Settlement development caused by tunnelling in soil

by P.B. ATTEWELL*, PhD, DEng, FICE & M.R. HURRELL*, BSc

If fully-developed, ground-loss generated settlement profiles normal to a tunnel centre line are described in terms of a normal probability function then it seems a logical progression to assign a cumulative probability curve to the form of the centre line settlement profile. It is concluded from an examination of some case history evidence that there is practical justification for doing this.

Introduction

PREDICTIONS OF ground settlement above tunnels driven in soil have tended to concentrate on the permanent deformations which occur at right angles (transverse) to the direction of tunnel advance. It is also generally realised that there is a wave of movement ahead of and behind the tunnel face. Thus, the mechanical behaviour of the ground in the vicinity of the tunnel depends on the tunnel face position and is compounded from movements that are resolved into and measured along lines transverse to and parallel to the tunnel centre line. The origins of these respective movements, and particularly the relative contributions of face loss and radial loss as a function of tunnel size, form of initial support and rate of advance are discussed in some detail in Attewell (1978). Ground above the tunnel face settles generally as shown in Fig. 1.

For prediction of ground movements and strains attributable to ground losses at the tunnel, several assumptions need to be made. These usually include:

(i) The form of the transverse *y*-coordinate settlement distribution each side of the tunnel centre line can be described by a normal probability (Gaussian) function, with the maximum surface settlement w_{max} (*z*-coordinate) being equivalent to the mean value of a statistical distribution and the distance i_y from the tunnel centre line to the point of maximum ground slope being equal to the standard deviation of a statistical distribution. This description is one of predictive convenience. An actual transverse settlement profile may not satisfy this form and may not be symmetric about the tunnel centre line.

(ii) The area displaced by the transverse surface settlement trough can be predicted. This is the area under a normal probability curve and it will depend on the type of soil at and above the tunnel face, the presence or otherwise of soil water, the sealing of the tunnel itself and the contact grouting of the voids around the lining, and the length of time after passage of the tunnel face.

(iii) The tunnel centre line *x*-coordinate settlement development profile can be described as a cumulative probability function based on the same statistical mean (w_{max}) and standard deviation (i_y) parameters as define the transverse normal probability settlement profile. Thus it is assumed that $i_x = i_y = i$.

Ground movement and ground strain equations based on assumption *(iii)* were formulated by Attewell & Woodman (1982), and design curves based on those equations were subsequently published by Attewell & Yeates (1984). The purpose of the present Paper is to explore in a non-rigorous and preliminary manner the reasonableness of assuming that $i_x = i_y$.

Curve matching

Perhaps the simplest method of checking the validity of the assumed equality between i_x and i_y is to compare a transformed cumulative probability centre line (*xz* plane) settlement curve based on the i_y parameter

*Department of Engineering, University of Durham, South Road, Durham DH1 3LE

TABLE I: Percentage of maximum settlement as a function of position on the tunnel centre line settlement curve as expressed ideally in terms of the transverse settlement parameter i_y

(Note: it is assumed that 50% w_{max} will have developed directly over the tunnel face on this centre line settlement profile; $+x$ distances are ahead of the face position and $-x$ distances are behind the face position.)

Location on centre line settlement profile	*Approximate theoretical percentage of maximum settlement assuming that a cumulative probability function applies*
$+3i_y$	$0\% w_{max}$
$+2i_y$	$2.3\% w_{max}$
$+\ i_y$	$15.9\% w_{max}$
0	$50\% w_{max}$
$-\ i_y$	$84.1\% w_{max}$
$-2i_y$	$97.7\% w_{max}$
$-3i_y$	$100\% w_{max}$

