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A Low-Rise Hospital Development on Restored Opencast Fill

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SYNOPSIS: An extensive low-rise hospital development has taken place since 1982 on a former opencast coal mining site 30 years after working and restoration. The clay and shale fill some 20 metres deep was placed without systematic compaction. Five separate site investigations have been carried out at different times during the last 20 years and a detailed engineering geological mapping exercise was completed in 1977. Drainage works and the main buildings in Scheme 1 and 2 have been monitored during and after construction. Significant settlements have occurred requiring some remedial work. A surcharge and inundation trial has been undertaken prior to the design and construction of Scheme 3. The response of the buildings and the drainage to ground movement is described and design recommendations made for future development.

INTRODUCION

Opencast coal mining began in earnest in the United Kingdom in 1942. Early developments were slow due to inexperience and the absence of suitable plant. Operations were often restricted to around 20 metres depth. Plant and technical assistance were provided by the United States and some 43 million tons were extracted in the 5 year period 1942-1947. United Kingdom opencast coal mining has often taken place in areas of past deep mine activity in or around population centres. The opencast mines in the United Kingdom are characterised by multiple thin seams giving overburden/coal ratios up to 20, excavation frequently below the water table requiring dewatering and back-filling without systematic compaction. The geology is commonly stiff glacial till overlying the sandstones, shales and mudstones of the Upper Carboniferous Coal Measures. The over-burden is excavated either by dragline and cast to one side or where face shovels are used, loaded into trucks and end tipped in high lifts. Superficial soils are usually removed using scraper equipment and stored on site until restoration. Occasionally coal washing plant and associated lagoons or settlement ponds for general site drainage alter groundwater conditions locally. While all opencast sites are restored for agricultural purposes it is only recently that controlled compaction in layers for all or part of the fill has been adopted for specific sites. Elsewhere the vertical restored profile can be expected to show large variations in material type, grading, density and moisture content. There have been many modest commercial, industrial and residential structures with their associated roads and services constructed on uncompacted fills up to 60 metres deep since the late 1950's. Most of this development has been trouble free and where it has not the response of the clay/ shale/sandstone backfill to increasing moisture content has been the most probable source of settlement-related damage. The prediction and control of the post-construction groundwater regime is the most challenging aspect of developing these sites.

SETTLEMENT OF OPENCAST FILL: GENERAL

There are three main components of settlement

A. Settlement due to self-weight at original moisture content.
B. Settlement due to building loads.
C. Settlement due to wetting-up.

While A always occurs first, B and C may occur in any order or contemporaneously.

A. Most settlement due to self-weight occurs rapidly as the fill is dumped. This primary compression is therefore of little interest long term. Measurements of surface movements on backfilled opencast sites have shown that settlement continues but the rate of settlement of the ground surface decreases rapidly with time and often appears to be negligible several years after backfilling, Kilkenny, W.M. (1968), Leigh, W.J.P. and Rainbow, K.R. (1981). This long-term compression due to self-weight may be defined by the creep compression rate parameter, α, which is the percentage vertical compression of the fill that occurs during a $\log_{10}$ cycle of time from one to ten years after restoration. Typical values of for opencast fill lie in the range 0.5-1.0 per cent reflecting the proportion of weak rocks to superficials in the backfill and the predominant rock type e.g. sandstones or limestones as opposed to shales or mudstones. These values relate to unsaturated conditions. Modest heave has been recorded where a location has been pre-loaded by a spoil heap.

B. Most settlement due to building loads will occur as the load is applied.

This immediate settlement may be estimated by the use of the constrained vertical drained modulus $E'_V = \frac{1}{m_V}$. The compressibility of opencast fills without pre-compaction or wetting-up may vary from E'_V = 2000-6000 KN/m^2 and a value of E'_V = 4000 KN/m^2 is often recommended for preliminary design [Building Research Establishment (1983)]. The long-term creep settlement under building loads may be estimated using the creep compression rate parameter with zero time taken at completion of the building.

