Cintec Strengthening and Repair for the Rail Industry.

CINTEC

A Selection of Cintec Case Histories Spanning Three Decades.


Bridge and Viaduct Reinforcement


Station Refurbishment


Tunnel Repairs


Ground Anchoring


Signal Anchoring


Parapet Strengthening

Tried & Tested

BRE

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Preface

This document brings together the three key elements being used by UK Rail authorities for a period spanning three decades that now comprise Cintec's bridge strengthening method (Archtec). First, the Cintec anchor has provided the basis of repair and reinforcement for a variety of structures including bridges and viaducts; included as an example is the significant work in stabilising the Kennett Bridge (Paddington-Reading line) in 1987 – the work illustrates the Cintec anchor's long term durability. Second, the various methods of installation demonstrates the versatility of the design of the anchor to meet specific needs of the responsible engineer – the whole method is based on the principle of designing the anchor to meet the need. Third is the use of the Discrete Element Analysis System (ELFEN) by Gifford; this method of determining the rating of masonry arch barrels is accepted by various Network Rail clients.

* Please note that prior to December 1998, Cintec International Ltd was registered under the name of Cavity Lock Systems Ltd. This booklet contains references to Cavity Lock Systems, however, other than this change in name, the management and structure of the company has remained exactly the same in all other respects.

FILE COPY

CERTIFICATE OF INCORPORATION

ON CHANGE OF NAME

Company No. 1884730

The Registrar of Companies for England and Wales hereby certifies that

CAVITY LOCK SYSTEMS LIMITED

having by special resolution changed its name, is now incorporated

under the name of

CINTEC INTERNATIONAL LIMITED

Given at Companies House, Cardiff, the 1st December 1998

C018847300

THE OFFICIAL SEAL OF THE
REGISTRAR OF COMPANIES

COMPANIES HOUSE

HC006B

Selection of railway viaduct case histories

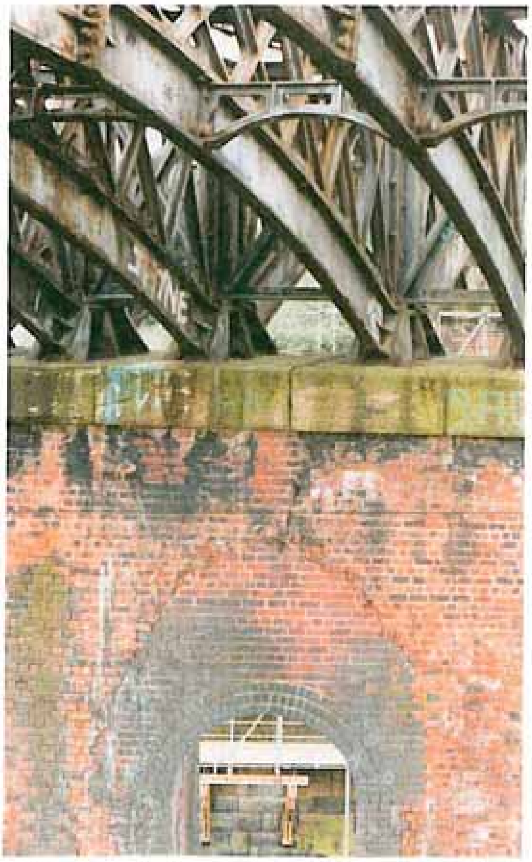
Outwood Viaduct - Radcliffe

Following its closure in 1966, Outwood Viaduct had fallen into dereliction, however its proposed demolition by British Rail was forestalled due to public objection led by the Railway Heritage Trust and it was eventually given Grade II listed status.

It spans the river Irwell at the western edge of Radcliffe, Greater Manchester, the spans were fabricated and erected in 1881 and have an overall length of over 100 metres.

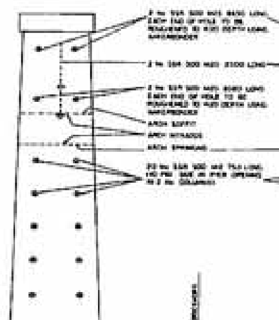
Each span comprises six cast iron open spandrel arch ribs with lateral bracing.

British Rail had previously attempted to strengthen the four tapering brickwork pillars by adding new masonry to the original single archway piercings located in each pier. This new work had however began to detach from the original structure and extensive cracking was visible between the new and the old (see right).

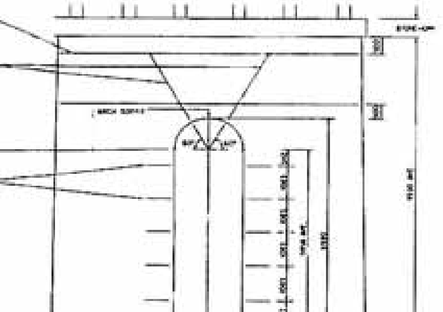


Cintec supplied 108 stud and rebar stitching anchors ranging from 1 to 10 metres in length. These were installed through the cracks to re-connect the inner reinforcement brickwork to the original structure as indicated in the design proposal below.

After renovation Outwood Viaduct was formally opened as a footpath, bridleway and cycleway in 1999 by Sir William McAlpine, President of the Railway Heritage Trust.

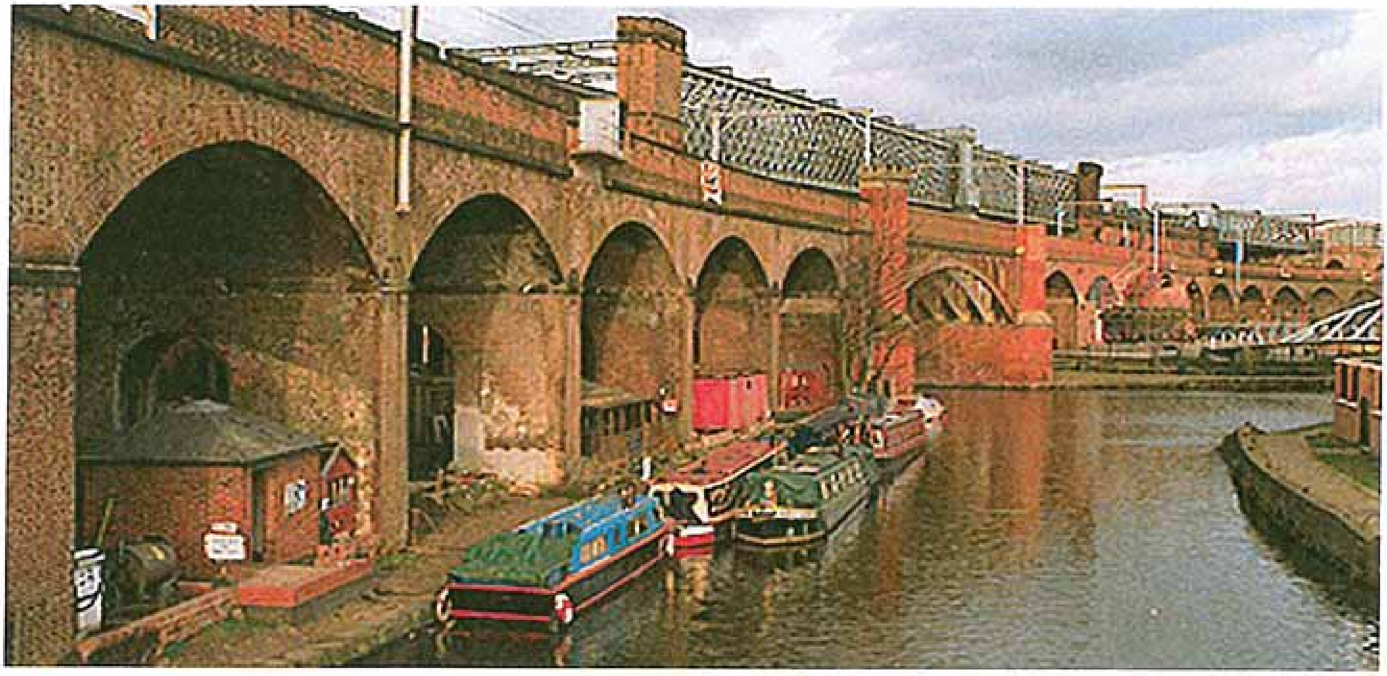


TYPICAL PIER SIDE ELEVATION

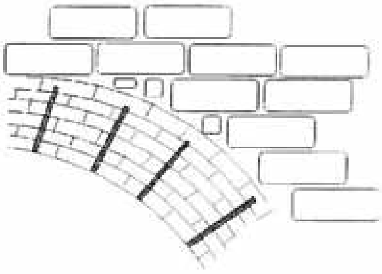


TYPICAL PIER ELEVATION SHOWING PROPOSED 'CINTEC' ANCHOR LOCATIONS

Deansgate Viaduct - Manchester

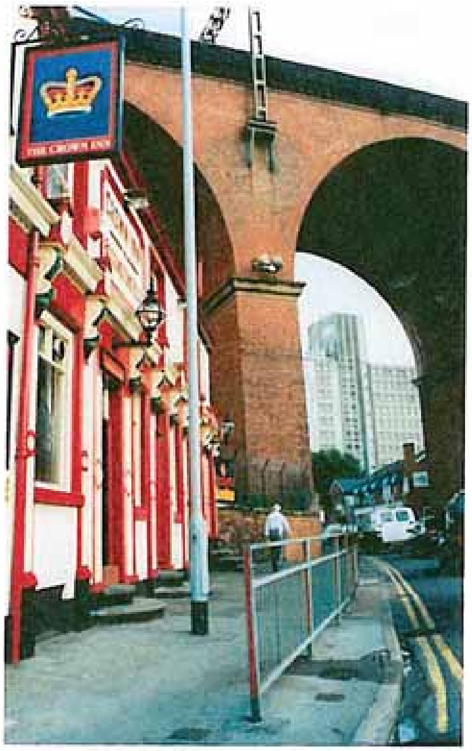


The busy Deansgate rail viaduct is situated in the heart of Manchester spanning numerous buildings, roads and canals. In 1997 the normal daily flow of rail traffic was disrupted by a destructive fire which took hold in a workshop located directly underneath. The subsequent heat generated by the blaze caused extensive damage and a weakening of the seven rings of masonry that form the arch barrels. The surface ring of brickwork completely delaminated and collapsed to the ground below.

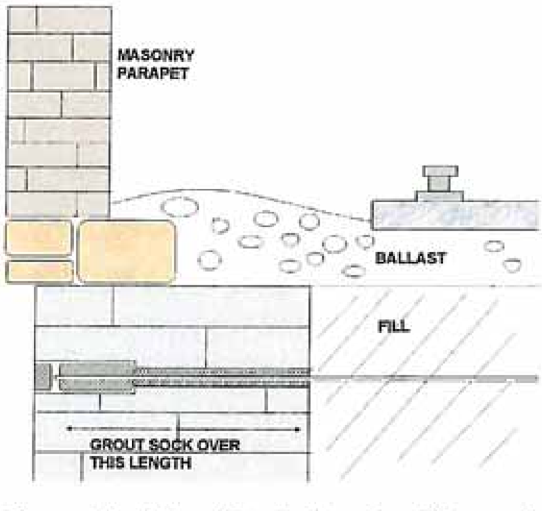


A team of consulting engineers assessed the damage and recommended a Cintec reinforcement solution. Any remnants of the outer ring were completely removed and the remaining six rings were hammer tested to locate the extent and area of internal delamination. Two arches were found to be in need of repair. In total approximately 500, 60 cm long RAC Cintec anchors were installed, perpendicular to the arch and at spacings of 50 cm. The anchors were staggered to avoid the formation of shear lines and because of their vertical aspect, each anchor was fitted with an air-vent tube to ensure full grout inflation without risk of air pockets being formed at their remote end. All anchors went no further than half way through the sixth ring so as not to puncture the original waterproof membrane that protects the arch barrel from the arch infill.

Finally the original appearance of the arches was restored by grouting an original piece from the drilled cores back into the mouth of each anchor hole. The completed work was rendered invisible to the naked eye and the viaduct was once again in operation servicing Deansgate station and the G-Mex conference centre.



Killiecrankie Viaduct - Tayside



Cross sectional view of 5 metre three piece Cintec anchor.

In1998 Killiecrankie Viaduct was both repaired and strengthened. The work was intended to increase maximum track speed and accommodate Intercity trains travelling up to 125 mph. These improvements were part of an extensive program covering the entire length of the Highlands Railway from Perth to Inverness in Scotland.

Following the contours of Glen Garry, the curvature of the multi-arch structure added to the engineering challenge. Engineering consultants Scott Wilson Glasgow assessed that strengthening would be required in order that the viaduct withstand the increased lateral forces being exerted by high speed trains.

The solution was provided in the form of 30mm Cintec deformed rebar anchors in lengths between 1 to 5 metres. Installed horizontally under the full width of the viaduct, the anchors passed from the masonry spandrel wall through the springing vee joints to the opposite spandrel wall. Only the anchor sections located within the spandrel walls were socked and inflated with grout (see above). To increase tension values the anchors were installed in stepped bore holes allowing the sock to expand beyond the diameter of the inner bore hole. Other anchors were installed through the voussoir stones into the masonry arch barrels. In total 230 cintec anchors were installed by the experienced drilling company Ritchies of Kilsyth.

Public footpath at base of Viaduct. Killiecrankie is situated in National Trust land popular with walkers.

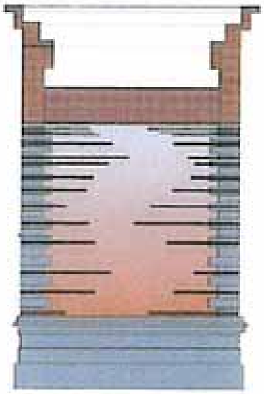
Royal Border Bridge - Northumberland

As part of Railtrack's major programme of repair and refurbishment, work was authorized on fifteen arches of the Royal Border Bridge. The bridge carries the main intercity East Coast rail line between Edinburgh (Waverley Street) and London (Kings Cross). George Stephenson's magnificent 28-arch, 39m (128 ft) high viaduct spans the tidal estuary of the River Tweed. Queen Victoria and Prince Albert opened the 658m (2160 ft) long bridge in 1850.

The project was complicated by both environmental and technical factors. Green nylon based Debri-Mesh surrounded the main work areas to contain dust and debris from the drilling which, if uncontained, would cause environmental problems to the residents of the Riverdene Estate situated directly below the bridge. The covering material also provided a degree of shelter from the strong prevailing winds which blow eastwards down the Tweed River's valley. Furthermore, certain areas of the 19m (61' 6") span brick arches provided roosting areas for galleries of bats. Provision was made to minimise disturbance to this protected species as well as keeping clear exits for their use. The ornamental stonework which forms the top parapet of the viaduct, is also a nesting site for House Martins and a pair of Kestrels were observed nesting under one of the electricity catenary poles.

A total of 1256 Cintec anchors were installed during 1995 and 1996. These were installed horizontally through the voussoirs to varying sizes and drill depths in order to prevent the creation of a shear line in the parent material

Apart from the erection of the electrification gantries for the high-speed Inter-City 125 express trains, some years earlier, this refurbishment is the first major repair work to be carried out in its entire 150 year existence a tribute to the engineering skills of the Victorian builders and an indication of the faith placed in the Cintec Anchor System. The project was partially funded by English Heritage.