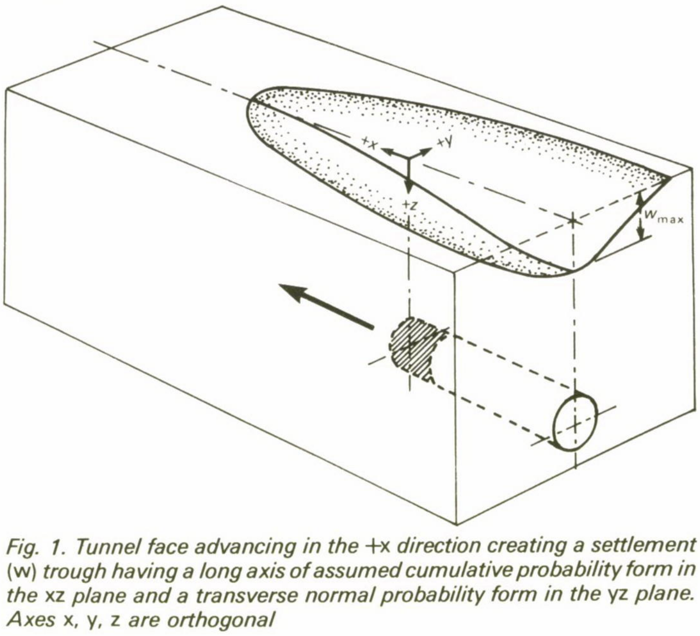

Fig. 1. Tunnel face advancing in the +x direction creating a settlement (w) trough having a long axis of assumed cumulative probability form in the xz plane and a transverse normal probability form in the yz plane. Axes x, y, z are orthogonal

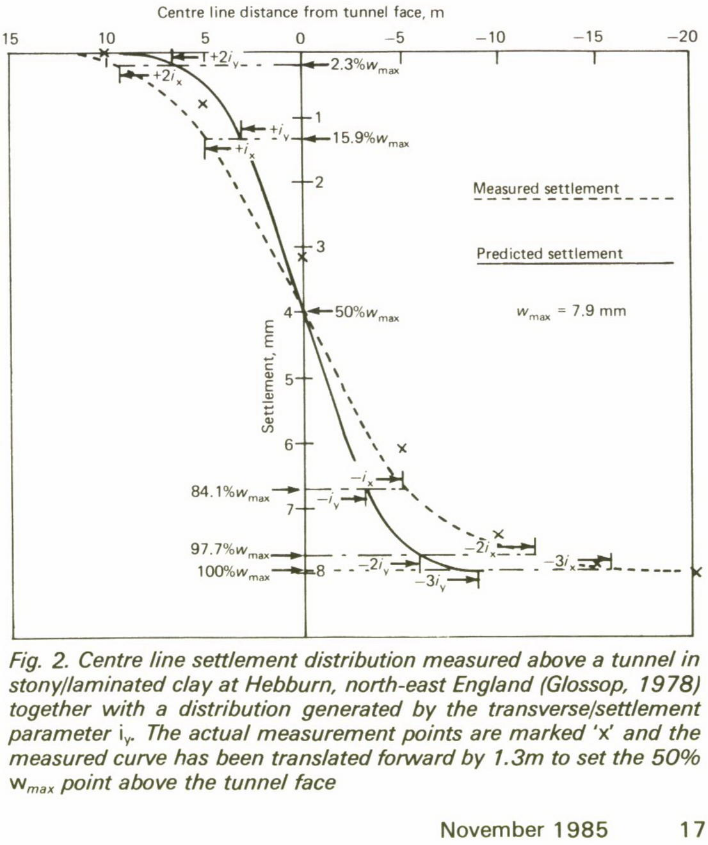

Fig. 2. Centre line settlement distribution measured above a tunnel in stony/laminated clay at Hebburn, north-east England (Glossop, 1978) together with a distribution generated by the transverse/settlement parameter i_y. The actual measurement points are marked 'x' and the measured curve has been translated forward by 1.3m to set the 50% w_{max} point above the tunnel face

with a curve fitted to measured centre line settlement data and to which an i_x value can be assigned. Table I and Fig. 2 show one method of doing this. In many instances the 50% w_{max} point on the measured centre line settlement development curve will not coincide with the tunnel face position ($x=0$). A precondition for subsequent measurements is that the origin of the x- coordinate system be set to the 50% w_{max} point for the transformed normal probability curve and the 50% w_{max} point on the measured centre line settlement profile be translated in x to achieve compatibility.