C. Significant additional settlement can be caused by an increase in the moisture content of the opencast backfill, Charles, J.A., Naismith, W.A., Burford, D. (1977). This increase may be caused by precipitation, surface ponding, ingress of water from flooded surface or near surface construction such as drainage trenches, broken services or by a general restoration of the water table. Episodic exposure to relatively small quantities of water, e.g. precipitation, has a modest effect on overall settlement. The settlement of recent fill has been observed to accelerate during the winter. However, inundation from surface or groundwater sources has caused initial collapse compression of up to 6 per cent. The creep compression rate is enhanced following inundation but generally the fill is less susceptible to dramatic settlement thereafter.

PROJECT DESCRIPTION

The North Tyneside District General Hospital is situated in Tyne & Wear, England, (National Grid Reference NZ 342703). The hospital site covers an area of about 13 hectares which formed part of the larger Whitley Bay opencast coal site on which work began in 1948 and which was fully restored in 1952. The hospital consists of single and two-storey buildings of load-bearing brick and blockwork cavity wall construction on flexible reinforced concrete rafts which incorporate the strip footings for internal and external walls as ground beams in a composite foundation. The buildings are divided into smaller structures by movement joints. The complex stormwater and separate foul drainage systems use concrete and clayware pipes ranging from 100-700mm diameter with rubber ring joints laid at shallow gradients with backdrops to brick manholes cast on reinforced concrete slabs. The first two phases of construction are complete, Scheme 3 is at the design stage. This paper reviews the ground conditions, the settlements and the structural response of a sub-section of the buildings, Area 39 and the site drainage, Figure 1.

GROUND CONDITIONS

Mining records show that in addition to the opencast coal operations which worked the Yard seam, 0.79 metres thick, five other coal seams exist within 155 metres of the surface. The closest to the pavement of the restored opencast working is the Bensham seam, 1.38 metres thick, which lies some 26 metres below it. This seam was extensively worked using deep mine techniques up till 1929. The old workings in this seam, together with the bed separation associated with them allows underdrainage to take place drawing down the groundwater level such that no positive porewater pressures have

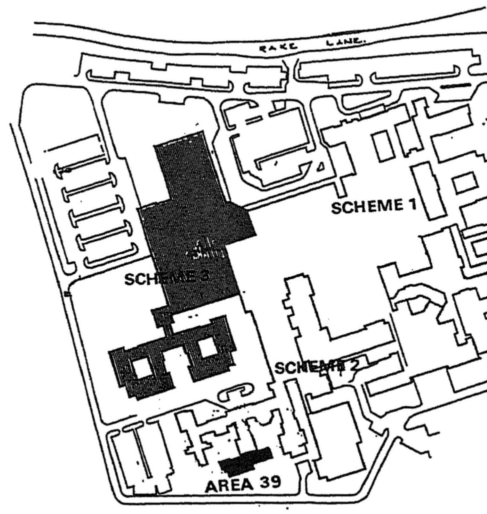

Fig. 1 Site Plan

been observed in the backfill. Interconnected old workings appear to drain to the adjacent North Sea and River Tyne causing permanent drawdown. At different times, some 5 site investigations have been carried out together with an engineering geological mapping exercise, Dearman, W.R. et al (1977), which has defined accurately the position, extent and depth of the opencast workings, Figure 2.

