Appendix T.5

Cross Sections

Appendix T.5.1

Transfer Sections

(8 Sections)

27000 to 30500 @ 500 m intervals

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

D COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHD86007 DRILL HOLE

0 50 100 200
METRES

0	23	03	88	ORIGINAL DRAFT	RJ	DJ	DJ
Rev.	D	M	Y	Revision Description	Drn.	Des.	App.

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION
27000

Scale 1:2500

Drawing No. 88-903-21-001

Rev. 0

0	23	03	88	ORIGINAL DRAFT	R J	D J	D J
Rev.	D	M	Y	Revision Description	Drn.	Des.	App.

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 27500

Scale 1:2500

Drawing No. 88-903-21-002

Rev. 0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

D — COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHD86007 — DRILL HOLE

0 50 100 200
METRES

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	23	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER

Category CROSS-SECTIONS

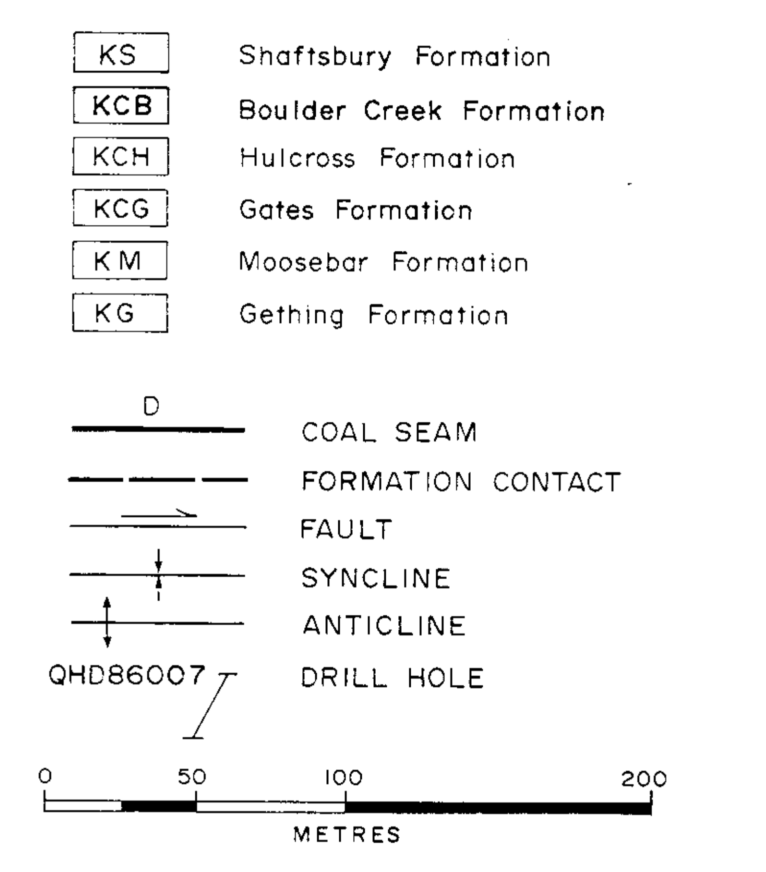
Drawing Title

CROSS-SECTION 28 000

Scale 1:2500

Drawing No. 88-903-21-003

Rev. 0



Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	23	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 28 500

Scale 1:2500

Drawing No. 88-903-21-004

Rev. 0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

D COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHD86007 DRILL HOLE

0 50 100 200
METRES

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	24	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER — Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 29 000

Scale 1:2500 — Drawing No. 88-903-21-005 — Rev. 0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

D COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHD86007 DRILL HOLE

0 50 100 200
METRES

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	24	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION
29 500

Scale 1:2500

Drawing No. 88-903-21-006

Rev. 0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

D COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHD86007 DRILL HOLE

0 50 100 200
METRES

0	24	03	88	ORIGINAL DRAFT	RJ	DJ	DJ
Rev.	D	M	Y	Revision Description	Drn.	Des.	App.

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER | Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 30 000

Scale 1:2500 | Drawing No. 88-903-21-007 | Rev. 0

KS Shaftsbury Formation
KCB Boulder Creek Formation
KCH Hulcross Formation
KCG Gates Formation
KM Moosebar Formation
KG Gething Formation

D COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHD86007 DRILL HOLE

0 50 100 200
METRES

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	24	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area TRANSFER

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 30 500

Scale 1:2500

Drawing No. 88-903-21-008

Rev. 0

Appendix T.5.2

Grizzly Sections

(5 Sections)

26000 to 28000 @ 500 m intervals

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	22	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area GRIZZLY

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 26000

Scale 1:2500

Drawing No. 88-905-21-001

Rev. 0

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	22	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

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COAL DIVISION

Area GRIZZLY — Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 26500

Scale 1:2500 — Drawing No. 88-905-21-002 — Rev. 0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

D — COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHC86007 — DRILL HOLE

0 50 100 200
METRES

0	22	03	88	ORIGINAL DRAFT	RJ	DJ	DJ
Rev.	D	M	Y	Revision Description	Drn.	Des.	App.

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area GRIZZLY

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 27000

Scale	Drawing No	Rev.
1:2500	88-905-21-003	0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

D COAL SEAM

FORMATION CONTACT

FAULT

SYNCLINE

ANTICLINE

QHD86007 DRILL HOLE

0 50 100 200

METRES

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	23	03	88	ORIGINAL DRAFT	RJ	DJ	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area GRIZZLY

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION
27500

Scale 1:2500

Drawing No. 88-905-21-004

Rev. 0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QHC86CO7 + DRILL HOLE

0 50 100 200
METRES

0	23	03	88	ORIGINAL DRAFT	RJ	DJ	DJ
Rev.	D	M	Y	Revision Description	Drn.	Des.	App.

QUINTETTE COAL LIMITED

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DENISON MINES LIMITED

COAL DIVISION

Area GRIZZLY

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION
28 000

Scale	Drawing No	Rev.
1:2500	88-905-21-005	0

Appendix T.5.3

Perry Creek Sections

(6 Sections)

41,800, 42000 to 44000 @ 500 m intervals


21500
22500
23500
24500
1600
1400
1200
1000
800
600
400
ELEVATION IN METRES
SW
NE
PERRY CREEK ANTICLINE
PERRY CREEK SYNCLINE
KCG
D3
E4
G1
J1,2&3
KCG
KM
KG
G1
J1,2&3
KCG
KM
KG
?
KCB
Boulder Creek Formation
Gates Formation
SYNCLINE
DRILL HOLE
R87003
100
200
400
METRES
03 88
TW
TW
DJ
Rev
D
M
Y
Revision Description
Drn.
Des.
App.
QUINTETTE COAL LIMITED
Project Manager
DENISON MINES LIMITED
COAL DIVISION
Area PERRY CREEK
Category CROSS-SECTIONS
Drawing Title
CROSS-SECTION
41800
Scale
1:5000
Drawing No
88-906-21-001
Rev.
0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QPR87003 DRILL HOLE

0 100 200 400
METRES

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	31	03	88	ORIGINAL DRAFT	TW	TW	DJ

QUINTETTE COAL LIMITED

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DENISON MINES LIMITED

COAL DIVISION

Area PERRY CREEK

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 42000

Scale	Drawing No.	Rev.
1:5000	88-906-21-002	0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QPR 87003 DRILL HOLE

100 200 400
METRES

Rev	D	M	Y	Revision Description	Drn.	Des.	App.
0	31	03	88	ORIGINAL DRAFT	TW	TW	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area PERRY CREEK

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION
42500

Scale 1:5000

Drawing No 88-906-21-003

Rev. 0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

COAL SEAM
FORMATION CONTACT
FAULT
SYNCLINE
ANTICLINE
QPR 87003 DRILL HOLE

0 100 200 400
METRES

0	31	03	88	ORIGINAL DRAFT	TW	TW	DJ
Rev.	D	M	Y	Revision Description	Drn.	Des.	App.

QUINTETTE COAL LIMITED

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DENISON MINES LIMITED

COAL DIVISION

Area PERRY CREEK

Category CROSS-SECTIONS

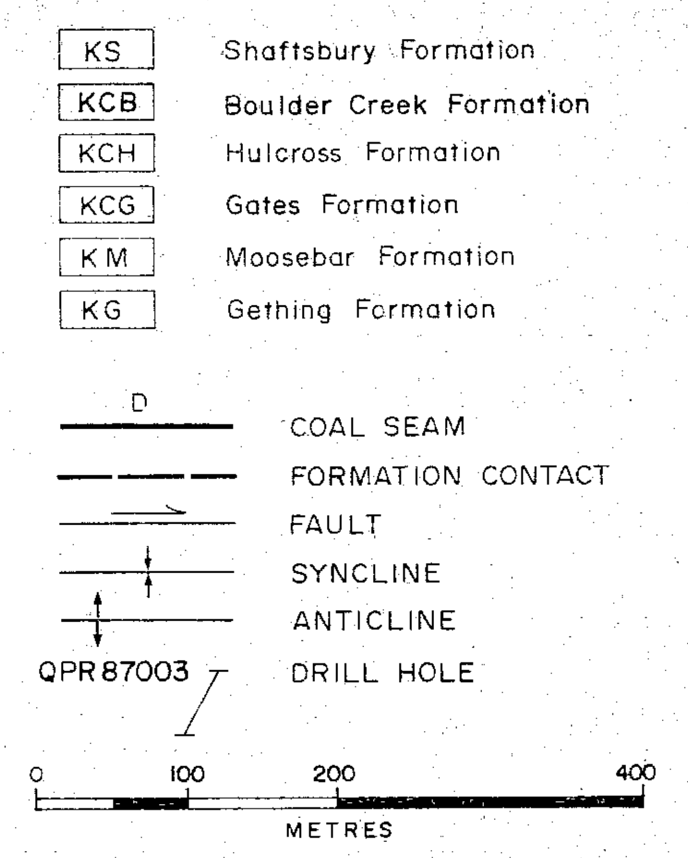
Drawing Title

CROSS-SECTION
43000

Scale 1:5000

Drawing No 88-906-21-004

Rev. 0



Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	31	03	88	ORIGINAL DRAFT	TW	TW	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area PERRY CREEK | Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 43500

Scale	Drawing No.	Rev.
1:5000	88-906-21-005	0

KS	Shaftsbury Formation
KCB	Boulder Creek Formation
KCH	Hulcross Formation
KCG	Gates Formation
KM	Moosebar Formation
KG	Gething Formation

- COAL SEAM
- FORMATION CONTACT
- FAULT
- SYNCLINE
- ANTICLINE
- QPR87003 DRILL HOLE

0 100 200 400
METRES

Rev.	D	M	Y	Revision Description	Drn.	Des.	App.
0	31	03	88	ORIGINAL DRAFT	TW	TW	DJ

QUINTETTE COAL LIMITED

Project Manager

DENISON MINES LIMITED

COAL DIVISION

Area PERRY CREEK

Category CROSS-SECTIONS

Drawing Title

CROSS-SECTION 44000

Scale 1:5000

Drawing No. 88-906-21-006

Rev. 0

VERTICAL DEVIATION

COMPU-LOG V8L1 DEVIATION
DATA FROM : V8L2*A

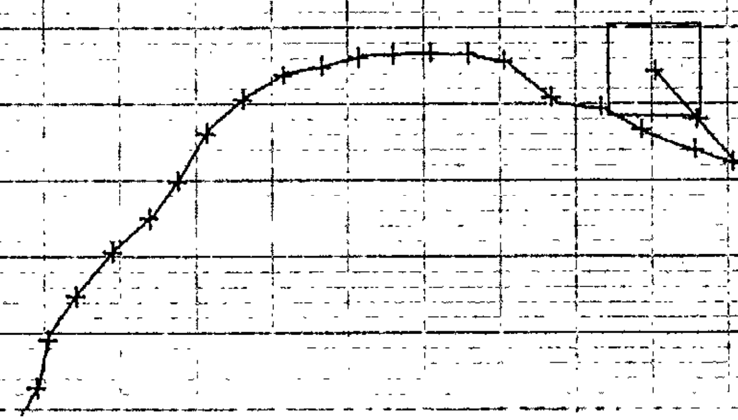
739

CLIENT : GAMMA NEUTRON DEV.
LOCATION : WOLV. VALLEY
HOLE ID : QPR87005
DATE OF LOG : 10-15-80
PROBE : 9055A 0079

SCALE: .10 M/DIV
MAG DECL: 24.5
TRUE DEPTH: 43.3 M
AZIMUTH: 241.8
DISTANCE: .94 M

+ = 2.0 M INCR
△ = TOP OF ZONE
◇ = BOTTOM OF ZONE

TRUE NORTH ↑



CENTURY GEOPHYSICAL CORP. PART N

277

739

COMPU-LOG V8L2 PLOT 11-13-80

QMR87135

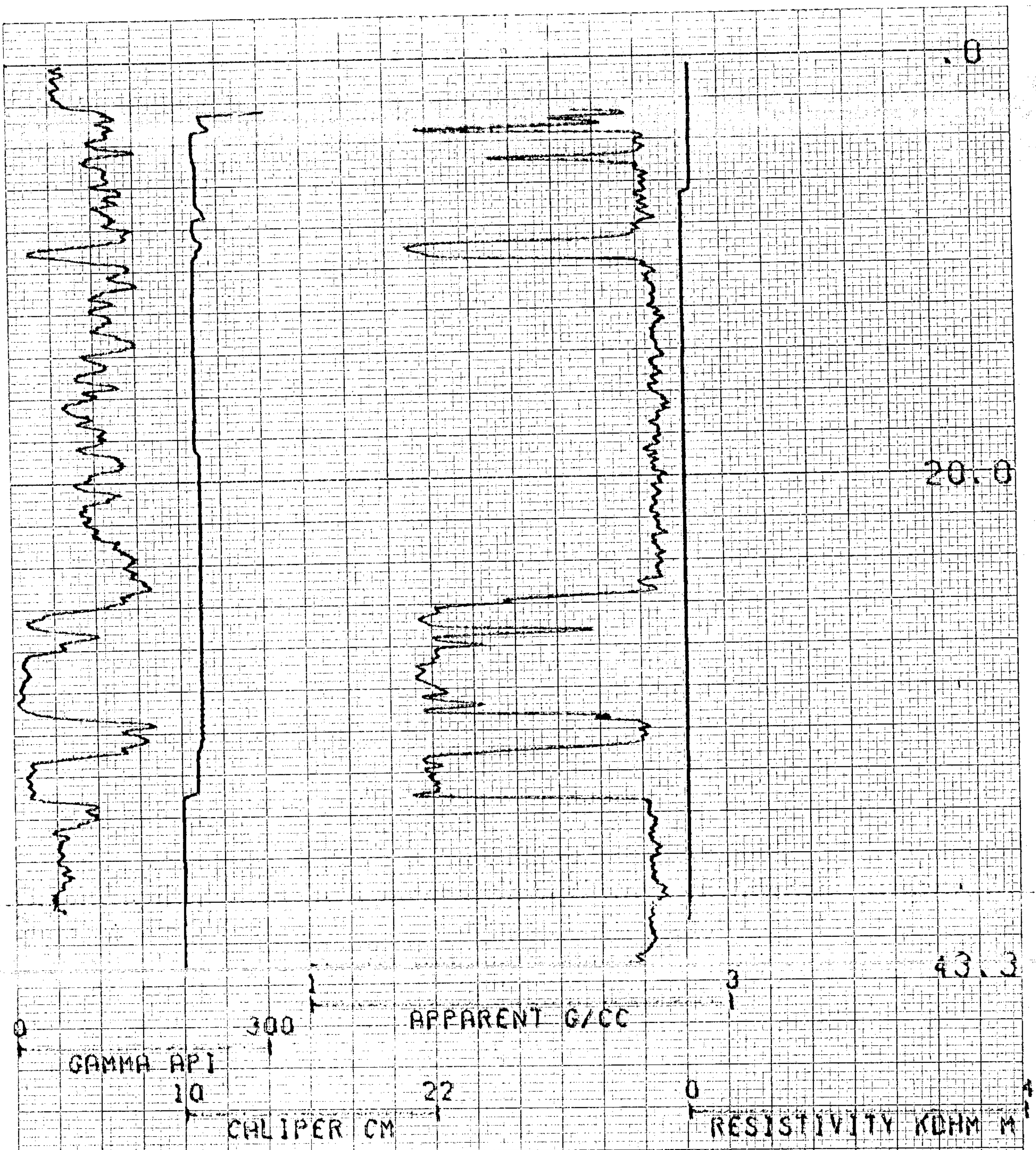
GAMMA NEUTRON RODS

SED-A FAILURE AREA

HOLE DIAMETER : 13.0
PROBE # 9055A - 079
SENSOR #4 CAL STD CPS = 152
SENSOR #4 CAL RUN CPS = 186
SENSOR #4 CAL BIAS = 0
DATA V8L2 TRUCK # L600
G.HOLMLUND APPL.#7 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040
CENTURY GEOPHYSICAL CORP. PART NO. 786-0040
CENTURY GEOPHYSICAL CORP. PART NO. 786-0040
CENTURY GEOPHYSICAL CORP. PART NO. 786-0040
CENTURY GEOPHYSICAL

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040



CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

445

739

COMPU-LOG V8L2 PLOT 10-15-80

QPR87005

GAMMA DENSITY CALIPR

WOLV. VALLEY

HOLE DIAMETER : 13.1

PROBE # 9030A1- 403

SENSOR #1 CAL STD CPS = 6588

SENSOR #1 CAL RUN CPS = 7500

SENSOR #1 CAL BIAS = 98

DATA V8L2*A TRUCK # 601

M.HEALY APPL.#30 L1

443

CENTURY GEOF

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

739

COMPU-LOG V8L2 PLOT 11-12-80

QMR87135

GAMMA NEUTRON RODS

SED A

HOLE DIAMETER : 13.1

PROBE # 9055A - 079

SENSOR #4 CAL STD CPS = 152

SENSOR #4 CAL RUN CPS = 186

SENSOR #4 CAL BIAS = 0

DATA V8L2 TRUCK # 600

M.HEALY APPL.#7 L1

739

COMPU-LOG V8L2 PLOT 11-18-80

QMR87135
GAMMA NEUTRON DEV'N
SED-A FAILURE AREA

HOLE DIAMETER : 13.0
PROBE # 9055A - 079
SENSOR #4 CAL STD CPS = 152
SENSOR #4 CAL RUN CPS = 186
SENSOR #4 CAL BIAS = 0
DATA V8L2*A TRUCK # L600
G.HOLMLUND APPL.#7 L1

VERTICAL DEVIATION

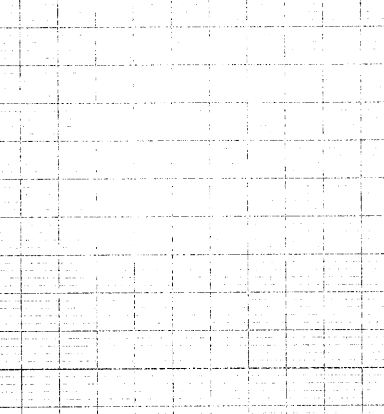
COMPU-LOG V8L1 DEVIATION
DATA FROM : V8L2*A

CLIENT : GAMMA NEUTRON DEV'N
LOCATION : SED-A FAILURE AREA
HOLE ID : QMR87135
DATE OF LOG : 11-18-80
PROBE : 9055A 0079

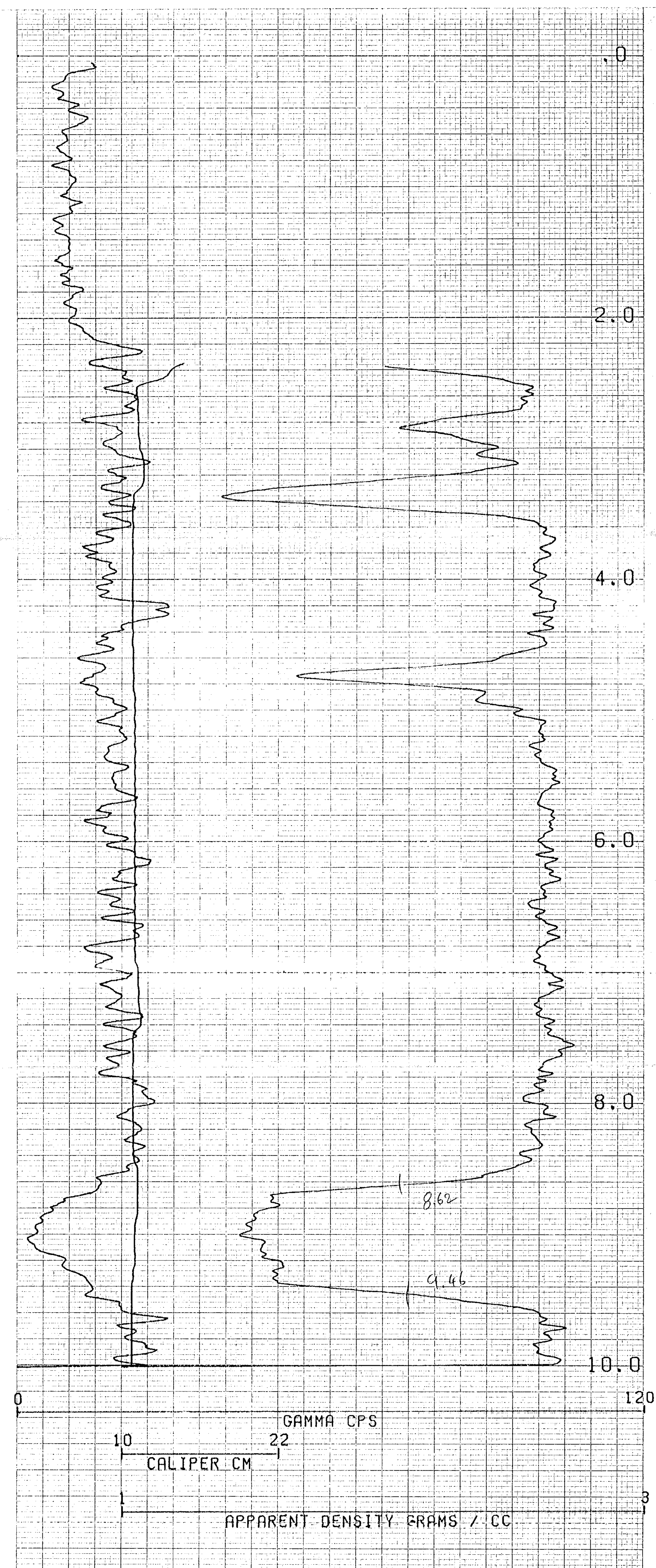
SCALE: 5.00 M/DIV
MAG DECL: 24.5
TRUE DEPTH: 283.2 M
AZIMUTH: 318.6
DISTANCE: 53.44 M

