CHAPTER 18

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CHAPTER 18

PROBABLE HYDROLOGIC CONSEQUENCES

Introduction

This chapter contains a discussion of the probable hydrologic consequences of the life-of-mine mining plan upon the quality and quantity of surface and ground water for the proposed permit and adjacent areas. The significance of each impact or potential impact is determined. The determination of significance has been made considering the impact of any probable hydrologic consequence on: (1) the quality of the human environment; (2) any critical habitats or important plant species; or (3) any threatened and endangered wildlife species within the proposed life-of-mine permit and adjacent areas.

Ground Water

Interruption of Ground-Water Flow and Drawdown. A comparison of five year average Wepo water level contours and isopach maps which show pit bottom contour elevations for all areas to be mined, along with review of historic and recent records, indicates that portions of the J-1/N-6, N-2, N-7, N-10, N-11, J-16, J-19/20 and J-21 pits have already or will intercept the upper part of the Wepo aquifer for some period during the life of the mining areas. Review of Wepo water level contours developed from recent data (1995-2000) and actual field observations during mining indicates that pits in the J-7, J-23, and N-14 mining areas will not intercept the Wepo aquifer. Flow in the portions of the Wepo aquifer truncated by overburden and coal removal will be intercepted since the ground-water gradient will rapidly orient itself in the direction of the sinks (pits).

Previously developed estimates of Wepo ground-water inflow to the above identified pits are presented in Tables 1 through 7, respectively. These estimates were prepared assuming that the total inflow would be derived from two principal sources: (1) the interception of pre-mining flow rates under a natural hydraulic gradient; and (2) the drainage of ground water from storage in the aquifers. It is assumed that the major portion of the Wepo ground-water inflow would be derived from lateral flow along bedding planes and fractures. Upward leakage from underlying aquifers was assumed to be negligible.

Two different techniques have been used to estimate the rates of groundwater inflow into the pits, depending on the technology available at the time the estimates were developed. Approach A was used for pits J-1/N-6, N-10, N-11, N-14, and J-16. This approach,

TABLE 1

Pit Inflows by Year for N-10

Pit Year	Total Length of Pit (Ft)	Constant Length in Water (L_W) (Ft)	Days in Water (t) (Day)	Constant Pit Adv./Day (Ft/Day)	Weighted T_F-Transmissivity (Gal/Day/Ft)	Weighted $T_{L,R}$ (Ft^2/Day)	I-Gradient (Ft/Ft)	Weighted Q_F (Gal/Yr)	Weighted Q_L (Gal/Yr)	Weighted Q_R (Gal/Yr)	Q_T (Gal/Yr)
2002*	8913	-	-	24.4	-	-	.018	-	-	-	-
2003*	8913	-	-	24.4	-	-	.018	-	-	-	-
2004*	8913	-	-	24.4	-	-	.018	-	-	-	-
2005	8913	1081	44	24.4	16.1	2.2	.018	20,833.0	12,541.0	832.0	34,206.0
2006	9566	2810	107	26.2	14.43	1.93	.018	123,834.2	34,176.8	271.0	158,282.0
2007	9566	2810	107	26.2	14.43	1.93	.018	243,574.7	67,223.8	533.1	311,331.6
2008	9566	2810	107	26.2	14.43	1.93	.018	331,589.0	91,514.8	725.8	423,829.6
2009	9566	2810	107	26.2	14.43	1.93	.018	324,425.1	89,537.6	710.1	414,672.8

*No mined area in water

described in more detail below, sums flow rates calculated from equations for steady flow under a hydraulic gradient, and transient, confined flow toward a linear drain (representing the sides of an approximately linear cut) and toward a well (representing the ends of the cut). The second approach (Approach B) was developed later, and applied to J-16, J-19/J-20, and J-21 in previous versions of this chapter, and to N-99 in the current version. This approach can be used to calculate inflow under unconfined and/or confined conditions.

Approach A - Aquifer and pit characteristics and the definitions of terms used in pit inflow calculations may be found in Attachment 1. Pre-mining flow calculations are based on the following form of Darcy's law:

Q = TIL

Where:

Q = Quantity of water flowing through the aquifer at the proposed highwall locations in gal./day.

T = Transmissivity of the exposed aquifer in gal./day/ft.

I = Natural hydraulic gradient in ft./ft.

L = Length of aquifer exposed in the highwall normal to the natural hydraulic gradient in ft.

Aquifer testing at Wepo monitoring wells indicates that water in the Wepo aquifer is under some confining pressure. Some of the coal seams have very low hydraulic conductivities and act as aquitards. Water in the alluvium is believed to be in both unconfined and confined conditions depending on depth and location. Those units in the Wepo aquifer believed to transmit water are most of the coal seams and sandstone units below the prevailing water level. Alluvial ground water is assumed to flow from the entire saturated thickness of the alluvium.

In Approach A, the removal of ground water from aquifer storage was calculated using two equations; one to compute the radial component of inflow to the ends of a pit and the other to compute the linear component of inflow to the longitudinal sections of the pit. Radial inflow to each end of the pit was calculated using the following constant drawdown-variable discharge equation (Jacob and Lohman 1952 and Lohman 1972, pp. 23-24).

$$Q = 2\pi TG(\alpha)s$$

$$\alpha = \frac{Tt}{Sr_w^2}$$

Where:

Q = Radial discharge into one end of the pit in ft^3/day

T = Transmissivity of the exposed aquifer in ft^2/day

S = Storage coefficient

s = Drawdown in the aquifer at the pit face in ft.

r_w = Radius of the pit opening in ft.; equal to ½ the width of the initial box cut

$G(\alpha)$ = The G function of α (see Lohman, 1972, p. 23)

t = Time since discharge began in days

The linear portion of inflow from aquifer storage was calculated using the constant drawdown-variable discharge drain equation derived by Stallman (Lohman, 1972, pp. 41-43):

$$q = \frac{2s\sqrt{ST}}{\sqrt{\pi t}}$$

Where:

q = Discharge from an aquifer to both sides of a drain per unit length of drain in ft^2/day

S = Storage coefficient

s = Drawdown in water level at drain in ft.

T = Transmissivity of exposed aquifer in ft^2/day

t = Time since drain began discharging in days

With confined aquifer conditions, lowering of the water level occurs with the lowering of hydrostatic head. The release of water from aquifer storage under confined conditions is small per unit area, because it is only a function of the secondary effects of water expansion and aquifer compaction. After some length of exposure, the hydrostatic head may decline far enough that the aquifer becomes unconfined. Further declines in the water level would then be accompanied by significantly greater quantities of ground water discharge per unit area. It is assumed that during the life of the pits, ground water flow in the affected portions of the Wepo aquifer will remain under confined conditions or that the unconfined area would only extend a short distance from the pit.

The equation for radial inflow assumes that a constant concentric head surrounds each end of the pit. The actual situation representing radial flow to the ends of the pit can be described as an arc of a circle whose center coincides with the center of the pit. If "x" is the arc of the circle intersected by the pit ends, then:

$$Q_R = \frac{xQ}{360}$$

should approximate the actual radial discharge into the ends of the pit.

The variables used in the above-mentioned equations were determined as follows:

1. Transmissivity and storage coefficients were determined from aquifer tests and the thickness of the portion of the aquifer being intercepted.
2. Gradients were determined from water level contours of the Wepo aquifer (Drawing No. 85610).
3. Drawdowns at the pit face ranged from 3.9 to 13.4 ft./day using the calculation technique derived by McWhorter (1982, p. 28).
4. Pit lengths, lengths below water level and the number of days when ground water discharges into the pit were determined by overlaying pit bottom isopachs, annual pit disturbance maps, and Wepo water level contour maps.

To date, no mining pits have directly intercepted the alluvial aquifer. Should this ever occur, the previously described pit discharge equations require the following modifications. Ground water through flow in the alluvial aquifer will be calculated from:

Q = PIA

Where:

Q = Quantity of water flowing through the aquifer into the ends of the pit in gal./day

P = Permeability of the exposed aquifer in gal./day/ft^2

I = Natural hydraulic gradient in ft./ft.

A = Average cross sectional aquifer area through which the flow occurs in ft^2

Ground-water contribution from storage was calculated using the linear and radial flow equations with the following modifications:

$$s_o = s_o - \frac{s_o^2}{2b}$$

Where:

s_o = Observed change in water level in the mine pit

b = Saturated thickness of the exposed aquifer prior to pit development and dewatering

A possible additional source of ground-water inflow is induced recharge from the alluvial aquifer where the water level in the alluvial aquifer is at a higher elevation than the pit bottom. Trial computations were performed using the average flow velocity equation described by Lohman (1972, pp. 10-11):

$$v = \frac{K\left(\dfrac{\Delta h}{\Delta l}\right)}{\theta}$$

Where:

v = Average flow velocity in the aquifer in ft./day

K = Hydraulic conductivity of the permeable units in the segment of the Wepo Formation that the induced recharge would have to flow through before reaching the pit in ft./day.

$\Delta h / \Delta l$ = Ground water gradient between a chosen elevation in the aquifer at the highwall and the recharge boundary in ft./ft.

θ = Porosity of the permeable units of the Wepo aquifer.

Pit inflow estimates were determined for that portion of the total pit length and associated time intervals that each pit was assumed to be below water level. Calculations for each component of inflow were based on the sum of daily values, which incorporated a continually increasing pit length. Each component of inflow from the Wepo aquifer as well as the totals of all inflow components for each year are presented in Tables 1 through 4.

Trial computations suggest that the hydraulic conductivity of the Wepo Formation is so low that induced recharge cannot reach the pit before one or two rows of spoil have been placed back in thus precluding the induced recharge from ever reaching the active pit.

Approach B - This approach was developed to be able to calculate inflow rates under confined or unconfined conditions. If the confined option is selected, it is assumed that conditions are initially confined, but can become unconfined as water levels decline. The flow equations for confined and unconfined conditions are solved by numerical integration. The algorithm uses information on the rate of pit advance to calculate the daily inflow, and reports the inflow on an annual basis. However, it does not consider the effects of antecedent dewatering, and therefore tends to conservatively overestimate the inflow rate. This approach is described in detail in Appendix 2. This method was used to predict inflow rates for J-16, J-19/J-20, and J-21 (Tables 5 through 7).

The following procedures were used and assumptions made in estimating inflow to the N99 pit for calendar years 2005-2013:

- Wepo wells in the area surrounding the N99 pit were selected, and recent water level data were evaluated to determine whether water table elevations had changed significantly from those used in the calculation of the 1985 water-table map. The Wepo wells evaluated include: 38, 39, 40, 41, 42, 43, 44, 49, 52, 53, 54, 159, 178. Data available through May of 2003 were used in this evaluation.

  Although there were obvious trends in the data for the majority of the 13 wells, the most recent data point was used in this evaluation, since this should be most representative of the water table at start of mining in N99. These data were compared to the 1985 water table map, and revisions made as necessary. As a result of these comparisons, Drawing No. 85611, 2003 Wepo Water Level Contour Map, has been constructed (see Volume 23, PAP).

- The May 2003 water-table map was then compared with the anticipated elevations for the bottom of the N99 pit, and a 'difference' contour map was constructed that identified those areas where the 2003 water table was above the bottom of N99. The difference map indicates that the water table will be above base of pit along the majority of the eastern boundary, and in the northwestern section of N99 (in the area between pits N11 and N6). The difference map was then overlaid on the projected cuts for Calendar Years (CY) 2005-2013, which indicated that only those cuts in the northwestern section of the pit will encounter water within this time period. Cuts to be completed in CY2005-2007 are all located within the southwestern section of N99, and will therefore encounter minimal water. In Calendar Years 2008-2013, cuts will be made both within the southwestern section of N99, and in the northwestern section where water inflow to the cuts is expected.

- The analytical code Minel-2_3 was used to estimate the amount of flux entering the cuts in the northwestern section of N99 for CY2008-2013. [Minel-2_3 is a modification of Minel-2 allowing pit geometry information to be input yearly, rather than using a single set of values for the entire mining period.] General parameters, and the selected values used as input to the code include:
  - o The Wepo was simulated as confined, based on the lithology of the formation, and the low values of storage coefficient determined from aquifer tests.

- o The hydraulic conductivity was set to 0.03432 ft/day, which is the geometric mean of the 24 hydraulic conductivity values for Wepo wells listed in Table 32 (Chapter 15, Hydrologic Description, PAP). The arithmetic average conductivity value was not used, since this weighted the calculated value towards the fewer, significantly higher values of conductivity, and would have overestimated this parameter.
- o The regional hydraulic Gradient (0.014) was estimated from the May 2003 water-table map.
- o A conservative value for the storage coefficient ($1x10^{-4}$) was estimated from the larger of the two values presented in Table 32. Use of a lower value would result in lower values of inflow.

The remaining parameters are specific to the cuts within each calendar year, and include: saturated area; average width of cut; average saturated thickness, days open, and whether this was the first cut in the pit (inflow is assumed through both sides of the initial cut only).

There are two components that contribute to inflow into the cuts: flux controlled by the regional hydraulic gradient (termed $Q_{natural}$ in the code), and flux from water in storage (termed $Q_{drainage}$ in the code). The code assumes that the regional hydraulic gradient, and therefore the regional flux component, is perpendicular to the long axis of each cut. This assumption is generally valid for the southern two-thirds of the cuts located within the northwestern section of N99; however, the gradient is not perpendicular in the northern one-third of the cuts. In this area, groundwater discharge into the cuts will be less than if the gradient was perpendicular, and a correction factor must be applied to decrease the inflow appropriately (this is done outside of the code). Therefore, an approximate *dividing line* was identified between these two areas, separating Area A representing the northern one-third of the cuts, from Area B representing the southern two-thirds of the cuts, and the *area, saturated thickness*, and *days open* parameters were calculated separately for the sections of the cuts located within areas A and B. The correction used to calculate the regional component of inflow to the cuts in Area A is:

Corrected $Q_{natural}$ = $Q_{natural}$*([width of cut]*sin(alpha) + [length of cut]*cos(alpha))

Alpha is the angle between a line perpendicular to the length of the cut, and the regional hydraulic gradient. The first component within the parentheses represents flux across the end of the cut, and the second component represents flux across the length of the cut. Maximum inflow to the cuts occurs when the regional hydraulic gradient is perpendicular to the length of the cut (angle alpha is 0 degrees in the above equation), and minimum inflow occurs when the gradient is parallel to the length of the cut (angle alpha is 90 degrees - this results in flux across the end of the cut only).

The *regional hydraulic gradient* is approximately parallel to the cuts in CY10-13, indicating that the regional flux component is minimal and is simulated as occurring across the end of the cuts only. The cut within CY08 does not extend north of the *dividing line*. For the cuts in CY09, an angle of 45 degrees was used to calculate the regional flux component.

Total lengths for all cuts within the northeastern section of N99 for each calendar year were measured and summed in ArcView, and total areas were calculated. These were used to calculate average widths for each of the cuts as input to Mine1-2_3.

- Output from Mine1-2_3 includes values for $Q_{natural}$, $Q_{drainage}$, and Q_{total} for Areas A and B. For each of the cuts in Area A, a corrected $Q_{natural}$ value was calculated using the equation above, this value was added to $Q_{drainage}$, and a corrected Q_{total} determined. The corrected Q_{total} values were summed for each calendar year, and added to the corresponding Q_{total} values for that calendar year from Area B to derive a total flux per calendar year.

Results for N-99 are presented in Table 7a. [This nomenclature was adopted to avoid changes in table number throughout the remainder of this Chapter.] The predicted inflow varies from year to year because of changes in the length of the pits beneath the water table, and the estimated depth below the water table. In addition, drainage from two directions is assumed for the first year (2008), but from only one side in later years. The maximum estimated rate, which occurs in 2008, is approximately 10 gallons per minute (gpm); the lowest rate is predicted to be approximately 2.5 gpm, in 2010.

Table 7a. Estimated annual inflow for pit N-99 and length of time the base of the pit is below the pre-mining water table.

Year	Inflow (gallons)	Total No. of Days in Water
2008	1170710	84
2009	2105469	226
2010	485396	135
2011	607995	106
2012	1050225	264
2013	783849	241

For all pits except N99, drawdowns in the Wepo and alluvial aquifers in the vicinity of the wet pits were theoretically projected and calculated for radial distances out to 5 feet of drawdown (see Figure 1) using the Theis equation and the greatest volumes of annual pit inflow (see Tables 1 through 7). Though ground water was projected to be intercepted by the N14 pits, this never occurred and Figure 1 has been modified to reflect this. The Theis drawdown analyses assumed horizontally contiguous aquifer units which is not the case, particularly in the J19/20 and J21/23 mining areas. The permeable units within the Wepo formation which will be disturbed by mining are perched aquifers in some locations (near Wepo wells 62R and 65), pinch out and/or are vertically displaced owing to some minor structure within the Peabody leasehold. Thus the actual extent of the drawdown will likely be considerably less than that shown on Figure 1. A 5-foot drawdown cutoff was selected because natural water level fluctuations measured in the Wepo and alluvial monitoring wells on the PWCC leasehold are of that magnitude.

Projected water-level changes predicted to occur because of dewatering of all the existing pits (excluding N99) are compared to historical variability in nearby monitoring wells in Table 8. Included on Table 8 are projected drawdown and historic completion and water level information for the two local wells (4K-389 and 8T-506) which are at least partially completed in the Wepo aquifer and are located within the projected 5-foot drawdown contours. Table 8 also includes a column with comparisons of projected drawdowns and actual measured water levels in alluvial and Wepo monitoring wells and a column with percent of available well bore water height loss for the two local wells as a result of theoretical pit pumpage drawdowns.

