

Foundations to Precast Marine Structures

Martin Hawkswood, Proserve Ltd, 80 Priory Road, Kenilworth, CV8 1LQ, UK
martin@proserveltd.co.uk,

William Allsop, Technical Director, HR Wallingford, Howbery Park, Wallingford, OX10 8BA, UK, w.allsop@hrwallingford.co.uk

1. Introduction

Precast concrete elements are increasingly used in the construction of Maritime Structures. They offer the prospect of efficient unit production and rapid construction, but that requires the efficient construction of adequate foundation restraint. Foundation design and constructability for these elements is therefore a critical area, but little guidance is available on the different forms of foundation available. Typical underwater foundation systems have been:-



Figure 1—Confederation Bridge Site

Pre-levelled Bed

- Stone Layers

Base Infill

- Tremie Concrete
- Open Grouting
- Grouted Fabric Formwork (Grout Bags)
- Pumped sand

Weak / Inadequate Strata

- Piled Foundations
- Ground Improvement

This paper will describe these different foundation systems used underwater, and will describe particular experiences of combinations of systems used on completed and current projects by way of example:-

- Cardiff Barrage, Wales
- Second Severn Crossing, UK
- Confederation Bridge, PEI, Canada
- Central Artery, Boston, USA
- Greystones Harbour, Ireland

Maritime construction is usually a high risk operation that needs efficient and suitably robust design and construction methods to be developed. The relative merits of different foundation systems are also outlined in terms of design and constructability, thus seeking to inform Designers, Contractors and Owners in considering alternative and combinations of foundation systems.

2. Structure & precast element types

Precast concrete element solutions are often used on the following range of maritime structures:-

- Harbour, quay walls and seawalls
- Bridge piers
- Breakwaters
- Immersed tube tunnels
- Barrages (particularly turbine or control sluice housings)
- Wind turbine mast bases
- Other general maritime construction

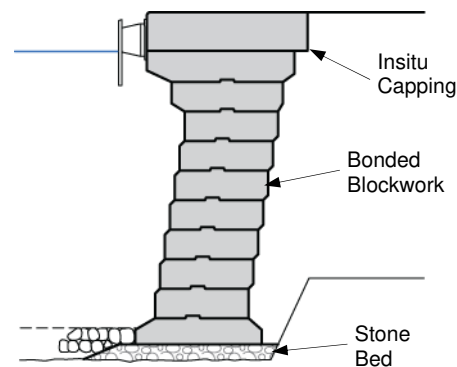


Figure 2—Quay Wall

Types of foundation elements often used:-

- Solid concrete blocks
- Hollow concrete blocks
- Open topped cell caissons
- Open base ‘shell’ caissons
- Immersed tubes
- Pier bases
- Mast bases
- Counterfort wall
- Other – purpose made

Closed bottom caissons and immersed tube elements are normally floated into place taking advantage of their natural buoyancy, before being lowered into place. Open base shell caissons can be lifted in by cranes or supported by pontoons (sometimes called “camels”).

Plain precast blocks are typically unreinforced giving advantages of increased longevity. Due to the generally aggressive exposure conditions of maritime works, reinforced concrete elements are often formed with a combination of protected reinforcement, increased cover and corrosion resistant concrete. This is particularly so in more extreme climates. Precasting often enhances quality control, allows economic repetitive production and a reduction of in-situ marine works to a minimum. Adequate plant and space is required within the precasting yard for casting, curing and storage. The elements can be used singularly, joined, stacked and arranged to form efficient and varied structures / foundations working at sea bed level.

3. Foundation Design

3.1 Structure Loadings

Typical load types for maritime structures

- Dead and imposed loads
- Wave momentum and impact loads, (+ve) and (-ve)
- Wave overtopping downfall loads
- Wave-driven internal and uplift pressures
- Current drag and lift forces
- Water pressures, tidal, uplift
- Seismic, wind loads, earth pressures, ice pressures
- Vessel berthing, mooring and impact

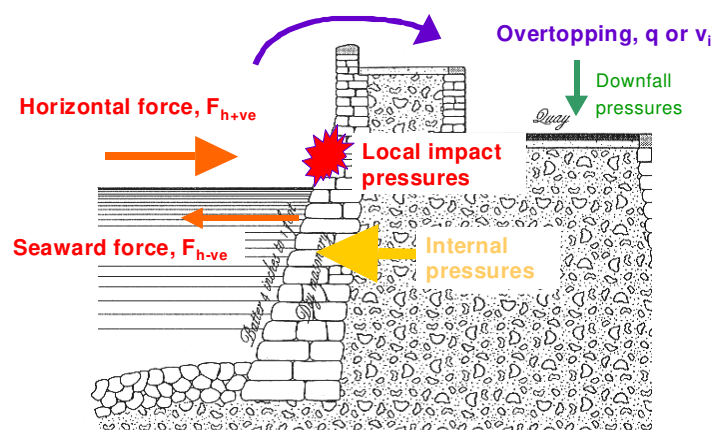


Figure 3—Wave-driven Loadings¹

Foundations need to be designed to accept all realistic loads imparted from the precast elements. This can often require design for:-

- Bearing pressures
- Overturning
- Sliding and slip circle failure where appropriate
- Settlement (overall or relative), short or longer-term deflection
- Scour protection
- Seismic and other dynamic effects (including effects of impulsive loadings)
- Suction pressures (from impermeable strata due to seismic / impulsive loadings).
- Filter failure, piping, wash out or suffusion (migration of fines).

In each instance, the full range of soil / foundation / structure interactions will need to be considered in the analysis. For structures subject to significant wave or tidal action, permeable foundation layers or strata can allow transmission of wave and hydraulic pressures and be at risk of filter failure, piping, washout or suffusion. Due to the very wide range of possible structures, foundation strata and load conditions, the above simplified list is only offered for initial guidance on foundation design and construction issues. Analysis / design procedures should be appropriate to the particular case considered, and should be in accordance with codes of practice and good practice.

3.2 Foundation Strata

The range of bed materials that may be encountered can be highly variable. Thorough site investigation should be carried out appropriate to the ground conditions, structure, environmental conditions and construction systems being considered^{2,3}.

