Ground and building settlement due to tunnelling in Hong Kong

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Abstract Extensive monitoring of the ground and buildings affected by the construction of the Island Line (ISL) of the Mass Transit Railway (MTR) in Hong Kong was carried out between 1982 and 1986. Predictions were based on the Peck method (Peck, 1969) and the monitoring enabled a comparison of prediction and performance to be made. The results suggested that tunnels in completely weathered granite behaved as in Peck’s sands (cohesive granular soils) whilst the shallow tunnels in colluvium, alluvium or marine clays behaved like Peck’s soft clay. This paper reports the results of some of the monitoring and provides a tunnelling settlement estimate method suitable for Hong Kong.

INTRODUCTION

Between 1982 and 1986 the Island Line (ISL) of the Mass Transit Railway (MTR) was constructed along the northern part of Hong Kong Island from Chai Wan in the east to Sheung Wan in the west. The alignment followed some of the busiest roads in this densely populated city and in order to minimize disruption to normal life, a decision was made to construct the bulk of the route underground by tunnelling methods.

Twin running tunnels, 6 m in diameter, and station tunnels, 8 m in diameter, were constructed using a variety of methods, principally tunnelling shields and compressed air in soft ground, hand mining methods in mixed ground, and conventional drill and blast techniques in rock. The station tunnels were connected to 12 new station concourses by adits. The concourses were excavations up to 35 m deep, often constructed by diaphragm wall techniques.

An extensive site investigation carried out before the commencement of construction revealed a geological succession consisting of a variable thickness of reclamation fill overlying marine deposits, alluvium, colluvium and weathered granite, before encountering fresh granite. A typical geological profile is shown in Fig. 1.

BUILDING CLASSIFICATION AND MONITORING

In most cases tunnelling and concourse excavations were carried out in very close
proximity to buildings. As a result a comprehensive building monitoring programme was carried out. Buildings were classified prior to construction of the ISL according to age and condition, and predictions were made of settlement and distortion, based on the Peck method (Peck, 1969), and hence structural damage. In a few cases buildings likely to have been severely damaged were demolished, although this was only carried out in exceptional circumstances on very old buildings on shallow footings.

The building classification adopted by the MTR Corporation is shown at Table 1. The condition of each structure was also noted on a four point scale.

Table 1 Building classification adopted by the MTR Corporation.

<table>
<thead>
<tr>
<th>Type</th>
<th>Age</th>
<th>Size</th>
<th>Foundations</th>
<th>Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>pre World War II</td>
<td>&lt;4 storeys</td>
<td>Spread footings or wooden piles?</td>
<td>Mainly brick or masonry load bearing walls</td>
</tr>
<tr>
<td>B</td>
<td>late 1940s to early 1960s</td>
<td>4-6 storeys</td>
<td>Spread footings or timber piles?</td>
<td>Often built in terraces using cross-wall infill to a r.c. frame.</td>
</tr>
<tr>
<td>C</td>
<td>early 1950s to mid 1960s</td>
<td>&gt;6 storeys</td>
<td>Usually concrete piles founded in or on weathered granite</td>
<td>r.c. framed buildings</td>
</tr>
<tr>
<td>D</td>
<td>since mid 1960s</td>
<td>&gt;15 storeys</td>
<td>Deep concrete or driven piles to weathered or fresh rock</td>
<td>r.c. framed buildings</td>
</tr>
</tbody>
</table>

TUNNELLING EFFECTS

Settlement data

Figures 2 and 3 present the ground and building settlements recorded above the centreline of a 5.8 m diameter tunnel constructed using a shield and compressed air. The
settlements are due to tunnelling only and were measured prior to decompression of the drive. A geological profile determined from site investigation boreholes is also shown in Fig. 2. The data are related where possible to site investigation information as it is on this that the original designs and estimates are based. Noticeably from the chainage-settlement plot in Fig. 2, the settlements appear to follow a definite trend which is dependent on their location. Accordingly the settlements have been separated into Groups A to E according to their relative magnitudes and location. These five locations are shown as cross-sections in Fig. 3 with all building and ground points included for each group. The curves shown in Fig. 3 represent gaussian curves of best fit calculated by the method of least squares.

Figure 4 shows for a 5.8 m diameter tunnel (shield and compressed air) the building and adjacent ground settlement resulting from the driving of the tunnel below the buildings central piles. The excavation of the tunnel involved cutting off several of the pile-tips below the central raft. The settlement shown is prior to decompression. Preventive works to the building foundation included grouting of the fill strata below the central pile cap and structural extension of the cap itself.