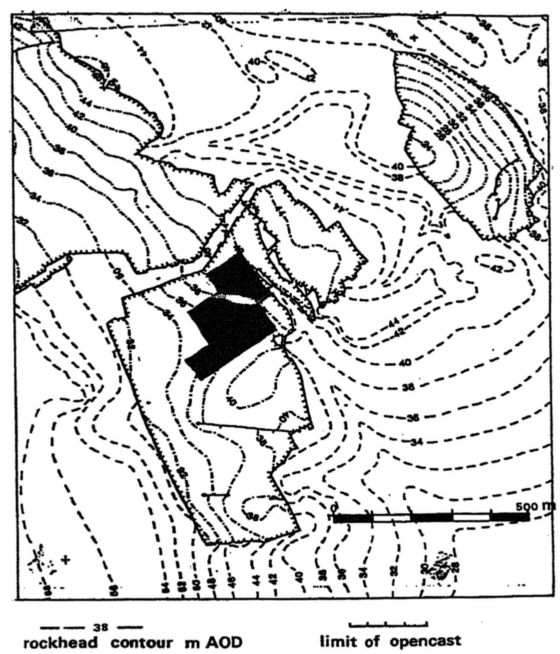

Fig. 2 Rockhead Contours

The low point in the excavation and accordingly the deepest fill, some 26 metres, lay on the Western boundary of the opencast site decreasing to 6 metres on the Eastern boundary. The 'average' depth of fill under Scheme 1 and 2 are 10 and 15 metres respectively. The opencast excavation was not continuous over the whole area but mining factors and the modus operandi combined to leave a ridge and a spur of undisturbed ground running in a NW-SE direction immediately to the East of Scheme 1 across which the site drainage ran. The commercial site investigations variously describe the backfill as "grey shale fill", "stiff grey clay with boulder and mudstone fragments" and "clayey sandy mudstone and shales". The compressibility of the opencast fill in its present condition has been assessed by in-situ testing using the Standard Penetration Test, Figure 3, by laboratory consolidation tests on undisturbed 75mm diameter samples of the finer grained material and on 250mm diameter samples recompacted at in-situ moisture content to field densities, Table 1. Field densities recorded on samples from two 75mm diameter cored holes ranged from 1884-2285 Kg/m^3 and the degree of saturation from 45-95 per cent.

Sample Size mm	Initial Moisture Content %	Bulk Density Kg/m^3	Constrained Modulus, E'_V MN/m^2	Effective Stress Range KN/m^2
75 (Undisturbed)	10-14	2000-2220	4.8-6.6	50-400
250 (Recompacted)	8-14	1970-2220	12.0-20.0	135-570
	8-12	1800	1.4-5.0	68-375

Table 1 Laboratory Compressibility Determination

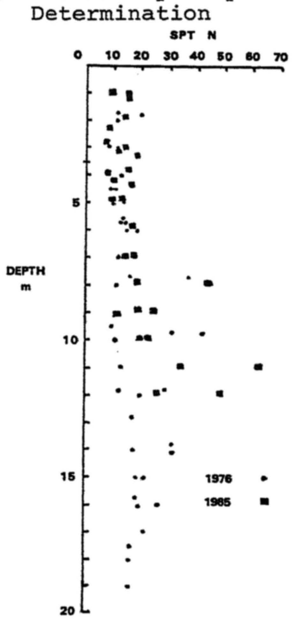

Fig. 3 Standard Penetration Test 'N' v Depth

A surcharge trial has been undertaken involving the construction of three trial embankments, each 20 metre square and 2, 4 and 6 metres high respectively, in the area of the proposed Scheme 3. Magnetic extensometers enabled the settlement at the surface and at 2, 3, 5, 7, 10, 13 and 16 metres below ground level to be determined. Values of constrained vertical modulus were computed by estimating the stress levels at the mid-height of each layer after Giroud and dividing by the observed vertical strain, Table 2. The response of the fill to inundation has been measured in the field by flooding a granular blanket under the 4 metre high embankment after near equilibrium had been established under the surcharge load in the dry conditions. This resulted in collapse compressions occurring but only in the previously identifiable zone of high compressibility between 5 and 13 metres below the surface, Figure 4.

Embankment Height Metre	Depth Metre	Constrained Modulus E'_V MN/m^2	After Inundation	Effective Stress Range KN/m^2
2	0-5	52.8		52-95
	5-7	19.4		126-165
	7-10	10.6		178-210
	10-20	40.5		315-335
4	0-5	35.9	19.6	52-139
	5-7	2.1	4.0	126-203
	7-10	1.6	1.5	178-236
	10-13	4.0	1.7	253-302
	13-20	7.9	6.0	346-376
6	0-5	101.2		52-174
	5-7	70.4		126-232
	7-10	4.2		178-252