Table 1—Typical Foundation Characteristics

<i>Strata Type</i>	<i>Marine Foundation Characteristics</i>	<i>Typical Allowable Bearing Pressures (kN/ m²)</i>
Hard Rock	Often with Steps or Trenches	> 2,000
Soft Rock	Can be dressed to level/ slope	500—2,000
Gravel to Sands	Settlement is short term May erode under wave or current action unless protected	75—500 Loose—Compact
Fine Sands and Silts	Increasingly prone to settlement Likely to erode under wave or current action	50—250
Organic Silts & Clays	Highly prone to settlement and erosion	Low and Variable
Clays	Unconsolidated clays particularly prone to long-term settlement	50—600
Fill Strata	Prone to variability, settlement and erosion	Low and Variable

The marine foundation characteristics for common strata shown in Table 1 are for initial guidance only. Bearing Pressures for submerged granular soils are reduced due to their relative density^{4,5}. Some precast systems may require the geotechnical assessment of relative and overall settlement during the construction period as well as the long term condition. Over-consolidated soils are far less prone to settlement. Soils with inadequate bearing capacity, or weak soil strata that are prone to high settlement, may be strengthened by ground improvement techniques or piling (see section 4.3).

4. Foundation systems and materials

4.1 Pre-levelled bed, stone layer

A stone layer is pre-levelled accurately on the sea bed to allow direct and rapid placement of precast elements. The stone material is usually a crushed quarry rock with a narrow size range to allow water to flow through and avoid small particle loss⁶. The size range should allow it to be readily screeded accurately into place. Common screeding methods are:-

Travelling Screed Hopper

The frame is positioned and levelled under diver control. Aggregates are supplied via tremie tube or lowered in bottom release skips which discharge into the screed hopper by diver operation. The travelling screed hopper is then motor driven along the frame monitored by the diver. The system is generally the most efficient diver operated system and usually produces good bed level control (Greystones). It is now probably the most common system for medium sized projects.



Figure 4—Traveling Screed Hopper (Khor Fakkan)

Screed Beam

Depending upon the size and type of frame system, the screed beam can be operated by hand by the diver for small areas, by long reach excavator bucket blade, by crane lift into place and then a crane or winch operated pulley system to drag the screed beam and level the aggregate (Cardiff). A number of passes is usually required. The systems are slow, diver intensive, but normally produce reasonable bed level control. They are normally used on smaller scale projects with limited repetition.

Stone Tremie Tube

Stone is fed down a large diameter tube whose bottom end is moved in a controlled way over the bed to be created. The bottom of the tube must be kept topped up. The system does not require diver operation and can be useful on all sizes of project. A jack up barge mounted system has recently been used for the Busan Geoje I.T.T., Korea.

Specialist Screeding

The stone bed for the Oresund and 2nd Benelux Tunnels were laid using a specialist ‘Scrader’ stone laying technique⁷. This formed a level bed with frequent drainage trenches to accommodate the lowering exit flow and silt. The system comprises a stone tremie tube with a sliding tube foot (scrade) which self adjusts for level under surface lazer control. The tube is moved along the side of the barge whilst it is held in position with pin piles. Other mechanical and automated systems can also be used. All these systems are normally applicable to reasonably large projects.

The top screeded layer of smaller stone is often laid on a foundation layer of larger stone that can act as a structural distribution layer or as a regulating layer to make up levels after excavation / dredging. This layer can sometimes be compacted by a vibrating plate to reduce final foundation and structure settlement. For good consistent stone layer compaction, the top surface may need to be installed to an overall tolerance less than the amount of surface compaction drop, to avoid areas of low compaction. The screeded layer cannot normally be compacted (and achieve level tolerances). The cost to construct the screeded layer is, often some 2 ½ times more expensive than an uncompacted foundation layer of twice its thickness.

Settlement

Stone aggregate layers are prone to consolidation, which generally occurs during the construction and initial loading period. The settlement occurs principally within the stone layer, sometimes with secondary bedding-in settlement at top and bottom contact surfaces. Usually, shortly after construction is complete, the settlement of uncompacted stone layers is complete and in balance with the load applied. Load increases, seismic or dynamic effects can cause further settlement or deflections and should be checked⁵ (Fig. 5).

Stone bed foundations are often used for Quay Walls (Fig. 2) and Breakwaters (Fig 13 & 29). Breakwaters can often accommodate greater degrees of settlement. Quay walls need much tighter control of settlement and forward rotation, particularly those supporting dockside crane rails. Walls formed of bonded blockwork on a stone bed can have good load distributive properties due to their height and interlock but are less tolerant of differential settlement, which can create loose blocks¹. Stacked column unbonded block walls accommodate initial settlement but are very susceptible to further relative settlement after the block or slab capping is cast. Many block quay walls are preloaded to ultimate design load levels before casting the insitu capping to reduce future settlement. For many structures relative settlement is more important than overall consolidation. Both cases need to include the settlement characteristics of the natural seabed strata⁴. The stone layer thickness can be optimised to distribute and lower the bearing pressures and settlement within the sea bed strata, whilst managing the consolidation within the stone layer thickness. A tremie concrete blinding layer can be used to reduced layer settlement or prevent formation softening or erosion.

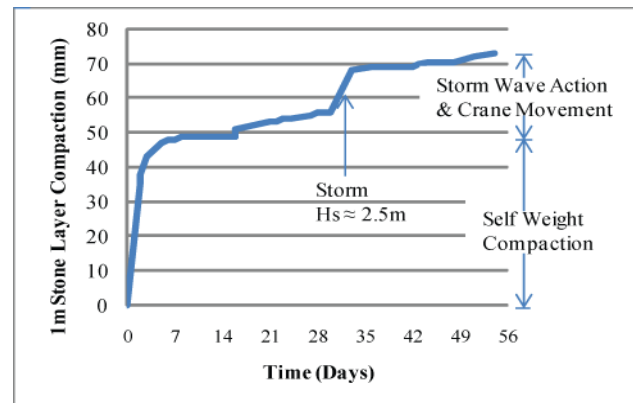


Figure 5 - Typical Settlement, Greystones.

Foundation Material Suitability

The stone for the top screed layer is normally sized to be small enough to allow screeding, but large enough to resist the initial wave and current wash-out effects. The screeded stone layer and the foundation stone layer if used, need to be adequately graded for filter performance, thus preventing migration of fines from the finer layer into the coarser layer⁶. Careful control of bottom placement is required to avoid size segregation when falling through water. Selection of stone for a specific purpose is important, in terms of size, grading, shape and material hardness. For instance, rounded (narrow-graded) gravels can be used to lessen settlement, but may have higher costs and slightly lower parameters for sliding, load distribution and wash out. Early compaction and shear tests are recommended where required on the actual materials proposed. Siltation effects can also be incorporated into sliding tests. Where the seabed comprises sands or silt prone to liquefaction, this can hinder the compaction of a stone layer under vibration plate action. Variation of vibration frequency, number of passes and pre-compaction of the sand can be trialed. The seabed formation may require a smaller stone size filter layer or geotextile separator depending upon the nature of the sea bed. Geotextiles should be weighted or negatively buoyant to aid formation of reliable overlap joints.

