

# The failure of a twenty-one year old anchored sheet pile quay wall on the Thames

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## 1.0 INTRODUCTION

Early in the week beginning 26 Feb 1990, an employee working on the quay, noticed a crack in the concrete paving behind the sheet pile quay wall and in front of the large service duct running along the quay edge. Two days later engineers visited the site and, by this time, a 10m length of sheet piling had moved away from the service duct, by up to 2 metres. A day later, the sheet piles complete with heavy R.C. capping beam and outer crane rail had fallen further away from the quay leaving a gap in between the piles and the quay behind (Photo 1). Delivery of a pump was arranged to dewater behind the piles after high tide to reduce the quantity of water retained which was itself accelerating failure. A slight tear appeared in the top of the sheet piles at the final downstream limit of failure.

Arrangements were made to bring heavy chains to the site to provide external restraint to the sheet pile wall and prevent the failure progressing. This was particularly critical in the upstream direction towards the location of the existing crane. Excessive settlement was observed over a large area of the concrete paving (up to approx 500mm) extending some 15m "in shore" from the sheet pile wall settlement and a further 30 m in an upstream direction from the initial wall failure zone.



**Photo 1: Wall Failure after 6 days (right)**

**Photo 2: Overall view of wall failure after about 2 weeks (left)**

The situation was monitored over the weekend and the first failure of the 1.5 m square R.C. service duct was observed. Failure was by a combination of vertical settlement and outward collapse into the void formed behind the sheet pile wall.

Two sets of chains and tie rods were provided the following week and fixed from the two bollards in the failed capping beam to two exposed pile caps some 30m behind the quay. Plant was then mobilised to drive a line of sheet piles normal to the quay in an effort to prevent the failure progressing closer to the crane. (Photo 2).

Investigation to establish factors contributing towards the wall failure commenced by removal of a number of complete anchor head units from the sheet pile in-pans in the area where the collapse had started. (Photo 3 and 4).

Access to the outside of the wall to inspect all exposed anchor heads was provided by the contractors vessel whilst during the low tide period safe access to the inner void allowed a detailed study of all exposed anchors, the inner steel work, and the R C components.

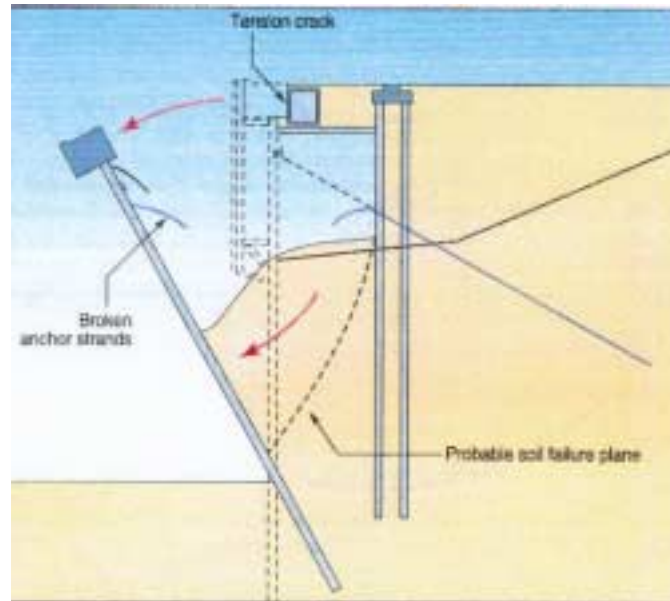


**Photo 3: Severed anchor tendons after wall failure**

**Photo 4: Anchor heads removed from the wall for inspection.**

## **2.0 THE STRUCTURE AND CONSTRUCTION TECHNIQUES**

The structure was designed to provide a 15 m deep shipping berth complete with crane rail and service duct, and is consistent with those commonly utilised (Fig 1). The 142m length of sheet piles, complete with heavy R C capping beam, provided the quay edge and supported the outer crane rail; whilst pairs of driven "H" piles, complete with R C beam, supported the inner crane rail. In between was located the rectangular R C service duct, and a thin R C slab covered the entire quay area. Lateral restraint was provided by a combination of a row of ground anchors installed 3 m below quay level, the toe fixity of the driven sheet piles, the pairs of driven H piles and the general stiffness of the composite structure.