Vertical cross section of arch stitching.

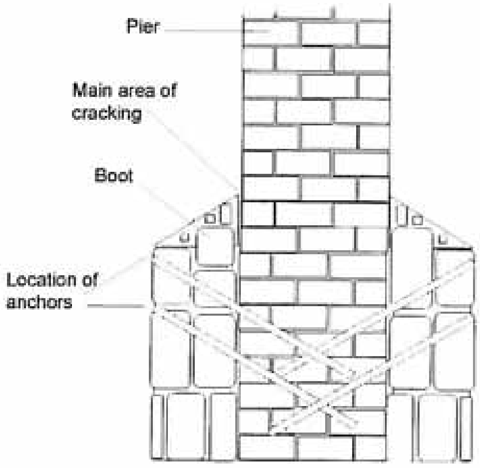
Leaderfoot Viaduct - Scottish Borders

View of drilling platform

Located in the Scottish Border country, the Leaderfoot Viaduct has four of its piers with foundations in the river Tweed, the brick masonry is protected by stone block 'boots' designed to deflect water and flood debris. However in 1994, after more than a century of service, extensive cracking had developed between the stonework and the brick masonry both above and below the water line. Although not exceeding 1.5m in depth, divers were required to assess the extent of damage underwater. A remedial solution for re-securing the two elements was devised by the installation of sixteen; 20mm Cintec rebar anchors, 2m in length and four per pier.

The uncontaminated river is popular with salmon fishermen and the necessity to avoid any environmental pollution was uppermost in the minds of all those involved. As alternative un-contained methods of anchoring and grouting were out of the question, Cintec was the clear choice.

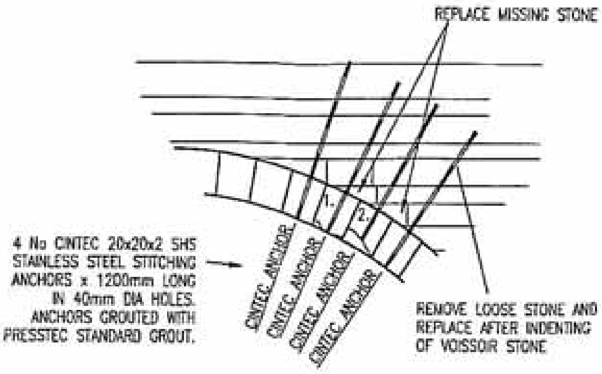
Under the supervision of the local river authority, holes were drilled at a downward angle through the boots and into the piers. These instantly filled with river water, however due to the unique nature of the Cintec anchor - filling a mesh fabric sleeve from the rear to the front, all water was fully displaced upon grout injection. The visible cracks were sealed manually by inserting lengths of sock into the fissures and expanding them. The subsequent watertight seal allowed conventional grouting to be injected into any remaining internal voids without danger of release into the water system.



Teviot Viaduct - Roxburgh

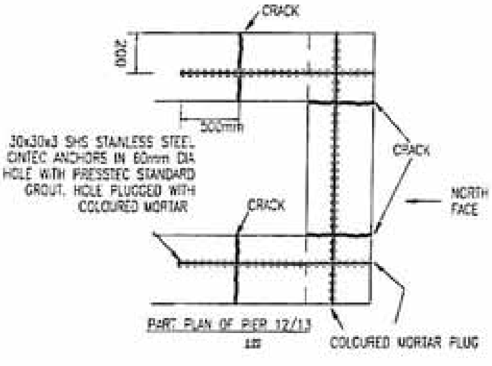
Built in 1847, the Teviot Viaduct spans the river Teviot at Roxburgh in the Scottish Borders. As a consequence of no longer being part of the rail system, the stone masonry structure had fallen into disrepair with extensive cracking to both the arches and the piers. A number of stone blocks had also come loose and were missing. However becasuse of its significance to local heritage, the viaduct was considered worthy of preservation and funding was made available by the British Railways Board and the Railway Heritage Trust.

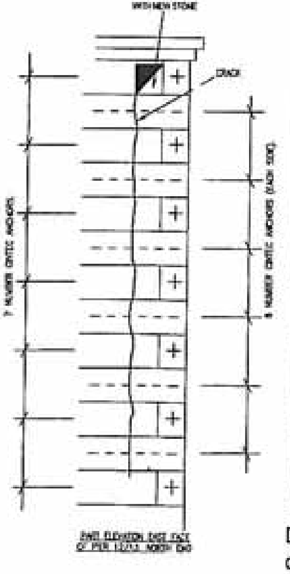
The first phase of restoration involved the replacement of broken and missing voissoir stones from the arch barrels. In order to reduce the risk of a progressive collapse, neighbouring stones were held in position by square hollow section stitching anchors 1.5m in length, this consolidated the arch while the replacement stones were installed.



PART ELEVATION OF VOUSSOIRS/SPANDREL FACE
1:50

The second phase of work involved interlocking the outer masonry walls of each pier. The original design drawings and the photograph (below and right) reveal the extent of the cracking and the subsequent Cintec solution. In total 112 anchors were installed.





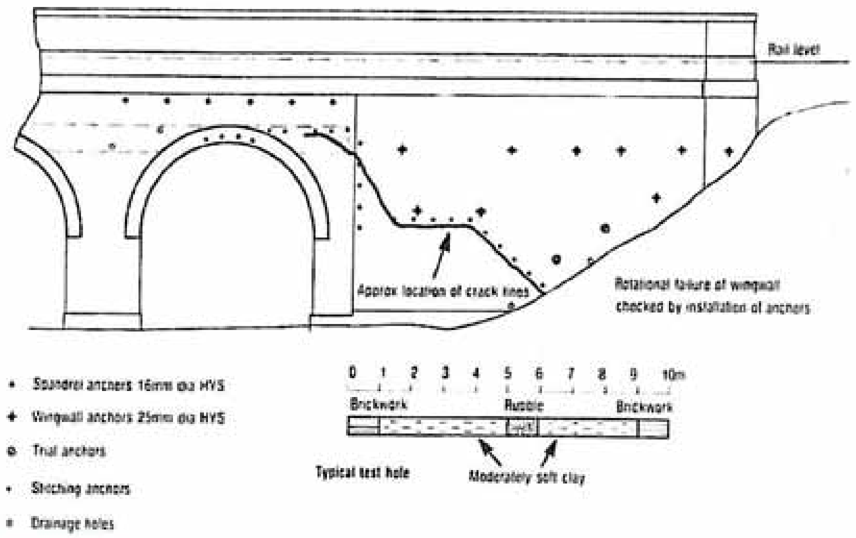
During this project a new team of installers were trained on site by an experienced Cintec Technical Advisor.

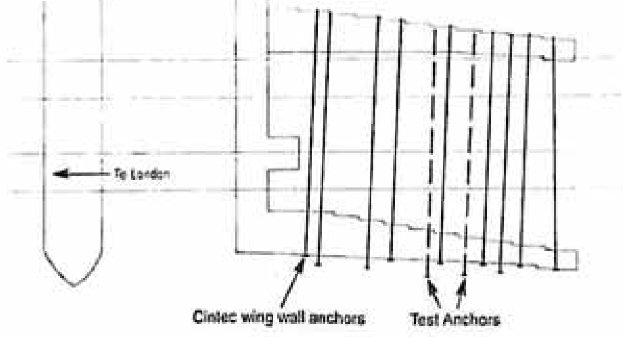
Kennet Bridge - Reading

River Kennet Bridge is located on the main rail line from Paddington to Reading. In 1987 it became clear that the bridge required stabilisation of the north side wing wall and to the adjoining spandrel wall, secondary work was also required for stitching and filling of cracks in the brickwork. Under the control of the regional engineering projects manager, Mr W G Grant of the Civil Engineering department - British Railways Board, Western Region, engineering analysis was carried out and a proposal put forward which involved tying the north side wing wall to an original buried wing wall with anchors 9.5m long. The spandrel wall would be anchored to the corresponding buried spandrel wall.

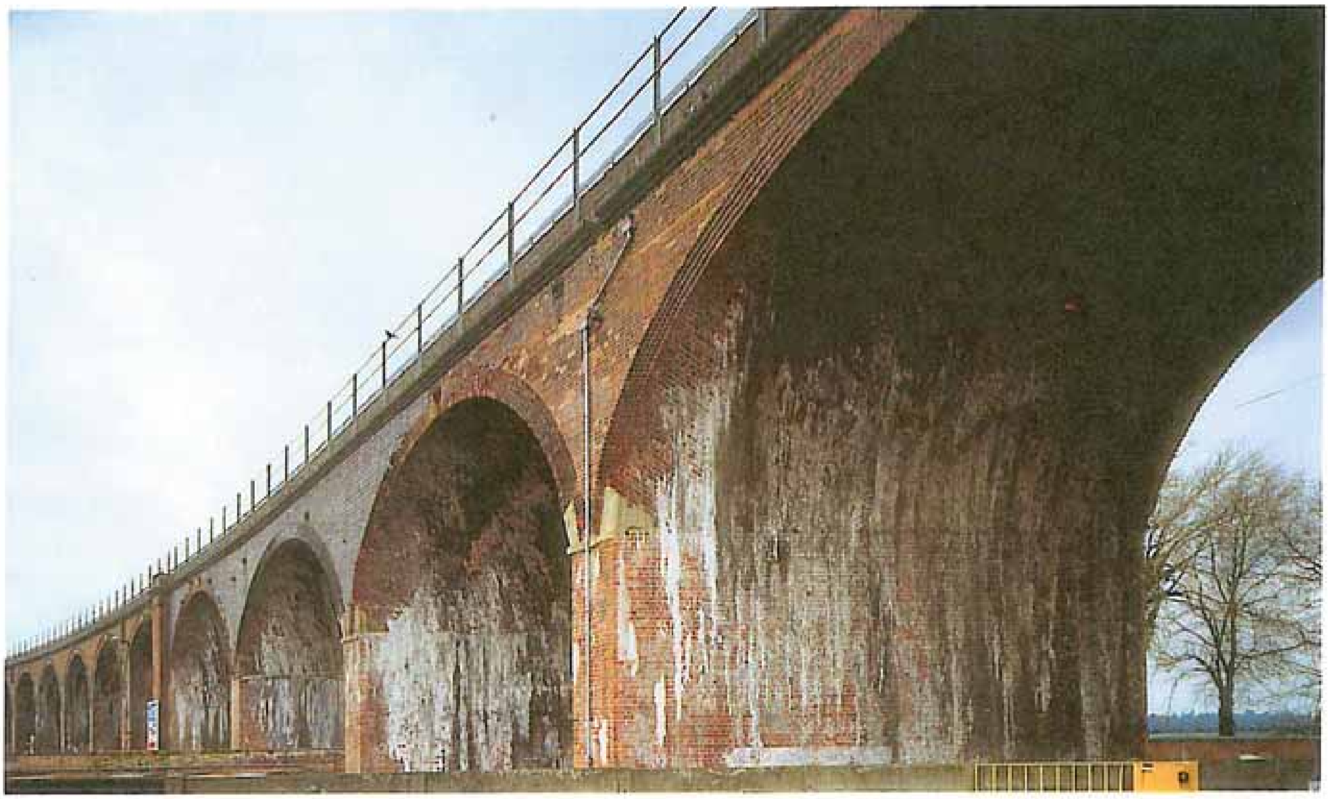
Two test anchors were installed made out of 25mm diameter high tensile reinforcing bars with separate grout feed pipes for the both the rear and front lengths of the anchor. These were installed into 76mm diameter drilled holes with the intension of testing both the anchors capability and to ascertain the load capability of the buried wing wall. A rotary drag bit with percussive rotary drilling produced the holes through the saturated fill material. A typical section through the wing walls and fill material is shown in Fig 1. The anchors were installed manually without difficulty and the rear 5m was grout injected. The required test loads of 10 tonnes were achieved and satisfactorily monitored for 24 hours. Loading and reloading the anchors at lower loads had shown essentially elastic behaviour.

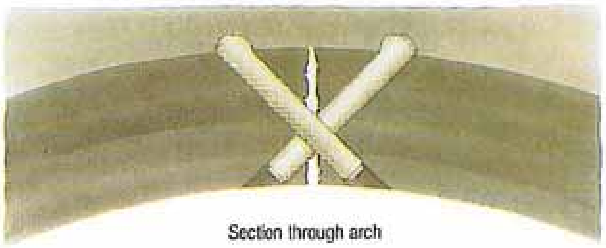
The successful tenderer for the remedial works adopted the Cintec Anchor System in preference to a resin alternative. The anchors used were high tensile 25mm diameter, 10m long bars, surrounded in a mesh fabric sleeve and grout injected following installation. The anchors within the ring wall were required to penetrate the buried wing wall so that grout plug was formed at the back of the wall. The remedial stitching anchors were 15 x 15 x 1000mm long stainless steel SHS anchors in a sleeve. With the use of a steam driven compressed-air auger, a 10m hole could be drilled every 40 minuets enabling the project to be completed ahead of schedule.





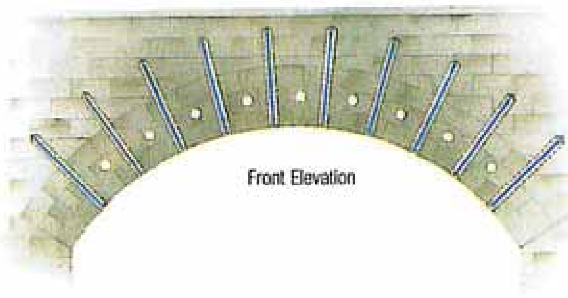
Worcester Viaduct





Section through arch

Plan view of stitching anchors



Front Elevation

Worcester Viaduct comprises sixty-five brickwork arches rising from approximately two storey height near the railway station to over three storey height as it approaches the river. Lack of proper draining within the arch had led to the spandrel walls being forced away from the intrados arch with longitudinal cracks close to the longitudinal edges of the bridge. Water penetration had contributed to cracking at the springings of some spans and delamination of external parts of some columns. These problems had been exacerbated by weathering, particularly freezing and thawing. Previous efforts to restore the structural integrity were evident, but had proved ineffective.

Transverse 30 x 30 x 3 SHS stainless steel WSA anchors were installed to restore the integrity of the spandrel wall/intrados arch connection at approximately 750mm centre-to-centre and alternate lengths of 2.0m and 2.5m. Stitching anchors were angled across the longitudinal cracks to restore structural integrity and the cracks were then filled. Transverse and diagonal stitching anchors, type RWT, 15 x 15 x 1.5 SHS stainless steel, were installed to restore the strength of the delaminated columns and the cracks filled. Drainage holes were drilled through the intrados and plastic pipes were installed to help relieve the existing water pressure. To date five spans have been renovated using the Cintec system and further spans will be renovated as part of an ongoing maintenance programme.

Mousewater Bridge

The slight curve in Mousewater Viaduct caused trains exert centrifugal forces within the structure. In consequence, radial cracking developed within the spandrel walls above the piers. 16mm stainless steel rebar anchors were installed in 50mm diameter holes to repair cracking and 32mm grip bars and pipe anchors were installed into stepped holes and secured with 35mm end plates in order to 'clamp' the spandrel walls and contain bulging and internal settlement.

