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 Boshokran 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 sloping 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, it 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 Boshokran 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,

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Photo 1: Aerial view of rockface and protective shelter

Photo 2: End-on view of rockface

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DIE SIVIELE INGENIEUR in Suid-Afrika — September 1982

477
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 agglomeratic 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. A geological map of the area and stratigraphic section are shown in Fig 1. A generalized section through the rock face is shown in Fig 2.

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 reconnaissance 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 agglomeratic lavas. The joints in this zone delineate blocks ranging from a few cubic metres to approximately 125 m³ 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 m³. 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 m³ in volume will be produced from it. A major fall of rock occurred from the granite

Fig 2: Generalized section through rock face showing approximate positions of potentially unstable zones

Fig 3: Elevation of rockface showing potentially unstable zones and relative position of rockfall shelter
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 detective 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>
<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 continuation 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 is 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
rockfill behind the rear wall. Gabions around the top perimeter and an uncompacted earthfill on the top. Horizontal sway arresting anchors tie the structure at the level of the deck slab to the rock face.

**Fundamental structural characteristics**

The intensity of an impactive blow is a function of the rigidity of the struck object. The stiffer the struck object the more severe the intensity of the blow. In view of this phenomenon the structure was designed to be as flexible as possible without exceeding maximum deflection. The cavity behind the rear wall allows, for example, a rotation of the joint 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,

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**Fig 4: Principal geometric detail of shelter**
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 dissolved. These boulders were readily identifiable and were limited in number to approximately twenty. If they were dissolved 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. Allowing for a temporary reduction in the factor of safety, the effect of these large boulders would be equivalent to the impactive effect of the design boulder. The factor of safety would be reduced for as long as it would take the boulder to find its way down the inclined surface of the earthfill and progress further down the hill below the level of the road.

If the boulders along the top escarpment face fell and were to break up they would produce blocks of approximately 25 m³ in volume. Boulders of this size could be expected to bounce down the slope and far exceed the impactive effect of the design boulder of 8 m³ in volume. Due to the uncertainties with regard to the stability of the top escarpment face and the effects which boulders from this zone would have on the structure, preventive remedial measures in the form of anchored dawels and laced cables were installed in this zone.

Based on the jointing in the rock the design boulder was taken to be approximately cubic. For the size of the possible boulders only the cubically shaped ones were considered likely to reach the shelter in an unbroken condition. Tabularly shaped rocks of 8 m³ in volume are not likely to reach the shelter in an unbroken state.

The main direction of travel of the design boulder was assumed to be on dip of the rock face. The maximum height of free fall amounted to a height of approximately 46 m above the elevation of the road and to a height of approximately 32 m above the top of the earthfill. The velocity on impact was assumed to be equal to the speed of free fall given by

\[(2gh)^{0.5} = (2 \times 9.81 \times 32)^{0.5} = 25\ m/s.\]

Spinning of the design boulder was not considered. It was considered to be unlikely that the design boulder would reach appreciably large angular velocities. This is confirmed retrospectively by the findings of the Japanese Railway Authority that the rotational energy is approximately 10 per cent of the translational kinetic energy of a falling boulder. The boulder was considered to strike the sloping surface of the earthfill vertically.

**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:

\[m_1v_1 + m_2v_2 = m_1v_1' + m_2v_2'\]
\[m_1v_1^2 + m_2v_2^2 = m_1v_1'^2 + m_2v_2'^2\]

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:

\[E_2 = 0.5m_2v_2^2\]

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:

\[R = \frac{m_2v_2^2}{m_1v_1^2}\]

It can be shown from the above conditions that

\[R_0 = \frac{4m_1/m_2}{(m_1/m_2 + 1)^2}\]

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:

\[m_1v_1 + m_2v_2 = (m_1 + m_2) v_2'\]

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

\[E_2 = 0.5(m_1 + m_2)v_2'^2\]

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

\[R_p = \frac{m_1/m_2}{(m_1/m_2 + 1)}\]

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

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**Fig 5: Energy absorbed ratio for elastic and plastic impacts**

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**DIE SIVILE INGENIEUR** in Suid-Afrika — September 1982

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483
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 shelf with the maximum practical depth of soil in the least appropriate state of compaction relative to the 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:

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

The relationship between the mass ratio and the depth of fill is illustrated for the design boulder of 8 m³ 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 resisting the displacement of the struck mass. The net energy was taken to be 80 per cent of the work done by the incident boulder at maximum penetration to allow for losses of energy in heat, sound, bearing failure, compaction and shearing of the fill and attrition of the soil particles. The supporting structure is illustrated in Fig 6. The work done by the distributed equivalent static load was determined from the following expression:

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

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

It can be shown after appropriate manipulation that:

\[
W = wR P
\]

where

\[
F = \frac{b(2a+b)(-60L \frac{R \gamma}{w}(a^2+(a+b)^2-2L)^2+2L(a+b)^4-5(L-a)^4+5(L-a-b)^4)}{480LE^{1/4}B}
\]

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:

\[
E = \frac{3(m_1 \gamma)}{2(B+2d \tan \Phi)(3m_1+3 \rho dB^2+6 \rho dB \tan \Phi+4 \rho D \tan^2 \Phi)}
\]

It follows from the preceding arguments that:

\[
W = 0.8E
\]

\[
\therefore \quad w = \left( \frac{0.8E(d)}{F} \right)^{0.5}
\]
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 may for example be expressed as follows in terms of the symbols adopted here:

\[
\begin{align*}
\hat{w} & = 24m_0,07/0,4Hd/0,6 \quad \text{kPa} \quad (1) \\
\end{align*}
\]

where, \( \hat{w} \) denotes Lame'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 also gives an alternative formula based on, \( \alpha \), the deceleration upon impact of the incident boulder. In terms of the current notation:

\[
\begin{align*}
\hat{w} & = m_0 g (\alpha) \quad (2) \\
\end{align*}
\]

