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## XI

### **The South Abutment at Kessock Bridge, Scotland**

La culée sud du pont de Kessock en Ecosse

Das südliche Widerlager der Kessock Brücke in Schottland

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### **SUMMARY**

The predictions of movement of a large bridge abutment, based on a consideration of the forces in the piles, indicated that it would move outwards and tilt forwards. However, taking into account the ground displacement due to the large mass of fill which was placed behind the abutment, inwards and backwards tilting movements were predicted. Since this latter mode of movement could have introduced undesirable stresses, the abutment movements were measured.

### **RESUME**

La prévision des déplacements d'une culée de pont de grande taille, basée sur l'analyse conventionnelle des forces appliquées, indiqua que la culée se déplacerait et basculerait en avant. Cependant, en considérant toutes les forces, la grande masse de remblai placée derrière la culée étant incluse, un déplacement et un basculement vers l'arrière furent prévus. Comme ce dernier type de mouvement pouvait introduire des contraintes indésirables, les déplacements effectifs de la culée furent mesurés.

### **ZUSAMMENFASSUNG**

Bezüglich der Bewegung eines grossen Widerlagers der Kessock Brücke in Schottland wurde vorausgesagt, es würde nach aussen gleiten und nach vorn kippen. Trotz Berücksichtigung der Bodenbewegungen und der grossen Füllmasse, die hinter dem Widerlager angebracht wurde, wird vermutet, dass das Widerlager nach innen gleiten und nach hinten kippen wird. Da dies unerwünschte Spannungen auslösen würde, sind an dem Widerlager Messungen durchgeführt worden.



## 1. INTRODUCTION

The Kessock Bridge when completed will span the Beaully Firth at Inverness, Scotland. As shown in Figure 1 the abutment to the southern approach viaduct retains a selected granular fill embankment some 12m high, and supports one end of the first 64m span. The bearings of the viaduct are designed to transmit vertical load only, longitudinal forces being absorbed in the piers and at the northern abutment.

