APPENDIX U

Probable Construction Cost Estimate, Miner Flat Dam, February 1995

MORRISON-MAIERLE, INC.

FEBRUARY 2007

PROBABLE CONSTRUCTION

COST ESTIMATE

MINER FLAT DAM

JANUARY 1995

PREPARED FOR:

WHITE MOUNTAIN APACHE TRIBE

WHITERIVER, ARIZONA

PREPARED BY:

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PROBABLE CONSTRUCTION COST ESTIMATE *
MINER FLAT DAM

ITEM NO.	DESCRIPTION	QTY	UNIT	UNIT PRICE	TOTAL PRICE
	SITE PREPARATION				
101	Mobilization/Demobilization	1.00	L.S.	$352,600.00	$352,600.00
102	Diversion and care of stream	1.00	L.S.	110,700.00	$110,700.00
103	Clearing and Grubbing	185.00	ACRE	750.00	$138,750.00
104	Dewatering	12.00	MON.	16,225.00	$194,700.00
105	Rock excavation	113,750.00	C.Y.	18.75	$2,132,812.50
106	Unclassified excavation	14,325.00	C.Y.	6.15	$88,098.75
107	Upstream and Downstream Coffer dams	1.00	L.S.	105,000.00	$105,000.00
108	Access road	1.00	L.S.	481,500.00	$481,500.00
109	Bridge to powerhouse	2,400.00	S.F.	60.00	$144,000.00
110	Abutment and foundation preparation	3,000.00	S.Y.	65.00	$195,000.00
111	Foundation and abutment grouting	1.00	L.S.	811,000.00	$811,000.00
112	Dental concrete	100.00	C.Y.	150.00	$15,000.00
113	Visitor viewpoint	1.00	L.S.	62,500.00	$62,500.00
	RCC DAM				
114	Bedding mix	2,500.00	C.Y.	76.00	$190,000.00
115	RCC for dam	120,702.00	C.Y.	42.00	$5,069,484.00
116	Structural Concrete for stilling basin	2,000.00	C.Y.	120.00	$240,000.00
117	Rock bolts for RCC stilling basin	1.00	L.S.	105,000.00	$105,000.00
118	Facing concrete	5,091.00	C.Y.	45.00	$229,095.00
119	Ogee crest concrete	359.00	C.Y.	150.00	$53,850.00
120	Non-overflow crest concrete	189.00	C.Y.	150.00	$28,350.00
121	Upstream face panels	40,384.00	S.F.	15.50	$625,952.00
122	Impermeable liner	40,384.00	S.F.	5.00	$201,920.00
123	Galleries	300.00	L.F.	390.00	$117,000.00
124	Drain holes from crest	3,050.00	L.F.	10.00	$30,500.00
125	Drain holes from gallery	1,325.00	L.F.	30.00	$39,750.00
126	Security Fence	1.00	L.S.	33,159.00	$33,159.00
127	Instrumentation	1.00	L.S.	50,800.00	$50,800.00
	OUTLET, TOWER, GATES, OPERATORS				
128	Structural Concrete, 4000 psi	1,262.00	C.Y.	650.00	$820,300.00
129	Reinforcing Steel	69,110.00	LBS.	.53	$36,628.30
130	Steel Transition Liner (10 foot dia.)	30.00	L.F.	1,105.00	$33,150.00
131	10 foot diameter RCP	135.00	L.F.	953.00	$128,655.00
132	6 foot diameter steel penstock	180.00	L.F.	750.00	$135,000.00
133	6 foot diameter air shaft	170.00	L.F.	750.00	$127,500.00
134	Bridge to tower	1,200.00	S.F.	60.00	$72,000.00
135	Trashracks	3.00	EACH	5,000.00	$15,000.00
136	Slide gates w/ operators	1.00	L.S.	1,000,000.00	$1,000,000.00
137	Handrails and miscellaneous metal	1.00	L.S.	30,620.00	$30,620.00
138	Emergency Warning System	1.00	L.S.	150,000.00	$150,000.00
	SITE CLEAN-UP				
139	Finish grading and seeding	185.00	ACRE	1,600.00	$296,000.00
				Subtotal	14,691,375.00
				Cont & unlisted items	2,938,275.00
	Probable Construction Cost				$17,629,650.00
	Field survey				100,000.00
	Geotechnical/RCC mix design				352,593.00
	Civil design				1,057,779.00
	Construct management				1,762,965.00
	Probable Total Project Cost				$20,902,987.00

* *The computation details of each bid item are included following this probable cost estimate.*

PROBABLE CONSTRUCTION COST ESTIMATE
MINER FLAT DAM

The following is a discussion of the probable construction cost estimate for Miner Flat Dam. Computation sheets and unit cost data are included in the Appendix.

Bid Item No. 101: Mobilization/Demobilization

The cost of this lump sum bid item was estimated by using 2.5 percent times the total probable construction cost including the contingency and unlisted items. The 2.5 percent was determined by studying bid tabulations and developing a relationship of project mobilization/demobilization cost versus total project cost.

Bid Item No. 102: Diversion and Care of Stream

The cost of this lump sum bid item was estimated as a crew B-6 from the 1994 Means Heavy Construction Cost Data[1]. This crew size would be comprised of two labors, one light equipment operator, and one backhoe operator. Crew B-6 would have an hourly rate of $41.18 per hour including overhead and profit.

The total number hours estimated for this crew on the project would be 8 hours per day at 28 days per month for 12 months. The total number of hours was calculated as: 8 hrs/day X 28 days/month X 12 months/year = 9,225 hrs/year.

The total lump sum was determined by multiplying the hours by the hourly rate or: $41.18/hr X 9,225 hrs/year = $110,691. The total cost was rounded to $110,700.

Bid Item No. 103: Clearing and Grubbing

The unit cost for this bid item was estimated based on the 1994 Montana Department of Transportation (MDT) bid summary. The MDT unit cost for clearing and grubbing was approximately $700/Acre.

The quantity of this bid item was determined from the 1986 project cost estimate developed by Morrison-Maierle, Inc (MMI). The quantity totalled 185 Acres.

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[1] 1994 Means Heavy Construction Cost Data, Means Southam Construction Information Network, 8th Annual Edition.

Bid Item No. 104: Dewatering

The unit cost for this bid item was estimated from the 1994 Means Heavy Construction Cost Data. It was estimated that a minimum of four pumps would be required to run daily. The Means Heavy Construction Cost Data lists this unit cost at $579/day. The monthly rate was computed by assuming that on the average, dewatering would be required for 28 days per month and the unit cost was computed as: $579/day X 28 days = $16,212/month. This cost was rounded off to $16,225/month.

The quantity for this bid item was estimated by assuming a total project duration of 12 months.

Bid Item No. 105: Rock Excavation

The unit cost for this bid item was estimated from the 1994 Means Heavy Construction Cost Data. It was estimated that average drilling and blasting pits would be required since most of the rock was a fractured basalt. The Means Heavy Construction Cost Data lists this unit cost at $18.75/cubic yard.

The quantity for this bid item was determined by delineating the approximate rock excavation limits on the Miner Flat Dam conceptual design plan sheets. The total volume was computed by the average end area method between the upstream and downstream cross sections of the dam. The total quantity of rock excavation computed was 113,742 cubic yards. This volume was rounded to 113,750 cubic yards.

Bid Item No. 106: Unclassified Excavation

The unit cost for this bid item was estimated from 1994 Means Heavy Construction Cost Data. It was estimated that 105 H.P. equipment, and a 300 foot haul, would be used to remove sand & gravelly material in the confined space of the canyon. The Means Heavy Construction Cost Data lists this unit cost at $6.15/cubic yard.

The quantity for this bid item was determined by delineating the approximate gravel material area from the Miner Flat Dam conceptual design plan sheets. The total volume was computed by the average end area method between the upstream and downstream limits of excavation. The total quantity of unclassified excavation computed was 14,326 cubic yards. This volume was rounded to 14,325 cubic yards.

Bid Item No. 107: Upstream and Downstream Coffer Dams

The lump sum cost for this bid item included two components: 1) the coffer dam earthwork; and 2) the temporary outlet pipe.

The unit cost for the earthwork was estimated from 1994 Means Heavy Construction Cost Data. The total unit cost was computed by adding the unit costs for borrow, embankment, and compaction from the Means Heavy Construction Cost Data. These unit costs are: $5.40/cubic yard, $2.48/cubic yard, and $0.96/cubic yard, respectively. The total computed unit cost would be: $5.40 + $2.48 + $0.96 = $8.84/cubic yard.

The unit cost for the temporary outlet pipe was determined by using Roscoe Steel's 1994 cost[2] for a ten foot diameter corrugated metal pipe, $207/lineal foot, and adding an estimated installation unit cost, $43/lineal foot. The total unit cost would be: $207 lineal foot + $43/lineal foot = $250/lineal foot.

The quantity of earthwork was estimated by using the average end area method for volume computation for both the upstream and downstream cofferdams. The total volume calculated was: 993 cubic yards (downstream cofferdam) + 1,515 cubic yards (upstream cofferdam) = 2,508 cubic yards (total).

The total length of temporary outlet pipe was determined by scaling the pipe length off the conceptual design drawings. The total scaled length of the temporary outlet pipe was 330 lineal feet.

The total lump sum cost was the sum of the cofferdam and the temporary outlet pipe costs. The total cost was: [($8.84/cubic yard X 2,508 cubic yards) + ($250/lineal feet X 330 lineal feet)] = $105,072. This cost was rounded to $105,000.

The total quantity for this bid item was one lump sum.

Bid Item No. 108: Access Road

The unit cost for this lump sum bid item was estimated using unit cost information found in the 1994 MDT bid summary. The unit costs for the components of the bid item are: 1) unclassified excavation - $6.15/cubic yard; 2) 18" diameter corrugated metal pipe culvert - $30.00/lineal foot; 3) crushed top surfacing $9.60/ton; 4) rail guard - $19.35/lineal foot; and 5) signs - $500.00/each.

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[2]Roscoe Steel & Culvert Company, 5405 Momont Road, Momont Industrial Park, Missoula, Montana, 59802, (406) 542-0345, 1994 Price Sheet.

The quantity for each component was computed by using the quantities determined in the 1986 cost estimate. These quantities are as follows: 1) unclassified excavation 67,500 cubic yards; 2) 18 inch diameter corrugated metal pipe culvert - 400 lineal feet; 3) crushed top surfacing - 4,500 tons; 4) guard rail - 500 lineal feet; 5) signs - 3 each.

The total estimated lump sum cost was: [($6.15/cubic yard X 67,500) + ($30/lineal foot X 400 lineal feet) + ($9.60/ton X 4,500 tons) + ($19.35 X 500 lineal feet) + ($500/sign X 3 signs)] = $481,500.

The quantity for this bid item was one lump sum.

Bid Item No. 109: Bridge to Powerhouse

The unit cost for this bid item was estimated by contacting the MDT to obtain a unit cost for precast concrete bridges[3]. MDT recommended a cost $50/square foot of bridge. A final unit cost of $60/square foot was used due to the remote location of this project.

The quantity of this bid item was determined by estimating that the bridge would be a 120 feet long by 20 feet wide. The total area would be 120 feet X 20 feet = 2,400 square feet.

Bid Item No. 110: Abutment and Foundation Preparation

The unit cost for this bid item was estimated from the 1994 Means Heavy Construction Cost Data. The unit cost for a track drill including operators was $1,400/day. The unit cost for two jackhammer operators and two laborers was $1,375/day. The total daily unit cost for this bid item was estimated by summing the drilling and jack-hammering items for a total unit cost of $1,400/day + $1,375/day = $2775/day.

This daily unit cost was converted to a unit cost per square yard by assuming that the daily productivity of the crew would be 400 square feet/day. Based on this assumption the unit cost was: [($2,775/day) X (9 square feet/square yard) X (1 day/400 square feet)] = $62.44/square yard. This unit cost was rounded to $65/square yard.

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[3]Telephone Communication with Deven Roberts, Montana Department of Transportation, Bridge Section.

The quantity of this bid item was estimated by using the quantity of 3,000 square feet calculated from the 1986 cost estimate developed by MMI.

Bid Item No. 111: Foundation and Abutment Grouting

This cost estimate includes a drilling and grouting program in each dam abutment and at the dam base. A total length of drilling was estimated to be 6,460 lineal feet in the abutments and 1,240 lineal feet at the dam base. The Bureau of Reclamation was consulted to obtain specific unit cost information for drilling, hook-up, pressure grouting, handling of the grout, and water testing.

The total lump sum cost was estimated as: [$558,593 (abutment grouting) + $80,208 (base grouting) + $10,000 (mobilization)] X 1.25 (contingency) = $811,000.

The quantity of this bid item was one lump sum.

Bid Item 112: Dental Concrete

The unit cost of this bid item was estimated at $150/cubic yard of concrete. The unit cost was determined by increasing the unit cost used in the 1986 cost estimate performed by MMI. The unit cost was $100/cubic yard in 1986.

The quantity of this bid item was estimated to equal 100 cubic yards. This quantity was obtained from the 1986 cost estimate.

Bid Item No. 113: Visitor Viewpoint

The unit cost for this bid item was calculated by taking the lump sum unit cost from the 1986 cost estimate performed by MMI, $49,000, and inflating it based on the Engineering News Record (ENR) cost index of 1.27. Based on this information, the adjusted lump sum unit cost was: $49,000 X 1.27 = $62,318. This unit cost was rounded to $62,500.

The quantity of this bid item was one lump sum.

Bid Item No. 114: RCC Bedding Mix

The unit cost for this bid item was estimated by using the unit cost information from the Upper Stillwater Dam project[4] which was constructed in 1986. The unit cost for RCC bedding for this project was $60/cubic yard. The updated unit cost was estimated by adjusting it with the ENR cost index of 1.27. Therefore, the adjusted unit cost was: $60/cubic yard X 1.27 = $76/cubic yard.

The quantity of this bid item was estimated at 2,500 cubic yards. This quantity was obtained from the 1986 cost estimate.

Bid Item No. 115: RCC for Dam

The unit cost for this bid item was estimated by using the unit cost information from the Lower Chase Creek Dam project[5] which was constructed in 1987. The unit cost for RCC for this project was $33.80/cubic yard. The adjusted unit cost was estimated by adjusting it with the ENR cost index of 1.23. Therefore, the adjusted unit cost was: $33.80/cubic yard X 1.23 = $42/cubic yard.

The quantity of this bid item was estimated at 120,701 cubic yards in the 1986 cost estimate.

Bid Item No. 116: Structural Concrete for Stilling Basin

The unit cost for this bid item was estimated by utilizing the unit cost of $80/cubic yard from the 1986 cost estimate and adjusting it with the ENR cost index of 1.27. The adjusted unit cost was $80/cubic yard X 1.27 = $102/cubic yard. This cost was rounded to $120/cubic yard to account for contingency items.

The quantity of this bid item was estimated by planimetering the stilling basin area shown on the Miner Flat Dam conceptual design drawings. The planimetered area totalled 10,675 square feet. The area was converted to a volume by multiplying the total area by 5 feet: (10,675 square feet X 5 feet) X 1 cubic yard/27 cubic feet = 1,977. The quantity was rounded to 2,000 cubic yards.

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[4] Roller Compacted Concrete III, Kenneth D. Hanson & Francis G. Mclean, American Society of Civil Engineers, 1992.

[5] Roller Compacted Concrete Dams, Kenneth D. Hansen & William G. Reinhardt, McGraw-Hill, 1991.

Bid Item No. 117: Rock Bolts for RCC Stilling Basin

The unit cost for this bid item was estimated by assuming that the rock bolts would consist of #10 reinforcing bar drilled ten feet into the rock for the full depth of the stilling basin and spaced at three feet on center. The total number of bolts was estimated by dividing the total area of the basin by the unit area of each bolt or: 10,675 square feet/9 square feet per bolt = 1,186 or 1,200 bolts. The total weight or steel anchors was estimated by multiplying the total length of #10 rebar by the unit weight of the rebar or: 1,200 bolts X 15 feet/bolt X 4.3 lbs/foot = 77,500 lbs. Finally, the total unit cost was determined by adding the drilling cost for the anchors with the material cost of the anchors or: 1,200 bolts X 15 feet/bolt X $3.53/lineal foot (drilling) + 77,500 lbs x $0.53/lb (bolts) = $104,615. This unit cost was rounded to $105,000.

The quantity of this bid item was one lump sum.

Bid Item No. 118: Facing Concrete

The unit cost for this bid item was estimated by using the unit cost information from the Upper Stillwater Dam project[6] which was constructed in 1986. The unit cost for facing concrete from this project was $35.53/cubic yard. The adjusted unit cost was estimated by adjusting it with the ENR cost index of 1.27. The adjusted unit cost became: $35.53/cubic yard X 1.27 = $45/cubic yard.

The quantity of this bid item was 5,091 cubic yards as estimated in the 1986 cost estimate.

Bid Item No. 119: Ogee Crest Concrete

The unit cost for this bid item was estimated by using the unit cost information from the Upper Stillwater Dam project[7] which was constructed in 1986. The unit cost for the ogee crest concrete from this project was $117.59/cubic yard. The adjusted unit cost was estimated by adjusting it with the ENR cost index of 1.27. The adjusted unit cost was: $117.59/cubic yard X 1.27 = $150/cubic yard.

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[6] Roller Compacted Concrete III, Kenneth D. Hanson & Francis G. Mclean, American Society of Civil Engineers, 1992.

[7] Roller Compacted Concrete III, Kenneth D. Hanson & Francis G. Mclean, American Society of Civil Engineers, 1992.

The quantity of this bid item was 359 cubic yards as estimated from the 1986 cost estimate.

Bid Item No. 120: Non Overflow Crest Concrete

The unit cost for this bid item was the same unit cost as Bid Item No. 119 or $150/cubic yard. The quantity of this bid item was 189 cubic yards as estimated from the 1986 cost estimate.

Bid Item No. 121: Upstream Face Panels

The unit cost for this bid item was estimated by using the unit cost of $12/square foot from the 1986 cost estimate and adjusting it with the ENR cost index of 1.27. The updated unit cost was $12/square foot X 1.27 = $15.50/square foot.

The quantity of this bid item was 40,384 square feet as estimated from the 1986 cost estimate.

Bid Item No. 122: Impermeable Liner

The unit cost for this bid item was estimated from the previous MM/CSSA bid tabulations. The unit cost for this project was $10/square foot.

The quantity of this bid item was 40,384 square feet as estimated from the 1986 cost estimate.

Bid Item No. 123: Galleries:

The unit cost for this bid item was estimated by assuming that the gallery structure would be slip-formed as the RCC would be placed on the dam and that the gallery structure would consist of 12 inch thick walls, 10 feet high. The roof of the gallery would be formed with half a corrugated metal pipe (CMP). Based on these assumptions, unit cost of the gallery was estimated by determining the unit cost of the concrete ($144/lineal foot) plus the unit cost of the CMP ($245/lineal foot). The total unit cost was: $144/lineal foot + $245/lineal foot = $389/lineal foot. This unit cost was rounded to $390/lineal foot.

The quantity of this bid item was 330 lineal feet. This was scaled from the Miner Flat Dam conceptual design drawings.

Bid Item No. 124: Drain Holes from Crest

The unit cost for this bid item was estimated at $10/lineal foot based on past project experience.

The quantity of this bid item was 3,050 lineal feet as estimated from the 1986 cost estimate.

Bid Item No. 125: Drain Holes from Gallery

The unit cost for this bid item was estimated to be $30/lineal foot based on past project experience.

The quantity of this bid item was 3,050 lineal feet as estimated from the 1986 cost estimate.

Bid Item No. 126: Security Fence

The unit cost for this lump sum bid item was estimated by calculating the total cost of the ten foot high chain link fence and the four foot high chain link fence independently and then summing the two. The 1994 Means Heavy Construction Cost Data lists unit costs for the ten foot and four foot high chain link fence at $15/lineal foot and $6/lineal foot, respectively. The quantity of each fence type was determined by scaling the total lineal feet of fencing from the Miner Flat Dam conceptual design drawings. The total fencing quantity for the ten foot and four foot high chain link fence was 1,025 and 2,964 lineal feet, respectively.

The quantity for this bid item was one lump sum.

Bid Item No. 127: Instrumentation

The cost for this lump sum bid item was estimated by adjusting the cost from the 1986 cost estimate. The 1986 cost was $40,000. The updated unit cost was determined by adjusting the 1986 cost, $40,000, with the ENR cost index, 1.27. Therefore, the adjusted unit cost became: $40,000 X 1.27 = $50,800.

The quantity for this bid item was one lump sum.

Bid Item No. 128: Structural Concrete, 4000 psi

The unit cost for this bid item was estimated from the 1994 Means Heavy Construction Cost Data. The unit cost for structural concrete including formwork and finishing was $650/cubic yard.

The quantity of this bid item was 1,262 cubic yards as estimated from the 1986 cost estimate.

Bid Item No. 129: Reinforcing Steel

The unit cost for concrete reinforcing was estimated from the 1994 Means Heavy Construction Cost Data at $0.53/lb.

The quantity of this bid item was estimated at 69,110 lbs in the 1986 cost estimate.

Bid Item No. 130: Steel Transition Liners

This bid item includes the ten foot diameter steel pipe liner that would be cast into the ten foot diameter reinforced concrete outlet pipe. The unit cost of this bid item was estimated at $1,105/lineal foot. This unit cost quotation was obtained from Roscoe Steel and Culvert Company.

The quantity of this bid item was 30 lineal feet and was scaled from the Miner Flat Dam conceptual design drawings.

Bid Item No. 131: 10 Foot Diameter Outlet Pipe

The unit cost for this lump sum bid item was estimated by adjusting the cost from the Miner Flat Dam 1986 cost estimate performed by MMI. The 1986 unit cost was $750/lineal foot. The adjusted unit cost was calculated by adjusting the 1986 cost, $750/lineal, foot by the ENR cost index, 1.27. The adjusted unit cost became: $750/lineal foot X 1.27 = $953/lineal foot.

The quantity of this bid item was 135 lineal feet and was scaled from the Miner Flat Dam conceptual design drawings.

Bid Item No. 132: 6 Foot Diameter Steel Penstock

The unit cost of this bid item was estimated at $750/lineal foot. This unit cost quotation was obtained from Roscoe Steel and Culvert Company.

The quantity of this bid item ia 180 lineal feet and was scaled from the Miner Flat Dam conceptual design drawings.

Bid Item No. 133: 6 Foot Diameter Steel Air Shaft

The unit cost of this bid item was estimated at $750/lineal foot. This unit cost quotation was obtained from Roscoe Steel and Culvert Company.

The quantity of this bid item ia 170 lineal feet and was scaled from the Miner Flat Dam conceptual design drawings.

Bid Item No. 134: Bridge to Tower

The unit cost for this bid item was estimated by contacting the MDT to obtain a unit cost for precast concrete bridges. MDT reported that $50/square foot of bridge was normally used. A final unit cost of $60/square foot was used due to the remote location of this project.

The quantity of this bid item was estimated by assuming a 75 feet long by 16 feet wide bridge. The total area for this bridge would be 75 feet X 16 feet = 1,200 square feet.

Bid Item No. 135: Trashracks

The unit cost for this bid item was estimated to be $5,000/each based on past project experience.

The quantity of this bid item was 3 based on the 1986 conceptual design sheets.

Bid Item No. 136: Slide Gates with Operators

The unit cost for this lump sum bid item was estimated at $1,000,000. This unit cost was determined by obtaining a material unit cost of $910,673 from the Rodney Hunt Company and adding an estimated installation cost of $89,327.

The quantity for this bid item was one lump sum.

Bid Item No. 137: Handrails and Miscellaneous Metal Items:

The unit cost for this lump sum bid item was estimated by combining the total handrail cost and including an estimated amount for other miscellaneous metal items. The total handrail cost for the project was $5,620. This cost was estimated by summing the total handrail on the project, 281 lineal feet, and multiplying it by the unit cost for handrail as listed in the 1994 Means Heavy Construction Cost Data, $20/lineal foot, or: 281 lineal feet X $20/lineal feet = $5,620. The estimated cost for the miscellaneous metal was $25,000. This was based on past project experience.

The total lump sum cost for this bid item was: $25,000 + $5,620 = $30,620.

The total quantity of this bid item was one lump sum.

Bid Item No. 138: Emergency Warning System:

The unit cost for this bid item was estimated to be $150,000 a piece based on past project experience.

The quantity of this bid item was one lump sum.

Bid Item No. 139: Finish Grading and Seeding:

The unit cost for this bid item was estimated at $1,600/acre. This unit cost was taken from the Flower Creek Dams Rehabilitation project which was bid in September 1994.

The total quantity of this bid item was 185 acres based on 1986 cost estimate.

Contingency and Unlisted Items:

Contingency and unlisted items was estimated as 20 percent of the construction cost subtotal.

Field Survey:

The field survey for final design of this project was estimated by allocating 4 crew months of labor and expenses for engineers, surveyors and drafting personnel.

Geotechnical/RCC Mix Design:

The geotechnical and RCC mix design cost for this project was estimated at 2 percent of the probable construction cost.

Civil Design:

The civil design for this project was estimated at 6 percent of the probable construction cost.

Construction Management:

The construction management cost for this project was estimated at 10 percent of the probable construction cost.

APPENDIX

COMPUTATION SHEETS

Morrison Maierle/CSSA	PROJECT: Miner Flat Dam		
	BY: MF	DATE 11/1/92	PROJ. NO.
	CHK:	DATE	PAGE: 1 OF 11

Probable Construction Cost Estimate

Item No. 101: Mobilization / Demobilization:

Assume 3.5% of Probable Construction Cost
Lump Sump. quantity.

Item No. 102: Diversion & Care of stream:

Assume Means guide Crew B-6 which includes:

2 Labors
1 Equip Oper. (Light)
1 Backhoe Loader, 48 H.P

} Hourly Rate (including O&P) = $41.18/Hr

∴ Assume needed full days during entire project.

Monthly rate = $41.18/Hr × 8 Hrs/day × 28 days/month ≈ 9,225/month

∴ Assume the project duration ≈ 12 months.

Total cost = $9,225/month × 12 months = $110,700

Item No. 103: Clearing & Grubbing:

Quantity = 185 Acres (from Barnett).

Use Unit Cost = $700/Acre {From 1992 MDT Bid Summary}

Item No. 104: Dewatering:

Quantity: Assume project duration = 12 months.

- Assume two 4" pumps running daily.

Unit cost = $579/day (Means). or $579/day × 28 days/month = $16,212/mo.

≈ $16,225/mo.

Morrison Maierle/CSSA

PROJECT: Miner Flat Dam		
BY: MF	DATE 11/2/94	PROJ. NO.
CHK:	DATE	PAGE: 2 OF 11

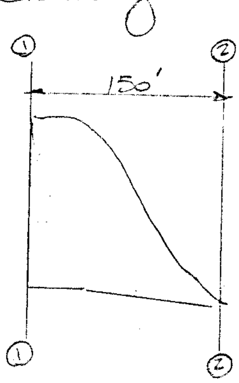
Probable Construction Cost (Cont)

Item No. 105 : Rock Excavation:

Assume Drilling & Blasting Pits average

Unit Cost = \$15.75/C.Y. (Means)

Quantity: Use Average End Area for x-section @ Dam Crest and at toe of RCC Section.



$$\text{Quantity}_{(rock)} = \left\{\left[\frac{17,860\ ft^3/ft + 23,087\ ft^3/ft}{2}\right] \times 150\ ft\right\} \times \frac{1\ yd^3}{27\ ft^3}$$

$$\text{Quantity}_{(rock)} = 113,742\ yd^3$$

Item No. 106 : Unclassified Excavation:

Assume 105 H.P. Sand & Gravel (300' Haul)

Unit Cost = \$6.15/C.Y. (Means)

Quantity : Use Average End Area for x-section @ Dam Crest and @ toe of RCC Section

Area of Talus & Terrace material is the same.

$$\text{Quantity} = \left[2,578.72\ ft^3/ft \times 150\ ft\right] \times \frac{1\ yd^3}{27\ ft^3} = 14,326\ yd^3$$

Item No. 107 : Upsteam & Downstream Coffer Dams:

(Use average end area method).

$$\text{Volume}_{\text{Lower DAM}} = \left(\frac{0 + 130\ ft^3/ft}{2}\right) 13\ ft + \left(\frac{130\ ft^3/ft + 475\ ft^3/ft}{2}\right) 11\ ft + \left(\frac{475\ ft^3/ft + 475\ ft^3/ft}{2}\right) 20\ ft$$

$$+ \left(\frac{475\ ft^3/ft + 113\ ft^3/ft}{2}\right) 27\ ft + \left(\frac{113\ ft^3/ft + 0}{2}\right) 25\ ft = 26,823\ ft^3 \text{ or } 993\ yd^3$$

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Probable Construction Cost Estimate (cont.)

Item No. 107 (cont): Upstream & Downstream Coffer Dams:

$$\text{Volume}_{\text{Upper Dam}} = \left(\frac{0 + 204\ ft^3/ft}{2}\right) 13\ ft + \left(\frac{204\ ft^3/ft + 468\ ft^3/ft}{2}\right) 9\ ft + \left(\frac{468\ ft^3/ft + 468\ ft^3/ft}{2}\right) \ldots$$

$$+ \left(\frac{468\ ft^3/ft + 220\ ft^3/ft}{2}\right) 37\ ft + \left(\frac{220\ ft^3/ft + 57\ ft^3/ft}{2}\right) 22\ ft$$

$$+ \left(\frac{57\ ft^3/ft + 0}{2}\right) 23\ ft = \underline{40{,}905\ ft^3 \text{ or } 1{,}515\ yd^3}$$

$$\therefore \text{Total Volume} = 993\ yd^3 + 1{,}515\ yd^3 = \underline{2508\ yd^3}$$

Unit cost = Borrow + Embankment (+ haul) + Compaction.

$$\text{Unit cost} = \$5.^{40}/C.Y. + \$2^{48}/C.Y. + \$0^{96}/C.Y. = \$8.84/C.Y. \sim \underline{\$9/C.Y.}$$

$$\therefore \text{Total cofferdam cost} = 2508\ yd^3 \times \$9/C.Y. = \underline{\$22{,}572}$$

Pipe culvert: 10' φ CMP: \$250/L.F. × 330 L.F. = \$82,500

Total Cost of this Item = \$82,500 + \$22,572 = \$105,072 or \$105,000

Item No. 108: Access Road:

(Take information compiled previously and update the Unit costs).

		Unit Cost	Quan	Amount
Unclassified Exc.	1994 MDT Bid Sheets	\$6.15/C.Y.	67,500 C.Y.	\$415,125
18" CMP Culvert		\$30.00/L.F.	400 L.F.	\$12,000
Crush Top Surfacing		\$9.60/Ton	4,500 Tons	\$43,200
Guard Rail		\$19.35/L.F.	500 L.F.	\$9,675
Signs		\$500/EACH	3 Each	\$1,500
			Total =	\$481,500

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Possible Construction
Cost Estimate (cont.)

Item 109: Bridge to Powerhouse:

Quantity = 120 ft × 20 ft = 2,400 ft²

Unit Cost = #60/ft² (MDOH uses #50, bump up to #60/ft for remote location).

Item 110: Abutment and foundation Preparation:

Quantity = 3,000 S.Y.

Unit Cost: Assume - 1 Track Drill w/ operator @ #1400/day

- 1 Jackhammer operators @ #1375/day
(2 Jackhammers, 2 Labors)

Total Unit Cost = #2775/day

Productivity = 400 ft²/day (assume 20' × 20' per day).

Number of days = 3,000 yd² × 9 ft²/yd² × 1 day/400 ft² ≈ 68 days

∴ Unit Cost = #2775/day × 1 day/400 ft² = #6.94/ft² or ~#65/yd²

Item 111: Foundation & Abutment Grouting:

(See detail calculations from mtg w/ Mike Kaczmarek)

Total Cost = (#558,593 (Angle holes) + #80,208 (Vertical holes) + #10,000 (Mobiliz Demob)) × 1.25 (contingency) = #811,001

use #811,000

Units = 1 L.S.

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Probable Construction Cost Estimate (Cont)

Item No. 112 : Dental Concrete :

Quantity = 100 C.Y. (from Barnett).

Unit Cost = $150/C.Y (Barnett use $100/C.Y in 1986).

Item No. 113 : Visitor Viewpoint :

Total Cost @ 1986 = $39,000 + $10,000 = $49,000

{Based on ENR Cost Index}: Inflation factor = $\frac{5381}{4231}$ = 1.27 or 27%

∴ Inflated Cost = $49,000 X 1.27 = 62,318 or $62,500

Item No. 114 : RCC Bedding Mix :

Quantity = 2500 C.Y. (from Barnett.)

Unit Cost = $60/C.Y @ 1986 (from Upper Stillwater Costs)

= $60/C.Y X 1.27 (Inflation) = $76/C.Y

Item No. 115 : RCC for Dam :

Quantity = 120,702 C.Y. (from Barnett).

Unit Cost = $33.80/C.Y. @ 1987 (from Lower Chase Creek)

= $33.80/C.Y X 1.23 (Inflation) = $42/C.Y.

Item No. 116 : Structural concrete for Stilling Basin :

Stilling Basin Area = 10675 ft^2 (from planimeter).

Volume = 10,675 ft^2 X 5 ft = 53,375 ft^3 or 1,977 or 2000 cy

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Probable Construction
Cost Estimate (cont)

Item No. 116: (cont)

Unit Cost = 1.27x Barnett's mass concrete cost.

Unit Cost = 1.27 x \$80/C.Y. ≅ \$102/C.Y. use \$120/C.Y.

Item No. 117: Rock Bolts for RCC Stilling Basin:

Unit cost 2" hole drilling rock = \$3.53/L.F. ; Bolts - #10 bar @ 3 ft each way.

Number of Bolts = 10,682 ft² / 9 ft²/bolt ≅ 1,200 bolts   length per bolt = 5' + 10' = 15'/bolt

Wt. steel = 1,200 bolts x 15 ft/bolt x 4.3 #/ft = 77,5[illegible]

Total Cost = [1,200 bolts x 15 ft/bolt] x \$3.53/L.F. + 77,500 lbs x \$0.53/lb ≅ \$65,[illegible]

Item No. 118: Facing Concrete:

Quantity = 5,091 C.Y. (from Barnett).

Unit Cost = \$35.53/C.Y. @ 1986 (from Upper Stillwater Costs)

= \$35.53/C.Y. x 1.27 (inflation) = \$45/C.Y

Item No. 119: Ogee Crest Concrete:

Quantity = 359 C.Y. (from Barnett)

Unit Cost = \$117.59/C.Y. @ 1986 (from Upper Stillwater Costs)

= \$117.59/C.Y. x 1.27 (Inflation) = \$150/C.Y.

Item No. 120: Non-Overflow Crest Concrete:

Quantity = 189 C.Y. (from Barnett)

Unit Cost = \$150/C.Y. (same as above).

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Probable Construction Cost Estimate (cont.)

Item No. 129: Reinforcing Steel:

Quantity = 69,110 lbs (from Barnett).

Unit Cost = \$1,050/Ton (Means) or \$0.53/lb

Item No. 121 Upstream Face Panels:

Quantity = 40,384 ft² (from Barnett).

Unit Cost = \$15.50/ft²

Item No. 122: Impermeable Liner:

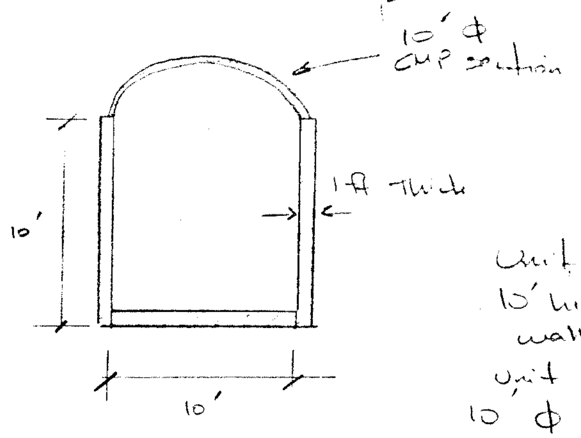
Quantity = 40,384 ft² (Same as above Item 122)

Unit Cost = \$10.00/ft². (from Flower Creek Bid Tabs).
(Assume PVC or Hypalon Liner)

Item No. 123: Galleries:

Quantity = 300 L.F. (scaled from drawing)

Unit Cost = Assume slip form walls and provide 1/2 CMP for roof.



$Vol_{concrete} = (10 ft \times 1 ft) 3 \times 300 ft = 9{,}000 ft^3$ or $333\ yd^3$
(Area) (Length)

Unit cost 10' high retaining walls = \$129/C.Y. (Means)

Unit cost 10' Φ CMP (10 gage) = \$240/LF (Material) + 5/LF Installation = \$245/LF.

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Probable Construction Cost Estimate (cont)

Item No. 123 (cont)

Total Unit Cost = (\$215/LF/Pipe × 300 L.F.) + (\$129/C.Y. Concrete × 335 C.Y.) ÷ 300 L.F.

Total Unit Cost = \$390/L.F.

Item No. 124: Drain Holes from Crest:

Quantity = 3050 L.F. (from Barnett)

Unit Cost = \$15/LF (from Mtg w/ Salo).

Item No. 125: Drain Holes from Gallery:

Quantity = 1325 L.F. (from Barnett).

Unit Cost = \$30/LF (from Mtg w/ Salo).

Item No. 126: Security Fence:

Quantity @ Dam = 100' + 65' + 120' + 50' + 30 + 15' + 15' + 15' + 43' + 10 (visitors viewpoint) + $\underbrace{200' + 200'}_{\text{View Point Rd}}$ + $\underbrace{75' + 75'}_{\text{access bridge}}$ = 1,013 ft ≈ 1025 ft

Unit Cost (10' chain link) = \$15.00/L.F. (w/ gates)

Quantities @ Faunce Dam Access Road = 1482 ft + 1482 ft = 2964 ft

Unit Cost (4'-0" chain link) = 6.00/L.F. (w/ gates)

Total Cost = 1025 ft (\$15.00/ft) + 2964 ft (6.00/ft) = \$33,159

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Probable Construction Cost Estimate (cont).

Item No. 127 Instrumentation:

Cost = \$40,000 (from Barnett)

Cost = \$40,000 X 1.27 Inflation = 50,800

Item No. 128: Structural Concrete, 4000 psi

Quantity = 1,262 yd^3

Cost = \$650/yd^3 (Means)

Item No. 130: Steel Transition Liner:

Quantity = 30 ft of 10 ft ϕ steel Pipe.

Unit Price = \$1105/L.F. (from Roscoe Steel)

Item No 131: 10 foot ϕ outlet Pipe:

Quantity = 135 ft (Scaled from Drawing)

Unit Price = \$750/LF (from Barnett) X 1.27 (Inflation) = \$953/LF

Item No. 132: 6 foot ϕ steel Penstock:

Quantity = 180 feet (Scaled from Drawing)

Unit Price = \$750/L.F. (from Roscoe Steel)

Item No 133: 6 foot ϕ Steel Air shaft:

Quantity = 170 feet (scaled from Drawing).

Unit Price = \$750/LF. (from Roscoe Steel)

Morrison Maierle/CSSA	PROJECT: Miner Flat Dam		
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Probable Construction Cost Estimate (cont)

Item No. 134: Bridge to Tower:

Quantity = 16 ft × 75 ft = 1,200 ft^2 (from Drawings)

Unit Cost = \$60/ft^2 (From MDT for precast/prestressed Bridge)

Item No. 135: Trashracks:

Quantity = 3 (from Drawings)

Unit Cost = \$5,000 Each

Item No. 136: Slide Gates w/ Operators

Quantity = 1 LS. (From Rodney-Hunt)

Unit Cost = \$910,673 (Material) + 90,000 (Installation) = \$1,000,000

Item No. 137: Handrails & Miscellaneous Items:

Quantity (handrails) = 75' + 75' + 12' + 15' + 12' + 30' + 32' + 30' = 281 ft

Unit Cost (Handrail) = \$20/L.F. (Means)

Total Cost (Handrail) = 281 ft × \$20/ft = \$5,620

Assume other Miscellaneous Items Cost = \$25,000

∴ Total L.S. Cost = \$25,000 + \$5,620 = \$30,620 ~ \$30,000

Morrison Maierle/CSSA

PROJECT: Minor Flat Dam		
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Probable Construction Cost Estimate (cont).

Item No. 138: Emergency warning System:

Quantity = 1

Unit cost = $150,000 (from City w/ Salo).

Item No. 139: Finish grading and seeding:

Quantity = 135 Acres

Unit Cost = $1600/Acre (from Flower Creek Bid tab).

MORRISON-MAIERLE/CSSA   11/ 9/94
9:38 AM

EARTHWORK COMPUTATIONS
MINER FLAT DAM, UPPER COFFERDAM

% SHK/S   .00

	AREAS		AVE AREAS (SQFT)			YARDAGE	VOLUMES IN C.Y.		
STA	EMB	EXC	EMB	EXC	DIST	FACTOR	EMB	EMB (%)	EXC
===	===	===	===	===	====	======	===	========	===
0	.00								
13	204.00		102.00	.00	13.00	.0370	49.11	49.11	.00
22	468.00		336.00	.00	9.00	.0370	112.00	112.00	.00
65	468.00		468.00	.00	43.00	.0370	745.33	745.33	.00
102	220.00		344.00	.00	37.00	.0370	471.41	471.41	.00
124	57.00		138.50	.00	22.00	.0370	112.85	112.85	.00
147	.00		28.50	.00	23.00	.0370	24.28	24.28	.00
			.00	.00	-147.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
							-----	-------	------
							1514.98	1514.98	.00

(Checks)

DIFFERENCE IN QUANTITY =   -1514.98

MORRISON-MAIERLE/CSSA [illegible]
[illegible] AM

EARTHWORK COMPUTATIONS
MINER FLAT DAM, DOWNSTREAM COFFER DAM

% SHRK/S .00

	AREAS		AVE AREAS SQFT			YARDAGE	VOLUMES IN C.Y.		
STA	EMB	EXC	EMB	EXC	DIST	FACTOR	EMB	EMB (%)	EXC
0	.00								
13	150.00		65.00	.00	13.00	.0370	31.30	31.30	.00
24	455.00		302.50	.00	11.00	.0370	123.24	123.24	.00
52	475.00		475.00	.00	28.00	.0370	492.59	492.59	.00
79	113.00		294.00	.00	27.00	.0370	294.00	294.00	.00
104	.00		56.50	.00	25.00	.0370	52.31	52.31	.00
			.00	.00	-104.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
			.00	.00	.00	.0370	.00	.00	.00
							-----	-------	------
							993.44	993.44	.00

(Checks!)

DIFFERENCE IN QUANTITY = -993.44

UNIT COST
DATA SHEETS

TABLE 1
UPPER STILLWATER DAM FINAL RCC COSTS (1986)

DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL PRICE	% RCC TOTAL
RCC MIX A	CY	1,163,000	$10.40	$12,095,200	21.9%
	CM	889,177	$13.60		
RCC MIX B	CY	307,000	$13.65	4,190,550	7.6%
	CM	234,718	$17.85		
SUBTOTAL @ BID PRICE	CY	1,470,000	$11.08	$16,285,750	29.4%
	CM	1,123,895	$14.49		
RCC SETTLEMENT	LS			$15,692,000	28.4%
SUBTOTAL RCC COST NO CEMENT, FLYASH	CY	1,470,000	$21.75	$31,977,750	57.8%
	CM	1,123,895	$28.45		
CEMENT FOR RCC	TON	102,348	$81.50	8,341,399	15.1%
	MT	92,849	$89.84		
FLY ASH FOR RCC	TON	223,216	$45.00	10,044,720	18.2%
	MT	202,498	$49.60		
SUBTOTAL CEMENT & FLYASH FOR RCC	CY	1,470,000	$12.51	$18,386,119	33.2%
	CM	1,123,895	$16.36		
TOTAL RCC INCL CEMENT & FLYASH	CY	1,470,000	$34.26	$50,363,869	91.0%
	CM	1,123,895	$44.81		
FACING CONCRETE UP & DOWNSTREAM	CY	87,000	$36.00	3,132,000	5.7%
	CM	66,516	$47.09		
CEMENT FOR FACING CONCRETE	TON	16,313	$81.50	1,329,510	2.4%
	MT	14,799	$89.84		
FLYASH FOR FACING CONCRETE	TON	11,093	$45.00	499,185	0.9%
	MT	10,063	$49.60		
SUBTOTAL FACING W/CEMENT & FLYASH	CY	87,000	$57.02	$ 4,960,695	9.0%
	CM	66,516	$74.58		
TOTAL, RCC INCL FACING CONCRETE	CY	1,557,000	$35.53	$55,324,563	100.0%
	CM	1,190,412	$46.48		
DAM STRUCTRUAL CONCRETE	CY	11,326	117.59	1,331,794	2.4%
	CM	8,666	153.67		
FDN. LEVELING CONCRETE	CY	76,653	$60.78	4,659,115	8.4%
	CM	58,653	$79.44		
TOT. DAM CONCRETE	CY	1,644,979	$37.27	$61,315,472	110.8%
	CM	1,258,688	$48.71		

Figure 10.4 RCC prices by low bidder.

Prospect	Bid date	RCC construction date	As bid, yd^3	Aggregate and process	Cost per yd^3 (U.S. $) Cement	Fly ash	Total
1. Willow Creek Dam, Oregon	10-23-81	1982	401,000 Average of four mixes	11.56	6.23 (117# average @ 106.20/ton)	1.21 (39# @ 61.60/ton)	19.00
2. Austin Detention Dams, Texas	6-28-83	1984	20,670	18.00	7.00 (200# @ 70.00/ton)	1.20 (80# @ 30.00/ton)	26.20
3. Upper Stillwater Dam, Utah	10-17-83	1983-87	1,357,000 Average of two mixes	10.78 1.07*	5.37 (132# @ 81.50/ton)	6.59 (293# @ 45.00/ton)	23.81
4. Winchester Dam, Kentucky	12-13-83	1984	32,000		Bid at lump sum item (175#)	—	32.50
5. Dolet Hills Plant Spillway, Louisiana	12-20-83	1984	26,123 Average of two mixes	27.00	5.60 (160# @ 70.00/ton)	0.91 (64# @ 28.50/ton)	33.51
6. Galesville Dam, Oregon	3-14-84	1983	210,500 Average of two mixes	15.56†	3.95 (91# average @ 87.00/ton)	1.91 (87# average @ 44.00/ton)	21.42
7. Monksville Dam, New Jersey	4-10-84	1986	289,000 Average of two mixes	13.72	3.70 (108# average @ 68.58/ton)	—	17.42‡
8. Middle Fork Dam, Colorado	3-21-84	1984	35,000			—	less than 25.00
9. Grindstone Canyon Dam, New Mexico	6-20-85	1986	114,500	20.20	4.38 (125# @ 70.00)§	1.00 (50# @ 40.00)§	25.58
10. Elk Creek Dam, Oregon	1-16-86	1987-88	999,000 + 41,860	14.00	4.13 (118# @ 70.00/ton)	1.14 (56# @ 2.90/ft^3) + 0.29 admix	19.56
11. Lower Chase Creek Dam, Arizona	2-3-87	1987	26,830		Bid as lump sum contract (105#)	(70#)	33.80
12. Stacy Dam Spillway, Texas	3-5-87	1988-89	103,800	18.05	4.62 (200# @ 46.17/ton)	1.20 (100# @ 23.83/ton)	23.87¶
13. Stagecoach Dam, Colorado	3-5-87	1988	43,500	23.00	4.80 (120# @ 80.00/ton)	1.54 (88# @ 35.00/ton)	29.34
14. Cuchillo Negro Dam, New Mexico	9-19-89	1990	103,700	15.00	5.20 (130# @ 80/ton)	2.00 (100# @ 40/ton) + 0.68 Admix	22.88

NOTE: 1 yd^3 = 0.765 m^3; 1 lb/yd^3 = 0.593 kg/m^3.
*Cost of government-furnished sand for RCC mix.
†6-in. (−) aggregate furnished from previous road contract—cost \$6.75/yd^3 (not included in bid).
‡Includes 5.5 percent increase from actual bid due to one-year delay in award.
§Actual mix contained an average of 135 lb of cement and no fly ash.
¶Average of three lowest bidders—low bidder at \$19.41/yd^3.

Crew B-5	Bare Costs Hr.	Bare Costs Daily	Incl. Subs O & P Hr.	Incl. Subs O & P Daily	Cost Per Man-Hour Bare Costs	Cost Per Man-Hour Incl. O&P
1 Labor Foreman (outside)	$21.00	$168.00	$33.25	$266.00	$21.46	$33.52
Laborers	19.00	608.00	30.10	963.20		
2 Equip. Oper. (med.)	24.35	389.60	37.35	597.60		
1 Mechanic	25.95	207.60	39.80	318.40		
1 Air Compr., 250 C.F.M.		105.40		115.95		
2 Air Tools & Accessories		29.60		32.55		
2-50 Ft. Air Hoses, 1.5" Dia.		12.80		14.10		
1 F.E. Loader, T.M., 2.5 C.Y.		784.60		863.05	14.57	16.03
64 M.H., Daily Totals		$2305.60		$3170.85	$36.03	$49.55

Crew B-6	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
2 Laborers	$19.00	$304.00	$30.10	$481.60	$20.47	$32.03
1 Equip. Oper. (light)	23.40	187.20	35.90	287.20		
1 Backhoe Loader, 48 H.P.		199.60		219.55	8.32	9.15
24 M.H., Daily Totals		$690.80		$988.35	$28.79	$41.18

Crew B-7	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Labor Foreman (outside)	$21.00	$168.00	$33.25	$266.00	$20.23	$31.83
4 Laborers	19.00	608.00	30.10	963.20		
1 Equip. Oper. (med.)	24.35	194.80	37.35	298.80		
1 Chipping Machine		193.80		213.20		
1 F.E. Loader, T.M., 2.5 C.Y.		784.60		863.05		
2 Chain Saws		92.80		102.10	22.32	24.55
48 M.H., Daily Totals		$2042.00		$2706.35	$42.55	$56.38

Crew B-7A	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
2 Laborers	$19.00	$304.00	$30.10	$481.60	$20.47	$32.03
1 Equip. Oper. (light)	23.40	187.20	35.90	287.20		
1 Rake w/Tractor		203.60		223.95		
2 Chain Saws		44.40		48.85	10.33	11.37
24 M.H., Daily Totals		$739.20		$1041.60	$30.80	$43.40

Crew B-8	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Labor Foreman (outside)	$21.00	$168.00	$33.25	$266.00	$20.98	$32.59
2 Laborers	19.00	304.00	30.10	481.60		
2 Equip. Oper. (med.)	24.35	389.60	37.35	597.60		
1 Equip. Oper. Oiler	20.75	166.00	31.85	254.80		
2 Truck Drivers (heavy)	19.70	315.20	30.35	485.60		
1 Hyd. Crane, 25 Ton		491.80		541.00		
1 F.E. Loader, T.M., 2.5 C.Y.		784.60		863.05		
2 Dump Trucks, 16 Ton		797.20		876.90	32.40	35.64
64 M.H., Daily Totals		$3416.40		$4366.55	$53.38	$68.23

Crew B-9	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Labor Foreman (outside)	$21.00	$168.00	$33.25	$266.00	$19.40	$30.73
4 Laborers	19.00	608.00	30.10	963.20		
1 Air Compr., 250 C.F.M.		105.40		115.95		
2 Air Tools & Accessories		29.60		32.55		
2-50 Ft. Air Hoses, 1.5" Dia.		12.80		14.10	3.70	4.06
40 M.H., Daily Totals		$923.80		$1391.80	$23.10	$34.79

Crew B-10	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
12 M.H., Daily Totals		$270.80		$419.20	$22.57	$34.93

Crew B-10A	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Roll. Compact., 2K Lbs.		81.60		89.75	6.80	7.48
12 M.H., Daily Totals		$352.40		$508.95	$29.37	$42.41

Crew B-10B	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Dozer, 200 H.P.		819.60		901.55	68.30	75.13
12 M.H., Daily Totals		$1090.40		$1320.75	$90.87	$110.06

Crew B-10C	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Dozer, 200 H.P.		819.60		901.55		
1 Vibratory Roller, Towed		93.80		108.70	76.53	84.19
12 M.H., Daily Totals		$1189.20		$1429.45	$99.10	$119.12

Crew B-10D	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Dozer, 200 H.P.		819.60		901.55		
1 Sheepsft. Roller, Towed		110.60		121.65	77.52	85.27
12 M.H., Daily Totals		$1201.00		$1442.40	$100.09	$120.20

Crew B-10E	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Tandem Roller, 5 Ton		135.40		148.95	11.28	12.41
12 M.H., Daily Totals		$406.20		$568.15	$33.85	$47.34

Crew B-10F	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Tandem Roller, 10 Ton		220.80		242.90	18.40	20.24
12 M.H., Daily Totals		$491.60		$662.10	$40.97	$55.17

Crew B-10G	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Sheepsft. Roll., 130 H.P.		526.20		578.80	43.85	48.24
12 M.H., Daily Totals		$797.00		$998.00	$66.42	$83.17

Crew B-10H	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Diaphr. Water Pump, 2"		20.00		22.00		
1-20 Ft. Suction Hose, 2"		4.40		4.85		
2-50 Ft. Disch. Hoses, 2"		6.80		7.50	2.60	2.86
12 M.H., Daily Totals		$302.00		$453.55	$25.17	$37.79

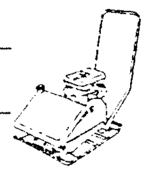
Crew B-10I	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Diaphr. Water Pump, 4"		55.40		60.95		
1-20 Ft. Suction Hose, 4"		11.40		12.55		
2-50 Ft. Disch. Hoses, 4"		12.80		14.10	6.63	7.30
12 M.H., Daily Totals		$350.40		$506.80	$29.20	$42.23

Crew B-10J	Hr.	Daily	Hr.	Daily	Bare Costs	Incl. O&P
1 Equip. Oper. (med.)	$24.35	$194.80	$37.35	$298.80	$22.57	$34.93
.5 Laborer	19.00	76.00	30.10	120.40		
1 Centr. Water Pump, 3"		30.80		33.90		
1-20 Ft. Suction Hose, 3"		7.40		8.15		
2-50 Ft. Disch. Hoses, 3"		8.80		9.70	3.92	4.31
12 M.H., Daily Totals		$317.80		$470.95	$26.49	$39.24

		021 100 | Site Clearing	CREW	DAILY OUTPUT	MAN-HOURS	UNIT	1994 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
116	1800	36" to 48" diameter, softwood	B-10M	70	.171	Ea.		3.87	14.20	18.07	21.50	116
	1850	Hardwood	↓	35	.343	↓		7.75	28.50	36.25	43.50	
		021 140 | Stripping										
144	0010	STRIPPING Topsoil, and stockpiling, sandy loam										144
	0020	200 H.P. dozer, ideal conditions	B-10B	2,300	.005	C.Y.		.12	.36	.48	.57	
	0100	Adverse conditions	"	1,150	.010			.24	.71	.95	1.14	
	0200	300 HP dozer, ideal conditions	B-10M	3,000	.004			.09	.33	.42	.51	
	0300	Adverse conditions	"	1,650	.007	↓		.16	.60	.76	.91	
		021 150 | Selective Clearing										
154	0010	SELECTIVE CLEARING										154
	1000	Stump removal on site by hydraulic backhoe, 1-1/2 C.Y.										
	1050	8" to 12" diameter	B-30	33	.727	Ea.		15.45	45	60.45	73.50	
	1100	14" to 24" diameter		25	.960			20.50	59	79.50	96.50	
	1150	26" to 36" diameter	↓	16	1.500	↓		32	92.50	124.50	151	
	2000	Remove selective trees, on site using chain saws and chipper,										
	2050	not incl. stumps, up to 6" diameter	B-7	18	2.667	Ea.		54	59.50	113.50	151	
	2100	8" to 12" diameter		12	4			81	89.50	170.50	225	
	2150	14" to 24" diameter		10	4.800			97	107	204	271	
	2200	26" to 36" diameter	↓	8	6			121	134	255	340	
	2300	Machine load, 2 mile haul to dump, 12" diam. tree, add				↓				40	60	
		021 200 | Structure Moving										
204	0010	MOVING BUILDINGS One day move, up to 24' wide										204
	0020	Reset on new foundation, patch & hook-up, average move				Total					8,500	
	0040	Wood or steel frame bldg., based on ground floor area	B-4	185	.259	S.F.		5.05	2.35	7.40	10.55	
	0060	Masonry bldg., based on ground floor area	"	137	.350			6.80	3.17	9.97	14.25	
	0200	For 24' to 42' wide, add				↓					15%	
	0220	For each additional day on road, add	B-4	1	48	Day		935	435	1,370	1,950	
	0240	Construct new basement, move building, 1 day										
	0300	move, patch & hook-up, based on ground floor area	B-3	155	.310	S.F.	5.30	6.35	10.20	21.85	27	
		021 400 | Dewatering										
404	0010	DEWATERING Excavate drainage trench, 2' wide, 2' deep R021-440	B-11C	90	.178	C.Y.		3.85	2.22	6.07	8.45	404
	0100	2' wide, 3' deep, with backhoe loader	"	135	.119			2.57	1.48	4.05	5.55	
	0200	Excavate sump pits by hand, light soil	1 Clab	7.10	1.127			21.50		21.50	34	
	0300	Heavy soil	"	3.50	2.286	↓		43.50		43.50	69	
	0500	Pumping 8 hr., attended 2 hrs. per day, including 20 L.F.										
	0550	of suction hose & 100 L.F. discharge hose										
	0600	2" diaphragm pump used for 8 hours	B-10H	4	3	Day		67.50	7.80	75.30	114	
	0620	Add per additional pump							30	26	33	
	0650	4" diaphragm pump used for 8 hours	B-10I	4	3			67.50	19.90	87.40	127	
	0670	Add per additional pump							63	68	69	
	0800	8 hrs. attended, 2" diaphragm pump	B-10H	1	12			271	31	302	455	
	0820	Add per additional pump							30	26	33	
	0900	3" centrifugal pump	B-10J	1	12			271	47	318	470	
	0920	Add per additional pump							39	43	43	
	1000	4" diaphragm pump	B-10I	1	12			271	79.50	350.50	510	
	1020	Add per additional pump							63	68	69	
	1100	6" centrifugal pump	B-10K	1	12			271	206	477	645	
	1120	Add per additional pump				↓			99	190	110	
	1300	Re-lay CMP, incl. excavation 3' deep, 12" diameter	B-6	115	.209	L.F.	5.65	4.27	1.74	11.66	14.80	
	1400	18" diameter	↓	100	.240	"	8.25	4.91	2	15.16	18.95	

022 200 | Excav./Backfill/Compact.

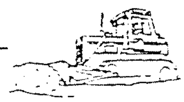
			CREW	DAILY OUTPUT	MAN-HOURS	UNIT	1994 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
226	6020	3 passes	B-10D	2,000	.006	C.Y.		.14	.47	.61	.72	226
	6030	4 passes		1,500	.008			.18	.62	.80	.95	
	6050	12" lifts, 2 passes		6,000	.002			.05	.16	.21	.24	
	6060	3 passes		4,000	.003			.07	.23	.30	.36	
	6070	4 passes	↓	3,000	.004			.09	.31	.40	.48	
	6200	Vibrating roller, 6" lifts, 2 passes	B-10C	2,600	.005			.10	.35	.45	.53	
	6210	3 passes		1,735	.007			.16	.53	.69	.82	
	6220	4 passes		1,300	.009			.21	.71	.92	1.10	
	6250	12" lifts, 2 passes		5,200	.002			.05	.18	.23	.27	
	6260	3 passes		3,465	.003			.08	.27	.35	.41	
	6270	4 passes	↓	2,600	.005			.10	.35	.45	.55	
	7000	Walk behind, vibrating plate 18" wide, 6" lifts, 2 passes	A-1	280	.029			.54	.21	.75	1.09	
	7020	3 passes		185	.043			.82	.32	1.14	1.55	
	7040	4 passes		140	.057			1.09	.42	1.51	2.13	
	7200	12" lifts, 2 passes		560	.014			.27	.10	.37	.54	
	7220	3 passes		375	.021			.41	.16	.57	.81	
	7240	4 passes	↓	280	.029			.54	.21	.75	1.09	
	7500	Vibrating roller 24" wide, 6" lifts, 2 passes	B-10A	420	.029			.64	.19	.83	1.21	
	7520	3 passes		280	.043			.97	.29	1.26	1.52	
	7540	4 passes		210	.057			1.29	.39	1.68	2.43	
	7600	12" lifts, 2 passes		840	.014			.32	.10	.42	.61	
	7620	3 passes		560	.021			.48	.15	.63	.91	
	7640	4 passes	↓	420	.029			.64	.19	.83	1.21	
	8000	Rammer tamper, 6" to 11", 4" lifts, 2 passes	A-1	130	.062			1.17	.45	1.62	2.34	
	8050	3 passes		97	.082			1.57	.60	2.17	3.14	
	8100	4 passes		65	.123			2.34	.90	3.24	4.69	
	8200	8" lifts, 2 passes		260	.031			.58	.22	.80	1.18	
	8250	3 passes		195	.041			.78	.30	1.08	1.56	
	8300	4 passes		130	.062			1.17	.45	1.62	2.34	
	8400	13" to 18", 4" lifts, 2 passes		390	.021			.39	.15	.54	.78	
	8450	3 passes		290	.028			.52	.20	.72	1.05	
	8500	4 passes		195	.041			.78	.30	1.08	1.56	
	8600	8" lifts, 2 passes		780	.010			.19	.07	.26	.39	
	8650	3 passes		585	.014			.26	.10	.36	.52	
	8700	4 passes	↓	390	.021	↓		.39	.15	.54	.78	
230	0010	DRILLING ONLY 2" hole for rock bolts, average	B-47	395	.061	L.F.		1.28	1.38	2.66	3.53	230
	0300	2-1/2" hole for pre-splitting, average		540	.044			.94	1.01	1.95	2.53	
	1600	Quarry operations, 2-1/2" to 3-1/2" diameter	↓	715	.034			.71	.76	1.47	1.95	
	1610	6" diameter drill holes	B-47A	1,350	.018	↓		.40	.32	.72	.97	
234	0010	DRILLING AND BLASTING Only, rock, open face, under 1500 C.Y.	B-47	225	.107	C.Y.	1.35	2.25	2.42	6.02	7.70	234
	0100	Over 1500 C.Y.		300	.080		1.35	1.69	1.81	4.85	6.15	
	0300	Bulk drilling and blasting, can vary greatly, average									3.65	
	0500	Pits, average									18.75	
	1300	Deep hole method, up to 1500 C.Y.	B-47	50	.480		1.35	10.15	10.90	22.40	29.50	
	1400	Over 1500 C.Y.		66	.364		1.35	7.70	8.25	17.30	22.50	
	1900	Restricted areas, up to 1500 C.Y.		13	1.846		1.35	39	42	82.35	108	
	2000	Over 1500 C.Y.		20	1.200		1.35	25.50	27	53.85	71	
	2200	Trenches, up to 1500 C.Y.		22	1.091		1.35	23	24.50	48.85	64.50	
	2300	Over 1500 C.Y.		26	.923		1.35	19.50	21	41.85	55	
	2500	Pier holes, up to 1500 C.Y.		22	1.091		1.35	23	24.50	48.85	64.50	
	2600	Over 1500 C.Y.	↓	31	.774		1.35	16.35	17.55	35.25	46.50	
	2800	Boulders under 1/2 C.Y., loaded on truck, no hauling	B-100	80	.150			3.39	5.90	9.29	11.75	
	2900	Drilled, blasted and loaded on truck, no hauling	B-47	30	.800	↓	1.35	16.90	18.15	36.40	48	
	3100	Jackhammer operators with foreman compressor, air tools	B-9	1	40	Day		775	148	923	1,375	
	3300	Track drill, compressor, operator and foreman	B-47	1	24			505	545	1,050	1,400	



SITE WORK 2

022 200 | Excav./Backfill/Compact.

			CREW	DAILY OUTPUT	MAN-HOURS	UNIT	1994 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
238	4400	Clamshell in sheeting or cofferdam, minimum	B-12H	160	.100	C.Y.		2.31	3.33	5.64	7.20	238
	4450	Maximum	"	60	.267	↓		6.15	8.90	15.05	19.25	
	8000	For hauling excavated material, see div. 022-266										
242	0010	**EXCAVATING, BULK, DOZER** Open site R022-240										242
	2000	75 H.P., 50' haul, sand & gravel	B-10L	460	.026	C.Y.		.59	.59	1.18	1.55	
	2020	Common earth		400	.030			.68	.68	1.36	1.80	
	2040	Clay		250	.048			1.08	1.09	2.17	2.88	
	2200	150' haul, sand & gravel		230	.052			1.18	1.18	2.36	3.12	
	2220	Common earth		200	.060			1.36	1.36	2.71	3.59	
	2240	Clay		125	.096			2.17	2.17	4.34	5.76	
	2400	300' haul, sand & gravel		120	.100			2.26	2.26	4.52	6	
	2420	Common earth		100	.120			2.71	2.72	5.43	7.20	
	2440	Clay	↓	65	.185			4.17	4.18	8.35	11.05	
	3000	105 H.P., 50' haul, sand & gravel	B-10W	700	.017			.39	.58	.97	1.23	
	3020	Common earth		610	.020			.44	.66	1.10	1.43	
	3040	Clay		385	.031			.70	1.05	1.75	2.24	
	3200	150' haul, sand & gravel		310	.039			.87	1.30	2.17	2.78	
	3220	Common earth		270	.044			1	1.49	2.49	3.19	
	3240	Clay		170	.071			1.59	2.37	3.96	5.10	
	3300	300' haul, sand & gravel		140	.086			1.93	2.88	4.81	6.15	
	3320	Common earth		120	.100			2.26	3.36	5.62	7.20	
	3340	Clay	↓	100	.120			2.71	4.03	6.74	8.65	
	4000	200 H.P., 50' haul, sand & gravel	B-10B	1,400	.009			.19	.59	.78	.94	
	4020	Common earth		1,230	.010			.22	.67	.89	1.07	
	4040	Clay		770	.016			.35	1.06	1.41	1.71	
	4200	150' haul, sand & gravel		595	.020			.46	1.38	1.84	2.22	
	4220	Common earth		515	.023			.52	1.59	2.11	2.55	
	4240	Clay		325	.037			.83	2.52	3.35	4.06	
	4400	300' haul, sand & gravel		310	.039			.87	2.64	3.51	4.26	
	4420	Common earth		270	.044			1	3.04	4.04	4.89	
	4440	Clay	↓	170	.071			1.59	4.82	6.41	7.75	
	5000	300 H.P., 50' haul, sand & gravel	B-10M	1,900	.006			.14	.52	.66	.80	
	5020	Common earth		1,650	.007			.16	.60	.76	.91	
	5040	Clay		1,025	.012			.26	.97	1.23	1.48	
	5200	150' haul, sand & gravel		920	.013			.29	1.08	1.37	1.66	
	5220	Common earth		800	.015			.34	1.24	1.58	1.89	
	5240	Clay		500	.024			.54	1.99	2.53	3.03	
	5400	300' haul, sand & gravel		470	.026			.58	2.12	2.70	3.22	
	5420	Common earth		410	.029			.66	2.43	3.09	3.69	
	5440	Clay	↓	250	.048			1.08	3.98	5.06	6.06	
	5500	460 H.P., 50' haul, sand & gravel	B-10X	1,930	.006			.14	.65	.79	.94	
	5510	Common earth		1,680	.007			.16	.75	.91	1.07	
	5520	Clay		1,050	.011			.26	1.20	1.46	1.72	
	5530	150' haul, sand & gravel		1,290	.009			.21	.97	1.18	1.39	
	5540	Common earth		1,120	.011			.24	1.12	1.36	1.60	
	5550	Clay		700	.017			.39	1.80	2.19	2.58	
	5560	300' haul, sand & gravel		660	.018			.41	1.90	2.31	2.74	
	5570	Common earth		575	.021			.47	2.19	2.66	3.13	
	5580	Clay	↓	350	.034			.77	3.59	4.36	5.15	
	6000	700 H.P., 50' haul, sand & gravel	B-10V	3,500	.003			.08	.79	.87	.99	
	6010	Common earth		3,035	.004			.09	.91	1	1.14	
	6020	Clay		1,925	.006			.14	1.43	1.57	1.80	
	6030	150' haul, sand & gravel		2,025	.006			.13	1.36	1.49	1.71	
	6040	Common earth		1,750	.007			.15	1.58	1.73	1.98	
	6050	Clay	↓	1,100	.011	↓		.25	2.51	2.76	3.14	



		022 100 | Grading	CREW	DAILY OUTPUT	MAN-HOURS	UNIT	1994 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
104	0010	GRADING Site excav. & fill, see div 022-200										104
	0020	Fine grading, see div 025-122										
		022 200 | Excav./Backfill/Compact.										
204	0010	BACKFILL By hand, no compaction, light soil R022-220	1 Clab	14	.571	C.Y.		10.35		10.35	17.20	204
	0100	Heavy soil		11	.727			13.80		13.80	22	
	0300	Compaction in 6" layers, hand tamp, add to above		20.60	.388			7.40		7.40	11.70	
	0400	Roller compaction operator walking, add	B-10A	100	.120			2.71	.82	3.53	5.10	
	0500	Air tamp, add	B-9	190	.211			4.08	.78	4.36	7.30	
	0600	Vibrating plate, add	A-1	60	.133			2.53	.97	3.50	5.10	
	0800	Compaction in 12" layers, hand tamp, add to above	1 Clab	34	.235			4.47		4.47	7.10	
	0900	Roller compaction operator walking, add	B-10A	150	.080			1.81	.54	2.35	3.39	
	1000	Air tamp, add	B-9	285	.140			2.72	.52	3.24	4.38	
	1100	Vibrating plate, add	A-1	90	.089			1.69	.65	2.34	3.39	
208	0010	BACKFILL, STRUCTURAL Dozer or F.E. loader										208
	0020	From existing stockpile, no compaction										
	2000	75 H.P., 50' haul, sand & gravel	B-10L	1,100	.011	C.Y.		.25	.25	.50	.65	
	2020	Common earth		975	.012			.28	.28	.56	.74	
	2040	Clay		850	.014			.32	.32	.64	.84	
	2200	150' haul, sand & gravel		550	.022			.49	.49	.98	1.30	
	2220	Common earth		490	.024			.55	.55	1.10	1.47	
	2240	Clay		425	.028			.64	.64	1.28	1.69	
	2400	300' haul, sand & gravel		370	.032			.73	.73	1.46	1.94	
	2420	Common earth		330	.036			.82	.82	1.64	2.18	
	2440	Clay		290	.041			.93	.94	1.87	2.48	
	3000	105 H.P., 50' haul, sand & gravel	B-10W	1,350	.009			.20	.30	.50	.64	
	3020	Common earth		1,225	.010			.22	.33	.55	.70	
	3040	Clay		1,100	.011			.25	.37	.62	.78	
	3200	150' haul, sand & gravel		670	.018			.40	.60	1	1.29	
	3220	Common earth		610	.020			.44	.66	1.10	1.42	
	3240	Clay		550	.022			.49	.73	1.22	1.57	
	3300	300' haul, sand & gravel		465	.026			.58	.87	1.45	1.85	
	3320	Common earth		415	.029			.65	.97	1.62	2.08	
	3340	Clay		370	.032			.73	1.09	1.82	2.33	
	4000	200 H.P., 50' haul, sand & gravel	B-10B	2,500	.005			.11	.33	.44	.53	
	4020	Common earth		2,200	.005			.12	.37	.49	.60	
	4040	Clay		1,950	.006			.14	.42	.56	.67	
	4200	150' haul, sand & gravel		1,225	.010			.22	.67	.89	1.08	
	4220	Common earth		1,100	.011			.25	.75	1	1.20	
	4240	Clay		975	.012			.28	.84	1.12	1.35	
	4400	300' haul, sand & gravel		805	.015			.34	1.02	1.36	1.64	
	4420	Common earth		735	.016			.37	1.12	1.49	1.80	
	4440	Clay		660	.018			.41	1.24	1.65	2.01	
	5000	300 H.P., 50' haul, sand & gravel	B-10M	3,170	.004			.09	.31	.40	.48	
	5020	Common earth		2,900	.004			.09	.34	.43	.52	
	5040	Clay		2,700	.004			.10	.37	.47	.57	
	5200	150' haul, sand & gravel		2,200	.005			.12	.45	.57	.69	
	5220	Common earth		1,950	.006			.14	.51	.65	.77	
	5240	Clay		1,700	.007			.16	.59	.75	.89	
	5400	300' haul, sand & gravel		1,500	.008			.18	.66	.84	1.01	
	5420	Common earth		1,350	.009			.20	.74	.94	1.12	
	5440	Clay		1,225	.010			.22	.81	1.03	1.23	
	6000	For compaction, see div. 022-226										
	6010	For trench backfill, see div. 022-254 & 258										
216	0011	BORROW Bank measure, loaded onto 12 C.Y. hauler, no haul incl.										216
	4000	Common earth, shovel, 1 C.Y. bucket	B-12N	840	.019	C.Y.	3.58	.44	.72	4.74	5.40	

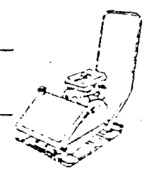
022 100 | Grading

	Line	Description	Crew	Daily Output	Man-Hours	Unit	1994 Bare Costs Mat.	Labor	Equip.	Total	Total Incl O&P	
104	0010	GRADING Site excav. & fill, see div 022-200										104
	0020	Fine grading, see div 025-122										

022 200 | Excav./Backfill/Compact.

	Line	Description	Crew	Daily Output	Man-Hours	Unit	Mat.	Labor	Equip.	Total	Total Incl O&P	
204	0010	BACKFILL By hand, no compaction, light soil R022-220	1 Clab	14	.571	C.Y.		10.35		10.35	17.20	204
	0100	Heavy soil		11	.727			13.80		13.80	22	
	0300	Compaction in 6" layers, hand tamp, add to above		20.60	.388			7.40		7.40	11.70	
	0400	Roller compaction operator walking, add	B-10A	100	.120			2.71	.82	3.53	5.10	
	0500	Air tamp, add	B-9	190	.211			4.08	.78	4.86	7.30	
	0600	Vibrating plate, add	A-1	60	.133			2.53	.97	3.50	5.10	
	0800	Compaction in 12" layers, hand tamp, add to above	1 Clab	34	.235			4.47		4.47	7.10	
	0900	Roller compaction operator walking, add	B-10A	150	.080			1.81	.54	2.35	3.39	
	1000	Air tamp, add	B-9	285	.140			2.72	.52	3.24	4.88	
	1100	Vibrating plate, add	A-1	90	.089			1.69	.65	2.34	3.39	
208	0010	BACKFILL, STRUCTURAL Dozer or F.E. loader										208
	0020	From existing stockpile, no compaction										
	2000	75 H.P., 50' haul, sand & gravel	B-10L	1,100	.011	C.Y.		.25	.25	.50	.65	
	2020	Common earth		975	.012			.28	.28	.56	.74	
	2040	Clay		850	.014			.32	.32	.64	.84	
	2200	150' haul, sand & gravel		550	.022			.49	.49	.98	1.30	
	2220	Common earth		490	.024			.55	.55	1.10	1.47	
	2240	Clay		425	.028			.64	.64	1.28	1.69	
	2400	300' haul, sand & gravel		370	.032			.73	.73	1.46	1.94	
	2420	Common earth		330	.036			.82	.82	1.64	2.18	
	2440	Clay		290	.041			.93	.94	1.87	2.48	
	3000	105 H.P., 50' haul, sand & gravel	B-10W	1,350	.009			.20	.30	.50	.64	
	3020	Common earth		1,225	.010			.22	.33	.55	.70	
	3040	Clay		1,100	.011			.25	.37	.62	.78	
	3200	150' haul, sand & gravel		670	.018			.40	.60	1	1.29	
	3220	Common earth		610	.020			.44	.66	1.10	1.42	
	3240	Clay		550	.022			.49	.73	1.22	1.57	
	3300	300' haul, sand & gravel		465	.026			.58	.87	1.45	1.85	
	3320	Common earth		415	.029			.65	.97	1.62	2.08	
	3340	Clay		370	.032			.73	1.09	1.82	2.33	
	4000	200 H.P., 50' haul, sand & gravel	B-10B	2,500	.005			.11	.33	.44	.53	
	4020	Common earth		2,200	.005			.12	.37	.49	.60	
	4040	Clay		1,950	.006			.14	.42	.56	.67	
	4200	150' haul, sand & gravel		1,225	.010			.22	.67	.89	1.08	
	4220	Common earth		1,100	.011			.25	.75	1	1.20	
	4240	Clay		975	.012			.28	.84	1.12	1.35	
	4400	300' haul, sand & gravel		805	.015			.34	1.01	1.35	1.64	
	4420	Common earth		735	.016			.37	1.12	1.49	1.80	
	4440	Clay		660	.018			.41	1.24	1.65	2.01	
	5000	300 H.P., 50' haul, sand & gravel	B-10M	3,170	.004			.09	.31	.40	.48	
	5020	Common earth		2,900	.004			.09	.34	.43	.52	
	5040	Clay		2,700	.004			.10	.37	.47	.57	
	5200	150' haul, sand & gravel		2,200	.005			.12	.45	.57	.69	
	5220	Common earth		1,950	.006			.14	.51	.65	.77	
	5240	Clay		1,700	.007			.16	.59	.75	.89	
	5400	300' haul, sand & gravel		1,500	.008			.18	.66	.84	1.01	
	5420	Common earth		1,350	.009			.20	.74	.94	1.12	
	5440	Clay		1,225	.010			.22	.81	1.03	1.23	
	6000	For compaction, see div. 022-226										
	6010	For trench backfill, see div. 022-254 & 258										
216	0011	BORROW Bank measure, loaded onto 12 C.Y. hauler, no haul incl.										216
	4000	Common earth, shovel, 1 C.Y. bucket	B-12N	840	.019	C.Y.	3.58	.44	.72	4.74	5.40	

	022 200 | Excav./Backfill/Compact.		CREW	DAILY OUTPUT	MAN-HOURS	UNIT	1994 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
226	6020	3 passes	B-10D	2,000	.006	C.Y.		.14	.47	.61	.72	226
	6030	4 passes		1,600	.008			.18	.62	.80	.96	
	6050	12" lifts, 2 passes		6,000	.002			.05	.16	.21	.24	
	6060	3 passes		4,000	.003			.07	.23	.30	.36	
	6070	4 passes		3,000	.004			.09	.31	.40	.48	
	6200	Vibrating roller, 6" lifts, 2 passes	B-10C	2,600	.005			.10	.35	.45	.55	
	6210	3 passes		1,735	.007			.16	.53	.69	.82	
	6220	4 passes		1,300	.009			.21	.71	.92	1.10	
	6250	12" lifts, 2 passes		5,200	.002			.05	.18	.23	.27	
	6260	3 passes		3,465	.003			.08	.27	.35	.41	
	6270	4 passes		2,600	.005			.10	.35	.45	.55	
	7000	Walk behind, vibrating plate 18" wide, 6" lifts, 2 passes	A-1	280	.029			.54	.21	.75	1.09	
	7020	3 passes		185	.043			.82	.32	1.14	1.65	
	7040	4 passes		140	.057			1.09	.42	1.51	2.18	
	7100	12" lifts, 2 passes		560	.014			.27	.10	.37	.54	
	7120	3 passes		375	.021			.41	.16	.57	.81	
	7140	4 passes		280	.029			.54	.21	.75	1.09	
	7500	Vibrating roller 24" wide, 6" lifts, 2 passes	B-10A	420	.029			.64	.19	.83	1.21	
	7520	3 passes		280	.043			.97	.29	1.26	1.82	
	7540	4 passes		210	.057			1.29	.39	1.68	2.43	
	7600	12" lifts, 2 passes		840	.014			.32	.10	.42	.61	
	7620	3 passes		560	.021			.48	.15	.63	.91	
	7640	4 passes		420	.029			.64	.19	.83	1.21	
	8000	Rammer tamper, 6"to 11", 4" lifts, 2 passes	A-1	130	.062			1.17	.45	1.62	2.34	
	8050	3 passes		97	.082			1.57	.60	2.17	3.14	
	8100	4 passes		65	.123			2.34	.90	3.24	4.69	
	8200	8" lifts, 2 passes		260	.031			.58	.22	.80	1.18	
	8250	3 passes		195	.041			.78	.30	1.08	1.56	
	8300	4 passes		130	.062			1.17	.45	1.62	2.34	
	8400	13"to 18", 4" lifts, 2 passes		390	.021			.39	.15	.54	.78	
	8450	3 passes		290	.028			.52	.20	.72	1.05	
	8500	4 passes		195	.041			.78	.30	1.08	1.56	
	8600	8" lifts, 2 passes		780	.010			.19	.07	.26	.39	
	8650	3 passes		585	.014			.26	.10	.36	.52	
	8700	4 passes		390	.021			.39	.15	.54	.78	
230	0010	DRILLING ONLY 2" hole for rock bolts, average	B-47	395	.061	L.F.		1.28	1.38	2.66	3.53	230
	0800	2-1/2" hole for pre-splitting, average		540	.044			.94	1.01	1.95	2.53	
	1600	Quarry operations, 2-1/2" to 3-1/2" diameter		715	.034			.71	.76	1.47	1.95	
	1610	6" diameter drill holes	B-47A	1,350	.018			.40	.32	.72	.97	
234	0010	DRILLING AND BLASTING Only, rock, open face, under 1500 C.Y.	B-47	225	.107	C.Y.	1.35	2.25	2.42	6.02	7.70	234
	0100	Over 1500 C.Y.		300	.080		1.35	1.69	1.81	4.85	6.15	
	0300	Bulk drilling and blasting, can vary greatly, average									3.65	
	0500	Pits, average									18.75	
	1300	Deep hole method, up to 1500 C.Y.	B-47	50	.430		1.35	10.15	10.90	22.40	29.50	
	1400	Over 1500 C.Y.		66	.364		1.35	7.70	8.25	17.30	22.50	
	1900	Restricted areas, up to 1500 C.Y.		13	1.846		1.35	39	42	82.35	108	
	2000	Over 1500 C.Y.		20	1.200		1.35	25.50	27	53.85	71	
	2200	Trenches, up to 1500 C.Y.		22	1.091		1.35	23	24.50	48.85	64.50	
	2300	Over 1500 C.Y.		26	.923		1.35	19.50	21	41.85	55	
	2500	Pier holes, up to 1500 C.Y.		22	1.091		1.35	23	24.50	48.85	64.50	
	2600	Over 1500 C.Y.		31	.774		1.35	16.35	17.55	35.25	46.50	
	2800	Boulders under 1/2 C.Y., loaded on truck, no hauling	B-100	80	.150			3.39	5.90	9.29	11.75	
	2900	Drilled, blasted and loaded on truck, no hauling	B-47	30	.800		1.35	16.90	18.15	36.40	48	
	3100	Jackhammer operators with foreman compressor, air tools	B-9	1	40	Day		775	148	923	1,375	
	3300	Track drill, compressor, operator and foreman	B-47	1	24			505	545	1,050	1,400	



SITE WORK 2

PRICE SHEET

ROSCOE STEEL & CULVERT COMPANY

"SPEEDY"

BILLINGS PLANT
2847 HESPER ROAD
BILLINGS, MONTANA 59102-6735
TELEPHONE (406) 656-2253
FAX (406) 656-8576

MISSOULA PLANT
5405 MOMONT ROAD
MISSOULA, MONTANA 59802
TELEPHONE (406) 542-0345
FAX (406) 542-1941

EFFECTIVE: 3-1-94
SUPERSEDES: 3-1-91

5" X 1" CORRUGATED STEEL PIPE
OR
3" X 1" CORRUGATED STEEL PIPE

CULVERT PRICES ------------------F.O.B BILLINGS OR MISSOULA, MONTANA

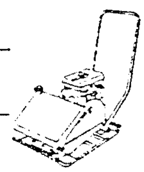
ARCH SIZE	PIPE DIA	PRICE/FT 14GA	WT. 14GA	PRICE/FT 12GA	WT. 12GA	PRICE/FT 10 GA	WT 10GA
40X31	36"	$40.75	41	$52.25	56	$62.89	71
46X36	42"	$46.69	47	$60.66	65	$73.53	83
53X41	48"	$53.68	54	$69.06	74	$84.16	95
60X46	54"	$60.64	61	$77.45	83	$93.90	106
66X51	60"	$66.61	67	$85.86	92	$104.53	118
73X55	66"	$73.86	74	$93.94	101	$114.13	129
81X59	72"	$80.51	81	$102.65	110	$124.01	140
87X63	78"	$86.83	87	$110.67	119	$134.47	152
95X67	84"	$93.43	94	$119.44	128	$145.26	164
103X71	90"	$99.81	100	$127.42	137	$154.82	175
112X75	96"	$106.37	107	$137.20	147	$166.53	188
117X79	102"	$113.78	114	$144.16	155	$175.17	198
128X83	108"	$125.24	120	$153.46	165	$186.54	211
137X87	114"	$126.76	127	$161.83	174	$196.40	222
142X91	120"	$145.13	146	$194.53	208	$207.30	234

BANDS 12" WIDE - SAME PRICE AS 1.5 FT. PIPE
20" WIDE - SAME PRICE AS 3.0 FT. PIPE

WE CARRY MOST OF ABOVE SIZES OF 5" X 1" IN STOCK FOR IMMEDIATE DELIVERY.
ALL 5" X 1" IS HELICAL PIPE. 3" X 1" CAN BE HELICAL OR ANNULAR.

