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High capacity O-cell pile load testing in Coal Measures Mudstone, Northern Spire Bridge, Sunderland U.K.

Haute capacité O-cellule test de charge de pieu dans les mesures de charbon mudstone, le Pont Northern Spire, Sunderland U.K.

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ABSTRACT: Sunderland's Northern Spire cable-stayed bridge is the first crossing of the Wear to be constructed in over 40 years. The steel pylon was prefabricated in Belgium, transported to the site by barge and erected to rest upon a large foundation base in the river constructed within a cofferdam. The pylon base is supported by ten 1.5m diameter, bored piles, each pile rock socketed into Coal Measures mudstone. The paper describes the geology and ground conditions at the bridge site, rationale for selection of the foundation type and detailed design of the foundation piles, including 14.5m long, 25MN design load rock sockets. Details of the test pile construction, 60 MN axial load capacity O-cell test layout, location and instrumentation arrangement to confirm the adequacy of design are also described. The design of the multilevel O-cell test pile assembly, instrumentation, testing sequence and summary of the results are provided. Interpretation of the data allowed an estimated prediction of the behaviour of the test pile and confirmed the design expectations for the working piles.

RÉSUMÉ: Le Pont Northern á Sunderland est un pont á haubans est le premier pont construit qui traverse la rivière depuis 40 ans. Le pilon,en acier était préfabriqué en Belgique, transporté en barge au chantier. La base du pilon est soutenu par 10 pieux forés de 1.5m de diamètre, chaque pieu ancré sur roche dans les mesures de charbon mudstone. L'article décrit la geologie et l'état du sol au chantier du pont. La logique pour la sélection du type et la conception détaillé des pieux de fondation comprennent 25MN d' ancres sur roche de 14.5m de long. Les détails de la construction des pieux de test, 60MN de charge axial capacité O-cellule test schéma, l'emplacement et l'organisation de l'instrumentation pour confirmer la compétence de la conception est aussi décrite. L'assemblage du design du multi-niveau O-cellule pieu de test, l'instrumentation, la séquence des tests et un sommaire des résultats est fourni. L'interprétation des données permettaient une prédiction estimée du comportement du fonctionnement des pieux pour confirmer les provisions de la conception pour les pieux de test et les pieux de fonctions.

Keywords: bored piles; Mudstone; static load test; multi-level O-cell; instrumentation.

1 INTRODUCTION

The Northern Spire Bridge in Sunderland is one of England's biggest civil engineering projects in recent years. Construction began in May 2015 and the official opening of the bridge was in August 2018. The 336m long cable-stay bridge is the first to have been constructed over the River Wear in over 40 years and it opens up large areas of land for regeneration. It comprises a 24m-wide deck, supported by a 1,550 tonne, 105m-high A-frame pylon. Built at a cost of £117.6m, it is estimated that the new bridge will carry 27,000 vehicles daily linking the A19 with Sunderland City Centre and the Port of Sunderland.

A highly innovative and unique erection sequence was developed for the bridge construction. This required pre-assembly and horizontal launch of the bridge deck in two phases from the southern approach, using temporary supports in the river. The main bridge pylon was fabricated in Belgium and transported via barge to the site. The pylon was erected vertically to rest upon its substructure and foundations using a temporary hinge and multiple temporary ground anchors for reaction (Figure 1). Once this critical phase was completed, the bridge deck was launched through the A frame pylon structure into its final position and permanently connected to the pylon via 14 pairs of cable stays.

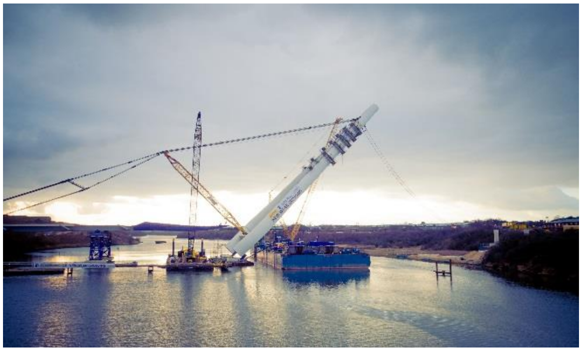

Figure 1. Bridge pylon during erection, February 2017.