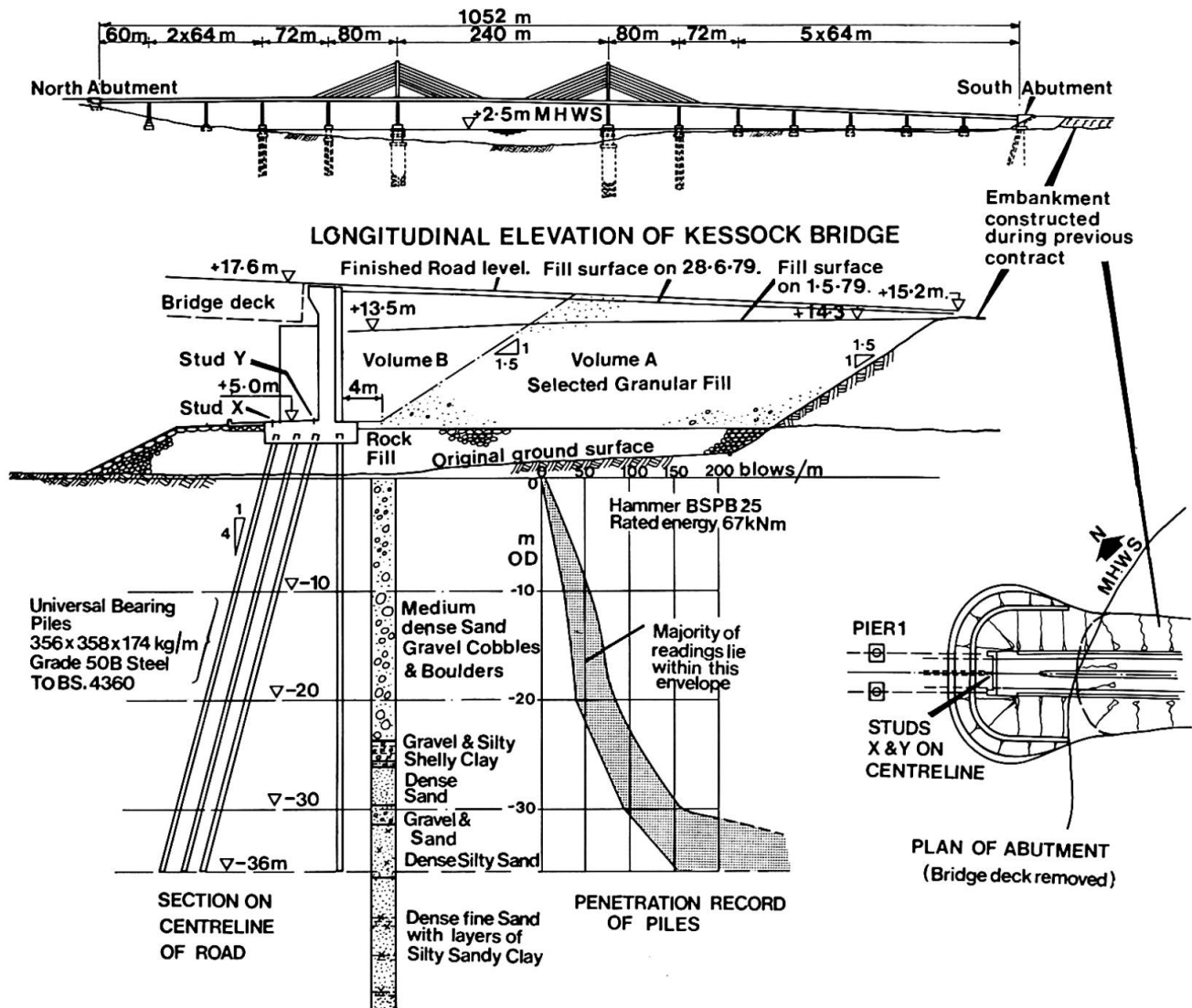


Figure 1 Location and general details

## 2. DESIGN OF FOUNDATIONS

### 2.1 Soil conditions

The great depth of alluvial deposits at the site have influenced the choice of foundations; a borehole for a pier adjacent to the navigation channel reached 95m below bed level, and a borehole 1 km away reached 89m below ground level, both encountering only alluvial deposits. The alluvial deposits increase in density with depth and pad foundations have been adopted where maximum bearing pressures can be economically kept to within 250 kPa. Where bearing pressures exceed this, or the depth of construction below water exceeds 10.5m, steel H-section bearing piles have been driven into the alluvial deposits.

## 2.2 Abutment foundations

Conventional analysis of the southern abutment foundation indicated that three rows of piles raked at 1 in 4, and a rear row of vertical piles were required to provide adequate support for the loads shown in Figure 2. The driving record of piles is given in Figure 1; test loading of a vertical pile gave 7mm settlement at the design working load of 1600 kN.

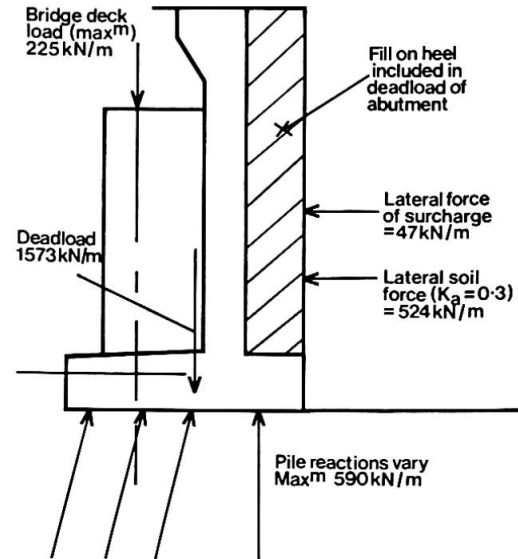


Figure 2 Loading diagram of forces on abutment (non-seismic loads)

## 3. MOVEMENT OF THE FOUNDATIONS

### 3.1 Analysis treating only forces on abutment

Analysis of the movement of the abutment, taking account of the forces shown in Figure 2, and treating the piled foundation as elastic springs, showed that abutment movements would tend to be downwards and forwards, the base of the abutment moving some 40mm towards the north.

However, it was evident that the analysis of movement of the abutment could not be considered in isolation from the ground movements that would be generated by the construction of the short length of granular fill embankment between the already placed embankment and the abutment.

