UNIVERSITY OF JOHANNESBURG

PROGRAM	:	BACCALAUREUS TECHNOLOGIAE: *ENGINEERING : CIVIL*
SUBJECT	:	**HYDROLOGY IV**
CODE	:	**THB 411**
ASSESSMENT	:	SUMMER EXAMINATION (MAIN PAPER)
DATE		22nd NOVEMBER 2016
DURATION	:	(SESSION 1) 08:30 - 11:30
WEIGHT	:	40:60
TOTAL MARKS	:	110

ASSESSOR	:	G.K. NKHONJERA
MODERATOR	:	PROF. F.M. ILUNGA
NUMBER OF PAGES:		PAGES: 20 including the cover page and Annexures.

INSTRUCTIONS :

1. This is a closed-book type of Exam.
2. This paper contains 7 questions. Four (4) in Section A and three (3) in Section B.
3. SECTION A : ANSWER **ALL** QUESTIONS.
   SECTION B : ANSWER **TWO** QUESTIONS ONLY.
4. Make sure that you understand what the question requires before attempting it.
5. Any additional material is to be placed in the answer book and must indicate clearly the question number, your name, and Student number.
5. Where necessary, answers without calculations will not be considered.

SECTION A
ANSWER ALL QUESTIONS

QUESTION 1 [10]

1.1 Discuss why Velocity-Area method may not be a proper method for discharge measurements in a small mountainous stream. (2)

1.2 In your own words, explain clearly why natural land, open up to agriculture, may result in more runoff from that land. (2)

1.3 A temporary cofferdam is being designed to protect a 5-year construction dam project from a 25-year flood. What do you think is the probability that the cofferdam will be overtopped:

a) At least once during the 5-year project. (2)
b) At least once in any one year. (2)
c) Not at all during the project. (2)

QUESTION 2 [15]

2.1 Briefly, discuss why the following factors may be important in selecting a site for a flow gauging station.

a) Control station must have firm and stable river banks and river bed. (2)
b) Flow gauging station should not be located just upstream of a confluence on any river reach. (2)

2.2 You are provided with **Fig 2.1** in Appendix E that represents a rating curve for the gauge station (A2H023) on the Jukskei River at Nietgedacht. This rating curve has been prepared from hydrological data collected over several years. During a particular storm event in the catchment of this station, records of stages were observed and recorded as shown in **Table 2.2** below. If the cross-section of the river bed and banks at the gauge station has remained relatively constant over all the years, generate and plot the hydrograph of the storm event. (11)

Table 2.2: Observed data for stage (m) at Nietgedacht on Jukskei River

Hour	08:00	09:00	10:00	11:00	12:00	13:00	14:00	15:00	16:00
Stage, H (m)	0.9	2.2	4.1	5.2	4.5	3.7	3.0	2.3	1.5

QUESTION 3 [15]

3.1 Where hydrological data may be available, statistical methods, like the Log-Pearson Type III Distribution method, are more reliable than deterministic methods. However, these statistical methods have limitations of their own. Explain briefly any THREE uncertainties regarding these statistical methods. (3)

3.2 A bridge is proposed to be constructed over the Orange River near a small town of Vioolsdrif. The bridge site is near the Vioolsdrif Gauge Station (D8H003) which has some hydrological data of annual flood flows spanning over 45 years. The data indicate a maximum flood flow of 8465.500 m^3/s and a minimum flood flow of 140.452 m^3/s. This data, when Hazen formula is used, plot as a straight line on a semi-logarithmic paper with return period plotted logarithmically. If regulations state that a 100-year flood value must be used for the design of such a bridge, determine the:

a) Actual design flood (m^3/s) that is to be used for the design of the bridge. (9)

b) Expected life span of the bridge if the hydrologic risk associated with the above design value is 10%. (3)

QUESTION 4 [20]

