SETTLEMENT OF OPEN CAST MINE BACKFILL

TWO LARGE SCALE FIELD TESTS

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DECLARATION

I declare that this dissertation is my own, unaided work. It is being submitted for the Degree of Master of Science in Engineering in the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination in any other University.

30 day of OCTOBER 1998.

ABSTRACT

The Electricity Supply Commission of South Africa (Eskom) have identified open cast coal mine backfill areas as potential disposal sites for the large volumes of coal ash produced by their power stations. As Eskom's power stations are mainly situated in agricultural and coal rich areas of the provinces of Mpumalanga and the Free State, the sterilisation of substantial areas of agricultural soil and coal deposits is thus reduced.

The construction of a tailings dam or dump on uncompacted open cast mine backfill creates various problems related to the settlement of the backfill. The scale of the operation, the large particle size and heterogeneous nature of the backfill and its method of placement complicates the prediction for settlement of the backfill.

Areas in excess of 74 000 ha could be subjected to opencast mining in Mpumalanga and for future development of these areas more information regarding the magnitude and mechanics of mine backfill settlement is required.

This dissertation describes two large scale field tests in which the settlement of mine backfill was studied during the construction of a test section of an ash tailings dam and the construction of a dry ash dump.

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1.1 CONSTRUCTION OF ASH DISPOSAL FACILITIES ON MINE BACKFILL

The Electricity Supply Commission of South Africa (Eskom) have identified open cast coal mine backfill areas as potential disposal sites for the large volumes of coal ash produced by their power stations. As low quality coal is generally burnt by Eskom and their 6 unit power stations are capable of producing 3 600 MW, up to 600 ton of ash per hour can be produced by a typical power station. The ash dams and dumps are planned to cover areas of up to 900 ha per site. As the power stations are mainly situated in agricultural and coal rich areas of the provinces of Mpumalanga and the Free State, the sterilisation of substantial areas of agricultural soil and coal deposits is thus reduced.

The construction of a tailings dam or an ash dump on uncompacted open cast mine backfill creates various problems related to the settlement of the backfill. The scale of the operation, the large particle size and heterogeneous nature of the backfill and its method of placement complicates the prediction for settlement of the backfill.

1.2 GENERAL DEVELOPMENT OF OPEN CAST MINE BACKFILL AREAS

At a seminar held on construction over mined areas Wagner [1] stated in his closing remarks that "The papers by P Day and W A Naismith dealt with an aspect of mining-related ground deformations which will become more important in the future, namely the settlement of backfill on restored open cast sites. From both these papers it is evident that a fuller understanding of the mechanism of settlement and its influence on the behaviour of surface strain profiles is needed to use rehabilitated areas for other purposes than agricultural use." and further "it is essential that more attention is given to this particular problem ".

Building up a database and properly documenting the findings would be of great importance to Eskom in the construction of future power stations if an in-pit ashing solution is adopted and for development of large areas of Mpumalanga, as estimates by Van Niekerk [2] concluded that areas in excess of 74 000 ha could be subjected to opencast mining.

1.3 LITERATURE SURVEY

1.3.1 Mechanisms of Settlement Of Opencast Mine Pit Backfill

Mine backfill settlement can be subdivided into three mechanisms, namely consolidation, collapse and creep settlements.

1.3.1.1 *Consolidation Settlement*

Day[3] described consolidation settlement as the settlement that occurs in response to changes in effective stress shortly after any change in applied load, in common with many granular materials. Due to mining sequences most backfill materials only become available for development or utilisation years after placement, when the consolidation settlement due to the self weight of the backfill has already occurred. The effects of changes in load therefore only need to be taken into account when development of backfilled areas is considered.

Naismith [4] described the mechanisms of consolidation settlement as a period in which the dry fill compacts under its own weight, due to the failure of contact points under high compressive stresses. As one failure will lead to movement of the material this will create another failure and thus the movement will be continuous. (Creep)

Jones and Wagener[5] concluded in their investigation for Lethabo Power Station, *"A large proportion of the observed settlement occurred shortly after loading and primary settlement was complete in less than two months."*

Burford[6] recorded that most of the consolidation settlement occurred during placement of a 7m high earthfill, on a 24 m backfill, near Corby in the United Kingdom to surcharge an area for the development of a housing estate. Thomson and Sonnenburg [7] found consolidation settlement to be completed within two weeks under a 40 kPa prototype foundation on mine backfill in western Canada.

Charles[8] stated that "The concept of bearing capacity as the controlling factor in foundation design is based on the assumption that the major cause of settlement is the weight of the structure built on the fill. Generally with deep uncompacted fills this is not the case and most settlement can be attributed to other causes." Significant settlement can however be expected due to the low constrained modulus of backfill. The settlement however occurs as the structure is being built which is less of a problem than long term creep settlement subsequent to completion of the structure.

1.3.1.2 *Collapse Settlement*

Day[3] describes collapse settlement in backfill as the result of softening of inter-particle contacts due to increase in moisture content leading to the rearrangement of particles and the assumption of a denser packing.

Naismith[5] describes the mechanisms of collapse settlement in open cast mine backfill as the following processes that occur when the water table re-establishes itself by infiltration and lateral ground water flow:

- Fine particles wash into voids causing reduced lateral support to larger particles and potentially larger contact stresses. This may lead to settlement above the water table.
- Effective shear strength is reduced due to the increase in water pressure thus creating the potential for internal shearing under the water table.
- Chemical and mechanical degradation may occur.

Jones and Wagener[9] estimated this collapse settlement to be in the order of 5 % at the Lethabo Power Station in-pit ashing facility. Charles et al [10] recorded up to 2 % compressions with an average of 1 % in 34 m of backfill at Horsley. Luker and Wates [11] predict a collapse settlement of between 5 % and 10 % for Kriel Colliery pit no. 3.

1.3.1.3 Creep Settlement

Day[3] described creep settlement as "...slow, long term settlement under constant conditions which generally decreases logarithmically with time." Day[3] recorded 40 mm of settlement over a five year period at Optimum Colliery. Burford[8] found that creep settlement decreased considerably after 10 days and virtually ceased after 28 days.

Charles[6] stated that long term settlement due to loading can be described using the logarithmic creep parameter $s\alpha$ defined by,

$$s = s_i + s\alpha \log (t/t_i) \qquad (1)$$

where s is the total settlement (consolidation and creep settlement) at time t after loading, s_i is the consolidation settlement measured at time (t_i). When a load is applied rapidly, Charles[6] states that $t_i = 0.1$ day might be appropriate and with a slower applied load recommends $t_i = 10$ days.

Alternatively expression (1) can be written as,

$$s\alpha = 2.303\ t\ ds/dt$$

where (ds/dt) is the settlement rate at time t.

A value in the order of 0.3 for $s\alpha/s_i$ is typical for poorly compacted granular fills, according to Charles[6].

1.3.2 Literature Survey of Recorded Mine Backfill Settlement

1.3.2.1 Wates and Wagner [12] Investigation at Matla Power Station

Wates and Wagner did a large scale field test on the mine backfill in Krial Pit 3 North. Luker and Wates[11] published detailed results of the same test.

An 8 by 8 m area was excavated 3.5 m into the backfill with 1 in 2 side slopes. The excavated pit was then lined with clay to enable a saturation experiment to be made.

After replacing the excavated backfill and installation of inclinometers the area was surcharged with a 7 m soil mound and the settlement was observed. Unfortunately the settlement within the pit was not recorded at this stage. The backfill in the pit was then flooded to measure the collapse settlement of the fill above the clay liner.

Observation made during this test were:

- There was a 33 mm rebound upon excavation of 3.5 m backfill.
- 65 mm of settlement occurred in the 20 m of backfill beneath the test site upon backfilling of the pit.
- 480 mm of settlement occurred within the pit upon saturation. This included the consolidation settlement due to the surcharge.
- Total settlement recorded in the 3.5 m fill was therefore 13.7 %.

To match the measured movements a program written by Luker was used to model increasing stiffness with depth and a decreasing Poisson's ratio with depth. The program was written for developing simple models of soil behavior and in this case it was concluded that it is justifiable to interpret the test measurements using the simple soil model of linear elasticity with no tension. For dry stiffness the following formula was proposed:

$$E = 670D + 4000\ (\text{kN/m}^2)$$

$$\nu = 0.25 - 0.014D\ (\text{D depth in metres})$$

For the 11.7 % collapse measured the following best fit was found:

$$E = 1060\ \text{kN/m}^2$$

That the test was done on backfill less dense than the overburden at depth, was recognised, and an E value of 2500 kN/m^2 was proposed. From triaxial tests, and the fact that E and ν are heavily stress related, an E value of 8000 kN/m^2 and a ν of 0.2 were proposed for consolidation settlement.

In the paper[11] Luker and Wates propose a collapse potential of between 5 % and 10 %.

1.3.2.2 *Jones and Wagener[4&13] Investigations at Lethabo Power Station*

Two studies were made on the expected settlement of approximately 33 m of uncompacted mine backfill in the open cast pit of New Vaal Colliery. Day [14] also reported on the findings of these two tests.

Consolidation and Creep Settlement Test

A starter platform of 11 m of sand was constructed. The width of the platform was approximately 30 m at the crest and 65 m at the base. Four concrete slabs of 1.2 m square and 250 mm thick were cast on the original surface of the backfill. The level of the surface was surveyed. On completion of the sand starter platform four small diameter holes were drilled through the platform to the slabs. 20 mm galvanised pipes were grouted into each slab and the top of each pipe was then used as a benchmark. The contractor installing anchors for the conveyors however destroyed the points and only two sets of data were available.

The first set of results obtained with the installation of the pipe showed 989 mm of consolidation settlement in a period of less than 3 months. A further 8 mm of creep settlement was recorded a further 3 months later.

Taken into account the finite width of the embankment and the 33 m depth of fill an elastic modulus of 5.6 MPa was calculated. Assuming that the elastic modulus increased with depth and had a variation of 0.5 E on the surface and 1.5 E at the base, an elastic modulus of 6 MPa was proposed.

Collapse Settlement Test

In order to establish the upper boundary of collapse settlement, laboratory tests were carried out. Values of collapse of the order of 9 to 12 % on average were recorded, depending on the applied pressure.

A controlled flooding experiment was done on the pit backfill. Prior to flooding a 15 m square 1 m deep moat and monitoring points were installed at depths of 5, 10, 15, 20 and 25 m in boreholes. Initially the fines within the moat almost sealed the moat and little penetration of water was achieved. To overcome this problem trenches were dug into the fill.

Six sets of surveys were carried out over a period of 9 months. The surface acted as if very stiff and settled the same as the 5 to 10 m layer, i.e. approximately 270 mm. At 10 m, 65 mm of settlement was recorded, reducing to 45 mm at 15 m, 12 mm at 20 m and 5 mm at a depth of 25 m. The greatest collapse was 4 % recorded at 5 to 10 m, reducing to 1 % at 25 m.

The inadequate penetration of water to the deeper layers may have played a role in the reduction of collapse with depth.

The lack of collapse in the surface layer was possibly due to low overburden pressure and possible previous collapse due to poor drainage in the area.

Jones and Wagener proposed using 6 MPa as an average elastic modulus and 5 % collapse settlement upon saturation for the design of the ash dump.

1.3.2.3 Settlement of Backfill at Horsley Restored Opencast Coal Mining Site, Charles et al[15]

Charles et al[15] describes an investigation into the collapse settlement of a 70 m depth of predominantly cohesionless fill, consisting of sandstone and mudstone material. Of particular interest is that de-watering of the pit continued after backfilling to keep the water level beneath the fill and monitoring was started before pumping was stopped giving the opportunity of monitoring the settlement that occurred with re-establishment of the water table.

During the installation of magnet extensometers in boreholes, less than 10 % of boulders were found. The borehole samples suggested that the unsaturated fill was in a loose condition. The permeability was found to be greater than 10^{-2} cm/sec. The extensometers were installed in fills of different ages.

The paper describes the results of this monitoring over a three year period. Findings were as follows:

- An increase in the rate at settlement was observed at every gauge, apparently related to the rise of the water table. Up to 1.4 % compression upon saturation was recorded. No correlation between backfill age and susceptibility to collapse settlement was established at this stage.
- Settlement measured during the period also gave an indication of creep settlement. Creep of between 1 and 4 mm/month was recorded. A heave of 2 mm/month was recorded for an area previously pre-loaded with an overburden heap.
- Continuous creep settlement at an increased rate was recorded after saturation of the area.
- An area previously wetted by a lagoon as well as the pre-loaded area showed significant less collapse settlement.

Settlement varied between 100 and 500 mm for the various areas.

In Charles et al[10] next paper recordings made at Horsley up to 1983 are detailed.

Observations made during this period are:

- Local vertical compression of up to 2 % was experienced.
- The pre-loaded area had significant less collapse and creep settlement.
- 0.5 m of settlement was recorded in the deepest area of the pit with more than 0.2 m as the result of creep in backfill above the water table.
- The newest fill showed a greater rate of settlement than older areas.

1.3.2.4 Settlement of a Factory On Opencast Backfill, Smyth-Osbourne and Mizon[16]

Osbourne and Mizon[16] describe an investigation into 0.21 m of differential settlement of a factory built on opencast backfill.

During the investigation two magnetic settlement gauges recorded 4.7 % and 2.4 % of collapse settlement following cessa... of pumping activities at closure of other local mines. A reduction in collapse settlement w 'h depth was recorded.

Continued monitoring of the settlement indi ated that after two years the settlement virtually ceased as the water table stabilised.

Total settlement recorded at that stage was 355 mm in 7.5 m saturated backfill. Therefore the total collapse was 4.7 %.

1.3.2.5 Settlement Studies of an Open Pit Mine Backfill In Western Canada, Thomson and Schultz[17]

The aim of this study was to determine the time after which mine backfill areas can be turned over for public utilization for urban or light industrial development as well as recommended foundation design procedure for such areas.

A mining site in Edmonton, Alberta straddling a highway named the Whitewood and Highvale mines, was investigated.

In the backfill, mudstone acted as a cohesive soil whereas the sandstone and siltstone acted as a cohesionless soil.

Instrumentation included multipoint magnet extensometers, single point settlement gauges and standpipe piezometers. Five 0.9 m^2 model footings were also placed close to instrumentation to simulate foundation pressures.

At Whitewood mine 30 mm of settlement was recorded in the one area with most settlement occurring between 8.4 and 12 m in depth. The authors noted that settlement became tolerable in 3 years and settlement of light structures occurs in a short time, 3 to 4 months. The water table level was constant at approximately 40 m depth.

At the Highvale mine up to 200 mm of settlement was recorded. The largest settlements were recorded in areas most recently placed. The collapse settlement played the major role in the recorded settlement. Load tests on three footings were also performed. Approximately 80 mm of settlement due to 44 kPa loading was recorded. Spring thaw contributed to collapse settlement of the footing.

Further findings were that age of the backfill played a smaller role than the rise of the water table.

1.3.2.6 Deflection of a Motorway Due To Settlement Of Backfilled Opencast Coal Site, Buist and Dutch[18]

Settlement of a highway was recorded due to deep-seated movements in backfill due to saturation of portions of the fill.

For financial and practical reasons continuous road maintenance was proposed as a remedial measures.

1.3.2.7 Land Reclamation By Opencast Mining, Waite and Knipe[19]

The paper describes reclamation of areas of shallow subsurface mining of up to 30 m depth by reclaiming the area and then backfilling in layers. The layers deeper than 15 m were deposited in 500 mm layers and compacted with one pass of a heavy roller. Layers shallower than 15 m were compacted with four passes of a towed vibratory compactor in layers not exceeding 225 mm.

Plate load tests done with a 600 mm diameter circular plate showed an average settlement of 1.33 mm with 100 kN/m^2 load and 2.7 mm with 200 kN/m^2 load, therefore an elastic modulus of approximately 17 MPa. The modulus was calculated using an influence coefficient of 0.785 to be consistent with Naderian and Williams[24] calculated moduli.

Settlement expressed as a percentage of fill thickness varied between 0.2 to 0.35 %, eighteen to thirty months after commencing monitoring.

1.3.2.8 Long Term Performance Of Houses Built On Opencast Backfill, Burford and Charles[20]

The paper compares the performance of houses built on three types of ground treatments, namely, pre-loading with surcharges, dynamic consolidation and saturation.

Average settlement (mm) produced by the ground treatment of approximately 24 m of fill is illustrated in the following table:

	Dynamic Consolidation	Saturation	9 m surcharge
At surface	240	100	410
At 4 m depth	90	40	230
At 10 m depth	<10	<10	40

Figure 1-1: Comparison Effect of Various Ground Treatments

An area left untreated as control, performed similarly or better than areas treated with dynamic compaction and saturation.

Saturation was found to be the least effective. If the saturation was throughout the fill is not mentioned in the paper. If the results are studied it appears as if saturation did not extend to the surface.

Washing out of fines in the surface layer may also have occurred leading to a larger void ratio close to the surface within the zone of influence of the house foundations.

