WEAK CLAYSTONE FLOORS AND THEIR IMPLICATIONS TO PILLAR DESIGN AND SETTLEMENT

by

Ross W. Seedman¹ and Nick Gordon²

ABSTRACT

The in situ behaviour of claystone floors associated with the Wallarah, Great Northern and Fassifern Seams has been studied using a comprehensive suite of stress and displacement monitors. The instrumentation was supplemented by in situ and laboratory testing of the claystone materials.

The instrumentation demonstrated that the presence of claystone floors is associated with floor heave, extra rib spall, and settlement of the pillar into the floor. Horizontal floor extensometers showed that the depth of extrusion of clay was limited to 1-2 m. A pore pressure transducer installed in a claystone floor demonstrated the possible existence of short-term undrained behaviour of the claystone. Long term swelling of distressed claystone was also documented.

Settlement of pillars into claystone floors can be analysed using simple elastic theory. The assumption of plane strain is valid for most claystone thickness/rib gap width aspect ratios. Floor heave can be analysed using bearing capacity concepts. However, it is considered that heave is the result of local bearing capacity failure in the rib spall zone. The advantage of this model is that the bearing capacity equation can be applied without resort to unrealistically high factors of safety.

INTRODUCTION

The Newcastle Coal Measures of New South Wales, Australia, contain a series of volcaniclastic rocks that have been called "buff" and are locally referred to as "claystones". These claystones are widespread and tend to occur immediately above or below coal seams. Of particular concern are the claystones associated with the economic reserves of steaming coal in Wallarah, Great Northern and Fassifern Seams.

Both roof and floor instability have characterized mining these seams in areas where the claystone is present. Mining issues related to the presence of claystone have been examined by Otten (1984) and Seedman (1988a,b) discussed their engineering properties in detail and outlined failure mechanisms. In 1989, ACIRL was awarded a NERDDC grant to continue the earlier work by conducting mine-site investigations to confirm failure mechanisms for the floor and propose design techniques to assist in maintaining economic extraction of the valuable steaming coal resource.

CLAYSTONE PROPERTIES

DRAINED AND UNDRAINED BEHAVIOUR

The claystones of the Newcastle Coal Measures are a highly variable group of materials. Their complexity is in part due to the inappropriate nomenclature used in the past. Some of the claystones can be highly silicified and hence chert-like in appearance. Such material is of very high strength and modulus and will not be considered further. Other claystones are noticeably very weak; they can be readily scratched with a fingernail and more significantly when placed in water will decompose (or slake) to form a liquid mud.

It is suggested that the problem claystones of the Newcastle Coal Measures can be looked upon more as overconsolidated clay-soils and that resort can be made to the soil mechanics literature to find explanations for their behavior. A feature of saturated clay soils is the change in their engineering behaviour over time which can be...
related to the movement of pore water. It is well known that clays have low permeability and it is this low permeability that can be used to introduce the concept of undrained and drained behaviour of clay.

Consider a saturated clay soil whereby there is water between the clay particles (note that this is not a bad model even for the Newcastle claystones). When an increment of stress is applied to the saturated clay, the stress is immediately carried by the pore water and the pore pressure rises. There will be some deformation of the clay due to deformation of the clay macrostructure but this will be relatively small. Since water cannot transmit shear stresses, the shear strength of the clay does not change. In fact, if one were to do a rapid triaxial test, the friction angle would be zero. Poisson’s ratio would be found to be equal to 0.5. Furthermore the cohesion (or in this case the undrained shear strength) would be half the measured unconfined compressive strength. This is called undrained behaviour.

Over time, and depending on the permeability and path length, the pore water which is under pressure will flow and the pore pressure will reduce. The incremental stress will then be transferred to the clay structure. The clay will consolidate and gain strength. In a slow triaxial test, the measured cohesion will decrease compared to the rapid test and the friction angle will not be zero. Given the extra consolidation that occurs, the deformation modulus is lower. This is called drained behaviour.

LABORATORY TESTING

Seedsman (1988a) summarises the properties of a typical claystone from the floor of the Great Northern Seam. Log pressure/void ratio and stress/strain plots are reproduced as Figure 1. Key results of the laboratory program were:

- Water contents of 7-14%
- One dimensional free swell of 21%-41%
- Swelling pressures in excess of 3.5 MPa
- Non-linear drained stress/strain plots
- Drained modulus averaging 200 MPa
- Undrained modulus averaging 1100 MPa
- Unconfined compressive strengths of 4 MPa-27 MPa
- Drained cohesion averaging 1 MPa
- Friction angle averaging 27°

Figure 1(a) - Log (pressure)/void ratio plot of free swell and constant volume tests.