A groundwater investigation study resulted in 3 discharge wells (X, Y, and Z) being dug in the study area. The wells, which penetrates the aquifer fully, are pumped at constant rates of 0.013 m^3/s, 0.019 m^3/s and 0.015 m^3/s respectively as shown in **Fig 4.3**. The steady-state drawdowns measured at observation wells A and B are 2.12 m and 1.54 m respectively. The relevant distances of these wells from one another are presented in **Table 4.4**. Making reasonable assumptions where necessary, determine the coefficient of permeability (m/day) if the aquifer is classified as:

a) Confined aquifer with a thickness of 40 m. (8)
b) Unconfined aquifer with an undisturbed water table level of 56 m above the impermeable layer. (12)

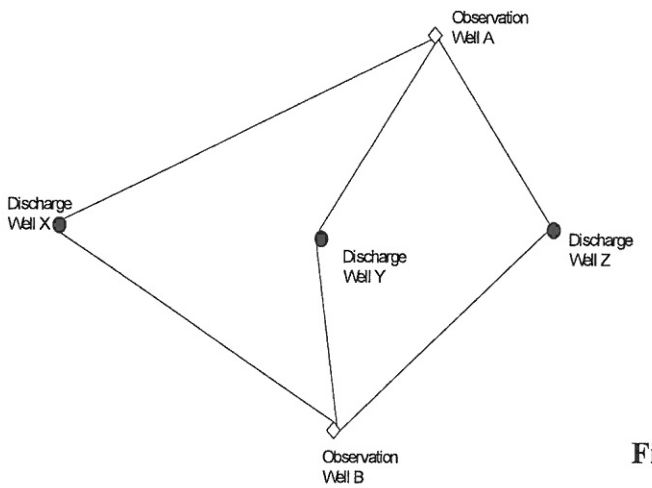

Fig. 4.3

Table 4.4: Distance of wells from one another.

Name of well distance	Distance (m)
AX	130
AY	105
AZ	217
BX	100
BY	145
BZ	200
XY	160
YZ	154

SECTION B
ANSWER <u>TWO</u> QUESTIONS ONLY

QUESTION 5 [25]

Table 5.6 together with **Fig 5.5** of Appendix D, show the relationship of elevation-outflow-storage data for a dam reservoir somewhere in Mpumalanga province. Then, a flood reaching the reservoir produced the inflow hydrograph as shown in **Table 5.7** below. Using the Modified Puls method, route the hydrograph through the reservoir and determine the following:

a) Peak outflow during the flood and its time of occurrence. (23)
b) Peak attenuation. (1)
c) Lag time (hrs). (1)
(For this reservoir, assume that initial outflow equals initial inflow)

Table 5.6: Elevation vs Outflow vs Storage relationship

Elevation (m)	Outflow, O (m^3/s)	Storage, S x 10^3 (m^3)
811	0	0
816	17.6	418
821	59.1	843
826	121	1280
831	122	1740

Table 5.7: Inflow hydrograph data

Time (hrs)	Inflow, I (m^3/s)
00:00	5
02:00	9
04:00	15
06:00	30
08:00	85
10:00	160
12:00	140
14:00	95
16:00	45
18:00	15

QUESTION 6 [25]

A certain river has a 3-hr unit hydrograph for a flow gauging station just as the river passes a gorge near the Nkhonjera Mountain as shown in **Table 6.8** below. From a 9-hr storm that occurred in the catchment the effective rainfall measured during the subsequent 3-hour intervals was 1.5 cm, 2.25 cm and 1.25 cm. Assuming a constant base flow of 12 m^3/s, determine the following:

a) Storm hydrograph that resulted from the 9-hour storm. (23)
b) Peak discharge and its time of occurrence. (2)

Table 6.8: 3-hr Unit Hydrograph for a gorge-station near Nkhonjera Mountain.

Hour	0	3	6	9	12	15	18	21	24
Q (m^3/s)	0	66.7	236.9	340.0	253.3	200	85.2	16	0

QUESTION 7 [25]

7.1 Annual peak flood discharges for the Orange River at Vioolsdrif Town (D8H003) from 1936 to 2016 are given in **Table 7.9** below. The statistical parameters (Mean, Standard deviation and skewness) for this annual series were computed and presented together with the table below. Determine the return period of the 1988 flood, 2005 flood and 2010 flood if this data is assumed to fit the following distribution trends:

a) Log-Normal Distribution. (6)
b) Log-Pearson Type III Distribution. (3)
c) Gumbel Distribution. (11)