2 SITE CONDITIONS

The bridge site was investigated using a range of investigative techniques including land and marine cable percussion and rotary cored boreholes, in situ pressuremeter testing, cone penetrations tests, vibrocores and grab sampling.

In general the following sequence of soils and bedrock strata were encountered beneath the bridge site (Figure 2).:

- Made Ground;
- Drift including soft alluvial silts and clays, loose alluvial sands, stiff glacial clays, medium dense glacial sands;
- Dense Permian Sands;
- Carboniferous Upper Coal Measures formation mudstones and sandstones.

At the main pylon location soft aluvium was directly underlain by Coal Measures mudstone.

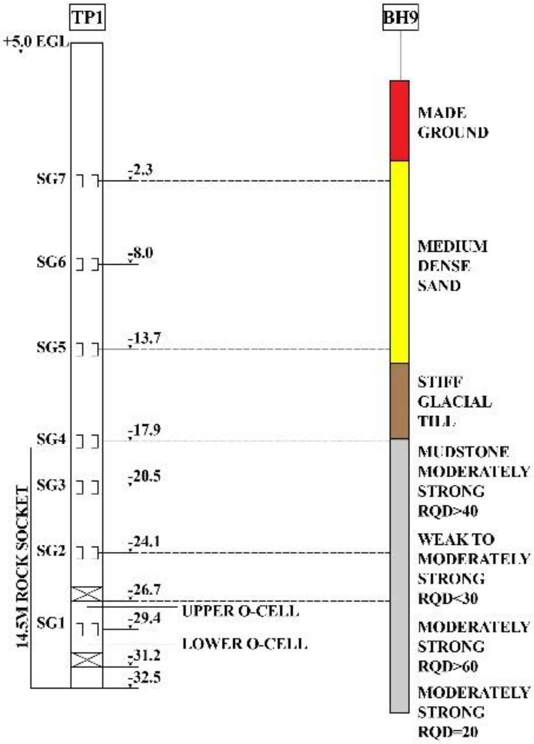

Figure 2. Geology & Instrumentaion at Test Pile

Table 1 Summary of Ground Model and design parameters at bridge pylon.

Location and levels (m. O.D.)		**Strata**	**Unit Weight**	**Shear strength $c_{u,k}$, c'_k or $q_{uc,k}$**	**Angle of effective stress, $\O'_k$**	**Soil or Rock Mass Modulus, $E_{m,k}$**
Upper	Lower		kN/m^3	(kPa) or *(MPa)*	(°)	(MPa)
-2.94 to -13.62	-2.57 to -16.64	Very soft CLAY & loose to very loose SAND (Alluvium)	18.0	15 to 20 kPa	27	4
-13.62 to -16.64	-16.77 to -19.54	Very weak to weak Weathered Mudstone (Coal Measures)	20.0	200 kPa 5 kPa *5 MPa*	27	100
-17.50 to -26.50		Moderately Weak to Strong Mudstone (Coal Measures)	22.0	10 kPa *10 MPa*	32	500
-26.50 to -33.00		Moderately Strong Mudstone (Coal Measures)	24.5	25 kPa *20 MPa*	40	1,000
-33.00 to -40.00		Moderately Weak to Strong Mudstone (Coal Measures)	24.5	10 kPa *5 MPa*	40	500
-40.00 to -44.00		Moderately Strong Mudstone (Coal Measures)	24.5	25 kPa *20 MPa*	40	1,000

Coal Authority records indicated that the site was potentially in the zone of influence of four seams of coal at 410m to 590m depth but concluded that risks were minimal. A rotary core borehole was extended to 50m depth at the pylon to confirm stable rock conditions were present.

The summary ground model and associated characteristic parameters used in design of the bridge pylon foundations are given in Table 1. A graph of Unconfined Compressive Strength (UCS) directly measured from rock core testing or derived from correlation to Point Load test (PLT) data (I_{s50}) assuming UCS = 17 times I_{s50} is presented in Figure 3. Rock mass moduli were derived from correlations to the in tact rock moduli measured in UCS tests assuming j=0.2. In situ rock modulus ranged from 500 to 2,250 MPa and were derived using equation 1

$$E_m = 2 \,.\, G_{PMT} / (1 + \nu) \tag{1}$$

Where Em is the rock modulus (MPa), G_{PMT} is the shear modulus (MPa) derived from in situ pressurementer testing and ν is poisson ratio measured in UCS tests and varied from 0.1 to 0.3.

