Design and construction of the Kowyn’s Pass rockfall shelter

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Synopsis

The site of the Kowyn’s Pass shelter is situated on the road traversing the escarpment between Graskop and Bosbokrand in the north-eastern Transvaal. The area was subject to a major fall of rock in 1973. Further falls, however, continued after the suspect area was anchored. The Transvaal Provincial Roads Authority decided towards the end of 1977 to construct a reinforced concrete rockfall shelter as a permanent and fail-safe protective measure. The design of the structure, of which construction was completed in August 1980, is discussed in the paper. A brief review of the state of the art of designing rockfall shelters against impactive loading is also presented.

Introduction

Rock slope problems can be divided broadly into two categories. The first comprises situations in which large masses of rock tend to be unstable by movement on relatively deeply seated readily identifiable planes or surfaces of weakness. By definition the failure surfaces are identifiable and of significant extent. In the second category can be placed all situations in which individual rocks or masses of rock on or close to the slopeing surface and comparatively small in relation to the slope height are potentially unstable. Failure planes, as such, are not a consideration in these problems.

To identify with reasonable certainty all the potentially unstable rocks on an extensive slope face in the second category of problems is difficult, if at all possible. To select in addition a suitable protective measure against the individual boulders is a somewhat intractable exercise. In principle one of two approaches may be adopted: either to anchor each boulder or to provide a barrier or protective structure at the endangered site. The usual dilemma is that all the potentially unstable rocks can either not be identified or cannot all be anchored, whilst the scope of the problem may also not justify the construction of an effective protective structure at the endangered site.

At the site on Kowyn’s Pass, potentially unstable boulders and imminently falling rocks occur on a thickly vegetated slope face which is approximately 133 m high and 170 m long. Aerial views of the face on which the path of ravelling rock debris and an end-on profile can be seen are shown in photos 1 and 2. The design considerations of a reinforced concrete rockfall shelter, which has been constructed as a fail-safe protective measure, are presented in principle in this paper. A brief review of the present state of the art of the selection and design of protective measures in such situations is also presented. The work of the Japanese Railway Authority, which was published in 1979 subsequent to completion of the design of the shelter at Kowyn’s Pass, receives the main attention in this regard.

Regional geology and profile of slope face

Kowyn’s Pass is situated on the road traversing the escarpment between Graskop and Bosbokrand in the north-eastern Transvaal. Basement archaen granites occur in the slope face at the elevation of the road. These are unconformably overlain by arkoses, lavas, siltstones and quartzites of the Dominion Reef Series. The granites vary from a black mottled white, highly weathered, medium to coarse grained, highly weathered, medium to coarse grained, and quartzites of the Dominion Reef Series. The granites vary from a black mottled white, highly weathered, medium to coarse grained,

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Photo 1: Aerial view of rockface and protective shelter

Photo 2: End-on view of rockface
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Friable, soft rock to a white mottled black, slightly weathered, medium to coarse grained, hard rock. The overlying arkose consists of a horizontally bedded, whitish brown, slightly weathered, fine to medium grained, hard rock. The bedding is medium to thick and the jointing medium to widely spaced. The lavas above the arkose are in general a grey, slightly weathered, very fine grained, hard rock.

In places less competent lava horizons are more deeply weathered and have as a result given rise to distinct breaks in the slope. The cap of the hill consists of an agglomeritic lava. The major structural trends are vertical and strike nearly perpendicular to the rock face. Occasional lineations strike along the slope at dip angles nearly parallel to the face, particularly in the granites at road level. The lineations tend to be separated by up to 25 mm in width for a distance of approximately 2 to 4 m into the face.

Identification of unstable zones
The general geological information was augmented by site specific investigations. This involved inspections of the top 50 m of the face and reconnaissances on foot to a height of about 30 m above the roadway. The four zones of imminently unstable boulders which were identified are indicated in Figs 2 and 3. Zone 1 forms an escarpment face approximately 105 m above the roadway. It is 10 m high, approximately 50 m long and somewhat below the crest of the natural slope. It consists of near vertically jointed and extensively weathered lavas and agglomeritic lavas. The joints in this zone delineate blocks ranging from a few cubic metres to approximately 125 m$^3$ in volume. The maximum size blocks measure typically 3.5 m into the face by 5 m on strike by 7 m high. There are approximately 20 of these blocks which in time could become unstable and cause considerable damage to the lower slopes and the roadway.

Zone 2 is situated directly below zone 1. It varies approximately from 65 to 85 m above the roadway and comprises a thickly vegetated surface on which the boulder rubble from the overlying slopes as well as from the in situ arkose accumulates. The volume of these boulders varies from 3 to 6 m$^3$. Typical maximum dimensions are 1 m thick by 2 m along strike by 3 m high. Periodic falls and slides of rock, involving a range of sizes, occur from this zone.

Zone 3 occurs beneath zone 2 from approximately 40 to 60 m above the road elevation. It comprises a competent layer of arkose on top of a very much weaker horizon of the same rock. The bedding and jointing in the zone are spaced from 0.2 to 2 m. The zone is highly fractured, as a result of which it is unlikely that boulders exceeding 3 to 4 m$^3$ in volume will be produced from it. A major fall of rock occurred from the granite...
below zone 3 in 1973.

Zone 4 occurs somewhat lower and to the east of zone 3, approximately 30 to 40 m above the road surface. It comprises a highly jointed rock nose and contains an interface between the granite and the arkose. This zone is likely to give rise to considerable volumes of boulder scree. There is evidence of fresh cracks and of water flows on the joints, which are widely open and deeply invaded by roots. The average joint spacing and slab thickness of boulders in this zone varies from 1 to 2 m.

Selection of protective shelter as remedial measure

The danger and the necessary action associated with falls of rock from slopes can be classified empirically on a six point scale as shown in Table 1. The particular class which applies to a given situation can be assessed in detail on a qualitative basis as enumerated by the Japanese Railway Authority. Factors such as the environmental conditions of the slope, the stability and support of potentially unstable rocks, the size and shape of such rocks, the route of fall, the kinetic energy of the falling rocks, the nature of the potential damage, traffic density, visibility, etc., are taken into consideration for this purpose.

The slope at Kowyn's Pass was recognized as potentially unstable and subject to periodic falls of rock. After the 1973 fall a contract was let for the anchoring of the unstable granites. As further falls occurred, stabilizing measures were again investigated. The difficulty under the circumstances was to find a minimum remedial measure which would provide a permanently safe solution. Remedial measures can in general be of a preventive, protective or a defective nature. In selecting a remedial measure, its particular function, durability, maintenance, complexity of installation, reliability and economy should be considered. The multiplicity of all the factors involved is an indication of the complexity associated with the selection of the optimum solution.

Table 1: Classification of rock slopes in terms of remedial action against rock falls

<table>
<thead>
<tr>
<th>Class</th>
<th>Condition of slope</th>
<th>Action required</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Evidently stable</td>
<td>None</td>
</tr>
<tr>
<td>2</td>
<td>Stable provided conditions remain unchanged</td>
<td>Periodic inspections</td>
</tr>
<tr>
<td>3</td>
<td>Marginally stable. Falls of rock may occur</td>
<td>Regular inspections and formal monitoring</td>
</tr>
<tr>
<td>4</td>
<td>Marginally unstable. Occasional falls of rock occur</td>
<td>Installation of remedial measures investigated</td>
</tr>
<tr>
<td>5</td>
<td>Evidently unstable. Frequent falls of rock occur</td>
<td>Remedial measures promptly installed</td>
</tr>
<tr>
<td>6</td>
<td>Highly unstable. Regular falls of rock occur</td>
<td>Temporary remedial measures installed immediately followed promptly by installation of permanent remedial measures</td>
</tr>
</tbody>
</table>

The situation at Kowyn's Pass was intuitively evaluated along the lines indicated by the Japanese Railway Authority. Prior to the construction of the shelter an attempt was made to anchor potentially unstable boulders and masses of rock. In view of the relative inadequacy of this attempt, the Transvaal Provincial Roads Authority proposed a permanent and fail-safe protective structure and requested that the feasibility of constructing it be investigated. A preliminary design of the structure indicated that the proposal was technically feasible and, although marginally more expensive, was selected because of its durability, reliability, comparative ease of construction and maintenance-free characteristics.