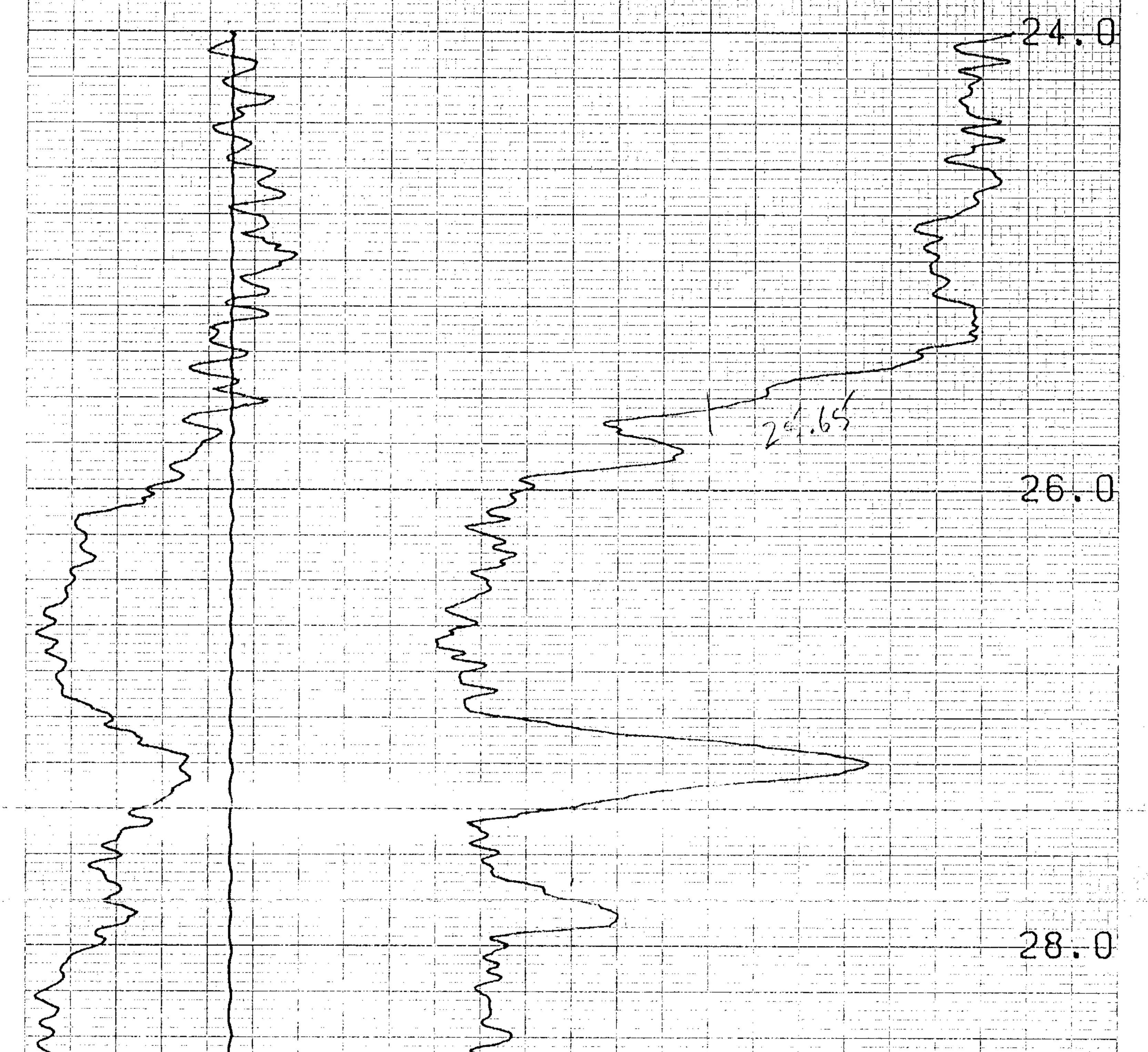
+ = 10.0 M INCR
Δ = TOP OF ZONE
◊ = BOTTOM OF ZONE

TRUE NORTH ↑



CENTURY GEOPHYSICAL CORP. PART NO. 786-0040





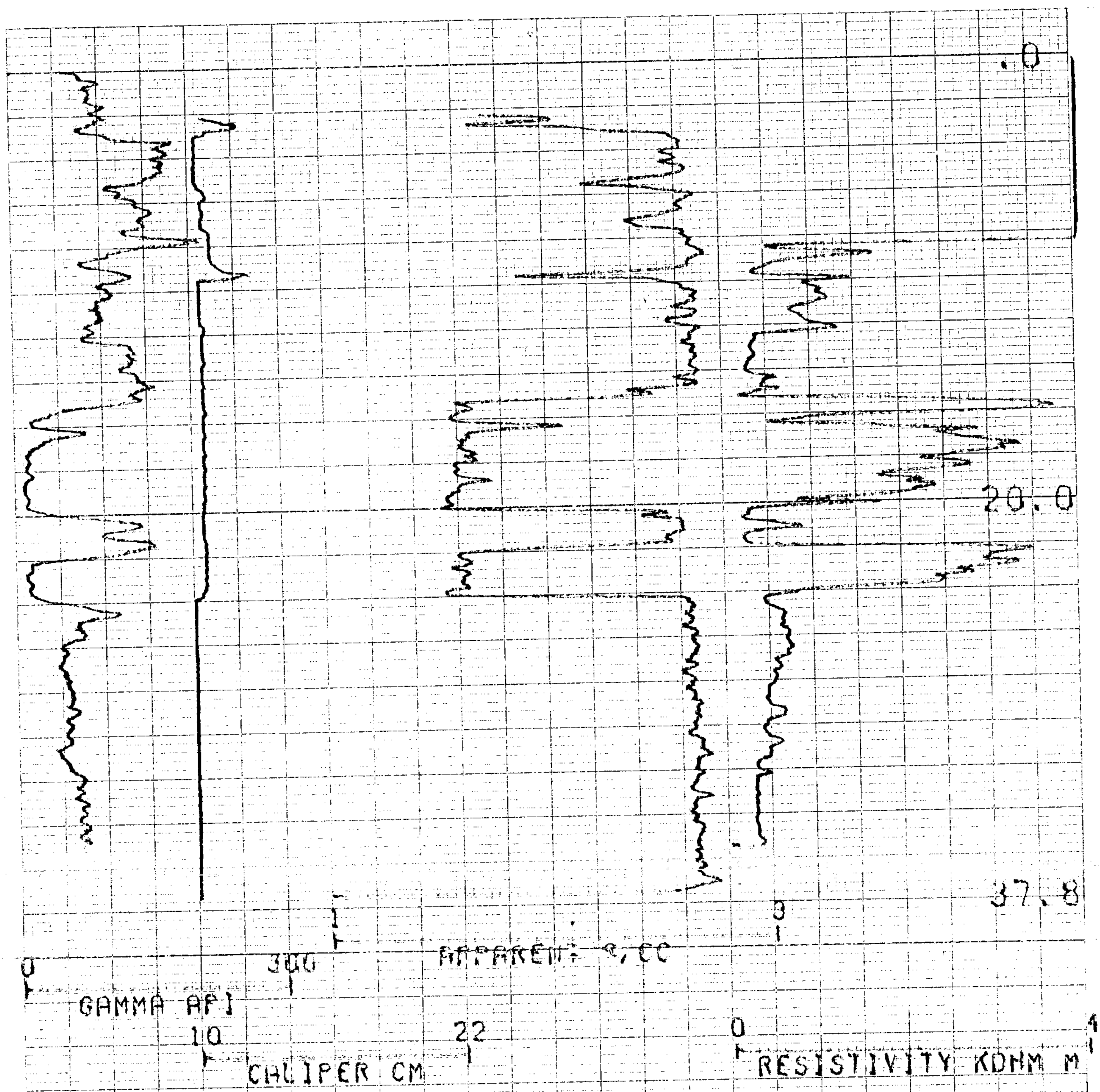
739

COMPU-LOG V8L2 PLOT 10-15-80

QPR87005
GAMMA DENSITY CALIPR
WOLV. VALLEY

HOLE DIAMETER : 13.1
PROBE # 9030A1- 403
SENSOR #4 CAL STD CPS = 6588
SENSOR #4 CAL RUN CPS = 7500
SENSOR #4 CAL BIAS = 98
DATA V8L2*A TRUCK # 604
M.HEALY APPL.#1708L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040



739

COMPU-LOG V8L2 PLOT 10-15-80

QPR87004

GAMMA DENSITY CALIPR

WOLV. VALLEY

HOLE DIAMETER : 13.1

PROBE # 9030A1- 103

SENSOR #1 CAL STD CPS = 6588

SENSOR #1 CAL RUN CPS = 7500

SENSOR #1 CAL BIAS = 98

DATA V8L2*A TRUCK # 601

M.HEALY APPL.#30 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040
CENTURY GEOPHYSICAL CORP. PART NO. 786-0040
CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

VERTICAL DEVIATION

COMPU-LOG V8L1 DEVIATION
DATA FROM : V8L2*A

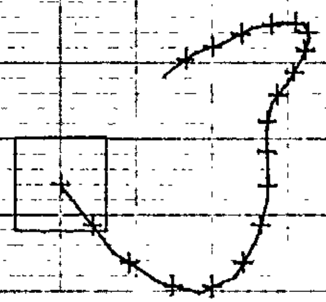
739

CLIENT : GAMMA NEUTRON DEV.
LOCATION : WOLV. VALLEY
HOLE ID : QPR87004
DATE OF LOG : 10-15-80
PROBE : 9055A 0079

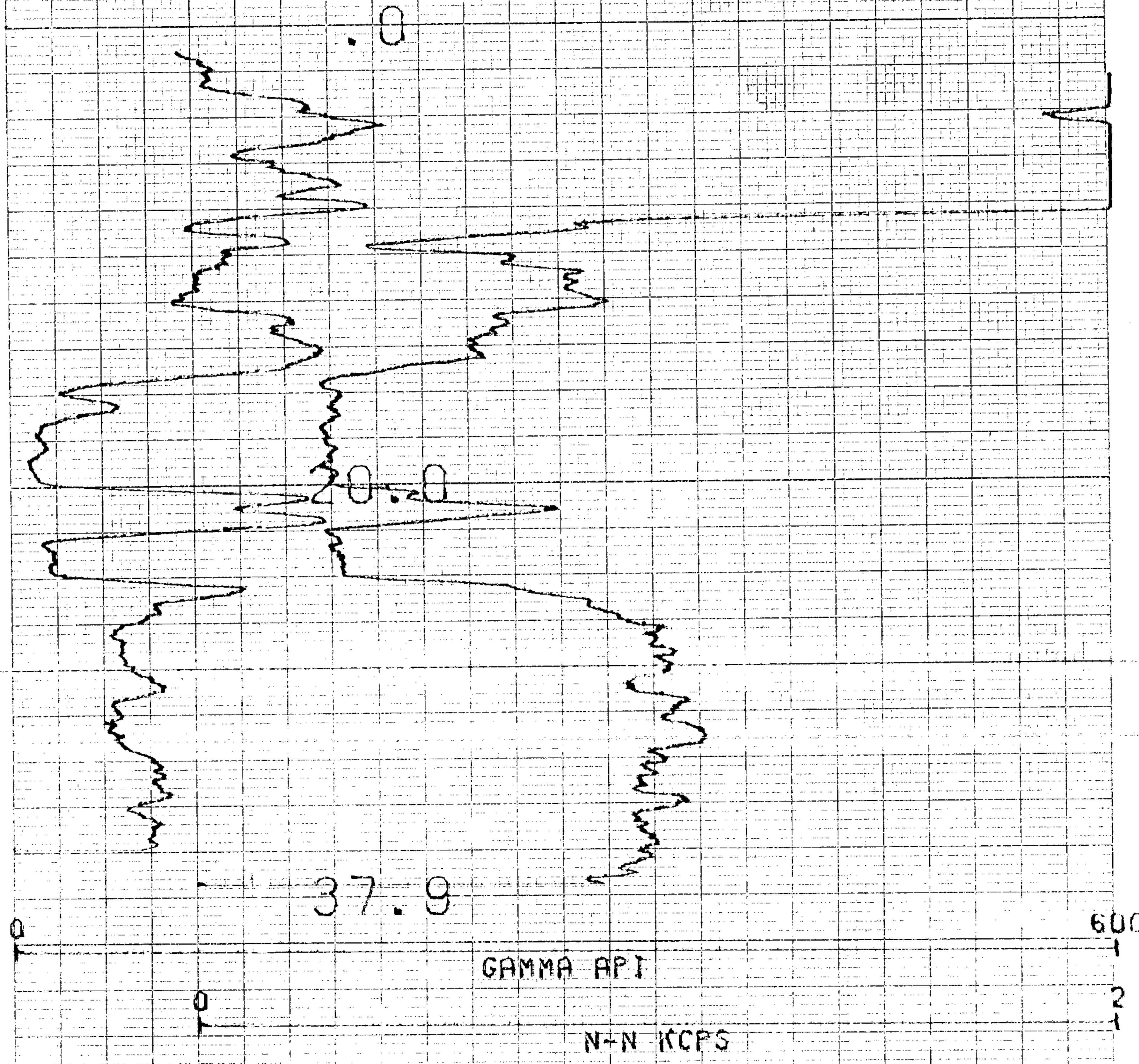
SCALE: .10 M/DIV
MAG DECL: 24.5
TRUE DEPTH: 37.8 M
AZIMUTH: 43.4
DISTANCE: .19 M

\+ = 2.0 M INCR
△ = TOP OF ZONE
◇ = BOTTOM OF ZONE

TRUE NORTH ↑



CENTURY GEOPHYSICAL CORP. PART NO. 786-0040



CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

COMPU-LOG V8L2 PLOT 10-15-80

739

QPR87004

GAMMA NEUTRON DEV.

WOLV. VALLEY

HOLE DIAMETER : 13.1
PROBE # 9055A - 079
SENSOR #1 CAL STD CPS = 152
SENSOR #1 CAL RUN CPS = 186
SENSOR #1 CAL BIAS = 0
DATA V8L2*A TRUCK # 601
M.HEALY APPL.#7 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

/ 739

CENTURY GEOPHYSICAL CORPORATION

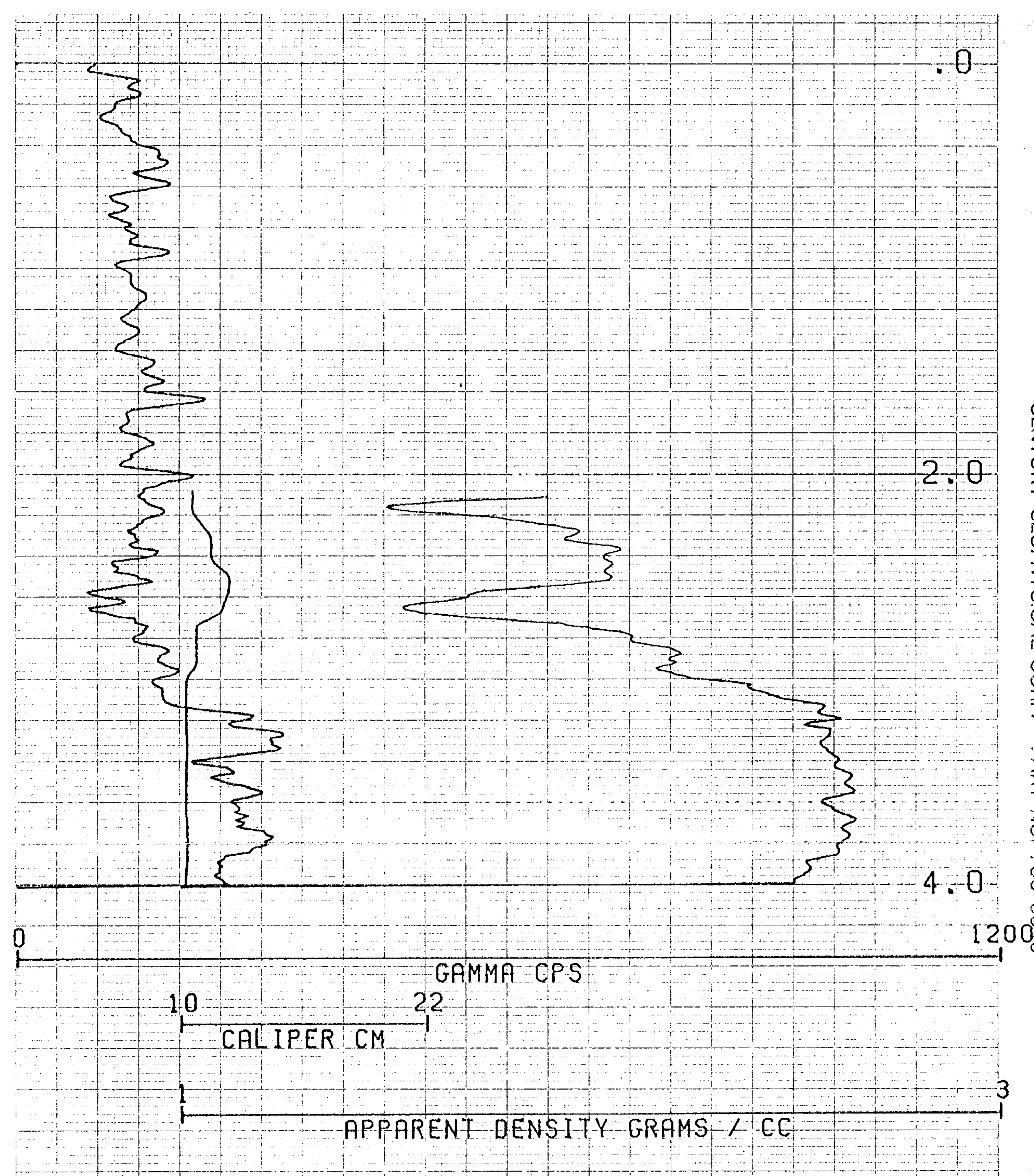
* * * * * * * * VERTICAL DEVIATION * * * * * * * *