Figure 1(b) - Typical stress/strain and stress path plots during (a) triaxial and (b) uniaxial compressive tests.
MINE SITE INVESTIGATIONS

SUMMARY

Three mines were studied being representative of conditions in the three seams of concern and the three mining companies operating in the area. A summary of the geological conditions at the three sites is given in Table 1. At each site a range of stress and displacement instruments were installed with emphasis being placed on instruments that could be remotely read. Full details are given in Seedman and Gordon (1991).

Table 1
Summary of Geotechnical Properties at the Instrumented Sites

<table>
<thead>
<tr>
<th></th>
<th>Chain Valley</th>
<th>Cooranbong</th>
<th>Wyee</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seam Mixed</td>
<td>Walfarah</td>
<td>Great</td>
<td>Fassifera</td>
</tr>
<tr>
<td>Roof Type</td>
<td>Conglomerate</td>
<td>Sandstone</td>
<td>Claystone</td>
</tr>
<tr>
<td>Roof Strength</td>
<td>44MPa</td>
<td>46MPa</td>
<td>58MPa</td>
</tr>
<tr>
<td>Coal Floor Thickness</td>
<td>0 - 300mm</td>
<td>0 - 300mm</td>
<td>1200mm</td>
</tr>
<tr>
<td>Coal Bed Seed</td>
<td>2.5GPa</td>
<td>2.5GPa</td>
<td>2.5GPa</td>
</tr>
<tr>
<td>Claystone Floor Thickness</td>
<td>1.8m</td>
<td>2.4m</td>
<td>2.6m in the immediate 2m</td>
</tr>
<tr>
<td>Claystone Floor Modulus (undrained)</td>
<td>15.6GPa</td>
<td>38.6GPa</td>
<td>90GPa</td>
</tr>
<tr>
<td>Claystone Floor Modulus (11MPa)</td>
<td>(11MPa, 17.7)</td>
<td>(2.6 - 6.4)</td>
<td></td>
</tr>
<tr>
<td>Claystone Floor Strength (undrained)</td>
<td>101MPa</td>
<td>22.6MPa</td>
<td>32.7MPa</td>
</tr>
<tr>
<td>Claystone Floor Strength (11MPa)</td>
<td>(13 - 23.4)</td>
<td>(8.5 - 11.6)</td>
<td></td>
</tr>
<tr>
<td>Depth of Cover</td>
<td>1.40m</td>
<td>1.30m</td>
<td>1.10m</td>
</tr>
</tbody>
</table>

* Seedman (1998)

KEY FINDINGS

Table 2 summarises the key findings of the three instrumentation exercises. At all three sites floor heave was measured with associated horizontal extension of claystone from beneath the pillar. However this extension extended for only 1m - 2m beneath the pillar.

The data obtained from the instrumentation at Cooranbong was particularly informative (Figures 2 and 3). During passage of longwall 1, the floor lifted at a uniform rate of about 0.45 mm/day with no evidence of horizontal extension of the floor. Roadway closure was of a similar rate. As longwall 2 approached, roadway closure rate increased significantly and this was observed to be due to increased floor heave. At about the same time, floor extension was observed. Also of interest was the increase in pore pressure recorded beneath the pillar as longwall 1 passed followed by a gradual decrease. Another increase was measured as longwall 2 approached; the very high levels recorded at 340 days may be related to cracking to the underlying Fassifera Seam which is a confined aquifer.

The Chain Valley site showed the pattern of floor heave, floor extension, and anomalous deep rib spall. A redistribution of vertical stress was also noted. Cables to the site were prematurely cut. The information from the Wyee site was complicated by multiple-seam interactions and will be presented elsewhere.