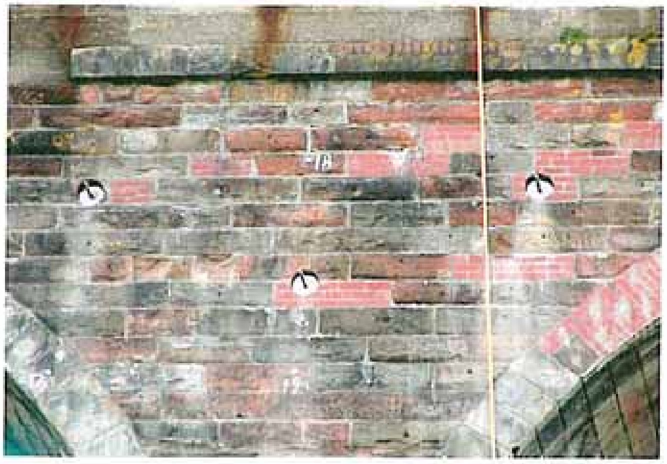
Burnton Bridge

Cracking in the barrels due to centrifugal forces meant the imposition of a 5 miles per hour speed limit. Both abseiling and a mast lifter were used to install strengthening anchors.

Core Drilling

Anchor Placement

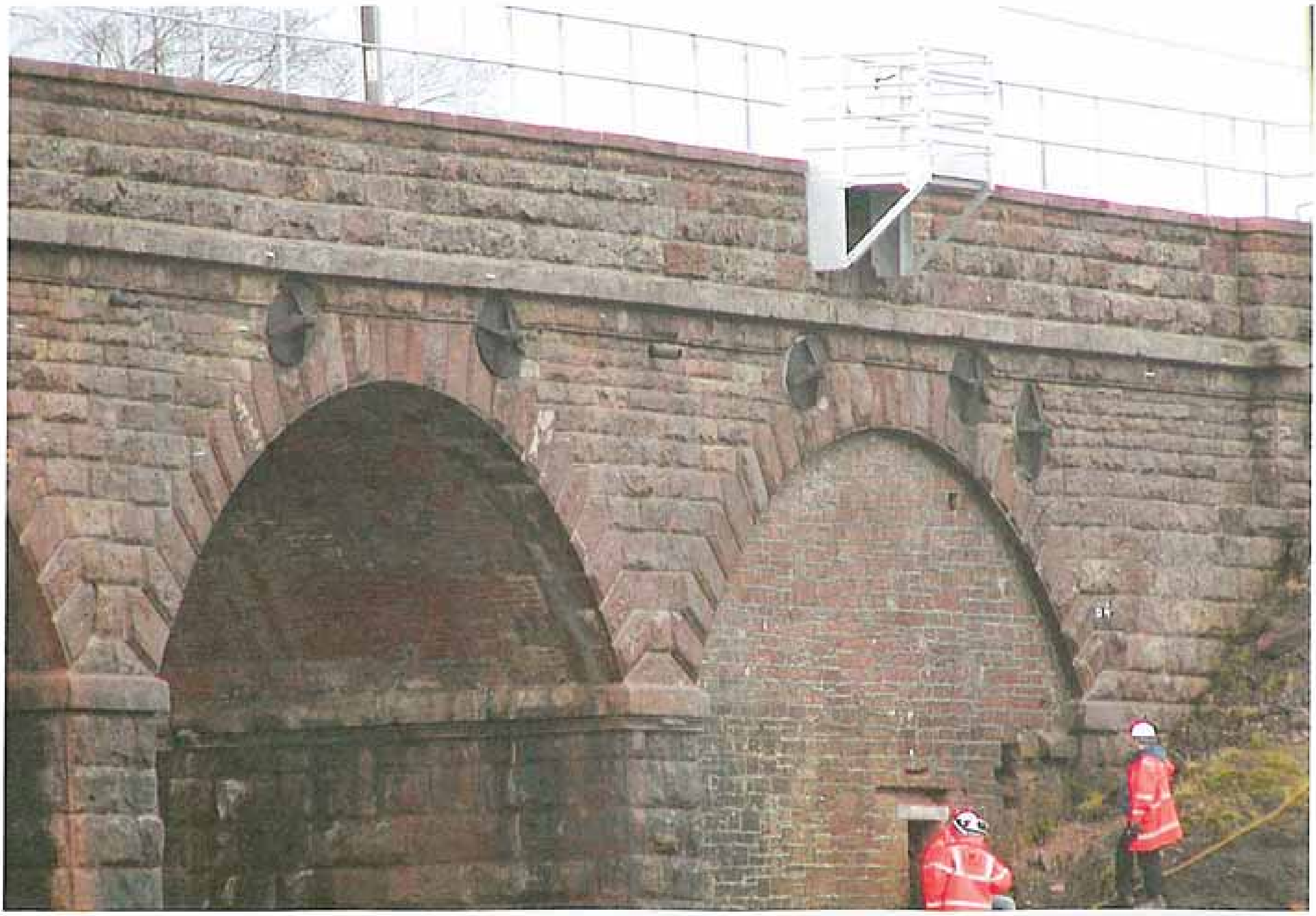
Anchor grout injection

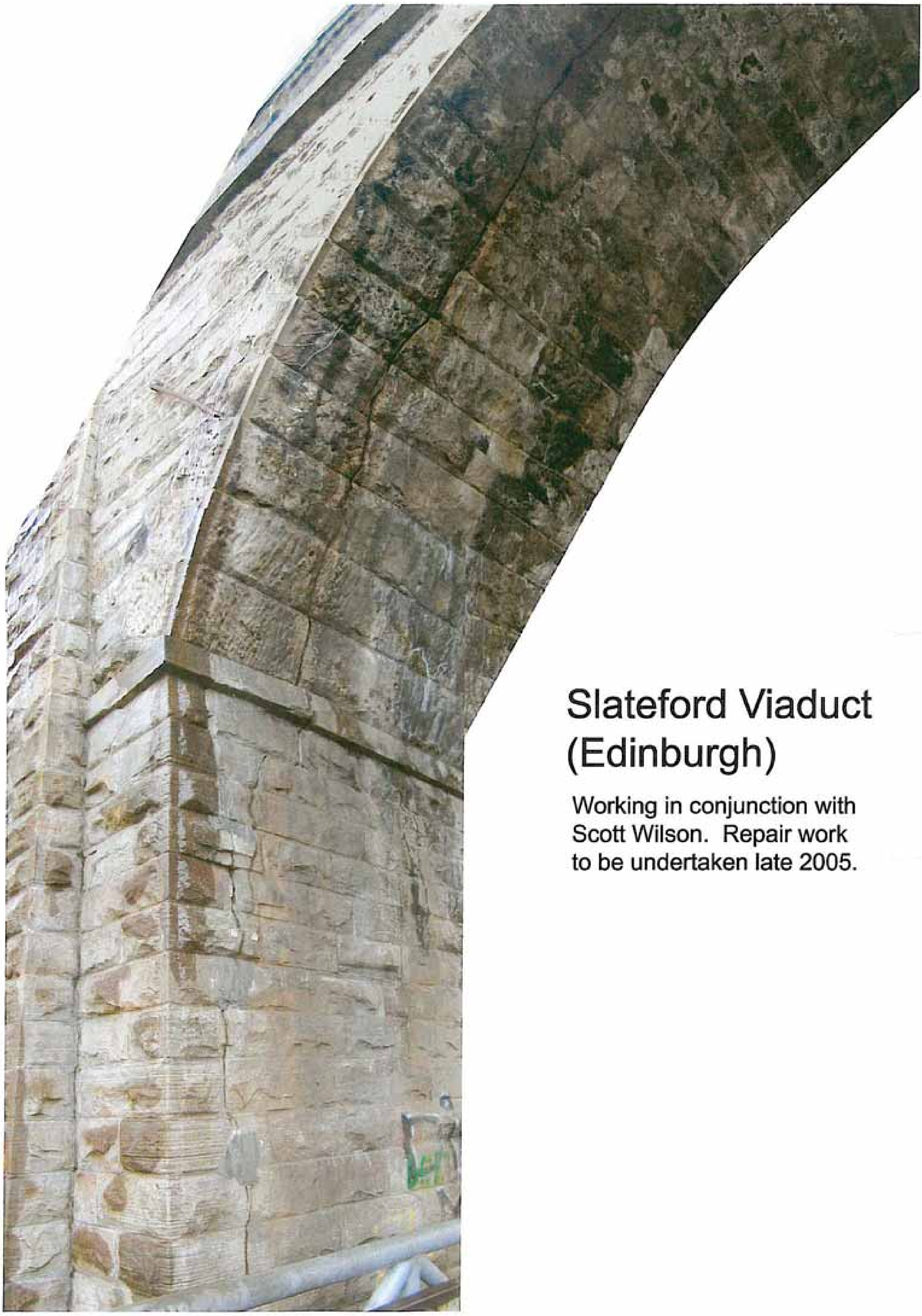


Inflated pipe anchors (Holes subsequently plugged and hidden).

Meinwater Bridge

Cintec Pipe anchors hidden within the stonework, counter forces imposed upon the spandrel walls.



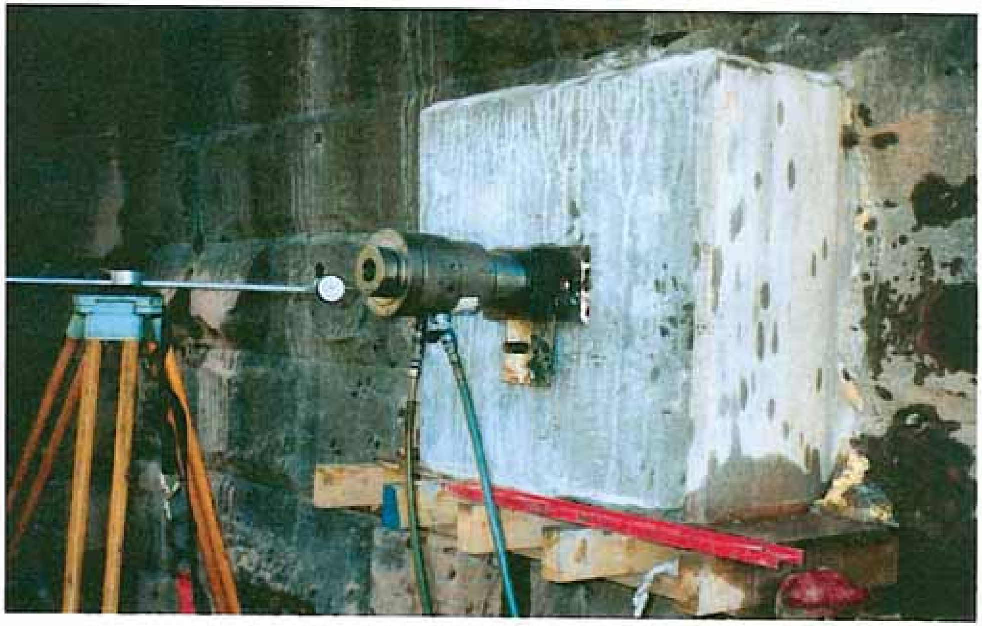


Slateford Viaduct (Edinburgh)

Working in conjunction with Scott Wilson. Repair work to be undertaken late 2005.

Harthope Bridge (Beatock Summit)

Part of the upgrading of the West Coastr main line. Replacement of bearings underneath steel girders. Concrete cast in situ against the abutment end and secured with 24 Macalloy Cintec anchors 5 metres in length and each under a 52 tonne tension load in order to increase friction. Pull out test (below) took an individual bar to 629Kn point without creep.



FENCHURCH STREET RAIL STATION, LONDON, U.K.


Cantilever Signal System

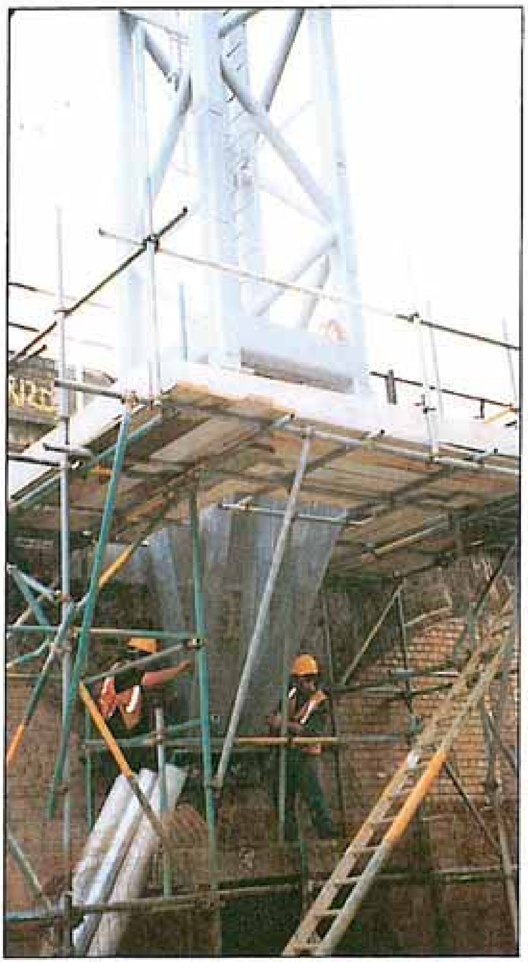

Viaduct

Fenchurch Street Station is one of London's busiest rail stations; it is the start point and terminus for the main tracks from the South of the U.K. to London. The construction itself is a remarkable example of Victorian 'railway' Architecture and was built at the height of rail travel era. The tracks carrying the service to the station travel over a Victorian Viaduct, comprising a series of arches. These arches support the cantilever system of signalling that guide trains to and from the station. The structure is a large steel gallows extending out over the track, with the signalling system suspended from it. The engineer had to recognise that any work on the structure had to address the problem of a live track running overhead.

The Problem

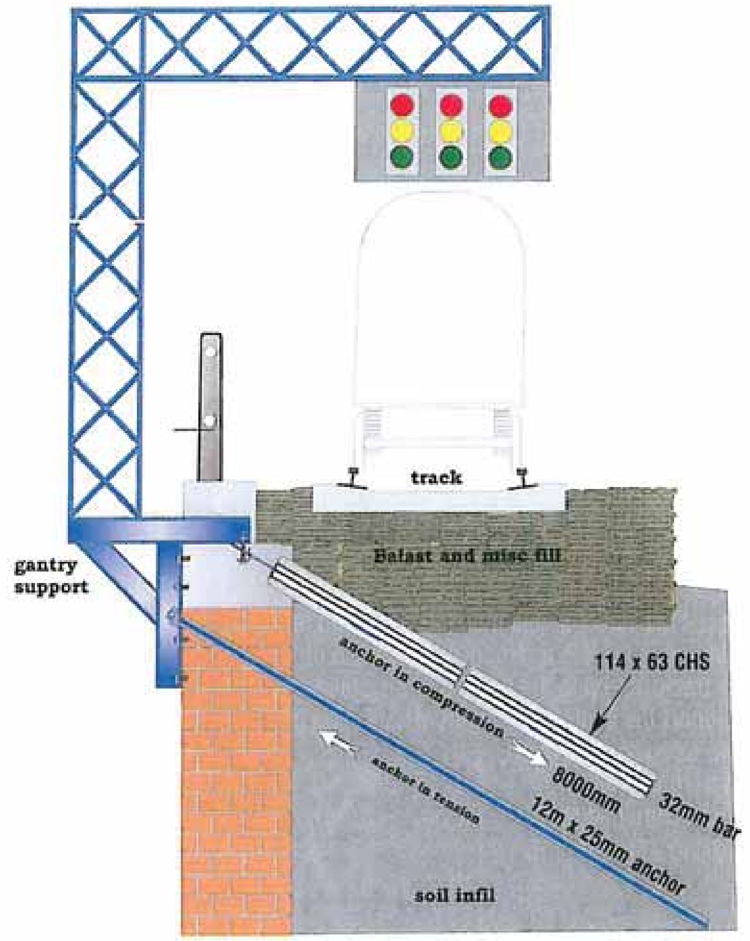
A system was required to secure the gallows to the bridge arches; in their preliminary planning, Railtrack anticipated a shut down of the tracks for 6 weeks. Such a closure would mean a chaotic time table, irate passengers and a loss of revenue. The CINTEC Anchoring System proposal provided a solution that would require only 2 days of rail shut down.