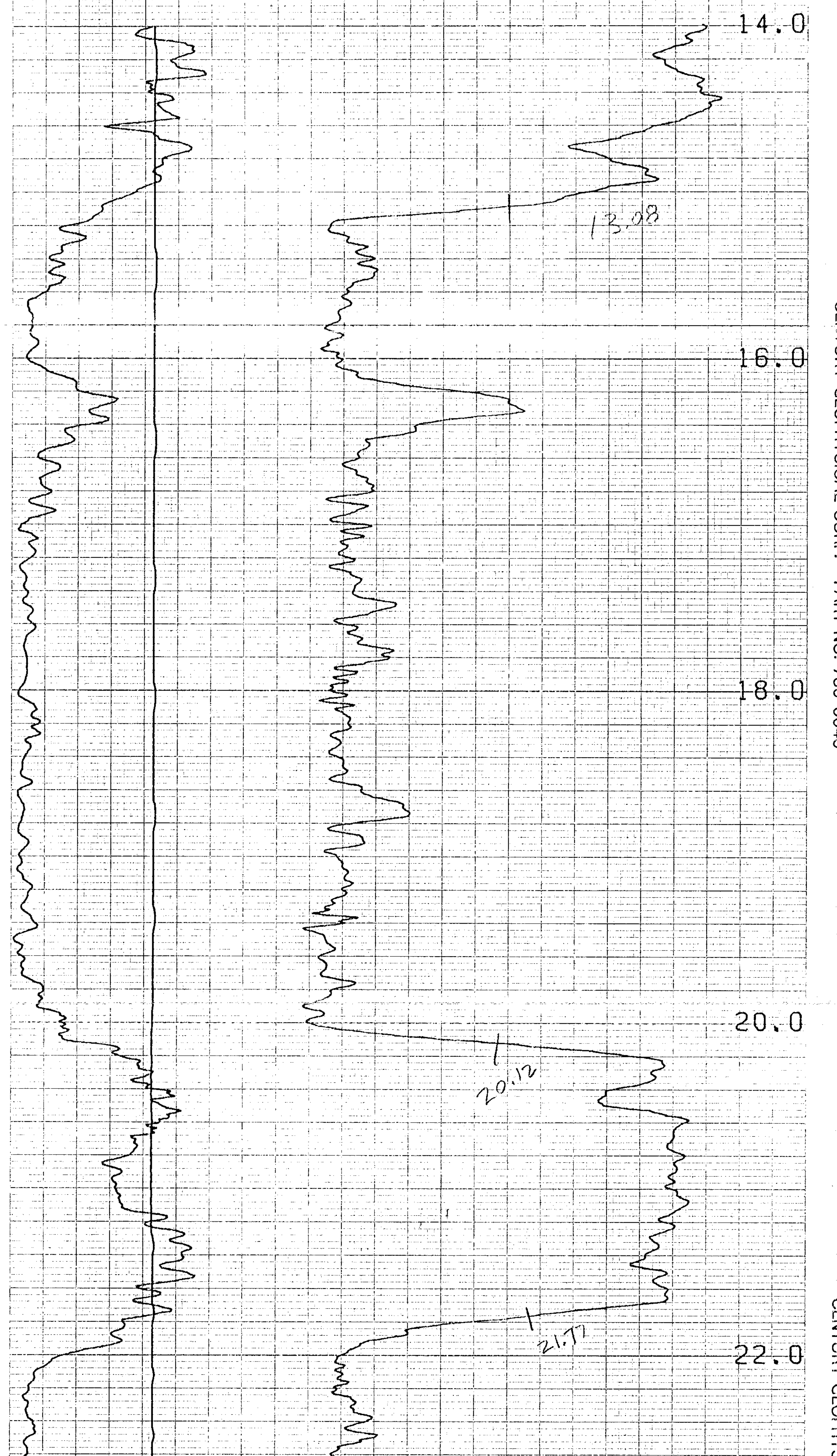
COMPU-LOG VSL1 DEVIATION

CLIENT : GAMMA NEUTRON DEV. HOLE ID : OPR87005
LOCATION : WOLV. VALLEY DATE OF LOG : 10-15-80
DATA FROM : VSL2*A PROBE : 9055A 0079

TD = TOTAL DEPTH
T = TOP OF ZONE
B = BOTTOM OF ZONE

	DEPTH	TRUE DEPTH	NORTH DEV	EAST DEV	DISTANCE	AZIMUTH	SA	SAB
	.00	.00	.00	.00	.00	.0	.0	.0
	2.00	1.99	-.06	.05	.08	138.6	2.4	138.6
	4.00	3.99	-.12	.10	.15	139.6	2.0	140.6
	6.00	5.99	-.10	.05	.11	153.2	1.5	288.2
	8.00	7.99	-.07	-.01	.07	193.3	2.1	290.4
	10.00	9.99	-.04	-.07	.08	235.5	1.7	298.2
	12.00	11.99	-.03	-.13	.14	255.3	1.8	281.6
	14.00	13.98	.01	-.19	.19	273.8	2.2	308.0
	16.00	15.98	.02	-.24	.24	275.3	1.3	281.8
	18.00	17.98	.02	-.29	.29	274.7	1.4	271.3
	20.00	19.98	.02	-.34	.34	273.6	1.4	267.2
	22.00	21.98	.01	-.38	.38	272.7	1.2	265.8
	24.00	23.98	.00	-.43	.43	270.8	1.4	255.7
	26.00	25.98	-.00	-.48	.48	269.4	1.4	257.1
	28.00	27.98	-.03	-.53	.54	266.0	1.7	238.2
	30.00	29.98	-.08	-.58	.59	262.1	1.8	227.8
	32.00	31.98	-.14	-.62	.64	257.0	2.1	211.3
	34.00	33.98	-.19	-.66	.69	253.8	1.7	216.8
	36.00	35.97	-.23	-.71	.75	251.6	1.8	228.2
	38.00	37.97	-.29	-.76	.81	248.9	2.1	220.4
	40.00	39.97	-.35	-.79	.87	246.2	1.9	212.6
	42.00	41.97	-.41	-.81	.91	243.0	1.8	193.7
TD	43.30	43.27	-.45	-.83	.94	241.6	1.8	209.4





739

COMPU-LOG V8L2 PLOT 10-15-80

QPR87004

GAMMA DENSITY CALIPR

WOLV. VALLEY

HOLE DIAMETER : 13.1

PROBE # 9030A1- 403

SENSOR #4 CAL STD CPS = 6588

SENSOR #4 CAL RUN CPS = 7500

SENSOR #4 CAL BIAS = 98

DATA V8L2*A TRUCK # 604

M.HEALY APPL.#1708L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

739

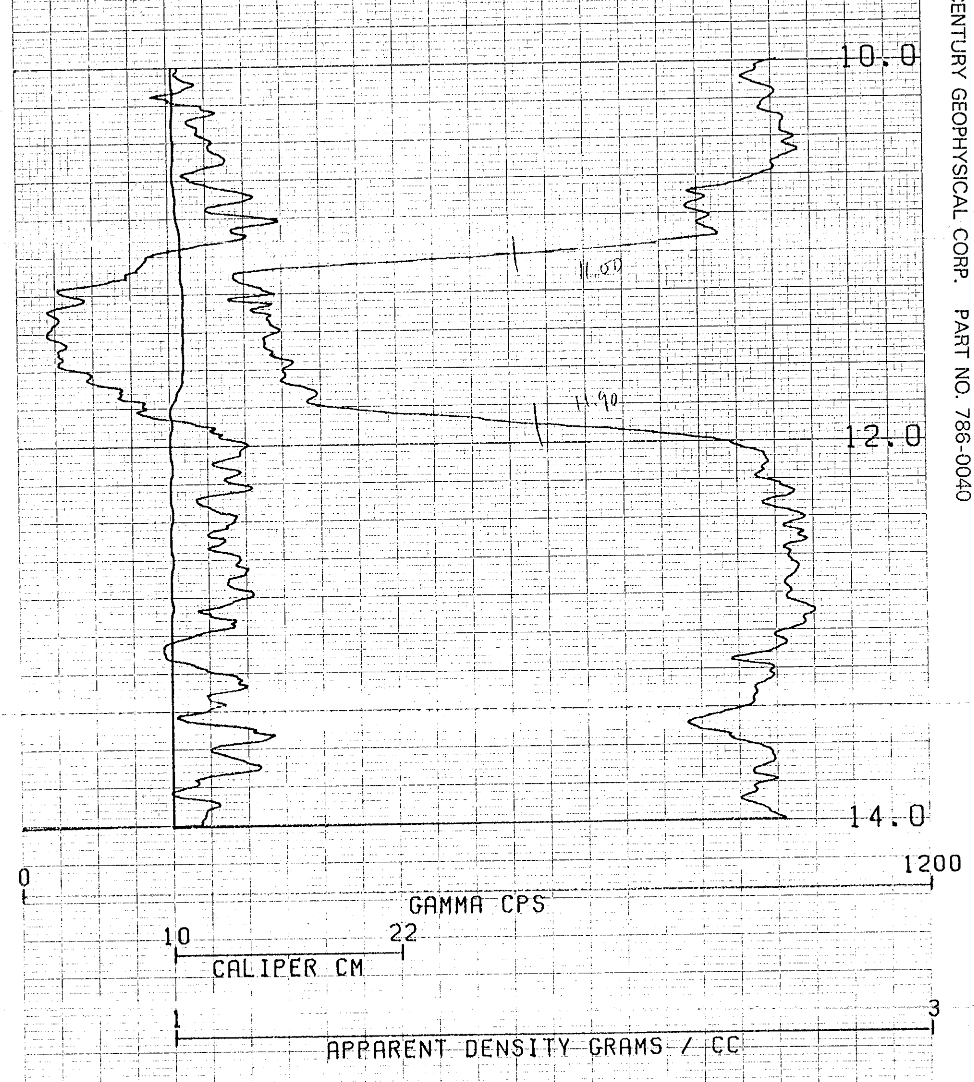
CENTURY GEOPHYSICAL CORPORATION
* * * * * * * * VERTICAL DEVIATION * * * * * * * *
COMPU-LOG V8L1 DEVIATION

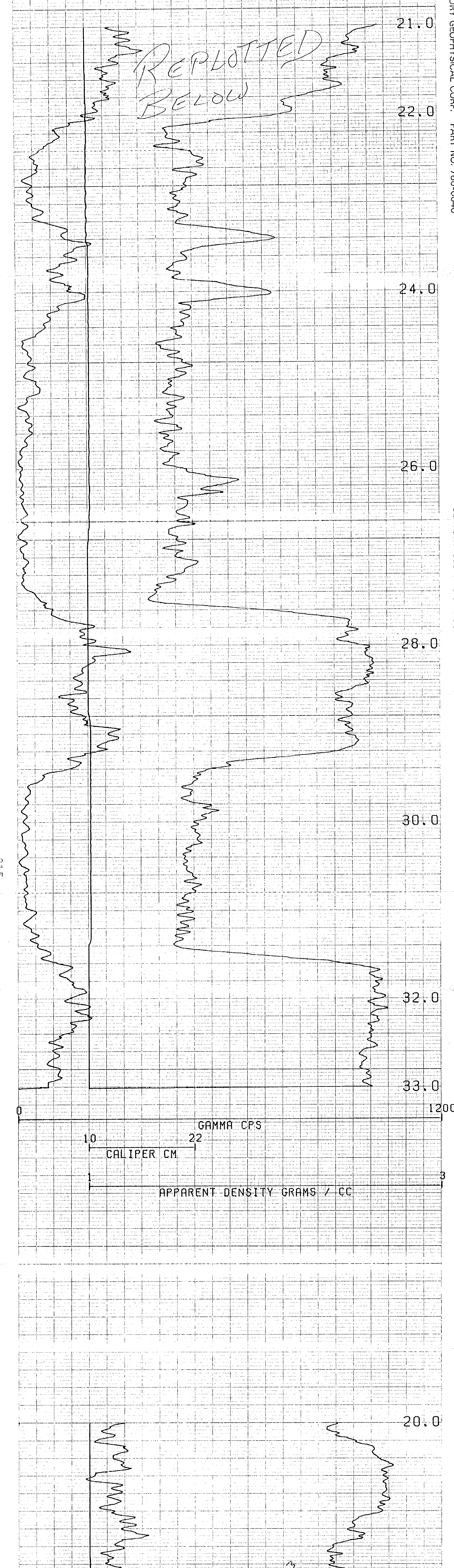
CLIENT : GAMMA NEUTRON DEV.
LOCATION : WOLV. VALLEY
DATA FROM : V8L2*A

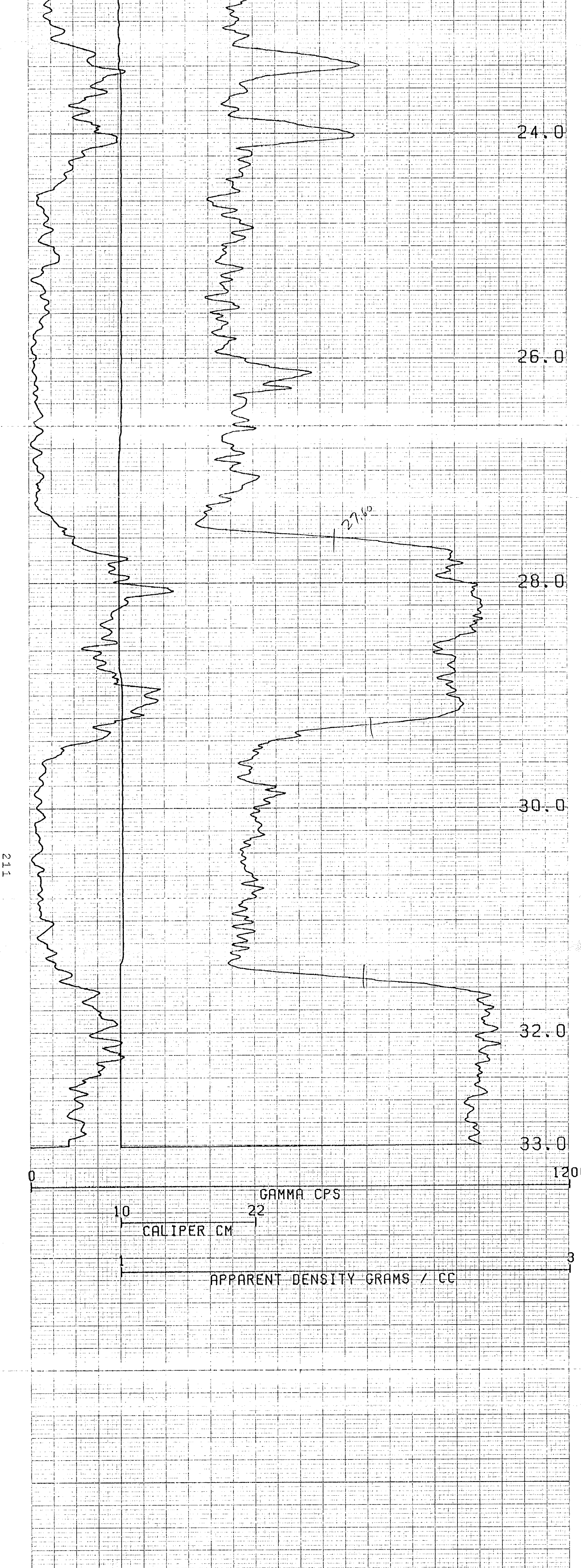
HOLE ID : QPR87004
DATE OF LOG : 10-15-80
PROBE : 9055A 0079

TD = TOTAL DEPTH
T = TOP OF ZONE
B = BOTTOM OF ZONE

	DEPTH	TRUE DEPTH	NORTH DEV	EAST DEV	DISTANCE	AZIMUTH	SA	SAB
	.00	.00	.00	.00	.00	.0	.0	.0
	2.00	1.99	-.05	.04	.06	141.9	1.9	141.8
	4.00	3.99	-.10	.09	.13	138.6	1.9	135.3
	6.00	5.99	-.13	.14	.19	132.1	1.8	117.9
	8.00	7.99	-.13	.19	.24	124.1	1.4	91.9
	10.00	9.99	-.10	.24	.26	113.5	1.4	52.8
	12.00	11.99	-.05	.26	.26	101.7	1.5	24.8
	14.00	13.99	-.00	.27	.27	90.9	1.4	10.2
	16.00	15.99	.04	.27	.27	81.2	1.3	357.9
	18.00	17.99	.08	.27	.28	73.1	1.1	2.3
	20.00	19.99	.11	.28	.30	67.7	1.0	21.5
	22.00	21.99	.14	.30	.34	64.4	1.0	35.3
	24.00	23.99	.17	.32	.36	61.3	.9	26.8
	26.00	25.99	.19	.32	.38	58.4	.6	5.9
	28.00	27.99	.21	.30	.37	55.5	.5	309.7
	30.00	29.99	.21	.27	.34	52.8	.9	266.5
	32.00	31.99	.19	.23	.30	50.2	1.1	252.7
	34.00	33.98	.18	.20	.26	47.9	1.1	245.5
	36.00	35.98	.16	.16	.23	45.0	1.1	245.2
TD	37.80	37.78	.14	.13	.19	43.4	1.1	233.8







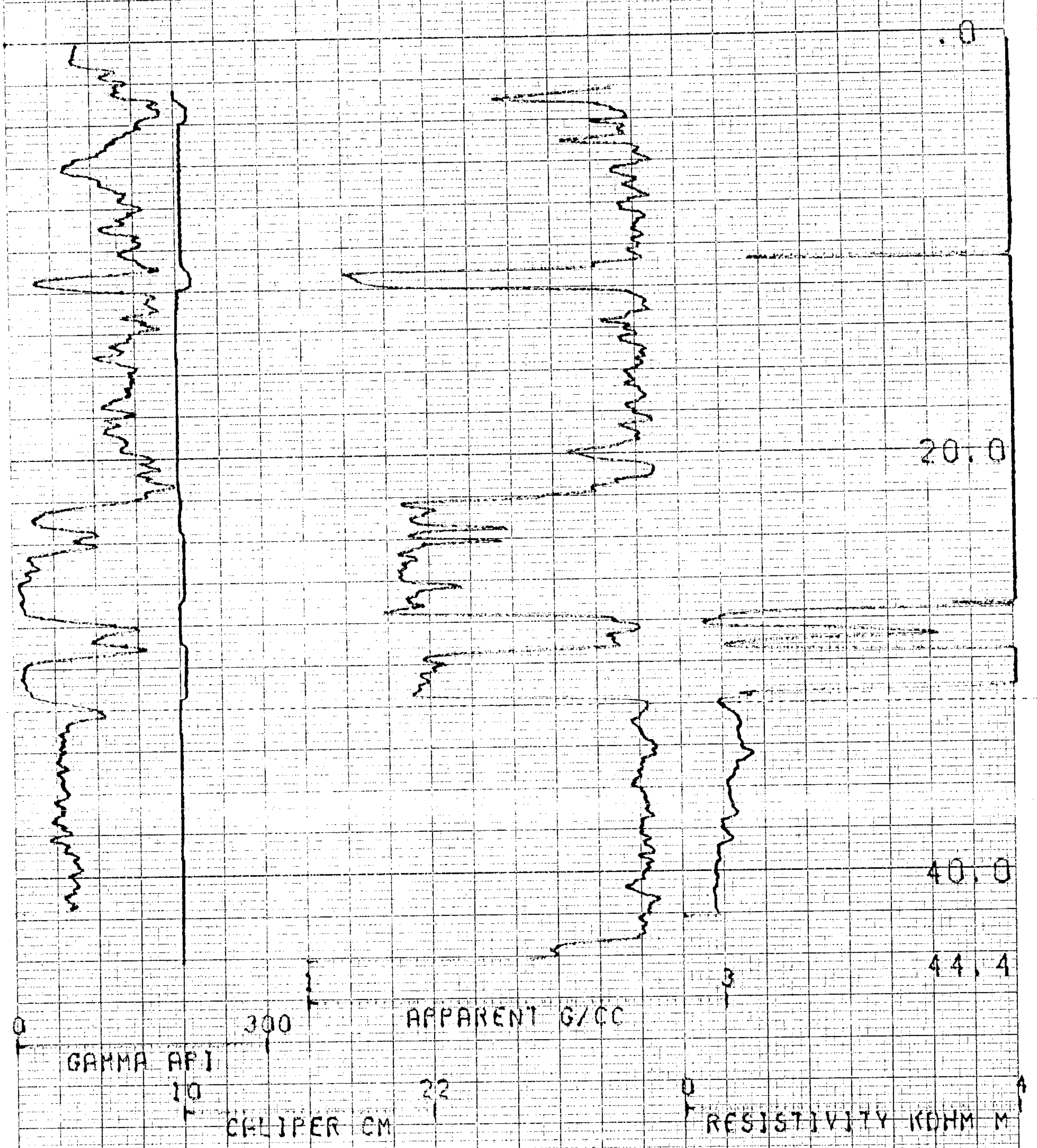
COMPU-LOG V8L2 PLOT 10-15-80

739

QPR87003
GAMMA DENSITY CALIPR
WOLV. VALLEY

HOLE DIAMETER : 13.1
PROBE # 9030A1- 403
SENSOR #4 CAL STD CPS = 6588
SENSOR #4 CAL RUN CPS = 7500
SENSOR #4 CAL BIAS = 98
DATA V8L2*A TRUCK # 604
M.HEALY APPL.#1708L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