Ritchie of the Washington State Highway Commission argues in favour of the installation of remedial measures at the endangered site. In view of the continued falls of rock and the apparent inability in many instances to prevent rocks from landing on highways, he is of the opinion that attempts to restrain every rock from falling are erroneous.

The Japanese Railway Authority is at direct variance with both Ritchie and our own experience with regard to the effectiveness of remedial measures. This authority claims that the measures in combating falls of rock are in order of reducing efficiency as follows: those implemented at the origin, the intermediate stage and at the endangered site. It is not clear whether this opinion is expressed with regard to structural or economic efficiency.

A protective structural gallery was under construction in 1980 across the A4 Hotwell Road in the County of Avon, Bristol. Alternative less-costly measures which would have involved extensive bolting and meshing were considered. The more expensive gallery was chosen because the required temporary remedial work was far and away easier to define and delimit. It was further considered that the gallery would have added to the scenic beauty of the Avon Gorge.

The installation of the bolting and meshing alternative would have required continued closure of the road throughout the British summer of 1979 and would as a result have given rise to hidden secondary costs in terms of transportation, disruption and overloading of alternative routes. The bolting and meshing alternative would also have been objectionable environmentally as had already been experienced in respect of other remedial installations elsewhere in the gorge. A further consideration against bolting and meshing was that future maintenance work on the unstable face would have been impossible without closing the road.

It is evident from these examples that the decision on the remedial action to be taken is a very complex one which involves a wide range of considerations. It would however appear, as experienced at Kowyn's Pass, that, given enough thought and perhaps some trial attempts, an appropriate solution can be found.

Description of shelter

General aspects

A plan view, longitudinal elevation and typical cross-section of the structure are shown in Fig 4. End and side views of the structure are shown in Photos 3 and 4. The road surface is super-elevated through the curved parts of the shelter and is inclined at a constant grade of 5 per cent through the structure. The super-elevation of the road surface is reflected in the deck slab. The cross-sectional dimensions are in accordance with TPA Specifications for Underpasses (Local Road; 2-lane width). The structure is 136 m long. The shelter comprises the following basic elements: horizontally anchored mass concrete foundations, reinforced
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between the deck slab and the wall. It further obviates the development of lateral earth pressures on the rear wall, in the absence of which the stiffness of the wall could be reduced. The rear wall was further ensured of maximum flexibility by arresting the side sway of the structure by means of horizontal anchors tying the deck slab to the rock face. The flexibility of the portal was further increased by providing a hinge at the joint between the deck slab and the outside columns.

The impact of a boulder with the earthfill on top of the shelter represents the collision of two bodies. The energy absorbed by the structure, comprising the earthfill and the reinforced concrete portal,
was minimized by increasing the mass of the structure and by ensuring as far as possible that the impact was plastic. The fill was consequently placed at the maximum slope angle and to a nominal degree of compaction only.

**Design considerations**

**Definition and properties of design boulder**

The design boulder is that boulder subject to the impact of which the shelter is designed, and of which the impactive effect on the shelter will not be exceeded.

Owing to the sloping surface of the earthfill, falling boulders will not accumulate directly on top of the structure. The cross-sectional shape of the structure and the earthfill will maintain a reasonably constant configuration. The stiffness and response of the structural system will therefore not change with time. Since the times of arrival of incident rocks from an extensive slope failure will in all probability differ, the impactive effect of only one boulder was considered at any one instant of time. With the information and knowledge available at the time it was not possible to evaluate rigorously all the properties of the design boulder, the required parameters with regard to which are as follows:

1. **Size and mass**
2. **Shape, that is, spherical, cubic or tabular**
3. **Three-dimensional translational velocity**
4. **Three-dimensional angular velocity, or spinning**
5. **Length of free falling trajectory**
6. **Shape of trajectory in free fall and successive contact with slope on bouncing**
7. **Angle of incidence on impact**
8. **Strength of the boulder subject to impactive bounces**

- Based on the joint spacing observed in the different zones on the rock slope, the size of the design boulder was taken as 2 m cube or 8 m³ in volume. It was only from the zone along the top escarpment face that boulders of approximately 125 m³ were expected to be dislodged. These boulders were readily identifiable and were limited in number to approximately twenty. If they were dislodged and fell down the slope without breaking up, they would not bounce over large distances, but would tend to roll and slide along the surface and eventually come to rest on the earthfill as a static load. All the effects of the large boulders would be equivalent to the impactive effect of the design boulder. The factor of safety would be calculated for a temporary reduction in the static load.

The stiffness and response of the structural system with the earthfill can be shown from the above conditions that the energy imparted to the earthfill is given by the expression:

\[ R_e = \frac{m_2 v_2^2}{m_1 v_1^2} \]

- The energy imparted to the fused mass is given by the expression:

\[ E_p = \frac{4m_1}{(m_1 + m_2)^2} \]

Consider next the impact to be plastic, that is, the bodies to fuse together upon impact and thereafter to move along as one combined mass. The conservation of momentum requires the following condition to be satisfied:

\[ m_1 v_1 + m_2 v_2 = (m_1 + m_2) v_2 \]

The ratio of energy absorbed by the structure which resists the movement of the fused mass before impact can be expressed as follows for \( v_2 = 0 \):

\[ R_p = \frac{m_1}{m_2 + m_1} \]

The variations in the energy absorbed ratios for elastic and plastic impacts against mass ratio are shown plotted in Fig 5. The following observations are evident:

1. When the striking mass, \( m_1 \), is large in comparison to the struck mass, \( m_2 \), the energy absorbed by the supporting structure is essentially the same for both types of collision.
2. When the striking mass is small compared to the struck mass, the

**Dissipation of impactive energy**

Consider two bodies of mass \( m_1 \) and \( m_2 \), velocity before impact of \( v_1 \) and \( v_2 \), and velocity after impact of \( v'_1 \) and \( v'_2 \), respectively. Further, consider the impact to be elastic, that is, for the bodies to separate after impact. For the conservation of momentum and energy the following respective conditions should be satisfied:

\[ m_1 v_1 + m_2 v_2 = m_1 v'_1 + m_2 v'_2 \]
\[ m_1 v'_1^2 + m_2 v'_2^2 = m_1 v_1^2 + m_2 v_2^2 \]

Let the subscript 2 denote the struck mass and assume this mass to be stationary before impact, that is, \( v_2 = 0 \). The energy imparted to the struck mass is given by the expression:

\[ E'_2 = 0.5m_2 v_2^2 \]

This amount of energy has to be absorbed by the structure resisting the movement of the struck mass. The ratio of energy absorbed to the energy of the striking mass before impact is given by:

\[ R'_e = \frac{m_2 v_2^2}{m_1 v_1^2} \]

It can be shown from the above conditions that

\[ R_e = \frac{4m_1}{(m_1 + m_2)^2} \]

Consider next the impact to be plastic, that is, the bodies to fuse together upon impact and thereafter to move along as one combined mass. The conservation of momentum requires the following condition to be satisfied:

\[ m_1 v_1 + m_2 v_2 = (m_1 + m_2) v_2 \]

The energy imparted to the fused mass is given by the expression:

\[ E'_p = 0.5(m_1 + m_2) v_2^2 \]

The ratio of energy absorbed by the structure which resists the movement of the fused mass before impact can be expressed as follows for \( v_2 = 0 \):

\[ R'_p = \frac{m_1}{m_2 + m_1} \]

The variations in the energy absorbed ratios for elastic and plastic impacts against mass ratio are shown plotted in Fig 5. The following observations are evident:

1. When the striking mass, \( m_1 \), is large in comparison to the struck mass, \( m_2 \), the energy absorbed by the supporting structure is essentially the same for both types of collision.
2. When the striking mass is small compared to the struck mass, the

**Fig 5: Energy absorbed ratio for elastic and plastic impacts**
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energy absorbed by the supporting structure is considerably less for the plastic impact than for the elastic impact.

