

A GEOHYDROLOGICAL INVESTIGATION AT  
CAROLUSPOORT, DE AAR FOR THE  
SOUTH AFRICAN RAILWAYS.

by

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## I. INTRODUCTION

In a report dated 24 October 1958 on the possible augmentation of the De Aar water supply from local sources, J.P. Kriel of the Department of Water Affairs recommended, inter alia, a geohydrological investigation of Caroluspoort. This investigation is reported in the following pages.

## II. PREVIOUS INVESTIGATIONS AND REPORTS BY THE GEOLOGICAL SURVEY

At the request of the Chief Civil Engineer, S.A. Railways, two bore-hole sites were selected on railway land at Caroluspoort in May 1948. Only small supplies were struck in bore-holes on these two sites as well as in a third one indicated by the driller (see table 3).

In April-May 1954 another investigation was made as a result of which four wells were constructed in the alluvial deposits. Three of them yield large supplies of water; the fourth was a failure (see table 4). Later in 1959 two more wells were constructed. The drilling of shallow observation bore-holes near the wells and pumping tests were recommended to determine the hydrological properties of the alluvial deposits. Two such tests were made by the Railways in February and May 1957, but unfortunately owing to insufficient and inaccurate water-level measurements no reliable calculations of permeability and storage could be made.

A reconnaissance of possible sources of supply in the vicinity of De Aar was made in February 1958 by representatives of the Railways, the Department of Water Affairs and the Geological Survey.

Knowledge of the following reports is assumed:-

- (a) Report on bore-hole sites by J.R. Vegter dated 29 May 1948.
- (b) Report on well sites by J.R. Vegter dated 14 June 1954.
- (c) "Verslag oor verkenningsopname van grondwaterbronne in die omgewing van De Aar" by J.R. Vegter and F.W. Schumann dated 30 May/6 June 1958.
- (d) "De Aar water supplies:- Possible augmentation of supply from local sources" by J.P. Kriel, Department of Water Affairs, dated 24 October 1958.

### III. THE PRESENT INVESTIGATION

The fieldwork was done by Messrs. M.E. Hauger and A.C.W. Mew, Junior Technical Officers under the supervision of Messrs. F.W. Schumann and J.R. Vegter, Chief Geologists, during the period 1 April to 4 December 1959. Supplementary work was done from 1-13 June 1960 and 1-23 December 1960. During the latter period Mr. A.P.C. Nieuwoudt, Junior Geologist, carried out a seismic refraction survey. The regional office at Prieska is responsible for the monthly servicing of water stage recorders and the measuring of water levels in bore-holes - work which is continuing except on land belonging to Mr. J.H. van der Merwe. Messrs. J.R. Vegter and M.E. Hauger were responsible for working up the data and making the calculations.

Fieldwork comprised the following:-

- (1) Forty seven bore-holes were drilled by the Railways with a percussion drill to determine the thickness and nature of the alluvial deposits.
- (2) Six bore-holes were drilled with a small diamond drill by the Soil Mechanics Laboratory of the S.A. Railways to obtain undisturbed samples for permeability measurements. Determination of the coefficients of permeability by the S.A.R. is gratefully acknowledged.
- (3) Twenty eight auger-holes were drilled for permeability measurements and water-level observations. Percolation tests were also made in some of the bore-holes.
- (4) Over 500 electrical depth-probes were taken to determine the thickness of the alluvium. In addition some 90 shallow seismic refraction lines were run.
- (5) Fifteen bore-holes were logged electrically.
- (6) About 500 magnetometer stations were read to locate, if present, dolerite intrusions.
- (7) Geological mapping was done and surface and bore-hole collar elevations were determined.
- (8) Over the last two years bore-hole water levels have been observed regularly. Water-level recorders have been erected on six holes.
- (9) Extensive water level measurements were made during two pumping tests to determine hydrological properties of the alluvial deposits. Fluorescein and salt were used to determine the velocity of flow between bore-holes during the second test. Assistance given by the Radioactivity section, N.P.L., C.S.I.R. in determining fluorescein content is gratefully acknowledged.

- (10) Moisture desorption determinations on eight samples were kindly made by Mr. C.M.A. de Bruijn of the National Building Research Institute.
- (11) Infiltration capacity was measured in small ponds constructed for that purpose. The water levels in a conservation dam and behind four contour-embankments were also observed.
- (12) Effluent ground-water flow was measured at two points in the Brak River by means of V-notches.

#### IV. PHYSIOGRAPHIC FEATURES AND GEOLOGY

These have been dealt with in previous reports (a) and (b) mentioned in chapter II. Although the present investigation covers a larger area than previously described, a good picture can be formed from plans G.132/61 A+B which show the geology, surface contours and eroded drainage lines.

#### V. NATURE AND THICKNESS OF THE ALLUVIAL DEPOSITS

The thickness of the deposits was determined by electrical depth-probes, seismic refraction lines and bore-holes.

Save for the dry surface layer, the alluvium is characterised by resistivities of between 2,000-5,000 ohm cm. The resistivity of the underlying Beaufort shale ranges from 5,000 to over 10,000 ohm cm. According to bore-hole data, the electrical depth determinations may be in error by as much as plus or minus 10 ft.

For this reason and owing to difficulties of interpreting electrical depth-probe curves, some seismic refraction profiles (of an experimental nature) were made. Seismic velocities in the alluvium are:-

upper (surface) layer	1,100-1,500 ft./sec.
lower layer	2,500-5,000 ft./sec.

The shale is characterised by a velocity of between 6,000 to 13,000 ft./sec., lower velocities obtaining where the cover of alluvium is thin and the shale probably weathered. Generally speaking, seismic depths are somewhat greater than the true ones.