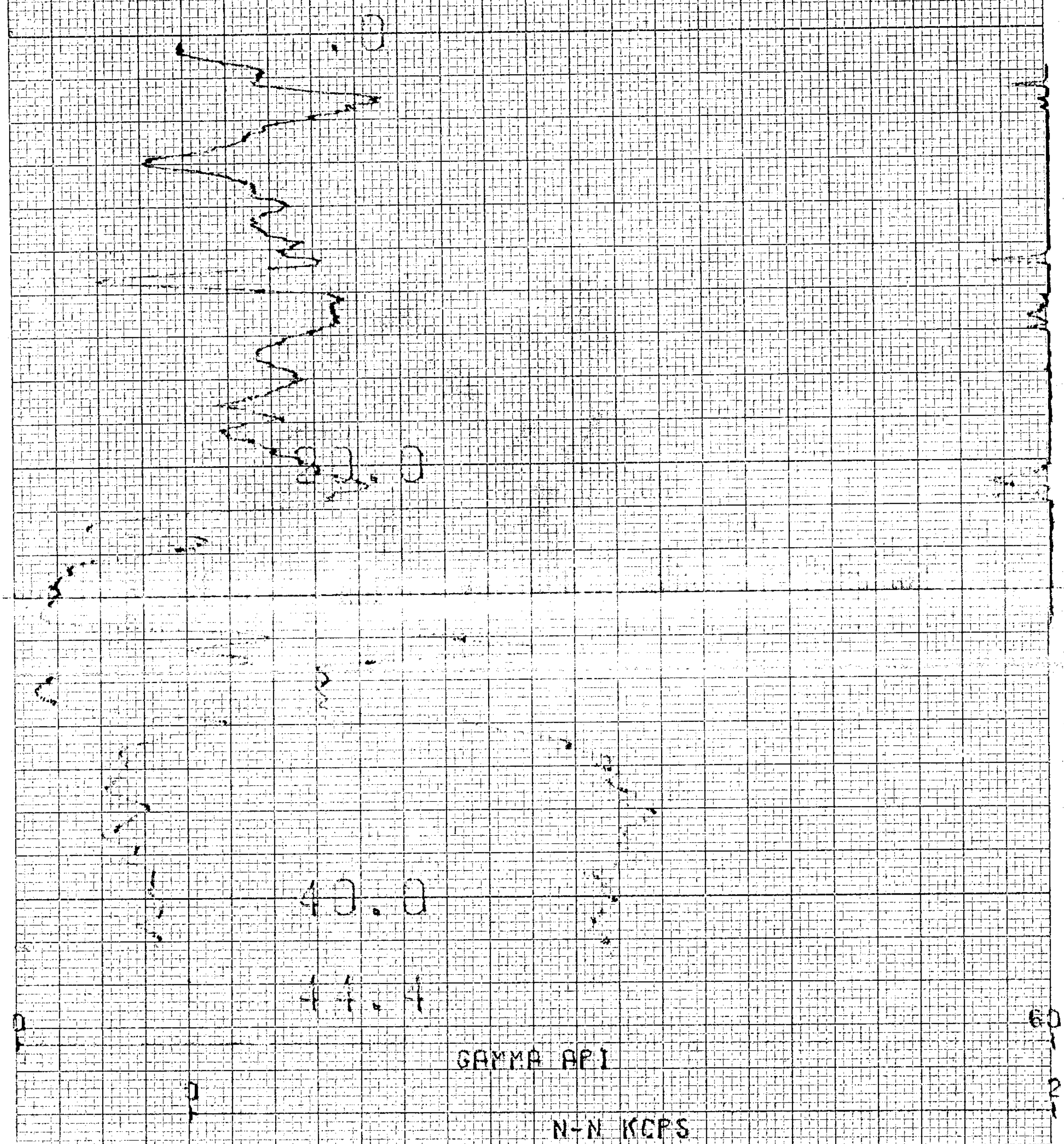


739

COMPU-LOG V8L2 PLOT 10-15-80

QPR87003
GAMMA DENSITY CALIPR
WOLV. VALLEY
HOLE DIAMETER : 13.1
PROBE # 9030A1- 403
SENSOR #1 CAL STD CPS = 6588
SENSOR #1 CAL RUN CPS = 7500
SENSOR #1 CAL BIAS = 98
DATA V8L2*A TRUCK # 601
M.HEALY APPL.#30 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040



739

COMPU-LOG V8L2 PLOT 10-15-80

QPRS7003

GAMMA NEUTRON DEV.

WOLV. VALLEY

HOLE DIAMETER : 13.1

PROBE # 9055A - 079

SENSOR #1 CAL STD CPS = 152

SENSOR #1 CAL RUN CPS = 186

SENSOR #1 CAL BIAS = 0

DATA V8L2*A TRUCK # 601

M. HEALY APPL. #7 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

/ 739

CENTURY GEOPHYSICAL CORPORATION

* * * * * * * * VERTICAL DEVIATION * * * * * * * *

COMPU-LOG V8L1 DEVIATION

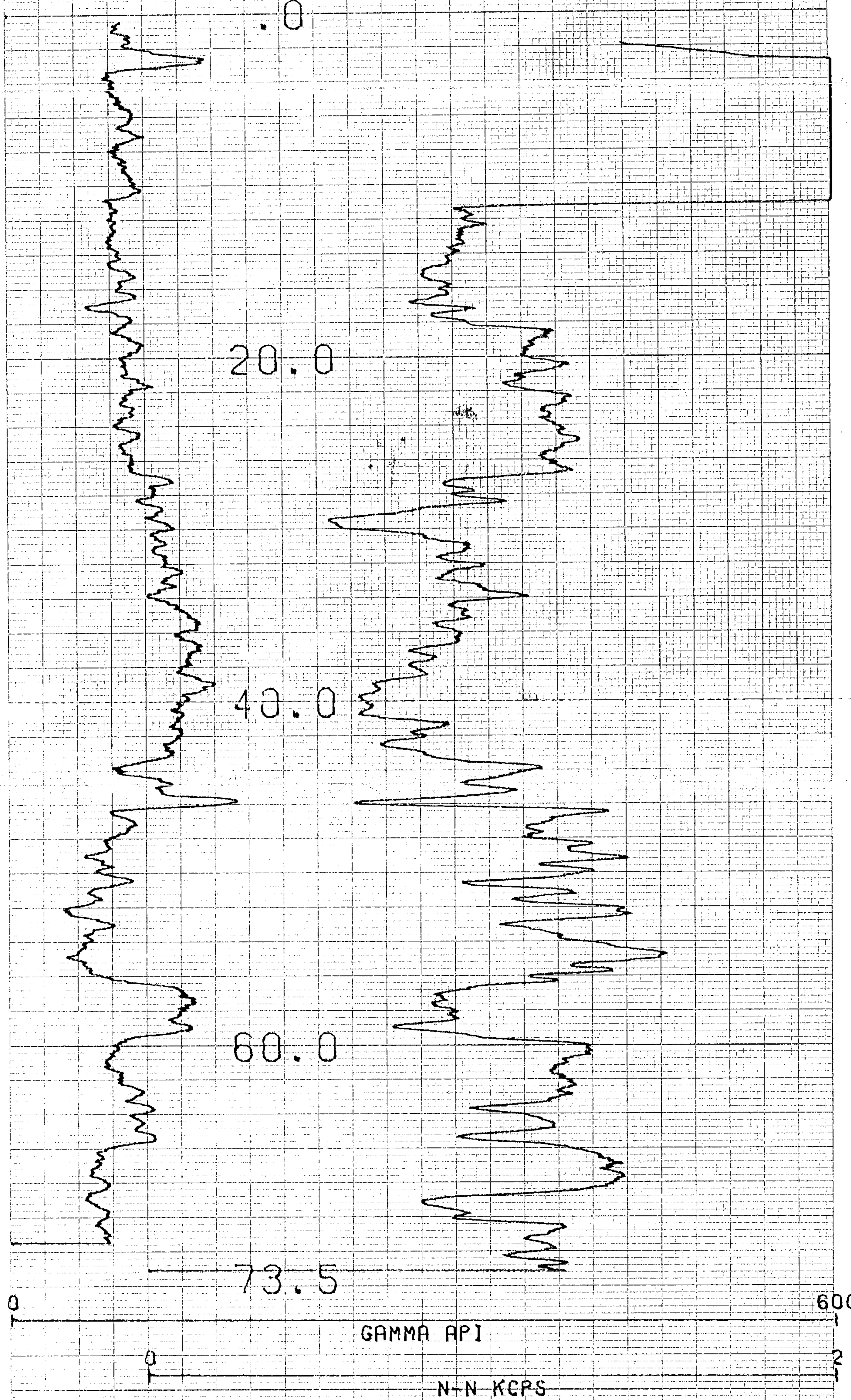
CLIENT : GAMMA NEUTRON DEV. HOLE ID : QFR87003

LOCATION : WOLV.VALLEY DATE OF LOG : 10-15-80

DATA FROM : V8L2*A PROBE : 9055A 0079

TD = TOTAL DEPTH
T = TOP OF ZONE
B = BOTTOM OF ZONE

	DEPTH	TRUE DEPTH	NORTH DEV	EAST DEV	DISTANCE	AZIMUTH	SA	SAB
	.00	.00	.00	.00	.00	.0	.0	.0
	2.00	1.99	.05	.07	.09	56.3	2.6	56.3
	4.00	3.99	.11	.15	.19	54.0	2.8	51.7
	6.00	5.99	.18	.22	.29	50.3	2.9	43.5
	8.00	7.99	.24	.29	.38	49.4	2.5	46.1
	10.00	9.98	.30	.35	.47	49.5	2.5	49.8
	12.00	11.98	.37	.43	.56	49.3	2.7	48.5
	14.00	13.98	.42	.51	.66	51.0	2.8	60.6
	16.00	15.98	.47	.59	.76	51.3	2.8	53.2
	18.00	17.97	.53	.67	.85	51.6	2.5	53.8
	20.00	19.97	.57	.74	.93	52.1	2.3	58.0
	22.00	21.97	.61	.80	1.01	52.8	2.1	61.6
	24.00	23.97	.64	.86	1.07	53.3	1.9	60.5
	26.00	25.97	.67	.93	1.15	54.0	2.1	63.6
	28.00	27.97	.71	1.00	1.22	54.6	2.1	63.4
	30.00	29.96	.73	1.07	1.30	55.4	2.1	68.8
	32.00	31.96	.76	1.14	1.37	56.2	2.1	70.9
	34.00	33.96	.80	1.21	1.45	56.4	2.4	59.6
	36.00	35.96	.85	1.28	1.54	56.4	2.4	55.3
	38.00	37.96	.90	1.35	1.63	56.4	2.4	57.8
	40.00	39.96	.94	1.43	1.71	56.5	2.5	57.5
	42.00	41.95	.99	1.51	1.80	56.6	2.6	58.3
	44.00	43.95	1.05	1.59	1.90	56.6	2.7	55.9
TD	44.30	44.25	1.05	1.60	1.92	56.6	3.2	58.2



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COMPU-LOG V8L2 PLOT 10-15-80

~~QHR87059~~ QPR 87001

GAMMA NEUTRON DEV.

WOLV. VALLEY

HOLE DIAMETER : 13.1

PROBE # 9055A - 079

SENSOR #4 CAL STD CPS = 152

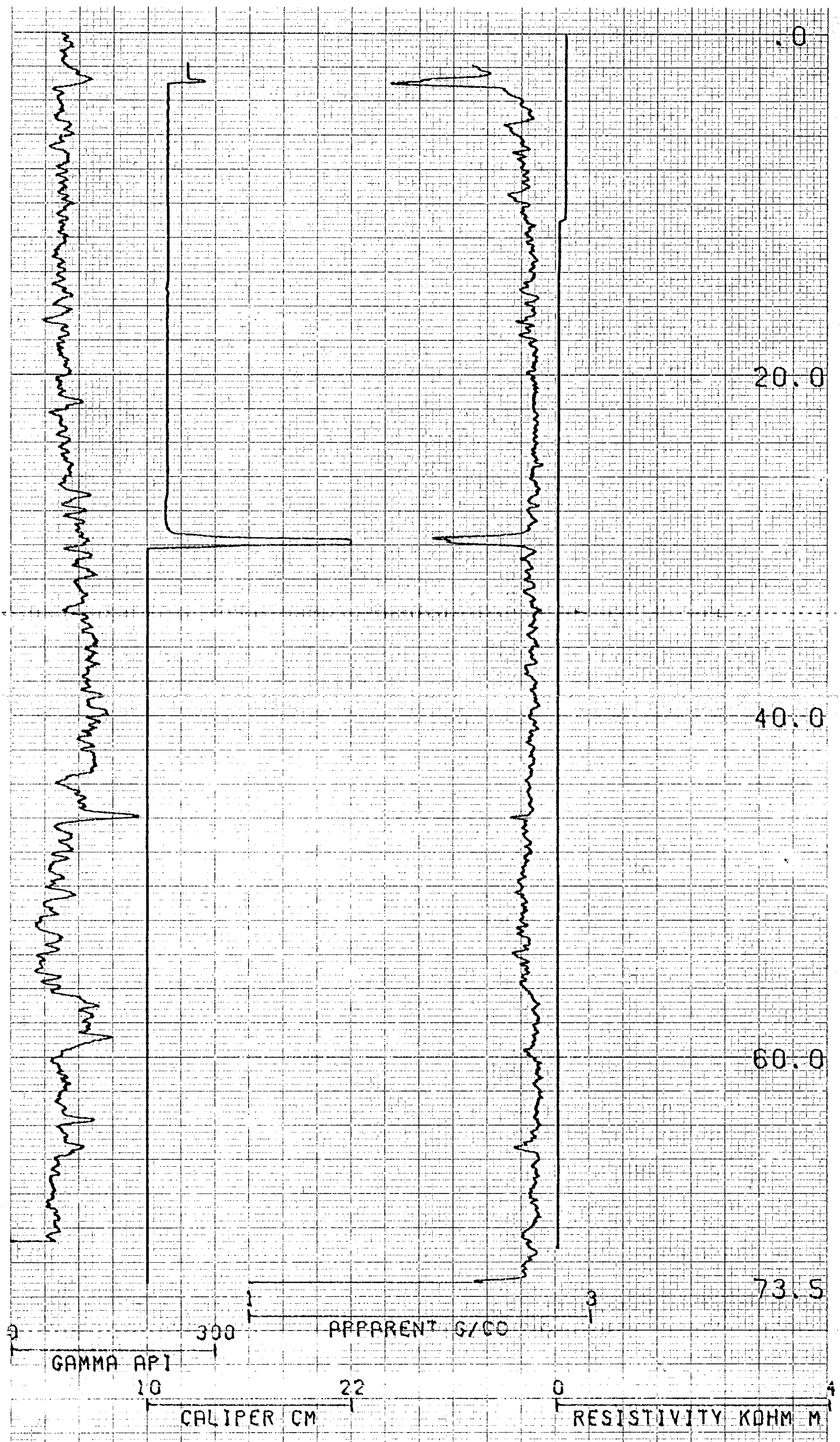
SENSOR #4 CAL RUN CPS = 186

SENSOR #4 CAL BIAS = 0

DATA V8L2*A TRUCK # 604

M.HEALY APPL.#7 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040



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COMPU-LOG V8L2 PLOT 10-15-80

~~QHR870459~~ QPR 87001

GAMMA DENSITY CALIPR

WOLV. VALLEY

HOLE DIAMETER : 13.1

PROBE # 9030A1- 403

SENSOR #4 CAL STD CPS = 6588

SENSOR #4 CAL RUN CPS = 7500

SENSOR #4 CAL BIAS = 98

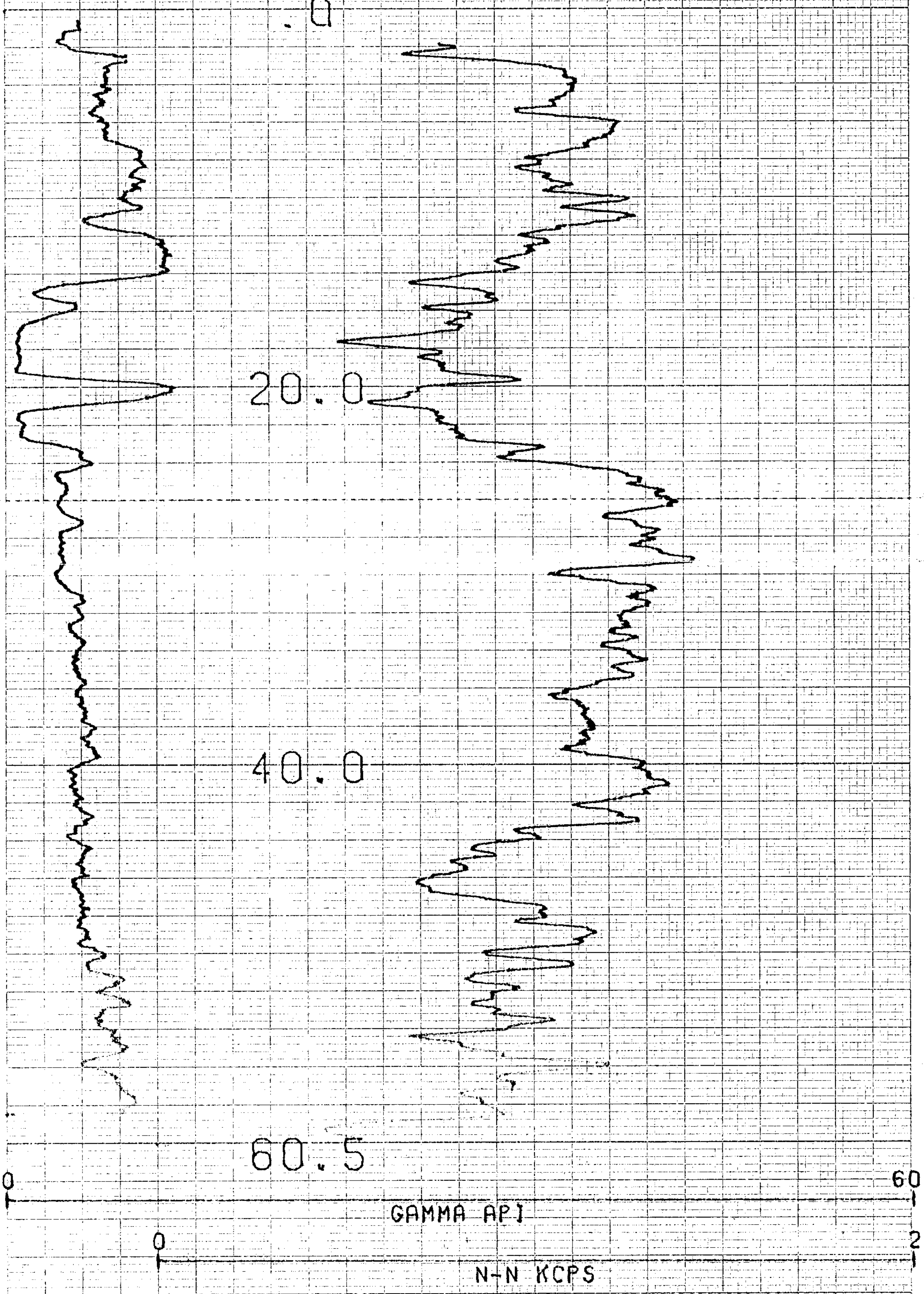
DATA V8L2*A TRUCK # 604

M.HEALY APPL.#30 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

487

485



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COMPU-LOG V8L2 PLOT 10-14-80

QPR87002

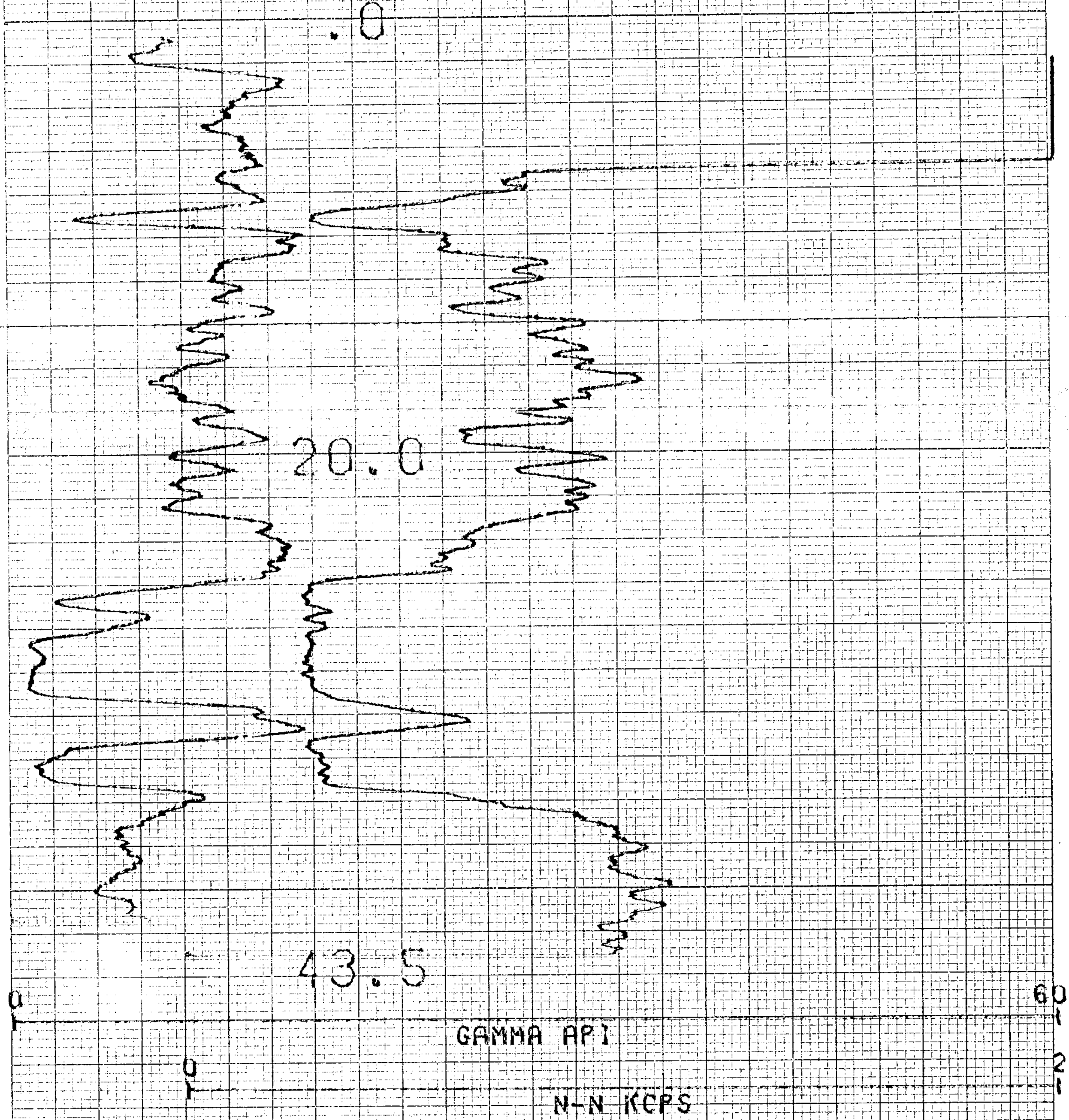
GAMMA NEUTRON RODS

WOLV. VALLEY

HOLE DIAMETER : 13.1
PROBE # 9055A - 079
SENSOR #4 CAL STD CPS = 152
SENSOR #4 CAL RUN CPS = 186
SENSOR #4 CAL BIAS = 0
DATA V8L2*A TRUCK # 604
M.HEALY APPL.#7 L1

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

GEOPHYSICAL CORP. PART NO. 786-0040



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447

739

COMPU-LOG V8L2 PLOT 10-15-80

QPR87005
GAMMA NEUTRON DEV.
WOLV. VALLEY

HOLE DIAMETER : 13.1
PROBE # 9055A - 079
SENSOR #1 CAL STD CPS = 152
SENSOR #1 CAL RUN CPS = 186
SENSOR #1 CAL BIAS = 0
DATA V8L2#A TRUCK # 601
M.HEALY APPL.#7 L1

VERTICAL DEVIATION

COMPU-LOG V8L1 DEVIATION
DATA FROM : V8L2*A

739

CLIENT : GAMMA NEUTRON DEV.
LOCATION : WOLV.VALLEY
HOLE ID : ~~[illegible]~~ QPR 87001
DATE OF LOG : 10-15-80
PROBE : 9055A 0079

SCALE: .25 M/DIV
MAG DECL: 24.5
TRUE DEPTH: 73.3 M
AZIMUTH: 178.3
DISTANCE: 2.65 M

+ = 5.0 M INCR
△ = TOP OF ZONE
◇ = BOTTOM OF ZONE

TRUE NORTH ↑

275

CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

CENTURY GEOPHYSICAL CORP. PART NO.