It follows from these observations that the energy absorbed by the supporting structure can be minimized by increasing the volume of the struck mass and by ensuring as far as is possible that the impact is plastic. Based on this principle it was decided to cover the shelter with the maximum practical depth of soil in the least appropriate state of compaction relative to long term consolidation settlement and side slope stability. The practical implications in this regard can be illustrated as follows. Upon impact, the volume of fill beneath the incident boulder will be accelerated. It may be assumed that this volume will be trapezoidal in shape as shown in Fig 6, where \( \Phi \) denotes the angle of internal friction for the fill. The mass, ratio, \( m_1/m_2 \), between the boulder and the struck volume is given by the expression:

\[
m_1/m_2 = 3B^2/2y/\rho/\gamma d(3B^2 + 6Bd\tan \Phi + 4d^2\tan^2 \Phi)
\]

The relationship between the mass ratio and the depth of fill is illustrated for the design boulder of 8 m\(^3\) in volume, in Fig 7. Also shown in the figure is the relationship between the mass ratio and the percentage energy absorbed by the supporting structure as given for plastic collisions by the expression for \( R_p \). The depth of fill across the shelter varies from 3 to 8 m. It can be seen from Fig 7 that the percentage energy absorbed by the supporting structure will vary correspondingly between 14 and 3 per cent.

**Static equivalent to impactive load**

A high velocity projectile which is small compared to the object on which it impinges causes local failure only at the point of impact. Due to its large inertia the bulk of the struck object is not significantly influenced by the impact. The inertia of the deck slab is small enough compared to the size and effective width of the design boulder for the entire slab to be accelerated upon impact. The effect of the impact on the structure was as a result stimulated by an equivalent static load. The magnitude of this load was determined by assuming that the net energy imparted to the struck mass was absorbed at maximum deflection by the structure resist ing the displacement of the struck mass. The energy imparted to the struck mass was absorbed at maximum deflection by the structure. Due to the inclined surface of the fill the volume of the struck mass will vary depending upon the point of impact, as illustrated in Fig 6. Therefore:

\[
E = E(d)
\]

where:

\[
E = \frac{3(m,v)^2}{2(B+2d\tan\Phi)} (3m_1+3\rho d B^2 + 2\rho d B \tan \Phi + 4\rho d \tan^2 \Phi)
\]

It follows from the preceding arguments that:

\[
W = 0.8E
\]

where:

\[
w = \frac{0.8E(d)^{0.5}}{F}
\]

**Fig 6: Illustration of incident boulder and struck mass of earthfill**

**Fig 7: Graphs of energy absorbed and depth of earthfill against ratio of incident mass to struck mass**

The work done by the distributed equivalent static load was determined from the following expression:

\[
W = \int_a^{(a+b)} \frac{w y}{2} dx,
\]

where \( y \) denotes the deflection caused by the load \( w \) distributed over length \( d \) as shown in Appendix B.

It can be shown after appropriate manipulation that:

\[
W = w^2F
\]

where

\[
F = \frac{b(2a+b)}{480LE/AB} \left(-60L(\frac{RA}{w}) (a^2 + (a+b)^2 - 2L^2) + \frac{2Lb^4}{(2a+b)} - 5(L-a)^4 + 5(L-a-b)^4\right)
\]

Let \( E \) denote the energy imparted to the struck mass per metre run of the length of the structure. Due to the inclined surface of the fill the volume of the struck mass will vary depending upon the point of impact, as illustrated in Fig 6. Therefore:

\[
E = E(d)
\]

where:

\[
E = \frac{3(m,v)^2}{2(B+2d\tan\Phi)} (3m_1+3\rho d B^2 + 2\rho d B \tan \Phi + 4\rho d \tan^2 \Phi)
\]

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\]

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\[
w = \frac{0.8E(d)^{0.5}}{F}
\]
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The relationship between \( w \) and \( d \), as a function of the distance from the left hand support, is illustrated in Fig 8. It is evident that the magnitude of \( w \) is a function of the stiffness of the supporting structure relative to the point of impact. For example, for points of impact close to the outside columns, which deflect very little under the impactive load, the load intensity \( w \) is the largest. The varying intensity of load which the earthfill imposes upon the structure is also shown in Fig 8.

There are a number of alternative ways in which the equivalent static load on the structure due to an impactive blow on top of an energy buffer can be determined. One such formula given by the Japanese Railway Authority\(^2\) may for example be expressed as follows in terms of the symbols adopted here:

\[
    w = \frac{24m_{\text{boulder}} \cdot 0.67 \cdot 0.4 \cdot \mu \cdot 0.8}{d^2} \text{ kPa}
\]

where \( \mu \) denotes Lamè's constant which for sand varies between 100 and 300 in appropriate units and \( H \) the height of fall of the incident boulder. The Japanese Railway Authority\(^2\) also gives an alternative formula based on \( \alpha \), the deceleration upon impact of the incident boulder. In terms of the current notation:

\[
    w = \frac{m_{\text{boulder}} \cdot \alpha}{b'^2} \text{ kPa}
\]

Values are given for the parameter \( (\omega/g) \) in the quoted reference up to fall heights of approximately 10 m only. By extrapolating the data on the basis of the impulse momentum principle the value for \( w \) from formula 2 for \( m_{\text{boulder}} = 207 \), a fall height of 30 m and a depth of sand buffer of 5 m can be shown to amount to approximately 85 kPa. For the same values of the parameters, \( w \) from formula 1 amounts to 80 kPa. The value of \( w \) for a depth of fill of 5 m for the Kowyn's Pass shelter can be interpreted from Fig 8 to be equal to approximately 84 kPa. The similarity in magnitude of \( w \) from the different approaches is remarkable.

It is further interesting to observe that the Japanese Railway Authority makes provision in the graph of \( (\omega/g) \) against \( H \), for the stiffness of the supporting structure relative to the point of impact. By appropriate interpolation and interpretation of the data a graph of \( w \) against distance from the left hand support can be constructed as shown in Fig 8. The overall correspondence with the \( w = w(d) \) function used in the design of the Kowyn's Pass shelter is again very remarkable.

Lateral thrust in fill

The lateral flexibility of the shelter gives rise to active stresses in the fill covering it. The corresponding lateral thrust was determined from Coulomb's formula for the active pressure of a sloping, dry, frictional material on an inclined retaining wall\(^2\). The situation is illustrated in Fig 9. The volume of fill ABDE was assumed to be a rigid body, and the surface BD to represent the inclined reverse side of the retaining wall for which the formula was developed. The active thrust on the soil to soil interface BD is given by the expression:

\[
    P_a = 0.5\rho gH \left[ \frac{\csc \theta \sin(\beta - \Phi)}{(\sin(\beta + \Phi) - \sin(\beta - \Phi))^{0.5} + (\sin(\beta + \Phi) - \sin(\beta - \Phi))^{0.5}} \right]^2
\]

The horizontal component of \( P_a \) can be shown to have a maximum value

![Fig 8: Variation in intensity of impactive load as a function of structural stiffness](image)

![Fig 9: Lateral forces acting in earthfill](image)
of 150 kN/m for $i = 27$ degrees, $\phi = 35$ degrees and $\beta = 110$ degrees.

**Force and displacement diagrams**

The derivations of bending moment, shearing force, thrust and displacement functions are given briefly in Appendices A and B for the static and equivalent impactive loads, respectively. Corresponding envelope diagrams for bending moment and shear are shown plotted in Fig 10. It can be seen from Fig 10(a) that the maximum bending moment for the impactive load is approximately equal to that due to the static loading.

**Stability of concrete sandwiched rockfill**

The volume between the shelter and the rockfill is filled with a self-supporting concrete reinforced rockfill as shown in Fig 11. A plain compacted soil or rockfill, bearing against the rear wall of the shelter, could have been used instead. This would, however, have given rise to the development of a lateral thrust on the shelter equivalent to the full depth of the fill of approximately 15 m. Furthermore, if the fill bore intimately against the wall it would have increased the overall stiffness of the structure with regard to impactive loads.

By providing a fill which stands clear of the rear wall, the lateral thrust on the shelter was reduced by approximately 65 per cent. In addition to allowing the shelter maximum flexibility, the opening between the fill and the structure obviates the development of any water pressures on the wall. By reinforcing the rockfill with layers of concrete the necessary amount of cohesion was ensured in the fill for stability requirements.