Bore-hole data are given in tables 1, 2 and 3. Based on the bore-hole data, electrical depth-probes and seismic refraction lines, contours of the floor of the alluvial deposits as shown on plan G.132/61B have been drawn.

Immediately east of the poort, adequate control is provided by bore-holes. However, further afield the contours are solely based on electrical and seismic soundings and only a general but not accurate idea can be obtained of the extent and thickness of the alluvium.

In table 2, descriptions are given of the alluvial material encountered in six bore-holes. The composition of a number of samples in terms of gravel, sand, silt and clay is shown in table 6. Two sieve analyses of fine sand and gravel which at one stage were thought as probably representative of the most permeable layers have been made (see table 5).

#### VI. LABORATORY AND FIELD DETERMINATIONS OF THE COEFFICIENT OF PERMEABILITY OF THE ALLUVIUM

As previously noted, six bore-holes were drilled by the Soil Mechanics Laboratory of the Railways to obtain undisturbed samples. The results of permeability determinations on them are given in table 6.

The coefficient of permeability of undisturbed samples of layers and lenses of the coarsest grained sediments exposed in the dongas was determined at the Geological Survey (see table 7). Reference to two sieve analyses of the coarse-grained material has already been made (table 5).

Although it was thought that these layers and lenses of coarse sediments are probably typical of those from which the high yields of the wells are obtained, it can be seen that the figures fall far short of those required to explain the high yields of the wells.

The results of precolation tests in auger-holes are summed up in table 8. The coefficients of permeability are higher than those determined on undisturbed samples. In a few instances it was noticed that after prolonged percolation of water out of the auger holes, heaving and cracking of the soil around the hole took place. This may account for the higher figures obtained with auger-holes.

Infiltration rates observed in small ponds - to be discussed in more detail in chapter XIII - however, compare very well with the permeabilities measured in shallow auger-holes (see table 13). As expected, infiltration rates are of the same order as the coefficient of permeability of the upper few feet of alluvium.

Moisture desorption determinations using the suction plate method in the  $pF^*$  range 2-3 were made by the Soil Mechanics Laboratory of the National Building Research Institute on eight disturbed samples taken at different depths. The results are given in

table 9. The data are of importance in estimating the storage capacity of the alluvium or, in other words, the extractable amount of water and will be discussed in more detail in chapter VIII.

\* The pF value is the logarithm of the soil suction measured in grams per square centimeter.

## VII. PUMPING TESTS

Two pumping tests were made on well G6778; the first one over the period 29 June to 4 July 1959 and the second from 13 to 18 December 1960. The decision to undertake a second test was made after the results of the permeability measurements by the Railways on undisturbed samples became available. The low values reported suggested that water was obtained from permeable layers, which could not be sampled, at or near the base of the alluvial deposits.

As the calculations according to the Thiem and Theis formulae yield average figures for the total thickness of saturated alluvium, it was thought that the coefficient of permeability of the more permeable layers could be calculated from the hydraulic gradient towards the well and the velocity of flow as indicated by fluorescein and salt. Before the second test, shallow auger-holes were drilled to just below the water level as it was expected that the water level in the auger-holes would recede very much more slowly during pumping than that in the bore-holes drilled down to the shale. Such an observation would corroborate the assumption of more permeable layers being present. Contrary to expectations this did not happen. The water-level in the auger-holes dropped just as fast as in the nearby bore-holes.

Water levels in the pumped well and observation holes prior to, during and after both tests are shown on plans G132/61F and G. From the water-level draw down the coefficients of storage and permeability have been calculated in several ways:-

(1) Assuming that the volume of dewatered alluvium is represented by the cone described by the water-levels in observation bore-holes surrounding well G6778, the percentage storage can be calculated at different stages of cone formation during the test.

(2) The direct inventory method by which storage is calculated as the difference in discharge through coaxial cylinders with the well as axis, was only used with the first test.

(3) The coefficient of permeability was calculated by Thiems' formula from the best fit straight line through the

points ( $h^2$ ,  $\log r$ ) where  $h = m - s$ , ( $m$  saturated thickness of aquifer,  $s$  draw down) and  $r$  distance from well. The values of  $h$  were those at the end of the test. The use of Thiem's formula presupposes equilibrium conditions. Although the rate of decline of the water level decreased towards the end of the tests, conditions of equilibrium were not attained.

(4) The coefficients of permeability and storage were also calculated according to Theis' non-equilibrium formula. Two methods of solution were used:-

(a) matching of curves ( $\log s$ ,  $\log r^2/t$  and ( $\log u$ ,  $\log W(u)$ ).

(b) that of Cooper and Jacob by which the most representative straight line through points ( $h^2$ ,  $\log r^2/t$ ) for values of  $r^2/t < 0.02$  is determined. ( $h = m - s$ ,  $m$  saturated thickness of aquifer before pumping started,  $s$  draw down,  $r$  distance from well,  $t$  time since pumping started,  $W(u)$  (well function) =  $\int_0^\infty \frac{e^{-u}}{u} du$ ).

In the second method, at least a partial correction for the reduction of the saturated thickness of the aquifer during the test is effected by plotting  $\log r^2/t$  against  $h^2$  instead of  $S$ .

The results of these calculations for both tests are given in tables 10 and 11.

Velocities of ground-water flow - obtained during the test by using fluorescein determinations by the National Physical Laboratories as tracer were as follows:-

(a) From bore-hole 40 to 45:-

Distance	45 ft
Mean Gradient	1:47.9
Time taken	1000 minutes
Velocity under unit gradient	2.2 ft/min.

(b) From bore-hole 40 to 44:-

Distance	74 ft
Mean gradient	1:25.8
Time taken	6000 minutes
Velocity under unit gradient	0.35 ft/min.