Figure 5 presents a time-settlement plot of three ground points above the centreline of a 6.3 m wide horseshoe-shaped tunnel constructed using NATM methods. Support
to the tunnel was steel-arch ribs and shotcrete and the drive was advanced using a top
ring cut and bench. The geology of the face ranged from marine deposits overlying rock.
The excavation was undertaken in free air through an annulus of grouted ground.
Cross-sections showing settlement and distance from the tunnel centreline for each of
the three locations are also included in Fig. 5. The best fit gaussian curves are shown
for the data points at each section. Figure 6 shows a summary of the settlement data
from the same drive with plots of chainage against maximum settlement ($S_m$), crown
settlement measured at a point on the crown of the arch ribs after installation ($c$), and
percentage equivalent face loss (shown as $A\%$). The calculated $i$ values expressed as a
function of tunnel depth and "diameter" are also shown in Fig. 6 plotted in the form
proposed by Peck (1969).

Percentage equivalent face loss is the area of the surface trough expressed as a
percentage of the tunnel cross-sectional area. This parameter is more aptly called the
tunnel settlement ratio by Cater et al. (1984) as it represents a relative area rather than
a specific loss into the tunnel. For the purpose of this paper, however, it is referred to
as the percentage face loss (EFL) for reasons of consistency.

The tunnelling settlement and decompression settlement due to two 5.8 m diameter
tunnels constructed by hand mining under compressed air is presented in Fig. 7. Both
ground and building data are included although no ground settlement data due to tunnelling alone were available. The best fit gaussian curve for the building points is included and in addition a best fit logarithmic curve for the decompression settlements. Published data to date (Cater et al., 1984) have suggested that decompression settlement, due to the migration of pore water back towards the tunnel on the release of the air pressures, forms a wider trough than the pure tunnelling settlement trough. No theory has been published to indicate a means of estimating the likely magnitudes or width of this trough in Hong Kong conditions. The reason for this is that all piezometers are backfilled prior to compressed air work to prevent water ingress to, and air escape from, the tunnel and consequently the behaviour of the groundwater during compressed air tunnelling is unknown. It would seem logical, however, that the settlement on decompression is due to groundwater movement towards the tunnel and therefore it may take the form of a classic dewatering curve which is generally a logarithmic function relating distance to head. Obviously the air pressure behaviour and consequent pore water migration is dependent on the specific geology and soil permeability of the tunnel alignment as shown in the data presented by Cater et al. (1984). However, the logarithmic curves in Fig. 7 show a reasonable agreement with the data. (The two curves
for each of the tunnels have been visually aligned at the centre-lines, as is the case with a normal symmetric dewatering profile.)

**ANALYSIS OF GROUND SETTLEMENT DATA**

**Forms of magnitude of settlement**

The data presented in Figs 2 to 7 represent the measured effects of various types of tunnelling on both ground and buildings. The estimates of settlement for these tunnels were originally based on the methods proposed by Peck (1969) with the parameters...
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assumed for Hong Kong conditions based on the previous experience of the MTRC and their consultants. Cater et al. (1984) have updated and refined these methods of estimation and proposed values of tunnelling settlement ratio (equivalent face loss) and values of \( i \) based on the tunnelling undertaken on the ISL. The relevant findings presented in their paper are:

- the proposal of values of "EFL" or "settlement ratio" which are related to the face conditions (degree of support to a face and crown is dependent on both the overall tunnelling method and the face geology and then surface trough area is suggested by Peck to be governed by this);
- the proposal of a value of \( i \) for Hong Kong to be equal to the tunnel depth divided by 2;
- highlighting of the phenomena of decompression settlement as distinct from tunnelling settlement and the provision of data to show the shape of the trough after each stage.
Considering the data in Figs 2 to 7, the initial conclusion is that the Peck method is indeed applicable to soft-ground tunnelling in Hong Kong. Figures 3 and 5 show reasonable agreement between the settlement data and a curve of best fit based on Peck's gaussian error curve. (In Fig. 3 the Group E curve is appreciably wider than for the other groups and this often indicates some dewatering effects during tunnelling or possibly the effects of outside sites.) Also the cross-sectional areas of the troughs presented agree with the values reported by Cater and also those used for estimating to date. The curves in Fig. 3 represent equivalent face losses (EFL) of 0.6% to 1.9% which agree with the range of 0% to 3.2% proposed by Cater for shield tunnels in both alluvium and CWG. Similarly, the curves in Fig. 5 represent EFLs of 0.2% to 0.8% compared to the range of 0 to 1% reported (although for compressed air). The twin-tunnel data in Fig. 7 has an EFL of 1.8, or 0.9% per tunnel, compared to the published range for a NATM driven tunnel under air of 0 to 1% or a shield driven tunnel under air of 0 to 3.2%. Therefore it would seem reasonable to assume that Peck curve describes the form of the settlement trough above tunnels in Hong Kong soils and that the area of this trough can be related to the data published to date.

