The dams of the Tugela-Vaal Project

H F W K Elges (Member)*

Synopsis
The construction of the first and second phases of the Tugela-Vaal Project, which includes the Drakensberg Pumped Storage Scheme, is nearing completion. The four major reservoirs forming part of this project are the Drielkoof, Kilburn, Woodstock and Sterkfontein Dams.

The paper describes some interesting engineering features of these four structures. In the construction of the Drielkoof Dam a soft sandstone was used as fill material and a baffled apron spillway was placed centrally over the embankment. In the case of the Kilburn Dam the available materials in conjunction with the rapidly varying water levels necessitated a rather flat slope on the Woodstock Dam and the design of the spillway consisting of an uncontrolled curved ogee overflow, chute and energy dissipator incorporated some novel ideas. A short description of the Sterkfontein embankment concludes the paper.

Somewatching
Die bou van die eerste en tweede fases van die Tugela-Vaal Projek, wat die Drakensberg Pomppozoskema insluit, nader voltooiing. Die vier vernaamste opgaarsehede wat deel uitmaak van hierdie projek is die Drielkoof-, die Kilburn-, die Woodstock- en die Sterkfonteindamme.

Hierdie verhandeling beskryf enkele interessante ingenieursaspekte van hierdie vier structure. ‘n Sage sandsteen is vir die bou van die Drielkoofdam gebruik en ‘n bufferblok oorloop is in die middel oor die waal geplaas. In die geval van die Kilburndam het die beskikbare materiaal en die vinnig varierende watervlak ’n plat strooimop helling genooodsaak.

‘n Tonne moes vir rivierverleggingsdoeleindes in die geval van die Woodstockdam gebo woorde. Daarbenewens is enkele nuwe gedagtes in die ontwerp van die vloëdbuurtboel bestaande uit ‘n gekromde, ongekoppelde ogee-oorloop, gout en energieverniegter opgeeneem. Ten slotte word ‘n kort beskrywing van die Sterkfonteindam bewys.

Introduction
The four major storage reservoirs of the Tugela-Vaal Project are the Drielkoof, Kilburn, Woodstock and Sterkfontein Dams. Each of these units has a specific function and some engineering problems which had to be overcome during design and construction. Some of the more interesting aspects of each of these dams are described in this paper.

Drielkoof Dam
Drielkoof Dam, the upper reservoir of the Drakensberg Pumped Storage Development, is located in an arm of the Sterkfontein basin. As such it will therefore be subjected to submergence up to 0.44 m below the non-overflow crest from time to time. Pumping from the Tugela River will first fill the Drielkoof Dam and the water for inter-branch transfer will flow over the spillway into the Sterkfontein Dam. There will be a rapid fluctuation of water levels up to 22 m below full supply in the Drielkoof reservoir over a weekly cycle of pumping and generation of electricity.

The dam is of the rockfill type with a central clay core. The spillway is a baffled apron placed centrally on the embankment (see photographs 1 and 2).

Main Data

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<td>Spillway crest</td>
<td>RL 2 700.0 m</td>
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<td>Maximum operating water level</td>
<td>RL 1 702.0 m</td>
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H F W K Elges was born in Estcourt in 1939 and matriculated at the Hermannsburg High School. He obtained the BSc degree in civil engineering from the University of Pretoria in 1962 and joined the then Department of Water Affairs. After working in the Design, Construction and Planning Divisions of the Department he undertook a work-study tour of the United States in 1976, spending some time with the US Army Corps of Engineers, the Bureau of Reclamation and a consulting firm concentrating on the design and construction of earth and rockfill dams. Since then he has been at the head of the earth and rockfill dams design team of the Department in his capacity as Assistant Chief Engineer (Design).

Minimum operating water level:
(a) normal RL 1 684.8 m
(b) emergency RL 1 680.0 m

The embankment
The commonest rock type underlying the site is a red-brown mudstone which is extensively slickensided. It slakes very rapidly and when saturated can be moulded in the hand. Layers of siltstone and sandstone, also of the Upper Beaufort Series, occur at greater depths.

The red-brown mudstone is particularly weak for a rock and will deform appreciably at relatively low stresses. Unfortunately there are thick layers of mudstone underlying the siltstones with the result that the possibility of large scale deep failure caused concern in the case of high stresses.

The selected site and a rockfill dam with a baffled apron spillway proved to be the most economic solution. The materials that were available for the construction of the embankment had to be evaluated with great care.

Weathered red-brown mudstones. The weathered red brown mudstones occurred within 2 km of the site and proved to be suitable for the impervious core of the dam.