The times of arrival were taken when the first signs of fluorescein showed and not when the rate of increase of fluorescein content is the greatest, as is customary in the use of tracers. If the time of the latter were used, velocities would decrease, especially the first figure which would be reduced to a third.

The relationship between coefficient of permeability  $P$  and Velocity  $v$  is expressed by the equation:-  $P = vp$  where  $p$  is

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The relationship between coefficient of permeability  $P$  and Velocity  $v$  is expressed by the equation:-  $P = vp$  where  $p$  is

the effective porosity. The latter, as will be seen further on, is of the order of 10%. The coefficients of permeability corresponding to the above-mentioned velocities are therefore 0.22 and 0.035 ft/min.

The first figure seems to be of the correct order, though perhaps a bit low. The second is definitely too small compared with the coefficients of permeability calculated according to Thiem's and Theis' formulae. There is some uncertainty about the time at which fluorescein reached bore-hole 44., because of the scatter of values which are not appreciably above the limits of detection. A single relatively high value was, for instance, determined at 3000 minutes. Even if this is taken into account, the velocity is still much lower than expected. The question thus arises as to what extent the transmission of fluorescein has been retarded by absorption. The streamline joining bore-holes 40 and 44 may also have been a devious one. During the second test the velocity of flow was also determined with salt between bore-hole 45 and auger-hole 2, and bore-hole 44 and auger-hole 3. The auger-holes were drilled to 2.5 and 2.7 ft below the water level respectively. As previously stated, the rates of water-level drop in the auger-holes were the same as in the nearby situated bore-holes. The following results were obtained:-

(a)	From bore-hole 45 to auger-hole 2	
	Distance	6 ft
	Hydraulic gradient	1:19.4
	Time taken	428 minutes
	Velocity under unit gradient	0:27 ft/min.
(b)	From bore-hole 44 to auger-hole 3	
	Distance	6 ft
	Hydraulic gradient	1:37.5
	Time taken	650 minutes
	Velocity under unit gradient	0.35 ft/min.

Much higher velocities were expected because both the bore-holes and the auger-holes seem to have struck the more permeable layers. If this is not so, similar rates of draw down in the auger-holes and in the bore-holes, cannot be explained.

#### VIII. DISCUSSION OF THE VARIOUS DETERMINATIONS OF THE COEFFICIENTS OF STORAGE AND PERMEABILITY

The figures for the coefficient of storage calculated from the volume of the cone of depletion, increase with the period of pumping. This is to be expected because drainage is a gradual process. After a lengthy period of pumping the figures may be expected to approach a constant value. This seems to be the case with

the first test', where the constant value seems to be 0.07. Towards the end of the second test, the increase in the storage figure was still appreciable. Extrapolation of the curve of storage coefficient versus time leads to a constant value of about 0.11.

The different values obtained for the two tests may possibly be ascribed to the fact that different layers were being dewatered, because the water level at the start of the second test was roughly 1.8 ft lower than at the beginning of the first.

On the other hand consideration must be given to the fact that the pumping rate of the second test was nearly half that of first. During the first test, the rate of draw down was therefore more rapid than in the second. Accordingly, drainage of the alluvium within the cone of depletion would lag further behind in the first than in the second test. A higher figure for the storage coefficient is therefore only to be expected for the second test. The reason, however, why the storage coefficient still increases appreciably towards the end of the second test, especially when compared with the first test, is not clear.

If the water-levels in the observation holes do not reflect the water-table but rather the pressure head in permeable layer(s) draining alluvium of lesser permeability, then the volumes of dewatered alluvium will be smaller than calculated and the storage coefficients accordingly higher.

Too much weight should not be attached to the coefficients of storage calculated according to the non-equilibrium formula, especially the figure derived by the straight line solution for the second test. The value of  $s$ , the Storage coefficient, is extremely sensitive to small changes of slope of the best-fit straight line. Although the method of least squares was used for determining the most representative straight line, considerable doubt remains owing to the scatter of the points ( $h^2$ ,  $\log r^2/t$ ). Same difficulty was also experienced with the method of curve matching also because of scatter of points.

Reference must be made to the moisture desorption determinations on samples of alluvium in the pF range 2-3, because they yield a separate estimate of the coefficient of storage. The effective porosity or storage coefficient can be calculated by subtracting the moisture content of drained alluvium (expressed as % volume) from the porosity. This calculation was made for the eight samples listed in table 9. The results are given in table 12. Under conditions prevailing at Caroluspoort, the equilibrium moisture content above the water-level corresponds to a soil suction of about 1300 g/cm<sup>2</sup> i.e. apF of 3.1. (assuming depth 25 ft, bulk density 110 lb/ft<sup>3</sup> and ignoring capillary water).

The effective porosity at a  $pF$  of about 2.8 - 2.9 averages 11.9% when the results of the eight samples taken at various depths above and below the water-level <sup>are</sup> considered and 13.0% when only the five samples from below the water-level are taken into account.

In the light of the foregoing discussion, it was decided to adopt a figure of 0.10 for the coefficient of storage in estimating the stored water in the alluvium.

As regards the coefficient of permeability determined by means of Thiem's and Theis' formulae, it should be noted that the underlying assumption is one of a homogeneous aquifer. Attention has already been called to the low values - 100 - 1000 times smaller than those calculated from the pumping test draw downs - measured by the Railways on undisturbed samples. Coefficients for coarse sediments also fall far short. The figures obtained with Thiem's and Theis' formulae are therefore to be taken as effective averages.

At present neither the nature and disposition of the highly permeable portion of the alluvial deposits are known, nor is the mechanics of flow fully understood. Ground-water flow in the alluvium appears to be more complex than was previously thought.

In calculating ground-water flow through the alluvium the figure of 0.15 ft/min. will be used for the coefficient of permeability. This figure is about the average coefficient for the two tests, that for the straight line solution of the second test excluded.