**Surface trough parameters**

To obtain a surface trough's shape, given its area, two parameters need to be defined; maximum settlement ($S_m$) and width ($i$). As previously described, Peck proposes the
parameter $i$ to be dependent on soil conditions and tunnel size and depth. Given this relationship, the maximum settlement is therefore interrelated to trough area and is dependent on tunnelling methods and the degree of support. This relationship of equivalent face loss or trough area to tunnelling methods is described above and appears to be now reasonably well defined for Hong Kong conditions (Cater et al., 1984). However along any tunnel drive the likely EFL will change with changing face conditions, as is evidenced by the chainage-settlement plots in Figs 2 and 6. The EFL of Group B points in Fig. 2 (refer also to Fig. 3) is 1.4% where the face intersects both alluvial and CWG material. In the group C and D areas, however, the EFL is 0.6% to 0.9% (Fig. 3) as the tunnel progresses through CWG only. Similarly, in Fig. 6 the plot of trough area (shown as $A_\%$) follows the crown settlement plot ($C$) along the tunnel drive but only approximately follows the maximum settlement trend ($Sm$). This is significant in that a direct measure of tunnel crown movement (albeit only after the support measure is installed) can be related to EFL. The variation compared to the
maximum settlement is because of the variation in $i$ values at each cross section as shown in the $2i$ vs. $Z$ plot, also in Fig. 6.

**Width of surface trough**

The determination of the value of $i$ for Hong Kong conditions is less well documented. Cater *et al.* (1984) have proposed that $i$ is equal to half the tunnel depth ($Z$). An earlier proposal (Howat & Cater, 1983) was that $i$ was equal to depth $Z$, although in the latter paper by Cater *et al.* (1984) they note that changing geology and possibly dewatering effects may have contributed to the wider troughs. Until now it has been generally considered that Hong Kong conditions approximated to the Peck "sands below groundwater level" zone (Peck, 1969). The $i$ equal to half the depth line described above partly follows this Peck limit. A set relationship, however, between $i$ and $Z$ assumes that the stress-strain relationship causing the location of the troughs point of inflexion is the same for all soil types encountered in Hong Kong. It is thought that the location of the point of inflexion $i$ (Mair, 1983) and therefore the development of this strain pattern depends on the soil properties and stress distributions around the tunnel face and crown. It is possible therefore that for any given depth $Z$, the $i$ value should be chosen depending on the face properties and their classification according to Peck’s general soil types. To address this point Fig. 2 shows a plot of data from Figs 2 to 7 and also published data relating depth $Z$ to width $i$. The Peck soil zones are included for reference. The mean line represents a line of best fit for all the points derived in Hong Kong. This line corresponds to an $i$ value equal to 0.6 tunnel depth and in that respect compares well with previously published data (Rankin, 1988; Mair *et al.*, 1993). However, for individual drives there appears to be a spread as evidenced by the plot in Fig. 6. This tunnel was of a constant depth and the gaussian curves of best fit showed varying $i$ values along its length. Similarly, $i$ values at specific sections of the drive represented by Fig. 3, and $i$ values from a drive of twin tunnels in CWG all show a spread at a constant tunnel depth (Fig. 7). This spread may be attributed to the spread of the original settlement data or it may indicate that, because of soil properties, there is a variation in $i$ for a constant depth tunnel. In this latter case Hong Kong soils will not exhibit a single $i$, $Z$ relationship but a band of values more in keeping with Peck’s bands for differing soil types. The data in Fig. 7 do not readily group themselves into classifications such as Marine soils and weathered granite soils. However, in many instances this is because of mixed soil conditions at the face. Furthermore, for a typical Hong Kong reclaimed land profile in CWG, whereas shallower tunnels will in many areas be located in the fill/marines/colluvium/alluvium strata (Fig. 1). Therefore a shallow tunnel, say $Z/D$ less than 2 or 3, and with $i$ equal to half the depth, will have an $i$ value of 4 to 9. This places the points right on the limit between Peck’s submerged sands and soft clays. A deeper tunnel, however, with $Z/D$ equal to say 5 or more, will have for $i$ equal to half the depth, an $i$ value of 15, which is well within the submerged sands zone defined by Peck. A unique relationship for Hong Kong may, therefore, be a result of the stratigraphy in that for the deeper CWG tunnels the settlement behaviour is more like Peck’s sands (also described as cohesive granular soils) and for the shallow depositional strata tunnels the settlement behaviour is closer to Peck’s soft clays behaviour. Without additional well-documented case histories it is difficult to further
determine the relationships for Hong Kong soils. The relationship by Cater et al. (1984) and the mean relationship shown in Fig. 7 agree reasonably well, however, keeping in mind the usual scatter of settlement points above a tunnel (Fig. 3), this line of $i$ equal to half the tunnel depth should be considered reasonable for estimating purposes in Hong Kong soils. Data do indicate however that the limits could be from $i$ equal to 0.4Z to $i$ equal to Z.