**Provisions for stormwater run-off**

The rainwater flows principally as a sheet from the rock slope onto the earthfill. The flow rate from the rock face is relatively small although the rainfall intensity is high. This is due to the limited extent of the catchment area. The run-off from the slope was estimated to be between 0.3 to 0.8 m$^3$/h for recurrence intervals of 4 and 100 years, respectively, and a time of concentration of 0.1 h. To obviate erosion damage of the earthfill and to convey the estimated maximum run-off, a trapezoidally shaped stone pitched channel of some 0.6 m$^2$ in cross-section was provided along the back of the fill against the rockface. A transverse stone pitched cut-off trench of similar dimensions was in addition provided along the edge of the fill above the Bosbokrand end of the structure.

**Aspects of construction**

**Foundations**

The foundations of the columns and the rear wall were explored beforehand by diamond drilling. The column footings comprised individual mass concrete piers varying in depth between 4 m and 12 m. The rear wall was founded on a strip footing which varied in depth between 0.7 m and 2.2 m. All the foundations are underlain by residual hard to very hard rock granite and were taken to a depth below which no voids or fissures were visible on the joints. A number of percussion holes were drilled in all the excavations for the column piers and filled with a lightly pressurized grout to ensure that no open joints remained. Every column footing was further secured against lateral instability by three horizontally installed 50 t anchors. No blasting was allowed on site.

**Formwork**

A steel shutter was used for the soffit of the deck slab. The soffit shutter for the arch openings between the columns was lined with galvanized sheet metal. All other formwork comprised regular timber
shutter boards, which in general were not re-used more than three to four times. Casting blemishes were minimal. The alignment and level of the formwork were set out from the centre line of the road to a tolerance of approximately 5 mm. The principal cross-sectional dimensions were fixed to a tolerance of ±5 mm to +10 mm. The columns and rear wall were plumbed to a tolerance of approximately 5 mm. The formwork for the deck slab was positively cambered by 50 mm, of which 5 mm to 10 mm was taken up by settlement of the scaffolding and a further 10 mm to 15 mm by the dead weight sag of the slab.

Reinforcement
Approximately 580 t of reinforcing steel was placed in the structure. Of this amount approximately 95 per cent was high tensile steel and the remainder mild steel. The cover to the reinforcement in the footings was 50 mm and that in the columns and rear wall 30 mm. The cover to the reinforcement in construction joints and on top of the deck slab was 100 mm and on the bottom of the slab 30 mm.

Concrete
Approximately 4 000 m³ of concrete was placed in the structure. A 40 MPa mix, containing ordinary Portland cement only, was used for the main structural elements which comprised approximately 60 per cent of the total amount of concrete.

The aggregate for the concrete and the stone for the gabions, pitching and rockfill were obtained from a granite quarry some 20 km from the site. Crusher sand was used in the concrete sandwiched in the rockfill. In all other concrete a washed river sand was used. The concrete was weigh batched in a rotating drum mixer. A maximum free fall of 1.5 m was not exceeded on placement. An average rate of placing of 8 m/h was achieved. Poker vibrators only were used. Vertical shutters were stripped after three days and soffit shutters after seven days. Curing of the concrete was mainly done by continuous water sprays. Three cubes were taken for each of 3, 7 and 28 day strength tests from every concrete pour. This adequately ensured the quality of the concrete.

Rock anchors
The sway arresting anchors at the elevation of the deck slab comprise untensioned 32 mm Dywidag bars installed at 2.5 m centres. The resin bonded anchorage of each bar is 1.5 m long. The remainder of the drill holes were filled with a lightly pressurized wet cement grout. As an added precaution against corrosion the rock surrounding each anchor was pressure grouted from an additional hole drilled 0.5 m above the anchor. The length of anchor between the rock face and the deck slab is protected against corrosion by means of a grease filled PVC sleeve. Anchorage in the concrete slab is ensured by end plates on the bars, in addition to a sufficient straightforward bond length. Eight of the 56 anchors installed were tested to 150 per cent of the working load and the remainder to 120 per cent. Elongation in excess of 0.3 per cent of 1.5 m, the length of the lower anchorage, was not accepted.

Each of the mass concrete column foundations is anchored by means of 3 x 50 t wire rope cables. The cables were epoxy coated and placed in grease-filled PVC sleeves, except for the lower 5.0 m. The drill holes varied in depths between 14 and 20 m and were filled completely with a cement grout after installation of the cable assembly. Two of the 50

Fig 12: Graphs of sway against time and depth of fill

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anchors installed were proof loaded to 150 per cent of the working load and the remainder to 120 per cent. A pre-stress of 5 t was left in every anchor. The anchor head assemblies were epoxy coated and cast in protective concrete blocks.

**Rockfill**

The boulders in the concrete sandwiched rockfill behind the rear wall varied from 200 to 600 mm in size. The fill is founded mostly on soft to hard rock. The boulders were placed in tightly fitting array in layers approximately 60 mm deep. The front 1,0 m width of the fill was fully grouted with a wet, small aggregate 10 MPa concrete. The sandwiching layers were approximately 100 mm thick and comprised a stiff 20 MPa concrete. The successive layers of boulders were placed immediately on top of the wet concrete. A cavity of 0,5 m was left between the rockfill and the rear wall of the shelter. Sufficient drainage holes were provided.

**Gabions, earthfill and seeding**

A gabion wall was built to a height of 3,0 m around the outside of the deck slab to contain the earthfill. The wall was built in lifts of 1,0 m. Special precautions were taken to ensure the quality of the walls. A filter cloth was provided between the gabions and the earthfill. Approximately 15,000 m² of end tipped fill was placed on top of the structure. The fill consisted of a clayey and siltly fine to coarse grained sand. It was placed to a reasonably uniform average moisture content and Mod AASHO density of 21 and 90 per cent respectively. The average cohesion and friction angle of the material was 20 kPa and 33° separately.

Longitudinal piles of evenly graded stone, covered with a filter cloth, were placed at regular intervals on top of the deck slab and the rockfill behind the shelter to provide the underdrainage to the earthfill. The slope of the fill behind the gabions is inclined at 27°. The topsoil was initially secured by means of timber wickerwork spaced at 1,0 m intervals. The topsoil was fertilized with dolomite lime, single super phosphate and commercial 2:3:2 fertilizer at 0,113, 0,025 and 0,063 kg/m², respectively. A mixture of teff, ergrostis and rye grass in the proportions 1:2:1 and at a coverage of 0,0078 kg/m² was used in hyroseeding the surface.

**Construction joints**

The structure of 135 m overall length was constructed in 8 completely separate bays. The rear wall in every bay was cast in one pour over an average period of 8 hours. The deck slab was cast in three pours: two pours each of 80 m and one smaller one of 20 m in the centre of the slab 24 hours later to allow for shrinkage. The green concrete was cleaned first before casting the central section. The joint was finally sealed with a hot poured bitumen on the top surface. The construction joints between bays were filled with a bituminous mastic sealer. The structure is completely watertight. Two minor leaks through the deck slab healed within the first year.

**Costs and rates of construction**

The total cost of construction amounted to about R1,2 million. The work took 19 months and involved a labour force of 90 persons on average.

**Monitoring of sway displacements**

Three single point rod extensometers were installed through the top of the rear wall in the second, fourth and fifth bays from the Graskop end of the structure, as shown in Fig 4(b). The extensometers were placed on top of the rockfill and were located approximately 2,0 m into the rockface. The sway of the structure was monitored during the actual placement of the fill and at regular intervals for the first year after completion, as shown plotted in Fig 12. It is evident from the figure that the amount of sway was approximately linearly related to the depth of fill. Ultimately it corresponded for all three extensometers within 10 per cent of the actual sway displacement calculated theoretically for the static load configuration.

**Acknowledgements**

The information in this paper is published with the kind permission of the Director of Roads of the Transvaal Provincial Administration. This privilege, as well as all the assistance obtained from the various offices of the Department of Roads, is sincerely appreciated. We further wish to express our gratitude for the cooperation received from the Contractor, Messrs Wilson Holmes, as well as the support provided by our colleagues in particular Mr Bruce James.