Discrepancies between the transformed and 'best fit' measured centre line settlement curves can be expected to develop at both low and high settlements. For comparison purposes, such discrepancies may be quantified in terms of:

$$i_y : \frac{+(3i_x)}{3} \; ; \; i_y : \frac{+(2i_x)}{2} \; ; \; i_y : +i_x \; ; \; i_y : -i_x \; ;$$
$$i_y : \frac{-(2i_x)}{2} \; ; \; i_y : \frac{-(3i_x)}{3}$$

These relations were originally examined by Gordon (1981). Because of the effect that the centre line settlement profile, and particularly the forward (of the tunnel face) settlement development, can have on buried pipes when a tunnel passes beneath and parallel to them this problem has been carefully re-examined in the light of the case histories listed in Table II. For most of these case histories original detailed site measurements were available for assessment. The results are shown graphically in Figs. 3-8 inclusive.

TABLE II: TUNNEL LOCATIONS

Case History number on graphs Figs. 3 to 8	*Tunnel Locations*	*Type of Ground*	*Reference*
1	Belfast, N. Ireland	Soft silt with overlying fill.	Glossop *et al.*, (1979)
2	Ouseburn, Newcastle upon Tyne, England.	Recent fill (rubble timber, household waste and ash) in a soft clay matrix.	Spencer (1978), Dobson *et al.* (1979)
3	Howdon, Newcastle upon Tyne, England.	Boulder clay.	Glossop (1978)
4	Willington Quay, Newcastle upon Tyne, England.	Silty organic alluvial clay.	Sizer (1976), Attewell *et al.* (1978)
5	Hebburn, Newcastle upon Tyne, England	Stony laminated clay.	Attewell and Farmer (1973)
6	Green Park, London, England.	London clay.	Attewell and Farmer (1974)
7	Grimsby B2, England.	Soft alluvial clay (marine 'warp')	Glossop (1980)
8	Grimsby B1, England.	do.	do.
9	Grimsby C, England.	do.	do.
10	Thunder Bay South, Canada.	Silt.	Morton and Dodds (1979)
11	Heathrow, England.	Upper ground section of London clay with 3.6m of clay cover to tunnel under wet gravel.	Wood and Gibb (1971), Smyth-Osborne (1971)
12	Norton, N.E. England.	Sandy silty clay.	Hurrell (1984)
13	San Francisco, USA.	Slightly cemented dense silty fine sand dewatered by deep wells.	Peck (1969)
14	Washington DC, USA.	Medium dense silty sand and gravel, interbedded with sandy silty clays.	Hansmire (1975)

Discussion

Settlement development actually measured above the centre of a tunnel as it advances rarely falls exactly on a well-defined sigmoidal curve. Several methods of curve fitting to isolated data points were attempted, but ultimately it was decided to perform a best-fit by inspection. This operation was undertaken independently by the two authors, with few differences in the choice of final fit because in most of the cases the data point scatter was low. However, because some scatter will always be present it does constitute one source of error.

In certain cases the measurement station from which the centre line settlement points were reported was displaced from, and therefore did not form part of, the transverse settlement array. This location difference could also contribute to any matching errors