Burford[6] describes a subsequent surcharge treatment of 7 m for housing development due to the success of the work described above. A stiffer surface layer was also recorded here with maximum settlements from 2 to 7 m below the surface.

The settlement also occurred shortly after placing of the surcharge. 600 mm diameter plate loading tests after surcharge showed immediate settlements of no greater than 7 mm at loads of 30 kPa, therefore an elastic modulus of approximately 2.7 MPa using an influence coefficient of 0.785.

1.3.2.9 Use Of Dynamic Compaction For Rehabilitation Of A Mining Backfill Area, Fourie And Wrench[21]

Fourie and Wrench describe the use of dynamic compaction of 20 m of backfill at Vogelstruisfontein Gold Mine. Surface rights did not belong to the mine and the area had to be given back to its owners ready for a proposed development. Surface settlement of less than 100 mm after 6 months was specified.

The backfill was placed and compacted in two layers, the first 8 to 12 m and the second approximately 8 m to the surface. A back analysis of the observed settlement provided a modulus of deformation of 7.2 MPa.

1.3.2.10 Effect Of Water On Settlement Of Opencast Pit Backfill, Day And Wardle[3]

Three case studies are described with emphasis on the effect of water on the settlement of the backfill. Observations made are:

- Pit backfill placed by dragline is stiffer than material bulldozed into position.

- Water ingress from the surface due to poor drainage creates localised collapse.
- More uniform settlement is experienced with a rising water table
- The long term position of the static water table must be defined for development purposes.

1.3.2.11 The Prediction Of Opencast Backfill Settlement, Hills and Denby[22]

A method of predicting opencast backfill settlement is described. Hills and Denby concentrate on creep and collapse settlement. From field observations they observe that saturation is a major cause of backfill settlement.

They concentrated on areas that have development planned after backfilling [illegible] areas have controlled backfilling operations with records of backfilling timing, [illegible] compaction as defined by method of placement for example dragline, end tipping [illegible] scrapers, type of backfill and saturation of backfill.

From their available information they make the following observations:

- Long term creep settlement occurs at a constant rate. Creep rate parameters (α) vary from 0.1 to 0.3. In areas with no compaction, values in excess of 1 were recorded. The creep rate parameter is inversely related to the compaction state.
- Rapid strains occur with saturation.
- Strain as measured at depth shows no apparent relationship to fill depth.
- Strain of the total fill depth is not simply proportional to backfill depth.
- Collapse settlement occurs even with compaction. Compacted areas record values of 0.1 to 0.4 % strain with poorly compacted areas recording up to 1.5 % and uncompacted areas up to 2.5 %. Collapse settlement shows an approximately inverse relationship to the compaction state of the backfill.
- Air void values up to 10 % were recorded.
- Collapse settlement measured at various depths shows no relationship with depth.

Hills and Denby predict mine backfill settlement using a computer model, estimating α with a relationship between α and dry density for UK Coal Measure strata established by Hodgetts et al [23] and collapse strain based on air voids in backfill.

A case study is described where 23 points were monitored over approximately 18 months and compared to predicted values. The values predicted were claimed to be on average, within 10 % of monitored values. However Hills and Denby show typical variations for creep parameters of between ± 40 and 70 % depending on compactive state and ± 100 % in collapse settlement percentage for a specific percentage air voids. How 10 % accuracy is obtained with such variable parameters is not explained.

1.3.2.12 *Bearing Capacity Of Open-Cut Coal-Mine Backfill Materials, Naderian and Williams[24].*

Naderian and Williams investigated the compressibility and bearing capacity of open cast backfill of different ages. Using laboratory tests the basic strength and deformation properties of sandstone and claystone were determined. In-situ plate bearing tests were carried out to assess bearing capacity.

The mining operation investigated was at Jeebropilly colliery in so ...st Queensland, Australia. The mining operation is by bench mining with excavators and 77 ton and 122 ton dump trucks. The backfill is therefore somewhat compacted after dumping by the earthmoving equipment.

From the laboratory tests a significant rapid consolidation settlement during placement is predicted with a significant long-term creep. Naderian and Williams believe rearrangement and crushing of particles to be the phenomena contributing to the long-term creep and propose a well graded material to reduce contact stresses.

The reduction in strength of the material on saturation is the major cause of collapse settlement according to Naderian and Williams.

A series of plate bearing tests was conducted on backfill with ages of one month, 12 months and five years. A steel plate of 750 mm in diameter was used loaded in 20 kPa increments to 200 kPa. An influence factor of 0.785 was used. Young's Moduli were calculated from the results. A high average E value for backfill placed longer than 1 month of 23 MPa was calculated with little variance. The freshly placed material recorded values of 6.3 and 11.1 MPa.

Naderian and Williams concluded that bearing capacity is a function of compaction and not the age of the backfill. The stiffness increased with loading of the mining trucks and will be able to support normal construction activities subject to movement of the water table.

The depth of the backfill layers as placed by the mining trucks is not mentioned in this paper but Naderian et al [25] state that the pit is backfilled by end tipping in a 60 m pit. As the plate bearing test only have a significant influence up to 600 mm any foundation with a larger footprint than 750 mm and therefore higher influence factors at deeper levels may still record significant movement. The results obtained from this paper is only valid for small footings in areas compacted with mine trucks.

1.3.2.13 Numerical Modeling Of Settlements In Back-Filled Open-Cut Coal Mines, Naderian et al[25].

Naderian et al[25], numerically models the differential settlements of the Jeebropilly Colliery using the program FLAC. The placement of the backfill by end tipping with mining trucks into a 60 m pit is modeled. Calculated differential movements varied from 0.3 m or 0.5 % of fill depth to 2.5 m or 4.1 % of fill depth. According to their model 90 % of consolidation settlement occurs when backfilling is completed.

Creep monitored by Naderian et al[25] at Jeebropilly was approaximately 64 mm in 2 years.

1.3.2.14 Simulation Of Water Recovery And Its Effect On Settlement Of Open-Cut Coal Mine Backfill, Naderian and Williams[26]

Naderian and Williams[26] also modeled collapse settlement with FLAC. They concluded that collapse settlement is the major cause of settlement. One-percent collapse was predicted. Minor differential settlement due to collapse settlement was also predicted. Similar collapse settlements were calculated for different depths of backfill.

1.3.3 Summary and Discussion of Literature Survey

In summarising the literature survey, consolidation settlement can be expected with backfill stiffness varying from 5 to 23 MPa. Collapse settlement varies from 1 to 5 % of fill thickness with proposed design values of up to 10 %. Creep settlement parameters vary from 0.1 to 1. General observations made in the literature are that backfill appears to be stiffer with increasing depth, older material appears to be stiffer and pre-loading has a significant impact on later settlement.

Limited information clearly illustrating the heterogeneous nature of the material is available regarding the settlement of open cast mine backfill. If one considers the South African situation the only information available is the work carried out by Jones and Wagener[3, 5, 9, 13, & 14], Wates and Wagner [11 & 12] and Fourie and Wrench[21]. Even though these studies provided good results and the proposed design criteria from these investigations proved to be accurate although sometimes conservative the field-testing had severe restrictions in completeness. The Jones and Wagener field investigations had restriction regarding time and data in their consolidation settlement investigation and regarding effective saturation in their collapse settlement investigation. The Wates and Wagner field investigation only provided results that was specific to the situation, that was the saturation of loosely placed backfill.

The investigations reported in international literature are very site and situation specific, with the exception of the work done by Hills and Denby[22]. Generally only relative small footings were investigated.

Models of settlement prediction by Hills and Denby[22] leave unanswered questions regarding possible accuracy. Naderian and Williams[25 & 26] do generalizations with computer models but correlation with actual data is missing in their work.

Consolidation settlement is generally assumed to have taken place and emphasis is on collapse and creep settlement, except for the work by Charles[20] and Burford[6] as well as the initial investigation by Jones and Wagener[9]. Since the utilisation of mine backfill will include loading of the backfill (except agricultural use) the study of the consolidation settlement is critical and largely missing.

In general recorded data can not be used with much confidence, due to difference in conditions and the spread of results.

2 PERFORMANCE OF AN HYDRAULIC FILL ASH DAM CONSTRUCTED ON MINE BACKFILL

2.1 PROJECT DESCRIPTION

2.1.1 Kriel Pit No. 3

In the preliminary geological and geotechnical evaluation of alternative sites for the proposed new ash disposal facility at Matla Power Station it was recommended by the consultants[27] that the backfilled open cast workings of Anglo American's, Kriel Colliery Pit No.3 was the preferred site. (See Figure 2-2 [28] for an A3 drawing of pit area)

Kriel Pit No. 3 was found most suitable for the ash dam for the following reasons:

- The ground was already disturbed.
- Although the site had been rehabilitated it was only suitable for grazing as an end use. The long-term viability and the quality of grazing on backfill are however viewed with circumspection by the local farmers.
- No viable coal deposits remain under the site.
- The site is topographically favourable.

Extensive investigation and staged development of the proposed ash dam with monitoring observations were recommended.

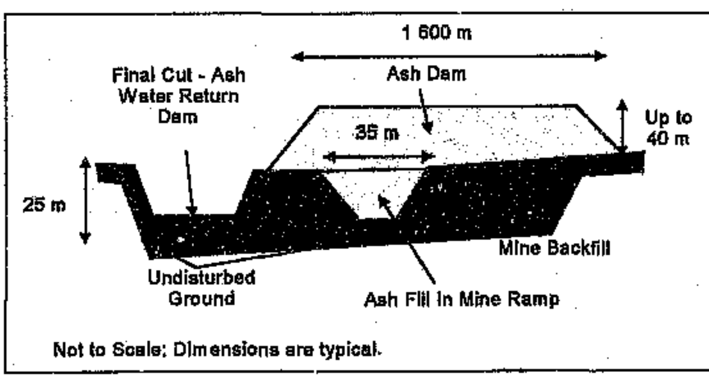
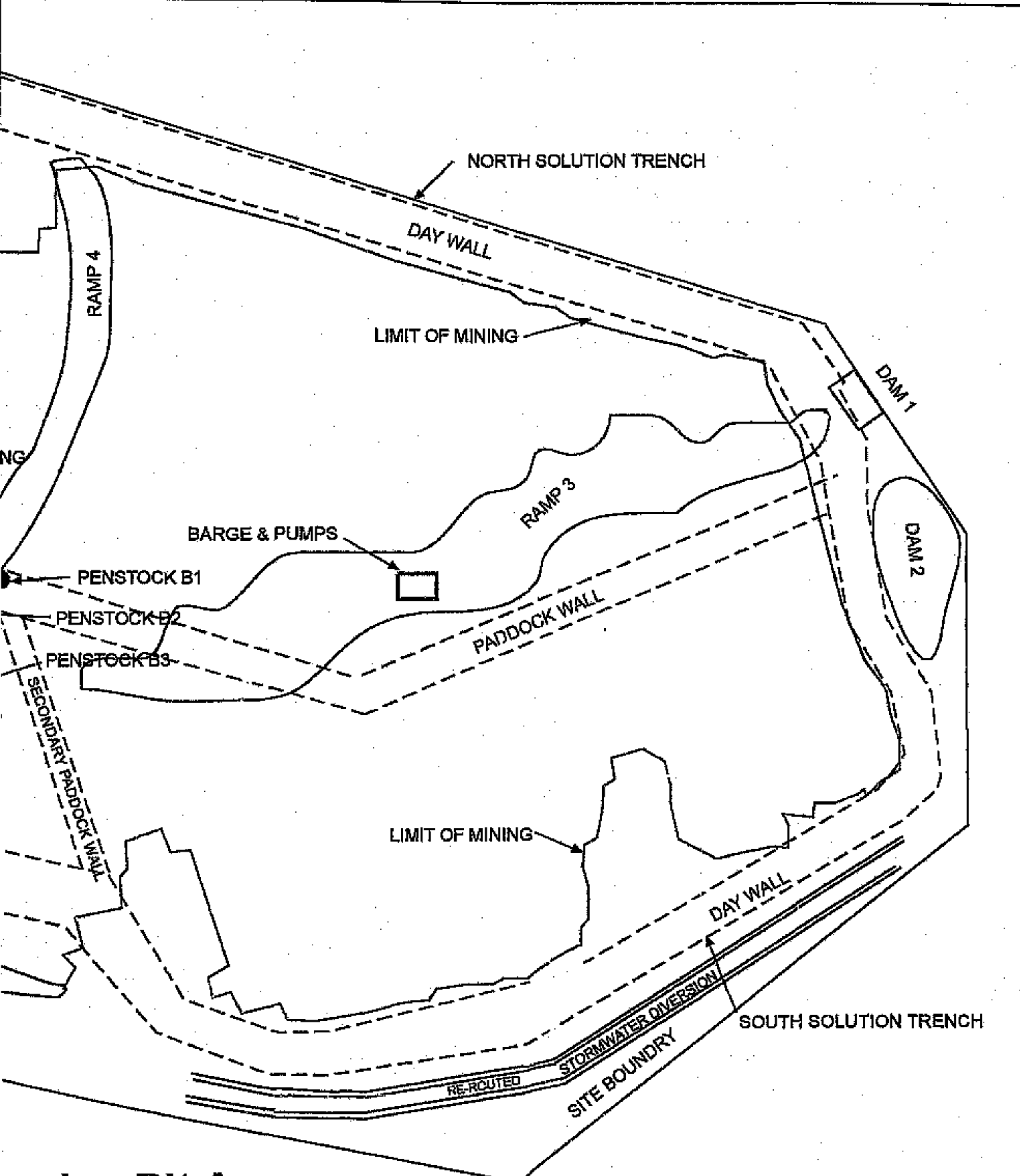


Figure 2-1: Typical Section Through Ash Dam

The subsequent geotechnical assessment[12] recommended that the entire toe or daywall of the ash dam be either located on spoils (mine backfill) or on undisturbed ground. Locating the entire toe on undisturbed ground was found to be technically preferable. The construction of the toe on mine backfill requires undisturbed ground to backfill interfaces, which was regarded as a severe problem. Eskom's (the owners of the ash dam) Civil and Building Division,



awing Pit Area

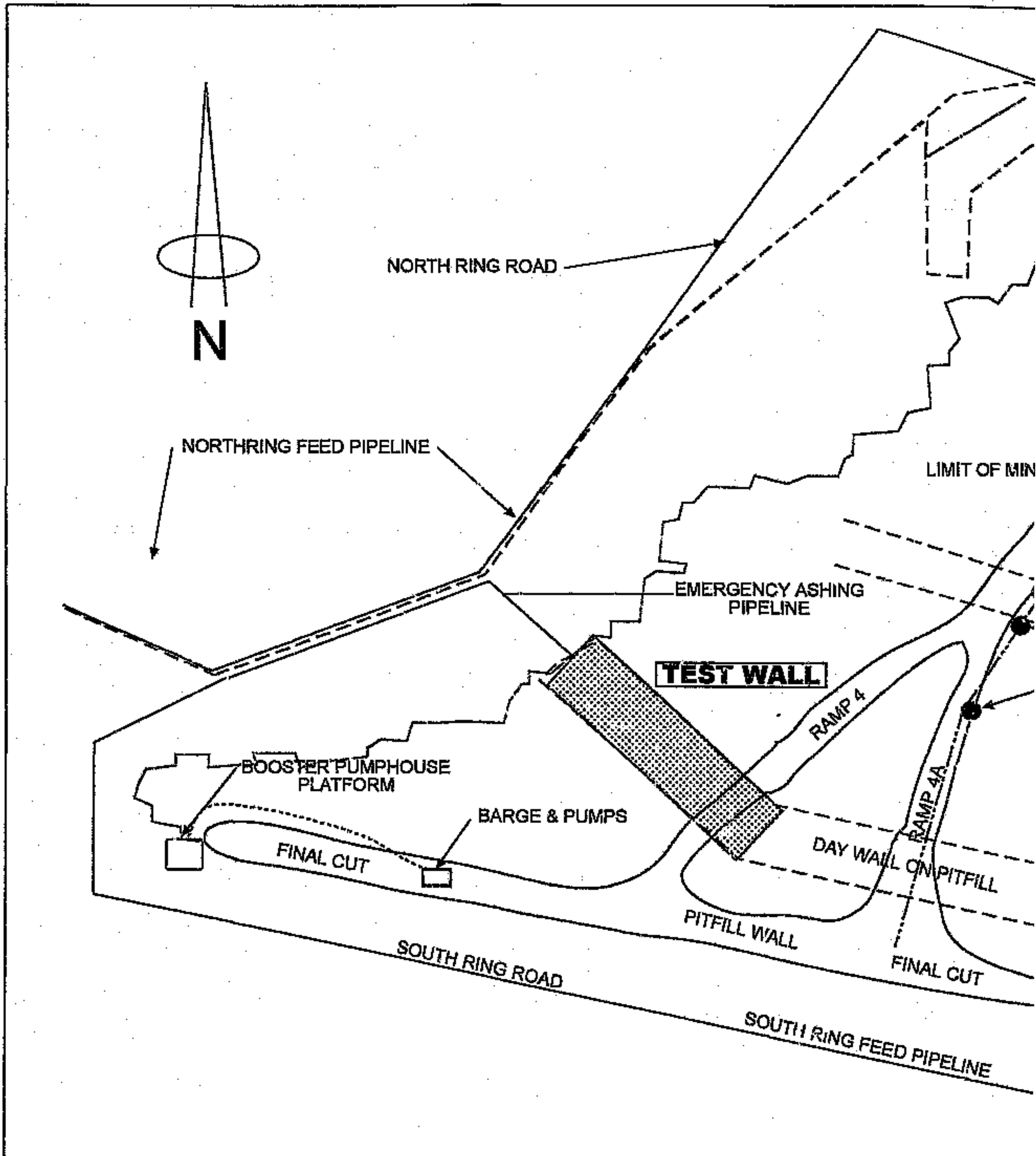

N
NORTH RING ROAD
NORTHRING FEED PIPELINE
LIMIT OF MIN
EMERGENCY ASHING PIPELINE
TEST WALL
RAMP 4
BOOSTER PUMPHOUSE PLATFORM
BARGE & PUMPS
FINAL CUT
RAMP 4A
DAY WALL ON PITFILL
PITFILL WALL
FINAL CUT
SOUTH RING ROAD
SOUTH RING FEED PIPELINE

Figure 2-2: D

responsible for the civil design of the ash dam used this information as their departure point.