PRICES SUBJECT TO CHANGE WITHOUT NOTICE.

022 200 | Excav./Backfill/Compact.



			CREW	DAILY OUTPUT	MAN-HOURS	UNIT	1994 BARE COSTS MAT.	LABOR	EQUIP.	TOTAL	TOTAL INCL O&P	
226	6020	3 passes	B-10D	2,000	.006	C.Y.		.14	.47	.61	.72	226
	6030	4 passes		1,500	.008			.18	.62	.80	.96	
	6050	12" lifts, 2 passes		6,000	.002			.05	.16	.21	.24	
	6060	3 passes		4,000	.003			.07	.23	.30	.36	
	6070	4 passes	↓	3,000	.004			.09	.31	.40	.48	
	6200	Vibrating roller, 6" lifts, 2 passes	B-10C	2,600	.005			.10	.35	.45	.55	
	6210	3 passes		1,735	.007			.16	.53	.69	.82	
	6220	4 passes		1,300	.009			.21	.71	.92	1.10	
	6250	12" lifts, 2 passes		5,200	.002			.05	.18	.23	.27	
	6260	3 passes		3,465	.003			.08	.27	.35	.41	
	6270	4 passes	↓	2,600	.005			.10	.35	.45	.55	
	7000	Walk behind, vibrating plate 18" wide, 6" lifts, 2 passes	A-1	280	.029			.54	.21	.75	1.09	
	7020	3 passes		185	.043			.82	.32	1.14	1.65	
	7040	4 passes		140	.057			1.09	.42	1.51	2.18	
	7200	12" lifts, 2 passes		560	.014			.27	.10	.37	.54	
	7220	3 passes		375	.021			.41	.16	.57	.81	
	7240	4 passes	↓	280	.029			.54	.21	.75	1.09	
	7500	Vibrating roller 24" wide, 6" lifts, 2 passes	B-10A	420	.029			.64	.19	.83	1.21	
	7520	3 passes		280	.043			.97	.29	1.26	1.82	
	7540	4 passes		210	.057			1.29	.39	1.68	2.43	
	7600	12" lifts, 2 passes		840	.014			.32	.10	.42	.61	
	7620	3 passes		560	.021			.48	.15	.63	.91	
	7640	4 passes	↓	420	.029			.64	.19	.83	1.21	
	8000	Rammer tamper, 6" to 11", 4" lifts, 2 passes	A-1	130	.062			1.17	.45	1.62	2.34	
	8050	3 passes		97	.082			1.57	.60	2.17	3.14	
	8100	4 passes		65	.123			2.34	.90	3.24	4.69	
	8200	8" lifts, 2 passes		260	.031			.58	.22	.80	1.18	
	8250	3 passes		195	.041			.78	.30	1.08	1.56	
	8300	4 passes		130	.062			1.17	.45	1.62	2.34	
	8400	13" to 18", 4" lifts, 2 passes		390	.021			.39	.15	.54	.78	
	8450	3 passes		290	.028			.52	.20	.72	1.05	
	8500	4 passes		195	.041			.78	.30	1.08	1.56	
	8600	8" lifts, 2 passes		780	.010			.19	.07	.26	.39	
	8650	3 passes		585	.014			.26	.10	.36	.52	
	8700	4 passes	↓	390	.021	↓		.39	.15	.54	.78	
230	0010	DRILLING ONLY 2" hole for rock bolts, average	B-47	395	.061	L.F.		1.28	1.38	2.66	3.53	230
	0800	2-1/2" hole for pre-splitting, average		540	.044			.94	1.01	1.95	2.58	
	1600	Quarry operations, 2-1/2" to 3-1/2" diameter	↓	715	.034			.71	.76	1.47	1.95	
	1610	6" diameter drill holes	B-47A	1,350	.018	↓		.40	.32	.72	.97	
234	0010	DRILLING AND BLASTING Only, rock, open face, under 1500 C.Y.	B-47	225	.107	C.Y.	1.35	2.25	2.42	6.02	7.70	234
	0100	Over 1500 C.Y.		300	.080		1.35	1.69	1.81	4.85	6.15	
	0300	Bulk drilling and blasting, can vary greatly, average									3.65	
	0500	Pits, average									18.75	
	1300	Deep hole method, up to 1500 C.Y.	B-47	50	.480		1.35	10.15	10.90	22.40	29.50	
	1400	Over 1500 C.Y.		66	.364		1.35	7.70	8.25	17.30	22.50	
	1900	Restricted areas, up to 1500 C.Y.		13	1.846		1.35	39	42	82.35	108	
	2000	Over 1500 C.Y.		20	1.200		1.35	25.50	27	53.85	71	
	2200	Trenches, up to 1500 C.Y.		22	1.091		1.35	23	24.50	48.85	64.50	
	2300	Over 1500 C.Y.		26	.923		1.35	19.50	21	41.85	55	
	2500	Pier holes, up to 1500 C.Y.		22	1.091		1.35	23	24.50	48.85	64.50	
	2600	Over 1500 C.Y.	↓	31	.774		1.35	16.35	17.55	35.25	46.50	
	2800	Boulders under 1/2 C.Y., loaded on truck, no hauling	B-100	80	.150			3.39	5.90	9.29	11.75	
	2900	~~Drilled, blasted and loaded on truck, no hauling~~	B-47	30	.800	↓	1.35	16.90	18.15	36.40	~~48~~	
	3100	Jackhammer operators with foreman compressor, air tools	B-9	1	40	Day		775	148	923	1,375	
	3300	Track drill, compressor, operator and foreman	B-47	1	24			505	545	1,050	1,400	

SITE WORK 2

APPENDIX V

Performance Evaluation, Miner Flat Wellfield
White Mountain Apache Tribe

MORRISON-MAIERLE, INC.

FEBRUARY 2007

PERFORMANCE EVALUATION
MINER FLAT WELLFIELD
WHITE MOUNTAIN APACHE TRIBE

Fort Apache Indian Reservation
Whiteriver, Arizona

Prepared for:

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FEBRUARY 2002

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EXECUTIVE SUMMARY

The Miner Flat Wellfield is the primary source of public water supply for the greater Whiteriver area of the Fort Apache Indian Reservation, providing water to Whiteriver and the surrounding housing areas in communities along the North Fork and East Fork, Fort Apache and Canyon Day, as well as to a 15-mile pipeline serving the community of Cedar Creek. The wellfield consists of 10 wells completed in the Coconino Aquifer System at Miner Flat approximately nine miles north of Whiteriver on the west side of the North Fork of the White River along State Route 73.

The wellfield was put into operation in December 1996 with three wells. In 1998, another five wells were added, increasing the initial design pumping capacity of the wellfield to 2,975 gpm. An investigation of wellfield conditions conducted December 2001 found that wellfield production has declined significantly since the wells were put into production. With Wells No. 2 and 8 out of service in December 2001, the design yield of the remaining eight wells was 2,750 gpm; however, tests conducted December 1, 2001 determined that the eight wells only produced 1,591 gpm, less than 58 percent of their original design capacity. Operation of the original pumping equipment with the water levels dropping in the wells has resulted in damage to the pumps due to air entrainment and cavitation. Daily demand for water from the wellfield ranges from approximately 1.8 to 2.0 million gallons per day which is equivalent to a 24-hour flow rate of 1,250 to 1,389 gpm from the wellfield. Accordingly, the wells operate from 78 to 87 percent of each day with some wells operating 24 hours per day.

The December 2001 investigations found that the 42 percent decrease in the wellfield yield (not including the two out-of-service wells) was due to declining groundwater levels and reduction of the saturated thickness of the aquifer in the wellfield area during the four to six years of well pumping. The four to six year decline in groundwater levels ranged from as little as 29 feet around the margins of the wellfield to 77 feet in the central area of the wellfield with the rate of decline ranging from 9.55 to 30.64 ft/year.

A thorough review of the original aquifer tests conducted during construction of the wellfield, interpreted in the context of the aquifer response to four to six years of well operations, indicates that the groundwater resource in the vicinity of the Miner Flat Wellfield is being depleted by groundwater withdrawal rates that greatly exceed recharge to the aquifer system. This being the case, it is anticipated that groundwater levels in the vicinity of the wellfield will continue to decline in the future. It is likewise anticipated that the future decline of groundwater levels will result in further reduction to the amount of water the wellfield can produce. The ongoing loss of production from the wellfield, combined with continued expansion of housing and increased demand for water in the communities around White River, will result in a shortage of water supply in the foreseeable future.

Based on the foregoing considerations, it is recommended that the Tribal Council of the White Mountain Apache Tribe begin studies to identify alternatives to replace the shrinking supply of water provided by the Miner Flat Wellfield and to provide additional water supply for future growth. Alternatives should include short-term expansion of the existing wellfield as well as selecting appropriate new sources of water.

1. INTRODUCTION

The Miner Flat Wellfield is located on the west side of Arizona State Highway 73 approximately nine miles north of the town of Whiteriver, Arizona, on the Fort Apache Indian Reservation (Figure 1.1) in Sections 16 and 21 of Township 7 North, Range 23 East. The wellfield is on the west edge of the basalt terrace along the west side of the North Fork of the White River, immediately downstream from the confluence of Cottonwood Creek and the North Fork of the White River.

The wellfield currently consists of 10 production wells operated by the White Mountain Apache Tribe to provide public water supply to the town of Whiteriver and its outlying communities, including a pipeline to the community of Cedar Creek which is approximately 15 road miles west of Whiteriver. Major residential housing areas in the vicinity of Whiteriver which are served by the wellfield include the North Fork, East Fork, Canyon Day, and the historic Fort Apache site.

The wellfield service area constitutes the principal and largest population concentration on the Fort Apache Indian Reservation. The wellfield presently services 2,600 hookups and an estimated population of 13,000 people. Construction of additional residential housing in the Whiteriver area is either planned or under way. In addition, the wellfield can provide supplemental water to the Fort Apache Timber Company (FATCO) mill at Whiteriver which is one of the principal sources of employment and income on the Fort Apache Indian Reservation.

1.1. Exploration History

The wellfield produces groundwater from the Coconino Sandstone and an underlying sandstone that forms the uppermost part of the Supai Group of formations in this area. Preliminary exploration for a groundwater source was sponsored by the Indian Health Service (I.H.S.) of the Office of Environmental Health and Engineering, Public Health Service. Dr. Charles S. Robinson, a professional geologist and owner of Mineral Systems of Golden, Colorado, had mapped the geology of the nearby proposed Miner Flat damsite, east of and contiguous to the wellfield location, and provided preliminary geologic advice to the I.H.S. for selection of exploration well sites with the assistance of Golder Associates of Denver, Colorado. Their recommendations were summarized in a report titled, "**Hydrogeologic Investigation North-Central part Fort Apache Indian Reservation, Navajo, Apache and Gila Counties, Arizona**," (Mineral Systems and Golder Associates, 1993). Mr. Keith Shortall, I.H.S. District Engineer, subsequently supervised exploratory drilling at 10 exploration sites within the future wellfield area in the summer and fall of 1993 and the summer of 1994. The exploration boreholes were drilled in the vicinity of the present Miner Flat Wellfield and along Cottonwood Creek from Highway 73 to about 1.7 miles upstream from Highway 73 along Cottonwood Creek.

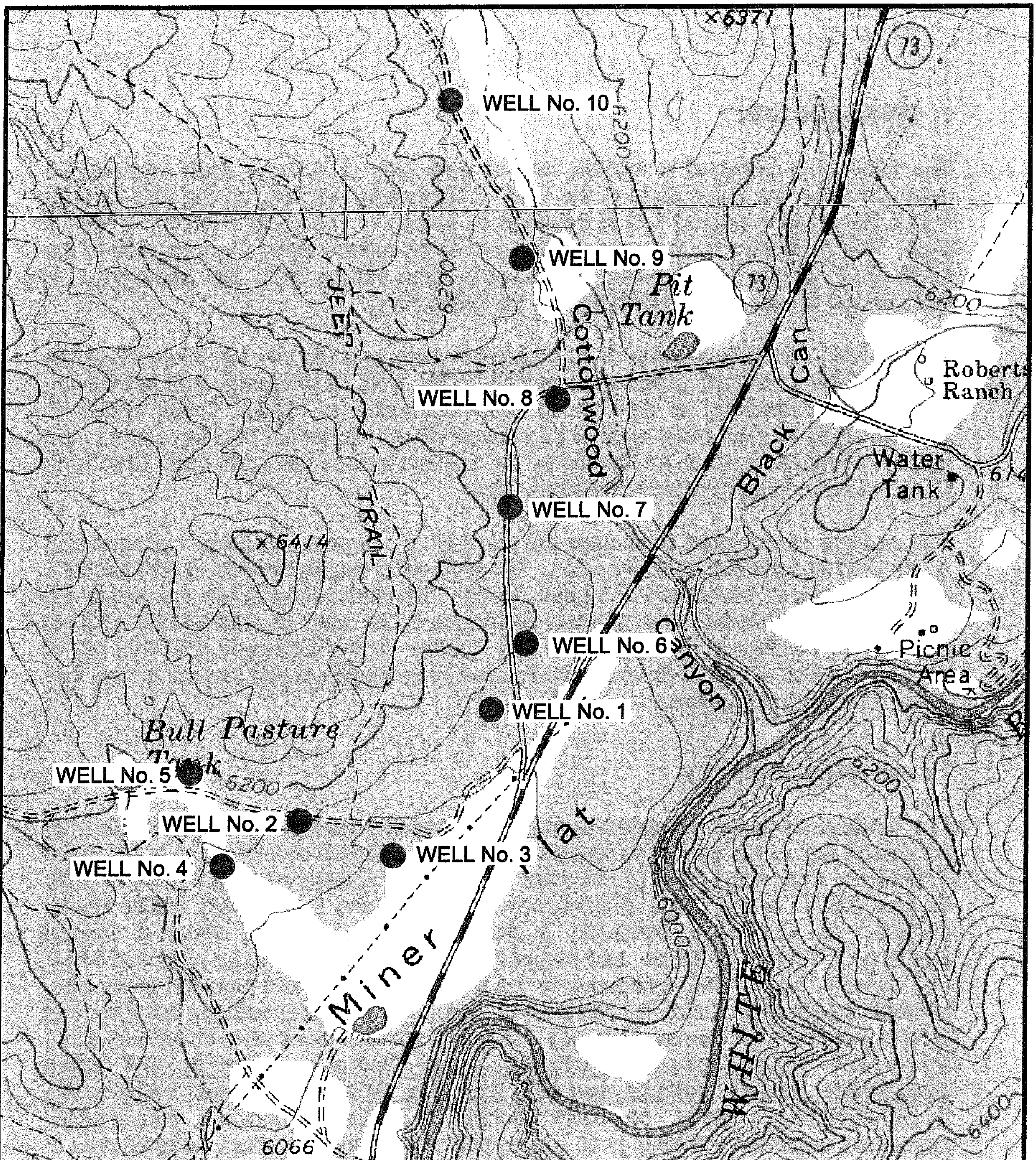

FIGURE 1.1: Well field vicinity map.

LEGEND

● WELL No. 5 - PRODUCTION WELL LOCATION

N

1000 0 1000

SCALE IN FEET

H:\WATER RESOURCES\1780 wmat\miner flat dam\MINER FLAT FIG 1.1.dwg Apr/09/2002

Some of the most productive exploration wells were located on the north and northwest edge of the Miner Flat. Accordingly, a test well was drilled in that location, a well which subsequently became production Well No. 3 of the Miner Flat Wellfield. On June 1, 1994, Well No. 3 was subjected to a 13.8-day constant rate test at an initial pumping rate of 325 gpm which decreased to 265 gpm as the load on the pump increased due to drawdown of water levels in the well. Although only eleven measurements of the pumping water level were collected during the test, the results indicated not only a favorable rate of drawdown for long-term use of the well, but indicated the well could produce more than the 265-325 gpm test rate used. A second test of 7-days duration was conducted on 9/16/94 at a starting rate of 500 gpm which decreased to 400 gpm over the seven-day period. These results led to a third test of Well No. 3 of 70 days duration, conducted at 400 gpm, in late 1994 or early 1995. Again, sparse data were collected during the test; however, the results were interpreted to indicate that the well performance was satisfactory for long-term sustainable yield. Plots of the test data from the 13.8-day and 70-day tests are shown on Figure 3.5 of this report.

The results of the tests of Well No. 3 justified detailed testing of the hydraulic properties of the aquifer materials in the Miner Flat area to obtain design criteria for a multiple-well source of groundwater to provide public water supply to the greater Whiteriver area. Golder Associates, of Denver Colorado, were hired for that purpose and issued a report in May 1994 titled, "**Pumping Test Analysis and Well Field Design, Miner Flat Area, Fort Apache Indian Reservation**" while working as a subcontractor to Mineral Systems, Inc. and Dr. Robinson. The results of Golder's tests and interpretations were favorable, and they concluded that the Coconino aquifer in the Miner Flat area, including the uppermost sandstone of the Supai Formation, would support a wellfield to provide as much as 4,000 gpm from eight to ten wells, spaced 500 feet apart.

A review of the history of exploration and testing of the wellfield area must make a distinction between hydraulic investigations and hydrologic investigations. The various exploration wells, test wells, and pumping tests conducted at Miner Flat before a decision was made to implement a wellfield in the area are not hydrologic investigations. Rather, the pumping tests determine the local hydraulic properties of the aquifer and the hydraulic properties of wells completed in the aquifer, all for the purpose of determining reasonable design yields for wells completed in the aquifer system. Therefore the tests are hydraulic tests.

The foregoing hydraulic tests presuppose that a supply of water is available to sustain abstraction of groundwater from the aquifer system. An aquifer system typically includes a recharge area where water enters the aquifer from precipitation and runoff and a discharge area where water leaves the aquifer through springs or interformational leakage into overlying strata. Water entering the aquifer in the recharge area flows to the discharge area. Wells such as the Miner Flat Wellfield are designed to intercept a portion of the flow between the recharge area and the discharge area.

As long as the amount of groundwater flow through the aquifer is large with respect to the amount of groundwater abstracted by wells, the wells are reliable. Drawdown of water levels caused by pumping recovers when pumping stops because part of the natural flow through the aquifer system replenishes the groundwater removed from storage around the pumped wells. If wells abstract water in amounts comprising a significant portion of the natural flow through the aquifer system, the local groundwater levels around the pumped wells will decline a certain amount and adjust to a new equilibrium between recharge, groundwater pumpage, and natural discharge out of the aquifer system. The amount of adjustment to the pumpage is dynamic, that is it fluctuates in response to variable pumping rates and variable recharge rates, but is governed by the hydraulic properties of the aquifer which are constant and which are measured by pumping tests.

Establishment of a new equilibrium in an aquifer in response to pumping of wells assumes the pumping does not exceed the long-term recharge that provides water to flow through the aquifer from the recharge area to the discharge area. If the amount of water pumped from wells abstracts groundwater at a rate that exceeds the rate of groundwater flow through the aquifer, and therefore exceeds the long-term available recharge, the pumping will cause the groundwater level around the pumped wells to decline continuously as groundwater is mined from the aquifer. Ultimately, the decline in groundwater levels will limit the ability of wells to abstract groundwater and well yields will decrease dramatically.

The exploration and initial testing of the Miner Flat Wellfield area was limited to hydraulic testing. Little or no hydrologic investigation of the volume of flow through the aquifer was conducted. The recharge areas and discharge areas remain unidentified and the annual groundwater flow through the aquifer strata at the wellfield remain unknown. The wellfield was constructed in response to a crucial demand to provide public water supply to the Whiteriver service area. In the face of hydraulic tests that indicated highly productive aquifer material, it was presumed that the flow of groundwater through these materials was commensurate with their hydraulic properties. Hydrologic investigations to determine regional groundwater gradients, directions of groundwater flow, sources of recharge, locations of natural discharge, and an estimate of flow volume through the aquifer were not accomplished. Now, after six years of operating three of the wells and four years of operation of the newer wells, it is increasingly evident that the natural groundwater flow through the aquifer strata is locally much smaller than the amount of water pumped from the Miner Flat Wellfield. Consequently, the wellfield produces water by virtue of depleting water stored in the strata at a rate that exceeds the flow of water back into the strata. This is causing the groundwater levels to decline with subsequent loss of well yield.

1.2. Wellfield Construction History

Based on the results of the I.H.S. 13.8-day and 70-day tests, and determination of the hydraulic parameters of the aquifer by Golder Associates, the I.H.S. and Dr. Robinson recommended to the Tribal Council of the White Mountain Apache Tribe that a wellfield should be constructed in the Miner Flat area to provide at least a short-term solution, if not a permanent solution, to satisfying part of the water requirements for the growing population in the area. One of the principal advantages of developing groundwater in the Miner Flat area was the minimal water treatment required for groundwater compared to the expensive and complicated treatment required for a surface water source, such as the North Fork of the White River. The Tribal Council, after considering the various aspects of using the groundwater source, passed resolutions authorizing construction of a pipeline and facilities for the Miner Flat Wellfield and authorizing drilling of the necessary wells.

Based on that authority, the I.H.S. proceeded with implementation of the project. At the end of 1996, three wells existed that were used as production wells for the next two years. These wells were Wells No. 1, 2, and 3. In 1997, the I.H.S. awarded a drilling contract for expansion of the wellfield. Drilling and testing of additional wells commenced in the summer of 1997. Morrison-Maierle was contracted to provide geologic logging during drilling of the wells, primarily to assist the I.H.S. District Engineer Keith Shortall in selecting the water-bearing zones to be screened, and to conduct the baseline tests of the wells. Wells 4 through10 were completed, not necessarily in numerical order, by the end of 1997 with the exception of Well No. 10 which was reconstructed in June and July of 1998. By January 1998, the pipeline connecting the wells to the Whiteriver water supply system had been completed and connected to the various wells. Accordingly, the wellfield in its current form was put into operation in January 1998, with the exception of Well No. 10 which was still being reconstructed.

Construction of Wells 4 through 10 of the Miner Flat Wellfield proved the initial conclusion that 4,000 gpm could be provided by eight to ten wells to be somewhat optimistic; however, the design yields of the 10 wells comprising the wellfield totaled 2,975 gpm with design yields of individual wells ranging from 200 to 350 gpm.

1.3. History of Wellfield Operation

All the wells in the wellfield were initially operated manually. This required the Tribal Utility Authority operator to be physically present to start and stop the wells as needed. A telemetry system was installed, using hardwired equipment; however, electrical interference from unidentified sources resulted in false signals which confounded automatic operation of the wellfield. The problem was solved by converting the telemetry controls to radio-control equipment over a period of time until automatic operation of the wellfield became a reality in the year 2001. A lightening strike in September 2001 damaged the telemetry system and the wells were again operated manually until December 2001 when most of the telemetry functions were restored.

Beginning in 1999, Dr. Laurel Lacher, Hydrologist for the White Mountain Apache Tribe, began collecting measurements of static water levels and pumping water levels in the wells at Miner Flat Wellfield. By the summer of 2001, the data were showing a continuous downward trend in water levels at the wellfield since the time the initial static water levels were collected in 1997. In addition, Frankie Williams, the Water System Operator, was reporting a number of problems at the wells including sand production at

Well No. 2, air in the water at Well No. 5, holes in pump columns at several wells, and a general perception that the wells were not producing the same amount of water that they originally produced. It was also observed that a number of pumps and pump columns removed from some of the wells for service or replacement exhibited thick incrustations of iron oxide, a condition that might indicate problems in the wells. In the fall of 2001, Dr. Lacher and I.H.S. Engineer Tom Moeller conducted a brief test of the yield of a number of wells in the wellfield and found them to be very sensitive to pumping water level changes as well as generally producing less water than the original design yields.

Based on the foregoing considerations, Tribal Engineer John Bereman, Tribal Hydrologist Laurel Lacher, and, I.H.S. Engineer Tom Moeller recommended that Morrison-Maierle, Inc. be contracted to perform investigations and tests of the wellfield to assess the nature and source of the various problems. This recommendation was approved by the Tribal Council and field investigations of the wells were conducted from 11/28/01 through 12/03/01. These investigations are referred to herein as the "December 2001" inspection.

1.4. December 2001 Investigations

The plan for conducting the December 2001 investigations included the following steps:

1. Replacement of damaged or missing standpipes in the production wells so that static and pumping water levels could be measured.

2. Installation of instruments to provide continuous records of water levels in the wells during tests proposed as part of the investigations.

3. Measurement of individual well yield into the existing system.

4. Stepped rate tests of well performance for comparison with comparable baseline tests conducted when the wells were first constructed.

5. Determination of electrical current draw at different operating rates for comparison to published values for pumps and motors.

The investigations were initiated on 11/29/01 with transportation of the standpipe materials from Phoenix to Whiteriver and initial installation of standpipes and data loggers. The following conditions were discovered that affected accomplishment of the investigations as originally planned:

1. It was discovered that Wells 1, 2, and 3 are equipped with older pitless unit spools which do not offer a cableway passage configuration that will accept a stand pipe or a data logger. It was therefore not possible to install either standpipes or data loggers in these wells.

2. Well No. 2 was out of service with the pump, motor and pump column lying on the ground beside the well. It was therefore not possible to step test Well No. 2 or determine its yield and current draw.

3. An attempt to install a standpipe into Well No. 5 was not successful. An alignment problem in the well casing and screen prevented the standpipe from advancing below 250 feet. The pumping water level in the well was below 250 feet so there was no merit to installing the standpipe. The difficulties encountered in lowering (an pushing) the standpipe to a depth of 250 feet indicated it would be very imprudent to attempt installation of an expensive data logger without a standpipe, so Well No. 5 was not instrumented with a data logger.

4. It was discovered that the gate valve between Well No. 6 and the main water transmission line could not be turned. It is necessary to use this valve to conduct a stepped rate test. Therefore, a stepped rate test of Well No. 6 was not attempted for fear that forcing the valve would cause unnecessary damage to the system.

5. The keys to unlock the control panel at Well No. 7 were never found during the investigations. Accordingly, a stepped rate test of Well No. 7 was not attempted for fear that regulating the discharge might cause the pump to stop due to an electrical overload. Without access to the control panel, it would be impossible to reset the electrical controls if the pump stopped due to an overload. Since the amount of water delivered by the wellfield was marginal compared to the system demands during the investigation, there was considerable concern that inadvertently dropping Well No. 7 off line without the capability to reset the controls and re-start the pump would result in a water shortage, so a stepped rate test of Well No. 7 was not attempted.

6. An obstruction was found in the existing standpipe in Well No. 7. A water level measuring instrument was dropped repeatedly on the obstruction and pushed it down the well to where the pumping water level could be observed. The obstruction was soft like tape or a plastic sack pushed into the standpipe.

7. The inspection found that the pump in Well No. 8 had severe mechanical damage which caused an overload and stopped the pump all but one time that it was operated. Accordingly, a stepped rate test was not conducted in Well No. 8.

8. When a stepped rate test was attempted in Well No. 9, a very slight throttling of the discharge rate at the gate valve to the main line resulted in an overload fault which stopped the pump. After numerous attempts to control the discharge rate with the valve, all of which stopped the pump on an overload, the stepped rate test attempt was abandoned.

9. It was not possible install either a standpipe or a data logger in Well No. 10. When standpipes were installed in Wells No. 4 through 10, it was found that the old-style pitless unit spool put a slight bend in the standpipe. The bend did not prevent the electronic water level measuring instrument or pressure transducers from passing through the standpipe. However, it was discovered that the new style of data logger rented for the inspection tests would not pass the slight bend in the standpipe. Accordingly, the data loggers were installed down the tested wells without a standpipe; however, the logger would not go below 134 to 139 feet in Well No. 10.

After discovery that the new type of data loggers will not pass through the slight bend that the pitless unit spool causes in the standpipes in Wells No. 4 through 10 and that the older type of pitless unit spools in Wells No. 1 through 3 do not provide enough room for either a logger or a standpipe, the Baker Monitor Division that manufactures the pitless units was contacted about this aspect of their spool design. They stated that the spool design was changed about mid-2000 to provide a 1-1/2 inch passage for standpipes without causing any bend or binding of the standpipes. Accordingly, if it is desirable at some time in the future to install dedicated groundwater level monitoring equipment on the wells, the existing pitless unit spools can be replaced with the new style of spools which do not interfere with the standpipe for the monitoring equipment.

Although it was not possible to conduct stepped rate tests of the wells, for the above reasons, considerable useful information was obtained during the wellfield investigations. Standpipes were replaced in the wells as necessary. The 12/01/01 well yields into the system were determined for all of the wells except Well No. 2 where the pump had been removed for service. The electrical current draw during 12/01/01 operating conditions was recorded from the control panel at the treatment building for each of the nine wells operating that day. Static water level and pumping water level information was collected from Wells No. 1, 4, 6, 7, 8, and 9. The condition of pump column pipes and pumps and motors removed from some of the wells was inspected.

The December 2001 investigations determined that the 12/01/01 production from all of the wells except Well No. 2 totaled 1,582 gpm and that although Well No. 8 did run for enough time to establish its pumping water level, it continually tripped out on a current overload during automatic operation of the wellfield. With the yield of Well No. 8 of 104 gpm, the absence of production from Wells No. 8 and No. 2 reduced the momentary wellfield yield during the December 2001 inspection to 1,442 gpm, or only 52 percent of the original design yield of 2,750 gpm. The 12/01/01 measurement of production from the wells indicated a decrease of 1,308 gpm from the design yield of the wellfield, with Wells No. 2 and 8 out of production.

1.5. Report Organization

Logical analysis of a problem involves the collection of information about the problem and a statement of the analytical methods applied to the collected information to arrive at a conclusion. The data collected and the analytical methods applied to the data are the foundation for the conclusions reached. Most reports are organized to provide the facts, discuss analytical principles, present the analysis, and provide the conclusions obtained from the foundation of facts and analysis. This report is organized slightly differently in that most of the conclusions are provided early in the report and the details of the analysis used to arrive at the conclusions are presented last. This organization is used to facilitate an easy grasp of the conditions at the Miner Flat Wellfield and why those conditions exist, without toiling through the extensive details of the highly technical analysis. The analysis is provided last for technical specialists to use in determining the basis for the conclusions presented in the early part of the report.

Therefore, Chapter 2 of this report summarizes the findings and conclusions that describe the condition of the Miner Flat Wellfield. Chapter 3 provides details and discussion of the analysis that supports the findings presented in Chapter 2.

2. WELLFIELD PERFORMANCE

Table 2.1 provides a summary of the yields of individual wells in the Miner Flat Wellfield. The yields shown include the 24-hour test pumping yields obtained during baseline tests of the wells immediately after their construction in 1997 compared to the yields measured on 12/01/01 after essentially four years of operation. The design yields selected by the I.H.S., based on reports of the baseline pumping test results provided by Morrison-Maierle, Inc., are also shown on Table 2.1. The design yields were the basis for sizes of the production pumps selected for permanent installation into the wells.

Table 2.1: Summary of Miner Flat Wellfield yield by well.

Well No.	Baseline Test Yield (gpm)	Initial Design Yield (gpm)	12/01/01 Observed Yield (gpm)	Decrease in Yield 1/1/98–12/01/01 (gpm)	Percent Design Yield Remaining (%)
1	<350>*	350	204	146	58
2	<225>*	225	---	---	---
3	400	350	220	130	63
4	150	225	88	137	39
5	200	225	140	85	62
6	350	350	162	188	46
7	375	350	188	162	54
8	300	350	104	246	30
9	400	350	338	12	97
10	190	200	147	53	74
Totals	**2,940**	**2,750****	**1,591**	**1,159****	**58**

* Well assumed to produce nominal design yield when new.
**Total does not include discharge from Well No. 2.

In a number of wells, the design yield selected was larger than the yield obtained by the baseline test, a fact that is discussed in detail in Chapter 3 of this report for each of the wells. The difference between the baseline test yields and the design yields is an apparent discrepancy in most cases because the design yields were a nominal value based on standardization of the pump sizes in the wellfield to two types of pumps. With a couple of exceptions that are discussed, the actual performance of the standardized pumps was within the design parameters of the wells.

Baseline tests of yields at Wells No. 1 and 2 are not reported in the records. The baseline test yield column on Table 2.1 assumes the wells initially produced the nominal design yields indicated for the wells in the I.H.S. records. The remaining columns on Table 2.1 omit the yields of Well No. 2 from the total flows at the bottom of each column because its yield on 12/01/01 was not determined. Accordingly, the collective decrease in wellfield yield from baseline conditions to 12/01/01 shown on Table 2.1 does not include the yield of Well No. 2. The collective decrease in wellfield yield is from baseline yield of 2,750 gpm, not including Well No. 2, to 1,591 gpm on 12/01/01, not including Well No. 2. This is a decrease in yield of 1,159 gpm or a loss of slightly more than 42 percent of the original capacity of the wellfield, not taking into consideration Well No. 2.

2.1. Pumping Operations

The measured decrease in well yields at the Miner Flat Wellfield verifies the concerns expressed by the Water System Operator, Frankie Williams, that the performance of the wellfield has been degrading. The loss of well yield has a profound effect on the operation of the wellfield.

The wells all pump to a storage tank at the water treatment plant for the wellfield. The storage tank is 42 feet in diameter and 54.3 feet high, providing an operational volume of 562,431 gallons, including fire flow storage, and a volume of 10,364.5 gallons per foot of storage. Demands for water downstream at Whiteriver are satisfied by release of water from the storage tank. The demands for water are triggered by storage levels in other storage tanks downstream in the water distribution system. Accordingly, the demands tend to be cyclic, requiring periods of maximum flow out of the storage tank at the wellfield interspersed with periods of no flow out of the tank during which time flow into the tank from the wellfield restores the water level in the tank.

2.1.1. Pump Control Set Points

Pumping at the individual wells in the Miner Flat Wellfield is triggered by the water levels in the storage tank at the wellfield. Table 2.2 shows the tank levels programmed into the telemetry controls to start and stop pumping at the wells.

Table 2.2: Storage tank levels programmed as set points to start and stop pumps at the wells in the Miner Flat Wellfield.

Well No.	Lower Set Point (feet)	Upper Set Point (feet)
1	52	55
2	44	54.5
3	51	54
4	49	53
5	47	52
6	52	55
7	50	54.5
8	48	54
9	46	53
10	45	52

Table 2.2 shows that Wells No. 1 and 6 are the first wells programmed to start pumping as the water level in the storage tank declines to 52 feet. The full tank level is 54.3 feet so a decline in tank level to 52 feet represents discharge of 23,838 gallons from storage in the tank before wells in the wellfield begin to pump. As the tank water level declines further to 51 feet, Well No. 3 is programmed to begin pumping and at a tank level of 50 feet, Well No. 7 is programmed to begin pumping.

Figure 2.1 shows the duration of pumping at each well during a representative 3.5-day period from 11/29/01 through 12/3/01. Figure 2.1 also shows the collective pumping rate from the wellfield compared to the pump operation periods. Figure 2.2 adds the storage tank water level record to the information shown on Figure 2.1. As shown on Figures 2.1 and 2.2, Wells No. 1, 3, 6, and 7 operated essentially 24 hours per day at a collective production rate of 774 gpm. The highest storage tank water level shown on Figure 2.2 is 54.2 feet. Accordingly, the storage tank level never reached the upper set points to shut off Wells 1, 6 and 7 during the 3.5-day period of observation. Well No. 3 did not stop pumping when the tank level increased to the upper set point of 54 feet (Table 2.2) because the well was operating on manual pending final repairs to the telemetry damaged by a lightning strike.

Table 2.2 shows that the next well to start pumping after Wells No. 1, 3, 6 and 7 is Well No. 4. Well No. 4 offers relatively low production, a fact that results in the next wells in succession starting shortly after Well No. 4 starts. The next set point below that for Well No. 4 is that for Well No. 8. Since Well No. 8 was stopping due to an electrical overload fault, the next wells to start after Well No. 4 were Well No. 5 followed by Well No. 9. Although Well No. 5 is nominally more productive than Well No. 4, it begins to pump air after a relatively short period of operation. Therefore, the upper set point for Well No. 5 is programmed to stop Well No. 5 at a tank storage level one foot lower than that for Well No. 4 as is the upper set point for Well No. 9.

Although the above arrangement appears to cause Well No. 4 to operate more than is desirable, the set point limits for Well No. 8 are programmed to start Well No. 8 before Well No. 4 and stop Well No. 8 after Well No. 4. Therefore, repair of the pump at Well No. 8 and restoration of production at Well No. 8 will result in a shorter duration of operation for Well No. 4.

Wells No. 9, 10 and 2 are used to increase the flow of water into the storage tank when its water level is approaching the lower limit of the operational storage, as limited by the need to retain fire flow storage in the water tank. Well No. 2 is programmed to operate only as a last resort, due to the fact it has historically produced excessive amounts of sand which are objectionable in the storage tank and which result in a short pump life at Well No. 2.

Figure 2.1: Well pump operation periods and total pumping rate from wellfield.

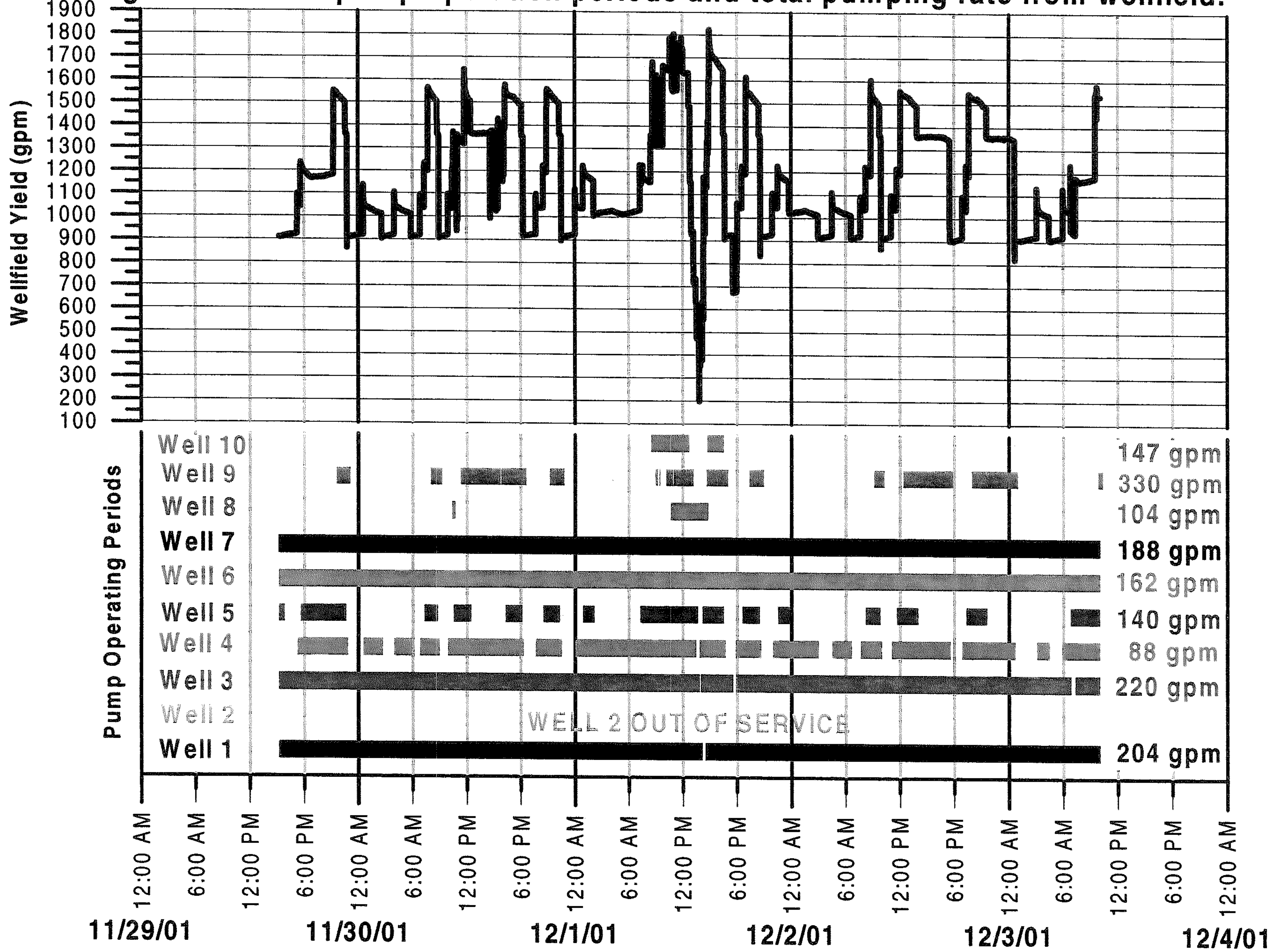

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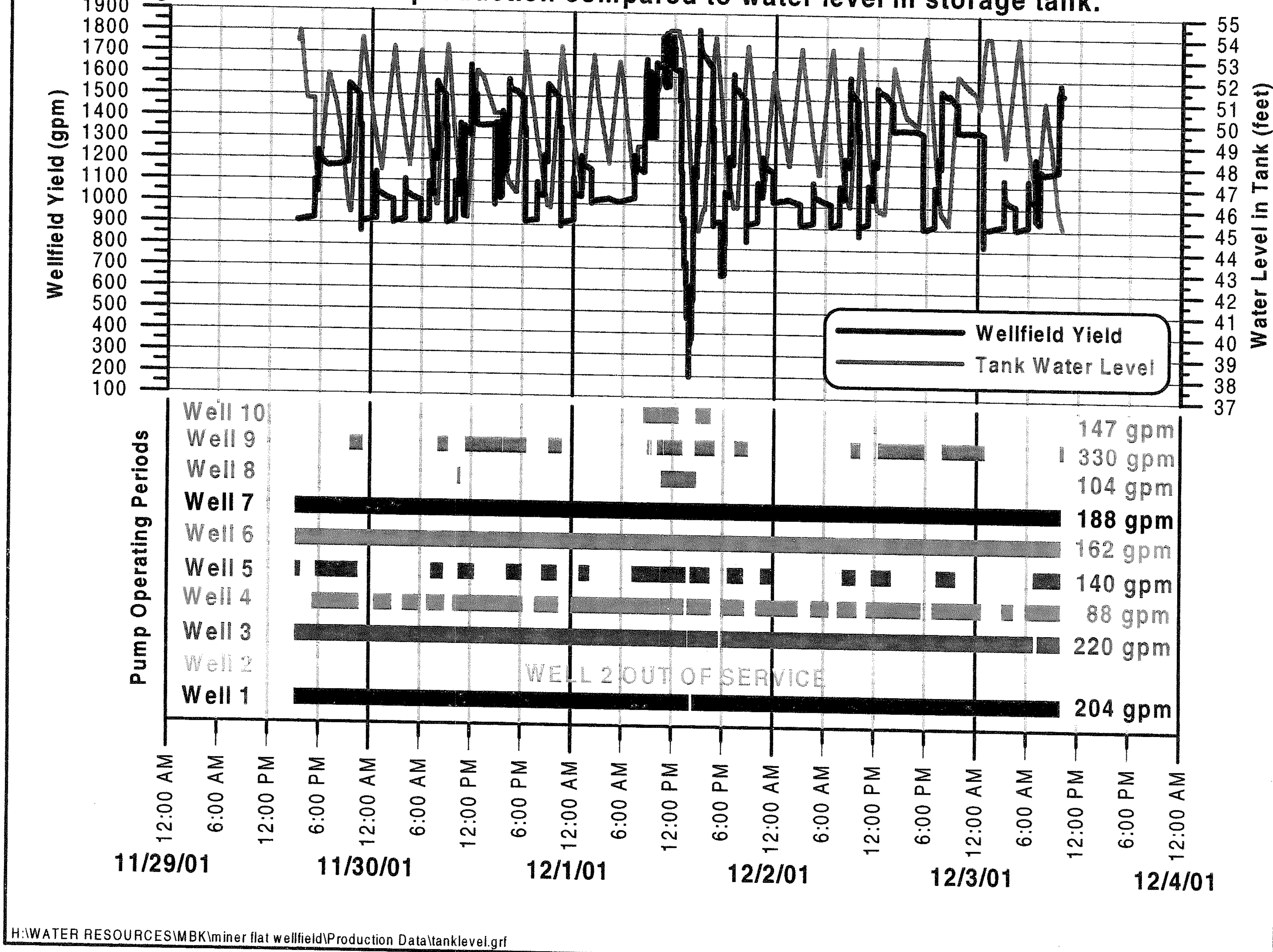
Figure 2.2: Wellfield production compared to water level in storage tank.



2.1.2. Tank Inflow and Outflow

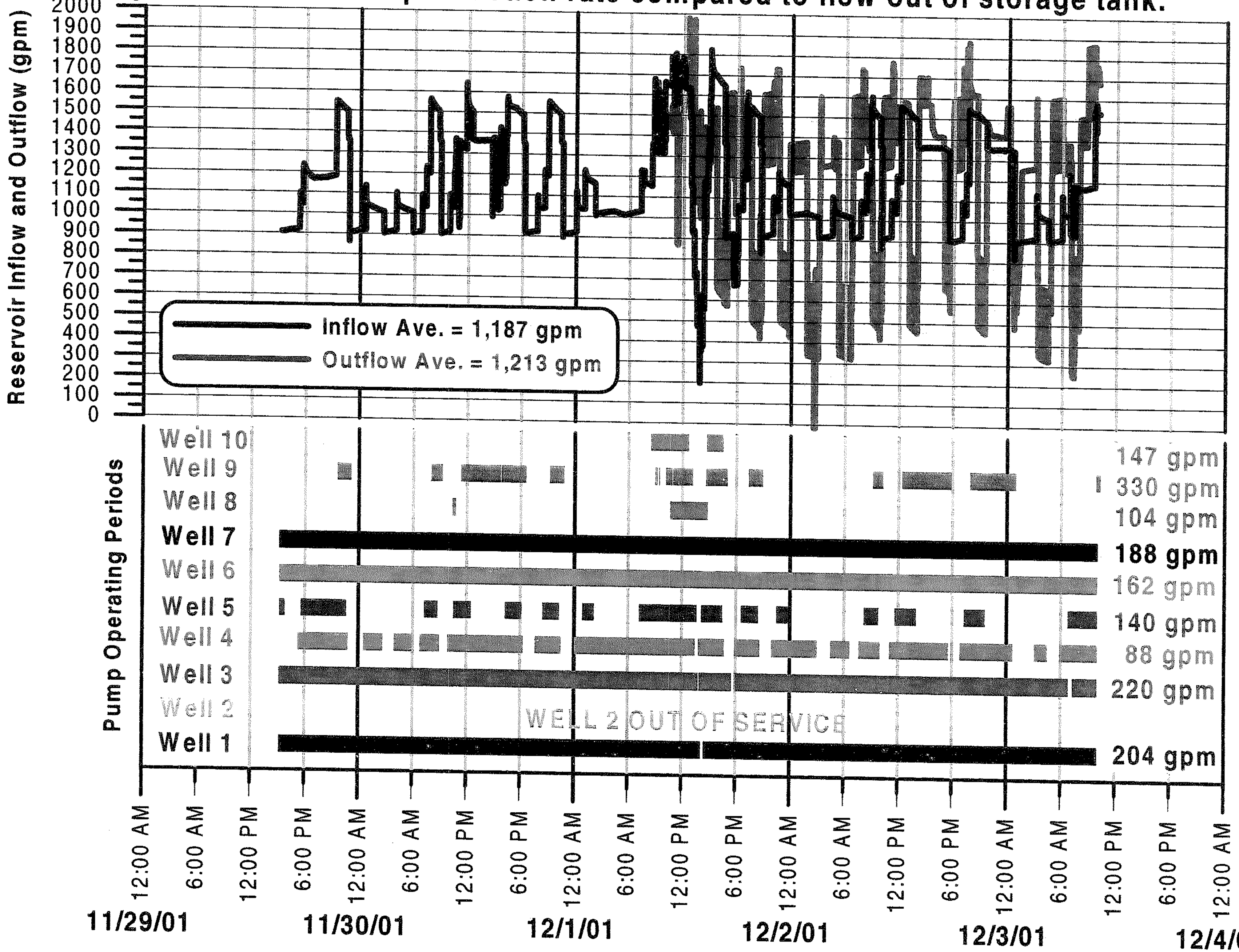
The foregoing review of the set points established to start and stop the wells at different times, depending on the water level in the storage tank at the wellfield, indicates the set points selected are about as reasonable an approach to operating the wellfield as possible, considering the demands placed on the wells. Although it is tempting to reprogram Well No. 9 to operate earlier in the sequence, for example to replace one of the first four wells to start or to replace Well No. 4 in the set point priority, there is little benefit, if any, to be gained by such a change. Prolonged operation of a number of the wells will remain necessary so long as the overall demand for water remains large with respect to the capacity of the wellfield.

For example, Figure 2.3 compares inflow from the wellfield into the storage tank to the outflow from the storage tank to downstream uses from 8:42 a.m. on 12/01/01 to 10:48 a.m. on 12/03/01. The flow hydrographs show the large fluctuations in flow out of the tank as driven by sudden calls for water from downstream storage tanks. The average inflow shown on Figure 2.3 is 1,187 gpm and the average outflow is 1,213 gpm. The 26-gpm discrepancy between inflow and outflow would presumably disappear over a longer period of observations. Comparison of the average demand of 1,200 gpm for the 3-day period on Figure 2.3 to the 12/01/01 wellfield yield of 1,591 gpm shown on Table 2.1, indicates the demand for water is equal to 75 percent of the total wellfield capacity. Accordingly, most of the pumps in the wellfield will have to operate a relatively high percentage of the time. This is not a desirable way to operate the wellfield but it is a necessity under prevailing conditions.

Since the December 2001 investigation was completed, a replacement pump has been provided for Well No. 8. Likewise, a sand separator is being installed in Well No.2 which, if successful, will render operation of that well attractive again. Restoration of the presently unknown yield of Wells No. 2 and 8 will decrease some of the use for all the wells in the wellfield. Depending on the yield and hydraulic performance of Wells No. 2 and 8, there may some merit to re-evaluating the sequencing of the wells and the priority established by the set point limits, in an effort to reduce the demand for marginal wells such as Well No. 5 to pump. The current plans by the I.H.S. to complete at least two more wells for the wellfield will also influence the pumping duration at each well and require additional consideration of how to program the set points to start and stop the well pumps.

As will be discussed later in this report, restoring Wells No. 2 and 8 to service and adding two or more additional wells to the wellfield will not solve the long-term problem at the wellfield. The long-term problem is that of declining groundwater levels and depletion of the groundwater available in the aquifer. Operation of more wells in the wellfield may spread the depletions over a larger area and thereby extend the life of the wellfield somewhat, presuming annual pumping withdrawals are not increased, but a continued decline in the groundwater levels due to aquifer depletion will ultimately result in a continued decline in the collective capacity of the wellfield.

Figure 2.3: Wellfield production rate compared to flow out of storage tank.



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2.1.3. Historic Pumping

The average 24-hour demand of 1,200 gpm satisfied by the wellfield during the 12/01/01 through 12/03/01 observations compares favorably with the current I.H.S. identification of a nominal average 24-hour demand of 1,300 gpm levied against the wellfield. The I.H.S. determination is based on the flow meter records of flow discharged from the storage tank at the Miner Flat Wellfield since the wellfield was put into operation in January 1998. The flow meter readings and calculated average daily flow rates are summarized on Table 2.3.

Table 2.3: Flow records from flow meter on Miner Flat Wellfield storage tank.

Date	Meter Reading (gallons)	Average Daily Flow (gallons)	Average 24-hour Flow Rate (gpm)
11/14/95	78,889,000		
12/14/95	93,052,800	472,127	328
7/09/99	2,044,298,000	1,497,502	1,040
7/20/99	2,087,224,000	2,044,095	1,420
3/06/00	2,481,766,000	1,793,373	1,245
8/30/01	3,504,789,000	1,887,496	1,341
Average Flows*		**1,805,617**	**1,254**

*Average flows do not include 12/14/95 reading.

The records summarized on Table 2.3 indicate an average daily demand of slightly more than 2 MGPD (million gallons per day) or 154 gallons per capita day (gpcd) in an eleven-day period in July 1999, a value which is probably close to the maximum day demand. The average daily demand over the period of record is 1.8 MGPD or 138 gpcd. Table 2.3 shows that more than 3.5 billion gallons of water (10,755 acre-feet) have been produced by the Miner Flat Wellfield at an average annual rate of 2,022 acre-feet per year (a-f/yr).

These statistics indicate there is not much fluctuation in demand compared to the type of seasonal demand observed in many public water supply systems. The significant difference between maximum day demand and average day demand in most public water supply systems is driven by irrigation of lawns in the summer months. Lawn and garden irrigation is relatively limited in the Whiteriver service area. Therefore, the average daily flow and the maximum day flow are relatively similar and the flow records provided on Table 2.3 reflect a relatively constant year-round demand for water. It is anticipated this demand will grow as the population in the greater Whiteriver area increases.

The data summarized on Figure 2.3 are significant in that they reflect very little change in the demand for water in the past four years of operation of the Miner Flat Wellfield. When suspicions developed that the groundwater levels at the wellfield might exhibit significant decline, there was speculation that such decline was caused by excessive pumping of the wellfield. Part of this speculation was based on the fact that casual observations from time-to-time noted water spilling from a full storage tank at the wellfield treatment building. This suggested that during the manual operation of the wellfield, before implementation of the telemetry system, water might have been wasted by leaving the pumps operating when the storage tank was full.

Although the meter records on Table 2.3 do not record water that was wasted through the tank overflow, it is doubtful that the amount of water spilled was significant compared to overall pumping. As discussed in Chapter 3 of this report, pumping water level conditions at the wells probably began limiting the yield of the wellfield within the first year or two of pumping operations, such that the pumping capacity of the wellfield was not greatly excess to the demands for water. This being the case, excessive wasting of water due to manual operation of the pumps while the storage tank was full could not have been very significant.

As shown by the analysis of aquifer response to pumping shown in Chapter 3, there is no need to assume that excessive pumping during the early life of the wellfield is necessary to explain the conditions prevailing in December 2001. The physical response of the aquifer to the pumping demands reported on Table 2.3 are sufficient to explain the December 2001 conditions in the aquifer and in the wells.

2.2. Causes of Lost Yield

Any water well involves the three following components that influence the yield of the well:

1. The aquifer.
2. The well casing and screens or perforations.
3. The pump and motor and appurtenances such as the pump column pipe.

The analyses performed following the December 2001 investigation of the wellfield indicate the decreased production capacity of the Miner Flat Wellfield is primarily a function of the aquifer conditions rather than the result of a change in the well screens or the pumps and motors. The principal changed condition in the aquifer is a decrease in saturated thickness resulting from decline of the groundwater levels in the wellfield area since the wells were put into operation. A second cause of decreased well yield is damage to the individual pumps in the wells; however, this second problem is a direct result of the decline in groundwater levels and operation of the pumps without a sufficient water level to support their operation. The analyses show that the hydraulic performance of the well screens has probably not changed since the wells were constructed, even though overall hydraulic performance of the wells has decreased significantly due to the decrease in saturated aquifer thickness caused by the decline in groundwater levels.

2.2.1. Aquifer Conditions

Figure 2.4 shows the alignment of a line of section through the Miner Flat Wellfield from south to north. The salient details of each production well are projected onto the line of section as shown on Figure 2.5. Figure 2.5 depicts the thickness of the various geologic strata penetrated by each of the wells and the initial position of the static water levels and pumping water levels with respect to the subsurface strata at the time of well construction. Although basalt was penetrated by Wells No. 1, 3, 4, 9 and 10, the simplified cross section on Figure 2.5 does not distinguish between the unconsolidated overburden and the basalt because the clayey overburden and the basalt are both considered to be confining layers.

Figure 2.5 indicates that when the wells were initially constructed, confined conditions prevailed at Wells No. 1, 3, 7, 8, 9, and 10. Figure 2.5 also shows the range of water level fluctuations from static conditions to the pumping water level after 12 hours of pumping during baseline pumping tests. In all of the wells except Well No. 9, the pumping water levels in the well declined below the base of the confining units. Although the 12-hour pumping water levels include a component of well loss drawdown that is not present in the aquifer outside the pumped well, baseline pumping test responses shown in Chapter 3 provide positive evidence that the portions of the cones of depression nearest the pumped wells went from an initially confined state to an unconfined state, consistent with the conditions depicted on Figure 2.5. This means that during the constant rate baseline test yields summarized on Table 2.1, the aquifer response at the wells in the confined part of the aquifer was initially a confined aquifer response which was subsequently modified when a portion of the cone of depression became unconfined.

Figure 2.6 shows the same information as Figure 2.5 with the addition of the December 2001 static water levels and pumping water levels. Table 2.4 summarizes the decline in the static water level at each well where data are available.

Table 2.4: Change in static water levels between 1997 and 2001.

Well Number	Static Water Level 1997 (feet BTOC*)	Static Water Level 12/01/01 (feet BTOC*)	Groundwater Level Change (feet)	Loss of Aquifer Thickness
1	72	Unknown	---	---
2	162	201.35	39	---
3	83	Unknown	---	---
4	117	168.85	52	20.0%
5	162	Unknown	---	---
6	132	207.11	75	36.6%
7	121	197.76	77	36.2%
8	76.5	130.82	54	---
9	55	98.1	43	14.8%
10	76.5	105.65	29	10.4%

* BTOC = Below Top of Casing

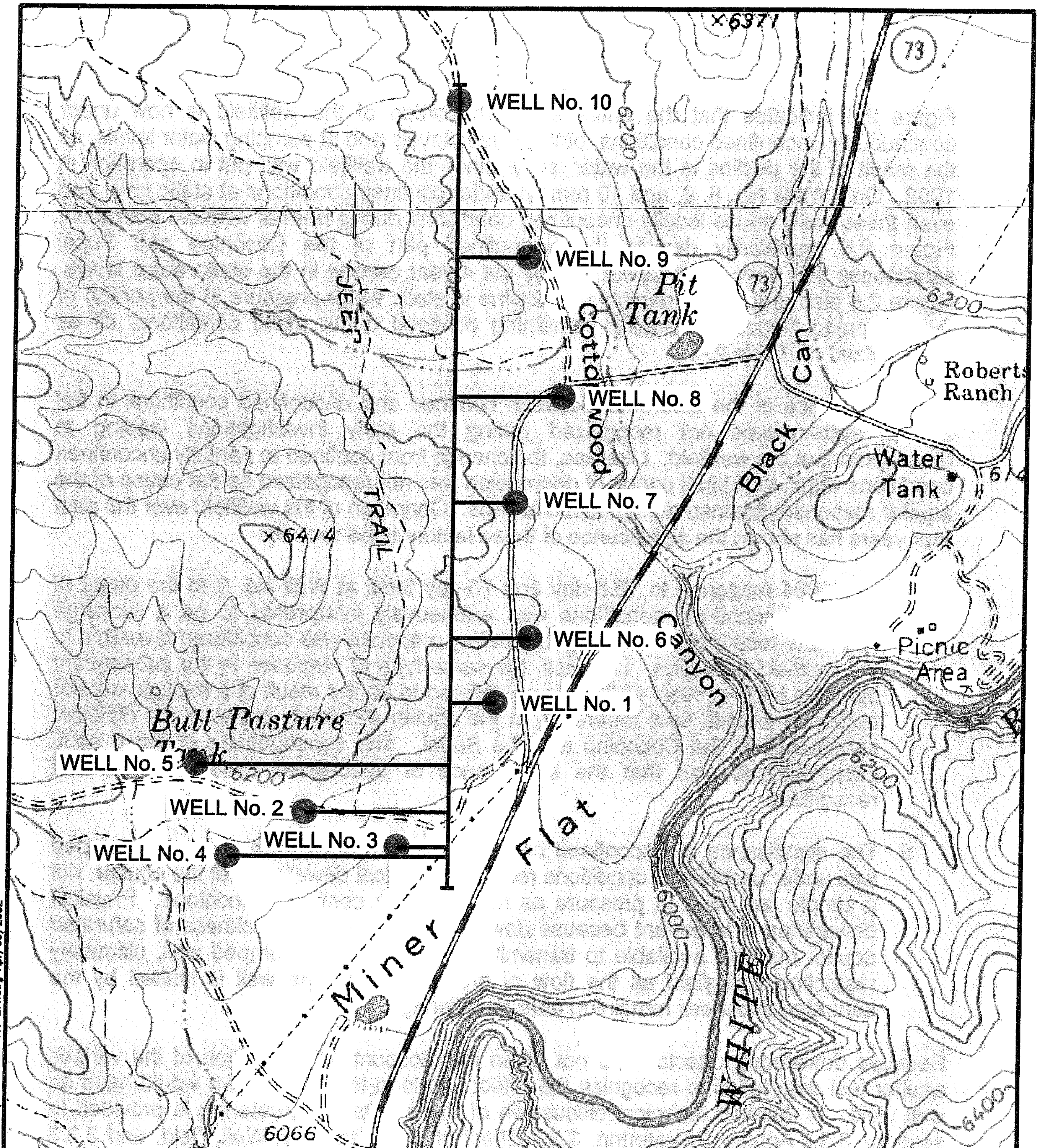


FIGURE 2.4: Alignment of Line of section through well field.

LEGEND

WELL No. 5 - PRODUCTION WELL LOCATION

LINE OF SECTION

N

1000 0 1000

SCALE IN FEET

H:\WATER RESOURCES\1780 wmat\miner flat dam\MINER FLAT FIG 2.4.dwg Apr/09/2002

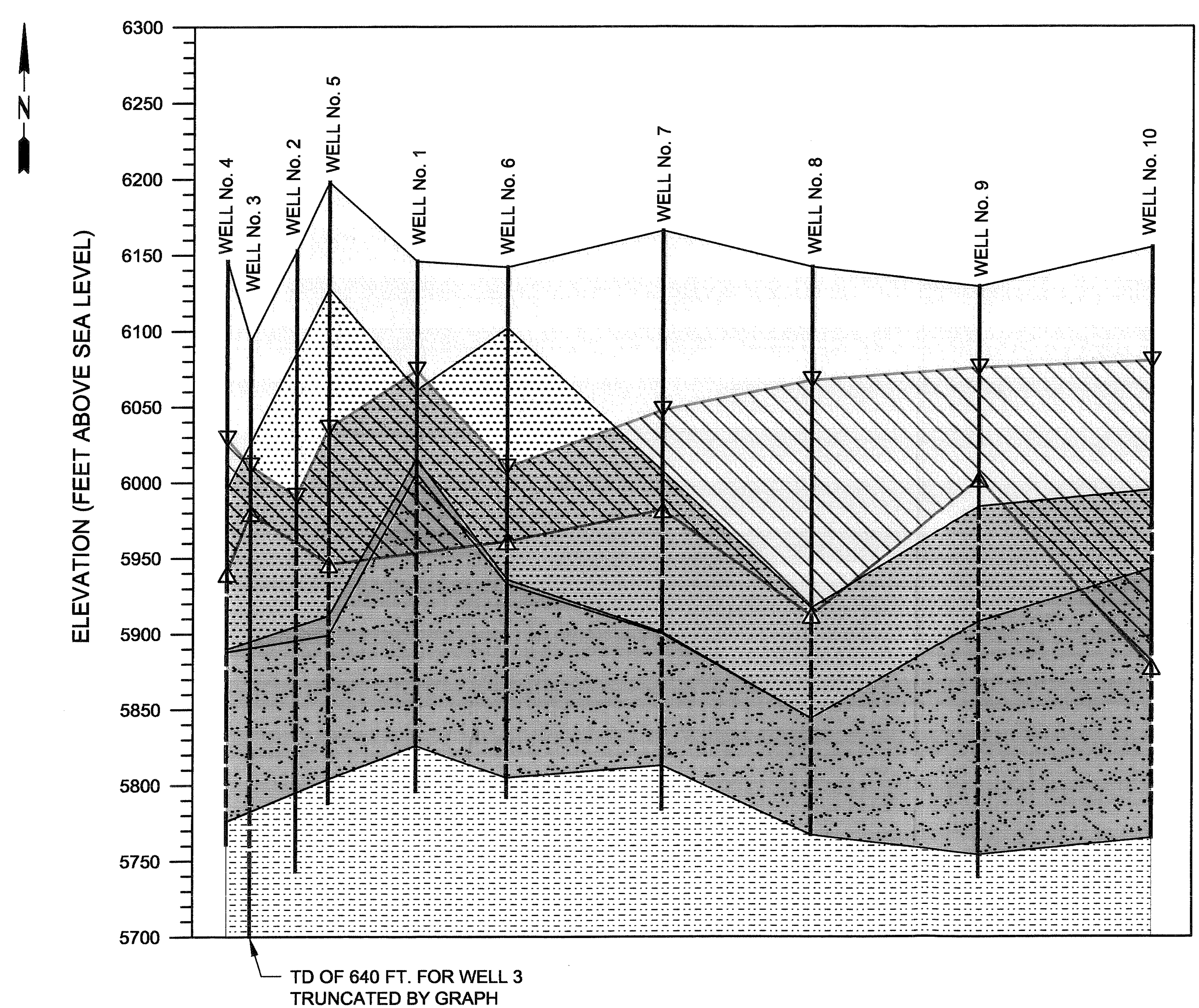
Figure 2.6 indicates that the entire southern portion of the wellfield is now under continuously unconfined conditions, both at static levels and at pumping water levels, as the result of the decline in the water levels since the wellfield was put in operation in 1998. Only Wells No. 8, 9, and 10 remain under confined conditions at static level and even these wells cause locally unconfined conditions during normal wellfield operation. Figure 2.6 graphically depicts the unconfined part of the Coconino and Supai sandstones that have been dewatered by the 4-year decline in the static water levels. Figure 2.6 also graphically depicts the decline in static water pressure in the portion of the Coconino Sandstone aquifer remaining confined under static conditions, all as summarized on Table 2.4.

The significance of the difference between confined and unconfined conditions in the aquifer system was not recognized during the early investigations leading to development of the wellfield. Likewise, the change from confined to partially unconfined conditions within individual cones of depression was not recognized as the cause of the aquifer response obtained during baseline tests. Operation of the wellfield over the past four years has shown the significance of these factors to be twofold:

1. The 1994 response to 13.8-day and 70-day tests at Well No. 3 to the onset of partially unconfined conditions was erroneously interpreted to be a recharge boundary response. A recharge boundary response was considered favorable to the wellfield operation. Likewise, the same type of response in the subsequent baseline tests of other wells was considered to be the result of a multiple-aquifer response caused by a difference in the aquifer storativity between the different sandstones in the Coconino and the Supai. The consequence of these early interpretations was that the significance of unconfined conditions was not recognized.

2. The significance of unconfined conditions is that drawdown around a pumped well under unconfined conditions results in physical dewatering of the aquifer, not a simple reduction in pressure as results under confined conditions. Physical dewatering is important because dewatering reduces the thickness of saturated aquifer material available to transmit groundwater to a pumped well, ultimately restricting well yield as the flow of groundwater to the well is limited by the saturated thickness remaining after dewatering.

Because dewatering effects were not taken into account, interpretation of the various aquifer test data failed to recognize the effect that long-term drawdown would have on well yield. A detailed technical discussion of the effects of dewatering is provided in sections 3.3.5 Aquifer Dewatering, 3.3.7 Effect of Dewatering on Well Yield, and 3.3.8 Erroneous Assumptions of Chapter 3 of this report. The foregoing sections of the report also explain how the long-duration tests of Well No. 3 in 1994 provide diagnostic evidence of dewatering effects.



LEGEND

- LAND SURFACE
- STATIC WATER LEVELS
- PUMPING WATER LEVELS
- WELL CASING
- WELL SCREEN

- UNCONSOLIDATED OVERBURDEN AND BASALT (WHERE PRESENT)
- UNSATURATED COCONINO
- SATURATED AQUIFER IN COCONINO
- LAMINATED RED CLAY AND SILT
- SATURATED AQUIFER IN SUPAI
- ZONE OF WATER LEVEL FLUCTUATION DURING PUMPING OPERATIONS
- NON-AQUIFER SUPAI

THE COCONINO AQUIFER SYSTEM IS CONFINED IN THE NORTH HALF OF THE WELLFIELD AND UNCONFINED IN THE SOUTH HALF OF THE WELLFIELD.

OPERATIONAL PUMPING WATER LEVELS DRAW GROUNDWATER SURFACE AT WELLS BELOW BASE OF CONFINING LAYERS AND DRAWDOWN IS SIGNIFICANT WITH RESPECT TO SATURATED THICKNESS AND/OR TOTAL HEAD AVAILABLE IN AQUIFER.

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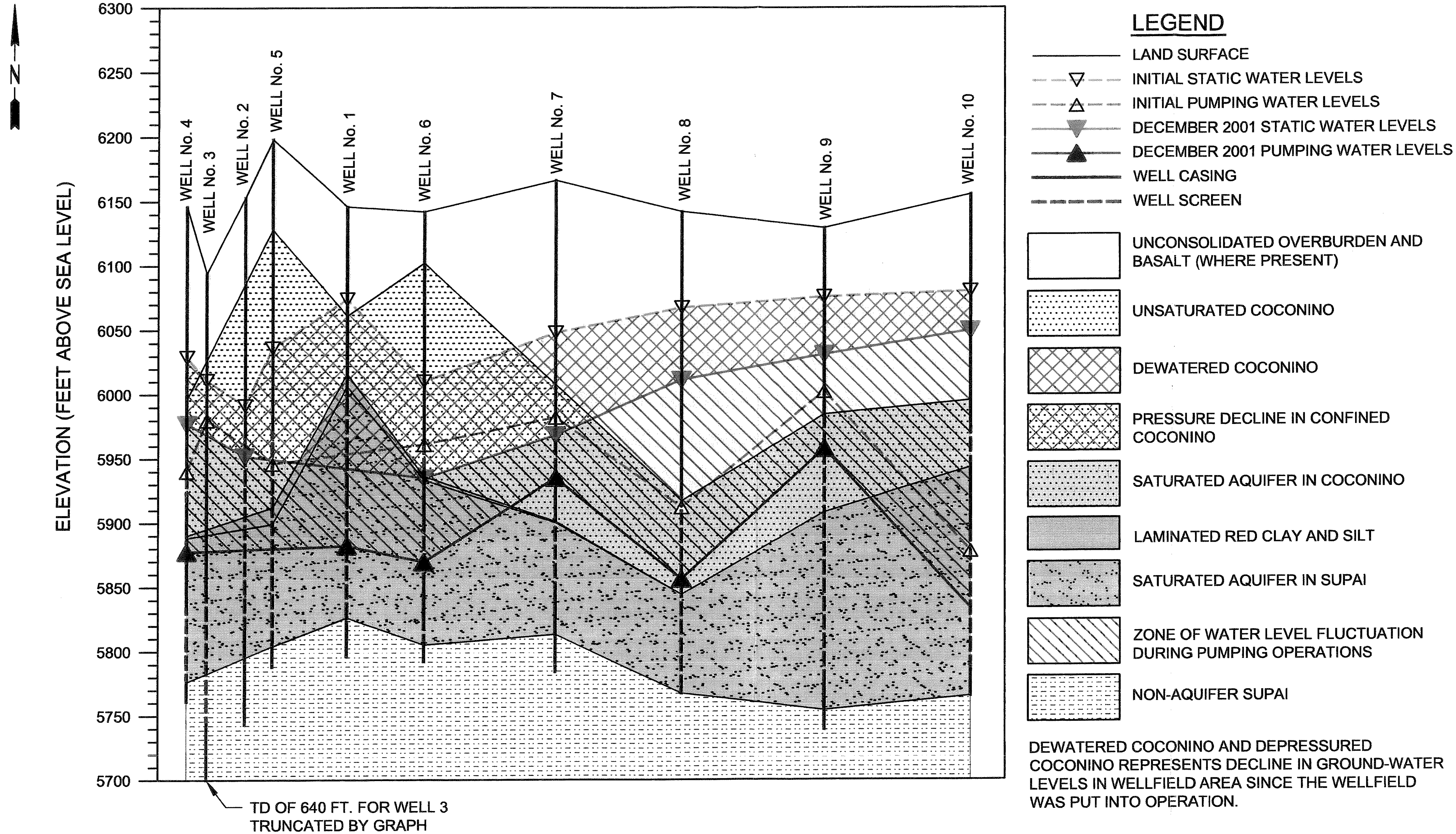
FIGURE 2.5
AQUIFER WATER LEVELS AND SATURATED THICKNESS
AT TIME OF WELL CONSTRUCTION



H:\WATER RESOURCES\1780 wmat\miner flat dam\MINER FLAT FIG 2.6.dwg Apr/08/2002

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FIGURE 2.6
DECLINE IN AQUIFER WATER LEVELS SINCE WELLS CONSTRUCTED

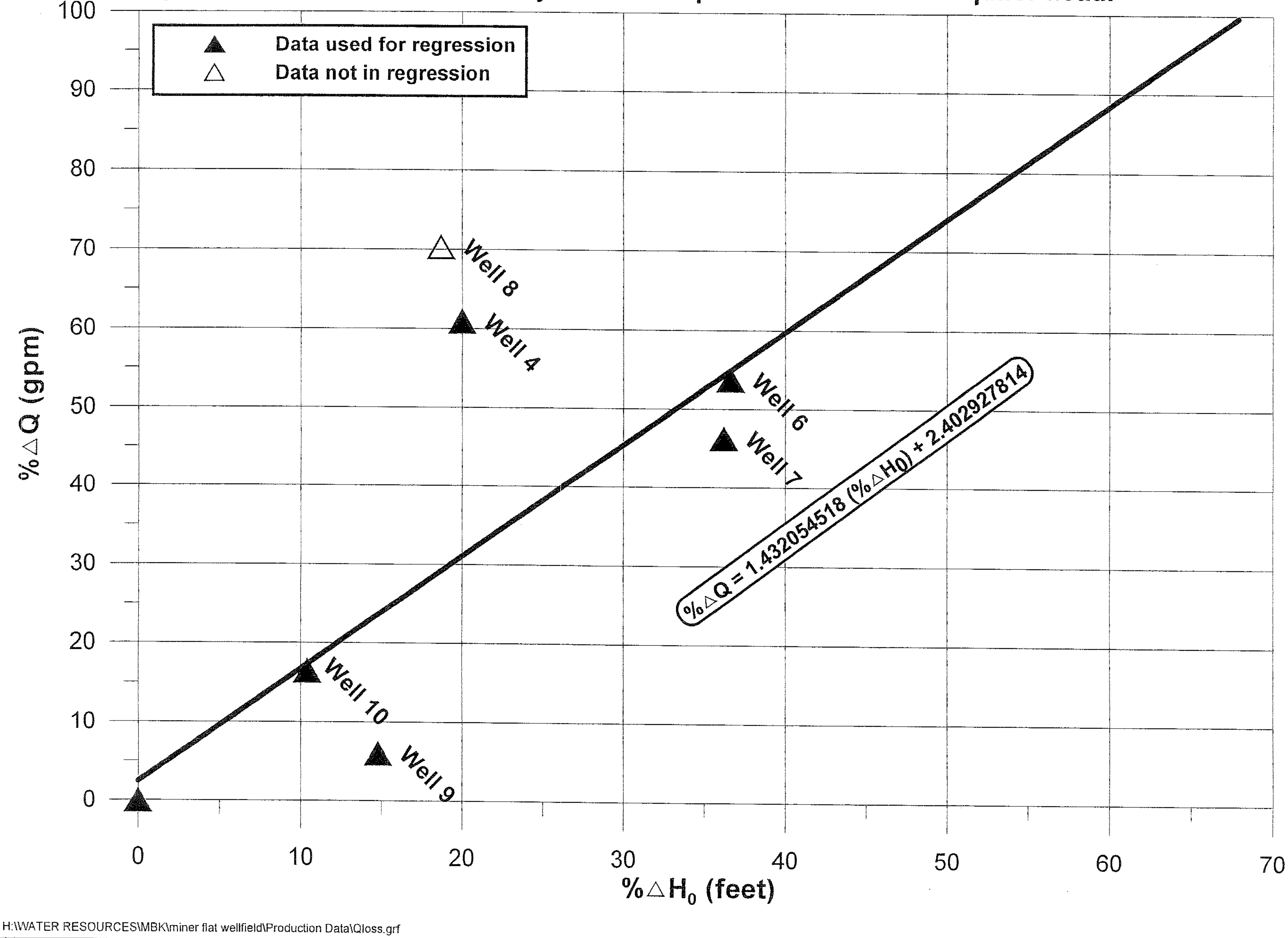
The failure to properly recognize dewatering effects in the early aquifer tests was not in itself sufficient reason to not predict the drawdown and dewatering that occurred after the wellfield was put into operation. Even if dewatering effects had been considered in the projection of wellfield water levels into the future, the type of response obtained during the past four years of operation would not have been predicted. The decline in water levels resulting from the past four years of wellfield operation would not have been predicted because an assumption was made that the wells were withdrawing groundwater from a dynamic groundwater system in which water was flowing from a recharge area to a discharge area and the wells were simply intercepting part of that flow. Implicit to the assumption of groundwater flow through the wellfield area was the corollary conclusion that drawdown caused by pumping the wells would recover due to the flow of groundwater into the wellfield area from the upgradient part of the flow system.

Subsequent experience has shown that a significant groundwater flow through the wellfield area does not exist, at least with respect to the volume of groundwater abstracted by operation of the wellfield. The decline in groundwater levels that has occurred in the past four years is essentially identical to the decline predicted by projection of the baseline test drawdown rates into the future, assuming no recharge to the system. The coincidence between projections of the baseline test responses and the subsequent decline of water levels during four years of wellfield operation is therefore considered to indicate the Miner Flat Wellfield is developed in a hydrologically isolated block of aquifer material which receives only limited recharge in an amount that is insignificant with respect to the rate of withdrawals by the wellfield, at least in the past four years. Consequently, operation of the wellfield has been mining groundwater from the aquifer system for the past four years.

Mining of the groundwater has decreased the saturated thickness of the aquifer, as shown on Table 2.4, resulting in a reduction of the pumping rates the aquifer will support at the various wells. Although the data to support a quantitative analysis of the dewatering effect are very limited, Figure 2.7 shows the relationship between decreasing head (saturated thickness) in the aquifer and decreasing well yield, expressed as percentages.

Recognition of these factors after four years of wellfield operation points out another significant conclusion – that is the conclusion that continued use of the wellfield will result in continued dewatering and loss of well capacity unless the amount of recharge to the aquifer increases from what it has been the past four years. At the present time, it appears to be a foregone conclusion that continued use of the wellfield will cause a continued decline in the groundwater levels in the aquifer. Declining groundwater levels in the future will result in a progressive and continued loss of yield from the wellfield.

Figure 2.7: Percent loss of well yield versus percent decrease in aquifer head.



2.2.2. Groundwater Level Trends

In addition to the data shown on Table 2.4, trends of groundwater levels within the Miner Flat Wellfield are shown by data collected by Dr. Laurel Lacher and the staff of the Hydrology Section of the Environmental Planning Office of the White Mountain Apache Tribe. Those data are shown on Figures 2.8 through 2.17 where linear regression analysis of the static water level records show groundwater decline trends ranging from 9.55 to 30.64 feet per year. Figure 2.18 is a map showing the locations of the wells, including monitoring wells and production wells, and the rates of groundwater decline associated with those wells where data is available.

Although the pumping rates at each of the wells are significantly different, the rates of groundwater level declines at the wells exhibit a clear pattern on Figure 2.18. The pattern is not related to the pumping rates but is directly related to the distance of the wells from the geometric center of the wellfield. The rate of groundwater level decline is greatest at the geometric center of the wellfield and least around the margins of the wellfield as shown on Figure 2.18. The only apparent exception to this relationship is provided by Monitoring Well No. 6; however, close examination of the data on Figure 2.17 suggests there is more than one way to interpret the data.

A second relationship may exist between the rate of groundwater level decline and confined versus unconfined conditions; however, it is not possible to make a positive distinction between a possible relationship of groundwater level decline and unconfined conditions and the obvious relationship with distance from the center of the wellfield. The most likely possibility is that both relationships exist. This is because the onset of unconfined conditions near the geometric center of the wellfield should result in an acceleration of the decline of groundwater levels due to the dewatering effect that takes place under unconfined conditions.

