Investigation of the failure of permanent anchorages adjacent to a tidal surge barrier

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Abstract
To provide protection against flooding of densely populated areas, a tidal barrier and anchored sheet pile walls were constructed as part of overall flood protection scheme across a tidal estuary in England. The piled structures, completed in 1979, were supported by thirty four, 48m long permanent ground anchorages installed below the capping beam at an angle of 40 degrees to the horizontal.

In September 1991 a 25m length of the pile wall was subject to a lateral deflection of up to 450mm at the pile head. The paper describes the visual examination of the wall, exhumation behind the wall to anchorage level and inspection of the exposed anchorage components. These included failure of the anchor tendon and pull out of the tendon from the anchor head. In addition there were indications of the use of inappropriate construction techniques employed at the time of installation.

Given anchored structures of similar type and age exist at many UK locations, there is concern that similar standards of construction and design specification may have been adopted elsewhere. By highlighting the features faced at this site, the paper gives an insight into the issues that could potentially be presented in other structures and provides recommendations on forward strategies that could be adopted to prevent failures in the future.

Introduction
The vulnerability of the east coast of England and its estuaries to flooding has been an age old concern. During the mid to late 1970s tidal barriers were constructed on the Thames and on estuaries to the north of England. Downstream of these severe tide controlled dams were the river bank raising schemes, many of which used ground anchored steel piles as the new barriers to prevent tidal overtopping. The barrier in question was completed in 1979 and involved some 34 ground anchorages retaining new sheet pile wing walls adjacent to the barrier and for 50m downstream (Figure 1).
Figure 1 Elevation of anchored sheetpile wall located adjacent to tidal surge barrier

Figure 2 South-east pile wall showing deflection immediately after failure

Figure 3 Cross-section through South-east pile wall
Structure and movement
In September 1991 it was noted that an approximately 25m length of the south-eastern sheet pile wall had undergone a sudden lateral deflection of up to 450mm (Figure 2). The cross-section through the south east wall is shown in Figure 3, comprising a line of Larssen 4B sheet piles with a concrete capping beam. The piles were generally located some 2m in front of the former river wall which comprises a mass concrete or masonry structure with timber fenders. The gap between the old and new walls was stage backfilled with gravel. To support the sheet piles, permanent ground anchorages were installed, comprising 48m long anchorages installed at an angle of 40° to the horizontal. Each anchorage was formed of two Dyform prestressing strands (15.2mm diameter, 300kN ultimate tensile capacity) fixed to the river side of the sheet piles by an anchor head and gusset plates and the fixed length founded by grouting into a dense sand stratum.

Initial Inspection
Excavation of the retained soil behind the wall relieved the wall loading and exposed the ground anchor tendons (Figure 4). In the area of the failure a small number of the anchor tendons could be seen to have broken whereas the majority appeared to have pulled through the restraining anchor heads located on the river side of the sheet piles.

Figure 4 Exposed anchor tendons after excavation to relieve load on piled wall
Construction and Construction Techniques

Sheet Piling
The south-eastern sheet pile wall was installed a short distance (1-2m) in front of the existing timber and concrete river wall. Due to a number of problems which would appear to have included the presence of the old wall, a setting out error and over correction for the error, the resulting line of the sheet piles was not straight. The magnitude of the mis-alignment is not known but it may be assumed to have been significant.

Anchorage installation, backfilling and stressing
The sequence of events after the anchorages were installed is of particular importance to the subsequent failure. Following anchorage installation, the anchorages were stressed and locked off at low loads of approximately 1 tonne per strand. This would have been done to ensure that the tendons were tight and straight and providing some support to the wall during backfilling. In an effort to improve the alignment of the sheet piles at the northern end of the wall a number of anchors were loaded to 20 tonnes. This apparently gave a noticeable improvement in the wall alignment. Backfilling was then carried out up to the level of the waling.

The backfilling appeared to have been undertaken by simply end tipping from trucks or by dropping the fill in with a crane mounted grab. This operation caused a significant amount of damage to the tendon sheaths where they bridged the gap between the old and the new walls. Repairs to the sheaths were made by wrapping either the damaged sheaths or the exposed strand itself in grease impregnated tape. There was evidence of the presence of strand couplers fitted between the sheet pile and the rear wall indicating either strand damage or installation of strands too short to allow stressing. These were also protected only by grease impregnated tape.

When approaching the completion of backfilling it was decided that further adjustments would be made to the wall alignment by altering some of the anchorage loads to 10 tonnes per anchorage compared to the design load of 34 tonnes.