Ground conditions at the proposed preliminary static load test pile located on the southern river bank were explored by two rotary core boreholes BH09 & BH703, the latter being coincident with the test pile. Variation of UCS with level at the preliminary test location is presented on Figure 4 together with the design profile (shown dashed) adopted for the pylon foundation but corrected for the difference in levels of rockhead. The design profile can be seen to be a reasonable fit for a conservative characteristic mean to both sets of in tact rock strength data. RQD in the mudstone typically ranged from 25 to 70 % but was locally poorer at the preliminary test pile base with RQD of 20%.

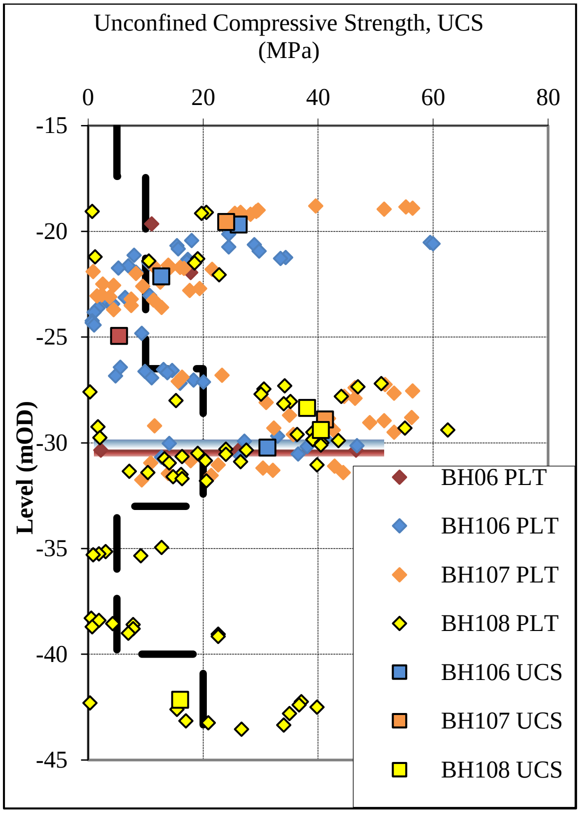

Figure 3. UCS v Level at bridge pylon

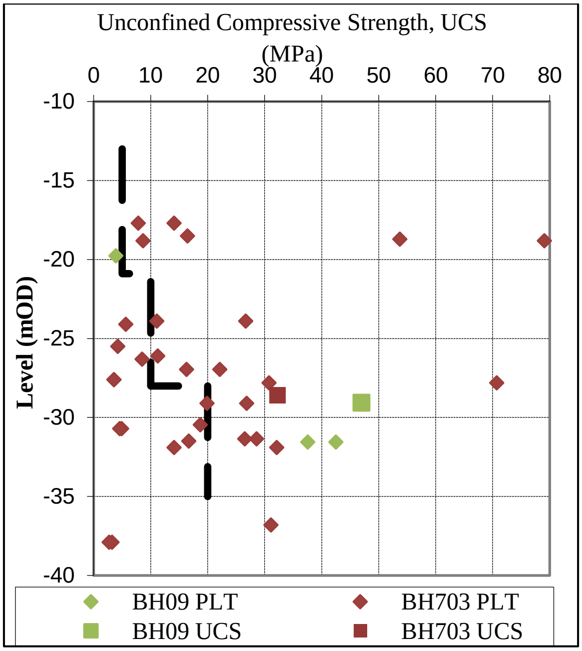

Figure 4 UCS v Level at Preliminary Test Pile

3 PYLON FOUNDATION DESIGN & CONSTRUCTION

The bridge pylon sustains a large variety of load conditions both during construction, erection and in final service. The critical load condition for axial design of the piles was during in service loads with a combination of dead, live and wind loads. The selected foundation layout was highly efficient and resulted in only ten 1.5m nominal diameter rock socketed bored piles supporting the pylon and its substructure, Each pile included a 1.524m outside diameter permanent steel casing extending from underside of the pile cap at -6m OD to the rock surface at -15.75m OD and was designed for a maximum axial SLS load of 24.5 MN.