The strong correlation shown on Figure 2.18 between well location and the rate of groundwater level decline is consistent with and supports the conclusion that the primary factor resulting in a loss of capacity at the Miner Flat Wellfield is the diminishment of saturated thickness resulting from mining of groundwater from the aquifer.

There may be a temptation to look at the data on Figures 2.10, 2.14, 2.15, and 2.17 and conclude that the downward trend of groundwater levels at those well sites has ceased or reversed. However, the pumping activities at each site portrayed on those hydrographs must be taken into consideration before any conclusion can be reached. For example, the pump in Well No. 8 is out of service and has been for some time. There has been a substantial period of time when the pump in Well No. 8 operated at a very diminished pumping rate. These conditions affect the rate of groundwater level change at that well. Likewise, Monitoring Well No. 3 is located near Well No. 4 which has suffered a substantial reduction in pumping rate. Monitor Well No. 4 shows some recovery of the static water level over an eight-month period; however, the well is located next to Well No. 2 which is used very little because of its sand production.

Figure 2.8: Water level trend at Well No. 5 - TOC 6201.1 FT.

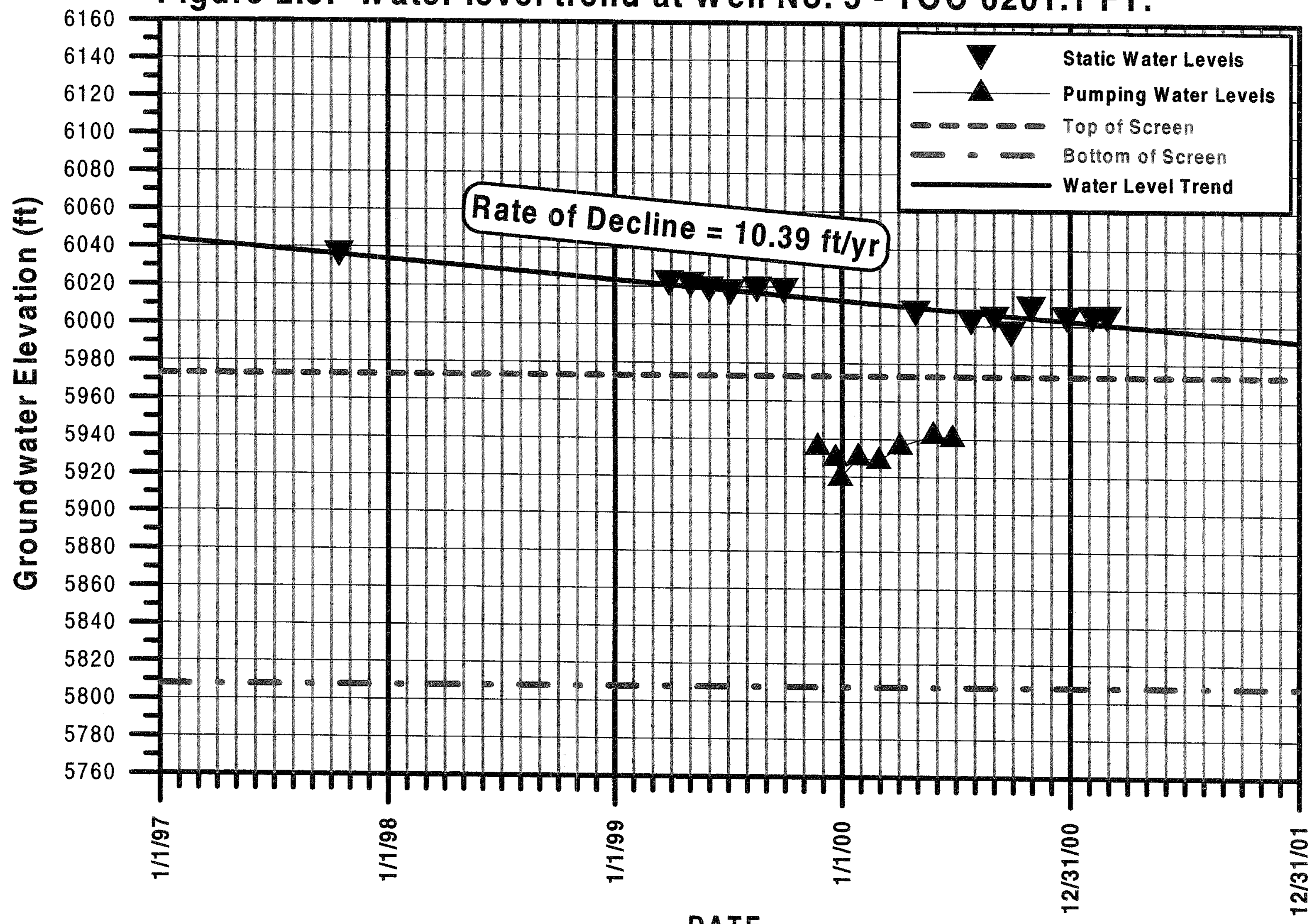


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Figure 2.9: Water level trend at Well No. 7 - TOC 6168.8 FT.

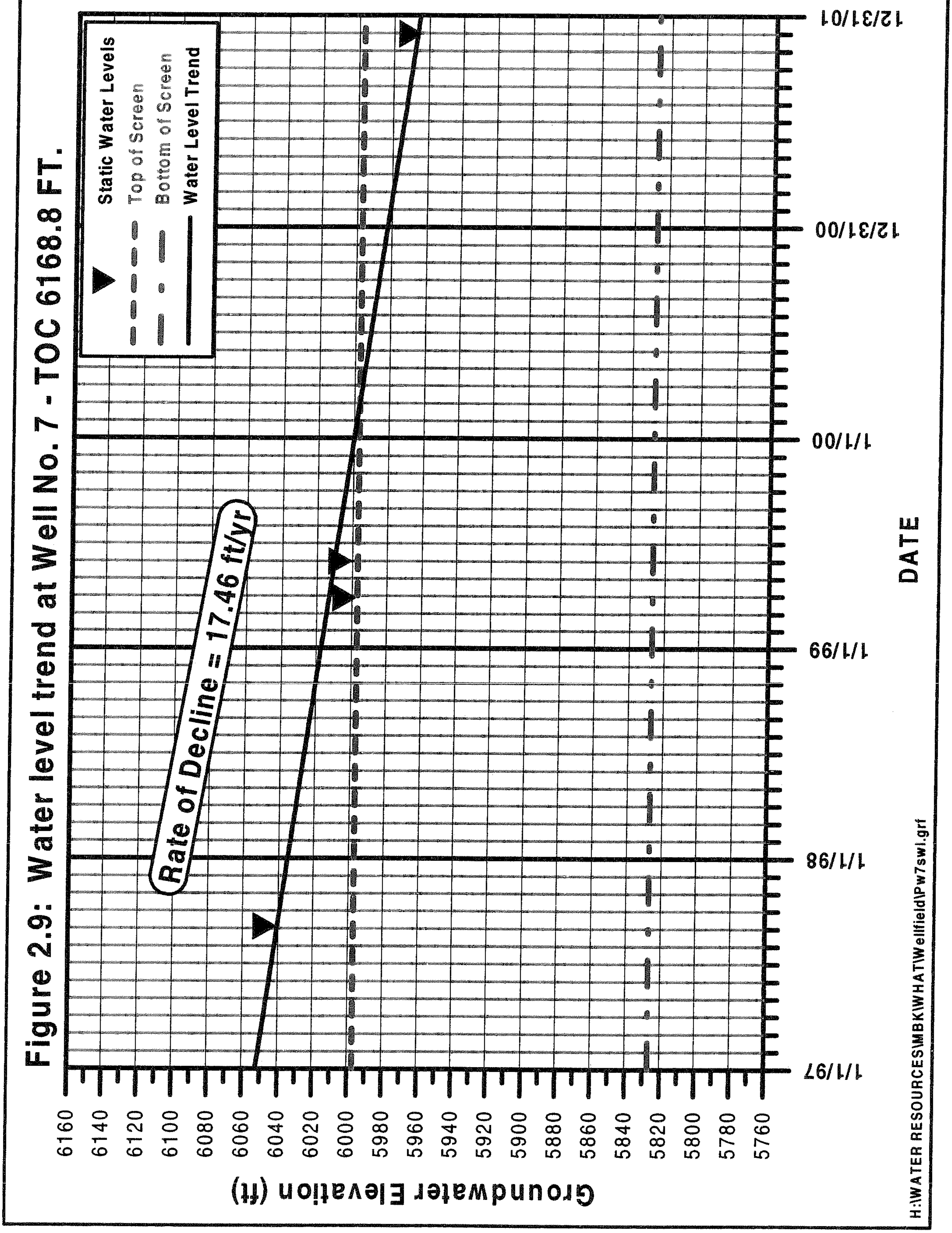

Figure 2.10: Water level trend at Well No. 8 - TOC 6143.5 FT.

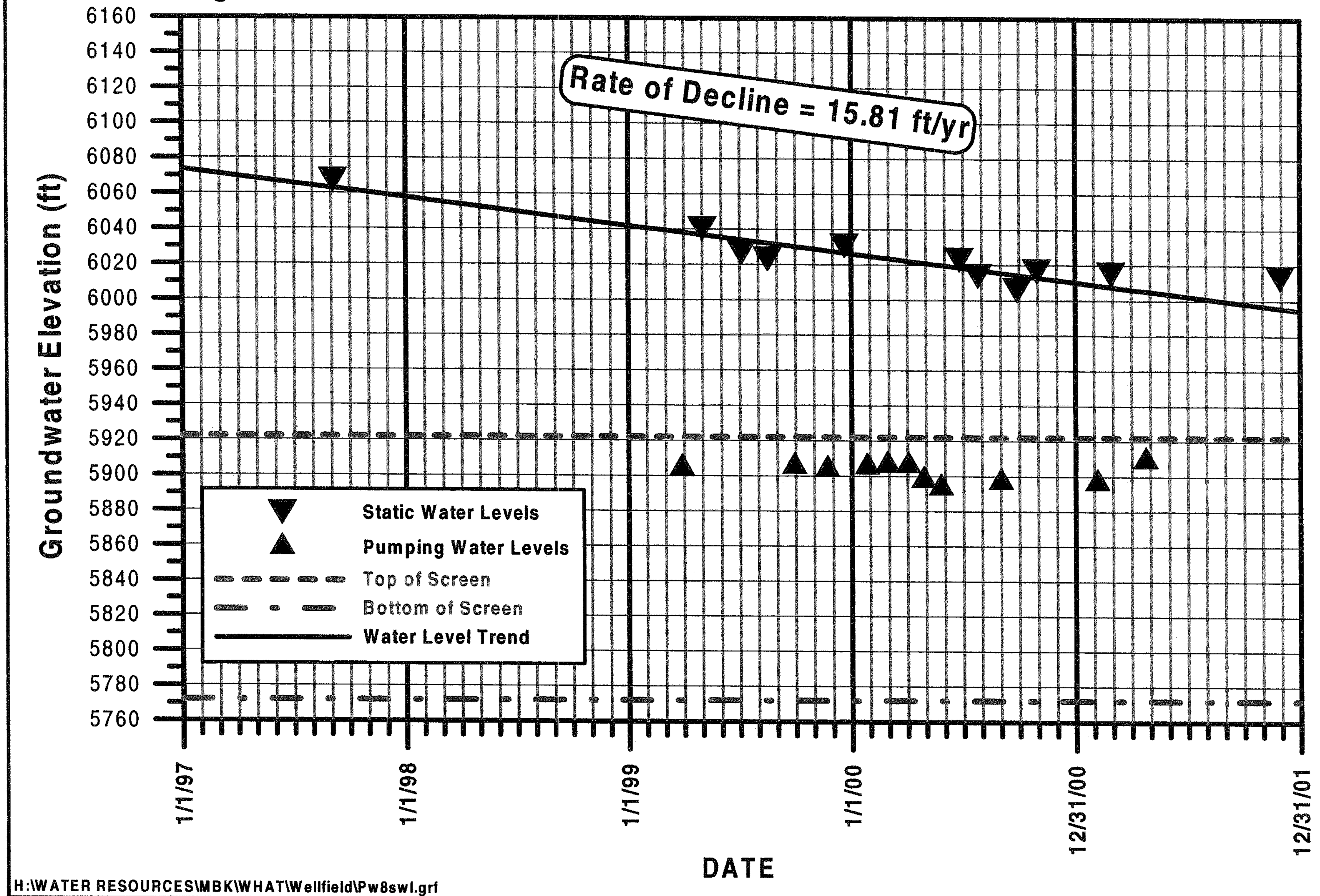

Figure 2.11: Water level trend at Well No. 9 - TOC 6130.5 FT.

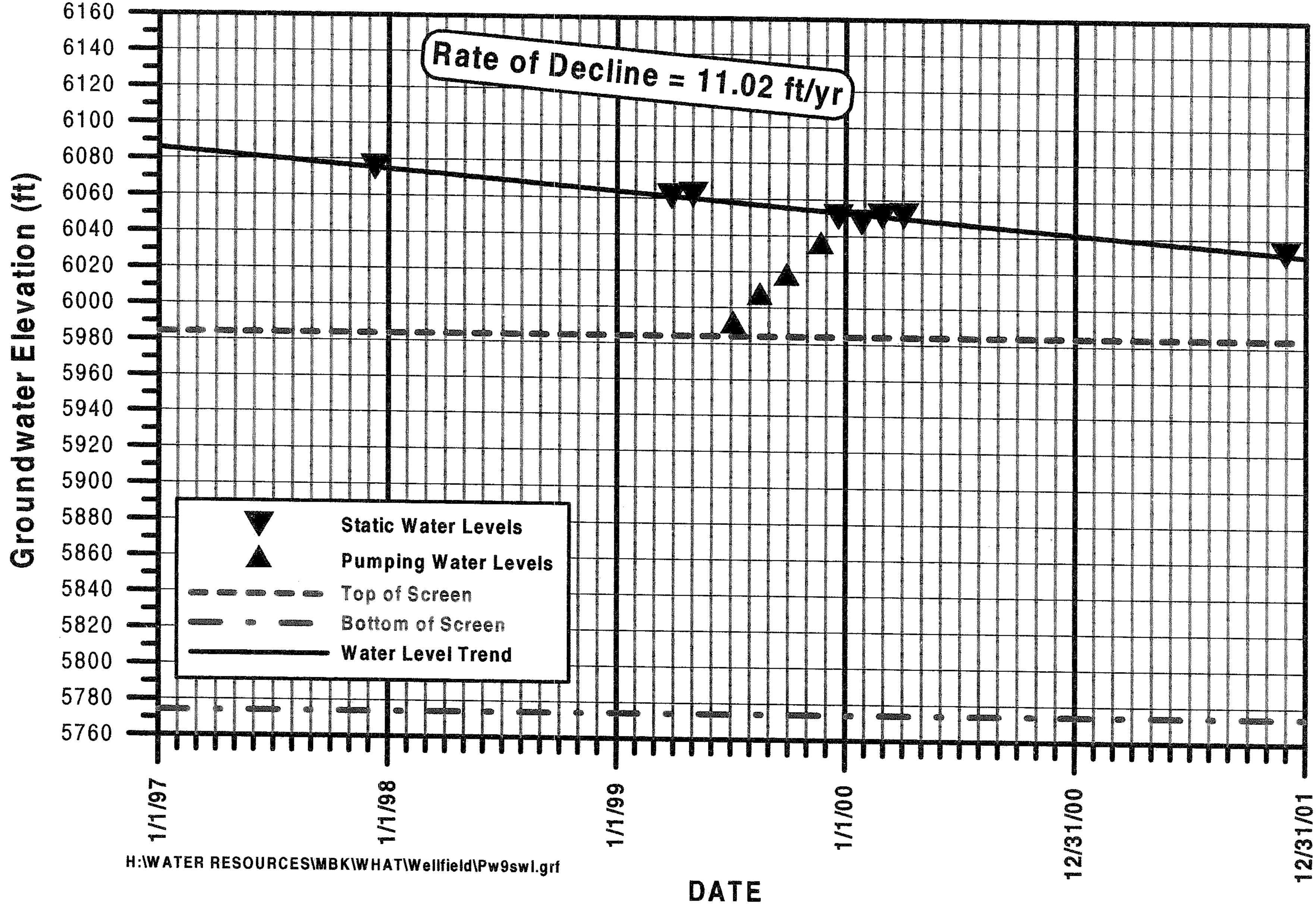

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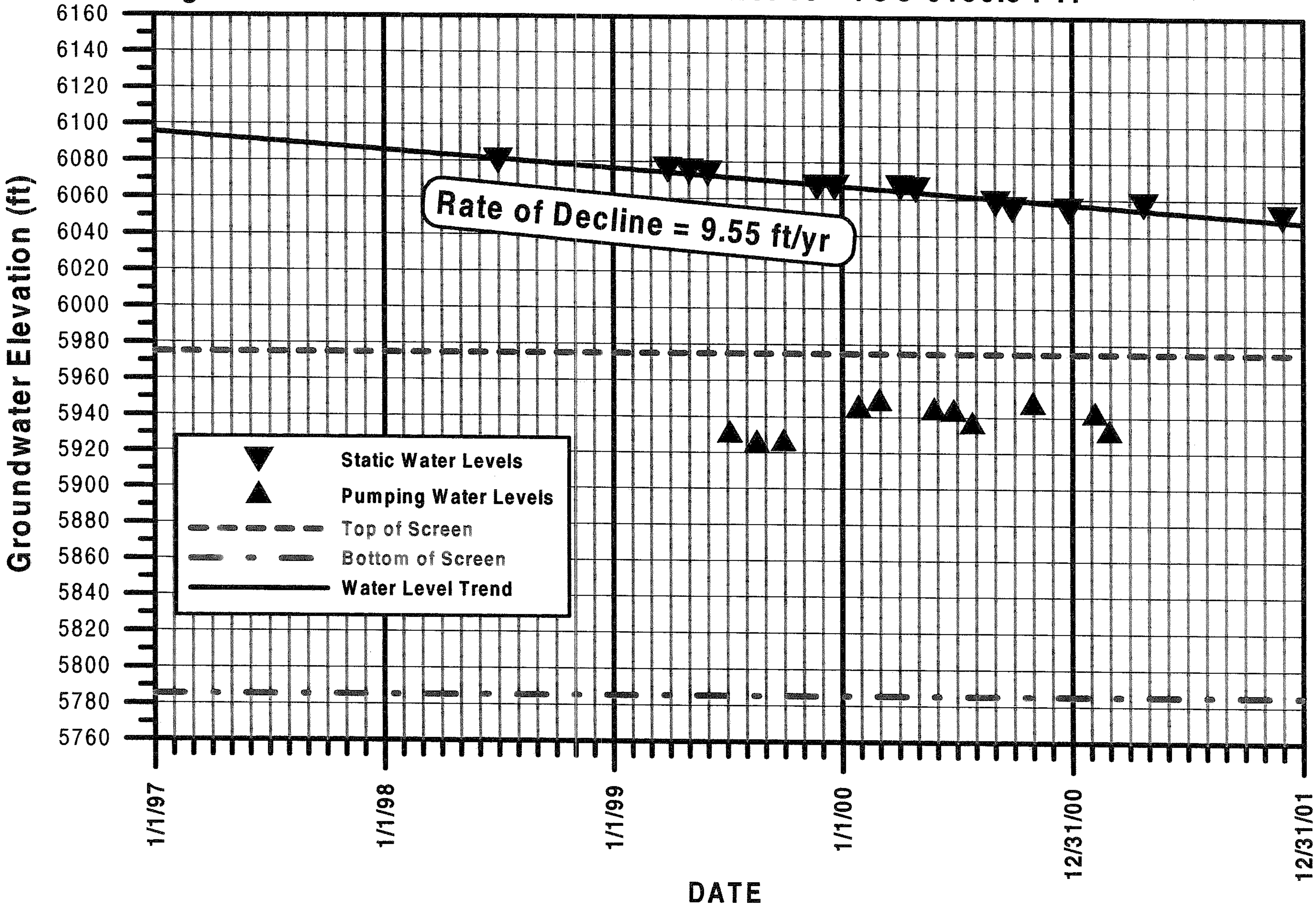
Figure 2.12: Water level trend at Well No. 10 - TOC 6156.5 FT.



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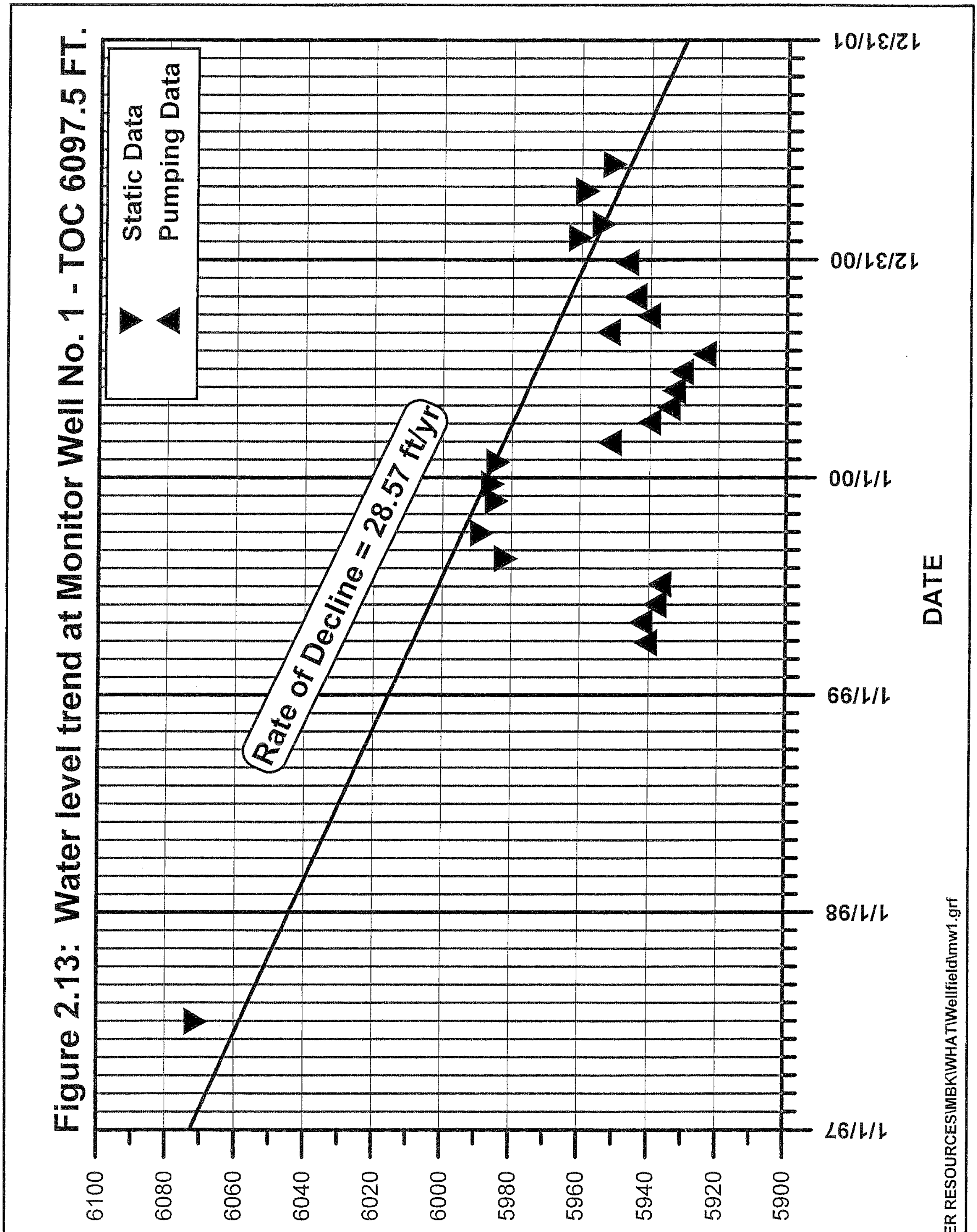

Figure 2.13: Water level trend at Monitor Well No. 1 - TOC 6097.5 FT.

Figure 2.14: Water level trend at Monitor Well No. 3 - TOC 6147.4 FT.

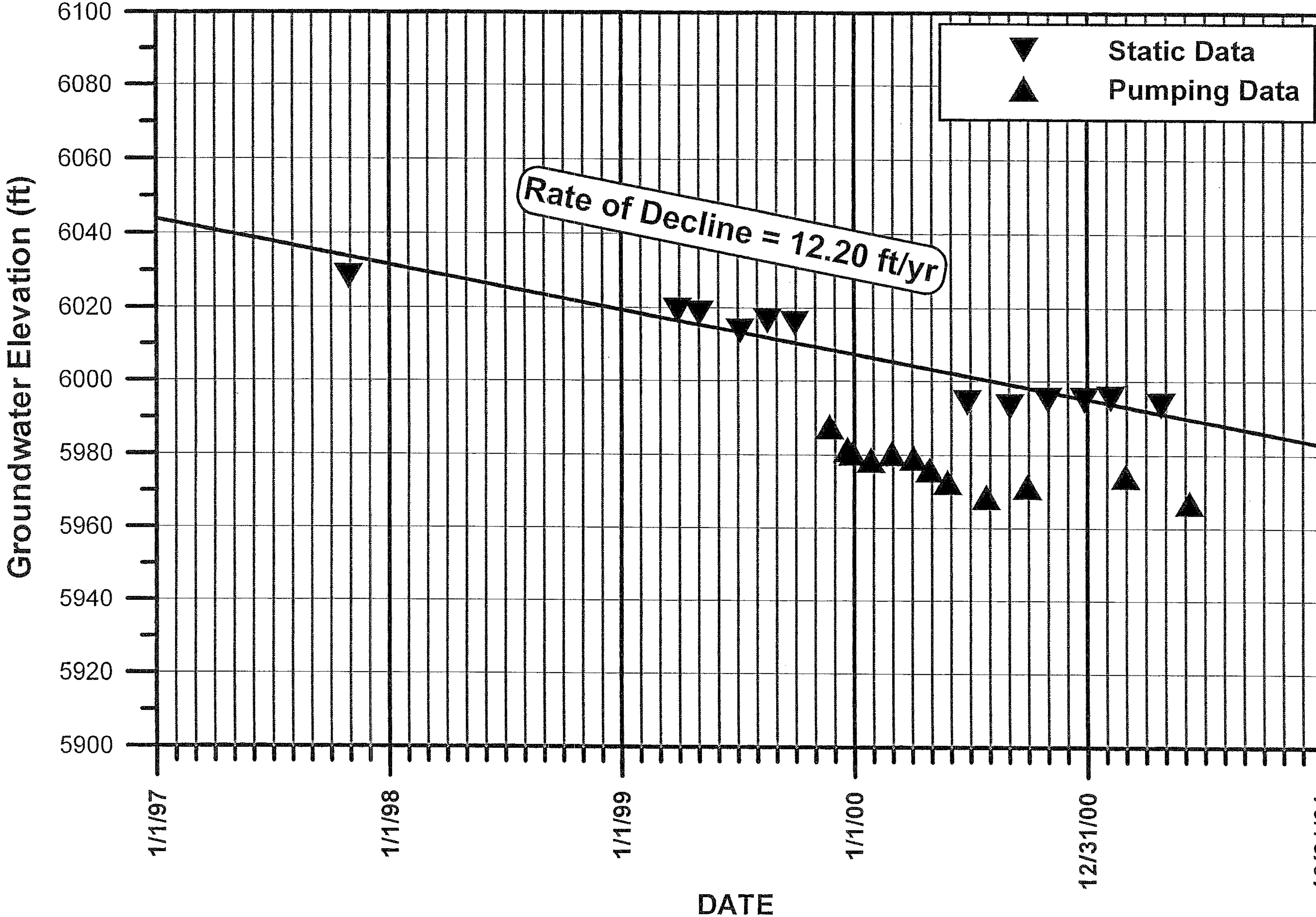

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Figure 2.15: Water level trend at Monitor Well No. 4 - TOC 6156.1 FT.

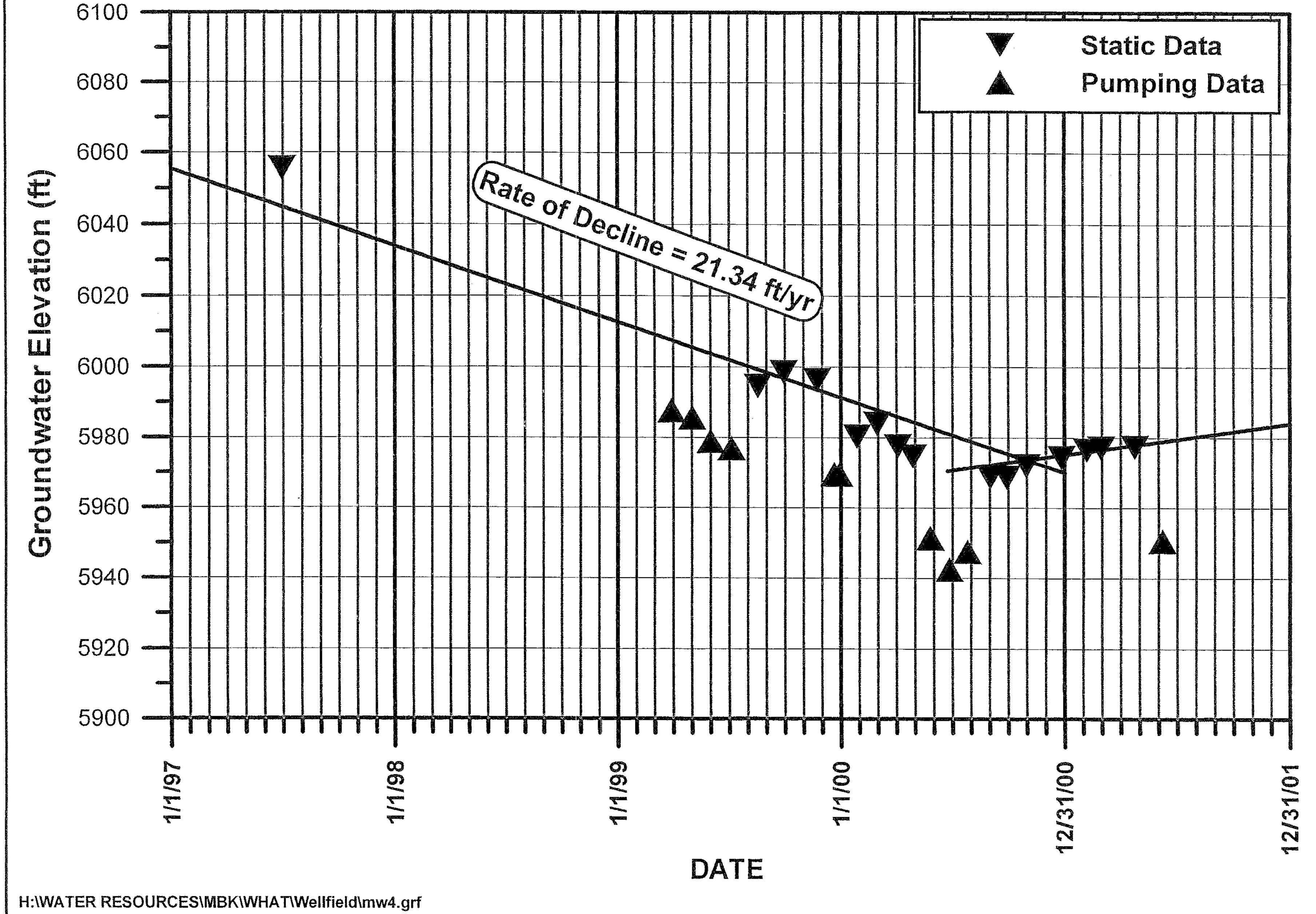
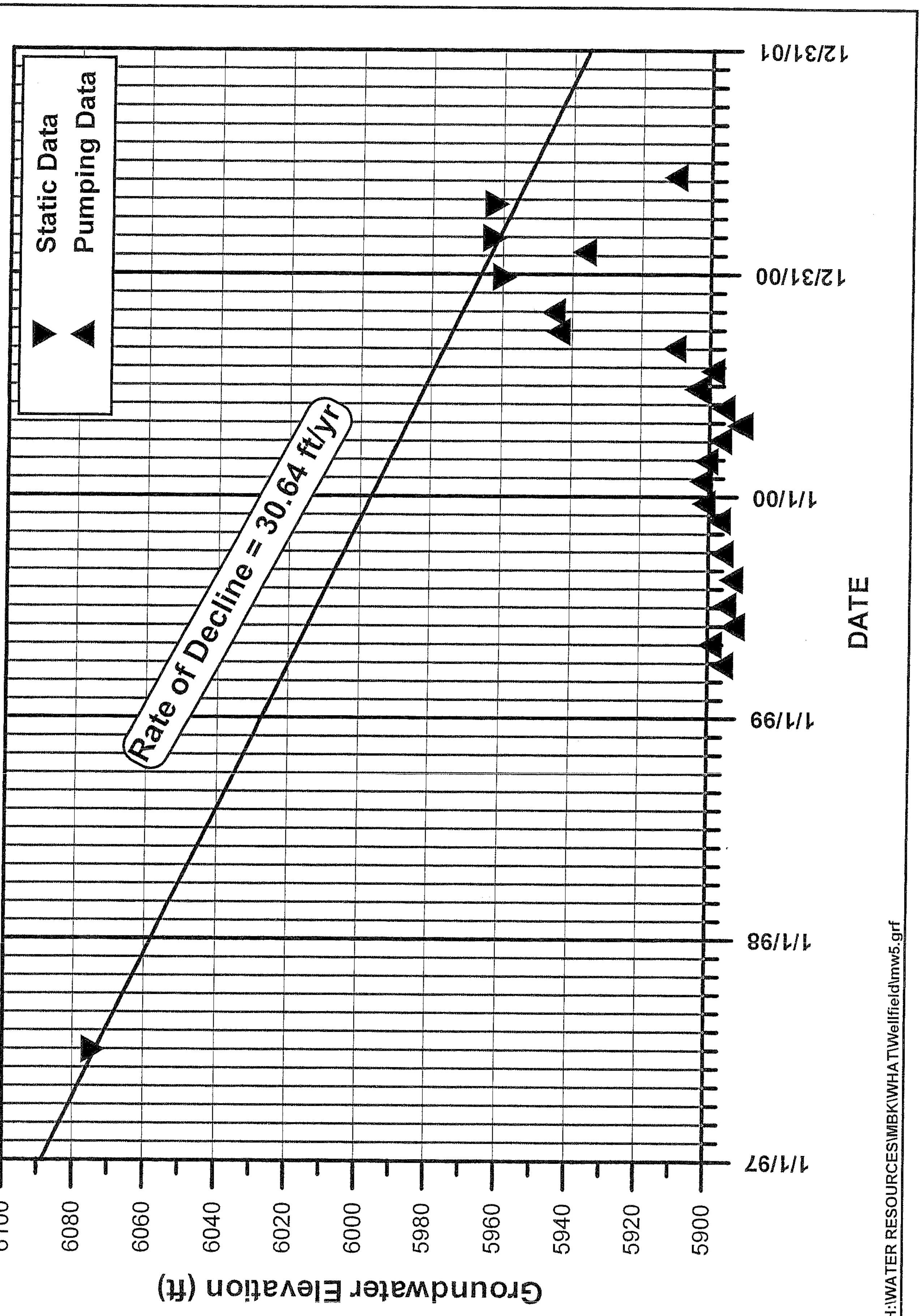


Figure 2.16: Water level trend at Monitor Well No. 5 - TOC 6147.03 FT.



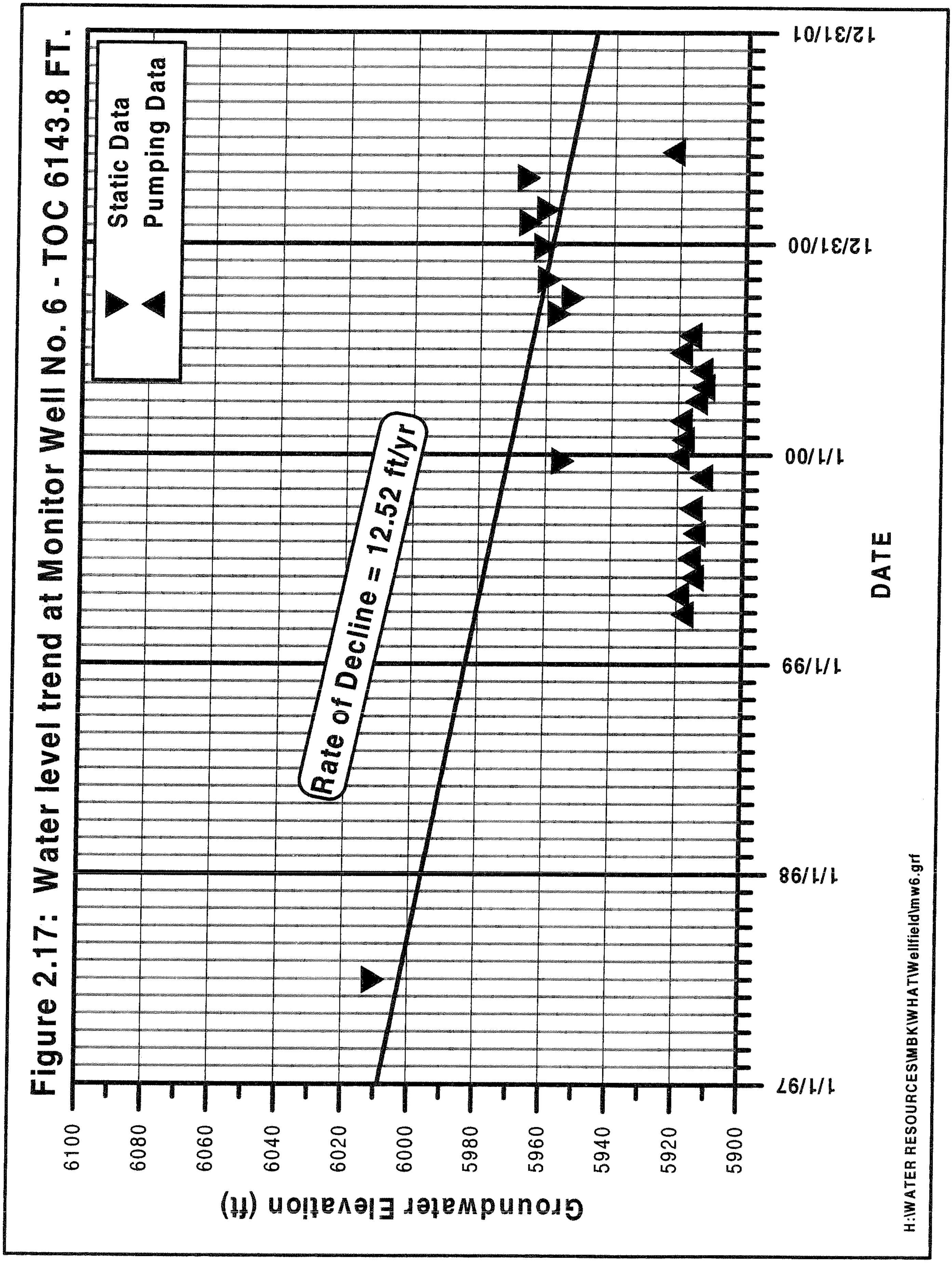


Figure 2.17: Water level trend at Monitor Well No. 6 - TOC 6143.8 FT.

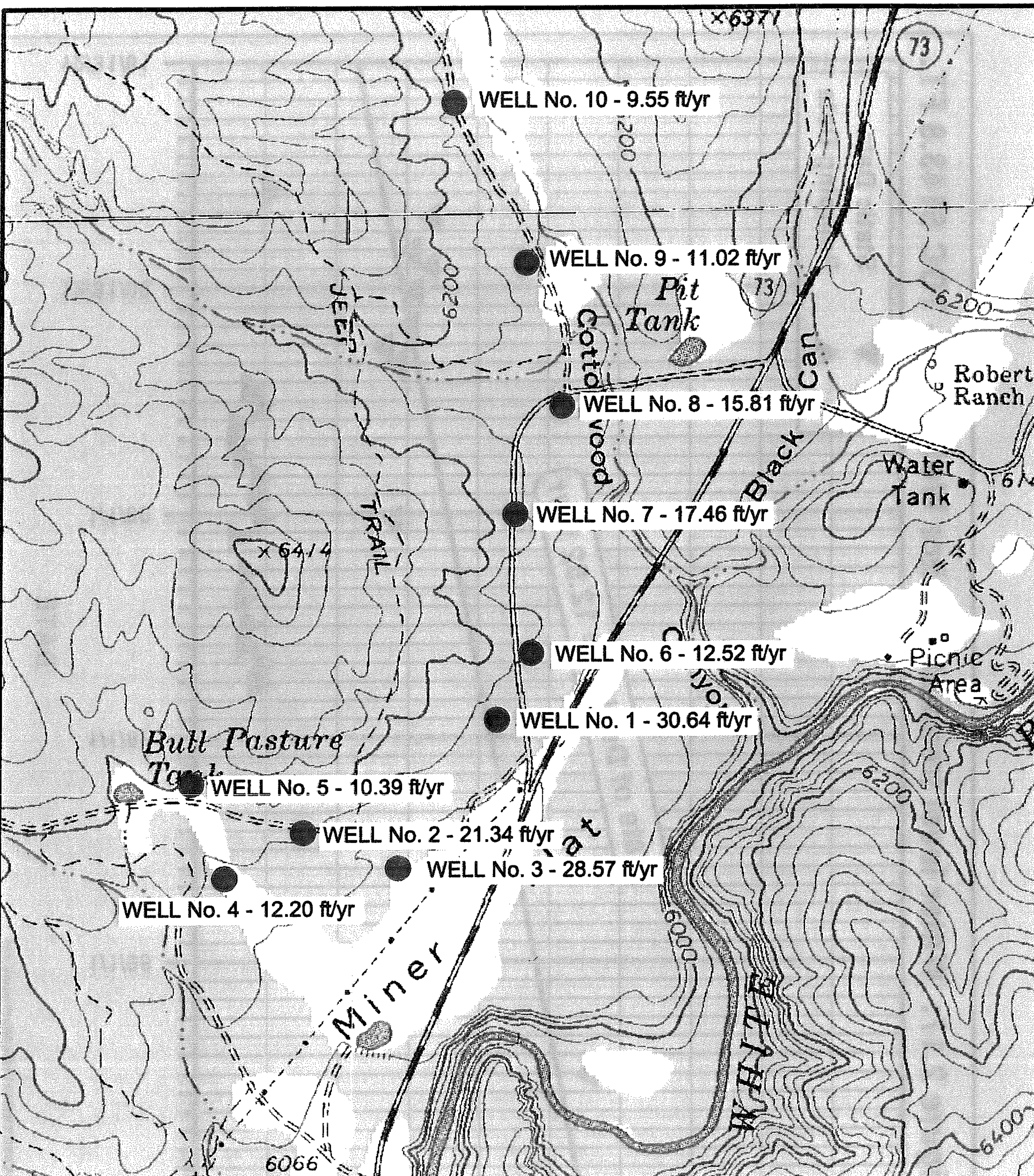


FIGURE 2.18: Rate of groundwater level decline at well sites.

LEGEND

● WELL No. 5 - PRODUCTION WELL LOCATION

10.29 NUMBER INDICATES RATE OF DECLINE OF GROUNDWATER IN FEET PER YEAR

N

1000 0 1000

SCALE IN FEET

H:\WATER RESOURCES\1780 wmat\miner flat dam\MINER FLAT FIG 2.18.dwg Apr/09/2002

2.2.3. Well Screen Conditions

Chapter 3 of this report provides considerable discussion of analytical methods used to evaluate the hydraulic efficiency of well screens and associated filter packs to determine if mineral incrustation on the well screens or plugging of the filter pack is a significant factor in the loss of yield at the Miner Flat Wellfield. Strictly applied, the analytical methodology for measuring well loss does not work on the wells in the Miner Flat Wellfield. This is because dewatering of the aquifer thickness causes a loss of specific capacity and an increase in specific drawdown that cannot be distinguished from the same effects caused by loss of hydraulic performance in the screen and filter pack.

Chemical analyses of the groundwater from the Miner Flat Wellfield show dissolved iron in concentrations sufficient to cause precipitation of iron oxide when the water is aerated or oxidized. Pumping equipment removed from several of the wells where water is cascading down the well from well screens exposed above the pumping water levels exhibit significant incrustations of iron oxide deposits. It is possible that iron oxide deposited by the cascading water might affect the performance of the well screens. However; downhole video logs of Wells No. 2 and 8, taken after the December 2001 investigations, did not reveal any significant plugging of the wells screens (Moeller, 2002). The video logs revealed deposits of iron oxide on the screens above the pumping water level, but not in amounts sufficient to cause a loss of production at this time, and generally above the depths where most of the water now enters the wells since the groundwater levels have declined.

2.2.4. Pump and Motor Conditions

One of the goals of the December 2001 investigation was to evaluate the performance of the pumps and motors in the wells. The ultimate goal of this type of evaluation is to determine the wire-to-water efficiency of the pumping system. In order to complete this type of analysis, it is necessary to record kilowatt hour use corresponding to delivery of a known volume of water. It was known at the onset of the investigations that power meters were not present on the different pumps so it would not be possible to determine the prevailing wire-to-water efficiency. However, it was perceived that observation of the current draw at each of the well pumps under different loads during a stepped rate test would provide some knowledge about the system conditions. Unfortunately, it was not possible to conduct stepped rate tests at different discharge rates, so the current draw under different loads was not determined. However, the current draw associated with specific pumps while they discharged water into the distribution system was observed and provides some useful information.

Table 2.5 shows the full load amperage published by the manufacturer for the submersible pump motors. Table 2.5 also shows the operating amperage observed during operation of the pumps for comparison to the published full load amperage limit. The values shown are the average of current readings on all three legs of the three-phase power to each pump and were recorded directly from the pump telemetry control panel in the treatment building at the wellfield. Independent readings of current draw at the panel at each well site were not made.

Table 2.5: 12/01/01 average operating current at Miner Flat Wellfield wells.

Well No.	Normal Amperage	Full Load Amperage	Observed Amperage	Percent Manufacture's Full Load Amperage
1	47	53.5	52.0	83.9
3	47	53.5	50.6	81.6
4	47	33.5	27.2	72.5
5	47	33.5	27.4	73.1
6	47	53.5	48.5	78.2
7	47	53.5	48.8	78.7
8	47	53.5	47.0	75.8
9	47	53.5	58.0	93.5
10	47	33.5	31.6	84.3

The "normal" and "full load" amperages were recorded from the display on the telemetry control panel; however, the basis for those values is not known. The maximum amperage ratings for 4-inch diameter 25 and 40 horsepower submersible motors, operating on 460-volt power, are 37.5 and 62.0 amperes, respectively, as published by the supplier of the pumps. Accordingly, the full load amperage recorded from the telemetry control panel and shown on Table 2.5 is evidently 89 percent of full load for the 25-horsepower motor and 86 percent of full load for the 40-horsepower motor.

The electrical current draw by an electrical motor depends on the work being done by the motor. The amperage requirement increases as the amount of work increases. Therefore, a submersible motor connected to a pump with a nominal capacity of 350 gpm and rated to draw 47 amps at 350 gpm will draw less current at pumping rates less than 350 gpm. This concept is useful in interpreting the information shown on Table 2.5. For example, motors in Wells No. 1 and 3 are operating at near full load amperage despite somewhat diminished yield. The current draw for Wells No. 1 and 3 is not unreasonable considering the increase in pumping lift associated with the decrease in discharge rate and allowing for some increase in mechanical resistance due to wear in the pumps.

Wells No. 6 and 7 are somewhat similar to Wells No. 1 and 3 but with considerably less discharge rate. Accordingly, the observed operating amperage appears somewhat high in these wells, suggesting the possibility that the pumps may in fact be producing more water than is reaching the flow meter, with some of the water circulating back into the wells through holes in the pumps or pump column pipes.

Wells No. 4 and 5 both exhibit current loads consistent with their reduced discharge rates. Well No. 8 produces very little water, a fact that suggests most of current load is needed to turn the badly damaged pump. Operation of Well No. 8 on automatic generally results in an overload fault that stops the pump. Wells No. 9 and 10 exhibit current loads reasonably consistent with their production rates.

Chapter 3 provides detailed analyses of pump performance, comparing the 12/01/01 performance of the pumps to their published pump performance curves. Table 2.6 shows the loss of pumping capacity as changed from the rated capacities under the prevailing head and drawdown conditions during the 1997 baseline tests. Because the losses are based on the pump performance curves and 1997 water levels, they do not match the nominal design yields assigned by I.H.S. and lost capacity shown on Table 2.1. Where data are available to support the analysis, Table 2.6 shows the loss of pumping capacity separated into two components – loss due to damage to the pumps and loss due to increased pumping lift caused by the decline in groundwater levels.

Table 2.6: Summary of factors reducing individual pump capacities.

Well No.	Pump Yield Loss Due to Increased Lift (gpm)	Pump Yield Loss Due to Damage to Pump (gpm)	Total Lost Pumping Capacity* (gpm)
1	55	91	146
3	104	26	130
4	58	54	112
5	---	---	85
6	73	115	188
7	0	187	187
8	---	---	246
9	18	94	112
10	---	---	53

* Based on rated pump capacity at 1997 total dynamic head conditions. These values are different than the design yields in Table 2.1 which are the nominal design yields assigned by the I.H.S. engineers.

Table 2.6 is provided only to demonstrate the effect of cavitation damage on the pumps in the Miner Flat Wellfield. Comparison of the lost pumping capacity on this table to the loss of well yield shown on Table 2.1 is comparing apples to oranges because the rated pump capacities at 1997 water levels exceeded the I.H.S. design yields in Wells No. 7 and 9 and was less than the I.H.S. design yield in Well No. 4.

2.3. Future Considerations

The foregoing conditions identified at the Miner Flat Wellfield indicate that groundwater levels at the wellfield are likely to continue declining in the future, due to operation of the wellfield. The continued decline of the groundwater levels will result in continued loss of capacity at the wellfield, with the collective production from the 10 wells becoming less in the future than in December 2001. Future loss of capacity at the wellfield, combined with future growth and increased demand for water in the Whiteriver service area will result in shortages in water supplies in the foreseeable future.

Some steps are already underway to offset these problems. One step is to resume use of an estimated 350-400 gpm water supply from the Columbine Spring. The Columbine Spring was used as a source of water supply in the past, but use was temporarily suspended until problems with deteriorated transmission line and treatment issues were resolved.

Another step is plans by the I.H.S. to drill at least two additional wells in the Miner Flat Wellfield in the spring of 2002. This step will provide welcome short-term relief for the loss of production in existing wells and will add backup pumping capability to the system. However, it must be recognized that the new wells will be subject to the same limitations as the existing wells, namely the lack of recharge to support the abstraction rates at the wellfield. Accordingly, groundwater levels at the new wells will suffer a long-term permanent decline just as have groundwater levels at the existing wells.

Recognition that the wellfield is mining water from the aquifer and that groundwater levels will decline in the future should not discourage continued use and development of the wellfield, at least as an interim step until a more permanent source of water can be developed. However, continued use of the wellfield, including expansion of the number of wells in order to sustain necessary productivity, should be accomplished in conjunction with plans to either develop alternative water supplies, such as surface water from the North Fork of the White River, or to restore the wellfield function by providing artificial recharge to the groundwater system.

2.3.1. Wellfield Expansion and Reduced Pumping Rates

In the absence of an alternative water supply that can be readily implemented, it will be necessary to depend on the Miner Flat Wellfield for water supply for a number of years. Therefore, it will be necessary to take steps to ensure the wellfield produces an adequate supply until alternatives can be implemented.

Although declining groundwater levels will continue to reduce the yields of individual wells in the wellfield, pumping of the wells at reduced rates will allow them to remain in operation for a considerable time, even as groundwater levels continue to decline. Operation of the wells at reduced pumping rates will generate less momentary drawdown during pumping, thus allowing the wells to continue to function, despite the lower groundwater levels. Due to the initial design of the wells, cascading water and damage to pumps may be a problem requiring greater than normal maintenance and replacement costs. The role of iron oxide incrustation under such an operation cannot be predicted.

Operation of the existing wells at 50 to 75 percent of presently delivered pumping rates will not satisfy current demands placed on the wellfield for water, let alone provide for future expansion. Therefore, reducing the pumping rates will require installation of more wells so that a collective production of 1,200 to 1,300 gpm is maintained at present and can be reconsidered in the future.

Reduced pumping rates for existing wells are shown on Table 2.7. The rates recommended on Table 2.7 are adjusted downward from the observed 12/01/01 pumping rates to account for continued decline of groundwater levels in the aquifer and anticipated loss of well yield. The pumping rates recommended on Table 2.7 are a compromise between obtaining the maximum amount of water possible from each well and a pumping rate that will extend the operational life of the well as groundwater levels continue to decline. Reduction of the present pumping rates at each well will also have the effect of slowing the decline of groundwater levels and the dewatering of the aquifer.

The collective production of the recommended pumping rates is 1,120 gpm which is less than the present demands of 1,250 to 1,300 gpm. However, addition of the Columbine Spring water source and construction of two additional wells in the wellfield, as currently planned, will more than enable the reduced pumping rates at the existing wells to satisfy the current demand, thus prolonging the useful life of the Miner Flat Wellfield.

Table 2.7: Recommended reduced pumping rates for existing wells in expanded wellfield operation.

Well No.	Original Design Rate (gpm)	Observed 12/01/01 Rate (gpm)	Reduced Rate for Long-Term Operation (gpm)	Recommended Pump Inlet Depth (ft BTOC)
1	350	204	100	299
2	225	---	100	---
3	350	220	100	320
4	225	88	50	299
5	225	140	50	320
6	350	162	120	318
7	350	188	140	318
8	350	104	100	340
9	350	338	250	278
10	200	147	110	362
Totals	**2,975**	**1,591**	**1,120**	

2.3.2. Geologic Mapping and Exploration Drilling

Addition of more wells, each to be pumped at much lower rates than the initial rates used when the wellfield was put into operation in January 1998, requires a better knowledge of the geology and hydrology of the local portion of the Coconino Aquifer System than is presently available. The present knowledge of the local aquifer system does not include knowledge of its distribution and extent, its hydraulic gradient and flow, or the boundaries of the system. It is strongly recommended that geologic mapping be accomplished to identify, if possible, the geologic conditions determining the boundaries of the aquifer system developed by the wellfield.

It is already known from the work accomplished in the early and mid 1990s that the eastern side of the aquifer is bounded by the basalt along the North Fork of the White River. There is preliminary information suggesting the western side of the aquifer may be bounded by a structural change in the Coconino and Supai strata. Existing geologic mapping has identified a fault, located north of the Lower Log Road crossing of the White River, which may extend west-northwesterly into the headwaters of Cottonwood Canyon or First and Second Hollows at the head of Big Canyon in East Cedar Creek, forming a northern boundary to the aquifer. The aquifer system is bounded to the south where the Coconino and Supai strata are truncated by the terrain; however, this boundary has not been mapped.

It is therefore recommended that drilling of additional wells to extend the life of the Miner Flat Dam Wellfield and to allow prudent management of the groundwater abstractions be preceded by geologic mapping. The geologic mapping should provide the basis for drilling of some additional exploratory boreholes to supplement and verify the geologic mapping and to provide information about the depth to groundwater, elevation of the potentiometric surface in the aquifer, and definition of the hydraulic gradient and direction of groundwater flow in the aquifer.

The geologic mapping effort should encompass at least the entire extent of the Cottonwood Canyon up to the southern edge of the basalt and cinder deposits to the north. The mapping should extend into the First, Second, Third, and Fourth Hollow areas and down Big Canyon to encompass Deer Spring Creek, the Little Round Top Mountain mass, and Post Office Canyon as necessary to search for a fault boundary to the north, to search for a structural boundary along the west side of the aquifer in the headwaters of Cottonwood Canyon and Big Canyon, and to define the southern boundary of the aquifer where it is truncated by the terrain.

The mapping and exploration drilling should provide accurate elevations on the upper and lower contacts of the strata so that the structure can be accurately depicted on geologic cross sections. The mapping effort should include a visual inspection of the watershed areas to identify potential recharge areas and to identify any discharge of water through springs that might be the discharge area for that portion of the aquifer developed by the Miner Flat Wellfield.

It is anticipated that the recommended geologic mapping, followed by associated exploratory drilling, will provide the basis for decisions about where to site additional wells to prolong the life of the wellfield and satisfy demands for water in the Whiteriver service area, while alternatives to the groundwater source are studied and developed.

2.3.3. North Fork of the White River

The North Fork of the White River is the only perennial surface flow upstream from Whiteriver. The flow in the North Fork is subject to enough seasonal variation in flow that it is not sufficiently reliable to support direct diversion for use as a public water supply without regulation. Recognizing the need for regulation and storage to provide a reliable water supply to the region around Whiteriver, the White Mountain Apache Tribe has proposed a dam on the North Fork of the White River in the vicinity of the Miner Flat. Implementation of this project would provide the reliability required of the surface water flow in the form of storage.

A schedule for implementation of the Miner Flat Dam project has not been established and numerous environmental and regulatory issues remain to be addressed before final design of the project can begin. In the interim, the communities in the Whiteriver area are dependent on a water supply from a wellfield that is running out of water and which will be insufficient for present and future needs within a foreseeable amount of time.

2.3.4. Artificial Recharge

In addition to reducing pumping rates from individual wells and adding more wells to the wellfield to spread the depletion of the groundwater resource over a larger area, thus extending the life of the wellfield, artificial recharge of the aquifer in the Coconino and Supai strata may be one alternative to provide a reliable source of water. As previously noted, it is necessary to provide storage of water in the North Fork of the White River to obtain the reliability of source necessary for a community water supply system. One alternative to a surface water impoundment on the North Fork of the White River, at least to the limited extent necessary to provide municipal and industrial water supply, may be to divert water from the North Fork and recharge it into the Coconino Aquifer System to support the Miner Flat Wellfield. Artificial recharge of the aquifer would use the groundwater system for storage in lieu of a surface water reservoir.

Artificial recharge is accomplished by collecting or diverting surface water and infiltrating or injecting it into the aquifer to supplement natural recharge. Typical methods have included water spreading, recharge basins, and injection wells. Water spreading and recharge basin methods are most applicable to unconsolidated alluvial aquifers which offer favorable surface infiltration rates. Bedrock aquifers such as the Coconino and Supai sandstone strata comprising the aquifer system at the Miner Flat Wellfield offer much lower infiltration capacity than alluvial materials and probably do not present favorable conditions for water spreading or recharge basins. Recharge basins typically offer initially high infiltration capacity which decreases with time due to clogging of the surface materials with sediments. This fact, combined with the intrinsically low infiltration capacity of the bedrock strata suggests recharge basins are not a realistic approach to providing recharge to the aquifer system at Miner Flat. This conclusion is reinforced by the fact much of the Coconino Aquifer System at the wellfield is a confined aquifer. Even if artificial recharge were introduced through recharge basins located over unconfined portions of the aquifer, they would have limited ability to restore groundwater levels to the confined aquifer pressures that prevailed prior to implementation of the Miner Flat Wellfield.

Another alternative for artificial recharge is injection of water into the aquifer through wells. Injection wells (recharge wells) offer the ability to manage a confined aquifer by using pumps to inject water into the aquifer under pressure. In this type of scheme, water from the North Fork would be diverted, cleaned of sediment, and pumped into the aquifer during periods of availability and stored in the aquifer for later abstraction through the existing wellfield or an expanded version of the wellfield. The injection wells used for recharge would be similar to the present water supply wells except that they would be constructed to inject water into the aquifer under pressure.

Injection wells are not trouble free. Their largest problem is that they are sensitive to clogging. The most likely potential causes of clogging of sandstone aquifers like the Coconino and Supai sandstones include the following factors:

1. Air entrainment caused by water allowed to cascade into the well,

2. Suspended sediment and/or organic matter in the water,

3. Formation of biofilm due to microbial growth in the well or aquifer, including growth of iron bacteria,

4. Reactions between the recharge water and the groundwater resulting in formation of precipitates in the interstices of the aquifer, and

5. Precipitation of iron from the groundwater due to reaction with the recharge water.

Most of the foregoing problems are solved by proper well design, removal of suspended sediment and organic matter from the recharge water by treatment, and scheduled maintenance of the wells to remove biofilm and suppress iron bacteria growth. The aquifer at the Miner Flat Wellfield appears to be susceptible to the problem of precipitation of iron from the groundwater. A solution to this problem may require some research but several possibilities to deal with the problem may exist including pH control and control of dissolved oxygen in the recharge water until recharge flushes enough of the native groundwater from the recharge/wellfield area to mitigate the potential for iron precipitation.

3. INDIVIDUAL WELL PERFORMANCE

Each of the wells in the Miner Flat Wellfield exhibit a significant decrease in the well yield provided by the pumps originally selected for each well. Baseline yield and drawdown tests were the basis for pump selection for most of the wells. Comparison of yield and drawdown conditions in December 2001 to baseline yield and drawdown conditions indicates the decrease in yield is due to at least two factors. One factor is a decline in the groundwater levels since the wells were put into production. The decline in groundwater levels has increased the pumping lift at each well, resulting in a commensurate decrease in pumping production as the lift at each well increased. The second factor is pump wear and/or damage. Comparison of pump yields at December 2001 pumping water levels to the pump performance curves indicates degraded pump performance.

A third factor affecting well yield might be loss of well hydraulic performance. The December 2001 investigations did not prove or disprove loss of hydraulic performance. A determination of the hydraulic performance of the wells under currently prevailing conditions in the wellfield would necessarily require completion of new yield and drawdown tests, specifically stepped rate tests of the wells. It was not possible to conduct stepped rate tests of the wells in December 2001 with the existing pumping equipment. If step tests had been conducted in December 2001 and if the step tests had indicated degraded hydraulic performance in the wells, they would not have indicated the cause of degraded hydraulic performance with any specificity. This is because the pumping water levels in the wells are now below at least a portion of the well screens in most of the wells.

The change in well hydraulics that occurs when pumping water levels decline below a well screen results from increased head loss due to turbulent flow through the well screen or, if the flow through the screen ceases as the water level in the aquifer drops below the screen, the decrease in open area providing water into the well results in a degradation of hydraulic performance. Therefore, there is no question that hydraulic performance of the Miner Flat Wellfield wells has decreased in each case where the pumping water level has declined below a portion of the well screens, due to increased well loss.

However, there are two other factors observed at the Miner Flat Wellfield during the December 2001 investigations which affect the hydraulic performance of the wells. One factor is the dewatering of the saturated thickness of the aquifer. The decrease in the saturated thickness of the aquifer since the wellfield was put into production is undoubtedly the single largest factor causing the decrease in the yield of the wells. This is because the yield of the wells depends on the transmissivity of the aquifer penetrated by the wells. Decreased saturated thickness causes decreased aquifer transmissivity, resulting in a decrease in well specific capacity. Specific capacity is the gallons per minute per foot of drawdown in the well. Therefore, if new step tests of the wells were performed in December 2001, they would have shown a marked decrease in the specific capacity of the wells simply because of the decrease in saturated thickness of the aquifer. However, such tests would still have identified well loss separately from loss of specific capacity and that well loss could have been compared to the baseline well loss.

As previously concluded, the well loss in December 2001 would necessarily have increased compared to the baseline tests due to dewatering of the well screens. However, step tests in December 2001 would not have distinguished well loss due screen dewatering from well loss due to mineral incrustation on the wells screens. Thick deposits of iron oxides were observed on pumping equipment during the December 2001 investigations. This is consistent with chemical analyses of the groundwater at the wellfield which indicate a propensity to precipitate iron and with dewatering of screened intervals which promotes aeration of water in the wells with attendant oxidation and precipitation of the dissolved iron concentration in the water. Inspection of pump column pipes removed from several of the wells revealed corrosion pits diagnostic of iron bacteria growth under the iron oxide coatings on the pipes. All of these factors taken into consideration, it is very likely that there has been a build up of iron oxide incrustation on the dewatered portions of the wells screens.

However, it is not known to what extent, if any, that incrustation has affected the hydraulic performance of the wells. Performance of new stepped rate tests of the well hydraulics would probably not provide a distinction between well loss caused by screen dewatering versus well loss caused by incrustation. It is clear, however, that the most important cause of decreased well productivity is the decrease in saturated thickness of the aquifer caused by a decline in groundwater levels during the history of the wellfield. The decline in groundwater levels is sufficient reason to explain all of the loss of well capacity without invoking loss of hydraulic capacity due to dewatering of screens or plugging of screens with mineral incrustation. Accordingly, incrustation on the well screens is probably not a significant factor in the loss of well yields in the wellfield, although incrustation is likely present in the form of iron oxides.

3.1. Well No. 1

Well No. 1 was completed in October, 1995 and the well was put into service in November, 1995. The well provided essentially trouble-free operation until September 2001 when lightening damaged the pump motor and controls (Frankie Williams, 2001). In November 2001, the pump controls were repaired and the motor and two lowermost sections of pump column were replaced. The following paragraphs provide information pertinent to the aquifer, the well screen and casing, and the pump and motor.

3.1.1. Geologic Log

Table 3.1 summarizes the geologic information logged during drilling of the well. The geologic information was not logged by a geologist, but is consistent with subsurface data from nearby wells logged by a geologist and is thought to be an accurate representation of the materials penetrated by Well No. 1.

Table 3.1: Production Well No. 1 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 50	CLAY, brown MALPAIS*, gray	Colluvium Basalt flow
50 - 85	SANDSTONE, red/tan	Coconino Sandstone
85 -130	CLAY, red w/gravel	Surface of upper member of Supai
130 - 140	SANDSTONE, red, water-bearing	Upper member of Supai
140 – 320	CLAY w/gypsum	Second member from top of Supai
320 - 350		

**malpais is derived from the Spanish "mal pais" for "bad lands" and was used by early southwestern explorers to describe land associated with lava flows which offers rough terrain as well as a harsh surface for unshod horse hooves and feet clad in moccasins. The term is still used by Anglos who often corrupt the term to the phonetic pronunciation "mala-pie".*

The red and tan sandstone from 85-130 feet is interpreted to most likely consist of Coconino Sandstone and the water-bearing red sandstone from 140-320 feet is interpreted as the uppermost unit of the Supai Group. The 130-140 foot interval, described as "red clay with gravel", is present in the southern half of the well field and, when logged by a professional geologist in nearby wells, is described as a sequence of laminated of silt and clay containing traces of lithic debris, primarily yellow sandstone fragments and a few chert chips and fragments of calcareous fracture fillings. The laminated silt and clay unit is considered to be the uppermost part of the Supai Group at the Miner Flat Wellfield area.

3.1.2. Construction Data

The well was reportedly drilled to a total depth of 350 feet and completed with 10-inch diameter steel casing to depth of 150 feet;10-inch diameter pipe sized, 20-slot stainless steel well screen from 150 to 320 feet; and Colorado Silica 10-20 filter pack was installed around the well screen and to an unreported height above the well screen and the remainder of the annulus sealed with neat cement grout to the bottom of the Baker-Monitor pitless unit.

Table 3.2 provides completion data reported at the time the well was put into service. The tank overflow elevation is for the storage reservoir that receives water from all of the wells in the Miner Flat Wellfield. The pumping water level of 225 feet is the assumed design pumping water level used in conjunction with the tank overflow elevation to obtain a nominal total dynamic head of 337 feet for sizing of the pump and motor for the well. Pump column and transmission line friction losses were assumed to be negligible for the purpose of pump and motor selection.

The depth to the well screens shown on Table 3.2 is depth below ground level (BGL) whereas the static water level and depth to the pump intake are shown as depth below top of casing (BTOC). Well casing height above land surface is about 2 feet. No attempt is made in this report to normalize all of the data to one reference level such as BTOC because the resulting values reported herein would differ from the records used to compile these data and therefore obscure the link between this report and the various original sources of records for these statistics.

Table 3.2: Production Well No. 1 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6146
Tank overflow		6258
Static water level (swl)	72	6074
Top of well screens (BGL)	150	5996
Bottom of well screens (BGL)	320	5826
Pumping water level (pwl)	225	5921
Intake depth	278	5868
Drop pipe length	273	
Total cased depth	350	
Nominal pump capacity (gpm)	350	
Pump horsepower	40	

The zone from 140 to 320 feet identified as "water-bearing" on the geologic log is contained entirely within the strata interpreted to be Supai Group; however, the static water level of 72 feet on Table 3.2 is above the base of the basalt layer that confines the top of the Coconino Sandstone. This indicates the groundwater in the Supai sandstone (and Coconino Sandstone) was under confined conditions at the time the well was put into service. It was not possible to measure the static water level in the well during the December 2001 investigations; however, the decline in the pumping water level since the well was put into service in November 1995 suggests the groundwater may no longer be under confined conditions at this well.

3.1.3. Water Levels

The static water level of 72 feet shown in Table 3.2 is for an unknown reporting date assumed to be essentially at the time the well was completed. A current static water level was not determined during the December 2001 investigations because the well was in operation nearly the entire time of the investigations, not allowing time for recovery to static conditions; however a pumping water level was measured. It is presumed that the static water level at Well No. 1 has declined similarly to the static water levels in all the other wells in the wellfield.

The records do not indicate that a baseline pumping test of yield and drawdown performance was conducted at Well No. 1. Accordingly, the original pumping water level at the well is unknown. However, it is clear from the design parameters set forth in Table 3.2 that a maximum depth of 225 feet to the pumping water level was intended. On 12/01/01 at 1610 hours, the pumping water level measured with the IHS hand-held electronic water level indicator was 264.2 feet (BTOC) or 39.2 feet greater than the maximum design lift for the pump installed in the well.

Because the original pumping water level is not known, this information does not indicate how much the static water level or pumping water level at Well No. 1 has declined since the well was put into operation. However, the 12/01/01 pumping water level of 264.2 feet does indicate at least a 39-foot decline in static water level, assuming no significant loss of well screen performance. The water level decline is probably considerably more than 39 feet considering that the design yield was 350 gpm and the well presently produces about 204 gpm, depending on the level of water in the storage reservoir. If these speculations are correct, the aquifer at the well has gone from a slightly confined condition, with the static water level above the base of the confining basalt beds, to unconfined conditions. This is a significant change in conditions in that unconfined conditions are subject to dewatering of the aquifer by drawdown associated with pumping and are therefore subject to the attendant reduction in aquifer transmissivity at the pumped well.

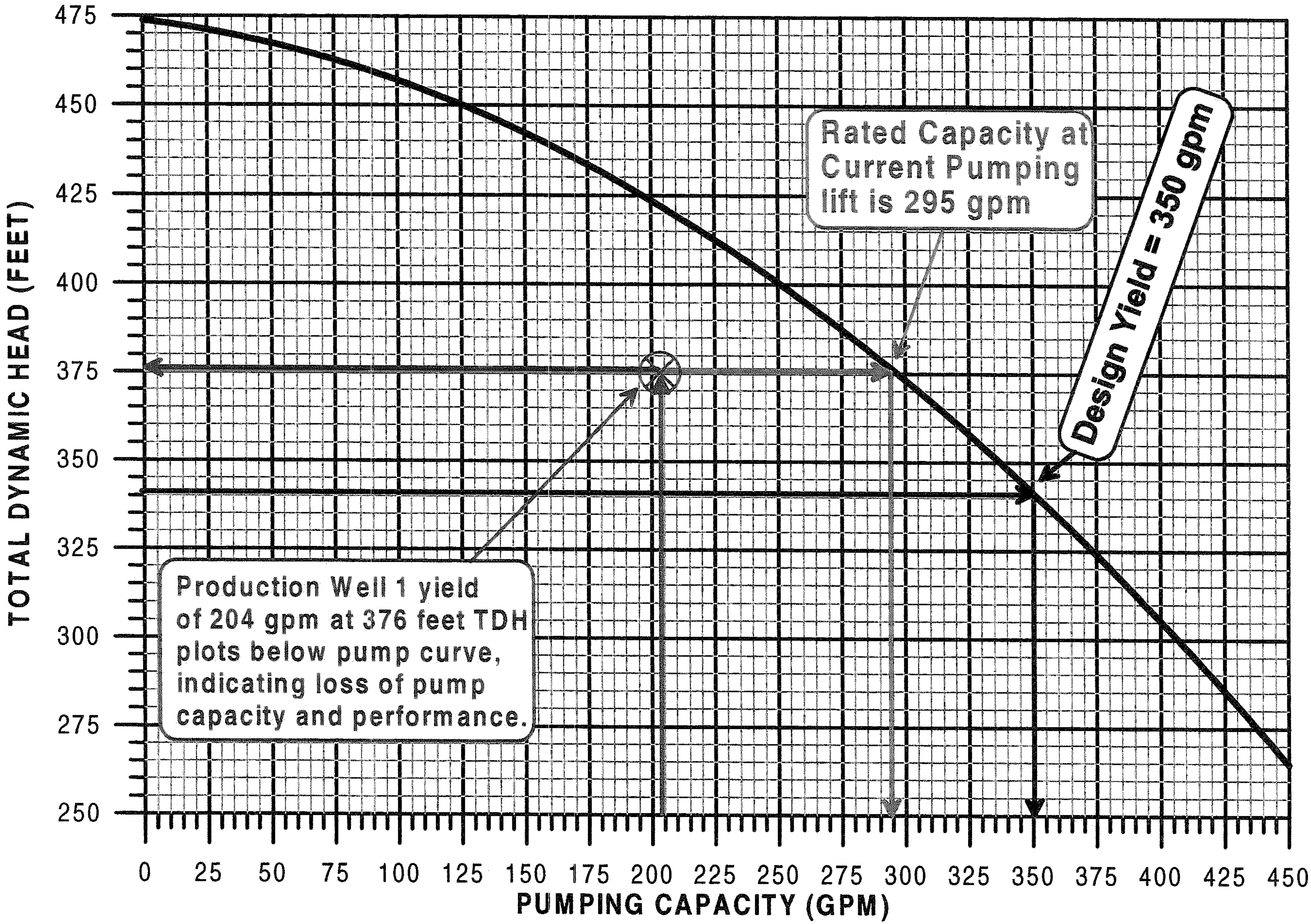
3.1.4. Well Yield and Pump Condition

Measurements of well yield conducted on 11/30/01 indicate an average yield of 204 gpm from Well No. 1 after the maximum pumping water level had been obtained. Accordingly, the 12/01/01 yield of the well was 146 gpm less than the design yield of 350 gpm shown on Table 3.2.

The design yield is based on the published pump performance curve for the Goulds 7CLC040 pump which indicates a 350 gpm discharge rate at a total dynamic head (TDH) of 337 feet (Figure 3.1). The basis for a 337 feet design TDH is the difference between the tank overflow elevation of 6258 feet and the design pumping water level of 225 feet or elevation 5921 (Table 3.2). The 12/01/01 pumping water level of 264.2 feet increases the TDH by 39 feet to 376 feet. The pump performance curve (Figure 3.1) indicates the pump is rated to provide approximately 295 gpm with a TDH of 376 feet. Therefore, the decline in the pumping water level at Well No. 1 has resulted in a decrease in well yield of 55 gpm due to the increased lift requirement imposed on the pump by the deeper pumping water level.

On 11/30/01, the discharge rate from Well No. 1 was measured at 204 gpm after the pumping water level in the well was reasonably stabilized at maximum depth. Although the 204-gpm discharge rate is subject to some fluctuation caused by the 10-foot fluctuation in head within the operating level of the storage reservoir, it is nominally 91 gpm less than the rated discharge of 295 gpm for the pump under the prevailing TDH of about 376 feet. This indicates a loss of 91 gpm of pumping capacity due to wear or damage on the existing pump.

Figure 3.1: Pump performance curve for Goulds 7CLC040 pump shows 91 gpm loss of capacity due to pump wear at Well No. 1.



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The decrease in pumping capacity of 55 gpm due to increased pumping lift plus the decrease in pumping capacity of 91 gpm due to wear on the pump is a total loss of pumping capacity of 146 gpm at Well No. 1 compared to the original design parameters.

3.1.5. Cause of Damage to Pump in Well No. 1

The probable cause of a 91-gpm loss of pumping capacity for the pump in Well No. 1 is damage to the pump caused by air in the water flowing through the pump. The measurements taken on 12/1/01 indicate the pump was necessarily breaking suction at some time in the past. The evidence for this conclusion is the specific capacity of the well compared to the rated capacity of the pump. For example, the elevations of the static water levels in nearby wells indicates the static water level in Well No. 1 was an estimated 206 feet BTOC. The 12/1/01 pumping water level of 264.2 feet therefore represents an estimated 58.2 feet of drawdown at the 204-gpm pumping rate. This provides a specific capacity of 3.51 gpm/ft (gallons per minute per foot of drawdown) as follows:

$$\frac{204\,\text{gpm}}{58.2\,\text{ft}} = 3.51\,\text{gpm/ft} \qquad \textbf{(Equation. 3.1)}$$

Therefore, pumping the well at the rated capacity of 295 gpm, divided by the specific capacity of 3.51 gpm/ft, results in an estimated drawdown of 84.2 feet or an increase of 26.2 feet in drawdown between the 204-gpm and 295-gpm pumping rates:

$$\frac{295\,\text{gpm}}{3.51\,\text{gpm/ft}} = 84.2\,\text{ft} \quad \text{and} \quad 84.2\,\text{ft} - 58\,\text{ft} = 26.2\,\text{ft} \quad \textbf{(Equations. 3.2 and 3.3)}$$

If an increase in the pumping rate from 204 gpm observed on 12/1/01 to the rated performance of 295 gpm would cause an estimated 26.2-feet increase in drawdown, the 12/1/01 observed pumping water level of 264.2 feet would increase to 290.4 feet. This is 12.4 feet deeper than the pump inlet depth of 278 feet. These data therefore indicate that when the pump was performing at its rated capacity, it necessarily began pumping a certain amount of air as the water level at Well No. 1 declined.

The resultant damage to the pump over a period of time resulted in the 91 gpm loss of pump capacity. The progressive loss of pump capacity due to cavitation eventually converged with the decline of the water level at the well to arrive at the 12/31/01 discharge rate of 204 gpm. The 204 gpm pumping rate represents an essentially self-regulating yield from the well that provides the minimum submergence of the pump inlet at the maximum rate the well can sustain under the prevailing groundwater levels and saturated thickness of the aquifer.

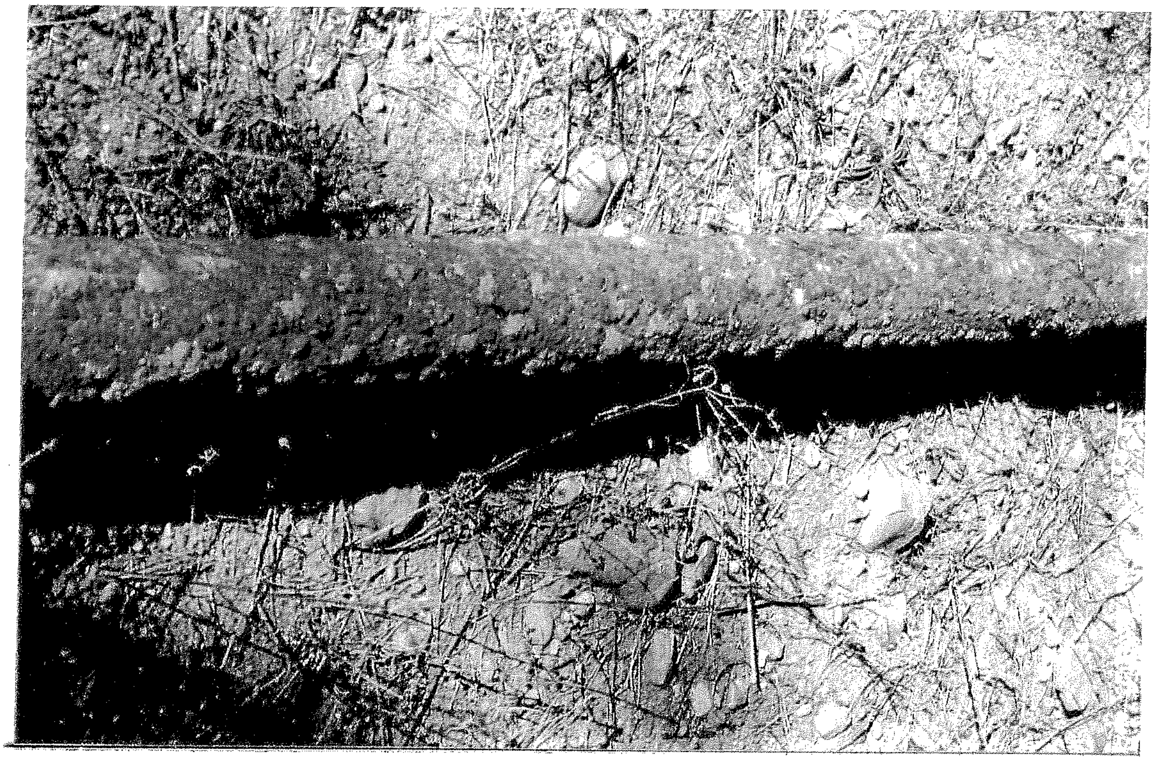
3.1.6. Cascading Water

The decline of the pumping water level until the pump began pumping air is not necessarily the only reason the pump in Well No. 1 pumped air until it was damaged. A second contributing factor may have been air entrained in water cascading into the well. The potential for cascading water is inherent in the well design. A pumping inlet depth of 278 feet associated with the top of the well screen at 150 feet (Table 3.2) gives the pump the opportunity to draw the pumping water level down below the top of the screen. If the groundwater level outside the well remains above the pumping water level in the well, water flowing through the portion of the screen above the pumping water level has the potential to entrain air. The portion of the well screen above the pumping water level is referred to as "dewatered" well screen. Air entrained in water cascading from dewatered screen may separate out of the water column before it enters the pump inlet, if the water column above the pump inlet is sufficiently long. However, a declining water level eventually reduces the height of the water column until entrained air does not separate and enters the pump, causing damage to the pump by cavitation.

Photographs 3.1 and 3.2 show the two pump column pipes from immediately above the pump in Well No. 1 which were replaced in November 2001. The thick incrustation of iron oxides and hydroxides on the pump column pipes reflect a highly aerated environment. The simple rise and fall of the water level in the well during pumping is unlikely to cause the type of incrustation present in the well. Most of the water samples collected from the Miner Flat Wellfield wells exhibit dissolved iron in concentrations that will cause precipitation of iron oxides when the water is exposed to air. Oxidized iron is insoluble and therefore precipitates. The type of incrustation observed in Well No. 1 is consistent with cascading water, oxidation of the dissolved iron by highly aerated water, and deposition of the iron oxide precipitates on the internal surfaces of the well.