Scour Protection

Where current and wave velocities or other accelerated flows (ship propeller wash etc) exceed the stability level of the foundation stone or natural bed strata, scour protection

should be provided^{8,9}. There are many examples of bow thrusters and propeller wash scouring below quay walls. For high currents, propeller races, bow thruster jets and HSS jets, concrete mattress slabs or tremie concrete should be considered along with rock armour as they are often more effective against high flows.

4.2 Base Infill

Base infill systems rely on the foundation element being supported on temporary jacks or initial pad foundations whilst the foundation void is infilled. Infill systems are often irreversible so need suitable planning, preparation and control. These systems are particularly common to bridge piers, caissons, immersed tube tunnels and sometimes blockwork wall foundations. The main systems used are considered to be:-

Tremie Concrete

Tremie concrete is a natural choice for foundations where the following conditions apply:-

- Sheltered Conditions – to avoid wash out during placement
- Relatively Narrow Base Width – to suit tremie concretes restricted fluidity
- Side Confinement – via dredge pockets or similar

For marine work, concrete placement can be undertaken by pumping or a hopper and tremie pipe¹⁰. The hose or tremie end should always be submerged to avoid segregation and wash out. Cement paste washout can commence in low currents. Anti washout additives can improve washout resistance, but in other than still or sheltered conditions, the washout resistance should be verified or tested. During tremie pours it is important to create top contact and keep an ongoing wave or concrete slope moving, to ensure there is no entrapment of water resulting in voids and loss of soffit contact. The mix should be safely retarded well past the filling period and back up plant provided as required. Often there is only one opportunity to pump efficiently a base infill pour and a detailed filling plan with suitable back up arrangements should be made.

Tremie pours are often relatively thick to cater for foundation excavation tolerance, clearance and side fluid head required for travel under the base. This combined with positive construction tolerances needed when forming the length and width of dredge pockets, can lead to relatively large marine concrete pours. Fluidity is normally measured and controlled by a flow table. The arrangement of tremie head and fluidity for the base travel width is best confirmed by full scale testing along with other concrete testing. The system is not suited to continuous foundations without a system of stop end provision.

Concrete infill foundations are naturally compatible with the use of precast elements. Distribution of load from the precast elements can be taken through the insitu concrete to the foundation strata. For base shear design, it is usual to take a safe coefficient of friction relative to the roughness of the precast soffit and ignore variable bond strengths as they can be unreliable and bond may be broken in the construction process. Surface roughness and friction can be increased and combinations of shear upstands or downstands can be created to suit. Shear downstands should be shaped not to trap water.

Open Grouting

Open grouting is similar to tremie concrete apart from its greater fluidity allows it to be used on much wider bases. Grout is more prone to wash out and needs sheltered conditions before its use should be considered. One of the Oresund Bridge piers suffered from storm washout with some 50% loss of contact.

Grouts historically have been formed of neat cement, occasionally with a blend of sand and cement and currently often with substantial replacement by PFA, GBFS or fillers. Grouts are often 2 to 3 times the material cost of concrete. Grout fluidity is normally measured by flow cone. Neat cement grouts are more readily and reliably pumped than concretes or sand : cement mixes. Initially, grout should be discharged onto the bed relatively slowly until the hose end is well submerged to avoid jet mixing and the creation of a light and weak grout. Grouts do travel into open bed materials such as gravels and may lead to a resulting loss of top contact. The marine grouting compartment needs to be reliable and grout tight up to the grouting level. Good filling practice needs to be employed to avoid jet mixing and entrapped water on the top contact surface. Generally, the top contact profile of fluid grout needs to be pushed past future filling positions before filling is transferred. Sea water can normally be used for mixing purposes to unreinforced pours, subject to a ASR check on mixes with aggregate. Sand : cement mixes can be pre-dried and blown into silos. Output of 20m³/hr/plant is common for barge mounted production.

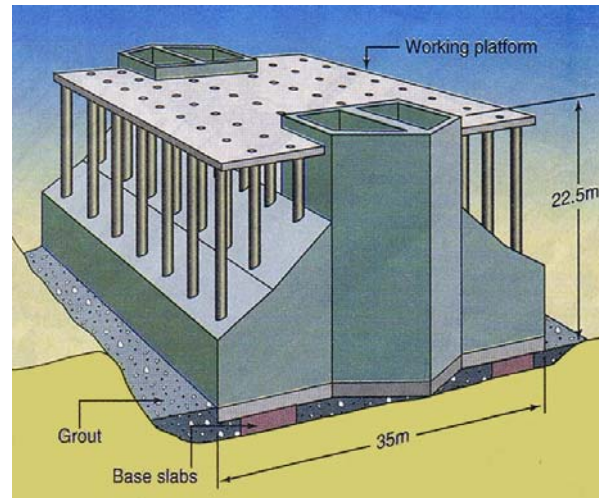


Figure 6 *—Grout Washout, Oresund Bridge Pier

Most design aspects are similar to tremie concrete with the general exception and extension of the following. Neat cement grout strength usually exceeds that of normal concrete. Where strength is not critical, increasing cement replacement or fillers can be used. Where sulphate resistance is required, either sulphate resistant cement can be used or a blend of PFA or GBFS where higher resistance is required. Neat cement grouts typically shrink underwater by approx 1% of their thickness. On sensitive structures, to avoid uneven load distribution, a shrinkage control additive is normally added.

Grouts are prone to thermal cracking and where this would be undesirable, various precautions can be taken¹⁰. Grout mix design and selection of additives should normally undergo a development testing process to ensure the required parameters are met. A pumping and filling trial could also be required.