Table 8

Projected Pit Inflow Drawdowns at Well Locations Versus Measured Water Level Ranges at Alluvial and Wepo Monitoring Wells and Static Water Levels at Local Wells

PWCC Well Id	Pit Inflow Analysis Maximum Projected Drawdown (Feet)	Background Water Level Range		Historic Water Level Range (1/88-1/95)		Current Water Level Range (1998)		Current Maximum Versus Background/ Historic Maximum
		Max	Min	Max	Min	Max	Min	
ALUV17	4	7.4	5.0	8.1	5.4	8.9	6.3	1.5 Ft deeper
ALUV23	30	18.6	15.6	18.1	16.0	17.6	16.6	No change
ALUV27R	42	-	-	26.7	21.5	28.3	26.7	1.6 Ft deeper
ALUV32R	30	-	-	Dry	1.1	Dry	Dry	No change
ALUV33R	10	5.3	3.1	5.0	2.9	5.0	3.3	No change
ALUV80R	4	11.7	8.9	12.9	10.6	10.8	10.5	No change
ALUV87	12	22.5	14.2	23.1	17.8	22.2	21.5	No change
ALUV88	50	4.3	1.3	3.6	1.2	3.7	1.6	No change
ALUV89R	50	-	-	5.0	2.5	5.0	3.0	No change
ALUV98R	42	-	-	14.3	9.6	13.0	11.6	No change
ALUV99R	23	-	-	13.8	9.8	15.0	13.0	1.2 Ft deeper
ALUV101R	30	Dry	Dry	Dry	Dry	Dry	Dry	No change
ALUV102	12	12.7	11.1	13.8	11.0	13.6	13.5	1.1 Ft deeper
ALUV108R	13	-	-	10.9	7.1	10.3	10.1	No change

Table 8 (Cont.)

Projected Pit Inflow Drawdowns at Well Locations Versus Measured Water Level Ranges at Alluvial and Wepo Monitoring Wells and Static Water Levels at Local Wells

PWCC Well Id	Pit Inflow Analysis Maximum Projected Drawdown (Feet)	Background Water Level Range		Historic Water Level Range (1/88-1/95)		Current Water Level Range (1998)		Current Maximum Versus Background/ Historic Maximum
		Max	Min	Max	Min	Max	Min	
ALUV165	41	-	-	28.7	20.3	29.8	28.6	1.1 Ft deeper
ALUV168	15	-	-	1.4	+0.4	1.6	1.3	0.2 Ft deeper
ALUV169	12	-	-	9.0	7.2	9.0	8.1	No change
ALUV170	10	-	-	5.8	4.5	6.0	4.2	0.2 Ft deeper
ALUV193	5	-	-	12.4	10.9	11.6	9.8	No change
ALUV199	45	-	-	17.2	13.5	15.4	12.5	No change
ALUV200	5	5.9	4.1	6.4	3.8	5.1	4.6	No change
WEPO42	5	-2.5	-0.1	-2.5	-1.1	-1.6	-1.4	No change
WEPO43R	7	-	-	138.1	137.6	136.5	135.6	No change
WEPO49	32	8.7	4.3	5.7	1.8	1.6	1.1	No change
WEPO53	10	68.7	37.8	66.0	46.4	61.6	60.5	No change
WEPO54	25	55.2	47.4	51.8	46.8	51.4	50.9	No change
WEPO62R*	65	159.7	114.0	227.6	133.0	217.8	213.5	68 Ft deeper**
WEPO63R	38	52.9	50.6	74.5	66.3	78.6	78.6	26 Ft deeper

* Background and historic water levels through 1996 are from WEPO62 corrected for ground surface elevation.

** Rather than use the current maximum versus the background maximum, the all-time maximum in 1996 versus background was used.

Table 8 (Cont.)

Projected Pit Inflow Drawdowns at Well Locations Versus Measured Water Level Ranges at Alluvial and Wepo Monitoring Wells and Static Water Levels at Local Wells

PWCC Well Id	Pit Inflow Analysis Maximum Projected Drawdown (Feet)	Background Water Level Range		Historic Water Level Range (1/88-1/95)		Current Water Level Range (1988)		Current Maximum Versus Background/ Historic Maximum
		Max	Min	Max	Min	Max	Min	
WEPO64R	60	46.2	36.0	42.4	35.2	40.2	37.3	No change
WEPO65	45	146.4	71.9	139.3	113.8	139.4	135.2	No change
WEPO66	15	89.1	75.4	88.0	82.0	87.5	87.4	No change
WEPO67	<5	204.5	129.5	208.3	181.5	182.3	182.0	No change
WEPO68	25	-	-	110.8	110.1	110.1	108.6	No change

Local Well Id	Pit Inflow Analysis Maximum Projected Drawdown (Feet)	Well TD	Static Water Level	Percent of Potential Water Height in Well Bore Lost to Pit Pumpage
4K-389	8	417	356	13
8T-506	14	552	34	2.7

Maximum water levels at only two of the twelve Wepo monitoring wells within or at the 5-foot theoretical pit inflow drawdown contours are greater than background or historic maximum water levels. At WEPO62R, maximum water levels are 68 feet deeper than background maximum water levels for WEPO62. This deepening exceeds the theoretical maximum projected drawdown for WEPO62 and 62R by 3 feet. WEPO62 appears to have been open to one or more perched zones. These perched zones are usually of limited aerial extent and can exhibit large well bore water level changes which are not indicative of true aquifer water level changes. At WEPO63R, current maximum water levels are 26 feet deeper than background or earlier maximum water levels but are still less than the theoretical projected maximum drawdown for this well which is 38 feet.

Maximum water levels at 5 of the 21 alluvial wells within or at the 5-foot theoretical pit inflow drawdown contours are greater than background or historic maximum water levels measured at these wells. They exceed the earlier maximum levels by only 0.2 to 1.6 feet and all drawdowns are less than the theoretical maximum projected drawdowns.

Based on the theoretical pit inflow drawdown contours, local well 4K-389 is projected to have its water level deepened by 8 feet or 13 percent of its total available water height of 61 feet. Local well 8T-506 is projected to have its water level deepened by 14 feet or 2.7 percent of its available water height of 518 feet. From the historic and current water levels at Wepo and alluvial monitoring wells in the vicinity of the two local wells, it appears likely that the projected water level declines at the two local wells will be less than that theoretically calculated. The likely drawdown at the two local wells from pit inflows will not be significant.

Estimates of drawdown caused by dewatering of N-99 were developed using the analytical-element simulation program TWODAN, version 5.0. This program solves the groundwater flow equations in two dimensions based on spatial and temporal superposition. Time-varying withdrawals can be simulated using wells. The estimated annual pit inflows were tabulated for N-99, and the average inflow rate was calculated.

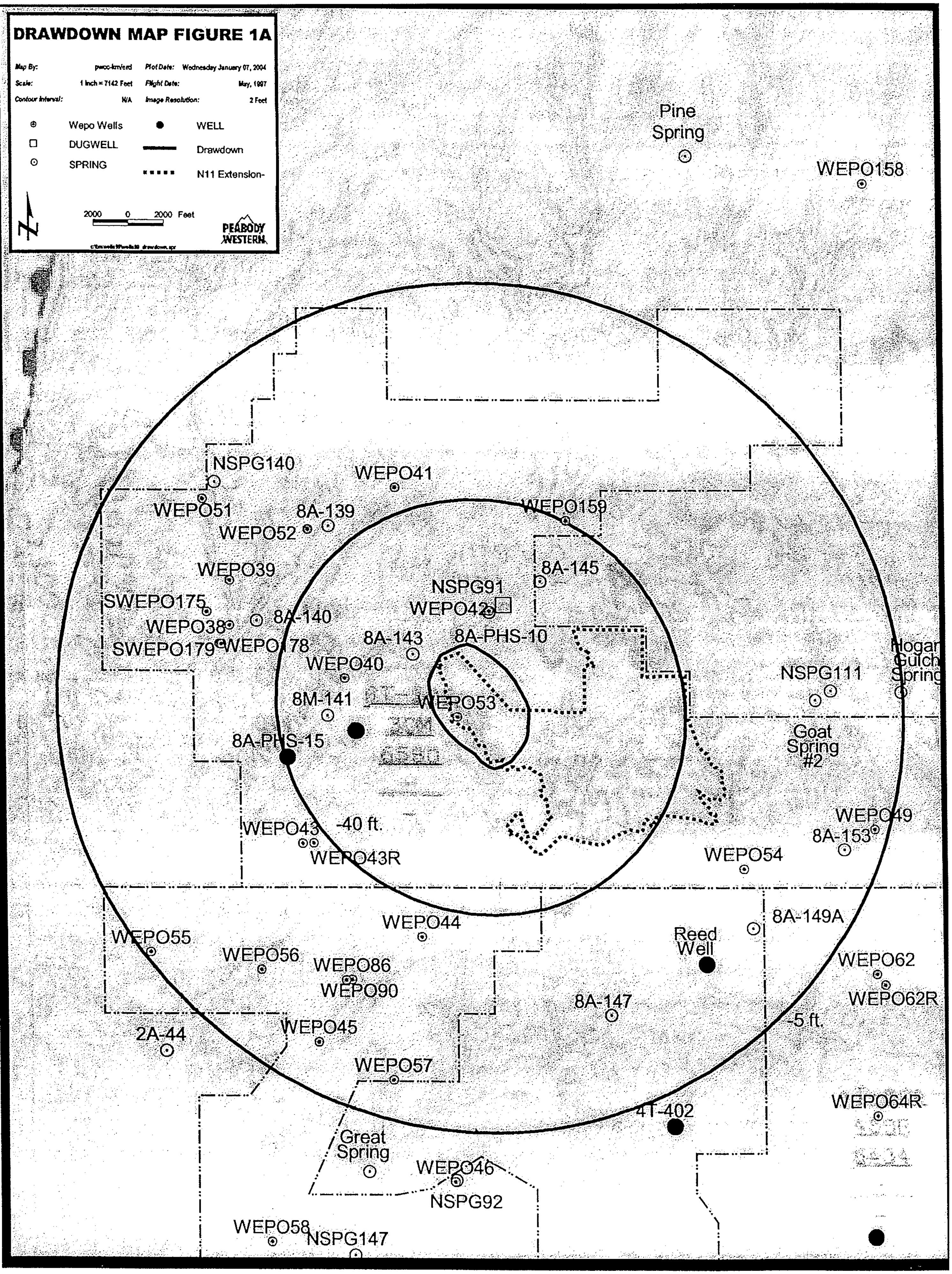
The inflow into the pits was simulated by using ten wells for the submerged part of the pit based on the mining plan through 2013. These wells were distributed around the perimeter of the pit and in the interior of the pit. The discharge rate for each well was one-tenth of the average inflow rate, resulting in an approximately even distribution of inflow over the pit.

The geometric mean of the hydraulic conductivities determined from aquifer tests of Wepo monitoring wells (Table 32, Chapter 15, Hydrologic Description, PAP), 0.03432 ft/d was used for the hydraulic conductivity of the Wepo, along with a specific storage value of 0.000001/ft. The Wepo was assumed to be 150 feet thick, even though it is over 300 feet thick in the vicinity of these pits, because of the limited depth of the pits. This value was chosen to approximate the effect of partial penetration of the pits into the saturated Wepo, and to subtract the thickness of the Wepo above the water table.

Figure 1a shows the simulated locations of the 5- and 20-foot drawdown contours at the end of 2013, when mining of N-99 below the water table and south of the beltline is scheduled for completion. Because the approach used to estimate the inflow rates does not take into consideration the decline in water levels caused by inflow into the pit in previous years, it will tend to over-estimate the inflow rate in the later years. In addition, the predicted inflow rates have tended to be considerably higher than observed during mining. For example, Western Water & Land (Water Waste and Land, 2003) noted

> The total [annual] inflows for pit J-1/N-6 were projected to range from approximately 50,000 gallons in 1972 to 3,182,179 gallons in 2003. As mining has progressed over the last several decades, it has generally been observed that pit inflows were overestimated, and in some cases no inflow has occurred at all. For example, initial mining of the southern portion of the N-6 Pit saw enough inflow to require pumping, but subsequent mining of this pit to the north has not resulted in any observed pit inflows.

The projected drawdown that is projected (though not expected) to occur is limited to a relatively small area. Drawdown in the N-99 pit is projected to be greater than 20 feet. Based on the predicted inflow rates, the drawdown beneath Coal Mine Wash will be between 5 and 20 feet. This projection raises a question of whether discharge rates into Coal Mine Wash from the Wepo will be affected. The Wepo is believed to be the source of discharge into the wash downstream from where Coal Mine Wash passes beneath the overland conveyor. Peabody does not believe that there will be significant impacts on this discharge for several reasons. First, observations of pit discharge suggest that the technique overestimates the inflow rate, as noted above. Second, the mining of N-6 has not caused a noticeable impact on the locations of discharge into Coal Mine Wash. Although the baseflow of Coal Mine Wash is not measured, a reduction in discharge caused by declining water levels beneath the wash would be also manifested by downstream movement of the location of the uppermost area of discharge. This has not been observed over many years of mining. Third, the water levels in Wepo 40, a well close to both N-6 and Coal Mine Wash, appear to be affected more by changes in local recharge than by dewatering.



DRAWDOWN MAP FIGURE 1A
Map By: pwcc-km/erd
Plot Date: Wednesday January 07, 2004
Scale: 1 Inch = 7142 Feet
Flight Date: May, 1997
Contour Interval: N/A
Image Resolution: 2 Feet
Wepo Wells
WELL
DUGWELL
Drawdown
SPRING
N11 Extension-
2000 0 2000 Feet
PEABODY WESTERN
Pine Spring
WEPO158
NSPG140
WEPO41
WEPO51
8A-139
WEPO159
WEPO52
WEPO39
8A-145
NSPG91
SWEPO175
WEPO42
WEPO38
8A-140
8A-143
8A-PHS-10
SWEPO179
WEPO178
WEPO40
Hogan Gulch Spring
NSPG111
8M-141
WEPO53
Goat Spring #2
8A-PHS-15
WEPO49
WEPO43
-40 ft.
8A-153
WEPO43R
WEPO54
8A-149A
WEPO44
Reed Well
WEPO55
WEPO56
WEPO86
WEPO62
WEPO90
WEPO62R
8A-147
-5 ft.
2A-44
WEPO45
WEPO57
WEPO64R
4T-402
Great Spring
WEPO46
NSPG92
WEPO58
NSPG147

In summary, Wepo water is expected to enter N99 (and other) pits. Based on operational experience, the inflow rates have generally been lower than predicted by the techniques described here. Similarly, the simulated drawdowns caused by dewatering are probably higher than will be encountered. Monitoring wells in the immediate vicinity of pits exhibit declines in water levels, but the drawdowns in other areas are likely to be low. Thus, inflow in the N99 pit is likely to be less than indicated in Table 7a, and the drawdowns are similarly to be less than portrayed in Figure 1a.

Removal of Local Wells and Springs. One local well (4T-404), completed in the Toreva aquifer, is located within the proposed life-of-mine mining plan area. In addition, another local well (4T-403), completed in the Toreva aquifer, was removed in advance of the mining operation in the J-7 mining area. One local spring (Site #97) was removed in advance of mining at N-14. The impacts have been mitigated during mining by providing alternative water sources (N-aquifer public water standpipes). The two wells will be replaced with ones of comparable quality and yield following the completion of mining and reclamation in the respective mining areas. The spring will be mitigated by retention of a permanent impoundment (see Chapter 19).

Containment of Pit Inflow Pumpage. It is sometimes necessary to pump ground water which seeps into pits to allow work to continue and to prevent slumping of spoil piles resulting from saturation near the bottom of the pit. Several sediment ponds and large dams (see Table 9) exist or will exist around the pits to contain all pit pumpage as well as storm water runoff and sediment from the disturbed areas up-watershed from the ponds. Referring to Tables 1 through 7a, it can be seen that the maximum pit pumpage in any one year will be 19 to 37 acre-feet and will occur in the J-19/20 pit. Typical quantities of pit pumpage will be on the order of 2 or less acre-feet per year. The larger dams are designed to contain this additional volume of water with adequate freeboard. Reed Valley Dam has been designed to impound 475 acre-feet of water and J-7JR dam will hold an estimated 700 acre-feet of water. The capacity of smaller sediment ponds to contain storm runoff will be maintained by pumpage from the ponds. The current NPDES Permit (Chapter 16, Attachment 3) allows for pond dewatering or pond to pond pumpage.

Impact of Replaced Spoil Material on Ground-Water Flow and Recharge Capacity. Pits remain open only until the coal has been removed. Following the short-term impacts on the ground-water system associated with open pits, a longer term impact is experienced due to the placement of spoil material in the mined-out pits. A wide range in permeabilities for spoil material can occur depending on how it is placed.

TABLE 9

Sediment Ponds and Dams to be Used to

Contain Pit Pumpage

Mining Area	Sediment Ponds and Dams Containing Pit Pumpage
N10	N10-G Series Ponds
N11	N11-A Series Ponds
N14	N14-D, -E, -F and -G Dams
N99	N11-H, -I, and -J Ponds
J1/N6	Wild Ram Valley Dam
J16	Reed Valley and J16-A Dams
J19/20	Reed Valley and J7-Jr. Dams
J21	Reed Valley and J21 Dams

Rahn (1976) reported that spoil material replaced using a dragline in one instance and a scraper in another, yielded hydraulic conductivities of 35.3 ft./day and 0.4 ft./day, respectively. Van Voast and Hedges (1975) concluded that greater porosities and hydraulic conductivities will result from volume changes (approximately one-fourth greater) between the spoil material in its original compacted, stratified state, and in its rearranged state following replacement, regardless of the method of replacement used.

Spoil material will be regraded by dozers and scrapers and final contouring will be accomplished with dozers. Based on the conclusions of the above studies, the spoil material should have higher porosities and permeabilities than it did in its original state. The topsoil surface will be disked as part of the reclamation activity; this procedure should further enhance the rainfall and overland flow infiltration rates.

However, regardless of the infiltration rates of regraded spoil, infiltration in reclaimed areas will provide little or no recharge to the Wepo aquifer. The distance from the reshaped land surface to the saturated portions of the Wepo aquifer and the limited annual precipitation preclude rainfall and snowmelt recharge other than in burn and clinker or highly fractured areas. These areas are found adjacent to rather than in the reclaimed coal fields.