The construction of the ash dam was divided into two phases. Phase one included all the mechanical and civil works to enable the filling of the mine access ramps with ash (See Figure 2-1 for typical section). Phase two was planned to include all the mechanical and civil works for the construction of the ash dam above mine backfill level as well as a relocated stream diversion.

2.1.2 Motivation for Test Wall Monitoring Program

Wates and Wagner[12] installed various instruments during 1993 and surveys on these instruments continued to be performed. These instruments were however located for long term monitoring of the ash dam. As the ash dam growth in these areas was slow, no loading of the extensometers occurred and the data was insufficient for a reassessment of the design assumptions.

2.1.2.1 Stream Diversion

Considerable amounts of ash had apparently entered the mine backfill through small sinkholes during phase 1 of the construction. This combined with normal consolidation settlement and collapse settlement (due to increases in water table level) occurring under the self weight of the backfill led the ash-handling contractor (Roshcon) to believe that the settlement originally anticipated in the design phase might be far smaller in practice.

During 1994 Roshcon proposed an investigation into the settlement of the mine backfill under the ash dam at Matla Power Station. Roshcon proposed this investigation as they believed from their practical operating experience on the ash dam that such an investigation might prove it practical to construct the ash dam wall across mine backfill and undisturbed ground interfaces. If this proved practical the daywall could be moved onto mine backfill. The planned relocation of the stream diversion South of the ash dam would then be eliminated with a potential cost saving of R 6 000 000.

2.1.2.2 Slope Stability and Daywall Integrity

The influence on wall stability due to the construction of the ash dam wall on mine backfill was not fully understood at the design stage. Before full scale construction of a small section in the southwestern area commenced, where the daywall was planned to be on backfill, more information was required. The construction of the wall on backfill in this area was necessary to enable the use of the final cut as an ash water return dam.

The development of tension cracks due to differential settlement could have lead to piping erosion as well as a reduction in the factor of safety against slope failure and was a major concern.

2.1.3 Site Conditions

2.1.3.1 Area Geomorphology

The geomorphology of the area surrounding the ash disposal site is dominated by the African erosion cycle with oval shaped pans being a noticeable feature on the flattened planar surfaces. The Bakenlaagte River dominates drainage of the area. The river has been diverted around Kriel Pit No. 3 via a large stream diversion.

2.1.3.2 Geology

Horizontally disposed sedimentary rocks of the Vryheid Formation of the Karoo Sequence underlie the pit area to a depth of 120 m. Cyclic successions of sandstone, shale, siltstone, mudstone and coal occur. Overall the Vryheid Formation consists of regular, horizontally disposed strata essentially free of faulting except for occasional intrusions of dolerite dykes.

2.1.3.3 Pit Stratigraphy

The opencast mining operations and subsequent rehabilitation created a unique stratigraphy. A large pit filled with mine spoils except for the final cut and mine access ramps, ramps 3, 4 and 4a was created with the mining of the coal layers. The backfill comprises of a mixture of residual and transported soils and blasted residual rocks such as shale, siltstone and mudstone. The material was initially placed with a dragline in parallel conical piles creating a series of peaks and valleys. The material was flattened using bulldozers and then most of the area was rehabilitated with a layer of topsoil using mobile mining machinery. The method of placement of the backfill led to a particle size varying from 2 m boulders to silt size material. Some segregation of the backfill has occurred at the bases of the original spoil heaps.

2.1.3.4 Layout and Construction of Test Wall

A settlement-monitoring site was chosen in the southwestern corner of the ash dam. The instrumentation was installed in the area of the planned daywall between ramp 4 and the western edge of the ash dam.

The test wall is a portion of the daywall planned on the mine backfill and was identified as the test section for the following reasons:

- Ease of access.
- Minimal extra operating cost.
- No sterilisation of topsoil. (Parts of the mine backfill had already been rehabilitated with topsoil before the final decision on the site of the ash dam was made.)

- The test area would form a part of the planned daywall. The full size of the test would eliminate all scale problems.

The test area overlies a section of mine backfill measuring 120 m x 280 m. The average backfill depth is 25 m. The average water table depth under the test section was 15.6 m at the time of the test.

The ash daywall is constructed in layers or lifts approximately 700 mm thick. The edge of the daywall is lifted approximately 1 000 mm at a time by mechanical means followed by depositing fly ash slurry into the area. The water/ash ratio is 1:1 by mass.

2.1.3.5 Ground Water Re-establishment

The mine artificially lowered the ground water table in the mine pit during mining. As the mine operation has moved away from this area it was anticipated that the water table would slowly start to rise and re-establish itself due to water from the ashing operation and rainfall recharging the ground water from the surface.

2.2 INSTRUMENTATION OF TEST WALL

2.2.1 Instrument Selection and Design

2.2.1.1 Requirements of Monitoring Program

The essential requirement for a settlement-monitoring program is that it must enable the creation of a reliable and accurate subsidence model. Thus a program that gives an indication of the following is required:

a) Consolidation settlement of the mine backfill layer under its own weight at first and later under the overburden weight of the ash layers.

b) Collapse settlement of the mine backfill layer as the water table re-establishes itself.

c) Settlement above the water table induced by the washing out of fines.

d) Determination of the nature and extent of the ash infiltration into the mine backfill.

e) The effect of the infiltration of ash particles into the mine backfill on:

   - ash dam settlement as described in points a, b and c.
   - surface volume required for the ash dam.

2.2.1.2 Parameters to be Monitored

Three parameters were monitored with time:

a) Vertical deformation of the mine backfill.

b) Fluctuations of the water table.

c) Ash infiltration into mine backfill.

2.2.1.3 *Predictions of Magnitude of Settlement in Test Area*

The basis for the original design work was an initial investigation by Wates and Wagner[12]. A 10 % settlement after saturation was predicted in this study. Maximum settlement expected according to this prediction in the area where the monitoring was carried out was approximately 2.5 m.

2.2.2 Instrument Description and Installation

Settlement monitoring instrumentation can be unreliable unless well designed for its specific purpose. With the scale of operation, the large particle size, the heterogeneous nature of the material and the magnitude of settlement expected, this problem was expected to manifest itself. A very rugged, simple and reliable method of settlement monitoring was therefore designed and installed.

Three types of instrumentation were decided upon:

2.2.2.1 *Extensometers*

Five sleeved extendible 12 mm diameter aluminium rods were anchored at various levels within three boreholes (See Figure 2-4 for schematic cross section and Figure 2-3 for relative positions). The boreholes (numbered E1 to E3) were drilled at approximately the centre of the test wall at approximately 50 m horizontal intervals. The holes were drilled to at least 25 m deep. (See Figure 2-3 for schematic layout of test wall)

The aluminium rods or extensometers were numbered E1/1 to E1/5 for borehole E1, E2/1 to E2/5 for borehole E2 and E3/1 to E3/5 for borehole 3. The number 1 benchmark was approximately 1 m from the surface with benchmark 5 the deepest at 24 m.

The following installation procedure was followed (See Figure 2-4):

- Measure borehole depth.
- Lower sleeved aluminium rod (3 m lengths threaded at both ends and connected) into borehole. The rod was sleeved with 20 mm diameter PVC piping and taped to the bottom of the rods to prevent separation during installation. A reinforcement bar was screwed to the bottom of the aluminium rod to anchor the rod into the grout.
- Anchor rod with grout. Grout was placed at bottom of hole through a 50 mm PVC pipe to protect the sides of the borehole where it was not sleeved and to allow for better control over the placement of the grout anchors. The grout

Schematic Layout

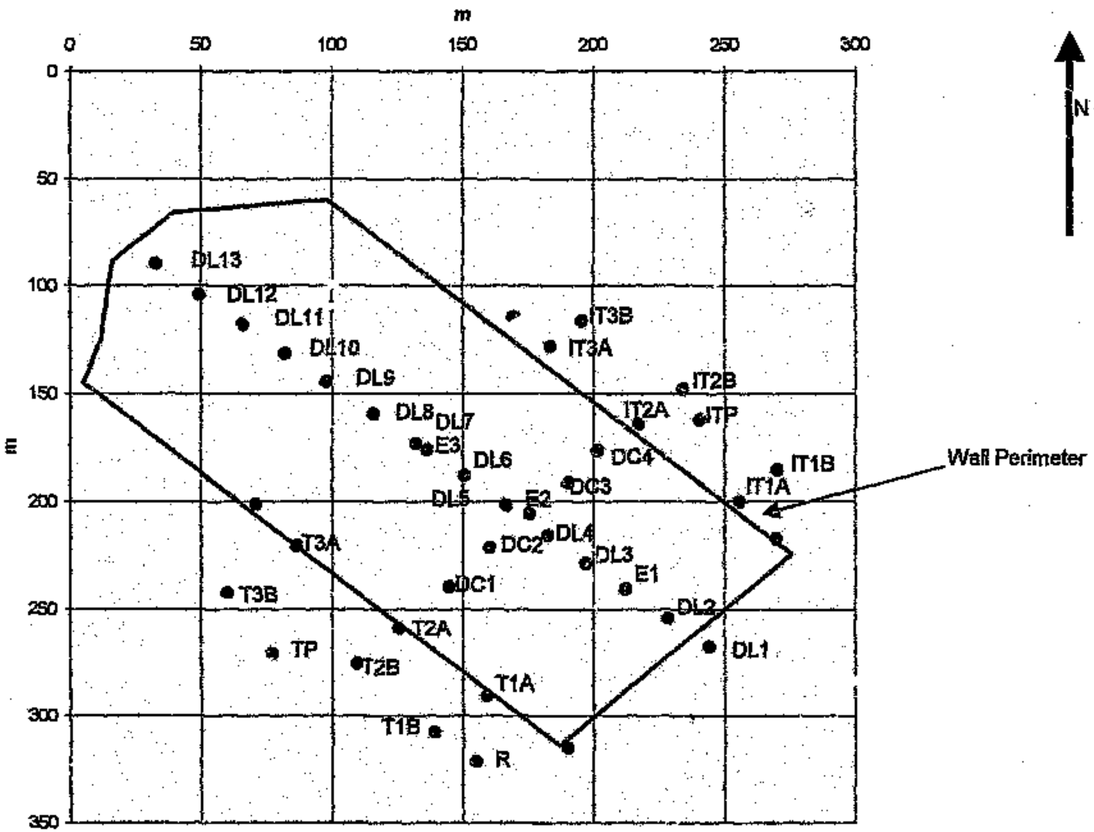


Figure 2-3: Schematic Instrumentation Layout

was then given a one day curing period to ensure that the rod was securely anchored. During anchoring the rod was suspended approximately 100 mm from the bottom of the borehole to ensure grout penetration beneath the rod.

- Test to see if rod anchored. If not, add more grout. Rod E3/5 had to be grouted three times before the rod was anchored. This was possibly due to the presence of a substantial void above the bedrock.
- Measure borehole depth.
- Place layer of 6 mm gravel in borehole.
- Install piezometer
- Place further layer of 6 mm gravel to just below level of next benchmark.
- Place PVC sleeve over piezometer and aluminium rod sleeves. (This was only done at 1, 5 and 10 m benchmarks to reduce the risk of failure of the instrumentation due to possible large differential settlements within these top layers.)
- Add 6 mm gravel up to the level of next benchmark.
- Anchor next rod.
- Repeat process at all the required anchoring levels. Benchmark 4 was planned to be placed just at the existing water table to differentiate between settlement in saturated and unsaturated material. Due to a significant fluctuation in the water table level during installation these benchmarks were however placed above the water table of the base survey.
- Add extendible casing at surface to prevent ash infiltration into the borehole.

2.2.2.2 *Piezometers*

Five open standpipe piezometers were installed, one in each of the extensometer boreholes (Numbered EP1, EP2 and EP3) and two in purpose-drilled boreholes (ITP upstream and TP downstream).

The piezometers consist of 25 mm PVC pipes coupled with PVC sockets glued together with PVC glue. The bottom of the piezometer consisted of a 300 mm piece of 25 mm diameter PVC, drilled with twelve 10 mm diameter holes which were covered by a porous polyethylene filter.

The piezometer installation consisted of the following steps:

- Measure borehole depth.

- Add layer of 6 mm gravel.
- Place piezometer in hole.
- Add 6 mm gravel to above filter tip by measuring borehole depth as the gravel is added. (Gravel to act as filter)
- Fill borehole to surface with material from drilling.
- A lockable cap was placed over piezometer TP, as this piezometer was the most accessible to vandalism.

2.2.2.3 Settlement Rods

A series of extendible settlement rods consisting of 25 mm galvanised pipe with a 100 mm T- piece anchor, cast into 500 x 500 x 300 mm thick concrete blocks were installed along the longitudinal and cross sections of the dyke. They were numbered DL1 to DL13 for the long section and DC1 to DC4 for the cross section.

A series of reference benchmarks was installed outside the wall area with a reference benchmark R outside the estimated influence area of the dyke. Benchmark R was monitored from benchmarks outside the pit area.

2.2.2.4 Borehole Data

Rotary percussion drilling was used for the boreholes. The boreholes were flushed with air and various problems were experienced with the substantial voids intersected in the fill. Placing steel casings into the boreholes reduced the problem of loss of air pressure into voids. The casings were extracted as the instrumentation was installed.

Identification of the intersection with bedrock was a major problem. Due to the use of rotary percussion drilling the identification of discontinuities was difficult. From previous surveys the depth of bedrock was estimated to be not more than 25 m. It can therefore not be guaranteed that the deepest anchors are anchored into bedrock. These anchors must, however, be very close to bedrock, in saturated and pre-loaded material and therefore the settlement potential of the anchors is very small. Other problems (See "Instrument Performance") however nullified the use of these benchmarks.

All the boreholes passed through a fairly homogeneous shale material with areas of sandstone boulders.

The borehole for one of the piezometers outside the daywall showed extensive ash infiltration at depths of 6 to 10 m. The borehole was drilled approximately 20 m from a substantial ash infiltration recorded by the construction supervisor. An interesting observation is that this infiltration occurred in line with the rows of spoil heaps placed by the dragline during the backfilling operation.

For detailed drilling logs see Appendix B, page 67.