Grouted Fabric Formwork (Grout Bags)

This system uses purpose made fabric forms to act as grouting compartments and are often known as Grout Bags. They are made from porous fabric which is grout tight yet water permeable and therefore avoids trapped water voids¹¹. The system can be engineered to offer the following:-

- Reliable grouting compartments – Factory made
- Protection against washout
- Control of compartment size
- Control of fluid grout filling and uplift forces
- Risk management via multiple compartments

The forms are normally prefixed to the precast foundation element before immersion or lowering. They are often condensed with side break ties or diver release ties. Form compartments are zipped to one another whilst fixing for large bases. Once the element is lowered and positioned on jacks and/or temporary foundations, the compartments can be pump filled. Compartments are normally filled with neat cement grout or with sand cement grout / micro concrete. Filler sleeves sewn into the bottom dictate where initial top contact is made with a wave of grout travelling to the side vents. The side vents control the compartment pressure which protect against failure of the fabric form and control uplift pressures, which can be important on larger bases. This allows a reduction of the percentage

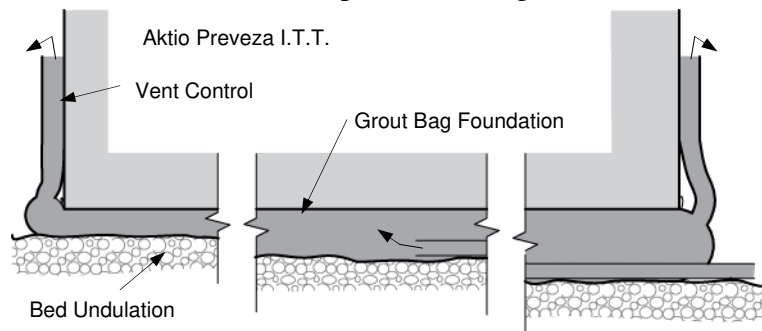


Figure 7—Typical Grouted Fabric Formwork Foundation

of negative buoyancy of the element and reduces the size of temporary support jacks and/or foundations. Formwork systems are designed to cope with foundation excavation tolerances, withstand grouting pressures, control uplift forces and give good base contact and reliability.

The system has been used for foundations on major marine projects for many years. It can typically cope with bed tolerances of ± 150 mm to ± 450 mm. The system can be diver worked or completely automated with prefixed hoses, grout monitors to the vents and automatic releases (PEI). To obtain the benefits of the system, the following processes and costs are encountered:-

- Design and development
- Prototype testing on major projects or for adaptations
- Fabric formwork supply and fitting cost

The forms are made from polypropylene and polyester fabrics generally, with designed, tested and quality controlled seams. Fabrics of a 2 layer format are used for increased resilience where performance is critical. The system has been used for grout travel lengths up to 18 m with current proposals based upon 24m for 48m wide bases (Venice Barrage). The introduction of a fabric layer above and below the foundation grout infill affects the shear interface. The top coefficient of friction is principally dependent upon element soffit roughness. This can be increased by selection of the concrete shuttering roughness. Recent laboratory tests for ply shuttered concrete achieved a coefficient of 0.61. Tests should be made where required to engineer the contact surface. Shear keys can be utilised where required. Base interface friction is normally aided by undulation and bed particle interlock. Where foundation beds are prone to fine particle siltation, this should be checked. Base shear analysis or tests should be conducted where shear is a consideration.

Pumped Sand

This infill system is normally only used on immersed tube tunnels. A 10% to 20% suspension of sand in water is pumped under the element to form a sequence of touching 'pancakes' of sand. The sand drops out of suspension when the flow slows. The system is a specialist technique requiring experience. The sand is partially compacted by pump filling pressures which in turn are managed to avoid uplift of the element. The system is not a foundation system in general use.

Pumped sand infill has widespread use on immersed tube tunnels where the required bearing capacity is normally very low and sheltered trench working conditions prevail. It is not generally suitable to seismic zones because of seismic liquefaction, unless a suitable and pumpable sand grading can be found. Due to the low compaction of the sand foundation layer, short term settlement can be a problem as it may delay the following stages of construction. Pumped sand is unlikely to prove a realistic option for other structures with a greater range of loading, exposure and sensitivity to settlement. However, where conditions are similar to immersed tube tunnels, the system could be considered.

4.3 Weak / Inadequate Strata Piled Foundations

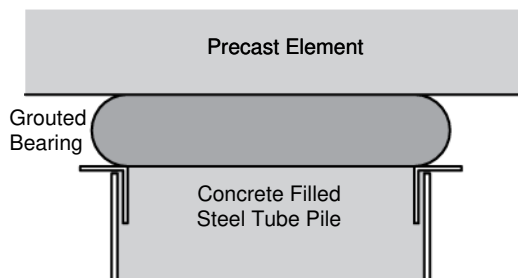


Figure 8—Pile Bearing

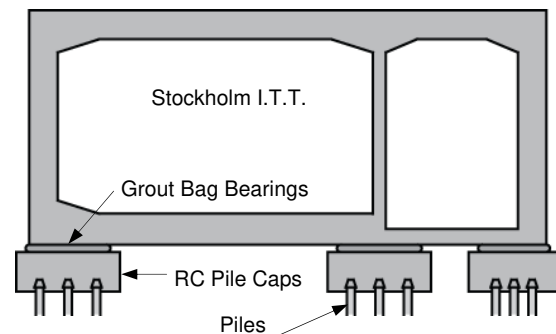


Figure 9—Pile Cap Foundations

Structural piles or shafts can be used to support precast elements although this is not common. Grouted top bearings can be used to large diameter piles. Alternately pile caps can be cast to pile groups and grouted fabric formwork or other bearings cast after the element is positioned. Precast pile cap shells are often used in America to form pile caps around water level.

Maritime piled foundations need to be designed with consideration for marine construction techniques. For high strength bearings, grouts up to 90 N/mm² have been used (Monaco). Due to the lack of structural continuity below the element, sliding resistance may need to be considered. Raking piles, shear key downstands or side bearings may be required.

Ground Improvement

Weak or inadequate soil strata can be improved by various Compaction and Replacement techniques¹⁰. Where a particular design calls for Ground Improvement to reduce high settlement characteristics, bearing failure or to cope with seismic effects, the following methods have been used in the marine environment:-

- Surface Vibration Compaction (Great Belt)
- Surcharging / Preloading (Quay Walls)
- Ground replacement (Quay Walls)
- Vibro Compaction (Aktio)
- Dynamic Compaction (Menard system)
- Sand Drains and Wicks (Pascagoula)
- Grouting (Jet Grouting, Cape Girardeau Bridge)
- Closely Spaced Piles (Soil Pinning) (Rion Antiron)
- Deep Probe Vibratory Compaction (Japan)

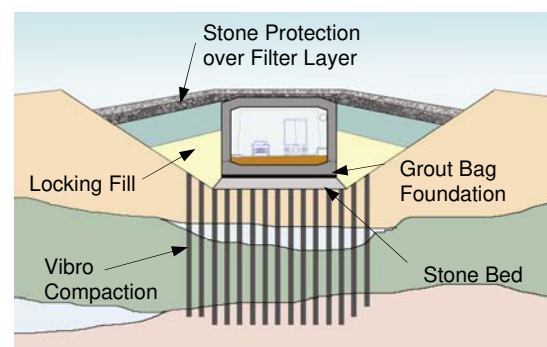


Figure 10*—Aktio-Preveza I.T.T.