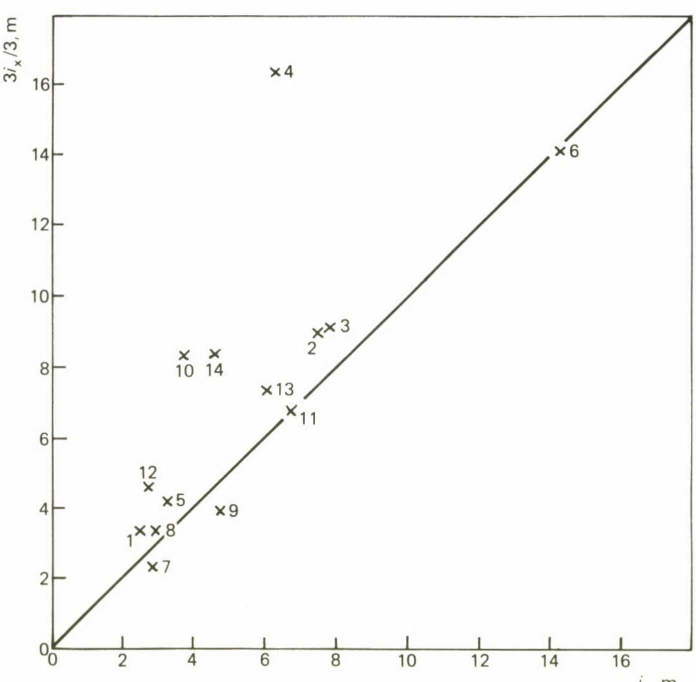

Fig. 3. Quality of fit between predicted and measured tunnel centre line settlement curves just at the onset of settlement

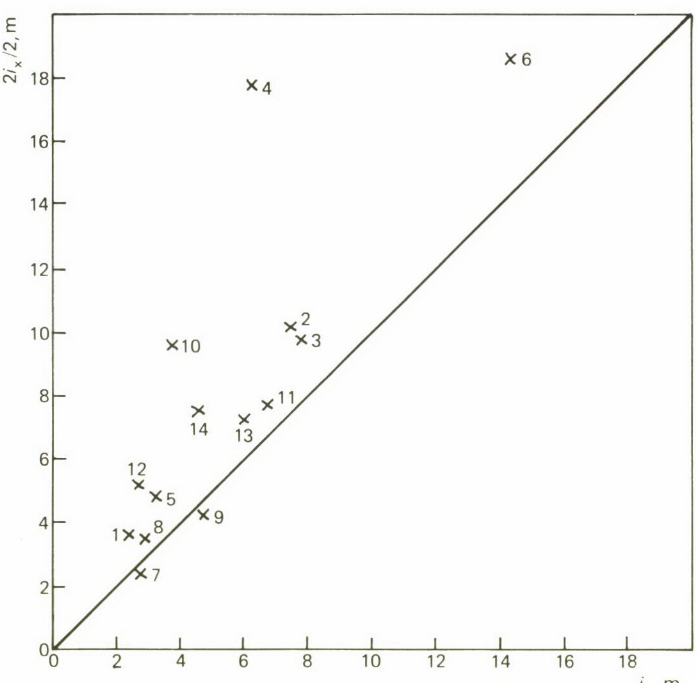

Fig. 4. Quality of fit between predicted and measured tunnel centre line settlement curves at the point where the settlement is 2.3% maximum

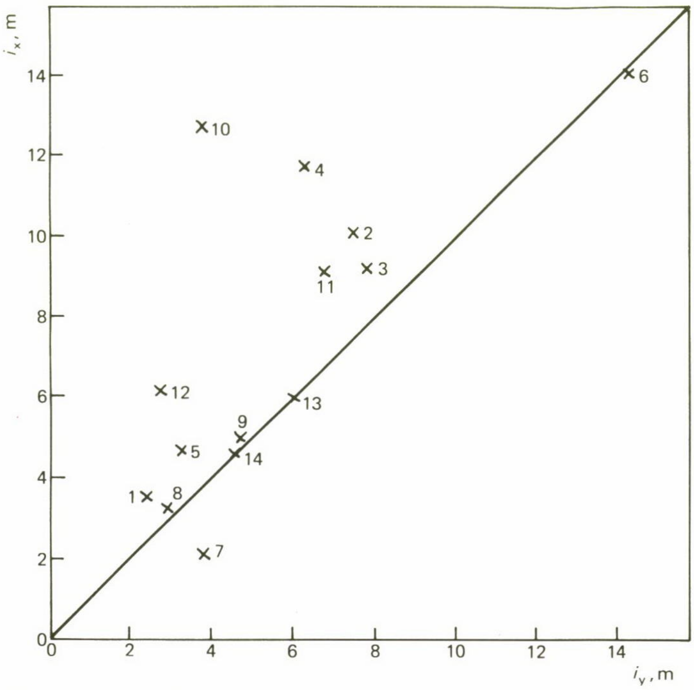

Fig. 5. Quality of fit between predicted and measured tunnel centre line settlement curves at the point where the settlement is 15.9% maximum