The rock socket length was estimated based on unit ultimate skin friction derived from correlations to unconfined compressive strength using equation 2.

$$Q_s = \alpha\ \beta\ \text{UCS}\ A_s \qquad (2)$$

Where Qs is the ultimate skin friction capacity, reduction factor α varies with UCS from 0.1 to 0.18, correlation factor β = 0.65, UCS is the characteristic unconfined compressive strength of rock and A_s = perimeter area of pile section, (Tomlinson, 2008). This approach yielded a range of estimated unit ultimate skin friction from 585 to 1,300 kPa in mudstone rock corresponding to differing design UCS values. A large range of ultimate unit end bearing capacity from 5 MPa to 20 MPa was considered feasible in practice. A 14.5m long rock socket length was initially estimated. Since there is a very limited number of full scale in-situ tests which have been undertaken in Coal Measures in the past, a minimum 60MN axial capacity static load test was specified to verify the design in accordance with BS EN 1997. Integrity testing was specified for all test and working piles with four tubes installed in each pile full length and tested by cross hole sonic testing methods.

4 PRELIMINARY TEST PILE DESIGN & INSTRUMENTATION

It was considered impractical to construct a reaction frame and anchorages within the main pylon cofferdam due to the high capacity test load and extended programme delay required to construct and test a non production preliminary test pile in the pylon cofferdam prior to progressing pylon production piles. The introduction of the bi-directional O-cell load test method has assisted greatly in providing actual full scale behaviour results, specifically for high loads, in a safer and more cost effective manner than is possible with traditional top down loading tests. Bi-directional load tests are carried out by casting a loading arrangement within the test pile itself. Consequently a location for the preliminary O-cell static test was selected at the southern river bank shown in Figure 5 where comparable mudstone bedrock conditions were investigated and proven to be present.

Figure 5 Site of Preliminary Test pile

The preliminary test pile was constructed over a period of seven days from 1st to 7th July 2015 with a 14.6m long, 1.45m nominal diameter rock socket in mudstone drilled wet during a two day period on the 3rd and 5th July and subsequently cleaned immediately prior to pouring concrete by tremmie on 7th July 2015. A total of 70 m^3 of C35/45 grade concrete was placed over a 5 hour period with minimal overbreak in the rock socket being measured, after allowing for the volume of steel reinforcement and hydraulic load cells.

The O-cell® is a hydraulically driven, calibrated, sacrificial jacking device installed within the pile. They can be arranged in multiples at the same elevation and also can be arranged at several different elevations. O-cells work in two directions, simultaneously loading upwards and downwards, thus performing two separate but concurrent static load tests. To complement the evaluation of the friction in Coal Measures, a multilevel bi-directional loading test was considered most appropriate. The multilevel loading arrangement was designed to utilise two levels of 2 x 540mm O-cells located 1.3 m and 5.8 m above the pile toe, with each loading assembly being capable of applying 20 MN in each direction at rated pressures.

At the test pile head a steel reference beam, supported some distance away from the test pile, is used to determine the pile head movements, as shown in Figure 6.

Figure 6 Instruments and reference beam over the test pile head

Each Osterberg Cell assembly is specially instrumented to allow for direct measurement of the O-cell expansion. By measuring the pile head movement and compression of each section, the upward and downward movements from each O-cell elevation is determined. Instrumentation also included 7 levels of sister-bar strain gauges (SG1 to SG7), as illustrated in Figure 2.

5 PRELIMINARY TEST PILE RESULTS

As is necessary for a multilevel bi-directional loading test, each O-cell assembly at each elevation is loaded in stages.

In Stage 1, the lower O-cell assembly is pressurised and the load displacement results are obtained from the friction and end bearing below reacting against the entire friction length above. Figure 7 shows the load displacement results from the start of the loading of the lower O-cell assembly for each pile element being moved upwards and downwards. A total of 46 MN was mobilised.