#### IX. THE QUANTITY OF WATER STORED IN THE ALLUVIAL DEPOSITISTS

The volume of saturated alluvium was calculated from the water-level (September 1959) and bedrock contours. Results for different portions of the area underlain by alluvium are as follows:-

- (a) The poort section i.e. from the western S.A.R. boundary to just below main pumping station at the eastern end of the poort.  
.....34.6 x 10<sup>6</sup>ft<sup>3</sup>
- (b) The area east of the poort to Mr. J.H. van der Merwe's conservation dam in the south and the 80 ft water-level contour to the north of well G6778.  
.....380.7 x 10<sup>6</sup>ft<sup>3</sup>
- (c) The area south of the conservation dam indicated on plans G132/61A and B  
.....627.3 x 10<sup>6</sup>ft<sup>3</sup>.

With a storage coefficient of 0.10, the extractable supplies amount to (a) 22, (b) 240 and (c) 390 million gallons. The last figure is only a rough estimate as water-level and bed-

rock depths are not known accurately.

X. WATER-LEVEL FLUCTUATIONS OVER THE PERIOD APRIL  
1959 - JUNE 1961

Water-levels in a number of bore-holes over the period April 1959 - June 1961 are shown on plan G132/61I.

According to the rainfall records of Caroluspoort, De Aar and four other gauges in the catchment east of Caroluspoort, precipitation over the season July 1958 - June 1959 was roughly 70 mm. above the average. The following season the rainfall was about 50 mm below the average. Complete records for the season July 1960 - June 1961 are as yet not available. Rainfall up to April - May is, however, already considerably (90 mm) in excess of the average. The average is about 280 mm. p.a.

The water-levels fluctuate in sympathy with the seasonal rainfall. When measurements started in May - June 1959 water-levels were at their peak. Thence they declined until a low was reached in the first quarter of 1961. In bore-hole 50, the drop amounted to 3 ft. After the copious rains of March 1961, a considerable rise took place and with the exception of observation bore-holes near wells G6778 and G6779, levels generally above the May - June 1959 maxima, were reached. In the vicinity of well G6778 water-levels were still 0.25 to 1.0 ft short (in April 1961). Near well G6779 the short fall is about 0.25 ft.

XI. THE GROUND-WATER INVENTORY FOR THE PERIOD  
10 JULY TO 20 OCTOBER 1959

As facilities for a complete study of the hydrological cycle were not available, the above-mentioned period was selected. The reasons for selecting this period are firstly that with the exception of 12 mm. on the 7th July, no rain occurred from 6 June until 20 October 1959. The assumption that little or no replenishment occurred from direct infiltration of rainfall during the period under consideration seems reasonable. Secondly there was no surface flow, except effluent seepage, which could have caused replenishment.

The area for which the inventory was drawn up extends from a line across the poort below the main pumping station and wells No's 4, 5 and 6, to the conservation dam in the south and the 75 ft water-level contour in the north. (see plan G132/61C for outline of area).

The general storage equation for an aquifer is

$$F + R_s + R_u + R_L + R_w = E + D_s + D_u + D_L + D_w + \Delta S$$

where F = recharge from infiltration

$R_s$  = recharge from surface bodies of water  
 $R_u$  = recharge from lateral underflow  
 $R_L$  = recharge by leakage through an aquiclude  
 $R_w$  = recharge by wells, trenches or other un-  
 filtration devices.

and

$E$  = discharge by evapo-transpiration  
 $D_s$  = discharge to surface bodies of water  
 $D_u$  = discharge by lateral underflow  
 $D_L$  = discharge by leakage through an aquiclude  
 $D_w$  = Pumpage from wells etc.  
 $\Delta S$  = change in storage volume.

Let us consider each item separately:-

- (1)  $F$ :- As already pointed out above  $F$  is taken as nil.
- (2)  $R_s$ :- The levels of the four small water bodies behind conservation embankments were observed regularly. The drop in level corrected for evaporation, is infiltration. Infiltration is estimated at 1.3 million gallons. Figures of the Class A evaporation pan at De Aar were used with a conversion factor of 0.67.
- (3)  $R_u$ :- Ground-water enters the area through the alluvial deposits and the Beaufort shale. Consider the alluvium first. The cross-sectional area of water-bearing alluvium just below the conservation dam of Mr. J.H. van der Merwe is 33,000 sq. feet. With a hydraulic gradient of 1:420 the flow through the cross-section was calculated to be 105,000 gallons per day. The cross-sectional area of water-bearing alluvium northeast of well G6778 is 43,000 sq. feet. The hydraulic gradient is 1:210 and the flow across the section (taking the coefficient of permeability as 0.15 ft/min) was calculated at 268,000 gallons per day.

Recharge will also take place by ground-water flow through the shale underlying the alluvial deposits and between the two cross-sections of alluvium just considered. The coefficient of permeability of the shale is not known. According to several bore-hole results the coefficient is of the order of  $10^{-4}$  to  $10^{-3}$  ft/min, for the first hundred feet below the surface. Beyond that depth, very little flow if any takes place. Taking the coefficient of permeability as  $5 \times 10^{-4}$  ft/min, and the hydraulic gradient as 1:300, the daily flow through the shale is estimated at 14,000 g.p.d.

- (4)  $R_L$ :- Nil.
- (5)  $R_w$ :- Nil.
- (6)  $E$ :- This could not be measured and is very difficult to estimate. The greater part of the evapotranspiration loss takes place in the beds of the dongas where there is open water or where the water table is at the surface and where vegetation (reeds and poplar trees) is present. The area thus involved comprises  $4.2 \times 10^5$  sq. feet and assuming that evapo-transpiration equals evaporation from an open water body, the total loss over the period is 4.1 million gallons. (Evaporation from Class A pan at De Aar over the period is 28.1 inches. The conversion factor to open water evaporation is taken as 0.67) Evapo-transpiration losses away from the dongas are taken to be negligible.