The assembling and installation of a compression anchor


The torqueing of the anchors

Fenchurch Street Station signal anchoring

FENCHURCH STREET RAIL STATION, LONDON, U.K.



FOUNDATION DETAILS OF ANCHOR ARRANGEMENT TO SIGNAL CANTILEVER FOR BRITISH RAILWAYS AT FENCHURCH STREET STATION LONDON

The Solution

The proposed solution involved three CINTEC Anchor types. The central one was a compression anchor of stainless steel comprising a 32mm shell rebar inside a 114 x 6.3 CHS installed in a 200mm hole, 8000mm deep, at an angle of 30 degrees to the horizontal. Below it was a tension anchor, comprising a solid stainless steel body, 12m x 25mm installed in a 50mm drilled hole, and attached to the gantry support to prevent any rotation. Two smaller shear anchors 20mm x 800mm were similarly installed to complete the support. Load tests were carried out, with the placing of a 20m steel beam in position.

As a result of the use of a CINTEC installation, disruption was reduced from 6 weeks to 2 days together with a 50% saving on the original budget Railtrack had allocated to the project.

Shear anchors shown at the top and tension anchors at the bottom

Testing

In shear

In tension

Ground anchor case history (Rail bridge 325)

Cintec Ground Anchor Installation at bridge 325 Abington

INTRODUCTION:

Cintec International Ltd has developed a system of ground anchors incorporating the patented grout techniques utilised in the Cintec System of anchor fixings. The bridge section of the Civil Engineering Department of Intercity Railways, British Rail, permitted the installation of trail ground anchors through the abutments of bridge number 325 on the Edinburgh / Carlisle Railway line for testing.

GENERAL DESCRIPTION:

In general terms the anchors have the following features:

a) A high tensile steel bar (ribbed type 2) forming the central element and load transferral mechanism to the abutment wall.
b) The reinforcement bar has been epoxy coated to provide the first layer of corrosion resistance in accordance with British Standard for Ground Anchors BS8081: 1989.
c) The corrugated sleeve of UPVC forms the second barrier against moisture and therefore corrosion resistance. The corrugations form a shear key to permit the transfer of forces from the ground to the central bar and then back to the structure.
d) The elements in a,b and c above are within a polyester fabric sock which expands to contain the pressurised grout, the sock becomes formed to the shape of the cored or drilled hole. Plastic centralisers are used to ensure the correct positioning of the corrugation relative to the bar. Drawings and sketches are attached showing details.
e) The grout forms the interlocking mechanism between the steel bar and the grout interface. The grout is a patented formulation developed specifically for anchor applications, it is delivered under pressure and is designed to obtain compressive strength capabilities of between 40 – 50 N/mm^2. Shrinkage is avoided by the use of additives premixed with the grout. The grout itself, being cementitious provides a highly alkaline protective environment against potential corrosion of the steel and the passage of moisture in the unstressed areas.
f) The sock arrangement used in the trial anchors has features such that the remote end (that which is in contact with the soil) can be inflated independently of the near sock (that which is contact with the structure). With this arrangement the remote end was tested in order to establish the load capabilities. After testing the outer sock was inflated to form the bond with the abutment structure.
g) Relatively low steel stresses were involved in the anchor testing to eliminate unnecessary elastic extension and subsequential relaxation losses may be neglected.
h) The outer sock forms a secure bond with the abutment structure thus avoiding the need for unsightly anchor heads visible on the outside.
i) Each stage of the inflation process is monitored by a 'check sock', that is a small sock that inflates at the external end of the anchor indicating that the remote or unseen sock is fully inflated.

The anchor component parts and design with regard to corrosion resistance comply with the requirements of BS8081: 1989 the British Standard for Ground Anchorage for Permanent Anchors.

INSTALLATION:

From a scaffolded access platform, a mining barrel was used to core the hole through the abutment structure and into the embankment behind. The anchors were inclined at 20° to the horizontal beneath the bridge structure, and at 30° to the horizontal at wing wall locations. The anchors were inserted into the preformed holes and the two sections of the inner sock inflated. The grout is inserted at pressure from a pressurised container (89 PSI, 0.61 N/mm^2). The outer sock was not inflated in order that each of the anchors could be subsequently be test loaded.

Sufficient time was permitted for the cementitious grout to cure before any load testing operations were carried out.

GROUND CONDITIONS:

The abutments are located either side of a vehicular access route through the railway embankment. The embankment was built approximately 100 years ago from nearby materials and consisted of gravel, sands with clay and silt. Given the soil profile found, the behaviour of the anchors would inevitably be unpredictable and large resultant test loadings were not anticipated.

TESTING:

The testing was carried out using a hydraulic jack with a calibrated dial gauge measuring the tensile load applied in tonnes. Each of the anchors was tested with the resulting loads tabulated in the following tables. The loads were applied in 4 tonne increments with a minimum of 10 minutes between each rise in the load. Several of the anchors were left for extended periods at the higher loads which coincided with the limit of the testing equipment. One anchor number 2 with the load applied overnight to see if any slippage had occurred. A small relaxation was apparent, although it could not be established if this was due to anchor creep or the testing apparatus deflecting.
The location of anchors is indicated in drawing C2162/Sk 1.

The results obtained were of larger magnitude than could have been anticipated given the actual ground conditions. In general the loads obtained varied between 13 – 20 tonnes. The bond stress or cohesion at the soil / interface has been calculated to vary between 81.3 and 219.7 KN/m^2. Anchor number 1 has an unusually low value of 93.8 KN/m^2, however this particular hole was left exposed for some considerable time after the mining barrel was removed before the anchors were fitted due to an equipment malfunction which may have led to some localised collapse of the substrate. Anchor number 5 also has an unusually low bond stress of 81.3 KN/m^2, this anchor was inserted into the area of the sloping embankment, which would not have had the benefit of the loading consolidation as the area underneath the railway tracks. The remaining results varied between 140.6 to a maximum of 219.7 KN/m^2 which reflects the variable nature of the substrate.

As the sock is inflated under pressure with grout, it expands to fill the shape of the hole, thus filling any irregularities in shape and size. A combination of different factors is anticipated to develop the load capacities obtained as follows.

1) Forming an irregular wedge by the shape of the hole and sock inflation, thus creating the need to shear the soil in order for the anchor to fail.
2) The grout 'milk' extrudes through the sock and partially bonds to the surrounding granular material, thus enlarging the effective diameter of the anchor.
3) Localised compaction of the surrounding material due to the pressurised grout inflation.

The installation and testing was witnessed by:

Mr Kader of British Rail Intercity Civil Engineering Dept.
Mr Barnet of British Rail Intercity Civil Engineering Dept
Mr Dimmick of Cavity Lock Systems (now Cintec International).
Mr Parry of Cavity Lock Systems (now Cintec International).
Mr Woodhouse of Fordham: Johns Partnership.

The anchors were installed in the period February – May 1992 and tested between June 1992 and December 1992.

DESIGN OF ANCHORS:

The following outlines the basic principals involved in assessing the design parameters and considerations in relation to the capacity of the ground anchors.

STEEL TENDON

The steel tendon in the anchors tested comprised of a high tensile steel bar, (epoxy coated for protection).

The bar area was established by the formula: $\text{Area required} = \dfrac{\text{Load}}{\text{Fy}}$

Where:- Load = working load multiplied by an appropriate factor of safety (200Kn)
Fy = characteristic strength of the steel (460 N/mm^2).

For the test anchors, the area required = $\dfrac{200\times10^3}{460} = 434.8\,\text{mm}^2$

Bar diameter 40mm provides area of 1256 mm^2, F.O.S. = 2.88
Bar diameter 32mm provides area of 804 mm^2, F.O.S. = 1.85

The steel stresses in this case were maintained at the low levels shown in order to avoid significant elastic extensions and therefore potential relaxation losses.
The steel bar utilised in the tests was a high yield ribbed bar (type 2) which has raised ribs on the surface for increased bond capability.

The bond between the grout and the bar can be established from the equation:-

$Fbu = B\sqrt{fcu}$ where fbu = the design ultimate anchorage bond stress.

$Fbu = 0.7\sqrt{40}$ B = coefficient dependent on type (0.5 x 1.4 = 0.7)

= 4.43 N/mm² fcu = compressive strength of grout (40 N/mm²)

DESIGN OF FIXED ANCHOR LENGTH:

The pull out capacity of the test anchors can be shown as:- $\mathrm{Tf} = \pi\,\mathrm{D\,L\,S}$

Where S = the shear, bond and skin friction at Substrate/rock interface (Kn/mm²)

D = diameter of fixed anchor (m)

L = Length of fixed anchor (m)

Tf = pull out capacity in (Kn)

The values of S varied between 81.3 to 219.7 Kn/m². For design purposes the lowest value should be used and a factor of safety of 4 utilised to limit ground creep in permanent anchors.

For design of anchors at specific locations the nature and behaviour of the substrate must be established by testing. Full-scale load tests are recommended to confirm laboratory results.

FIXED ANCHOR DESIGN IN ROCK

$$\mathrm{Tf} = \frac{\pi\,\mathrm{D\,L\,Tult}}{\text{Factor of Safety}}$$

Where Tult = the ultimate bond or skin friction at sock / rock interface.

The value of Tult will vary dependant on rock type, condition and discontinuities. A minimum fixed anchor length of 3m is recommended to account for local variations and a factor of safety of 3 to 4 be applied dependent upon the circumstances of usage.

FIXED ANCHOR DESIGN IN COHESIONLESS SOILS

The substrate at the testing location falls into this category although clay and silts were present.

$$\mathrm{Tf} = \frac{\pi\,\mathrm{D\,L\,S}}{\text{Factor of Safety}}$$

The value of S must be found by testing. A factor of safety of 4 should be used and a minimum length of 4m is recommended.

FIXED ANCHOR DESIGN IN COHESIVE SOILS

$$\mathrm{Tf} = \frac{\pi\,\mathrm{D\,L}\,\alpha\,\mathrm{Cu}}{\text{Factor of Safety}}$$

Where α = adhesion factor 0.3 – 0.45 verified by testing.
Cu = average undrained shear strength of substrate.

The value α and Cu must be found by laboratory tests or full-scale tests. The factor of safety should be of the order of 3 to 4 and a minimum length of 3m is recommended dependent upon consistency.

ANCHOR BOND TO STRUCTURE

Should the anchor be required to bond to the structure (as opposed to an anchor head arrangement) the following equation may be used:-

$$\mathrm{Ts} = \frac{\pi\,\mathrm{D\,L\,B}}{\text{Factor of Safety}}$$

Where Ts = ultimate bond to the structure material (Kn)

B = bond between sock and structure (Kn/m ²)

The value of B will vary dependent upon material, values of 600Kn/m² are reasonable (subject to testing) for solid concrete or masonry.

DISCUSSION

The general conditions at each location will dictate the design stresses to be used in assessing the ultimate capacity of an individual anchor. Where laboratory tests are not available, full-scale insitu tests are required to establish the lower bounds of the substrate capacity.

A minimum fixed anchor length of three metres is recommended to account for local variables in substrate conditions.

In order to reduce the possibility of long term ground creep, factors of safety should be applied. These factors should be of the order of 3 to 4 dependent on soil consistency, life expectancy and their importance to the structure.

The fixed anchor length must be located beyond the critical zone, such as the wedge failure, slip circle, rock discontinuities in order to be effective. The free anchor length will depend upon the geometry of the location.

The anchors can act as a restraint, only accepting load if movement occurs, or they can be pre-stressed to a set load to provide an active force.

A feature of the Cintec System is that a choice of connections can be achieved with regard to fixing to structure. Traditional anchor head details may be used where periodic re-stressing or monitoring is required. Where the structure is suitable, the anchor may be bonded to the material as a permanent fixing, without the requirement for surface apparatus.

GENERAL DESIGN CONSIDERATIONS

Where ground anchors are being utilised, careful consideration should be given by the designer to the following points:-

a) Detailed field and laboratory tests to establish soil characteristics.
b) Full-scale load tests to confirm laboratory predictions.
c) Assessment of consequences of potential long-term creep.
d) Overall length of anchor, fixed anchor length, failure planes.
e) Effects of anchor groups if anchors closely spaced.
f) Likely stress losses due to tendon relaxation.
g) The free anchor length can be released from the grout by use of smooth tubes forming the second barrier of corrosion resistance, thus avoiding stressing ground close to structure.
h) The factor of safety to be applied.
i) Reference should be applied to the British Standard BS.8081 : 1989 or other appropriate document for advice on usage and design.

CONCLUSION

The testing of the ground anchors showed that the Cintec System could be successfully used in even the most difficult of ground conditions and achieve results in excess of expectations.

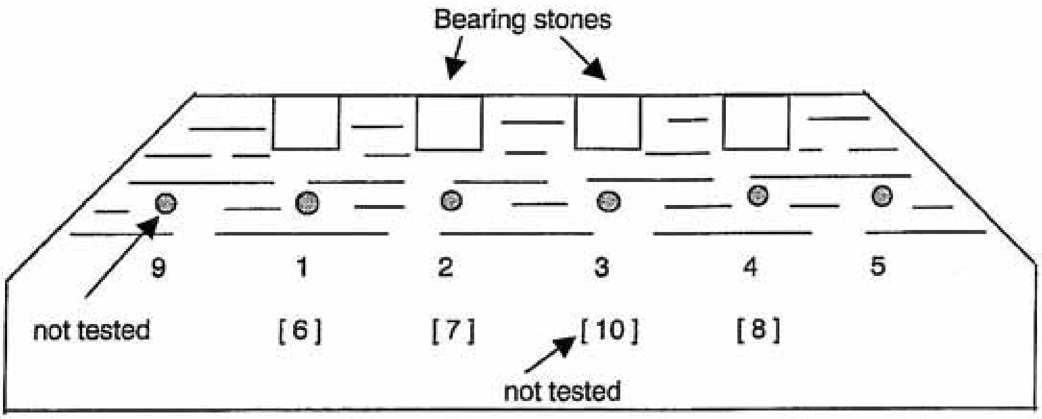
Careful appraisal of all factors must be given by the designer, to the points raised in the design considerations section, in order to fully realize the potential of the system.