7.2 Based on the Gumbel Distribution method, and that if the 1988 flood value were to be used for the design of a hydraulic structure, what would be the probability that this structure will not be:

a) Washed away in any one year. (2)
b) Reliable during the 50 years of its lifespan. (3)

Table 7.9: Annual Peak discharges of the Orange River @ Vioolsdrif

Year	Peak Flow (m^3/s)	LogQ
1936	2190.536	3.3406
1937	5781.75	3.7621
1938	3548.545	3.5501
1939	224.01	2.3503
1940	2344.496	3.3700
1941	3598.92	3.5562
1942	4109.369	3.6138
1943	3843.396	3.5847
1944	5381.73	3.7309
1945	2941.697	3.4686
1946	1732.909	3.2388
1947	1056.325	3.0238
1948	5337.519	3.7273
1949	461.087	2.6638
1950	2992.697	3.4761
1951	2888.837	3.4607
1952	2245.156	3.3512
1953	1849.791	3.2671
1954	2732.317	3.4365
1955	5693.26	3.7554
1956	2837.477	3.4529
1957	4746.599	3.6764
1958	6776.73	3.8310
1959	3499.7	3.5440
1960	1180.7	3.0721
1961	2732.317	3.4365
1962	4672.199	3.6695

Year	Peak Flow (m^3/s)	LogQ
1963	4090.52	3.6118
1964	1602.279	3.2047
1965	2448.168	3.3888
1966	3905.476	3.5917
1967	6447.29	3.8094
1968	781.025	2.8927
1969	1972.4	3.2950
1970	907.597	2.9579
1971	305.262	2.4847
1972	2582.417	3.4120
1973	445.398	2.6487
1974	2357.1	3.3724
1975	1991.1	3.2991
1976	3051.9	3.4846
1977	1647.4	3.2168
1978	1180.7	3.0721
1979	401.25	2.6034
1980	274.99	2.4393
1981	400.087	2.6022
1982	359.06	2.5552
1983	739.007	2.8686
1984	300.74	2.4782
1985	229.51	2.3608
1986	321.989	2.5078
1987	347.643	2.5411
1988	8465.5	3.9277
1989	2442.837	3.3879

Year	Peak Flow (m^3/s)	LogQ
1990	417.911	2.6211
1991	912.196	2.9601
1992	508.33	2.7061
1994	332.626	2.5220
1995	157.251	2.1966
1996	2261.465	3.3544
1997	2582.417	3.4120
1998	1157.36	3.0635
1999	489.775	2.6900
2000	1775.27	3.2493
2001	714.618	2.8541
2002	1783	3.2512
2003	897.33	2.9530
2004	203.17	2.3079
2005	140.452	2.1475
2006	2054.393	3.3127
2007	721.109	2.8580
2008	148.712	2.1723
2009	886.041	2.9475
2010	3078.778	3.4884
2011	5076.334	3.7056
2012	193.46	2.2866
2013	232.327	2.3661
2014	949.735	2.9776
2015	200.1	2.3012
2016	280.826	2.4484

Mean = 2069.42
Standard Deviation = 1864.7
Log Mean = 3.0947
Log Standard Deviation = 0.4884
Log Skewness Co-efficient = -0.3328
Data size, N = 80

MERRY CHRISTMAS TO YOU ALL !!!!!

APPENDIX A

CALCULATION FORMULAS

1) Water Balance Equation

$$P - R - E - I - T = \Delta S \text{ or } P + Q_{in} - Q_{out} - E - I - T = \pm \Delta S$$

2) The general formulas for the return period using the plotting position methods

Return Period: $T_r = \dfrac{a \times N + b}{c \times m + d}$

Table 5.3: Values of constants used to calculate the return period *(Haarhoff & Cassa, 2009)*

Method	a	b	c	d	T (if N = 50, m = 1)
California (1923)	1	0	1	0	51
Hazen (1930)	2	0	2	-1	102
Weibull (1939)	1	1.00	1	0	51
Beard (1962)	1	0.40	1	-0.30	73
Blom (1958)	1	0.25	1	-0.38	82
Gringorten (1963)	1	0.12	1	-0.44	91
Cunane (1978)	1	0.20	1	-0.40	85
Greenwood (1979)	1	0	1	-0.35	78