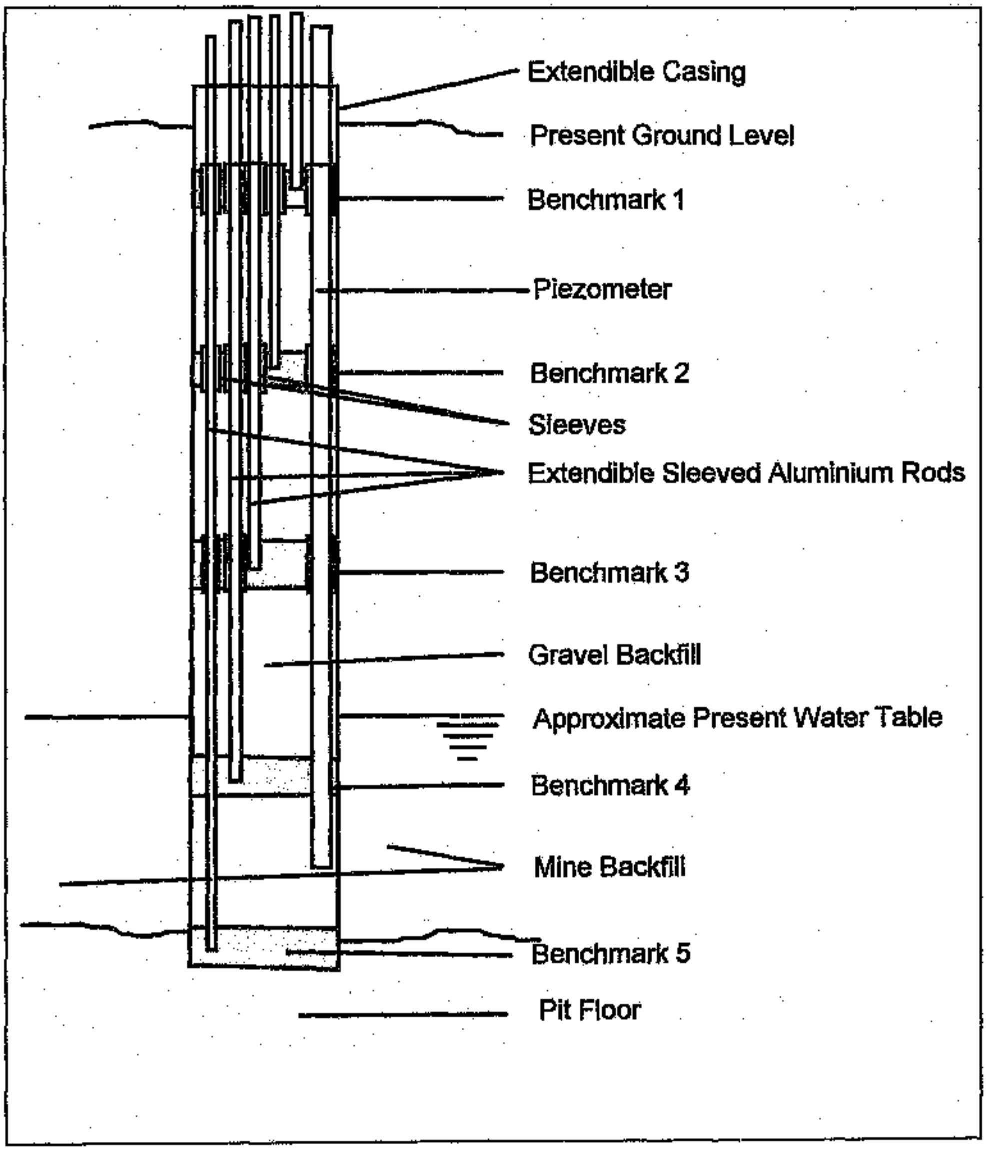


Figure 2-4: Typical Extensometer

2.2.3 Instrument Calibration

2.2.3.1 Installation depths

All measurements in the boreholes were done using a tape measure weighted at the zero end.

The aluminium rod lengths were known and therefore anchoring depths could be calculated fairly accurately. Installation depths as supplied by the installation subcontractor are included in Appendix C.

An electrical dipmeter was used to measure water table levels.

2.2.3.2 Base Survey

A professional land surveyor carried out a base survey on 9 May 1994. All the anchor depths were translated to levels using the installation information. Water level readings were also carried out on the same day. This information was regarded as the base information upon which the monitoring program was based.

A list of all the measured and calculated levels is included in Appendix D. (The "concrete" levels indicate the levels of the concrete anchors of the benchmarks.)

2.2.4 Instrument Performance

2.2.4.1 Extensometers

All three extensometers performed well and provided reliable results. Problems experienced are that the deepest extensometer anchors have shown erratic movements. A possible explanation for this is that during installation these anchors were not sleeved from Benchmark 4. This may explain the similar settlements recorded for these two levels. The strains in layers 4 to 5 have therefore also been evaluated ignoring the data for benchmark 5 and assuming that solid bedrock is at 25 m depth. The results of the fourth benchmark should however not be influenced by this problem due to the low stiffness of an 8 m long, 12 mm diameter aluminium rod.

The agreement of the extensometers with adjacent settlement rods was very good and indicated that the pea gravel used to separate the benchmarks conformed to the backfill settlement.

2.2.4.2 Settlement Rods

No problems were experienced with the settlement rods.

2.2.4.3 Piezometers

The piezometers proved to be unreliable and of the five installed only one survived. The piezometers in the wall were all lost due to differential settlement. Piezometer (ITP) outside the wall was lost due to an accumulation of unforeseen events. Water infiltrated the borehole during ashing at an emergency ashing area, washing out the backfill material near the surface of the borehole and then scraping the standpipe against the edge of the borehole, thus puncturing it and then flooding it with ashing water from the surface.

The one remaining outside piezometer (TP) is still operational and enough information is available to correlate its readings with the water table in the area under the test wall.

In future the installation of vibrating wire piezometers with cables leading to data loggers should be considered for similar applications.

2.3 OBSERVATIONS AT TEST WALL

Surveys were carried out on days 0, 30, 50, 80, 99, 142, 184, 275, 338, 463 and 625.

During this period the ash daywall grew in height to approximately 7 m (See appendix A for schematic cross section of wall growth). This chapter only illustrates the observed settlement of the various instruments. Evaluations of the results and stress strain graphs follow in chapter 4.

The settlement versus the stress imposed by the ash wall is illustrated in figures 2-5 to 2-9. Steinbrenner influence factors were calculated for the different positions of the instruments in the wall. The loading was calculated using surveyed wall height, the influence factor and an assumed ash density of 1 300 kg/m^3.

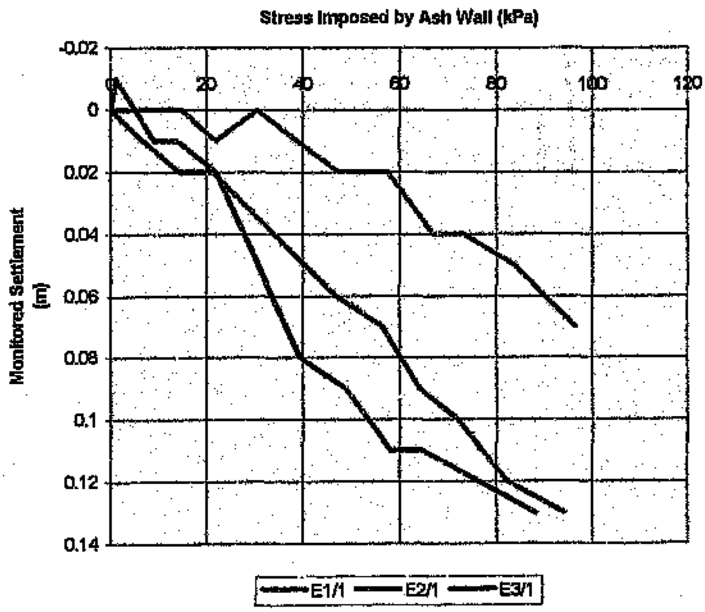


Figure 2-5: Comparison Of Settlement Of Mine Backfill Layer Depth 1 To 5 m

2.3.1 Recorded Data

2.3.1.1 Extensometers

Figures 2-5 to 2-8 illustrate the settlement of the layers of mine backfill as recorded by the extensometers.

Good correlation between extensometers 1 and 3 were recorded in the upper layers (Figure 2-5 and 2-6). The significant difference with extensometer 2 in the upper layers is possibly due to the presence of sandstone boulders in the area.

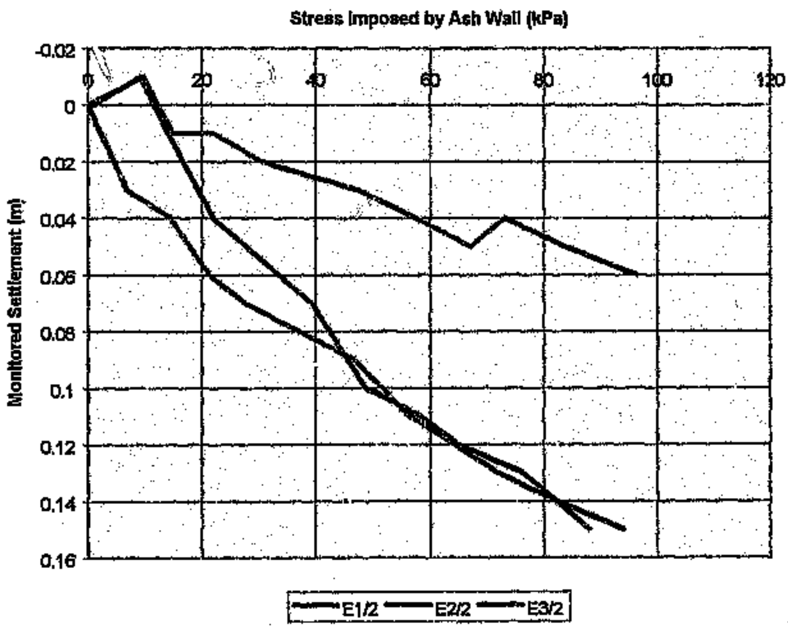


Figure 2-6: Comparison Of Settlement Of Mine Backfill Layer Depth 5 To 10 m

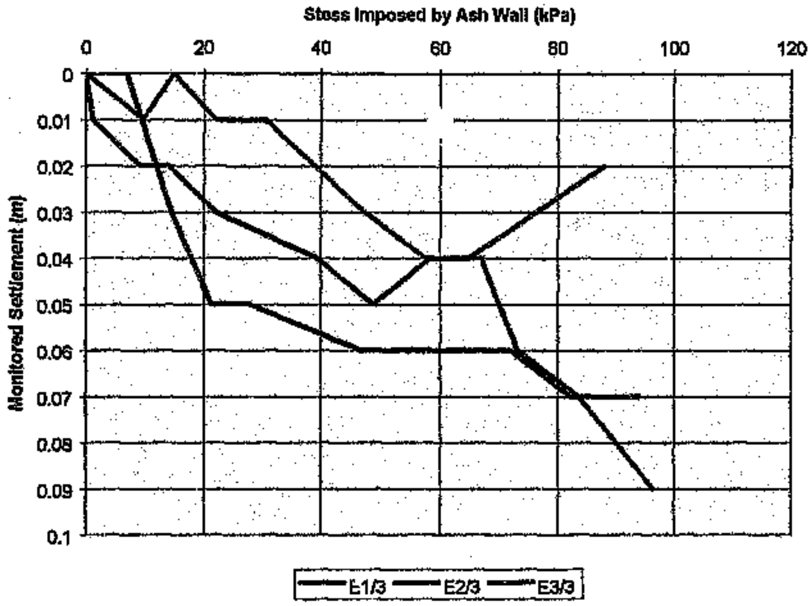


Figure 2-7: Comparison Of Settlement Of Mine Backfill Layer Depth 10 To 16 m

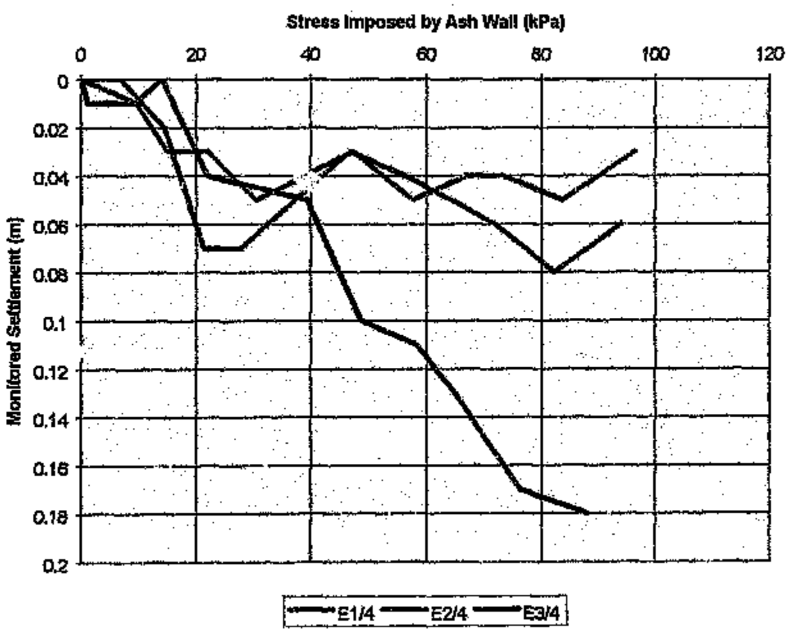


Figure 2-8: Comparison Of Settlement Of Mine Backfill Layer Depth 16 To 25 m

Extensometers 1 and 2 again correlate better in the lower layers (Figure 2-7 and 2-8). The large settlement recorded in the lowest layer of extensometer 3 is possibly due to the presence of a void or low-density area which was recorded during drilling.

The illustrated settlement of layer 4 ignores settlement recorded by deepest extensometers (E1/5 to E3/5).

2.3.1.2 Settlement Rods

Comparative settlement of all the settlement rods as well as the three surface (1m deep) extensometers is illustrated in Figure 2-9.

A survey done in March 1997 with no further loading of the wall recorded an average creep settlement of 6 cm after 14 months under a consistent load of approximately 73 kPa.

2.3.1.3 Piezometers

Water table fluctuations of up to 1,5 m were recorded. The ash water return dam in the final cut governed these fluctuations and no significant influence on the settlement was recorded.

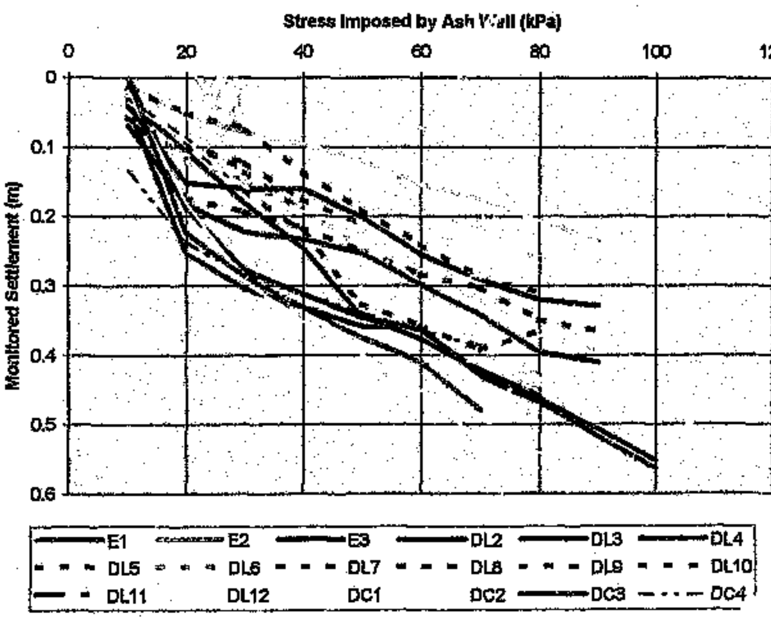


Figure 2-9: Recorded Settlement All Surface Extensometers And Settlement Rods.

2.3.2 Visual Observations

A very important observation made during the construction of the test wall is that no tension cracks formed in the wall or on the edges. If one considers the fact that the eastern slope was constructed at an angle steeper than 1 in 1 and the northern slope steeper than 1 in 2 this is quite remarkable. Furthermore, 350 mm of measured differential settlement that occurred between two points which are approximately 20 m apart did not create any sign of distress within the wall.

2.4 OBSERVATIONS OF WATES AND WAGNER INSTRUMENTATION

Surveys were carried out from 30 March 1992 to 20 November 1997 at regular intervals on existing instrumentation installed by Wates and Wagner[12] for ash dam performance monitoring. A problem with this data is that no analysis was done on the information during the latter part of this period. When the data was analysed in November 97 various irregularities appeared. From the information it appears as if benchmarks, assumed to be stable, were creating various problems with the data.

2.4.1 Extensometer MB1

One extensometer MB1 similar to the ones installed in the test wall, was installed with 4 aluminium rods, the deepest anchor in bedrock or very close to it. An assumption that the deepest anchor was stable was made and the relative movement of the other three extensometers relative to this one is illustrated on Figure 2-10. MB1/1 is the deepest anchor in bedrock, MB1/2 at 12.5 m deep, MB1/3 at 7 m deep and MB1/4 at 4 m deep.

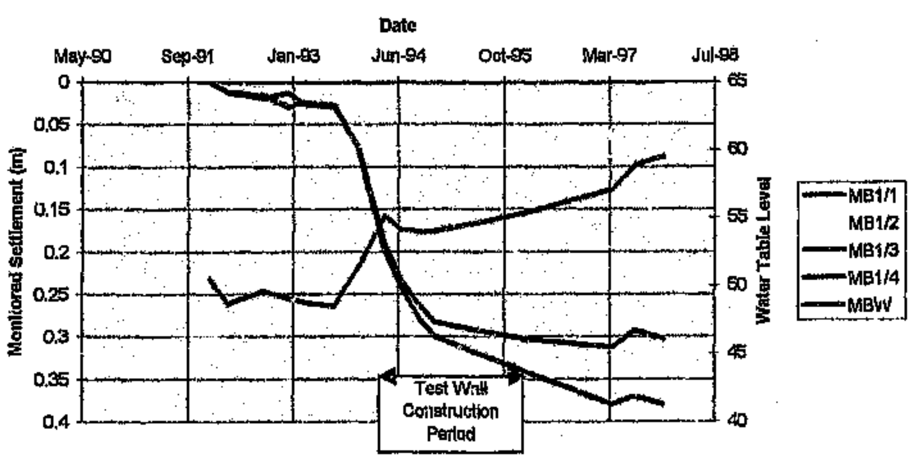


Figure 2-10: Settlement Extensometer MB1 With Saturation Of Backfill

Initially from January 1992 to August 1993 a total creep settlement of 30 mm was recorded. The major part of this settlement, 20 mm occurred between MB1/1 and MB1/2. Fluctuations in the water table may have led to some collapse settlement in this layer.