Surface vibration compaction, surcharging or pre-loading, and partial replacement can be effective solutions.

Where a bed strata has been consolidated, it is often usual for a capping layer of crushed rock to be laid. A Pre-Levelled Bed or Base Infill system is then used to found the precast element foundation contact. Full site investigation details are needed to allow analysis, design and improvement method selection for a particular soil. Vibro compaction can aid seismic liquefaction resistance of vulnerable soils, by pore water pressure dissipation via the stone columns (Fig. 10). This system, along with a compacted stone capping layer and grouted fabric formwork infill layer, was used to overcome seismic effects at the Aktio-Preveza ITT in Greece. Settlement and seismic liquefaction problems to the Rion Antiron bridge piers were overcome by very large base design of some 90m diameter and ground improvement by soil pinning. Soil pinning is similarly proposed to the Venice Barrage (Fig. 11).

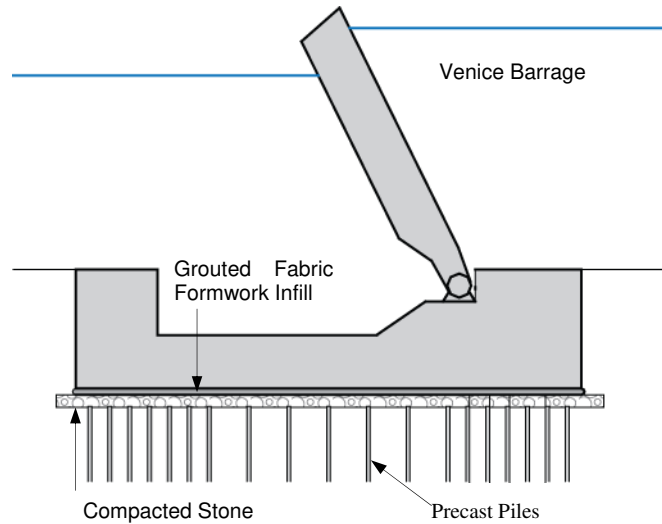


Figure 11—Soil Pinning

Ground improvement cost can sometimes be avoided by appropriate design. For example the improvement of the soft alluvium at the Limerick Tunnel has been avoided by design tolerating settlements up to 100mm.

5. Case studies

5.1 Cardiff Barrage Entrance Harbour, Wales

The Cardiff Bay Barrage is 1.1 km long and extends from Cardiff Docks in the North to Penarth in the South. It has created a freshwater lake with over 13 km of waterfront. To the south end of the barrage a protective harbour of 2 curved breakwaters was constructed to protect the locks.

The breakwater arms were formed by a series of pre-cast closed bottom caissons, each 17m high and weighing up to 4,500t, which were floated into position with buoyancy 'camels'. The dredge trench was filled with graded rock layers: first



Figure 12—Aerial View

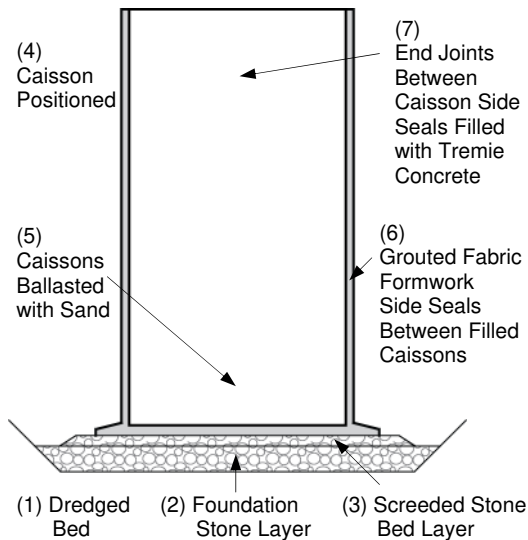


Figure 13 – Construction Sequence

a larger rock foundation layer; then a screeded bedding stone layer to accept the caissons, lowered and ballasted into place (Fig. 13). The caissons were filled with water, then sand, and the end joints between caissons were sealed with grouted fabric forms (Fig. 14). The pairs of seals were located in protective recesses and allowed tremie concrete infilling of the end joints between caissons.



Figure 14 – Side Seals

5.2 Second Severn Crossing, UK

The Second Severn Crossing comprises a 5.2km crossing of the Severn Estuary. The crossing consists of a 0.9km cable stayed bridge with the main span being 0.5km long and gives a clearance of more than 37m over the highest tide level. The viaducts connecting the bridge to the shores are 2.2km and 1.9km long.



Figure 15 – Second Severn Crossing

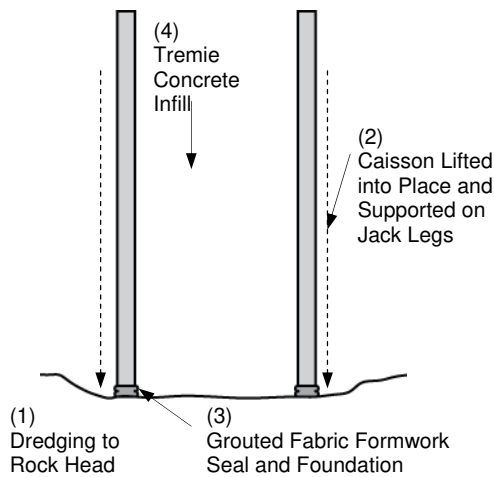


Figure 16 – Construction Sequence



Figure 17— Lowering



Figure 18 – Grout Bag Foundations

each weighing up to 2,000 tonnes. Fabric formwork units were fixed onto the underside of the caissons before they were lifted into position by a jack up barge mounted crane (Fig. 17). They were supported on temporary jack legs in the pockets excavated into the Sandstone / Mudstone bedrock (Fig. 16). The grouted fabric formwork system was used to found the caissons onto the dredged rockhead, and was designed to be part of the bearing area of the foundation (Fig. 18). After grouting the caisson supports were removed and the caisson cells were then tremie filled with mass concrete. The foundation system coped with the extreme tide and working conditions.

Bridge piers were made of 27m to 35m long precast concrete shell caissons,

5.3 Confederation Bridge, PEI, Canada

Confederation Bridge is 13km long and links Prince Edward Island to mainland Canada. The bridge was formed from precast elements including 64 pairs of piers bases and shafts plus cantilever and infill beams. The bridge and precast site were shown earlier in Fig. 1. All precast elements were lifted into place by floating crane.