**Fig 1: Original anchored sheet pile wall with supported crane rails and service duct. Failure mode portrayed.**

Construction commenced in Autumn 1968 and was completed by Autumn 1969. The original site consisted of a paved sloping river embankment, and the new sheet pile line was driven some 25 m out from the original crest, with an upstream and downstream return.

The returns were restrained by use of tie rods and anchor beams positioned in the fill, and the corners by use of diagonal ties. At a later date the quay wall line was extended downstream so, at the time of collapse, only the upstream return was exposed. It was in fact the buried downstream return which appeared to restrict failure from progressing downstream.

The basic construction technique adopted involved:-

- 1) Progressive back filling from the embankment crest towards the line of the new quay.
- 2) Driving pairs of H piles for the inner crane rail support. (These would provide some retention of the backfill).
- 3) Driving sheet piles.
- 4) Positioning horizontal RSJ beams at 3m centres, spanning between outer H pile and the sheet piles, seated on brackets and welded to steelwork. The service duct was eventually seated on these beams and the underside fill, but during construction these beams stiffened the composite structure, to allow high level backfilling prior to anchor construction.
- 5) Fixing of twin R C waling behind sheet piles immediately above anchor level.
- 6) Backfilling to waling level (RSJ beams provided sheet pile restraint until anchors stressed).
- 7) Constructing and Stressing anchors including testing of 5% to 1.5 x working load.
- 8) Finishing backfill, constructing service duct, R C beams and crane rails etc.
- 9) Dredging to 6.8 m below original bed level with final exposed face of sheet piles 15 m high above dredge.

### **3.0 HISTORY OF GROUND ANCHORS AND ASSOCIATED CORROSION PROTECTION SYSTEMS IN THE UNITED KINGDOM 1965 - 1970**

**(Refs 1, 2, 3, 4, 5)**

Although ground or rock anchors have been used in a few engineering projects for over half a century, it is only in the last two decades that adequate demand, resources, and expertise have been brought to bear on the complex problems involved to allow the development of the art to its present sophisticated state. The first British Standards Publication regarding Ground Anchors did not arrive until 1982. Therefore, in order to arrive at a reasonable understanding of the rationale behind the planning and execution of this anchor project over twenty five years ago, it is useful to take a brief look at the history of the art of ground anchoring in the United Kingdom during the preceding period.

While rock bolting and simple forms of rock anchoring had been used in the United Kingdom prior to the 1960's, the use of anchoring techniques in the form of Ground Anchors, founded in soils, commenced only in the mid-60's period. Initial usage followed practices developed in Continental Europe, particularly in Germany, where pressure grouting techniques in granular materials had been developed. Associated Tunnelling Company became the United Kingdom licensees of the German Bauer System. This system had been investigated and proven in the early sixties, and allowed economic production of soil anchors utilising a "lost bit" method. The drill bit was knocked off the end of the drill rod on completion of drilling by the advancement of the bar tendon bond down the centre of the rod. In some cases, to aid the tendon bond, the bar was threaded into the bit prior to it being displaced from the drill rod. This system was particularly suited to short anchors of length up to 40 ft (12 m). For lengths greater than this, bulky couplers were required and their installation down the inside of the drill rod became impracticable.

During the middle and the late 60s, UK companies began developing their own techniques; notably Cementation, Soil Mechanics, Universal Anchorage Company and Hughes Company. Anchor capacities in general soils were enhanced by the use of such systems as "chemical injection" and "post grouting", and in clays by "gravel placement" and "underreaming".