Hazen formual,

$$T = \frac{2N}{2m - 1}$$

3) Flood frequency analysis

$$Q_T = \overline{Q} + K\sigma_Q$$

$$Log_{10}Q_T = \overline{Log_{10}Q} + K\sigma_{\log Q}$$

$$\sigma_Q = \sqrt{\frac{\sum \left(Q - \overline{Q}\right)^2}{N - 1}}$$

$$Cs = \frac{N\sum_{i=1}^{N}\left(Q-\overline{Q}\right)^{3}}{(N-1)(N-2)\sigma_Q^3}$$

$$K = \frac{YT - Yn}{Sn}$$

Reduce variable,

$$YT = -\ln\left(-\ln\left(1-\frac{1}{T}\right)\right)$$

Hydrologic risk,

$$J = 1-\left(1-\frac{1}{T}\right)^{n}$$

Exceedance probability, $p = \frac{1}{T}$

Reliability,

$$K = 1 - J$$

4) Precipitation formulas

- Theissen Polygon method,

$$\overline{p} = \frac{\sum_{i=1}^{i=n} P_i A_i}{\sum A}$$

- Isohyetal Map method,

$$\overline{p} = \frac{\sum_{i=1}^{i=n} P_{i-j} A_{i-j}}{\sum A_{i-j}}$$

5) Standard Shapes

Area of triangle,

$$A = \frac{1}{2}bh$$

Area of a trapezium,

$$A = \frac{1}{2}(B + C)h$$

Where

b = base of triangle; h = height of triangle and trapezium

B & C = two parallel sides of a trapezium.

6) **Groundwater hydrology formulas.**

a) Dupuit equation:

$$q = \frac{K}{2L}\left(h_1^2 - h_2^2\right)$$

$$q = \frac{K}{2L}\left(h_2^2 - h_1^2\right) + W\left(x - \frac{L}{2}\right)$$

b) Thiem equation - Confined Aquifer:

$$T = Kb = \frac{Q}{2\pi\left(h_2 - h_1\right)}\ln\left(\frac{r_2}{r_1}\right)$$

c) Thiem equation - Unconfined Aquifer:

$$Q = \pi K \frac{h_2^2 - h_1^2}{\ln\left({r_2}/{r_1}\right)}$$

d) Theis method (Unsteady-state Well hydraulics)

$$s' = \frac{Q}{4\pi T}\int_u^{\infty}\frac{e^{-u}du}{u} = \frac{Q}{4\pi T}W(u)$$

where:

$$u = \frac{r^2 S}{4Tt}$$

$$W(u) = -0.5772 - \ln(u) + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + \frac{u^5}{5.5!} - \dots\dots$$

e) Multiple discharging wells (Steady-state Well hydraulics)

Confined aquifer:

$$s = s_{ob} + \frac{Q_A}{2\pi T}\ln\left(\frac{r_{Ao}}{r_A}\right) + \frac{Q_B}{2\pi T}\ln\left(\frac{r_{Bo}}{r_B}\right) + ...$$

Or,

$$s = s_{ob} + \sum_{i=1}^{M}\frac{Q_i}{2\pi T}\ln\left(\frac{r_{io}}{r_i}\right)$$

Unconfined aquifer:

$$h^2 = h_{ob}^2 - \frac{Q_A}{\pi K}\ln\left(\frac{r_{Ao}}{r_A}\right) - \frac{Q_B}{\pi K}\ln\left(\frac{r_{Bo}}{r_B}\right) - .....$$

Or,

$$h^2 = h_{ob}^2 - \sum_{i=1}^{M}\frac{Q_i}{\pi K}\ln\left(\frac{r_{io}}{r_i}\right)$$

Where:

s = Drawdown of observation well where *s* is sought.

s_{ob} = Drawdown of observation well where *s* is known.

Q_A = Discharge of pumping well at A.

Q_B = Discharge of pumping well at B.