- (7)  $D_s$ :- Effluent seepage was measured downstream of the main pumping station with a V-notch. The flow was as follows:-
- |              |            |         |          |
|--------------|------------|---------|----------|
| 10-31 July   | 3.3        | million | gallons. |
| August       | 5.9        | "       | "        |
| September    | 2.7        | "       | "        |
| 1-20 October | <u>0.7</u> | "       | "        |
| Total        | 12.6       | "       | "        |
- (8)  $D_u$ :- The cross-sectional area of the water bearing alluvium downstream of the main pumping station is 5,500 sq. ft. The gradient is 1:315 and taking the coefficient of permeability as 0.15 ft/min, loss by lateral underflow through the alluvium is calculated to be 23,500 gallons per day. Flow through the shale need not be considered, as it is presumably very little.
- (9)  $D_L$ :- Nil
- (10)  $D_w$ :- Pumpage over the whole period from wells G6778, G6779 and 4 and of effluent seepage in the Stoffpoort tributary amounts to 16.4 million gallons.
- (11)  $\Delta S$ :- From the drop in water-levels of various bore-holes over the whole area, the decrease in storage is estimated at 6 million gallons.

The inventory can now be drawn up:-

Gains:-

F	0.0	m.g.
$R_s$	1.3	"
$R_u$ 387,000 g.p.d. for 102 days	39.5	"
$R_L$	0.0	"
$R_w$	0.0	"
Total	<u>40.8</u>	"

Losses:-

E	4.1	m.g.
$D_s$	12.6	"
$D_u$ $2.35 \times 10^3$ g.p.d. for 102 days	2.4	"
$D_L$	0.0	"
$D_w$	16.4	"
Total	<u>35.5</u>	"

As the storage decreased by 6 million gallons, there is a discrepancy of 11.3 m.g. Either the gains have been over-estimated or the losses under-estimated. Balance can be achieved by either increasing E or decreasing  $R_u$ . (Items  $D_s$  and  $D_L$  have been measured and as  $R_s$  and  $D_u$  are relatively small even large errors in their estimation, will only have a small effect).

If the average coefficient of permeability is reduced to 0.10 ft/min. gains and losses will balance. Ground-water inflow would then amount to about 265,000 gallons per day. Should evapo-transpiration losses be nil, which they obviously cannot be, balance

of the inventory requires a ground-water inflow of 225,000 g.p.d.

It would thus appear that ground-water inflow during the period for which the inventory was drawn up, was of the order of a quarter million gallons per day. The flow will be reduced as the water-table drops. The water-level in bore-hole 51 fluctuated nearby 2 feet during the two years of observation. A drop of 2 feet along the whole northern section would mean a reduction of about 13% of the inflow. Indications are that the drop of the water-table along the section below the conservation dam, did not exceed 1 ft, which means a reduction of only about 7% of the inflow.

## XII. AVAILABLE GROUND-WATER

The maximum quantity of ground-water available, if evapotranspiration losses and underground outflow could be obviated and all the effluent seepage salvaged, would be equal to the ground-water inflow plus replenishment from direct infiltration of rainfall and of floodwaters. The following discussion deals again only with the area east of the poort to the 75 ft. water-level contour and southwards to the wall of the conservation dam.

As inflow and outflow of runoff could not be measured as a number of gauging weirs would have <sup>had</sup> to be built and as effluent seepage was only measured for a few months, a ground-water inventory for the last two years from which replenishment could have been estimated can not be drawn up. This was stated previously. There are, however, indications that replenishment by infiltration of rainfall and flood waters is of lesser importance compared with the inflow of ground-water through the alluvial deposits.

Consider March 1961, when rainfall amounted to nearly 150 mm. - about half the annual average. It is estimated from the rise of the water-level as measured in April 1961, that storage increased by about 11 million gallons. Judging by the above inventory, no great error will be made if it is assumed that pumpage and evapotranspiration losses were more or less balanced by subsurface inflow during the period March - April in which recharge took place. Flow observations during the period June - October 1959, can be used to estimate the loss of ground-water by effluent seepage during the same period, that is before water-levels reached <sup>5</sup> their peak by middle April. The loss is probably of the order of 5 million gallons. During March - April 1961 replenishment by rainfall and flood waters was therefore of the order of 16 million gallons.

With the high rainfall over a short period, conditions for replenishment were at their optimum. On the average, one can expect that replenishment will be much less, for an equal rainfall over a longer period. Therefore, although proof by actual

measurement is lacking, one can safely assume that annual replenishment by infiltration from rainfall and floodwaters, under present conditions, does not exceed 30 million gallons and is most likely considerably lower on the average. Compared to this, the subsurface inflow at the relatively low rate of 200,000 gallons per day amounts to 73 million gallons per annum.

### XIII. INFILTRATION EXPERIMENTS WITH PONDS AND GROUND-WATER RECHARGE BY DAMS

Unless recharge can be improved artificially, the maximum ground-water supply in all probability will not exceed 350,000 g.p.d., that is if all the losses are salvaged.

It was for this reason that some attention was given to the question of artificial recharge. Firstly, the levels of water contained by conservation embankments were observed over a period of several months. The areas of the water bodies of which these were six, range from 0.06 to 3.1 morgen. The drop in level was corrected for evaporation loss using figures of a Class A evaporation pan at De Aar (10 miles away) and a conversion factor of 0.67, as previously mentioned. The following infiltration rates were thus obtained:-

Gauge A	0.011 ft/day	or	$7.3 \times 10^{-6}$	ft/min
" B	0.010 "	" "	$7.0 \times 10^{-6}$	" "
" C	0.004 "	" "	$2.8 \times 10^{-6}$	" "
" D	0.023 "	" "	$1.6 \times 10^{-5}$	" "

For positions of the gauges (and the water bodies) see plan G132/61C. The drop in level of two remaining bodies contained in the two stream channels behind the conservation dam could be accounted for completely by evaporation.