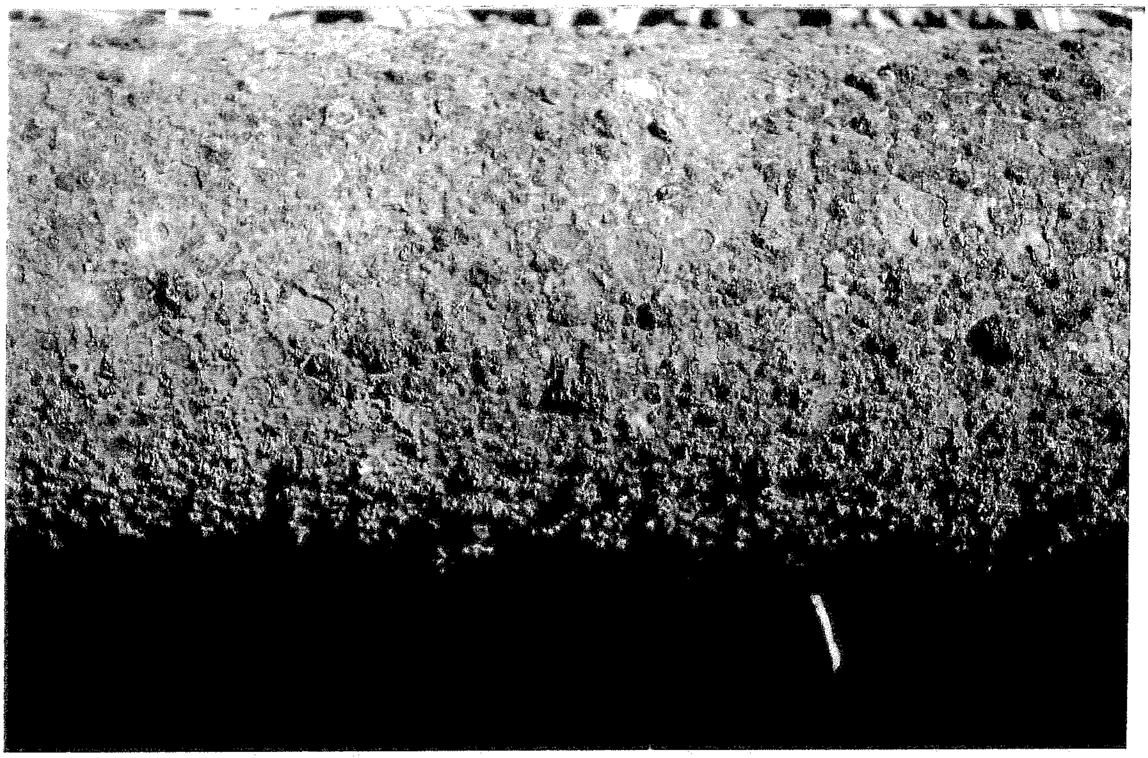
Photograph 3.3 shows pitting on the outside of the pump column in association with the iron oxide deposits. Photograph 3.4 shows deep pitting in an area of minimal encrustation where corrosion tubercles have formed on the pipe. Both photographs show the effect of the iron oxide deposits in providing a protected environment on the surface of the pipe where anaerobic bacteria can grow. The anaerobic iron bacteria cause transfer of ferric iron away from the pipe, causing corrosion pits in the pipe, and deposit the ferric iron as mineral crusts of ferrous iron and iron hydrates that form an armored tubercle around the microbiologic activity that is corroding the pipe. The rounded bumps and lumps in the iron oxide crust shown in Photograph 3.3 are such tubercles whereas Photograph 3.4 shows the pits corroded into the pump column under the tubercles.

The pits corroded into the pump column, and presumably into the well casing, are the direct result of deposits of iron oxide from cascading water providing an environment where a biofilm can develop to protect anaerobic microbiological growth that in turn corrodes the pump column and casing. In Well No. 1, the iron oxides are deposited on the well screen rather than on well casing, raising the issue of plugging of the well



Photograph 3.1: Iron oxide incrustation on Well No. 1 pump column.

Photograph 3.2: View of thickness of iron oxides on Well No. 1 pump column.



Photograph 3.3: Tubercles and corrosion pits on pump column, Well No. 1.

Photograph 3.4: Corrosion pits formed under tubercles.

screen by incrustation as well as the concern that corrosion may be attacking the stainless steel well screen. A mitigating factor is that the flow of water through productive parts of a well screen will generally prevent incrustation from totally blocking the screen; however, the flow may allow a certain amount of incrustation that results in a loss in hydraulic performance, even though an open area is maintained to accommodate prevailing flow through the well screen slots. Based on these considerations, it is recommended that Well No. 1 be logged with a down-hole video camera at some convenient time in the future to ascertain the extent of incrustation and plugging of the well screen by iron oxides and iron hydrates.

3.1.7. Recommended Pump Size

It is recommended that the pump in Well No. 1 be reduced to a pump limited to 100-gpm at 377 feet total dynamic head. The basis for this recommendation is discussed below.

The 12/1/01 pumping water level of approximately 264 feet at 204 gpm is only 14 feet above the pump inlet depth of 278 feet (Table 3.2). The pump inlet submergence of 14 feet is the result of progressive damage to the pump and loss of pump capacity until the minimum submergence required to satisfy the Net Positive Suction Head Requirement (NPSHR) of the pump at 204 gpm was established. Accordingly, it may be assumed that the 12/1/01 pumping water level of 264.2 feet represents the minimum NPSHR at 204 gpm. Therefore, installation of a replacement pump capable of the original design yield of 350 gpm, with the pump inlet at any reasonable depth, will result in the new pump breaking suction and becoming damaged, since the well now produces only 204 gpm under prevailing water level conditions.

The original recommendation for operation of the well was to provide a 350-gpm pump for peak pumping capacity, but to limit the average pumping duration to 12 hours per day or less. This is the equivalent of an average pumping rate of 175 gpm or less. In actual operation, demand for water has been such that the well has operated closer to 24 hours per day than the recommended pumping duration of 12-hours per day. The result was that the average pumping rate was closer to 350 gpm than to the recommended average rate of 175 gpm, until declining groundwater levels and damage to the pump resulted in a decrease in the pumping rate.

Taking into consideration the original average design yield of 175 gpm, the present yield of 204 gpm from 264.2 feet, and the rate of groundwater decline, it is recommended the pump in Well No. 1 be replaced with a pump sized to produce 100 gpm with a maximum pumping water level of 265 feet. A pumping water level of 265 feet is equivalent to an elevation of 5881 which provides a total dynamic head requirement of approximately 377 feet when compared to the tank overflow elevation of 6258. Pump column and transmission line loss is considered negligible within these general parameters.

The pump inlet for a 100-gpm pump should be set at a deeper setting than the present pump inlet depth of 278 feet in order to provide a larger margin for satisfying the pump submergence requirement (NPSHR). However, the pump motor should not be set below the bottom of the well screen unless a shroud is used to direct water around the motor to maintain required cooling of the motor. A nominal pump setting of 299 feet is recommended to increase pump inlet submergence while maintaining the pump motor above the bottom of the well screen at 320 feet and providing flow past the motor to the pump inlet. A pump inlet depth of 299 feet will require addition of one 21-foot joint of pump column to the existing pump column.

To summarize the recommendations above, it is recommended the existing pump in Well No. 1 be replaced with a pump capable of delivering 100 gpm with 377 feet of total dynamic head. It is likewise recommended the replacement pump be installed on 14 joints of pump column, each 21 feet long, for a total pump column length of 294 feet (one new joint plus the existing pump column) and a pump inlet setting of 299 feet (present pump column of 273 feet plus one 21-foot joint). Based on a specific capacity of 3.51 gpm/ft, the pumping water level at 100 gpm is estimated to be approximately 234.5 feet as follows, until affected by continued groundwater level decline:

$$\frac{\textbf{175 gpm}}{\textbf{3.51 gpm / ft}} = \textbf{28.5 ft of drawdown} \quad \textbf{(Equation. 3.4)}$$

$$\textbf{Pumping Water Level} = \textbf{28.5 ft drawdown} + \textbf{206 ft Static} = \textbf{234.5 ft} \left(\textbf{Equation. 3.5}\right)$$

The recommended pump inlet setting of 299 feet minus the estimated pumping water level of 234.5 feet provides an estimated 55-60 feet of pump inlet submergence at 12/1/01 water levels and a 100-gpm discharge rate. This is adequate to satisfy NPSHR at 100 gpm plus provide a safety margin for two years more of groundwater level decline at the well, assuming the current rate of 30.6 feet per year slows with less pumping.

3.1.8. Well No. 1 Alignment

Photograph 3.5 shows an imprint of the vertical rods in the well screen in Well No. 1 worn into the side of a check valve in the pump column. Presumably, the check valve was installed on top of the pump as is a common practice. If the check valve was not installed on top of the pump, it was on one of the first two joints above the pump when the two pipes were replaced in the pump column. The wear marks on the pump column check valve indicate the well is either not plumb (vertical) or is not straight or both. This might cause some problems when a replacement pump is installed, particularly if it is installed to a deeper setting as recommended.

The contact between the pump or pump column and the well screen may ultimately cause damage to the well screen. One solution to this problem may be to install a pump column centralizer or torque arrester on the pump column about 5 to 10 feet above the pump. However, the centralizing device must be constructed of synthetic material that is softer than the well screen so that it will not tend to wear a hole in the well screen.

Photograph 3.5: Well screen rods imprint on Well No. 1 pump column check valve.

3.2. Well No. 2

Well No. 2 was put into service in November 1995. Frankie Williams, Water System Operator, reports that the well has always pumped sand and therefore wears out pumps in a relatively short time. Therefore, use of the well since 1995 has been limited to periods of acute water shortage. At the time of the December, 2001 investigations, the pump had been removed from Well No. 2 for replacement with a new pump and installation of a Lycos sand separator. Accordingly, it was not possible to measure yield or drawdown in the well; however, the static water level was measured and examination of the pumping equipment provided some information about conditions in the well.

Photograph 3.6 shows the well head and the pump and pump column stacked beside the well. The stack of pipe includes 4-inch diameter pipe from the pump column presently in use and 6-inch pump column from a larger pump that was installed in the well when it was originally put into service. The large green pipe on the ground to the left of the pump column is the new Lycos sand separator which will be mounted in the well below the pump to remove sand from the water before it flows through the pump. The rusty pipe immediately to the right of the sand separator is a 10-foot piece of pump column attached to the used 4-inch pump and motor which are to be replaced.

Photograph 3.6: Pump column and sand separator stacked at Well No. 2.

Iron oxide encrustation on the pump column, pump, and motor from Well No. 2 is evident in Photograph 3.6. The iron oxide encrustation is similar to that observed at Well No. 2, provided on Table 3.3, indicates unconfined aquifer conditions at the well. Well No. 1 and is interpreted to be the result of dewatering of the well screen during pumping and aeration of water flowing out of the well screen above the pumping water level when the well is in operation.

3.2.1. Groundwater Level Decline

The static water level measured at 0830 hours on 11/28/01 was 201.35 feet BTOC as compared to 164 feet BTOC when the well was put into service in November 1995. Although the well had been out of service for nearly a month on 11/28/01, the static water level was more than 37 feet below the original static water level. The 37-foot decline in the groundwater level at Well No. 2 since November 1995 has taken place despite the fact the well has been used only for short periods of time since it was put in service in November 1995. The decline in the static water level at Well No. 2, where very little pumping has taken place, indicates a decline in groundwater levels throughout the wellfield, not just in the immediate vicinity of each pumped well

Table 3.3 summarizes the geologic information logged during drilling of the well. The drill cuttings were not logged by a geologist; however, the log is considered to provide a good representation of the major units penetrated by the well. The description of "gravel" associated with two of the sandstone layers is a non sequitur, yet nonetheless important in indicating relatively large pieces of fractured and broken sandstone fragments in the drill cuttings. The interpretation in this report that the log indicates the base of the Coconino Sandstone at 160 feet is based solely on the change to predominantly red sandstone at 160 feet and deeper.

Table 3.3: Production Well No. 2 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 10	CLAY, brown	Colluvium
	SANDSTONE, yellow/brown	Coconino Sandstone
10 - 30	SANDSTONE, light brown	Coconino Sandstone
30 - 40	SANDSTONE, hard red	Coconino Sandstone
40 - 110	SANDSTONE, light brown	Coconino Sandstone
110 - 140	SANDSTONE, yellow	Coconino Sandstone
140 - 160	SANDSTONE, red w/gravel	Supai
160 - 190	SANDSTONE, light red	Supai
190 - 220	SANDSTONE, red, water-bearing	Supai
220 - 370	SANDSTONE, red w/gravel	Supai
370 - 400	CLAY	Supai
400 - 410		

3.2.2. Construction Data

The well was reportedly drilled to a total depth of 410 feet and completed with 10-inch diameter steel casing to depth of 185 feet;10-inch diameter pipe sized, 20-slot stainless steel well screen from 185 to 335 feet; and Colorado Silica 10-20 filter pack was installed around the well screen and to an unreported height above the well screen and the remainder of the annulus sealed with neat cement grout to the bottom of the Baker-Monitor pitless unit.

Table 3.4 provides completion data reported at the time the well was put into service in November 1995, with the exception of the static water level of 164 feet which was measured on 12/5/97. The tank overflow elevation is for the storage reservoir that receives water from all of the wells in the Miner Flat Wellfield. The pumping water level of 200 feet is the assumed design pumping water level used in conjunction with the tank overflow elevation to obtain a nominal total dynamic head of 303 feet for sizing of the pump and motor for the well. Pump column and transmission line friction losses were assumed to be negligible for the purpose of pump and motor selection.

The depth to the well screens shown on Table 3.4 is depth below ground level (BGL) whereas the static water level and depth to the pump intake are shown as depth below top of casing (BTOC). Well casing height above land surface is about 2 feet. No

Table 3.4: Production Well No. 2 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6155
Tank overflow		6258
Static water level (swl)	164	5991
Top of well screens (BGL)	185	5970
Bottom of well screens (BGL)	335	5820
Pumping water level (pwl)	200	5955
Intake depth	300	5855
Drop pipe length	294	
Total cased depth	335	
Nominal pump capacity (gpm)	225	
Pump horsepower	25	

attempt is made in this report to normalize all of the data to one reference level such as BTOC because the resulting values reported herein would differ from the records used to compile these data and therefore obscure the link between this report and the various original sources of records for these statistics.

Comparison of the completion data on Table 3.4 to the geologic interpretation on Table 3.3 indicates that the saturated zone from the static water level to the bottom of the water-bearing materials was contained entirely within strata interpreted to be the red sandstone at the top of the Supai Group by 12/5/97 when the static water level was measured and two years after the well was put into production.

3.2.3. Well Performance Test

A stepped rate test of baseline hydraulic performance of Well No. 2 was conducted on 12/5/97, prior to replacement of the original 6-inch pump with a smaller 4-inch pump. The stepped rate test was the basis for the peak pumping design yield of 225 gpm shown in Table 3.4. Figure 3.2 shows a conventional Hantush-Bierschenk plot (Hantush, 1964 and Bierschenk, 1963) of the step test data for pumping rates of 200, 225, 250, 275, and 300 gpm. In conventional interpretation of the Hantush-Bierschenk plot, the slope of the line is equal to the well loss coefficient, C, and the well loss for any given pumping rate is the product of the well loss coefficient times the square of the discharge rate, as follows:

$$s_w = CQ^2 \qquad \textbf{(Equation 3.6)}$$

where s_w = well loss drawdown
C = non-linear well loss coefficient
and **Q** = discharge rate

Figure 3.2: Hantush-Bierschenk plot for Well No. 2 step test on 12/5/97.

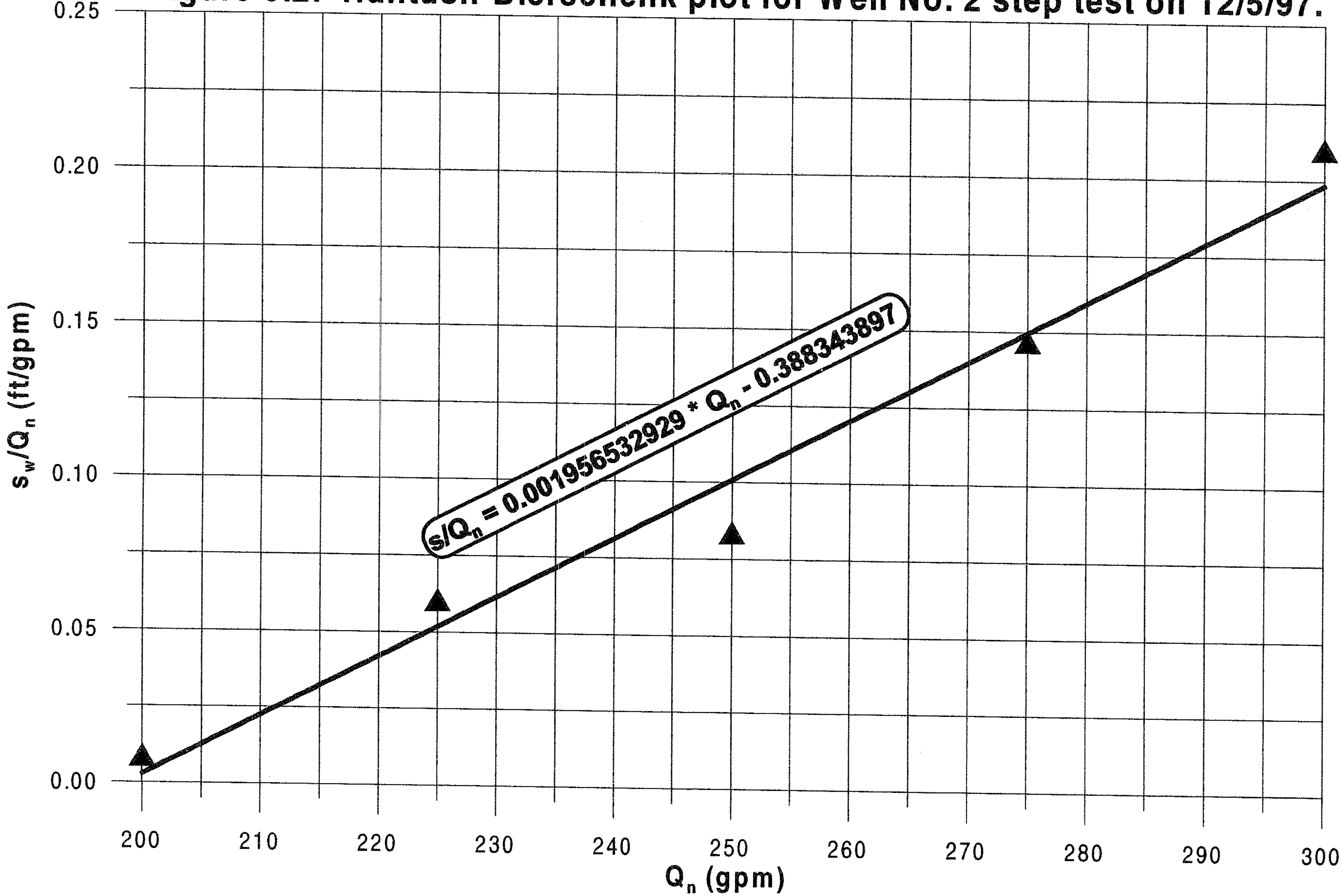


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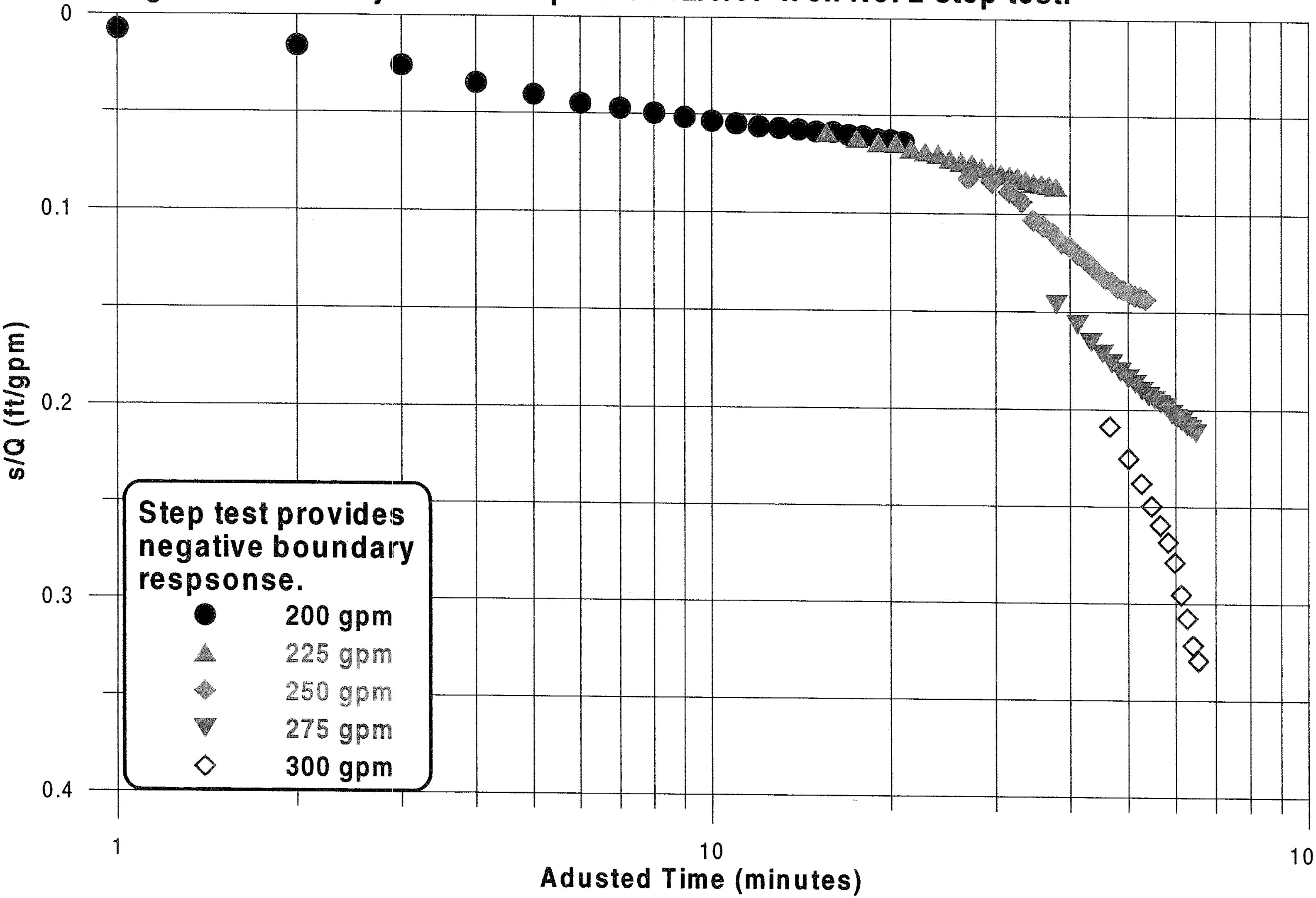
Figure 3.3: Birsoy-Summers plot for 12/5/97 Well No. 2 step test.



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From Figure 3.2, C = 0.001956532929; however, additional interpretation of the step test data indicates the results of the test will predict excessive well loss drawdown. The basis for this conclusion is the Birsoy-Summers (Birsoy and Summers, 1980) interpretation of the December 5, 1997 test data, as shown on Figure 3.3. The Birsoy-Summers plot exhibits progressive increase in specific drawdown in a manner that indicates a "negative boundary" response to pumping. A negative boundary response may be the result of the cone of depression encountering a zone of lesser transmissivity in the aquifer, a no-flow barrier, dewatering of an unconfined aquifer, or interference from a nearby pumped well. The presence of a negative boundary response in the 12/5/97 stepped rate test indicates that an unknown but significant amount of the well loss predicted by the well loss coefficient, C, is not well loss but is instead the effect on the drawdown rate caused by the boundary conditions. Accordingly, Equation 3.6 does not predict well loss, but instead predicts well loss plus boundary effects and well loss is unknown.

3.3. Well No. 3

Well No. 3 was put into service in November 1995. Frankie Williams, Water System Operator, reports that until the SCADA system became operational about the beginning of 1999, providing telemetry controls for the wells, Well No. 3 operated almost continuously on manual control. That condition continued to a large extent after the SCADA system was installed due to electrical interference problems with the hard-wired telemetry which necessitated operating the well on the manual control setting much of the time. Accordingly, Well No. 3 has been operated on an average duration close to 24 hours per day over much of its history. The submersible motor has been replaced three times in the history of the well, possibly due to the relatively high utilization of the well.

Frankie Williams reports that Well No. 3 has been reliable except that the electrical controls occasionally trip it off line, both on automatic and manual control settings. This indicates the motor is stopped by the Motor Saver™, not by the telemetry system. The Motor Saver™ is present to protect the motor from undercurrent, overcurrent, and unbalanced power conditions. Undercurrent can be due to the electrical power service delivered to the well or due to the pump breaking suction and taking the load off the motor (dry well condition). Accordingly, the occasional fault that stops the motor may be due to undercurrent from an excessively deep pumping water level in the well or due to transient conditions in the electrical service to the well site.

3.3.1. Geologic Log

Well No. 3 was the first well at the Miner Flat Wellfield completed with casing and screen for pumping. Although the well was not put into service until November 1995, it was completed 10/15/93 as a test hole with a total borehole depth of 640 feet. Based on the air lift discharge obtained from the uncased borehole during drilling, the well was completed to a total depth of 350 feet with 10-inch casing from the surface to 250 feet, 8-inch pipe-sized 20-slot stainless steel screen from 250 to 350 feet, and Colorado Silica 10-20 filter pack around the well screen to an unrecorded depth in the annulus. The annulus above the filter pack was sealed with neat cement grout. Table 3.5 provides a geologic log of the drill cutting samples collected when the well was drilled. The samples were logged nearly six months after the borehole was drilled, on April 5, 1994, by Dr. Charles S. Robinson, a professional geologist who was instrumental in assisting the Indian Health Service in selecting test drilling sites during exploration for groundwater in the area.

Table 3.5: Production Well No. 3 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 10	CLAY, brown	Colluvium
	CLAY, tan/brown	Colluvium
10 - 20	CLAY, brown/gray	Colluvium
20 - 40	MALPAIS, gray/brown	Basalt
40 - 90	MALPAIS, brown	Basalt
90 - 120	MALPAIS, gray	Basalt
120 - 140	MALPAIS, dark gray	Basalt
140 - 150	SANDSTONE, red, water-bearing	Upper member of Supai
150 - 350	GYPSUM and CLAY Lenses	Second member from top of Supai
350 - 520	SHALE, gray	Undifferentiated Supai
520 - 540	SHALE, gray/brown	Undifferentiated Supai
540 - 560	MUDSTONE, brown	Undifferentiated Supai
560 - 580	SANDSTONE, gray	Undifferentiated Supai
580 - 590	MUDSTONE, gray/brown	Undifferentiated Supai
590 - 640		

In a discussion about the samples, Dr. Robinson expressed the opinion (Robinson, 1994) that the samples were not adequate for the purpose of determining stratigraphy. A review of Robinson's April 5, 1994 geologic log of the samples (not shown here) reveals that a number of the samples contained such a diverse mixture of materials that Robinson refrained from assigning a major lithologic description to the samples and simply described them as a "heterogenous mixture". Samples thus described were from 350-520 feet and all samples from 560-640 feet.

Accordingly, the descriptions of the strata provided on Table 3.5 are based on the field notes of the Indian Health Service engineer who observed the drilling and identified the major changes in lithology, working in conjunction with the experienced well driller. It is likely the field engineer who was working with the driller on the site had a much better idea of the materials penetrated than could be determined later from the composite samples of drill cuttings. The characterization herein of the strata as Coconino or Supai on Table 3.5 are based on the engineer's field notes and are made in the light of the same engineer's field notes for additional wells in the area combined with this author's experience as a geologist in logging the nearby wells. All of this latter information was not available to Robinson in 1994, a fact that made interpretation of the samples out of context quite difficult as properly noted by Robinson on his log.

3.3.2. Construction Data

Table 3.6 provides a summary of well completion data for Well No. 3 compiled in 1998 by Indian Health Service engineers from various sources. Comparison of data on Tables 3.5 and 3.6 shows that the original static water level of 84 feet BTOC was 66 feet above the top of the red sandstone aquifer material in the Supai, therefore indicating confined aquifer conditions. The top of the well screen at a depth of 250 feet shows that only the lower half of the aquifer thickness was screened in order to provide as much water column as reasonable for drawdown during pumping.

Table 3.6: Production Well No. 3 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6095
Tank overflow		6258
Static water level (swl)	84	6011
Top of well screens (BGL)	250	5845
Bottom of well screens (BGL)	350	5745
Pumping water level (pwl)	200	5896
Intake depth	283	5812
Drop pipe length	273	
Total cased depth	350	5745
Nominal pump capacity (gpm)	350	
Pump horsepower	40	

3.3.3. Water Levels

The static water level in Well No. 3 has not been recorded since the well was tested in 1994. Likewise, the pumping water level is unknown. The lack of water level information from Well No. 3 is due in part to a characteristic shared by Wells No. 1 through 3, namely, the type of pitless unit installed on the well head. Measurements of water levels in the wells are made by lowering a measuring instrument through one of the motor cable passages in the spool on the pitless unit. In a number of the wells, a 1-inch diameter PVC standpipe has been installed through one of the motor cable passages for this purpose. The shape of the motor cable passage through the spools on Wells No. 1 through 3 will not allow installation of a standpipe and is at an angle such that only a flexible electrical tape can be lowered through the opening to measure water levels.

This configuration requires the electrical tape to be lowered down the annulus between the well casing and the pump column without a standpipe. The result is that the tape frequently comes to rest on top of a pump column coupler or on top of a portion of the motor cable where it is taped to the pump column and therefore will not go down the well to the depth of the water level. If the electrical tape is jigged and shook to get it off the obstacle, it often becomes stuck below the coupler or wedged between the motor cable and the pump column and therefore stuck in the well where it is necessary to break the electrical tape to remove it from the well. This was the case on 12/01/01 when an attempt was made to measure the pumping water level in Well No. 3 and the probe end of the Indian Health Service electrical tape became stuck at 100 feet and was left in the well after the tape broke.

3.3.4. Well Yield and Pump Condition

The actual yield of Well No. 3 when the production pump was new and the well was first put into service was never recorded. The Indian Health Service statistics on Table 3.6 indicate a design yield of 350 gpm with a pumping water level of 200 feet. This is a total dynamic head of 363 feet static plus unknown friction losses. The production pump is rated to produce only 317 gpm at 363 feet of TDH. Therefore, the original discharge rate of Well No. 3 was necessarily less than 350 gpm and is unknown. Accordingly, the decrease in well yield between an observed 220 gpm on 12/01/01 and the original yield is not known. However, the available information supports some general conclusions.

At the end of 10,000 minutes during a 265 gpm test of Well No. 3 in 1994, the pumping water level in the well was approximately 209 feet, assuming a static water level of 84 feet. A second test at 400 gpm provided a pumping water level of approximately 279 feet in 10,000 minutes. On 12/01/01, the discharge rate from Well No. 3 was observed to be 220 gpm. Recognizing that drawdown is directly proportional to pumping rate, the pro-rated pumping water level for a 220 gpm discharge rate should be 185.7 feet, assuming a static water level of 84 feet. A pumping water level of 185.7 feet in Well No. 3 provides a system TDH of approximately 349 feet. The performance curve for the pump in Well No. 3, shown on Figure 3.1 (Well 3 contains the same type of pump as Well No. 1), indicates the pump is rated to produce approximately 338 gpm at 349 feet of TDH.

The difference between the theoretical well yield of 338 gpm and the observed yield of 220 gpm is a loss of 118 gpm in well yield. A decrease in yield of 118 gpm is equivalent to a decline in the pumping water level of 66 feet (from pump performance curve on Figure 3.1) or a lesser amount of pumping water level decline combined with wear or damage to the pump. The decline in groundwater levels in nearby Wells Nos. 1 and 2 is approximately 40 feet. Therefore, the observed yield of 220 gpm in Well No. 3 on 12/01/01 is consistent with a decline in groundwater levels of 40 feet and pump wear or damage equivalent to 26 feet of head. An alternative interpretation is a groundwater level decline of approximately 40 feet at the well combined with 26 feet of increased well loss drawdown due to iron oxide incrustation on the well screen. Other intermediate alternatives involving a combination of water level decline, increased well loss, and pump wear are possible.

3.3.5. Aquifer Dewatering

Well No. 3 was tested for 13.8 days from 5/18/94 to 6/1/94 at an initial discharge rate of 325 gpm declining to a stabilized rate of 265 gpm by the end of the sixth day of the test. Figure 3.4 shows the time-drawdown response of the aquifer in the pumped well. Although sparse data were collected during the test, the aquifer response exhibits a remarkable decrease in the rate of drawdown after about 200 minutes of pumping. The time-drawdown response is repeated in the residual drawdown during recovery, thus demonstrating the response is controlled by the aquifer hydraulic parameters and boundary conditions.

The geologic log on Table 3.5 indicates the abrupt decrease in the rate of drawdown occurred shortly after the pumping water level in the well declined below the bottom of the basalt that provides a confining layer at the top of the saturated sandstone aquifer. There is good correlation between the onset of drawdown below the confining layer and the decrease in the rate of drawdown shown on Figure 3.4, even considering that the drawdown in the well is not the same as drawdown in the aquifer outside the well, due to well loss drawdown.

This indicates the rate of drawdown during the early response was controlled by release of groundwater from confined storage whereas the late response was controlled by release of groundwater from unconfined storage. Residual drawdown during recovery from pumping exhibits the same type of response. An additional aspect of the aquifer response is that recovery of the groundwater to zero drawdown occurs in less time than the preceding pumping duration ($t/t' > 1$ where t = pumping time and t' = time since pumping stopped).

The 13.8-day pumping test indicated Well No. 3 could be pumped at a greater rate than the 325- to 265-gpm test. Therefore, a second test was performed at a constant discharge rate of 400 gpm maintained for a pumping duration of approximately 70 days. The time-drawdown response during the 400-gpm, 70-day test is shown on Figure 3.5 where it is compared to the response of the earlier 13.8-day test.

The results shown on Figure 3.5 are highly significant in that the slope of the late time-drawdown response is steeper at 400 gpm than at the lower average rate of 273 gpm (325 gpm declining to 265 gpm). The slope of the time-drawdown response is a direct function of the transmissivity of the aquifer. Therefore, the slope of the time-drawdown curve should be the same for all different pumping rates. In other words, the time-drawdown curve at 400 gpm should show more drawdown than that for 273 gpm, but the two should be parallel and separated by the difference in well loss drawdown at the two different pumping rates. The significance of the non-parallel time-drawdown curves shown on Figure 3.5 is that the aquifer response was affected by something in addition to aquifer transmissivity.

Figure 3.4: Time-drawdown response of Well No. 3 at 325 gpm decreasing to 265 gpm during 5/18/94 - 6/1/94 test (13.8 days) averaging 273 gpm.

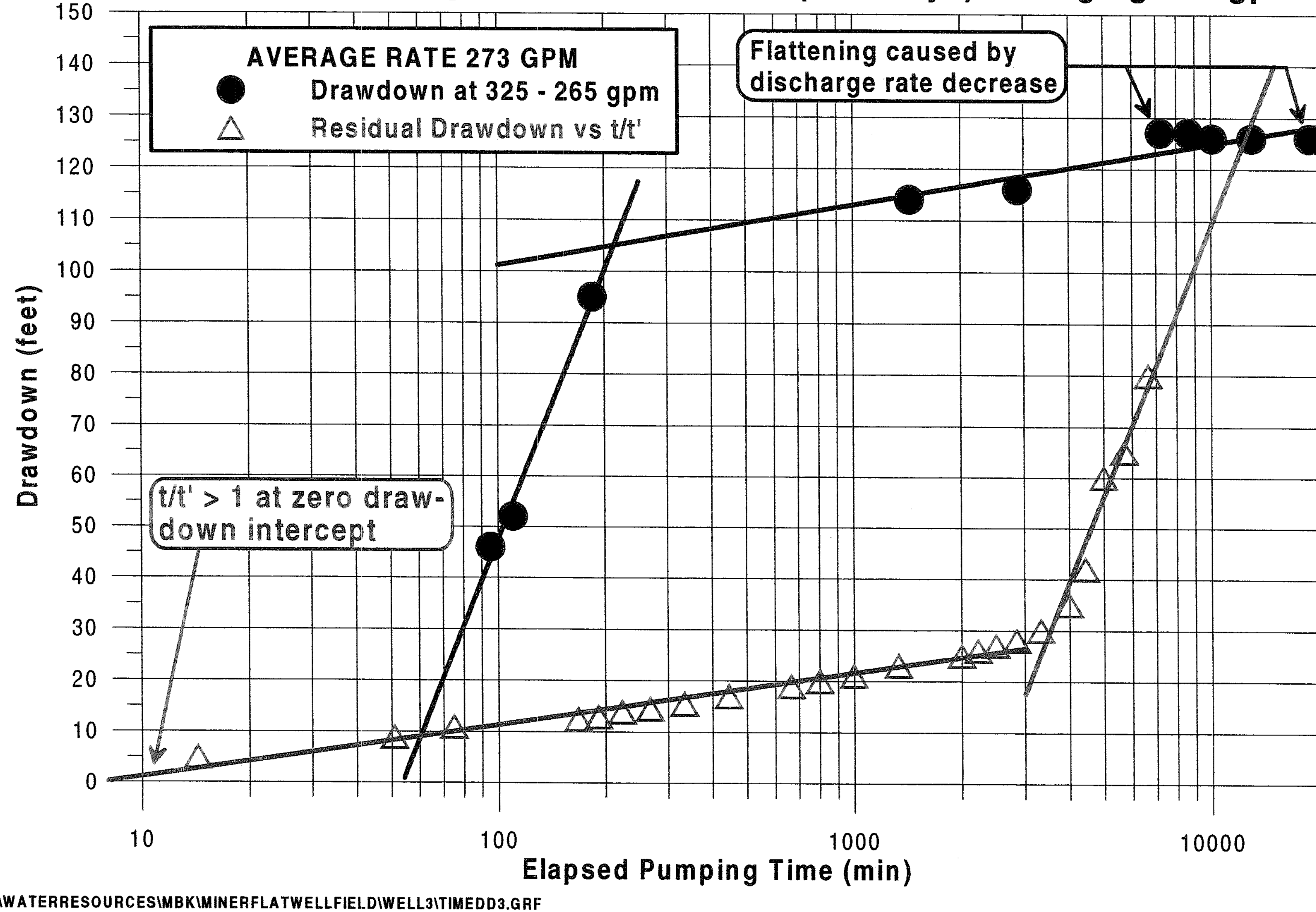
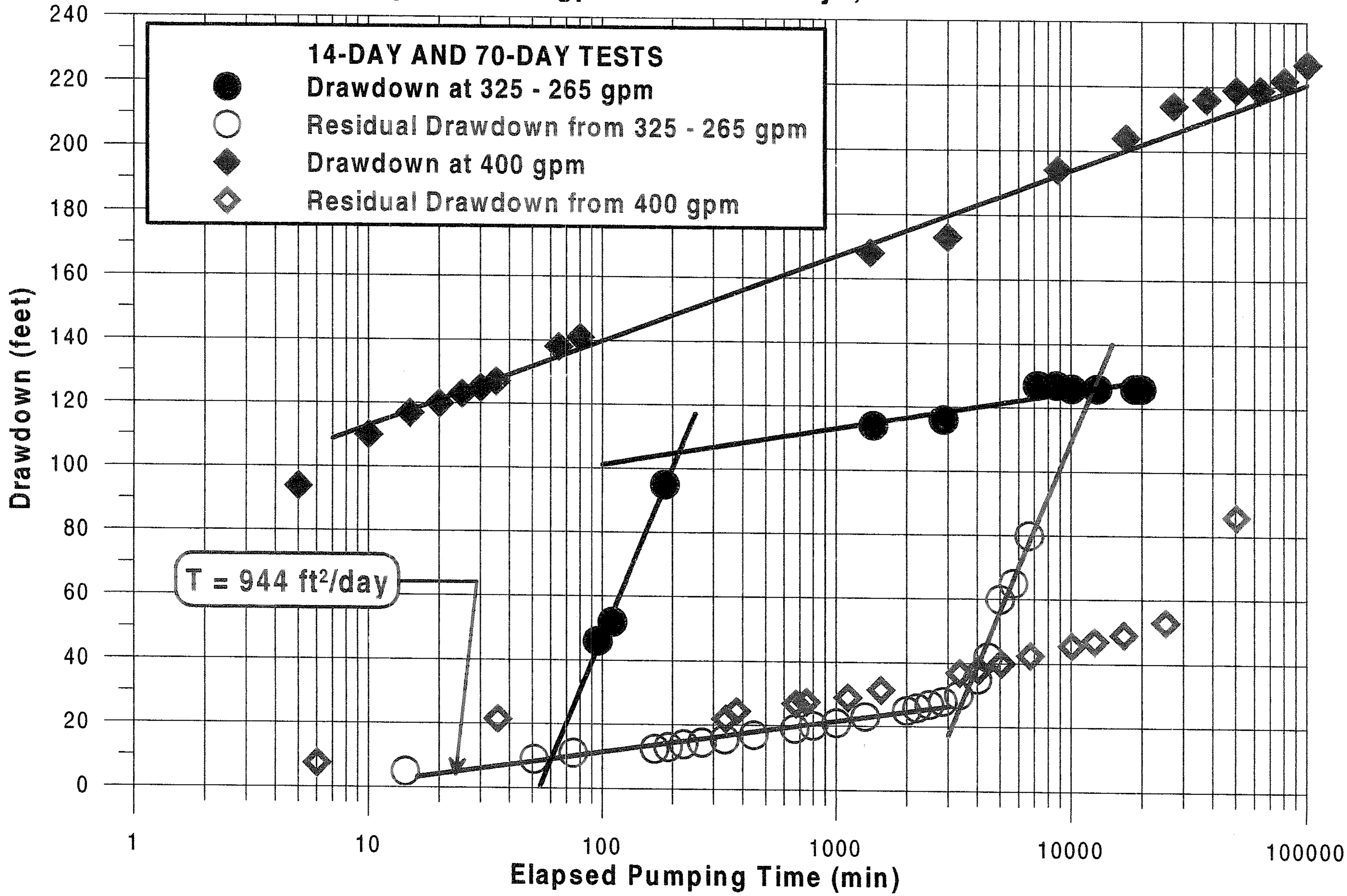


Figure 3.5: Time-drawdown response at Well No. 3 during 325 -265 gpm test for 13.8 days and 400 gpm test for 70 days, both in 1994.



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This conclusion, coupled with the recognition of a change from confined to unconfined flow during the tests, indicates the additional factor is dewatering of the aquifer thickness within the unconfined flow portion of the cone of depression around the pumped well. The drawdown, s_t, in a well pumping an unconfined aquifer is the sum of drawdown due to confined flow, s_c, and drawdown due to dewatering, s_d, as follows, if well loss drawdown, s_w, is ignored:

$$s_t = s_c + s_d \quad \textbf{(Equation 3.7)}$$

Jacob (1963) determined that the portion of drawdown produced by dewatering, s_d, in an unconfined aquifer is:

$$s_d = \frac{s_t^2}{2b} \quad \textbf{(Equation 3.8)}$$

where b = saturated thickness of aquifer

Equation 3.8 shows that as total drawdown increases in a given pumping period, drawdown due to dewatering increases exponentially with respect to total drawdown. The addition of the dewatering drawdown, s_d, to the confined drawdown at Well No. 3 therefore causes a greater change in the slope of the time-drawdown curve for large drawdowns (greater pumping rates) than for small drawdowns (lesser pumping rates). Therefore the slope of the time-drawdown curve is steeper at 400 gpm than at the average 273 gpm rate.

3.3.6. Aquifer Transmissivity

Figure 3.5 shows that the slope of the residual drawdown curve, after confined flow conditions are restored, is the same for both the average 273 gpm test and the 400 gpm test, demonstrating that confined aquifer transmissivity was the same at both pumping rates. Therefore, the slope of the recovery curve can be used to calculate the aquifer transmissivity indicated by the aquifer response at Well No. 3, utilizing the Cooper-Jacob assumptions (Cooper and Jacob, 1946) of the modified nonequilibrium method. The semilogarithmic Cooper-Jacob solution provides an aquifer transmissivity value of 944 ft^2/day from both tests of Well No. 3 as shown on Figure 3.5.

3.3.7. Effect of Dewatering on Well Yield

Based on the foregoing considerations, the results of the 13.8-day and 70-day pumping tests of Well No. 3 (Figure 3.5) indicate a change from confined to unconfined flow during normal operation of the well. The change to unconfined flow introduces an additional component of drawdown due to dewatering of aquifer saturated thickness. Because the dewatering component of drawdown is related exponentially to total drawdown as shown by Equation 3.8, it has two adverse effects:

1. It results in a greater rate of drawdown than that associated with confined flow at a given pumping rate, i.e, a steeper time-drawdown curve than confined flow.

2. It causes the slope of unconfined drawdown to increase as total drawdown increases, thus, time-drawdown curves become steeper as pumping rate increases rather than remain parallel at different pumping rates as in confined flow.

The latter effects have serious implications with regards to well yield. The transmissivity, T, of an aquifer is the product of hydraulic conductivity, K, and aquifer thickness, b, where T = Kb. Therefore, as dewatering makes b smaller, T becomes smaller. Reduction of aquifer saturated thickness, b, causes aquifer transmissivity, T, to decrease in direct proportion.

Theis et al. (1935) demonstrated that the maximum yield of a well is directly related to transmissivity, T, as follows:

$$\mathbf{T} = \frac{\mathbf{Q}}{\mathbf{4\Pi s_c}}\mathbf{W}(\mu) \qquad \textbf{(Theis nonequilibrium equation)}$$

where Q = discharge rate (well yield)
and W(μ) = Theis well function

so

$$\mathbf{Q} = \frac{\mathbf{4\Pi T s_c}}{\mathbf{W}(\mu)} \qquad \textbf{(Equation 3.9)}$$

Equation 3.9 demonstrates that well yield is directly proportional to transmissivity (ignoring the effects of well loss drawdown, s_w). Therefore, the reduction of aquifer transmissivity by dewatering drawdown around a pumped well in unconfined aquifer conditions decreases the maximum yield potentially available from the well.

Kasenow and Pare (2001) provide the following equation for predicting total drawdown, s_t, in an unconfined aquifer, based on values of s_c for confined conditions:

$$\mathbf{s_t} = \mathbf{b}\left[\mathbf{1} - \left[\mathbf{1} - \left(\frac{\mathbf{2s_c}}{\mathbf{b}}\right)\right]^{\frac{1}{2}}\right] \qquad \textbf{(Equation 3.10)}$$

The values of s_c used as input for Equation 3.10 can be projected by conventional equations for confined nonsteady or steady state groundwater flow and drawdown around a pumped well.

3.3.8. Erroneous Assumptions

Recognition of the dewatering effect in the 13.8-day and 70-day tests is significant, because dewatering predicts a future limitation on well yields, as demonstrated by the above equations, unless groundwater levels are allowed to recover during periods of non-pumping in the use of the wells. The dewatering effect was not recognized in the early interpretations of the tests and was therefore not taken into consideration in assessing the long-term performance of the wells. Even if the dewatering effect had been recognized in the earlier test interpretations, it is doubtful that the earlier conclusions about an abundant source of long-term water supply from the Coconino and Supai sandstones at the site would have been any different. This is because the tests, despite the indication of dewatering effect, could not predict the apparent lack of recharge that has prevailed at the wellfield site.

The early interpretations of the test data made an assumption common to pumping test interpretation in a remote area such as the Miner Flat, where groundwater has not been developed by wells and is in a state of nature. The assumption is that the aquifer is in a long-term steady-state flow that is in equilibrium with the long-term reliable recharge to the aquifer. In other words, it is assumed that the natural flow of groundwater through the area is equal to the long-term average recharge. It is therefore assumed that when wells are pumped at a total rate equal to something less than the estimated flow through the aquifer, they will have long-term reliability and that the groundwater levels at the pumped wells will stabilize in a dynamic equilibrium with the aquifer after an initial period of declining levels.

These assumptions are implicit in the recommendations provided by the interpretations of the 13.8-day and 70-day pumping tests. In a February 7, 1995 letter summarizing the results of the tests, Golder Associates stated:

> "***Attached are our analysis of the 70-day pumping test data. These analyses were performed using log-log plots and by matching to type curves generated with the FLOWDIM software package. We attempted to match the data to several different aquifer models including partial penetration, leaky aquifer, and recharge boundary models. The best fit to the data was obtained with the infinite aquifer type curves (Theis).***" (Golder Associates, February 7, 1995 letter to Mineral Systems, Inc.)

The Golder Associates analyses document the progressive change from confined to unconfined conditions in the cone of depression around the pumped well as it expanded outward towards the observation wells. Aquifer storativities determined by Golder Associates were largest (unconfined) at the pumped well and smallest (confined) at the most distant observation well. However, the change from confined storativity early in the test to unconfined conditions late in the test was not recognized by Golder Associates who dismissed the early test response as due to "non-standard" well constructions. Failure to recognize the change in aquifer response during the tests resulted in calculated values of storativity that were incorrect in terms of absolute values but which provided a correct indication of relative differences in confined versus unconfined conditions in the cone of depression. The values of storativity determined by Golder Associates were as follows:

> "*. . . **The deviation of the early time data from an ideal type curve may be the result of the non-standard well constructions. However, because the late time data is more representative of the true aquifer response, all these analyses are considered to produce reliable estimates of transmissivity.***
>
> ***Due to the lack of good quality early time data for OB-1 and OB-2, curve fitting along the x-axis (elapsed time) is subject to a wide range of variation. Storativities for the pumping well, OB-1, and OB-2 were calculated at $5.2x10^{-3}$, $3.6x10^{-4}$ and $5.3x10^{-5}$, respectively. The non-unique fit to the time axis results in the wide range of storativities determined from these analyses. Storativities calculated from the observation well data are typically considered more representative of aquifer conditions than pumping well results.***" (Golder Associates, February 7, 1995 letter to Mineral Systems, Inc.)

The above statements show that Golder Associates ignored the early part of the aquifer response and ignored the differences between calculated storativity values at different distances from the pumped well which documented a cone of depression that was unconfined near the pumped well while still confined at a greater distance from the pumped well.

The failure by Golder Associates to recognize the type of aquifer response obtained was not a fatal flaw in their analysis. Even if they had recognized the change from confined to unconfined flow in the cone of depression, their conclusions probably would not have been any different. This is because, in the absence of data to the contrary, they assumed that steady-state flow of groundwater from a recharge area to the Miner Flat area would support recovery of groundwater levels at the wells during non-pumping periods. This assumption is stated clearly in their February 7, 1995 letter as follows:

> "***The late time data from both the 14 and 70-day pumping tests show an infinite aquifer response. These results are not significantly different from results presented in our earlier report.***" (Golder Associates, February 7, 1995 letter to Mineral Systems, Inc.)

The assumption of an "infinite aquifer", relative to the cone of depression during the two tests and relative to cones of depression during tests conducted by Golder Associates in 1994, was not unreasonable in the context of the local geology and in the context of the test responses. The only thing that would have alerted Golder Associates (or anyone else) to the lack of recharge would have been test wells showing that there was not a significant hydraulic gradient in the area, thus implying a lack of groundwater flow and therefore an absence of significant recharge. The few test holes available indicated a hydraulic gradient, albeit towards the North Fork of the White River, a fact that obscured the lack of recharge revealed by the subsequent response of the groundwater levels to use of the wellfield.

Accordingly, it is only with hindsight that the flaws in the assumptions made by Golder Associates in 1994 and 1995 are evident. The results of the 70-day test lulled those involved into a sense of complacency. This is evident in the February 7, 1995 letter from Mineral Systems, Inc. to the Tribal Engineer wherein Mineral Systems, Inc., who used Golder Associates as a subconsultant, stated:

> "***The analyses*** (sic) ***of the data from the 70-day pumping test does not significantly change the conclusion presented by Golder Associates, Inc. in their report, "Pumping Test Analysis and Well Field Design, Miner Flat Area, Fort Apache Indian Reservation,: of May 1994. The transmissivity and storativity are all within the same order of magnitude. Based on the geology and the transmissivity and storativity, assuming that the total yield desired from the well field is 4,000 gallons per minute and that the individual well yields would be 400 gpm for a 10 to 12 hour pumping period, the number of wells needed would be eight or ten, spaced at least 500 feet apart. These well should be drilled to the west or northwest of the Miner Flat well.***" (Mineral Systems, Inc., February 7, 1995 letter to Tribal Engineer)

The highly optimistic concept that eight to ten wells would provide 4,000 gpm was soon proven erroneous as the wellfield was constructed. Only Well No. 9 could be pumped at 400 gpm during a 24-hour test. Wells No. 4, 5, 6, 7, 8, and 10 yielded from 150 gpm (two wells) to 375 gpm (one well) with three of the wells yielding 200, 300 and 350 gpm, respectively. However, this experience still had no bearing on identifying the limitations of recharge to the wellfield. The erroneous assumption of long-term steady-state recharge would be disproved only by the subsequent experience of operating the wellfield.

This experience serves to prove the need for widespread assessment of a groundwater system to quantify groundwater throughflow as an indication of available recharge. Localized aquifer tests of the type conducted at the wellfield serve only to identify well hydraulics and local aquifer hydraulic parameters. They do not quantify the available resource or long-term recharge. A reliable assessment of the long-term availability of groundwater requires extensive investigations of flow through the system over a broad area as well as hydrographs of groundwater fluctuations of at least 10-years duration. Obviously, it was not possible to wait for this type of detailed data before implementing the Miner Flat Wellfield to meet the crucial demands for water supplies in the Whiteriver area. The wellfield was constructed as an immediate solution to an existing problem. Whether the wellfield was to be a long-term solution or simply an intermediate solution remained to be determined when the wellfield was constructed. The history of the aquifer response to the wellfield operation now indicates the wellfield is probably only an intermediate solution with a predictably limited life.

3.3.9. Recommended Pump Size

More information is required to select a proper pump capacity for Well No. 3 under current and future conditions. The 12/01/01 yield of 220 gpm indicates the yield of the well may have declined significantly, due to dewatering effects, as compared to the original design yield of 350 gpm. However, the pumping water level at 220 gpm must be determined and the question of increased well loss due to oxide incrustation on well screen versus loss of capacity due to pump wear must be addressed before a properly sized pump can be selected for this well. These preliminary data, and the performance of the other wells in the wellfield, suggest a properly sized pump for this well would be 100 gpm, taking into account the rate of annual decline in water levels of 28.6 ft/yr at the well.

3.4. Well No. 4

Well No. 4 was put into service in January 1998. Frankie Williams, Water System Operator, reports that there has not been any problems with the well.

3.4.1. Geologic Log

A geologic log of the materials penetrated by Well No. 4 is provided on Table 3.7. The well penetrates 70 feet of colluvial deposits on top of a basalt flow. The base of the basalt flow at 95 feet rests on paleoalluvium or paleocolluvium of the ancestral White River valley. The base of the paleo-channel is at the top of the Coconino Sandstone at 150 feet and the channel is filled with alluvial clay. The original static water level of 117 feet BTOC indicates confined aquifer conditions before pumping began. The log shows the initial air lift yield from the well during drilling was obtained at 155 feet, only five feet below the top of the Coconino Sandstone.

3.4.2. Construction Data

Table 3.8 provides a summary of well completion data for Well No. 4. Comparison of data on Tables 3.7 and 3.8 shows that the original static water level of 117.4 feet BTOC on 9/23/97 was 32.4 feet above the base of a clay-filled paleo-channel at 150 feet, therefore indicating confined aquifer conditions. The well was completed to a total cased depth of 375 feet with 8-inch nominal diameter steel casing including 8-inch pipe sized 20-slot stainless steel well screen in the following intervals:

210 - 230 feet
258.5 - 278.5 feet
310 - 370 feet

Table 3.7: Production Well No. 4 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 70		Colluvium
	CLAY, tan to brown	Basalt
70 - 95	BASALT, gray	Paleoalluvium/colluvium
95 - 150	CLAY, red and tan	Coconino Sandstone
150 - 210	SANDSTONE, red and tan	
	Minor water-bearing zone 155 ft	
	Good water-bearing zone 200 ft	Coconino Sandstone
210 - 233	SANDSTONE, red and tan	Coconino Sandstone
233 - 241		Coconino Sandstone
241 - 256	SILTSTONE w/SANDSTONE and CLAY	
	SANDSTONE, yellow/white, good water-	Surface of of Supai
256 - 258	bearing zone	Supai
258 - 287	CLAYSTONE, dark red	
	SANDSTONE, reddish-brown, good	Supai
287 - 310	water-bearing zone 258-287 ft.	Supai
310 - 374	SANDSTONE, brown	
	SANDSTONE, white, pink and tan, good	Supai
374 - 385	water-bearing zone	
	SANDSTONE & SILTSTONE, red	

Colorado Silica 10-20 silica sand filter pack was installed in the annulus between the 12-inch diameter borehole and the casing and well screen from total depth to near the base of the pitless unit.

Table 3.8: Production Well No. 4 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6146
Tank overflow		6258
Static water level (swl)	117	6029
Top of well screens (BGL)	210	5936
Bottom of well screens (BGL)	370	5776
Pumping water level (pwl)	230	5916
Intake depth	320	5826
Drop pipe length	315	
Total cased depth	375	5771
Nominal pump capacity (gpm)	225	
Pump horsepower	25	

3.4.3. Water Levels

Figure 3.6 shows the initial aquifer response to a baseline 24-hour constant rate test at 150 gpm and compares that response to the 12/01/01 response at a pumping rate of 88 gpm. Figure 3.6 shows that there has been approximately 50 feet of decline in the static water level at Well No. 4 since September 1997. Figure 3.6 also shows that the pumping water level, rather than remaining above the top of the first screened interval at 210 feet, now is at the top of the second screened interval at 260 feet and sometimes declines below the top of the second well screen, even though the pumping rate has declined from 150 gpm to 88 gpm.

3.4.4. Well Yield and Pump Condition

This raises a question about the design pumping water level of 230 feet and the design pumping rate of 225 gpm shown on Table 3.8, which indicate an original design pumping water level that would dewater the entire uppermost well screen. Designing for a pumping water level that dewaters a well screen is not a good design practice. The design pumping water level of 230 feet appears to be the result of a projection of the 2-day baseline pumping test results to predict the pumping water level that would be caused by a 7-day period of continuous pumping during some undefined peak pumping demand. The projection, at 150 gpm, predicted a pumping water level near the bottom of the first well screen after one week of continuously sustained pumping at 150 gpm. While drawdown of the water level in the well to below the well screen is not a desirable design practice, it could be accepted on the basis of a few brief periods of such use during the life of the well, presuming water levels were allowed to recover following such a pumping episode.

The 12-hour pumping water level measured during the September, 1997, baseline test of the well at a constant rate of 150 gpm was approximately 206 feet. During the same 150-gpm test, the pumping water level at the end of 24 hours was 207.645 feet BTOC, still above the top of the first screened interval at 210 feet. These data indicated the well could be pumped at rates up to 200 gpm for 24 hours, if lowering of the pumping water level about 8 feet into the uppermost well screen after 24 hours of continuous pumping was acceptable to obtain a short-term peak pumping rate. A design for 200 gpm did not take into consideration the potential for iron oxide precipitation from the well water as chemical analyses of the groundwater had not been performed.

Based on the foregoing considerations, use of a pump with a nominal capacity of 225 gpm, as shown on Table 3.8, was not supported by the baseline tests. Figure 3.7 shows the pump performance curve for the 225 gpm pump. Table 3.8 indicates a total dynamic head of 322 feet when the pumping water level is at the top of the well screens. The pump performance curve (Figure 3.7) indicates the pump will produce 230 gpm with a total dynamic head of 322 feet and about 200 gpm when the pumping water level declines to the bottom of the first screened interval with 342 feet total dynamic head. Accordingly, sizing the pump to produce the maximum recommended yield of 200 gpm

Figure 3.6: Comparison of 1997 and 2001 static water and pumping water levels at Well No. 4.

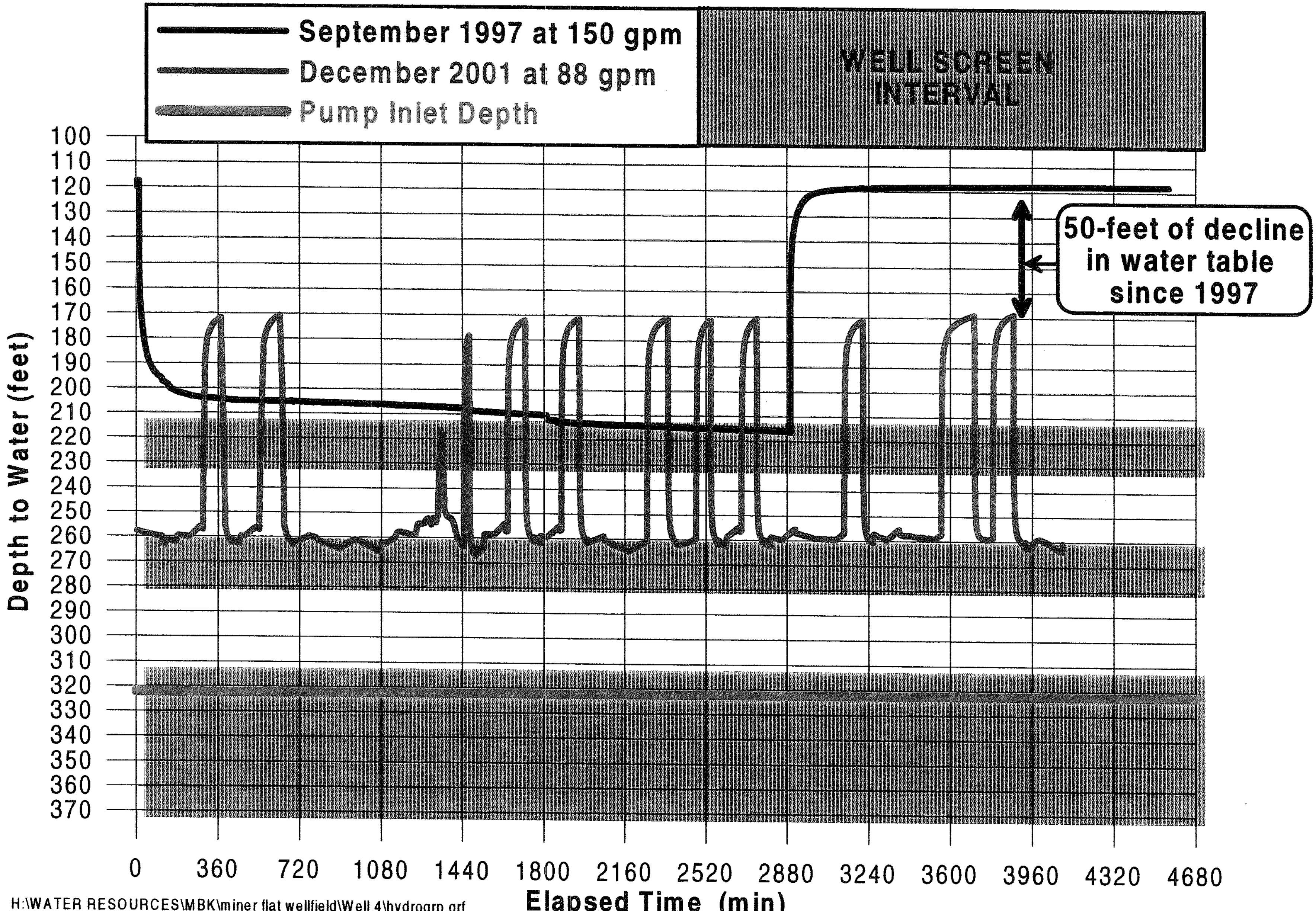
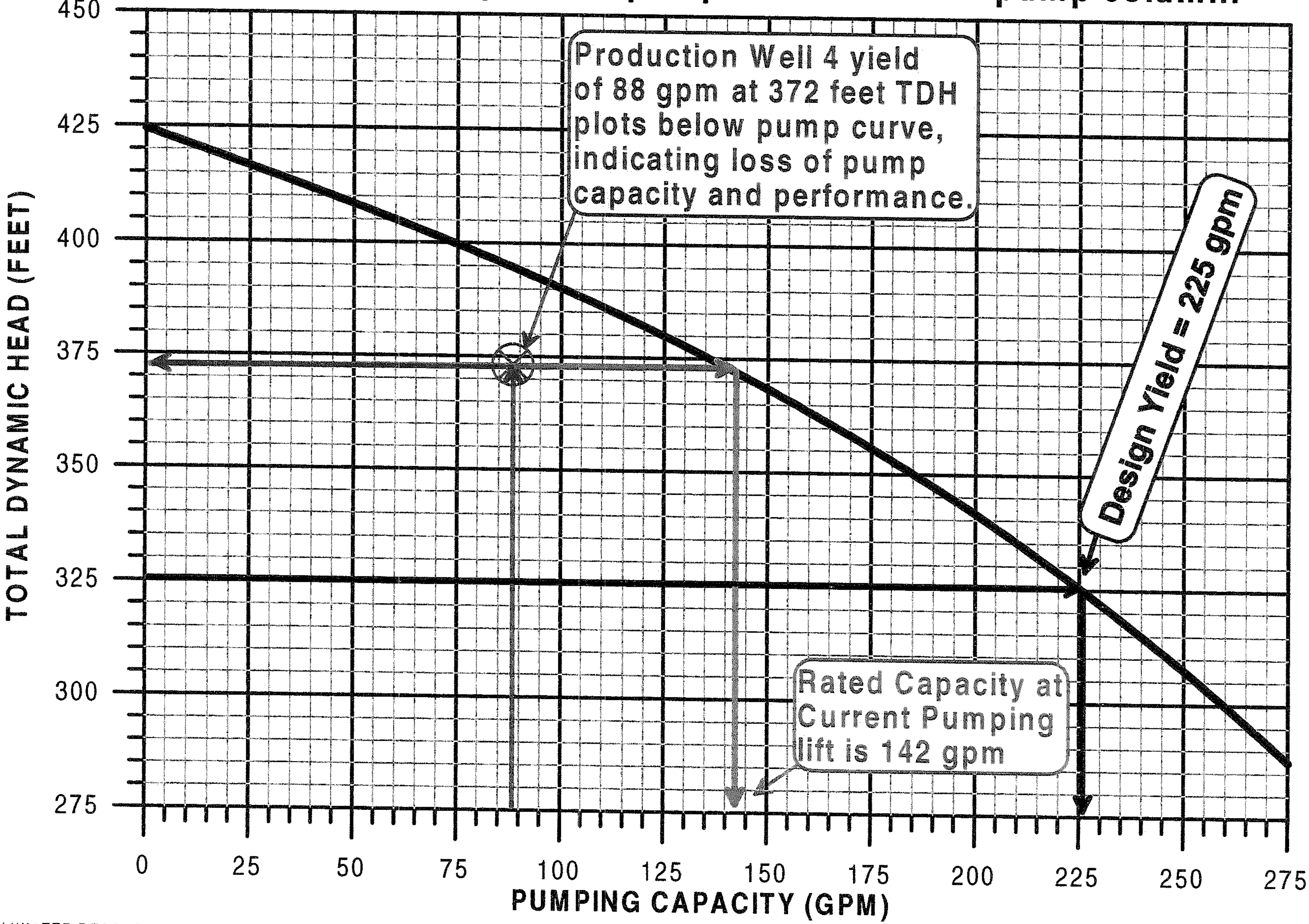


Figure 3.7: Pump performance curve for Goulds 6CHC025 pump shows 54 gpm loss of capacity due to pump wear or hole in pump column.



H:\WATER RESOURCES\MBK\miner flat wellfield\Well 4\60CHC025.grf with some drawdown results in a pump that dewaters the uppermost screened interval during normal operation of the well.

Preliminary design considerations aside, the pump presently produces 88 gpm with the pumping water level at approximately 260 feet. From Table 3.8, a pumping water level of 260 feet is equivalent to a total dynamic head of 372 feet, ignoring line losses in the pump column and distribution system. The pump performance curve on Figure 3.7 shows the pump is rated to produce 142 gpm at a total dynamic head of 372 feet.

The pump is therefore rated to produce at least 200 gpm under the original static water level and baseline pumping conditions. The present pump yield of 88 gpm represents a loss of 112 gpm in pumping capacity, or about 56 percent of the original pump capacity. The difference between the rated yield of 142 gpm at the present pumping water level and 200 gpm, a loss of 58 gpm, is due to the decline in groundwater level and increased pumping lift at the well. The remainder of the decrease in capacity, the difference between 142 gpm and 88 gpm, a loss of 54 gpm, is due to wear or damage to the pump.

A final consideration in evaluating the performance of the existing pump in Well No. 4 is the shape of the plot of water levels during the December 2001 operations, as shown on Figure 3.6. When the pump starts, initial drawdown to a pumping water level exhibits the normal response anticipated in a pumped well. However, the initial drawdown is followed by an abrupt change in the hydrograph with the pumping water level abruptly departing from the initial drawdown curve and fluctuating somewhat erratically in the uppermost few feet of the second well screen. This is not typical drawdown behavior and may indicate an undetected problem with the pumping equipment.

For example, the response displayed on Figure 3.6 might indicate a hole in the pump column at about 262 feet BTOC. The abrupt departure from the initial drawdown rate always occurs at this depth. If this is the case, the pump is producing more than 88 gpm, but all but 88 gpm is circulating back into the well. When the pump in Well No. 4 was in operation during December 2001, the sound of water cascading or spraying out into the well could be heard from the land surface. At the time, the noise was interpreted to be water cascading down the well from the uppermost screened interval; however, it may have been partly or solely due to water spraying out of a leak in the pump column.

3.4.5. Recommended Pump Size

Irrespective of the possibility of a leak in the pump column, a new pump size for Well No. 4 can be recommended, based on the pragmatic observation of the 12/01/01 yield of the well at 88 gpm with the pumping water level slightly below the top of the second screened interval at 258.5 feet. It is evident from Figure 3.6 that it is no longer practical to maintain a pumping water level above the top of the first screened interval while obtaining any reasonably useful well yield. However, it is desirable to maintain the pumping water level above the second screened interval at a depth of approximately 255 feet BTOC and take into account the annual rate of groundwater level decline of 12.2 ft/yr at this well.

The maximum possible pumping rate with the pumping water level maintained at or above 255 feet can be estimated from the present specific capacity of the well. Inspection of the initial drawdown response during the December 2001 pumping indicates the pumping water level at 88 gpm or less would have stabilized at an estimated 270 to 275 feet in 12 to 24 hours pumping time. Assuming the discharge rate would have remained at 88 gpm, the specific capacity at a pumping water level of 270 to 275 feet and a static water level of approximately 170 feet is approximately 0.84 gpm/ft (gallons per minute per foot of drawdown). The drawdown from a static water level of 170 feet to a pumping water level of 255 feet is 85 feet, which at 0.84 gpm/ft provides a well yield of 71.4 gpm. Provision for declining water levels reduces this to 50 gpm.

The foregoing factors indicate Well No. 4 should be equipped with a 50 gpm pump at prevailing groundwater levels. The 50 gpm pump should be sized to deliver 50 gpm at a total dynamic head of 367 feet. The pump inlet should be installed at approximately 299 feet (14 joints of pump column each 21 feet long plus a 5-foot pump length) in order to keep the pump motor in a section of well casing instead of well screen so that the pump motor will receive proper cooling.

Uncertainties in recommending a 50 gpm pump include the fact that the groundwater level at Well No. 4 may continue to decline in the future, thus increasing the pumping lift, and the fact that well loss due to incrustation on the well screens is unknown. Removal of incrustation from the well screen, if present, might decrease well loss; however, this factor is a complete unknown with the information available at this time.

3.4.6. Aquifer Response

A typical single-well aquifer test analysis presents a determination of well loss from step test data; correction of pumped well drawdown for well loss; correction for dewatering drawdown, if appropriate; and determination of aquifer hydraulic parameters from the constant rate test data starting with presentation of the diagnostic log-log plot. The analysis of baseline tests conducted on Well No. 4 in September 1997 does not follow the typical organization. This is because step test data normally used to show well loss reveal an aquifer boundary condition that is not inherently obvious in the constant rate data and which influences how the response must be interpreted. The boundary condition is the onset of partially unconfined conditions in the cone of depression.

Figure 3.8 presents a Birsoy-Summers plot of the baseline step test reponse at Well No. 4 in 1997. The plot indicates that negative boundary conditions began to affect the aquifer response within the 20-minute duration of the first step at 150 gpm. Recognition of the early onset of negative boundary conditions is the first step in understanding the significance of the diagnostic log-log plot shown on Figure 3.9.

The diagnostic plot on Figure 3.9 exhibits the shape of a time-drawdown curve that can represent several different types of aquifers and aquifer response in nature. The shape of the time-drawdown curve is characterized by an initial period of early drawdown

Figure 3.8: Birsoy-Summers solution for 9/25/97 baseline test of Well No. 4.

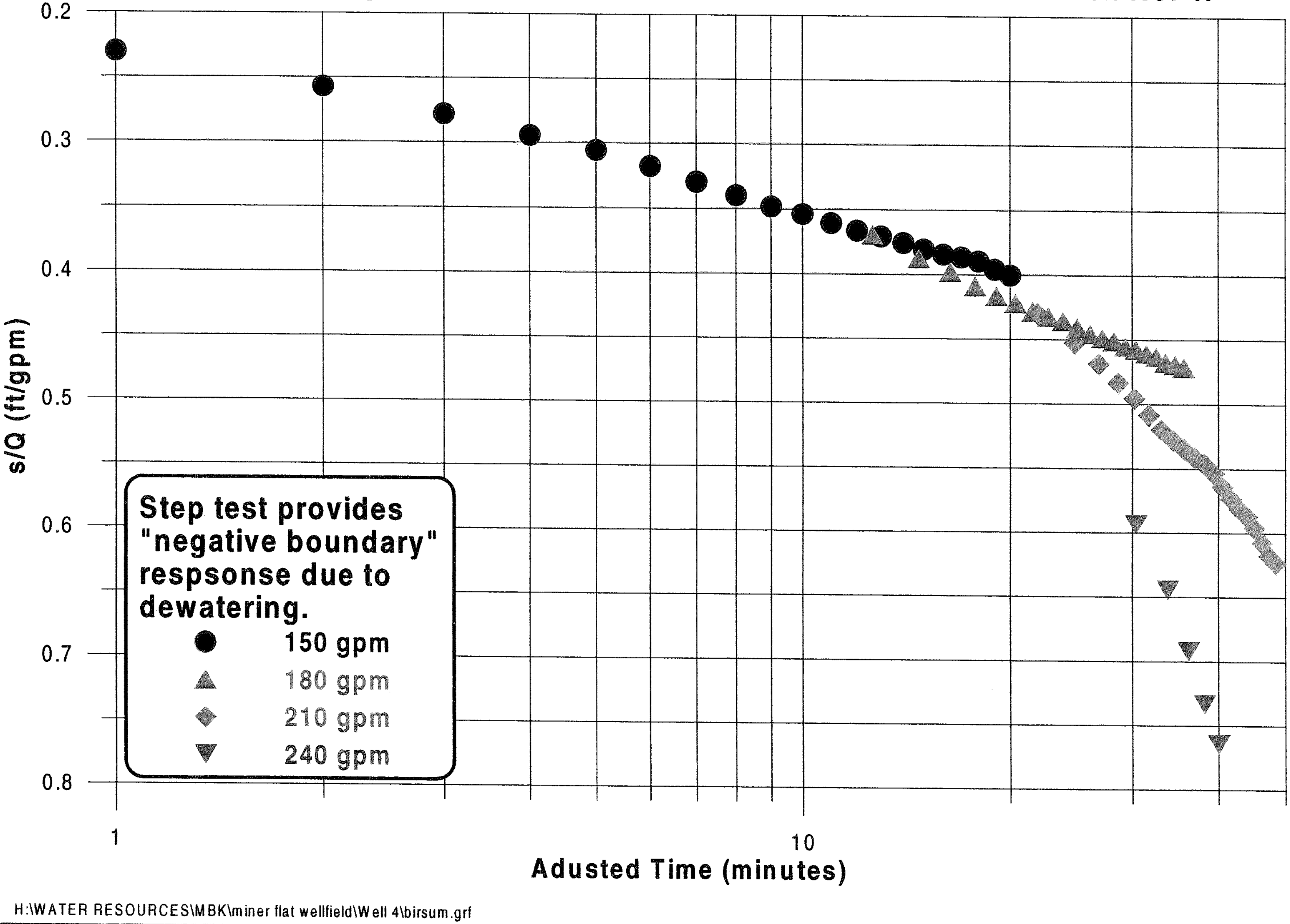
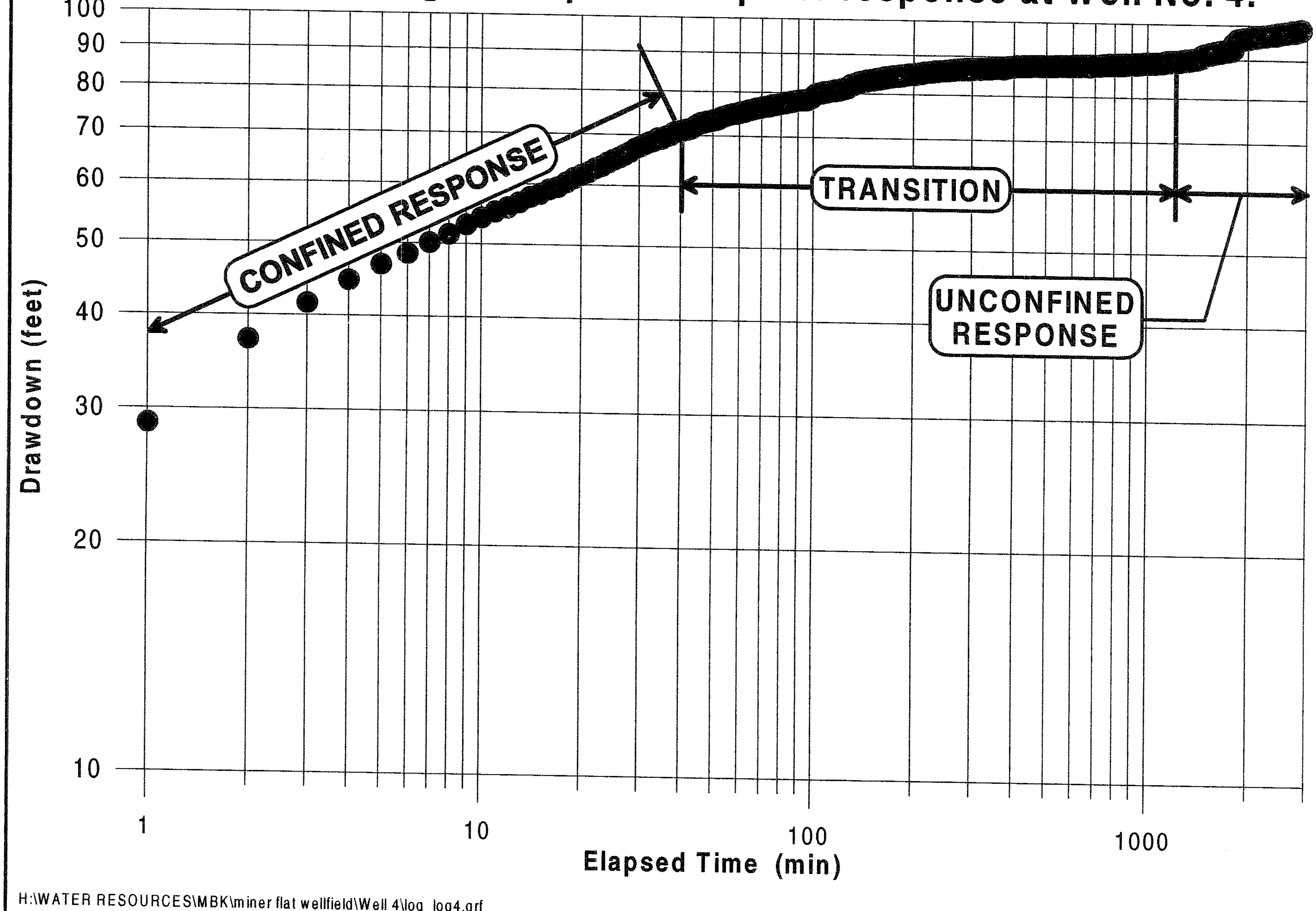


Figure 3.9: Diagnostic plot for aquifer response at Well No. 4.



followed by a transition into an intermediate period of stabilized drawdown followed by resumption of drawdown in the late part of the response. This type of time-drawdown curve can represent delayed yield in semi-confined flow, dual porosity in confined or unconfined fractured porous rock, response to multiple aquifers penetrated by one well, and partial dewatering of the cone of depression an otherwise confined aquifer.

In the initial interpretations of the baseline data in 1997, the diagnostic curve on Figure 3.9 was considered to represent the response of a two-layer aquifer system, one layer being the Coconino Sandstone and the other being the Supai Sandstone. It was thought that the transition from early drawdown response to late drawdown response represented a change from the storativity of one zone controlling the drawdown to the storativity of the other zone controlling drawdown. However, that early interpretation did not take into account the negative boundary response which was not recognized at that time.

The Birsoy-Summers plot on Figure 3.8 was used following the December 2001 investigation of the wellfield in an attempt to explain why the stepped rate test response apparently indicated well losses too large to be consistent with the drawdown-recovery relationship during the constant rate test. This resulted in recognition of the negative boundary effects for the first time. Negative boundary effects are any limitation in aquifer transmissivity, thickness, or extent that impinges on the cone of depression during a pumping test and requires an acceleration of drawdown throughout the remainder of the cone of depression. Introduction of negative boundary conditions as an additional factor in interpreting the test response rules out the multiple-layer aquifer interpretation and requires reassessment of the choice of analytical model used to explain the aquifer response.

Reinterpretation of the test in view of the physical relationship between the test pumping water levels and the geology of the aquifer at Well No. 4 indicate that the three-part time-drawdown curve response was due to partial dewatering of the cone of depression in a confined aquifer. Partial dewatering causes a negative boundary response in the Birsoy-Summers plot, similar to a no-flow boundary or other causes of negative boundary conditions.