S. Woodhouse

S. WOODHOUSE B. Eng (Hons) C.Eng M.I.Struct.E.

23rd APRIL 1993

Date: **April 1993** Scale: / Drawing No: **C2162/Sk 1**
Drawn: **J.S.** Design **S.W.** Project: **BRIDGE 325, ABINGDON**
Drawing Title: **GROUND ANCHOR DETAIL TO ABUTMENTS**



ELEVATION OF NORTH & SOUTH ABUTMENT SHOWING GROUND ANCHORS
SOUTHERN ANCHORS 1 – 5
NORTHERN ANCHORS 6 - 8

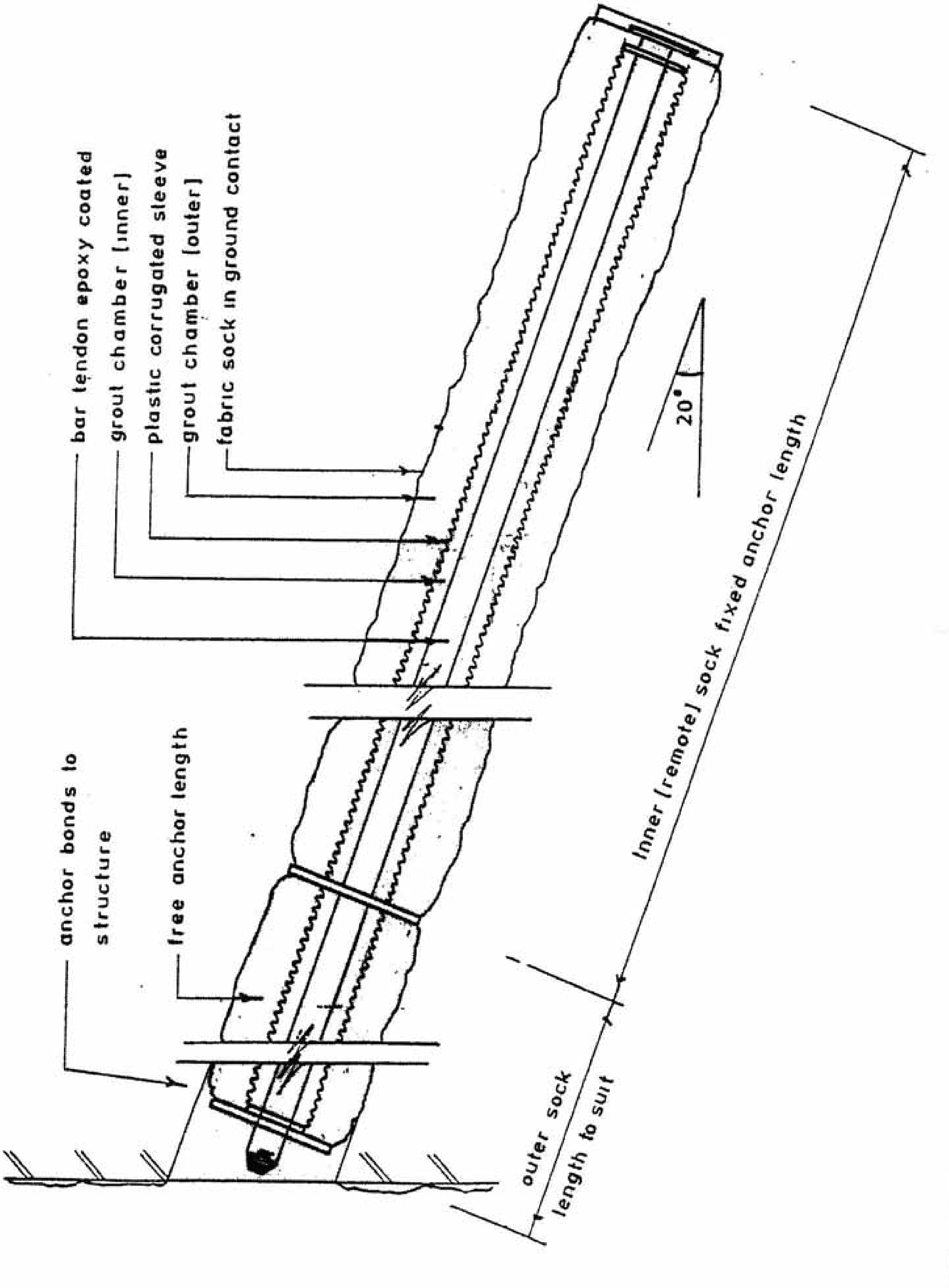
ANCHOR NUMBER	ANGLE OF INCLINATION	TOTAL LENGTH (M)	FIXED ANCHOR LENGTH OR LENGTH OF EMBEDMENT (M)	HOLE DIAMETER (MM)	TEST LOAD [T]
1	20°	5.45	4.1	124	15
2	20°	3.95	2.6	124	18
3	20°	3.45	2.1	124	18
4	20°	3.95	2.6	124	19
5	30°	5.45	4.1	124	13
6	20°	4.45	3.1	124	18
7	20°	4.45	3.1	124	17
8	20°	4.95	3.6	124	20

Date: **April 1993** Scale: / Drawing No: **C2162/Sk 3**
Drawn: **J.S.** Design **S.W.** Project: **BRIDGE 325, ABINGDON**
Drawing Title: **GROUND ANCHOR TEST RESULTS**

Anchor number	Angle of inclination	Total Length (m)	Fixed anchor length or length of embedment (m)	Hole diameter (mm)	Soil anchor Interface (mm^2)	Test Load (T)	Test Load (KN)	Shear stress Soil / anchor Interface (N/mm^2)	Shear stress soil anchor interface (KN/m^2)
1	20°	5.45	4.1	124	1.599x10^6	15	150	0.0938	93.8
2	20°	3.95	2.6	124	1.014 x10^6	18	180	0.1775	177.5
3	20°	3.45	2.1	124	0.819 x10^6	18	180	0.2197	219.7
4	20°	3.95	2.6	124	1.014 x10^6	19	190	0.1873	187.3
5	30°	5.45	4.1	124	1.599 x10^6	13	130	0.0813	81.3
6	20°	4.45	3.1	124	1.209 x10^6	18	180	0.1488	148.8
7	20°	4.45	3.1	124	1.209 x10^6	17	170	0.1406	140.6
8	20°	4.95	3.6	124	1.404 x10^6	20	200	0.1424	142.4

Date:	April 1993	Scale:	/	Drawing No:	C2162/Sk 2
Drawn:	J.S.	Design	S.W.	Project:	BRIDGE 325, ABINGDON

Drawing Title: **GROUND ANCHOR DETAILS**

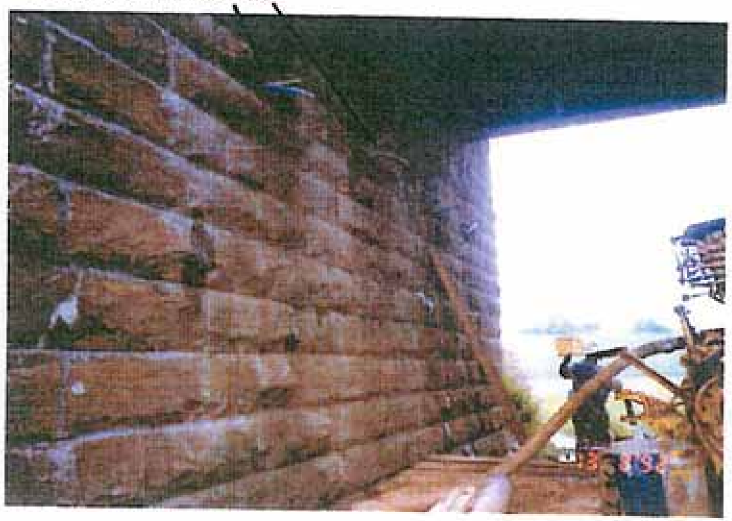


Bridge 325 Abington

Diamond Core Drilling Rig

Anchor Insertion

Installed Anchors



Anchor Testing

Parapet wall testing (London Underground Rail)

PRESTRESSING UNDERSTRENGTH WALLS AND PARAPETS

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Gifford and Partners
Carlton House, Ringwood Road
Woodlands
Southampton, SO40 7HT

INTRODUCTION

There are many brick walls and parapets, supported on elevated structures and located at the sides of roads and railways, that are now over 100 years old, some over 150 years, and in need of maintenance. One of the main causes of deterioration is through ageing and degradation of the mortar which is invariably lime based in these older structures.

The walls must be able to withstand wind loading, and where they are located beside roads or railways, there are extra windage effects caused by passing traffic. Additionally, there are traffic induced vibrations which can exacerbate the live loading and accelerate deterioration of the walls by loosening the old mortar. It is also not uncommon for walls, particularly those beside railway lines, to experience additional dead load effects due to utility pipes being bolted on, see Figure 1.

Figure 1 Parapet wall with bolted on utilities

Currently, old walls are failing strength assessments and it is necessary to undertake remedial measures by a suitable method. In addition many older walls are prone to inadequate stability. Differences in flexibility between the walls and their supporting structure, thermal movements, lateral loading effects, bed joint degradation and loss of adhesion can all lead to the walls becoming detached from their substrata. The supporting structure's contribution towards stability is thus lost, resulting in a reduced factor of safety for stability.

Any strengthening proposal must satisfy the requirements for both strength and stability as necessary. This papers presents such a strengthening solution using Cintec Anchors, a system of post-tensioning that has been tailored to meet the requirements of brick walls. An in situ test to confirm the performance of a post-tensioned 100 year old wall in the field is described.

OPTIONS FOR STRENGTHENING

The appearances of old brick walls do not necessarily have intrinsic value on their own, but taken alongside other structures such as adjacent bridges and buildings of similar age, the collection often merits heritage status. It follows that methods of refurbishment and strengthening should be acceptable in a cosmetic as well as structural sense.

In the normal course of events, deep raking out and repointing of the mortar joints using a stronger cementitious material is the most straightforward refurbishment. The lateral bending strength of the wall can be raised some 70 per cent using this method. However, using conventional methods of analysis and assessment, it is difficult to justify that such repairs provide adequate strength and factors of safety. Furthermore, repointing is a time consuming activity made additionally expensive by costs of access and the need to have lane closures or track possessions to satisfy safety requirements. In any event, repointing is unlikely to satisfy the requirements for stability.

In situations where walls are located on top of retaining structures (a fairly common occurrence), strengthening is sometimes carried out by bolting vertical channel-section steel girders to the brickwork and retaining structure, see Figure 2. This is an unsightly method requiring regular maintenance painting and unpopular with heritage bodies. Furthermore, it is often impracticable to fit the girders when utilities are bolted to the walls.

Figure 2 Externally strengthened wall

Post-tensioning the brickwork into the substructure provides a mode of strengthening having none of the above objections. It is quick to carry out, leaves no external evidence on the brickwork and is economic. The post-tensioning can be designed to suit local conditions and strength and stability can be calculated with adequate accuracy.

POST-TENSIONING SYSTEM

The required level of post-tensioning, spacing of tendons and lengths of anchorages are calculated to meet the requirements of the local conditions. The main components of the Cintec system comprise stainless steel tendons, cementitious grout and a sock to contain the grout. Stainless steel tendons are required to provide long term durability in an environment that is akin to post-tensioned segmental bridges where serious corrosion can occur at the mortar joints. Cementitious grout is used in preference to an epoxy based material as it is considered important to have materials that are compatible with the wall. The sock has been designed to contain the grout and prevent any loose brickwork being displaced by the injection pressures of 3 to 4 bar. It also prevents unsightly leakage through cracks that may be present. The sock permits controlled leakage of grout to enable a structural connection to be formed with the surrounding brickwork.

The construction activities are similar to conventional post-tensioning:

- Remove capping pieces and bore vertical holes through the centre of the wall to the required depth in the substructure.
- Introduce the sock and stainless steel tendon into the anchorage length within the supporting structure and inject grout to form the anchorage.
- When the grout has fully hardened, post-tension and lock off against a plate fitted to the top of the wall.
- Inject grout into a second sock occupying the remaining space in the masonry wall.
- When the second injection of grout has hardened, remove the end piece and plate, replace the capping pieces to leave a cosmetically acceptable appearance to the wall.

STRENGTHENING DESIGN PARAMETERS

No specific standards are available for the assessment or refurbishment of existing masonry parapets. The following parameters have therefore been developed for this Cintec anchor strengthening system:

- The post-tensioned wall is designed in accordance with BS5628:Part 2 assuming bonded tendons.
- Due account is to be given to the make-up and condition of the wall in deriving an appropriate characteristic compressive strength of the masonry.
- In accordance with BS5628:Part 2, at the Serviceability Limit State, no tension is to be permitted in the masonry.
- Wind loading to BS6399:Part 2 and additional loading due to dynamic pressure and suction from railway traffic derived from European Rail Research Institute Report ERRI D 189/RPI.
- Consideration of crowd loading.
- Bond stress between the Cintec anchor and masonry or other material is based on pull-out tests on similar materials.
- The prestress force must be sufficient to ensure a factor of safety against overturning greater than 2.

The governing criteria in the design of post-tensioned masonry structures is usually the restriction of no tension at the Serviceability Limit State. Accurate values for material properties are therefore not always necessary. This is fortunate since little data appear to be available for typical strengths and stiffnesses of old brick walls.

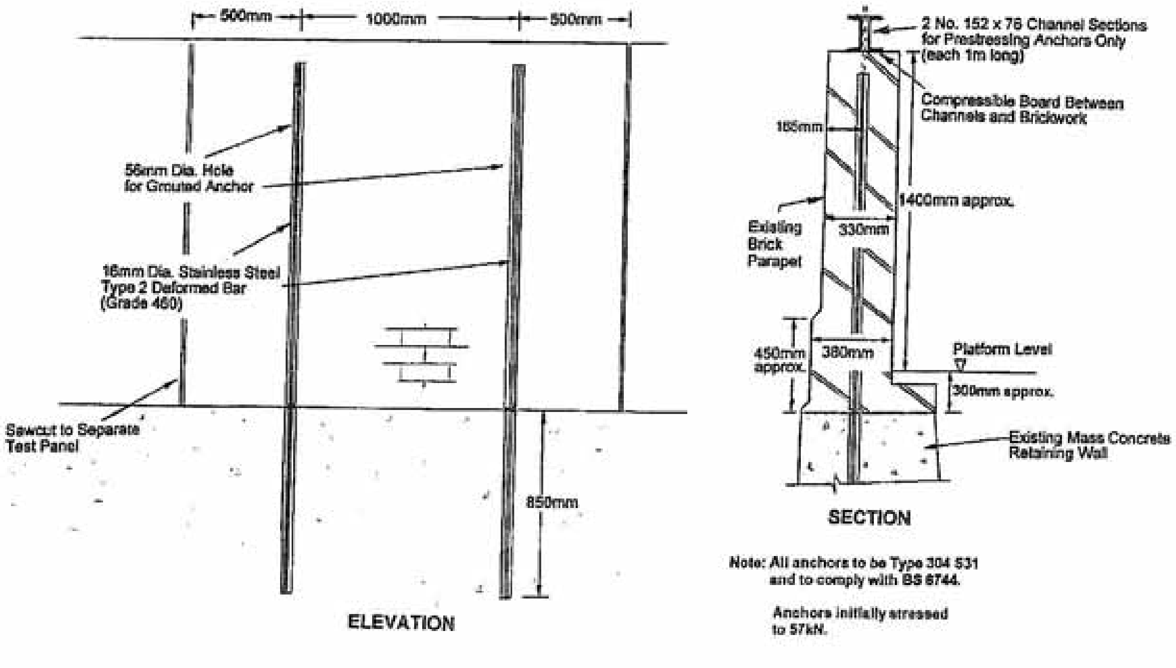
SUPPLEMENTARY LOAD TEST

A load test was carried out to confirm that the performance of the strengthened wall had been correctly calculated and provide assurance on the method. It was debated beforehand whether to do the test in the laboratory or in situ. A laboratory test has the merit that it can be accurately controlled and loaded to collapse so that the full non-linear load-deflection relationship is recorded. It is not affected by bad weather and a laboratory environment is conducive to good workmanship. On the other hand it was felt that a test on a freshly constructed model would be unlikely to reproduce the true conditions presented by a deteriorated wall containing weakened mortar and weathered bricks. It was therefore decided to test an existing wall in situ and accept the various shortfalls.