During the early stages of development of anchor systems, the available tendons consisted of a single bar (up to 1" diameter in High Yield Steel), multibar, multistrand (a multiple of 0.5" or 0.6" diameter prestressing strands) or multiwire (a multiple of 0.25" diameter prestressing wires). Attitudes to corrosion protection were, in the initial days, consistent with the zero protection applied to the tie bar systems which anchors were replacing. (Tie bars have, even in recent years, still been installed without any corrosion protection). However, during the sixties, a simple corrosion protection system for bar tendons was developed; the tendon would be surrounded by a PVC sheath or densotape in the free length to serve the dual purpose of protecting the steel from the corrosive elements and debonding the tendons from the surrounding grout; the fixed length protection was provided solely by the cement grout. As multistrand anchors were introduced for permanent works, their protection development followed similar lines, but was also influenced to some extent by the corrosion protection afforded by cement grout around the strand tendons as recommended in the pre-stressed concrete industry. Certainly, anchor specifications, even in the early 1970's, required application of primary grouting of the fixed length and a secondary grouting system for the free length, as generated from that industry.

There are indications that research resulted in the installation of some specially protected permanent anchors at Vauxhall Bridge, London, as early as autumn 1968. The tendons consisted of multistrands incorporating individual PVC sheathing containing a grease filling around each strand within the anchor free length. The manufacturers report that such factory applied coatings were not available on the market until late 1969. This would certainly indicate that the use of this superior form of corrosion protection had not become a state of the art system until this date.

Multiwire systems were also available for anchor work, and may have been the first tendons available with a plastic protective coating (approx 1968). However, since the coating was bonded directly to the wire, considerable frictional problems in the coated free length were encountered and this system did not develop to such an extent as to eliminate this problem.

Development of a system to provide corrosion protection to the fixed anchor length which would supplement that provided by the cementitious environment also commenced in 1968. However, it was not until the mid to late 1970's that such supplementary protection became generally adopted as a requirement in the industry.

Research into corrosion protection systems used at the anchor head pre-1970 has revealed little in the way of protective media other than bitumen or epoxy paints. Bar anchors installed in 1968 along the riverside in Bath were coated with the same bitumen paint as applied to the piles (in two coats). Since tie rod end nuts were rarely even painted, it is probable that such paint coating was considered more than adequate. With regard to strand anchors, the first reference to anchorhead protection, was in the form of a steel anchor cap filled with grease, but this recommendation was not published until 1970 (Ref 1).

#### **4.0 THE ANCHOR AND CORROSION PROTECTION SYSTEM**

In total 79 anchors were installed through the sheet pile in-pans at 1.68m centres. Each carried a working load of 500kN at 30° inclination. The 17m long anchors were constructed using end of casing pressure grouting techniques and were founded in Thames Ballast (sand and gravel). The tendon generally consisted of 5 No. 0.6" normal seven wire prestressing strand as supplied on coils. Tendons were fabricated on site and, to enhance the tendon bond within the fixed length, the wires were unwound and rewound to form "bushes". The protection against corrosion of these bushes in the fixed length would be entirely provided by the cement grout injected in the surrounding ballast.

In acknowledgement that the free length tendon was vulnerable to corrosion, a 60mm diameter corrugated plastic duct was threaded over the strands for a 9m length prior to installation (Photo 5).

It is likely that the grouting of both the fixed length and the free length, both inside and outside the corrugated duct, was executed as a single continuous grouting operation during casing withdrawal. However, it is inevitable that a considerable amount of grout would have permeated out from the free length bore into the fill in many cases leaving only the grouted duct around the strands. This would in fact have been beneficial during stressing to reduce the amount of load transferred by bond to this soil in the intended "free" length.

The free length duct by intent extended into a recess behind the anchor head unit, which itself was weld-sealed to the sheet piles. The sealed recess, and any fall in level of grout in the plastic duct, was to be grout filled on completion of stressing via a central hole in the anchor head plate.