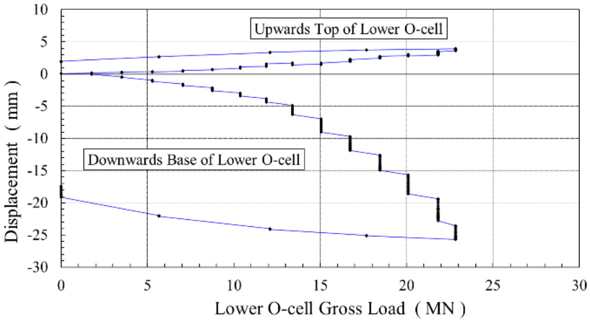

Figure 7- Load displacement of lower O-cell Stage 1

Stage 2 pressurises the upper O-cell assembly, mobilising the frictional resistance above the upper O-cell elevation and the frictional resistance between the O-cell elevations (noting that in Stage 1 a gap has been formed and, with the lower O-cell free to drain hydraulically, offers no resistance to closing). Figure 8 shows the load displacement results obtained from Stage 2, again mobilising over 46 MN.

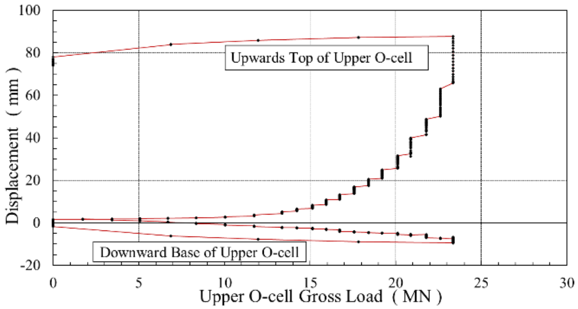

Figure 8 Load displacement of upper O-cell Stage 2

Back analysis of the strain gauge results, making appropriate assumptions of the structural stiffness, show the mobilised loads with depth for each of the loading stages in Figures 9 and 10 respectively. The corresponding maximum unit skin friction mobilised in mudstone during Stage 2 was over 600 kPa upwards above the upper cell and over 1000 kPa downwards between O-cell elevations. Unit skin friction against displacement for each incremental length between strain gauge elevations is shown in Figures 11 & 12. Note that the maximum displacement of the test pile segment between the upper and lower O-cells measured in Stage 2 is less than 10mm and an ultimate skin friction capacity has not been reached in the test.