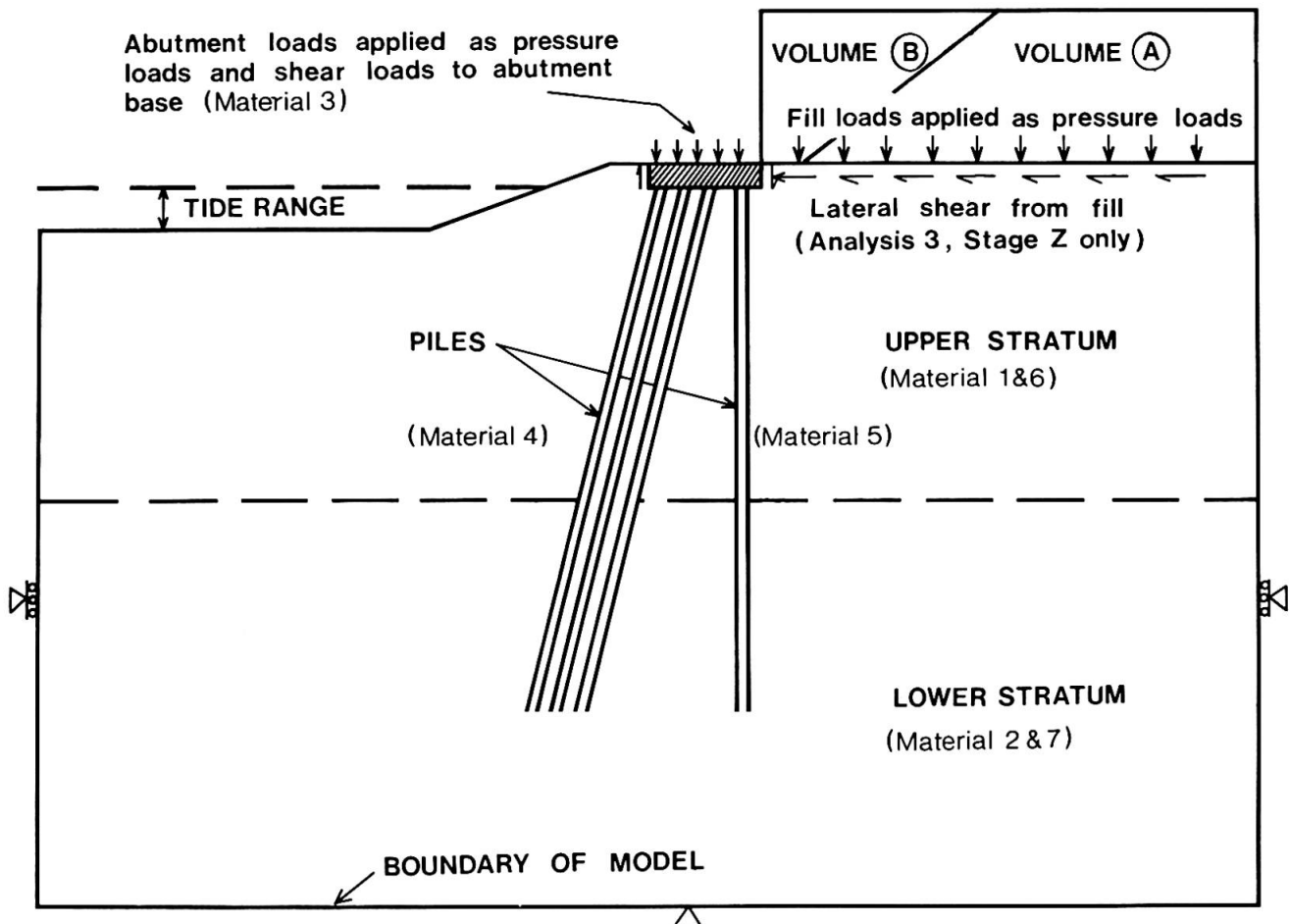


Figure 3. Finite element model. For properties of materials 1 to 7 see Table 1.