The quick re-establishment of the water table (MBW) experienced in this area due to an excess water problem is also illustrated on Figure 2-10. One can clearly see the collapse settl[illegible]t occurring as the water table rises. Figure 2-11 illustrates the water table movement relative to the depths of the various anchors.

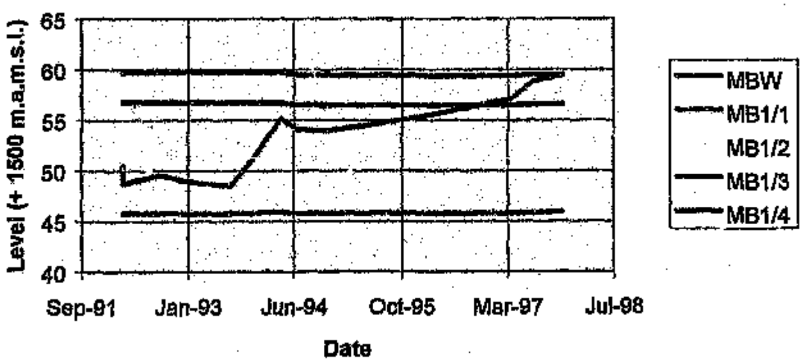


Figure 2-11: Re-Establishment Of Water Table At Extensometer MB1

3 PERFORMANCE OF A DRY PLACED ASH DUMP CONSTRUCTED ON MINE BACKFILL

3.1 PROJECT DESCRIPTION

3.1.1 Introduction

In the initial investigation on the disposal of ash from Lethabo Power Station[29] it was concluded that in general, dry ash disposal on open cast strip mine backfill is an environmentally correct solution, avoiding the sterilisation of good arable land. Due to the proximity of the Vaal River and a major water resource underlying the pit they however believed that it was the correct disposal method at the wrong place. Subsequent research by Hodgson [30] indicated that there would be a nett loss of rainfall due to evaporation on the ash dump which gave Eskom the confidence to continue with the design of the ash dump on the backfilled pit of New Vaal Colliery.

This method creates technical problems of which the influence of the settlement of the loosely placed mine backfill on surface drainage of the completed ash dump is the major one.

In the subsequent investigation by Jones and Wagener[9] on compressibility of the mine backfill at Lethabo Power Station, the conclusion was drawn that *"A large proportion of the observed settlement occurred shortly after loading and primary settlement was complete in less than two months."*. Jones and Wagener further recommended additional monitoring on variations in consolidation and collapse settlement. Their subsequent report[12] addressed collapse settlement but the degree of confidence in these results was low due to the very wide range of results from various sources.

3.1.2 Motivation for a Monitoring Program

A formal settlement monitoring program was therefore needed for gathering of essential information to create an accurate subsidence model. The information is essential with regards to long-term environmental impact assessment, storm water drainage design, ash dump construction, the possibility of future land utilisation and the future level of the mine backfill to ash interface surface.

3.1.2.1 Environmental Impact

The permission to construct an ash dump over undermined ground was given to Eskom by the Government Mining Engineer. The following were however conditions for the construction of the ash dump:

- Eskom is liable for any environmental problems that may arise during and after the completion of the ash dump.

- The ash dump must be constructed in accordance with the procedures contained in Eskom's Environmental Impact Control Section's report on "The Proposed Method to Dispose of Ash".

In the report "The Proposed Method to Dispose of Ash" it is recommended that the ash be placed above the spoil and soft overburden and also above *the future water table* that is estimated to be at 1421 m a.m.s.l. This will reduce the likelihood of pollution of the Vaal River and underlying aquifer system by leachate.

These considerations were the deciding factors in choosing the 1429 m level as the final level to which the mine backfill is spread. An 8 m tolerance between mine backfill level and estimated future water table was decided upon. This tolerance will cater for an estimated 6 m of settlement at that stage and a 2 m safety margin.

Subsequently Jones and Wagener recommended the use of a Young's modulus of 6 MPa for consolidation settlement and 5 % of backfill thickness for collapse settlement. On a full height ash dump (50 m) these parameters give 6.55 m of settlement for 50 m of mine backfill. Considering the mine backfill tolerance of – 0.5 m only a 1 m safety margin remains.(On this estimate).

Due to the low degree of confidence in the settlement estimates, it was therefore essential to monitor the settlement continuously. The monitoring will either confirm the previous results or provide an early warning system that the ash will settle into the future ground water table.

3.1.2.2 Storm Water Drainage Facilities

Settlement monitoring is also needed to ensure the continued functioning of the ash dump's storm water drainage facilities. Differential settlement of the ash dump could influence the storm water runoff patterns, possibly rendering the existing and planned storm water facilities ineffective. Local depressions could cut off the flow of canals and create ponds resulting in further localised settlement and the generation of ash leachate into the mine backfill. Larger scale settlements could change the runoff directions of entire catchment areas, requiring major ongoing remedial works.

Accurate prediction of settlement is therefore a requirement for any future storm water drainage planning.

3.1.2.3 *Ash Dump Construction*

Accurate prediction of the total and differential settlement of the ash dump and mine backfill will make it possible to anticipate settlements and account for them by overstacking ash in the problem areas. This will result in savings on bulldozing and hauling when re-contouring for final shaping of the ash dump before rehabilitation.

3.1.2.4 *Ash Dump Stability*

Tension cracks as wide as 13 cm have developed due to differential settlement of the ash. These cracks lead to an increase in probability of a deep seated slope failure within the ash dump. Drainage problems due to these cracks have also occurred.

Understanding the mechanics of these cracks should make it possible to predict their size and impact on ash dump stability.

3.1.2.5 *Level of Handover of Mine Backfill*

With more accurate information available on mine backfill settlement, a re-evaluation of the 1429 m a.m.s.l. level can be done. It may be possible to use a larger tolerance, which may lead to considerable savings in mining costs.

3.1.2.6 *Legislation*

The Water Act No 54 of 1956 is very important to the construction of the ash dump. Government Notice R287 of February 1976 contains the following relevant regulation:

> Regulation 12 "The manager of a mine or works shall ensure that any minerals, tailings and waste rock dump and any slimes, barrier or evaporation dam, storm water drain or other waterway is as far as practicable located and designed so as to minimise the possibility of damage thereto by subsidence, settlement, shock and cracking due to past, present or future operations at the mine or works."

The monitoring of the settlement to confirm that the storm water canals and the drainage concept of the ash dump will not be "damaged" by settlement is therefore a legal requirement.

3.1.2.7 *Possible Future Land Utilisation*

There is the possibility that the in-pit ash dump after the closure of the Power Station could become prime land for developments due to its proximity to the Vaal River and Vereeniging.

Construction over this area will be dependent on the reliability of information on the possible remaining settlement of the area.

3.1.3 Site Conditions

3.1.3.1 Area Geomorphology

The New Vaal Colliery pit is situated on the Free State bank of the Vaal River, south east of Vereeniging. The topography of the area is flat and undulating. Maccauvlei, situated on the north eastern side is a low lying poorly drained area.

3.1.3.2 Geology

The site is situated on the Vryheid Formation of the Ecca Group, Karoo Sequence. Mainly sandstone with interbedded mudstone and coal seams are encountered. The Vryheid formation is underlain by Dwyka tillites and Malmani dolomite(Which is the aquifer referred to earlier).

The entire area is overlain by approximately 10 m of sandy alluvium.

3.1.3.3 Construction Method for Mine Backfill

The basic mining operation is illustrated in the schematic cross section Figure 3-1, and is as follows.

Initially the alluvial sands are stripped and stockpiled. Thereafter the overburden and interburden are blasted into the previous cut and the coal is removed from between these layers up to a depth of approximately 50 m.

The over- and interburden are then cleared by a dragline that dumps this material in front of the ashing operation creating the sharp crested rock backfill heaps shown in Figure 3-1.

The alluvial sands are then hauled by mobile plant or a conveyor system and loosely dumped onto the dragline spoil heaps and leveled with mobile plant to the required 1429 m a.m.s.l.

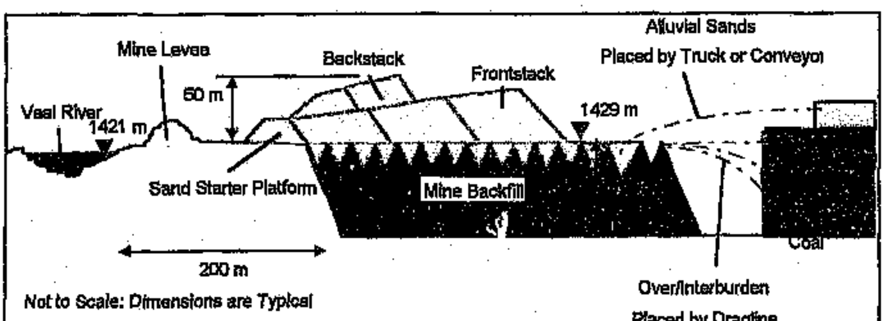


Figure 3-1: Typical Cross Section Lethabo Ash Dump And New Vaal Colliery Pit

3.1.4 Ash Dump Development

After an area of leveled mine backfill has been handed over to Lethabo, ashing commences on the area. The ashing operation sequence is illustrated in the schematic cross section Figure 3-2.

An 11m high sand starter platform was constructed on the mine backfill running from north to south. Shiftable conveyors were erected on this platform.

Ash is transported from the power station at an average of 500 ton/hr in a semi dry state (13 % moisture) via overland conveyors to the in-pit ash dump. At a transfer house the ash is transferred to either one of the two extendible conveyors. These conveyors are extended at regular intervals as the ash dump develops in a westerly direction. Each extendible conveyor ends with a headstation that transfers the ash to a shiftable conveyor. Stacker machines are connected to the shiftable conveyors from which the ash is stacked.

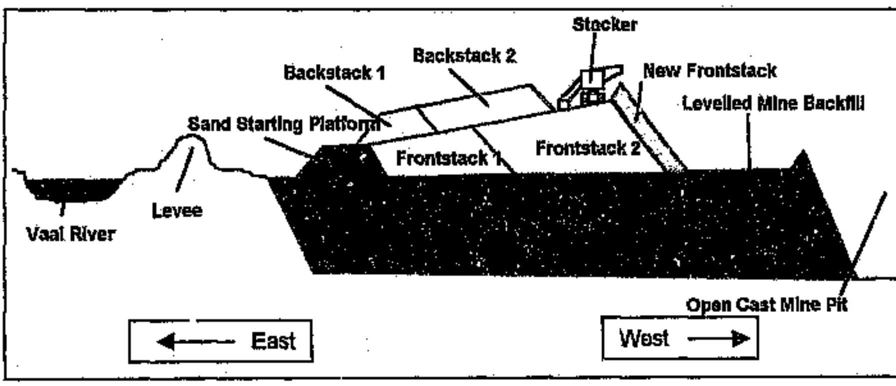


Figure 3-2: Schematic Cross Section In-Pit Ashing

Initially, only a frontstack was constructed. This involved dumping the ash over the western side of the starter platform onto the mine backfill. This newly ashed area was flattened, as the ash was stacked. When the stacker system could no longer reach over the edge the extendible conveyors were extended and the shiftable conveyors were shifted parallel in a westerly direction by approximately 42 m. The ashing then started again by firstly ashing behind (backstacking) on the eastern side of the conveyors and then again over the edge (frontstacking) until the stackers could not reach any further. The conveyors were then shifted again. This process will continue in repeating cycles until the closure of the Power Station.

The frontstack grows at an incline and will reach a maximum of 38 m above the mine backfill level. The backstack will vary in height between 8 m and 12 m. The backstack will be contoured and sloped to provide an acceptable storm water and erosion control system.

3.1.4.1 *Ground Water Re-establishment*

The mine artificially lowers the ground water table to de-water the mine. As the mine operation will move westwards faster than the ashing operation, the mine pit will gradually move away from the ash dump. The water table will then slowly start to rise (but to a very limited extent due to the high permeability of the backfill) and re-establish itself beneath the ash dump.

When the mining operation stops the water table will fully re-establish in the area. Due to the permeability of the mine backfill the final phreatic surface will be nearly horizontal and will establish itself to the level of the Vaal River or approximately 1421 m a.m.s.l. This level will be governed by the water level in a planned mine lake north of the ash dump.

3.1.4.2 *Environmental Conditions*

The area of the ashing operation is very environmentally sensitive due to the close proximity of the Vaal River. Water for the PWV region is pumped from the river downstream of the Power Station. A further environmental constraint is a large water resource beneath the mine pit floor in form of the Malmani dolomitic aquifer.

Environmental protection thus prohibits leaching of the ash water into the mine backfill. No unlined ponding is allowed on the ash dump surface as this water may leach through the ash into the mine backfill.

3.1.4.3 *Mechanisms of Mine Backfill Settlement*

Similar mechanisms of mine backfill settlement are expected as discussed in chapter 1. In the Lethabo case the mechanism may vary due to the water table re-establishing itself from below the ash dump area at the floor of the pit and not from the surface. The different method of re-establishment of the water table with continuous movement of the water in a horizontal plane caused by the de-watering of the pit may lead to differential movement of fine particles. Geotechnical properties of the materials may also vary.

3.2 INSTRUMENTATION OF ASH DUMP

The work was broken down into three distinct phases.

3.2.1 Design and Installation

3.2.1.1 *Essential Requirements for the Monitoring Program*

The requirements for a settlement monitoring program are the same as for the program at Matla and require giving an indication of:

- Consolidation settlement of the different mine backfill layers under their own weight at first and later under the overburden weight of the frontstack and finally under the backstack of the ash dump.

- Collapse settlement of the different mine backfill layers as the water table re-establishes itself. Problems were foreseen in obtaining accurate collapse settlement results due to the high permeability in the backfill.

- Consolidation settlement of the ash dump under its own weight.

It was further proposed that the movement within the ash front face also be investigated to throw more light on the tension cracks experienced on the frontstacks. It was believed that the following sequence of events led to this cracking. (See Fig. 3-4)

- The frontstack is placed and the mine backfill settles rapidly. (A)

- Pozzolanic action takes place within the settled ash and the ash's cohesion increases substantially.

- The new frontstack is placed.

- The mine backfill beneath the new frontstack settles. (C)

- The previous frontstack face settles under new frontstack weight. (B)

- Tensile stresses develop.

- Tension cracks occur.

With an increase in height of the ash dump this problem is likely to increase in the future. A full understanding of this process will enable the prediction of the size and impact of this problem in the future.

3.2.1.2 Parameters to be Monitored

Five basic parameters are being monitored:

- Vertical deformation of the mine backfill.

- Differential vertical deformation of the mine backfill under the front face of the ash dump

- Vertical deformation of the ash dump.

- The re-establishment of the water table.

- The time these deformations take to manifest themselves.

Other information required for the creation of a subsidence model is:

- Mine pit floor levels.
- Dragline heap and sand backfill geometry.

3.2.1.3 Prediction of the Magnitudes of Settlement

The investigation by Jones and Wagener [9] was used as the basis for desig.. work at this stage. A 1 m settlement in two months under 11 m overburden was recorded in their study.

Allowance was made for an average collapse settlement on saturation of 5 % of the backfill thickness; this is additional to normal settlement, the magnitude of which was calculated assuming an average elastic modulus of 6 MPa.

Maximum settlement expected was approximately 6.55 m.

Maximum total differential settlement expected in the area of monitoring between mine backfill with no overburden and mine backfill with 31 m of ash overburden is approximately 5 m. The total settlement is expected to take several years to materialise.

The settlement expected in the ash layers is difficult to calculate due to the compaction action of the ash falling through an average height of 20 m. Based on experience on ash dumps built on solid ground it is likely that the settlement will be much less than that of the mine backfill.

3.2.2 Instrument Selection, Location, Purpose and Installation

Instrument selection and location are very sensitive and problematic due to the scale of the operation, the method of construction of the ash dump and the magnitudes of settlement expected. A very rugged, simple and reliable method of settlement monitoring was therefore required.

3.2.2.1 Instrument Location

The locations of the proposed instrumentation are illustrated on Figure 3.3.

3.2.2.2 Survey Techniques

Forty benchmarks (settlement monitoring points) will eventually be placed on top of the ash dump to monitor the movement of the ash surface. Twenty-six of these benchmarks will be covered by the backstack, six will be extended to protrude above the backstack before the backstack covers this area. These will include the 3 benchmarks above the settlement cells. This will enable measuring the settlement within the backstack as well. The other benchmarks will be replaced on the surface of the backstack with new benchmarks. The line of benchmarks (Fig. 3-3) parallel to the conveyor will be chosen to include all possible combinations of mine backfill layers. The line may be repeated at sensible intervals, depending on the speed of advance of the ash face and the composition of the mine backfill layers.