These figures are not very encouraging and show that artificial recharge by constructing dams is not efficient.

Infiltration experiments were conducted in a number of ponds 10 x 10 ft which were covered by 9 inches of water. The time for the water to seep away was observed. Clear water from well G6778 was used and the tests were repeated a number of times. The results are summarized in table 13. It is important to note that the floors of the ponds were natural undisturbed surfaces. As a continuous supply of water was not available, the ponds could not be kept flooded for a lengthy period and each test had to be limited to one day. In general, infiltration rates, decreased with each flooding. A hole down dug alongside pond 6 showed that at a depth of about ten feet, soil had been wetted to a distance of 5 feet away from the edge of the pond.

Roughly, the infiltration figures are about a 1000 times larger than for the water-bodies contained by conservation embankments, where silt is being deposited and the surface is under water for at least several months.

It is evident that artificial recharge can only be achieved effectively by a system of floodwater spreading whereby the natural veld is submerged for very short periods only, so that vegetation is not killed. The newly laid-down silt deposit must become part of the soil through drying out, and the action of soil organisms and vegetation.

#### XIV. NOTE ON THE QUALITY OF THE WATER

Recent analyses of the water from Caroluspoot are not available. Samples were collected and dispatched but unfortunately became mixed up and damaged in transit. In report (a) Chapter II, two analyses are given which were made in 1946 and 1948. As far as is known water from Caroluspoot is suitable for railway purposes and amenable to treatment. The Railway Administration presumably has sufficient information at its disposal about this aspect.

#### XV. CONCLUSIONS AND RECOMMENDATIONS

The supply obtainable from the area east of Caroluspoot (i.e. the area for which the inventory has been drawn up and as delineated on plan G132/61C) is limited to the subsurface inflow and the replenishment from rainfall and runoff. Subsurface inflow under high watertable conditions (e.g. during the winter of 1959) has been calculated at 265,000 g.p.d. and under present conditions, replenishment from rainfall and runoff has been shown not to exceed 30 million gallons per annum (i.e. 82,000 g.p.d.).

The maximum obtainable supply can therefore not exceed 350,000 g.p.d. As both the average subsurface inflow and replenishment from rainfall and runoff will certainly be less than the figures given above it seems reasonable to assume that the maximum available supply, without embarking on a scheme of artificial recharge, is of the order of 250,000 - 300,000 g.p.d. These figures are based on the assumption that all ground-water losses by evapotranspiration, effluent seepage and subsurface outflow can be eliminated.

By cutting-off the latter i.e. the subsurface outflow to the alluvial deposits in the poort, the available supply from these deposits will theoretically be limited to the storage (about 22 million gallons). In practice however, the deposits can not be completely dewatered and ground-water is furthermore lost by evapotranspiration, and subsurface outflow. In addition, the depleted storage must be restored by replenishment from rainfall and runoff to the tune of 22 million gallons annually if the supply in the

poort is to be maintained. Without recourse to artificial replenishment this seems unlikely. A reasonable guess at the supply available from the poort alluvium may be 30,000 gallons per day (sub-surface inflow not taken into account, as stated above).

Since September 1960, pumpage from the wells and of the effluent seepage in the Stofpoort tributary has averaged just over 200,000 g.p.d. This quantity, at first sight, can easily be raised to 300,000 g.p.d. without enlarging the Railway land, by merely pumping more water from the wells. Two points are, however, to be noted:- Firstly, with the present arrangement of wells, loss of water by effluent seepage down the Brak River and by evapotranspiration can not be effectively decreased or eliminated. Secondly, there is the danger of locally depleting the supply near the wells, because ground-water flow to the wells may not be sufficient to establish and maintain equilibrium. This seems to be the case with well G6778, where in spite of the abundant rainfall during the 1960 - 61 season recovery to the May 1959 levels was not complete.

If the supply from Caroluspoort is to be stepped up to over 300,000 g.p.d., replenishment will have to be increased. In the first place, infiltration of floodwater can be increased through lowering the watertable along the river by pumping strategically placed wells. At the present time the watertable lies at the surface in the bed of the river.

Secondly, recourse will have to be taken to artificial recharge by floodwater spreading. As the mean annual runoff at Caroluspoort is estimated at 1,000 morgen feet (see report (d) Chapter II) sufficient water is available for this purpose and although infiltration of silt-laden water spread over a large area, will be considerably less than that measured in the ponds (table 13), recharge by spreading and doubling the available supply seems quite feasible. Economics and engineering considerations will of course have to be taken into account.

It would be undesirable to raise the watertable by more than 5 feet above the present highest level and even this may be too much, as effluent seepage will increase tremendously as well as evapotranspiration losses. (The area above the poort to the 80 ft. water-level contour in the north and to the conservation dam in the south is about 250 morgen in extent. A rise of 5 ft. of the watertable over this area would require recharge of 72 million gallons). The effluent seepage of course could be recovered for use, but unfortunately it is not a constant supply. To maintain effluent seepage at all times above requirements would mean large wastage of water. On the other hand if effluent seepage is to be obviated completely, water-levels will have to be drawn down consider-

ably by pumping wells, to accommodate the total natural and artificial recharge. This will raise pumping costs. A happy balance seems use of effluent seepage, when available, plus pumping. The storage position could be improved by silting-up of the dongas.