T = Transmissivity

r_A = Distance from pumping well at A to a point where *s* is sought.

r_{Ao} = Distance from pumping well at A to an observation well where *s* is known.

r_B = Distance from pumping well at B to a point where *s* is sought.

r_{Bo} = Distance from pumping well at B to an observation well where *s* is known.

h = Piezometric head of observation well where *h* is sought.

h_{ob} = Piezometric head of observation well where *h* is known.

7) Muskingum Method

$$S = K[XI + (1 - X)O]$$

$$O_2 = C_1 I_2 + C_2 I_1 + C_3 O_1$$

$$1 = C_1 + C_2 + C_3$$

$$C_1 = \frac{0.5\Delta t - KX}{K - KX + 0.5\Delta t}$$

$$C_2 = \frac{0.5\Delta t + KX}{K - KX + 0.5\Delta t}$$

$$C_3 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t}$$

8) Modified Puls Method

$$\frac{S_j - S_i}{\Delta t} = \frac{I_j - I_i}{2} - \frac{O_j - O_i}{2}$$

$$(I_j - I_i) + \left[\left(\frac{2S_i}{\Delta t}\right) - O_i\right] = \left[\left(\frac{2S_j}{\Delta t}\right) + O_j\right]$$

APPENDIX B & C

Appendix B

Table 5.4: Values of the reduced mean (Yn) and standard deviation (Sn) as a function of the sample size N (Baban 1995)

N	Yn	Sn
20	0.52	1.06
30	0.54	1.11
40	0.54	1.14
50	0.55	1.16
60	0.55	1.17
70	0.55	1.19
80	0.56	1.19
90	0.56	1.20
100	0.56	1.21
150	0.56	1.23
200	0.57	1.24
Infinity	0.57	1.28

Appendix C

Table 9.4: Values of K for a Normal Distribution (Wilson, 2011)

Probability of exceedance (%)	K	Probability of exceedance (%)	K
0.1	3.09	50	0
0.5	2.58	55	-0.13
1	2.33	60	-0.25
2.5	1.96	65	-0.385
5	1.645	70	-0.52
10	1.28	75	-0.67
15	1.04	80	-0.84
20	0.84	85	-1.04
25	0.67	90	-1.28
30	0.52	95	-1.645
35	0.385	97.5	-1.96
40	0.25	99	-2.33
45	0.13	99.5	-2.58
50	0	99.99	-3.09