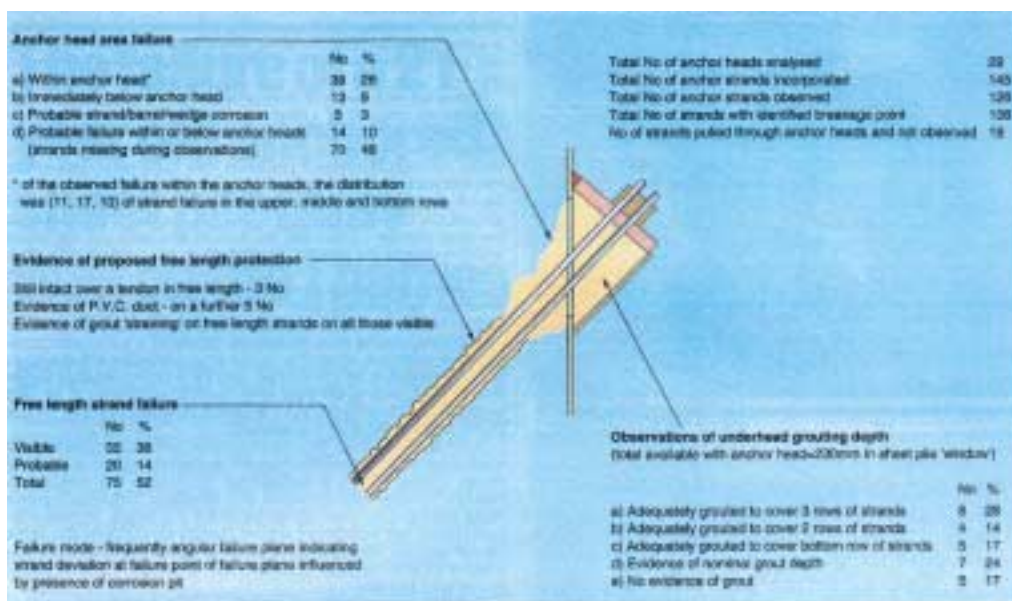
The outer protection to the anchor, including the exposed strand ends and the prestressing barrels/wedges, was by the application of three coats of epoxy resin on completion of the anchor work.

(It is interesting to note that the "going" rate at the time for such anchors was £77 14s 0d, and the drilling rig costs were £12 5s 0d per hour).

Although in comparison with the modern refinement of corrosion protection systems to eliminate corrosion of anchor components, the above system appears extremely primitive, it must be acknowledged that there was clear intent to protect the vulnerable areas. In comparison with current tie bar corrosion protection, the system was good, and its development of the use of grout as a protective medium from the prestressing industry is apparent.



**Photo 5: Strands encapsulated in grouted duct for free length protection.**



**Fig 2: Section through anchor head and summary of location of strand failures (weakest link).**

Furthermore, the utilisation of five strands with ultimate capacity of 1124kN to provide a working load of 550kN and hence a factor of safety of 2.25, was extremely conservative in relation to factors utilised in the prestressing industry at that time (1.25 to 1.33).

'Quality Control' would certainly have been a foreign term in the construction industry at that time, but there are records to confirm that the Main Contractor demanded the anchor specialist to excavate and expose the free length underhead junction (i.e. underhead grouting) at two separate locations. Reports stated that the presence of grout was extensive, and immediately behind the pile much volume of fill above and below the anchor line could not be penetrated or removed. One observation of "splitting" of the plastic duct, where it was visible, "3 to 4 feet" below the anchor head was recorded, and for this reason the MC demanded a regrouting of all anchor heads prior to acceptance.

**5.0 SUMMARY OF OBSERVATION OF THE ANCHORS AND ANCHOR CORROSION PROTECTION SYSTEM AFTER WALL COLLAPSE**

Detailed inspection of the anchors and other structural components was not possible until 6 weeks after initiation of failure. Investigation continued for several months after that visit and various anchor components were taken from site for testing and very detailed inspection. The actual results of these tests, the studies of corrosion of the strands and the close observation of the protection system are covered in a separate report (Ref 6).

This paper summarises the state of the anchor components and protective layers visible on 29 anchors after wall collapse. Clearly the collapse was progressive and, after initial failure of one or two anchors, the adjacent anchors would be overloaded. Thus, the location of failure of the strands in the majority of cases identified the "weakest link", but not necessarily the inadequacy of that anchor to provide the designated working load at the time of failure.

Fig 3 presents a plan view of the locations of the anchors and the wall both prior to and after failure. In addition to the 29 wall anchors, the two bollards were each restrained by a pair of anchors which also failed as collapse progressed.