The time necessary for the replaced spoil material to become resaturated and for flow patterns to be reestablished will depend on the porosity and permeability of the replaced spoil material. Recharge of previously saturated areas may take from a few years to 100 years; but, the impact will be of little significance to the local well users. There are no local wells completed in the Wepo aquifer in the areas to be mined and local wells which do exist in the vicinity will not be significantly impacted (See Table 8 and Figure 1). The only exceptions to this are the two Toreva wells (4T-403 and 4T-404) which are discussed in the previous section "Removal of Local Wells". The maximum drawdowns will be at specific points within individual pits in a particular year and are estimated to range from 14 feet to 115 feet with the greatest drawdown in the J-16 and J-19 pits. Following the resaturation period, ground-water levels will recover to near premining levels.

<u>Impact of Replaced Spoil on Ground-Water Quality</u>. The replacement of spoil material in the areas of the pits where portions of the Wepo aquifer and in one case, the alluvial aquifer, are to be removed will have a long-term, localized impact on the ground-water quality in these areas. Two types of chemical reactions will probably occur as the spoil resaturates resulting in a change in the local ground-water quality - dissolution and oxidation and reduction of sulfides and organic sulfur.

The first chemical reaction will be an increase in the major ions as a result of dissolution of readily soluble materials in the spoil. Various leaching processes acting over geologic time remove most of the readily soluble constituents from the permeable unsaturated and saturated units in the undisturbed overburden. In contrast, a considerable quantity of soluble constituents may still remain in the relatively impermeable strata, such as the finer grained clay, siltstones and shales. Fracturing and mixing of materials during pit excavation and reclamation exposes many new chemically reactive surfaces and mineral constituents that may readily release ions to the ground water during resaturation.

Studies performed by Van Voast et al. (1978) and McWhorter et al. (1979) in western mine spoils suggest that increases in TDS from 50 to 130 percent could be expected in the disturbed portions of the Wepo aquifer following resaturation of the spoil material. Based on the Wepo aquifer water quality types, the more soluble salts (principal ions) that would account for these increases in TDS are Ca, Mg, Na, SO_4 and HCO_3.

On a related matter, Montana Department of State Lands personnel have noticed in their review of mine overburden data that materials with high salinity are generally quite shallow (less than 15 meters). Normal dragline operation would generally place some of the near surface overburden in the lower portions of the pit. This mining practice could cause the placement of some of the more saline materials in the resaturated zone and result in a greater degree of ground-water degradation. A review of overburden core data for portions of the pits that will intercept the Wepo aquifer (N-6, N-10, N-11, N-14, N99, J-16, J-19/20 and J-21) indicates that there are no significantly high conductivity zones in the overburden material. Therefore, significant salinity increases are not expected in resaturated graded spoil on the Black Mesa leasehold.

The second principal chemical reaction that occurs in spoil material and could affect ground-water quality is the oxidation and reduction of sulfides and organic sulfur. In the west, waters which contact spoil are rarely acidic. Acid zones will probably form in the spoil; however, sufficient carbonate materials and alkaline salts are available to neutralize acid production resulting from the oxidation of sulfides.

Cores from within or immediately adjacent to the wet portions of the pits have been analyzed to determine the acid potential of the overburden (see Appendix B). The overall acid-forming potential of core material involves a comparison of the acid potential and the neutralization potential expressed in terms of tons of $CaCO_3$ required per 1000 tons of material for neutralization (acid potential) and tons of $CaCO_3$ excess per 1000 tons of material (neutralization potential). Table 10 is a summary of: (1) the percent of the total core that is comprised of material with acid potential; (2) the mean weighted acid potential; and (3) the mean weighted neutralization potential. Cores from within or adjacent to wet pits, and new cores (2003) drilled in the J2, J4, J6, J9, J14, J15, J23, N9, N12, and N99 coal resource areas are also included. Only 1 core; Core #30356EO in the N-9 mining area had a higher mean weighted acid potential. All other cores indicate excess ($CaCO_3$) neutralization potential. The neutralization of the acid produced from the oxidation of sulfides and sulfates does have an adverse water quality related side effect. In the process of the carbonate minerals reacting to achieve neutralization, there is increased dissolution of alkaline salts and consequently elevated TDS levels.

Considerable controversy surrounds the potential activity of the different forms of sulfur and the significance of organic sulfur. In western mine settings as much as 70% of the total sulfur analyzed has been found to be organic sulfur. According to Dollhopf (1984), organic sulfur when oxidized produces approximately one-third less acid than the sulfide forms of sulfur in a low (<4) pH environment. A comparison of total sulfur versus pyritic sulfur in cores taken on Black Mesa suggests that organic sulfur is approximately 20 percent of the total sulfur. In this comparison it was assumed that only the above two forms comprised the total amount of sulfur. Whether it is pyritic or organic sulfur, not all the forms of either will react to form acid. Considerable research remains to be done in this area.

Oxidation of sulfides primarily occurs above the water table in the zone of water level fluctuations or in zones of significant infiltration of precipitation. As was explained previously, significant recharge will not occur to the aquifer through the spoil material, so the potential of this as a mechanism for additional leachate movement and acid production on the leasehold is minimal. Also, the typical Wepo water level fluctuations range from 2 to 3 feet or less. This does not constitute a significant zone in which alternate weathering and leaching of ions could occur.

Below the water table, less oxygen may be available than in the overlying unsaturated vadose zone resulting in less sulfide oxidation-reduction increases in salinity or acidity of the water. Pionke and Rogowski (1979) state that water has an oxygen diffusion coefficient four magnitudes less than for sulfides in air. The opportunity exists during the mining process to minimize the oxidation of pyrites and the production of sulfates by burying localized pyritic zones in the postmining saturated zone. Sulfide reduction may be the dominant process occurring below the water table if substantial populations of sulfate reducing bacteria are present. No information exists regarding the possibility of the presence of these bacteria on the leasehold.

TABLE 10

Summary of Acid and Neutralization Potential for Cores in Mining Areas Projected to Intercept the Wepo Aquifer

Overburden Core No.	% of Core With Negative Potential	Mean Weighted Acid Potential (Tons $CaCO_3$ Needed for Neutrality per 1000 Tons Material)	Mean Weighted Neutralization Potential (Tons $CaCO_3$ Excess per 1000 Tons Material)
J2 Mining Area			
30362EO	13.67	7.77	33.11
J4 Mining Area			
30359EO	20.21	10.27	29.21
J6 Mining Area			
30366EO	22.31	5.25	19.16
30367EO	36.42	8.82	20.63
J9 Mining Area			
30364EO	7.61	3.00	30.34
J14 Mining Area			
30360EO	8.32	5.20	42.03
30361EO	21.88	3.51	25.68
J15 Mining Area			
30363EO	11.97	5.82	40.87
N6 Mining Area			
21104C	16.63	9.76	40.94
23163C	4.48	7.98	45.01
23164C	15.38	11.26	39.39
23165C	26.35	10.36	39.51
23166C	14.97	7.41	62.12
24093C	14.42	8.21	44.63
24094C	12.98	7.13	61.89
24095C	12.60	6.94	50.53
24096C	5.39	6.92	52.68
24097C	22.77	8.61	40.35
24098C	23.32	7.21	38.85
24099C	11.93	2.82	36.39
24400C	12.50	9.23	51.70
24401C	20.14	10.90	21.81
24402C	21.67	12.54	38.14
J16 Mining Area			
23146C	44.57	24.37	32.29
23147C	33.14	17.81	28.66
23148C	41.22	30.79	39.28
23149C	1.42	4.59	24.60
23325C	37.64	13.89	28.80
23326C	32.34	11.06	40.85
23327C	45.26	23.06	39.89
23328C	34.72	24.12	39.41
26462C	12.28	2.65	27.30
J19 Mining Area			
24406C	33.23	5.05	27.74
24407C	32.03	16.48	32.03
24408C	17.97	4.34	32.01
24418C	24.09	15.39	34.28
J21 Mining Area			
24403C	12.02	7.44	79.73
24404C	11.98	4.97	73.07
24405C	12.36	8.49	54.99
J23 Mining Area			
30365EO	13.04	7.71	48.83
N9 Mining Area			
30355EO	29.64	16.10	51.16
30356EO	54.64	21.25	20.63
30357EO	34.30	18.57	41.57
30358EO	32.14	17.42	72.61

Revised 11/21/03

TABLE 10 (Continued)

Summary of Acid and Neutralization Potential for Cores in Mining Areas Projected to Intercept the Wepo Aquifer

Overburden Core No.	% of Core With Negative Potential	Mean Weighted Acid Potential (Tons $CaCO_3$ Needed for Neutrality per 1000 Tons Material)	Mean Weighted Neutralization Potential (Tons $CaCO_3$ Excess per 1000 Tons Material)
N10 Mining Area			
21099C	46.63	20.02	21.97
21100C	40.09	23.89	28.40
21101C	38.21	20.86	24.10
30354EO	12.32	15.81	43.99
N11 Mining Area			
26272C	29.61	18.73	42.57
26364C	25.91	18.50	49.32
26367C	20.76	14.00	69.67
26463C	37.84	17.98	58.24
N12 Mining Area			
30370EO	17.19	15.12	33.15
N14 Mining Area			
26269C	31.41	18.73	30.73
26271C	40.04	16.51	19.65
N99 Mining Area			
30351EO	11.06	10.09	34.62
30352EO	32.00	14.47	28.76
30353EO	18.88	14.12	33.72
30368EO	28.11	15.11	33.91
30369EO	32.48	16.34	24.77
30381EO	26.65	15.72	46.39

A final concern associated with the oxidation and reduction of sulfides and sulfates is the mobilization of trace metals in the ground-water system. Dollhopf et al. (1979, 1981) compared column leach extracts with spoil water quality. They found that the statistical means and ranges for the comparisons between column leachates and water from spoil wells often differed by as much as a factor of ten. Though they did state that column leachates were comparable to well water concentrations to a degree, they allowed that these correlations would have to be made at many mines with contrasting chemical conditions in order to verify the usefulness of this method for judging which overburden materials would be most suitable for aquifer reestablishment.

Evaluation of cores taken in the N-11, N-14, J-16, J-19/20 and J-21 mining areas for B, As, Se, Mo, Hg, Cu, Cd, Cr and Zn indicates that there are not high concentrations of any of these chemical constituents in the overburden material. During the oxidation and reduction stages of the sulfide zones in the saturated portions of the pits, trace metals will be alternately taken into solution as the pH drops and precipitated out as the acid is neutralized and additional alkali salts go into solution. Total recoverable metal analyses performed on Wepo and alluvial ground-water samples collected at below-mining monitors also support the core chemistry. Wepo and alluvial ground-water trace metal analyses presented in the annual "Hydrological Data Reports" and summarized in Table 11 indicate that both the dissolved and total recoverable concentrations of trace constituents at monitoring sites downgradient of wet pits are typically well below the livestock drinking water limits.

The above discussion has addressed the sources of potential ground-water quality degradation. In order to assess the significance of this potential degradation, the historic and potential uses of the Wepo and alluvial ground water is considered. Table 12 is a summary of the principal constituents in both aquifers that render the water sources unsuitable for livestock drinking water. Those monitoring sites chosen for Table 12 are either at or in the immediate vicinity (downgradient) of a pit that will intersect the Wepo and or alluvial aquifer. Livestock drinking water limits were taken from three sources. Tribal water quality standards (NNEPA, 1999; Hopi, 1998) were principally used and other livestock standards (F, NO3, NO2, and TDS) recommended by the National Academy of Science (NAS, 1974) and the Environmental Protection Agency (1995) were also included. All chemical parameter values listed are on water quality sampling at each site from 1986 through 2002.

The principal constituent rendering Wepo aquifer water unsuitable for use as livestock drinking water is fluoride. The NO3, Se and TDS standards were also exceeded at one site (WEPO46). Fluoride levels above 2 mg/l have been shown to cause mottling of teeth and

Table 11.

Summary of Dissolved and Total Recoverable Trace Metal Concentrations in Portions of the Wepo and Alluvial Aquifers Below Mining Black Mesa Leasehold (1986 – 2002)

Wepo Aquifer

Chemical Constituent	Range of Minimum Values (mg/l)	Range of Mean Values (mg/l)	Range of Maximum Values (mg/l)	Livestock Standards (mg/l)
Arsenic (D)	<.001-.002	.001-.002	<.001-.003	0.2
Arsenic (TR)	.001-.003	.001-.004	<.001-.005	0.2
Boron (D)	.03-.79	.065-.88	.08-1.2	5.0
Cadmium (D)	<.003-.008	.003-.011	<.003-.02	0.05
Cadmium (TR)	<.003-.009	.005-.009	<.005-.009	0.05
Chromium (D)	<.01-.01	.01-.01	<.01-.01	1.0
Chromium (TR)	<.01-.01	.01-.01	<.01-.01	1.0
Copper (D)*	<.01-.01	.01-.023	<.01-.02	0.5
Copper (TR)	<.01-.02	.01-.037	<.01-.06	0.5
Lead (D)*	<.02-.02	.02-.02	<.02-.02	0.1
Lead (TR)	<.02-.08	.02-.08	<.02-.08	0.1
Mercury (D)*	<.0001-.0003	.0003 - .0003	<.0001-.0003	0.01
Mercury (TR)	<.0001-<.0001	-	<.0002-<.0002	0.01
Molybdenum (D)	<.001-.002	.001-.003	<.001-.003	N/A
Molybdenum (TR)	<.001-.002	.001-.003	.001-.005	N/A
Selenium (D)*	<.001-.005	.001-.09	<.001-.21	0.05
Selenium (TR)	<.001-.007	.001-.09	<.001-.21	0.05
Zinc (D)	<.01-.03	.01-.05	.01-.07	25
Zinc (TR)	.01-.03	.02-.20	<.01-.53	25

Alluvial Aquifer

Chemical Constituent	Range of Minimum Values (mg/l)	Range of Mean Values (mg/l)	Range of Maximum Values (mg/l)	Livestock Standards (mg/l)
Arsenic (D)	<.001-.001	.001-.004	<.002-.015	0.2
Arsenic (TR)	<.001-.006	.001-.008	.001-.03	0.2
Boron (D)	<.02-.32	.088-.431	.11-.85	5.0
Cadmium (D)*	<.003-.02	.003-.02	<.01-.02	0.05
Cadmium (TR)	<.003-.02	.003-.02	<.01-.021	0.05
Chromium (D)*	<.01-.03	.01-.038	<.01-.07	1.0
Chromium (TR)	<.01-.03	.01-.11	<.01-.35	1.0
Copper (D)*	<.01-.04	.01-.043	<.01-<.08	0.5
Copper (TR)	<.01-.02	.01-.062	<.01-.22	0.5
Lead (D)*	<.02-.08	.02-.08	.02-.12	0.1
Lead (TR)	<.02-.04	.02-.14	<.02-.59	0.1
Mercury (D)*	<.0001-.0009	.0002-.002	<.0002-.003	0.01
Mercury (TR)*	<.0001-.0004	.0001-.0007	<.0001-.0013	0.01
Molybdenum (D)	<.001-.002	.001-.004	<.001-.01	N/A
Molybdenum (TR)	<.001-.002	.002-.008	<.001-.016	N/A
Selenium (D)	<.001-.009	.001-.013	<.002-.023	0.05
Selenium (TR)	<.001-.004	.001-.011	.002-.024	0.05
Zinc (D)	<.01-.05	.02-.08	.02-.13	25
Zinc (TR)	<.01-.02	.02-.08	<.01-.47	25

* Range adjusted to exclude suspected outliers. Criteria used for identifying suspected outliers include measurable dissolved concentrations yet the pH is alkaline; dissolved concentrations higher than total recoverable concentrations; and one or two abnormally high dissolved values mixed with 40 below detection limit values.