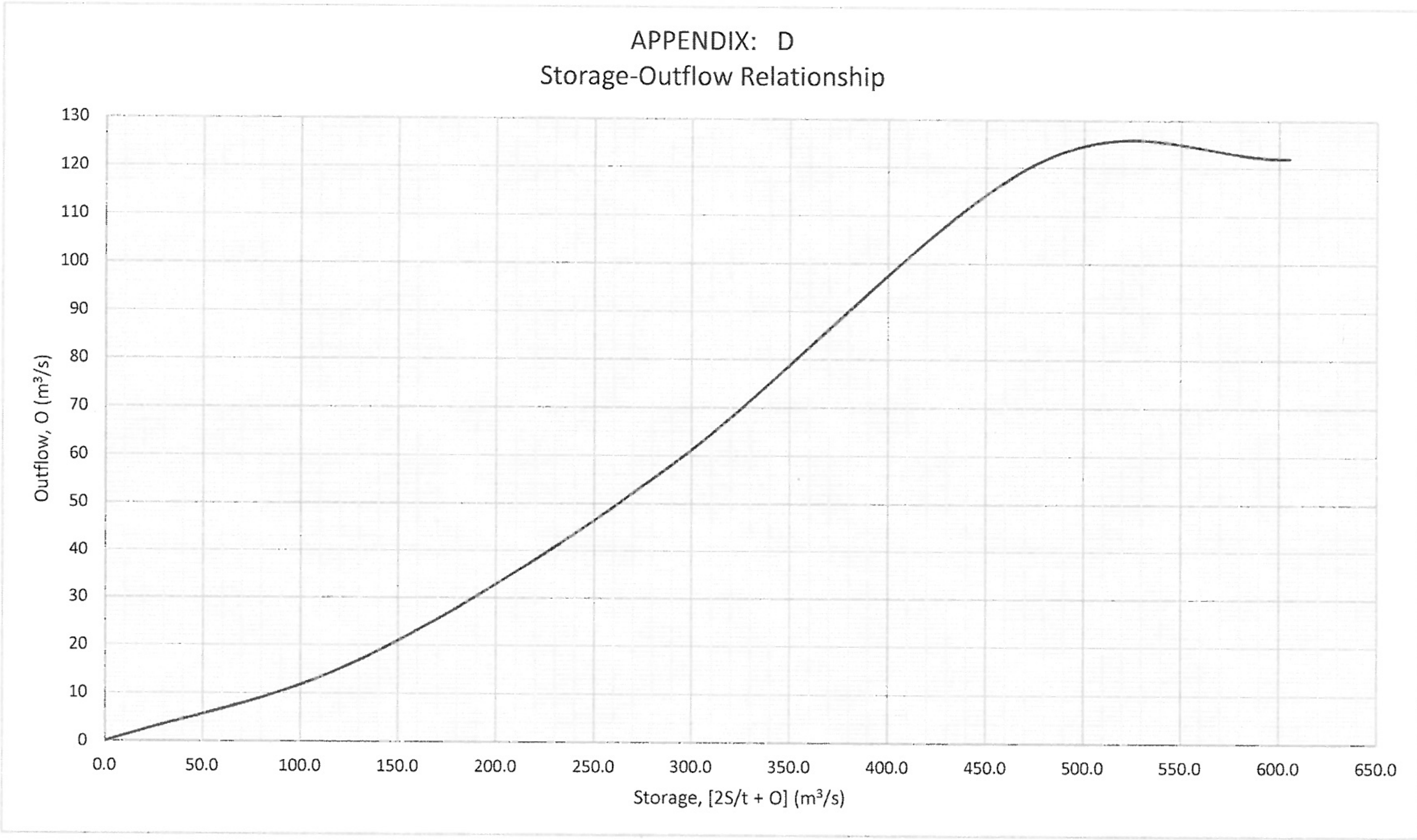
APPENDIX: D
Storage-Outflow Relationship
Outflow, O (m³/s)
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
400.0
450.0
500.0
550.0
600.0
650.0
Storage, [2S/t + O] (m³/s)