The partial dewatering model fits the physical aquifer conditions of the test. The initial static water level of 117.5 feet was 32.5 feet above the base of the confining clay beds. Initial drawdown in the well exceeded 32.5 feet in less than two minutes of pumping time. Although drawdown outside the well casing would lag behind drawdown inside the well casing due to casing storage effects and well loss drawdown, the Birsoy-Summers plot on Figure 3.8 indicates that the onset of unconfined conditions in the portion of the cone of depression nearest the well had occurred 8 to 10 minutes into the first step of the stepped rate test. Since the first step was conducted at 150 gpm, the same rate as the constant rate test, it follows that the cone of depression during the 150 gpm constant rate test became partially unconfined within 8 to 10 minutes pumping time.

Although unconfined conditions (and therefore partial dewatering of the aquifer thickness) started in 8 to 10 minutes of pumping, the extent of the unconfined portion of the aquifer did not become significant with respect to the size of the cone of depression until about 40 minutes of elapsed pumping time as shown by the confined response on Figure 3.9. After 40 minutes of pumping at 150 gpm, the unconfined area is large enough that it begins to affect the shape of the time-drawdown curve as control of the rate of drawdown shifts from the confined storativity to the unconfined storativity. This transition continues through the intermediate part of the test with drawdown essentially stabilizing until more than 1000 minutes of pumping time. Between 1000 and 1100 minutes, drawdown resumes but is controlled by the unconfined storativity.

Figure 3.9 shows that after unconfined response began, the pumping water level in the well declined to the top of the first screened interval at 210 feet where drawdown was equal to 92.5 feet. The rate of drawdown abruptly increased at this depth due to the additional increment of well loss added to the drawdown as the well screen was dewatered.

3.4.7. Well Loss

Figure 3.10 shows total drawdown plotted versus pumping rate for the stepped rate test of Well No. 4. Two specific capacity curves are provided on the plot, one for the 1-minute response of each pumping rate and one for the 10-minute response. The difference between the two curves is that the pumping water level dropped below the top of the uppermost well screen during the last and greatest pumping rate after 10 minutes. The significant increase in well loss due to turbulent flow through the dewatered portion of well screen is reflected by the abrupt departure of the 240-gpm step from the specific capacity curve after 10 minutes of pumping time. The well performance on 12/02/01 is compared to the baseline data and reflects a 41.5 percent decrease in the specific capacity of the well.

Figure 3.11 shows the same data as Figure 3.10, but plotted as a conventional Hantush-Bierschenk presentation of the step test data; specific drawdown versus pumping rate. The 1-minute and 10-minute data define the same curve with the exception of the departure after the well screen begins to dewater during the 240-gpm rate. Again, a plot of the 12/02/01 data shows a marked loss of well performance since 9/25/97.

The slope of the specific drawdown versus pumping rate curve on Figure 3.11 multiplied times the square of the pumping rate provides well loss in the Hantush-Bierschenk method. For example, Figure 3.11 shows the slope of the plot, prior to dewatering of the well screen, to be 0.00383; therefore at a pumping rate of 150 gpm, the Hantush-Bierschenk solution for well loss is:

$$\textbf{Well loss} = s_w = (0.00383)(150)^2 = 86 \textbf{ feet} \qquad \textbf{(Equation 3.11)}$$

Figure 3.10: Comparison of specific capacity curve for 9/25/97 to present specific capacity shows loss of performance.

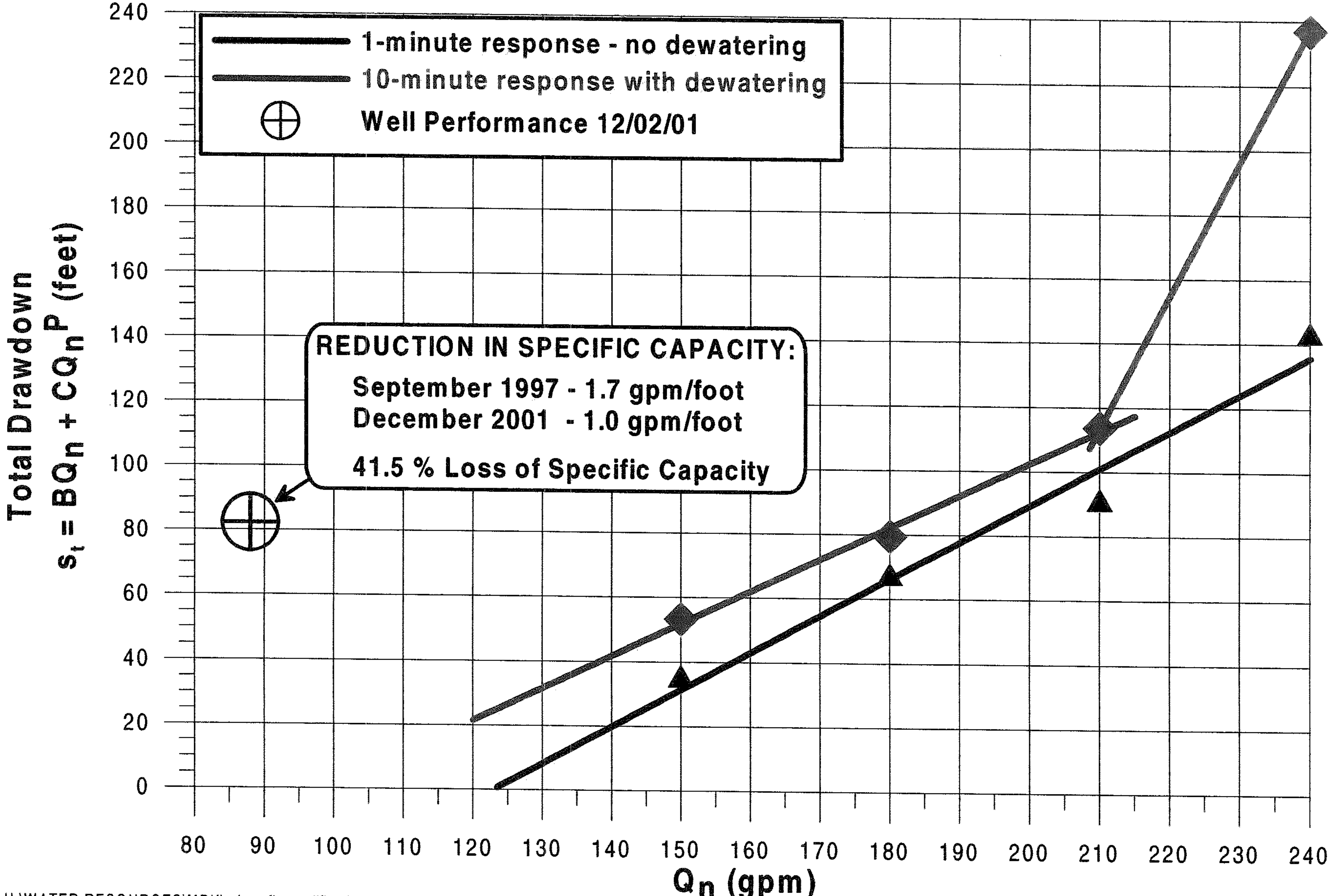


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Figure 3.11: Comparison of specific drawdown in 1997 to specific drawdown 12/02/01 at Well No. 4.

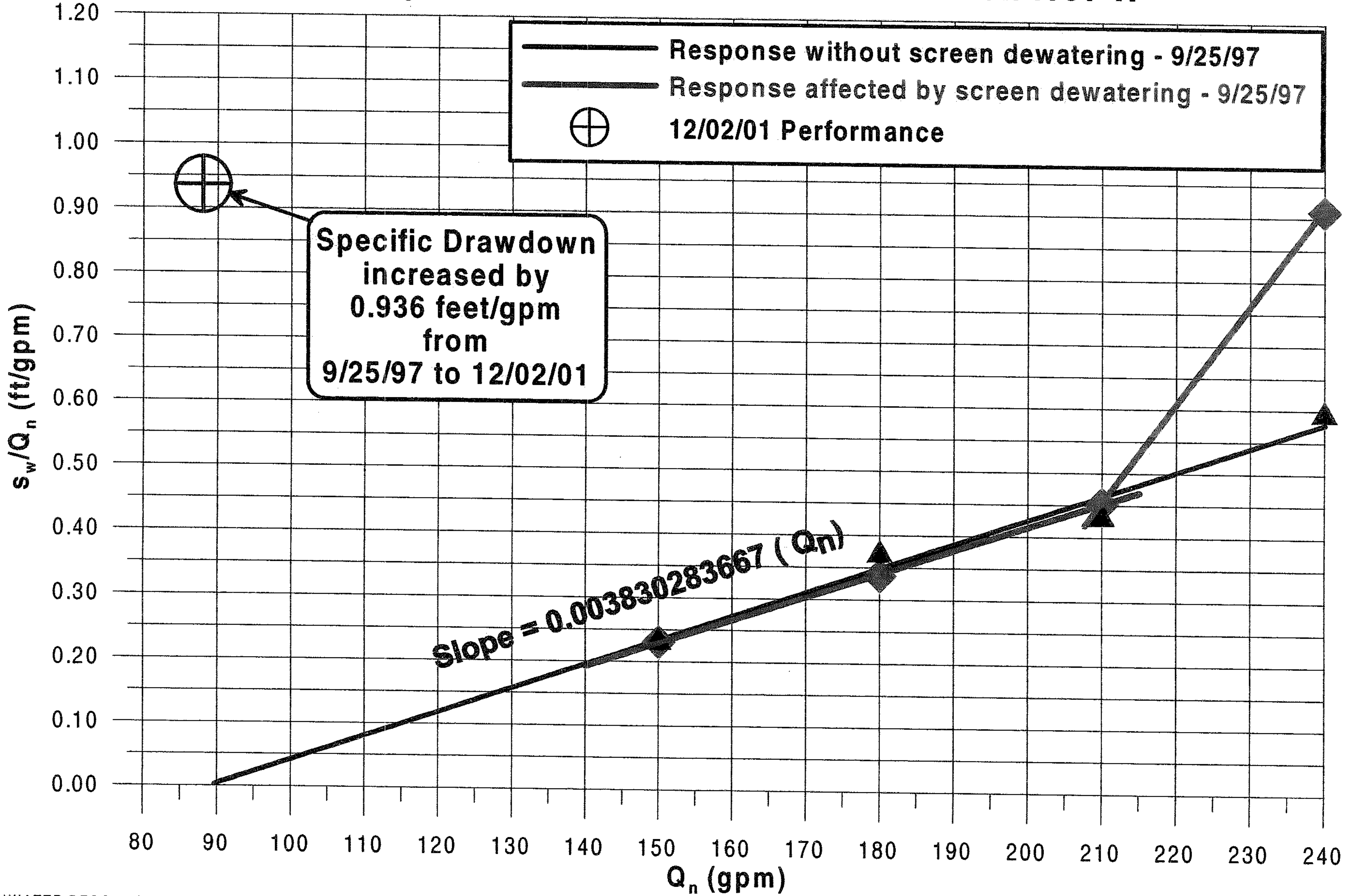

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Figure 3.12 shows drawdown and residual drawdown (recovery) for Well No. 4 pumped at a constant 150 gpm. The difference along the drawdown axis between drawdown during pumping and residual drawdown during recovery is usually a good measure of well loss, for the specific pumping rate used. Figure 3.12 indicates a separation of 66 feet between drawdown and recovery. This value is considerably smaller than the 86 feet of well loss provided by the Hantush-Bierschenk solution, Equation 3.11.

The reason the Hantush-Bierschenk solution grossly overestimates well loss drawdown is that each specific drawdown value used in the solution includes a significant amount of drawdown due to dewatering effects in addition to well loss. This fact is demonstrated by the Birsoy-Summers plot of the same data on Figure 3.8 which shows dewatering effects after the first 8 to 10 minutes of the stepped rate test. Obviously, the uncorrected drawdown response during constant rate test, as shown on Figure 3.12, also includes a component of dewatering drawdown. Therefore, the 66 feet of separation between drawdown and recovery on Figure 3.12 also overestimates well loss.

Based on the complications provided by the foregoing considerations, the standard methods of calculating well loss from stepped rate tests do not work. The best estimate of well loss is provided by the diagnostic curve on Figure 3.9. Recalling the static water level was 117.5 feet at the start of the test and the depth to the base of the confining layer was 150 feet, the confined head was equal to 32.5 feet. Figure 3.9 shows that when the transition from confined to unconfined response started, the drawdown was approximately 70 feet. The beginning of the transition is the time at which the water level outside the well casing declined far enough below the confining bed, 32.5 feet below static water level, to cause the unconfined response. Accordingly, a drawdown of 70 feet inside the well and of 32.5 feet outside the well indicates a well loss of 37.5 feet. This is a maximum value for well loss because the water level outside the casing may have declined below the confining bed well before the drawdown in the well increased to 70 feet but simply did not cause a response because the drawdown at that time was still controlled by confined storativity and would remain so until later in the test.

The summary of the above analysis is that well loss at a pumping rate of 150 gpm was 37.5 feet or less. If drawdown outside the well casing reached the bottom of the confining layer before drawdown inside the well increased to 70 feet, which is likely, well loss is less than 37.5 feet. Figure 3.13 shows the drawdown measured in the field at 150 gpm, drawdown corrected for 37.5 feet of well loss, and the drawdown corrected for well loss and further corrected to confined drawdown without dewatering effects (Equation 3.8).

3.4.8. Projected Drawdown

The confined drawdown on Figure 3.13, corrected for well loss of 37.5 feet and dewatering, can be projected into the future as unconfined drawdown. The projection must be made from data after the onset of partial dewatering of the uppermost well screen and the attendant increase in well loss. Figure 3.14 shows such a projection,

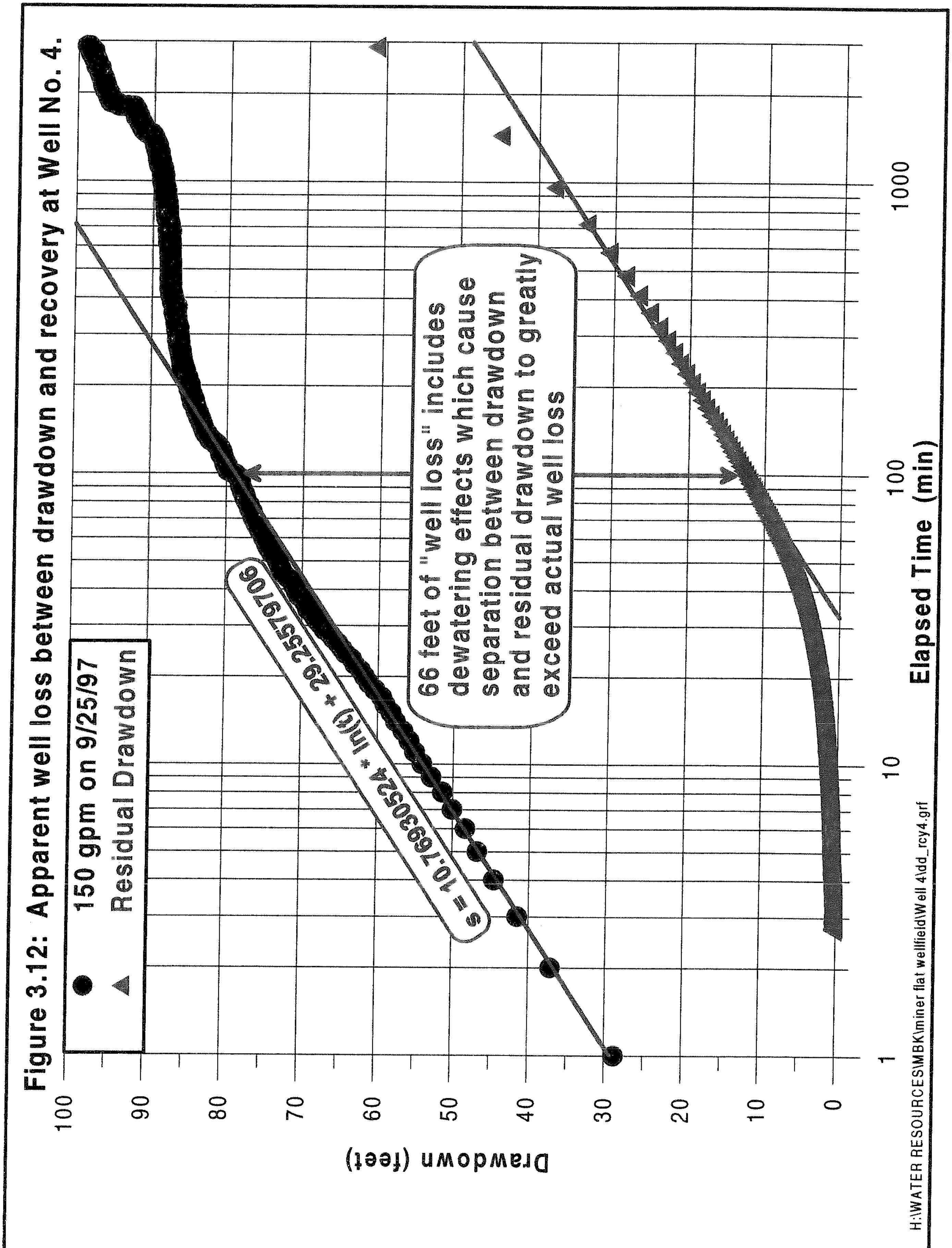


Figure 3.12: Apparent well loss between drawdown and recovery at Well No. 4.

Figure 3.13: Components of drawdown in Well No. 4.

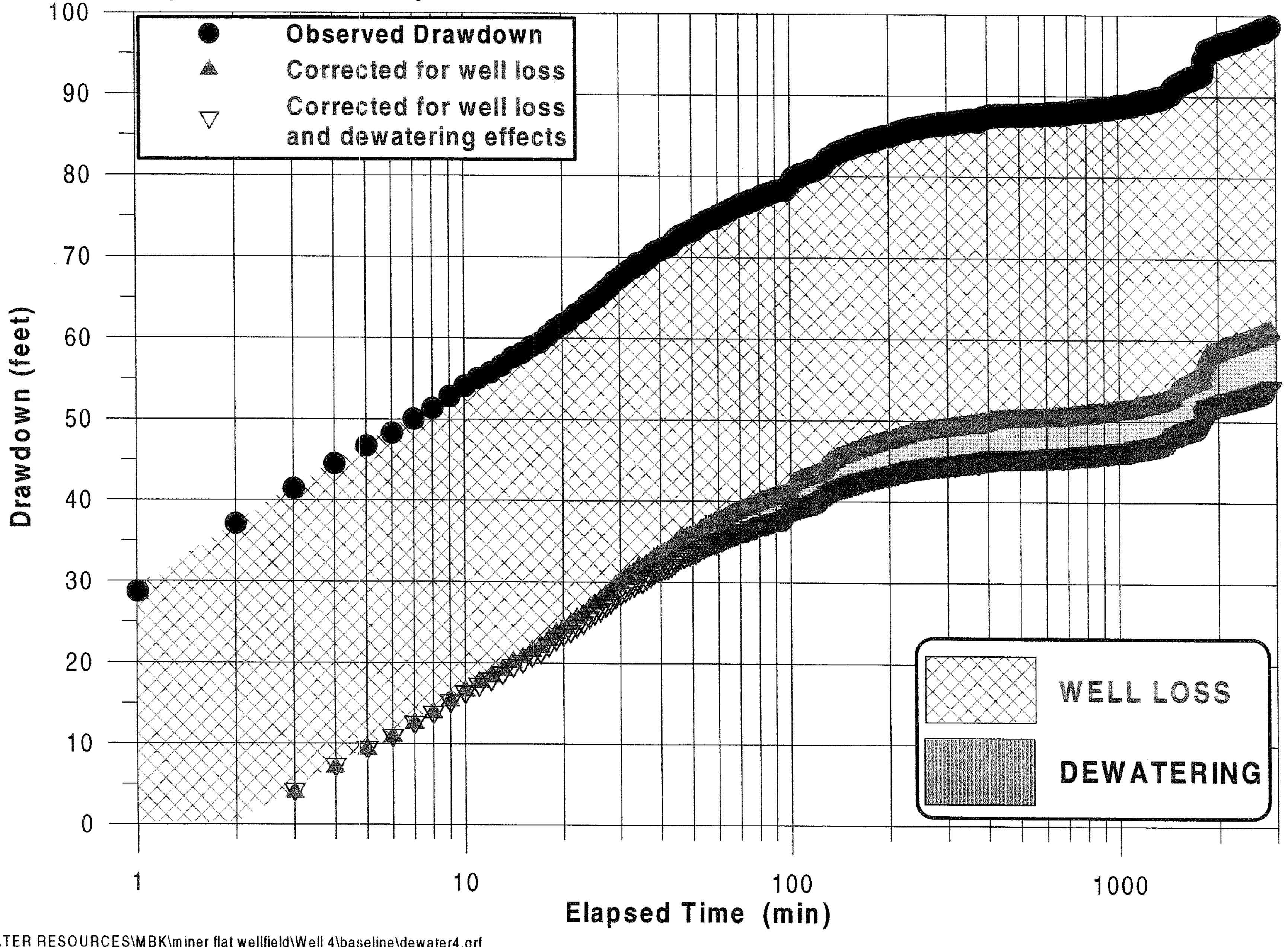
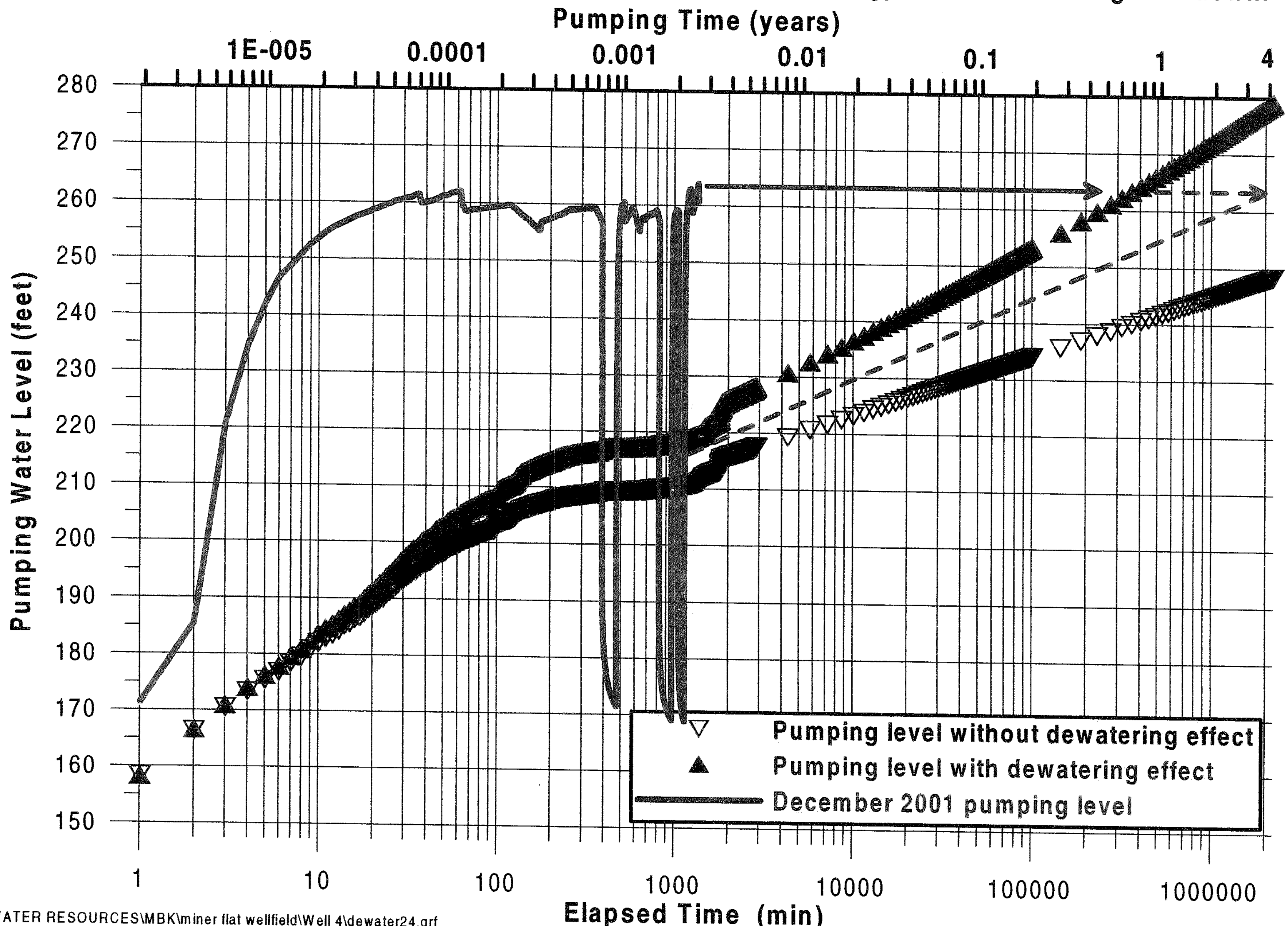


Figure 3.14: Projected pumping water level in Well No. 4 at 150 gpm with dewatering drawdown.



one for drawdown with well loss but no dewatering, and one for combined well loss of 37.5 feet and dewatering. The total drawdown with dewatering is obtained by application of Equation 3.10 to the confined drawdown prior to adding well loss.

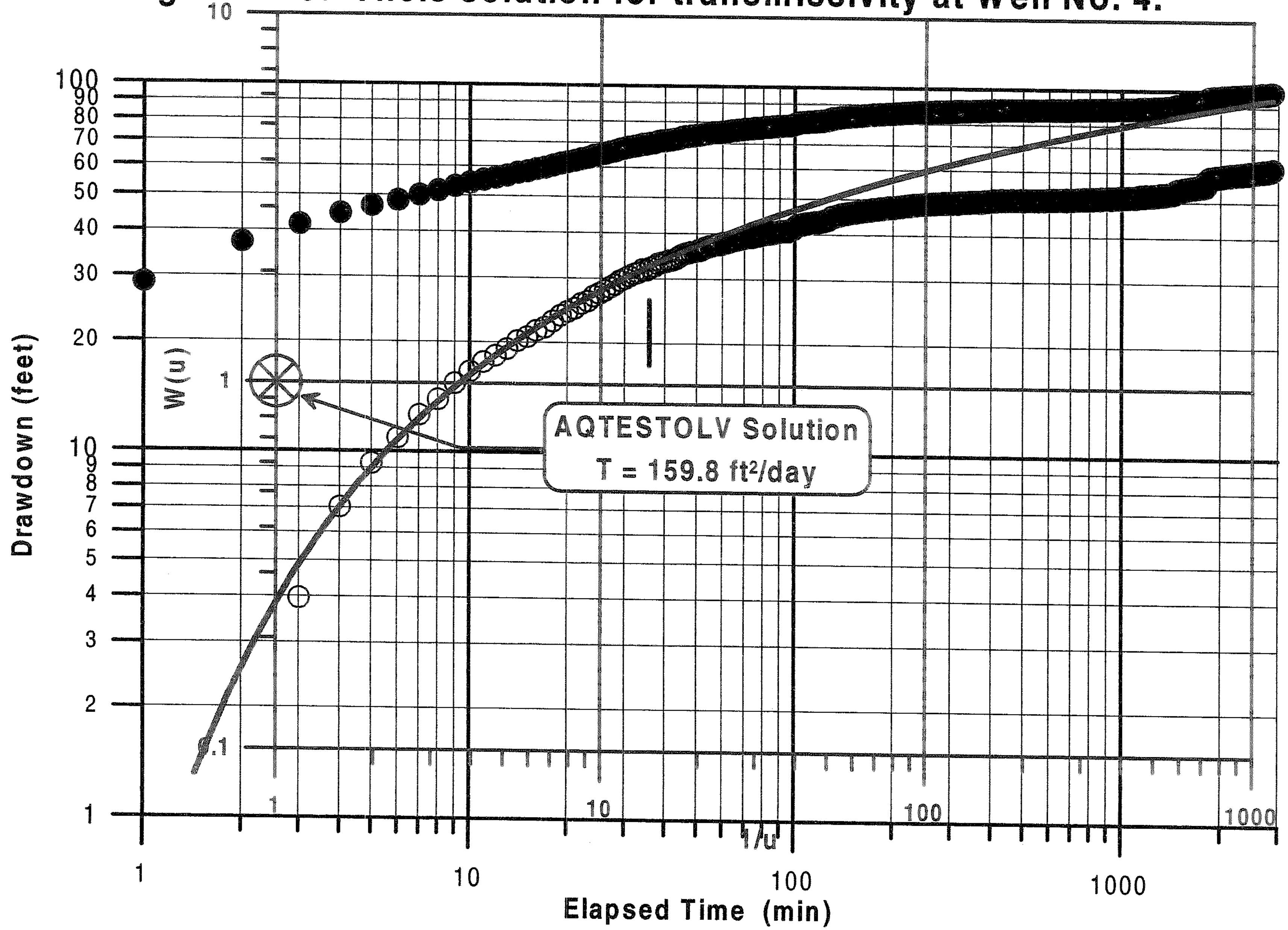
The December 2001 pumping water level plots about half way between the projections. The difference between the December 2001 pumping water level and the pumping water level at four years of projected drawdown with dewatering is due mostly to the fact that 37.5 feet of well loss is probably more than the actual well loss, as previously discussed. The decline in pumping rate from 150 gpm in 1997 to 88 gpm in 2001 as well as non-pumping periods may also play a role in the difference between projected drawdown and actual drawdown. A third factor resulting in overestimation of the 4-year drawdown is that the unconfined drawdown rate shown at the end of the 150 gpm test in 1997 probably would have decreased with increased pumping duration, following the pattern of the non-equilibrium type curve. However, application of the dewatering effect (Equation 3.10) to the confined flow drawdown predicts that continuously sustained pumping without recharge or recovery would result in a pumping water level essentially like that in December 2001, taking into account the foregoing three sources of over-estimation.

Accordingly, the December 2001 pumping water level is reasonably consistent with the drawdown that would have been predicted from the baseline tests, if the assumption were made that the aquifer would not receive recharge. In the absence of recharge, the baseline test projections do not include recovery due to recharge. Therefore, the reasonable agreement between actual pumping water level decline during a pumping duration of approximately four years and the projections of baseline tests without recharge, is consistent with the conclusion that recharge to the aquifer has been minimal since Well No. 4 was put into operation in 1997. is due mostly to the fact that 37.5 feet of well loss is

3.4.9. Aquifer Transmissivity

The transmissivity of the aquifer at Well No. 4 can be estimated, taking into consideration the onset of partially unconfined flow after about 40 minutes pumping time. The Theis nonequilibrium solution is applied to the confined flow response from the beginning of the test to 40 minutes, as shown on Figure 3.15. The type-curve fit exhibited on Figure 3.15 was obtained by applying AQTESOLV® software to the data, after the data were corrected for an estimated well loss of 37.5 feet. The value of transmissivity equal to 159.8 ft^2/day thus obtained is a minimum value of transmissivity. This is because the well loss correction of 37.5 feet is for the maximum possible estimated well loss. Application of smaller values of well loss would result in larger values of transmissivity. The well yield of 150 gpm at Well No. 4 compared to the well yield of 400 gpm at Well No. 3, where the transmissivity before unconfined flow occurred was 944 ft^2/day, suggests the transmissivity at Well No. 4 probably does not exceed about 350 ft^2/day, even if the well loss correction is reduced significantly.

Figure 3.15: Theis solution for transmissivity at Well No. 4.



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3.5. Well No. 5

Well No. 5 was put into service in January 1998. The well was tested at 200 gpm for a period of 24 hours on 10/29/97. Frankie Williams, Water Systems Operator, reports that the well typically produced "white water" if pumped for more than a few hours at a time or if it was pumped to the atmosphere through the local blow-off line. This indicates the well has historically pumped air throughout its service history. On 12/01/01, the discharge rate provided by the existing pump was 140 gpm and considerable air could be heard in the discharge line when the well was in operation. As described in detail below, this well exhibits a number of problems which indicate it should be replaced with a new well.

3.5.1. Geologic Log

A geologic log of the materials penetrated by Well No. 5 is provided on Table 3.7. Well No. 5 was logged by a Keith Shortall, Indian Health Service Engineer, from surface to 245 feet, and from 245 feet to total depth by Trevor Haig, geologist for Morrison-Maierle. The information on Table 3.7 is a highly condensed version of the geologic log that summarizes the most important information. The well penetrates 70 feet of colluvial deposits resting on top of the Coconino Sandstone. The base of the Coconino Sandstone at 286 feet rests on clayey siltstone with minor amounts of interbedded sandstone from 286 to 299 feet which comprise the uppermost part of the Supai Group. From 299 to 358 feet, sandstone in the Supai offers two water-bearing zones and some yield was obtained from a calcareous sandstone from 358 to 394 feet. Supai siltstone from 394 to 410 feet did not yield appreciable water. The static water level at 162.32 feet BGL implies an unconfined aquifer in the Coconino and Supai sandstones at Well No. 5.

Table 3.9: Production Well No. 5 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 70	CLAY, tan to brown	Colluvium
70 – 286	SANDSTONE, Tan, pink, brown, & reddish-brown with good water-bearing zone 220-248 ft	Coconino
286 – 299	SILTSTONE, dark brown clayey	Top of Supai
299 - 358	SANDSTONE, tan and brown, good water-bearing zone 337-345 ft	Supai
358 – 394	SANDSTONE, reddish-brown, calcareous	Supai
394 - 410	SILTSTONE, dark reddish-brown	Supai

3.5.2. Construction Data

Table 3.10 provides a summary of well completion data for Well No. 5. The well was completed to a total cased depth of 395 feet with 8-inch nominal diameter steel casing including 8-inch pipe sized 20-slot stainless steel well screen in the following intervals:

225 – 285 feet
300 - 390 feet

Colorado Silica 10-20 silica sand filter pack was installed in the annulus between the 12-inch diameter borehole and the casing and well screen from total depth to near the base of the pitless unit.

Table 3.10: Production Well No. 5 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6198
Tank overflow		6258
Static water level (swl)	162	6036
Top of well screens (BGL)	225	5973
Bottom of well screens (BGL)	390	5808
Pumping water level (pwl)	255	5943
Intake depth	320	5878
Drop pipe length	315	
Total cased depth	395	5803
Nominal pump capacity (gpm)	225	
Pump horsepower	25	

3.5.3. Construction History

Review of the field notes collected during drilling and construction of Well No. 5 provides an insight to some of the conditions observed at the well during the December 2001 investigations. The field notes were compiled by Trevor Haig, a geologist working for Morrison-Maierle, Inc.; under contract to Indian Health Services to provide geologic logging and assistance in selecting intervals to be screened and to conduct baseline pumping tests of the new wells. The notes were collected over a period of days as Well No. 5 was drilled and cased and while yield and drawdown tests were conducted concurrently at other well locations. Table 3.11 does not include all the field notes taken during construction and testing of Well No. 5 or their exact language, but paraphrases the notes pertinent to interpretation of December 2001 conditions at the well.

Table 3.11: Abstracts of field notes for Production Well No. 5.

9/24/97	12-inch surface casing set to 230 feet. Cement plug from 230-245 ft after cementing surface casing. Start drilling out plug and new hole w/12-inch nominal bit. At 256 feet stop drilling with engine problem, oil in radiator, cracked block. Take rig off-site for repair. Finish drilling 12-inch hole on 10/1/97 and 10/2/97. Total depth drilled is 410 feet.
10/14/97	Step test Production Well No. 5. Install test pump inlet at 388 feet BGL and PXD at 365 ft BGL. SWL = 165.32 BTOC (162.32 BGL). Also monitor test well No. 10 which is 18.5 ft from Production Well No. 5. At step 4, break suction at 210 gpm. Stop test. *[test stopped due to poor well performance]*
10/15/97	Pull pump inlet up 60 feet to 328 ft. At 160-170 gpm makes 0.1 ml/l sand. At 180 gpm, <0.1 ml/l sand but turbidity from air and possibly bentonite. After 25 minutes at 180 gpm, break suction and valve back to 150 gpm, "slug of sand coming up". Surge with pump for about one hour.
10/15/97	Prod. Well #5: Development. Crew has pulled pump ran SAPP into hole, swab w/surge block up & down screened interval.
10/16/97	Prod. Well #5: Development. Crew runs surge block up & down screened interval. About 3' fill in bottom of well since last night. Top of gravel pack at 115 feet.
10/28/97	12:30 Pump test guys arrive. Run swab along screen, remove sand with bailer. Install test pump with inlet at 364 feet. Sounded hole while bailing with dart valve bailer. Top of sand at 384 ft. Bail to 385 ft after another hour of bailing. Stop bailing, not effective. Run pump inlet to 364 feet.
10/29/97	The voluminous pumping test notes are not replicated herein; however, pertinent notes are summarized. Sand production at rates of 150, 160, 170, 180, 190, 200, and 210 gpm ranged from 0.1 - 0.3 ml/l and increased to 0.5 ml/l at 220 gpm. Moderate to high turbidity characterized as "Turbidity from bentonite" was observed through all steps. Some air was noted at 210 and 220 gpm steps. Step 9 at 230 gpm broke suction after 14 minutes and produced "sand, dirty discharge" throughout its duration. The well was surged with the test pump for about two hours, finally producing 0.1 ml/l sand at 230 gpm. After 2.5 hours of recovery, a 200 gpm constant rate test was started, initially producing 2.0 ml/l sand, decreasing to 0.1 ml/l after 8 minutes of pumping. The following day the discharge was "clean, free of sand"

The field notes summarized on Table 3.11 indicate some potentially serious problems with the well. Although the top of the filter pack outside the screen is measured at 115 feet after development with a surge block, the well continues to produce sand to the extent that the lowermost 5 feet of the well screen is full of sand when the test pump is installed. The fact that the bailer used on 10/28/97 could not remove the sand indicates sand was coming into the well as fast as the bailer could remove it.

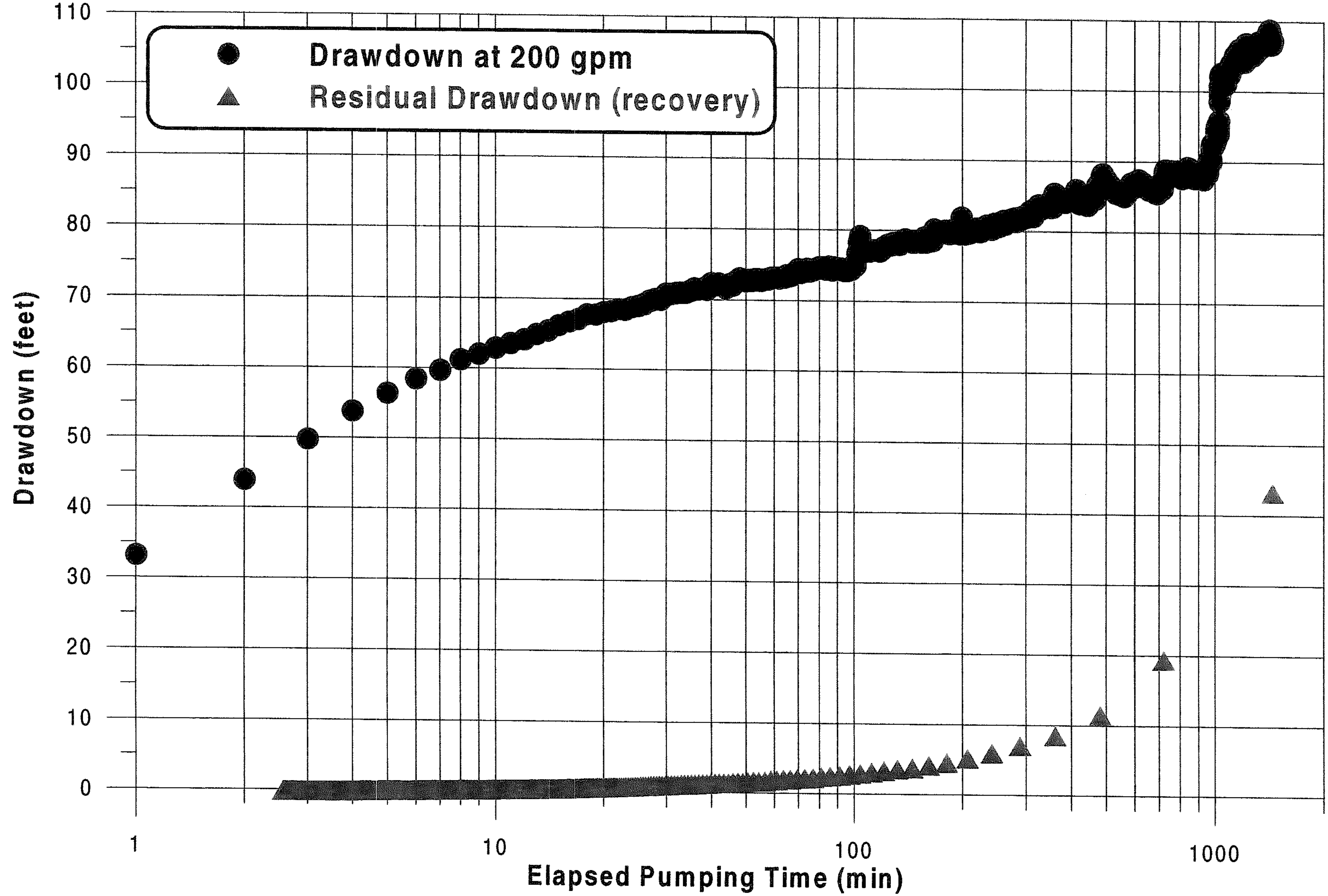
It is unlikely that sand could flow from the aquifer formation, through the commercial silica sand filter pack, and into the well screen at the rate necessary to render bailing ineffective. Accordingly, the ineffectiveness of the bailer on 10/28/97 suggests that sand from the formation was flowing into the well through an interval where filter pack was not present along the entire length of the well screen. It is not unusual for filter pack to form a hanging bridge in the annulus. This allows pack below the bridged material to settle during development. The use of 20-slot screen and 10-20 Colorado Silica Sand indicates the screen would retain more than 95 percent of the filter pack. Accordingly, settlement of the filter pack along the entire length of the 150 feet of screen in the well would be less than 7.5 feet during development.

On 10/15/97, during the second of three stepped rate tests, the sudden onset of "a slug of sand" being produced after throttling the discharge rate from 180 to 150 gpm, is not entirely consistent with a bridged pack. Initially, the well seems to clean up, although at a lesser discharge rate than initially anticipated, and then suddenly begins to produce "a slug of sand". This later response may be related to a buildup of formation sand on the outside of the well screen suddenly being sucked through the screen. This suspicion is reinforced by the performance of the well during the 200-gpm constant rate pumping test. After 1027 minutes (17 hours and 7 minutes) of pumping at 200 gpm, the pumping water level in the well abruptly departed from the time-drawdown rate, going down at an alarming rate as shown on Figure 3.16. In a 12/9/97 memorandum to the Indian Health Service, Morrison-Maierle explained this as a pump problem, stating:

> "***The rate of drawdown increased drastically after 1027 minutes of pumping. The apparent increase in drawdown has the characteristics of interference from a nearby well being turned on. The nearest well (Production Well #2) is 1,000 feet away and subsequent testing proved that impact while pumping is negligible. The other possibility is a negative boundary, but the sharp increase of 7 feet over a 5 minute interval is more abrupt than what could be attributed to a negative boundary.***
>
> ***The erratic data after 1027 minutes is probably due to electronic interference between the pump cable and transducer cable. The pump used in this test broke down during later testing of another well where the problems were attributed to worn-out bearings. It is possible that the pump had begun to wear out after 1027 minutes of testing production of well #5. The increase in amps needed to turn the failing pump were effecting the transducer cable through induction.***" (December 9, 1997 memo from Trevor Haig, Morrison-Maierle to Keith Shortall, Indian Health Service District Engineer)

Apparently the sudden increase in drawdown rate occurred when no one was present at the well to notice if it was accompanied by a transient increase in suspended sediment in the discharge. Likewise, there is no manual groundwater level measurement to verify

Figure 3.16: Time-drawdown response of Well No. 5 at 200 gpm on 10/29/97.



H:\WATER RESOURCES\MBK\miner flat wellfield\Well 5\dd5_resid.grf the speculation that the logger produced erroneous data. It is apparent that the latter conclusion was reached only after the test was stopped and the aberrant well response was discovered while processing the data. It is just as likely that the data are correct and the well was responding to an actual event during the test. An alternative explanation for the sudden increase in drawdown rate, not considered in the 12/9/97 letter, is that bridged filter pack material suddenly collapsed into a void that had formed around the well screen. Equally plausible is that a void around the well screen was suddenly filled with sandstone rubble caused by a collapse of the aquifer formation. The significant increase in the rate of drawdown indicates that the phenomenon was not restricted to a small length of the well screen.

3.5.4. December 2001 Well Conditions

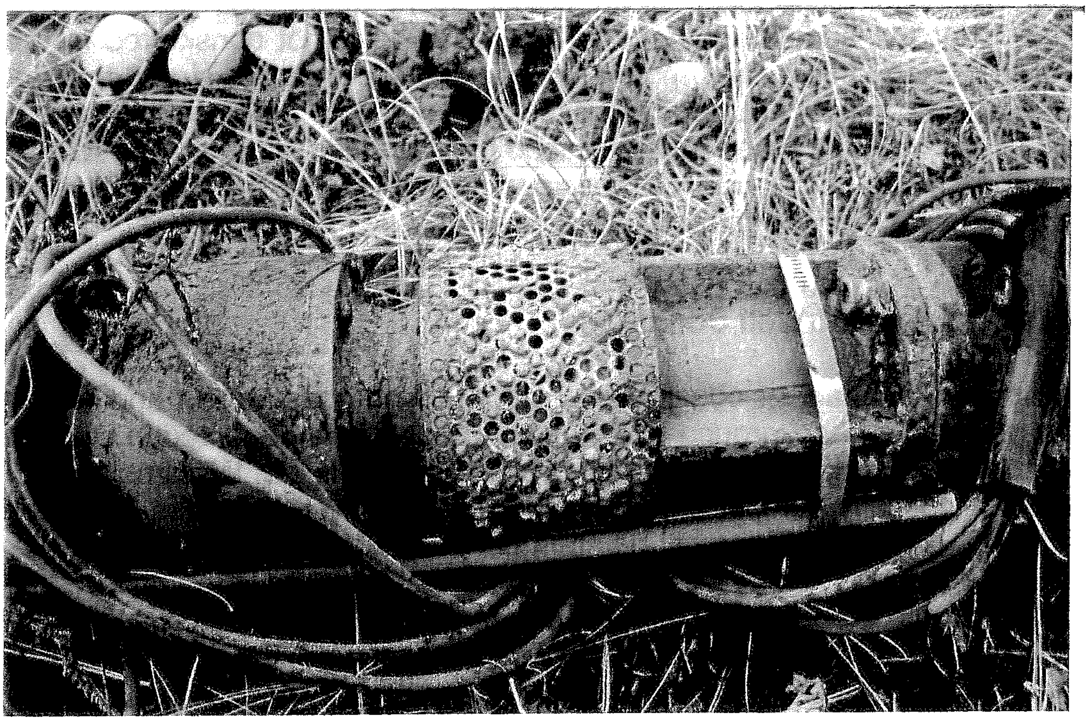
The foregoing summary of observations made during construction and testing of Well No. 5 is provided as background information for conditions observed in December 2001 and reported by Frank Williams, Water System Operator. Mr. Williams states that Well No. 5 has historically pumped aerated water into the system unless its operating time was limited. This is considerably different performance than predicted by the October 1997 pumping tests and may indicate that the abrupt increase in the rate of drawdown at 1027 minutes into the 24-hour constant rate test was a real phenomenon, not a logger failure. Again, collapse of material into a void around the well screen would be consistent with the loss in observed production.

A December 2001 inspection of the pump pulled from the well earlier in 2001 provided the final clue as to the problems with Well No. 5. Photographs 3.7, 3.8, and 3.9 show the strainer around the pump inlet on the pump removed from Well No. 5. Although some sand from the land surface has attached to the strainer where it was resting on the ground prior to the photographs being taken, the photographs show rounded fragments of sandstone from the aquifer matrix plugging the holes in the strainer. The rounded fragments of sandstone are much larger than the well screen slot size. Therefore, there is only one way they could have entered the well and that is through a damaged well screen.

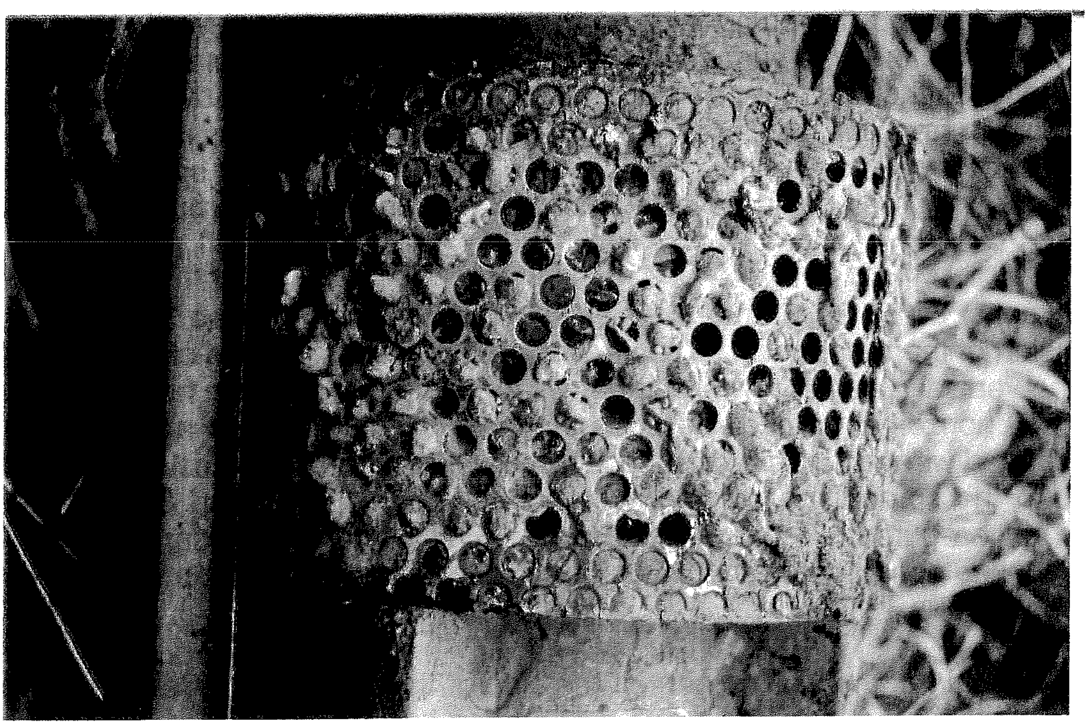
Photographs 3.10, 3.11 and 3.12 show that one side of the pump contains an impression of the well screen. The vertical grooves in the outside of the pump body are from the vertical rods in the well screen wearing into the cast iron housing of the pump. The small horizontal grooves between the large vertical grooves are an impression of the v-shaped wire which is continuously wrapped around the vertical rods to provide the slots into the screen. Accordingly, Photographs 3.10, 3.11 and 3.12 show that the motion of the pump, each time it torques during starts and stops, and as it vibrates slightly during operation, has worn an impression of the inside of the well screen into the pump body.

The impression of the well screen on the side of the pump from Well No. 5 shows that either the well is not straight or the well screen has partially collapsed. Although the

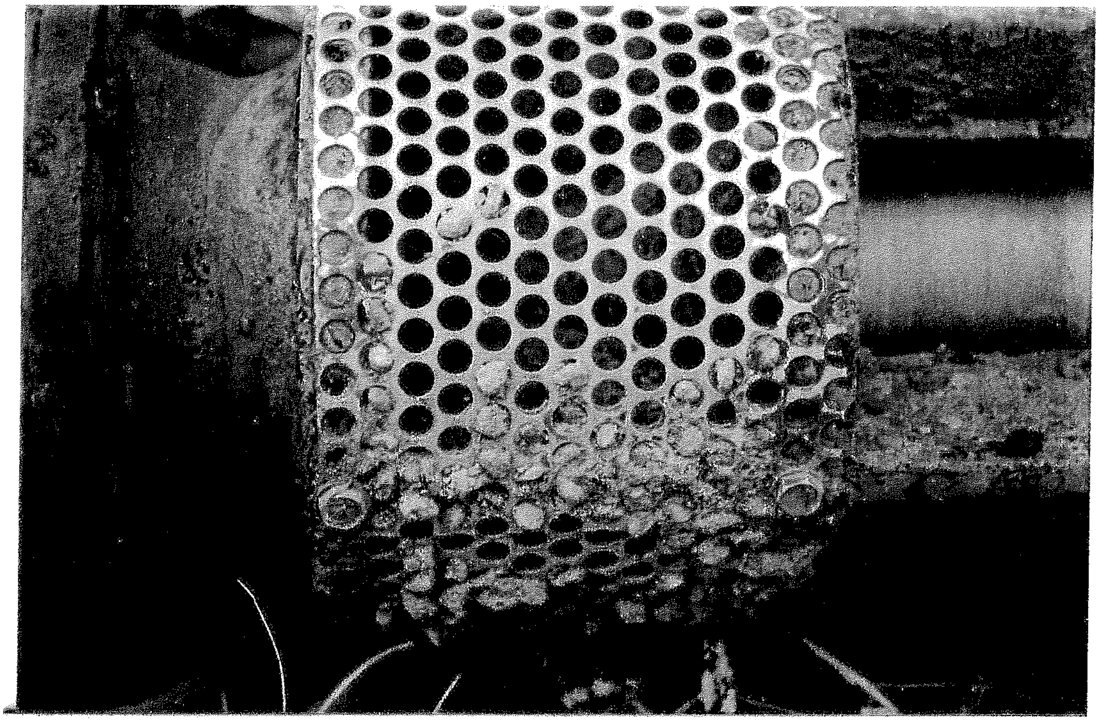
Photograph 3.7: Strainer on Well No. 5 pump inlet partially plugged with sandstone fragments.



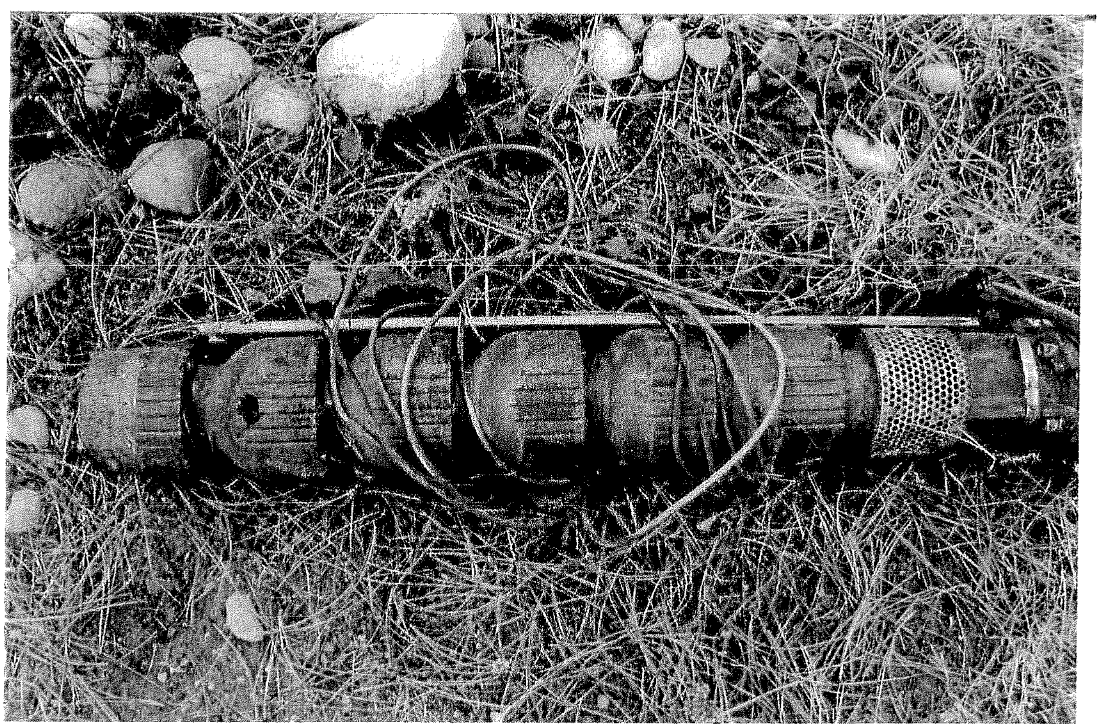
Photograph 3.8: Close-up view of sandstone fragments in pump inlet strainer.



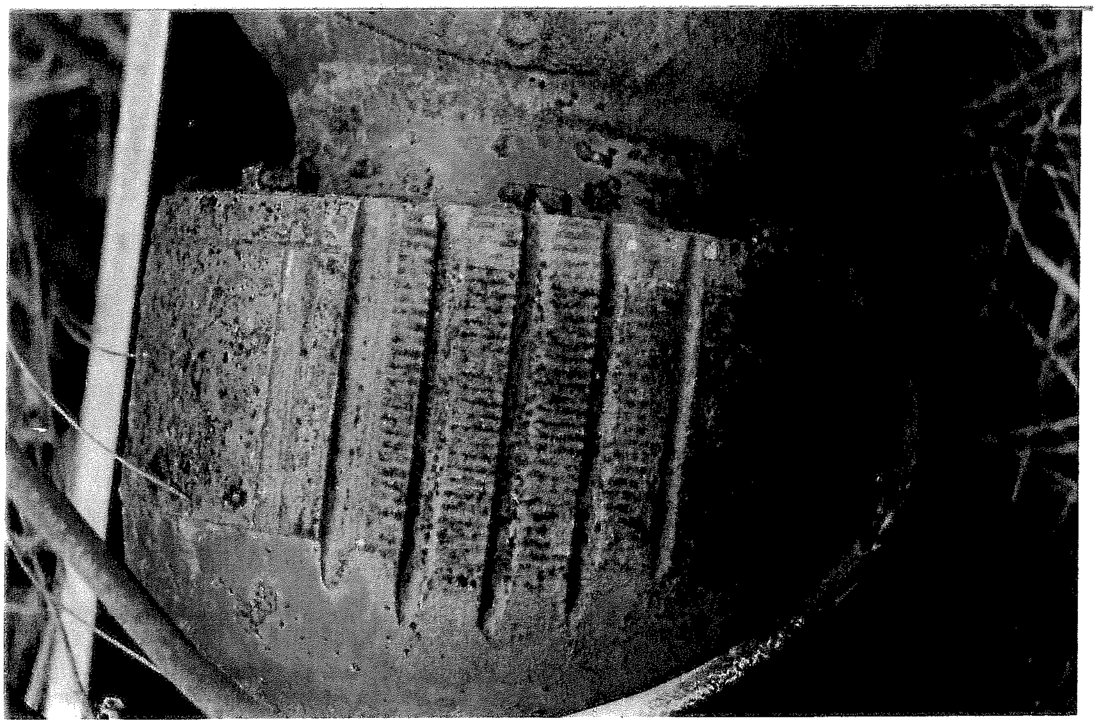
Photograph 3.9: Close-up of Well No. 5 pump inlet strainer.



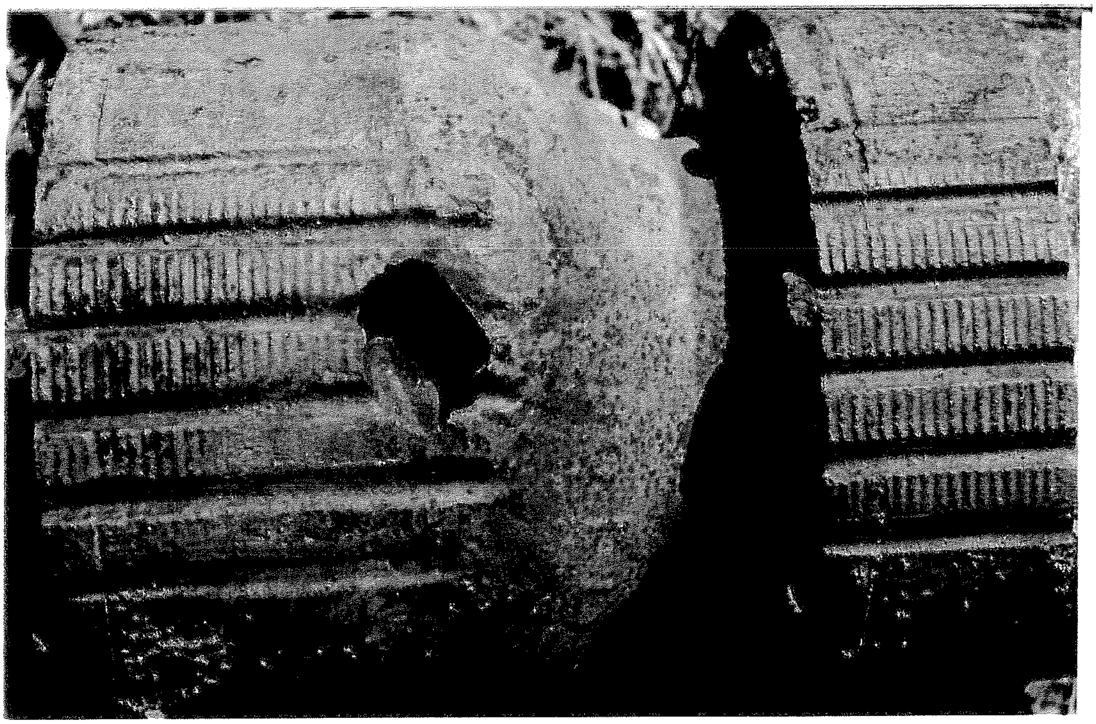
Photograph 3.10: Pump from Well No. 5, strainer at bottom end.



Photograph 3.11: Close up of screen imprint in pump body at Well No. 5.



Photograph 3.12: Hole through top impeller housing of Well No. 5 pump.



available information is not diagnostic, it is well known that operating a surge block inside a well screen, as documented in the field notes on Table 3.11, offers a risk that suction on the low-pressure side of the surge block may partially or fully collapse the well screen. Partial collapse of the screen into an oval shape is usually accompanied by separation of some of the v-shaped wire from the vertical rods, thus providing openings into the well screen that are much larger than the original slot size.

The impression of the well screen on one side of the production pump and the presence of relatively large fragments of sandstone in the pump strainer on Well No. 5 are conclusive evidence of damage to the well screen. It is not known if the screen was damaged due to partial collapse during application of the surge block for development or simply damaged by rough handling when it was installed. Another possibility is that it has been damaged by contact with the pump, although there is no evidence of this in the impression on the pump. The field notes indicating that SAPP (sodium acid pyrophosphate) was applied during development indicates that bentonite fluid was used during drilling of the borehole since SAPP is used to break down bentonite additives. This in turn indicates the drillers switched from air rotary drilling to mud rotary drilling, a change that is usually the result of unstable, caving formation that will not support an open borehole when drilling with air. Accordingly, this is evidence of serious borehole instability and caving when the well was drilled.

The presence of sandstone fragments in the pump strainer indicates that filter pack is not present around a significant area of the screen, perhaps not anywhere above the damaged portion of the well screen. This, and the evidence of a damaged well screen, indicates the well probably should be replaced by a properly constructed well for the simple reason it is pumping sand. As long as the well is pumping sand, it will damage pumps. It would be worthwhile to obtain a downhole video log of this well to determine the nature of the damage to the screen before making a final decision about a course of action, but the most likely outcome is that the well should be replaced.

Additional information about the well condition was obtained during an attempt to install a 1-inch diameter standpipe into Well No. 5 on 11/29/01. The existing pump was lifted with a pump truck until the pitless unit spool was five to six feet above the top of the well casing. This permitted installation of the 1-inch standpipe down the annulus between the 8-inch well casing and the 4-inch pump column. The standpipe encountered an obstruction at about 250 feet where no amount of maneuvering of the pump from side to side, up and down, and so forth (including rotating the pump and standpipe) would allow the 1-inch standpipe to advance below 250 feet from any location around the well. The tight spot at 250 feet BTOC is about 25 feet below the top of the uppermost well screen and may indicate partial collapse or bending of the screen.

After the attempt to install a standpipe was abandoned, the pump and pump column was lowered back into the well on the afternoon of 11/29/01. The field notes show that the pump stopped on an obstruction at 300 feet. Evidently, one piece of pump column pipe was left out of the well when the new pump was installed, or else our field notes show the pump 20 or 21 feet too shallow. We have lost track of the reason the field notes show the pump hanging up at 300 feet, rather than 315 feet. The depth of 300 feet is the top of the lowermost well screen, a fact that may have influenced the field notes. In any event, when the pump and pump column were rotated manually with a pipe wrench, the pump motor fell off whatever it was resting on and was lowered down the well until the pitless unit spool seated in the pitless unit body. This experience shows that something is wrong at the top of the lower well screen or 21 feet below the top of that well screen as the inside of the well screen and/or casing should not provide enough of an irregularity to catch the bottom of the pump motor, even if the well is grossly out of plumb. There is either an alignment problem, a partial collapse, a damaged well screen, or a separated weld where the pump and motor stopped going down the well.

3.5.5. Pump Condition

Photographs 3.10 and 3.12 show the impression of a well screen worn into the pump removed from Well No. 5 as well as a hole through the housing around the impellers on the uppermost stage of the 5-stage pump. Although sand may have played a role in enlarging the hole by erosion, the hole is typical of the damage caused by air entrained in the water passing through the pump. The damage is caused by a process called cavitation.

Cavitation can occur from two different but similar causes. One cause is any flow condition in the pump that reduces local pressure in the pump below the vapor pressure of the water. This causes part of the water flowing through the pump to vaporize (boil) at the low pressure area. Low pressure typically occurs at an area of excessive increase in the flow velocity through the pump. Although several different things may cause in the increase in velocity, the drop in pressure below the vapor pressure results in a stream of vapor pockets flowing through the pump. When the vapor pockets, which are in themselves low pressure pockets, flow into an area of the pump with higher pressure, they collapse. The collapse is so violent it plucks metal from the pump bowel and impeller surfaces and causes pitting. The collapse of the vapor pockets is called cavitation.

A second way cavitation occurs is entrainment of air bubbles in the water entering the pump inlet. As the pressure in the column of water flowing through the pump is increased by the force of the impellers, the entrained bubbles collapse, causing cavitation and damage to the pump. In view of Frank William's observations of "white water" discharging from the pump, there is little doubt that the hole in the uppermost pump bowl at Well No. 5 (Figures 3.10 and 3.12) was caused by entrained air and cavitation. It should be noted that the location of the highest pressure in a vertical turbine submersible pump like that used in Well No. 5 is in the uppermost pump bowl and is therefore where cavitation is likely to be most severe with entrained air. When cavitation is due to excess velocity at the pump impeller (instead of entrained air), the cavitation damage usually occurs in the first or lowermost stage of the pump bowls. Therefore, the fact the hole is in the uppermost stage of the pump is an indication that the cause is probably entrained air, rather than cavitation caused by excess flow velocity at an impeller.

3.5.6. Recommended Pump Size

In view of all the condition of Well No. 5, it is not desirable to pump the well as the sand production will continue to damage the pumps. In part this is because it was not possible to determine the pumping water level during operation of the well during its operation in December 2001. However, it is mostly because of the evidence that the well not only pumps air at 140 gpm, a condition damaging to the pump, but that the well also is pumping rather large fragments of sandstone derived from the aquifer matrix.

The amount of decrease from the original design rate of 225 gpm due to increased head on the pump versus damage to the pump cannot be determined without a measured pumping water level. The original pump was a Goulds 6CHC025 for which a performance curve is shown on Figure 3.7. If the replacement pump is the same pump, it should have the capability to produce 165 gpm with the pumping water level in the well at 320 feet (assuming one piece of pump column was left out of the well). Thus, the observed yield of 140 gpm either indicates damage to the replacement pump or that the pump is pumping entrained air and will soon be damaged. Accordingly, a replacement pump capacity of 50 gpm is recommended, taking into account the uncertainties involved and the annual water level decline of 10.4 ft/yr at this well.

3.5.7. Baseline Performance

The poor performance of Well No. 5 and the abrupt increase in drawdown at the end of the 200 gpm constant rate test indicate the performance of the well has not been equal the baseline test performance. However, part of the results of the baseline tests are provided herein to document the strong unconfined response of the aquifer to pumping, including dewatering effects. Figure 3.17 shows a Hantush-Bierschenk plot of the step test data which exhibits considerable departure from a straight-line fit. The data do not plot as a straight line; however the nature of the departure cannot be explained by as simple an explanation as changing the exponential on the discharge rate from a value of 2 (rate squared) to some other exponential. The compound nature of the departure of the Hantush-Bierschenk plot from a straight line implies complex factors affecting the aquifer response, even in the limited duration of the stepped rate test.

Figure 3.18 shows a Birsoy-Summers plot of the step test which exhibits a classic dewatering response for an unconfined aquifer which begins after 12 minutes of pumping time and does not abate throughout the rest of the test. The steps are essentially continuous from one pumping rate to the next, indicating there is no initial adjustment of a confined versus unconfined part of the cone of depression at the beginning of each new pumping rate as occurred at the other wells discussed thus far in this report. Figure 3.19 shows the constant rate test response at 200 gpm projected into the future for four years of continuous pumping. A second projection is shown with the projected pumping water level adjusted for the increased drawdown that would occur due to dewatering effects predicted by Equation 3.10.

Figure 3.17: Hantush-Bierschenk plot of Prod. Well No. 5 Step test on 10/29/97.

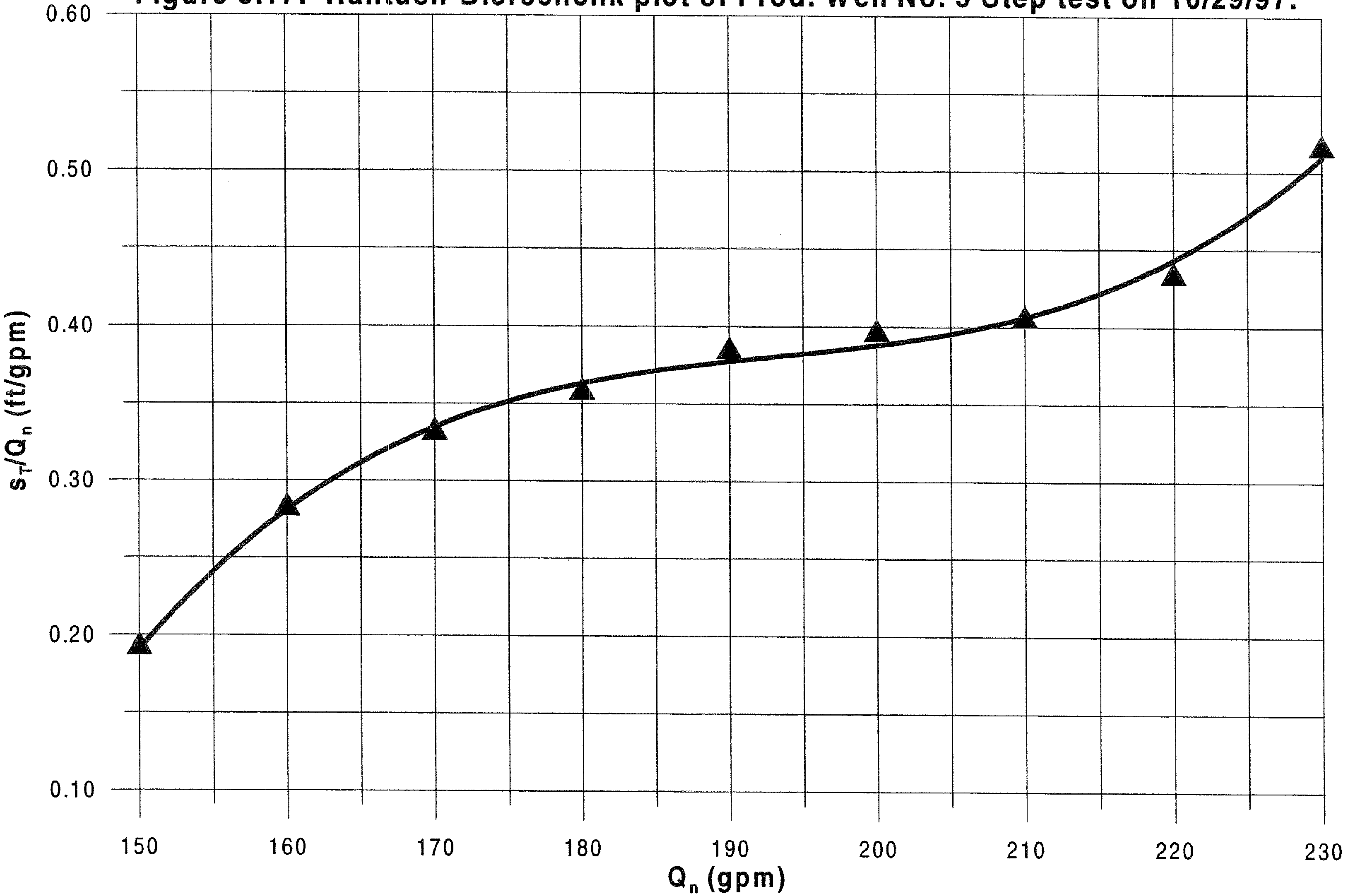


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Figure 3.18: Birsoy-Summers plot of 10/15/97 step test Prod. Well No. 5.

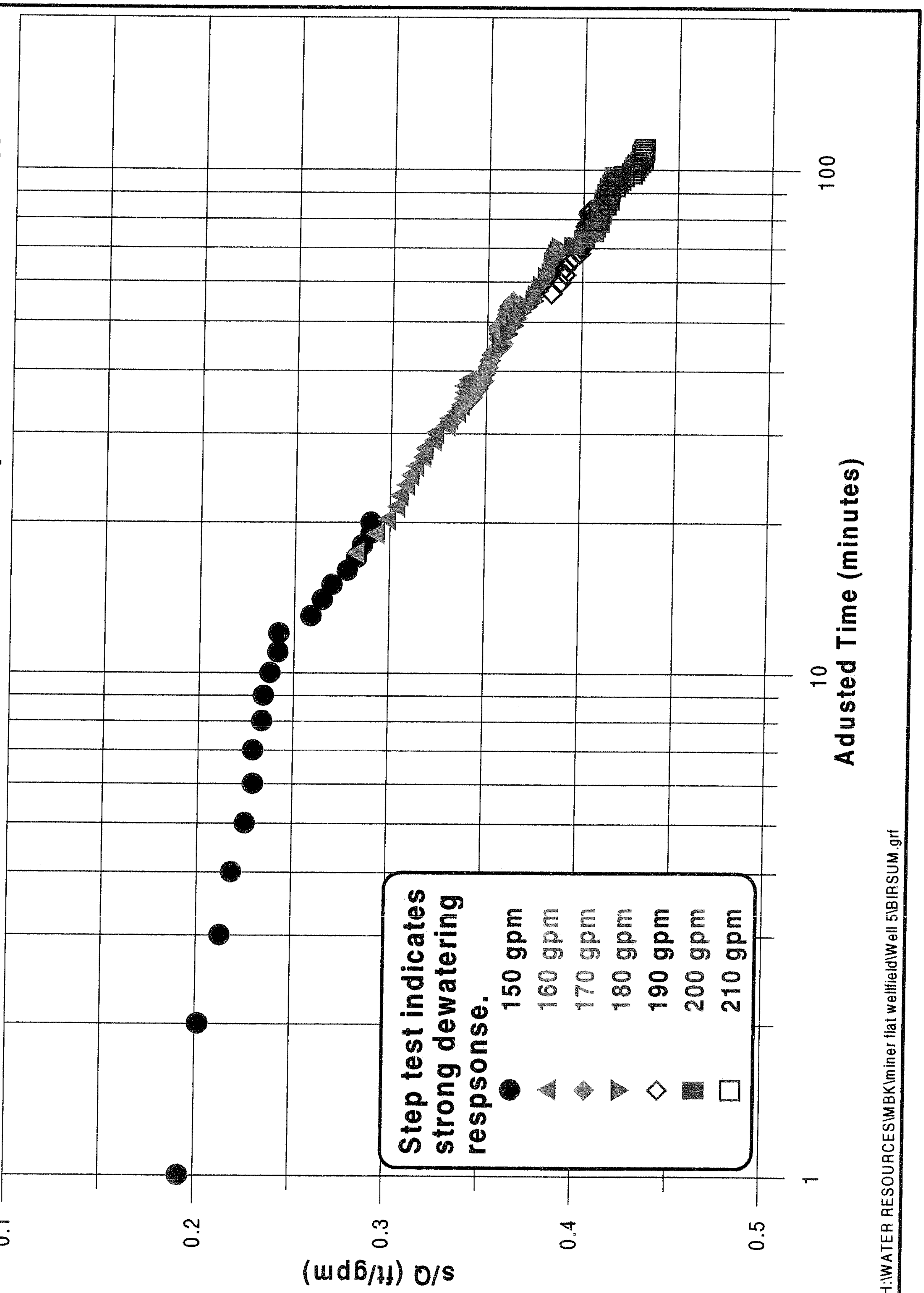
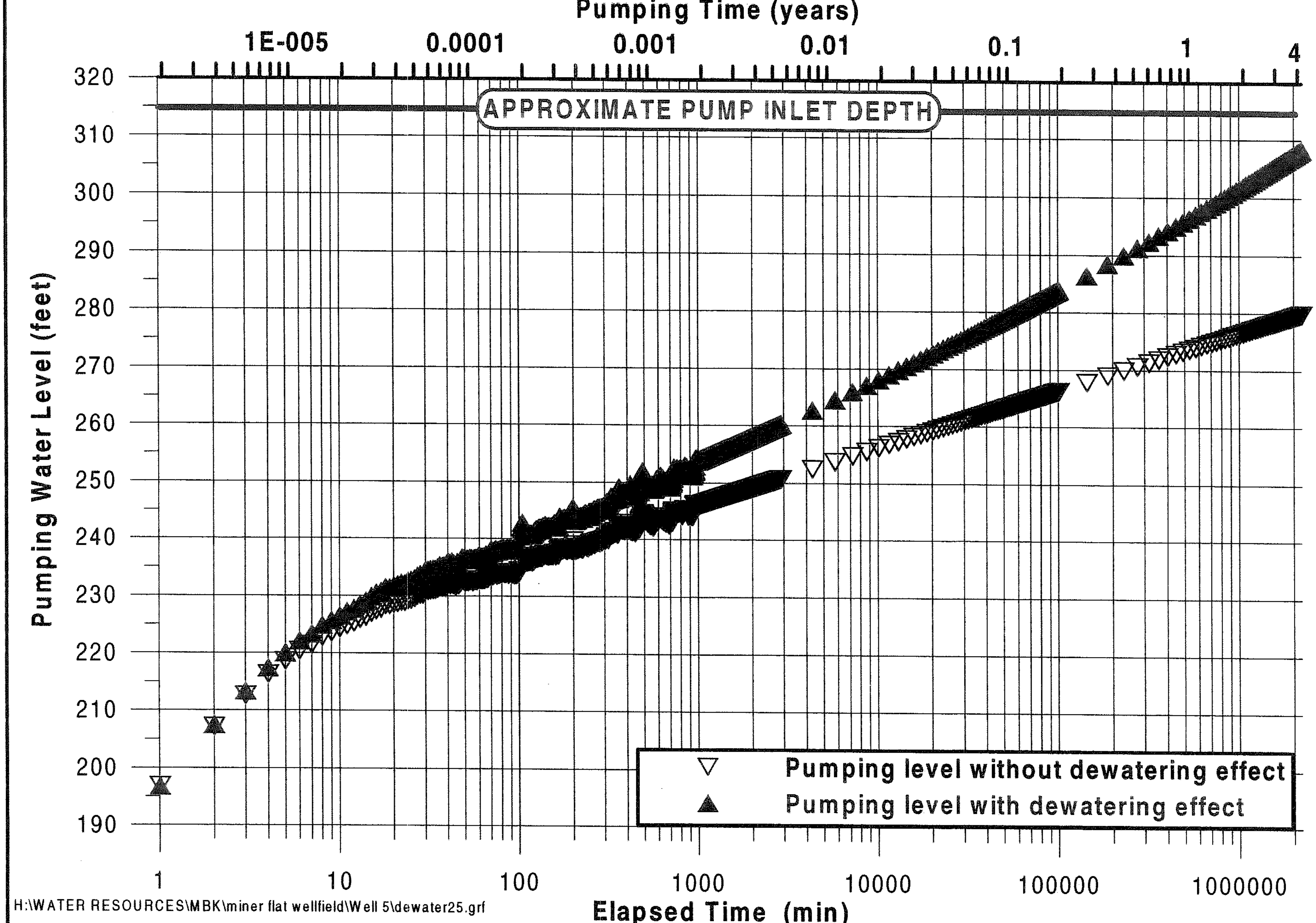


Figure 3.19: Well No. 5 projected PWL with and without dewatering effects.



The projections on Figure 3.19 do not take into account the abrupt increase in drawdown observed after 1027 minutes of the baseline test (Figure 3.16). If the increase in drawdown at the end of the constant rate test is regarded as an abrupt increase in well loss of about 15 feet, the pumping water level projected at the end of four years of pumping is at the design pump inlet depth of 320 feet (Table 3.10). Even without the estimated 15 feet of increased well loss, the projection indicates that with dewatering effect, the pump in Well No. 5 would start to break suction and pump air after about 4 years of continuous pumping, notwithstanding the effects of cascading water (if any) from dewatered well screen.

3.6. Well No. 6

Well No. 6 was completed 6/28/97 and put into service in January 1998. Baseline stepped rate testing of the well was completed on 7/2/97 at rates from 300 to 500 gpm. On 7/3/97 a constant rate test was started at 400 gpm. After 127 minutes, the constant rate was reduced to 350 gpm and the test was continued for a total pumping time of 24 hours. Based on the baseline tests, a peak pumping rate of 350 gpm was recommended for the well for up to 69 days, presuming the long-term average pumping rate would be one-half to two-thirds the peak rate. Frankie Williams, Water Systems Operator, reports that this well has operated with no problems.

The December 2001 evaluation of the well determined that the gate valve which isolates the well from the main water transmission line was stuck in the open position. Accordingly, the well could not be isolated from the system nor could its discharge rate be regulated with the valve. It was observed that the well operated continuously, 24-hours per day, during the period 11/28/01 through 12/04/01 while the wellfield evaluation was in progress. Measurement of the well discharge rate from the flow meter at the control panel at the main treatment building indicated the well was producing 162 gpm on 12/01/01, after the pumping water level stabilized.