Inundation beneath 4 metre embankment

Depth metres	Collapse Compression per cent
0-5	0.18
5-7	0.5
10-13	2.0
13-20	0.14

Table 2 Field Compressibility Determination

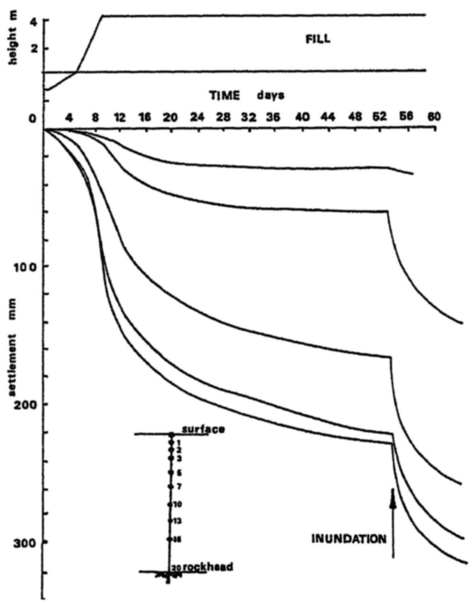

Fig. 4 4m Embankment, Settlement v Time

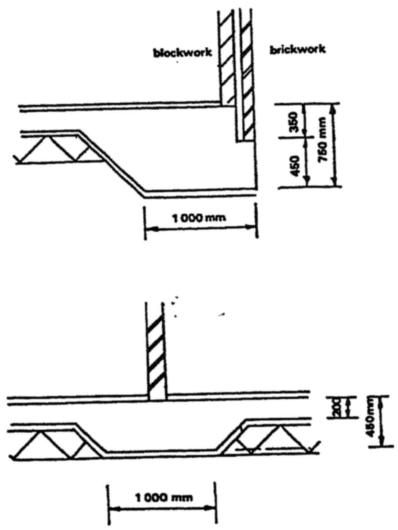

Fig. 5 Foundation Details

SETTLEMENT, DIFFERENTIAL SETTLEMENT AND STRUCTURAL RESPONSE

Buildings

The modest building loads transferred to the opencast backfill through the flexible raft foundations are of the order of 30-50 KN/m^2, Figure 5. Some slight cracking has occurred in places throughout Scheme 1 and 2 which may be settlement related but this has not required remedial work. Observed settlements due to initial structural loads varied from 25-70mm. Some significant additional movement was first noticed in the summer of 1985 after completion of the structural frame, across the movement joint between Area 36/39 and along the South wall of Area 39. Monitoring pins were installed at damp proof course level throughout Scheme 2. Settlement in Area 39 proceeded rapidly initially and then continued at a diminishing rate, Figure 6. The relationships between the maximum total settlement 161mm at W34 and minimum total settlement 23mm at W23, giving a maximum differential settlement of 138mm which plots well outside quoted limits for building on raft foundations. The relative rotation across the building is much higher than load bearing walls would be expected to accommodate, Table 3.

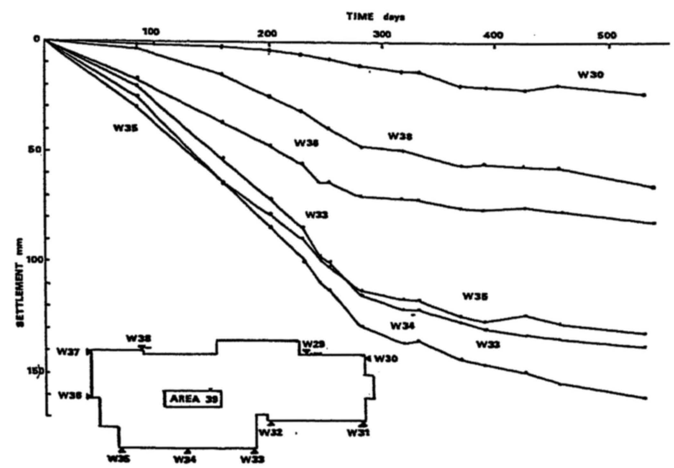

Fig. 6 Area 39: Settlement v Time

Scheme 2 : Area 39

Section	L/H	$\frac{\Delta}{L} \times 10^3$	Settlement Mode
W35/W37	4.2	6.6	Hogging
W33/35	6.1	1.3	Sagging
W31/32	6.2	4.2	Hogging

Table 3 Deflection Ratios

Internally, some minor cracks have appeared in the plaster finish of the long corridor walls at floor level adjacent to doors and windows and along the south elevation at the locations indicated, Figure 7.