Table 12 Downgradient Wepo and Alluvial Well Chemistry vs Livestock Standards

Analyte	Standard	No. Sites	Sites	Frequency	Exceedence Date Range		Exceedence Value Range	Exceedence Median
Aluminum, Dissolved	0.0000 - 5.0000	1	ALUV199	1/0/0/37	07/03/2001-07/03/2001		8.2000 - 8.2000	8.2000
Arsenic, Dissolved	0.0000 - 200.0000	0	none					
Boron, Dissolved	0.0000 - 5000.0000	0	none					
Cadmium, Dissolved	0.0000 - 50.0000	7	ALUV180	0/0/1/37	08/25/1997-08/25/1997	(<)	200.0000 - 200.0000	200.0000
			ALUV181	0/0/1/36	08/25/1997-08/25/1997	(<)	200.0000 - 200.0000	200.0000
			ALUV182	0/0/1/37	03/24/1998-03/24/1998	(<)	60.0000 - 60.0000	60.0000
			ALUV19	0/0/1/53	01/27/1998-01/27/1998	(<)	80.0000 - 80.0000	80.0000
			ALUV193	0/0/1/38	11/03/1997-11/03/1997	(<)	400.0000 - 400.0000	400.0000
			ALUV197	0/0/2/38	11/03/1997-01/27/1998	(<)	80.0000 - 400.0000	240.0000
			ALUV29	0/0/1/37	07/30/1999-07/30/1999	(<)	80.0000 - 80.0000	80.0000
Chromium, Dissolved	0.0000 - 1000.0000	0	none					
Copper, Dissolved	0.0000 - 500.0000	0	none					
Fluoride	0.0000 - 2.0000	10	ALUV169	1/0/0/36	11/20/1997-11/20/1997		4.5000 - 4.5000	4.5000
			ALUV199	25/5/0/37	09/23/1993-07/29/2002		2.1000 - 7.8100	3.1000
						(B)	3.0000 - 5.0000	4.0000
			WEPO178	1/0/0/22	09/20/1993-09/20/1993		2.5000 - 2.5000	2.5000
			WEPO40	20/0/0/22	04/23/1986-10/14/2002		6.6000 - 12.0000	9.0500
			WEPO42	13/0/0/22	08/26/1987-07/24/2002		2.1000 - 2.4000	2.1000
			WEPO44	20/0/0/21	06/13/1986-05/20/2002		8.9000 - 22.7000	12.0000
			WEPO45	20/0/0/20	06/11/1986-05/13/2002		5.2000 - 8.1000	7.0000
			WEPO46	2/0/0/20	04/20/1990-05/14/2002		2.2000 - 2.7000	2.4500
			WEPO55	18/0/0/19	04/16/1986-05/13/2002		6.1000 - 10.3000	7.4000
			WEPO56	1/0/0/20	12/03/1986-12/03/1986		6.3000 - 6.3000	6.3000
Lead, Dissolved	0.0000 - 100.0000	20	ALUV169	0/0/10/36	11/20/1997-10/16/2002	(<)	200.0000 - 200.0000	200.0000
			ALUV170	0/1/17/39	04/21/1997-08/06/2002	(B)	200.0000 - 200.0000	200.0000
						(<)	200.0000 - 200.0000	200.0000
			ALUV180	0/0/18/37	03/07/1997-05/20/2002	(<)	200.0000 - 2000.0000	200.0000
			ALUV181	0/0/18/36	03/07/1997-08/17/2001	(<)	200.0000 - 2000.0000	200.0000
			ALUV182	0/0/16/37	03/07/1997-11/20/2002	(<)	200.0000 - 800.0000	200.0000
			ALUV19	0/0/11/52	02/04/1997-10/05/2000	(<)	200.0000 - 200.0000	200.0000
			ALUV193	0/0/16/38	11/03/1997-07/31/2002	(<)	200.0000 - 1000.0000	200.0000
			ALUV197	0/0/17/38	08/26/1997-07/26/2002	(<)	200.0000 - 1000.0000	200.0000
			ALUV199	0/0/17/37	07/16/1996-07/29/2002	(<)	200.0000 - 200.0000	200.0000
			ALUV27R	0/0/16/50	11/06/1997-07/29/2002	(<)	200.0000 - 200.0000	200.0000
			ALUV29	0/0/6/36	07/08/1997-04/29/2002	(<)	200.0000 - 1000.0000	200.0000
			ALUV33R	1/0/13/47	09/14/1990-04/17/2001		120.0000 - 120.0000	120.0000
						(<)	200.0000 - 200.0000	200.0000
			ALUV80R	0/0/2/51	07/19/1999-07/31/2002	(<)	200.0000 - 400.0000	300.0000
			ALUV82	1/0/17/46	09/13/1990-04/24/2001		160.0000 - 160.0000	160.0000
						(<)	200.0000 - 200.0000	200.0000
			ALUV83	0/0/21/55	07/16/1996-07/31/2002	(<)	200.0000 - 200.0000	200.0000
			ALUV88	1/0/16/51	09/11/1990-04/19/2001		120.0000 - 120.0000	120.0000
						(<)	200.0000 - 200.0000	200.0000
			ALUV89R	0/0/15/49	03/02/1998-07/25/2002	(<)	200.0000 - 200.0000	200.0000

Table 12 (cont.) Downgradient Wepo and Alluvial Well Chemistry vs Livestock Standards

Analyte	Standard	No. Sites	Sites	Frequency	Exceedence Date Range		Exceedence Value Range	Exceedence Median
Lead, Dissolved	0.0000 - 100.0000	20 (Cont.)	WEPO178	0/0/8/22	03/10/1997-05/20/2002	(<)	200.0000 - 200.0000	200.0000
			WEPO46	0/0/2/20	09/18/1997-11/06/1997	(<)	200.0000 - 200.0000	200.0000
			WEPO55	0/0/1/19	08/26/1997-08/26/1997	(<)	200.0000 - 200.0000	200.0000
Nitrate Nitrogen_N	0.0000 - 100.0000	1	WEPO46	2/0/0/20	09/18/1997-11/06/1997		202.0000 - 269.0000	235.5000
Nitrite Nitrogen_N	0.0000 - 10.0000	0	none					
Ph At 25 Deg. Cent.	6.5000 - 9.0000	1	ALUV199	6/0/0/37	05/22/1995-07/29/2002		6.0000 - 6.4200	6.1000
Selenium, Dissolved	0.0000 - 50.0000	1	WEPO46	10/0/0/20	06/24/1986-08/20/1998		51.0000 - 560.0000	187.5000
Solids, Dissolved	0.0000 - 6999.0000	7	ALUV170	13/0/0/39	03/18/1993-04/17/2001		7010.0000 - 9540.0000	7470.0000
			ALUV197	11/0/0/38	08/28/1998-07/26/2002		7040.0000 - 7730.0000	7300.0000
			ALUV199	35/0/0/37	12/17/1992-07/29/2002		7270.0000 - 9692.0000	8500.0000
			ALUV29	4/0/0/37	10/25/1993-04/29/2002		7070.0000 - 8030.0000	7445.0000
			ALUV82	32/0/0/47	10/23/1987-01/19/2001		7010.0000 - 8340.0000	7423.0000
			ALUV83	27/0/0/56	10/23/1987-07/31/2002		7002.0000 - 7670.0000	7236.0000
			WEPO46	2/0/0/50	09/18/1997-11/06/1997		7840.0000 - 8010.0000	7925.0000
Total Recoverable Hg	0.0000 - 10.0000	0	none					
Vanadium, Dissolved	0.0000 - 100.0000	2	WEPO40	0/0/1/21	04/23/1986-04/23/1986	(<)	500.0000 - 500.0000	500.0000
			WEPO55	0/0/1/19	04/16/1986-04/16/1986	(<)	500.0000 - 500.0000	500.0000
Zinc, Dissolved	0.0000 - 25.0000	7	ALUV19	0/0/1/52	01/12/2000-01/12/2000	(<)	50.0000 - 50.0000	50.0000
			ALUV199	0/0/1/37	01/12/2000-01/12/2000	(<)	50.0000 - 50.0000	50.0000
			ALUV33R	0/0/1/47	01/18/2000-01/18/2000	(<)	50.0000 - 50.0000	50.0000
			ALUV82	0/0/1/46	01/14/2000-01/14/2000	(<)	50.0000 - 50.0000	50.0000
			ALUV83	0/0/1/55	01/14/2000-01/14/2000	(<)	50.0000 - 50.0000	50.0000
			ALUV88	0/0/1/51	01/13/2000-01/13/2000	(<)	50.0000 - 50.0000	50.0000
			ALUV89R	0/0/1/49	01/13/2000-01/13/2000	(<)	50.0000 - 50.0000	50.0000

Frequency = uncensored/between MDL&PQL/censored/no. samples, (B) = Between MDL&PQL range, (<) = Censored range

Revised 11/21/03

skeletal damage in dogs, sheep and cattle. Elevated NO3 levels can lead to methemoglobinemia and impaired liver function, whereas elevated Se can cause white muscle disease in livestock. Principal constituents in the alluvial aquifer that preclude livestock use are F, pH, TDS and Pb. Consumption of lead in concentrations above 0.1 mg/l by animals can result in deleterious affects on the central nervous system and to the animal's principal organs, as heavy metals tend to concentrate in these parts of the animals. Livestock ingestion of water with high TDS concentrations (above 7000 mg/l) can cause diarrhea, rundown ragged appearances, weakening and eventually even death. Those portions of the Wepo and alluvial aquifers potentially affected by pit interception of the Wepo aquifer do not appear to be significantly affected as eight of the twelve Wepo wells and three of the 22 alluvial wells have always had unsuitable livestock water use potential owing to fluoride. Also, six of the 22 alluvial wells and one of the twelve Wepo wells have always had high TDS levels.

In summary, increases in concentrations of Ca, Mg, Na, SO_4 and HCO_3 and TDS will occur regardless of the nature of the spoil material placed in the saturated zone. The potential for acid formation and acid and trace metal migration is minimal, because of the overall buffering capacity of the overburden material. There will be some amount of additional TDS increases as a result of the neutralization of acid forming material placed in the saturated zones. Acid formation will occur primarily in response to oxidation of sulfides in advance of the wetting front during spoil resaturation. Reduction of sulfates will primarily occur following resaturation. Based on climatic conditions and the transmissivities of the material, resaturation and reestablishment of premining ground water flow gradients could take 10 years or more. The magnitude of the impact to either aquifer should be limited to the immediate pit areas, because gradients and transmissivities are very low.

The overall significance of this impact is minor. There are no present water users of the Wepo aquifer within the leasehold. In fact, only two wells (4K-389 and 4T-405) in the region are reported to be completed only in the Wepo aquifer (see Chapter 17). An inspection of the lithologic log for one of the wells suggests that it is actually completed in the upper member of the Toreva (155 feet of sandstone at the bottom of the well). No log could be found for the other well. Local wells are not completed in the Wepo aquifer for two reasons; (1) the yields are too low, and (2) the quality of the water is may be unsuitable for domestic or livestock purposes

<u>Interception of Wepo Recharge to the Alluvial Aquifer by Pits</u>. Based on Drawing No. 85610, Wepo Water Level Contour Map, ground-water flow is from the Wepo aquifer to the

alluvial aquifer system. Pit interception of portions of the Wepo aquifer in the N-10, N-11, N-6, J-16, J-19/20 and J-21 pits can potentially cause local decline in the alluvial aquifer system. Distance drawdown projections for the combined pit pumpage (Figure 1 and Table 8) suggest portions of the alluvial aquifer system (Reed Valley, Red Peak Valley, Upper Moenkopi and Dinnebito alluvial aquifers) could potentially be affected to the extent that drawdowns exceed natural water level fluctuations.

It is difficult to predict the magnitude of the drawdowns as the alluvial aquifers have a large range of transmissivities and storage coefficients. Comparing this situation to the N-7/8 pit pumpage effects on the Yellow Water Canyon alluvial aquifer (Alluvial Well 74 and 75), it is estimated that drawdowns in the alluvial aquifer near the N-14, J-16 and J-19/20 pit areas could range from 8 to 20 feet during the period of maximum combined pit interception (1980 to 1983). Also, drawing on what was experienced at the N-7/8 pit, the alluvial aquifer drawdowns should be quite localized and limited in extent (less than one mile downgradient). These impacts should be partially offset by recharge to the aquifers from water impounded in Reed Valley, N-14D, N-14E, N-14F and J-16A dams. The significance of this impact is minimal because of the limited portions of the alluvial aquifer system affected and the absence of local use of the alluvial aquifer. As with the Wepo aquifer, the alluvial aquifer is low yielding throughout most of the leasehold and the quality is not suitable for domestic purposes and is marginal to unsuitable for livestock use. Therefore, water from the alluvium does not support the pre- or postming land use nor does it support any critical habitats or plant species (see Chapters 9 and 10).

<u>Interception of Channel Runoff Recharge to Alluvial Aquifers by Dams and Sediment Ponds</u>. Dams, sediment ponds and internal permanent impoundments will intercept the runoff from about 29 and 12 percent, respectively, of the Moenkopi and Dinnebito watersheds to the <u>down drainage lease boundaries</u>. These structures will remove some potential channel bottom transmission loss recharge to the alluvial aquifers downstream from the structures. Downstream aquifer recharge impacts associated with the dams should be offset by the impounded water recharge to the alluvial aquifer. The alluvial aquifer water level monitoring program indicates that the impact of the structures on alluvial water levels is insignificant. There is no evidence suggesting gradual water level declines in the alluvial aquifer system over time (see Chapter 15).

<u>Truncation of Portions of the Alluvial Aquifers by Dams</u>. Eight large dams have been constructed such that the embankments cut through the entire thickness of alluvium to bedrock. The embankments are designed and constructed to be impervious. These structures impact the alluvial aquifer system by disrupting the ground-water flow. A review of the

five-year alluvial ground-water level hydrographs (Chapter 15) indicates that these impacts are of no significance probably owing to the following reasons. All dams, with the exception of J-7 Dam are on small tributaries, which only contribute minimal amounts of water to the alluvial ground-water system. Seepage occurs around J-7 Dam along sandstone bedding planes. The Wepo aquifer discharges to the alluvial aquifer all along the channel reaches. Any localized ground-water flow disruptions would be offset within short distances below the dams.

Effects of Altered Wepo Aquifer Water Quality on Alluvial Aquifer Water Quality. The effects of higher TDS water from resaturated spoil in the Wepo aquifer recharging the alluvial aquifer are expected to be minimal. The pits will require anywhere from several years to 100 years to resaturate and reestablish ground-water flow gradients because of limited precipitation recharge and very low Wepo ground-water flow rates. These same low transmissivities will continue to limit the Wepo feed and contaminant transport into the alluvial aquifer. In contrast, responses to snowmelt and rainfall runoff recharge are rapid and greater than Wepo feed during three seasons of the year. The potential for rapid dilution of elevated TDS inputs from the Wepo would be quite high during these significant recharge periods.

The significance will be minimal because, the alluvial aquifer water within the leasehold is unsuitable for domestic purposes and marginal to unsuitable for livestock drinking water. Water from the alluvial aquifer is not essential to support the postmining land use or critical habitats or plant species.

Mining Interruption of Spring Flow. To date, only eight natural and one artificial spring of any significance (more than just a damp spot along the side of a channel) have been identified and monitored within and immediately adjacent to the leasehold. Of these, one spring (Monitoring Site #97) at the northwest edge of N-14 has been removed by mining activities (N-14 channel realignment). Reference to the statistical water quality summary for springs in Chapter 15, Hydrologic Description, indicates that the water quality of the spring was unsuitable for livestock use. Those parameters and parameter concentrations above the livestock drinking water limits are presented in Table 13. Peabody has provided two alternate water supplies for this spring: (1) water impounded in the N14-D dam; and (2) two public water outlets on the leasehold. The alternate water supplied is greater in quantity and better in quality than the spring water. The water supplied at the public water outlets meets domestic drinking water requirements. No other springs are expected to be impacted by the proposed mining.

TABLE 13

Chemical Parameters and Concentrations at Spring 97

Which Exceed Livestock Drinking Water Limits

Parameter	Mean Concentration (mg/l)	Recommended Livestock Limits[1] (mg/l)
Fluoride	2.1	2.0
Lead	0.167	0.1
Total Dissolved Solids	6846[2]	6999

(1) Limits are based on Navajo Nation (1999), Hopi Tribe (1998), National Academy of Science (1974), and USEPA (1996).

(2) One of four TDS values was greater than 6999 mg/l.

Impact of Peabody Wellfield Pumpage on Regional Water Levels and Stream and Spring Flows. Peabody operates a wellfield consisting of eight wells completed in the D aquifer and N aquifer (Navajo Sandstone, Kayenta Sandstone, and Wingate Sandstone) to provide water for the coal slurry pipeline serving the Mohave Generating Station and for other operational uses. Pumpage was initiated in 1969 and has averaged about 3,900 acre-feet per year (1969-2000).

The pumping of water from the N aquifer by Peabody since 1969 has produced one of the longest term pumping tests ever. Water-level changes have been measured in wells at considerable distances and in several directions from the PWCC wellfield. The rates of pumping at the well field have been measured throughout the period of pumping. The result is a data set which, if properly evaluated, provides considerable information about the aquifer, and about the response of the aquifer to pumping. These measurements also provide information with which to estimate the effects of future water use. It is important to use appropriate tools to interpret this information. The analytical models, such as the Theis, Cooper-Jacob, Hantush, or other solutions of the flow equations, while appropriate for short-term tests, are commonly not suitable for longer tests because many of their simplifications affect long-term results. Material properties can vary over reasonably short distances, and boundaries can affect aquifer responses to pumping. Therefore, numerical models are better tools with which to properly interpret these long-term pumping tests, and to predict the effects of future pumping. In short, monitoring the effects of past water use provides information with which to predict future effects. This approach was first applied in the Black Mesa area in 1985 and 1987 by the USGS, through the development of a ground water flow model of the N aquifer beneath and surrounding the Black Mesa basin, and use of the calibrated model to predict the effects of future pumping. In 1998, consultants for Peabody started development and calibration of a more realistic, three-dimensional model of the aquifer and incorporating more recently collected information; this improved model is used to predict the effects of N aquifer water use by Peabody.

The following analysis of the effects of Peabody's pumping of the N Aquifer is based on data measured before and during the period of pumping, and on models based on these data. It considers the effect of pumping on drawdown at existing locations of groundwater use, groundwater discharge at springs and to streams, the structural integrity of the N aquifer, and water quality of the N aquifer that might be affected by increased leakance of water through the overlying Carmel.