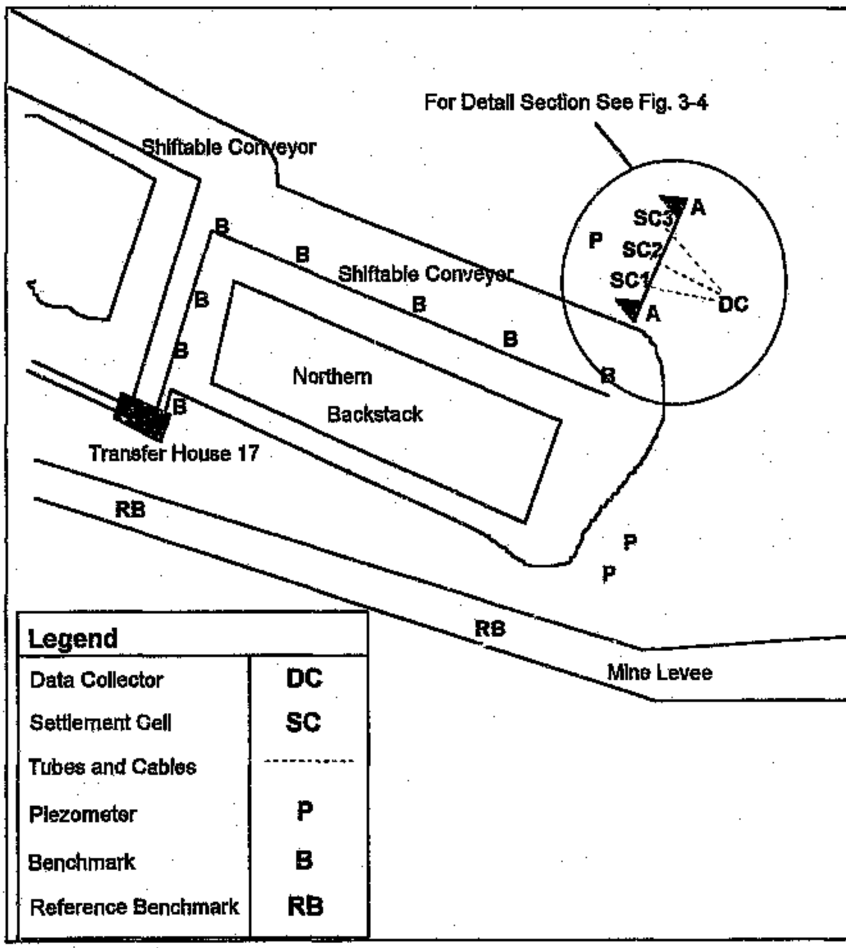


Figure 3-3: Schematic Layout Instrumentation Position

The normal benchmarks will be monitored monthly. A [illegible] surveyor will monitor the reference benchmarks, which are existing, at 3 month intervals; this will serve as a regular calibration of the system.

The reference point at the data collector and the top of the existing piezometers will be surveyed from the reference benchmark on a monthly basis.

3.2.2.3 Vibrating Wire Settlement Cells

A vibrating wire settlement system (Geokon Model 4650) was installed to measure the settlement of the mine backfill surface beneath the ash layer once the ash dump covers the mine backfill. The initial consolidation settlement takes place very quickly and this system made it possible to measure the settlement beneath the ash while the surface area is not available due to the ashing operation.

It will be possible to calculate the settlement in the ash layer after final leveling by using a benchmark placed directly over the cell on the frontstack surface. If the relatively small settlement of the ash layer can be accurately predicted, only survey techniques on the existing surface will be required in the future to calculate the larger differential settlements of the much less uniform backfill material.

Figure 3-4 shows a schematic diagram of a vibrating wire settlement system as well as cross section AA of Figure 3-3.

Three trenches were dug to insure that the cells are under positive pressure from the readout terminal. The cells were placed at approximately one meter under the level of the mine backfill surface. As the total mine backfill settles this will have no influence on the data but will insure instrument operation and accuracy over a longer period.

The cells were connected with flexible tubes and electrical cable to a readout terminal north of the ash dump. This terminal includes an automatic data collector or logger, which take readings every 15 minutes.

The three settlement cells were placed next to 2 x 2 x 0,5 m concrete blocks in front of the advancing ash face. Three cells are used to get some idea of the precision of the measurements. The placement of the cells also throws light on the tension cracks experienced on the frontstack. With this placement of cells the differential movement under the front face of the frontstack can be measured, as the ash is stacked. The initial concept of placing the cell within the concrete block was discarded as possible differential settlement between the blocks and the surrounding area might have sheared the cables and tubes.

The concrete blocks placed next to the cells act as a backup system if the vibrating wire system fails. It will then be possible to drill down to these blocks and grout a rod into the slab. The rod can then be used for further measurements.

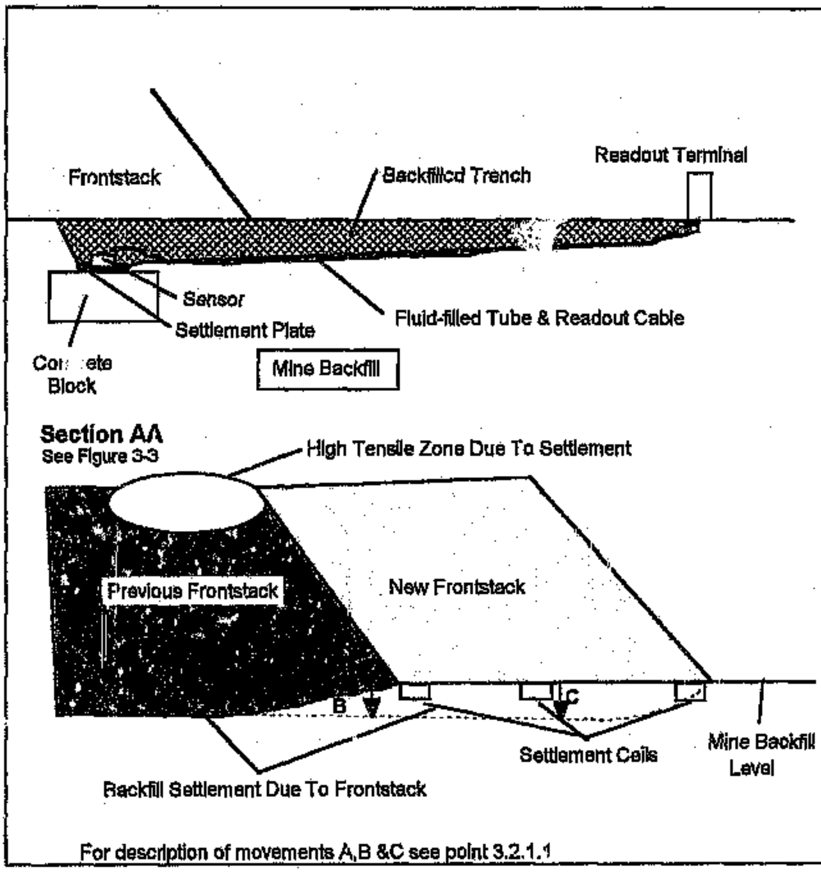


Figure 3-4: Schematic Cross Section Ash Dump Development And Instrumentation Position

3.2.2.4 Piezometers

A piezometer has been installed to measure the ground water level. This information is required to predict the percentage of collapse settlement that has already occurred. Due to the permeability of the mine backfill the water level in the piezometer will correspond with the local ground water level.

One piezometer was installed in a borehole drilled for that purpose next to the middle cell. The piezometer is also of the vibrating wire type and the data logger records the water levels. Installation consisted of the following steps:

- Measure borehole depth.
- Place piezometer in hole.
- Add ash to above piezometer tip to act as filter.
- Fill borehole to surface with material from drilling.

- Lay cable along trench of middle settlement cell to readout terminal.

3.2.3 Borehole Data

Rotary percussion drilling was used for the borehole. The borehole was flushed with air and various problems were encountered with substantial voids in the fill. Placing steel sleeves into the borehole reduced the problem of loss of air pressure into voids. The sleeves were extracted with the installation of the instrumentation.

The borehole produced no samples for the first 7 metres due to loss of air pressure in the sandy layer. After this a fairly homogeneous shale and sand material was encountered with signs of water at approximately 40 m. Due to continuous collapsing of the hole the piezometer was installed at a depth of 38.9 m.

3.2.4 Instrument Calibration

3.2.4.1 Calibration Procedures

All measurements in the borehole were done using a tape measure weighted at the bottom.

A professional land surveyor did a base survey of all benchmarks for Roshcon.

3.2.4.2 Initial Readings

Base Survey

A base survey was carried out on 27 June 1996. All the cell and piezometer depths were translated to levels using the installation information. This information will be regarded as the base information from where the monitoring program will depart.

3.3 OBSERVATIONS

3.3.1 Recorded Data

The settlement cells provided excellent data regarding the speed at which consolidation settlement occurs and have confirmed the predicted ash movement.

The surface benchmarks must still be installed due to geometric and operational limitations. These benchmarks will in the future provide information on differential settlement within the backfill.

The data shown in Figure 3-5 shows monitored settlement up to November 1997.

- During the loading with the first frontstack consolidation settlement of 800 mm was recorded for SC2. SC1, where the fill has been previously influenced by the ash dump, settled 650 mm and SC3 on the toe settled 150 mm.

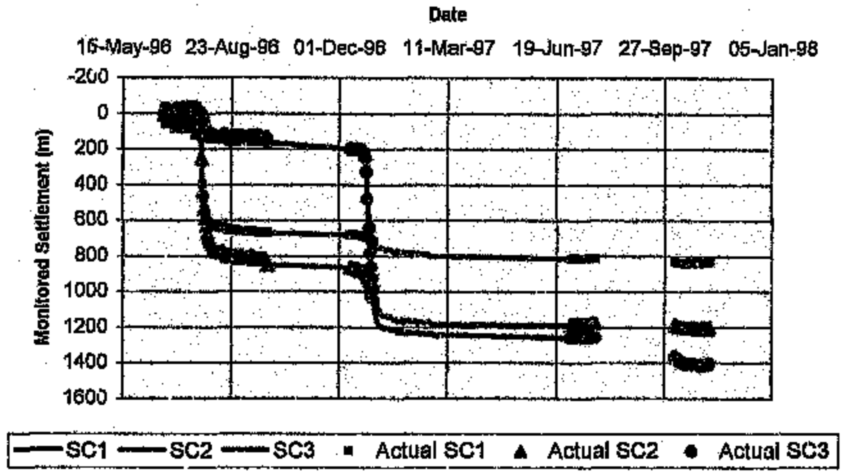


Figure 3-5: Monitored Settlement Vibrating Wire Cells

- Fairly consistent creep occurred adding a further 80 mm of settlement before the consolidation settlement with the second frontstack occurred.
- The continuous lines on the graph are modeled settlement as problems with the data logger lead to the loss of some data.
- SC3 recorded more than a meter of settlement when loaded with the second frontstack.
- Under subsequent frontstack loads SC3 has settled a total of 1400mm.
- Limited fluctuations in the water table were recorded.
- The small fluctuations in the data initiali, not visible on this graph are thought to be temperature linked. These fluctuations reduced after the ash was placed.

4 INTERPRETATION OF SETTLEMENT DATA

4.1 CONSOLIDATION SETTLEMENT

4.1.1 Monitored Settlement

4.1.1.1 Matla Extensometers

The consolidation settlement of the various layers of mine backfill, as defined by the different benchmarks, with depth and increase in stress, due to increase in wall height, is illustrated in Figure 4-1. For example layer L1 is the layer of mine backfill defined by benchmarks E1/1, E2/1 and E3/1 at 1m beneath the surface level and E1/2, E?'2 and E3/1 at an approximate depth of 5.1 m.

Included in these results is the creep settlement that has occurred during the time between surveys. Due to the intervals and loading that took place between surveys it is not possible to calculate the creep settlements between loadings.

Only the average data of the two benchmarks with the best correlation, as illustrated in Figures 2-5 to 2-8 were used in Figure 4-1 to 4-3.

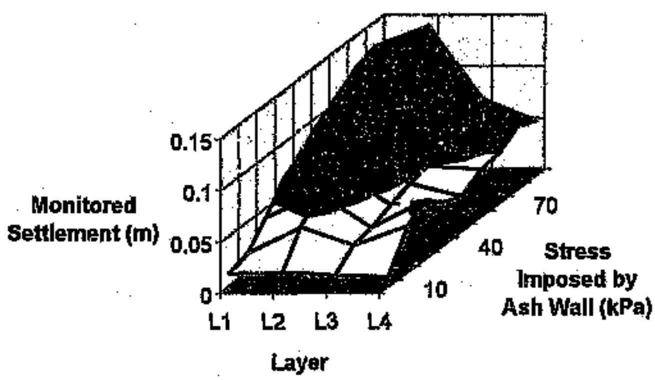


Figure 4-1: Settlement Of Various Layers With Increasing Stress

The strain versus stress of the different layers of mine backfill as defined by the different benchmarks is illustrated in Figure 4-2.

An increase in stiffness with depth and increase in stress can be seen from these Figures. Initially at low stress this was not as apparent. This is possibly due to influence factors not being correct for low stress conditions due to slope of the site. For Figure 4-3 an apparent elastic modulus was calculated using the using the simple relationship.

$E = \delta\sigma/\delta\varepsilon$
Where:
$\delta\sigma$ was the applied overburden stress for an ash density of 1 300 kg/m3 and

$\delta\varepsilon = \delta z/z$

Where:

δz = total settlement up that stage
z = thickness of layer

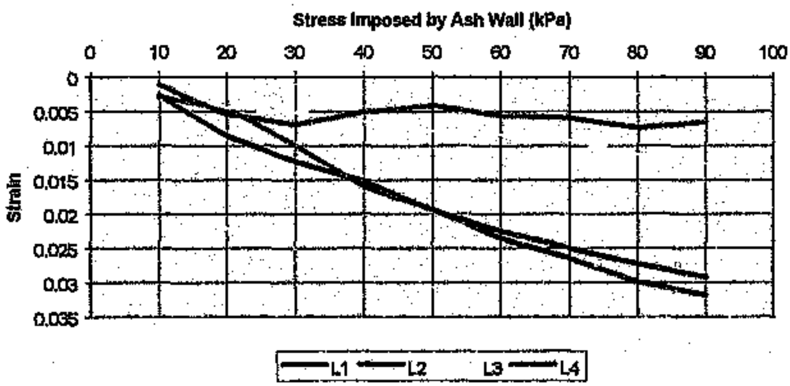


Figure 4-2: Strain versus Stress Relationship of Various Layers

Figure 4-3 therefore models the stiffness with depth and increase in stress. The increase in stiffness with depth is probably due to various reasons:

- The placing of the backfill with a dragline lead to dynamic compaction of the lower layers.

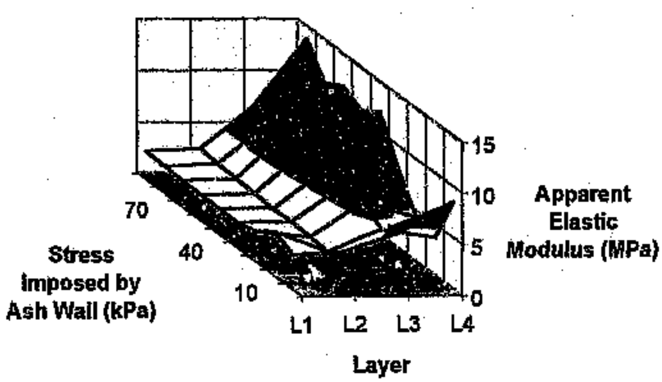


Figure 4-3: Apparent Elastic Modulus of Various Layers with Increasing Stress

- The overburden weight of the upper layers compacted the lower layers.

- The washing out of fines in the upper layers due to the continuous percolation of water through the ashing operation reduced the stiffness of the surface layer. As illustrated in Figure 4-2 the surface layer L1 was initially stiffer than the second layer L2, possibly due to compaction by mobile plant during the shaping of the mine backfill. Both Day[3] and Burford[6] recorded similar results. The layer reduced in stiffness due to the washing out of fines and only during the later stages of the test became stiffer again.
- As layer L4 was saturated during the monitoring period, it appears as if the saturation did not have an influence of the stiffness of the layer.

4.1.1.2 Heterogeneous Nature Of Backfill

The heterogeneous nature of the mine backfill has a significant influence on the accuracy of predicting mine backfill settlement. If one studies Figure 2-9 one can see that there is a significant spread in the recorded information. If one works in the range from 30 to 70 kPa where the most reliable data falls, the minimum recorded values vary on average by 56 % from the average recorded value. The maximum recorded values vary on average by 48 % of the average recorded value. These values are respectively 1.95 and 1.66 *standard deviations from the average.*

The area that this data was recorded in is approximately 33 600 m^2 in extent, and therefore a relative small area with similar placed material.

4.1.1.3 Matla Settlement Rods

Average apparent elastic moduli of all the settlement rods as well as the surface extensometers are illustrated on Figure 4-4. An initial decrease, followed by an increase in stiffness with increasing stress is again apparent in this Figure.