3.6.1. Geologic Log

A geologic log of the materials penetrated by Well No. 6 is provided on Table 3.12. Well No. 6 was logged by a Keith Shortall, Indian Health Service Engineer, from surface to 190 feet, and from 190 feet to a total depth of 350 feet by Trevor Haig, geologist for Morrison-Maierle. The information on Table 3.12 is a highly condensed version of the geologic log and summarizes the most important information. The well penetrates 40 feet of colluvial deposits resting on top of the Coconino Sandstone. The base of the Coconino Sandstone at 206 feet rests on three feet of laminated sandstone interbedded with red clay from 206 to 209 feet which comprise the uppermost part of the Supai Group. Below 209 feet, to the penetrated depth of 350 feet, the Supai consists of red sandstone and siltstone. Principal water-bearing zones are from 185 to 200 feet in the Coconino Sandstone and 230 to 290 feet and 310 to 337 feet in the Supai. The 1997 static water level at 132 feet BGL implies an unconfined aquifer in the Coconino and Supai sandstones at Well No. 6.

Table 3.12: Production Well No. 6 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 40	CLAY, tan to brown	Colluvium
40 - 200	SANDSTONE, tan and red with good water-bearing zone 185-200 ft	Coconino
200 - 201	CLAY, red and gray with sandstone fragments, probable shear zone.	Shear zone in Coconino
201 - 206	SANDSTONE, tan and red with good water-bearing zone.	Coconino
206 - 209	CLAY & SANDSTONE, bright red.	Supai
209 - 226	SANDSTONE, red, fine-grained	Supai
226 - 230	SILTSTONE & SANDSTONE, red, tight	Supai
230 - 305	SANDSTONE, red, fine-grained, good water-bearing zone 230-290 ft, clayey laminae below 290 ft	Supai
305 - 310	SILTSTONE, dark red, some clay	Supai
310 - 337	SANDSTONE, red, fine-grained, good water-bearing zone.	Supai
337 - 350	SILTSTONE, dark red, laminated	Supai

3.6.2. Construction Data

Table 3.13 provides a summary of well completion data for Well No. 6. The well was completed to a total cased depth of 342 feet with 8-inch nominal diameter steel casing including 8-inch pipe sized 20-slot stainless steel well screen in the following intervals:

180 - 200 feet
240 - 290 feet
317 - 337 feet

Colorado Silica 10-20 silica sand filter pack was installed in the annulus between the 12-inch diameter borehole and the casing and well screen from total depth to near the base of the pitless unit.

The 6/27/97 field notes indicate the interval from 250 to 260 feet required 75 sacks (75 cubic feet) of 10-20 silica sand to fill a void around the well screen.

Table 3.13: Production Well No. 6 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6142
Tank overflow		6258
Static water level (swl)	132	6010
Top of well screens (BGL)	180	5962
Bottom of well screens (BGL)	337	5805
Pumping water level (pwl)	225	5943
Intake depth	297	5845
Drop pipe length	294	
Total cased depth	342	5800
Nominal pump capacity (gpm)	350	
Pump horsepower	40	

3.6.3. Water Levels

Figure 3.20 shows the 7/03/97 static water level and pumping water level at 400 gpm and 350 gpm in Well No. 6 compared to the static water level and pumping water level at 162 gpm on 12/01/01. The static water level on 12/01/01 was obtained by stopping the pump in Well No. 6 for 1 hour and 45 minutes. This was the only time the well was not pumping during the 7-day evaluation of the wellfield. Figure 3.20 shows that Well No. 6 exhibits the same pattern of decreased well yield and decreased hydraulic performance exhibited by the other wells in the wellfield, all associated with a decrease in saturated aquifer thickness. The 12/01/01 static water level was approximately 75 feet lower than the 7/03/97 level. Although the yield of the well with about 25 feet of submergence remaining over the pump inlet was only 162 gpm, the 12/01/01 pumping water level was 50 to 55 feet deeper than that on 7/03/97 at 350 gpm.

Figure 3.20 shows the pumping water level in Well No. 6 fluctuating between five and eight feet with abrupt five-foot increases in the pumping water level occurring several times in a one-day period. During baseline tests of Well No. 7 on September 5 and 6, 1997, the observation well next to Well No. 6 was monitored for the effects of pumping Well No. 7. It was observed at that time that Well No. 7 did not have an effect on Well No 6; however, Well No. 1 caused as much as 14 feet of interference drawdown at Well No. 6 in a 24-hour period. Accordingly, the fluctuations in the pumping water level at Well No. 6, as depicted on Figure 3.20, are thought to reflect the influence of Well No. 1 on Well No. 6.

Figure 3.21 shows a specialized plot used to evaluate the aquifer response for evidence of linear flow such as might be caused by fractures or boundary conditions. The aquifer response on Figure 3.21 for the 7/03/97 and 12/01/01 pumping is a radial flow response. Figure 3.22 is a Birsoy-Summers plot of the 7/02/97 step test data exhibiting a

Figure 3.20: Comparison of 1997 and 2001 static water and pumping water levels at Well No. 6.

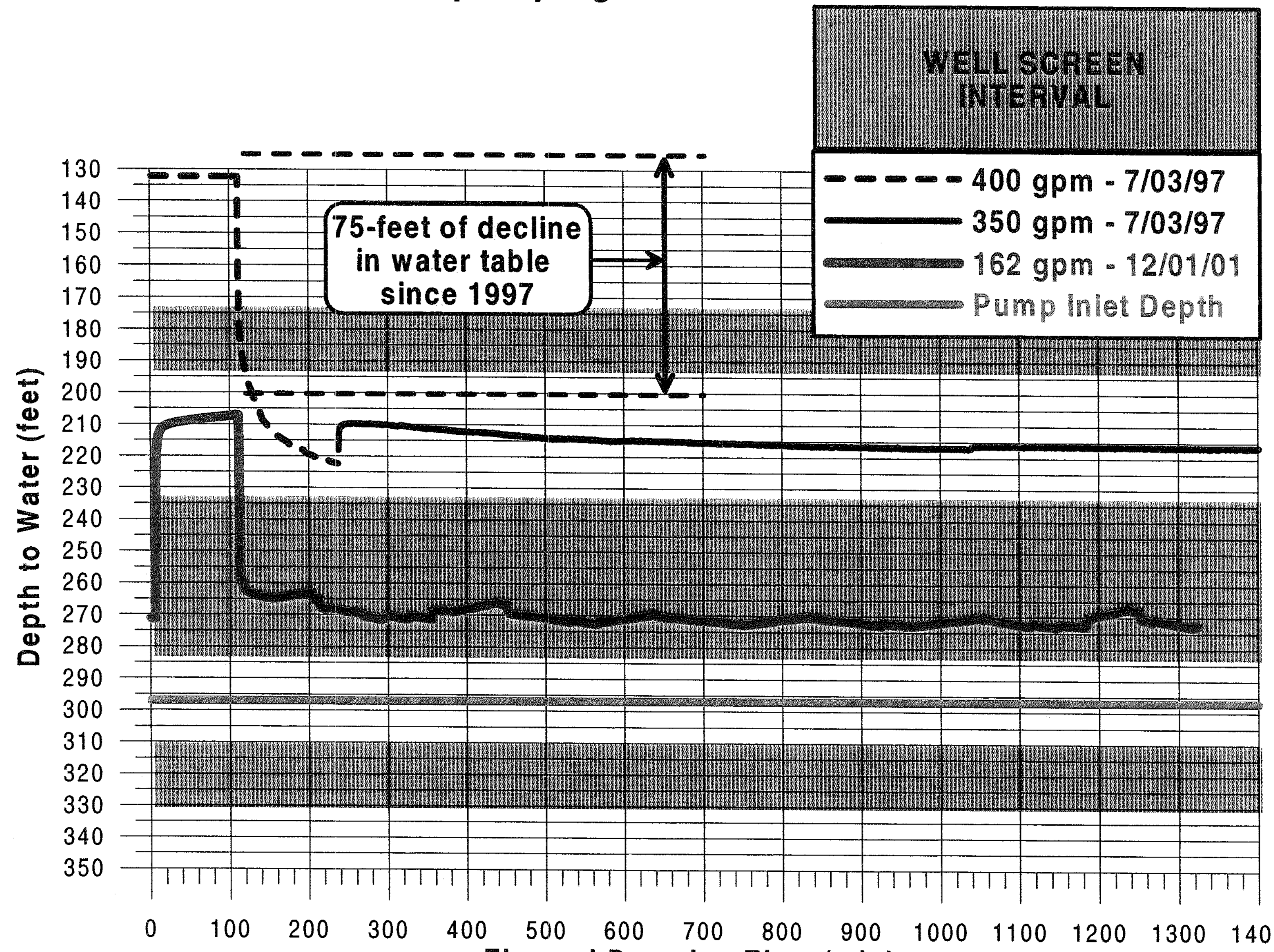


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Figure 3.21: Specialized plot for linear flow at Well No. 6.

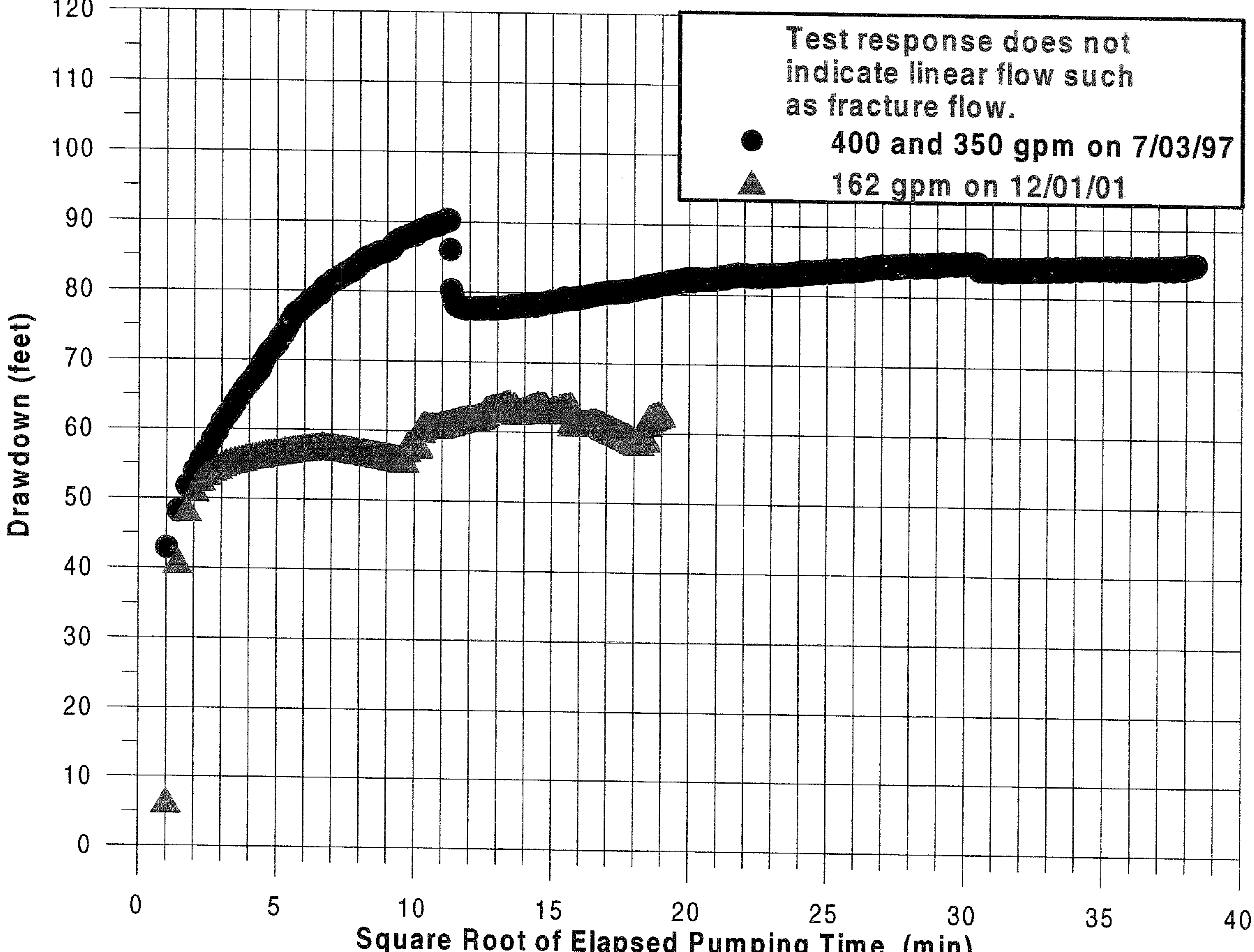

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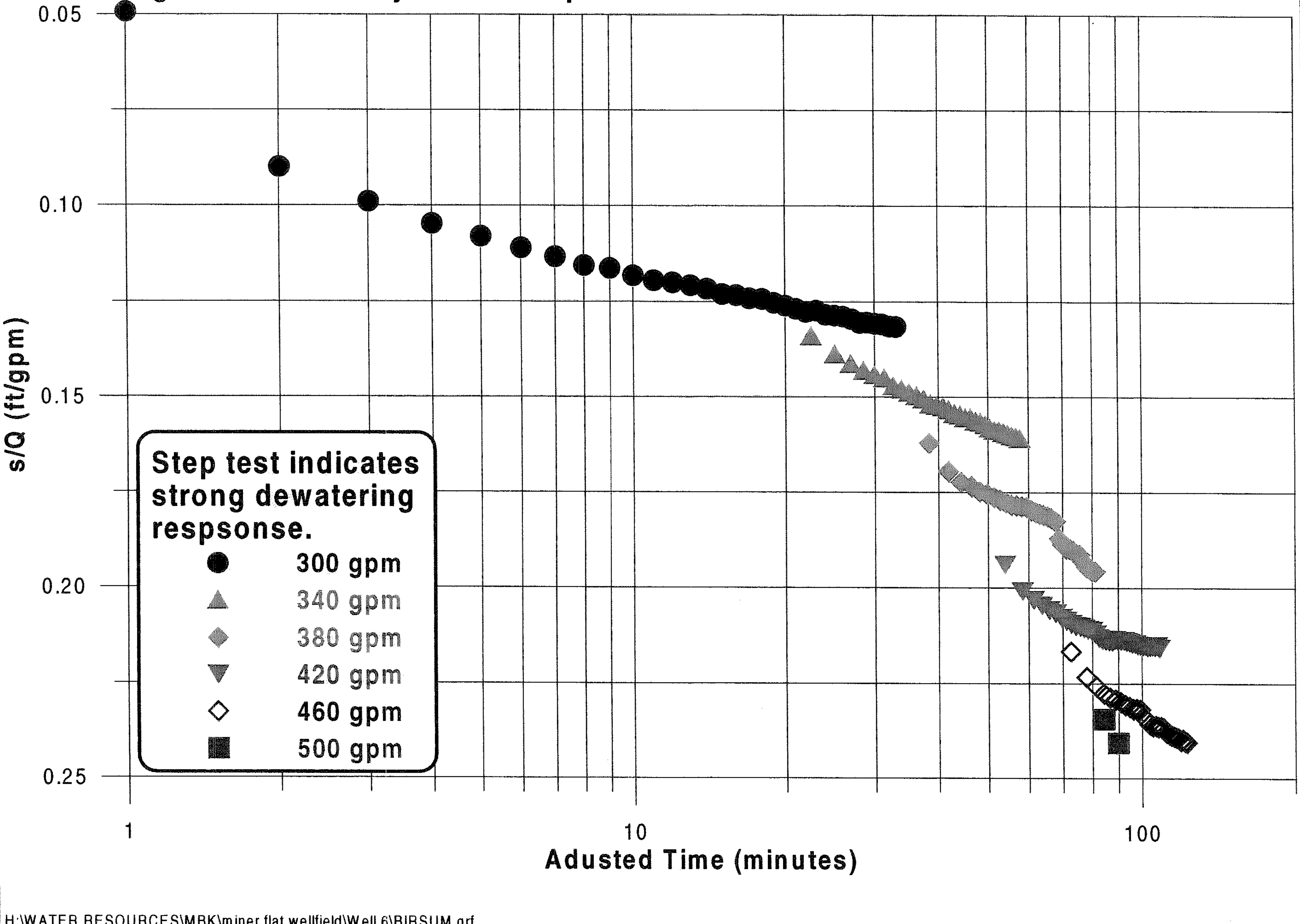
Figure 3.22: Birsoy-Summers plot for 7/02/97 test of Well No. 6.



pronounced dewatering effect superimposed over increased well loss between each increase in the pumping rate. This plot is diagnostic of dewatering of the cone of depression in an unconfined aquifer. Figure 3.23 is a Hantush-Bierschenk plot of the step test data. The specific drawdown on Figure 3.23 includes a significant component of dewatering effect added to the well loss drawdown and therefore does not accurately distinguish well loss drawdown from well loss plus dewatering effect. The specific drawdown of the well on 12/01/01 at 162 gpm is approximately 10 times greater than that in 7/02/97. The decrease in saturated aquifer thickness is probably the greatest cause of the increase in specific drawdown although dewatering of well screen is undoubtedly a contributing factor. Plugging of well screen with iron oxide incrustation may also be a factor in the decreased hydraulic performance of the well.

Figure 3.24 shows the depth to water versus elapsed pumping time for the Well No. 6 July 1997 baseline test. The early part of the test response, at 400 gpm, is adjusted to a pumping water level commensurate with a 162-gpm pumping rate, based on application of the Hantush-Bierschenk plot to correct for well loss and dewatering effect. The trend of the data from the July 1997 response at 400 gpm is projected into the future by regression analysis and intercepts the 12/01/01 pumping water level of 272.5 feet at 17,829 minutes (12 days and 9 hours). This projection forecasts excessive drawdown because it is based on the first 100 minutes of pumping when drawdown was still in a non-steady state condition. Therefore, transformation of the same projection to the data corrected for a 162-gpm response should also overestimate drawdown versus time at 162 gpm. The projected time to drawdown to a pumping water level of 272.5 feet at 162 gpm is approximately 97 years.

Since the pumping water level has declined to 272.5 feet in approximately four years, not 12 days and not 97 years; it is evident that the 12/01/01 pumping water level is the result of a combination of drawdown with dewatering effects and loss of well yield as saturated thickness in the aquifer decreased. Retrograde projection of the regression line (Figure 3.24) backwards from four years indicates the average pumping rate over the four-year period of groundwater level decline was 293.4 gpm. The average rate of 293.4 gpm is the result of an initial well yield of 350 gpm decreasing to 162 gpm as the saturated thickness of the aquifer decreased. Because the slope of the projections shown on figure 3.24 are conservatively steep, and therefore overestimate the rate of drawdown, the average pumping rate for the period from January 1998 through December 2001 was probably somewhat more than the estimate of 293.4 gpm provided on Figure 3.24; however, the conclusion remains the same, namely that dewatering of the saturated thickness of the aquifer has caused a decline in well yield from 350 gpm to 162 gpm over a period of approximately four years of well operation.

The significance of the foregoing conclusion is that the baseline test projections would only predict the dewatering and decrease in well yield, if it is assumed there is no recharge to the aquifer during the four-year period of projected drawdown. The fact that the historic performance of the well is consistent with such a prediction indicates that

Figure 3.23: Hantush-Bierschenk plot for 7/2/97 test of Well No. 6.

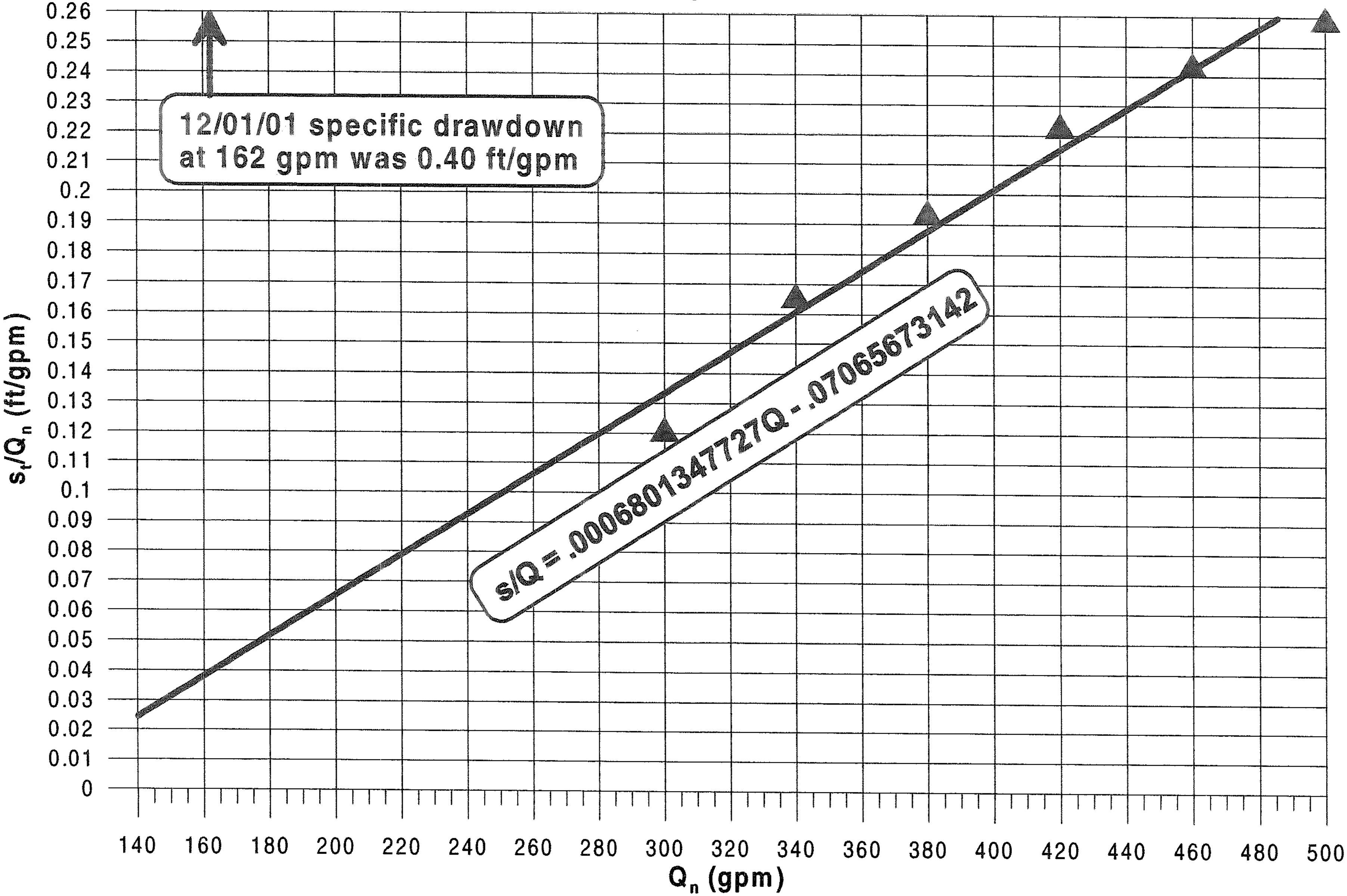


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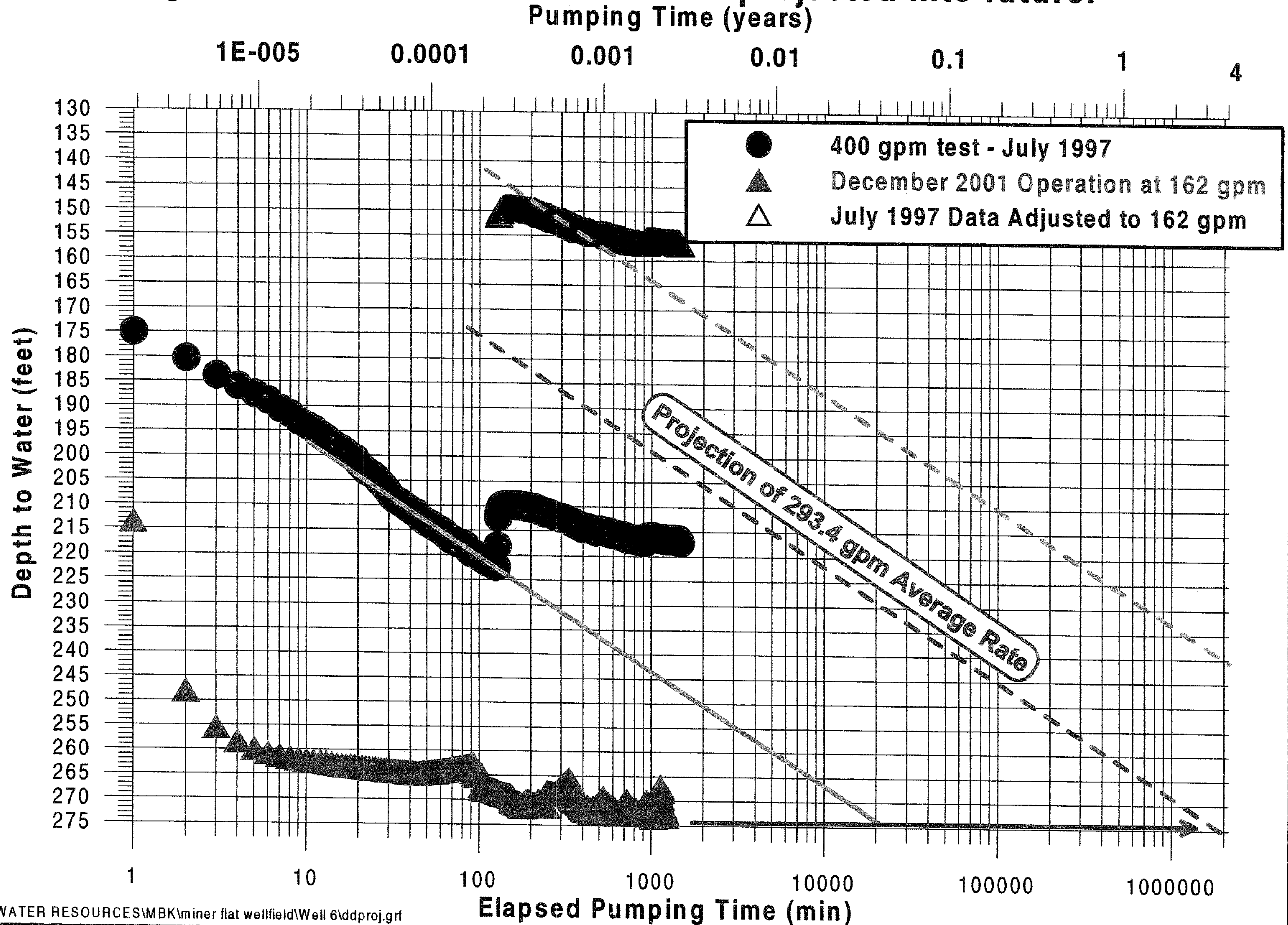
Figure 3.24: Well No 6 baseline test projected into future.



H:\WATER RESOURCES\MBK\miner flat wellfield\Well 6\ddproj.grf groundwater levels at Well No. 6 during the past four years of operation have been affected little or not at all by recharge to the aquifer or by flow of water from some other part of the aquifer into the cone of depression around Well No. 6.

3.6.4. Pump Condition

Figure 3.25 shows the pump performance curve for the Goulds 7CLC040 pump installed in Well No. 6. The difference in elevation between the land surface elevation at the well and the storage tank overflow is 116 feet. When this static lift is subtracted from the pump's rated lift of 343 feet at the design yield of 350 gpm, the pump was capable of delivering the design yield of 350 gpm from a pumping water level of 226 feet, assuming negligible transmission line and pump column friction loss. These numbers are consistent with the nominal design data shown on Table 3.13.

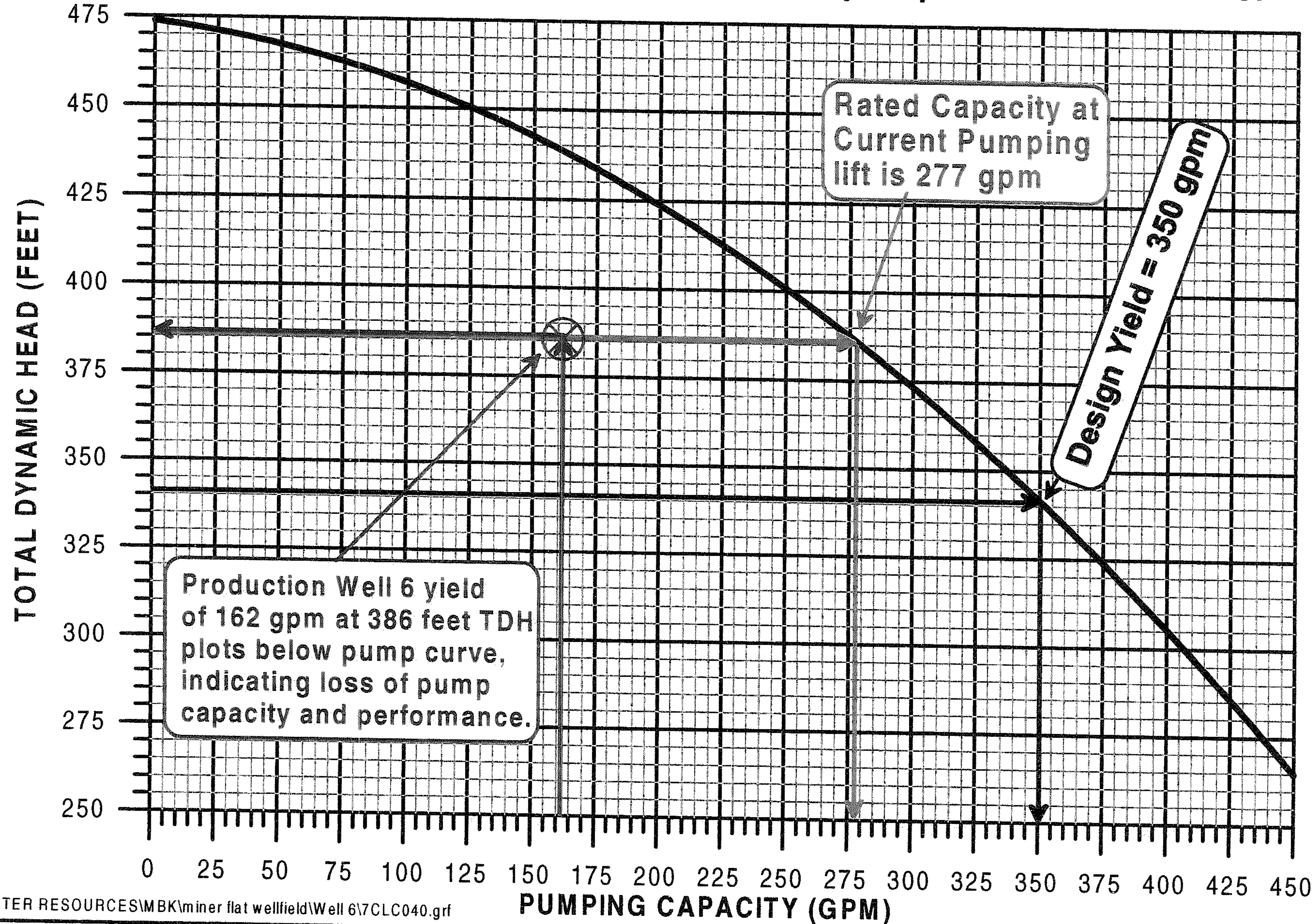
Assuming a 12/01/01 pumping water level averaging 270 feet, the total dynamic head imposed on the pump (assuming negligible friction loss) is 386 feet. At 386 feet of total dynamic head, the pump is rated to deliver 277 gpm, as shown on Figure 3.25. This indicates the reduction in yield at Well No. 6 due to increased pumping lift caused by declining water levels is the difference between 350 gpm and 277 gpm, or a loss of 73 gpm. However, the pump only delivers 162 gpm. The difference between 162 gpm and the rated yield of 277 gpm, a loss of 115 gpm, indicates the pump has damage and/or wear that has reduced its capacity.

The most likely cause of damage to the pump is cavitation due to entrained air entering the pump sometime over the past four years. As groundwater levels declined, the pump continued to attempt to pump 350 gpm, but without sufficient submergence to satisfy net positive suction head requirements (NPSHR). Consequently, the pump was progressively damaged by entrained air. Eventually, the damage to the pump decreased its yield to a rate commensurate with the yield of the well and the available submergence of the pump inlet. It would not be surprising to find a hole in the uppermost pump bowl or somewhere in the first pump column pipe above the pump at Well No. 6, similar to that observed at Well No. 5.

3.6.5. Recommended Pump Size

The data collected on 12/01/01 show from a pragmatic standpoint that Well No. 6 will presently deliver approximately 160 gpm with 386 feet total dynamic head. On 12/01/01, this provided about 25 feet of submergence over the pump inlet. If the downward trend of water levels continues in the future, the maximum yield of the well will decrease. Accordingly, installation of a new pump, capable of delivering 120 gpm with a total dynamic head of 386 feet and a pump inlet set at 318 feet, will provide 120 gpm under 12/01/01 conditions, and for a few years as water levels at the well continue to decline. A pump inlet setting of 318 feet puts the pump motor inside the top of the lowermost screen in the well, a fact that might reduce the flow of water past the submersible motor

Figure 3.25: Pump performance curve for Goulds 7CLC040 pump shows 115 gpm loss of capacity due to pump wear at Well No. 6.



H:\WATER RESOURCES\MBK\miner flat wellfield\Well 6\7CLC040.grf to provide cooling. The total cased depth of 342 feet was not verified in December 2001, but if correct, does not provide enough room to add another full 21-foot pipe to the pump column to set pump below 318 feet; however, a new 120-gpm pump could be installed at a maximum depth in the well of 335 feet to the pump inlet by adding a 17-foot and a 21-foot pipe to the existing pump column.

3.7. Well No. 7

Construction and baseline tests of Well No. 7 were competed 9/6/97 and the well was put into service in January 1998. Frankie Williams, Water Systems Operator, reports that the well has operated since 1998 with no problems. However, the December 2001 investigations found that the yield of the well has decreased from an initial design yield of 350 gpm to a yield of 188 gpm on 12/01/01. Likewise, it was found that the static water level at Well No. 7 had declined 70 to 75 feet, as shown on Figure 3.26, and the pumping water level had declined about 40 feet. The decline in the pumping water level is limited by the fact it has declined to essentially the pump inlet depth. Similarly to Well No. 6, Well No. 7 operated continuously for the 7-day period of investigation of the wellfield with the exception of a couple of hours when the well was stopped manually in order to determine the static water level.

The decline in the groundwater level and the loss of capacity at Well No. 7 reflects the conditions throughout the wellfield and is caused by the progressive dewatering of the aquifer as the result of inadequate recharge to the wellfield area. Ancillary effects include damage to the pumping equipment as well as loss of well yield irrespective of the condition of pumping equipment.

3.7.1. Geologic Log

A geologic log of the materials penetrated by Well No. 7 is provided on Table 3.14. Well No. 7 was logged by Trevor Haig, geologist for Morrison-Maierle. The information on Table 3.14 is a highly condensed version of the geologic log and summarizes the most important information. The well penetrates Cocnino Sandstone from land surface to 265 feet and Supai sandstone and siltstone from 265 feet to the total drilled depth of 382 feet. The interval from 265 – 266 feet consisted of interlaminated red clay and brown fine-grained sandstone, typical of the contact between the Coconino and the Supai throughout most of the wellfield area. The borehole barely penetrated an orange and brown sticky clay at 382 feet. Principal water-bearing zones started at 170 feet in the Coconino Sandstone with a large increase in flow at 190 feet and again at 300 feet in the Supai. A gradual increase in yield was obtained between the intervals of abrupt increase in yield. The 9/4/97 static water was 112.36 ft BTOC.

Figure 3.26: Comparison of 1997 and 2001 static water and pumping water levels at Well No. 7.

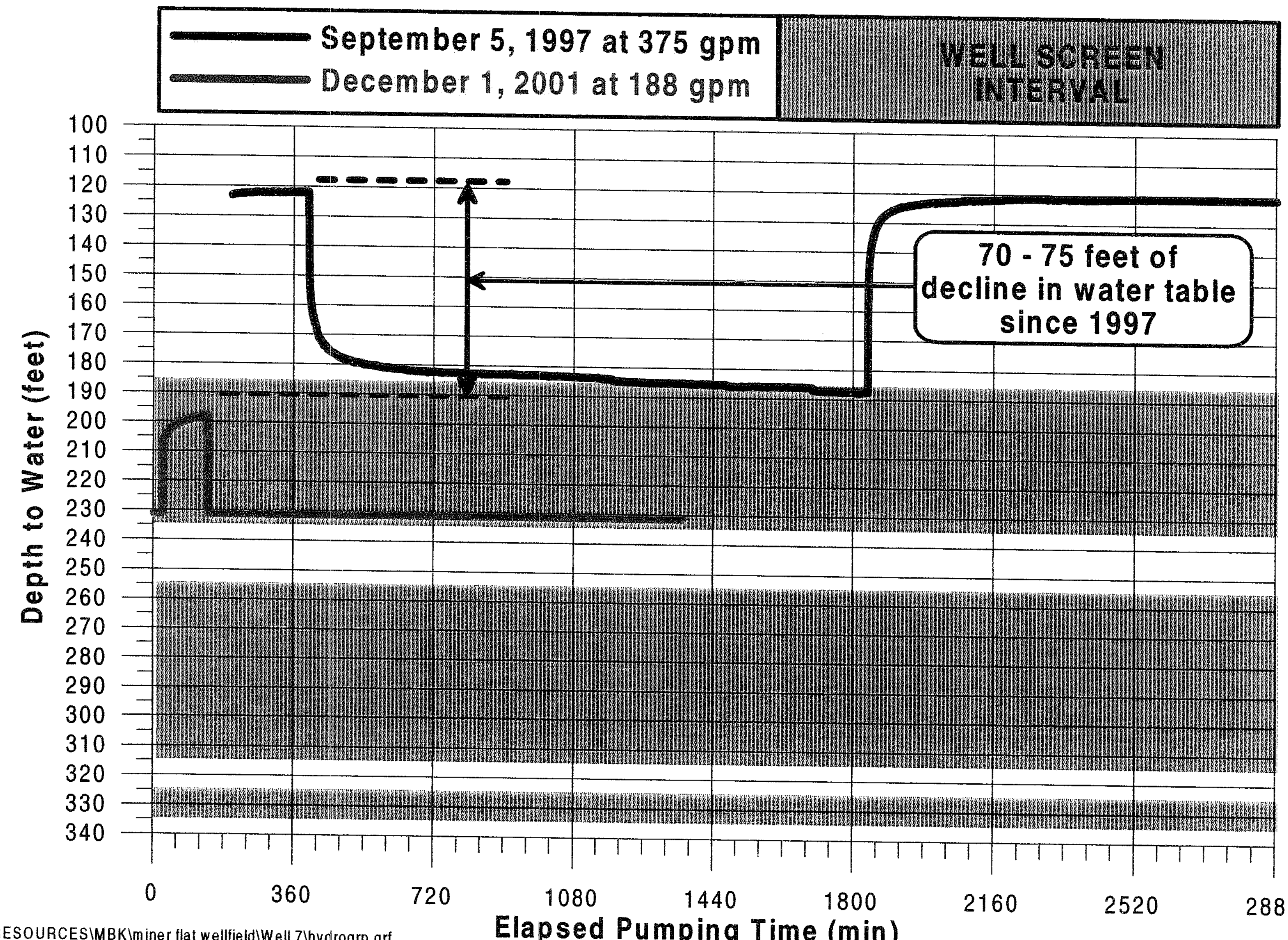


Table 3.14: Production Well No. 7 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 – 195	SANDSTONE, tan to brown, first water at 170 ft with increase at 190 ft.	Colluvium
	SANDSTONE, dark reddish brown.	Coconino
195 - 227	SANDSTONE, yellow to lt. brown.	Coconino
227 - 265		Supai
265 - 266	CLAY, red, interlaminated with brown sandstone	Supai
266 - 278	SANDSTONE, red to reddish brown, water-bearing zone.	Supai
278 – 339	SANDSTONE, reddish-brown w/traces clay, good increase in water at 300 ft.	Supai
339 - 382	SANDSTONE & SILTSTONE, red, fine-grained, with thin layers of brown to red clay interbedded with sandstone layers. Not very productive of water.	

3.7.2. Construction Data

Table 3.15 provides construction data for Well No. 7. Field notes from 7/8/97 record 12-inch surface casing to 190 feet. An Indian Health Service compendium of wellfield information titled, "*Miner Flat Well Logs*", dated August 1998, assembled under the supervision of District Engineer Keith Shortall, shows 12-inch surface casing to a depth of 180 feet. The field notes do not document the surface casing being pulled back and the 12-inch surface casings were cemented into 15-inch diameter boreholes so it is unlikely the casing was pulled back.

The discrepancy between the field notes recorded on-site and the construction drawing prepared by Indian Health Service is not explained. It is worth noting that after the holes were logged, surface casing installation and cementing was usually observed and recorded by Keith Shortall, so the information provided in the Indian Health Service compendium may be the more reliable in regards to the depth of the surface casing. On the other hand, the field record showing a surface casing depth of 190 feet was prepared while the cement plug remaining from cementing the casing was being drilled out. Accordingly, the record was that of an on-site report by the drilling contractor. The drilling contractor operated two shifts per day with alternating crews. This may have caused confusion if the report of surface casing to 190 feet was provided by a different crew then that which installed the casing. The only conclusion that can be reached is that the surface casing is installed to either 180 or 190 feet below the land surface.

Table 3.15 Production Well No. 7 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6166
Tank overflow		6258
Static water level (swl)	122	6044
Top of well screens (BGL)	169*	5997
Bottom of well screens (BGL)	339	5827
Pumping water level (pwl)	185	5981
Intake depth	234	5932
Drop pipe length	231	
Total cased depth	339	5827
Nominal pump capacity (gpm)	350	
Pump horsepower	40	

**Bottom of surface casing at 180 or 190 feet (see text)*

The main well casing consists of 8-inch steel casing and 8-inch pipe size stainless steel well screen installed to a total depth of 339 feet. The screened intervals are as follows:

169 – 190 or 180 ft (Well screen behind surface casing)
190 – 239 ft
259 – 319 ft
329 –339 ft

3.7.3. Construction History

The initial completion of Well No. 7 specified well screens from 282-342 feet and 352-362 ft with blank steel casing from 362-372 feet below the well screens. Before the well screen and casing was installed into the borehole, the drilling contractor lost a bit and stabilizer in the hole. Attempts to fish for the lost tools were unsuccessful so a decision was made to leave the blank casing off the bottom of the lowermost screen and complete the well with well screens as planned and put a plate bottom on the lowermost screen at 362 feet.

As the 8-inch diameter casing and screen assembly was lowered into the well, it stopped on an obstruction at 240 feet. A tremie pipe was installed to the "ledge" at 240 feet and compressed air was used to loosen the material until the screen assembly passed the blockage. After the filter pack was installed around the well screen, preliminary test pumping determined that the yield of the well at maximum drawdown was limited to 160 gpm and the well continuously produced an unacceptable amount of very fine-grained brown sand aptly described by the drillers as "sugar sand". Removal of the test pump and additional development of the well did not improve the yield or stop the sand production.

On 8/10/97, a tremie pipe was used to inject compressed air into the filter pack between the borehole wall and the outside of the 8-inch casing until the filter pack was removed from the annulus to a depth of 290 feet. The well screen and casing were then removed from the well. On 8/18/97, multiple passes were made through the 12-inch borehole with a bit and stabilizer to remove the "ledge" at 240 feet. This effort determined that the borehole remained "tight" from 240 to 260 feet and that a copious amount of loose sand continued to flow into the open borehole despite continued development of the open hole.

A decision was made to revise the screened intervals to 192-262 feet, 282-342 feet, and 352-362 feet with a plate bottom on the lowermost screen. When this assembly was lowered into the borehole, it encountered about 20 feet of loose sand backfill on top of the lost tools. Attempts to wash the casing and screen assembly down and to blow it down with an outside air pipe were unsuccessful. Finally, the entire assembly was pulled back until the bottom of the lowermost well screen and the plate bottom were at 339 feet, in order to match the screened intervals with the water-bearing zones as well as possible. This resulted in the upper part of the uppermost well screen overlapping the surface casing.

Addition of filter pack around the well screens continued through mid-afternoon on 8/20/97 at which time 22 pallets of silica sand had been placed in the annulus to bring the filter pack to a depth of 100 feet from the surface. Using a true-hole diameter of 12 inches and an outside diameter of 8-5/8 inches for steel casing, the true-hole volume of 229 feet of annulus is 90.7 ft^3 and the volume of the 12-inch borehole from 339 feet to the top of the sand backfill at about 355 feet is 12.6 ft^3 for a total theoretical volume of 103.3 ft^3 of filter pack. With 30 ft^3 of sand per 30-sack pallet, the theoretical requirement for filter sand was 3.4 pallets. The 22 pallets of sand installed into the well was 660 ft^3. This provides some idea of the volume of loose sand that flowed into the well and the void it left behind. The 660 ft^3 volume of filter pack installed was equivalent to the annulus volume of a 24.3-inch diameter borehole.

After filter pack was installed, the well was developed with a surge block followed by air lift pumping. Although the well screen was designed to retain more than 95 percent of the filter pack, fine sand from the formation came through the pack and into the well where it was discharged by the air lifting. The filter pack in the annulus settled into the void left behind by the sand and more filter pack was added to the annulus to replace the settlement. By the end of 8/22/97, 5 additional pallets plus 30 sacks of filter pack had been added to the well and the top of the pack had settled to 170 feet in the annulus. The field notes end on 8/22/97, a Friday afternoon, at which time the Morrison-Maierle staff turned the construction observation over to the Indian Health Service staff and started yield and drawdown tests at another well. Accordingly, the total amount of filter pack added to Well No. 7 is not available in the Morrison-Maierle field notes; however, the notes available document a very unstable formation, presumably in the Coconino Sandstone, as indicated by the color of the sand. Similar problems occurred at Wells No. 8, 9, and 10, indicating that the formation in the northern portion of the wellfield is less stable than in the southern portion.

As shown on Figure 3.26, the baseline test of Well No. 7 on 9/05/97 provided a yield of 375 gpm for more than 24 hours of continuous pumping without the pumping water level declining below the top of the uppermost well screen.

3.7.4. Aquifer Response

The response of the aquifer system at Well No. 7 was analogous to that at the other wells in the wellfield, exhibiting initially confined response followed by unconfined response and the onset of a component of drawdown due to dewatering of the aquifer thickness near the pumped well. Figure 3.27 is a specialized plot which will provide a straight line if flow to the pumped well is linear, such as occurs in fracture-controlled flow and in narrow strip aquifers. The response shown on Figure 3.27 is that of radial flow. It is interesting to note, that although Well No. 7 penetrated the Coconino Sandstone at the land surface and the first indication of water yield was at 170 feet, the static water level was 122 feet, indicating locally confined conditions within the sandstone strata. This phenomenon was observed to some extent in other wells in the wellfield and an a deep well drilled into the Coconino Sandstone for a Tribal monitoring well in the Pinetop-Lakeside area.

Figure 3.28 is a Birsoy-Summers plot showing a "negative" boundary response caused by dewatering of the aquifer in a partially unconfined cone of depression, superimposed over increased well loss with increased pumping rates. The vertical separation between the plot of each step is the measure of well loss in a conventional Birsoy-Summers solution. On Figure 3.28, the vertical separation includes both well loss drawdown and drawdown due to dewatering effect. Accordingly, well loss cannot be determined directly from the plot, a problem that existed with every one of the wells in the wellfield.

Figure 3.29 is a conventional Hantush-Bierschenk plot of the 9/04/97 step test data showing specific drawdown versus pumping rate. The specific drawdown values include a component of drawdown due to dewatering effects. Figure 3.29 shows that by 12/01/01, the specific drawdown at the 188 gpm pumping rate had increased from an estimated 0.07 ft/gpm in 1997 to 0.1098 feet at the end of 2001. This increase in unit drawdown is undoubtedly attributable mostly to the decrease in the saturated thickness of the aquifer due to dewatering. Figure 3.30 shows the same information expressed as a specific capacity curve of total drawdown versus pumping rate, indicating a 71 percent loss of specific capacity between initiation of production in January 1998 and the December 2001 inspection, a mere four-year period.

Figure 3.31 shows the pumping water level in Well No. 7 at 375 gpm on 9/05/97 as well as an estimate of what the pumping water level would have been on 9/05/97 at 188 gpm. Figure 3.31 also shows the 12/01/01 pumping water level at 188 gpm for comparison. Projection of the confined portion of the baseline response at 375 gpm indicates that continuous pumping, without recharge, would draw the pumping water

Figure 3.27: Specialized plot for linear flow at Well No. 7.

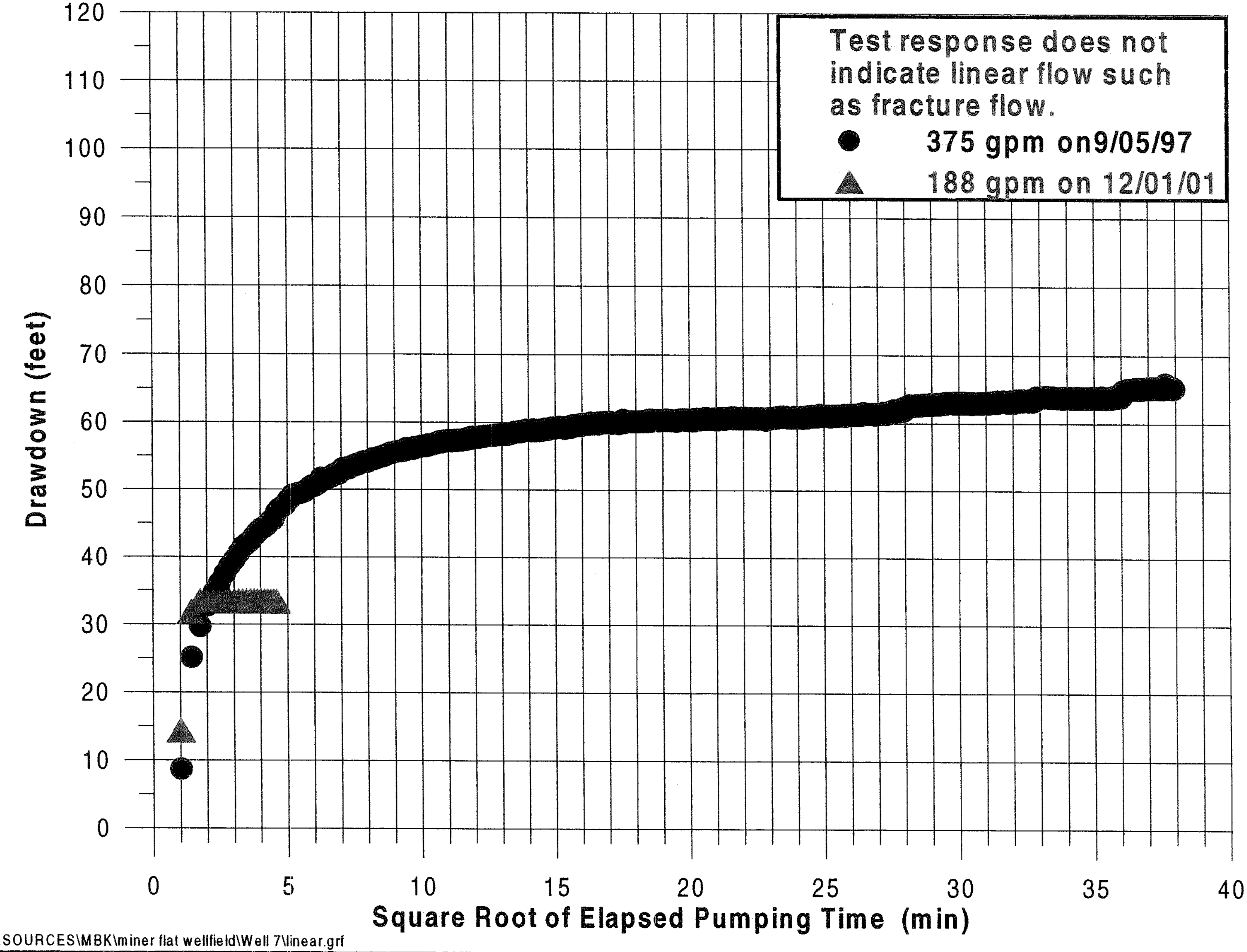


Figure 3.28: Birsoy-Summers plot for 9/05/97 test of Well No. 7.

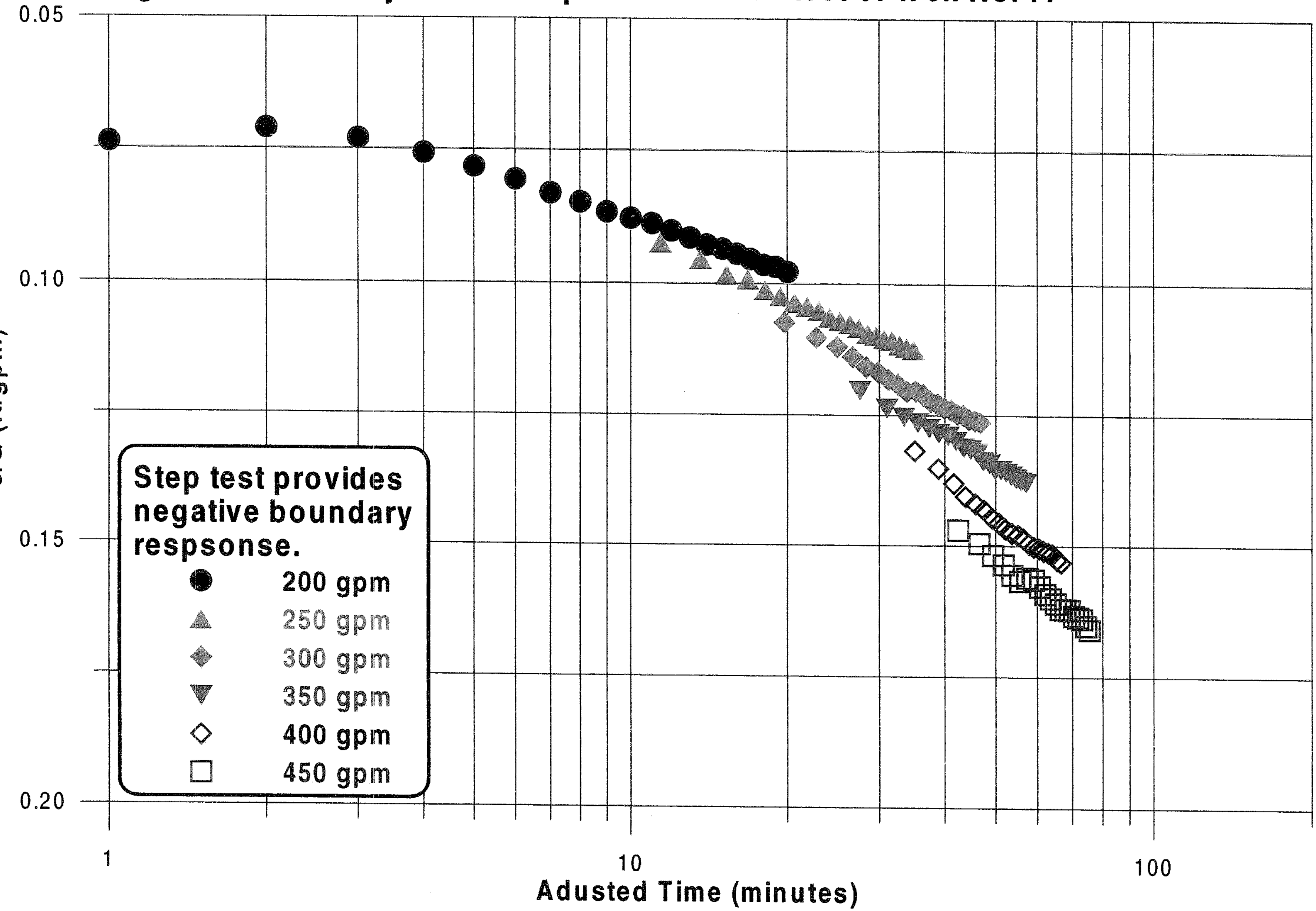


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Figure 3.29: Comparison of specific drawdown curve for 9/05/97 to present specific drawdown shows loss of performance.

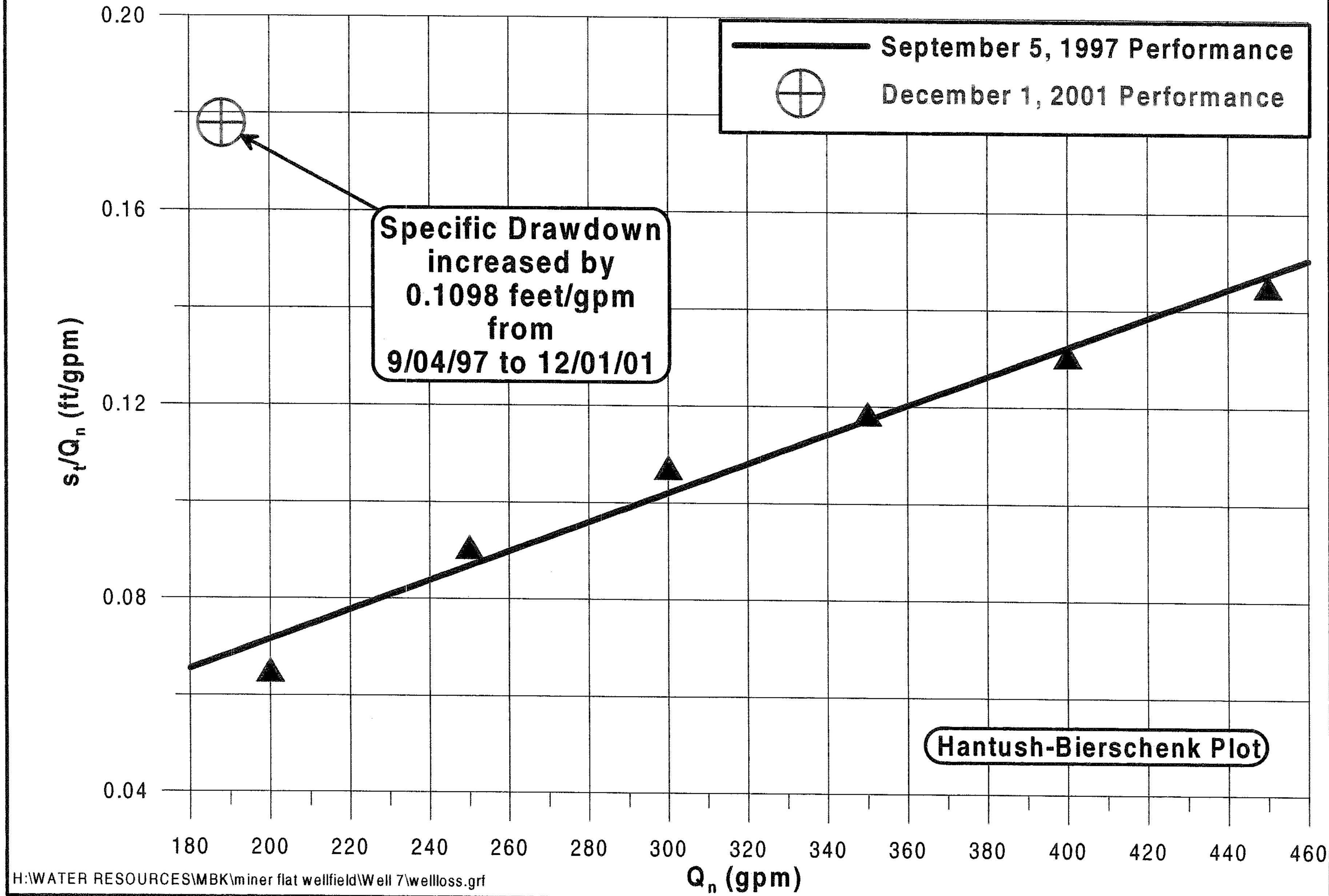


Figure 3.30: Comparison of specific capacity curve for 9/05/97 to present specific capacity shows loss of performance.

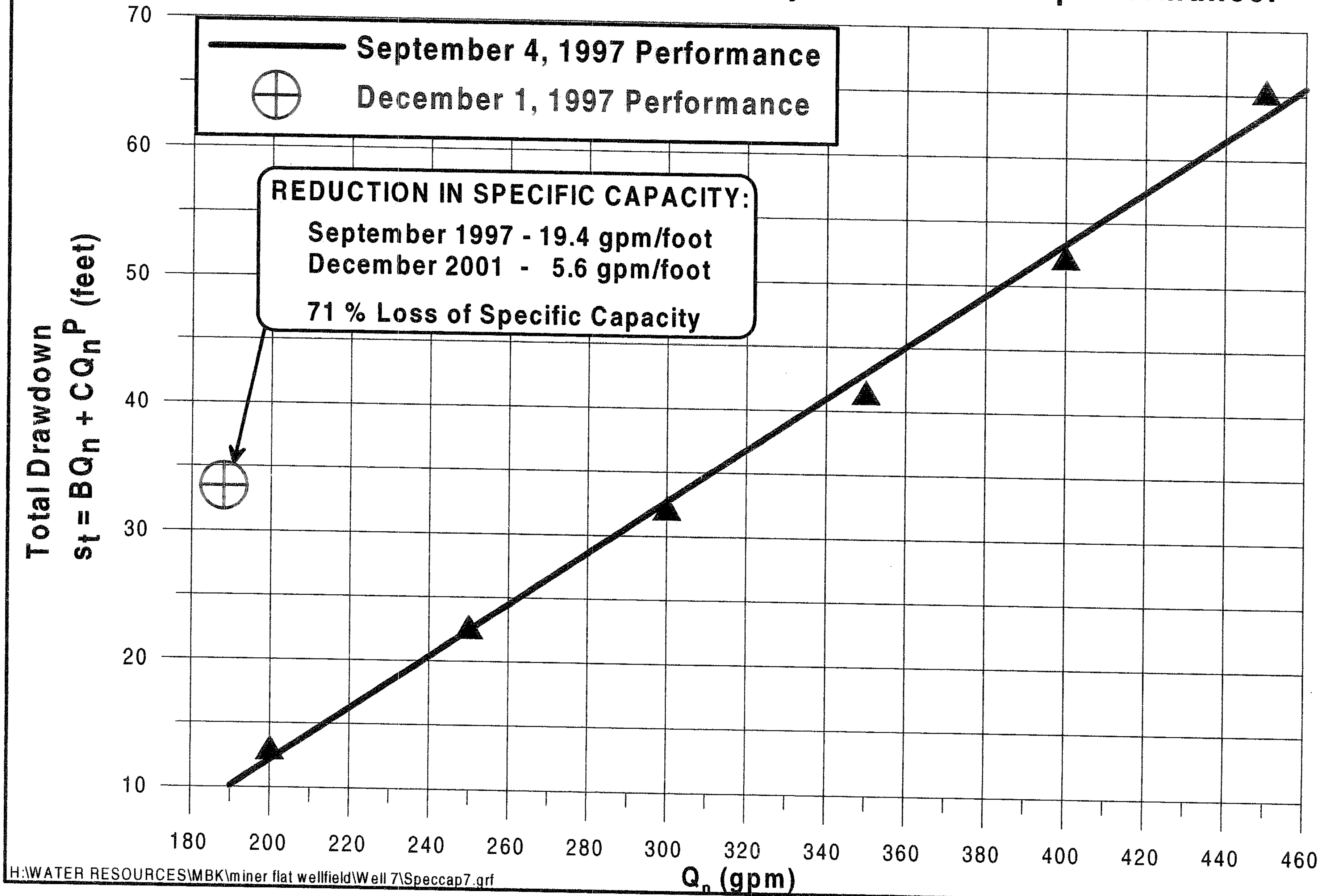
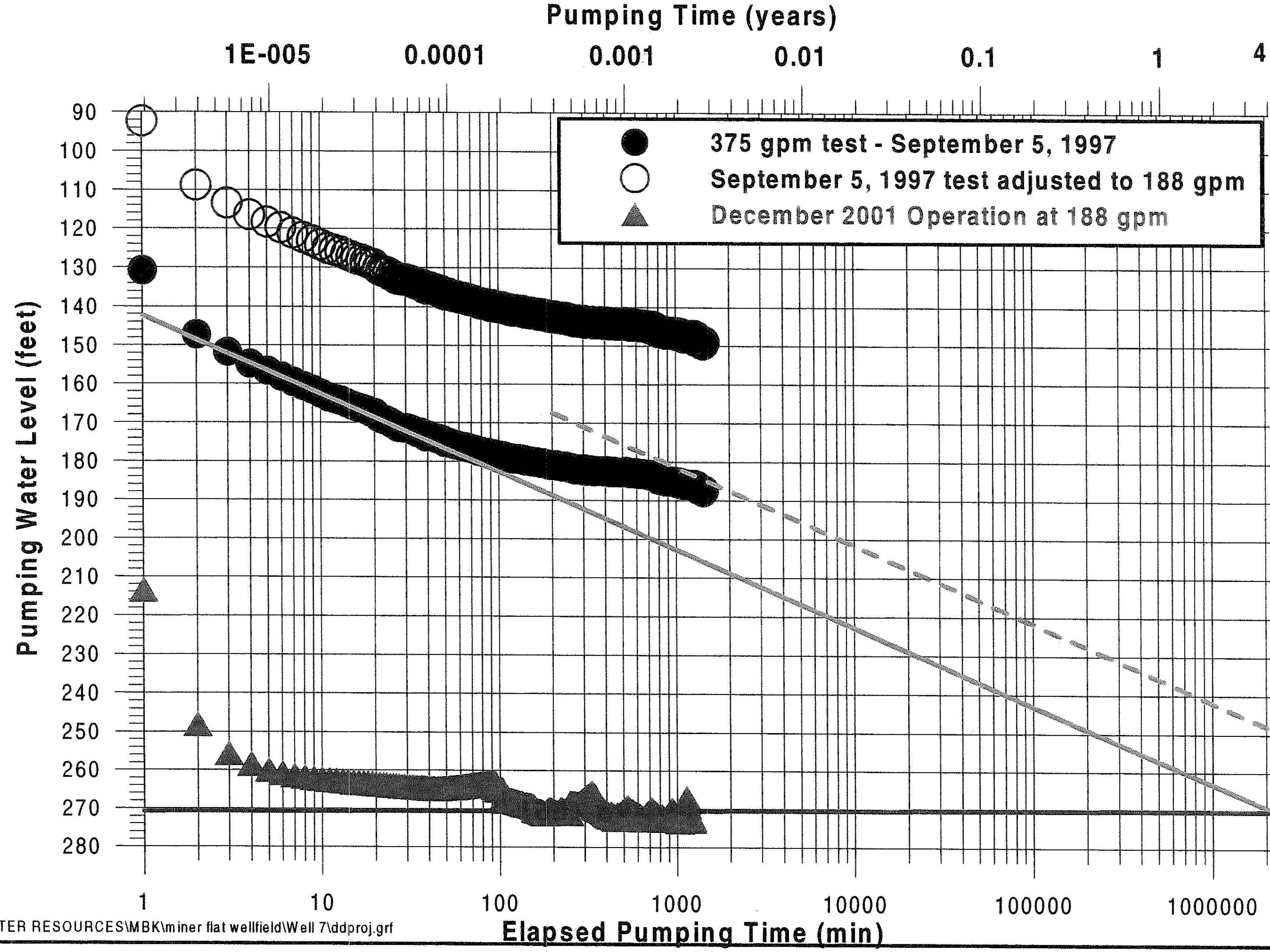


Figure 3.31: Well No. 7 baseline test projected into future.



H:\WATER RESOURCES\MBK\miner flat wellfield\Well 7\ddproj.grf level down to the December 2001 level in a four-year period. The agreement between the projected drawdown, without recharge, and the actual well performance indicates the aquifer did not receive significant recharge since January 1998.

3.7.5. Pump Condition

Figure 3.32 shows the pump performance curve for the pump installed into Well No. 7. The design pumping water level of 185 feet shown on Table 3.15 does not match the design yield of 350 gpm shown on Table 3.15 when plotted on the pump performance curve. The total dynamic head at a pumping water level of 185 feet, is 277 feet, assuming negligible friction loss in the pump column and distribution system. As shown on Figure 3.32, the pump is rated to produce 430 gpm at a total dynamic head of 277 feet and was therefore oversized for the design yield.

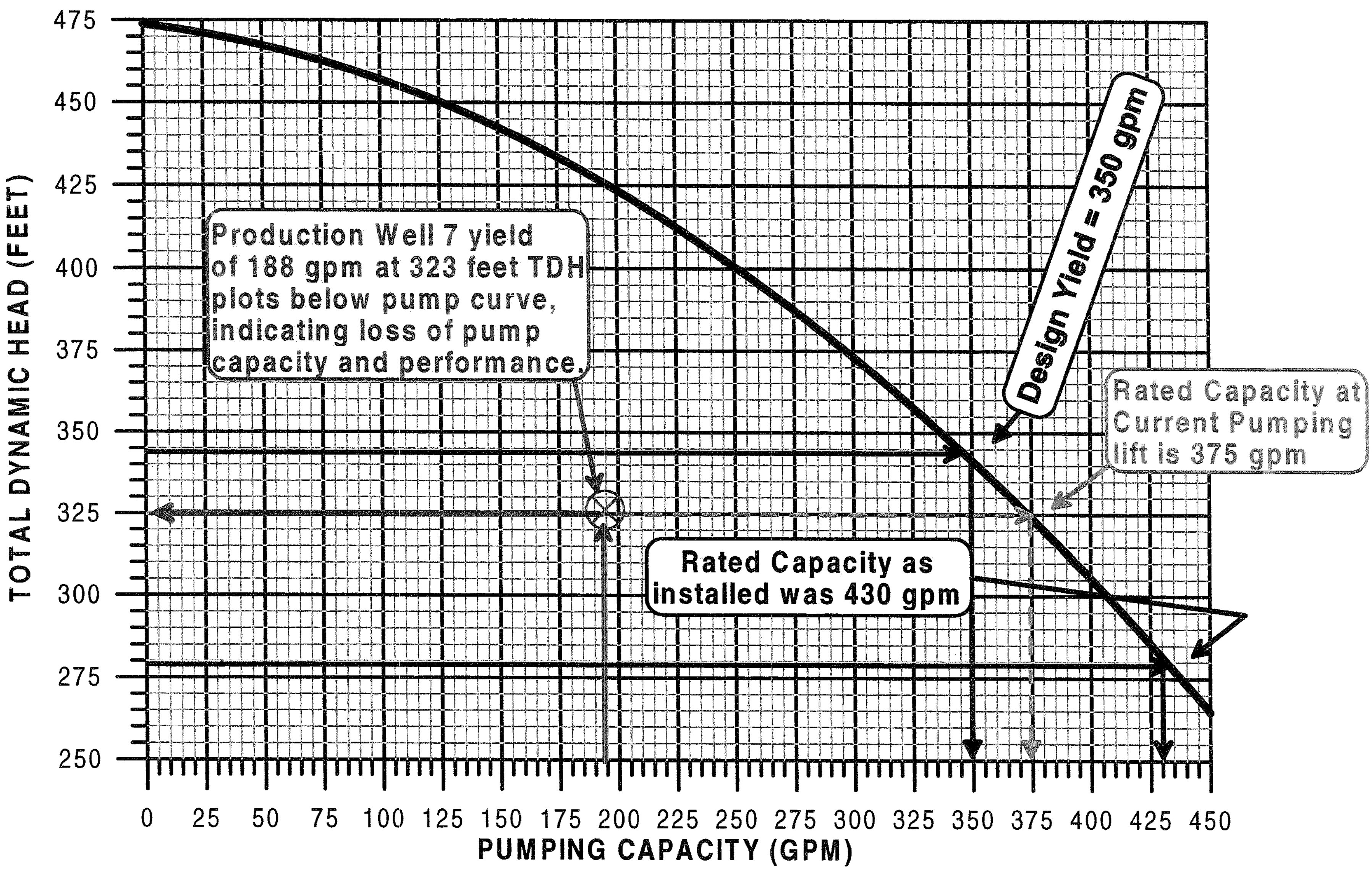
Figure 3.32 shows that at the design yield of 350 gpm, the pump was rated to produce 342 feet of total dynamic head, 65 feet more than the design pumping water level. At the present pumping water level of 231 feet, total dynamic head is 323 feet, for which the pump is rated to produce 375 gpm. The difference between the rated production of 375 gpm and the observed production of 188 gpm indicates that damage to the pump has reduced its capacity by 187 gpm, a loss of 50 percent of its capacity. As in the case of Well No. 6, the damage to the pump in Well No. 7 is assumed to have resulted primarily from cavitation due to entrained air. Figure 3.32 shows that the oversized pump would have drawn the pumping water level down to the pump inlet at a pumping rate of 375 gpm, as limited by the pump inlet depth. This would have resulted in the pump breaking suction and pumping entrained air beginning with operation in January 1998. Damage to the pump due to cavitation continued until the capacity of the pump was reduced to the 12/01/01 yield of the well with the pumping water level at the pump inlet.

3.7.6. Recommended Pump Size

The data collected on 12/01/01 show from a pragmatic standpoint that Well No. 7 will presently deliver approximately 188 gpm with the pumping water level essentially at the pump inlet depth. Accordingly, the pumping rate should be reduced to provide adequate submergence over the pump inlet to satisfy net positive suction head requirements (NPSHR). The Goulds pump book does not show NPSHR for the 7CLC040 pump, so the minimum submergence requirement is unknown at the time of this writing.

If the downward trend of water levels continues in the future, the maximum yield of the well will decrease to less than 188 gpm. Installation of a new pump, capable of delivering 160 gpm with a total dynamic head of 386 feet and a pump inlet set at 318 feet, will provide 160 gpm under 12/01/01 conditions, but yield will decrease if water levels at the well continue to decline. The average rate of groundwater level decline has been 17.5 feet per year since September 1997. Accordingly, a pump rated to produce

Figure 3.32: Pump performance curve for Goulds 7CLC040 pump shows 187 gpm loss of capacity due to pump wear at Well No. 7.



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140 gpm with a total dynamic head of 323 feet is recommended with a pump inlet setting of 318 feet. A 318-foot pump inlet setting puts the pump motor inside an interval of blank well casing where flow past the motor will provide cooling.

3.8. Well No. 8

Construction and testing of Well No. 8 was completed 12/02/97 and the well was put into service in January 1998. During the December 2001 inspection, the pump in Well No. 8 would not operate and caused an overload within a minute or two of being switched on in all but one attempt. It was obvious that the pump had suffered extreme mechanical damage, most likely due to excessive pumping of air, as discussed further in this report. Baseline tests of the well produced 300 gpm with a final pumping water level at about 240 feet after 24 hours on 12/02/97. During the December 2001 inspection, the well produced 104 gpm with a pumping water level of about 177 feet. The static water level at Well No. 8 declined approximately 53 feet between December 1997 and December 2001.

3.8.1. Geologic Log

A brief geologic log of Well No. 8 is provided on Table 3.16. The well penetrated 75 feet of unconsolidated overburden on top of a basalt layer from 75 to 225 feet. The 12/02/97 static water level of 76.5 feet indicates confined conditions before the well was put into production. Immediately after the well penetrated from 225 to 245 feet in the Coconino, the borehole began caving on 10/31/97. On 11/1/97, the borehole was advanced to a total depth of 320 feet; however, caving problems persisted. Work was further complicated by the failure of the seals in the hydraulic pump on 11/1/97. In the period from 11/1/97 to 11/17/97, work was slowed by repeated failures of the hydraulic equipment on the rig and the borehole repeatedly caved back to 230 feet.

Table 3.16: Production Well No. 8 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 75	CLAY, brown, with sand, gravel and cobbles.	Alluvium
75 - 225	MALPAIS	Basalt
225 - 245	SANDSTONE, tan to white, first water at 225 ft, making 200 gpm+ at 245 ft.	Coconino
245 - 298	SANDSTONE, tan to white, silica cemented and hard, almost quartzite	Coconino
298 - 390	SANDSTONE, brown to light brown, very poor samples obtained	Supai

On 11/17/97, the drilling contractor changed from air rotary to mud rotary drilling methods to stabilize the caving sandstone and cleaned the borehole to the 320-foot depth originally obtained on 11/1/97. On 11/18/97, the mud pump failed. Following repair of the mud pump, the contractor was unable to restore circulation of drilling fluid to the land surface and by mid-afternoon on 11/18/97, the borehole had caved back to 230 feet. By 3:00 a.m. on 11/19/97, the bit was stuck in caved material at 270 feet and the hydraulic system failed again. By the evening of 11/19/97, drilling of caved material resumed with mud rotary drilling but lost circulation problems continued and very little if any sample of the formation was obtained until circulation resumed at a depth of 355 feet. Lost circulation problems continued intermittently until the borehole reached a total depth of 390 feet and drilling operations were stopped.

It is clear from the history of lost circulation and blind drilling at Well No. 8 that the geologic log is based on imprecise samples. The contact between the Coconino and the Supai is interpreted to be at 298 feet, as shown on Table 3.16, but the interpretation is not based on very reliable information.

3.8.2. Construction Data

Table 3.17 provides a summary of construction data for Well No. 8. A 12-inch diameter steel surface casing was installed to a total depth of 190 feet and cemented into a 15-inch diameter borehole. An 8-inch diameter casing and screen was installed to a total depth of 375 feet with the screened interval from 220 to 370 feet, leaving a 5-foot piece of casing below the screen. The well screen was 20-slot stainless steel with a filter pack of 10-20 Colorado Sillica production sand.

Table 3.17 Production Well No. 8 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6142
Tank overflow		6258
Static water level (swl)	76	6066
Top of well screens (BGL)	220	5922
Bottom of well screens (BGL)	370	5772
Pumping water level (pwl)	250	5892
Intake depth	340	5802
Drop pipe length	336	
Total cased depth	375	5767
Nominal pump capacity (gpm)	350	
Pump horsepower	40	

3.8.3. Aquifer Response

Figure 3.33 shows that the static water level at Well No. 8 declined approximately 53 feet between December 1997 and December 2001. Figure 3.34 is a specialized plot used to evaluate aquifer response for linear flow. The 12/02/97 step test and 11/30/01 constant rate responses shown on Figure 3.34 indicate radial flow to the well, not linear flow such as produced by fracture control of aquifer response. Figure 3.35 is a conventional Birsoy-Summers plot of the 12/02/97 step test. The Birsoy-Summers plot shows the onset of dewatering of a portion of the cone of depression in the confined aquifer after the first four minutes of pumping. Dewatering effects are continuous from the fifth minute of the test until the end of the test.

Figure 3.36 shows the step test data presented on a Hantush-Bierschenk plot. This type of plot should provide a straight line through the data; however, the response of Well No. 8 provided an "S-shaped" response, similar to the response obtained at Well No. 5 and shown on Figure 3.17. The shape of the curve indicates that the shift in drawdown between the individual pumping rates is not due solely to increases in well loss, but includes a significant component of total drawdown which is influenced by two things. One influence is the change from a confined response to a partially unconfined response. The increase in the storativity from confined to partially unconfined conditions caused a reduction in the change in drawdown per unit increase in pumping rate during the middle part of the step tests. The second influence is that of a component of dewatering drawdown becoming progressively more significant as the pumping rate and duration increased. Accordingly, the relationship on Figure 3.36 reflects the influence of the aquifer response superimposed over the hydraulic response of the well and includes considerably more drawdown for each pumping rate than just the well loss drawdown.

Figure 3.37 shows the 12/02/97 baseline test response plotted as a time-drawdown curve. The early part of the test consisted of stepped rates, starting at 200 gpm and increasing in 25-gpm increments to 300 gpm, after which time the 300-gpm rate was maintained until a total pumping time of 24 hours (1440 minutes) elapsed. The last step of the baseline test at 300 gpm provided a final pumping water level at about 240 feet after 24 hours on 12/03/97. During the December 2001 inspection, the well produced 104 gpm with a pumping water level of about 177 feet, as also shown on Figure 3.37. The pumping water level at 104 gpm was not a function of the well performance but was instead determined by the condition of the pump. The pump exhibited all the signs of extreme mechanical damage and only operated for more than a few minutes for one test. When the pump was operating, it made a great deal of mechanical noise, with the impellers and shaft obviously rattling in the casing. In all but one instance, the pump caused an overload error at the control panel, and the motor saver would turn the pump off after less than a minute or two of operation.

The static water level of 76.5 feet in December 1997 is shown on Figure 3.37 as well as the December 2001 static level. A dashed line is drawn between the two static water levels to show the static level change relative to the steady-state pumping water level at

Figure 3.33: Comparison of 1997 and 2001 static water and pumping water levels at Well No. 8.

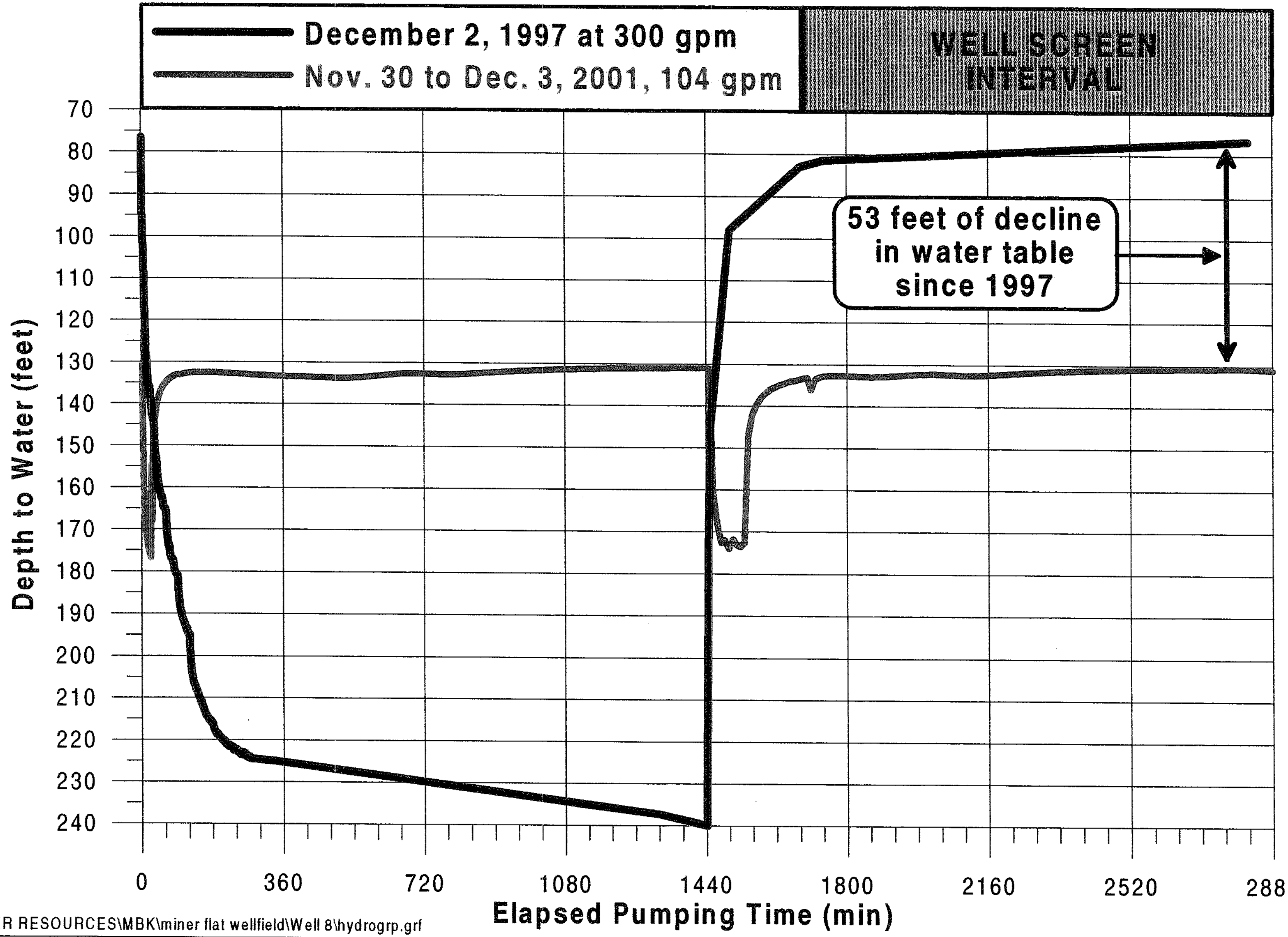

Figure 3.34: Specialized plot for linear flow at Well No. 8.