From this discussion it follows that additional land will have to be purchased to develop larger ground-water supplies. Although the alluvium continues upstream another 2 miles beyond the conservation dam, it is recommended that only the land underlain by alluvium up to the conservation dam be obtained. The dam itself, may be required to store floodwater for artificial recharge.

To make a better assessment of artificial recharge by floodwater-spreading a large scale experiment with silt-laden water is deemed necessary.

Should the Railway Administration decide to improve the underground supplies rather than go for surface storage, the Geological Survey would like to continue its study at Caroluspoort. For a proper study of the complete water cycle, gauging weirs will have to be built so that inflow and outflow of runoff can be measured. This could be undertaken in co-operation with the Department of Water Affairs.

Sites for additional wells have not been indicated. This can easily be done as further work is not required.

#### SUMMARY

By means of bore-holes and geophysical methods, the thickness of the alluvial deposits has been determined both on railway property and on private land east of the Caroluspoort proper. Bedrock contours (of the underlying Beaufort shale) are depicted on plan G132/61B. On the same plan are also shown water-level contours for September 1959. From both sets of contours the volume of water-bearing alluvium was calculated.

The coefficient of storage (or effective porosity) was calculated from water-level draw downs in observation wells during two pumping tests made on well G6778. Moisture desorption determinations were made by the National Building Research Institute on eight samples of the alluvial deposits, also with the view of determining the coefficient of storage. It was found that the storage coefficient is of the order of 10%. The quantity of water stored in the alluvium, was accordingly calculated to be (a) 22 million gallons in the poort, (b) 240 million gallons east of the poort to the 80 ft. water-level contour north of well G6778 and to the large conservation dam of Mr. J.H. van der Merwe to the south and (c) 390 million gallons south of the conservation dam as shown on plans

G132/61A and B. The last figure can only be taken as a very rough estimate.

The coefficient of permeability of the alluvium was also calculated from data of the two pumping tests. The average value is 0.15 ft/min.

A ground-water inventory for the period 10 July to 20 October 1959 for the area outlined on plan G132/61C shows that this value is too high and that it <sup>is</sup> probably of the order of 0.10 ft/min. Using the latter figure, ground-water inflow under high watertable conditions is calculated at about  $\frac{1}{4}$  million gallons per day.

Replenishment by infiltration of rainfall and runoff over the same area, was shown not to exceed 30 million gallons per annum. Probably it is considerably less. Taking into account that the average subsurface inflow is less than stated above, (was 10% lower at lowest water-table position after the dry 1959 - 1960 season), the maximum perennial quantity of water, available in the area outlined on plan G132/61C, is thus about 250,000 - 300,000 gallons per day. No allowance has been made for losses by evapotranspiration, effluent seepage and subsurface outflow. These losses can be minimized by pumping strategically placed wells. It will, however, be necessary to purchase land up to the conservation dam if the available supply is to be developed to its fullest extent. The supply obtainable from the alluvial deposits in the poort is relatively small being estimated of the order of 30,000 gallons per day.

If larger supplies are required, artificial recharge will have to be resorted to. Sufficient runoff seems to be available and it would appear that by artificial recharge the supply could easily be doubled if not more. Experiments with ponds and observations of the water-levels in the conservation dam and behind embankments have shown that artificial recharge can only be done effectively by water spreading over natural veld and not by infiltration from dams. The conservation dam may play an important role in temporary storing water for spreading.

Should the Railways Administration decide to undertake further development of the underground supplies, the Geological Survey would be glad to continue with its studies at Caroluspoort and offers its assistance. A complete study of the water cycle could be undertaken in co-operation with the Department of Water Affairs.

PRETORIA  
12 July, 1961.

J.R. Vegter  
CHIEF GEOLOGIST.

TABLE NO. 1

## DATA ON TEST BORE-HOLE

Bore-hole No.	Collar elevation ft.	Depth (ft)	Thickness of alluvium (ft)
1	85.39	48	44
2	87.16	53	50
3	89.26	46	44
4	90.37	50	48
5	<sup>3024 CA 306</sup> 91.43	48	44
7	63.07	21	-
8	63.48	30	-
9	64.20	27	-
10	66.82	23	-
11	66.52	27	-
14	96.01	40	38
15	<sup>3024 CA 352</sup> 96.77	36	34
16	97.75	36	34
17	<sup>3024 CA 304</sup> 102.08	40	38
18	97.81	37	35
19	95.86	39	37
20	<sup>3024 CA 353</sup> 96.64	41	38
21	<sup>3024 CA 307</sup> 97.28	46	46
22	<sup>3024 CA 354</sup> 86.84	47	47
23	83.77	47	47

Bore-hole No.	Collar elevation ft.	Dept (ft)	Thickness of alluvium (ft)
24	3024 CA 347 / 355 90.29	45 (20 m)	45 ?
25	3024 CA 312 93.09	38	38 ?
26	3024 CA 86.44	45 (20 m)	45 ?
27	3024 CA 309 84.56	43	43 ?
28	90.42	45	45 ?
29	3024 CA 308 90.74	40	40 ?
30	87.66	45	45 ?
31	3024 CA 348 84.10	45	45 ?
32	93.33	49	49
33	89.11	37	37 ?
34	89.98	47	47 ?
40	97.15	43	38
41	98.58	44	36
43	95.40	38	32
44	99.23	43	41
45	99.71	43	38
46	96.74	40	36
47	96.43	40	38
48	97.85	41	38
49	97.81	27	?
50	98.49	41	38

Bore-hole No.	Collar elevation ft.	Depth (ft)	Thickness of alluvium (ft)
51	98.03	40	36
52	3024 CA 345 94.48	65	40
53	89.62	44	40
79	78.73	-	-