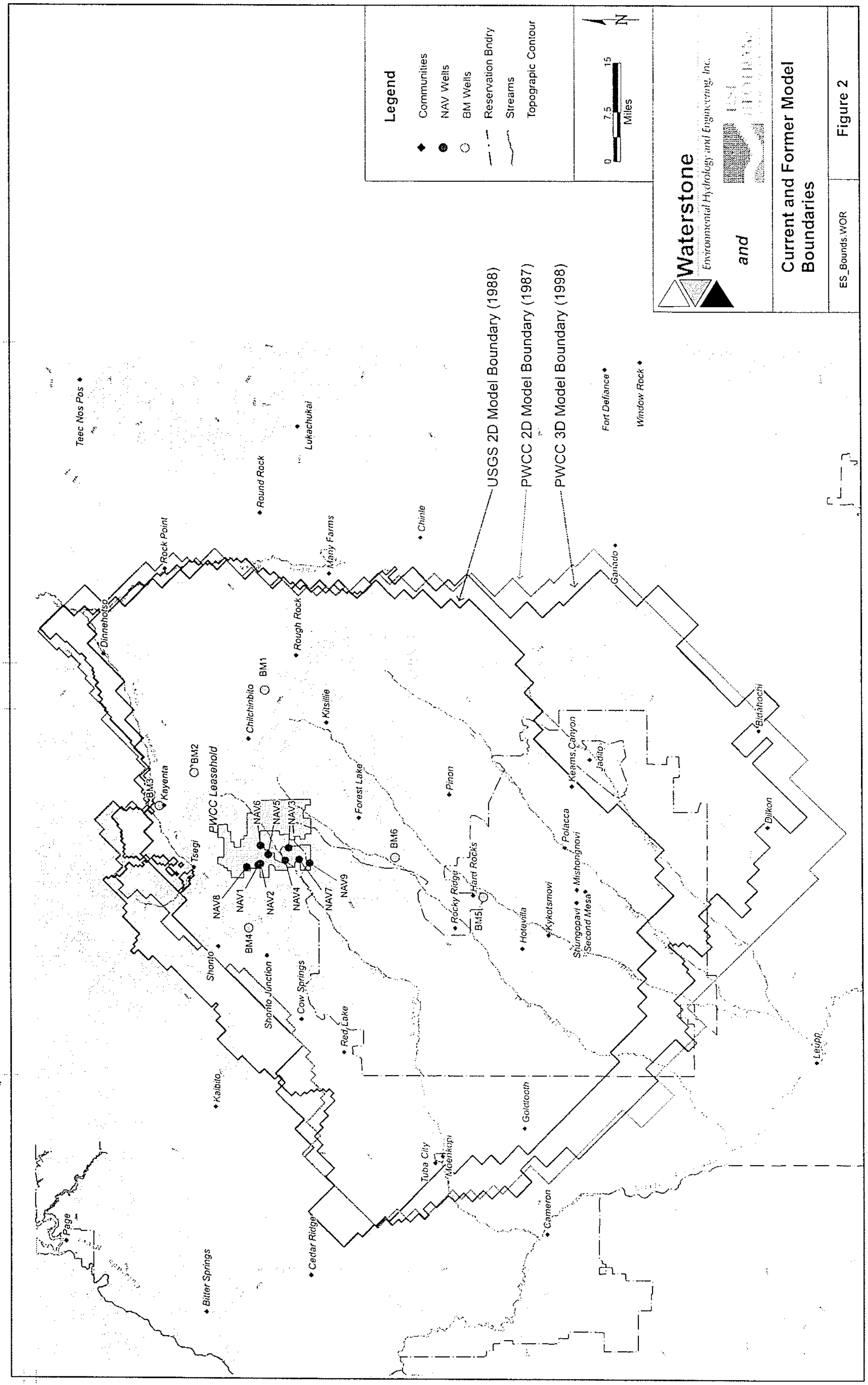
Numerical Modeling. Several numerical models have been developed to estimate the impacts of pumping by Peabody and the tribal communities on the N Aquifer, beginning in 1983 (Eychaner, 1983). Most recently, Peabody has developed a model that includes the overlying D Aquifer (PWCC, 1999). The D Aquifer is also used as a water resource, but to a much lesser extent than the N Aquifer. These models are the best tools available for determining the individual contribution of each pumping stress on the observed or measured effects (i.e., water levels and stream flows). The models are not of sufficient resolution to simulate flow at individual springs, but can be used to make intelligent observations of regional spring flow. Each model includes:

- Development of a basic description of the real system, including geologic controls on material properties (i.e., geometry of the rock layers, deformation of the rocks, etc.), areas and amounts of recharge and discharge, and distribution of water levels.
- Formulation of a mathematical description of the system to be modeled. This formulation is based on
  - o Darcy's Law - a mathematical expression that relates the rate of groundwater flow to observable differences in water levels.
  - o Mass balance - a mathematical expression of conservation of mass. For a groundwater-flow system, this means that flow into the system (recharge) must equal flow out of the system (pumping or discharge to streams or springs) plus the change in the amount of water held or released from storage as water levels change.
  - o Boundary conditions - mathematical statements of various conditions that exist on the boundaries of the modeled system. These require knowledge of the geometry of the rock formations and the processes and locations through which water enters and exits the system.
  - o Initial conditions - description of the water levels everywhere in the system at the beginning of the modeled time period.
- Development of a set of numerical values for all parameters appearing in the mathematical formulation. These include hydraulic conductivity, specific storage, and specific yield, all of which may be spatially variable.
- Application of a numerical algorithm that "solves" the mathematical formulation for different applied stresses. The algorithm calculates the spatial and temporal distribution of water levels and groundwater flow rates that satisfy the mathematical model for different pumping rates, recharge rates, etc.

Each model is put through a calibration process whereby model parameters are adjusted by either manual or automated methods until simulated results reasonably match measurements. This usually means matching historic water-level measurements at wells against model output. The model parameters adjusted towards calibration are typically flow and storage properties of the geologic material. They are adjusted within ranges reported in the scientific literature for the specific rock type. Boundary conditions such as recharge may also be adjusted if calibration can not be achieved with the independently derived estimates. The geometry of the flow system is typically held fixed during this process. Calibration can be performed for non-pumping (steady state) and pumping (transient) conditions whereby a single set of flow properties is derived to match water levels representing both conditions.

Each of the groundwater models developed for the Black Mesa area was based on the USGS's finite-difference computer codes, covered most or all of the Black Mesa Basin, included all known pumping stresses and was calibrated under non-pumping and pumping conditions using the USGS's six monitoring wells as key calibration targets.

The first model developed was by Eychaner of the USGS in 1983 using the computer code written by Trescott and others (1976) which simulates flow in one or two dimensions. The N Aquifer was simulated as a single flow unit (using one model layer) in two-dimensional space and the transient calibration period was from 1965 through 1977. Soon thereafter, the USGS (Brown and Eychaner, 1988) updated this model using a refined grid spacing and the USGS modeling code MODFLOW (McDonald and Harbaugh, 1988). After calibration, this model was used to predict the effects of pumpage on the flow system. Concurrently, an independent modeling effort was begun by Peabody. Both groups chose to simulate the flow system in two dimensions and to represent the N Aquifer as one flow unit. This was reasonable given the N Aquifer's large regional-scale, relatively uniform and continuous nature, and its predominantly horizontal groundwater flow. The USGS (Brown and Eychaner, 1988) kept their same model boundaries while Peabody's version (GeoTrans, 1987) extended the boundaries to cover more of Black Mesa basin (Figure 2), particularly to the southeast. This difference in extent reflects the fact that the USGS only includes the Navajo Sandstone in the southwestern part of the model, while the Peabody model also includes the Kayenta and Wingate. The USGS extended their calibration period from 1965 through 1984, whereas the Peabody model was calibrated from 1956 through 1985. The earlier time period (pre-1965) was chosen to simulate community pumping at Kayenta. The Peabody model also increased the number of transient calibration-target locations from six to nine. Although the two models were developed independently and differed from each other in details, their results were similar.



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In 1993, under the auspices of the coal leases and authorized by the Secretary of Interior, the Hopi Tribe, the Navajo Nation, and Peabody funded a study by S. S. Papadopulos & Associates, Inc. (SSPA) that reviewed the USGS model (Brown and Eychaner, 1988) for appropriateness, application, calibration, and results. They concluded that:

> "A mathematical model, such as the one constructed by the USGS, is the only method available for separately determining impacts of Peabody's pumping on groundwater conditions in the N-Aquifer. This method insures consistency among the different facets of the hydrogeologic system such as recharge, discharge, ground-water levels, pumping, aquifer properties and aquifer extent. Consequently, the method used in the USGS studies is a standard method and is clearly appropriate for the purpose of evaluating impacts due to pumping by Peabody."

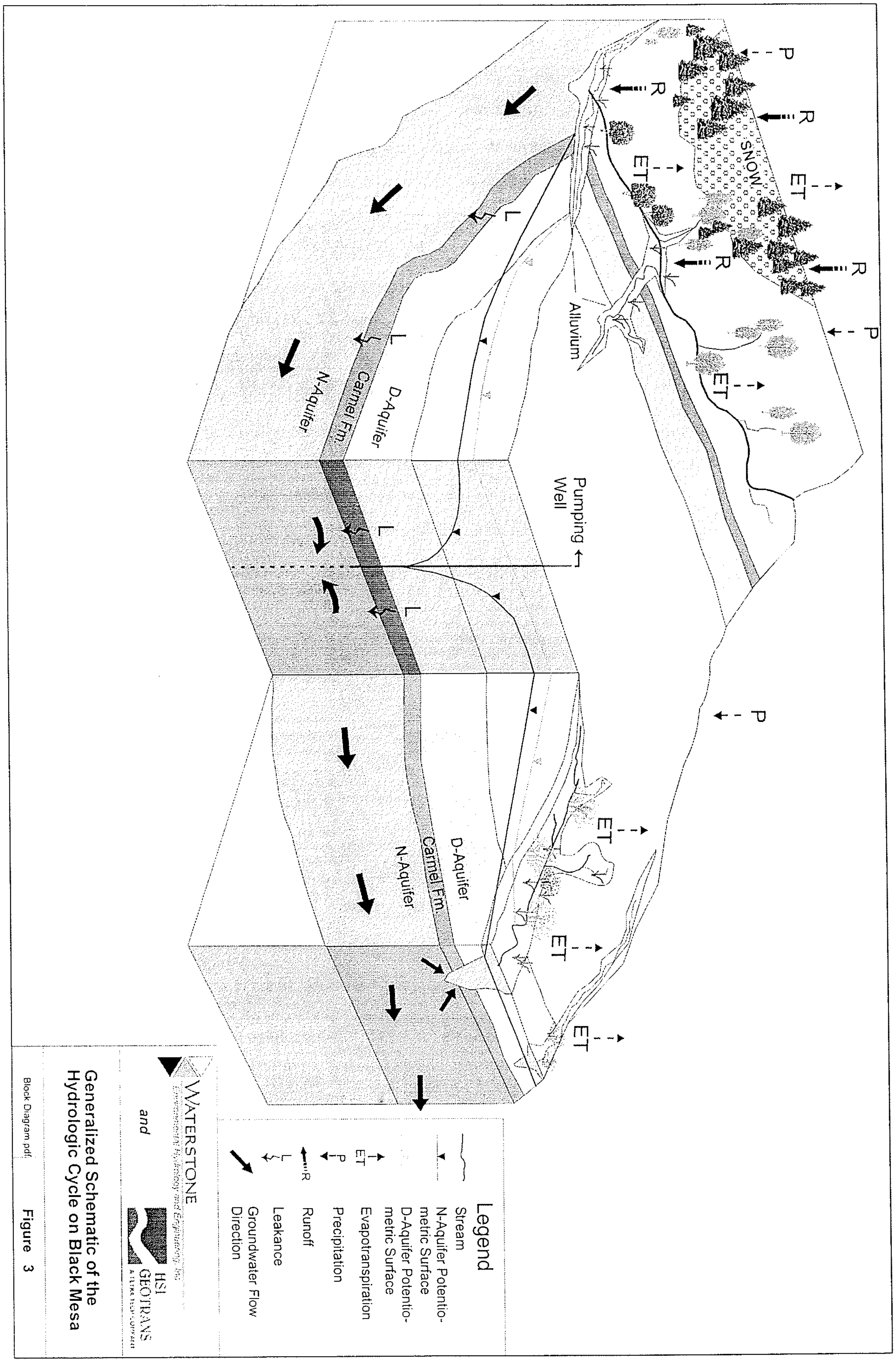
They further concluded that application of the modeling method was appropriate, the calibration was reasonable, and that Peabody's impacts on surface-water features (such as streams and springs) was minimal because water was predominantly coming from aquifer storage. An important contribution of SSPA (1993) towards the understanding of the N Aquifer system was the development of a database for the N aquifer that was more comprehensive than the one used by Brown and Eychaner (1988).

Following the completion of the SSPA report and because of the concern expressed by some of the participants in the co-operative study, the Secretary of Interior requested that the USGS conduct a fresh review of the Brown and Eychaner report. The reviewer from the USGS believed that Brown and Eychaner did not sufficiently document the water budget estimates, and that the model was less credible as a result. To evaluate this concern, Peabody initiated a modeling study in which the sensitivity of modeling predictions to the estimated recharge rate was tested (Peabody, 1994). First, this study compared the USGS and Peabody models. Second, it documented the conversion of both the USGS and Peabody models to an automated calibration modeling code, MODFLOWP (Hill, 1991). The converted USGS model was then successfully re-calibrated for pre-pumping conditions using recharge values that were 0.5, 0.75, 1.5, and 2.0 times the base recharge value used by Brown and Eychaner (1988) of 13,380 ac-ft/yr. The best agreement with water-level data occurred with the base-case recharge value. Pumping simulations were then performed, and results indicated the importance of calibrating the model to both pre-pumping and pumping conditions. MODFLOWP did not have this capability. Similar tests were performed with the Peabody model (1987), with similar results.

In 1999, Peabody substantially revised the modeling of the flow system by developing a three-dimensional representation of the N and D Aquifers, separated by the intervening low-permeability Carmel Formation (Figure 3). The revised three-dimensional model, including detailed discussions on the model database, conceptual model, geology, hydrogeology, numerical modeling, and future pumping effects, are presented in the report entitled "A Three-Dimensional Flow Model of the D and N Aquifers, Black Mesa Basin, Arizona" (PWCC, 1999). Each of the seven rock units that comprise the D and N Aquifers are explicitly included in this 7-layer model. Moreover, the changes in material properties caused by different depositional environments within the rock units are incorporated, allowing their properties to be explicitly adjusted individually during the model-calibration process. In previous models, the model parameters represented a lumped average for the properties of several different formations. The calibration period was extended from 1956 through 1996 and the number of wells providing information on changes in water levels caused by pumping increased from nine to 47. This work was based on a database that included and went beyond the one compiled by SSPA (1993), in part, by adding information for the Carmel Formation and the D Aquifer, and including eleven additional years of pumping stresses, water-level measurements, and spring and streamflow measurements.

When the 3D model was developed, it was calibrated to both non-pumping (pre-1956) and pumping (1956 through 1996) conditions. Temporal changes in measured water levels were compared with changes in the simulated water levels. The calibration process relied more on data from wells BM-1, -2, -3, -4, -5 and -6 than from other wells, because (1) these wells were specifically chosen for monitoring the effects of pumping at the Peabody leasehold, (2) the higher quality and greater quantity of data from the BM-series, and (3) because detailed information on pumping of community wells was not available. The calibrated model provides good agreement with the measured changes in water levels for the BM-series wells.

An automated calibration process that used both pre-pumping and pumping datasets was used. This facilitated the development of multiple calibrated models, each one calibrated to different estimates of recharge or other model parameters. In 1997, Lopes and Hoffmann (1997) used geochemical data to estimate the recharge rate near Shonto. Their estimated rate was approximately one-half that proposed by Brown and Eychaner for this area. Using a larger geochemical data set and a numerical transport model, Zhu and others (1998) and Zhu (2000) showed that the geochemical data are consistent with the higher, earlier estimates of recharge rates based on hydrologic data. Still, uncertainty in recharge rates remain. To address this uncertainty, the model was calibrated twice, first using a



Generalized Schematic of the Hydrologic Cycle on Black Mesa

Figure 3

recharge value similar to Brown and Eychaner's and again, using a value similar to Lopes and Hoffmann's. In addition, two different approaches (full ET and low ET) to simulating discharge in non-wash settings were used, resulting in four calibrated models. These are termed FR/FET (full recharge and full ET), HR/FET (half recharge and full ET), FR/LET (full recharge and low ET), and HR/LET (half recharge, low ET). The use of different recharge estimates and different non-wash discharge approaches in the four calibrated models explicitly answers questions about the sensitivity of the models' predictions to uncertainty in these items. All four calibrated models matched measurements of drawdown better than the USGS's 2D model at all but one (BM-1) of the six USGS's monitoring wells (Figures 4 through 9). Each of the four models is shown on each figure. The figures show the change in water levels, measured from the last measurement. The plots portray drawdown in the usual way, such that lowering of water levels (increase in drawdown) is shown as the point moving toward the bottom of the plot with increasing time. It is the definition of zero change that differs. By definition, the change is zero for the last measurement, and for the last corresponding simulated value. Thus, for the purpose of determining whether the model is providing good agreement with the measured changes in water levels, it is important to evaluate the trends for early times.

These four models were used to estimate impacts of Peabody and tribal pumping on the D and N Aquifers. Unless otherwise indicated, the term "base-case model" refers to Peabody's FR/FET 3D model of the D and N aquifers using a recharge rate similar to that used by Eychaner (1983) and Brown and Eychaner (1988), and using MODFLOW's ET package to simulate discharge in the non-wash settings.

The 3D model was developed to improve the confidence in predictions of future effects of Peabody's pumping. The fact that the new model matched water-level information better than older models, while reassuring, does not necessarily mean that the predictions will be accurate. Earlier models produced reasonably good agreement with water-level change information available at the time of their calibration, but the agreement of measured and simulated water-level changes degraded with increasing time.

