Monitoring and Back-analysis of a Road Cutting with Locked-in Horizontal Stress Field

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Summary: The completed West Charlestown Bypass project includes 6.5km of urban freeway south west of the Newcastle CBD. The project included a cutting at its northern extent of up to 17m depth and 450m in length. Geologically, the cutting is located within the Newcastle Coal Measures comprising interbedded sandstone and siltstone, tuffaceous claystones and coal seams throughout. The cutting design incorporated a combination of 2:1 (H:V) cut batters underlain by an anchored piled retaining wall with infill panels and a pattern of sub-horizontal batter drainage. During excavations, and when the cutting was at shallow depth, certain borehole inclinometers identified horizontal movements at depths significantly lower than the earthworks level. Additional instrumentation and investigative boreholes were subsequently implemented. Numerical back-analyses were also carried out to assist in assessing the cause and potential impact of this movement. This back-analysis work inferred that the horizontal movements were caused by the relief of locked-in in-situ horizontal stresses in the weathered bedrock and sliding along gently dipping claystone seams. Based on this additional work, design changes were made to cater for the predicted future movements.

INTRODUCTION

The West Charlestown Bypass in Newcastle, New South Wales, Australia provides a link in the inner city bypass of Newcastle. This section comprises a 6.5 km length of dual carriageway motonvay in an urban environment extending from the Pacific Highway at Windale in the south to Charlestown Road, South Kotara, in the north.

This paper considers the major cutting at the northern extent of the bypass route, Cut 7 oriented north-south. Due to constraints of vertical geometry associated with the Myall Road Bridge to the north and the Hillsborough Road bridge to the south of this cutting, the maximum total cut height of 17m could not be reduced. In fact the vertical gradients associated with this height of cut were already at upper limits for motonvay design.

The Cut 7 geometry was further constrained by its proximity to adjacent properties to the east of the southbound carriageway. This limited horizontal distance, when combined with the geotechnical requirement for a maximum cut batter angle of 2:1(H:V) in this geological environment, required the construction of a retaining wall to achieve the design road cross section. This geometrically constrained eastern batter is the primary focus of this paper.

SITE SETTING

Regional Setting

The subject Cut 7 is founded on a topographic "spur". This "spur" is formed by a major escarpment located some 350m to the east of the cutting, which provides significant topographic relief toward the coast, and the gentle relief across the site, typically in a south-westerly direction. Thus the northern end of the cutting forms a topographic high.

The West Charlestown Bypass Route is located within the Newcastle Coal Measures. The Cut 7 area is located within the Tickhole formation, which includes in descending stratigraphic order, the Australasian, Montrose and Wave Hill Coal Seams. The intermediate rock units associated with these seams consist of siltstone, sandstone and claystone many of which are of tuffaceous origin. The Australasian Coal Seam beneath the Bypass alignment was mined in the late 1800's and early 1900's.

Landslides associated with failure along low strength claystone seams are known to be a feature of the geology of this locality within the Newcastle Coal Measures (Fell, 1995). Many of the known landslides have been well documented including the detailed investigations undertaken by Coffey Partners (and reviewed by GHD-LongMac) for the Tickhole Tunnel failure located to the north of the Bypass alignment. GHD-LongMac has also developed stress related designs for rock cuttings of the Sydney-Newcastle, F3 Freeway to cater for shear movements along weak claystone seams resulting from stress relief (Leventhal and Stone, 1995). These low strength claystone seams are often difficult to detect during investigation, due to their low strength and small thickness, which results in minimal recovery during core drilling.

Geological Model

The cutting profile typically consists of a thin topsoil/fill layer (<0.5m) overlying residual soils grading to highly to extremely weathered bedrock over a depth varying up to 8m. These residual soils typically consist of medium to high plasticity clays and vary from stiff to very stiff.

The lowest rock unit intersected in Cut 7 is the Australasian Coal Seam, which consists of 9 to 12m of coal of varying quality and includes numerous interbedded claystone seams. The units overlying the Australasian Coal Seam are encountered over the majority of the cutting, and consist of siltstone and sandstone both interbedded and as distinct units, with occasional carbonaceous bands and claystones. These units vary from highly weathered to fresh, and increase in strength with depth from weak to strong. The regional dip direction was assessed to be south-westerly, resulting in an adverse dip out of the eastern batter into the bypass alignment. This adverse clip angle varied from 2" to 8°, but was typically 3". Figure 1 shows a typical geotechnical cross section of the cutting.