TABLE NO. 2

BORE-HOLES DRILLED FOR UNDISTURBED SAMPLES

Bore-hole No.	Collar elevation (ft)	Depth (ft)	Geological Profile
54	101.49	41	<p>3024 CA 366</p> 0-4'5" brown, clayey, silt sand 4'5"-10'2" red brown, sandy silt with pebbles 10'2"-11'8" yellow-brown, sandy silt with pebbles 11'8"-16'2" limestone 16'2"-18'9" yellow-brown, clayey sand with lime nodules 18'9"-32'8" brown, sandy silt 32'8"-35'9" boulders and gravel 35'9"-36'11" dense, brown, clayey sand 36'11"-41'0" hard blue shale.
55	90.26	47	<p>3024 CA 367</p> 0-2'4" grey brown, clayey sand 2'4"-11'9" grey sandy silt with lime nodules 11'9"-15'4" brown silt sand with lime nodules 15'4"-23'5" brown, sandy silt 23'6"-33'8" yellow-brown, sandy silt 33'8"-34'4" yellow brown, sandy silt with shale cobbles. 34'4"-40'8" yellow brown, silt sand 40'8"-41'11" grey silty sand and shale pebbles 41'11"-43'4" yellow grey, sandy-silt 43'4"-46'4" boulders and gravel 46'4"-47'4" hard blue shale.
56	83.87	45	0'-19" light brown, clayey sand 1'9"-7'5" grey brown, clayey silt with lime nodules 7'5"-11'9" brown, sandy silt with lime nodules 11'9"-16'4" brown, silt sand with lime nodules 16'4"-23'7" grey brown, sandy silt 23'7"-24'9" yellow-brown, silty sand 24'9"-25'4" limestone 25'4"-29'9" brown, sandy silt 29'9"-32'0" grey brown, silty sand, gravel and cobbles 32'0"-37'10" yellow brown silt. 37'10"-40'4" brown, silt sand 40'4"-42'1" grey brown, silt sand and gravel 42'1"-42'8" boulders and sand 42'8"-45'2" hard blue shale

Bore-hole No.	Collar elevation (ft)	Depth (ft)	Geological profile
57	30 <sup>24</sup> CA 368 38.82	41	0'-4'3" brown, clayey sand 4'3"-11'9" brown, sandy silt with lime nodules 11'9"-13'0" limestone 13'0"-14'9" brown, sandy silt with lime nodules 14'9"-19'6" brown, silt sand 19'6"-30'3" grey brown sand and pebbles 30'3"-39'9" grey brown sand and boulders 39'9"-41'1" decomposed shale.
58	95.31	38	0'-0'9" brown sand and pebbles 0'9"-8'3" reddish brown, silty sand with pebbles 8'3"-10'5" brown, sandy silt with lime nodules 10'5"-15'10" brown, silty sand with pebbles 15'10"-16'8" limestone 16'8"-22'2" reddish brown, sandy silt 22'2"-23'6" reddish brown, sandy silt with lime nodules 23'6"-27'0" grey brown, silty sand 27'0"-30'9" grey, silty sand and gravel 30'9"-36'2" boulders and sand 36'2"-37'5" decomposed shale 37'5"-37'10" hard blue shale
59	3024CA 346 70.12	38	0-6'3" grey brown, silty sand with lime nodules 6'3"-10'9" brown, sandy silt with pebbles 10'9"-14'0" brown sandy silt 14'6"-19'10" grey brown, sandy silt 19'10"-23'2" grey brown, silty sand and pebbles 23'2"-25'9" grey brown, silty sand 25'9"-34'10" boulders, sand and gravel 34'10"-37'11" hard blue shale.

TABLE NO. 3

OTHER BORE-HOLES

Bore-hole No.	Collar elevation (ft.)	Depth (ft.)	Elevation of water-level and date	
G2632 (S.A.R.)	-	100	16'	240 g.p.h.; 0-29 alluvial deposits and boulders, 29-100 shale
G2633 (S.A.R.)	47.37	100	32.11 12/11/59	0-3' soil; 3-14' limestone; 14-100 shale. Yield about 2000 g.p.h.
Bore-hole at S.A.R. well G6780	-	36'	-	600-700 g.p.h.
1	130.32	-	111.65 Sept. 1959	Windmill Mr. Altema
2	98.24	-	74.07 26/6/59	Windmill Mr. P. v.d. Merwe
3	110.18	-	72.45 30/11/59	" Mr. J.H. v.d. Merwe
4	147.62	-	109.16 30/11/59	"
5	108.23	-	87.35 30/11/59	" " " "
6	108.72	-	88.99 30/11/59	" " " "
7	97.53	-	78.40 30/11/59	" " " "
8	94.80	-	77.54 30/11/59	Open bore-hole Mr. J.H. v.d. Merwe
9	50.21	-	38.19 26/5/59	Open bore-hole Mr. P. v.d. Merwe
10	48.86	-	37.25 26/5/59	Open bore-hole Mr. P. v.d. Merwe
11	47.86	-	-	Engine

TABLE 4

Data on Wells

Well No.	Elevation on top of concrete cover at manhole (ft)	Depth (ft)	Dia- meter (ft)	Yield g.p.h.	Date of test
<sup>3024 CA94</sup> G6778 (S.A.R. No.1)	100.00	40.5	11	8000	5/57
<sup>CA95</sup> G6779 (S.A.R. No.2)	89.09	51	11	7800	2/57
G6780 (S.A.R. No.3)	65.68	29	11	failure	-
S.A.R. No.4	53.04	18	11	8500 has de- creased consider- ably	?
S.A.R. No.5	79.36	35	11	1,200	10/59
S.A.R. No.6	77.86	37	11	3,200	9/59
No.7 Old S.A.R. Well)	52.06	18	-	-	-
No.8 (P.v.d. Merwe)	-	22	-	9,600	-