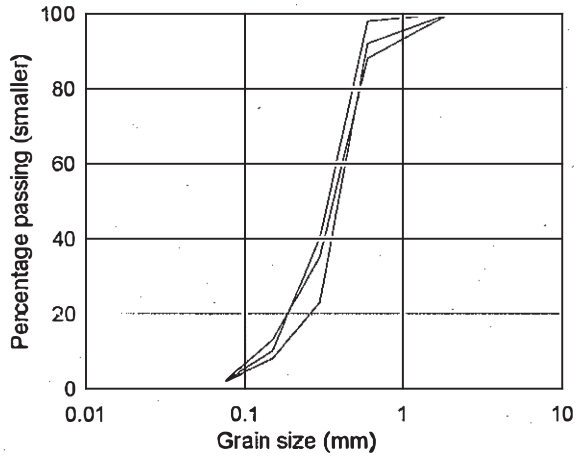

Figure 4. Grading of Sand Fill

A minimum concrete strength for the facing panels of 40MPa was specified, together with a mix design to enhance the durability of the panels.Key elements of the concrete specification were:

1. Binder content 350 kg/m^3 (90% ACSE Type GP cement, 10% Silica Fume)
2. Max. water/cement ratio 0.4.
3. Max. shrinkage 600 microstrain at 56 days.
4. Max. chloride permeability 1,000 coulombs at 56 days.

The minimum strength achieved with this mix was in excess of 60 MPa.

The steel earth reinforcement was structural grade mild steel with a minimum yield stress of 275 MPa (ultimate strength 410 MPa), produced to The Reinforced Earth Compny's specification.

3.3.2 Stability

The external stability of the Reinforced Earth structure was checked for sliding and overturning. The minimum factors of safety calculated were 1.5 (sliding) and 3.7 (overturning). The minimum factors of safety for internal stability were 3.1 (panel capacity), 2.2 (strip tension) and 1.8 (strip friction).

4. DURABILITY

4.1 *Background to durability design*

The durability of marine structures is well documented. The marine environment is relatively well known and steel has been commonly used in these structures for a long time. Sheet piling, in particular, has a long history.

In stable buried environments, the reduction of corrosion rates with time is well established. This can be defined by the relationship

$$P = A \, . \, T^{n} \qquad (1)$$

where P is the corrosion at time T, and A and n are constants (Darbin et al, 1988).

In an exposed, dynamic environment, the corrosion by-products are taken away from the metal surface, thereby maintaining its fresh state. The corrosion is always beginning and maintains its initial high rate. In a stable, buried environment on the other hand, the soil particles and the gangue which forms induce a chemical and physical change on the properties of the environment around the metal which impact on the corrosion rate. A non-linear corrosion rate is valid for long term behaviour.

For buried, marine environments, a value of n equal to 0.5 has been suggested as a valid prediction of the reduction of corrosion rate with time. For this project, a value of n = 0.65 was adopted, conservatively. The constant, A (representing the corrosion at 1 year), was taken as 130 um per year. This resulted in the corrosion prediction shown in Figure 5.

4.2 *Basis of corrosion estimates*

There is extensive data on the use of steel in marine structures. This is generally described by Morely and Bruce(1983), Boyd and Fink (1975) and Kinson, Lloyd and Eadie (1981) amongst others. More specifically, there are three relevant actual cases which validate the corrosion estimates we have used for this project. These are:

1. Examination of steel piling removed from Port Adelaide, Earth seawall at Gaspe, Canada (Quebec Ministry of Transport, 1983)
3. Investigation of the Australia, after 52 years service (Eadie and Kinson, 1980)
2. Study of the corrosion of the reinforcing strips in a Reinforced corrosion of a steel sheet pile wharf in Honiara, Solomon Islands (Cameron McNamara, 1986)

The data from the above studies is compared with our predictions in Figure 5.

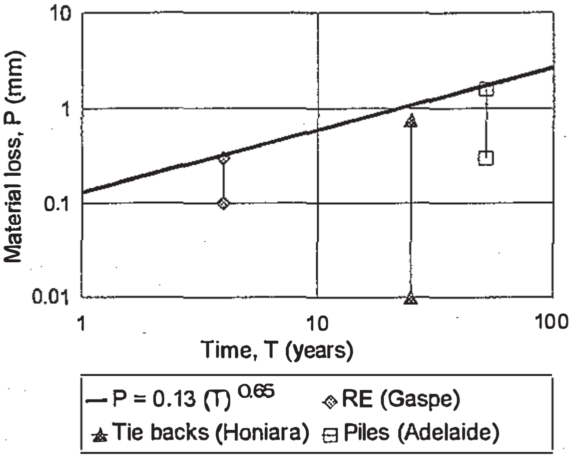

Figure 5. Corrosion/time relationship

4.3 *Effect of corrosion on loss of strength*

An innovative contribution to corrosion design by the Reinforced Earth Group is to relate the loss of thickness to loss of strength. Generally, pile structures have only been designed for a simple thickness reduction which takes no account of the variability of the corrosion process. Loss of thickness and loss of tension are related by the factor 'K' which is around 2.5 for mild steel and below 2 for galvanised steel in aggressive environments. For this project we have assumed a K value of 2.5.