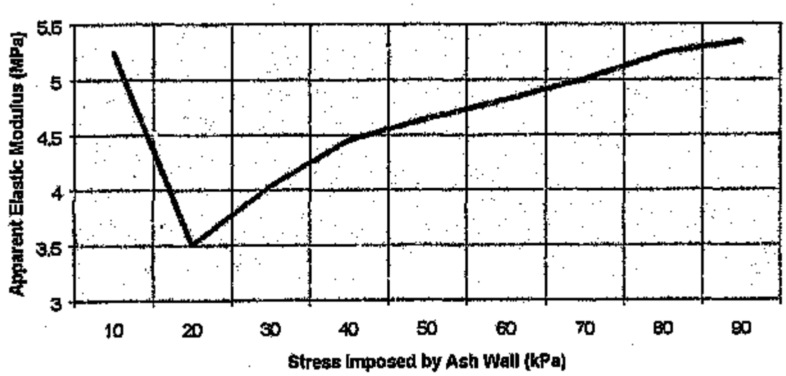


Figure 4-4: Apparent Elastic Modulus with Increasing Stress

The initial reduction in stiffness is due to the limited influence of the ash at low loads due to the slope of the test area. (As the area was initially not flat the ash started to load the rods from the southern end. The area was therefore not completely covered with ash and the influence factors used for the graph for low loads were therefore incorrect.)

This, combined with the washing out of fines from the surface layer reducing the stiffness of the layer creates the incorrect impression that there is a large reduction in stiffness over the total depth at low loads.

4.1.1.4 Lethabo Vibrating Wire Settlement Cells

As stated previously excellent data regarding the rate of consolidation and creep settlement were recorded at Lethabo. SC1 and 2 settled 650 and 800 mm under approximately 260 kPa and 210 kPa respectively. The lower settlement of SC1 under higher load is due the fill in this area being previously loaded. The settlement at SC3 of 150 mm should be similar to this previous settlement. Both SC1 and SC2 settled in excess of 600 mm in ten days.

From the 22 December 1996 to the 10 January 1997 SC3 settled a metre under the second loading of the frontstack (an increase in load of 200 kPa, approximately). The loading could not be calculated precisely as the nature of the operation make access to the top surface impossible during placement of the ash. The order of the load is however correct as the final level of the ash dump is known and the overstack of ash was estimated.

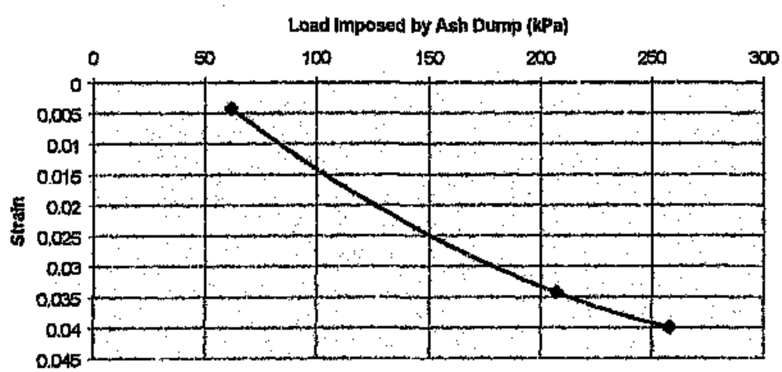


Figure 4-5: Strain Versus Stress Relationship Lethabo

Figure 4-5 illustrates the strain versus stress behavior of the three points calculated from the Lethabo data. The total backfill depth in this area is approximately 35 m. Total load estimated on the cells after the placement of the third frontstack is 260 kPa. Using these figures and a total settlement of 1.4 m an elastic modulus value of 6.5 MPa was calculated for maximum settlement. This value agrees with the 6 MPa proposed by Jones and Wagener[9].

4.1.2 Modeling of Consolidation Settlement

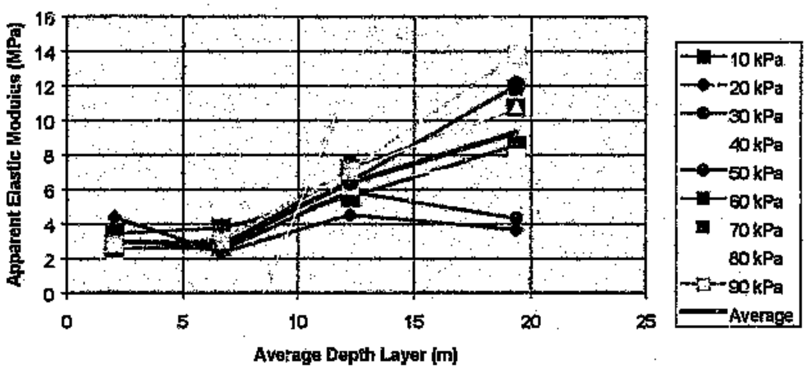


Figure 4-6: Apparent Elastic Modulus Increase with Depth for Various Loads

4.1.2.1 Increase in Stiffness with Depth

From the consolidation settlement data it is obvious that there is an increase of the stiffness of the backfill with depth and increase in load.

Figure 4.2, 4-3 and 4-6 illustrate the increase in stiffness with depth. Apparent elastic moduli at specific strain were calculated from the extensometer data to create these graphs with the simple relationship discussed under Matla Extensometers page 51.

Up to 10 m the material is very constant in stiffness. However the influence of the wet ashing on the surface is difficult to quantify. The layers beneath this 10 m layer increase in stiffness with depth approximately linearly with layer L3, 2.6 times stiffer and L4, 4.2 times stiffer. Hills and Denby observed no apparent relationship between strain and depth whereas; Jones and Wagener[13], Osboume and Mizon[16], Burford and Charles[20] as well as Burford[6] recorded an increase in stiffness with depth under consolidation and collapse settlement.

4.1.2.2 Increase in Stiffness with Load

Duncan and Chang[31] state that it is commonly found that soil behavior over a wide range of stresses is non-linear, inelastic and dependant upon the magnitude of the confining pressures. They show (based on experimental studies by Janbu [32]) that the variation in modulus value may be represented by

$$E = K.P_a.(\sigma_3/P_a)^n$$

Where

E = Elastic Modulus Value.

σ_3 = The minor principal stress or confining pressure.

K = Corresponding modulus value.

n = Exponent determining the rate of variation of E.

P_a = Atmospheric pressure expressed in the same units as E and σ_3

Applied to the settlement one-dimensionally, this relationship gives a very good correlation with the recorded data as observed. Figure 4-7 illustrates all surface data points as well as a modeled hyperbola (solid red line). The average of the recorded data are the red points. At high and low loads the correlation is not as good due to inaccurate influence factors at low loads and lack of data at high loads.

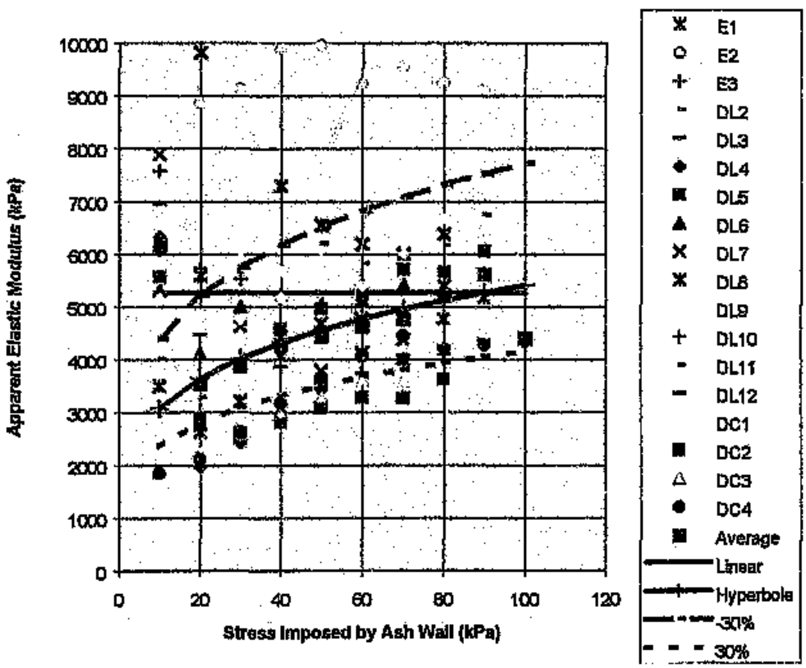


Figure 4-7: Model Stiffness with Increase Stress

As the strain versus stress graphs are not that far from a linear relationship, the least squares method was used to calculate from the available data an expected settlement for the maximum ash load (40 m of ash overburden or 510 kPa). A maximum settlement of 2.4 m was calculated, giving an Elastic Modulus (E) of 5.27 MPa. This graph also appears in Figure 4.7(blue line).

4.1.2.3 Heterogeneous Nature Of Backfill

To predict consolidation settlement to within 10 %, as Hills and Denby[22] claim possible for collapse and creep settlement, is not possible from the data recorded at Matla. The probability, using a normal distribution and all the surface point data, of two points settling within 10 % of the average is only about 27 %.

To predict the possible range of settlement for any specific load accurate to 70 % of the time, the variation in expected settlement according to the data is $\pm$ 30 %. On Figure 4-7 a range for 70 % accuracy has been added (red dotted lines).

Visually from the data it appears if there is a reduction in variation with increasing load. Because there are less data points at higher stresses this can, however, not be substantiated.

4.1.3 Comparison of Recorded Consolidation Settlement

All recorded data for mine backfill consolidation settlement under a surcharge is illustrated in Figure 4-8. Calculated settlement values using the hyperbolic approach with the same K and n values and only varying backfill depth, models the recorded data on Figure 4-8.

The recorded data from Matla was simplified for this graph by eliminating the highest and lowest values in the average settlement calculation. There is therefore a better apparent fit than for Figure 4-7.

The Burford[6] and Charles[20] recorded data should be compared to Matla calculated values as the backfill are similar in depth. As the method of placement is not described one cannot comment on the low correlation. The Charles[20] value however falls within 30 % of the Matla calculated graph.

The elastic parameters proposed by Luker and Wates[11] correlate well with the Matla data. There is however a significant difference between predicted values at higher stresses.

Good correlation with the data recorded at Lethabo by the author and Jones and Wagener[9] is obtained with both the linear model of 6 MPa proposed by Jones and Wagener and the graph based on Matla data (Lethabo calculated). Prediction to within 100 mm was made. At higher stresses the difference will however increase in magnitude. This information will become available when the backstack loads the cells at Lethabo.

4.2 COLLAPSE SETTLEMENT

4.2.1 Matla Test Wall

Due to limited changes in water table no information regarding collapse settlement was gathered at the test wall. Due to the proximity of the final cut the water level was governed by that in the final cut, which in turn is controlled to a reasonable constant level by a pump system.

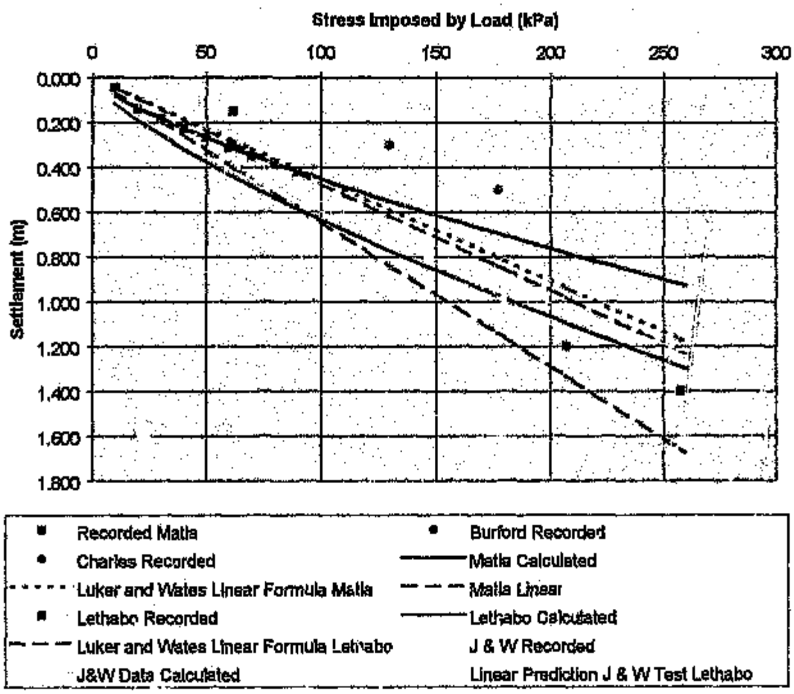


Figure 4-8: Comparison of Consolidation Settlement Data

4.2.2 Matla Extensometer MB1

Extensometer MB1 from the Wates and Wagner instrumentation provided some data on collapse settlement. Figure 4-9 illustrates the collapse percentage of the various layers as well as a percentage for the total depth saturated. The fill was saturated at the start of the test to a level of 1 549 m.a.m.s.l. The solid red graph marked MB1/1 –1/2b illustrates the collapse percentage in that layer assuming that the settlement only occurred above the old water table, which is most likely, whereas the broken red line assumes that collapse settlement occurred over the total depth of the layer.

From this graph it appears that the collapse percentage increases with depth which is possibly due to higher loading conditions. The apparent increase in stiffness appears to have no significant influence in reducing the collapse potential, which does not agree with results from Day[3]. The confidence in Day's results are however not high as the extent of saturation is unknown.

A total collapse of 3.8 % was recorded over the full depth.

Hills and Denby[22] also found no correlation between collapse and depth of backfill. Charles et al[15] four d no influence on collapse settlement of the age of the material.

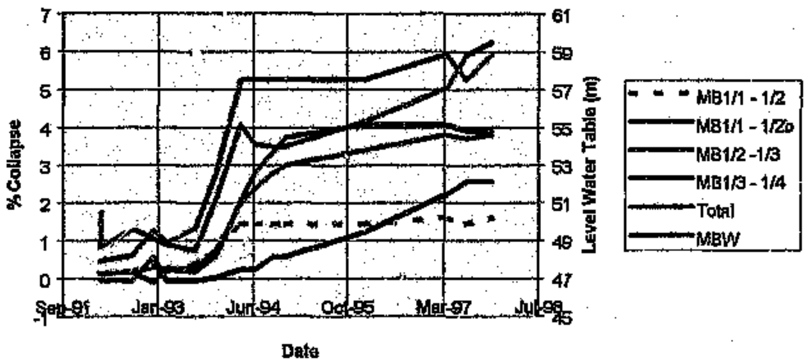


Figure 4-9: Collapse Settlement with Saturation

4.2.3 Comparison of Recorded Collapse Settlement

The significance of the total collapse recorded is that it correlates with the 4 % measured by Day[14] at Lethabo in a layer 5 m to 10 m deep which was likely to have been saturated.

Charles[10] recorded 2 % collapse settlement at Horsley and Hills and Denby[22] prediction range for uncompacted backfill is 0.75 % to 4 %, averaging 2.5 %.

Smyth-Osbourne and Mizon[18] recorded values between 2.4 % and 4.7 %.

From the data it appears that collapse settlement is a rapid process linked to the speed of rise of the water table. The collapse settlement continued at a steep rate in layer MB1/2 – 1/3 even after a period during which the water table stayed approximately constant from June 94 to October 95.

4.3 CREEP SETTLEMENT

4.3.1 Matla Extensometers

The survey of March 1997 mentioned in paragraph, 2.3.1.2 Settlement Rods provided interesting information regarding creep within the backfill over a period of 14 months.

A trend line in Figure 4-10 indicates a reduction in settlement due to creep for areas under higher stress. Not enough data is available at lower loadings to give support to the trend line. The trend might actually be due to the method of construction of the daywall. As the daywall is loaded in layers, areas under higher stress have been loaded longer and would have become stiffer. As creep reduces with time under constant loading the higher stressed, stiffer backfill creeps less. The reduction of void ratio at higher loads might also reduce the creep.

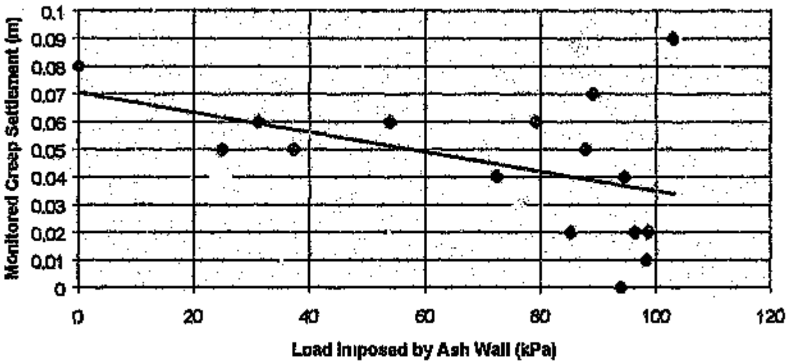


Figure 4-10: Creep Settlement of Test Wall

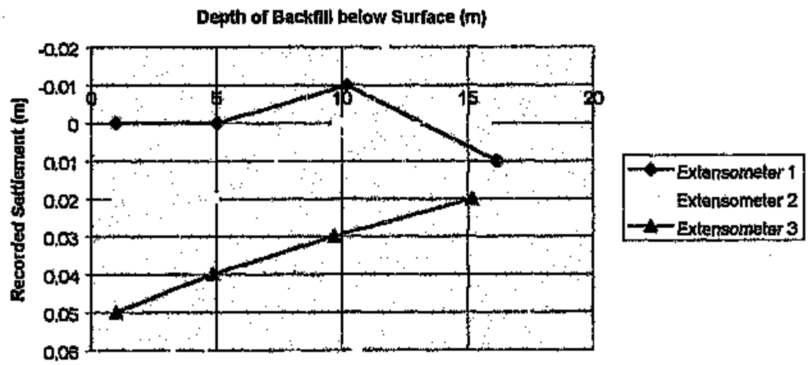


Figure 4-11: Recorded Creep Settlement With Depth

From Figure 4-11 it appears as if there is a reduction of creep settlement with depth.