VERTICAL DEVIATION

739

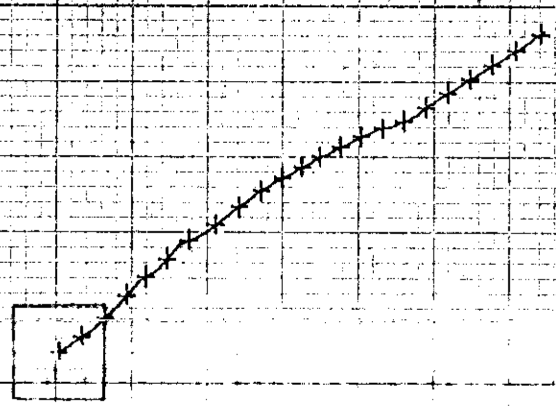
COMPU-LOG V8L1 DEVIATION
DATA FROM : V8L2*A

CLIENT : GAMMA NEUTRON DEV.
LOCATION : WOLV.VALLEY
HOLE ID : QPR87003
DATE OF LOG : 10-15-80
PROBE : 9055A 0079

SCALE: .25 M/DIV
MAG DECL: 24.5
TRUE DEPTH: 144.3 M
AZIMUTH: 56.6
DISTANCE: 1.92 M

+ = 2.0 M INCR
△ = TOP OF ZONE
◇ = BOTTOM OF ZONE

TRUE NORTH ↑



CENTURY GEOPHYSICAL CORP. PART NO. 786-0040

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~~CONFIDENTIAL~~

QUINTETTE COAL LIMITED

1987 EXPLORATION REPORT

TRANSFER, GRIZZLY AND PERRY CREEK AREAS

FEBRUARY, 1988

APPENDIX 2

739

1987 EXPLORATION REPORT
TRANSFER, GRIZZLY AND PERRY CREEK AREAS

Table of Contents

APPENDIX 2

DESCRIPTION

Appendix 2.1 of this report contains coal quality data, and remains confidential under the terms of the *Coal Act Regulation*, Section 2(1). It has been removed from the public version.

http://www.bclaws.ca/EPLibraries/bclaws_new/document/ID/freeside/10_251_2004

Appendix 2

Section 2.1

Analytical Results

Appendix 2

Section 2.1.1

Core Analysis

Appendix 2

Section 2.1.2

Adit Sample Analysis

Appendix 2

Section 2.2

Geotechnical and Hydrogeological Assessment

for Grizzly/Transfer Project

QUINTETTE COAL LIMITED

GEOTECHNICAL AND HYDROGEOLOGICAL ASSESSMENTS FOR THE GRIZZLY/TRANSFER PROJECT

Prepared by

PITEAU ASSOCIATES ENGINEERING LTD.

PROJECT 81-339GT

NOVEMBER, 1987

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FIGURES

TABLES

1. INTRODUCTION

This report describes the rock mechanics, geotechnical and hydrogeological studies carried out to prepare preliminary open pit slope and waste dump design guidelines, and to evaluate foundation conditions for sedimentation ponds for the proposed Grizzly-Transfer Project. Terms of reference for this work are described in our proposal dated February 19, 1987 and Quintette Coal Limited's (QCL) Purchase Order Q8710700-00-CE issued April 28, 1987.

Field work was carried out during June and July, 1987. Office analysis, development of design guidelines and report preparation were ongoing during August and September. A summary draft report was issued for review by QCL October 8, 1987. This report was subsequently finalized and submitted on November 27, 1987.

Due to the timing of the exploration program and mine planning requirements, a detailed geological interpretation, as well as detailed proposed pit plans, cross sections and mining layouts were not available for this study. In addition, only information available prior to conclusion of the geotechnical field program in mid-July, 1987 was assessed for this study. Consequently, as discussed with Mr. G. Gormley of QCL, detailed design of specific pit slopes or waste dumps was not conducted. Rather, study results are presented in terms of guidelines or design concepts which may be applied to specific pit slopes or waste dumps, as required. We strongly favour this approach as it allows for incorporation of geotechnical and hydrogeological considerations and design criteria at an early stage of mine planning and economic evaluation. Once detailed geologic interpretations are available and preliminary pit slope and waste dump designs have been prepared, design criteria should be reviewed and modified, if necessary, prior to final design.

It should be noted that due to the proximity and similarity of the proposed Grizzly/Transfer pits to the existing Shikano Pit, much of the information contained in our previous study of the Shikano Project (Piteau Associates, 1985) is considered relevant to the current study.

2. DESCRIPTION OF THE INVESTIGATION

2.1 FIELD STUDIES

2.1.1 Engineering Geology and Rock Mechanics

Field work for the engineering geology and rock mechanics studies was conducted in June and July, 1987. Sufficient data were collected to prepare preliminary design guidelines for proposed open pit slopes and to assess potential waste materials. Field studies included geotechnical core logging, field reconnaissance, geologic structural mapping and slope documentation.

Geotechnical core logging was conducted to assess variations in bedding orientations and mechanical properties of the rock mass. Approximately 1200m of drill core from nine diamond drillholes were geotechnically logged and photographed. Five of the logged holes were from previous drilling programs conducted in 1985 and 1986, and the remaining four were drilled in June and July, 1987. Bedding dip, rock type, recovery, RQD, frequency of natural bedding joints and cross joints, Degree of Breakage, weathering and Hardness were recorded for each core run (i.e. approximately every 10 feet) for all new core. Because of general weathering, slaking and repeated rehandling, RQD, bedding joint frequency and cross joint frequency could not be reliably determined for old core. Consequently, only bedding dip, Hardness, rock type and Degree of Breakage were recorded for old core.

General field reconnaissance and limited geologic structural mapping of available outcrops were carried out to obtain an appreciation for the character of the rock mass and spatial relationship between bedding and the various joint sets. In addition, examination and documentation of rock mass and slope conditions on existing benches in the Shikano Pit were conducted to assess the behaviour of excavated slopes in similar geologic materials.

2.1.2 Hydrogeology

Field work for the hydrogeology studies was carried out in conjunction with the engineering geology and rock mechanics studies discussed above. The field program was designed to establish a preliminary data base for evaluation of hydrogeologic conditions and basic pre-mining groundwater quality. Sufficient data were collected for preliminary assessments of pit slope depressurization requirements and quantities of seepage inflow. The program included limited hydrogeological mapping, instrumentation, in situ permeability testing, water level monitoring and limited groundwater sampling.

A total of 15 sealed standpipe piezometers and eight open standpipes were installed in seven new rotary and one existing diamond drillhole. All piezometer installations were falling head tested to assess responsiveness and hydraulic conductivity of the formation. With the exception of P2 and S3 in Drillhole QHR87007, which appear to be hydraulically connected, all installations appear to be responsive and operating properly.

Prior to conducting falling head tests, static water levels were measured in all piezometers and standpipes. In addition, open hole water levels were measured in all drillholes (rotary and diamond) which were drilled as part of the 1987 exploration program and which had not caved. Water levels were also measured in some existing drillholes in the Transfer area. Existing holes in the Grizzly area were found to have caved or could not be located.

Groundwater samples were obtained from three piezometers and shipped to ASL Laboratories in Vancouver for inorganic chemistry testing.

2.1.3 Surficial Geology

Surficial geology studies were conducted to assess potential geotechnical hazards and foundation conditions in the vicinity of proposed waste dumps and sedimentation ponds. Studies included a preliminary airphoto interpretation followed by field reconnaissance, surficial soils mapping, test pitting and sampling.

A preliminary airphoto interpretation was carried out prior to the field program to assess general soils types and distribution. Reconnaissance traverses and mapping of soils exposures were conducted in the vicinity of each proposed sedimentation pond and waste dump sites where significant soil deposits were identified in the airphoto interpretation. Based on this reconnaissance, the preliminary airphoto interpretation was updated.

Using a backhoe, 13 test pits were excavated to depths of up to about 6m. The test pits were logged and samples of the various soil strata were obtained for laboratory classification and testing. Borehole logs prepared by Hardy Associates (1982) for the portion of the conveyor route adjacent to the east limb of the Grizzly Pit were also examined.

2.2 OFFICE STUDIES

2.2.1 Engineering Geology and Pit Design

i) Geologic Structural Analysis

Geotechnical core logging and geological mapping data were compiled and processed using computer techniques. Logs of all geotechnically logged diamond drillholes were prepared. Representative geotechnical sections were prepared for each area based on preliminary geological interpretations provided by QCL.

Geologic structural data from geological mapping in the Grizzly, Transfer and Shikano areas were analyzed. Spatial relationships of discontinuities were assessed using computer sorting and statistical analysis techniques. Bedding dips from drill core were assessed statistically and compared with preliminary geological interpretations provided by QCL.

ii) Stability Analyses

Based on the results of the geologic structural analysis, assessments were carried out to determine kinematically possible failure modes which could be expected on the various types of walls. Detailed slope stability analyses were carried out and preliminary slope design guidelines were established for all potential footwall, hanging wall and endwall slopes.

2.2.2 Hydrogeological Assessment

i) Data Review and Compilation

Hydrogeological logs were prepared for each drillhole in which piezometers were installed. Water level readings were reviewed and falling head tests in piezometers were analyzed using standard procedures. Hydrogeologic information available from piezometer installations, water level measurements and hydrogeological reconnaissance were plotted on typical geotechnical cross sections. Representative longitudinal hydrogeological sections were constructed along the anticlinal axis in the Transfer Pit and along the northeast limb of the Grizzly Pit. Baseline groundwater quality data were also summarized.

Results of hydrogeological studies carried out for the Shikano Project were also reviewed.

ii) Computer Modelling and Analysis

Anticipated ranges of hydraulic conductivity and anisotropy of the rock mass as a whole were determined. A two-dimensional, steady-state, finite-element computer model was used to model present condition (i.e. pre-mining) groundwater flow along the longitudinal section through the proposed Transfer Pit. The purpose of this modelling was to provide a numerical check of the ranges of hydraulic conductivity and anisotropy estimated from the field program.

Additional modelling was conducted using the same computer model to estimate post-mining quantities of seepage inflow to the proposed Transfer Pit and post mining piezometric levels in the west end of the Transfer Pit.

Based on a comparison of the similar hydrogeologic regimes in the Transfer and Grizzly Pits, results of modelling studies in the Transfer Pit were also used to estimate post-mining seepage inflows in the Grizzly Pit and piezometric levels in the west end wall of the Grizzly Pit.

iii) Conclusions and Recommendations

Best estimates of potential seepage inflow to the pits from various sources were prepared. Groundwater conditions in proposed final pit walls were evaluated and potential problem areas defined. Recommendations concerning groundwater monitoring and possible remedial measures were prepared.

2.2.3 Assessment of Surficial Soils and Waste Dumps

i) Surficial Soils Assessment

On the basis of the results of the airphoto study, field reconnaissance and test pitting, the approximate extent and types of near surface (i.e.

within 6m of surface) soils were delineated in the vicinity of the proposed waste dump and sedimentation pond sites. Preliminary test pit logs were prepared.

ii) Laboratory Testing

Soil samples were classified according to the Unified Soil Classification System. Pertinent features such as colour, particle size, consistency, particle angularity, degree of weathering and plasticity were recorded, as appropriate. Moisture content, Atterberg limit and gradation tests were conducted on representative samples. Preliminary field classifications were reviewed based on results of laboratory classification and testing, and test pit logs were finalized.

iii) Assessment of Materials Properties

Preliminary shear strengths for the foundation materials were estimated. Strength parameters for the waste rock were estimated based on the behaviour of existing waste dumps at the McConkey Mine, as well as on experience gained from other coal mine waste dumps in the Rocky Mountains region.

iv) Assessment of Waste Dumps

Based on the above (i.e. i to iii), assessments were made as to the suitability of the proposed waste dump sites and the need for rock drains under two of the dumps. Precautions that should be taken into consideration and further investigations that should be completed before dump designs are finalized were also evaluated.

v) Assessment of Sedimentation Pond Sites

Assessment of general foundation conditions in the vicinity of the three sedimentation pond sites was carried out.

3. ENGINEERING GEOLOGY AND SURFICIAL SOILS

3.1 SETTING

3.1.1 Regional Geology

The Grizzly-Transfer Project area (see Fig. 1) lies within the Peace River Coal Field of northeastern British Columbia. This coal field is characterized by structural disturbances that resulted from its proximity to the Rocky Mountain structural zone. All major structural features follow a general northwest-southeast trend, reflecting the Rocky Mountain fold structure. The main geological structures in the Quintette area are broad synclines and anticlines which are separated by low to medium angle thrust faults which dip to the southwest.

The regional stratigraphy is summarized on Fig. 2. A brief description of the more relevant lithologic units is given below in Section 3.2.

3.1.2 Location and Topography

The Grizzly-Transfer Project area is located on the northwestern side of the Murray River, about 1km to 2km northwest of the existing Shikano Pit. General site location and layout is given in Fig. 1.

Natural topography is variable, with ground elevations ranging between about 770m and 1670m. Relatively flat alluvial flood plains and low alluvial-glaciofluvial terraces associated with the Murray River characterize the southeastern portion of the project area. Proposed Sedimentation Ponds SP-1, SP-2 and SP-3 will be located in this area. Beyond the flood plain and terraces, slopes initially rise moderately to steeply (i.e. 20^{o} to 40^{o}), then more moderately (i.e. 20^{o} to 30^{o}) towards the northwest to west. Both the Grizzly and Transfer open pits and Waste Dump Sites TD1, TD2, GD1 and GD3 are located in this area. The steepest slopes generally occur in the southern portion of the Transfer Pit area where slopes up to about 45° were observed. Waste Dump Site GD2, located northeast of the Grizzly Pit, is characterized by relatively flat topography, with most slopes generally less than 10° to 20°.

The project area is crossed by a number of drainage courses. M18 Creek drains the northern portion of the project area, originating on the northern flank of the Transfer area, flowing northeastward around the Grizzly Pit, then turning eastward to flow through the GD2 Waste Dump Site, to the Murray River. This creek apparently experiences continuous flow for most of the year, with loss of flow in late summer/early autumn through to spring. Smaller, intermittent or seasonal creeks drain the southern slopes in the project area, flowing southeastward to the Murray River. The most southerly creek, M-14, discharged throughout the 1987 exploration season.

3.2 LITHOLOGY

The stratigraphic units exposed in the Grizzly-Transfer area belong to the Lower Cretaceous Commotion Formation (see Fig. 2). The coal bearing sequence is part of the Gates Member, which is composed of an interbedded sequence ranging from coal and carbonaceous shales to sandstones with some zones of conglomerate. A particularly thick sequence of conglomerate was observed in core holes along the northeast limb of Grizzly Pit. A number of coal seams (i.e. D, E, F, G, J and K1 and K2) have been identified.

Immediately underlying the K2 Seam is a thin (i.e. generally less than 3.0m true stratigraphic thickness) sequence composed primarily of shales and carbonaceous shales with minor coal splits, siltstone and sandstones. This zone is observed throughout the project area. For approximately 20m below this zone, available core information indicates lithology varies from an interbedded succession of sandstones, siltstones and shales in the Grizzly Pit area and eastern limb of the Transfer Pit, to a relatively massive, competent sandstone in the west limb of the Transfer Pit. Detailed lithologic composition of these various stratigraphic units identified within the Gates Member, based on available core information, are given in Table I.

Marine shales of the Moosebar Formation underlie the Gates Member; however, these rocks will not be exposed in the open pit. Overlying the Gates Member are the Hulcross and Boulder Creek Members of the Commotion Formation. The Hulcross Member, primarily a marine shale, is about 90m thick. It is anticipated that only the upper portions of the pits may be comprised of Hulcross Member rocks. The bulk of the Boulder Creek Member is composed of carbonaceous shales and siltstones with resistant sandstones and conglomerates occurring towards the base. No Boulder Creek rocks will likely be exposed in the pit.

3.3 STRUCTURAL GEOLOGY

Rational slope stability analysis and slope design requires that the proposed pit be subdivided into areas of similar geologic structural and/or mechanical characteristics. The engineering behaviour of the slope forming materials can be expected to differ in areas of the pit in which these characteristics are appreciably different.

The most important structural geology features of the Grizzly-Transfer area are a series of anticline/syncline folds which plunge shallowly towards the northwest (see Fig. 1). The Grizzly Pit occurs along the northwestward extension of the Shikano Anticline (see Figs. 1 and 3), the same structure which is currently being mined in the north limb of the Shikano Pit. The Transfer Pit occurs along a similar, parallel anticline (the Transfer Anticline) to the southwest of the Shikano Anticline (see Figs. 1 and 4).

Insufficient mapping information was available to assess geologic structural conditions in each limb of each pit independently and retain a reasonable degree of statistical confidence. However, because of the similarity in fold structures and probable pit configurations, the limbs of each of the proposed pits may be divided into two Structural Domains on the basis of their general orientation (see Figs. 3 and 4). Structural Domain 1 includes all strata on southwest dipping fold limbs, and Structural Domain 2 includes all strata on northeast dipping fold limbs. Fold structures in the Shikano Pit may be similarly grouped. The boundaries between structural domains correspond to the locations of fold axial planes. Within each structural domain, the relative spatial orientation of the various discontinuity sets are expected to be relatively consistent with respect to bedding.

Joint data from similarly dipping fold limbs from the Grizzly, Transfer and Shikano areas were considered together in assessing discontinuity populations and spatial relationships. Lower hemisphere, equal area projections of poles to bedding and bedding joints and cross joints on both southwest dipping limbs (Structural Domain 1) and northeast dipping limbs (Structural Domain 2) are given in Figs. 5 and 6, respectively.

3.3.1 Bedding

As a consequence of the plunging folds, bedding orientations vary throughout the pit areas. Because bedding is a controlling geologic structure, a detailed knowledge of bedding orientation is essential for design. An assessment of the variation of bedding dip in proposed walls was conducted by statistically analyzing bedding dip logged in several diamond drillholes. Locations of logged drillholes are shown on Figs. 3 and 4.