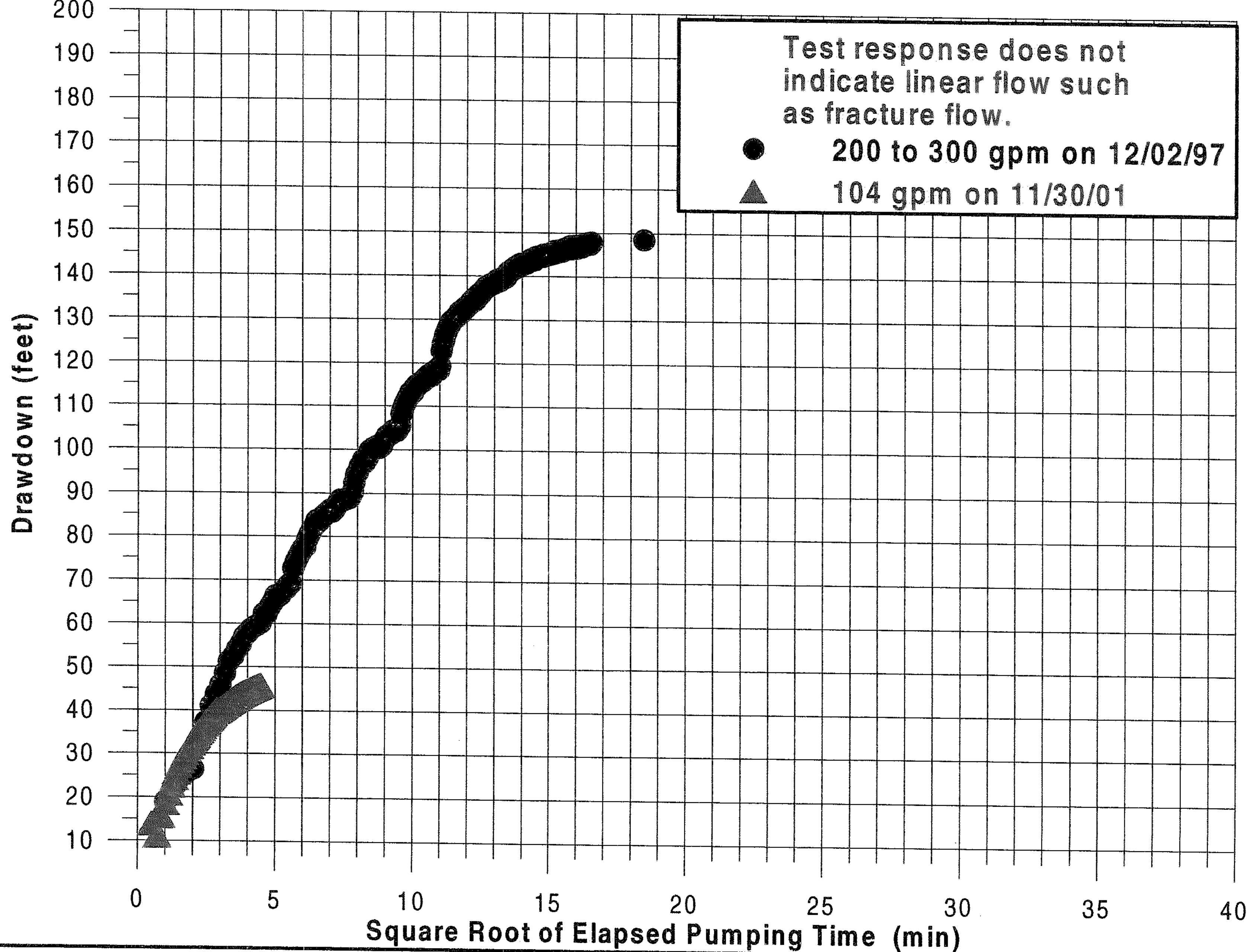

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Figure 3.35: Birsoy-Summers plot for 12/02/97 test of Well No. 8.

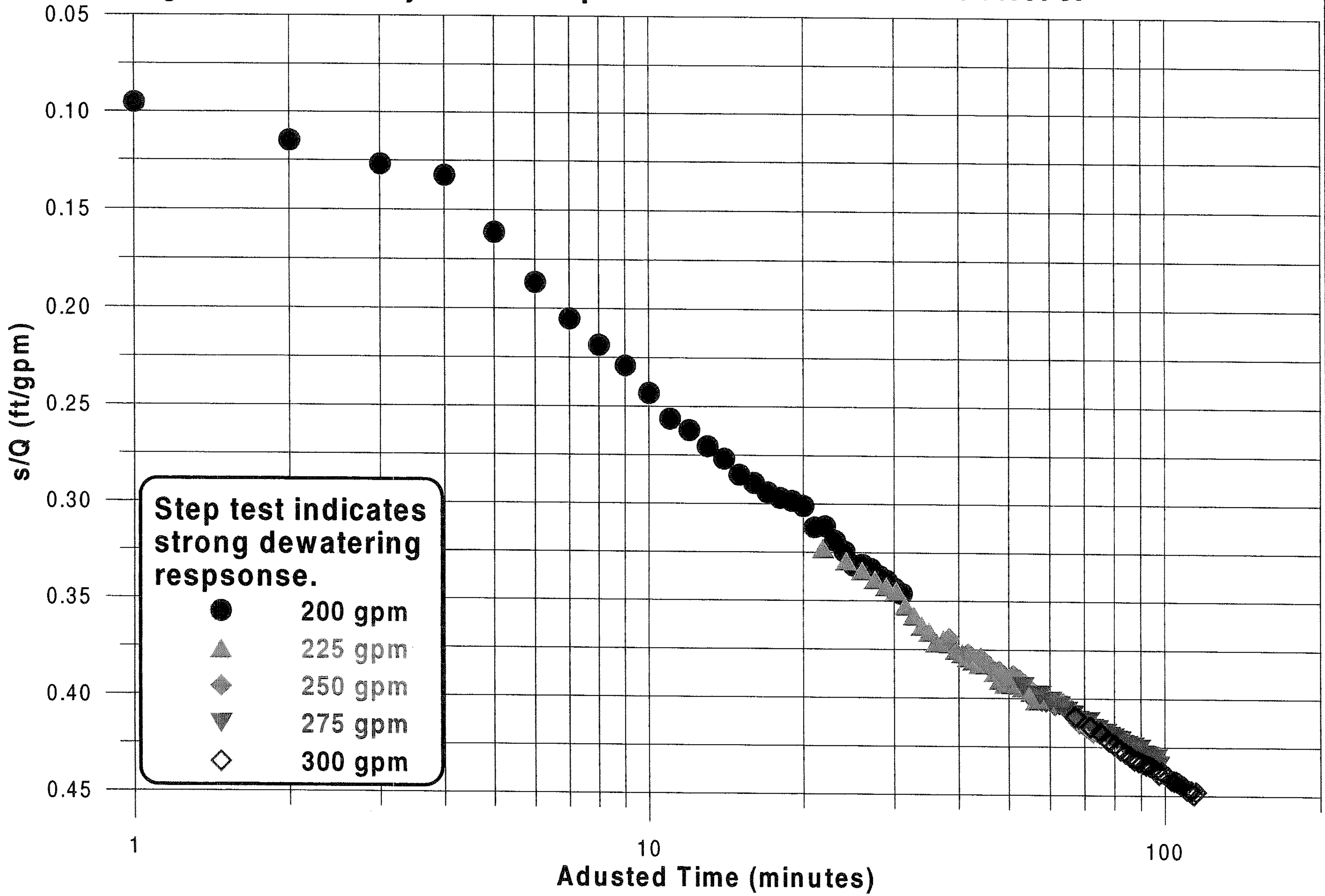

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Figure 3.36: Hantush-Bierschenk plot of 12/02/97 test of Well No. 8.

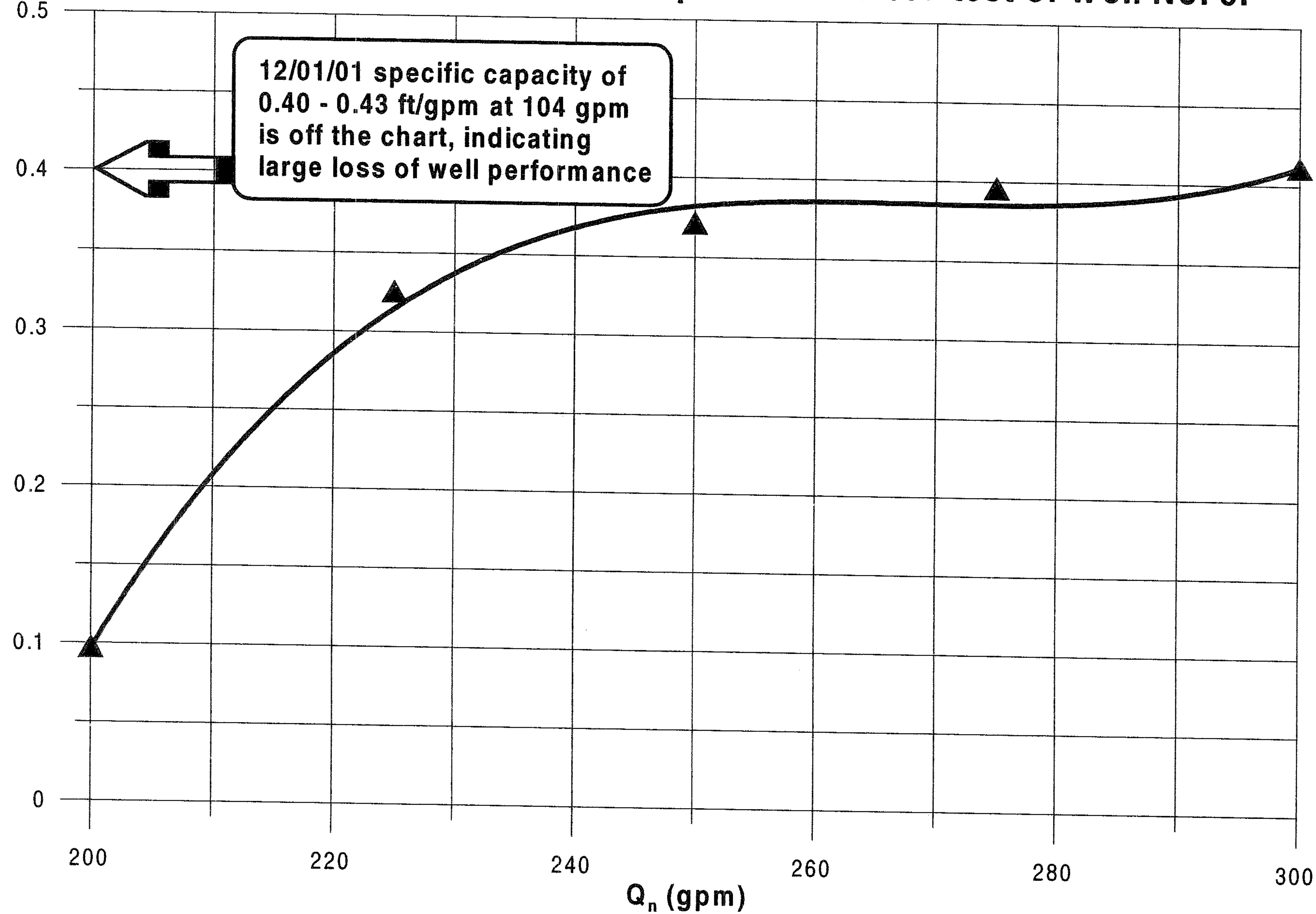

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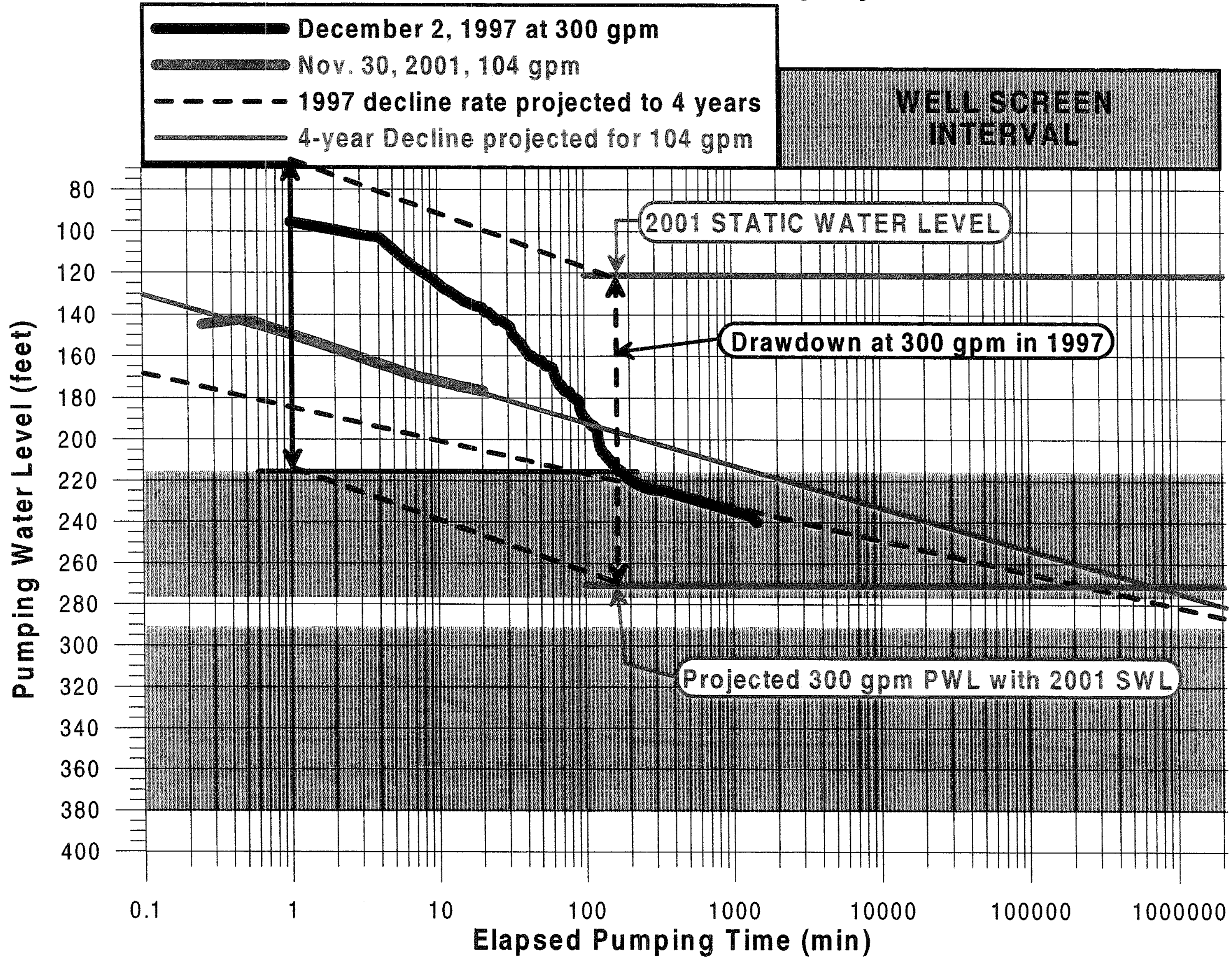
Figure 3.37: Well No. 8 baseline test projected into future.



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300 gpm. Figure 3.37 shows the 1997 static level offset along the y-axis of the graph by the amount of drawdown during steady-state response of the 300-gpm test. Projection of the latter value along the slope of the line between static water levels relative to the steady-state drawdown shows where the 300-gpm pumping water level would theoretically be, starting from the December 2001 static water level and assuming no dewatering effects. Figure 3.37 shows this theoretical pumping water level to be the same as the pumping water level obtained by projecting the 300-gpm time-drawdown curve from December 1997 for four years into the future. Likewise, the slope of the time-drawdown response at 104 gpm in December 2001, subject to dewatering effects and loss of aquifer transmissivity and well yield, projects over a four-year period to the theoretical pumping water level as offset from the December 2001 static water level. The projections of the December 1997 baseline data and the December 2001 response both indicate the pumping water level of Well No. 8 at its maximum yield under December 2001 conditions will result in a pumping water level of approximately 281 feet. The maximum well yield associated with the projected pumping water level is not known.

3.8.4. Recommended Pump Size

The foregoing data do not directly provide the yield of Well No. 8 under December 2001 conditions. The nominal pumping rate of 104 gpm obtained during the December 2001 inspection caused a drawdown of 45.9 feet. This drawdown divided by the pumping rate provides a specific capacity value of approximately 2.27 gpm/ft; however, the sudden stabilization of the time-drawdown curve is unexplained and suggests that a hole is present in the pump or pump column. If the December 2001 time-drawdown response of Well No. 8 prior to stabilization of the drawdown is projected as a semi-logarithmic projection, as on Figure 3.37, the 24-hour specific capacity is estimated to be 1.2 gpm/ft. The projection to 24-hours pumping time eliminates the possibility that the calculated specific capacity value is in error due to stabilized drawdown caused by water circulating back into the well from a leak in the pump column pipe, as is a possibility suggested by Figure 3.33. Assuming that increased well loss and dewatering effects will reduce that specific capacity by 50 percent if the pumping rate is increased or if pumping water levels decline, a presumptive specific capacity for estimating well yield is approximately 0.6 gpm/ft.

Assuming the maximum pumping water level of 281 feet projected by the data on Figure 3.37, starting from a pumping water level of approximately 131 feet (December 2001 static level), 150 feet of drawdown is available. The presumptive value of specific capacity of 0.6 gpm/ft times 150 feet provides a presumptive maximum yield of 90 gpm. This is not much different than the 104 gpm obtained by the damaged pump, but with about 100 feet more drawdown. The available data indicate the rate of decline of the groundwater level at Well No. 8 has been about 15.8 feet per year. Accordingly, continuation of this trend will use up whatever available water column remains in the well at the present pumping water level in just a few years.

Considering the foregoing factors, an estimated yield of 100 gpm from Well No. 8 with a pumping water level of 281 feet and the pump inlet at about 340 feet, is not an unreasonable design estimate for future operation of Well No. 8. This assumes a design capacity of 100 gpm at 397 feet of total dynamic head. Accordingly, it is recommended that the pump in Well No. 8 be replaced with a pump rated to produce 100 gpm with 397 feet of total dynamic head and which will provide less than 100 gpm as the pumping water level in the future declines from 281 feet to some deeper level. The pump inlet for the 100-gpm pump should be set at 340 feet and consideration should be given to placing a shroud around the pump and motor to ensure adequate cooling of the submersible pump motor.

3.9. Well No. 9

Drilling of Well No. 9 started 10/15/97 and the baseline pumping test was completed 12/11/97-12/12/97. The well was put into service in January 1998. Frankie Williams, Water System Operator, reports that the only trouble at the well has been a hole in the pump column pipe. The baseline test of the well was conducted at 400 gpm and the pump installed in the well in January 1998 was rated for a nominal design capacity of 350 gpm. The well yield on 11/30/01 was 338 gpm and 345 gpm on 12/01/01, depending on the duration of pumping and the level in the storage reservoir. However, the December 2001 inspection indicates the pump in the well has suffered damage or excessive wear and the groundwater level has declined about 43 feet since December 1997.

3.9.1. Geologic Log

Field notes logged by Trevor Haig, a geologist on the staff of Morrison-Maierle, Inc., provide a 10/15/97 geologic log for Well No. 9 from land surface to 150 feet, Evidently, borehole below 150 feet was logged by Keith Shortall, Indian Health Service District Engineer at that time. The geologic log, as summarized from notes by Keith Shortall, is provided on Table 3.18.

Table 3.18: Production Well No. 9 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 80	CLAY, brown and gray, silty, with cobbles.	Alluvium
	MALPAIS	Basalt
80 - 145	SANDSTONE, tan, white, brown first water at	Coconino
145 - 227	150 ft.	
		Supai
227 - 230	CLAY & SILTSTONE, red	Supai
230 - 235	SILTSTONE, reddish-brown	Supai
235 - 390	SANDSTONE, reddish-brown, fine-grained, lots of water below 270 feet	

The well penetrates 80 feet of unconsolidated overburden consisting primarily of alluvial deposits. Basalt is present from 80 to 145 feet where the well penetrates the Coconino Sandstone, according to Trevor Haig's notes. The compendium of information compiled for the wellfield by Keith Shortall indicates the base of the basalt at 135 feet and "cooked sandstone" or "quartzite" from 135 to 145 feet. Both logs indicate that at a depth of 150 feet, the borehole was producing groundwater.

The I.H.S. log shows red siltstone and red clay from 227-230 feet, which is interpreted to be the uppermost part of the Supai strata. The Supai strata from 230-235 feet was logged as siltstone and the remainder of the borehole was logged as reddish-brown fine-grained sandstone with "lots of water below 270 feet".

3.9.2. Construction Data

Table 3.19 provides a summary of construction data for Well No. 9. The 10/15/97 field notes indicate the drillers were instructed to install a 12-inch diameter steel surface casing to a depth of 145 feet. However; a 10/17/97 field note indicates the surface casing would not advance past 80 feet (the top of the basalt) and was terminated at that depth where it was cemented into 15-inch diameter borehole. The as-built drawing of the well indicates a total cased and screened depth of 360 feet; however, the lengths shown for individual sections of casing and screen add up to 389 feet. Screened intervals are from 145-225 feet and either 275-355 feet or 275-385 feet, depending on how the as-built drawing is interpreted. The well screen was 20-slot stainless steel with a filter pack of 10-20 Colorado Silica production sand.

Table 3.19 Production Well No. 9 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6129
Tank overflow		6258
Static water level (swl)	55	6074
Top of well screens (BGL)	145	5984
Bottom of well screens (BGL)	355*	5774*
Pumping water level (pwl)	125	6004
Intake depth	215	5914
Drop pipe length	210	
Total cased depth	360**	5769**
Nominal pump capacity (gpm)	350	
Pump horsepower	40	

* *Possibly 385 feet and elevation 5744 feet.*
***Possibly 389 feet and elevation 5740.*

3.9.3. Water Levels

Figure 3.38 shows the static water level for the 12/11/97 baseline test and on 12/01/01, indicating the static water level at Well No. 9 has declined 43 feet in four years, an average of slightly more than 10 feet per year. Figure 3.38 shows that whereas the pumping water level at 400 gpm was above the top of the well screens in December 1997, it was 5 to 6 feet below the top of the well screens in December 2001 at a pumping rate of 338-345 gpm.

The 12/11/97 static water level of 55 feet was above the base of the basalt or baked sandstone at 145 feet, indicating confined aquifer conditions. The 12/01/01 static water level of about 98 feet is still above the base of the confining unit.

3.9.4. Aquifer Response

Figure 3.39 shows the specialized plot used to evaluate the aquifer response for linear flow. The aquifer response on Figure 3.39 indicates radial flow to Well No. 9. Figure 3.40 is a Birsoy-Summers plot of the 12/11/97 step test data which shows a strong dewatering response. Both Figures 3.38 and 3.39 indicate that the pumping water level at the pumped well, including well loss drawdown, did not decline below the base of the confining unit during the 12/11/97 test; however, the strong dewatering response on the Birsoy-Summers plot indicates that some portion of the cone of depression was responding to unconfined aquifer storativity from essentially the beginning of the test.

The same effect, that of a partially confined and partially unconfined cone of depression, is reflected in the Hantush-Bierschenk plot of the 12/11/97 step test data which shows a progressive decrease in the slope of the specific drawdown versus pumping rate curve. The progressive decrease in the slope of the specific drawdown versus pumping rate curve reflects the progressive increase in the percentage of the area of the cone of depression that is unconfined versus the confined portion. Figure 3.41 shows the specific drawdown versus pumping rate curves at 1 minute and 30 minutes pumping time. The separation between the two curves indicates the "well loss" increase caused by each increase in the pumping rate includes a component of dewatering effect in addition to well loss, otherwise, the two curves should plot the same.

3.9.5. Hydraulic Performance

The 12/01/01 data from Well No. 9 provide an opportunity to compare the baseline hydraulic performance of a well in 1997 to that in 2001 where dewatering of the static conditions has not taken place, i.e., where the aquifer remains confined under static conditions and saturated thickness and transmissivity have not been diminished. As shown on Figure 3.41, the 12/01/01 performance at 330 gpm, after 30 minutes of pumping, is better than the original baseline performance. A specific capacity plot of

Figure 3.38: Comparison of 1997 and 2001 static water and pumping water levels at Well No. 9.

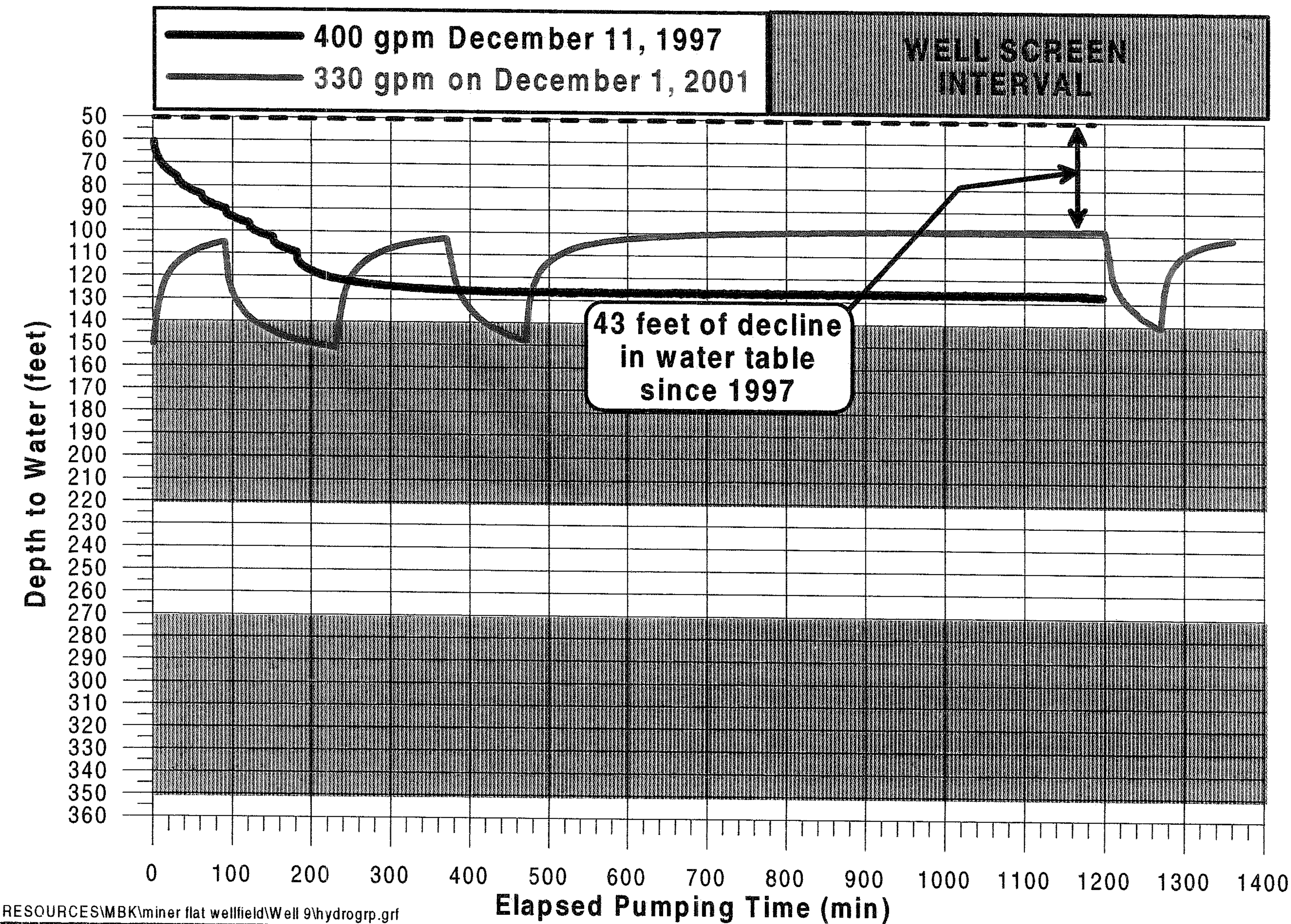

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Figure 3.39: Specialized plot for linear flow at Well No. 9.

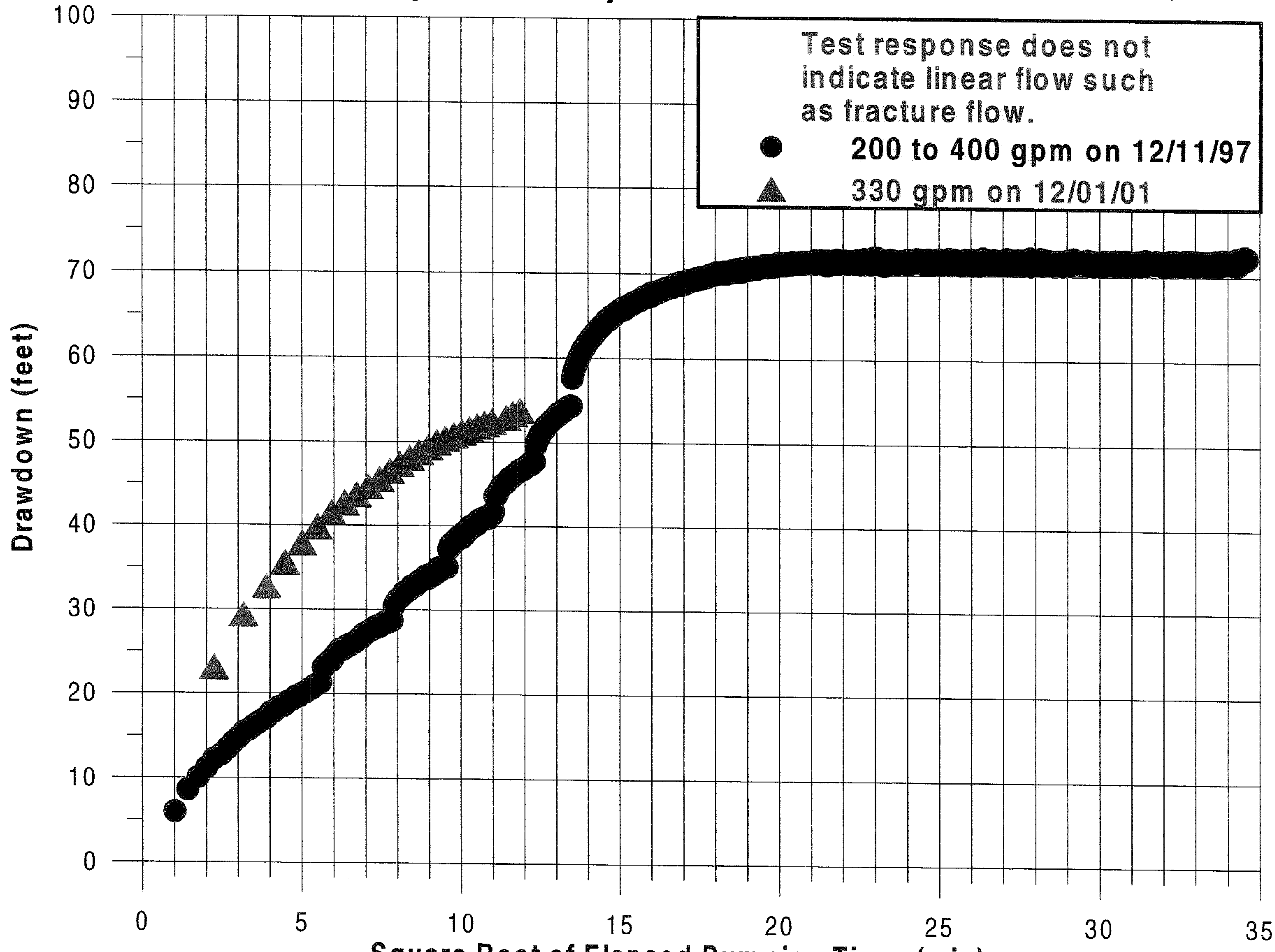

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Figure 3.40: Birsoy-Summers plot for 12/11/97 test of Well No. 9.

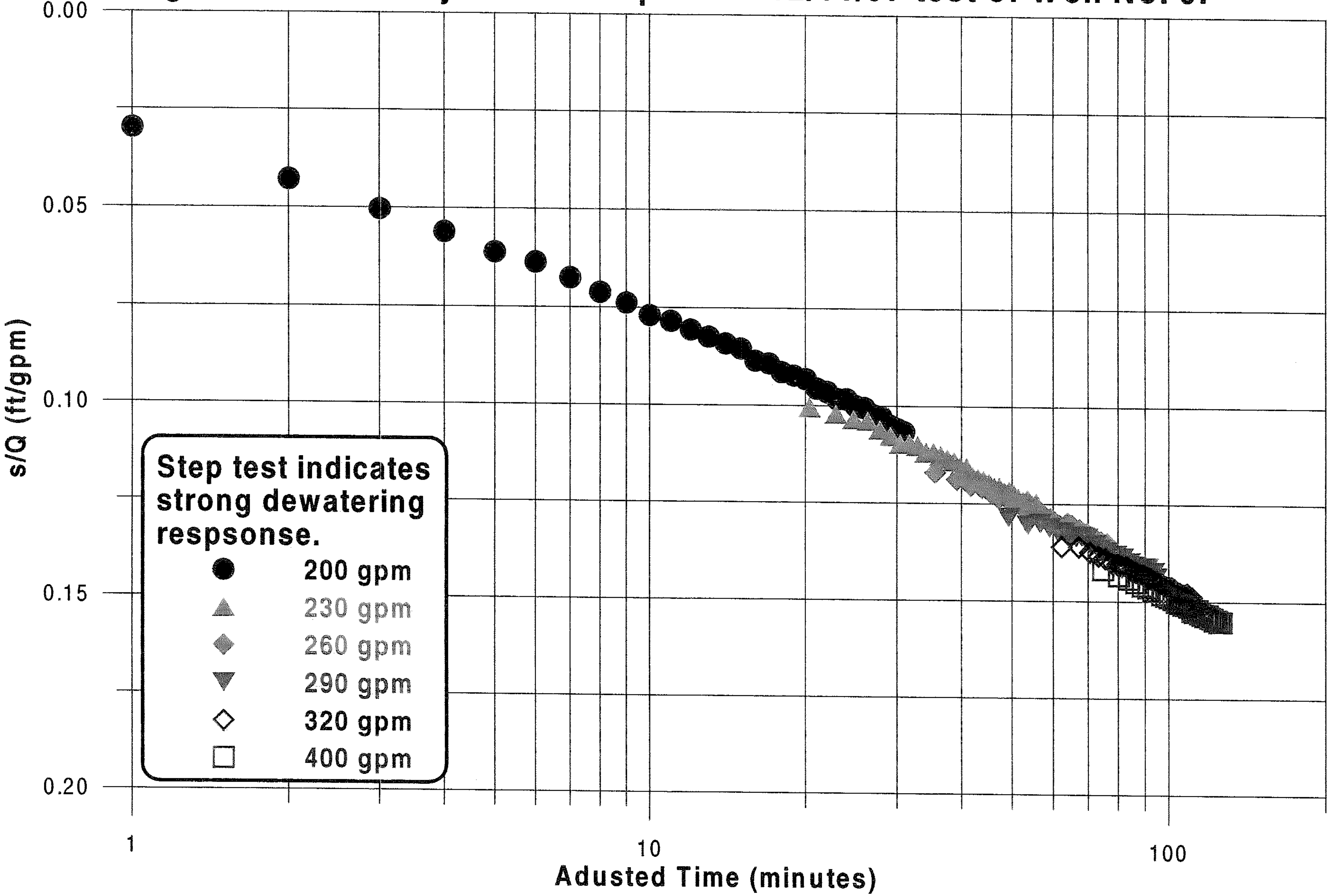

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Figure 3.41: Hantush-Bierschenk plot of 12/11/97 test of Well No. 9.

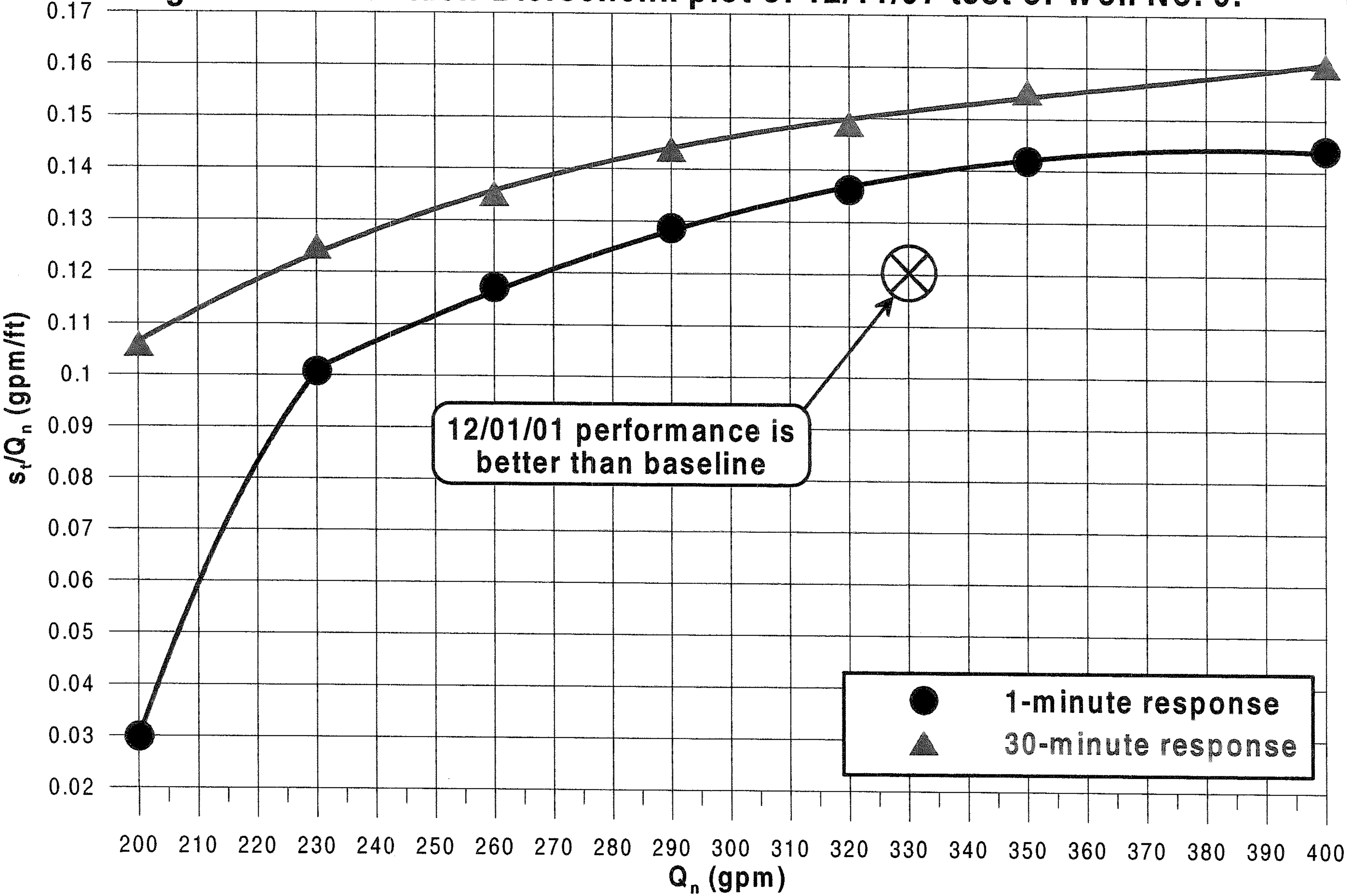

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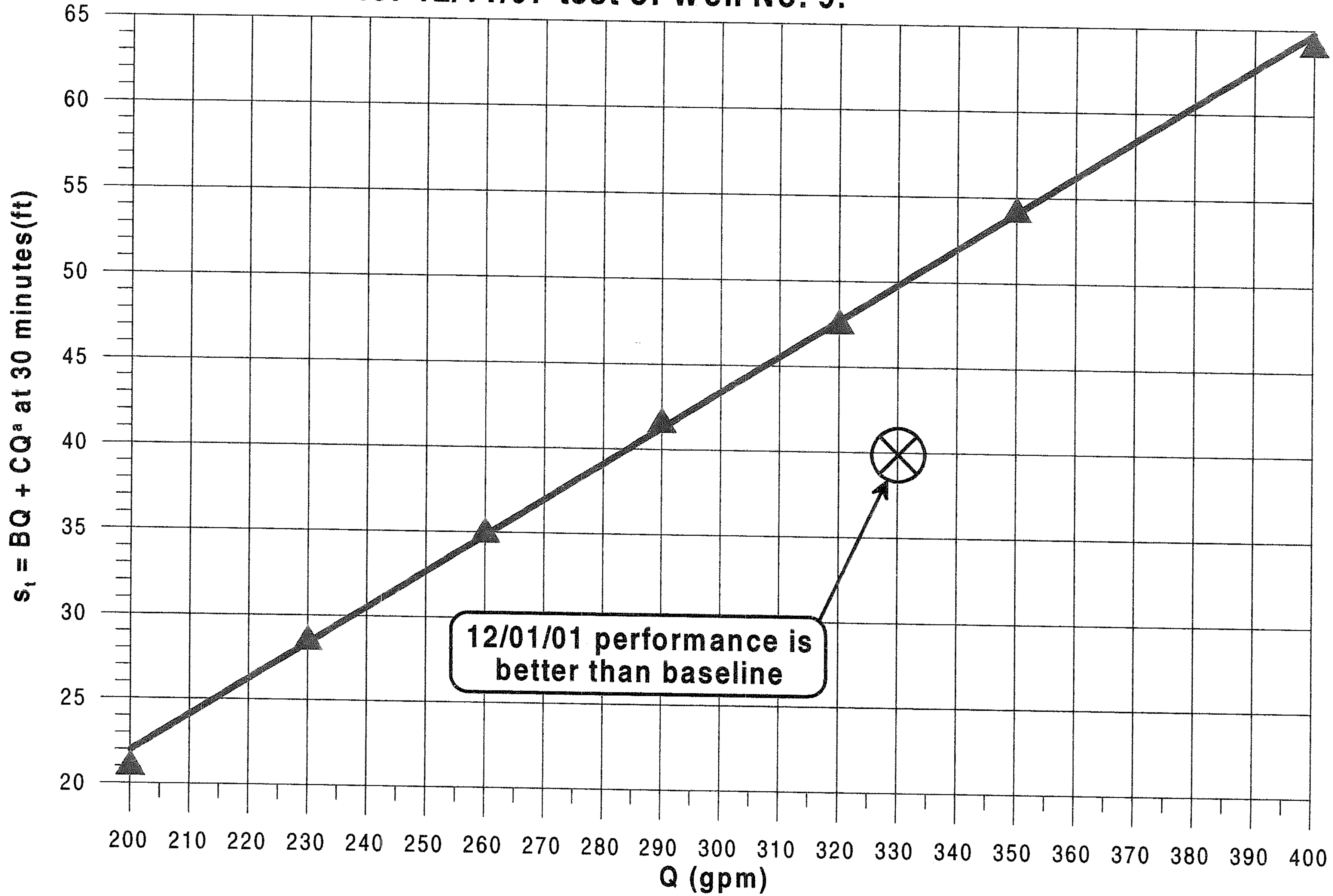
Figure 3.42: Total drawdown at 30 minutes versus pumping rate for 12/11/97 test of Well No. 9.



H:\WATER RESOURCES\MBK\miner flat wellfield\Well 9\speccap.grf total drawdown versus pumping rate on Figure 3.42 also shows improved hydraulic performance since 1997. It is not unusual for hydraulic performance of a well to improve after the well is put into service. The improvement in performance results from continued development of the well during operational pumping due to removal of fines and drill cuttings from the formation as well as restoration of borehole damage by removal of wall cake where bentonite drilling fluid was used.

The hydraulic performance of Well No. 9 is significant as evidence that plugging of well screens with iron oxide incrustation or other factors are not the cause of decreased yield from the well. As will be shown, Well No. 9 has suffered a loss of pumping capacity due to excessive wear or damage in the pump. The type of damage to the pump indicates that the pumping water level in the well was near the pump inlet depth and below the top of the well screens over a significant part of the historic operation of the well. Therefore, conditions in Well No. 9 were not significantly different than in the other wells in the Miner Flat Wellfield. Therefore, the fact that increased well loss and loss of hydraulic performance due to mineral encrustation or other factors did not occur at Well No. 9 suggests it is unlikely they occurred at the other wells. This is consistent with the indications that degraded well performance at the other wells is caused primarily by dewatering of the aquifer.

Figure 3.43 shows the baseline time-drawdown curve for the 400-gpm step of the 12/11/97 stepped rate test. The specific capacity curve on Figure 3.42 is used to calculate the time-drawdown curve that results from adjusting the 400-gpm drawdown to a hypothetical pumping rate of 300 gpm. The time-drawdown curve adjusted to 300 gpm is also shown on Figure 3.43 where it is compared to the time-drawdown curve produced on 12/01/01 during normal operation of Well No. 9. The 12/01/01 time-drawdown response exhibits a steeper slope than the baseline tests from 1997 because it was not preceded by previous stepped rates. Projection of the 12/01/01 time-drawdown relationship at 330 gpm indicates it will begin to parallel the slope of the baseline plots with less drawdown than the 330-gpm time-drawdown curve adjusted from the baseline test. This response again indicates the hydraulic performance of Well No. 9 is equal to or better than the 1997 baseline performance, despite a 43-foot decline in the static water level at the well.

3.9.6. Pump Condition

The original design data summarized on Table 3.19 for Well No. 9 indicates a nominal design pumping water level of 125 feet at 350 gpm which is equivalent to a total dynamic head of 254 feet, ignoring minor transmission losses in the pump column and transmission line. The design pumping water level of 125 feet appears to be very conservative, based on the baseline test which showed an essentially stabilized pumping water level of 125 feet after 24-hours of pumping at 400 gpm. However, the pump performance curve on Figure 3.44 indicates that the rated capacity of the pump

Figure 3.43: Time-drawdown response at 330 gpm on 12/01/01 equals or exceeds baseline performance on 12/11/97.

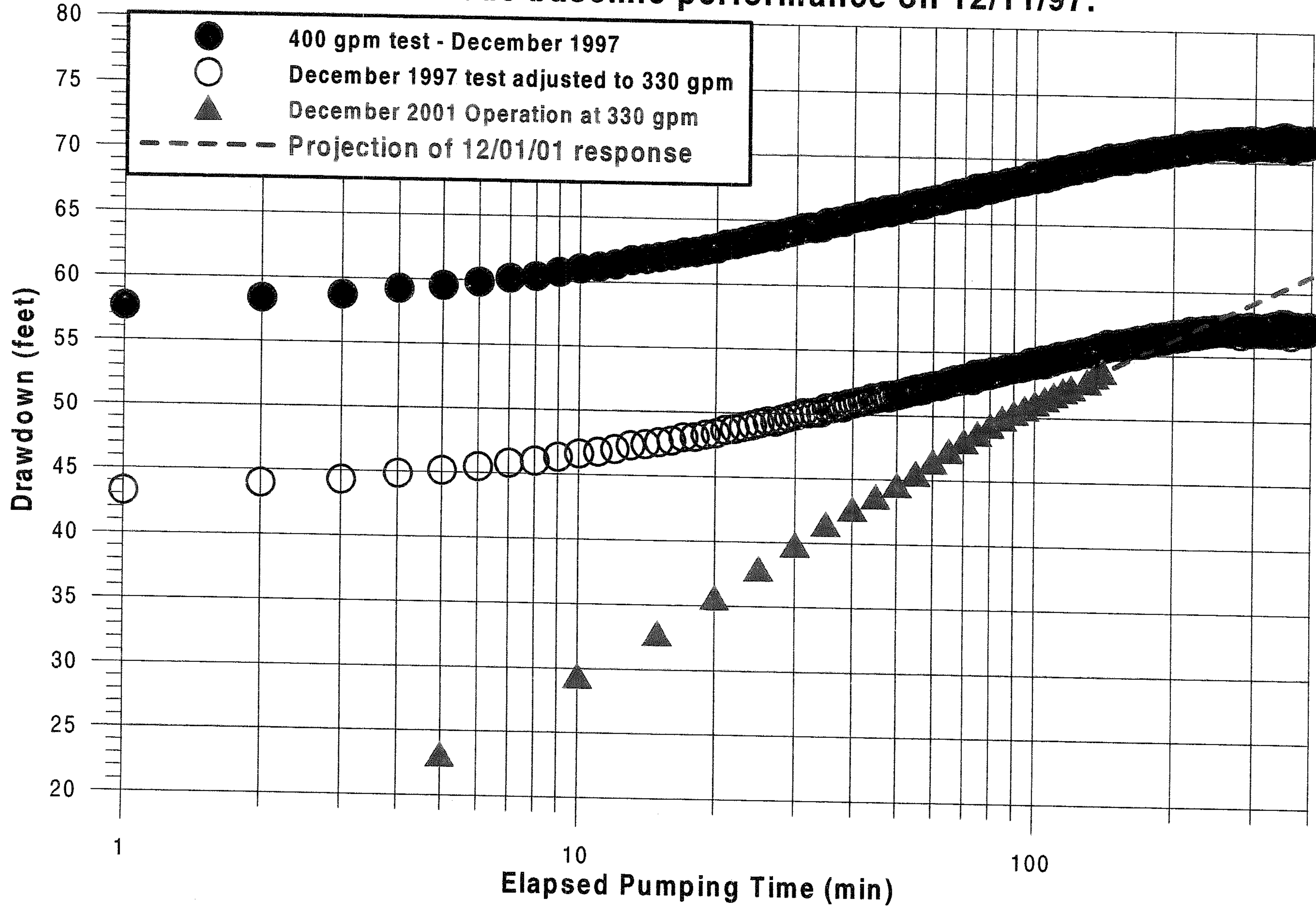


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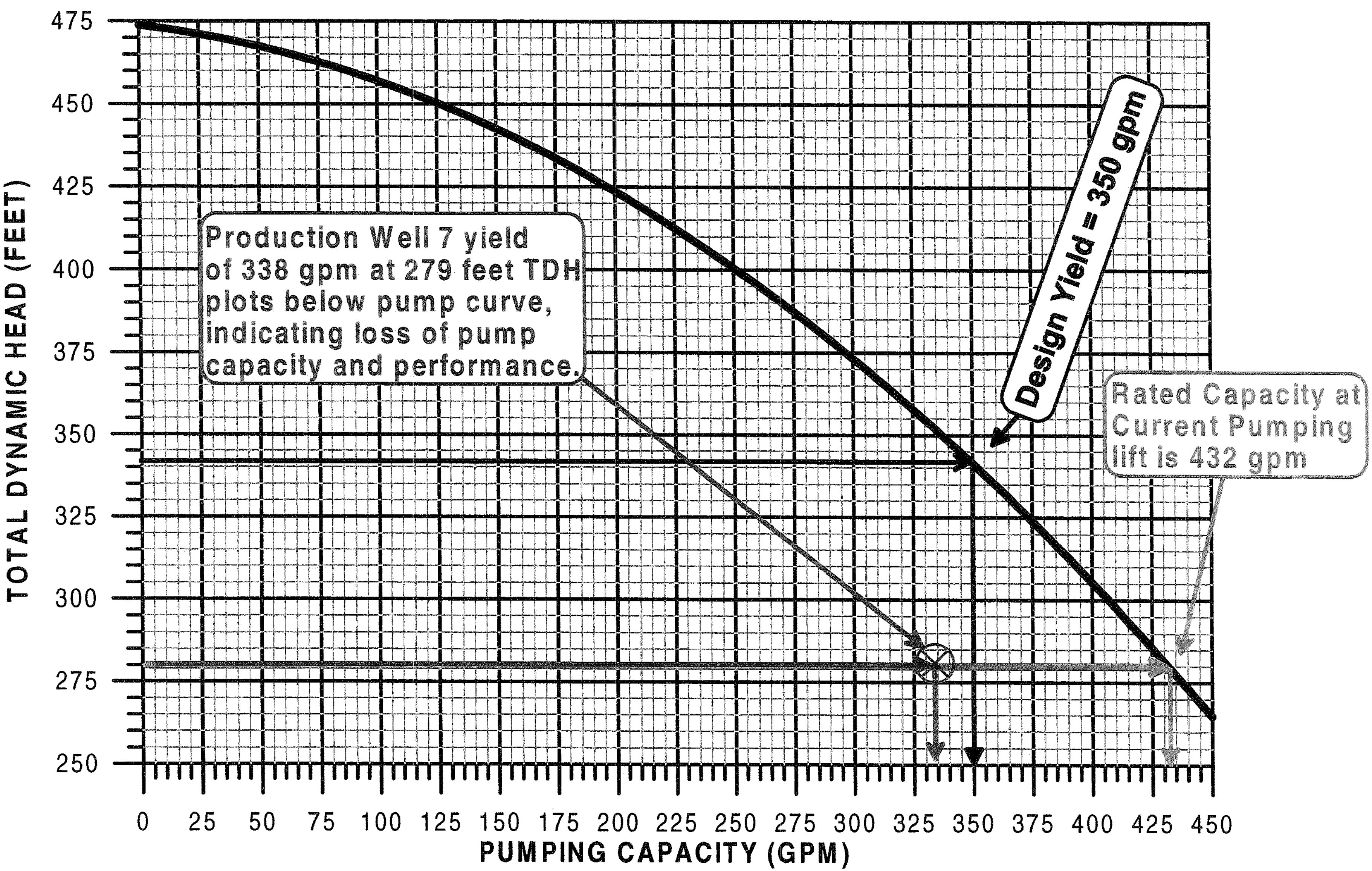
Figure 3.44: Pump performance curve for Goulds 7CLC040 pump shows 94 gpm loss of capacity due to pump wear at Well No. 9.



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selected for installation in the well is at least 450 gpm at the total dynamic head of 254 feet. Accordingly, the pump was oversized for the design flow of 350 gpm at a pumping water level of 125 feet, as shown on Table 3.19.

The pump performance curve also shows that the pump will produce 350 gpm with the pumping water level at 215 feet and a total dynamic head of 344 feet. Conversely, this means the pump that was installed in the well was rated to lower the pumping water level down to the pump inlet at 215 feet, if drawdown caused the water level to drop that far. The baseline tests indicate the pumping water level should not have been below 125 feet at 400 gpm and about 132 feet at 450 gpm, all at the prevailing groundwater levels in December 1997 which were approximately 43 feet higher than in December 2001. Thus, an explanation is required as to why the pumping water levels would drop so far below the top of the screen as to result in damage due to cavitation, as is evident from the December 2001 inspection.

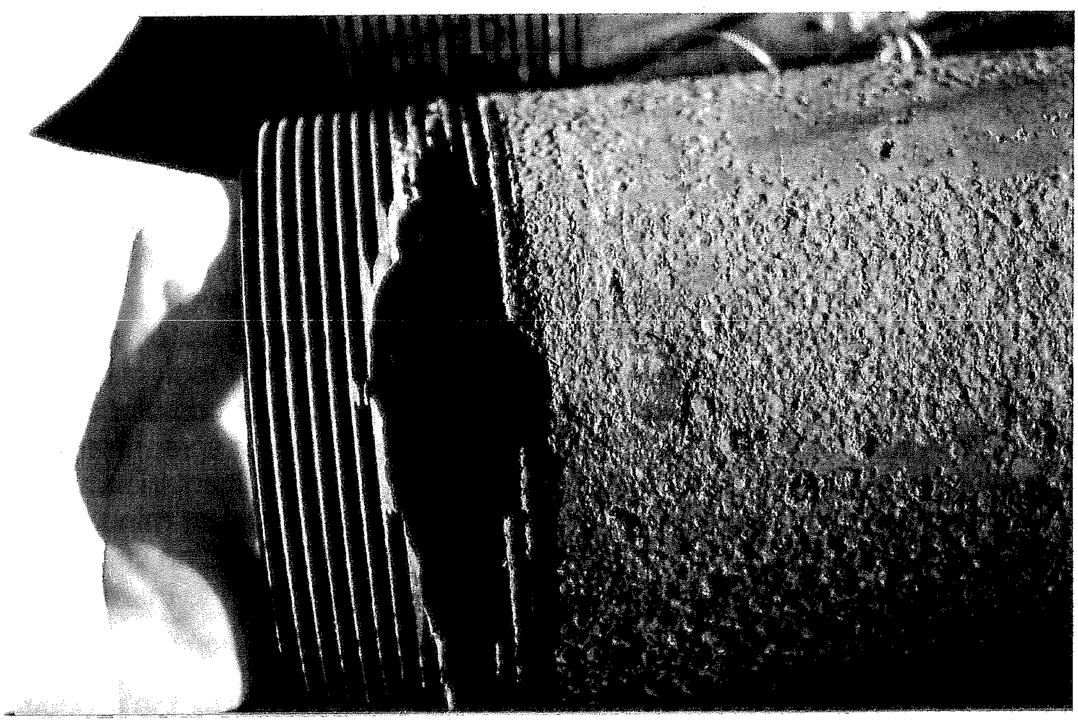
Photographs 3.13 and 3.14 show a hole that developed in the pump column pipe above the pump and check valve as the result of air entrained in the discharge water. The December 2001 pump production of 338-345 gpm from a pumping water level of about 150-151 feet, combined with the physical evidence of damage in the pump column by entrained air, indicates the pump impellers and seats were damaged over a period of time by entrained air. The damage ultimately reduced the capacity of the pump to the present production rate. It is probably not a coincidence that the present yield and drawdown performance maintains a pumping water level slightly below the top of the well screens as shown on Figure 3.38, exactly the maximum pumping level below which cascading water and air entrainment will begin to occur in the well. Progressive damage to the pump and associated loss of capacity eventually brought the yield and drawdown performance into equilibrium with a pumping water level at which cascading water and entrained air ceased.

The decline of 43 feet in static water level since December 1997 provides the explanation for foregoing conditions. Addition of 43 feet of decline to the 1997 pumping water level of 132 feet at 450 gpm provides a pumping water level of 175 feet. This is well below the top of the well screens at 145 feet. Cascading water and entrained air would occur well before the entire water level decline of 43 feet took place. In addition, turbulent flow through the dewatered portion of the well screen, combined with the need for the 350 to 400 gpm flow to pass through a shorter length of screen remaining below the dewatered part, would have resulted in an abrupt and significant increase in well loss drawdown, causing an increase in the unit drawdown per gallon per minute pumped, i.e., a decrease in specific capacity. The combined factors of cascading water and abruptly increasing well loss drawdown provide a reasonable explanation to the onset of damage to the pump and pump column in Well No. 9 by entrained air.

The pump performance curve on Figure 3.44 indicates that at the December 2001 pumping water level, the rated capacity of the pump is 432 gpm. The observed performance of 338 gpm indicates damage and excessive wear in the pump have resulted in a loss of pumping capacity of 94 gpm.

Photograph 3.13: Hole in pump column at Well No. 9 due to entrained air.

Photograph 3.14: View through hole in pump column at Well No. 9.



3.9.7. Recommended Pump Size

The present pumping water level of 150-151 feet is about 5-6 feet below the top of the well screen at an average yield of about 340 gpm. Increasing the pumping rate to more than about 340 gpm will repeat the history surmised for the pump presently installed in the well. In addition, the rate of groundwater level decline of 11.2 feet per year at this well will probably continue into the future. This suggests damage to the existing pump will continue to decrease the yield of the pump as the groundwater level declines. Under these circumstances, it is impossible to predict the short-term yield of the well for the next year or two because the well loss associated with dewatered well screen is not known.

Under these circumstances, the best approach may be to leave the existing pump in the well and let it continue to adjust to the declining water level until it fails. If it is necessary to replace the pump in the next two years, a pump capable of providing 250 gpm with a total dynamic head of 274 feet might be a good choice, depending on the shape of the pump performance curve. A pump with the appropriate performance curve will automatically pump less water as lift on the pump increases due to a declining groundwater level in the future. This will ultimately result in the pump operating in an inefficient head-discharge range, but will allow the pump yield to decrease in response to declining groundwater level. For example, a Grundfos 300S00-5 (30 hp) will produce about 300 gpm at 274 feet total dynamic head (TDH) and 78 percent efficiency; 240 gpm at 314 feet TDH and 76 percent efficiency; and 195 gpm at 354 feet TDH and 71 percent efficiency. The latter pump is offered as an example of a pump selection principle, not as a recommendation for that specific pump for Well No. 9.

3.10. Well No. 10

Drilling of Well No. 10 started 10/18/97. After a checkered history, which included recovering the 8-inch casing and screen from the well, re-cementing the surface casing and replacing the 8-inch casing and screen back into the well, a final baseline pumping test was conducted on 7/7/98 and the well was put into service some time after that date. The construction history is described below. Although the final baseline test was limited to 190 gpm for 15 hours, resulting in a pumping water level of 208.45 feet, there was evidence that the well yield was improving and a 200-gpm design rate was selected for the well. Frankie Williams, Water System Operator, reports that the well has not presented any problems in the past 3-1/2 years except for a hole in the pump column pipe. The 12/01/01 yield of the well was 147 gpm at an unknown pumping water level.

3.10.1. Geologic Log

Well No. 10 was logged by Trevor Haig, geologist for Morrison-Maierle, Inc. A highly condensed summary of the detailed geologic log is provided in Table 3.20.

Table 3.20: Production Well No. 10 geologic log.

Depth Interval (feet)	Description	Geologic Interpretation
0 - 135	CLAY, brown and gray, silty, with cobbles.	Alluvium
135 - 160	BASALT	Basalt
160 - 212	SANDSTONE, tan, white, brown first water at 180 ft, increases to 150 gpm by 212 ft.	Coconino
212 - 270	SILTSTONE & CLAYSTONE, light brown, silty.	Supai
270 - 280	CLAYSTONE, light brown, silty, with water-bearing sandstone layers, discharge increases from 150 to 250 gpm.	Supai
280 - 390	SANDSTONE, reddish-brown, silty, fine-grained, discharge increases to 300 feet	Supai

Well No. 10 penetrates unconsolidated overburden, consisting of alluvial clay containing lenses of sand, silt, and cobbles, to a depth of 135 feet. Basalt was penetrated from 135 to 160 feet. The uppermost 10 feet of the Coconino Sandstone, from 160-170 feet, were baked by the overlying basalt and included some clayey material. Although the top of the Supai was penetrated at 212 feet, it consisted of claystone and siltstone from 212 to 285 feet and it did not yield water until the 270-280 foot zone was penetrated and an increase in yield of about 100 gpm (150 gpm increased to 250 gpm) was observed. Supai sandstone from 285 to 390 feet produced a gradual but noticeable increase in yield from 286 to about 300 feet.

3.10.2. Construction Data

Table 3.21 provides a summary of construction data for Well No. 10. The surface casing depth is not recorded in the field notes. The I.H.S. as-built drawing of the well indicates 12-inch nominal diameter surface casing from land surface to 180 feet. The as-built and the field notes indicate the well was completed with 8-inch nominal diameter steel well casing and 8-inch pipe size stainless steel well screen to a total depth of 375 feet. The screened intervals reported in the field notes are as follows:

180 – 210 feet
260 – 370 feet

The well screen consisted of 20-slot stainless steel screen installed with a Colorado Silica 10-20 production sand filter pack. The filter pack was reportedly installed to within 60 feet of the land surface in the annulus outside the well casing.

Table 3.21 Production Well No. 10 depths and elevations.

Parameter	Depth (feet)	Elevation (feet)
Ground elevation	0	6155
Tank overflow		6258
Static water level (swl)	73	6082
Top of well screens (BGL)	180	5975
Bottom of well screens (BGL)	370	5785
Pumping water level (pwl)	290	5865
Intake depth	362	5793
Drop pipe length	357	
Total cased depth	375	5780
Nominal pump capacity (gpm)	200	
Pump horsepower	25	

3.10.3.Construction History

Initial drilling of Well No. 10 began at an unspecified date, prior to 10/18/97. The first field note for the well is on 10/18/97 and states, "Surface casing loose, will re-cement today." A 10/19/97 field note indicates the surface casing was set to only 120 feet and provides a geologic log to a total depth of 200 feet of drilling that was accomplished prior to setting surface casing. The depth of the 12-inch borehole was extended from 200 feet to 390 feet on 10/19/97 with direct air rotary drilling. On 10/20/87, the borehole was found to be blocked with caved material at 115 feet. In a subsequent letter report, the caving is described as blocking the hole at 120 feet, which is the bottom of the surface casing, and is attributed to the uncased alluvium from 120 to 135 feet.

The notes for 10/20/97 and 10/21/97 indicate that the drilling method was changed from direct air rotary to direct mud rotary using bentonite and EZ Mud (a polymer additive) and that circulation back to the surface was never achieved even though the caved material was eventually pushed to the bottom of the hole and ground up. The drilling contractor began to install 8-inch casing and screen at 1600 hours on 10/21/97. Although a tight spot was encountered at 125 feet, the casing and screen was eventually installed to a total depth of 375 feet. The tremie pipe used to install filter pack would not advance past 315 feet, so filter pack was installed from that depth with the hole taking only 14 sacks of pack. The true-hole size annulus volume between 12-inch borehole and 8-inch casing is equal to 22.8 sacks of filter pack from 315 to 375 feet and any open hole remaining below 375 feet below the casing would require more pack. Therefore, it is obvious the lower part of the well screen was not filter packed. The total filter pack volume installed in the annulus was 14 pallets. At 30 sacks per pallet (30 ft^3/pallet), this totals 420 ft^3 which is enough sand volume to fill 1,127 feet of true-hole size annulus in Well No. 10. This indicates the individual caved areas, voids, and rugosity of the borehole required more than a 300 percent overrun on filter pack.

A test pump was installed on 10/31/97 and a stepped rate test was conducted on that date. The duration of each pumping rate of the step test ranged from 18 to 25 minutes and rates of 150, 180, 210, 240, 270, and 300 gpm were achieved. The pump broke suction after 10 minutes of pumping at 300 gpm. This test is significant in that it shows the aquifer was capable of supporting much larger pumping rates than were subsequently possible after a series of events which appear to have damaged the borehole wall and filter pack such that well yield was reduced to 190 gpm or less.

Before a constant rate test of Well No. 10 was undertaken, it was discovered on 11/20/97 that the 8-inch well casing and screen assembly was slipping down into the borehole, evidently into the void remaining below the casing assembly where filter pack was not installed. On 6/27/98, the casing and screen were removed from the borehole and the borehole was plugged with cement from 170 feet back to the surface casing at 120 feet. Cement was then displaced up the annulus from 120 feet to the surface to re-seal the surface casing. After the cement cured, the 12-inch diameter borehole was re-drilled to the original depth of 390 feet with mud rotary methods, removing the cement plug and all cavings and cuttings. The original casing and screen was reinstalled in the well to 375 feet, all of this in borehole filled with bentonite-based drilling fluid which circulated back to the surface, unlike the prior work when circulation could not be established. This indicates the drilling fluid, as well as clay from the caved alluvium, finally formed a filter cake on the borehole wall that prevented loss of water into the sandstone and vice versa.

A tremie pipe was installed to an initial depth of 360 feet and used to install filter pack. There is no record of the volume of sand used to fill the annulus and open borehole below 360 feet. Presumably it was filled with filter pack. The field notes from 6/29/98 indicate that the interval from 300 to 360 feet required 5-1/2 sacks of pack per 20-foot length of annulus and the interval from 40 to 300 feet required 9-1/2 sacks of pack per 20-foot length of annulus. This totals 149.5 sacks of filter pack (149.5 ft^3) for an annulus with a true-hole volume of 121.5 ft^3. An additional four sacks of filter pack were added to the annulus during development, bringing the total pack in the 40 to 360 foot interval to a volume of 153.5 ft^3.

Comparison of the 153.5 ft^3 of pack used after the hole was cleaned to the 420 ft^3 of pack used the first time the well was constructed, indicates that more than 260 ft^3 of filter pack, mixed with cuttings and bentonite, remained around the borehole the second time the well was cased, screened and filter packed. This fact became obvious when baseline tests of the reconstructed well were attempted. After the well was developed by air-lift surging until little or no improvement was gained with additional surging, a pump was installed in the well on 7/2/98. The pump was used to develop the well from 7/2/98 through 7/6/98. The field notes indicate the well continuously produced water turbid with bentonite and containing large amounts of sand at rates of 100 to 150 gpm on 7/2/98 and 7/3/98. Late in the afternoon of 7/3/98, the pumping water level at 150 gpm was 259.5 feet with water cascading in the well and the pump ultimately breaking suction with the pumping water level at 270 feet. This demonstrates the effect of entrained air, considering that the test pump inlet was at 320 feet, some 50 feet below the pumping water level when entrained air from cascading water caused the pump to break suction at 150 gpm.

A stepped rate test was conducted on 7/4/98 to provide data from which to judge the progress of the development by overpumping. The field notes indicate the well discharged "concentrated bentonite" at pumping rates as low as 80 gpm. At 160 gpm, the pumping water level was 260 feet, and four minutes into a 180-gpm step, the pump broke suction. The well was surged and overpumped the remainder of 7/4/98 to continue development.

On 7/5/98, the field notes at 0600 hours state, "*discharge clear, very little to no sand, discharge rate steady at 150 gpm*," evidently after the well had been pumped overnight at a constant discharge rate. The pump was stopped at 0625 hours and the notes state, "*Have pulled LOTS of bentonite out of hole*". A 180-gpm constant rate test was started at 0850 hours with the static level recovered to 76.66 feet BTOC and the well discharged water turbid with bentonite until the pump was stopped at 0950 due to cascading water and air entrainment with the pumping water level at more than 279 feet. Development with the pump continued through the remainder of 7/5/98 and 7/6/98.

On 7/7/98, a final constant rate test was conducted at 190 gpm. The test was terminated after 15 hours of pumping at 190 gpm because the pumping water level declined to 285.35 feet BTOC and the resultant cascading water and entrained air was likely to damage the test pump. The conclusions resulting from the 190-gpm test were that the well was limited to a 15-hour pumping duration at the 190-gpm pumping rate and that more development of the well should be performed in an effort to improve the yield and drawdown characteristics.

Based on the first pumping test of the well on 10/31/97, a design rate of 250 gpm had originally been established as a target for production. It is clear that the borehole and filter pack conditions resulting from the various construction activities damaged the potential yield of the well, causing a reduction in step test rates from 150 to 300 gpm obtained on 10/31/97 to rates limited to 100 to 200 gpm on 7/6/98. The most likely cause of this reduction in step test yields is a mixture of filter pack and bentonite-based drilling mud remaining in enlarged portions of the borehole from the 420 ft^3 of filter pack installed during the initial construction effort.

3.10.4. Water Levels

When Well No. 10 was inspected in December 2001, the static water level was measured at 105.65 feet BTOC. It was not possible to measure the pumping water level below 139 feet. An attempt was made to put both a digital logger and a manually operated electrical water level measuring device down the well, through the pump motor cableway openings on each side of the pitless unit spool. The devices would not descend below 139 feet through the cableway not in use and would not descend below 134 feet through the cableway containing the pump motor cable. The devices tended to momentarily catch on a restriction at 126 feet on both sides of the well.

A static water level of 73.2 feet BGL was measured at Well No. 10 on 10/31/97. Accordingly, the static water level of 105.65 feet BTOC indicates a decline in the groundwater level at Well No. 10 of about 32 feet in the four-year period since the well was drilled. The difference between the ground level and top of casing reference points (BGL and BTOC) is less than one foot. Comparison of the 10/31/97 and 12/01/01 static water levels to the base of the basalt layer at 160 feet (Table 3.20) indicates the aquifer at Well No. 10 is confined during static conditions.

Figure 3.45 displays the 10/31/97 baseline test response at Well No. 10, prior to reconstruction of the well. The response, similarly to that at the other wells in the Miner Flat Wellfield, indicates the onset of dewatering effects after about 10 minutes of pumping at 150 gpm and throughout subsequent steps of the test. The difference between the 210-, 240- and 270-gpm curves on Figure 3.45 indicates an abrupt increase in well loss drawdown at rates greater than the 210-gpm step, indicating the maximum yield of the well prior to reconstruction was in the range of 200 to 210 gpm.

The 7/6/98 stepped rate test of the well, after reconstruction, indicated the pump was subject to cascading water and was discharging entrained air after 15 hours of pumping at 190 gpm, with a pumping water level of 285.35 feet and a test pump inlet setting of 320 feet. Although the pumping water level could not be measured during the December 2001 inspection, the report by Frankie Williams of a hole in the pump column pipe indicates that the pumping water level is deep enough that the existing production pump, set with an inlet depth of 362 feet, remains subject to the problem of cascading water and entrained air.

3.10.5. Pump Condition

The statement by Frankie Williams, Water System Operator, that a hole was found in the pump column at Well No. 10, similar to that shown on Photographs 3.13 and 3.14 at Well No. 9, is consistent with the original design rate of 200 gpm (Table 3.21) and air entrainment problems at the final baseline test rate of 190 gpm. However, it is necessary to review the pump performance curve of the pump selected as a production pump before concluding that the nominal design rate of 200 gpm was excessive.

Figure 3.46 shows the pump performance curve. The pump performance curve indicates that the pump was rated for 341 feet of total dynamic head at the design yield of 200 gpm. However, Table 3.21 shows that the design pumping water level assumed for the 200-gpm design yield was 290 feet. A pumping water level of 290 feet in Well No. 10 is equivalent to a total dynamic head of 393 feet, ignoring minor head losses in the pump column and distribution lines. The pump is rated to produce 91 gpm with a total dynamic head of 393 feet. Considering that the pumping water level associated with the 190-gpm test rate was 285.5 feet, equivalent to 388.5 feet of total dynamic head, the selected pump could not possibly produce the design yield of 200 gpm from Well No. 10 unless the conditions limiting yield from the well improved due to ongoing

Figure 3.45: Birsoy-Summers solution for 10/31/97 baseline test of Well No. 10. The well was subsequently reconstructed and re-tested.

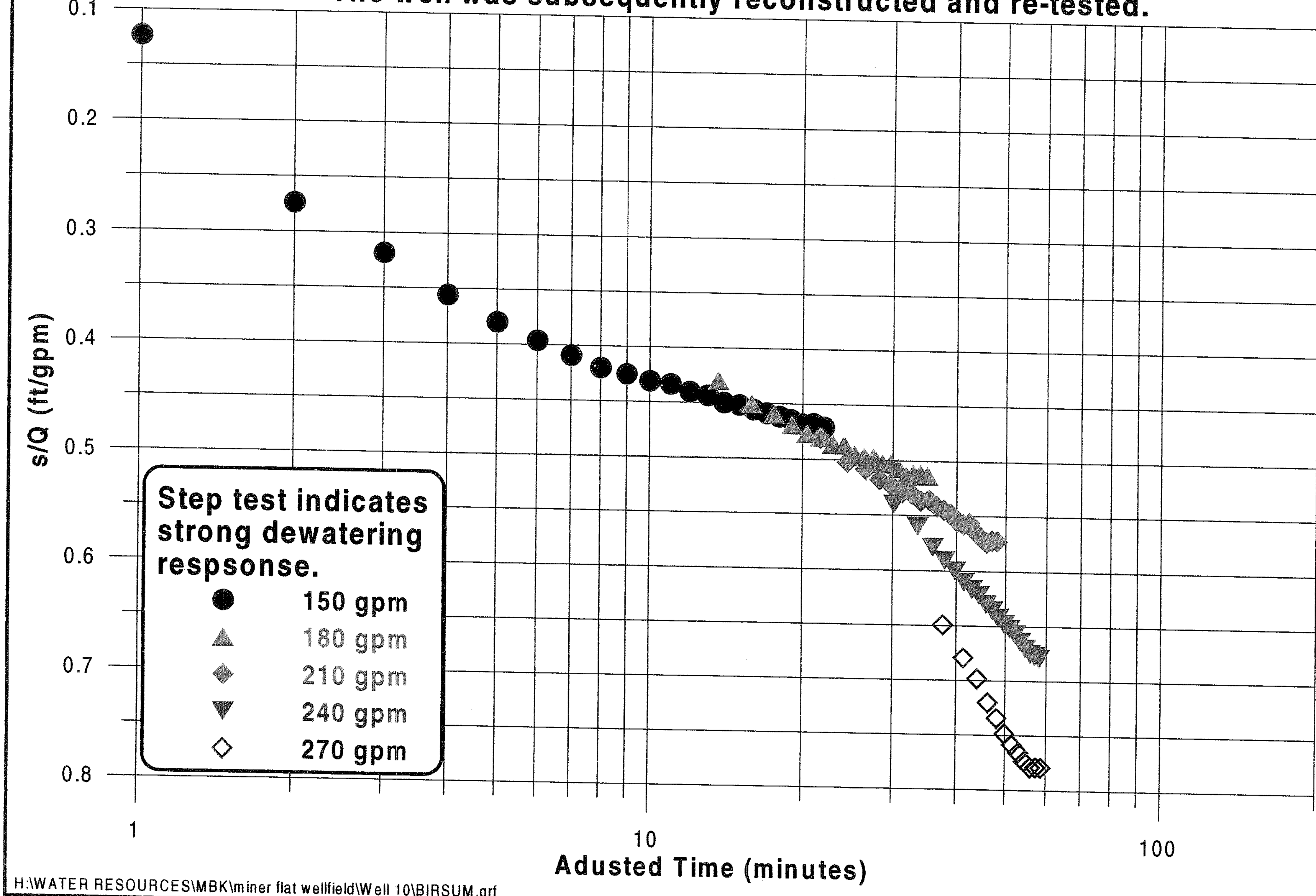
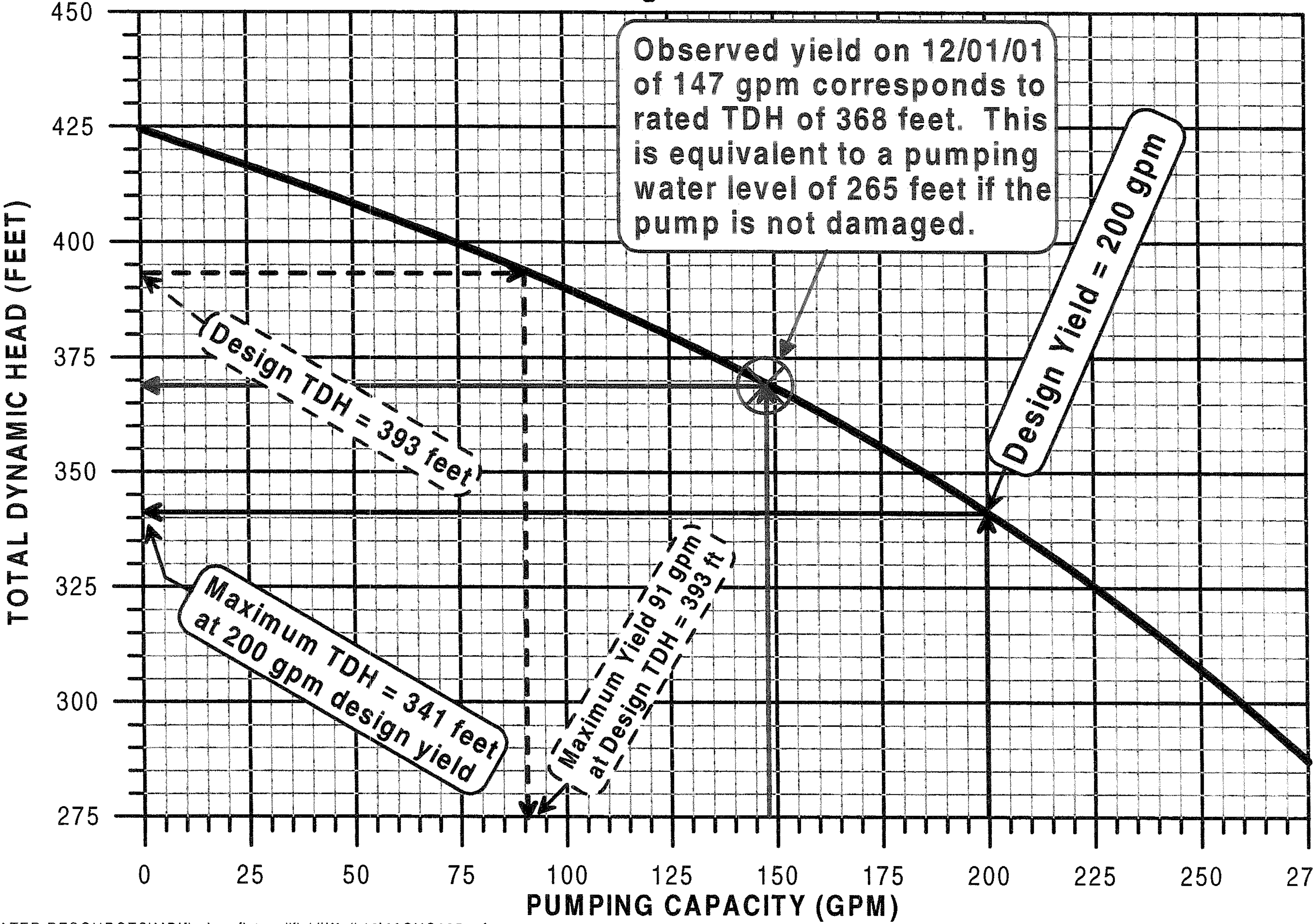


Figure 3.46: Pump performance curve for Goulds 6CHC025 pump compared to observed discharge of Well No. 10 on 12/01/01.