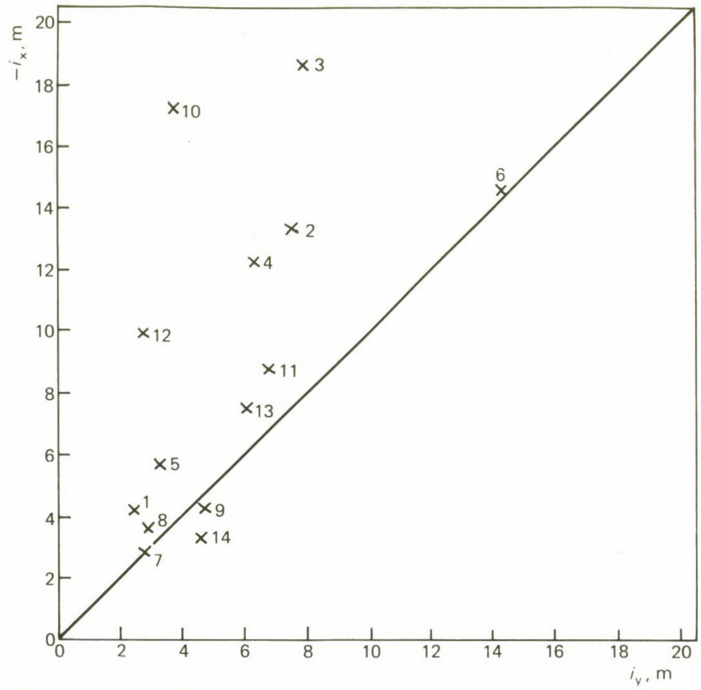

Fig. 6. Quality of fit between predicted and measured tunnel centre line settlement curves at the point where the settlement is 84.1% maximum

between i_x and i_y, particularly since there could be slight differences in the recorded maximum centre line settlements at the two locations.

The plots on Figs. 3-8 are seen generally to lie above the equality line. This means that the length of the measured centre line settlement field ($\frac{dw}{dx} \neq 0$) almost always exceeds the length ($6i_y$ metres) of the cumulative probability curve generated from the transverse settlement trough i_y parameter. The measured curve is closer to the predicted line ahead of the face than it is behind the face, where the incremental settlements are much attenuated.

However, the symmetry of the predicted ($i_x = i_y$) centre line settlement curves about the tunnel face position could only be satisfied if the tunnel actually behaved as the point source of linearly translating ground loss specified by Attewell & Woodman (1982) for their analyses. While it may be reasonable to idealise a tunnel face proper as a point source of loss for settlements ahead of the face, those continuing settlements above a lined tunnel owe their origins more to distributed radial losses at the tunnel, these losses being progressively inhibited and delayed as contact grout sets behind a segmental lining and the lining ring stiffness is fully mobilised. Because consolidation effects in clay soils should be additive to ground loss settlements, the differences between measured and predicted centre line settlement curves behind the face could be expected to be less than actually indicated by the points on Figs. 6, 7, 8 graphs.

The practical implications of the curve mismatches are not serious. Since all points in the vicinity of the advancing tunnel face experience the same wave of x-axis ground movement and strain the preliminary translation in x on the graphs may be ignored in any practical appraisal. Ahead of the tunnel face the ground around buried pipes and building foundations lying along the centre line is strained such that they experience their worst longitudinal (x-axis) tensions and superimposed bending tensions. In this area the curve mismatches are usually not large, and so any structural analysis based on i_y and an x-coordinate cumulative probability curve is reasonable. Behind the face, where the mismatches are