![Figure 2 - Instrumentation Layout at Cooranbong Colliery](image-url)
Table 2

Summary of Mining Parameters and Monitoring Results at the Instrumented Sites

<table>
<thead>
<tr>
<th></th>
<th>Chain Valley</th>
<th>Cooranbong</th>
<th>Wyee</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pillar Dimensions</strong></td>
<td>24 x 20</td>
<td>27 x 95</td>
<td>30 x 95</td>
</tr>
<tr>
<td><strong>Roadway Width</strong></td>
<td>5.5m</td>
<td>5.5m</td>
<td>5.5m</td>
</tr>
<tr>
<td><strong>Roadway Height</strong></td>
<td>2.4m</td>
<td>2.6m</td>
<td>2.9-3.1m</td>
</tr>
<tr>
<td><strong>Virgin Stress</strong></td>
<td>3.55MPa</td>
<td>3.06MPa</td>
<td>0.78MPa(Site 1)</td>
</tr>
<tr>
<td><strong>Tributary Area Stress</strong></td>
<td>10.55MPa</td>
<td>7.83MPa</td>
<td>20.47MPa (Site 2)</td>
</tr>
<tr>
<td><strong>Pillar Stress</strong></td>
<td>6.99MPa</td>
<td>4.77MPa</td>
<td>19.1MPa(Site 1)</td>
</tr>
<tr>
<td><strong>Panel Span</strong></td>
<td>43m</td>
<td>143m</td>
<td>211m</td>
</tr>
<tr>
<td><strong>Max Roof Heave</strong></td>
<td>286mm</td>
<td>&gt;300mm</td>
<td>550mm</td>
</tr>
<tr>
<td><strong>Horizontal Floor Movement</strong></td>
<td>85mm</td>
<td>34mm</td>
<td>Not Measured</td>
</tr>
<tr>
<td><strong>Depth of Horizontal Floor Movement</strong></td>
<td>2m</td>
<td>1m</td>
<td>Not Measured</td>
</tr>
<tr>
<td><strong>Max Pillar Settling</strong></td>
<td>16mm</td>
<td>37mm</td>
<td>13mm</td>
</tr>
<tr>
<td><strong>Rift Spall</strong></td>
<td>140mm</td>
<td>4 - 7mm</td>
<td>20 - 75mm</td>
</tr>
<tr>
<td><strong>Depth of Spall</strong></td>
<td>2m</td>
<td>1m</td>
<td>1 - &gt;7m</td>
</tr>
<tr>
<td><strong>Roof Movement</strong></td>
<td>0mm</td>
<td>28mm</td>
<td>0 - 36mm</td>
</tr>
<tr>
<td><strong>Height of Roof Movement</strong></td>
<td>-</td>
<td>2m</td>
<td>0 - 4m</td>
</tr>
</tbody>
</table>

**FAILURE MECHANISMS AND DESIGN TECHNIQUES**

Mine operators and planners require techniques that allow them to predict conditions ahead of mining and devise remedial or preventative measures. Any failure mechanism proposed in geotechnical engineering is only of real value if it can be translated into a predictive design technique. For this claystone material, the proposed mechanisms have been developed in the context of the established knowledge of foundation engineering. Other mechanisms may be possible but can you design with them?
SETTLEMENT INTO FLOOR

When a load is placed on a clay, three components of total settlement can be recognised: immediate settlement, primary consolidation, and secondary consolidation or creep. The immediate settlement results from the idealised distortion of the soil (the deformation of the macrostructure referred to above), primary consolidation results from the flow of pore water to equalise the pore pressures, and creep relates to deformation under constant effective stress.

Elastic theory provides a convenient way of simplifying the prediction of immediate settlement and primary consolidation. The accuracy depending on the technique for determining representative linear values for modulus and Poisson's ratio (Figure 2 shows the striking non-linear behaviour that is typical for clays).

Elastic theory states that settlement S at a point on the surface is given by:

\[ S = \frac{q B (1 - \nu^2)}{E} \]

where:
- \( q \) = uniform applied pressure
- \( B \) = footing width
- \( \nu \) = representative Poisson's ratio
- \( E \) = representative elastic modulus
- \( \lambda \) = influence factor = function of aspect ratio of layer, shape and rigidity of footing, and location of the point

Values of \( \lambda \) are tabulated for both rigid and flexible footings with the rigid footings of perhaps most use for coal mine applications.

At low layer aspect ratios, the influence factor is approximately equal to the aspect ratio. In such cases, the settlement equation above can be approximated as:

\[ S = \frac{q B h}{E} \]

where:
- \( h \) = thickness of layer
- \( B \) = width of footing

for \( h/B < 0.2 \)

In foundation engineering, the applied pressure can usually be reliably estimated. This is not the case for pillars, especially in longwall environments. Tributary area and an abutment angle model (Mark, 1989) has been used in this work. Based on laboratory tests, Seedman (1983) suggested values of 600 MPa and 100 MPa for the in situ undrained and drained moduli of the Awaba Tuff claystone at Cooranbong. These values represent a 50% reduction on the measured values.