H:\WATER RESOURCES\MBK\miner flat wellfield\Well 10\60CHC025.grf development as the well was pumped. At the end of the baseline test in July 1998, the selected pump would have produced a pumping rate somewhere between 91 and 200 gpm, depending on the convergence of the drawdown or pumping water level with the total dynamic head on the pump performance curve.

This means the unknown pumping water level would be somewhere between 238 and 290 feet under prevailing groundwater levels in July 1998. The discharge rate of 147 gpm observed on 12/01/01 corresponds to a rated total dynamic head of 368 feet, as shown on Figure 3.46, assuming the pump is in good working order. A total dynamic head of 368 feet at Well No. 10 is equivalent to a pumping water level of 265 feet which is about midway between the extremes defined by the design yield and design head on Table 3.21.

The construction data summarized on Table 3.21 show that the potential range of pumping water levels associated with the selected pump, 238 to 290 feet, is below the bottom of the uppermost well screen which is in the 180 to 210 foot depth interval of the well. Therefore, operation of the selected production pump would automatically result in dewatering of the upper well screen, cascading water, and air entrainment at any potential discharge rate the pump would potentially produce when it was first installed.

If the 32-foot decline in the static water level since 10/31/97 is added to the rated pumping water level of 265 feet for the 147-gpm discharge rate, the pumping water level in December 2001 was theoretically at 297 feet. This is considerably deeper in the well, i.e., more drawdown, than would be predicted for a 147-gpm pumping rate based on the 7/6/98 and 7/7/98 test performance of the well. It is therefore more likely that entrained air, resulting from dewatering of the upper well screen, has damaged the pump over a period of time and the 12/01/01 pumping water level is unknown, but shallower than 265 feet.

The short-term pumping water level on 7/6/98 was about 180 feet at 140 gpm and no more than 200 or 205 feet at 160 gpm. The 147-gpm rate observed 12/01/01 therefore suggests a pumping water level at about 190 feet. Again, it may not be a coincidence that the pumping rate of the damaged pump is equal to the maximum rate the well can produce without causing cascading water and entrained air. Addition of the last three and one-half years of decline in static water level to the 190-foot estimate suggests the pumping water level under present conditions may be closer to 222 feet; however, the hydraulic performance of the well may have improved enough over the past three and one-half years to compensate for the decline in water level. The short-term drawdown at 150 gpm projected to about 155 feet during the 10/31/97 step test, prior to reconstruction of the well. This indicates that development of Well No. 10 due to operation since July 1997 could improve the well back to at least the 10/31/97 performance. Addition of 32 feet of water level decline to the latter pumping water level projects it to 187 feet by December 2001.

3.10.6. Recommended Pump Size

Although the interpretation indicating a12/01/01 pumping water level of about 187 to 190 feet is speculative, it is consistent with the present well yield and the fact that there was no sound of cascading water and no sound of entrained air when the pump was in operation on 12/01/01. The best course of action may be to simply continue operating the existing pump until it fails. All indications are that the groundwater level at Well No. 10, and at the other wells in the wellfield, will continue to decline. The rate of decline since the wellfield was put into operation has been 9.55 ft/yr. Taking all of the above considerations into account, a replacement pump capable of delivering 110 gpm at the presently unknown total dynamic head is recommended when the pump presently in the well fails.

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APPENDIX W

Evaluation of the Structural Geology of the Miner Flat Wellfield

MORRISON-MAIERLE, INC.

FEBRUARY 2007

EVALUATION OF STRUCTURAL GEOLOGY MINER FLAT WELLFIELD

Fort Apache Indian Reservation
Whiteriver, Arizona

Prepared for:

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OCTOBER 2005

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1. SUMMARY

Localized folds and faults imposed over the regional structure of the Coconino aquifer are highly significant to the availability of groundwater on the Fort Apache Indian Reservation. This is because the local folds and/or faults in some parts of the reservation modify the regional structure enough to create subsurface traps for groundwater or to divert the flow of groundwater into reservation springs, away from the regional dip. The area of C-aquifer developed by the Miner Flat Wellfield is in a low part of the geologic structure that contains stored groundwater.

An investigation of the Miner Flat Wellfield geologic structure supports the following conclusions:

1. The groundwater body penetrated by the Miner Flat Wellfield is bounded on the west by the base of the Coconino Sandstone as it rises up westward out of a small syncline (downward fold) that contains the wellfield.

2. The southern margin of the groundwater body penetrated by the Miner Flat Wellfield is likewise bounded by the intercept of the water table and the base of the Coconino Sandstone as the strata rise up-dip to the south. An additional southern boundary is Post Office Canyon where it penetrates the sandstone and provides a potential drain, if groundwater levels should ever rise to the elevation of the canyon floor.

3. The eastern boundary of the groundwater body exploited by the wellfield is locally the basalt in the ancestral river channel of the North Fork of the White River that prevents the groundwater from draining into the modern North Fork of the White River where its valley is incised through the Coconino Sandstone east of the wellfield.

4. The northern boundary of the groundwater body cannot be determined from the available data. Various factors discussed herein, combined with the declining yield of the Miner Flat Wellfield since it was placed into service 7.5 years ago, indicate that whatever the boundary is, there is not a significant flow of groundwater to the wellfield area from the Coconino Sandstone north of the wellfield. The absence of significant natural discharge from the aquifer, available to be diverted to sustain a wellfield, may be due to a fault severing the strata, drainage of the discharge to surface watersheds upstream from the wellfield, a simple lack of significant flow from the Mogollon Rim, or all three factors.

2. INTRODUCTION

Whiteriver, Arizona, (Figure 1) is the seat of government for the White Mountain Apache Indian Tribe on the Fort Apache Indian Reservation. Whiteriver and nearby reservation communities, referred to herein as the greater Whiteriver area, are the center of the

H:\WATER RESOURCES\1780 wmat\016.01\dwgs\VICINITY figure.DWG Plotted by jerry m. hallford on 9/19/2005

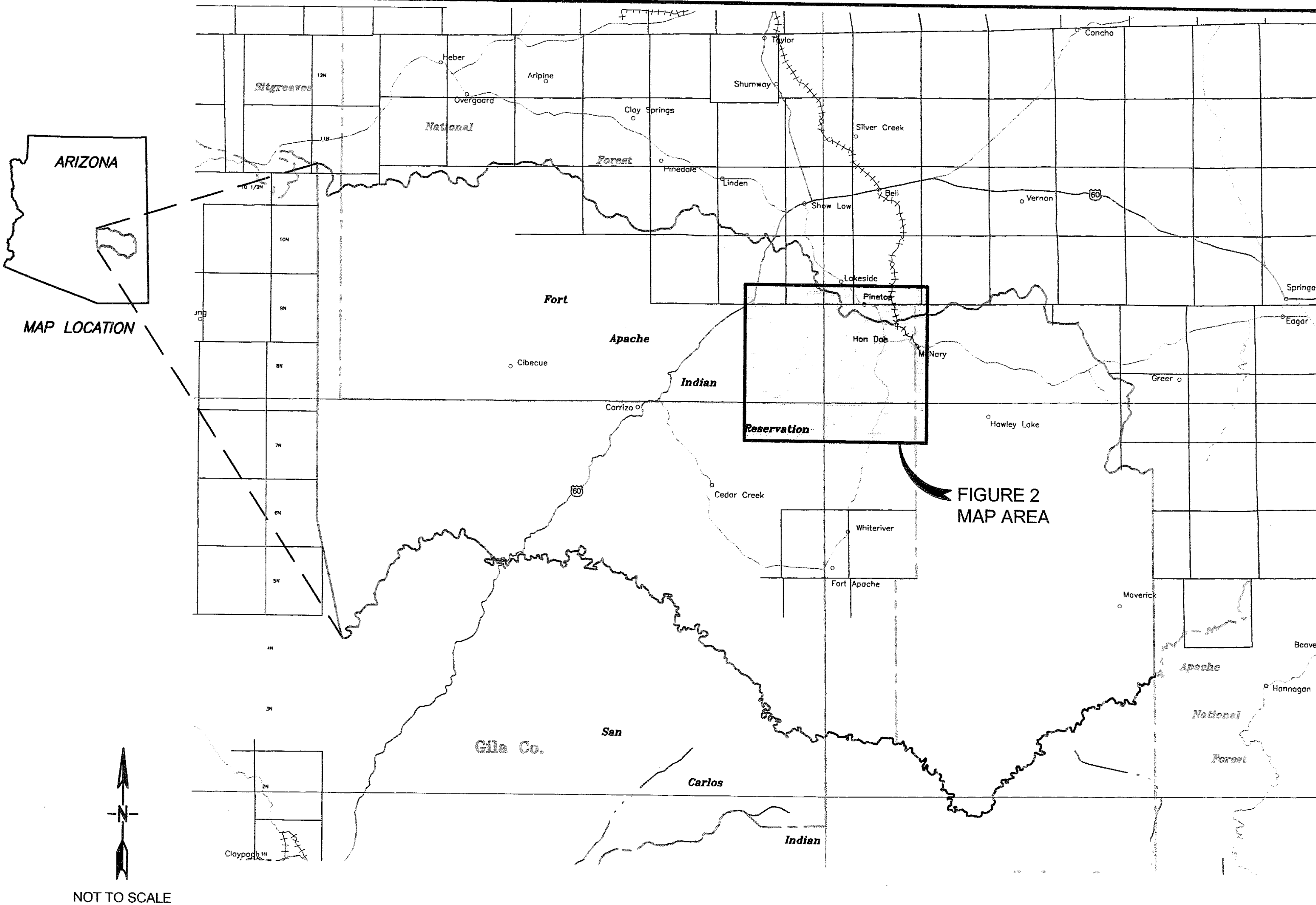


FIGURE 1
VICINITY MAP

largest concentration of population on the reservation. The principal source of municipal water supply to the greater Whiteriver area is the Miner Flat Wellfield, located approximately nine miles north of Whiteriver on State Highway 73 in Sections 16 and 21 of Township 7 North, Range 23 East (Figure 1).

Well yields in the Miner Flat Wellfield have steadily decreased since the wellfield was put into full-time production with 10 operating wells in January 1998. Baseline pumping tests conducted as each well was completed indicated a collective wellfield yield of 2,940 gpm in January 1998. The wellfield was initially equipped with pumps collectively producing 2,750 gpm. The White Mountain Apache Tribe sponsored an evaluation of the wellfield in December 2001 to quantify the wellfield yield and conditions in view of a decline in yield. The December 2001 study (Kaczmarek, 2002) determined that the collective yield of wells in the Miner Flat Wellfield had decreased to 1,591 gpm or about 58 percent of the original pumping equipment yield in January 1998. The decline in yield was due to declining groundwater levels in the wellfield.

In April 2005, the collective yield of the wellfield was reported to be 1,000 gpm in the winter months, decreasing to 900 gpm in the summer months of longer daily pumping duration (Frankie Williams, verbal communication, 2005). Addition of three new wells north of Cottonwood Creek, put into service after April 2005, increased the wellfield production to an average rate of 1,400 gpm in July 2005 (Frankie Williams, verbal communication, 2005). Columbine Spring, the only additional water source in the system, produced an additional 350 gpm in July 2005. Storage in the 562,431-gallon reservoir at the wellfield declined to about 25 percent of total capacity on a daily basis in the first two weeks of July 2005, indicating daily demand was exceeding Columbine Spring and wellfield yield of 1,750 gpm.

This report was sponsored by the White Mountain Apache Tribe as part of an on-going effort to determine the physical boundaries of the groundwater resource developed by the Miner Flat Wellfield. The goal was to identify any geologic limitations that have contributed to the marked decrease in wellfield production during a relatively short seven-year history of production. The investigation was initiated as a geologic mapping effort to identify the extent and distribution of the principal geologic units providing groundwater to the wellfield, i.e., the Coconino Sandstone and a thin red sandstone at the top of the Supai Group. However, it became evident in the first few days of fieldwork that mapping the distribution of surface outcrops of the Coconino Sandstone and related water-bearing strata would not provide sufficient information to determine how the geology might exert limitations on the groundwater resource available to the wellfield. Consequently, the effort was redirected to evaluation of the structural geology of the wellfield and surrounding area, instead of detailed mapping of the Coconino outcrop.

Through the 1990s there was an unresolved question respecting the ability of the well field to provide a sustainable and dependable yield. This history is presented in Appendix A that provides an account of the early investigations and the uncertainty associated with the well field. Regardless of the unresolved question about long-term sustainability of the wellfield at that time, the fact was that the Tribe was experiencing critical water supply shortages in the early 1990s. Preliminary indications were that wells of sufficient yield to relieve the shortages, if only in the interim, could be drilled in the Miner Flat area (Appendix A). After Well No. 3 was pumped continuously at 400 to 500 gpm for 70-days, plans to move forward with the wellfield were implemented with an extensive drilling program and construction of production wells in 1997. In January, 1998, the wellfield was put into service with 10 operating wells.

Minimum well spacing recommendations (Appendix A) were used to site the wells, with individual spacing between wells ranging from 600 to 1,200 feet, thus exceeding by a liberal margin the 500-foot spacing recommended for wells producing less than 350 gpm and falling within the range of recommended spacing for 500-gpm wells. In addition, the wells were arrayed in a roughly "L-shaped" pattern to further limit the potential for drawdown interference between individual wells as compared to the rectangular well arrays recommended in preliminary planning studies (Golder Associates, 1994; Figure 25). The large spacing between wells and distribution of the wells over a relatively large area as compared to the Golder conceptual layout were intended to minimize the collective local effect of the wellfield on groundwater levels in the aquifer and to prolong the life of the wellfield.

As previously described, the completed wells initially produced 2,940 gpm collectively, considerably less than the hypothetical 4,000-gpm production discussed in the initial studies that led to implementation of the wellfield construction (Golder Associates, 1994 and Robinson, 1995). More significantly, the wellfield yield declined from the initially installed pumping capacity of 2,750 gpm in January 1998 to 900 gpm by the summer of 2005.

It is within the context of the foregoing experience in developing the groundwater resource at the Miner Flat Wellfield area that the White Mountain Apache Tribe sponsored the investigation reported herein for the purpose of identifying, if possible, the geologic and hydrologic conditions contributing to the groundwater resource limitations experienced at the wellfield. It was also hoped that the study would identify any areas into which the wellfield could be expanded to extend its useful life as an interim water source until the North Fork of the White River can be developed to provide a long-term reliable water supply.

3. FIELDWORK

The fieldwork for this report did not include field mapping of the distribution of geologic units or their extended geologic contacts. Instead, existing geologic maps were used where available and fieldwork consisted of measurement of elevations at the base of the Fort Apache Limestone and the base of the Coconino Sandstone at locations selected to define the geologic structure around the Miner Flat Wellfield area. The GPS measurements were conducted from March 24 through March 28, 2005 by Mike Kaczmarek, Chief Geologist for Morrison-Maierle, Inc., and Cheryl Pailzote, Tribal Hydrologist.

Measurement of elevations on geologic contacts was performed with a WAAS capable, hand-held, Garmin® GPS unit. The locations were subsequently plotted electronically on All Topo Maps electronic versions of the USGS 7.5-minute topographic quadrangles and the GPS elevations were compared to the elevations interpolated from the electronic versions of the topographic maps by the All Topo Maps V7 Pro software. In most cases, the GPS elevations were in good agreement with the elevations interpolated electronically from the topographic maps. If the difference between the GPS elevation and the map elevation was more than a meter or two, the topographic map elevation was used instead of the GPS elevation.

Measurements were made on the base of the Fort Apache Limestone at the cliffs along the east side of the North Fork of the White River, near Whiteriver, Arizona, as a prelude to constructing a regional structural map. Subsequent measurements on the base of the Fort Apache Limestone in Limestone Canyon and Corduroy Creek combined with inspection of outcrops in Middle Cedar Creek and Big Canyon verified the presence of two large faults that make it difficult to use the Fort Apache Limestone as the marker for an analysis of the regional geologic structure containing the wellfield. Accordingly, the effort to make GPS measurements on outcrop contacts was shifted to the geologic contact at the base of the Coconino Sandstone where new measurements were made east of the North Fork of the White River, west of the North Fork of the White River in Post Office Canyon and on the Round Top Road, in Big Canyon (east fork of Cedar Creek), and along Forestdale Creek near the old Forestdale Trading Post.

Additional information included elevations from the core hole logs for the Miner Flat dam geotechnical investigations, elevations from the well logs for the Miner Flat Wellfield, the elevation at the Tribe's deep monitoring well near Lakeside, and Coconino elevations estimated off the 15-minute geologic map of the Show Low quadrangle (McKay, 1972) and Springerville Volcanic Field (Condit, 1991). The review of the above-described data indicated they are adequate to support a reasonable interpretation of the geologic structure influencing the Miner Flat Wellfield.

4. REGIONAL GEOLOGIC STRUCTURE

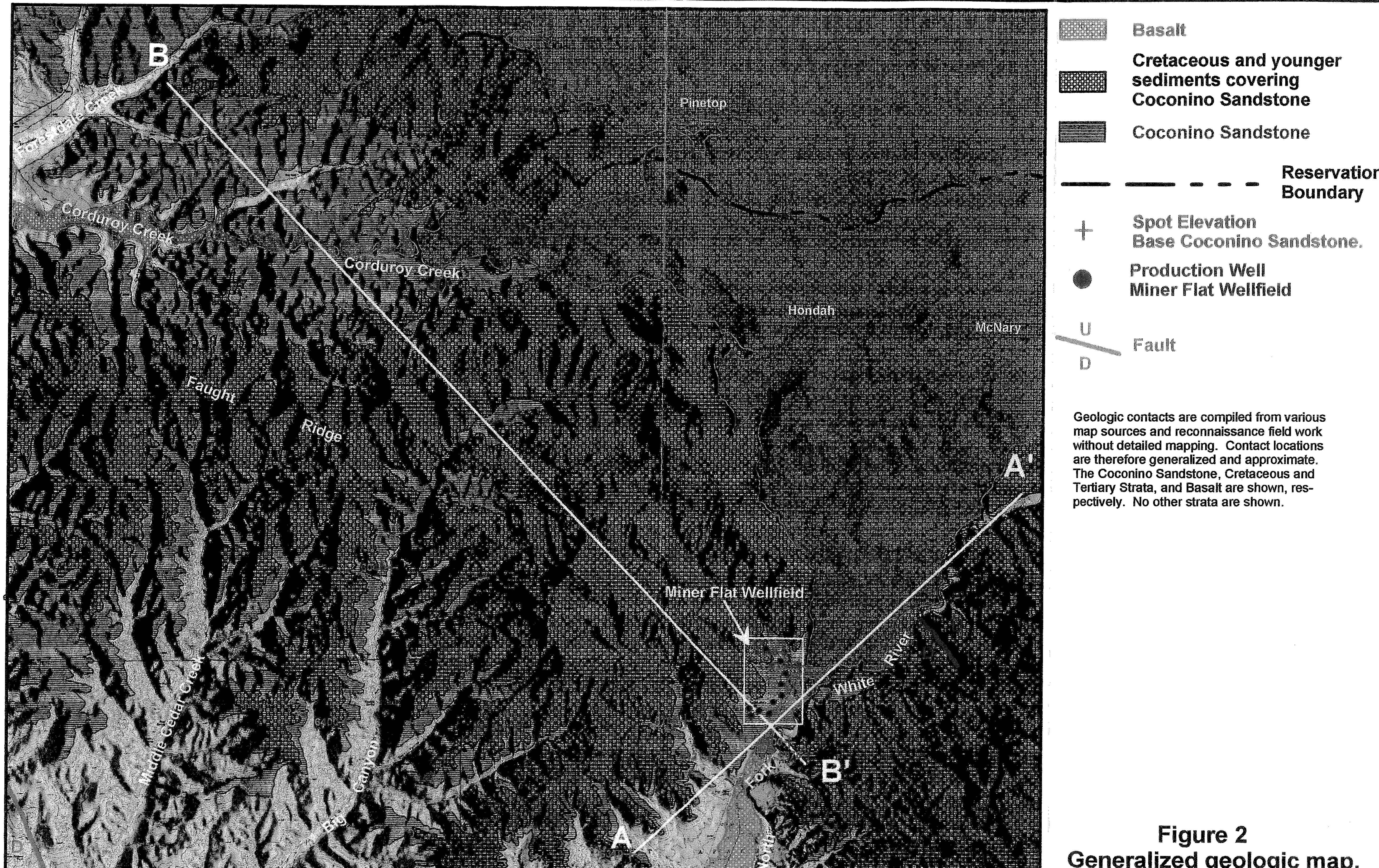
The Coconino Sandstone and other sedimentary strata underlying the Fort Apache Indian Reservation are the southernmost extension of the relatively structurally stable and flat-laying sedimentary strata of the Colorado Plateau. The Colorado Plateau and its underlying strata are truncated by the Mogollon Rim that forms the southern boundary of the plateau and is the northern boundary of the Fort Apache Indian Reservation. The Fort Apache Indian Reservation is therefore located where the terrain sloping southward from the Mogollon Rim into the Basin and Range structural province of Arizona truncates the relatively flat-laying sedimentary strata. Whereas the land surface north of the Mogollon Rim, referred to as the Mogollon Slope, generally coincides with the dip of the sedimentary strata into the Black Mesa structural basin underlying the Little Colorado River basin, most of the land surface of the Fort Apache Indian Reservation cuts across the truncated ends of the sedimentary strata. Only a few small areas of the reservation rest on the Mogollon Slope where southward draining canyons have not dissected the terrain. Figure 2 distinguishes between areas drained by the southward flowing canyons and areas where the strata are not dissected and drained.

North of the Mogollon Rim, the sedimentary strata under the Mogollon Slope dip northeasterly toward the Little Colorado River at about 50 to 55 feet per mile or 0.54 to 0.60 degrees as indicated by structural contour maps presented in Mc Kay (1972) and Mann (1976). Although the truncated ends of these same strata are present under the Fort Apache Indian Reservation, detailed geologic maps of the reservation show local dips of as much as 3 to 5 degrees to the northeast (Finnell, 1966a and 1966b; McKay, 1972). The strata under the reservation locally exhibit steeper dips than those north of the Mogollon slope because the geologic structure under the Fort Apache Indian Reservation has been modified where small folds and faults are imposed over the regional structure. For example, Robinson (1994) reported dips of 3 to 7 degrees northeast at Miner Flat and hypothesized a local fault or fold north of the damsite to explain the anomalous dip.

In general, precipitation, snowmelt, and runoff enter the sedimentary strata as recharge along the Mogollon Rim where the strata are exposed and infiltration becomes sufficiently concentrated to percolate downward to the saturated zones in the strata. Water infiltrating into the strata moves downward through permeable layers until a less permeable layer is encountered. Water then flows sub-horizontally through the permeable strata as groundwater along the surfaces of the relatively impermeable strata, following the dip of the strata northeasterly or northerly toward the Little Colorado River.

Some water entering the strata along the Mogollon Rim flows south, particularly where canyons south of the rim are incised through the sedimentary layers, thus providing drains where water infiltrating down through the strata can drain to the south. The groundwater draining out at the base of the permeable layers and into the southward-flowing canyons forms the springs on the Fort Apache Indian Reservation. Because most of the strata on the reservation are dissected by canyons tributary to the Salt River, most of the strata on the reservation have drained out through the canyons and offer little or no potential for significant development of groundwater. Only the small areas of reservation lands near the northern boundary of the reservation rest over strata containing stored groundwater that has not drained out to the south through the canyons cut into the terrain. Therefore, wells may be developed in those relatively small areas. The areas of Coconino Sandstone to the south do not offer good potential for developing wells because they are drained.

Localized folds and faults imposed over the regional structure are therefore significant to the availability of groundwater on the Fort Apache Indian Reservation in the context of the foregoing relationship between geologic structure and groundwater. This is because the local folds and/or faults in some parts of the reservation modify the



Geologic contacts are compiled from various map sources and reconnaissance field work without detailed mapping. Contact locations are therefore generalized and approximate. The Coconino Sandstone, Cretaceous and Tertiary Strata, and Basalt are shown, respectively. No other strata are shown.

Figure 2
Generalized geologic map.

regional structure enough to create subsurface traps for groundwater or to divert the flow of groundwater into reservation springs, away from the regional dip.

5. MINER FLAT WELLFIELD STRUCTURE

The principal strata yielding water to wells in the Miner Flat Wellfield are the Coconino Sandstone and a red sandstone unit in the uppermost part of the Supai Group strata. The red sandstone is likely a member of the Schnebly Hill Formation defined by Blakey (1990); a subject deserving review but beyond the scope of this report. Therefore, the historic assignment of the red sandstone to the uppermost Supai strata will be used in this report. The distribution of the water-bearing red sandstone assigned to the uppermost Supai Group corresponds closely to that of the Coconino Sandstone. Accordingly, a map showing the distribution of the Coconino is, for all practical purposes, a map of the distribution of the red sandstone unit. In much of the area of investigation, the sandstone strata in the Coconino and Supai may be above the groundwater level or drained out and, therefore, not part of the aquifer system. Thus, a map of the Coconino Sandstone distribution is not necessarily a map of the Coconino aquifer because some of the Coconino and related Supai strata do not contain groundwater. However, the distribution of the Coconino Sandstone is relevant to identifying the geologic structure of the area that includes the Miner Flat Wellfield.

Figure 2 shows the generalized distribution of the Coconino Sandstone. Figure 2 also shows where the Coconino Sandstone is exposed at the land surface as an outcrop and where it is buried under Cretaceous-aged or younger strata. The maps relied on to identify the distribution of the Coconino Sandstone and Cretaceous and younger strata include Wilson et al. (1969), McKay (1972), and Condit (1991) as well as the geologic maps of the Miner Flat damsite prepared during the geotechnical investigations of the site.

The alignments of two geologic cross sections, A-A' and B-B', are shown on Figure 2. Cross section A-A' is drawn through the Miner Flat Wellfield and is intended to generally align with the dip of the geologic strata. Cross section B-B' is drawn from the Forestdale area through the Miner Flat Wellfield area and is intended to generally align with the strike of the geologic strata. The two cross sections deviate slightly from alignment with the regional strike and dip of the sedimentary strata in order to coincide with the locations of measured or mapped elevations on the base of the Coconino Sandstone.

Figure 3 displays cross sections along the strike and dip of the Coconino Sandstone prior to localized folding and faulting. Cross section A-A' on Figure 3 shows the strata dipping to the northeast (the dip shown is greater than the regional dip, based on contemporary structural elevations for the purpose of simplification). Cross section B-B' on Figure 3 shows the strata essentially horizontal along the strike.

Figure 4 shows how local folding and faulting modified the regional structure. On cross section A-A', a significant fault completely offsets the Coconino Sandstone, severing the sandstone north of the fault from that south of the fault. The fault depicted on cross section A-A' is shown on the geologic map prepared by Condit (1991) and is about 4,500 feet upstream from the Lower Log Road bridge over the North Fork of the White River.

The U.S. Geological Survey (USGS) version of the Condit (1991) geologic map shows Cretaceous and Tertiary strata on the south side of the fault displaced downward to the same elevation as the top of Coconino Strata north of the fault. The WMAT #1 monitoring well drilled along the Mogollon Rim near Lakeside, Arizona, penetrated a partial section of the Cretaceous strata 540 feet thick. The Condit (1991) map shows at least 280 feet of Cretaceous strata at the fault. Accordingly, the juxtaposition of Tertiary strata on top of the Cretaceous to the Coconino across the fault implies at least 280 feet of displacement along the fault and potentially as much as 540 feet of displacement. Since the Coconino Sandstone in this area is only about 280 feet thick, the fault must completely displace the sandstone unit, severing the continuity of the sandstone. The fault disappears under basalt flows north and west of the North Fork of the White River. Therefore, the extent of the fault to the northwest, towards Wheat Field Cienega, is unknown.

It should be noted that the USGS version of Condit's map shows the north side of the fault displaced downward with respect to the south side. However, the map also shows the contact between the Cretaceous sediments and the Coconino Sandstone north of the fault at approximately elevation 6360 feet whereas the contact between the Cretaceous sediments and the overlying Tertiary gravels south of the fault is at approximately elevation 6160. This indicates the south side of the fault is displaced downward with respect to the north side, even if a significant amount of the Cretaceous sediments has been removed by pre-Tertiary erosion. Clearly, the direction of relative displacement is mislabeled on the USGS version of the map. This conclusion was verified by Condit (2005), who provided a faxed copy of that portion of his original field map showing the south side of the fault is displaced downward with respect to the north side.

Cross section B-B' on Figure 4 shows modification of the regional structure by a gentle fold at the east end of the cross section. The fold is indicated by the relative elevations on the base of the Coconino measured by GPS near Forestdale, in Big Canyon, in Post Office Canyon, in the North Fork of the White River downstream from the proposed Miner Flat Dam, and estimated from the McKay (1972) map along upper Corduroy Creek, compared to the elevations provided by the wellfield water well logs and the geotechnical corehole logs of the Miner Flat damsite. The fold is a subtle structural feature with an axis generally aligned along the North Fork of the White River. Plotting the known elevations on the base of the Coconino produces the cross-section profile along the strike of the strata shown on Figure 4, cross section B-B'. The upper surface of the Coconino Sandstone is projected at a constant thickness of 280 feet.

Figure 3: Cross sections of pre-fault structure.

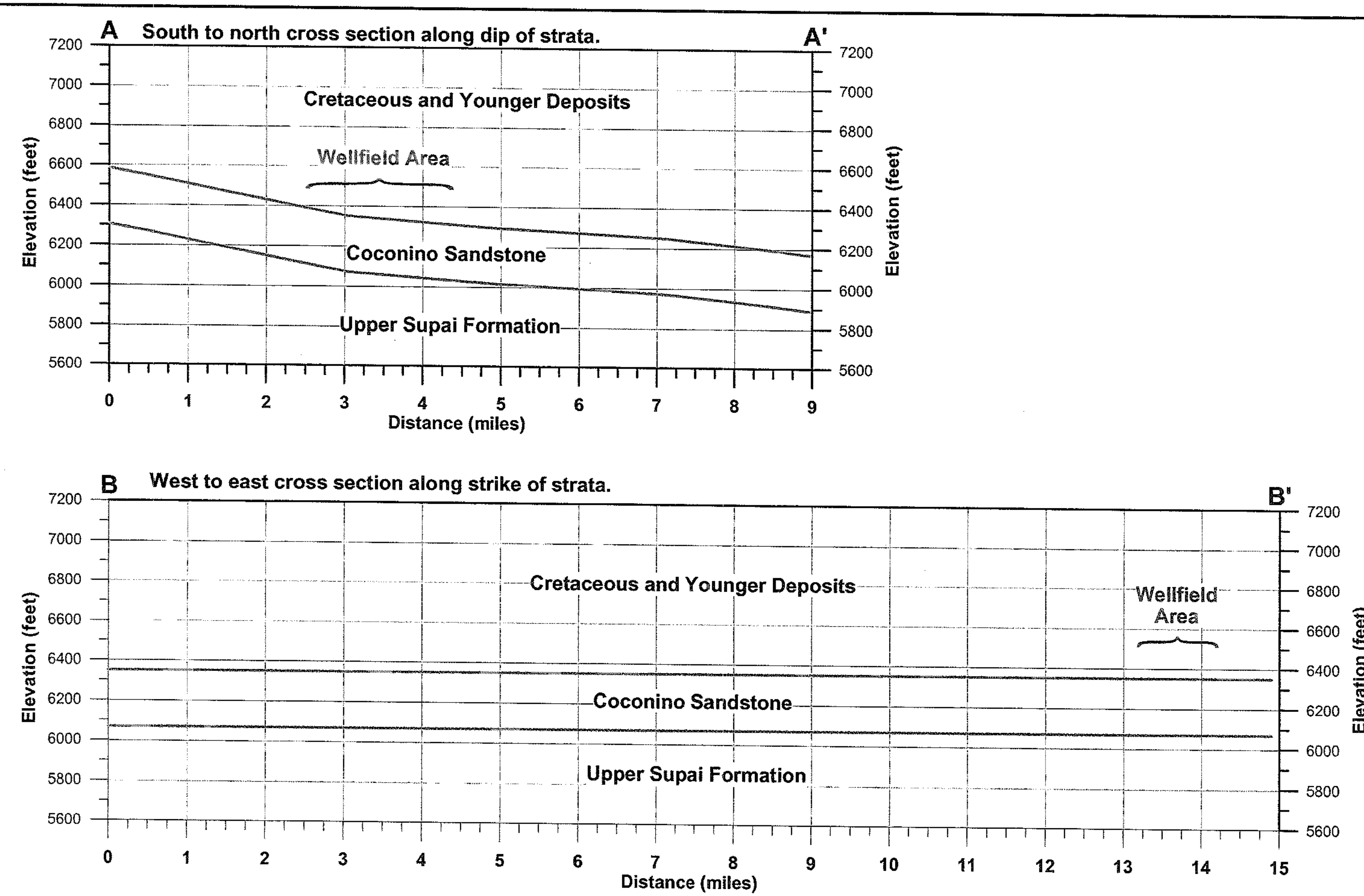


Figure 4: Cross sections of structure modified by local folds and faults.

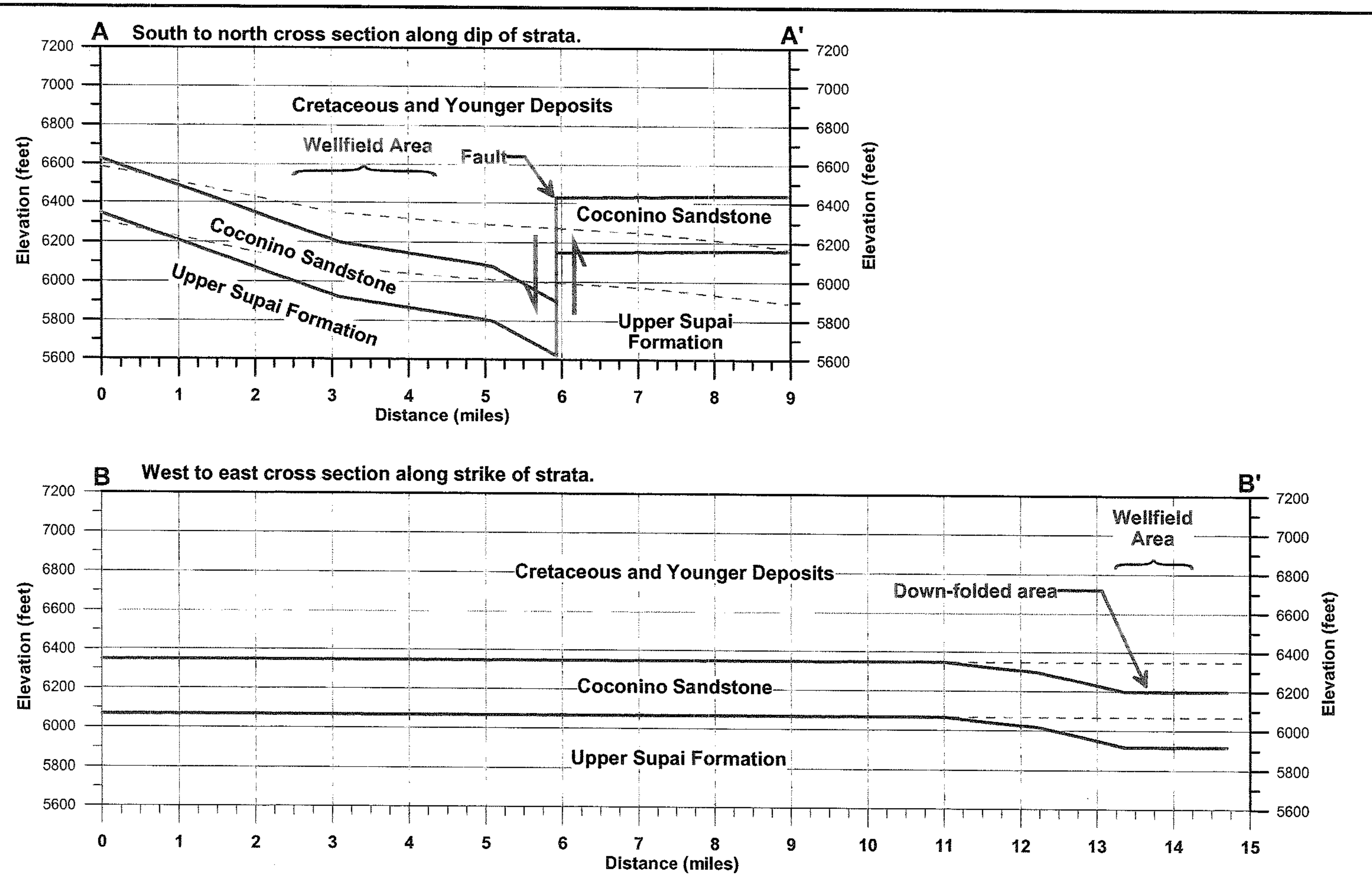


Figure 5 shows cross sections A-A' and B-B' with the modern topography along the lines of section added and cross-hatching added to differentiate the different strata. In addition, the channel of the ancestral North Fork of the White River is shown incised into the Coconino and Supai strata and filled with basalt.

Figure 6 shows the cross sections expanded for greater detail. A basalt flow confining part of the aquifer in the Miner Flat Wellfield area is added to cross section A-A'. Cross section B-B' is expanded to show how the basalt occupying the ancestral river valley provides an essentially impermeable dam across the east side of the Coconino and upper Supai water-bearing units at the wellfield. The modern channel of the North Fork of the White River is shown incised into the basalt.

Figure 7 shows the location of the Miner Flat Wellfield with respect to the geologic cross sections. The saturated Coconino/Supai strata (as of December 2001) are distinguished from the unsaturated part of the strata above the water table. In addition, the pre-pumping static water level is shown and compared to the December 2001 static water level after essentially three years of pumping from the wellfield.

The change between the pre-pumping and December 2001 post-pumping surface is exaggerated slightly on cross section A-A' of Figure 7 in order to demonstrate the change from confined to unconfined aquifer conditions as the groundwater surface dropped below the confining layer of basalt. The change from pre-pumping to post-pumping (December 2001) levels on cross section B-B' of Figure 7 is plotted by actual elevations and is representative of aquifer conditions between January 1998 and December 2001.

The most important concept to be gained from the structural interpretation is that the groundwater at the Miner Flat Wellfield is present because of three factors. One factor is the gentle downward fold of the strata shown on cross section B-B' in Figures 4 through 7 that creates a low spot in the regional structure and causes the base of the Coconino west of the wellfield to rise above the wellfield elevation. The second factor is the deposit of basalt in the ancestral river channel that prevents groundwater from draining out of the Coconino/Supai strata and into the North Fork of the White River. The third factor is that the Coconino/Supai strata increase in elevation to the south, owing to the regional dip, causing the base of the water-bearing strata to rise above the groundwater elevation. This condition is shown on Figure 7, cross section A-A', where the base of the water-bearing layers becomes the southern boundary of the groundwater system between the wellfield and Post Office Canyon.

The role of the fault shown on cross section A-A' (Figures 3 through 7) is less clear. The fault, which is depicted on the Condit (1991) map, is approximately 2.6 miles down dip from the wellfield. The dip of the strata at the wellfield is from 3 to 7 degrees (Robinson, 1994) as compared to less than one degree for the regional structure. Robinson (1994) concluded a fault somewhere north of the wellfield is required to explain the local dip but did not specifically mention the fault shown on the Condit (1991) map.

Figure 5: Geologic cross sections with land surface and basalt flow in North Fork of White River.

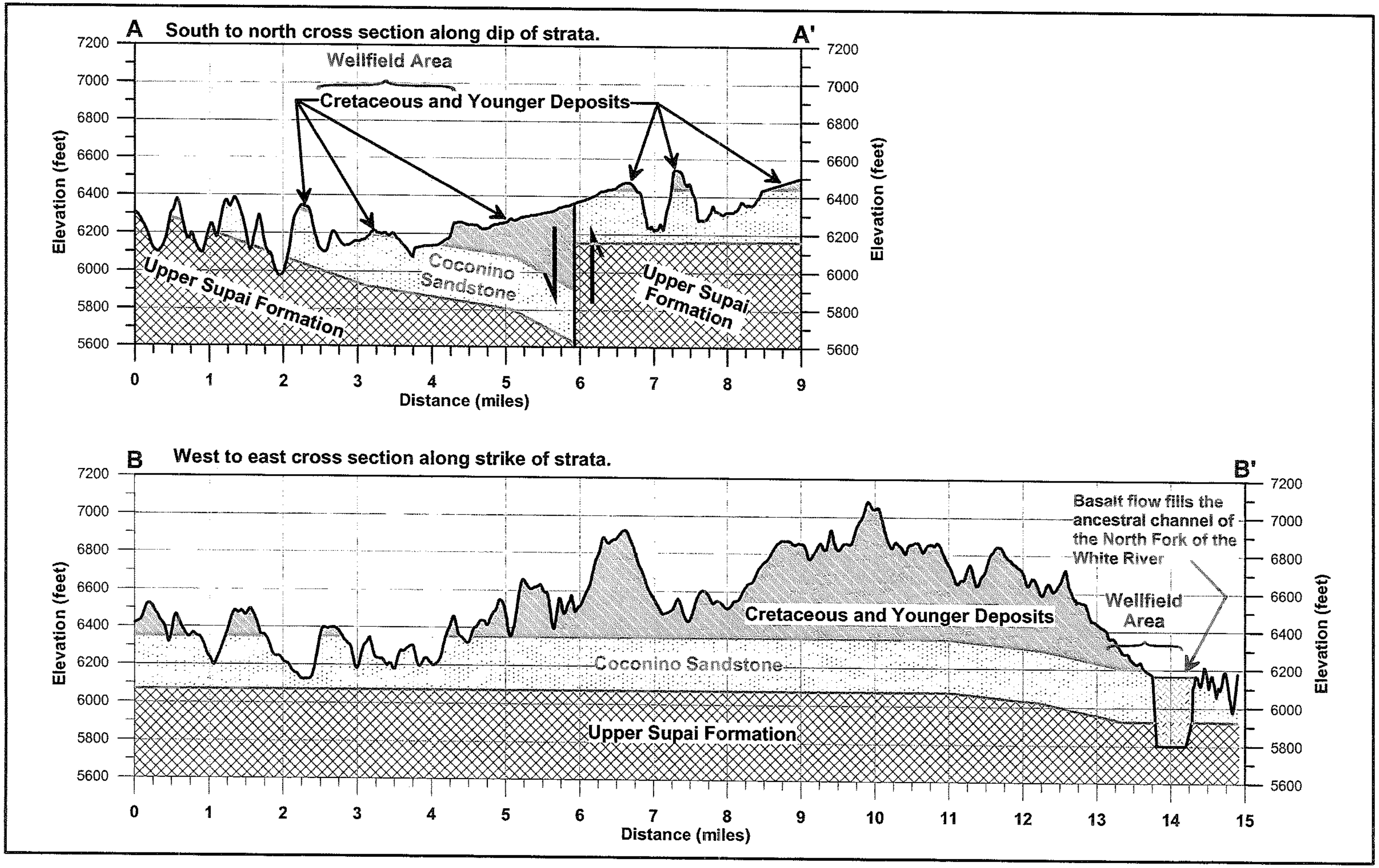


Figure 6: Expanded cross section showing how basalt acts as a dam between the aquifer and the river.

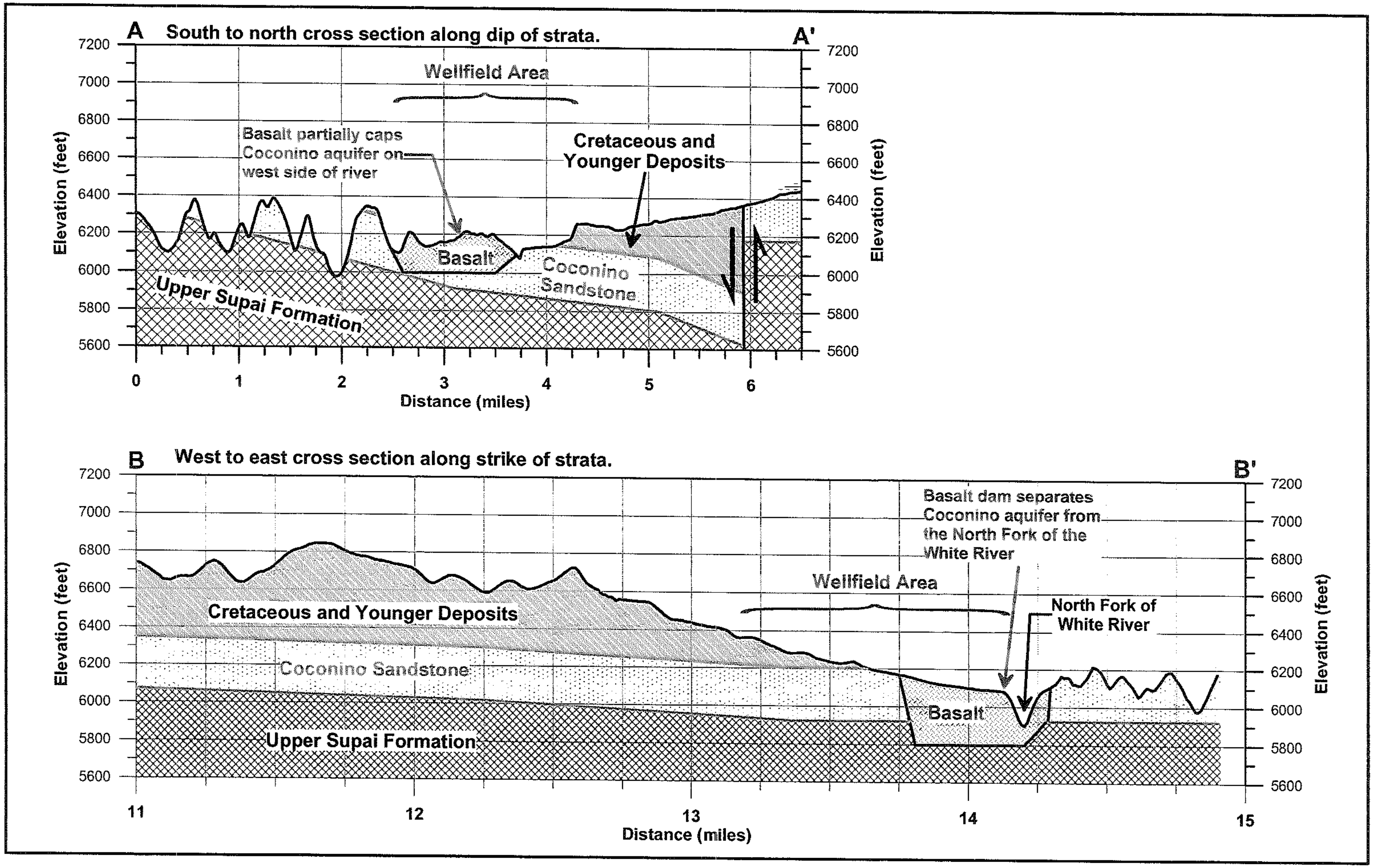
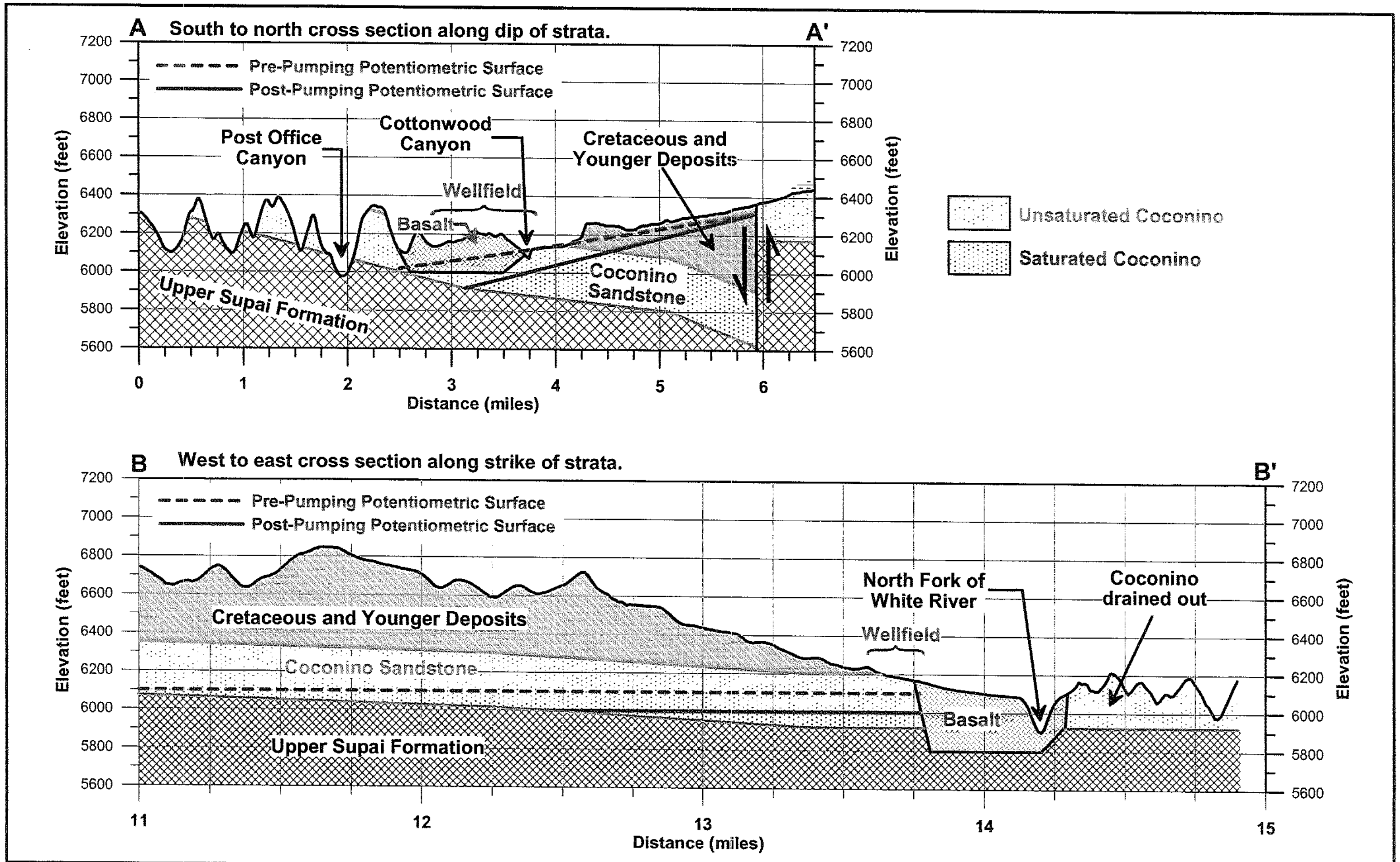


Figure 7: Cross section showing relationship of geologic structure to groundwater distribution.



The Condit (1991) map shows strata dipping northeasterly 30 to 40 degrees about 3,900 to 4,000 feet southwest of the fault, between the fault and the wellfield, in the area of the Lower Log Road bridge. The latter dips indicate a steep downward flexure in the Coconino and related strata at this location. The dips are too steep to be consistent with the strata exposed along the south side of the fault and therefore cannot persist continuously in the subsurface for any significant distance. Accordingly, the strata concealed in the subsurface between the steeply dipping strata and the fault must decrease in dip back to 3 to 7 degrees or less between the Condit (1991) fault and the Lower Log Road bridge. The steep dips are very anomalous and may indicate the effect of drag along a fault. If the steep dips are due to fault drag, the area between the steeply dipping strata and the fault shown by Condit (1991) may be a down-dropped block. The latter interpretation requires a fault just upstream from the steeply dipping strata, a fault concealed by the overburden of basalt and/or Tertiary sediments.

Regardless of the cause of the steep dips in the strata near the Lower Log Road bridge, the structural relationship indicates an area of Coconino Sandstone at an elevation much lower than that of the surrounding geologic structure, with the Coconino at a lower elevation than at the Miner Flat Wellfield. The structurally low area is a potential groundwater reservoir of limited extent that has not yet been explored. The principal uncertainty with regards to this potential reservoir, aside from whether or not it will produce groundwater, is the width of the structurally low area. The observed width of the down-dropped area south of the fault is about 2,800 feet. As previously mentioned, the fault is concealed by basalt on the northwest side of the river and by Tertiary gravel on the southeast side such that the down-dropped area may be wider than the observed 2,800 feet.

If a structural reservoir for groundwater exists along the south side of the Condit (1991) fault, it is severed from groundwater flow from the Mogollon Rim by the fault. Likewise, it is severed from recharge by infiltration directly into the sandstone from surface flow in the North Fork of the White River. The apparent limits of the structural low are about 2,800 feet wide by less than 4,000 feet long, i.e., approximately the size of the area presently developed by the existing wellfield. If the fault extends through the subsurface northwest of the river, toward Wheat Field Cienega, the width of the structural low may be greater than 2,800 feet.

The foregoing considerations indicate it may be possible to develop additional groundwater from the Coconino Sandstone in the structurally low area described above, along the south side of the fault. Available information does not indicate a significant natural groundwater discharge associated with the area. Consequently, groundwater pumped from the area south of the fault will not be diverted from some other discharge area. It will be mined from local groundwater storage with subsequent depletion of the resource and loss of water well capacities, similar to the historic performance of the Miner Flat Wellfield. Other considerations include the fact that the potential groundwater reservoir associated with the fault is on the east side of the river, requiring a minimum of about 2.6 miles of water transmission line plus laterals to individual wells plus a river crossing.

6. HYDROGEOLOGIC CONSIDERATIONS

Figure 2 shows that Corduroy Creek valley upstream from its confluence with Forestdale Canyon to about the mouth of Dry Valley penetrates completely through the Coconino Sandstone and into the top of the Supai strata. Corduroy Creek penetrates through much of the thickness of the Coconino Sandstone from the mouth of Dry Valley to nearly the Amos Ranch. Forestdale Canyon penetrates through the thickness of the Coconino strata throughout nearly its entire length. Groundwater from the Coconino aquifer discharges into Forestdale Canyon at Ruin Springs and Corduroy Creek acts as a drain intercepting any groundwater flowing south from the Mogollon Rim through the Coconino/Supai aquifer past Forestdale Canyon. Corduroy Creek also drains the northeast-dipping strata under Faught Ridge. Likewise, the headwater tributaries of Big Canyon drain the Coconino strata west of the Miner Flat Wellfield. Therefore, any significant flow of groundwater from the Mogollon Rim recharge area, along the northern boundary of the Fort Apache Indian Reservation, toward the area west of the Miner Flat Wellfield is either intercepted by Corduroy Creek where it penetrates through the aquifer strata or drains out through Big Canyon.

The geologic terrain between Hondah and the Miner Flat Wellfield, east of Highway 73, most likely drains to the North Fork of the White River. As shown on Figure 2, the North Fork of the White River is incised into the upper part of the Coconino Sandstone from the confluence with Trout Creek and as far downstream as the fault about 4,500 feet upstream from the Lower Log Road bridge. The Coconino Sandstone is not separated from the river valley by basalt along this reach. Therefore, any groundwater in the sandstone can drain freely into the river. The rate and volume of drainage into the river is essentially negligible, as demonstrated by measurement of stream flow gains and losses in this reach. Accordingly, the natural discharge of groundwater from this part of the Coconino aquifer available for capture to sustain a long-term discharge from a wellfield is negligible. This indicates a wellfield located east of Highway 73 and between the Hon Dah and McNary communities and the North Fork of the White River down to the Lower Log Road bridge will mine groundwater from the aquifer until the resource is depleted and the yield of the wellfield declines, similar to the experience at the Miner Flat Wellfield.

Forestdale, Corduroy, and East Cedar (Big Canyon) Creeks drain the Coconino Sandstone northwest and west of the Miner Flat Well Field. The North Fork of the White River drains the sandstone northeast of the wellfield. The only potential flow path from the Mogollon Rim to the wellfield is through a narrow neck of Coconino Sandstone preserved under Cretaceous strata and basalt flows, straight south of Hon Dah and Pinetop, in the area west of Highway 73 and east of the head of Big Canyon and its eastern tributaries such as Deer Spring Creek. The latter canyons drain groundwater out of the western part of the narrow zone of preserved sandstone, further reducing the width of the potential flow path from the rim. The only reason the upper reaches of Post

Office Canyon and Cottonwood Canyon do not drain the sandstone is the gentle fold depicted on cross section B-B' of Figures 3 through 7.

In addition to the likelihood that drainage of water from the Coconino Sandstone into upper Corduroy Creek and Big Canyon diverts groundwater from away from the Miner Flat Wellfield, other factors suggest any potential groundwater flow through the strip of Coconino Sandstone preserved due north of the wellfield may be very small or insignificant. For example, the Coconino Sandstone up-gradient from the river, east of Highway 73, does not appear to provide any detectable discharge to the North Fork of the White River. This is similar to the discharge from the Coconino into Forestdale Canyon and upper Corduroy Creek which ranges from undetectable in dry periods to a small flow from Ruin Spring in wet periods, i.e., essentially negligible flow.

Since the block of sandstone generally south of the area between Hon Dah and McNary is not materially different from that due north of the wellfield, there is no basis to expect groundwater flow to the wellfield is any greater than that to the North Fork of the White Rver or to Forestdale Creek and upper Corduroy Creek where discharge from the Coconino Sandstone is negligible. A similar argument can be made based on the relatively small amount of discharge from the aquifer into the head of Big Canyon and its tributaries. This argument is simply that the flow of groundwater from the Mogollon Rim through the Coconino Sandstone is too small to sustain a wellfield. This conclusion is supported by the fact the largest springs on the reservation discharge from the Fort Apache Limestone where they receive recharge by vertical leakage from the Coconino and other overlying aquifers. The discharge from the Fort Apache Limestone indicates the flow of groundwater from the rim is mostly vertical interformational flow to the Fort Apache Limestone followed by horizontal flow to the reservation springs that support surface water baseflow.

Another factor potentially affecting groundwater flow through the narrow zone of sandstone north of the wellfield is the possibility of a fault or faults that displace the sandstone sufficiently to sever its continuity between the Mogollon Rim and the wellfield. This possibility is indicated by the existing fault upstream from the Lower Log Road bridge, as previously described, and its alignment with similar faults at the head of Dry Valley and areas west of Dry Valley. This fault or a similar fault may prevent groundwater from flowing southward to the wellfield area and explain the relative absence of observable natural discharge out of the Coconino aquifer around the wellfield area, i.e., the absence of natural discharge that can be diverted to sustain a wellfield.

7. CONCLUSION

All of the foregoing considerations indicate the potential to develop a reliable and sustainable supply of water from the Coconino aquifer to meet the needs of the growing greater Whiteriver area is unfavorable. The body of groundwater developed from the lowermost Coconino Sandstone and red sandstone in the uppermost part of the Supai Group by the Miner Flat Wellfield is physically bounded to the west and south by the geologic structure where the water-bearing sandstone rises above the groundwater elevation. It is bounded to the east by a basalt flow occupying an ancestral river channel cut through the water-bearing strata. The decline in groundwater elevations and well yields during 7.5 years of use of the wellfield indicates that the principal component of the groundwater produced by the wellfield has been groundwater mined from storage in the aquifer with little or no capture of groundwater flow away from natural discharge areas. Hence, the decline in groundwater levels and well yields is indicative of depletion of the water resource in the aquifer.

The foregoing conclusions must not be misconstrued to indicate that additional groundwater cannot be developed from some portions of the Coconino aquifer within the Fort Apache Indian Reservation boundaries along the Mogollon Rim. They simply mean that the amount of groundwater that can be developed in that geologic setting is far less than the present and future requirements for continued population growth in the greater Whiteriver area.

The Miner Flat Wellfield is located in a relatively unique part of the Coconino aquifer where groundwater storage was atypical and enhanced by a downward fold in the strata and a basalt dam blocking drainage out of the strata in the fold. The volume of groundwater stored in this unique setting has supported mining of groundwater for much longer than would be possible in a typical Coconino aquifer setting within the Fort Apache Indian Reservation. In a typical Coconino aquifer setting in the reservation, groundwater entering the aquifer strata in the recharge area along the Mogollon Rim at the northern boundary to the reservation flows southward to springs along the base of the Coconino Sandstone with no restriction other than the hydraulic properties of the aquifer. As shown on Figure 2, southward flowing canyons have dissected the truncated end of the Coconino strata, draining much of the strata located south of the rim, and leaving unaffected only a narrow strip of sandstone within the reservation boundaries. Wells drilled into the Coconino Sandstone can capture a part of the flow to the natural springs and redirect it to flow out of the wells. However, the unrestricted drainage allows most of the groundwater to drain out of the formation, rather than remain as stored groundwater, particularly between the canyons dissecting the strata. Accordingly, the amount of groundwater remaining stored in the aquifer at any one location is limited and the saturated thickness in the water-bearing strata is limited.

Intermittent inspection of the springs and seeps in the foregoing areas where the Coconino Sandstone discharges groundwater to the canyons and springs on the Fort Apache Indian Reservation has shown that the discharge of groundwater available for capture to support long-term sustainable production for a wellfield in the Coconino Sandstone has been negligible in the period of inspection from 1996 to the present. For example, Photo 1 shows Ruin Springs in December 1996 when the discharge was limited to a flow too small to measure and which disappeared into the ground downstream from the spring. In September 2005, the same area shown in Photo 1 was dry. Photo 2 shows discharge from the base of the Coconino Sandstone in Hop Canyon. The discharge rate into the tanks was less than 1 gpm with additional unmeasured seepage creating the wet area in the foreground of the photograph.

Photo 3 shows some of the Cottonwood trees and other phreatophytes growing at the old Forestdale Trading Post in Skiddy Canyon, tributary to Forestdale Creek. This vegetation indicates a small discharge of groundwater out of the Coconino that would be undetectable if not for the few deciduous trees growing just downstream from the end of the Coconino outcrop.

In September 2005, upper Forestdale and Corduroy Creeks did not exhibit any discharge from the Coconino Sandstone, including their tributaries such as Dry Valley. Photo 4 shows water pooled in the channel of Middle Cedar Creek, immediately downstream from 24-Dart Spring where the creek crosses the base of the Coconino Sandstone. Although the pooled water appeared to result from the discharge of groundwater out of the Coconino Sandstone under Faught Ridge, there was no visible flow of water through the pools shown in the photograph. In September 2005, inspection of Post Office Canyon where it cuts through the Coconino Sandstone did not find any discharge of groundwater from the sandstone.

The foregoing observations indicate the Coconino Sandstone does discharge small amounts of groundwater to the surface drainages flowing south on the Fort Apache Indian Reservation. However, the discharges from different locations along the southern boundary of the Coconino Sandstone, considered individually or collectively, are considerably less than the water requirements for the greater Whiteriver area and, in fact, do not appear to total as much as the discharge rate from one well in the existing Miner Flat Wellfield. Considering the fact that sustainable yield from the Coconino Sandstone up-gradient from these discharge areas is limited to that part of the natural discharge flows that can be captured by wells, it is clear that abstraction of groundwater from the Coconino Sandstone through wells pumped at rates equal to the demands for the Whiteriver area would mine considerable groundwater from storage in this aquifer. The mining of groundwater from the aquifer at the rates necessary to meet the present and future demands for the greater Whiteriver area will quickly deplete the groundwater storage in the aquifer and result in loss of yield from the wells.

In other words, the limited amount of groundwater flow naturally discharging from the Coconino aquifer indicates the flow through the aquifer is not adequate to sustain the demands for relatively high-capacity groundwater withdrawals required for the communities in the Whiteriver service area. However, the Coconino groundwater up gradient from the natural discharge areas can be used to support small water demands for local housing projects, if used judiciously, and if annual pumping volumes are limited to less than the estimated long-term annual natural discharge out of the aquifer.

Photo 1: Ruin springs in December 1996.

Photo 2: Little Spring in Hop Canyon.

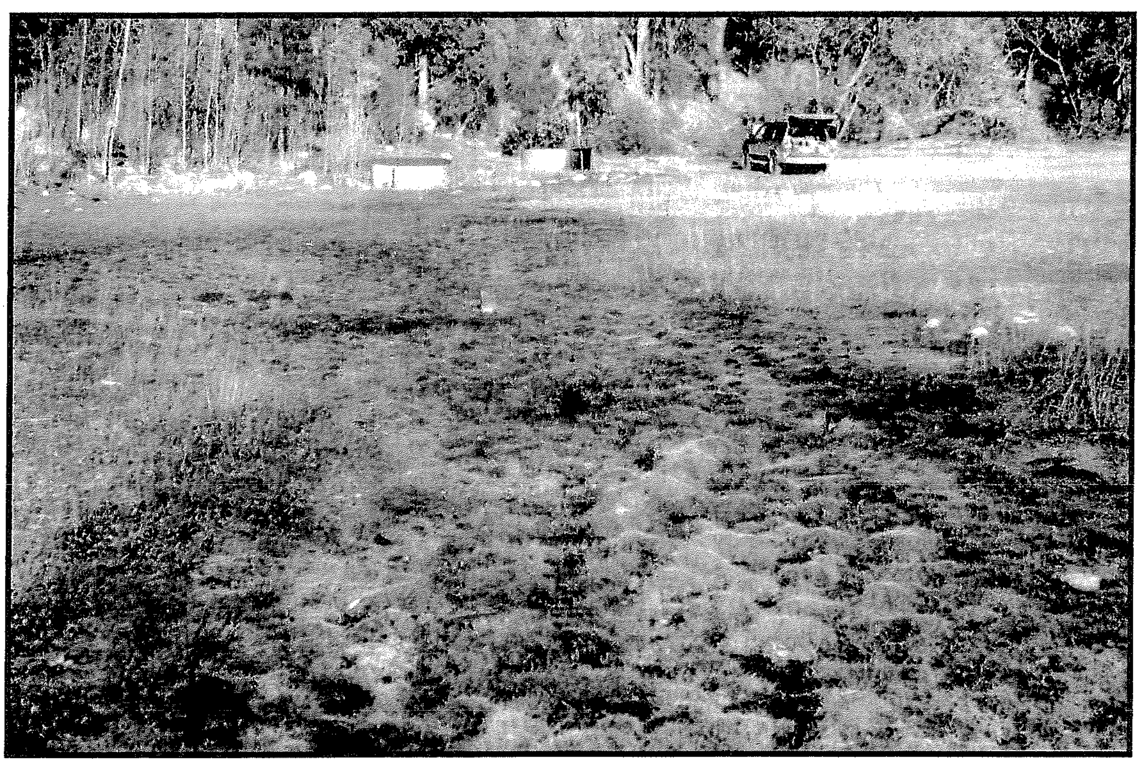
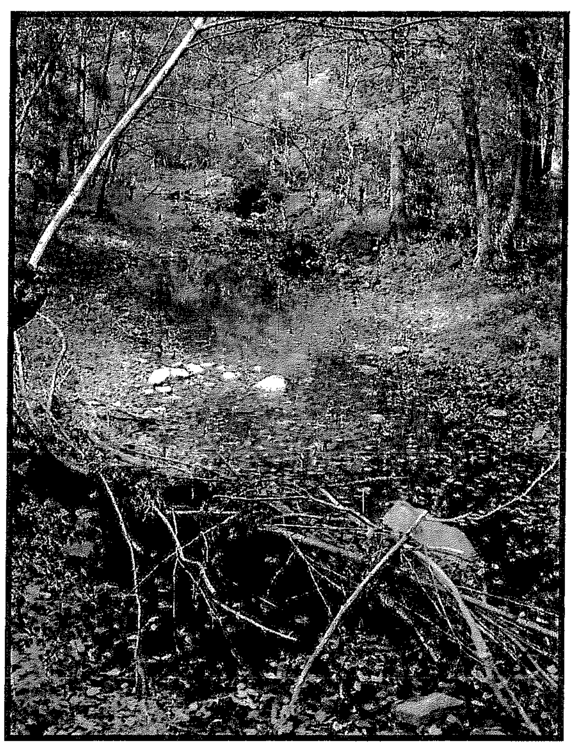


Photo 3: Cottonwoods in Skiddy Canyon tributary to Forestdale Creek.

Photo 4: Pooled water below 24-Dart Spring in East Cedar Creek in Big Canyon.



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APPENDIX A

HISTORICAL DEVELOPMENT ALL THE MINER FLAT WELL FIELD

In order to understand the significance of geologic structure to the hydrology of the wellfield, it is necessary to understand the history behind development of the wellfield. The Miner Flat Wellfield was conceived in the early 1990s as an attempt to solve critical municipal water supply shortages facing the Tribe at that time in the Whiteriver area. The supply of municipal water to the Whiteriver area in the 1980s and early 1990s was provided by a number of individual wells and springs scattered throughout the area to serve the historical population concentrations. By the 1980s, the local wells were becoming integrated into a water supply system for the larger Whiteriver area; however, many of the wells were potentially subject to influence by surface water, thus requiring surface water treatment. Other wells were failing. Consequently, the White Mountain Apache Tribe sponsored a hydrogeologic investigation of the north-central part of the Fort Apache Indian Reservation to evaluate the groundwater resources potentially available to the greater Whiteriver area. The hydrogeologic study supplemented other on-going long-term plans to develop surface water, including construction of a dam at Miner Flat to provide storage needed to supply future water demands.

The groundwater resources evaluation was prepared by the team of Mineral Systems, Inc. and Golder Associates, Inc., dated July 1993 and titled, "**Hydrogeologic Investigation North-Central Part Fort Apache Indian Reservation, Navajo, Apache, and Gila Counties, Arizona**." (Mineral Systems and Golder Associates, 1993). The study produced a recommendation for a test drilling program north of Whiteriver, stating:

> "***The recommended drilling program consists of completing test well(s) in the northern portion of the site. The objective of the northern test well(s) is to evaluate the Kaibab/Coconino aquifers relative to their potential for adequate water resource development for Whiteriver. Test wells in the southern portion of the site are not recommended at this time due to the potential for surface water/groundwater interaction.***" (Golder Associates, 1993; p.19)

In response to the Golder Associates (1993) recommendation, the Indian Health Service worked with the White Mountain Apache Tribe to explore for groundwater, completing 15 exploratory boreholes, including one test well, in the period from the summer of 1993 through the summer of 1994. One of the exploratory boreholes was located at the Roberts Ranch, two were located along Cottonwood Canyon, and the others were located in the present Miner Flat Wellfield area. In December 1993, Golder Associates conducted stepped rate tests at the well currently referred to as production Well No. 3, using rates ranging from 200 to 300 gpm. Again in May 1994, the well was subjected to a constant rate test at 275 gpm for approximately two weeks; however, data collected by the Indian Health Service during that latter test were very sparse and do not adequately define the aquifer response. The pumping test results were given in a report dated August 1994 and titled, "**Pumping Test Analysis and Well Field**

<u>Design, Miner Flat Area, Fort Apache Indian Reservation.</u>" (Golder Associates, 1994)

Based on the pumping test results, Golder Associates concluded that the aquifer at the Miner Flat could support a 4,000-gpm wellfield, stating:

> "***If the White Mountain Apaches wish to minimize the distance between pumping wells, a pumping rate of about 333 gpm from each well appears reasonable. At this pumping rate, a total of 12 pumping wells will be needed to meet the peak pumping rate of 4,000 gpm. The estimated distance between pumping wells needed to prevent the wells from going dry is about 250 feet.***
>
> ***If the White Mountain Apaches wish to minimize the number of pumping wells, a pumping rate of 500 gpm appears appropriate. Under this scenario, 8 pumping wells would be needed at a well spacing of about 500 feet. Given the maximum yield of PW-1*** [now used as Well No. 3] ***during the pumping tests, a pumping rate of 1,000 gpm from each well, although theoretically possible, does not appear to be practical.***" (Golder Associates, 1994; pp.20-21)

and,

> "***The transmissivity and storativity values can be used to determine the approximate spacing of pumping wells at various pumping rates to provide a total yield of 4,000 gpm from a well field. Twelve wells pumping at 333 gpm each for 10 to 12 hours per day would need to be spaced about 150 to 500 feet apart. If each well were pumped at 500 gpm for 10 to 12 hours per day, eight wells spaced about 350 to 1,000 feet apart would be needed. Data collected during the constant-discharge test confirm these recommended well spacings.***" (Golder Associates, 1994; p.23)

One of the fundamental concepts of groundwater hydrology is that the long-term sustainable yield of a well or wellfield is limited to the amount of groundwater flow that the well or wellfield can capture from the natural recharge and discharge areas of the aquifer. A review of the Miner Flat Wellfield site quickly established the conclusion that the wellfield would not capture rejected recharge in a recharge area, i.e., induce an increase in recharge due to pumping. The latter conclusion was based on the fact the aquifer at the wellfield is separated from surface water such as the North Fork of the White River by a natural barrier or "dam" of basalt and the water table is separated from other surface waters in the area by a thick unsaturated zone above the aquifer strata. Moreover, a source of rejected recharge water, available for induced capture, is not available in the recharge area up-gradient from the wellfield.

Likewise, a review of the various springs and surface water streams in the area failed to detect any significant discharge of groundwater out of the Coconino and Supai strata around the margins of the wellfield or down gradient from the wellfield. These observations indicated that a major discharge area for the portion of the Coconino aquifer system contributing to the Miner Flat Wellfield did not (and does not) exist and, therefore, there was no opportunity to capture a substantial amount of groundwater flow away from a natural discharge area and redirect it to discharge through the proposed wellfield. In view of these facts, it was evident early in the wellfield development that the principal source of the groundwater to be pumped from the proposed wellfield would be water mined from groundwater stored in the aquifer. Consequently, the wellfield would have a finite life and would exhibit declining yield as the groundwater levels in the aquifer declined in response to depletion of groundwater from storage in the aquifer.

Following release of the Golder Associates' 1994 pumping test report, Morrison-Maierle, Inc. was asked by the Tribe to review the report and comment on the efforts to develop a municipal water supply from the Coconino/Supai strata in the Miner Flat Wellfield area. At the time, the Golder Associates and Mineral Systems work characterized the aquifer as upper Supai strata and did not refer to the Coconino aquifer. In a critical review of the Golder Associates work and the plans to develop a wellfield, Morrison-Maierle, Inc. Chief Geologist, Mike Kaczmarek, transmitted the following comments in an October 24, 1994 letter:

> "***Pursuant to our telephone conversations today, the following is a summary of thoughts in regards to the IHS development of a well field to provide municipal water supplies to the town of Whiteriver, based on a review of the August 1994 report.***
>
> ***Firstly, the report is limited to the subject of aquifer hydraulics and does not address the issue of groundwater resources availability and reliability with respect to the demand to be levied by the well field. As stated on page 24 of the report, a principal assumption of the analysis is that the aquifer is infinite. This is another way of saying the groundwater will always recover to the pre-pumping level when pumping stops. In fact, analysis of the aquifer hydraulics does not indicate what the response of the aquifer will be to long-term abstractions*** [of groundwater]***.***
>
> ***A reliable assessment of the long-term availability of groundwater will require extensive investigations of the storativity, recharge, and through flow*** [in other words, natural discharge that can be captured] ***of the aquifer including compilation of hydrographs of at least 10-years duration. Obviously, it is not desirable to wait for 10 years to develop a badly needed resource. However, under the prevailing circumstances, the Tribe may be abandoning a resource of known reliability (the North Fork of the White River and the Miner Flat Dam)***

for a resource of questionable longevity (the Supai groundwater system).

It is my understanding that there are presently no long-term records of groundwater level fluctuations in the Supai or any information regarding the full extent and distribution of the aquifer system or its recharge areas. Having had more than 20 years experience with arid zone hydrology, I can point to a number of examples where complete and total reliance on a groundwater resource began with pumping tests that showed it was relatively easy to abstract large amounts of water from the groundwater system but subsequently the resource failed because the abstraction rates exceeded the long-term recharge and through-flow. Thus, individual aquifer tests are simply hydraulic tests and do not ultimately reveal the extent to which the abstracted groundwater is mined from groundwater storage with a resultant long-term depletion of the resource.

It may very well be that investigations conducted properly over a period of years may indicate a groundwater resource adequate for the Tribe's long-term requirements. However, until such investigations have been completed to a level of detail adequate to resolve the availability and reliability of the groundwater resource for a defined level of development, the well field currently proposed should prudently be regarded as a temporary and interim solution to a long-term problem." (Letter from Mike Kaczmarek to William Veeder dated October 24, 1994)

Following the October 24, 1994 comments, which also suggested that Golder Associates' aquifer test interpretation methods were technically flawed, the IHS conducted a 70-day test of the well tested by Golder Associates, pumping at an initial rate of about 500 gpm that declined to about 400 gpm over the 70-day period. Again, data collection was sparse. The limited data set appeared to indicate radial flow to the pumped well and a relatively high-yield confined aquifer, but were too sparse to support detailed analysis. The 70-day test was simply a repetition of the original error of substituting a local pumping test for hydrologic evaluation of aquifer sustainability. Dr. Charles Robinson, principle of Mineral Systems, Inc. wrote to the Tribe following the 70-day test with the following conclusion:

"***The analyses of the data from the 70-day pumping test does not significantly change the conclusion presented by Golder Associates, Inc. in their report, "Pumping Test Analysis and Well Field Design, Miner Flat Area, Fort Apache Indian Reservation," of May*** [August] ***1994. The transmissivity and storativity are all within the same order of magnitude. Based on the geology and the transmissivity and storativity, assuming that the total yield desired from the well field is 4,000 gallons per minute and that the individual well yields would be***

400 gpm for a 10 to 12 hour pumping period, the number of wells needed would be eight or ten, spaced at least 500 feet apart. These wells should be drilled to the west or northwest of the Miner Flat Well [Well No. 3 or PW-1 of the 1994 Golder Associates report]." (Letter from Charles S. Robinson, Mineral Systems, to John Bereman, Tribal Engineer, dated February 7, 1995).

Construction of the Miner Flat Wellfield proceeded based on the foregoing information and in response to a pressing need to provide an alternative to the failing historic sources of water supply for the greater Whiteriver area. Although the questions about the sustainability of the resource were recognized, they were of almost academic concern in the face of the immediate need for additional water for the Whiteriver area. The various pumping tests showing very favorable yields from the aquifer at the proposed wellfield area encouraged development of groundwater as a timely solution to the water supply problem. Moreover, aquifer systems capable of relatively large, short-term yields during pumping tests often offer relatively good long-term reliability, simply because aquifers capable of supporting high-capacity wells may also support considerable groundwater flow to natural discharge areas. Accordingly, the initially high yields obtained at the Miner Flat Wellfield site encouraged an optimistic outlook for sustainability, although the issue of sustainability had not truly been addressed by the tests and investigations conducted up to January 1998.

In view of the subsequent decline of the wellfield yield over a relatively few years of production, it is now easy to criticize the decision to develop the wellfield, particularly to the extent it detracted from the effort to develop a reliable long-term source of surface water from the North Fork of the White River. However, it must be recognized that the wellfield fulfilled a critical and immediate need at the time, however short the life of the solution. As the population and demand for water supply to the greater Whiteriver area continues to grow, the problem facing the Tribe remains the same as it did in the 1990s. To that extent, groundwater may continue to provide a short-term and interim solution to the water supply shortage, while a long-term solution to develop water from the North Fork of the White River is implemented. However, the unsustainable nature of the groundwater resource must be recognized in any plans to develop additional groundwater production as a short-term solution to current and future demands for water. Assuming the existing wellfield can be expanded again to satisfy existing demands until a long-term solution is implemented, such expansion will not provide a permanent or sustainable supply of groundwater. Future expansion of the wellfield will support short-term growth that will result in an increased future population in the greater Whiteriver area suffering water shortages when the groundwater resource is depleted, as clearly will happen.

Moreover, expansion of the existing wellfield to support the population centered around the Whiteriver area may have the effect of depleting groundwater resources that can be used more wisely for smaller, localized population clusters that are not likely to be serviced by the water supply system for the greater Whiteriver area. Localized population clusters can be supplied water by relatively low-capacity, local wells, if the resource in the Coconino aquifer is not depleted by heavy use.

It is very important to the future of the White Mountain Apache Tribe that the groundwater resource is properly managed. The small long-term natural discharge of groundwater from the Coconino aquifer system through the historic springs on the Fort Apache Indian Reservation indicates the limited volume of groundwater production that can be sustained from this aquifer. Wells drilled into the aquifer in the parts of the aquifer not drained out by canyons cutting through the aquifer will capture groundwater that otherwise would drain as discharge to the natural springs. Use of the wells will therefore divert the natural discharge from springs to discharge through the pumped wells. If the volume pumped from the wells is maintained equal to or less than the natural discharge from the aquifer, the yields from the wells will be sustainable. If the volume pumped from the wells exceeds the natural discharge through the aquifer, groundwater will be mined in the amount exceeding the natural discharge, and groundwater storage in the aquifer will be depleted. The rate of depletion will be commensurate with the amount which annual pumping exceeds the long-term, historic natural discharge out of the aquifer through springs and natural discharge and the storage remaining in the aquifer.

It is therefore important that the Coconino aquifer within the reservation boundaries be managed for long-term sustainability. Housing developments and other potential demands for groundwater along the Mogollon Rim should be controlled to limit growth to a population that the groundwater supply can support on a sustainable basis. Short-term, high capacity withdrawals of groundwater from the system for municipal use or irrigation should be avoided as they will quickly mine the local groundwater storage in excess of the long-term sustainable yield of the aquifer system.

Several issues must be considered in any attempt to manage the Coconino aquifer for long-term sustainable groundwater development. One issue is the impact that long-term diversion of groundwater through wells will have on natural discharge through springs. Although development and groundwater use may be regulated to a sustainable rate of use, that use will divert water from some of the natural springs and to the wells over a long period of sustainable well use. Another major issue in developing the groundwater resource for use on a long-term sustainable basis is the lack of long-term measurements of the natural discharge from the aquifer, particularly from the Coconino Sandstone at specific areas, as needed to quantify the amount of groundwater use that can be sustained.

Another issue is the dynamic nature of the direction of groundwater flow along the Mogollon Rim. The simultaneous flow of groundwater north toward the Little Colorado River and south into the Fort Apache Indian Reservation dictates the presence of a groundwater divide along the Mogollon Rim. Mining of groundwater on one side or the other of the groundwater divide will ultimately shift the position of the divide and divert groundwater flow towards the center of excessive pumping and drawdown of water levels. This in turn will reduce the natural discharge and sustainable resource on the opposite side of the groundwater divide. Accordingly, the Tribe must be concerned about the development of groundwater from the Coconino aquifer along the northern boundary of the Fort Apache Indian Reservation where such use may adversely affect the long-term sustainability of public water supply wells drilled by the Tribe.

Therefore, the lessons provided by the historic development of groundwater on the reservation, particularly the reasons for the decline of the relatively high-yield wells at the Miner Flat Wellfield, are important. Although groundwater provided an immediate, albeit unsustainable, solution to the on-going water supply shortage in the Whiteriver area; aquifer conditions are not favorable to development of a sustainable high capacity source of groundwater for the entire reservation.