### 3.2 Analyses of interaction of soil and structure

A finite element programme, SAFE, (Reference 1) incorporating facilities to take account of soil and structural material properties was used to compute ground movements and stresses in the structural elements. Previous computations using the SAFE programme including modelling of anchored retaining walls, had shown that the programme gave reasonably accurate predictions of movements when compared with field measurements.

A two dimensional finite element model of the abutment and the soil was established, see Figure 3, and a series of analyses undertaken, the assumed soil and structural material properties for Analyses 3, 4 and 5 being shown in Table 1.

**Table 1** "SAFE" finite element input data of material parameters.

All strength and modulus properties in MN/m<sup>2</sup>

Material parameters	Soil (Analyses 3 & 4)		Concrete (All analyses)	Piles (Analyses 3 & 5)		Soils (Analysis 5)	
Material number	1	2	3	4	5	6	7
Material type	LE	LE	LE	LE	LE	LE(s)	LE(s)
Drainage	Drained	Drained	Drained	Drained	Drained	Drained	Drained
E <sub>1</sub>	50	30	30 000	50	50	50	30
$\nu_{12}$	0.3	0.3	0.1	0.3	0.3	0.3	0.3
E <sub>2</sub>	50	30	30 000	4809	7642	50	30
$\nu_{12}$	0.3	0.3	0.1	0.3	0.3	0.3	0.3
G <sub>12</sub>	19.20	11.54	13636	19.20	19.20	19.20	11.54
$\alpha$	0	0	0	-14.036	0	0	0
c'	—	—	—	—	—	0.017	0.017
$\phi'$	—	—	—	—	—	0	0

#### Note

LE = linear elastic model

LE(s) = linear elastic model with strength limitation

Drained = excess pore water pressures assumed dissipated

E<sub>1</sub>, E<sub>2</sub> = principal Young's moduli in x, y plane, suffix 1 denoting horizontal (in plane), 2 denoting vertical and 3 denoting horizontal (normal to plane) directions when  $\alpha = 0$

G<sub>12</sub> = shear modulus between directions 1 and 2

$\alpha_1$  = anticlockwise inclination of direction 1 to horizontal (degrees)

c' = cohesive resistance (limiting value)

$\phi'$  = frictional resistance (limiting value)

The soils were given linear elastic properties and assumed to be drained, since their rate of consolidation was expected to be rapid. The assumptions concerning the pile elements are stated in Figure 5; for Analysis 4 the piles were assumed to be absent in order to demonstrate the general validity of the method. The main defect of the model was that the pile elements had to be considered as having the same unit "depth" (of 1 metre) as the soil and concrete elements, thus prohibiting relative movement in the line of the bridge between the piles and adjacent soil.

### 3.3 Loading sequence

The loading sequence used is shown in Figure 4, the construction process being simulated by four stages, A to D. To check on the effect of the simplifying assumption that the embankment load applied only a vertical load, Stage Z in which horizontal shear loads from the embankment were applied was run in addition to Stage B in Analysis 3.