Figure 19 – Confederation Bridge



Figure 22 – Pier Bases



Figure 20 – Installation Frame



Figure 21 – Initial Pad, Forms Condensed

Weathered and weak mudstone was removed by clam shell dredging. Initial pad foundations (Fig. 21) were installed onto a weak mudstone rock head in water depths up to 33m by an Installation Frame (Fig. 20). It carried three precast pads with condensed fabric forms and was levelled by three hydraulic rams. The Installation frame was fully automated for levelling, grout filling, vent monitoring and removal. Divers were only required to undertake monitoring duties. The hard pads were each formed in 5 compartments to limit risks of local failures, and were filled with a neat cement grout including an anti-shrink additive. Fig 30 shows an initial hard pad trial.

The conical pier bases (Fig. 22) weighing up to 4,000 t were carried by the Svanen floating crane and lowered accurately into place onto the hardpads. The sheltered dredge pockets were mass filled with a plain concrete, via prefixed tremie tubes, to form the foundation. The pier bases remained unfilled. The precast pier shaft was then lifted onto the pier base and sealed with a grouted fabric formwork seal. This allowed the structural joint between the two elements to be grouted with neat cement grout vertically within the protected void. This joint was below water level to accommodate the ice shield cone detail which aids the local break up of winter ice floes.

This large scale, automated and sophisticated construction system was purpose designed. During the second season of element installation, a progress rate of one element per day was regularly achieved, giving a peak bridge construction rate of some 250m per 4 days.

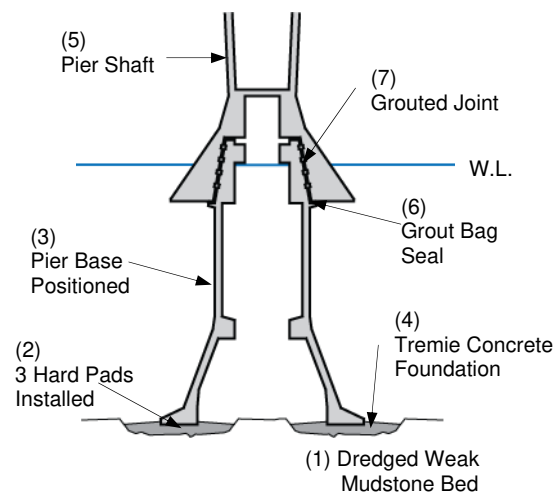


Figure 23 – Construction Sequence

5.4 Central Artery Immersed Tube Tunnel, Boston, USA.

The Central Artery project (Big Dig) in Boston required a road crossing to the Fort Point Channel harbour crossing, the site of the ‘Boston Tea Party’. A precast immersed tube tunnel system was chosen for the first time in America.

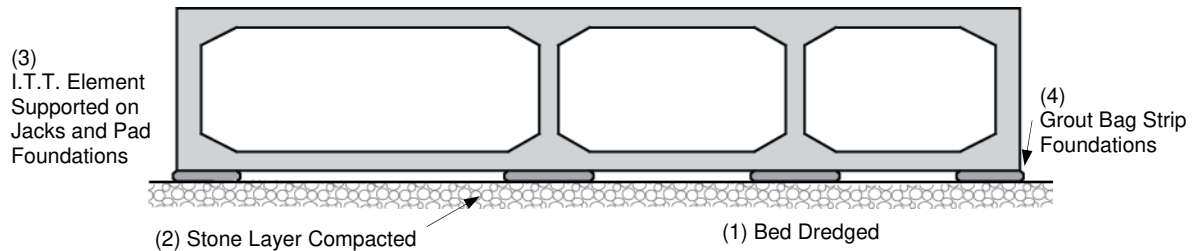


Figure 24—Construction Sequence

Where good ground conditions occurred, the elements were supported on strip foundations (Fig. 24). Fabric forms were filled with neat cement grout whilst the element was supported on jacks and temporary foundations. The condensed form system was pre-fixed to the bottom of the tunnel elements by divers before lowering. The central strip foundations had grout supply and vent hoses prefixed to each grout bag as they were not accessible after lowering. A foundation layer of crushed stone was compacted by vibration plate.



Figure 25 – Grout Bag Trials

To manage the risk of irreversible grouting operations, construction in non accessible central areas, development and reliability testing was undertaken. A 95% contact requirement was achieved with all compartments reported filled.

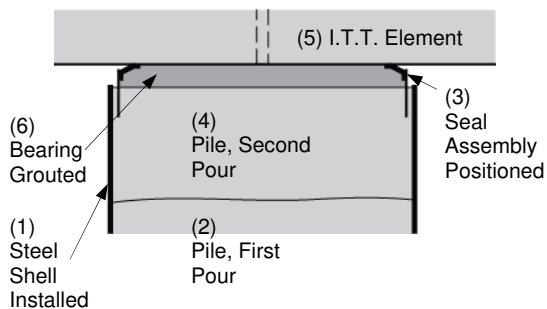


Figure 26 – Pile Bearing Detail

The precast tube elements were supported on piles due to poor ground conditions on one side and to avoid loading the underground train tunnels under of the crossing.

The piles were 1.8m Ø steel shafts with a reinforced concrete infill. The critical bearing detail was formed by diver placement of a rubber gasket seal arrangement (Fig. 26). Once the tunnel element had been positioned, the bearing compartment was grouted. Trial grouting tests were carried out to check for trapped water voids as this would have presented a significant hazard.

5.5 Greystones Harbour, Ireland

The harbour is currently being extended (2008/09) and improved by the construction of extensive breakwaters. The breakwaters are stacked precast block walls with a seaward embankment of stone and Antifer wave protection units. The breakwaters are being built by a build out method with a crane working from the end of the stacked precast blocks for work sequence stages 2 – 7 (Fig. 29). The great reduction in the use of marine plant has potential for significant savings.

The solid concrete blocks are unreinforced, weighing up to 60 t and have side recesses to allow the formation of vertical shear keys (Fig. 28). Blocks are placed onto a screeded stone bedding layer, laid by a travelling screed hopper, on top of an uncompacted stone foundation layer. The stone bed is laid 100mm high to allow for bedding in, stone layer compaction and slight strata settlement. Easterly storms during construction did cause some initial scour to the stone bedding layer and unprotected stacked blocks. Construction has proceeded with some improvements to foundation stone grading, temporary scour protection and shear key concreting sequence.