Observations identified serious shortcomings in the corrosion protection after 22 years of "service", although in some areas it was apparent that underhead grouting had never attained the intended void filling, nor had the free length duct diameter been compatible with the strand distribution required in the anchor head plate. A statistical summary of the locations of strand failures, and of the effectiveness of the underhead grouting, and the state of the prestressing barrels/wedges is presented in Fig 2.



**Fig 3: Plan view of anchors before and after wall failure.**

## 6.0 STABILITY ANALYSIS AND CONCLUSION

An extensive study was carried out into the overall stability of the anchored wall, into the influence of the construction technique, and the potential wall loading conditions with the following conclusions.

Although calculations in accordance with BS 8081 were indicative that the overall stability of the anchored structure could be placed in question, due to the short length of the anchors (17m) in relation to free face wall height of 15.6m, there could be no doubt that no such failure mode characteristics were exhibited either during or after failure. The restraint provided by the pairs of bearing piles installed to support the rear crane rail may have contributed greatly to maintaining the overall stability. Observations were clearly indicative that the wall failed by overturning as a result of overloading of the anchor components, or of anchor component failure.

There were indications that the original calculations based on the relevant BS document for Earth Retaining Structures (CP 2) were satisfactory in establishing the required waling load to be provided by the ground anchors. However, analysis of potential mechanisms and external loading conditions which could have resulted in increase in the required anchor load during the life of the structure could not be ignored, viz:-

- i) the dredge depth appeared to have been increased by 6.8m during the construction period but after the anchors were originally loaded;
- ii) temporary tie beams installed below the service duct during construction could have failed due to weld joint corrosion, or due to overloading as a consequence of backfill settlement and subsequent service duct loading;
- iii) failure or partial failure of the flap valves and/or limitation in free drainage of the backfill material;
- iv) overloading of the quay due to storage of scrap metal (which had been observed);
- v) slab settlement due to iv) resulting in excessive local drainage of surface water. (High rainfall run-off being coincident with extreme tidal range at time of failure).

Despite the above potential contributions to anchor overloading and/or load fluctuation, there are no indications that failure within the fixed anchor had taken place. However, such conditions may have contributed to the failures of the anchor tendons and other components

which were observed. Analysis identified three main areas where some failures occurred, either solely due to corrosion, or due to load increase in the anchor tendon identifying the weakest component:-

- i) the failure of the epoxy resin coating over the external anchorhead;
- ii) the inadequacy of the underhead grouting;
- iii) a weakness at the junction between the underhead grouting and the upper free length protection.

Where tendon failures within the anchor free length occurred, they probably resulted from a combination of overloading and angular deviation due to change in geometry during wall failure, although the occurrence of corrosion, where some cracking in the corrosion protection may have taken place, could not be eliminated.

The anchor system and associated corrosion protection provided during this early period of anchor introduction in the United Kingdom were either consistent with, or in advance of, the State-of-the-



**Photo 6:**      **The completed rebuilt quay**  
**Main Contractor: Costain Civil Engineering**  
**Anchor Sub-Contractor: Keller Colcrete**



**Photo 7: Modern corrosion protection system protects the entire anchor. Grease filled glass reinforced anchor caps used in the marine environment**

Art in the UK at that time (1968). Investigation revealed that attention was paid to the protection of all components, and it is only during the continual development over the intermediate 20 year period that the shortcomings in certain protective mediums then used have been realised.

It cannot be fully established whether anchor failure during week beginning 26 February 1990 was solely due to corrosion of certain anchor components, or whether load increase in the anchors caused by failure of other structural components or by increased pressure on the wall, also contributed. However, it is inevitable that failure due to corrosion would eventually have taken place.

Subsequent to failure, the quay wall was reconstructed and restrained by the use of a tie rod system which was easily facilitated in the open site. However, the existing length of wall upstream of the failure was stabilised by the installation of an additional row of ground anchors, fully protected against corrosion by the double plastic protection system. The collapse and inspection demonstrated that the provision of a high quality corrosion protection, in which at least one proven barrier must exist to protect all steel components when in situ, and this barrier must outlive the intended lifespan of the structure.

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