**Building settlement**

The previous sections consider mainly the ground settlement above a tunnel. Settlement of buildings is influenced not only by ground surface movement but also by ground movement at the location of the foundations, be they piles or footings. Building data are shown in Figs 3, 4 and 7. From Fig. 3 it is evident that generally building settlements follow ground settlements. The exception is for the wider trough shown as Group E. In all cases the buildings shown are on piles founded above or close to the tunnel level. No relationship appears to exist between foundation depth and settlement for this drive except that for all buildings the magnitude and form of settlement can be said to approximate to ground settlement. It is possible that the lower building settlements recorded for Group E may be due to a larger $i$ value at this location. If this is due to dewatering effects the stresses causing settlement may have less effect on a piled building than on the ground, especially if the dewatering was less at the pile founding level.

Figure 4 shows building and ground settlement due to a shield tunnel intersecting the buildings piles. It is of interest that building settlement and ground settlement troughs are approximately the same width but with the building suffering approximately 70% greater settlement at the centre-line. This suggests that the ground movement at tunnel level and just above may be greater than indicated by surface movement and therefore the downward force extended on the piles, or the pile-tip movement, results in a larger settlement. The similar width troughs may be a result of similar development of strain patterns close to the tunnel as at the surface, which suggests that the bands of maximum strain achieve their maximum width in the area just above the tunnel, or alternatively the building has redistributed some load across its width. Figure 7 shows ground and building settlements due to decompression. No ground data due to tunnelling were available for this drive so this aspect cannot be studied. On decompression, however, it is noted that generally, within the limits of the data, the building movement approximated to ground movement. There again appears no definite relationship to foundation level for these data. The several points showing zero decompression adjacent the E tunnel axis (Fig. 7) were in a zone where annulus grouting had been carried out around the tunnel and therefore decompression or dewatering effects were possibly prevented.

**CONCLUSIONS**

From the data analysed, and the information and proposed methods available from other publications, a tunnelling settlement estimate method that appears suitable for Hong Kong conditions is as follows:
For a given tunnelling method and likely face conditions, choose an equivalent percentage face loss (EFL) or tunnel settlement ratio (refer to Cater et al., 1984).

Using the above value and the tunnel cross-sectional area ($At$), calculate the area of surface trough ($As$) from:

$$EFL = \frac{As}{At} \%$$  \hspace{1cm} (1)

Assume a value of $i$, say equal to half the tunnel depth ($Z$), and calculate maximum settlement ($Sm$) as follows:

$$i = \frac{Z}{2}$$  \hspace{1cm} (2)

$$As = 2.5i Sm$$  \hspace{1cm} (3)

Obtain surface settlements ($S$) at any distance ($X$) from the tunnel centreline according to equation (4):

$$S = Sm \exp \left(-\frac{X^2}{2i^2}\right)$$  \hspace{1cm} (4)

This method applies to ground settlements. For building settlements, considering the maximum values likely of only 5 to 15 mm, it seems reasonable to assume that they can be approximated in magnitude and form to ground settlements.

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REFERENCES