Values are given for the parameter \( \alpha \) 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 \( m_0 = 20T \), 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 \( \alpha \) 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. 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:

\[
P_a = 0.5 \rho H^2 \frac{\cot \phi \sin (\beta - \phi) \sin^2 (\beta - \phi)}{(\sin (\beta - \phi) \sin (\beta - \phi) \cot \phi + \sin 2 \phi \sin (\beta - \phi) \cot \phi)^{0.5}}^2
\]

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

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**Fig 8:** Variation in intensity of impactive load as a function of structural stiffness

**Fig 9:** Lateral forces acting in earthfill
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 m³/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 rockface 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 6,0 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
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 600 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 placed 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 silty 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° respectively.

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 rey grass in the proportions 1:2:1 and at a coverage of 0.0078 kg/m² was used in hydroseeding 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 m³ and one smaller one of 20 m³ 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 water tight. 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 on 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.

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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:

DIE SIVIELE INGENIEUR in Suid-Afrika — September 1982 491
\[ SV = 0, \text{vertical equilibrium of forces} \]
\[ SH = 0, \text{horizontal equilibrium of forces} \]
\[ SM = 0, \text{rotational equilibrium of moments} \]
\[ \theta_{BA} = \theta_{BC}, \text{compatibility of rotations at B} \]

Sway at \( A = -\Delta \) Sway at \( B = -\Delta \)
\[ K_{HB} = -\Delta, \text{for sway to left} \]
\[ H_B = 0, \text{for sway to right} \]

It can be shown that:
\[ R_A = al(\frac{3}{8} + \frac{12b}{8+24b} + \frac{b}{10+30b}) + yH, \]

where
\[ \alpha = \frac{1}{l_{AB}} \frac{l_{BC}}{l_{AD} + h^3} \]
\[ \theta = \frac{4l_t}{3L} \left( \frac{3LK_{EBC} + 3K_{AD} + h^3}{3L} \right), \text{and} \]
\[ 4L \left( 3LK_{EBC} + 3K_{AD} + h^3 \right) \]
\[ y = 2L \left( 3LK_{EBC} + 3K_{AD} + h^3 \right) \]
\[ H_B = \frac{-3L(\frac{R_A}{l_{AB}} + \frac{a+l}{24} + \frac{b}{24}) - 2hH}{2h(s+1)} \]
\[ H_C = \frac{l}{60a} \left[ 120R_A(L^3 + 3ah) - 15aL(L^3 + 3ah) - 4bL(L^3 + 3ah) \right] \]
\[ H_A = H_B - hC + H \]
\[ R_C = -R_A + a + bL/2 \]
\[ M_C = -R_A l + H_C h + aL/2 + bL/6 \]

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

**Member DA:**
\[ T = R_A \]
\[ Q = H_A \]
\[ M = H_A (x - h) \]
\[ y = \frac{H_A (x^3 - hx^2)}{E I_{DA} \left( \frac{x}{2} \right)} \]

**Member AB:**
\[ O = -R_A + ax + bx/2L \]
\[ M = -R_A x + ax^2/2 + bx^2/6L \]
\[ y = \frac{1}{E I_{AB}} \left[ \frac{R_A x^3}{6} (1 + x/2) \right] + \frac{b}{24} (x^3 + 2x^2) \]

**Member BC:**
\[ T = R_C \]
\[ Q = H_C \]
\[ M = R_A l + aL + bL/2 + H_C x \]
\[ y = \frac{1}{E I_{BC}} \left[ \left( -aL \frac{(x^3 + bL^2)}{6} + \frac{(x^3 + h^2)}{6} + \frac{(x^3 + h^2)}{3} \right) \right] \]

**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:
\[ R_A = \frac{w(L-a)^2(12BZ^2 + 3LZ^2 - 2aL - aL + h^3)}{8L(Z + 30)} \]
\[ H_B = \frac{3LZ^2 + w(L-a)^2(12BZ^2 + 3LZ^2 - 2aL - aL + h^3)}{12KE_{EB} + 12KE_{AD} + h^3} \]
\[ H_A = \frac{3K_{EB} - H_B}{h^2} \]
\[ H_C = -H_A + H_B \]
\[ R_C = w(b-R_A) \]
\[ M_C = w(bL/2 + bL) + H_C h \]

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

**Member DA:**
\[ T = R_A \]

\[ Q = H_A \]

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

\[ y = \frac{H_A (x^3 - hx^2)}{E I_{DA} \left( \frac{x}{2} \right)} \]

**Member AB:**
\[ O = -R_A + w(x-a) - w(x-a) \]

\[ M = -R_A x + w(x-a) - 0.5w(x-a)^2 \]

\[ y = \frac{1}{E I_{AB}} \left[ \left( -aL \frac{(x^3 + bL^2)}{6} + \frac{(x^3 + h^2)}{6} + \frac{(x^3 + h^2)}{3} \right) \right] \]

**Member BC:**
\[ T = R_C \]

\[ Q = H_B \]

\[ M = -R_A l + 0.5w(L-a)^2 - 0.5w(L-a)^2 + H_C x \]

\[ y = \frac{1}{E I_{BC}} \left[ \left( -aL \frac{(x^3 + bL^2)}{6} + \frac{(x^3 + h^2)}{6} + \frac{(x^3 + h^2)}{3} \right) \right] \]

**Discussion on papers**

Written discussion on the papers in this issue will be accepted until 15 November 1982. This, together with the authors' replies, 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 conform to the requirements laid down in the 'Notes for the Guidance of Authors and Contributors' as published in the September 1971 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.