**References**


**Appendix A**

**Derivation of expressions for force and displacement distributions for static earthfill loading**

The fundamental characteristics of the shelter and the configuration of static loads are illustrated in Fig A1. The reactions in the structure may be defined as shown in Fig A2. The six unknown reactions are denoted by $R_A$, $R_C$, $H_A$, $H_B$, $H_C$ and $M_C$. These may be determined from the following conditions:

**FIG A1**

**FIG A2**

**FIG A3**

**FIG A4**

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The distributions of thrust, shearing force, bending moment and deflection are

\[ T = RA + \alpha L + \beta L^2 \]

\[ Q = MC + \gamma L + \delta L^2 \]

\[ M = \frac{RC}{H} + H \cdot \frac{x}{6} \]

\[ H = \frac{H_A(x-h)}{60a^3} \]

\[ R_C = RA + AL + BL/2 \]

\[ M_C = RA + HC + AL/2 + BL/6 \]

The distributions of thrust, shearing force, bending moment and deflection in the structure are then given by the following respective expressions:

Member DA:

\[ T = RA \]

\[ Q = HA \]

\[ M = H_A(x-h) \]

\[ y = \frac{HA}{E_{DA}} \frac{x^3}{6} - \frac{hx^2}{2} \]

Member AB:

\[ M = R_A + ax + bx^2/2L \]

\[ y = \frac{RA}{E_{AB}} \frac{x^2}{6} + \frac{b}{120L} (x^3 - ax^2) \]

Member BC:

\[ T = RC \]

\[ Q = HC \]

\[ M = RL + AL/2 + BL/6 + HC \]

\[ y = \frac{RC}{E_{BC}} \left( -\frac{RA}{2} + \frac{ax}{6} + \frac{b}{12} \right) + \frac{b}{24L} (x^3 - ax^2) \]

**Appendix B**

**Derivation of expressions for force and displacement distributions for dynamic loading**

The distributed equivalent static load is illustrated in Fig B1. The reactions, \( R_A \), \( R_C \), \( H_A \), \( H_B \), \( H_C \) and \( M_C \) are defined in Fig B2. It can be shown for the same boundary conditions given in Appendix A that:

\[ R_A = \frac{w(L-a)(128L^3+256L^2-24L^3)}{8L^2(1+3B)} \]

\[ R_C = \frac{w(L-a)^2}{2L^2(1+3B)} \]

\[ H_B = \frac{w(L-a)^2}{2L^2(1+3B)} \]

The distributions of thrust, shearing force, bending moment and deflection are then given by the following respective expressions:

Member DA:

\[ T = RA \]

\[ 492 \]

**Discussion on papers**

Written discussion on the papers in this issue will be accepted until 15 November 1982. This, together with the authors' reply, will be published in the May 1983 issue of The Civil Engineer in South Africa, or later.

Such written discussion, which must be submitted in duplicate, should be in the third person present tense, and should be typed in double spacing. It should be as short as possible and should not normally exceed 600 words in length. It should also conformed to the requirements laid down in the 'Notes for the Guidance of Authors and Contributors' as published in the September 1974 issue of The Civil Engineer in South Africa.

**Reference**

Whenever reference is made to the above paper this publication should be referred to as The Civil Engineer in South Africa and the volume and date given thus: Civ Engr S Afr, Vol 24, No. 9, 1982.
Use of ferricrete for road construction in the South-Western Cape Province

B ROSSOUW* (Member)

Synopsis

As a result of the tremendous variation of the properties of ferricretes found in the south-western Cape, especially with regard to their uses for roadbuilding, it was decided to conduct an in-depth investigation to establish the reasons and extent of this variation in formation. The investigation was conducted on the in situ material in two borrowpits located in a road contract, as also on the ferricrete in the road pavement extracted from these two sources. All the tests discussed were conducted in the normal site laboratory.

The discussion contained herein reviews the origin of ferricrete, the results of the tests conducted and some recommendations as to extraction and construction procedures. The tests conducted on the in situ ferricrete deposits included such aspects as the California Bearing Ratio, the Atterberg Limits, moisture contents, stabilization and the reaction of certain primes.

Introduction

Due to the extensive use of ferricrete for basecourse and subbase, on a 30 km road contract in the Hermanus area, it was essential to determine the properties and behaviour of this material for roadbuilding purposes.

Ferricrete has been used for road construction throughout the world for many years. Because of its generally superior quality when compared with other natural gravels, it has been incorporated in the upper pavement layers. Although it is normal to group all ferric oxide cemented material under the banner of ferricrete these materials vary extensively in characteristics and properties. Amongst the more commonly used terminology the following are referred to: ferruginized soil, nodular ferricrete, honeycomb ferricrete, hardpan ferricrete and boulder ferricrete.

As most of the above were found on the abovementioned contract it was decided to conduct a number of tests, within the capabilities of a field laboratory, in an endeavour to gain a better understanding of the properties of ferricrete. Included in this paper are some observations made by the author and the solutions to some difficulties which arose.

Formation

In order to understand the variability of the properties of ferricrete material a brief discussion on the formation of this pedogenic material is included. Pedogenic materials are formed by alluvial processes in a soil profile which have altered the physical properties of the soil, in some cases to the extent of making it a potentially useful construction material.

The cementing agent in ferricrete is hydrated ferric oxide which may be combined with an alumina oxide. Ferricrete formation is dependent on the climatic conditions which prevail or prevailed in the area and is generally associated with a relatively humid climate. During the wet seasons the iron compounds are reduced to their more soluble ferrous state below the water table. In the dry season the iron compounds are drawn up towards the surface by the capillary rise from the water table. The ferrous compounds dehydrate and oxidize during the evaporation of the water and revert to the immobile ferric state. The ferric oxide is hence deposited in or around the upper layer of material, depending on its physical state at the time this action occurs.

In some instances the ferric oxide may be deposited on the surface of the ground, resulting in the formation of a crust of "hardpan ferricrete". The difference in mobility of the iron and alumina compounds explains why the high iron content ferricretes generally increase with depth of clayey soils, as the less mobile alumina remains combined in the form of clay below the ferricrete. In the tropics, where leaching is at a maximum, some iron is also lost in solution, resulting in an increase in the alumina content of the weathering complex and the subsequent formation of laterites.

The colour of ferricrete varies considerably, even within a fairly restricted area. Colours range from yellow, brown and red to a deep purple and depend on the degree of hydration and the presence of elements such as manganese and titanium. Photographs 1 to 7 illustrate the degree of variance of colour and characteristics of the ferricrete deposits located in two borrow areas situated along the length of the road contract.

In this case the ferrous solutions originate from the shales, but it is not necessary for ferricrete to overlie shales as the ferrous oxides can be transported some distance before the alluvial process takes place - as on hill slopes where there is a downward flow of the subsurface water (see Fig 1).

This explains why ferricrete is frequently found over relatively impermeable sandstones which may be completely dry, even though the ferricretes are saturated with water. A common feature of this movement is found where ferricrete deposits occur in the sides of hills as this is the point where the water table is close to the surface (see Fig 1). From tests conducted it has been found that the further away the ferricretes are from the shales, the lower the PI values become. It was also noted that where deposits of ferricrete are relatively deep the PI values become progressively lower from the clay layer to the top reaches of the ferricrete sometimes even becoming non-plastic at the surface.

Properties

The properties of ferricretes varied considerably with regard to plasticity and linear shrinkage. In order to illustrate the variability of ferricrete within a single source, a number of field and laboratory tests were conducted, the results of which are discussed below:

California Bearing Ratio and compaction characteristics

Ferricrete produced a range of CBR values from very low, where clay was predominant and the ferricrete still very young and undeveloped, to extremely high values when crust ferricrete was encountered. The following results were obtained from ferricrete crushed from the same borrow pit.

<table>
<thead>
<tr>
<th>CBR at 100% Mod AASHTO</th>
<th>Mod density</th>
<th>OMC Per cent</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>2360</td>
<td>11,9</td>
</tr>
<tr>
<td>184</td>
<td>2400</td>
<td>9,0</td>
</tr>
<tr>
<td>245</td>
<td>2466</td>
<td>10,6</td>
</tr>
<tr>
<td>320</td>
<td>2550</td>
<td>7,6</td>
</tr>
</tbody>
</table>

As discussed above the properties of the ferricrete vary with depth. This is best illustrated in Fig 2 where the CBR values were determined at different depths in the test holes. It can be seen that it is extremely important to sample the material in keeping with the use for which it is intended and the means of extraction from its source.