The load test was carried out according to the guidelines published by the National Steering Committee for the Load Testing of Bridges [1]. Although written for bridge testing, the principles of the guidelines are fundamental and generally applicable to other structures. The guidelines define three types of load testing; supplementary, proof and proving tests. In this investigation the load tests were supplementary and, as the name implies, were planned to supplement the structural analysis. The level of loading was to be sufficient to produce measurable responses from the structure without causing any permanent damage.

The available space and access constrained the load test to being as simple as possible. The supplementary load test was undertaken on a section of wall identified as being understrength for wind loading. The brick wall was believed to be constructed from London stocks with a lime mortar and the strengthening scheme was therefore designed assuming a characteristic compressive strength of the masonry of 2.3N/mm^2. Built in English bond, the wall was supported on a mass concrete retaining wall. For the purposes of the test a 2m panel was separated from the rest of the wall by vertically saw cutting the parapet down to the top of the retaining wall.

Details of the wall together with the strengthening scheme using two 16mm diameter Cintec anchors are shown in Figure 3.



Note: All anchors to be Type 304 S31 and to comply with BS 6744.

Anchors initially stressed to 57kN.

Figure 3 Details of Strengthening Scheme for Test Panel

The vertical Cintec anchors were each tensioned up to 57kN and, during jacking, the behaviour of the wall and anchors was monitored, see Figure 4. The elongation of the anchor between the jack and the anchored end in the retaining wall was as expected. However, the wall itself was also found to compress by about 4mm at the top which was significantly greater than the 0.2mm expected. The wall was also found to deflect by about 4mm towards the platform.

Figure 4 Post-tensioning Anchor in Test Panel

For the supplementary load test, the applied wind load was simulated by the application of a lateral point load on a horizontal spreader beam positioned vertically at the centroid of the wind pressure. The lateral load was applied with a hydraulic jack pushing against a jacking frame anchored to the retaining wall supporting the parapet, see Figure 5.

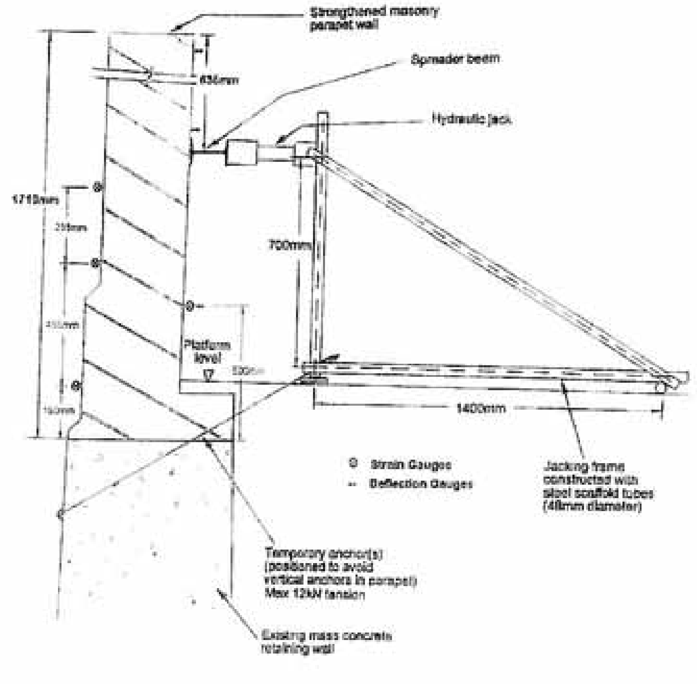
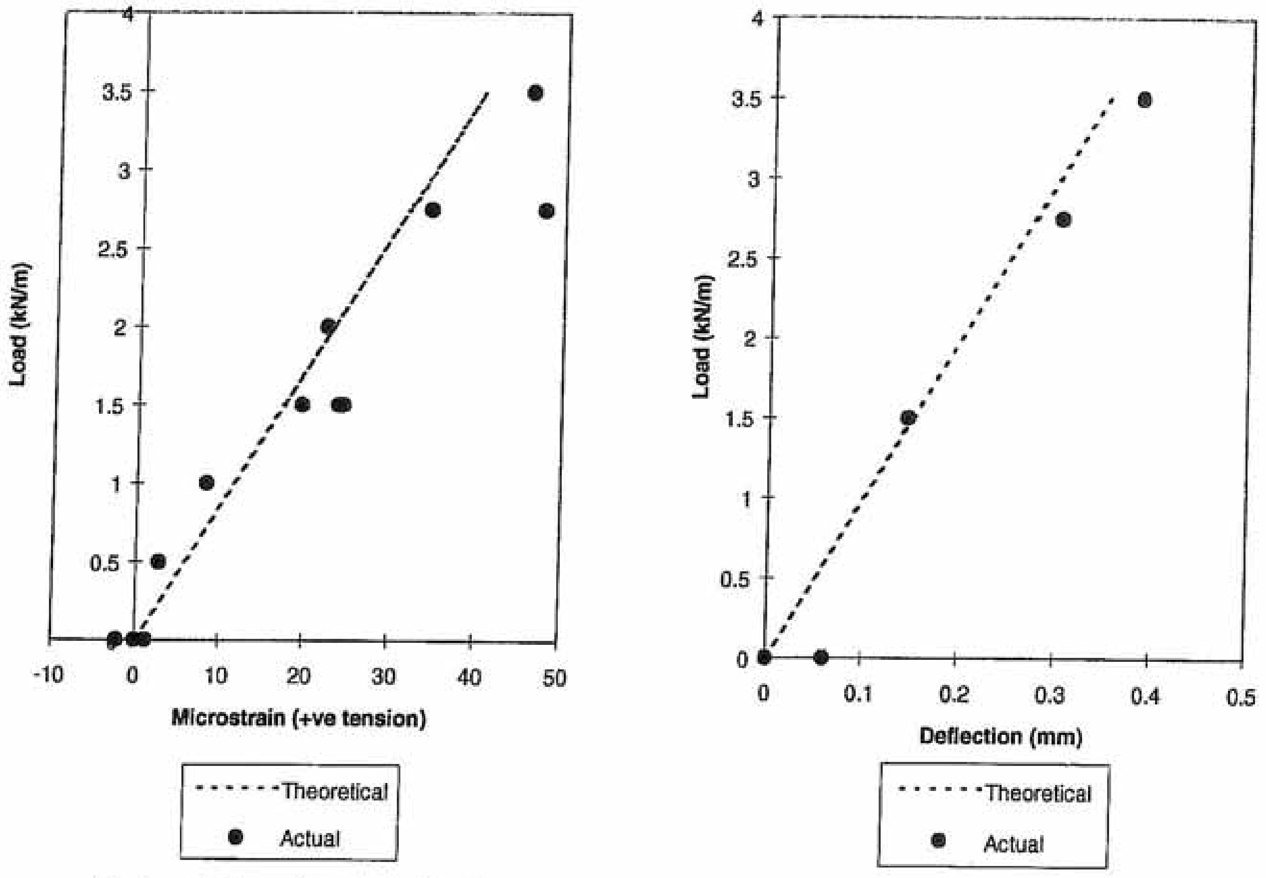


Figure 5 Testing Arrangement

The behaviour of the test panel under lateral loading was monitored using twelve 5½ inch strain gauges located on both the tensile and compressive faces and six dial gauges on the tensile face. The test was undertaken with an incremental increase in applied lateral load up to 3.5kN/m, equivalent to 1.6 x nominal wind pressure. The maximum deflection at the top of the wall was 0.38mm while the maximum tensile strain at a position 500mm above the base of the wall was 48 micro strain.

The test results are plotted in Figure 6 against the predicted values which were based on a characteristic masonry strength f_k of 2.3N/mm2 and an elastic modulus E of 2.07kN/mm^2 (=0.9f_k). The measured results identified that the loading was not uniform across the full panel so, for comparison with predicted values, weighted averages were determined in proportion to the tested area. The measured results demonstrate linear behaviour but give greater strains and deflections than those predicted. This suggests that the masonry strength and/or stiffness assumed for the strengthening calculations were too high. Nevertheless, considering the possible variation of material parameters for old masonry walls, the measured results match the predicted very well.



Strains at 500mm from base of wall

Deflections at top of wall

Figure 6 Comparison of Test Results against Predicted

At the end of the test there was no evidence that the loading had caused any damage such as cracking or spalling.

CONCLUSIONS

The 2m long test panel of 100 year old parapet wall was post-tensioned against wind and dynamic pressure and suction loading, using two Cintec anchors. The supplementary load test with an applied load up to 3.5kN/m (1.6 x nominal loads) demonstrated a linear elastic response.

The predicted response of the strengthened wall, calculated beforehand and based on assumed values for the material properties, were within 30% of the measured values. Bearing in mind the wide range of uncertainties in relation to the wall stiffness and strength, this is surprisingly close. On completion of the test there was no damage such as cracking or spalling.

It is concluded that the supplementary load test was successful in demonstrating the efficacy of strengthening an old brick wall in a poor state of repair. The strengthening scheme presented is an economic and aesthetic solution to the refurbishment of understrength and unstable masonry walls and parapets.

ACKNOWLEDGEMENTS

The authors would like to thank Dr S Mehrkar-Asl for his assistance during the supplementary load tests and analysis of the results.

REFERENCES

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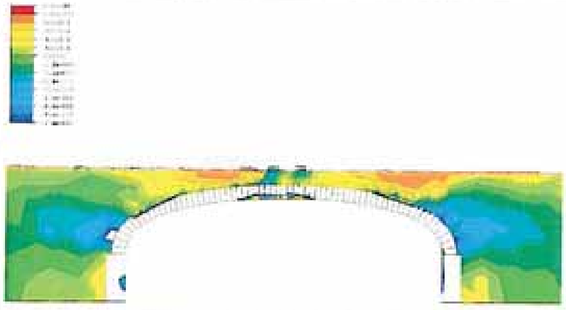
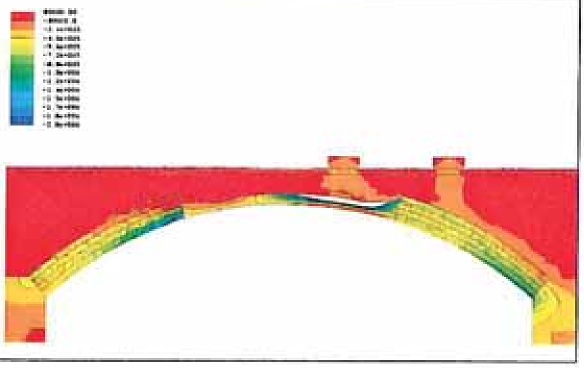
[1] ICE National Steering Committee for the Load Testing of Bridges '*Supplementary Load Testing of Bridges*'. Thomas Telford 1998.

ELFEN
Discrete Element Analysis

Bridge Name:	**ROMAN RIDGE BRIDGE**	Structure No:	HUL4/14
Client:	Bridgeguard 3 (York)	Territory:	London North Eastern
Location:	Garforth, West Yorkshire	ELR:	11m 59ch
Contract:	G400/258	Service:	Special Assessment
Completed:	2003	Software:	ELFEN
Structure Type:	Single span flat elliptical masonry arch	Outcome:	Rating increased from failing Section 117 (BE4) to 40/44 tonnes ALL

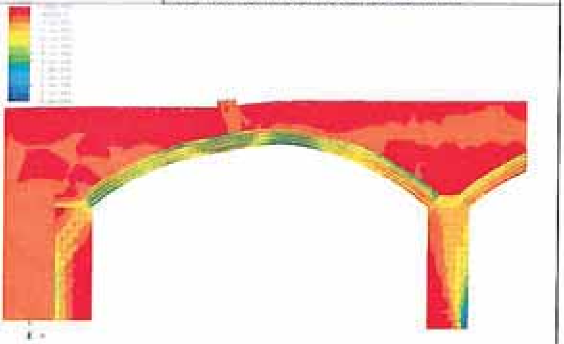


Bridge Name:	**HOPPERS ROAD BRIDGE**	Structure No:	HDB/10
Client:	Bridgeguard 3 (York)	Territory:	London North Eastern
Location:	Enfield, London	ELR:	7m 42ch
Contract:	G400/258	Service:	Special Assessment
Completed:	2003	Software:	ELFEN, DISPLAY3/NISA2
Structure Type:	Single span skew masonry arch with cast iron crown plate	Outcome:	Rating increased from 3 tonnes to 40/44 tonnes ALL



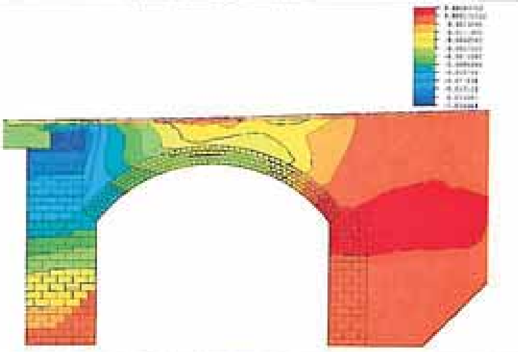
Bridge Name:	**LLANHARAN BRIDGE**	Structure No:	SWM2
Client:	Glamorgan Engineering Consultancy	Territory:	Unknown
Location:	Llahharan, Rhondda Cynon Taff	ELR:	183m 65ch
Contract:	N/A	Service:	Special Assessment
Completed:	2004	Software:	ELFEN
Structure Type:	Single span masonry arch with intermediate supports	Outcome:	Rating increased from 17 tonnes to 40/44 tonnes ALL

Bridge Name:	**SIDE BRIDGE**	Structure No:	CHR/109A
Client:	Bridgeguard 3 (York)	Territory:	London North Eastern
Location:	Renishaw, Derbyshire	ELR:	152m 44ch
Contract:	G400/258	Service:	Special Assessment
Completed:	2003	Software:	ELFEN
Structure Type:	Two span multi-ring brick masonry arch	Outcome:	Rating increased from 10 tonnes to 13 tonnes restricted ALL