Fresh dolerite. The fresh dolerite constituted the best fill material available for the upstream and downstream zones of the embankment. Owing to the fact that the nearest quarry was about 22 km from the site, the use of the fresh dolerites was limited to the most critical sections of the dam, viz the range of operating levels on the upstream slope. The fresh dolerite was further used as filter material and for slope protection.

Weathered dolerite. Due to the relative ease of borrowing, placing and compaction the weathered dolerite deposits were investigated as well. They were ultimately discarded because the deposits were small, though numerous, and haulage and handling costs therefore too high.

Hard sandstone. The use of the hard Upper Beaufort sandstones proved to be uneconomic due to the distance from the site and the excessive overburden covering these formations. Although these sandstones comply with the strength requirements for a rockfill, their durability has not been proven conclusively.

Soft sandstone. The soft Lower Molteno sandstone occurs within 1.5 km of the site and the overburden was minimal. Although these sandstones broke up completely during the quarrying, handling, placing and compaction operations, the shear strength of the compacted material was comparable with that of a hard rockfill though the permeability was considerably lower, viz 5 \times 10^{-8} to 1 \times 10^{-4} cm/s in comparison with 1 \times 10^{-2} to 1 \times 10^{-1} cm/s for normal hard rockfill.

The suitability of the soft sandstones as a fill material was further investigated and proven in the laboratory as well as in the field by the construction of a trial embankment. The placement and construction procedures were also finalized during these trials.

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The final design of the embankment and the zones where the different materials were used are shown in Fig 1 and 2. Placement and compaction control during construction was strict in order to minimize settlement of the embankment especially along the spillway section. Settlements were predicted utilizing the finite element programme ISBUILD. Subsequent settlement measurements have shown the predicted values to be of the right magnitude. Photograph 3 shows the embankment during construction.

The spillway

The run-off from the Driekloof catchment is insignificant and does not require a spillway. But, as the secondary function of the dam is to convey the water for the inter-basin transfer to the Sterkfontein basin, Driekloof had to be provided with a spillway. The solution adopted after extensive investigations is a baffled apron spillway centrally placed on the embankment as illustrated in Figs 1, 3 and 4, and photographs 4 and 5. The spillway has to perform a dual function, viz:

1. To convey water being pumped from the Tugela River for the inter-basin transfer from Driekloof to Sterkfontein. The maximum dis-

charge of water over the spillway in this direction is 200 m³/s.

2. To convey 312 m³/s from Sterkfontein to Driekloof when water levels in the Driekloof reservoir drop below the spillway crest during power generating cycles and when the Sterkfontein Dam is at full supply level.

Alternative methods which were investigated for conveying water from Driekloof to Sterkfontein included a side channel spillway on either
The right or left abutments, a 'morning glory' or shaft spillway, a central concrete gravity section with an ogee crest as spillway and a gate controlled conduit through the Driekloof embankment. A concrete dam with a central ogee spillway section was considered as well.

Although feasible, all these alternatives proved to be far more costly than the baffled apron spillway, mainly owing to the poor foundation conditions and the elaborate energy dissipation arrangements required. A literature survey revealed that baffled apron spillways had been model tested for slopes of 1 vertical to 2 horizontal and flatter and for a maximum drop in elevation of 16.76 m.

Model tests were therefore undertaken in the Directorate of Water Affairs' Hydraulics Laboratory to determine the effectiveness of this type of spillway on a slope of 1:1.6 and to assess the scouring pattern at the toe of a spillway chute dropping vertically through 42 m.

The steeper slope necessitated a closer spacing of the baffles to obtain the same flow conditions recorded with the flatter slope. In addition the scour depths were marginally deeper for the steeper slope. Under these conditions the flatter slope of 1:2 proved to be both more practicable and economic and was therefore adopted.

To provide the necessary resistance against sliding of the baffled apron slabs, precast concrete beams were placed horizontally into the rockfill during the fill operation. The slabs were then cast in situ on a blending layer after the completion of the fill and after allowing for initial settlement of the embankment and the foundations. Subsequently the baffles were cast in place as well.

Energy dissipation for flows from Sterkfontein to Driekloof presented far less of a problem even though the maximum flow would be 312 m³/s. The lowest operating level for the Driekloof Dam under normal conditions is RL 1 684.8 m and in case of emergency RL 1 680 m. Hydraulic model tests confirmed that it was sufficient to line the rockfill slope from spillway level at RL 1 700 m to the level of the 25 m berm at RL 1 675 m with concrete. The minimum water depth of 10 m on the berm proved to be sufficient to absorb all the energy and to protect the structure against erosion.