B Rossouw graduated from the University of Natal in 1963 with a BSc in Civil Engineering. He was employed by Asphalt and General (Pty) Limited in 1964 constructing the Kimberley Airport. In 1967 he joined Keeve, Steyn and Partners in their roads section. In 1965 he was employed by Jeffares & Green in their roads section specializing in transportation and freeway design. He was employed in 1970 by a consultant in Britain, associated with Jeffares & Green, furthering his knowledge in traffic engineering and computer programming. He was promoted to an Associate in 1974. He also holds the following qualifications: MSAICE (1967), PrEng (1969), MSc (Civil UCT) (1980), MIHE (1970), MICE (1977), CEng (1978), FEANI (1979).

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The variability of the ferricrete dumped on the road from a single source produced CBR values ranging from 105 to 350 at 100% Mod AASHTO density. This in itself caused difficulties when testing the compacted layers on the road, as the maximum dry density and CBR values could not be assumed to persist over any acceptable length of road. It was found that to obtain the correct compaction at any point on the road a separate modified density sample had to be taken at each density hole. This involved a substantial increase in the work load in a field laboratory. (See photos 8 and 9 which show the variability in colour associated with variability of properties.)

Atterberg Limits

The iron oxides originate from a shale strata and are carried in solution either in an upward direction due to the capillary action or away from the original source by the movement of the subsurface water. The movement due to subsurface flow can be quite considerable in many instances and can carry the iron oxides some distance. The clay in the underlying strata in a ferricrete borrowpit is not carried upwards by the capillary action of the subsurface water to the same extent as the iron oxides. This results in a decrease of the plasticity index the further away, both horizontally and vertically, the ferruginized material is located from the original shale source. With this in mind tests were conducted at various depths in trial holes. Results from a typical hole appear in Fig 2.

Having been faced with the problem of obtaining from a series of test holes, higher plasticity index values, which would normally have been rejected, corrective measures such as gridroiling and crushing were resorted to. It was established that by crushing the ferricrete a plasticity index of 12 was reduced to eight and after compaction reduced even further to five. The explanation is that the parent material in the ferricrete investigated was sandstone which produces fines each time the material is processed, adding coarser grains of non-plastic granular particles into the -0.425 mm fraction which reduces the clay content.

Water content

The natural in situ moisture content can vary considerably within a ferricrete deposit depending on the host material. Values between 6 per cent and 13 per cent were observed. The determination of the moisture content was found to be sensitive to the method of drying. The methods used were a bare flame, a sand bath or an oven, with the last-mentioned giving the best results. Quick drying of ferricrete should not be resorted to as it is a relatively lengthy operation to extract all the moisture from the porous cavities in the ferruginized material. During certain of the tests conducted using a flame or the sand bath, a latent water content of up to 4 per cent was recorded after the drying process. The drying procedure can thus affect the acceptance or rejection of a field density.

With the moisture problem in mind tests were conducted on an existing road built of ferricrete. The results are shown in Fig 3, these being the mean lines drawn through the results obtained. It was found that the moisture can be up to 4 per cent higher under the centreline of the road than in the adjacent gravel shoulder. Moisture contents of 11 and 12 per cent were recorded under the road where the optimum moisture content was only 7 to 9 per cent respectively. This was especially interesting as the tests were conducted during the dry season. Another interesting fact was that the maximum moisture content was never found just under the bituminous surface as is the popular belief but at about 200 to 300 mm below the surface.

Stabilization

A normal reaction when dealing with gravels which may include ferricretes, is to apply stabilization by means of lime or cement. With this in mind tests were conducted on ferricrete using both lime and cement with contents of 4.6 and 10 per cent, by mass. The results of the tests conducted are shown in Fig 4.

From the results it was found that there was no increase in strength by adding lime or cement; in fact a decrease in strength with increased stabilizing agent added was established. This tendency was particularly evident with respect to the lime stabilization. A possible reason for the lack of positive effect is that the lime and cement hydroxides may not react with the iron oxide, especially since the ferricrete samples used were only slightly plastic. It was found that the only positive contribution that the lime made to the samples tested was that the PI was lowered and the liquid limit increased marginally. This has also been verified by other authors on the subject.2

Fig 1: Formation of ferricrete on slopes

Fig 2: Comparison CBR with depth

Fig 3: Moisture content in existing roads

Fig 4: Comparison of stabilized ferricrete

Self-stabilization

It was found that if a pavement layer of ferricrete is left exposed for any length of time and subjected to several wetting and drying cycles it recements into a hard crust. This recementing action takes place when some of the iron oxides go into solution during the wetting stage and precipitate out during the drying period. The tendency would therefore be to form a new bond between adjacent particles in the material. To illustrate this phenomenon crushed unstabilized ferricrete was
compacted at optimum moisture content into 152 mm concrete moulds, then stripped and sealed into water-tight plastic bags. These cubes were then stored at regular intervals for up to 100 days. The results have been presented in Fig. 5. This clearly shows increased strength with time. Perhaps as a result of this phenomenon ferricrete constructed roads tend to display less problems of material fatigue with time than other roads, all things being equal.

In addition to the recementing action of ferricretes by the wetting and drying cycles, it has been found by some authors on the subject that there is a significant correlation between the ratio of the oxides of silica to iron and the mechanical strength. The iron oxides tend to replace the silica oxides during the formation of the ferricrete. The degree of replacement will determine the quality of the ferricrete and also the potential to recement effectively.

**Penetration by bitumen and tar**

Bitumen based products were observed to meet with varying results on a ferricrete basecourse. In the case of prime, better penetration appears to occur when the ferricrete is lightly watered prior to the spray. The reason for the lack of penetration can be traced to the fines that form part of the natural ferricrete which would be absent in the normal rock crusher run. Two types of prime were used on a section of ferricrete basecourse, namely MC30 and MSP1. The MC30 did not penetrate as well as the MSP1 but tended to bind the surface and not ravel under traffic. The MSP1 penetrated rapidly but kept the surface relatively soft.

A short section of ferricrete basecourse was not primed and a tack coat of 150/200 pen bitumen was applied with a slight increase in the spraying rate. In this case it was found that the tack coat did not penetrate and could easily be lifted like a carpet. It is firmly advocated that all ferricrete basecourse surfaces should be primed to avoid surface failure.

It is normal practice to allow the basecourse to dry before prime is applied. Due to the presence of the latent moisture which is inherent in the ferricrete this drying process cannot be strictly adhered to. The criterion adopted by the writer was that priming could be undertaken once the moisture content had dropped by more than 4 per cent below the optimum moisture content. There is a danger to this as moisture bubbles have been noticed under the first tack coat on a very hot day. To date the moisture bubbles have not extended through the full surface treatment.

A phenomenon which can possibly be directly related to this latent moisture is that when the moisture is drawn out of the ferricrete modulus it tends to lower the strength of the layer directly under the surface. This causes a tendency to slight punching of the chips into this layer under heavy traffic, thus reducing the average least dimension of the chips. As bitumen application is directly determined by the ALD of the chip the resultant effective excess bitumen will cause bleeding.

**Construction**

*Extraction from borrowpit*

It is very important to stipulate the manner by which ferricrete should be excavated from a source. Although it is more economical to rip a borrowpit and load directly to the road, it is far safer to instruct a contractor to stockpile the material before use. Where stockpiling is encouraged the following benefits will be realized:

1. Better mixing of the variable ferricrete.
2. Better control on a more homogeneous material on the road.
3. Reduction of high PI values possibly avoiding additional processing.
4. The possibility of excavating greater depths of ferricrete which may exhibit higher PI values.
5. Greater confidence in the maximum use of a source especially where limited supplies occur.
6. Whether crushed or not improved gradings can be obtained.

As discussed above ferricrete varies considerably and in certain instances crushing may be necessary to produce a workable layer material. Normally gradings can be taken after crushing but these will vary considerably within a borrowpit, thus causing some problem in setting the crusher screens. It is strongly advocated that short test sections of crushed material be compacted on the road and gradings taken to establish the pattern of breakdown compared with the crusher gradings. This could influence the crushing pattern.