Figure 27—Build Out System

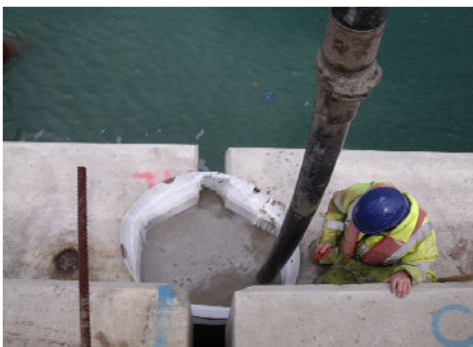


Figure 28—Shear Key

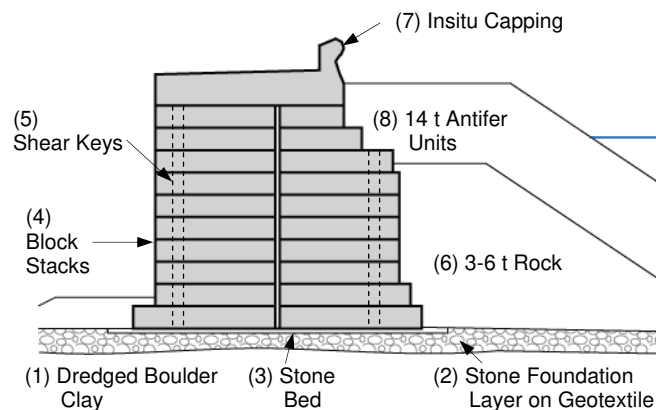


Figure 29—Construction Sequence

Stone layer compaction (Fig. 5) of typically 70mm occurs in some 2 to 3 months before the final lines of shear keys are concreted. This allows management of settlement and differential settlement before the insitu capping slab is formed.

6. Foundation System Selection

6.1. Foundation Influences

The choice of foundation type can be highly influenced by:-

- Diver working conditions, health and safety
- Marine plant cost and availability, construction speed, risk management and cost.
- Sea conditions, currents, waves, depth, tidal range, sediment transportation, visibility, environmental restraints, obstructions, location and draft to the casting basin or yard.
- Structures sensitivity to settlement
- Seabed strata type and profile
- Environmental impact and decommissioning
- Material availability, durability and disposal
- Degree of construction repetition
- Seismic, ice flows or other dynamic action
- Required accuracy of installation

The safety and efficiency of divers is often dependent upon the constructability of the design and the working conditions. Apart from repeat structures, divers with relevant experience should be involved in the foundation system selection, providing diving methodology

advice. Automated construction options should be considered. Where there is sufficient repetition, installation frames, positioning guides and surface control systems can be considered.

Marine Plant

Most modest floating marine plant will have difficulties in operating in significant wave heights of greater than 1m. Jack-up platforms can operate at greater wave heights, but are still vulnerable when being moved. Marine plant when needed is often the most significant cost. The cost of marine plant for dredging, stone laying and craneage can typically range from £2,000—£150,000/day for small to large plant, with very large specialist crane barges up to £500,000/day.

6.2. Relative Merits of Foundation Systems

The case studies show some arrangements and many combinations of foundation construction systems. This, in conjunction with the range of precast element types and possible element arrangements, gives good scope for solutions. Proposed solutions should take advantage of the relative merits of the various systems.

Table 2—General Relative Merits of Foundation Systems.

Foundation System	Advantages	Disadvantages	Typical Structure / Element Types
Pre-levelled Bed Stone Layers	<ul style="list-style-type: none"> Allows rapid placement of elements Quarry material is generally readily available Generally good sliding resistance 	<ul style="list-style-type: none"> Prone to compaction settlement and possible seismic settlement, filter failure, piping or suffusion (migration of fines) May not be suitable for highly loaded foundations Can be prone to wash out of screeded bedding layer in construction and may require permanent edge scour protection Requires use of screeding frame generally Requires larger level tolerance and structure level tolerances 	Bridge Piers Harbour/ Quay Walls Caisson Breakwaters I.T.T.'s Barrages Mast Bases
Base Infill Tremie Concrete	<ul style="list-style-type: none"> Good compressive strength and sliding resistance Cost effective system when conditions allow 	<ul style="list-style-type: none"> Prone to wash out before set Difficult to divide size of pour Fluidity insufficient for wide bases High wastage in dredge pockets Concrete plant/ pumping required Temporary support needed 	Bridge Piers Caisson Breakwaters Harbour/ Quay Walls
Open Grouting	<ul style="list-style-type: none"> Good fluidity for wide bases Good compressive strength and sliding resistance Cost effective in sheltered conditions 	<ul style="list-style-type: none"> Highly prone to washout (<i>Oresund</i>) Possible washout environmental risk Difficult to divide size of pour, control uplift and avoid entrapped water to large pours Permeates open bedding layers Grout provision & pumping required Temporary support needed 	I.T.T.'s Caisson Breakwaters Bridge Piers

Table 2 continued.

	Advantages	Disadvantages	
Base Infill Grouted Fabric Formwork (Grout Bags)	<ul style="list-style-type: none"> • Grout wash out prevented • Good fluidity for wide bases • Compartment size and uplift controlled • Allows engineered risk management (<i>PEI</i>) • Good compressive strength and designed sliding resistance 	<ul style="list-style-type: none"> • Relative cost of the system, protection in transit and lowering required after fixing • Grout provision & pumping required • Often a Specialist system • Temporary support needed 	Bridge Piers I.T.T.'s Barrages Caisson Breakwaters Harbour/ Quay Walls
Pumped Sand	<ul style="list-style-type: none"> • Cost effective • Suitable for wide bases 	<ul style="list-style-type: none"> • Prone to washout • Prone to compaction settlement • Prone to seismic liquefaction • Specialist technique • Cast-in pumping pipes often required 	I.T.T.'s
Weak / Inadequate Strata Piled Foundations	<ul style="list-style-type: none"> • High load capacity • Minimal settlement 	<ul style="list-style-type: none"> • High cost • May require pile cap construction • Grouted bearings required • Shear keys or bearings may be needed 	I.T.T.'s Bridge Piers
Ground Improvement	<ul style="list-style-type: none"> • Useful when the most cost-effective option^{4,10} 	<ul style="list-style-type: none"> • All systems are relatively costly. Surface compaction, surcharging and replacement may be effective. • Other systems tend to be specialist techniques. 	All Structures

6.3. Risk Management.

Risk is generally defined as the probability of a hazard multiplied by its consequence. Risks in maritime construction can be more likely than on land, and the consequences can be greater. Good management of risks is therefore essential. The development of good robust maritime construction systems for foundations has many benefits as problems can be costly. Appropriate risk management techniques are best used to manage risk during the design and construction system selection period. Design and system selection should be integrated as both invariably need to be developed together¹⁰. This requires the early formation of design and construction teams with relevant experience, for projects other than relatively simple or repeat ones. Risks are generally lower where systems have had similar previous use and where project teams have good experience. Similarly, risks are generally higher for teams with less experience and new bespoke solutions. Multiple partner joint venture groups generally attain better risk management due to their sharing of wider experiences. Engagement of Specialists should be considered where required. For Construction system design and selection, a formal risk management procedure could be adopted with at least two members of the team taking responsibility for areas where they have proven relevant expertise and experience. This is particularly important where construction is irreversible, difficult or costly.