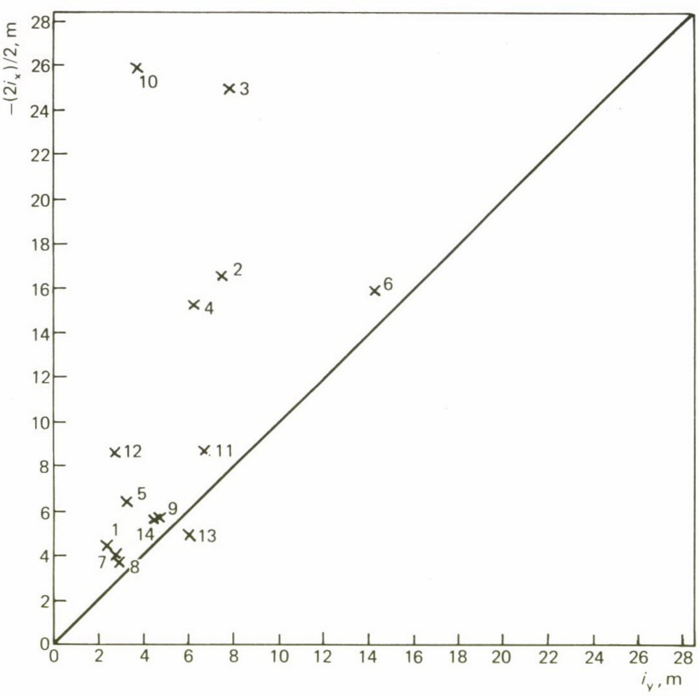

Fig. 7. Quality of fit between predicted and measured tunnel centre line settlement curves at the point where the settlement is 97.7% maximum

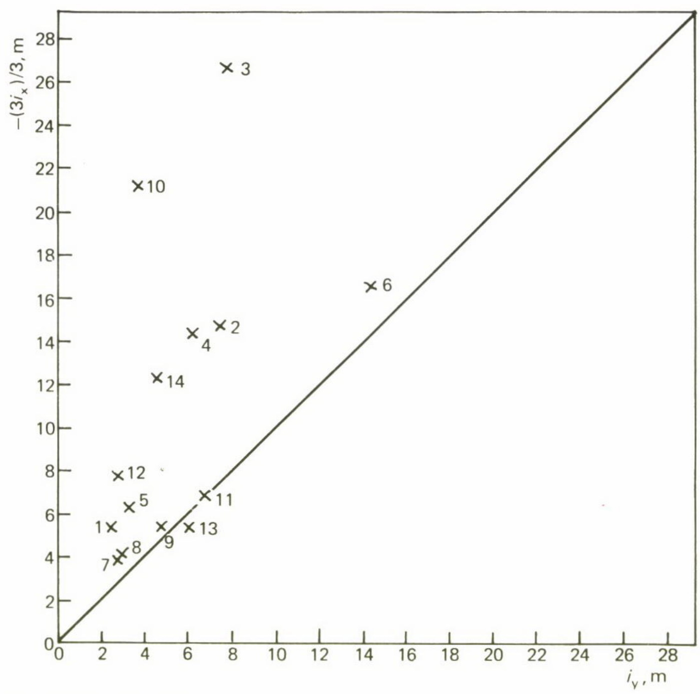

Fig. 8. Quality of fit between predicted and measured tunnel centre line settlement curves at the point where the settlement has maximised

greater, the cumulative probability centre line curve based on i_y predicts early post-shield settlements and rates of settlement that are greater than those measured. Accordingly, the predicted (and temporary for any given element of ground) radii of curvature are usually greater than those actually measured, and any structural damage assessment based on the equation generating the curve, or on the curve itself, would tend to be conservative*.

It is concluded that the adoption of parameter $i = i_y = i_x$ for the prediction of settlement and its derivatives parallel to a tunnel line is generally valid for most practical design problems.