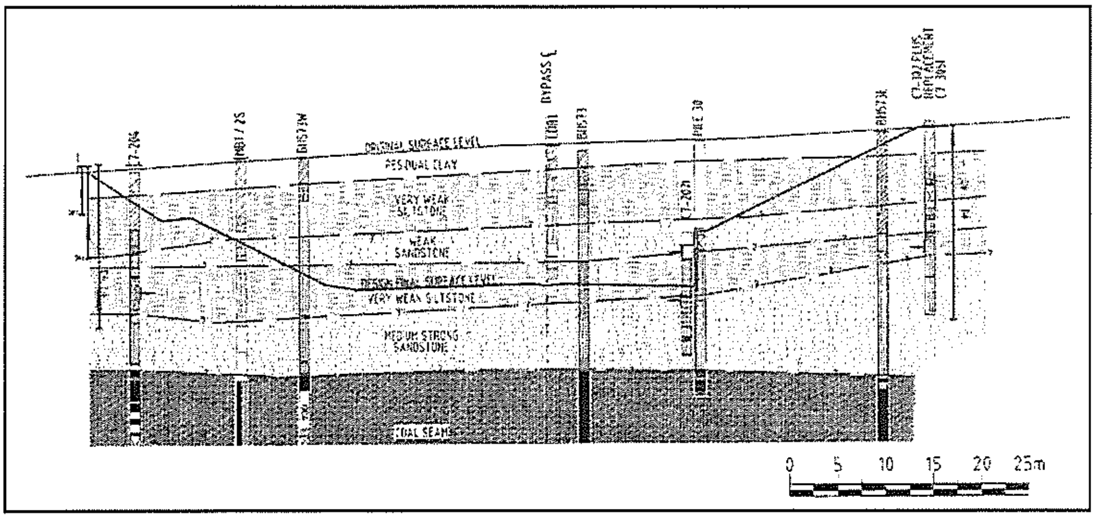

Figure 1. Section through cutting at CH12300m

From the investigations and construction phase mapping, a number of sub-vertical joints were identified throughout the cutting, with the two most prominent sets striking at 050" to 080° and approximately 150°. On the western batter these joints were observed to be open which was related to the underlying mine workings and associated subsidence effects.

From the results of the investigations, and our understanding of the regional geology, a "rogue" claystone seam was assumed to occur for design purposes. When combined with the adverse dip direction into the cutting and the formation of discrete blocks formed by the sub-vertical joints, a potential block slide mechanism along this claystone seam was recognised. The location of this "rogue" seam, for analysis purposes, was conservatively assumed to correspond with the lowest stability scenario.

The claystone seams were prudently assumed to be at residual strength assuming that movement along these seams would have occurred associated with previous geological disturbance of the rock strata and/or mining effects. This potential for past movement was further supported by the presence of slickensiding on structural features within the recovered core. The assumed strength of the claystone seams considered published data for local landslides (Fell, 1995), extensive data collated by GHD-Longmac for claystones associated with the Newcastle Coal Measures (Leventhal and Stone, 1995) and limited laboratory test results. A residual shear strength friction angle of 12° was assumed.

Based on the geological model described above, the potential for further stress relief movements resulting from excavation of the cutting was considered unlikely. This judgement recognised that cutting was located substantially within residual soils and extremely low strength weathered rock strata, in conjunction with the pre-existing topographic relief to the east and west.

ORIGINAL DESIGN

Retaining Wall Design Selection

The basic design approach was to prevent potential block sliding via a detailed drainage solution for the cut rock mass, with a retaining wall constructed at the toe of the batter to cater for local stability only. This approach catered for the cross section geometry in an expected low stress environment.

A number of retaining wall options were considered for the cutting including anchored pile, cantilever and anchored shotcrete walls. The selected option was an anchored pile retaining wall as it provided the benefit of being better able to control risks of block sliding through staggered or sequential excavation, progressive anchoring and capacity for monitoring of excavation movements as work progressed. The anchored pile wall consisted of bored piles at regular spacings with active rock anchors at the pile head to provide lateral restraint. The pile span was then infilled with a shotcrete arch to transfer the load to the piles with architectural panels erected in front of the wall to allow application of a design motif for the wall length.