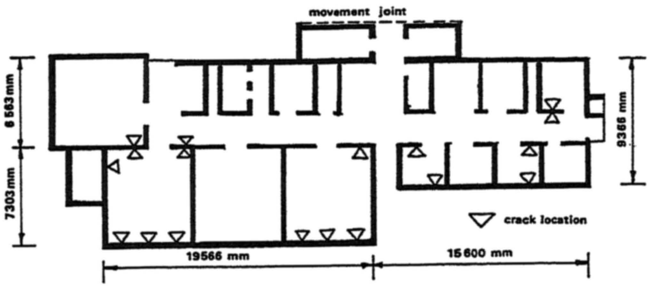

Fig. 7 Area 39: Crack Locations

Openings reduce wall stiffness and act as stress concentrators. No cracking has been observed in the external brickwork other than hairline cracks in the mortar bed joints around the windows on the south elevation. This reactivation of settlement under constant load has been attributed by the Author to wetting-up of the opencast fill from an, as yet, unidentified source. Because the foundation raft is relatively flexible it is the configuration and stiffnesses of the walls which control redistribution of contact stress, differential settlement and resulting crack patterns. This subject forms a continuing area of research at the University of Newcastle upon Tyne.

<u>Drainage</u>

Light linear structures such as pipelines are vulnerable to settlement damage particularly where they pass from undisturbed ground to fill, from a supported position within a structure to fill, or from a stressed condition under a structure to an unstressed condition outside. As early as 1980, during installation of the drainage on this site movements of manholes and foul and stormwater sewers were reported. A level survey in November 1982 suggested that manholes and associated drainage on the western and southern boundary of the site had settled up to 150mm. Pipeline inspections, prior to handover, revealed a number of bending and shear failures in the 100-250mm diameter clayware pipes of the foul drainage and back falls and changes of gradient in the larger diameter concrete pipes, together with manhole settlement.

Five categories of failure were identified which were settlement related and associated with the high compressibility of the opencast backfill and its sensitivity to moisture content changes.

(i) Pipe and pipe joint failure immediately adjacent to or under loaded structures due to shear forces and stress related settlement.

(ii) Pipe failures caused by differential movement at manhole connections. Both through pipes and backdrops were affected.

(iii) Backfalls and pipe failures due to bending stresses at trench crossings caused by localised settlements in trench backfill or trench formations.

(iv) Backfalls and pipe failures at boundaries between fill and undisturbed ground.

Records and on-site inspection showed ample evidence of precipitation causing collapse of trench backfill, groundwater movement through the site backfill and groundwater movement within the granular trench bedding. While most of the installed drainage has performed satisfactorily, considerable expense was incurred in reinstatement of failed sections.

CONCLUSIONS

The upper layers of opencast fill on this site appear to have relatively low compressibility which is probably due to the precompaction caused by landscaping operations and precipitation causing volume reduction in the near surface zone by wetting-up. The majority of the fill below this level is of medium to high compressibility and is the seat of settlement at existing in-situ moisture content as well as being the zone most susceptible to collapse settlement. The acceleration of settlements at constant stress which have been observed on this site can only be attributed to wetting-up. Any design should recognise the limitations of uncompacted opencast backfill sites and seek to provide and appropriate structure and sufficient flexibility in the drainage to accommodate settlements and differential settlements of the order experienced here without significant damage or loss of serviceability. On this site the flexible raft used on Scheme 1 and 2 has to a large extent achieved this. Surcharging such sites prior to construction will reduce total settlements, reduce variability and hence differential settlements and reduce but not eliminate susceptibility to wetting-up. Drainage design should incorporate steeper gradients rather than shallow gradients and backdrops, increased flexibility or sleeving where pipes exit from building foundations or manholes and where pipes pass closely under loaded areas. Increasing the bending strength of the pipes and the pipe diameter beyond that necessary for hydraulic design would also reduce the incidence of failures.

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