Calibration of the 3D model benefited from the collection of approximately eleven additional years of data since development of the earlier 2D models. These data provided additional, indirect information about the groundwater system through a model-development process. Groundwater models are widely acknowledged to be "non-unique". Different models (boundary conditions, geometries, material properties, solution techniques) can produce equally good agreement with available information. However, they <u>may</u> yield different results when used to make predictions. Thus, an important aspect of using models to guide

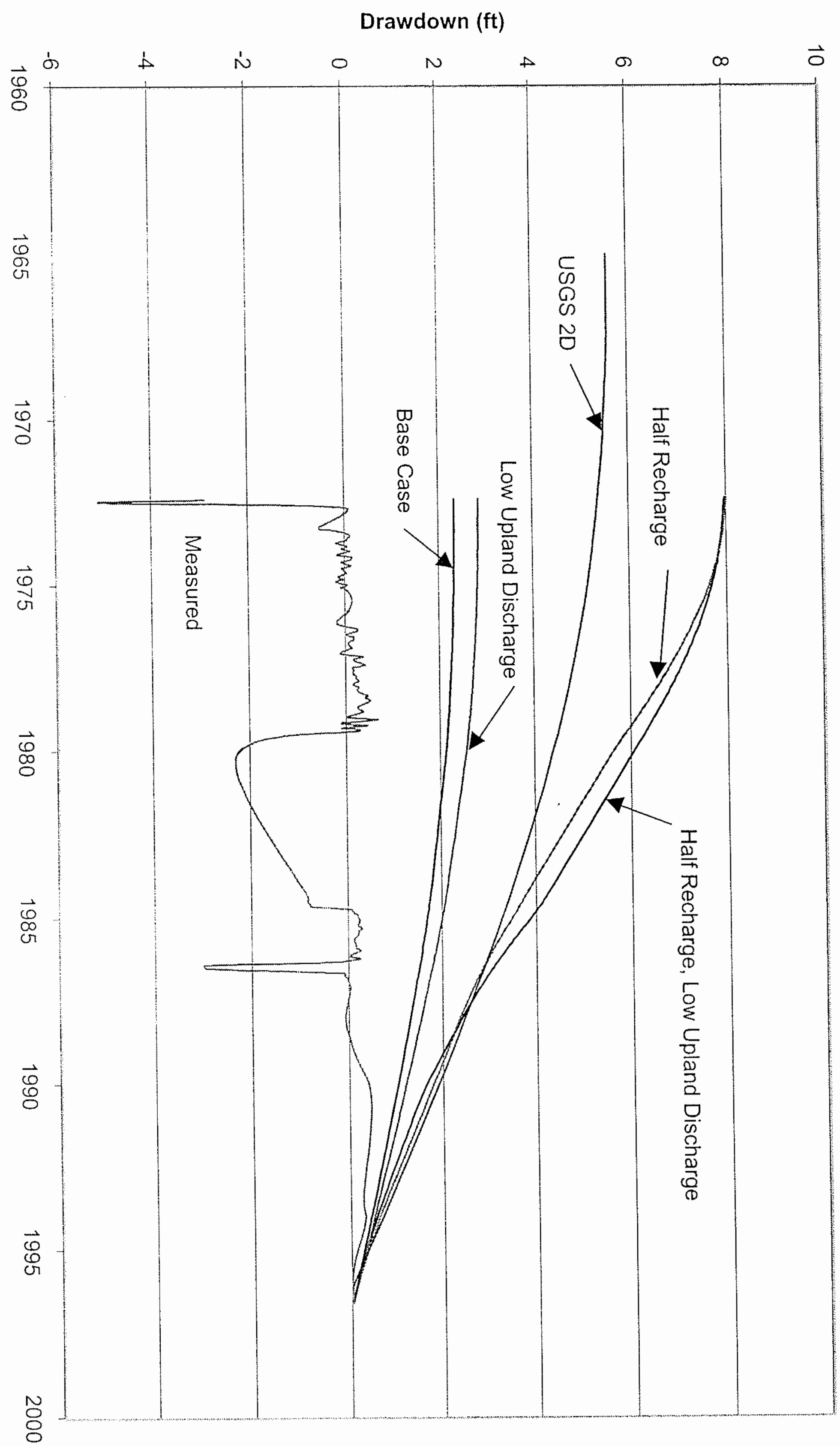


Figure 4. Measured vs. Simulated Water Levels Through 1996 - BM-1

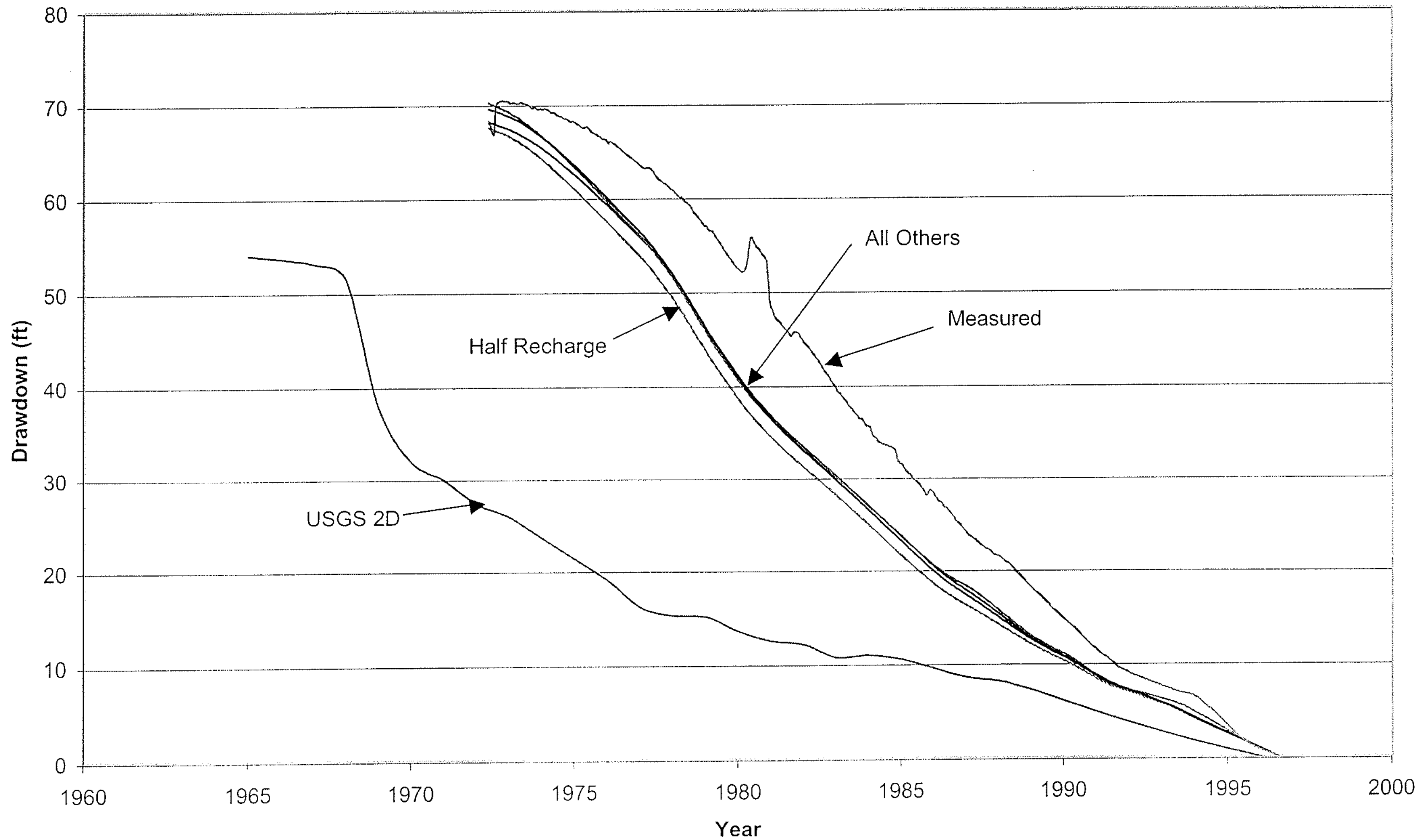


Figure 5. Measured vs. Simulated Water Levels Through 1996 - BM-2

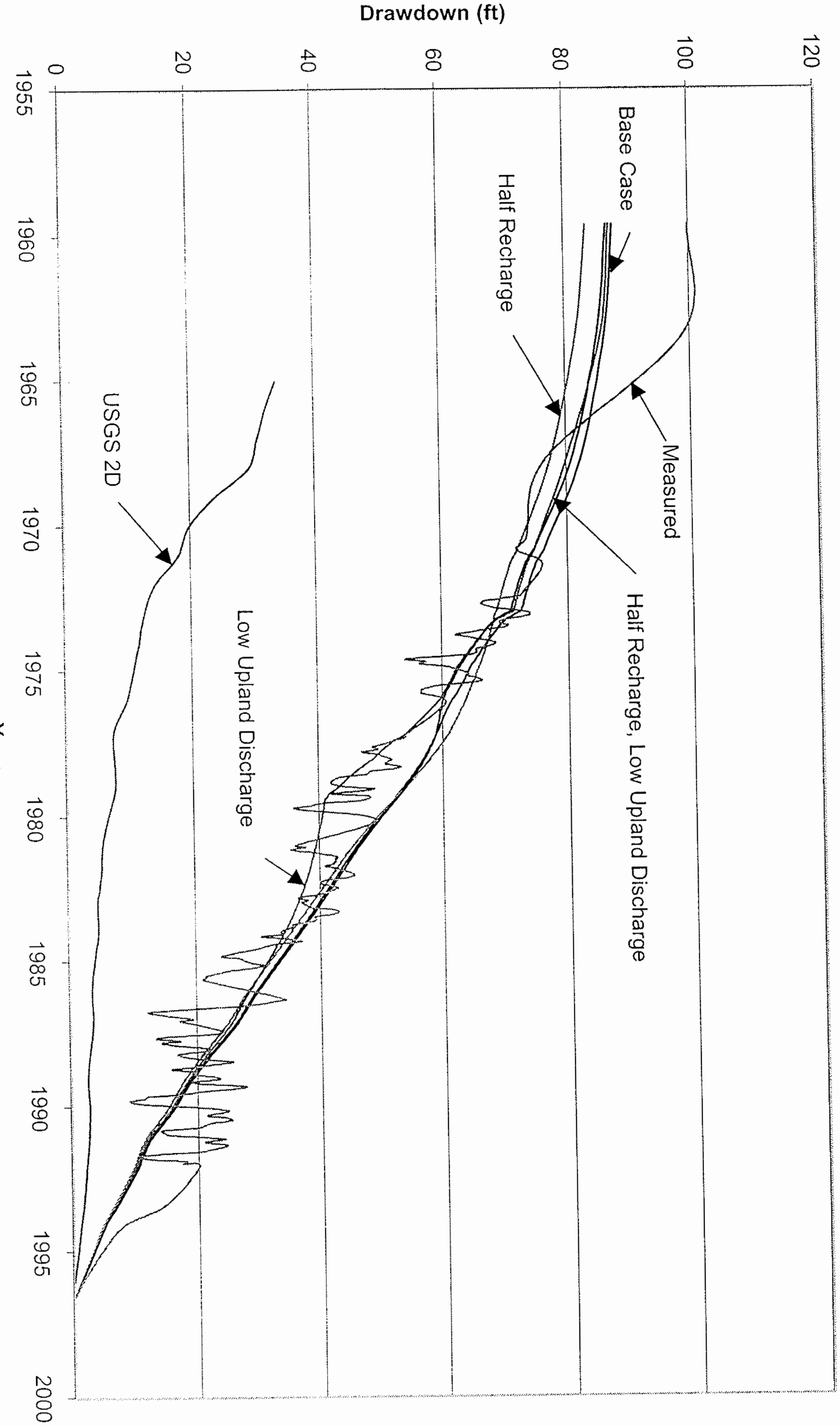


Figure 6. Measured vs. Simulated Water Levels Through 1996 - BM-3

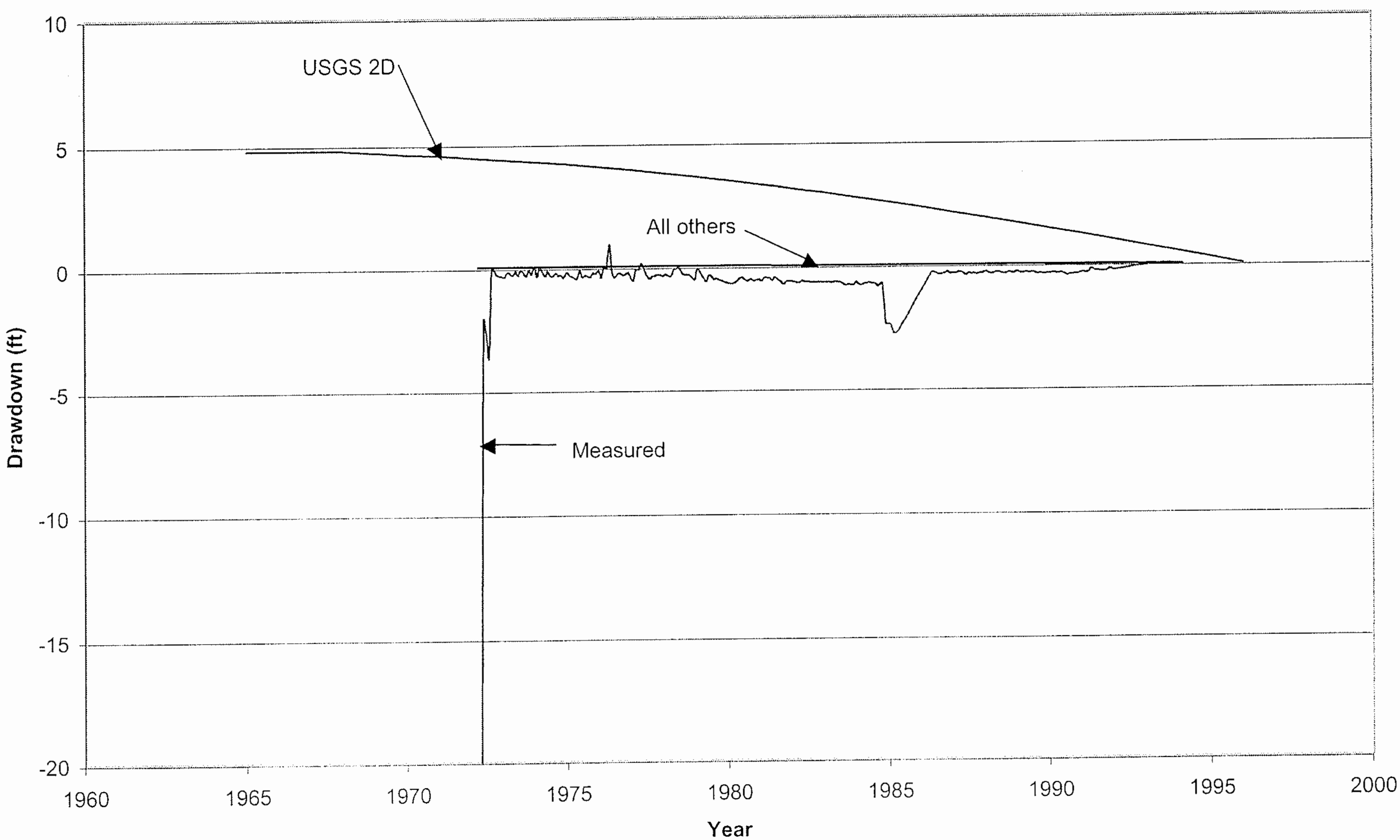


Figure 7. Measured vs. Simulated Water Levels Through 1996 - BM-4

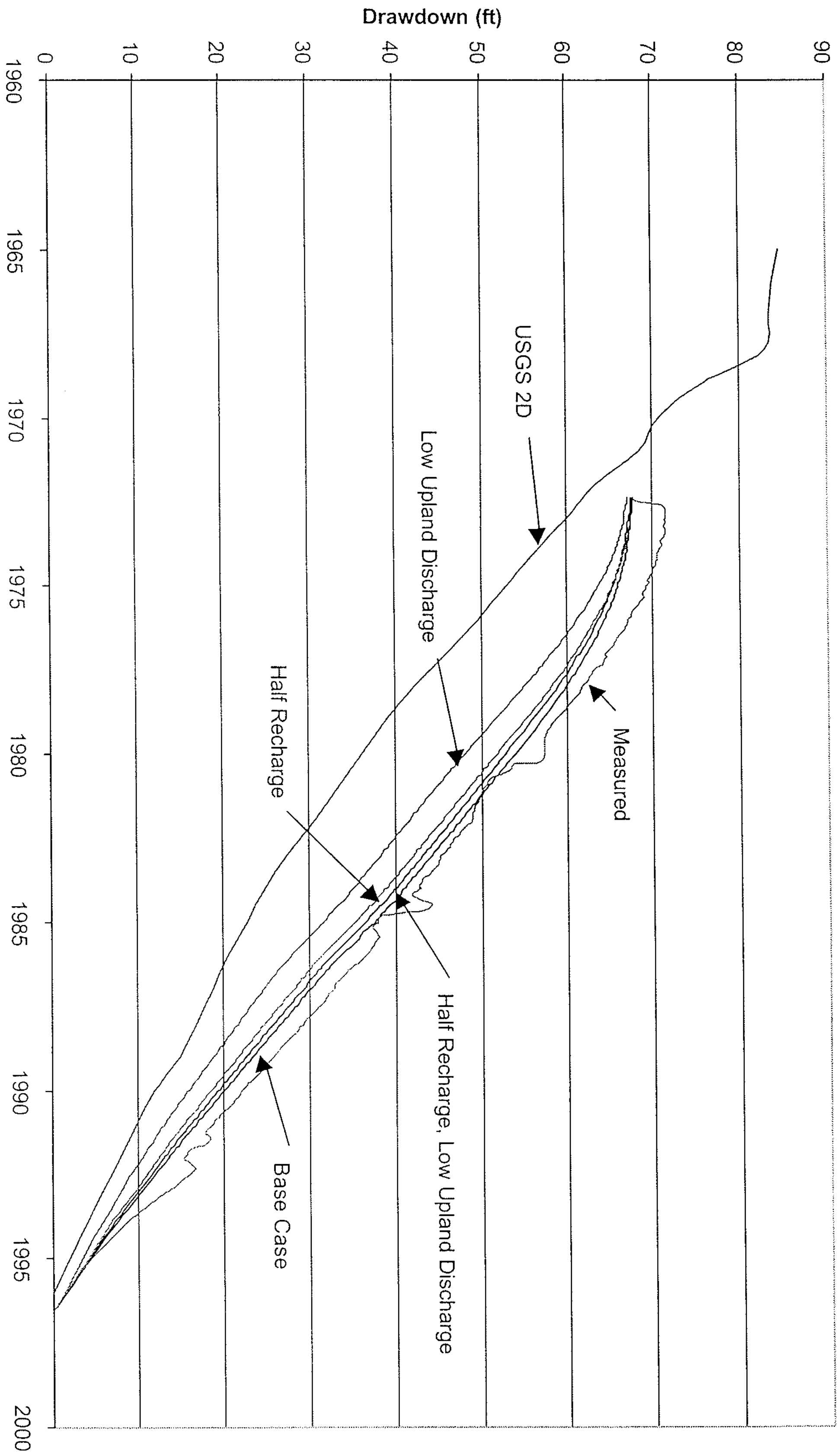


Figure 8. Measured vs. Simulated Water Levels Through 1996 - BM-5

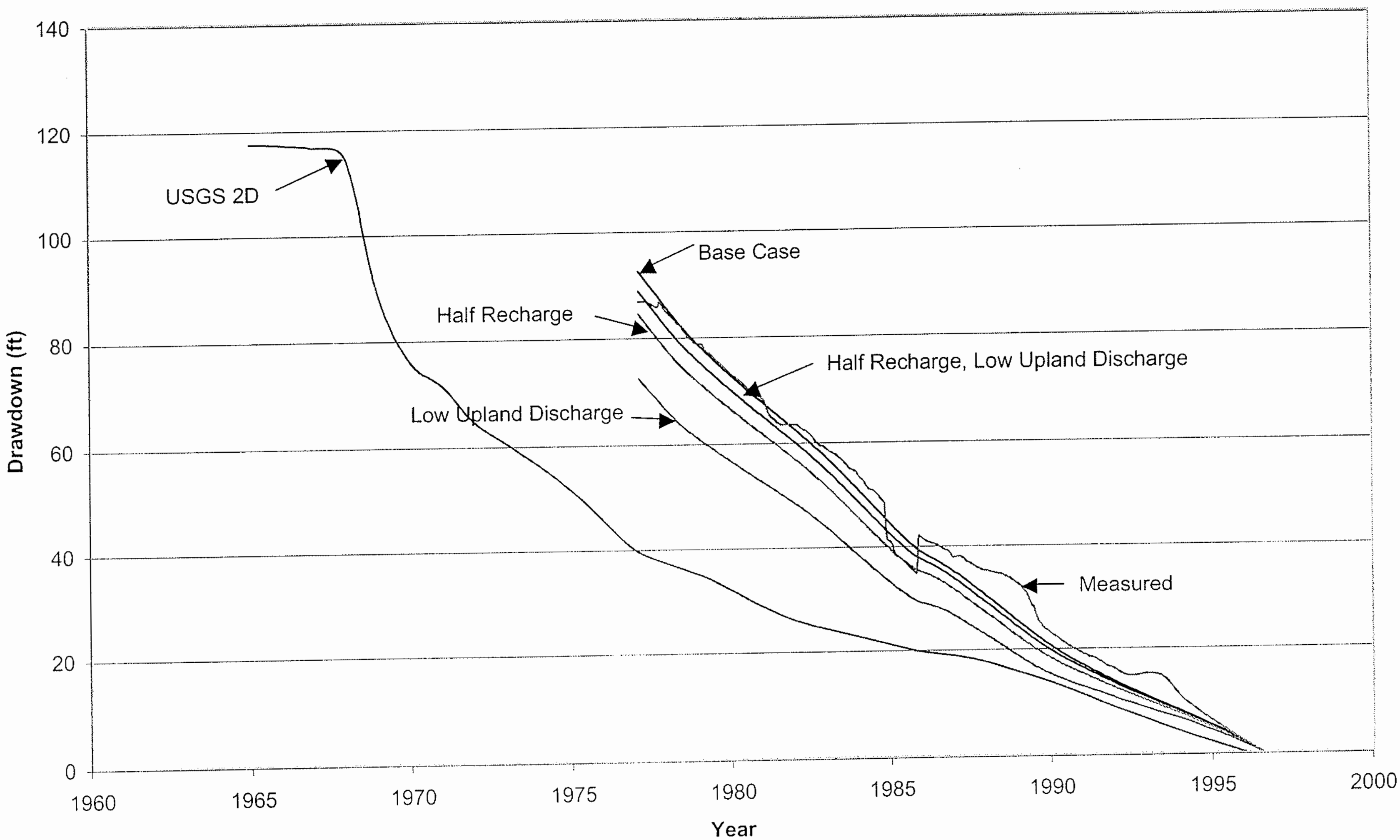


Figure 9. Measured vs. Simulated Water Levels Through 1996 - BM-6

resource management decisions is to evaluate whether the model results agree with data not used to calibrate the model, such as newly collected water-level data. If the agreement is good, confidence in the model's predictive ability is increased. However, if the agreement is poor, the need for additional calibration work is indicated.

The accuracy of the 3D model to simulate water-level changes beyond the calibration period was tested using data collected by several organizations from 1996 through 2000. Water-level data from the BM-series wells were obtained from the U.S. Geological Survey (Thomas, B., USGS, written communication, 2001) through the end of 2000. Pumping rates were also updated. Monthly pumpage data from each of the PWCC production wells were used in the simulations. For community pumping, information was obtained from the USGS, who compiles information provided by BIA and NTUA. These pumping data were added to the pumping data set.

Simulations were performed using four different models, as described in Peabody (1999). These four models, each individually calibrated, use two different recharge rates and two different upland (non-stream) discharge values simulated using different maximum ET rates. For the model validation tests, only the pumping rates for the period 1997 through 2000 were updated; no other changes were made.

Figures 10 through 15 provide comparisons of measured and simulated water-level changes through 2000, based on the updated pumping information, for the BM-series wells. At BM1, the agreements of the two models using the full recharge values are better than for the two models using half the full recharge values. The trend of the recent measurements suggests a lowering of water levels. Whether this is due to pumping, changes in the instrumentation, recent climate, or other cause is not clear. Earlier measurements do not suggest drawdown effects, but do indicate some variability. The four models simulate drawdown effects at BM1 and are thus likely to overpredict water level changes in the future.

The four models have similar responses at BM2. The simulated total change agrees well with the measured total change, but the predicted change occurs earlier than the measured change. The rate of measured change had decreased after 1998, so that the simulated rate is very similar to the measured rate. Stated differently, the agreement of the model appears to have improved slightly after 1997 or 1998.

Comparison of simulated with measured values is more difficult at BM3 because of the impacts of variable, local pumping and the resultant high variability of water levels in