In addition to the above anchored pile wall, the requirement for sub-horizontal drainage within the cutting was determined from consideration of a simple block-sliding model. The intention of the drainage in the design model was to reduce both driving and uplift piezometric pressures. The initial design included a staggered drainage pattern with drainage lengths of up to 15m into the cutting.

Monitoring

Prior to construction, piezometers were installed to enable validation of the design assumptions and ensure compliance with the design intent, including assessment of the groundwater levels as the staged construction progressed. A limited number of borehole inclinometers were also installed along the crest of the cutting to enable monitoring of potential slope movements. This instrumentation reflected the uncertainty regarding the adopted model including the location, orientation and condition of low strength claystone seams and the potential for movement along these seams.

Additional instrumentation including anchor load cells were specified for the retaining wall anchors to assess the performance of the 150 anchors along the retaining wall during construction. These load cells also enable measurement of the anchor load gain predicted from the modelling as the excavation and construction progresses through the modelled stages.

CONSTRUCTION PERFORMANCE

At the commencement of Cut 7 excavation, significant and prolonged rainfall occurred, which continued at varying intensity for the duration of the earthworks within this cutting. The excavations were at shallow depths (<5m) when the first indication of movement was identified at a depth of 11 to 12m in one inclinometer. Ongoing monitoring whilst excavation proceeded identified further movement at this depth. This movement at this early stage was not expected from the assumed design model.

The extent of monitoring was thus intensified via a stage of construction phase monitoring installations. These additional monitoring instruments were intended to identify the nature of the movement, its geometry and extent. The instruments included multi-level piezometers and inclinometers, with extensometers within the mine void area to detect possible subsidence effects. All installations included core drilling to assist with interpretation of structure and defects that may be associated with the observed movement. This was further supplemented with detailed geological mapping of the exposed cut batters.

The construction stage monitoring was intended to provide data to facilitate assessment of various features of the cutting that included: possible artesian pressures; location of low strength shear/bedding planes; perched water table including multi level aquifers; ongoing mine subsidence and the extent and geometry of the movement.

Initial consideration of the cause of the movement identified possible: global block movement of the hillside; a local deep-seated subsidence related movement; or a combination of these two mechanisms. As these mechanisms were expected to be relatively slow with intermittent movement, a rapid "catastrophic" failure with a high safety risk was considered unlikely.

Monitoring Observations

Figure 2 below shows a typical inclinometer plot. The following observations relate to the observation and assessment of the monitoring data:

- o The measured inclinometer deflection correlates with a shear movement which could be projected between inclinometers to identify planes of movement;
- o At several inclinometers, planes of shear movement were identified at multiple levels;
- o The magnitude of the shear movement in any cross section reduced away from the exposed batter. That is movements on the same shear plane reduced away from the excavation;
- o The incremental movement corresponded with ongoing excavation and peaks of groundwater level;
- o Depths of shear movements could be correlated with claystone seams with thickness as small as 5mm within recovered core and excavation exposures;
- o Multi layer aquifers exist across the site corresponding with variation in strata type. Of particular note was perched water above a carbonaceous claystone/siltstone seam in the vicinity of the measured movement.
- o The extensometers did not indicate any vertical displacement, that is there was no subsidence effects.
- o The western batter experienced minor movements into the cutting, i.e. the two batters moved towards each other.

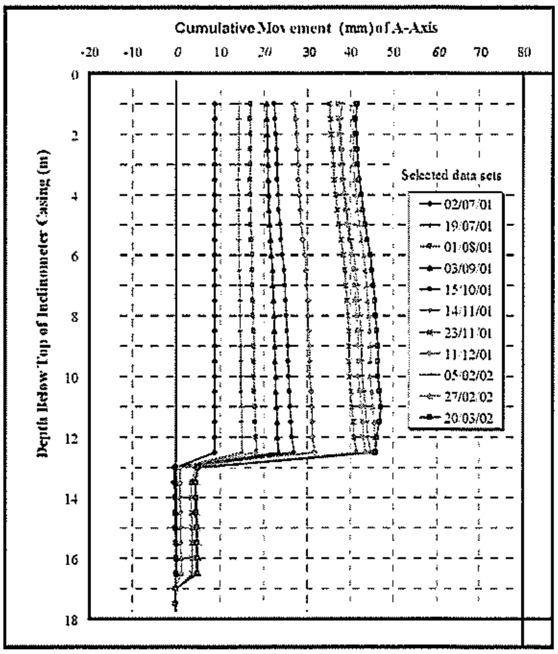

Figure 2. Inclinometer Plot (C7-1021)