An average creep rate parameter (α) of 0.25 was calculated using the expression on page 8, and assuming zero time as proposed by Sowers et al[33]. Hills and Denby[22] took their time zero as the time at which the half of the total height of the wall was completed. One must keep in mind that this α is not for uncompacted backfill as this backfill had been surcharged on the average with approximately 73 kPa. The spread of the values is however from approximately 0 to 0.4. (See Figure 4.12)

These values are in the lower range of values recorded by Hills and Denby[22] comparing to values of backfill placed with scrapers and compacted with vibrating rollers in 300 mm layers. Their values are however for creep under the self weight of the backfill and not after placement of a surcharge.

A creep rate parameter cannot be calculated for the 30 mm creep recorded before saturation at Extensometer MB1 as the settlement for t1 is not known.

4.3.1.1 Lethabo Vibrating Wire Settlement Cells

Much lower creep settlement rate parameters were recorded for Lethabo than for Matla. Calculated values varied from 0.05 for SC1and 0.07 for SC3 to 0.09 for SC2.

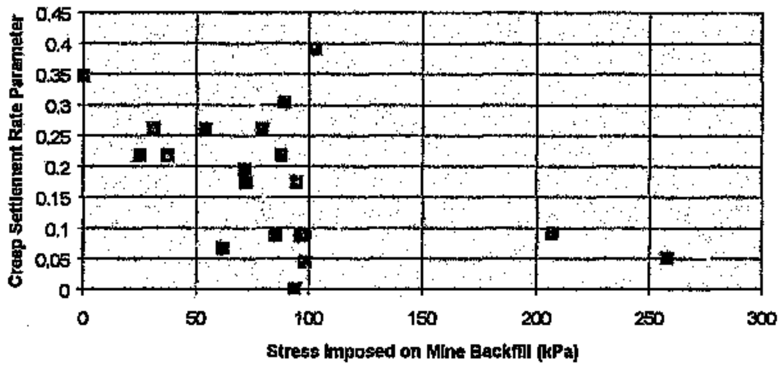


Figure 4-12: Creep Settlement Parameter versus Stress

Data recorded at Lethabo is also shown in Figure 4-12. The values are the two points under high stresses of 210 kPa and 260 kPa and a value at ?0 kPa. According to Charles et al[10] pre loading reduces creep settlement and Figure 4-12 may support this if one considers that the Lethabo data falls very near to the average predicted graphs.

5 CONCLUSION

5.1 SETTLEMENT PREDICTION

5.1.1 Consolidation Settlement

For the two specific cases considered consolidation settlement, for open cast mine backfill under surcharge can be estimated linearly or hyperbolically with acceptable results for stresses up to 260 kPa. Consolidation settlement is a rapid process and occurs for all practical purposes as the stress increases.

Young's Modulus varies between 5 and 6.5 MPa for the linear estimation.

To model the apparent increase in stiffness with increase in imposed stress due to surcharging the following relationship is proposed;

$$E = K.P_a.(\sigma/P_a)^n$$

Where

σ = Change in stress due to surcharge.

K = 0.056

n = 0.25

The heterogeneous nature of the backfill material requires a ± 30 % variation in predicted settlement for the Matla site to obtain a prediction with a 70 % accuracy.

5.1.2 Collapse Settlement

The Matla test recorded average collapse settlements of 3.8 %. 4 % collapse upon saturation would be an adequate figure for design purposes. The influence of the heterogeneous nature of the material on collapse settlement could not be measured during the tests.

5.1.3 Creep Settlement

Creep rate parameters for surcharged backfill of as low as 0.05 were recorded. For design purposes a creep rate parameter of 0.25 is proposed for low (100 kPa) loads. There appears to be an inverse relationship between creep rate parameter and stress imposed by surcharge. Creep obviously continues for a long time period.

5.1.4 Total Settlement Prediction

A combination of the three mechanisms and values discussed above should be adequate for typical building development to be expected on mine backfill.

The reduction of collapse settlement due to surcharge requires further investigation as well as the effect of previous collapse settlement on subsequent consolidation settlement under surcharge. Continued monitoring of the two sites will help to answer these questions.

5.2 INFLUENCE OF SETTLEMENT ON ASH DAM AND DUMP CONSTRUCTION AND BACKFILL DEVELOPMENT

Both Matla's wet ash dam and Lethabo's dry ash dump can be constructed on mine backfill with limited problems. The reason for this is the resilient nature of ash. Both ashes have cementing properties leading to very high strengths.

5.2.1 Matla Ash Dam

The Matla ash seems to heal itself from cracks forming due to differential movement. No significant cracks were recorded.

As the data indicates a possible reduction in differential movement, an increase in stiffness of the backfill and a reduction in creep settlement with higher stresses, the probability of stability problems due to backfill settlement should reduce with increasing ash dam height.

5.2.2 Lethabo Ash Dump

Cracking due to differential movement as the ash is placed will continue to create problems with drainage and may lead to stability problems if there are changes in ash properties. Continued monitoring of cracks and sealing will be required. Due to the fact that the water table will only re-establish itself after mining operations stop, development of the area should be delayed until after saturation of the backfill. After saturation has occurred, development of the ash dump area should, however, be no problem due to the high surcharge.

5.2.3 General Development

Important points to consider when development of open cast mine backfill is planned are:

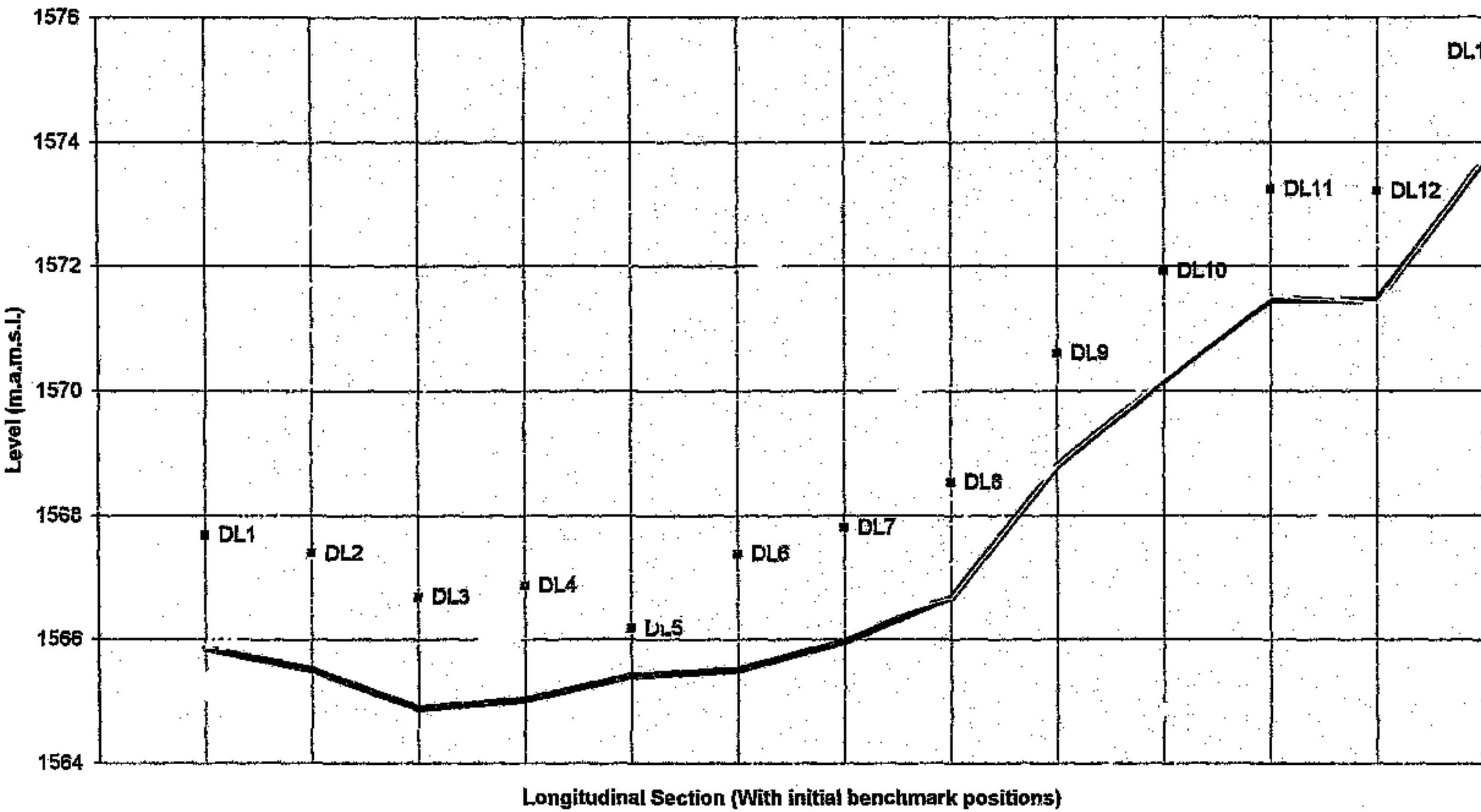
- Present water table level and possible future fluctuation in the water table.
- Small scale tests on the surface may be misleading.
- The size of the footprint and therefore the depth of the influence of a surcharge or foundation is very important as the typical backfill operation results in a loose layer at 5 to 10 m deep from the surface with large settlement potential.
- Creep settlement continues over a long period.

- Surcharging is a very good method for ground improvement. If areas of development are identified before mining, an attempt should be made to surcharge these areas as part of normal mining operations.
- Large differential movements will occur even in materials placed in a similar manner.

6 APPENDIX A

6.1 DRAWINGS

Schematic Cross Section Wall Growth



7 APPENDIX B

7.1 DETAILED DRILLING LOGS

7.1.1 E1

0 to 12 m	Shale, fairly homogeneous not too many drilling problems.
12 to 13 m	Void, loss of air pressure.
13 to 16 m	Sandstone.
16 to 24 m	Shale, fairly homogeneous not too many drilling problems.
19 m	Water table.
24 to 28 m	Consistent shale material. Bedrock.

7.1.2 E2

0 to 6 m	Sandstone and shale mixture.
7 to 14 m	Fairly homogenous sandstone with limited voids. This is possibly due to sandstone boulders in the area that may lead to failure of the borehole with any lateral movement.
14 to 25 m	Shale, fairly homogeneous, drilling problems due to air losses.
19 m	Water table.

7.1.3 E3

1 to 3 m	Shale material, air losses through surface.
3 to 7 m	Shale material, air loss problems.
7 to 8 m	Sandstone.
9 to 22 m	Shale material, air loss problems.
12 m	Water table.
22 to 23 m	Sandstone.
23 to 25 m	Shale, hole continues collapsing to 24.5 m.

7.1.4 TP

Borehole TP was the most interesting borehole from an ash infiltration point of view. The borehole was drilled in line with the dragline spoil heaps approximately 20 m from an area that substantial ash infiltration into the mine backfill took place, during May 1993. This borehole created the least problems, probably due to the infiltration of the ash and the filling of the voids.

0 to 5 m	Shale material.
5 to 6 m	Shale material with low ash percentage.
6 to 7 m	70 % Ash and shale.
7 to 10 m	Ash.
10 to 11 m	Shale and low percentage ash.
11 to 13 m	Shale material.
13 to 14 m	Shale material and ash 50/50 mixture.
14 to 17 m	Clay material. Use scraper to remove.
17 to 24 m	Shale material.
19 m	Water table.

7.1.5 ITP

1 to 2 m	Sandstone and shale.
2 to 24 m	Shale material limited air losses.
19 m	Water table.

8 APPENDIX C

8.1 INSTALLATION DEPTHS

Code	Description	m Beneath Ground Level
ITP	Inside toe piezometer	
	Borehole	24.0
	Piezometer filter	22.0
	Water table (8/4/94)	19.8
TP	Toe piezometer	
	Borehole	24.0
	Piezometer filter	19.0
	Water table (9/4/94)	17.8
E1	Extensometer 1	
	Borehole	26.0
EP1	Piezometer filter	22.0
	Water table (14/4/94)	15.5
	Anchor depths	
E1/1	1	1.0
E1/2	2	5.2
E1/3	3	10.5
E1/4	4	16.5
E1/5	5	25.0
E2	Extensometer 2	
	Borehole	25.0
EP2	Piezometer filter	18.0
	Water table (15/4/94)	15.6
	Anchor depths	
E2/1	1	1.0
E2/2	2	5.0
E2/3	3	10.0
E2/4	4	16.0
E2/5	5	24.0
E3	Extensometer 3	
	Borehole	25.5
EP3	Piezometer filter	18.0
	Water table (22/4/94)	15.6
	Anchor depths	
E3/1	1	1.0
E3/2	2	5.0
E3/3	3	10.0
E3/4	4	15.5
E3/5	5	23.0

9 APPENDIX D

9.1 BASE SURVEY

Code	Description	Level Benchmark or Water + 1 500.000 m	Concrete Level + 1 500.000 m	Audit Survey Ground Level 28/7/94 + 1 500.000 m
IT1A	Settlement Rod	67.632	65.90	65.85
IT1B	Settlement Rod	66.376	64.74	64.71
IT2A	Settlement Rod	68.106	66.34	66.32
IT2B	Settlement Rod	66.613	64.86	64.83
IT3A	Settlement Rod	68.321	66.52	66.49
IT3B	Settlement Rod	66.711	64.85	64.81
ITP	Piezometer	49.0	NA	
DL1	Settlement Rod	67.680	65.85	65.75
DL2	Settlement Rod	67.392	65.51	65.46
DL3	Settlement Rod	66.684	64.89	64.69
DL4	Settlement Rod	66.871	65.03	64.79
DL5	Settlement Rod	67.272	65.41	65.27
DL6	Settlement Rod	67.381	65.57	65.48
DL7	Settlement Rod	67.799	65.97	65.94
DL8	Settlement Rod	68.523	66.69	66.67
DL9	Settlement Rod	70.592	68.79	66.77
DL10	Settlement Rod	71.917	70.17	70.14
DL11	Settlement Rod	73.248	71.46	71.43
DL12	Settlement Rod	73.213	71.45	71.42
DL13	Settlement Rod	75.448	73.71	73.67
DC1	Settlement Rod	66.877	65.05	64.94
DC2	Settlement Rod	66.678	64.79	64.56
DC3	Settlement Rod	67.063	65.19	65.02
DC4	Settlement Rod	67.179	65.34	65.16
T1A	Settlement Rod	70.703	68.97	68.96
T1B	Surface Benchmark	70.746	70.11	70.11
T2A	Settlement Rod	68.083	66.31	66.30
T2B	Surface Benchmark	67.885	67.30	67.29
T3A	Settlement Rod	65.376	63.54	63.51
T3B	Surface Benchmark	64.791	63.16	64.16
TP	Piezometer To top of casing Water level	 66.832 49.1	NA	
EP1	Piezometer	48.8	NA	
E1/1	Extensometer	67.614	64.30	64.19
E1/2	Extensometer	67.615	60.10	60.01
E1/3	Extensometer	67.616	54.80	54.75
E1/4	Extensometer	67.613	48.80	48.78
E1/5	Extensometer	67.618	40.30	40.27
EP2	Piezometer	49.4	NA	
E2/1	Extensometer	67.614	64.30	64.26
E2/2	Extensometer	67.629	60.30	60.26
E2/3	Extensometer	67.637	55.30	55.27
E2/4	Extensometer	67.629	49.30	49.27
E2/5	Extensometer	67.627	41.30	41.28

Code	Description	Level Benchmark or Water + 1 500.000 m	,oncrete Level ، 1 500.000 m	Audit Survey Ground Level 28/7/94 + 1 500.000 m
EP3	Piezometer	50.2	NA	
E3/1	Extensometer	68.176	65.10	65.07
E3/2	Extensometer	68.197	61.10	61.08
E3/3	Extensometer	68.196	56.10	56.07
E3/4	Extensometer	68.192	50.60	50.59
E3/5	Extensometer	68.193	43.10	43.06
R	Reference Benchmark	72.623	71.93	71.93

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