Bedding dips measured at regular intervals in drill core were analyzed using the cumulative sums technique developed by Piteau and Russell (1971). The cumulative sums (cusums) technique provides a rapid and precise method of determining the location and magnitude of major trends in bedding dip. These major trends are designated "current mean bedding dips".

Current mean bedding dips in each drillhole are summarized on the geotechnical logs in Appendix A and have been used to prepare possible structural interpretations of the geology intersected in each drillhole. These interpretations are shown on the geotechnical sections on Figs. 7 to 11. For geotechnically logged diamond drillholes which do not occur on geotechnical sections, bedding dip interpretations are included in Appendix A.

Results of cumulative sums analysis, geologic structural mapping, and geologic interpretation provided by QCL were used to evaluate bedding variations on each limb in the two pit areas. Based on this information, it appears that the bulk of bedding which will be exposed on main pit limbs will range in dip from about 20° to 70°. Bedding dips on the southwest limb of the Grizzly Pit appear to range from 32° to 59° with an average dip of about 41°. On the northeast limb of Grizzly, dips range from 56° to 70° with an average of about 60°. In Transfer, bedding dips range from 20° to 54° and 25° to 65° with averages of about 40° and 38° for southwest and northeast dipping limbs, respectively.

3.3.2 Joints

As indicated above, in sedimentary sequences, the orientation of discontinuity sets is commonly fixed with respect to bedding. It is therefore important to evaluate discontinuity populations relative to a common bedding orientation. Within each structural domain, the azimuth or dip direction of bedding is expected to be relatively consistent; however, signficant variations in bedding dip, and hence discontinuity set orientations, may occur. To assess possible variations in discontinuity sets with respect to bedding, discontinuity data in each structural domain were rotated relative to the peak azimuth of bedding and to bedding dips of 30^{o}, 45^{o} and 60^{o}. Lower hemisphere, equal area projections of rotated discontinuity sets are given in Appendix B.

Examination of Figs. 5 and 6 and Appendix B indicates that, in addition to bedding joints which occur parallel to bedding, four sets of cross joints occur in both structural domains. Joint Set 1 is moderately to well developed, strikes about perpendicular to bedding and dips steeply. Joint Set 2 is well developed, strikes about parallel to bedding and dips about normal to bedding. Bedding joints, Joint Set 1 and Joint Set 2 appear to form an approximately orthogonal system of joints which are probably related to local folding mechanisms. Joint Sets 3 and 4 are moderately well developed, strike obliquely to bedding and dip moderately to steeply. Joint Sets 3 and 4 appear to form a conjugate set of joints and may be related to more regional deformations.

Bedding joints are expected to be relatively continuous. Statistical assessments of bedding joint spacing based on geotechnical core logging data (see Table II) indicates spacing varies somewhat, depending on rock type. Based on Table II and observations in surface outcrops and exposed slopes in Shikano and Mesa, bedding joint spacings of 0.5m to 1.0m in coal, carbonaceous shale and shale, and 1.0m to 3.0m or greater in siltstones, sandstones and conglomerates are considered appropriate for preliminary assessments.

Cross joints are expected to be less continuous than bedding joints and are often truncated or offset by throughgoing bedding joints. Typical natural cross joint frequencies in core range from 1.3/m for finer grained rocks (claystones/siltstones) to 1.6/m for conglomerates and 2.0/m for sandstones (see Table II). Higher cross joint frequences for the coarser grained rocks may result because these rocks tend to be harder, more brittle and less anisotropic than the finer grained rocks.

3.3.3 Faults

Several relatively continuous, moderately to steeply dipping reverse (thrust) and normal faults are indicated on the geological base plans and cross sections provided by QCL (see Figs. 3, 4 and 7 to 11). These faults appear to strike subparallel to the regional fold structures (i.e. northwest-southeast) and have been interpreted by QCL personnel to offset the coal measures, apparently on the basis of observations in core and geophysical correlation of the various coal seams. Detailed correlation and identification of major, throughgoing faults has not been carried out.

Based on our experience in the Shikano and Mesa open pits, it is likely that unfavourably oriented faults will occur in the Grizzly-Transfer pit walls. Such faults could have significant impacts on slope stability to pit design. Further definition of major fault trends and correlation of individual faults is required.

3.4 ROCK COMPETENCY AND CORE QUALITY

Because slope behaviour is a function of the mechanical properties as well as the geologic structural characteristics of the rock mass, an assessment of the relative rock competency and its variability within the rock mass is also required. In this regard, a statistical assessment of rock mechanics properties based on rock types logged in the core was carried out using the cumulative sums technique described in Section 3.3.3. Results of this assessment are summarized in Table II. On the basis of detailed stratigraphic and lithologic assessments described in Section 3.2, the rock mass may be divided into five basic rock mass units as follows:

i) Hulcross Hanging Wall Rocks - These are primarily closely bedded, friable marine shales of relatively low competency. These rocks are subject to significant weathering, slaking and general deterioration and are expected to be the least competent, softest rocks to be exposed in the pit. Hulcross rocks may occur near the crest of the pits in hanging walls and endwalls.

ii) Gates Hanging Wall Rocks - These are interbedded coals, carbonaceous shales, shales, siltstones and conglomerates of variable competency. Coal and shale components tend to be of poor to moderate competency; siltstones of moderate to good competency and sandstones and conglomerates of generally good competency. Overall, this unit is expected to be of poor to moderate quality with the exception of some significant good quality conglomerate sequences which occur in some drillholes in the Grizzly area.

iii) Immediate Footwall Rocks - Rocks within about three metres stratigraphically below the footwall of K2 Seam tend to be interbedded carbonaceous shales, with minor siltstones and sandstones and occasional coal splits. These rocks are of generally poor quality and may form the immediate footwall slopes. Bedding joint spacing within these rocks is expected to be about 0.5m to 1.0m and carbonaceous zones and coal splits may be correlated some distance between drillholes.

iv) Intermediate Footwall Rocks - These rocks consist of interbedded shales, siltstones and sandstones of moderate competency. Continuous bedding joint spacing is expected to be in eccess of 1.0m. These rocks occur in the southwest and northeast limbs of Grizzly and the northeast limb of Transfer, below the Immediate Footwall Rocks (unit iii above).

v) Competent Footwall Rocks - These rocks consist of relatively massive, competent sandstone. Continuous bedding joint spacing is expected to be in excess of 3.0m. These rocks occur in the footwall of the southwest limb of Transfer, below the Immediate Footwall Rocks (unit iii above).

3.5 ROCK STRENGTH PROPERTIES

3.5.1 Shear Strength of Discontinuities

A knowledge of the shear strength characteristics of the various discontinuity sets is required for slope stability analysis. Design shear strengths were based on the results of previous studies by Golder Associates (1982) and on our experience with similar rock masses. For stability analyses involving footwall slope failure mechanisms, bedding joints were assumed to be continuous and cohesionless, and to exhibit friction angles of 26° along the more carbonaceous bedding planes within Immediate Footwall Rocks, 30° within Intermediate Footwall Rocks and 34° within Competent Footwall Rocks. Cross joints, which tend to be slightly rougher than bedding joints, were assumed to have friction angles of 35° and negligible cohesion.

3.5.2 Rock Mass Strength

Certain footwall stability analyses also require a knowledge of the rock mass strength. Based on unconfined compressive strengths for individual rock types (see Golder Associates, 1978) and anticipated rock mass behaviour based on results of geotechnical core logging, mohr envelopes representing rock mass strengths for each rock type were developed using the criteria of Hoek and Brown (1980) (see Fig. 12). On the basis of these type curves, and percentage rock type composition as summarized in Table I, mohr envelopes representing rock mass strength for each of the three footwall rock mass units were derived (see Fig. 12).

3.6 DESCRIPTION OF SURFICIAL SOILS

The results of the airphoto interpretation and ground reconnaissance are illustrated on Fig. 13, along with the location of the test pits, proposed pit boundaries, waste dumps and sedimentation ponds. Logs of the test pits excavated for this study are contained in Appendix C. Laboratory test results are included on the test pit logs. Gradational analyses are contained at the end of Appendix C.

Typical cross sections through the five proposed dump sites are shown in Figs. 14 and 15. Dump configurations shown on these cross sections, and in plan on Fig. 13, are based on the preliminary dumping scheme provided by QCL in June, 1987. Similarly, sedimentation pond locations indicated on Fig. 13 are preliminary in nature. As such, it is understood that these waste dump and sedimentation pond configurations are intended only to indicate the general size and extent of such facilities and that revised configurations will be provided for assessment once detailed mine plans are available.

As can be seen in Fig. 13, the principal terrain types in the Grizzly/Transfer area include bedrock with minor weathered bedrock and colluvium, colluvium covered dip slopes and a glaciofulvial outwash terrace system along the Murray River. More specific descriptions of the soil types in the proposed waste dump and sedimentation pond sites are included below.

3.6.1 Grizzly Dump Sites

Three dump sites have been outlined for the Grizzly Pit. Two of these sites, referred to as GD1 and GD3, are relatively small, while the third, GD2, covers a substantial portion of the area between the overland conveyor and the Mesa Mine access road.

i) GD1 Dump Site

The GD1 site is located in a draw between about the 950m and 1100m elevation on the south side of the Grizzly Pit. Natural slope angles in the area are between about 20^{o} and 35^{o}. A variable thickness of colluvium, generally consisting of a mixture of silt, sand, gravel, cobbles and some angular boulders, is present over most of the site. In some areas, bedrock is exposed on surface. An intermittent, small stream channel is present in the natural draw.

ii) GD2 Dump Site

The large GD2 site is located between about the 1040m and 830m elevations on the north side of the Grizzly pit above the M19 Gravel Pit access road, and between the Mesa Mine access road and the overland conveyor. The overall slope angle of the topography is less than about 10°, with local slope angles ranging from flat to about 15°. M18 Creek, which flows through the centre of the dump site, becomes more incised at lower elevations, with the banks of the creek channel being up to about 30m to 40m high at a slope angle of up to about 50°.

In general, the GD2 dump site is blanketed by a layer of glacial till overlying bedrock. Based on field observations and test pits, the till may be described as compact to dense silty sand to sandy silt with gravel and cobbles. The thickness of this unit is variable, but is expected to be less than about 5m to 7m, with the thickness in many areas being less than 2m to 4m. Soft peat or silty peat overlies the till in a few flat lying (or depressed) areas within the dump site. These localized, boggy deposits (see Fig. 14) are anticipated to be no more than about 2m thick and for the most part to be located below about the 925m elevation. Colluvium, similar to that described above for the GD1 dump site, is present in the upper reaches of the GD2 site. A minor amount of sand, gravel and silt alluvium is present in the upper reaches of M18 Creek. Bedrock is exposed in the channel bottom and sides of M18 Creek between about the 915m and 825m elevations. Depending on the lower limit of the proposed GD2 Dump, the dump could encroach on a glaciolacustrine terrace deposit which has been observed between about the 825m and 790m elevations at numerous locations along the Murray River Valley. This material is typically a compact (i.e. medium dense) to dense silty fine sand with lenses of silt or clayey silt.

iii) GD3 Dump Site

The GD3 dump site is located downslope of the proposed Grizzly Pit between about elevations 870m and 785m. The topography in this area generally slopes at between about 5^{0} and 20^{0}. Colluvium or glacial till is present over the upper portion of this site (i.e. above about 825m to 830m elevation). Between elevation 825m to 830m and about 785m, remnants of the glaciolucustrine terrace material, discussed above for the GD2 dump site, are present. At the lower elevations (i.e. $<$ 785m), the proposed dump will be founded on a relatively flat lying glaciofluvial terrace consisting largely of compact sand and gravel with cobbles and occasional boulders.

3.6.2 Transfer Dump Sites

Two dump sites have been outlined for the Transfer Pit, with the TD1 site being proposed to contain the majority of the waste rock generated.

i) TD1 Dump Site

The TD1 site is located on the south side of the proposed Transfer Pit in the M14 Creek drainage between about elevations 1540m and 1000m. While the overall gradient of the drainage course is about 15^{0}, natural slopes within the proposed dump site range up to about 25^{0} to 30^{0}, with some locally steeper areas.

Bedrock, overlain by a thin veneer of silty, sandy, gravelly, cobbly colluvium in some areas, is present over much of the TD1 dump site. Below about the 1025m to 1050m elevation, thicker colluvium exists in the form of a colluvial fan (see Fig. 13).

ii) TD2 Dump Site

The TD2 site is located downslope of the proposed Transfer Pit area between about the 900m and 875m elevations. The natural topography in the area typically slopes at between about 15^{o} and 20^{o}, with slopes as flat as about 5^{o} to 10^{o} below about the 900m elevation. Surficial soils in the dump site appear to be very similar to those outlined above for the TD1 dump site, with the colluvium expected to be somewhat thicker, particularly at lower elevations. Some glacial till, similar to that described for the GD2 dump site is expected to be present in the lower areas of the site (see Fig. 13).

3.6.3 Sedimentation Pond Sites

Three sites (identified in Fig. 13 as SP1, SP2 and SP3) have been identified by QCL as potential locations for sedimentation ponds. These sites are all located on the glaciofluvial outwash terrace along the Murray River and are all located on near level ground at approximately elevation 780m to 790m. Although there are some differences between the sites, the glaciofluvial terrace materials are generally described as being compact sand and gravel with cobbles and occasional boulders. It is noteworthy that a veneer of stiff, fine grained (i.e. silty, clayey) soils overlies the granular glaciofluvial materials in some areas. This material, which ranges up to at least 2m thick and generally increases in thickness to the northwest (i.e. towards the hillside), is thought to be slopewash from the upper slopes. No seepage was noted in the test pits excavated for SP2 or SP3.

While the soils encountered in the vicinity of SP1 are similar to those described above for SP2 and SP3, some differences were observed. At Test Pit 12 the granular glaciofluvial soils underlying the 2m thick surface veneer of stiff, fine grained materials were saturated, with the water table being at the contact beween the two soil types. At Test Pit 11, sand and gravel was found on surface with the water table being at a depth of about 1m. Underlying this sand and gravel, from a depth of about 1.4m to at least 6m (i.e. the bottom of the test pit), is a layer of very soft to soft clayey silt/silty clay of medium plasticity. It is important to note that M14 Creek drains onto this portion of the glaciofluvial terrace.

3.6.4 Soils Within Pit Areas

As can be seen from Fig. 13, colluvium and/or bedrock is present over much of the proposed pit areas, with very little material interpreted as being of significant quantity and quality for reclamation purposes. The best reclamation materials appear to be located in the flats adjoining the two small lakes at about the 1315m elevation in the Transfer Pit area. These materials are interpreted as being ponded silts and may contain considerable colluvial debris as well as disseminated organic materials.

4. HYDROGEOLOGY

4.1 REGIONAL HYDROGEOLOGY

The folded structure of the sedimentary rocks common to the Peace River Coal Fields of northeastern B.C., and the well developed system of creeks and rivers which drain the area, tend to limit groundwater flow to local systems. The 200m to 300m stratigraphic thickness of Moosebar and Gething shales should act as a confining layer for any deep regional flow systems under the study area, thereby isolating the Quintette area from significant upward flow from deeper flow systems. Thrust faults present in the area could result in permeable zones through which deep, regional flow groundwaters could discharge to surface. This is unlikely, however, as the underlying shales are relatively soft and have a high clay content, both of which are not conducive to the development of zones of open fractures along faults.

The preferred direction for groundwater flow is parallel to bedding. Groundwater flow across bedding planes is limited by the hydraulic conductivity of the rock normal to bedding, which is generally much lower than the hydraulic conductivity along the bedding planes. This is due in part to the presence of bedding joints, but mainly to the interbedded nature of the rock, in which relatively soft shale and coal strata separate more brittle sandstone and siltstone strata. In the study area, which is characterized by shallow folds, the groundwater flow system should be fairly shallow, reflecting this stratigraphic control.

Most of the shallow geologic structure in the area is relatively small scale with respect to topographic features (e.g., both limbs of a fold are located on adjacent ridges or in adjacent valleys). Therefore, groundwater flow along bedding generally discharges relatively close to areas where groundwater recharge occurs.

4.2 PHYSICAL INFLUENCES ON LOCAL HYDROGEOLOGY

The physiography, climate and hydrology of the study area are all factors which must be considered in a discussion of the local groundwater regime.

4.2.1 Physiography

The study area is characterized by high relief. The elevation of the Murray River near the Transfer and Grizzly Pits is approximately 760m. The elevation of the ridge at the upper end of the proposed Grizzly Pit is approximately 1600m. Natural slopes in the study area range from nearly flat, to about 35°.

4.2.2 Climate

Average annual precipitation in the catchment area around the two pit areas ranges from 600mm/yr to 900mm/yr (Golder Associates, 1982a). The average precipitation over the entire catchment area around the Grizzly and Transfer Pits is estimated to be 700mm/yr.

4.2.3 Surface Water Hydrology and Infiltration

Surface water drainage in the immediate vicinity of the two pits is mainly to the east, towards the Murray River, and north towards a tributary of the Murray River, via a number of small creeks.

There is only limited flow monitoring data for two of these creeks. As this data is for spring flows, it does not provide an estimate of the base flows in either of these creeks.

A rough estimate of groundwater recharge can be made by assuming that low measured flow is the base flow, due entirely to groundwater discharge. Although there is limited information available for surface flows in the immediate area of the Grizzly and Transfer Pits, there is monitoring data available for the M-9 Creek in the Shikano Pit area, directly across the Murray River. Based on this data, an infiltration rate equal to 15% of average annual precipitation was determined (Piteau Associates, May 1985). Fifteen to twenty percent of average annual precipitation is generally considered to be a reasonable estimate of the infiltration rate in mountainous areas, and this range has been used for subsequent calculations of groundwater recharge to flow systems which will ultimately discharge into the two proposed pits.

4.3 GROUNDWATER FLOW IN SURFICIAL SEDIMENTS

Colluvium and till cover most of the catchment area on the slopes above the proposed pits. The thickness of these deposits is expected to generally be less than 3m, except in small depressions in the bedrock surface.

An estimate of groundwater flow through the surficial sediments into the pit was made, based on the following assumptions:

i) Precipitation recharge to surficial sediments equals 10% of average annual precipitation over the recharge area. (The other 5 to 10% of the precipitation which infiltrates is assumed to eventually become groundwater flow in bedrock).

ii) The recharge area for groundwater recharge to the surficial sediments equals the surface runoff catchment area outside the perimeter of the proposed pits.

Groundwater flow through surficials into the pit is therefore calculated as:

$Q = P.i.A$ where P = average annual precipitation (700mm/year)
i = infiltration factor (10%)
A = recharge area (see Table III)

Estimated inflows to the Grizzly and Transfer Pits, due to groundwater flow through surficial sediments, are 1.3 and 2.1 L/s, respectively (see Table III).

4.4 GROUNDWATER FLOW THROUGH BEDROCK

In order to quantitatively evaluate the flow of groundwater through bedrock, estimates of the hydrogeological properties of the rock mass must be made. These estimates, along with interpreted groundwater flow systems, are discussed below.

4.4.1 Hydrogeological Properties

Based on the local stratigraphy, the rock mass which underlies the Grizzly and Transfer Pit areas can be subdivided into two hydrogeological units, as follows:

i) Hanging wall rocks (above D Seam) and interseam rocks (D Seam to approximately 5m below K2 Seam, inclusive).
ii) Footwall rocks

The footwall rocks are expected to be more competent and less fractured than the hanging wall/interseam rocks, and are therefore expected to have slightly lower hydraulic conductivity than the overlying rock.

The bedded structure of the rock in the mine area should result in a high degree of anisotropy in the rock mass, which will have a great effect on groundwater flow systems. Because this anisotropy is common to all rocks in the area and because relatively little data are available for the hanging wall and footwall rocks, hydrogeologic modelling has been based on one hydrogeologic unit, having a high degree of anisotropy and a range of hydraulic conductivities. The range of hydraulic conductivities accounts for different intensities of fracturing in the various rock units.

Hydraulic conductivity for the rock mass was estimated from falling head tests performed in piezometers installed in drillholes in the area. Details of piezometer completions are included on the drillhole logs in Appendix D. The results of falling head tests performed in thirteen piezometers in the study area are summarized in Table IV. One of the hydraulic conductivity values estimated from falling head tests was less than 10^{-9} m/s, but the majority of values were between $5x10^{-9}$ m/s and $3x10^{-7}$ m/s. This is virtually the same range as was encountered when testing twenty-three piezometers in the Shikano area (Piteau Associates, May, 1985). Falling head tests in ten piezometers installed on Babcock Mountain indicate a range in permeability of 10^{-8} m/s to 10^{-6} m/s for the coal and interbeds (Golder Associates, 1982a). One test, performed in the Hulcross Shale, resulted in a hydraulic conductivity of $1x10^{-6}$ m/s. All the above falling head tests indicate that a range of between about $5x10^{-9}$ and $5x10^{-7}$ m/s is a reasonable estimate for the hydraulic conductivity of the rock mass in the Quintette area.