**Operational experience**

The Driekloof Dam was completed in 1979 and has been in operation since. Thus far the performance of the embankment and the operation of this spillway has been as predicted during the design of the dam.

**Kilburn Dam**

The Kilburn Dam is located in the foothills of the escarpment and forms the lower reservoir of the pumped storage development. Water for the inter-basin transfer is pumped into the Kilburn Dam from the Jagersrust forebay. Utilizing the pump-turbine station this additional water is transferred to the Sterkfontein reservoir. This operation will fill and empty the Kilburn reservoir over a weekly cycle thus causing a rapid fluctuation of water level over a height of 21 m.

**Main data**

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<td>Type of spillway</td>
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<td>Full supply level</td>
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<td>Lowest operational water level</td>
<td>RL 1 235.0 m</td>
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**The embankment**

The foundation on the right bank is formed by a wide alluvial plain dropping abruptly down into the river section. The overburden varies from 1.5 m to 12 m. In the river section up to 6 m of alluvial cobbles, gravel and sand overlie the rock formations. The overburden on the left flank consisted of residual soils from 1 m to 6 m deep. The solid geology is formed by a flat lying sedimentary sequence of alternating and intermingled sandstones, siltstones and mudstones.

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Fig 5: Kilburn Dam: plan

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Fig 6: Kilburn Dam: cross-section of embankment
3. The embankment during construction showing:
   (a) the upstream dolerite rockfill
   (b) the central core
   (c) the soft sandstone zone
   (d) the precast concrete beams embedded in the fill in the overspill section

Additional photos on p 383
In the river section the overburden was removed and the whole embankment was founded on rock. The outlet conduit was placed in this section as well. On the flanks the topsoil was removed over the total width of the embankment. The cut-off trench was excavated down to rock on the left flank and a 2 m deep cut-off was built on the right flank. Curtain grouting was done in the river section and along the left flank.

All the materials used for the construction of the embankment were borrowed in the basin of the dam. These consisted of the alluvium on the right bank of the river, weathered mudstones, shales and sandstones as well as moderately weathered shales. The clay materials were used in the core of the dam whilst the soils with the highest sand content were used in the upstream zone. The intermediate materials and the moderately weathered shales were placed in the downstream portion.

Due to the rapid fluctuation in water levels and the relatively impervious nature of the sandy materials rather flat slopes were required on the upstream face of the embankment. The upstream slope was protected with dumped dolerite rip rap from a quarry approximately 20 km from the site. The downstream slope was grassed for protection against erosion. Figs 5 and 6 show details of the embankment as built.

The spillway

The estimated maximum flood discharge from the Kilburn catchment is smaller than the discharge of 312 m³/s from the pump-turbine station in the generating mode. The spillway was therefore designed to pass the 312 m³/s with normal freeboard from the station during extended emergency generation cycles.

Topographic, economic and foundation conditions dictated a side channel chute spillway with an energy dissipating device at the downstream end. Severe erosion stretching to the downstream toe of the dam could not be avoided with a flip bucket type energy dissipator. A hydraulic jump stilling basin was therefore designed and model tested.

To minimize the length of the basin baffles were inserted. This arrangement proved to be satisfactory and was adopted as such (see photograph 6).

Water being pumped from the Jagersrust forebay is released into a baffled outlet on the crest of the Kilburn embankment and down a baffled apron drop into the dam as shown in photograph 7.

Operational experience

The dam has been in operation since 1980 and apart from some seepage along the outlet conduit performance has been satisfactory. The seepage was successfully stopped by grouting.

Woodstock Dam

The Woodstock Dam which is being built on the Tugela River to regulate the flow of the Upper Tugela for pumping to Sterkfontein Dam is nearing completion. Water from the dam will be released into the basin of the Driel Barrage some 5 km downstream from where it will be pumped into the main Tugela-Vaal canal which will convey the water to the Jagersrust forebay.

The dam consists of an earth embankment with an outlet tunnel and a chute spillway on the left flank. The general layout of the dam is shown in Figs 7 and 8.

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<td>Maximum discharge of spillway</td>
<td>2 x 500 m³/s</td>
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Type of spillway

One controlled tunnel outlet and one uncontrolled chute spillway

The embankment

The foundations of the embankment consist of layers of sandstone, silstone and mudstone. On the left flank a layer of alluvium covered the area. The embankment was founded on the overburden with the top soil removed. The cut-off trench below the core of the dam was taken down to rock.