BACK-ANALYSIS

Objectives and Analytical Model

Following the detection of the lateral/shear movements within the borehole inclinometers during the early stages of construction, numerical modelling was undertaken for the subject cutting. The modelling work was orientated to serve two main purposes. Firstly, it was used to back-analyse the measured movements in order to refine the geotechnical model of the site and to understand the likely mechanisms of the induced movements. Secondly, the modelling was extended using the refined model to assess the impact of the batter movements on the construction works and the design retaining structures. Based on the numerical modelling, an action plan was also devised to minimise any adverse impacts and to improve the stability of the cutting.

Apart fiom conventional limit equilibrium slope stability and wedge failure analyses, the majority of the numerical modelling work was undertaken using a 2-dimensional finite element program, Phase2 (Rocscience, 2001). The models assumed plane strain conditions and were discretised into 3 noded triangular elements. The cutting was sub-divided into different domains dependent upon the stratigraphy and the retaining structure design. A typical geological cross section was developed for each domain, as demonstrated in Fig. 1. Each rock unit was modelled as one material with elastic properties assigned to the rock mass. For the purpose of numerical modelling, it was assumed that claystone seams were present at levels where lateral movements were detected by the borehole inclinometers. Furthermore, it was assumed that a number of sub-vertical joints existed in the vicinity with a nominal horizontal spacing of 10m. These claystone seams and sub-vertical joints were treated as elasto-plastic discontinuities with Mohr-Coulomb failure criterion. The design bored piles and the shotcrete in-filled panels were also modelled as a composite elastic material with equivalent thickness to provide the appropriate rigidity.

Various model stages were analysed to simulate the history of the site and the construction sequencing. These stages included the original mining of the coal seam, assumed collapse of the mine pillars, initial excavation work when movements were detected, future excavation and construction stages.

Back-analysis Results

A likely range of typical properties was assigned to the rock mass and the discontinuities based on the investigative data and local experience. These properties were varied in different runs to assess the sensitivity of their values on the behaviour of the excavation. Due to the critical nature of the claystone seams, emphasis was given to their shear strength properties. Some of the initial runs adopted peak shear strength for the claystone seams. These initial runs confirmed the original geological model that the peak shear strength of the claystone seams would be exceeded as a result of mining activities and the assumed subsequent mine pillar collapse. For the yielded claystone seams, residual shear strength was used in subsequent runs. The back-figured properties of the rock mass and the discontinuities are shown in Tables 1 and 2 respectively.

A natural in-situ stress state was also specified for the models. The initial runs assumed minimal locked-in horizontal stress for the weathered bedrock profile. However, it was revealed from these initial results that the measured batter movements could not be simulated by varying the properties of the rock mass and discontinuities alone. Locked-in horizontal stresses were subsequently introduced to the models using the following relationship with the vertical stresses:

$$\sigma_H = a + K\,\sigma_v \qquad (1)$$

where σ_H and σ_v are the horizontal and vertical stresses respectively, 'a' is the locked-in horizontal stress and 'K' is the stress ratio. The values of 'a' and 'K' were varied until the magnitude and the profile of the modelled horizontal movements at the crest of the cutting approximated the movements measured by the borehole inclinometers. The back-figured values of 'a' and 'K' are about 0.5MPa and 0.5 respectively. For a nominal depth of 10m, the horizontal stress, σ_H was therefore estimated to be 0.6MPa. Whilst this level of stress is higher than our initial expectations for the weathered nature of the site materials and the proximity to the topographic spur, it is considered to be geotechnically admissible.