Falling head tests generally provide a low estimate of rock mass permeability. This is due partly to the detrimental effects of drilling on the permeability of the rock forming the walls of the drillhole and partly because falling head tests are small scale. Many test zones may not intersect open fractures or bedding planes, resulting in a low estimate of hydraulic conductivity. These low hydraulic conductivity values are not necessarily indicative of the overall permeability of a fractured rock mass. For example, an extensive open fracture can greatly increase the effective permeability of an otherwise tight or competent rock mass.

A reasonable estimate for the range of hydraulic conductivities of a rock mass should generally fall in the upper portion of the range determined from falling head tests. Hence, based on tests conducted in Grizzly, Transfer, Shikano and Babcock areas, the rock mass hydraulic conductivity in the Grizzly/Transfer area is expected to range between about

$5x10^{-8}$ m/s and 10^{-6} m/s. This range is for hydraulic conductivity parallel to bedding. As discussed above, sedimentary rocks are usually anisotropic to some degree (i.e., hydraulic conductivity along bedding (Kh) is greater than hydraulic conductivity across bedding (Kv)). Fractures perpendicular to bedding rarely extend over more than a few strata, resulting in a relatively low hydraulic conductivity in this direction. The anisotropy (Kh/Kv) of the rock mass in the study area is expected to range between 10 and 30.

4.4.2 Groundwater Flow System

Water levels measured in piezometers in the study area are summarized in Table IV. With the exceptions of QHR87033 in the Transfer area and QHR87007 in the Grizzly area, all monitoring holes show a downward (i.e. recharge) gradient. These gradients, which are across bedding, are generally quite high, ranging from about 0.1% to 80%, and are illustrated by the sometimes dramatic variations in piezometric levels measured in piezometers, as shown on the sections in Figs. 7 to 11. Hydraulic gradient along bedding generally parallels ground surface (see Figs. 16 and 17).

The above gradients are indicative of a stratigraphically controlled groundwater flow system in which most groundwater flow occurs along bedding, with only limited flow across bedding. The high downward gradient is a function of the anistropy of the rock mass and the varying elevations at which strata subcrop in the Murray River Valley. Groundwater will discharge from the strata where they subcrop. Thus the elevation of the subcrop defines the head of groundwater discharge. Because of the high anisotropy, the hydraulic head at any point along the flow system is more closely related to the elevation of recharge to, or groundwater discharge from, the particular stratum in which the groundwater is flowing, than to the hydraulic head in adjacent strata. Artesian conditions encountered in the upper portion of the Grizzly Pit and the middle area of the Transfer Pit are due to the confining effect imposed by the stratigraphic nature of the rock mass.

Anisotropy in the rock mass will limit the areal effect of pit development on local groundwater flow systems, thus limiting the rate of groundwater inflow into the pit. Rather than one large flow system in a homogeneous rock mass discharging into the pit, only flow in strata which daylight in the pit, or immediately adjacent strata, will discharge into the pit.

4.5 STEADY-STATE COMPUTER MODELLING OF GROUNDWATER FLOW

A steady-state, finite-element computer model was used to model both existing and post mining groundwater flow. The purpose of the modelling exercise was to predict the probable range of groundwater conditions in the Transfer Pit, based on the range of hydrogeologic properties discussed above.

The section analyzed with the steady-state model was constructed along the anticlinal axis which runs through the Transfer Pit (see Fig. 16). The purpose of modelling a section along the anticlinal axis was to check the validity of our estimated hydraulic conductivity range, to determine the range of expected groundwater inflow from the catchment area above the Transfer Pit, and to estimate the degree of drainage which should occur naturally in the end wall at the west end of the pit. The results of this modelling were also applied to the Grizzly Pit, which is of a similar geometry, but on a smaller scale (see Fig. 17).

The finite element mesh was based on the section shown in Fig. 16. The constructed mesh, and range of material properties assumed for the modelling are shown in Fig. 18. Initially, the premining situation was modelled.

Discharge to the surface was computed for both cases modelled, and then compared to estimates for precipitation recharge to the groundwater flow system. Equipotential plots for the best fit case and tabulated results for both cases are presented in Fig. 18. Modelling of the premining situation indicates that the average hydraulic conductivity of the rock mass is close to 10^{-7} m/s, and that the anisotropy is likely to be about 10. If the hydraulic conductivity were closer to 10^{-6} m/s, the recharge required to maintain the hydraulic heads measured in piezometers installed above the section would approach the entire average annual precipitation, which is not reasonable. If the anisotropy were much less than 10, vertical hydraulic gradients would be very low, which is not the case, based on piezometric levels recorded in the field.

Once the premining modelling was completed, the mesh was modified to account for excavation of the pit, and the model was run for the "best fit" case. The mesh used for this predictive modelling, and material properties used, are shown on Fig. 19. Computed steady-state inflow to the pit, through the end wall, was very low (approximately 1.7 L/s), due to the small catchment area above the pit.

The post mining position of the water table behind the west wall of the pit was also estimated during the predictive modelling. The computed water table position for the case modelled is shown on Fig. 16 along with the present water table position.

4.6 ESTIMATED INFLOWS TO THE PIT

Estimates of groundwater inflows to the pit were based on results of the computer modelling, volume calculations of water removed from storage in the rock mass around the pit, and water balance calculations. Surface flows into the pit (direct precipitation and surface runoff) were based on calculations involving catchment areas and average annual precipitation.

Groundwater inflow from bedrock in the hanging walls of the two pits was calculated by assuming that recharge equal to 10% of precipitation falling on an area extending 0.5km out from the pit crest would eventually flow into the pit. Estimated flows are tabulated on Table III.

Estimates of groundwater inflow from the west end walls of the two pits were based on the steady-state modelling discussed in Section 4.5. The computed flow from these walls (based on an approximate wall length of 500m) is 1.7 L/s.

Groundwater flow from the east walls of the two pits will be virtually zero due to the pit geometry.

All the above groundwater inflows are for steady-state conditions. There will also be a transient response to mining. This will involve removal of water from storage in the rock mass around the mine. An estimate of the water removed from storage was made by assuming that the effect of the pits on the groundwater flow system would not extend more than 0.5km from the pit crest, and that the average drawdown would be 40m. This was converted to a volume of water based on a 1% drainable porosity, and to a flow by assuming that the dewatering would occur over a period of 6 years for the Transfer Pit, and 2 years for the Grizzly Pit.

All groundwater inflows and surface water inflows estimated for the two pits are summarized in Table III.

The estimated inflows are very high, being 99 L/s and 48 L/s for the Transfer and Grizzly Pits, respectively. The major portion of these inflows is due to direct precipitation and surface runoff, hence ambient weather will have a major impact on the volume of water which must be removed from the pits. In this regard, experience gained in other pits in the area will be invaluable in estimating pit inflows during the freshet. Pit inflows during the dry periods of the year can be estimated by totalling the groundwater components of inflow for each of the pits.

4.7 GROUNDWATER CONDITIONS BEHIND THE PIT SLOPES

Computer modelling indicates that the west end walls of both the pits will be well drained. Computer modelling of the north and south walls was not performed as there is insufficient geological control to the north and south of the pits on which to base a modelling study. However, due to the limited recharge area located upslope from these pits, all slopes are expected to be moderately well drained. The worst groundwater conditions in terms of slope stability are expected to occur behind the eastern portion of the hanging wall slopes, as they will have the greatest recharge area behind and above them. Four multiple piezometers should be installed in each pit area (two in each limb) to monitor pore pressures behind the hanging wall slope. At this time, it is not anticipated that monitoring of pore pressures in footwall slopes will be required, although consideration should be given to drilling either relief wells (with a production drill) or horizontal drainholes over the lower portion of high footwall slopes.

Once preliminary pit planning is complete and slope heights are known, monitoring requirements recommended above should be reevaluated, as they are dependent on stability concerns which are in turn related to slope height.

4.8 MONITORING

Areas which require monitoring and recommended monitoring installations have been discussed above. All piezometers should be monitored on at least a seasonal, if not monthly, basis starting from the present, and continuing until the pit is completed. This should include piezometers in the pit area, which can be monitored until they are destroyed by the excavation.

If monitoring data indicates that groundwater conditions are adversely affecting stability of the slope, dewatering measures may be required.

4.9 GROUNDWATER QUALITY

Groundwater samples were collected on July 21, 1987 from QHR87013-P2 (interseam rock between J & G Seams) and QHR87017-P2 (J-Seam) in the Grizzly area. A sample was also obtained from QHD86006-P2 (J-Seam) in the Transfer area. These samples were submitted to ASL Laboratories in Vancouver for inorganic chemical analyses. Laboratory results are summarized in Appendix D. The sample from the Transfer area was affected by cement used in installation of the piezometer and was subsequently resampled on August 22, 1987. As the results of the resampling indicated the water chemistry was still affected by the cement, the following discussion is concerned mainly with the water samples from the Grizzly area. However, it is expected that quality of groundwater in the Transfer and Grizzly areas should be very similar. This is borne out by the similarity between the analysis results for parameters not affected by Portland cement (i.e. sulphate, nitrate, nitrite, TKN, NH_4, COD, TOC).

Groundwater at the site can be characterized as a calcium-sodium-magnesium bicarbonate water having a slightly alkaline pH (8 to 8.5) and a moderately high total dissolved solids (>425 mg/L). Nutrient concentrations were all very low, with the exception of phosphorous in the contaminated sample, and slightly anomalous ammonia concentrations (1 to 1.5 mg N/L) in all samples. The ammonia nitrogen is probably associated with the coal.

In general, the groundwater is of good quality, but is approaching the upper limits for acceptable drinking water in terms of total dissolved solids and hardness, and exceeds the acceptable concentration for barium (1 mg/L) in two of the samples collected (see Table D-II in Appendix D). The available data indicate that groundwater inflow to the two pits will not have a detrimental effect on receiving waters, provided suspended matter is allowed to settle prior to discharge to the Murray River.

5. SLOPE STABILITY ANALYSES AND DESIGN

5.1 BASIC SLOPE TYPES

Based on engineering geology assessments and proposed slope orientations, several basic slope configurations are apparent for the proposed Grizzly and Transfer open pits. These are: southwest and northeast dipping footwall slopes, where bedding strikes parallel to the slope and dips moderately in the same direction as the slope; southwest and northeast facing hanging wall slopes, where bedding strikes parallel to the slope and dips moderately into the slope; and northwest and southeast endwall slopes, where bedding strikes perpendicular to the slopes and dips shallowly to moderately towards the southwest or northeast. Because each of these slope configurations represents a unique geologic and geometric combination, separate kinematic assessments and stability analyses were carried out for each.

5.2 FOOTWALL SLOPES

5.2.1 Engineering Geology of the Final Wall

As shown on the typical geotechnical sections in Figs. 6 to 11, bedding dips on footwall slopes are expected to vary from about 20° to 70°. Depending on the location of the final wall, several different rock mass units (i.e. Immediate, Intermediate and Competent Footwall rocks), each with distinctively different engineering geology characteristics, may be exposed in the final wall.

5.2.2 Kinematic Assessments

To determine possible failure mechanisms involving discontinuities, kinematic assessments were carried out for a variety of bedding and discontinuity orientations. Peak orientations of discontinuity sets in

Structural Domains 1 and 2 were rotated relative to discrete bedding orientations representative of the range of bedding dips which are anticipated in the slopes (i.e. 30^{o}, 45^{o} and 60^{o}). Lower hemisphere projections of planes representing bedding and peak discontinuity set orientations at each of these bedding orientations in each structural domain were prepared and are given in Appendix E. These projections were assessed to determine possible plane, wedge or other failure modes which could occur at the various bedding dips. Simple limit equilibrium stability analyses were conducted for each kinematically possible failure mode using the strength criteria described in Section 3.5 to determine which failure mechanism controls stability. Based on these assessments, no significant kinematic difference was observed between southwest and northeast dipping footwalls (i.e. footwalls in Structural Domains 1 and 2, respectively). It is concluded that, for bedding dips of less than 30^{o}, 35^{o} and 40^{o} for Immediate, Intermediate and Competent Footwall Rocks, respectively, footwall slopes are kinematically stable, provided bedding is not undercut (i.e. daylighted). For bedding dips greater than these values, bilinear slab failure is considered to control stability as described below. Detailed descriptions of possible failure modes in footwall slopes are given in Hawley et al (1985).

5.2.3 Bilinear Slab Failure Analysis

In general, bilinear slab failure requires the presence of a flat lying discontinuity which dips out of the slope. Based on the engineering geology assessment described earlier and on the kinematic plots in Appendix E, a discontinuity set with this required orientation does not appear to be present. However, isolated discontinuities with this orientation, such as thrust faults, may occur, and these must be identified and assessed on an individual basis.

In high, relatively steep slopes in fractured rock masses, a discontinuity which dips shallowly out of the slope may not be required for bilinear slab failure. Failure in the toe area of the slope may occur by shearing through the rock mass. Based on this assumption and rock mass strength criteria described in Section 3.5, stability analyses were carried out using the analysis technique illustrated in Fig. 20. Because footwall slopes may occur in either Immediate, Intermediate or Competent Footwll Rocks, which exhibit substantially different rock mass characteristics, separate analyses were carried out for each of these rock mass units. Analysis results are given in Fig. 20 for a variety of potential slab thicknesses, bedding dips and slope heights.

Depending on the location of the slope within the stratigraphic sequence, the potential for encountering a continuous bedding discontinuity which could form a slab type failure may vary considerably. For preliminary assessment purposes it appears that a spacing of 0.5m to 1.0m between continuous bedding joints may be appropriate for Immediate Footwall Rocks. For Intermediate and Competent Footwall Rocks, a spacing of between 1.0m and 3.0m is considered appropriate.

5.3 HANGING WALL AND ENDWALL SLOPES

5.3.1 Engineering Geology of the Final Walls

The bulk of the hanging wall and endwall slopes will be excavated in the interbedded rocks of the Gates Formation. Depending on wall location, pit depth and pit slope angle, the upper portion of some endwall and hanging wall slopes may be located in Hulcross Formation shales. As no geologic structural mapping information is available for Hulcross rocks, detailed assessment of possible failure mechanisms involving joints in Hulcross rocks was not possible. However, because of the anticipated relatively poor quality of Hulcross rocks and their susceptibility to deterioration on exposure, ravelling and slaking considerations, rather than kinematically possible failures are expected to control slope design in these rocks.

As for footwall slopes, bedding dip in hanging walls and endwalls is expected to vary from about 20° to 70°.

5.3.2 Kinematics and Stability Assessments of Possible Wedge or Plane Failures

To determine possible failure mechanisms involving discontinuities, kinematic assessments similar to those conducted for footwall slopes were carried out. Lower hemisphere, equal area projections of planes representing peak discontinuity set orientations relative to discrete bedding orientations representative of the range of bedding dips expected, were prepared for each basic wall orientation. Based on these projections, which are included in Appendix E, all possible combinations of discontinuties which could form potential wedge or plane failures were identified. Simple limit equilibrium analyses were carried out to determine which of the potential wedges or planes could fail if its apparent plunge or dip were undercut by the slope. Based on the results of these stability analyses and on assessments of the importance of a given wedge or plane (i.e. the likelihood of its occurrence), the wedge or plane and its apparent plunge or dip considered to control bench stability was determined for each wall and bedding dip examined. Results of this assessment are summarized in Table V.

Because of the interbedded nature of the strata, discontinuities other then bedding joints or faults are expected to be relatively short and discontinuous. For such discontinuities to form continuous wedge or plane failures on a bench scale, they must combine with other discontinuities. The likely result of this combination is a failure plane or wedge which is stepped or ragged in appearance. All failure modes indicated in Table V are of this type (i.e. stepped wedge or stepped plane) because they all involve at least one discontinuous cross joint set.

Examination of Table V indicates that for northeast hanging wall slopes, stepped wedges involving various combinations of all discontinuity sets, and stepped planes on Joint Set 2, control bench stability. Furthermore, the apparent plunge or dip of failure considered to control bench stability appears to be relatively independent of bedding dip. Based on these observations, an apparent plunge or dip of failure of 52° is considered appropriate for preliminary bench design on northeast hanging wall slopes.

Similar stepped plane and wedge failures appear to control stability on southwest hanging wall slopes; however, apparent plunges of failures controlling stability are somewhat shallower than for northeast hanging wall slopes. Based on Table V, an apparent plunge or dip of failure of 45° is considered appropriate for preliminary bench design on southwest hanging wall slopes.

Kinematic controls on endwall slopes are much less clear, as illustrated in Table V and Appendix E. In general, endwall slopes are kinematically very favourable, in that relatively few plane or wedge failures appear to occur. Based on Table V, an apparent plunge or dip of failure of 72° is considered practical for design of endwall slopes, regardless of orientation or dip of bedding.

5.4 SLOPE DESIGN CONCEPTS

Because of the wide range of possible open pit slopes and engineering geology conditions which may occur within the proposed pits, and the preliminary nature of mine planning studies and geologic interpretations to date, slope designs for the proposed Grizzly/Transfer Project must be flexible and adaptable to a variety of conditions. This is particularly true for the footwall slopes, where changes in bedding dip and competency of footwall materials may result in substantial variations in kinematic conditions and slope stability considerations. In this regard, slope design for the proposed pits was approached as a series of general slope design concepts (Hawley & Stewart, 1986), rather than as a fixed recommended design for each potential slope. Based on the engineering geology assessments, results of kinematic assessments, and idealized slope stability analyses, a series of design concepts have been prepared which provide for all cases within the range of conditions anticipated.

Slope design concepts are described below and summarized in Table VI. Each slope design concept is valid for a given slope type, orientation, bedding dip and rock mass unit, and is identified using a simple coding system of three symbols. The first symbol refers to the basic wall type (F = footwall, H = hanging wall and E = endwall). The next group of symbols refers to the rock mass unit (A = Immediate Footwall Rocks, B = Intermediate Footwall Rocks, C = Competent Footwall Rocks, G = Gates Hanging Wall Rocks, and H = Hulcross Hanging Wall Rocks). The last symbol refers to a range of bedding dips and varies depending on the basic wall type and general orientation of bedding.

5.4.1 Footwall Slope Design Concepts (FA-1 to FA-6, FB-1 to FB-7 and FC-1 to FC-6

Slope design concepts for the footwall slopes are based on the results of kinematic assessments, plane and wedge failure analyses and bilinear slab analyses (see Fig. 2). For slopes where bedding dips less than about 30° to 40° (depending on the rock mass unit), no benches are required (Slope Design Concepts FA-1, FB-1 and FC-1). Where bedding dips between 30°/40° and 90° (Slope Design Concepts FA-2 to FA-6, FB-2 to FB-7 and FC-2 to FC-6), bench height is limited based on the potential for slab failures. Berms are provided of sufficient width to provide adequate access and catchment for small slab-type failures, ravelling and rockfalls, and to account for potential breakback of bench crests due to blasting or excavation technique. Bench faces are assumed excavated parallel to bedding and bedding is assumed not to be undercut (see Fig. 21).

For preliminary assessment purposes, a nominal breakback of footwall bench crests of 2m is assumed. This breakback was added to the minimum berm width (i.e. that berm width required to contain failures and ravelling debris and provide access) to determine the design berm width. Experience has shown that where ripping can be utilized to excavate the final slope, such as along the base of a coal seam, breakback of bench crests can be eliminated or substantially reduced. Where blasting is required to develop the final slope, breakback of bench crests can be substantial, and depends to a large degree on the blasting technique utilized. Depending on the actual breakback achieved, design berm widths could be wider or narrower than those indicated in Table VI. Additional comments regarding blasting, artificial support, etc. are given in Table VI and Section 5.5.

5.4.2 Hanging Wall and Endwall Slope Design Concepts (HG-1, HG-2, EG-1, HH-1 and EH-1)

Based on the apparent plunge or dip of the wedge or plane failure considered to control bench stability, and the berm width required to contain failures on the slope and to provide adequate access, detailed slope configurations were prepared for northeast hanging walls, southwest hanging walls and endwalls. Slope geometries, prepared in accordance with the definitions given in Fig. 21, are summarized on the right side of Table V. Results in Table V were then evaluated with respect to practical mining configurations, overall slope heights and consequences of bench scale failures. In addition, results of slope and bench documentation conducted in the Shikano Pit were considered, and slope design concepts were prepared for each wall. Recommended preliminary slope design concepts are summarized in Tables V and VI.

In all cases, 20m high (i.e. double) benches with 90° design bench face angles are recommended for endwall and hanging wall slopes. Design berm widths in Gates Formation rocks (i.e. Design Concepts EG1, HG1 and HG2) vary from 15.5m to 18m (depending on assumed breakback) and intermediate slope angles vary from 48° to 52°, depending on the wall orientation and actual breakback achieved. Recommended preliminary slope designs for end-walls or hanging walls in Hulcross rocks (Design Concepts HH-1 and EH-1) are based on an assumed long term breakback of benches due to ravelling and rockfalls to an effective bench face angle of 60-65°. Design berm widths of 19m and intermediate slope angles of 46.5° are recommended in Hulcross rocks, regardless of slope or bedding orientation.