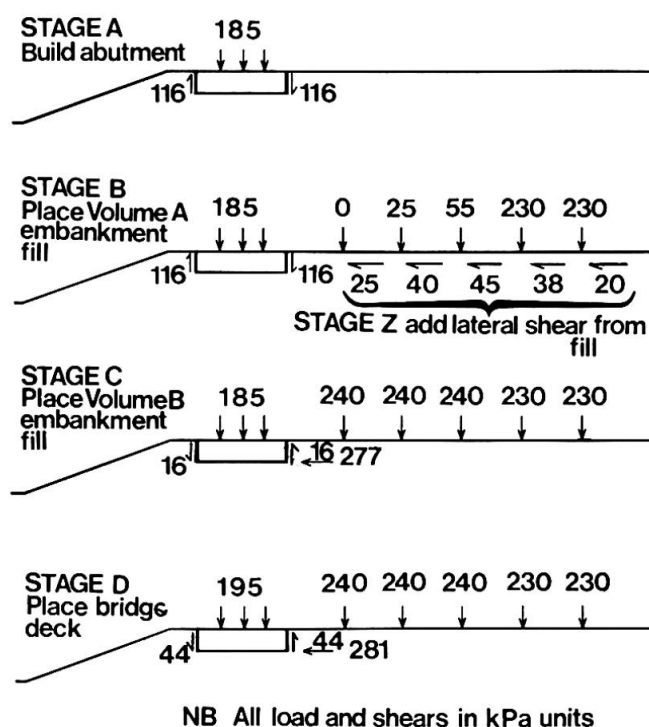


Figure 4 Loading diagram for finite element model

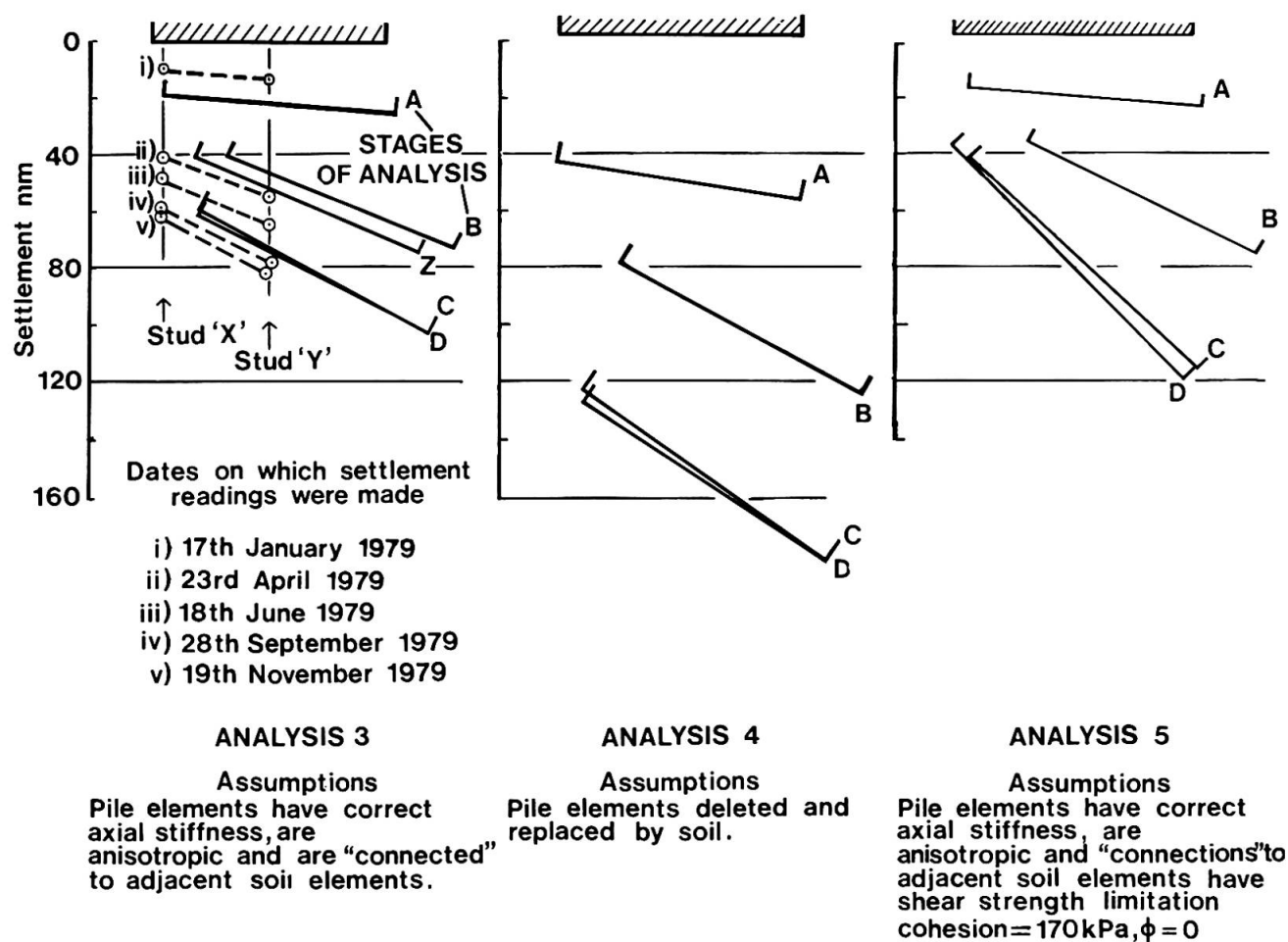


Figure 5 Computed and measured movements.