Corrosion protection provided and standards at time of construction
At the time the ground anchorages at this site were installed there was no British Standard relating to their design, construction or testing. BS8081:1989 was eventually published in 1989 which was first issued as a draft for development DD81 in 1982. More recently the Euro Norm EN 1537:2000 was published as an execution code to provide guidance on aspects relating to the construction of anchorages. There was, however, a significant amount of published information available at the time (notably the GLC Specification for anchor protection on the Thames Bank Raising Scheme) which recommended quite clearly types and extent of corrosion protection to be provided although this was not embodied in this Contract Specification.
Although the specification called for ground anchorages to be ‘sufficiently protected against corrosion to provide a working life of not less than 100 years, no reference was made in the specification to concepts such as single or double protection. The tender drawings showed bar anchorages with no specific corrosion protection measures to the anchor head.

Prior to the anchorage installation it was known that the anchorage contractor offered additional corrosion protection to the Client. This comprised a steel, bitumen coated, grease filled cap to cover the anchor wedges and exposed strand (Figure 6). This offer was not taken up, primarily on the grounds that it did not appear necessary to provide a greater degree of protection to that given to the ends of the anchor tie bars. It is unfortunate that the implications of this decision were not realised. This type of protection was subsequently developed into glass reinforced plastic caps as shown in Figure 7.

Figure 6 Grease filled bitumen coated steel caps at Greenock, 1981.  
Figure 7 Grease filled glass reinforced plastic caps at Lowestoft, 1987.

Overall examination
Examination of the failure comprised a visual inspection of the failed anchorages and strands, as well as the remaining intact anchorages, together with a more detailed examination of the recovered portions of some of the failed anchorages.

It was clear from the outset that the failure of the sheet pile wall was a result of a failure of the ground anchorages by one of three mechanisms:

- Failure of the anchor strands
- Pull out of the strand from the anchor head
- Pull out of strand from an in-situ free length coupler
The wedges and strand at the anchor head were completely exposed to the environment, with no covering or protective cap as shown in Figure 4. The corrosion protection applied to the anchor heads at the time of construction, in the form of bitumen coating, had long since cracked and peeled off.

The examination showed that the majority of tendon strands had pulled through the locking wedges (Figure 8). The number of strands that had actually failed in tension was limited with only four, of a total of 22 that had been affected.

Of the anchorages that had pulled through the locking wedges, a limited number of locking wedges and barrels were recovered, including two retrieved from the mud below the wall. The recovered wedges showed corrosion of the outer surface underneath the remains of the bitumen paint. The inner serrated surfaces of the wedges were also corroded. The wedges were generally corrosion “welded” into the barrel and virtually immovable. The strands that had pulled out were particularly heavily corroded on the short section that had been exposed outside the wedges. The degree of corrosion and the volume of corrosion products was such that these exposed sections of the wires, above the wedges, were forced apart as individual wires (Figure 9). Where the locking wedges had gripped the strand could be determined by an area of lesser corrosion, but still exhibiting a clearly reduced section of the strand.

Below the wedges the protective polypropylene sheath had been cut back over a short length and leaving some strands unnecessarily exposed. This had been done to allow a considerable amount of strand extension during stressing. Such “extension” would accommodate true free length extension and also pull-in of some out-of-line piles. This length of tendon was generally slightly corroded, but to a degree much less than the
length exposed outside the wedges probably a result of some of the initial grease remaining as a coating.

There were also two anchorage strands whose condition and location within the backfill suggested that they had been exposed for a considerable length of time. One strand was permanently bent into a curve over its exposed length. The section of strand that had been exposed by stripping back the sheath was relatively long, about 0.5m, and was uniformly more corroded that most other strands inspected.

The second strand had pulled back in behind the sheet piles and had pushed downwards so that it was resting against the rear of the sheet pile pointing vertically upwards. The length over which the strand had been cut back was very short, only about 150mm, and the exposed section was quite heavily corroded. The strand must have pulled back and pushed downwards by the backfill. It is believed that these strands were remains of the original sacrificial test anchorage.

An examination of the exposed anchor heads and tendons on the anchorages on the north-west wing wall showed them to be more heavily corroded than those on the south-east wall where failure had occurred. The individual wire segments were in general almost completely corroded through and could easily be broken off by hand above the wedges (Figure 9). The greater degree of corrosion was probably due to these anchorages being located at a slightly lower level within the splash zone. Below the anchor head plate could be seen the underhead protection in the form of an epoxy resin. Of the strand examined most showed that the polypropylene sheath had extended up into this resin. However, a number of the sheaths ended below the resin.