References

Attewell, P.B. (1978): "Ground movements caused by tunnelling in soil." In: Proc. Conf. on Large Ground Movements and Structures, Cardiff, July 1977, Ed. J.D. Geddes, Pentech Press, London, pp.812-948

Attewell, P.B. & Farmer, I.W. (1973): "Measurement and interpretation of ground movements during construction of a tunnel in laminated clay at Hebburn, Co. Durham". Report to Transport and Road Research Laboratory, DoE, Research Contract No. ES/GW/842/68

Attewell, P.B. & Farmer, I.W. (1974): "Ground deformation resulting from shield tunnelling in London Clay". Can. Geotech. J., Vol. **11**, pp.380-395

Attewell, P.B., Farmer, I.W. & Glossop, N.H. (1978): "Ground deformations caused by tunnelling in a silty alluvial clay." *Ground Engineering,* Vol. **11** (8), pp.32-41

Attewell, P.B. & Woodman, J.P. (1982): "Predicting the dynamics of ground settlement and its derivatives caused by tunnelling in soil." *Ground Engineering,* Vol. **15** (8), pp.13-22,36

Attewell, P.B. & Yeates, J. (1984): "Tunnelling in soil." In: Ground Movements and their Effects on Structures, Eds P.B. Attewell & R.K. Taylor, Surrey University Press, London, pp.132-215

Dobson, C., Cooper, I., Attewell, P.B. & Spencer, I.M. (1979): "Settlement caused by driving a tunnel through fill." Proc. Midland Geotech. Soc. Symp. on the Engineering Behaviour of Industrial and Urban Fill, Birmingham, pp.E41-E50

Glossop, N.H. (1978): "Soil deformations caused by soft ground tunnelling". PhD thesis, University of Durham.

Glossop, N.H. (1980): "Ground deformation caused by tunnelling in soft ground at Grimsby." Internal Report, Dept. of Mining Engineering, University of Newcastle upon Tyne

Glossop, N.H., Saville, D.E., Moore, J.S., Benson, A.P. & Farmer, I.W. (1979): "Geotechnical aspects of shallow tunnel construction in Belfast estuarine deposits." Tunnelling '79, Proc. 2nd Int. Conf., London. Ed. M.J. Jones, Published IMM, pp.45-50

Gordon, R.I.G. (1981): "Ground movement associated with soft ground tunnelling and its effects on buried services." Dissertation: M.Sc. Advanced Course in Engineering Geology, University of Durham

Hansmire, W.H. (1975): "Field measurements of ground displacements about a tunnel in soil". PhD thesis, University of Illinois, Urbana, USA

Hurrell, M.R. (1984): "Results of a programme of monitoring ground and pipe response to shield-driven tunnelling in soft ground at Norton, Stockton-on-Tees, Cleveland". Report to Northumbrian Water Authority and the Water Research Centre, Swindon, by the Department of Engineering, University of Durham

Morton, J.D. & Dodds, R.B. (1979): "Ground subsidence associated with machine tunnelling in fluvio deltaic sediments, Part 2." Tunnels and Tunnelling, November '79, pp.23-28

Peck, R.B. (1969): "Deep excavations and tunnelling in soft ground". State of the Art Report, Proc. 7th Int. Conf. on Soil Mechanics and Foundation Engng., Mexico City, pp.225-290

Sizer, K.H. (1976): "The determination and interpretation of ground movements caused by shield tunnelling in silty alluvium at Willington Quay, North East England". MSc thesis, University of Durham

Smyth-Osborne, K.R. (1971): Discussion on Muir-Wood & Gibb (1971). Proc. Inst. Civ. Engrs., Vol. **50**, pp.187-203

Spencer, I.M. (1978): "Soft ground tunnelling on contracts 16A and 276 of the Tyneside Sewerage Scheme." Dissertation: MSc Advanced Course in Engineering Geology, University of Durham

Wood, A.M. Muir & Gibb, T.R. (1971): "Design and construction of the cargo tunnel at Heathrow Airport, London". Proc. Inst. Civ. Engrs., Vol. **48**, pp.11-35

*Any tensile bending strains in the lower fibre of a buried structure will tend to be resisted by the direct (both *x*-axis and *y*-axis) compressions on the surrounding soil. This will increase the conservatism of any assessment based on bending only. The response of buried pipes is discussed in Attewell & Yeates (1984).