4.4 *Durability design*

For the seawalls, the minimum original thickness of the reinforcing strip is 15 mm and original strength is 338 kN. For a design life of 100 years, the loss of thickness, based on the above relationship, is predicted to be 5.2 mm and the residual strength, assuming a K value of 2.5, is predicted to be 44 kN.

For a maximum design tension in the reinforcing strip of 20 kN, there remains a residual long-term factor of safety of 2.2, which is in excess of the 1.7 specified.

4.5 *Corrosion Monitoring*

Corrosion monitoring stations have been incorporated in the Reinforced Earth structures to allow simple, non destructive testing of reinforcing strip specimens over the life of the structure in order to maintain confidence in the ongoing serviceability of the structure and to provide data for future applications.

5. CONSTRUCTION

5.1 *Reclamation*

The sand fill for the reclamation was dredged from the proposed Port Botany expansion area at a rate in excess of 5,000m³ per hour.

The method of reclamation involved the forming of 70 metre wide bunds along the line of the seawall until the surface was clear of the water. The sand was dewatered sufficiently to allow excavation of a 30 metre wide trench to the level of the base of the seawall, then erection of the seawall panels, placement of the backfill and earth reinforcement. On completion of the wall construction to a sufficient height, the water level was restored. The portion of the bund remaining in front of the wall was then transferred to the back of the wall and the final bed profile achieved.

5.2 *Dewatering*

Dewatering of the sand was undertaken using 6 metre spears at 1 metre centres in two tiers. The spears were connected to 150mm pumps at 60 metre centres. Each pump is capable of handling 87 litres per second. Up to 27,000 cubic metres per day was pumped from each 100 metre length of trench.

Over 2,000 wellpoints and 40 pumps were used in this project.

5.3 *Erection*

The Millstream Diversion was erected at the rate of approximately 50 panels per day, or 500 square metres per week. Concurrently, the placement and compaction of approximately 4,000 cubic metres of backfill was achieved.

The Seawall was erected at a rate of approximately 50 panels per day also, achieving a total of 1,300 square metres per week. The size and configuration of the seawall panels allowed for the construction of over 200 linear metres of completed seawall per week.

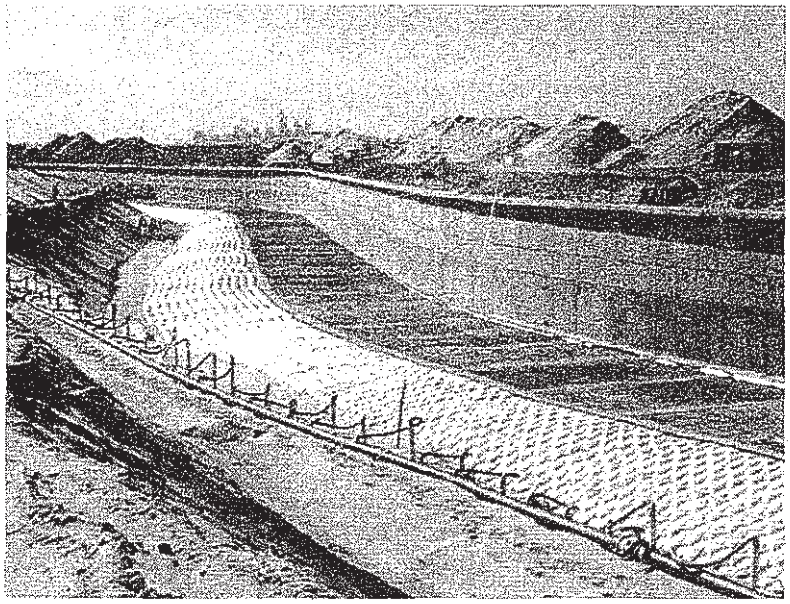

Figure 6. View of Seawall

6. PERFORMANCE

Some lateral movement of the seawall was observed during and after construction, particularly at the concave curves of the wall. The maximum movement was approximately 1% of the wall height, which was within expectations and most likely due to settlement of the sand and underlying material (Connell Wagner, 1995). This movement had essentially stopped by August 1995, with only one reading (out of 406) greater than 10mm measured over a four month period.

Observation of the toe of the walls by the Contractor has also indicated good stability of the general shape of the sand berms and no uncovered sections of the scour mat.

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