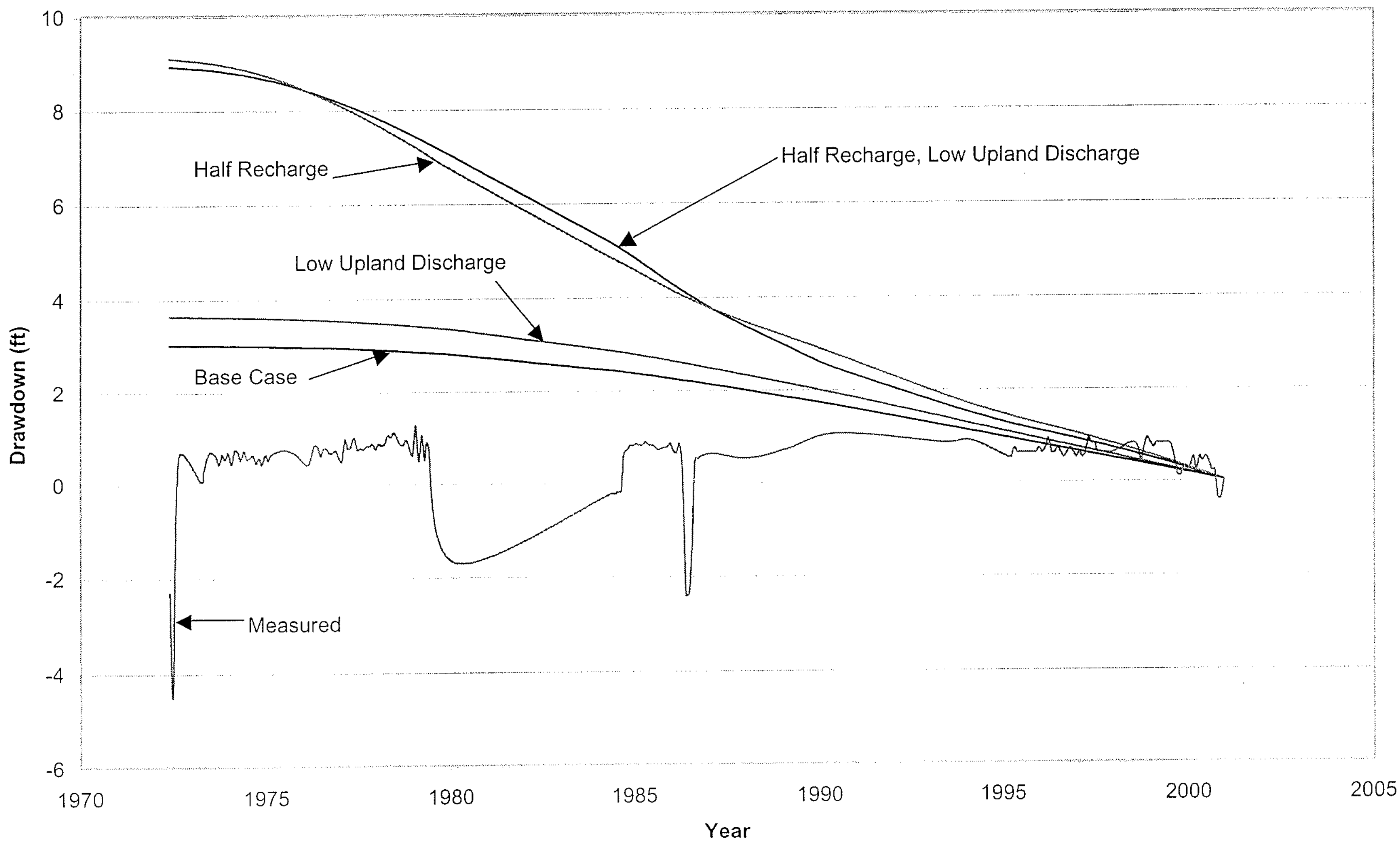


Figure 10. Measured vs. Simulated Water Levels Through 2000- BM-1

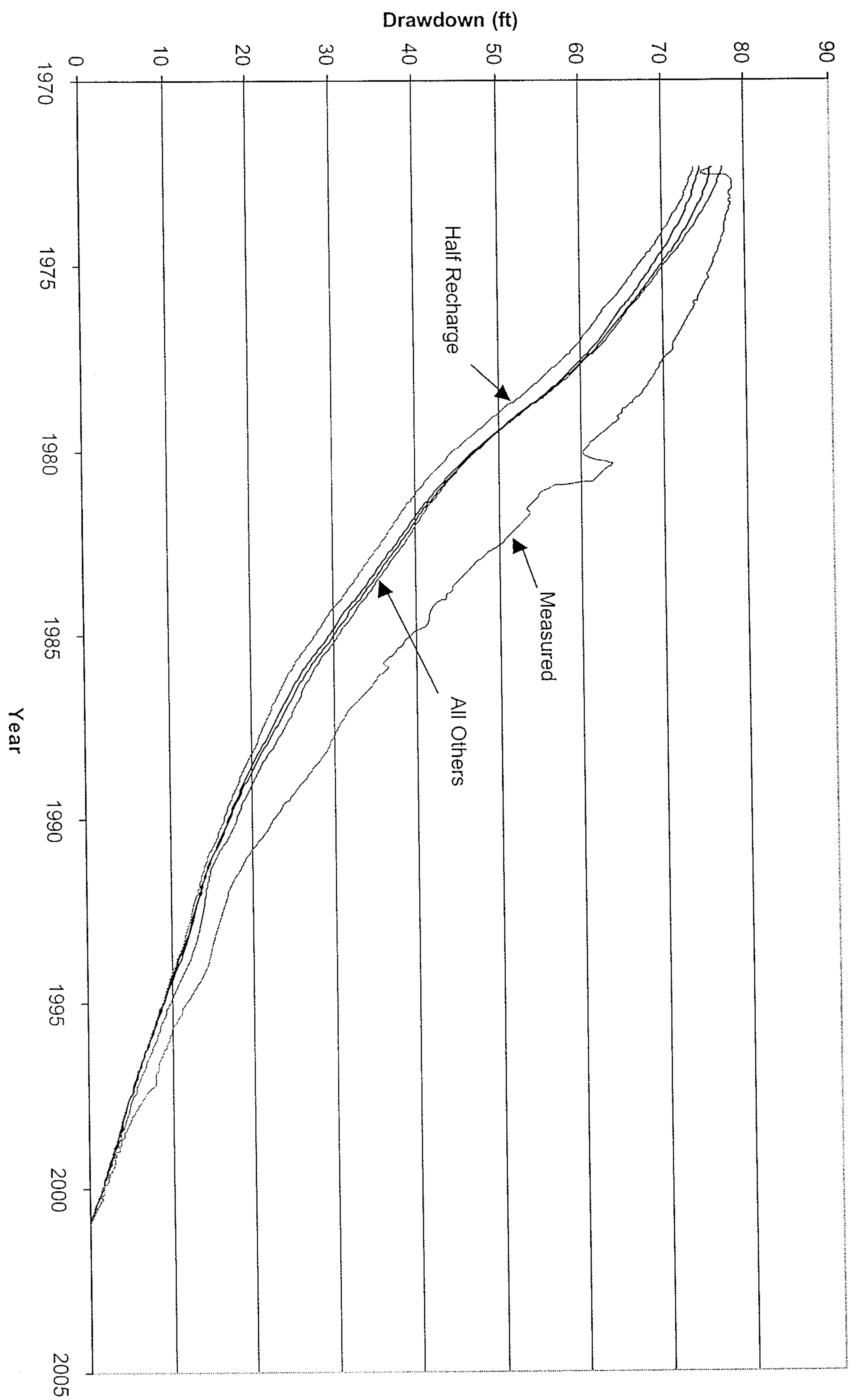


Figure 11. Measured vs. Simulated Water Levels Through 2000 - BM-2

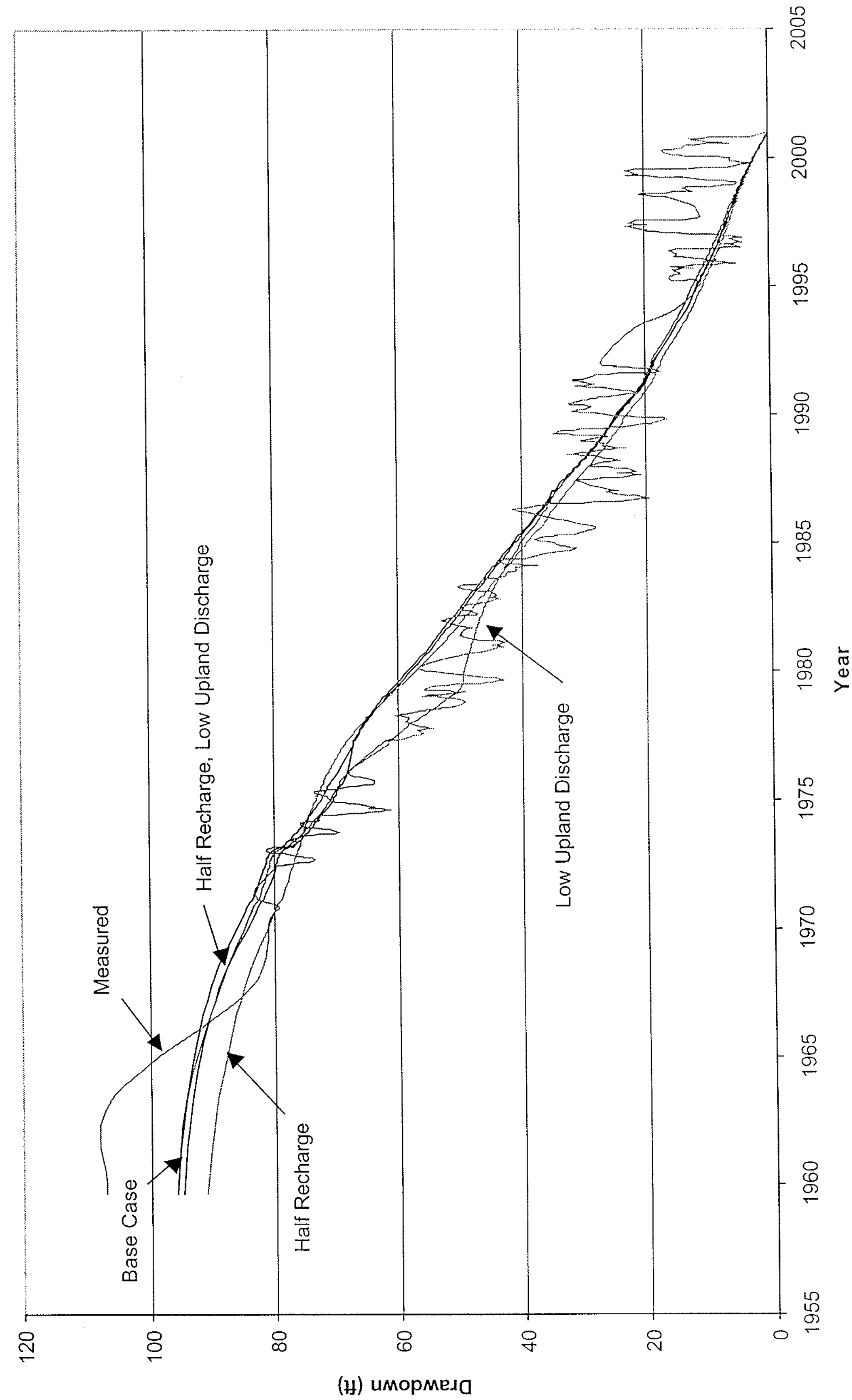


Figure 12. Measured vs. Simulated Water Levels Through 2000 - BM-3

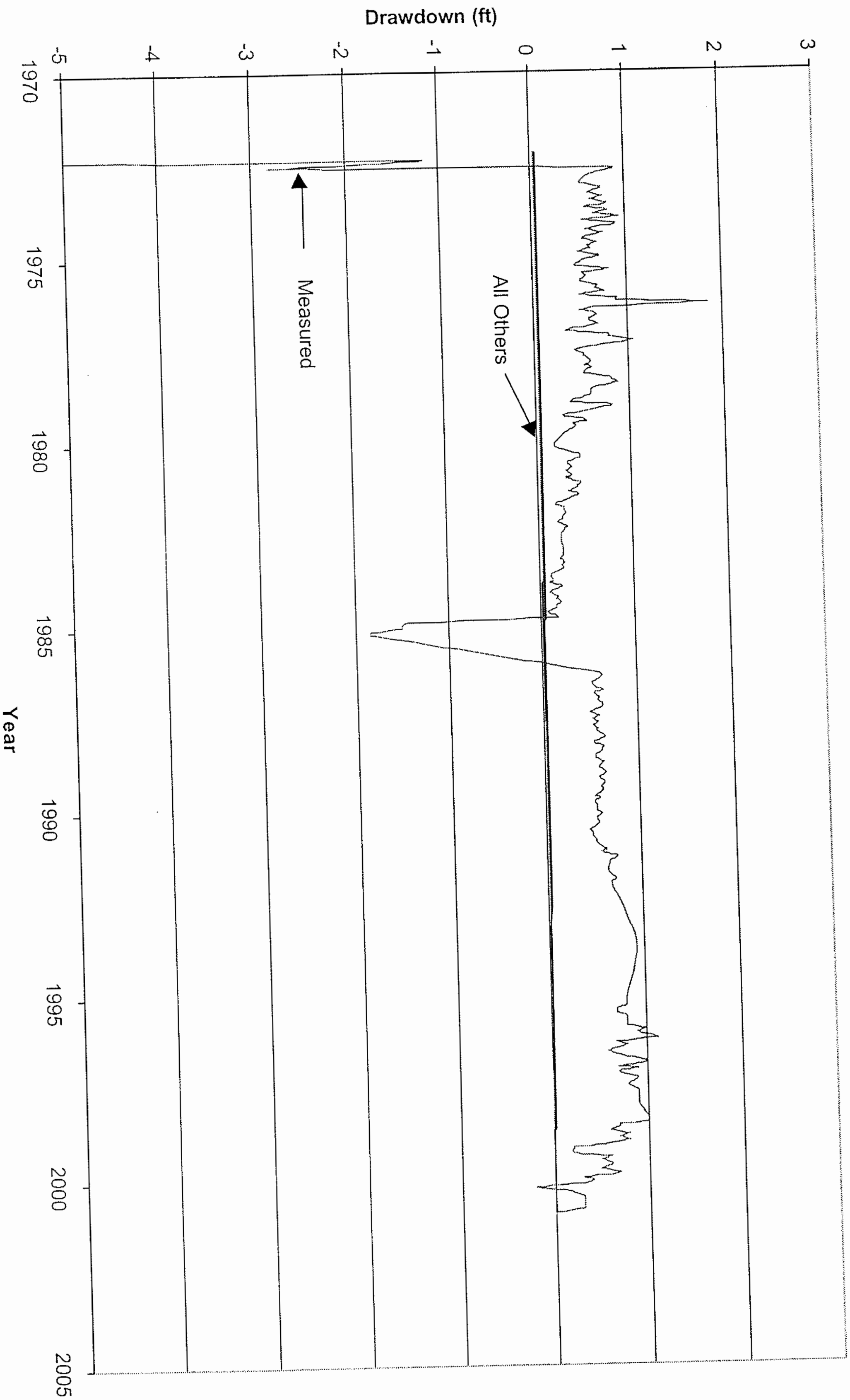


Figure 13. Measured vs. Simulated Water Levels Through 2000 - BM4

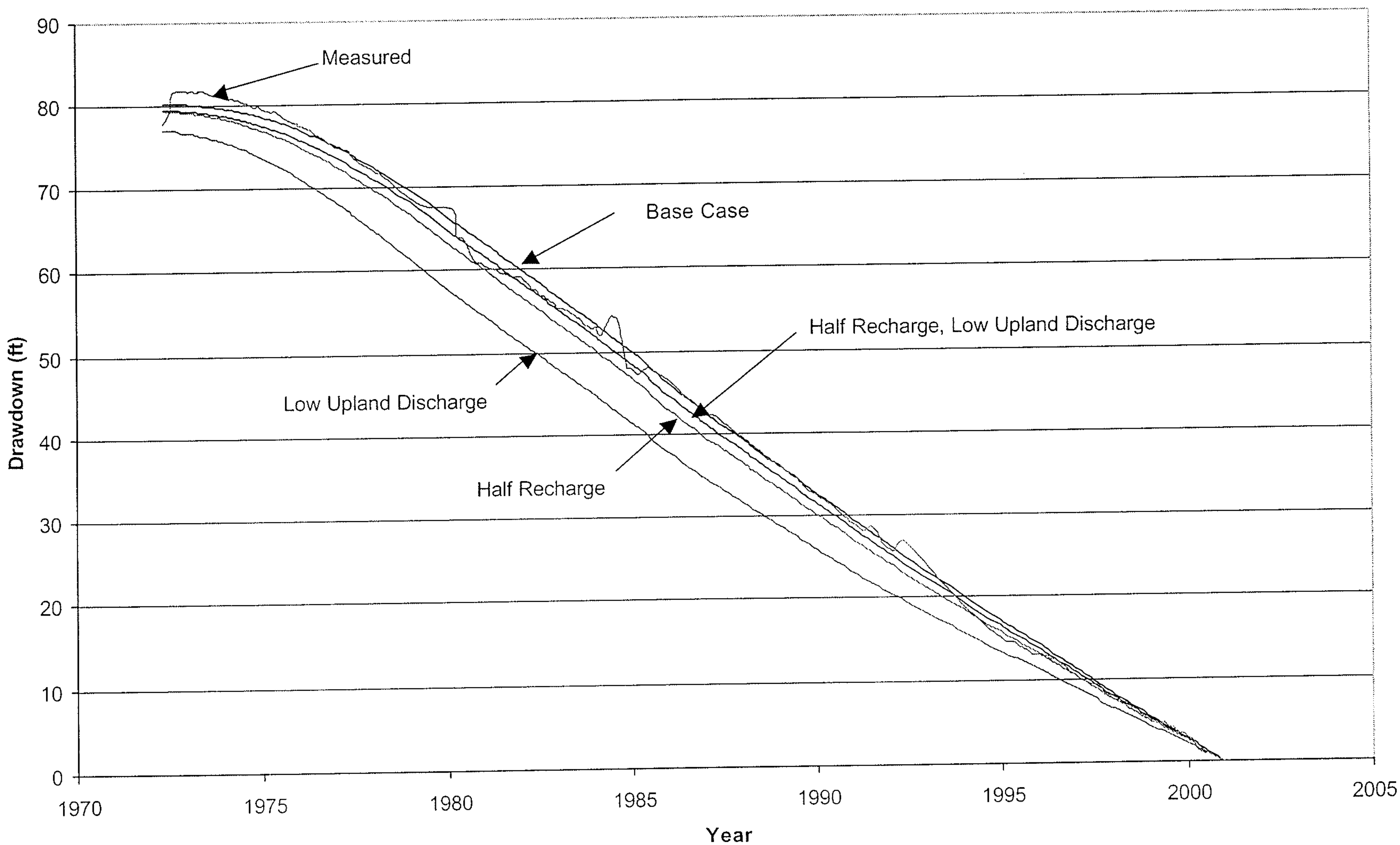


Figure 14. Measured vs. Simulated Water Levels Through 2000 - BM-5

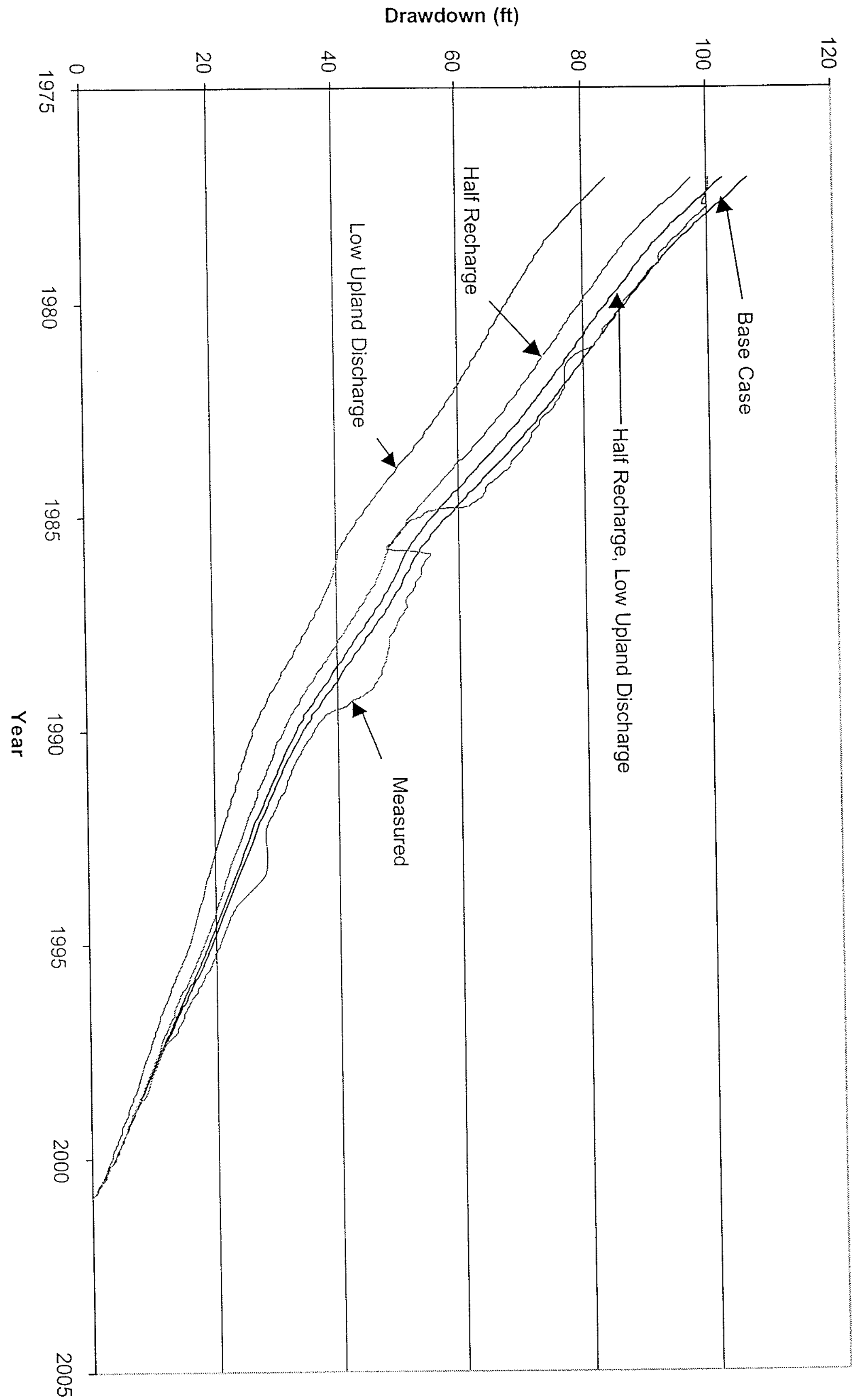


Figure 15. Measured vs. Simulated Water Levels Through 2000 - BM-6

BM3. The four models track the measured changes approximately equally well. Variability in the measured values make comparison with the simulated values questionable, but the model may be predicting greater drawdown from 1996 through 2000 than has occurred. BM4 is another well where little change has occurred. A recent decline in water levels of approximately 1 m occurred after 1998. As with BM1, the cause is unknown. Continued monitoring should determine whether this will be a long-term trend, or a short-term change that might be reversed.

Data collected at BM5 between 1996 and 2000 are tracked very well by the four models, although the agreement of the full recharge, low ET model is not quite as good as the other three. The models also agree well with the previous years.

Agreement at BM6 also continues to be excellent. The full recharge, low ET model, although providing a good fit to the measured changes, simulates about 20% less change than the measured change, and less than the other three models. The rate of change calculated by the other three models agrees very well with the measured rate of change. The base case model (full recharge, full ET) provides the best overall fit, reflecting the greater effort in its calibration. Its agreement with the measured changes from 1996 through 2000 (and earlier) suggests that its predictions of the changes in water levels within the area circumscribed by the BM wells will be reliable for many years. Two of the other three models also produce excellent agreement, and would be expected to also provide reliable predictions. The fit of the fourth (full recharge, low ET) is not quite as good, but still acceptable.

The four models match the observed water-level changes at the six BM monitoring wells quite well. Extension of the model from 2D to 3D, coupled with incorporation of additional information, produced a framework for evaluating the effects of Peabody's pumping on the groundwater system. The framework was used to calibrate four different models using reasonable estimates of the recharge rate and the magnitude of discharge in non-wash environments. All four of these models provide significant improvement over previous 2D models in the quality of agreement between simulated and observed changes in water levels due to Peabody's pumping. The predictive ability of the models was evaluated. The four models had been calibrated using water level and pumping data collected through 1996. In this evaluation, the water level and pumping data were updated through 2000, and the four models rerun without changing any other model input. The results indicated that recalibration is not warranted at this time, and are an indication of the ability of the model to accurately predict the effects of pumping by Peabody within the groundwater basin. As with all models used to guide decisions, the model should be periodically evaluated as more data are collected, and updates made when appropriate.

The base-case model is used in the predictive simulations presented below. Testing of the four models used longer pumping periods than evaluated in this PHC (Scenario A, PWCC, 1999), and indicated that all four models produce similar results. The predicted drawdowns are similar (because each model is calibrated to the same water-level and drawdown data), though not identical. Similarly, the predicted impacts on the discharge to streams are also quite similar. Obviously, for the half-recharge cases, the simulated discharge into the streams is less than for the full-recharge cases, and therefore the effect, expressed on a percentage basis, is slightly higher for the half-recharge cases. Because the effects of PWCC pumping on stream discharge are predicted to be low in Scenario A for all four cases, and because the pumping plan evaluated in the PHC envisions a decrease in both pumping rates and time, only the base-case model is evaluated below.