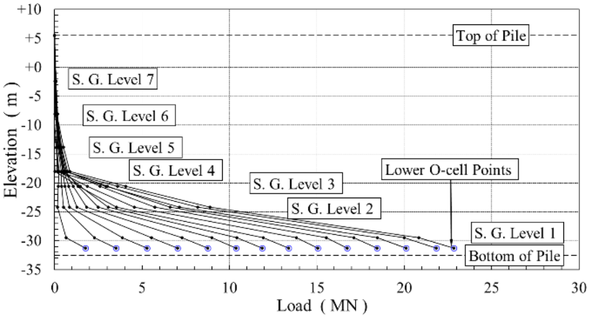

Figure 9 Strain Gauge load distribution Stage 1 for each loading step

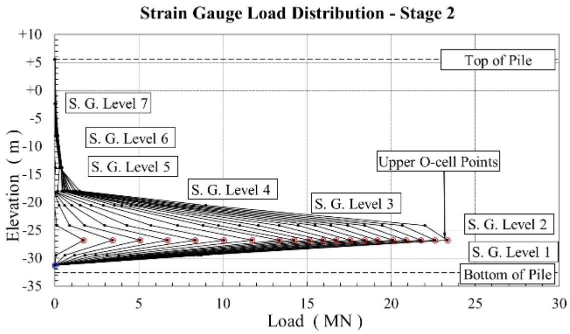

Figure 10 – Strain Gauge load distribution Stage 2 for each loading step

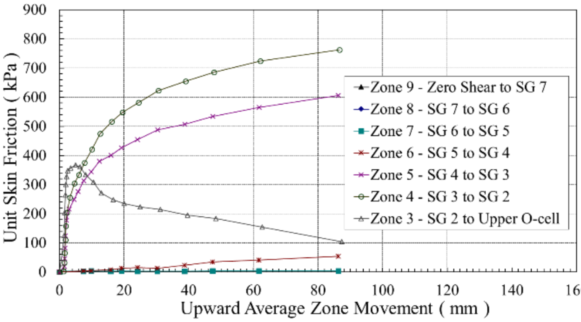

Figure 11 – Mobilised unit skin friction – Stage 2

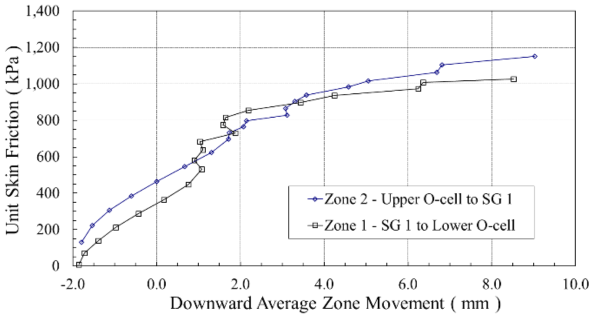

Figure 12 – Mobilised unit skin friction – Stage 2

In separating the end bearing from the lower skin friction using the strain gauge results, it is assumed that the unit skin friction below the lower O-cell is the same as that calculated for the zone immediately above the lower O-cell at the same displacement. The computed unit end bearing displacement graph is shown in Figure 13.

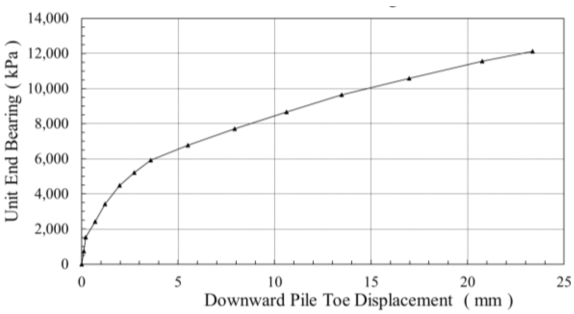

Figure 13 – Mobilised Unit end bearing in Stage 1

A Cemsolve analysis, (Fleming, 1992) was carried out for each of the load displacement results recorded for each tested section and these were recombined to assess the total load deformation behaviour of the test pile. The hyperbolic curve fitting procedure predicted two asymptotic ultimate resistances from Stage 1, comprising 11 MN of skin friction and 35 MN of end bearing beneath the lower O-cell at a stiffness of 300,000 kN/m^2. The skin friction parameters determined from Stage 2 indicate a total of 46 MN mobilised.

A significant regular variation of the vertical displacement data in time from the top of pile gauges was noted during the test of approximately ±1 mm. This was due to the tidal influence on the reference beam level which was determined by monitoring of the telltales installed from the toe of the test pile. and referenced to top of concrete pile head. The displacement measurements were corrected for these tidal variations in reference beam level.

6 PREDICTED BEHAVIOUR OF PYLON PILES

An equivalent top down load settlement curve was produced for the production piles, (England, 2005), based on the same 14.5 m rock socket length and load deformation characteristics, but with a reduced overburden and pile length to that constructed at the preliminary test pile location.

The assessment of skin friction in the overburden was made directly from the interpeted load mobilised at the relevant elevations. The effective length of permanently lined production piles in contact with soft / loose alluvium above the Coal Measures bedrock is reduced to around 10 m compared to over 23 m length in the soils surrounding the preliminary test pile. The predicted load-displacement analysis is presented in Figure 14 and a settlement of 9 mm is estimated at 24.5 MN axial SLS load.

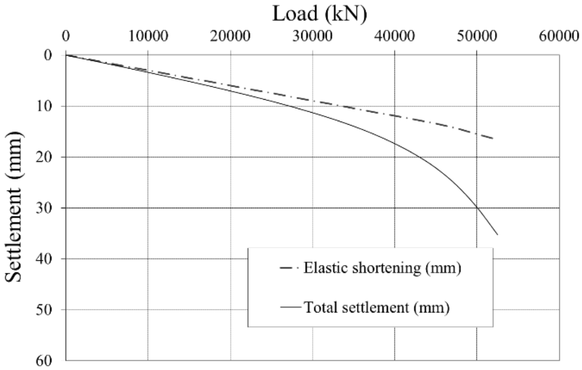

Figure 14 Cemset Pylon Pile head prediction

7 CONCLUSIONS

Foundation design optimisation in Coal Measures can provide significant savings if preliminary full scale load testing is carried out and actual representative in situ parameters are obtained from high quality results. The paper describes was is believed to be the highest capacity foundation load test yet performed in these mudstone deposits in north east England mobilising a total axial capacity in excess of 70 MN from two test stages. The test validated that the originally designed 14.5 m long rock socket in Coal Measures mudstone could adequately support the bridge pylon design loads with acceptable settlements of 9 mm.

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