On many projects the cost of marine plant is dominant. The cost of the Svanen floating crane at Confederation Bridge was reported to be \$100,000/day in 1996. Accurate assessment of marine plants capability relative to the sea environment climate and work tasks are crucial. Where divers are to undertake important works, video camera monitoring should be used routinely where visibility allows, to aid diver support and improve risk management. On

some projects the design of the construction system and temporary works can be significant. For foundation design, accurate site investigation information is vital, but is difficult and costly to obtain. The risk management or risk / benefit approach should identify key areas of plant, systems or materials that are required to be tested. Some testing of stone, grout or concrete for example may be routine, but key testing may be required to allow the design and development work to proceed. The risk management process should lead to the control and quality control system for the construction process.



Figure 30 – Trial Forms Filled.
P.E.I.

6.4. Future Developments

Precast marine systems are being increasingly used and applied to a greater scale and to more challenging depths. Improvements in the efficiency of forming foundations are expected to continue with the developing use of more automated systems for rock layer placement, precast element placement and grouting or concreting. These improved systems may be increasingly applied, both generally and to the further development of marine gravity structures for energy generation.

Acknowledgements

This paper presents the views of the authors, not necessarily of their employers or clients. The authors are however grateful for wise words and support from their colleagues, particularly Ernest Cannon for this pioneering work on maritime foundations before his retirement at Proserve, Drs Richard Whitehouse and Scott Dunn at HR Wallingford, Bruno Francou of Saipem (formally GTM / Vinci), Otto Thorup of Sisk (formally Christiani & Nielsen), Bill Wilcox of Norfolk Marine and Trevor Thielen of Halcrow. We thank also our clients who have supported this attempt to draw general guidance from their specific examples. Figures marked * courtesy of New Civil Engineer.

References

1. Allsop N.W.H. (2009) "Historical experience of vertical breakwaters in the UK", Proc ICE Conference on Coasts, Marine Structures & Breakwaters, Edinburgh, September 2009, publ. Thomas Telford, London.
2. Tomlinson M.J. (1954) Site Exploration for Maritime and River Works. *I.C.E. Proceedings, Engineering Divisions*; 3(3): pp. 225-255.
3. British Standards Institution. (1988) BS 6349-2:1988. *Maritime Structures: Design of quay walls, jetties and dolphins*. London: BSI.
4. Das B.M. (2007) *Principles of Foundation Engineering*. 6th Edition, Thomson; USA.
5. British Standards Institution (1986) BS 8004:1986. *Foundations—(Formerly CP 2004)*. London: BSI.
6. CIRIA; CUR; CETMEF (2007) *The Rock Manual. The use of rock in hydraulic engineering* (2nd edition) Publication C683, CIRIA, London.
7. Stemat/Boskalis V.O.F. *The Scraeder Concept*. [Online]. Available from: www.stemat.nl/var/filemanagement/files/fleet/stematscradewaybrochure.pdf [Accessed 13th May 2009]
8. Whitehouse R. (1998) *Scour at Marine Structures*, London: Thomas Telford.
9. McConnell K. (1998) *Revetment Systems Against Wave Attack: A Design Manual*, London: Thomas Telford.
10. Gerwick B.C.Jr. (2007) *Construction of Marine and Offshore Structures*, 3rd Edition. Florida: CRC Press.
11. Cannon E.W, Boyes R.G.H. (1987) Permeable Woven Fabric Formwork. *Civil Engineering. March 1987*; pp. 57-60

Giancarlo Passacantando, TECHNITAL S.p.A.:

Could you give any background information or test examples of shear coefficient values between grout bag foundations and caisson soffits that may be useful in shear stability design?

Authors Reply:-

Shear between foundation grout bags and precast elements is largely dependent upon the surface roughness of the element against which the grout bag is cast. Shear test results shown in Table 3 demonstrate this.

Table 3 Kinetic Coefficient of Friction (courtesy Proserve)		
Precast Element Surface	Grout Bag Material	
	2 Layer (with 5% interweave)	1 Layer
Rough Concrete (sand blasted, rough sawn boarding to exposed aggregate finish)	0.51 to 0.88	
Concrete (planed and coated timber boarding)	0.54	0.58
Smooth Concrete (steel shuttering)	0.43	0.48
Steel rusty	0.46	
Steel smooth	0.43	
Bed Material		
Crushed Rock (50mm down)	0.60	
Sand (Fine / medium sand)	0.51	
Rounded Gravel (10mm)	0.4	

Notes:

1. Results for neat cement grout. Results for sand : cement micro concrete $> 35\text{N/mm}^2$ are expected to be similar.
2. Grout bag fabric is 1100 desitex polypropylene which has long-term stability in contact with concrete (unlike polyester which suffers from pH degradation).
3. 2 layer fabrics are usually used for neat cement grouts, for additional grout tightness security and robustness.

The above is provided for initial guidance only, as differing project materials and conditions may give rise to significant variance and Engineers should arrange for appropriate testing. Testing should be conducted with appropriate levels of bearing stress.

The kinetic or sliding coefficient of friction should be used in design to ensure safe solutions with sometimes slightly higher static coefficients being ignored as they can be prone to progressive release and other construction issues. Fig 33 shows that sliding restraint can be engineered by the selection of suitable surface roughness and materials.

Where a higher degree of sliding restraint is required, shear key profiles can be adopted as shown in Fig 34 (precast element removed).



Fig 34, Grout Bag Shear Keys

The degree of bearing contact on the sloping bearing surfaces is normally established by test and the shear keys designed structurally to give the completed sliding resistance required.

For sliding restraint against bed materials, some test coefficients are shown in Table 3, the coefficient of friction for larger crushed and more open stone would undoubtedly be higher than for the 50mm down crushed rock. However, great care should be exercised in assessing the risk of siltation onto the bedding layer and the resultant lowering of the effective friction coefficient. Marine growth and other effects may also need to be considered. The effect of bed undulations is usually considered to be small for normal construction tolerances. Again in appropriate cases, suitable shear tests should be carried out.