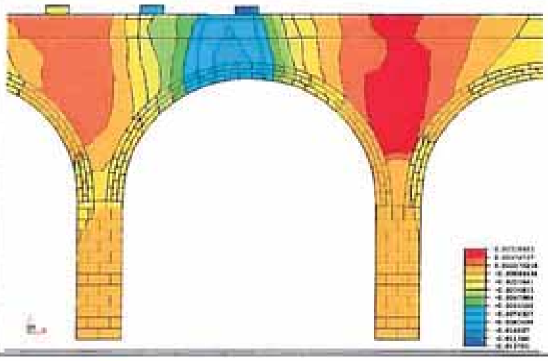


Bridge Name:	**WOODTHORPE BRIDGE**	Structure No:	BAC3/17
Client:	Bridgeguard 3 (York)	Territory:	London North Eastern
Location:	Woodthorpe, Derbyshire	ELR:	153m 75ch
Contract:	G400/258	Service:	Special Assessment
Completed:	2004	Software:	ELFEN
Structure Type:	Three span multi-ring brick masonry arch	Outcome:	Rating increased from 17 tonnes to 40/44 tonnes ALL

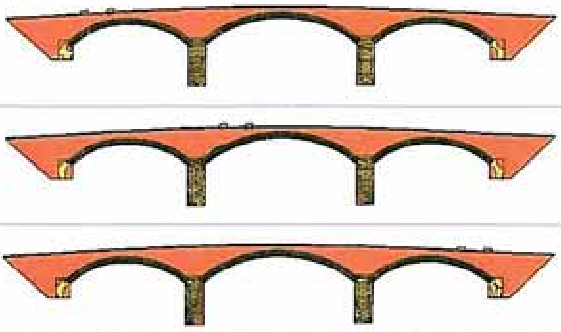
Bridge Name:	**CROUCH HILL BRIDGE**	Structure No:	TAH1/14
Client:	Bridgeguard 3 (York)	Territory:	London North Eastern
Location:	Islington, London	ELR:	3m 16ch
Contract:	G400/258	Service:	Special Assessment
Duration:	ongoing	Software:	ELFEN
Structure Type:	Three span brick masonry arch with centre arch replaced by pre-cast concrete beams.	Outcome:	Rating increased from 7.5 tonnes to 40/44 tonnes ALL dependant on findings of recommended site investigation



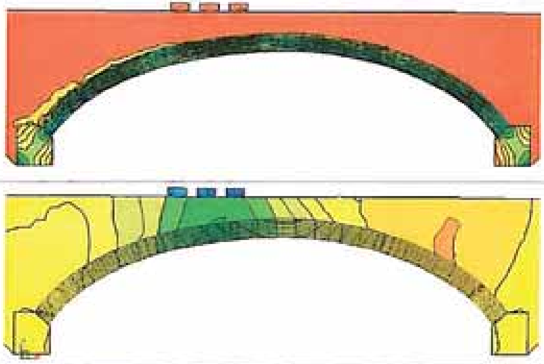
Bridge Name:	**CHILDERS STREET VIADUCT**	Structure No:	NKL/275
Client:	Network Rail Infrastructure Ltd.	Territory:	Unknown
Location:	Deptford, London	ELR:	4m 778yds
Contract:	BB98/322	Service:	Special Assessment
Completed:	2003	Software:	ELFEN
Structure Type:	Fire damaged multi-span brick masonry viaduct	Outcome:	Safe Ultimate Load Capacity of RA6



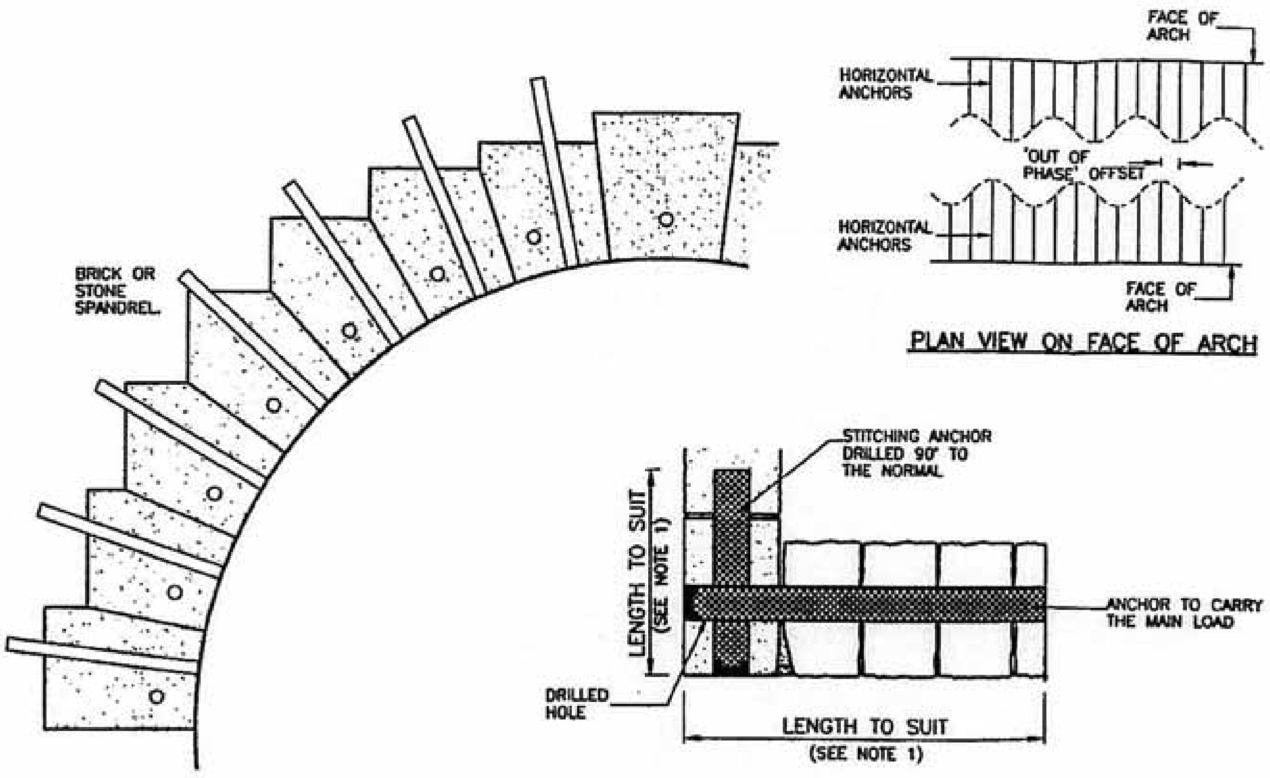
Bridge Name:	**HOLLINHILL LANE BRIDGE**	Structure No::	BAC3/28
Client:	Bridgeguard 3 / May Gurney	Territory:	Unknown
Location:	Clowne, Derbyshire	ELR:	Unknown
Contract:	Unknown	Service:	Strengthening Feasibility
Completed:	2002	Software:	ELFEN
Structure Type:	Three span multi-ring brick masonry arch in poor condition	Outcome:	25 tonne restricted ALL and recommendations for Archtec strengthening



Bridge Name:	**SWAINS PARK ROAD BRIDGE**	Structure No:	KSL/64
Client:	Bridgeguard 3 (York)	Territory:	London North Eastern
Location:	Swadlincote, Derbyshire	ELR:	Unknown
Contract:	Unknown	Service:	Special Assessment
Duration:	2001	Software:	ELFEN
Structure Type:	Single span multi-ring brick masonry arch in poor condition	Outcome:	25 tonnes restricted ALL and recommendations for Archtec strengthening – bridge replaced



Two engineering examples using the Cintec system



PLAN VIEW ON FACE OF ARCH

ELEVATION ON ARCH.

TYPICAL SECTION

STITCHING SYSTEM FOR LONGITUDINAL CRACK BETWEEN VOUSSOIRS AND ARCH BARREL.
(CINTEC METHOD)

NOTES

1. CONTRACTOR TO DECIDE ON LENGTH OF STITCHING BAR HOLES, RELATIVE TO THE POSITION OF THE CRACK, AFTER CONSULTATION WITH THE MANUFACTURER OF THE RELEVANT PROPRIETARY SYSTEM (NB LENGTH OF ADJACENT ANCHORS TO BE VARIED TO PREVENT A NEW LONGITUDINAL CRACK FORMING, BY PRODUCING AN ELONGATED 'S'-SHAPED LINE ON PLAN, AS SHOWN)

2. 'OUT OF PHASE' OFFSET TO BE PROPOSED BY THE CONTRACTOR.

3. ANCHOR GROUT

TO BE A MIXTURE OF CEMENT, GRADED AGGREGATE, FLOW AND ANTI-SHRINK ADDITIVES GIVING A PUMPABLE GROUT WITH AN INITIAL SETTING TIME OF 160 MINUTES (FINAL SET OF 180 MINUTES). GROUT TO HAVE A TYPICAL STRENGTH OF 40N/mm² AT 28 DAYS. (23N/mm² AT 1 DAY).

4. STEEL SECTION

THE STEEL SECTION IS TO BE A CIRCULAR HOLLOW SECTION/SQUARE HOLLOW SECTION/ DEFORMED BAR MADE FROM HIGH YIELD STAINLESS STEEL (GRADE 460) COMPLYING WITH BS6744 IN STEEL TYPES 304S31 OR 316S33.

5. FABRIC SOCK/SLEEVE

SOCK FABRIC TO BE POLYESTER BASED AND MANUFACTURED IN SIZES FROM 20 TO 300mm IN DIAMETER. MESH SIZE AND AMOUNT OF EXPANSION OF SOCK TO BE VARIED ACCORDING TO APPLICATION.

6. POINTING MORTAR

MORTAR FOR POINTING FACE OF CORE INTO DRILLED ANCHOR HOLE TO BE ONE OF THE FOLLOWING MIXTURES:-

1 : ¼ : 4 CEMENT/LIME/SAND WITH THE ADDITION OF AN APPROVED AIR ENTRAINING ADMIXTURE TO ENHANCE FROST RESISTANCE (N.B. CONTRACTOR TO CHECK WITH ENGINEER TO CONFIRM REQUIREMENTS).

7. INSTALLATION

a) THE CONTRACTOR SHALL SUBMIT A SCHEME DRAWING SHOWING THE LOCATION / EXTENT OF GENERAL REPAIRS & DETAILS OF HIS TEMPORARY WORKS, SUPPORTING CALCULATIONS & FORM 'B', AS APPROPRIATE.

b) CONTRACTOR TO SUBMIT TO THE ENGINEER THE NUMBER AND POSITION OF THE STITCHING/ANCHOR BARS REQUIRED AFTER CONSULTATION WITH THE SPECIALIST SUPPLIER MANUFACTURER. DEPENDING ON THE POSITION OF LONGITUDINAL CRACKS, CERTAIN STRUCTURES MAY REQUIRE THE OPTION OF A NUMBER OF STITCHING BARS TO BE DRILLED COMPLETELY THROUGH THE STRUCTURE/DRILLED FROM EACH FACE AND OVERLAPPED IN A CRACK-FREE ZONE.

c) ANCHOR POSITIONS TO BE MARKED OUT AND HOLES DRILLED USING DIAMOND CORING/ ROTARY/ROCK DRILLING EQUIPMENT. FOR STRUCTURES WHERE AESTHETIC CONSIDERATIONS HAVE A HIGH PRIORITY, DIAMOND CORING IS RECOMMENDED. CORES TO BE RETAINED (SEE 7h). FOR ALL HOLES AT AN ANGLE TO THE HORIZONTAL, A JIG SHALL BE USED. HOLE DIAMETER TO BE DRILLED WITH A TOLERANCE OF 0 TO +5%.

d) HOLES TO BE FLUSHED CLEAR OF ALL DUST AND DEBRIS, AND ANY SHARP PROTRUSIONS WHICH COULD DAMAGE THE FABRIC SOCKS ARE TO BE REMOVED.

e) THE FABRIC SOCK ON THE ANCHOR IS TO BE CHECKED FOR DAMAGE PRIOR TO PLACING IN THE DRILLED HOLE. DAMAGED SLEEVES AND/OR ANCHORS ARE NOT TO BE USED/TO BE REPAIRED.

f) GROUT TO BE INJECTED INTO THE ANCHOR AT A PRESSURE RECOMMENDED BY THE SUPPLIER/MANUFACTURER. (THE ANCHOR IS TO BE CLASSED AS COMPLETELY INFLATED WHEN "MILK" GROUT APPEARS THROUGH THE FRONT OF THE SLEEVE AND THE SLEEVE CANNOT BE COMPRESSED).

g) CLEAN UP SURPLUS GROUT FROM DRILLED HOLE AND FACE OF STRUCTURE.

h) DRILLED HOLE TO BE BLOCKED OFF BY REPLACING THE END SECTION OF THE CORE AND MORTARING UP TO MATCH THE EXISTING STRUCTURE.

* CRACK TO BE POINTED UP & GROUTED ON COMPLETION.

8. THE CONTRACTOR IS AT LIBERTY TO PROPOSE ALTERNATIVE METHODS AND / OR MATERIALS FOR CARRYING OUT THE WORKS PROVIDED THAT THE ALTERNATIVE PROPOSALS ARE SUBMITTED ALONG WITH THE TENDER FOR CONSIDERATION / APPROVAL BY THE ENGINEER.