Fig 5.5

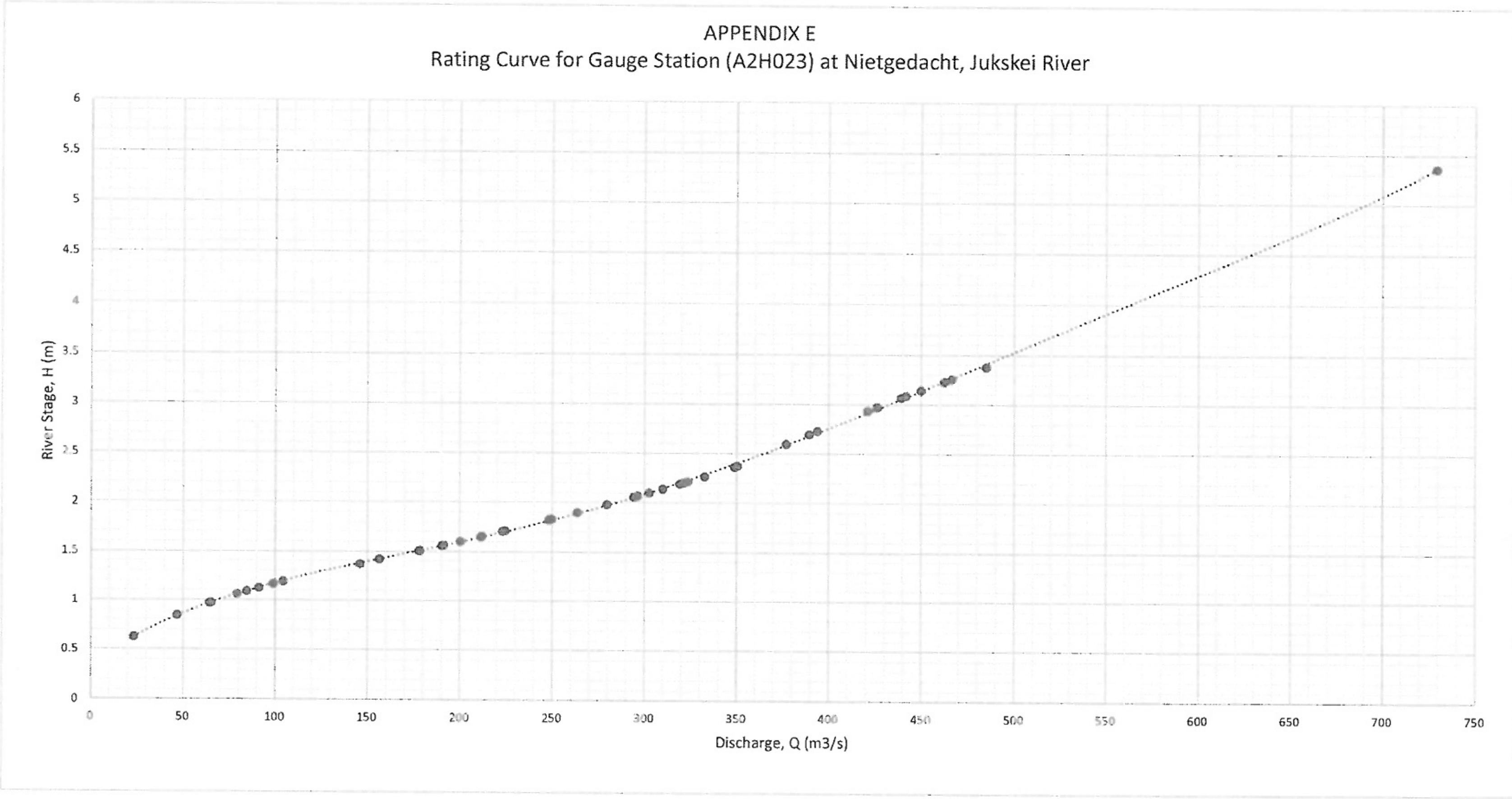
APPENDIX E
Rating Curve for Gauge Station (A2H023) at Nietgedacht, Jukskei River
River Stage, H (m)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
Discharge, Q (m3/s)

Fig 2.1

APPENDIX F

Frequency Factors K for Gamma and log-Pearson Type III Distributions (Haan, 1977, Table 7.7)