Back analysing immediate pillar settlement measured at two sites, and correcting for pillar compression, gives values of 1.89 GPa, and 420 MPa for Chuin Valley and Cooranbong respectively. The Wyec data could not be used due to possible errors of pillar failure. None of the instrumentation sites were able to supply information on the drained modulus of the claystone as cables could not be maintained.

BEARING CAPACITY AND FLOOR HEAVE

Drawing the analogy with footings, pillars can induce shear failure of the floor by generating stresses that exceed the shear strength of the claystone. This can be analysed by use of the well-established bearing capacity equation for strip footings which states:

\[ q_{\text{max}} = c N_1 + p' \left( N_p - 1 \right) + \gamma B N_s, \]

where:
- \( q_{\text{max}} \) = failure stress
- \( c \) = cohesion
- \( p' \) = effective overburden pressure
- \( \gamma \) = soil buoyancy density
- \( B \) = footing width

\( N_1, N_p, N_s \) and \( N_y \) are bearing capacity factors.

Pillars can be taken as surface footings so \( p' = 0 \). Furthermore, it can be shown that the most critical stability state for claystone is the undrained case where \( \lambda = 0 \). The bearing capacity equation then reduces to:

\[ q_{\text{max}} = c N_1 = (UCS/2)N_s, \]

Most practical cases are not those of a strip footing on an infinite half space. In such cases correction factors are included. The most significant of these in this context is the effect of layering.

Mandel and Salincov (1969) published a technique for calculating layer factors for various friction angles and aspect ratios. For aspect ratios greater than 8, \( N_1 \) tends to a value given by:

\[ N_1 = (\pi + 1) / (2h) \]

When applying foundation engineering concepts to an underground mining problem, it is necessary to recognize a significant difference in the systems. Foundation engineering usually concerns itself with gravity loads or dead loads. In mining, one often
deals with stiff loading systems in which the loads applied to a mine structure may be relieved if some deformation is permitted. The full development of bearing capacity failure with the consequent rotation of the ground requires large vertical displacement of the structure which applies the load, i.e. a soft loading system.

Applying the layer correction to the bearing capacity of the floor at Cooranbong and Wyee results in estimated failure stresses of 63 MPa or 530 MPa respectively. Applied stresses are 12 MPa and 16 MPa respectively, suggesting Factors of Safety of 5 and 15 respectively. Similar high factors of safety have been obtained by workers in the Illinois Coal basin. In order to apply bearing capacity concepts and have factors of safety approaching 1, workers have adopted the strategy of massively reducing the laboratory strength values.

Consider now how such a simple bearing capacity model would predict floor heave. Figure 4 shows an exaggerated cartoon of a pillar. The claystone at the edge of the pillar needs to move out and into the roadway and then lift the floor. If it is some form of rotation, then the floor should behave locally to a similar amount as the settlement of the pillar into the floor. Certainly, the ribs of the pillars at both Chinchilla and Cooranbong show greater than usual amount of rib spall in association with some horizontal movement of the floor. However, this is only to a depth of 1 - 2 m. It is not clear how such a failure mechanism in the floor can lead to the extensive floor heave that results in almost complete closure of the roadway.

SWELLING HEAVE

The gradual heave at Cooranbong seen during and after the retreat of Longwall 1 is considered to be by a different mechanism. Seedsman and Mullen (1988) predicted gradual heave due to swelling of 330 mm over 72 days for a 2.7 m clay layer assuming a 10 cm path length and a coefficient of consolidation of 1.01 m/day. Heave rate is inversely proportional to the square of the path length.

This prediction was based on the claystones in the roadway swelling in the presence of water. It is considered that the similarities between the production and actual occurrence of heave are sufficient to support such a mechanism. The swelling will have induced failure of the coal floor beam at the rib side(Figure 6) so that consequent overloading of the floor as Longwall 2 passed would not cause the rotational failure seen at Wyee.
CONCLUSIONS

It is concluded that many of the aspects of the behaviour of claystones in the underground coal mining environment are amenable to engineering analysis and design based on the established principles of foundation engineering. Both pillar settlement and floor heave can be analysed.

ACKNOWLEDGMENTS

This work was funded by National Energy Research Development and Demonstration Program (Project 1336). The co-operation of the managers of Chain Valley Cooranbong and Wyee Collieries is gratefully acknowledged. Mr. Bob Hedger installed the instrumentation.

11th International Conference on Ground Control in Mining, The University of Wollongong, N.S.W., July 1992.
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