Letter	Date	Descriptions of revisions	Drawn	Approved

British Steel

CEDG York Ltd.
Consulting Engineers & Quantity Surveyors
Hudson House
York YO1 1HP
Tel: 01904 523366

Approved
Date 30.05.96

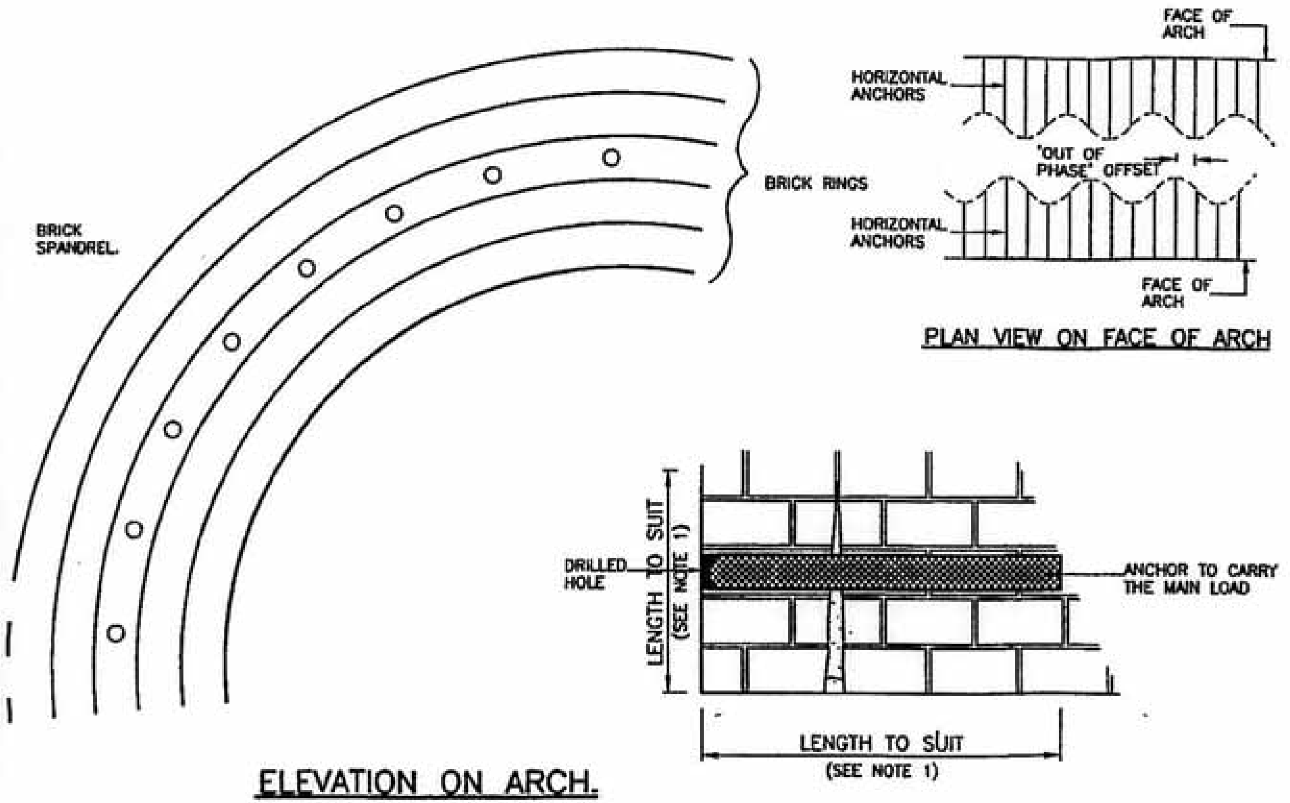
RAILTRACK
LONDON NORTH EASTERN
2nd Floor
Prudential House
Blossom Street
York YO2 2AJ
Tel.: 01904 525618
Fax: 01904 525607

Zone Structures Engineer
Date 10th June 96

Line
E.L.R.
U/O Bridge No
Mileage
Between/At
O.S. Grid Ref

Description BRICK & MASONRY REPAIRS
STITCHING OF LONGITUDINAL CRACK BETWEEN VOUSSOIR/BRICK SPANDREL & ARCH BARREL

Scales N.T.S.
File Number
Plan Room Reference
Drawing Number 10436-015
Revision



ELEVATION ON ARCH.

PLAN VIEW ON FACE OF ARCH

TYPICAL SECTION

STITCHING SYSTEM FOR LONGITUDINAL CRACK IN BRICK ARCH BARREL. (CINTEC METHOD)

NOTES

1. CONTRACTOR TO DECIDE ON LENGTH OF STITCHING BAR HOLES, RELATIVE TO THE POSITION OF THE CRACK, AFTER CONSULTATION WITH THE MANUFACTURER OF THE RELEVANT PROPRIETORY SYSTEM (NB LENGTH OF ADJACENT ANCHORS TO BE VARIED TO PREVENT A NEW LONGITUDINAL CRACK FORMING, BY PRODUCING AN ELONGATED 'S'-SHAPED LINE ON PLAN, AS SHOWN)

2. 'OUT OF PHASE' OFFSET TO BE PROPOSED BY THE CONTRACTOR.

3. **ANCHOR GROUT**

TO BE A MIXTURE OF CEMENT, GRADED AGGREGATE, FLOW AND ANTI-SHRINK ADDITIVES GIVING A PUMPABLE GROUT WITH AN INITIAL SETTING TIME OF 160 MINUTES (FINAL SET OF 180 MINUTES). GROUT TO HAVE A TYPICAL STRENGTH OF 40N/mm^2 AT 28 DAYS. (23N/mm AT 1 DAY).

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Letter	Date	Descriptions of revisions	Drawn	Approved

British Steel

CEDG York Ltd.
Consulting Engineers & Quantity Surveyors
Hudson House
York YO1 1HP
Tel: 01904 523366

Approved
Date 30.05.96

RAILTRACK
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York YO2 2AJ
Tel.: 01904 525618
Fax: 01904 525607

Alistair Wood
Zone Structures Engineer
10 th June 96
Date

Line
E.L.R.
U/D Bridge No
Mileage
Between/At
O.S. Grid Ref
Description BRICK & MASONRY REPAIRS LONGITUDINAL STITCHING SYSTEM FOR BRICK ARCH BARREL (PROPRIETARY)
Scales N.T.S.
File Number
Plan Room Reference
Drawing Number 10436-016
Revision

List of Cintec refurbishment rail projects during the period 1988 to 2002

Cintec Rail Refurbishment Projects
from 1988 to 2002

Project Name	Project Completion Date
KENNET BRIDGE READING	19/04/1988
SCOT RAIL VIADUCT	19/05/1988
FRAMPTON MANSEL VIADUCT	15/08/1988
KINGSCROSS UNDERGROUND	06/09/1988
WHITESHAPE VIADUCT	30/06/1989
LEWISHAM STATION	13/11/1989
BRIDGE NO 67 BR	28/02/1992
PETERBOROUGH VIADUCT	28/10/1992
O/D 6475817LD WORCHESTER	26/11/1992
GUIDE BRIDGE REDDISH	22/02/1993
BESTHILL BRIDGE	15/03/1993
SPATEFORD VIADUCT	18/03/1993
BRIDGES NO 6 & 29 WINDERMERE	27/05/1993
LIVERPOOL STATION	30/07/1993
O/N 6427004 ID CARLISLE	31/08/1993
O/N 6494656MD NORTHAMPTON	15/10/1993
O/N 6495052MD CARLISLE	05/11/1993
PRESTON BRIDGE	31/01/1994
WORCHESTER BRIDGE	24/03/1994
HITCHEN BRIDGE	31/03/1994
FENCHURCH STREET STATION	25/05/1994
BRIDGE 10 HECK	20/06/1994
BRIDGE PANDY	20/06/1994
WESTMINSTER TUNNEL	24/06/1994
LEADERFOOT VIADUCT	04/08/1994
ROYAL BORDER VIADUCT	21/08/1994
BERWICK ON TWEED VIADUCT	27/09/1994
BRUNSWICK WALL	15/12/1994
6699305MD	10/02/1995
6699163MD	15/02/1995
PATCHWAY COMPOUND	16/02/1995
669925OME	21/02/1995
66516263D	10/03/1995
6699826JD	15/03/1995
6699866KD	23/03/1995
6698344LD	23/03/1995
6661026LE	30/03/1995
BRIDGE 70 TRENT VALLEY LINE	18/09/1995
ROYAL BORDER BRIDGE	02/02/1996
WORCHESTER VIADUCT	13/10/1997
3 BRIDGES VARIOUS PLACES	31/01/1998
HOLMSLEY OVERBRIDGE	28/02/1998
CHELMSFORD VIADUCT	28/02/1998

BRAFFERTON BRD DARLINGTON	30/03/1998
DOVE HOLES BUXTON DERBY	31/03/1998
WARRINGTON VIADUCT SPAN 123	30/05/1998
BRIDGE 2C PRESTON/BLACKBURN LINE	30/06/1998
BRIDGE SAC 197 CROSBY GARRETT CUMBRIA	30/08/1998
DISTRICT LINE EAST PUTNEY STATION	30/09/1998
SOMERTON VIADUCT SOMERSET	30/09/1998
PONTYPRIDD STATION	30/11/1998
TEVIOT VIADUCT ROXBURGH	30/12/1998
BRENT CROSS TO GOLDERS GREEN LDN	30/12/1998
SHADWELL SHAFT	30/01/1999
THROPP BRIDGE TEMPLECOMBE	30/03/1999
SUSSEX PARK STATION D72 LDN	30/03/1999
RAVENS COURT STATION D79C LDN	30/03/1999
BIRKSLAND ST BRIDGE DUH/43 BRADFORD	31/03/1999
OUSE VALLEY VIADUCT	30/05/1999
BRIDGE NO 7BARNSLEY SOUTH YORK	30/06/1999
PUTNEY BRIDGE STATION D93C/93B LDN	30/08/1999
FESTINIOGG RAILWAY EMBANKMENT R/WALL	03/11/1999
BRIDGE 4 MAWN GREEN CREWE/STOCKPORT	30/11/1999
CHORLTON BRIDGE NO.72 CREWE-STAFFORD LINE	30/03/2000
RUNCORN VIADUCT	30/04/2000
BRIDGE 62 OXENHOLME NR. KENDAL CUMBRIA	30/05/2000
SHANKEND VIADUCT	30/06/2000
COLUMB JOHN OVERBRIDGE PADDLEFORD	30/08/2000
HARRINGWORTH VIADUCT RUTLAND LEICS	30/08/2000
RAIL BRIDGE SHARPNESS GLOUCESTERSHIRE	30/09/2000
GREEK STREET BRIDGE STOCKPORT	30/04/2001
HARTHORPE VIADUCT NORTH OF BEATTOCK	30/04/2001
ROY 26 HORBURY NR. WAKEFIELD - CRIGGLESTON	31/12/2001
LLANGOLLEN	31/03/2002
CORK TUNNEL IRELAND	07/08/2002
FARROW HOUSE LONDON	07/08/2002
BURNTON VIADUCT AYRSHIRE	24/09/2002
VIADUCT 91 STOKE ON TRENT	10/12/2002

BRE Report

Anchor Age Testing
Moisture / Temperature
Cycling Tests

Technical Consultancy

CLIENT REPORT:

Moisture/temperature cycling tests on Cintec Harke remedial wall tie

for: Cavity Lock Systems Ltd.,
Factory Road, Newport, Gwent NP9 5FA

by S K Arora

November 1990

Building Research Establishment

Department of the Environment

NTRODUCTION

This report gives results of pull-out tests on Cintec Harke remedial tie embedded in a clay brick, having been subjected to accelerated moisture/temperature cycling over a period of three months. The object of the exercise was to test the long term performance of the tie anchors under conditions of wetting by rain of the external walls of a structure into which they would be incorporated followed by drying.

THE ANCHOR SYSTEM

The literature supplied by the manufacturers of the system, Messrs Cavity Lock Systems Ltd. of Newport, Gwent, describes Cintec-Harke replacement wall tie as a cementitious anchor. The standard design is a long stainless steel hollow tube of 8mm O.D.[1] x 1mm thickness provided with a mesh polyester fabric sleeve or a 'sock' of required diameter at each end. A specially designed cementitious grout is injected into the socks through the tie under pressure in predrilled position(s) in the cavity wall requiring replacement tie(s). The pressure is maintained until the inflated socks are hard and the grout milk with bonding agents are driven out to give good bond between the inflated sock and the background material. The grout is a Presstec or S.T.M.A. grout[1].

EXPERIMENTAL DETAILS

The anchor used in the pull-out tests was a special design of 165-175mm long 8mm O.D. x 1mm stainless steel hollow tube, with an 85mm long sock provided at one end only which would inflate to a diameter of approximately 22mm. The background material chosen for the test specimens was a flat faced solid wire-cut facing clay brick of 212mm x 100mm x 65mm size. The anchor sock was embedded through one of the 212mm x 65mm faces to its full depth, with the steel tube coming out through the other face. Three spare specimens were also prepared with the anchor sock embedded to a lesser depth of around 60mm, with the remaining part providing a bulge of anchor material into a reamed out hole of 40mm diameter. This was done to test a situation where a positive re-entrant tension fixing is to be provided in a wall, in case the grout to brick bond fails.

The specimens made with the said brick supplied by BRE were prepared by the manufacturers at their own premises and delivered to BRE three days later.

The test programme assumed that a masonry wall in reality would be exposed to rain such as to saturate it fully with water at least once a year. Trials were made to ensure wetting of the brick in a water tank to saturation followed by drying in an electric oven heated to 40°C(±2°C) temperature, to a constant weight. A half hour soak in a water tank followed by a minimum of two days of drying was found sufficient to meet the requirements.

The BRE contract stipulated 20 pull-out tests on brick/anchor specimens, five each to be tested at: seven days cure after construction of the specimens, and then after 10, 20 and 40 cycles of wetting/drying of the specimens. A further three specimens of 60mm embedment length referred above were also tested after 40 wetting/drying cycles.

The pull-out testing was carried out on a standard Universal Testing machine with a maximum load capacity of 20 Tonnes, calibrated to BS 1610: 1985 Grade 2. The test brick was placed in a small restraining rig made out of a rectangular hollow steel section designed to hold the brick firmly along its 'anchored' face. A side load of about 3.5 N/mm^2 pressure was applied on the bed faces to simulate condition of confinement of the brick in a real wall. Vertical restraint was provided by small wedge strips keeping the top surface of the brick tightly parallel against the upper part of the frame.

TEST RESULTS

Clay brick

For the clay brick used, trial tests indicated a water absorption after a 1/2 hour soak of 15.0%, which approximates the full saturation value after a 24 hour soak of 17.5% for the same brick. Its compressive strength was indicated to be 43.3 N/mm^2.

Brick/anchor specimens

The pull-out values obtained in the 20 standard and three extra tests carried out are tabulated below.

Tie Pull-out values in KN

Specimen No.	After 7 days cure	Number of wetting/drying cycles		
		10	20	40
1	10.45	7.56	10.45	9.10 (9.79)
2	12.23	10.23	10.23	11.00 (6.23)
3	10.68	8.45	10.23	10.00 (8.01)
4	10.45	10.68	10.90	12.90
5	10.90	10.68	8.45	9.79
Mean	10.94	9.52	10.10	10.56 (8.01)
c.o.v. %	7.00	15.00	9.00	14.00 (22.00)

Note:- The bracketed values are for the three extra tests involving anchors of the limited embedment length of 60 mm.

A one way analysis of variance of the tabulated values for the 20 standard tests has shown that the wetting/drying treatment given did not affect the pull-out performance of the tie in the background material tested in any significant way. Mean pull-out value for these specimens was 10.28 KN. Regression analysis of the data (for a linear as well as polynomial fits) further confirmed a lack of a significant correlation between the pull-out performance and the wetting/drying treatment given.

The failure of the system tested was typically by a pull-out of the steel tube from the anchor grout (Figure 1), sometimes accompanied by splitting of the brick in the plane of the anchor.

As to the three extra specimens, the mean pull-out value of 8.01 KN, when compared with the corresponding value given for the standard specimens, suggests that the apparent deterioration in performance was only due to the reduced length of embedment of the anchor. The failure here was typically by a rupture of the anchor grout at the interface between the embedded part to the bulging part, accompanied by a pull-out of the steel tube again (Figure 2).

CONCLUSIONS

1. The experiments show that the pull-out performance of the test anchor/clay brick combination tested would not be affected adversely in any significant way in the conditions of exposure to rain simulated in the manner described.
2. The failure of the standard specimens was typically by pull-out of the steel tube from the anchor grout.
3. The pull-out performance of the anchor/brick system tested appears to be directly proportional to the length of embedment of the anchored sock.

REFERENCE

1. Private communication, Mr Owen/Mr James, Messrs Cavity Lock Systems, Factory Road, Newport, Gwent.