### 3.4 Computed and measured movements

The results of the computations of movement are presented in diagrammatic form in Figure 5, with the measured results to date shown on Analysis 3. Studs X and Y on which level readings have been made are located on the road centreline at the positions shown in Figure 1. Site measurements of movement of the abutment in the line of the road were attempted, but showing little if any movement and being difficult to make with sufficient accuracy they were discontinued. The abutment movements have therefore been presented on the assumption that only vertical movement took place.

### 3.5 Movements were time dependent

The diagram giving settlement against time, Figure 6, shows that realisation of full settlement at each stage of loading was delayed by classical consolidation involving the dissipation of excess pore water pressures from the fine grained soils, complete dissipation for each stage taking about 6 months. The specified sequence of embankment filling was that Volume A should be completely placed in Stage B (Figure 4) before the placing of Volume B was started in Stage C, the object being to eliminate the pressure of fill material against the stem of the retaining wall as the base took up

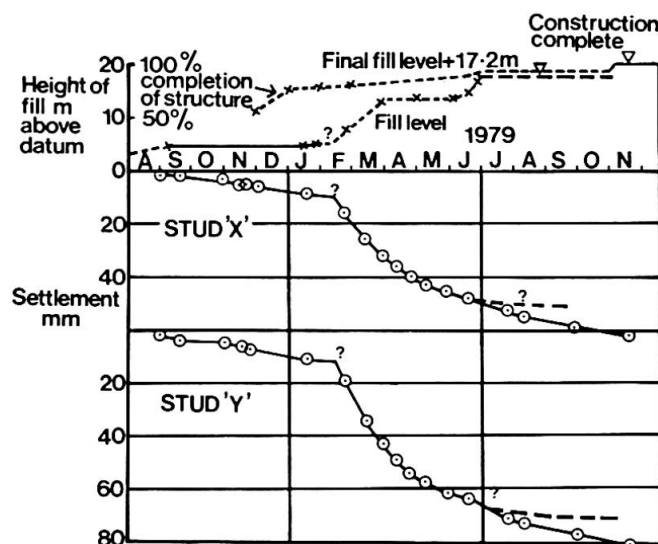


Figure 6 Loading and settlement with time.

the major part of its backward rotation during the placing of Volume A, see Figure 5. In the event, the fill surface was raised uniformly over the entire fill length from about 5m above datum to 13m above datum in one continuous lift, followed by further filling to road base level at 17m above datum after a delay of two months.

## 4. EFFECT OF MOVEMENTS ON STRUCTURE

### 4.1 Restraint on backward rotation

As a consequence of following the described sequence of filling the lateral pressure of retained soil restrained the tendency of the base to tilt backwards by developing the bending strength of the connection between stem and base, the measured average angular rotation between the stem and base being about 1 in 900 after the filling to 13m above datum. During the subsequent filling this rotation increased to about 1 in 525, It is thought that the average vertical movements of base will have not been affected by the difference between the intended and the actual sequences of filling, but the backward tilt of the base may have been reduced.

### 4.2 Stiffening effect of piled foundation

From Figures 5 and 6 reasonably good correlations are apparent between the predicted settlement from Analysis 3 and the actual settlement on 17th January 1979, when the loading was the abutment base and stem (Stage A) and on 18th June 1979 when all loads except the bridge deck had been placed and consolidation under those loads was virtually complete (Stage C). It is evident that the piles moved together with the surrounding soil to give much greater settlements than could have been predicted from the results of the pile test loading and considerations of group action; the movements were of the same





form but smaller than those that would have occurred if the foundation had not had piles. The smaller actual backward tilt than predicted by Analysis 5 indicates that the soil adjacent to the rear row of piles did not move differentially to the piles, and that shear failure played little part in the overall behaviour.

## 5. ACKNOWLEDGEMENTS

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The Joint Engineers for the Client are Crouch and Hogg of Glasgow and Ove Arup and Partners of London

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## 6. REFERENCE

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