**Surface finish**

Ferricrete will, if prepared properly, present a very good finished surface (refer to photos 10 and 11). Because of the stone shape the initial brooming should be completed during slushing, whilst the material is very wet. The final brooming can be done when dry but must be carefully controlled to avoid ravelling. In some cases handbrooming must be resorted to in place of the mechanical broom to safeguard the surface finish.

**Conclusion**

Although the tests conducted were limited by the equipment available in a site laboratory the objectives of the investigation were realized. The aim was to illustrate the properties and variability of the group of materials called ferricrete and to highlight some of the problems which can occur when dealing with these materials.

To summarize the above the following should be borne in mind:

1. Ferricrete is a collective name for different materials which have been cemented by iron oxides and as such possess varying physical and structural properties which must be taken into consideration when testing a borrowpit.
2. A ferricrete deposit varies in properties both with depth and location within that deposit. Exploration must therefore be extensive.
3. Ferricrete deposits may be located some distance away from the original source of the iron oxide. If this occurs the plasticity index of the material may be very low.
4. If high PI values are encountered the engineer should investigate the possibility of reducing this by crushing or gridROLLing. The addition of lime would assist in reducing the effect of the clay content.
5. Some ferricrete is very recent or exposed to wetting and drying cycles. The strengths achieved are often far in excess of a comparable rock crusher run or natural gravel stabilized with cement.
6. Construction techniques must be adopted to suit the ferricrete material to avoid segregation in the layers.
7. Before specifying different types of prime, tests should be conducted to determine the penetration effects.
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8. Ferricrete should not be surfaced without the use of prime.
9. Ferricrete bases should be allowed to dry out as much as possible before priming to avoid possible blistering of the prime by latent moisture in the ferricrete nodules.

Acknowledgements
I wish to thank Messrs Van Vuuren and Visagie of the Jeffares & Green Inc site laboratory at Hermanus for their time and effort in conducting the experiments necessary for the presentation of this paper. Appreciation must also be extended to Dr F Netterberg of the NITRR for his assistance and valuable advice. I would like to note that the views and observations contained in this paper are my own and, although not designed to cover the full spectrum of the ferricrete group of materials, must be viewed as an attempt to generate active thought on this subject.

References
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Soweto Project

To the Editor of The Civil Engineer

Sir: We refer to the interesting article on the Soweto Education Project in your April 1982 issue (Vol. 24, No. 4). We wish to thank the organizers and at the same time inform them that we support the scheme wholeheartedly.

There is, however, an aspect which we feel the organizers (SAICE/SAFCEC) are missing. It is all well and good to decide how many students are to be trained and how many are to be aided financially, etc.

What is important to realize is that all black students have to obtain a permit allowing them to study at 'open' universities such as Wits, for example. What this means is that the government has the final say as to how many graduates will be black.

Also, we feel it is important to include in the education project programme typical problems blacks in particular will face both in their lives as students and as professional engineers. Yes, we must move towards improving our relations and here we would be the last to disagree. The point, however, is that healthy attitudes on site are hard to come by at present. The young students should be made aware of this.

It is a known fact that the government distributes financial aid for education unequally among the different population groups, thus achieving different levels of education. It does seem a little absurd then for the private sector to spend large sums of money on 'improving' black education without at least making a public statement criticizing the present government's attitudes towards education, viz its refusal to accept all the recommendations of the De Lange Report.

C Hansa, H D Kuverjee, S B Msutwana and S L Manyatshe, 4th Year, BSc (Eng) (Civil), University of the Witwatersrand.

Editor's reply

In the Institution's official comment on the De Lange Report, fully documented in the Editorial in the April 1982 issue of this Journal, the main recommendations of the report are supported and the urgent need to implement the proposed reform of the education system is stressed. It was stated that tertiary engineering education institutions, both technikons and universities, should be free to admit students of all races at their own discretion and the importance of black students being accommodated on campus and being involved in an integrated environment during their studies was stressed. Regarding healthy attitudes in employment situations, we are encouraged by the evidence of continually improving attitudes and must emphasize that this will be greatly improved by proven success of black engineers and technicians, an aspect which the Institution has accepted as an important priority.

A 'mere' Doctor

To the Editor of The Civil Engineer

Sir: I should like to question a statement made by Michael Rees, PrEng, (letter to the Editor, June 1982) in which he states: '... Prof Ockleston (then a mere Dr) ...'. A mere Doctor (I presume of Engineering) is the highest recognition of one's academic achievement, while a Professor is an appointment and has no direct bearing on one's academic progress.

I remember Mr W H 'Concrete' King of the University of Cape Town, when still a mere Mr, before he was awarded his Doctorate, saying 'Anyone can be made a Professor, but you have to be good to be awarded a Doctorate', or words to that effect.

With the greatest respect to all Professors, being a professor is an appointment and is no greater honour than holding many other senior appointments. It is certainly not easily come by and does signify a sound academic record, subsequent experience, maturity and talent amongst other attributes. Being awarded a Doctorate is a completely different achievement and should not be confused with an academic appointment.

M S Judd, PrEng, Ulster Cottage, Thorpe House, Tokai. 7945.

future monitoring have no practical value. Once the project has been built in its present proposed form there will be no practical way to correct its permanent harmful effect.

Why was a full EIA, as recommended by Habitat, not done for such an important project? It would have required six to nine months and would have given a proper rational base for the decision makers.

Quote: 'The strongest justification for the establishment of the pipeline must be a socio-economic one...'.

Comments: On pages 181/182 of your issue, describing the Mondi project, we have the following on the socio-economic justification: 'The R520 million pulp mill... will employ more than 1 000 people...'. 'Employment' is a term used naively in similar debates but a heavily capitalized industry does not solve the employment problem, rather, it creates one! Examples of this are found well outside the industrialized parts of the world.

R520 million for 1 000 employees gives approximately R500 000 capital outlay to create one working place. Whereas a capital outlay of R1 000 to R5 000 per working place, in rural development of modern, small agricultural holdings, supported by decentralized small-scale industry, a superior type of employment, both economically and socially, could be produced. With R520 million of capital 100 000 to 300 000 working places could be created, making the whole region prosperous!

Unfortunately, in our present distorted philosophy of life, our economy does not allow for such a concept — it would not show in GNP statistics!

The Richards Bay industrial concept, perhaps unfortunately, has to be accepted as a fact. In which case a sea outfall, properly planned, could be an essential part of the infrastructure, as is electricity, water supply and transport. Waste disposal is an essential element of industrial production.

An area, approximately 3 000 ha, has been reserved for the proposed industrial development at Richards Bay. It can be expected that the industry occupying this land will be producing a proportionately greater volume of waste than Mondi and Trioom, who occupy less than 20 per cent of that area. Yet Mondi and Trioom alone utilize some 70 per cent of the total capacity of the proposed outfall.

It appears that present planning of the sea outfall is chiefly to serve Mondi, Trioom and the municipality. There is no plan for the rational disposal of the totality of the expected waste.

A sea outfall with properly established purification standards for the effluent, controlled and enforced at the intake end of the pipeline could be an acceptable part of such waste disposal plan. In its present form the proposed pipeline could become an impediment to such a rational solution.

I therefore believe that this project may be not only an 'Environmental blunder'.

Michal S Zakrzewski, PrEng, Environmental Consultant, PO Box 3540, Durban, 4000.

Editor's reply

For the past six months or so the Institution has been publishing more articles of general interest in the Journal as a service to members. Some of these articles have been written by our features writer from information given to her and from her own research. These articles, and those obtained from other sources, are written as reports, go unsigned and therefore do not pretend to represent the view of the Institution. The article on the Richards Bay sea effluent pipeline was written on information obtained by interviewing officials in provincial and government service as well as from the local authority. The Habitat Council and members of EPIC were also consulted. (Mr Zakrzewski's letter has been slightly shortened and actual references omitted to save space.)
Who is building South Africa’s most spectacular and technically advanced viaduct?
Right now, Concor is completing work on the spectacular, incrementally launched viaduct spanning the Hugo's River Valley, at the foot of the du Toits Kloof pass, near Paarl.

The viaduct will be 535 m long with a deck width of 16 m, a radius curve of 596 m, a super elevation of 10% and a 5.7% gradient, making it the steepest viaduct of its kind in the world.