**Detailed Examination**

A number of the affected anchor tendons and components were taken for detailed laboratory examination. One of the most important aspects shown by the detailed examination was the lack of wedge serration markings on the strand that had pulled out of the wedges. Strand subjected to high anchor forces had quite distinct serration marks where the teeth had gripped the tendon. Conversely those that had pulled out exhibited shallow and indistinct wedge serration markings. These strands also showed a greater degree of corrosion at the location of the wedges.

One particular strand showed a series of teeth marks over an approximately 250mm length. At the lower end these were quite deep and distinct. Moving up the strand there were further teeth marks but becoming progressively less distinct and shallower. The location that the wedge block had finally taken up was shown by the corrosion thinning immediately above and below. Within the area that the wedge block had been located, for what was obviously a long period of time, the wedge marks were almost non-existent. These same features were noted on several of the recovered strands which had pulled out of the anchor wedges.
Clearly the anchorages had been stressed to proof load before being locked off at lower loads. The application of the proof load resulted in deep teeth marks, the lower lock off resulting in virtually indistinct teeth marks, particularly after loss of section.

It is known from the construction records that although the anchorages had a design load of 34 tonnes, many were locked off at 10 tonnes load. The calculated elastic extension of the strand between these two loads almost exactly matched the distances between proof wedge marks and final wedge lock off wedge marks.

**Summary and Recommendations**

The construction techniques were extremely poor when considering the detrimental effect they had on the anchorages and the anchor corrosion protection. Realignment of the sheet pile utilising a variety of anchor loads resulted in misalignment of the anchor tendon from the original bore line and its direct contact with the sheet pile sheet and the rear wall. This was damaging to the strand sheathing.

The pouring of backfill behind the piles directly onto the strand caused extensive damage to the sheath, and where observed, necessitated repairs. The sheath repairs were poorly executed (grease impregnated tape only) and yet even afterwards direct backfilling over unprotected strands proceeded. It should be born in mind that the coupler would move extensively during testing and such movement, without coupler protection damage, must be accommodated.
The vulnerability of a single sheath around each individual strand free length is now generally acknowledged and double plastic sheathing is now frequently recommended. However even this system would not tolerate direct tipping of fill without damage.

Evidence suggests that on some anchorages the under head resin grout protection system was incomplete: the free length of strand did not extend fully into the resin thereby leaving a very vulnerable short length of strand below the resin. When applying under head protection and loading/unloading and strand tendons, the fact that nominal frictional forces may move the sheath up and down, must be considered. Furthermore the original cut-off length of sheath must be correct. In such circumstances the under head protection on a number of anchorages should be removed after setting, and broken out to confirm its suitability for its intended long term purpose.

The provision of bitumen coatings as permanent overhead protection is unsuitable for ground anchorages in any environment. Its effective lifespan on the anchor heads in the marine environment was probably a matter of months.

An example of the successful use of the proposed grease-filled capping system when applied in a similar environment is illustrated in Figure 7.

The principle of the taper wedge in the taper hole gripping the prestressing strand is that the wedge serrations indent in to the strand steel and the wedges slide further into the tapered hole. The sliding movement in turn increases the lateral force on the strands and the serrations indent deeper. However, the system only works efficiently if there is a lubricant between the wedge and hole interface.

Observation of the state of the wedges, the taper holes and the strand were consistent with those reported by Barley (1997) whilst analysing contributory factors to a major anchored wall collapse on the Thames in 1990. In exposed conditions the wedges become “corrosion bonded” within the taper hole; the strand corrodes within the wedges resulting in a reduced section. If the wedges could move freely they would advance into the taper hole and grip the reduced strand section.

However when wedge movement is not possible then the reduced section strands pull through the wedges leaving an empty hole (Figure 4). The anchorage then fails shedding increased load onto adjacent strands and then anchorages. Thus failure of unprotected anchor heads and the damage of strands behind the sheet pile occurred progressively in such a manner that total structural failure was avoided. In addition to the omission of outer protection of the anchorages, the poor construction techniques and failure to appreciate the necessary precautionary measures to prevent tendon damage, contributed to the short twelve year lifespan of anchorages.

All the in situ anchorages were subsequently replaced by new double corrosion protected anchorages complying with BS 8081:1989 requirements.
With regard to the future mechanism associated with anchorages on this contract, three matters should always be addressed:

- Any structural movement that occurs during anchorage stressing should be measured in terms of the effect of change of alignment of the drilled anchorages and potential damage to the corrosion protection.
- Complete anchor head protection is difficult to achieve in a marine environment yet without its guaranteed integrity the lifespan of anchorages may be relatively short.
- Anchor head protection performance should be checked periodically (Littlejohn and Mothersille, 2007).

References