Material Type	Unit Weight (kN/m3)	Young's Modulus (MPa)	Poisson's ratio
Residual soil	20	25	0.3
Very weak siltstone	23	100	0.25
Weak sandstone	23	350	0.25
Coal seam	23	500	0.25
Medium strong sandstone	23	700	0.2
Concrete piles	24	30000	0.2

Discontinuity Type	Normal Stiffness (MPa/m)	Shear Stiffness (MPa/m)	Friction Angle (deg.)
Claystone seam	500	200	20 (peak) 10 (residual)
Sub-vertical joint	1000	400	25

The above back-analysis has also demonstrated that the batter movements were caused by a combination of geotechnical/geological conditions and events. The factors associated with the batter movements included the mining activities (including possible mine pillar collapse and associated yielding and strain softening of 'rogue' claystone seams), the relatively high horizontal stresses being relieved by construction excavation and the presence of sub-vertical joints enabling the batter to move as a block. The initial batter movements also coincided with groundwater recharge via a heavy rainfall event immediately preceding the initial movements, which would have caused a rise of the pore pressure and a reduction of effective normal stresses along the claystone seams.

REVISED DESIGN/ADOPTED TREATMENT

From the modelling an action plan was developed. This included consideration of design alternatives, methods of construction and the critical nature of the construction program. The RTA also considered that changes to the overall design be minimised where possible for contractual purpose.

The back-analysed model was used to extrapolate the cutting behaviour during future construction stages. The analysis results indicated that the horizontal movements at the location of the anchored pile wall could be up to some 150mm. Separate analysis of the response of the piles caused by the predicted batter movements showed that the induced bending moment would exceed the bending moment capacity of the piles. The original design thus needed to be revised in order to prevent any damage to the anchored pile wall.

In addition, the design section included a large diameter stormwater drainage conduit along the cutting centreline to be constructed at a depth of up to 6m below the cutting floor. This drainage slot presented the deepest point of excavation and the maximum exposure of the cutting to stress relief movement. This proposed excavation constituted a key consideration of the revised design process.

Detailed modelling was undertaken to develop a staged excavation process of the drainage slot, which would reduce the remnant stresses and associated shear movements along weak bedding planes in a controlled manner prior to pile installation. This included maintaining berms of specific geometry against the vertical section of the design cutting. The berm and excavation stages were modelled to ensure that the residual stress and corresponding shear movements, post slot excavation, were reduced to a magnitude acceptable for pile or anchor installation. This excavation became known as the 'stress relief slot', which commenced at the shallow, less critical northern, end of the cutting where movements prior to slot excavation had been minor to insignificant.

Additional inclinometers were constructed adjacent to the pile retaining wall alignment to permit observation and comparison of the magnitude of shear movements as the slot excavation progressed. The location of these inclinometers corresponded with the existing instrumentation and selected modeled sections. The magnitude of the inclinometer movements in each section was assessed to understand the progress and extent of the stress relief as the slot excavation proceeded south.

When plotted on a log time scale, a distinct "turning over" of the rate of movement was recognised along the length of the retaining wall for the prevailing (groundwater) conditions. From the modelling, the remaining 'creep' movement post turn-over. was predicted to be acceptable for the bending moment capacity of the pile section. However, this prediction was extrapolated from early movements (with minimal excavations), and thus a level of uncertainty existed as to the magnitude of the remnant 'creep' movement.

However, to avoid extended construction delays, a risk management approach was developed for the pile installation in consultation with the RTA. To assess the pile performance under the influence of the expected remnant creep movement, inclinometers were installed within selected piles to enable measurement of the deflected pile shape and to calculate the pile bending moment.

On "release" of the waiting period for piling, the retaining wall construction and cutting excavation proceeded with a support berm remaining in front of the vertical wall section to maintain local stability. This berm was removed in stages to facilitate the installation of the batter drainage. The pre-construction drainage design methodology, assuming a potential basal, low strength shear plane formed a block slide mechanism, was revised utilising the monitoring results and geological mapping. The revised batter drainage design included a pattern of staggered sub-horizontal batter drainage at 5m centres with lengths extended up to 38m.

SUBSEQUENT MONITORING

Subsequent monitoring has demonstrated that the shear movements have essentially stabilised. As predicted some creep movement did occur resulting in pile deflection and anchor load gain. Figure 3 provides a plot of the inclinometer deflection, anchor load gain, rainfall and groundwater levels. This plot demonstrates that the creep movement correlated with groundwater levels. This reflects an incremental release of remnant stress where the normal effective stress on the claystone plane of movement is reduced as a result of groundwater peaks.

By determining the relationship between pile deflection and bending moment via the cracked pile modulus, the performance of the piles under the remnant 'creep' movement was assessed to be acceptable.