Ground engineering plant & equipment

Mobile 100ton drilling rig for very large diameters

TO MEET A DEMAND for a more powerful mobile drilling rig, Anderson Manufacturing and Drilling have developed "Big Stan" (see Front Cover), to provide the capability of boring up to 26ft diameter and 260ft deep.

Anderson Drilling was set up in 1945 as drilling contractors. Since the Manufacturing division was added in 1961, it has modified or built completely some 56 drilling rigs. "Big Stan" actually consists of two major units which travel independently but are assembled on site by in-built hydraulic equipment. One section is mounted on a five-axle carrier, while the upper tower and engine assembly are transported by a seven-axle truck-trailer outfit. Assembly of the units is claimed to take two men 30 minutes, resulting in a 100ton rig powered by a 600hp KTA 1150 Cummins diesel engine.

Rotary Drive

The double-pinion 48in dia. ring gear of the rotary is driven via a six-speed Allison transmission, a Clark final drive and twin 14in drive belts. Maximum torque is 534 000ft.lb. Standard equipment includes two 75ft long kelly bars, with 13in square outer shaft and 10in square inner. The tower stands 97ft high in the raised position and is capable of lifting 35 000lb, the maximum crane radius being 53ft. An inter kelly winch provides a single line pull of 42 000lb, with 1⅛in cable.

A downthrust of 70 000lb is provided by an endless reaving winch. The tower can be rotated through 360deg. and provides a reach of 13ft from the rear to the centre of drilled hole. Drill buckets with capacities up to 8yd^3 each pass can be handled. Height of the rotary above ground is 14ft, while the tail swing height is 7ft. Slewing action is provided by two KYB 5 piston motors, and controlled by an 8ft dia. mechanical brake.

Carrier unit

The 40ft long carrier unit is equipped with five hydraulic jacks, four in pairs on outriggers which can be extended to provide 30ft lateral support, and a front jack with a travel of 6ft for mating and levelling operations. A 9ft long stroke hydraulic ram provides fore-and-aft slide movement, plus the mating operation.

Further details can be obtained from: D.W. Fort, Director of Sales, Anderson Manufacturing, 10303 Channel Road, Lakeside, California 92040, USA.

10tonne vibratory compactor

A NEW 10tonne self-propelled vibratory compactor incorporating several features as standard which are normally found on larger machines was launched at the 1984 International Construction Equipment exhibition, Birmingham.

Produced by Hamm Walzenfabrik GmbH, Erthalstrasse 1, D-6500 Mainz, West Germany, the debut of this roller is of particular significance to Hamm since it is the first new development to come out of its Tirschenreuth factory following the company's acquisition by a private consortium following the collapse of its parent company, IBH.

Hamm, which has designed, engineered and manufactured rollers for over 100 years, has a range that totals 16 rollers with 1.6-32tonne operating weights and varying capabilities for all kinds of earth and road consolidation work. There are nine in the vibrating class, which include the tandem drum and the self-propelled types, and seven in the "static" class. Non-vibrating, this type includes rubber-tyred rollers (PTR) and traditional three-wheeled rollers.

The new 10tonne vibratory compactor, designated the 2410-S, has a rolling width of 2 100mm, a drum diameter of 1 500mm and is capable of exerting a 30tonnes compaction force. By having variable frequencies, this machine can compact soils as well as black-top. The high amplitude will, it is claimed, compact thick fills in depth and, in combination with a 30Hz frequency, ensure a high compaction density result with the minimum number of passes.

Another standard feature is that this roller is provided with a drive on both its drums and rear wheels. This is hydrostatic, infinitely variable, has two speed ranges and is controlled by a single lever. Working speed is 0-9km/hour, while travelling speed is 0-17km/hour.

The compactor is powered by a Deutz direct injection six-cylinder diesel engine rated at 103hp at 2 500rpm.

Contributing to operator comfort, and hence output, the cabin is soundproofed to 80dB(A) and is independently mounted to reduce vibration to a minimum.

During the five-day ICE exhibition, Hamm report that 30 of the 2410-S rollers were sold as well as many other machines in its range to both UK and European markets.