	Recurrence Interval In Years							
WEIGHTED	1.0101	2	5	10	25	50	100	200
SKEW COEFFICIENT	Percent Chance (>=) = p							
Cw	99	50	20	10	4	2	1	0.5
3	-0.667	-0.396	0.42	1.18	2.278	3.152	4.051	4.97
2.9	-0.69	-0.39	0.44	1.195	2.277	3.134	4.013	4.904
2.8	-0.714	-0.384	0.46	1.21	2.275	3.114	3.973	4.847
2.7	-0.74	-0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	-0.769	-0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	-0.799	-0.36	0.518	1.25	2.262	3.048	3.845	4.652
2.4	-0.832	-0.351	0.537	1.262	2.256	3.023	3.8	4.584
2.3	-0.867	-0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	-0.905	-0.33	0.574	1.284	2.24	2.97	3.705	4.444
2.1	-0.946	-0.319	0.592	1.294	2.23	2.942	3.656	4.372
2	-0.99	-0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	-1.037	-0.294	0.627	1.31	2.207	2.881	3.553	4.223
1.8	-1.087	-0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	-1.14	-0.268	0.66	1.324	2.179	2.815	3.444	4.069
1.6	-1.197	-0.254	0.675	1.329	2.163	2.78	3.388	3.99
1.5	-1.256	-0.24	0.69	1.333	2.146	2.743	3.33	3.91
1.4	-1.318	-0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	-1.383	-0.21	0.719	1.339	2.108	2.666	3.211	3.745
1.2	-1.449	-0.195	0.732	1.34	2.087	2.626	3.149	3.661
1.1	-1.518	-0.18	0.745	1.341	2.066	2.585	3.087	3.575
1	-1.588	-0.164	0.758	1.34	2.043	2.542	3.022	3.489
0.9	-1.66	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	-1.733	-0.132	0.78	1.336	1.993	2.453	2.891	3.312
0.7	-1.806	-0.116	0.79	1.333	1.967	2.407	2.824	3.223
0.6	-1.88	-0.099	0.8	1.328	1.939	2.359	2.755	3.132
0.5	-1.955	-0.083	0.808	1.323	1.91	2.311	2.686	3.041
0.4	-2.029	-0.066	0.816	1.317	1.88	2.261	2.615	2.949
0.3	-2.104	-0.05	0.824	1.309	1.849	2.211	2.544	2.856
0.2	-2.178	-0.033	0.83	1.301	1.818	2.159	2.472	2.763
0.1	-2.252	-0.017	0.836	1.292	1.785	2.107	2.4	2.67
0	-2.326	0	0.842	1.282	1.751	2.054	2.326	2.576
-0.1	-2.4	0.017	0.846	1.27	1.716	2	2.252	2.482
-0.2	-2.472	0.033	0.85	1.258	1.68	1.945	2.178	2.388
-0.3	-2.544	0.05	0.853	1.245	1.643	1.89	2.104	2.294
-0.4	-2.615	0.066	0.855	1.231	1.606	1.834	2.029	2.201
-0.5	-2.686	0.083	0.856	1.216	1.567	1.777	1.955	2.108
-0.6	-2.755	0.099	0.857	1.2	1.528	1.72	1.88	2.016
-0.7	-2.824	0.116	0.857	1.183	1.488	1.663	1.806	1.926
-0.8	-2.891	0.132	0.856	1.166	1.448	1.606	1.733	1.837
-0.9	-2.957	0.148	0.854	1.147	1.407	1.549	1.66	1.749
-1	-3.022	0.164	0.852	1.128	1.366	1.492	1.588	1.664
-1.1	-3.087	0.18	0.848	1.107	1.324	1.435	1.518	1.581
-1.2	-3.149	0.195	0.844	1.086	1.282	1.379	1.449	1.501
-1.3	-3.211	0.21	0.838	1.064	1.24	1.324	1.383	1.424
-1.4	-3.271	0.225	0.832	1.041	1.198	1.27	1.318	1.351
-1.5	-3.33	0.24	0.825	1.018	1.157	1.217	1.256	1.282
-1.6	-3.88	0.254	0.817	0.994	1.116	1.166	1.197	1.216
-1.7	-3.444	0.268	0.808	0.97	1.075	1.116	1.14	1.155
-1.8	-3.499	0.282	0.799	0.945	1.035	1.069	1.087	1.097
-1.9	-3.553	0.294	0.788	0.92	0.996	1.023	1.037	1.044
-2	-3.605	0.307	0.777	0.895	0.959	0.98	0.99	0.995
-2.1	-3.656	0.319	0.765	0.869	0.923	0.939	0.946	0.949
-2.2	-3.705	0.33	0.752	0.844	0.888	0.9	0.905	0.907

-2.3	-3.753	0.341	0.739	0.819	0.855	0.864	0.867	0.869
-2.4	-3.8	0.351	0.725	0.795	0.823	0.83	0.832	0.833
-2.5	-3.845	0.36	0.711	0.711	0.793	0.798	0.799	0.8
-2.6	-3.899	0.368	0.696	0.747	0.764	0.768	0.769	0.769
-2.7	-3.932	0.376	0.681	0.724	0.738	0.74	0.74	0.741
-2.8	-3.973	0.384	0.666	0.702	0.712	0.714	0.714	0.714
-2.9	-4.013	0.39	0.651	0.681	0.683	0.689	0.69	0.69
-3	-4.051	0.396	0.636	0.66	0.666	0.666	0.667	0.667

APPENDIX G

FLOOD ROUTING

MODIFIED PULS METHOD

Time (hrs, min, sec)	Inflow, I_i (m^3/s)	Inflow, I_j (m^3/s)	$[I_i + I_j]$ (m^3/s)	$[2S_i/\Delta t - O_i]$ (m^3/s)	$[2S_j/\Delta t + O_j]$ (m^3/s)	Outflow, O (m^3/s)

APPENDIX H

HYDROGRAPH ANALYSIS

Time							