CONCLUSION

This case study presents a technically complex combination of geotechnical conditions and construction phase events. The value of geotechnical instrumentation was demonstrated via the early detection of movement, which enabled application of an innovative approach to advance construction and minimise delays whilst managing an acceptable control of risk. This approach was made possible via a close working relationship between the client and designer during application of an unconventional design solution.

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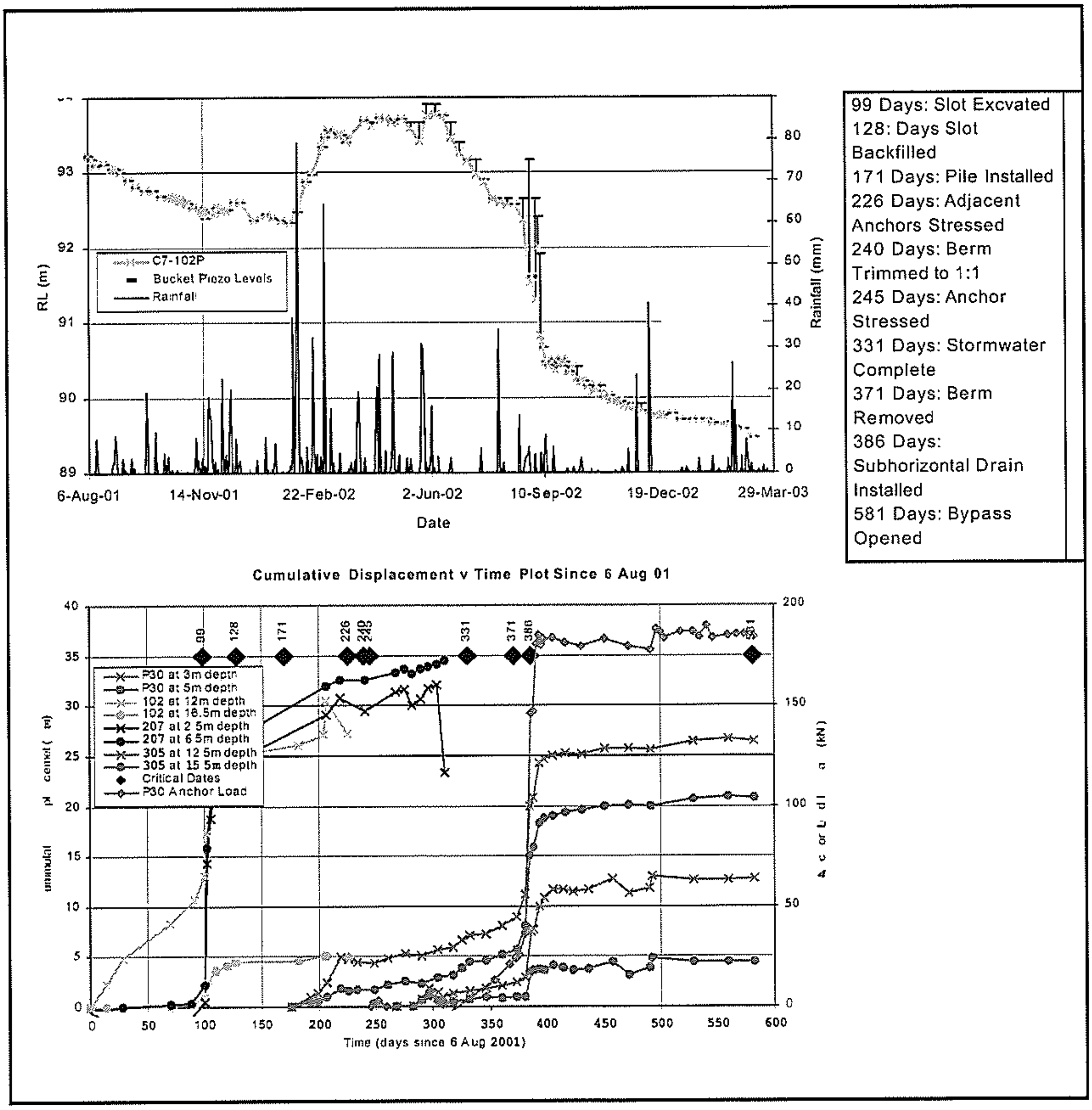

Figure 3. Compilation of Monitoring Results for Section at CH12300m

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