Single row curtain grouting was done beneath the core trench along the entire length of the dam. Normal curtain grouting procedures proved successful on both flanks. The river section on the other hand presented some problems. Further geological investigations revealed the presence of slickensided/micro-fractured and rapid slaking clays as well as the occurrence of micaceous bedding planes within the mudstones and some of the siltstones.12

Two alternative methods of sealing these fractures were considered. Firstly, a chemical grout which would penetrate and thereby seal the seepage paths or, secondly, a cement grout under high pressure opening up the fractures and consolidating the rock mass thus achieving closure were investigated. The second alternative was adopted for economic reasons and to save time during construction. Great care was taken during the grouting process not to cause any heaving of the rock mass. Subsequent water pressure tests have shown that closure has been obtained.

The zoned embankment was constructed utilizing the silty and sandy alluvial deposits and the in situ weathered materials occurring in the basin of the dam. Slope protection on the upstream slope was achieved using dumped rip rap and the downstream slope was grassed to combat erosion.

Because of the severity of the flash floods in the Drakensberg area a 25 m high cofferdam with a 5,5 m × 6,2 m horseshoe-shaped diversion tunnel was constructed. The initial capacity of this river diversion with the crest of the coffer dam at RL 1 156 m was 340 m³/s, which corresponds to an incoming flood of 1 300 m³/s. The cofferdam was incorporated in the upstream toe of the embankment.

The spillway

For the design flood the total spillway capacity required is 1 000 m³/s, of which 500 m³/s can be discharged through the tunnel. An additional spillway with a capacity of 500 m³/s had therefore to be built. The spillway was placed on the left flank and the original design consisted of a straight ogee crest, a converging channel and a 15 m wide chute discharging the water back into the river downstream of the dam. Hydraulic model tests13 however revealed that the waves formed in the chute were unacceptable.

After considerable redesign and experimentation with the model the solution was found. It consisted of a curved ogee spillway section, a transition zone with a floor elevated along the centreline and an 11 m wide chute. The flow conditions obtained were satisfactory and even under probable maximum flood conditions there was no submergence of the overflow crest. The ogee crest and transition as modelled are shown in photograph 8.

The energy dissipating device at the end of the chute is of the flip bucket type. The design of the bucket is such that, in addition to serving its primary purpose of energy dissipation, it will also tend to divert the direction of discharge in the chute, which is practically at right angles to the centreline of the river, towards the direction of flow in the river. The model is shown on photographs 9 and 10.

An auxiliary spillway to handle floods up to and including the probable maximum flood of 2 730 m³/s at the dam wall has also been provided on the far left flank adjacent to and beyond the chute spillway. The energy of the water being released through the tunnel is dissipated utilizing a standard hydraulic jump stilling basin. The tunnel is also being used for releases of water from the dam.

Progress on work

The design and construction of this dam posed some interesting and challenging problems. Water is already being stored in the dam as construction is nearing completion. The performance of the prototype will be watched with great interest in order to compare it with the results obtained in the model tests and with the predictions made.

Sterkfontein Dam

Situated on the Nuwejaarspruit a few kilometres from the edge of the Drakensberg escarpment, Sterkfontein Dam will on completion of its final stage be the largest dam in South Africa in respect of the volume of material in the wall. It will also be the only dam in the Republic so far to qualify for inclusion in the ICOLD Register of the World’s Largest Dams (see photographs 11 and 12).

Main data

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![Fig 9: Sterkfontein Dam: plan](image1)

![Fig 10: Sterkfontein Dam: cross-section through embankment](image2)
7. Kilburn Dam: baffled outlet and baffled apron drop

8. Woodstock Dam: model of ogee spillway and transition

9. Woodstock Dam: model of flip bucket energy dissipator

10. Woodstock Dam: model of flip bucket with discharge equivalent to 500 m/s

11. Sterkfontein Dam: aerial view

12. Sterkfontein Dam: raising of embankment
The embankment

The embankment is a typical earthfill design with an impervious core sloping upstream as shown in Figs 9 and 10. The foundations consist mainly of mudstones, shale, sandstone and some dolerite. Most of the fill material consists of weathered mudstones and shales with some weathered dolerite in the downstream zone of the embankment. Foundation treatment consisted of a single row grout curtain placed centrally in the cut-off trench.

The main challenge on this site was to make use of the fill materials which were quite variable. This gave rise to the design of a rather flat upstream slope.

Conclusion

The dams on the Tugela-Vaal Project posed some very interesting problems to both design and construction engineers. Indications so far are that the structures perform essentially as predicted and that the challenges have been met successfully.

Acknowledgements

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References