The graceful, slender lines of the viaduct blend with the topography whilst bridging the rugged South African landscape and allow the wildlife to move unhindered through the valley.

The Hugo's River Valley Viaduct — another example of Concor's innovative design expertise and engineering skill.
Dolosse oorskry die 100 000 kerf in Suid-Afrika

Die woord dolos in die sin van 'n golfbrekerblok is vandag algemeen bekend, ook in die buiteland. In 1983 het die eerste dolosse hulle verskyning in Oos-Londen gemaak waar 'n vindingryke haye-ingenieur, mnr. Eric Merrifield, die besondere vorm van beton gietstuk ontwikkel het vir gebruik in golfbrekers in plaas van konvensionele beton blokke. Hy het sy gedagte verder laat gaan en wel om 'n golfbrekerblok te ontwerp wat gelykstaande sou wees aan soortgelyke gepatenteerde blokke soos tetrapods (Frans), tetrahedrons (Japannees) en andere, wat reeds in die buiteland groot aansien verwerf het. Ook in Kaapstad se hawe is hulle in die sesjier jare gebruik.

Nadat 'n aantal 18-ton dolosse as eksperiment aan die kop van die Oos-Londense golfbrekerblok gebruik is, in plaas van aanvulling met 37 ton kubusvormige blokke waarmee die seehoof beskerm is, dan is daar besluit om 'n modelstudie te maak in samewerking met die WNNR om die kenmerke van die dolosse en verskeie ander vorms wat reeds in gebruik was, in 'n laboratorium op 'n vergelykende basis wetenskaplik te bestudeer. Die uitslag was beslis in die guns van die dolos!

Mnr. Merrifield hurk langs 'n dolosmodel

Wat dan is die geheim van die besondere vorm? Die eierskappe wat die uitvinder as uitgangspunt aan homself gestel het, was:

- Die blok moet stabiel wees in groepsverband.
- As deklaag moet dit poresus wees sodat golfenergie in 'n hoë mate selfdempend is.
- Die vorm moet so wees dat dit vir ekonomiese massaproduksie aangewend kan word.
- Die gietstuk moet voldoende sterk wees vir hanteerdoeleindes.

Aan elkeen van hierdie voorwaardes voldoen die dolosse besonder goed en dit is ook in die buiteland met behulp van modelstudies en in die praktiek bewys.

Meer as 100 000 dolosse is reeds met wetslae in Oos-Londen, Port Elizabeth, Mosselbaai, Sandy Point, Kaapstad, Gansbaai, Natal Suidkus, Richardsbaai en St. Lucia gebruik. Die grootste varieer van 3 ton tot 30 ton; die uitsondering is St. Lucia waar mini-dolosse van 1 ton gebruik is. Die 30 tonners kan aan die kop van die golfbreker te Richardsbaai gesien word. In die buiteland is hulle gebruik in die VSA, Kanada, Portugal, Spanje, Japan, Denemarke, Hong Kong, Tristan da Cunha en word oorweeg vir projekte in Holland, Australië, Hawaië, Israel en Egipet.

Van die grootste dolosse wat tot dusver gemaak is, is 39 tonners vir die seehoof van Humboldthawe, Kalifornië, 42 tonners vir die golfbreker in Sines, Portugal en in Japan seifs dolosse met 'n massa van 50 ton.

Eric Merrifield het van die staanspoor af besluit dat as hy daarin sou slaag om 'n meer doeltrefende beton blok te ontwerp, dit nie gepatenteer sal word nie, intende dat dit aan enigeen vrywel beskikbaar gestel word. Juis as gevolg van hierdie oprede is baie meer navorsing op dolosse in die buiteland gedoen as op enig ander van die blokke waarop wit 'n patent uitgemene is.

Lidmaatskap

Candidates elected to Corporate Membership

The names of the following candidates having been published in the July 1982 issue of The Civil Engineer in South Africa in accordance with the By-Laws, the Executive Committee has elected them to Corporate Membership:

P Huber, PrEng, Dip Ing (Zurich) (Civ), Durban
L J Rossette, PrEng, BSc (Eng) (Witwatersrand) (Civ), Johannesburg
J S Somloka, PrEng, Dip Ing (Bmp), Johannesburg
C Wilson, PrEng, BScEng ( Natal) (Civ) MS (California), Durban
R J E Nicholson, BA hon (Cantab) (Eng), Mbabane
L A K Blox, PrEng, BScEng ( Natal) (Civ) MSc (Florida), Pretoria
A Colenbrander, PrEng, BScEng ( Natal) (Civ), Komga
B A Faist, PrEng, BScEng ( Cape Town) (Nai-MBA) (Cape Town), Cape Town
M C Holton, PrEng, BSc Eng (Witwatersrand) (Civ) GDE (Witwatersrand), Johannesburg
B G McIntosh, PrEng, BScEng ( Natal) (Civ), Johannesburg
J Potgieter, PrIng, BSc Ing ( Pret) (Siv) BSc (hons) (Pret) (Vervoer) HBA (Puv in CH0) Polotschrestroom
H W Rothman, PrIng, BEng (hons) (Pret) (Siv), Johannesburg
S O Saether, PrEng, BSc Eng (Witwatersrand) (Civ), Johannesburg
D J Taylor, PrEng, BScEng ( Natal) (Civ), Richards Bay
W S Valentine, PrIng, BSc Ing (Pret) (Siv), Thabazimbi
N J van den Berg, PrIng, BSc Ing (Pret) (Siv), Kempton Park
P V van Huisjes, PrIng, DrIr (Delft) (Civ), Chamard
D Wiescomb, PrEng, BSc Eng (Witwatersrand) (Civ), Richards Bay
G S Wolvaardt, PrEng, BSc Eng (Witwatersrand) (Civ), Johannesburg

Applications for election

Applications for election to the grade of Corporate Membership have been received from the following candidates and have been approved by the Executive Committee:

R G Meyer, PrEng, BScBIng (Stell) (Siv) Dip in Adv, Public Admin., Pretoria
M J Baigel, PrEng, BSc (Cape Town) (Civ), Cape Town
R M Batchelder, PrEng, BSc Eng (Witwatersrand) (Civ), Johannesburg
G C Fisher, PrEng, BScEng ( Natal) (Civ), Durban
J A Griffioen, PrEng, BSc Eng (Witwatersrand) (Civ), Rivonia
M Kolesky, PrIng, BScBIng (Stell) (Siv), Cradock
M J Narun, BSc Eng (Witwatersrand) (Civ), Canada
P J F Oscroft, PrEng, BSc Ing (Witwatersrand) (Civ), Johannesburg
H J Tuczek, PrEng, BSc Eng (Cape Town) (Civ), Cape Town
J P van der Westhuizen, PrIng, BSc Ing (Pret) (Siv) BSc (hons) (Pret) (Konstruksiebestuur), Johannesburg

Applications for admission

Applications for admission to the following grades have been received from the following candidates and have been approved by the Executive Committee:

As Graduates
M I Douglas, BSc (Newcastle-Upon-Tyne) (Civ), Windhoek
A G Hall, BSc (Birmingham) (Civ) PhD (Nairobi) (Eng) Ing, Johannesburg
L J Pickering, MSc Eng (London) (Civ) B Admin (Univ of SA), Pietermaritzburg
M E Ross-Smith, BSc Eng (Cape Town) (Civ) MBA (Cape Town), Chloorkop
D W Shaw, BSc Eng (Cape Town) (Civ), Cape Town
P J Viljoen, BIng (Stell) (Siv) Ming (Stell) (Siv), Brits
J Biisch, BIng (Stell) (Siv)
P J Gouws, BIng (Pret) (Siv)
W W Maartens, BIng (Pret) (Siv)
M N Lear, BScEng ( Natal) (Civ)
P J Maré, BIng (Stell) (Siv)
M R Taat, BSc Eng (Witwatersrand) (Civ)
A d V Taljaard, BIng (Pret) (Siv)

As Graduate - Registered as Engineer-in-Training: R McLaren

As Graduate - On Advice from the University of Natal

As Student: W J J Pilliers

As a Companion: P D Santillano

Obituaries: It is with regret that we record the death of the following:
I H Lowe (Member), P R MacKay (Fellow), J P Smit (Member)

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