It should be noted that slope design concepts summarized in Table VI reflect maximum overall slopes based on results of kinematic assessments and assumed breakback of bench crests. If actual bench crest breakbacks are greater than those anticipated, wider berms and flatter intermediate slopes may be necessary. In this regard, some form of controlled blasting may be necessary to minimize breakback and achieve the optimum overall slope design. In general, higher benches provide for wider berms and better access and catchment for potential failures, although the size of potential failures may increase. Hence, double benches have been selected as providing the best compromise between access, catchment and size of potential failures.

5.4.3 Application of Slope Design Concepts

Based on the slope design concepts discussed above and summarized in Table VI, a geotechnical slope design can be developed for each of the proposed final slopes. That is, based on the estimated bedding orientation, structural domain, rock mass unit and wall type and orientation, an appropriate slope design can be developed from Table VI. In most cases, depending on wall type and location, the slope design concepts are also applicable to the design of interim slopes.

Design concepts given in Table VI for footwall slopes refer to bedding dip ranges. In these cases, the given design concept may be applied for any bedding dip less than or equal to the maximum value indicated for the range. For example, in the dip range where 40m high benches are acceptable, 30m, 20m or 10m bench heights would also be satisfactory, but 50m high benches would be unsatisfactory. In this case, the choice between 40m, 30m, 20m and 10m benches would be based on operational considerations and slope geometry required for stability on adjacent sections.

A schematic example of how the design concepts might be applied is given in Fig. 22. Initially, a pit bottom must be chosen on section. This decision is usually based on coal seam geometry and economics. The slope design is begun at this point and developed upwards. The geometry and rock mass conditions are assessed and a suitable slope design chosen from Table VI. This design is projected upwards for as far as conditions remain appropriate for the chosen slope design concept. When a point is reached where conditions have changed significantly, a new design concept must be chosen for the next slope segment. This process is repeated on all geologic sections along the slope until the slope design reaches surface. Differences or inconsistencies in design between adjacent sections must be resolved by blending (gradually changing from one design to the next), or by choosing the more conservative design. Operational considerations must also be considered to arrive at an efficient and practical slope geometry.

In terms of footwall slope design, an alternative approach may also be feasible. By removing the relatively incompetent Immediate Footwall Rock, it may be possible to extend the initial unbenched height of the footwall in Competent or Intermediate Rock, thereby reducing or possibly eliminating the need for benches on the slope. The decision to remove or retain the veneer of Immediate Footwall Rock should be based on a comparison of required stripping for the two options.

5.5 GENERAL SLOPE DESIGN RECOMMENDATIONS

In addition to the specific design guidelines related to slope geometry given above, the following general slope design recommendations are provided.

5.5.1 Excavation Techniques

The objective of controlled excavation along final walls is to minimize rockfall hazards and bench failures and reduce bench crest breakback. In this regard, free digging or ripping with a dozer, rather than blasting, should be utilized wherever practical, such as along the base of coal seams. Where this is not feasible, some form of controlled blasting should be utilized to minimize damage to the bench face and reduce potential breakback. If breakback can be reduced, and steeper bench face angles can be maintained, steeper intermediate slope angles may be feasible in some areas. The optimum system for controlled blasting should be determined by field blasting trials. In any case, all slopes should be thoroughly scaled to minimize rockfall hazards to personnel and equipment.

All benches must be thoroughly scaled and debris cleaned from berms during excavation. Berms must be kept reasonably clean of debris to remain effective as rockfall catchments. Cleaning berms will be particularly important on hanging walls and endwalls, where it is anticipated that significant ravelling may occur.

5.5.2 Remedial Measures

In some areas, remedial measures, possibly in the form of artificial support, may be required. Artificial support may be particularly useful in cases where unanticipated bedding rolls occur or where faults transect footwall slopes or otherwise affect stability. The usefullness of artificial support has already been demonstrated in the development of the

Marmot J Seam Footwall in the Mesa Pit. Typical situations where artificial support may be useful are illustrated schematically in Fig. 23. Actual implementation of artificial support is site specific and must be designed accordingly.

In some situations, provision of additional, strategic benches may be an alternative to artificial support. Such an approach would require a remedial benching margin to be incorporated into the overall slope design. Typical benching alternatives to artificial support are also illustrated schematically in Fig. 23.

Depending on the potential problems associated with faulting and folding, and the reliability of geologic interpretations, a combination of artificial support and remedial benching may be the optimum approach to remedial design.

5.5.3 Trial Slopes and Slope Documentation

Slope design concepts and recommendations are based on anticipated rock mass behaviour and should be confirmed through the use of trial slopes and slope documentation, particularly on the first few benches. Trial slopes will be particularly useful in assessing bench crest breakback. Trial slopes will also permit the operational practicability of recommended slope designs to be assessed. In this regard, depending on the results of field trials on interim and final slopes, modifications to the various slope design concepts and recommended slope designs may be made, if necessary.

5.5.4 Groundwater Monitoring and Control

Monitoring of piezometric levels in all major slopes should be carried out periodically. As discussed in Sections 4.7 and 4.8, existing and recommended piezometers should be monitored on a regular basis, both prior to and during mining operations. Pre-mining monitoring will provide a seasonally adjusted baseline from which the effects of excavation can be assessed. After development of final walls commences and preliminary monitoring data are available, groundwater control measures can be installed, as required.

5.5.5 Monitoring Slope Movement

Slope movements in open pit mining operations can usually be dealt with in such a way that operations can be effectively continued with little or no loss in production rates. It is most important that slope instability be noted at the earliest possible time so that, if necessary, plans may be altered without disruption of the mining process.

If slope movements are detected, it is most important to monitor displacements and to determine the type, geometry, cause, rate and direction of movement. While continuous, slow displacement of a slope may not suggest imminent danger of complete failure, appropriate movement monitoring will indicate accelerations which usually precede failure. When such accelerations are noted, implementation of immediate, previously planned remedial action may prevent or delay failure.

At Grizzly/Transfer, periodic visual inspections of all pit slopes should be conducted as a means of first identifying potential areas of slope movement. In addition, a system of movement monitoring should be immediately established in all walls where slope failures could adversely affect mine production or operations. A series of movement monitoring "hubs" or survey benchmarks should be established on selected benches at an initial spacing of about 50m to 100m. Hubs should be monitored and results plotted and evaluated at least twice each month. Movements should be plotted in terms of vertical movement, total movement and movement rate. If slope movements are detected, monitoring frequency and the number of hubs should be increased and appropriate remedial measures such as groundwater depressurization, slope flattening, buttressing, etc. should be considered, if necessary.

5.5.6 Ongoing Geotechnical Work

The optimum method of slope design for the proposed open pits is an iterative process whereby theoretical slope designs are prepared, evaluated with respect to operational constraints, and modified on the basis of updated geological interpretations and results of trial slopes and slope documentation.

Because of the interactive nature of this process, and the potential variability in geological conditions, periodic reviews of recommended slope designs and design concepts should be carried out as the geologic interpretation is updated. Designs should be confirmed or modified as necessary. In particular, because of the preliminary nature of present geologic interpretations and mine planning studies, a thorough review of proposed pit slopes should be conducted once detailed mine plans have been prepared and prior to commencement of mining.

6. WASTE DUMPS AND SEDIMENTATION PONDS

As discussed above in Sections 1 and 3.6, due to the timing of the exploration program and mine planning requirements, detailed proposed pit plans, waste dump layouts, etc. have not been completed for this study. Similarly, final sedimentation pond locations have not been selected. As such, the waste dump and sedimentation pond configurations illustrated on plan in Fig. 13 and on cross section in Figs. 14 and 15 are intended only to indicate the general location, size and extent of such facilities. It is understood that revised configurations will be provided for assessment once detailed mine plans are available. The discussion below is intended to assist with the detailed planning of the waste dumps and sedimentation ponds by providing general geotechnical guidelines, concepts and considerations that can be input into future work. Depending on final dump and pond configurations, further detailed geotechnical assessments may have to be undertaken in some areas.

The stability of the waste dumps will be controlled by the strength of the surficial soils and bedrock materials on which the dumps will rest. Bedrock would constitute an adequate supporting medium for waste dumps constructed at the proposed configurations and heights. However, the capability of the soil overburden to support the individual waste dumps may be limited, as discussed in the following sections.

6.1 MATERIAL PROPERTIES

6.1.1 Peat and Organic Silt

Peat and organic silt form a mantle on some of the wetland areas, the most noticeable being the flat areas under portions of the GD2 Waste Dump site between the overland conveyor and the mine access road (e.g. approximately 2m of soft peat was encountered in TP1). Such materials exhibit negligible strength.

6.1.2 Coarse Grained Soils

Coarse grained soils, defined as soil containing no more than 10% silt and clay, includes most of the colluvial and glaciofluvial deposits. These soils are expected to behave as drained, frictional (i.e. cohesionless) materials when subjected to foundation loading. Lower bound friction angles in the range of 30° to 34°, depending on the proportion of fines, are considered appropriate. Where dumps are founded on natural slopes of less than about 25° and underlain by colluvial or coarse grained glaciofluvial soils, Factors of Safety for potential sliding surfaces through foundations in these soils will generally exceed 1.2 regardless of dump height. Special dump placement or advancement procedures will generally not be required in these cases, except where construction of rock drains is required, as discussed below.

Where natural slopes in these foundation materials exceed about 25° (e.g. some portions of the GD1 and TD1 Dumps) special dump advancement procedures may be required, such as dumping along contours or directly down narrow draws, to maintain stable dump configurations.

In general, the gradation of the glaciofluvial material is relatively coarse. However, because of lateral variations in gradational characteristics, which are common to fluvial deposits, and in the absence of very detailed subsurface information, it may be prudent to assume that layers of fine material are present in some areas. In addition, up to about 3m of fine grained slopewash has been observed on top of the coarse grained glaciofluvial sand and gravel along the northwestern edge of the glaciofluvial terrace in the vicinity of GD3 and SP2. In such circumstances, the otherwise coarse grained soils should be assessed and treated as mixed or fine grained soils as described below.

In the vicinity of SP1, soft to very soft clayey silt to silty clay underlies saturated glaciofluvial sand and gravel in Test Pit 11 and stiff clayey silt to silty clay overlies saturated glaciofluvial sand and gravel in Test Pit 12. These soft fine grained soils within the glaciofluvial terrace may be remnants of a glaciolacustrine deposit. Without further detailed information regarding the location and size of SP1 and site specific subsurface conditions, the foundation materials in this area should be assumed and treated as fine grained soils, as discussed below.

6.1.3 Mixed Grained Soils

Mixed grained soils consist of a mixture of fine and coarse sizes, including silt and clay particles. Glacial till, some of of the fluvioglacial deposits, and some of the colluvial materials are in this category.

Exposures of glacial till are found mainly within the proposed GD2 Waste Dump site and in a portion of the GD3 site. In general, and based on the limited exposures available, the glacial till is expected to be dense to very dense and will tend to dilate upon shearing. Thus, any pore pressure development following placement of the dump is likely to be small. A friction angle of 35° is considered to be a suitable lower bound strength value, assuming drained conditions.

Both the till and the relatively limited mixed grained portions of the glaciofluvial and colluvial deposits can be fairly soft within about 2m of the ground surface, particularly where the water table is high. Such conditions occur within swampy depressions in the GD2 area and along the northwestern side of the glaciofluvial terrace in the vicinity of SP2. Although most of these materials are expected to drain fairly rapidly upon being loaded, in the short term, excess pore pressures may develop and undrained shear strengths in the range of 50 kPa to 150 kPa are probably applicable.

6.1.4 Fine Grained Soils

Fine grained soils are generally defined as stratified material, containing a high percentage of silt and clay sizes. This soil was usually deposited under lacustrine conditions.

Depending on the exact dump configurations, glaciolacustrine materials could be encountered under the lower edge of the GD2 Waste Dump and under about the lower half of the GD3 Waste Dump where the material ranges from fine sand to sandy silt to silty clay/clayey silt. The more granular sand to sandy silt material generally ranges from medium dense (i.e. compact) to dense, while the fine grained, silty clay/clayey silt was observed to range between firm to very stiff. A friction angle of 30° is considered appropriate for the granular glaciolacustrine materials. From field observations, an undrained shear strength of 50 kPa is considered appropriate for the bulk of the fine grained glaciolacustrine soils. However, the presence of lower strength zones or layers is considered likely. As discussed above, some very soft to soft fine grained soils, possibly of glaciolacustrine origin, exist in the vicinity of the proposed Sedimentation Pond 1. These materials could have undrained strengths substantially less than 50 kPa.

The glaciolacustrine soils are highly prone to fluvial erosion once the vegetative cover is removed.

6.1.5 Waste Rock

Based on the gradational characteristics, the repose angle of waste rock being placed in the existing dumps and experience from other coal waste dumps in the Rocky Mountain region, a minimum friction angle of 37° is considered to be appropriate for the waste rock.

6.1.6 Bedrock

Because of the generally favourable orientation of the bedding in the dump areas, bedding planes are unlikely to be potential slip surfaces below the waste dump. In addition, the bedrock is generally more competent than the surficial soil deposits, and in any case will not have lower friction angles than the value assumed for the colluvium (see Section 6.1.1).

6.2 GROUNDWATER AND SURFACE WATER

With the exception of Test Pits 11 and 12 in the vicinity of Sedimentation Pond 1, none of the test pits encountered a significant amount of seepage. Thus, groundwater is assumed not to be a significant consideration in the waste dump assessment. However, at least two of the proposed waste dumps will be situated in creeks (i.e. GD2 will cover M18 Creek and TD1 will cover M14 Creek), giving rise to the need to consider the requirements for a rock drain within these dumps. Further discussion concerning rock drains is included with the individual dump assessments.

6.3 ASSESSMENT OF WASTE DUMPS

Based on the discussions in Sections 3.6 and 6.1 concerning topography, surficial soils, etc., the following comments are made with regard to the suitability of the proposed waste dump sites. Precautions that should be taken into consideration and further investigations that should be completed before dump designs are finalized are also outlined.

6.3.1 GD1 Waste Dump

As it is presently envisaged, the GD1 Waste Dump would be a small capacity dump on the southwest side of the Grizzly Pit. If it remains at approximately its present size and its present location (see Figs. 13 and 14), it should be well keyed into the gully. Foundation failures should not be a problem. The only concern associated with this waste dump is that natural slope angles are relatively steep, being up to about 35°, and failures could occur within the dump material if appropriate dumping precautions are not taken. In this regard, it is suggested that where natural slopes exceed about 25°, dumping should be placed along contours, or else directly down the gully, where the gully is narrow and will confine the waste material.

6.3.2 GD2 Waste Dump

Present QCL plans call for the GD2 Waste Dump to cover a large surface area at a relatively shallow average depth of between about 30m and 50m. This shallow depth, along with the relatively flat topography in the area, constitute the main advantages of situating a waste dump at this site. However, there are also two main concerns that must be addressed before detailed dump designs can be finalized for the GD2 Waste Dump. The first concern relates to the presence of soft soils under the dump, particularly in the boggy low lying or depressed areas where deposits of peat and soft silt overlying softer tills have been observed (see Figs. 13 and 14 and Test Pit 1). While the full extent and nature of these deposits have not been delineated in this study, their presence (i.e. for the most part, below about the 925m elevation) indicates that some precautions will have to be taken. Such precautions may involve placing the waste in more than one lift over the soft soil areas, with the first lift being limited to about 10m to 20m in thickness. Depending on the actual extent of the soft soils and the nature of material that will be consigned to this dump, it may be more practical to design the dump in such a way that the soft soils are avoided. In any event, before the GD2 dump design is finalized, the full extent and nature of the soft soils should be determined.

The second concern with regard to the GD2 Waste Dump is the conveyance of M18 Creek through the dump. Due to the nature of the creek and the surrounding topography, diversion of the creek is not felt to be practical. Conveyance of the creek through the dump in a rock drain would appear to be a better solution. While it is understood that a hydrological study of M18 Creek has not been conducted, it is anticipated that the creek flow would be manageable in a rock drain. The relatively small catchment, which will be partially cut off with mining, would likely limit the design flow to in the order of $2m^3/s$. In any event, it is recommended that a hydrological assessment of M18 Creek be conducted to obtain actual data on which a suitable rock drain design can be based. Such parameters as cross-sectional area, block size, the need for an apron or buttress, etc. would be determined. Construction procedures would also be outlined. A hydrology study of M18 Creek will also be required to allow detailed planning and design of sedimentation facilities in the SP3 area.

Development of rock drains is often accomplished by dumping good quality waste rock over a relatively high dump to achieve good segregation of the larger blocks at the bottom of the dump, thus creating a free draining basal layer. At the GD2 Waste Dump, the limited height of the dump may hinder this segregation. However, this may be overcome by selectively allocating the most competent rock for the rock drain. In this regard, it is probable that the best rock available for the rock drain would be the very competent conglomerate unit between F and G Seams on the northeastern limb of the Grizzly Pit.

With regard to the proximity of the proposed GD2 Waste Dump to the overland conveyor, it is recommended that the dump toe be set back at least 10m to 20m from the conveyor and that an impact berm or windrow be created to prevent boulders from rolling out and possibly damaging the conveyor.

6.3.3 GD3 Waste Dump

The GD3 Waste Dump is relatively small in size and rests on fairly gentle topography. Maximum dump heights are in the order of 50m to 60m (see Fig.

14). In the present plans, this dump consists of two main lifts with the upper lift being founded on colluvium and till over bedrock. No stability concerns should exist in this area. However, the lower portion of the dump would appear to be at least partially founded on a glaciolacustrine terrace, which has been found to consist of silty fine sand with silty clay/clayey silt of varying strengths. Thus, to ensure stability, it is recommended that either the lower portion of the dump that would be founded on the fine grained soils be eliminated, or that further investigations be conducted to depths below which could be test pitted. These investigations would likely involve drilling, sampling and strength testing, and would be carried out once a more accurate evaluation of dump configurations has been determined. Should such investigations not be possible, an initial lift thickness would likely have to be restricted to about 10m.

6.3.4 TD1 Waste Dump

This waste dump has been planned as a large volume dump that would be founded essentially on colluvium and/or sound bedrock and would, for the most part, infill the upper reaches of M14 Creek. While side slopes in the M14 valley are up to about 25^{o} to 30^{o}, conditions are considered to be favourable for a stable waste dump. Some contour dumping or dumping directly down narrow draws may be necessary where natural slopes are steeper than about 25^{o}.

The main concern in the TD1 Waste Dump is M14 Creek itself and the need to convey the creek through the dump. However, most of the upper portion of the drainage basin will be covered by the dump and good surface drainage control on the dump platforms could prevent much of the water from reaching the creek, at least upstream of the lower part of the dump. Thus, in view of the fairly small catchment area and attenuation effects of the water seeping through the dump, it is rather doubtful that a placed rock drain will be needed. The key to conveying the water from M14 Creek would appear to be getting good quality rock into the lower portions of the dump and accept the natural drain that forms from segregation.

Notwithstanding the above, it is recommended that a hydrology study of M14 Creek be carried out to obtain reliable flow data. Such information, along with information as to the quality of the waste rock, would be used to confirm the assessment that a placed rock drain would be unnecessary and to evaluate the capability of a natural drain formed by segregation to adequately convey the flow. A hydrology study would also be used as input into the detailed evaluation of sedimentation facilities in the SP1 area.

6.3.5 TD2 Waste Dump

The TD2 Waste Dump, which is a relatively small dump, is founded primarily on colluvium and/or bedrock. Natural slopes are up to about 20° in the upper dump areas, and flatter at lower elevations. Based on these favourable site conditions, no stability problems are envisaged.

6.4 ASSESSMENT OF SEDIMENTATION POND SITES

As discussed in Section 3.6.3, the three general areas that have been identified as potential sedimentation pond sites are all located on the flat lying glaciofluvial terrace along the Murray River. The test pits for SP1 indicate a high variability of soils, ranging from at least 4m to 5m of very soft clayey silt/silty clay to sand and gravel. Thus, depending on the exact location for the pond, it is likely that site specific drilling will be required to conduct a detailed investigation of subsurface conditions at depths greater than that achievable with a backhoe. Such investigations should probably be done when the ground surface is frozen to overcome existing poor access conditions (i.e. boggy and swampy ground).

If soft soils are found to predominate at SP1, construction will be difficult because of the boggy conditions and the pond design will have to reflect the weak soils (i.e. flat, bermed slopes may be required). Furthermore, as several small creeks could be diverted into SP1 (i.e. including M14), the pond may have to be relatively large and the pond design may not be simple. Thus, if possible, the site selected for SP1 should be underlain by granular soils.

Foundation conditions at SP2 and SP3 are favourable, with granular soils existing within about 2m of ground surface.

As an alternative to constructing large, lined sedimentation ponds, it is recommended that consideration be given to alternative structures such as smaller exfiltration ponds. This type of structure would be particularly applicable to SP2 and SP3 where granular soils predominate. A second method of avoiding conventional larger sedimentation ponds would be to construct a series of filter berms, which could be combined with flocculation (if necessary) during high turbidity periods. Such structures, which may be particularly applicable at SP1, have been successfully utilized at other coal mines in the Rocky Mountains area.

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Respectfully submitted,

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