The effects of Peabody's withdrawals from the N aquifer have been simulated using conservative estimates of the annual pumping rate under two mining-plan scenarios (Table 14). Peabody's annual ground water withdrawals averaged 4,176 af/y from 1996 through 2000. Therefore, the selection of 4,400 af/y is considered a conservative yet realistic amount of water withdrawal to support the current coal production rates through 2007 for simulation purposes. While PWCC has not and does not relinquish or restrict any right it has or may have to continue to utilize water from the N aquifer in accordance with the terms of its tribal lease agreements, two long-term pumping scenarios were simulated (Table 14), evaluating the effect of N aquifer pumping assuming that water from an alternate Tribal source is available starting in 2008. The alternate tribal source will be from a source other than the N aquifer. In Scenario J, the alternate source provides water for operation of both the Kayenta and Black Mesa mines through 2025, and reclamation activities at both mines through 2028. The N-Aquifer wellfield would be maintained to provide water in the event that water was unavailable from the alternate wellfield. Maintenance of the N-aquifer wellfield is assumed to involve limited monthly pumping of each of the wells when not in regular use. For Scenario J, the maintenance pumping is 444 af/y. From 2029 through 2039, Scenario J assumes that the N-Aquifer wellfield is used to solely supply water to the local residents.

Scenario K is very similar to Scenario J. However, the N-Aquifer wellfield is assumed to supply the needs of the Kayenta Mine (928 af/y), using NAV2, NAV4, NAV7, and NAV9. Maintenance pumping of NAV3, NAV5, NAV6, and NAV8 is assume to total 247 af/y. Otherwise the pumping schedule is the same as Scenario J.

In both of these scenarios, it is assumed that the N Aquifer wellfield would be used to replace water from the alternate source during periods when the alternate source is not

Table 14

Simulated Peabody Pumping Rates for Two Predictive Scenarios

Scenario	Peabody N Aquifer Pumping Distribution
J	**1997-2003** - Actual (2003 estimated) **2004-2007** - 4400 af/y **2008-2025** - well maintenance (444 af/y, reduced every 3 years based on 6 month or 1 month supplemental pumping), public supply (61 af/y), and 6-month (2462 af/y) or 1-month 409 af/y) periods of supplemental pumping to replace alternate Tribal production (alternating on a three-year cycle) **2026-2028** - 430 af/y reclamation and 75 af/y public supply **2029-2039** - 100 af/y public supply
K	**1997-2003** - Actual (2003 estimated) **2004-2007** - 4400 af/y **2008-2025** - Kayenta mine supply (928 af/y), well maintenance (247 af/y, reduced every 3 years base on 6 month or 1 month supplemental pumping), public supply (61 af/y) and 6-month and 1-month periods of supplemental pumping to replace alternate Tribal production (alternating on a three-year cycle) **2026-2028** - 430 af/y reclamation and 75 af/y public supply **2029-2039** - 100 af/y public supply

available for either 1 month or 6 months, every three years. The first period of replacement pumping is assumed to be for 1 month in 2010. The second period is assumed to be 6 months long in 2013, three years later. This alternating pattern of periodic usage of the N aquifer to replace the alternate source, either using all eight N Aquifer wells in Scenario J, or only 4 N Aquifer wells in Scenario K, is continued through 2025. The community pumping is assumed to increase at a rate of 2.7% per year, as described in Chapter 6 of the 3D modeling report (PWCC, 1999) for the future pumping.

Impacts of Drawdown at Community Pumping Centers. Pumping of water from the N aquifer causes lowering of water levels or confined pressures within the aquifer. Drawdown is necessary in order for water to be withdrawn from the aquifer by wells and occurs due to pumping at the Peabody well field, as well as at the communities. However, excessive drawdown may cause wells to become unusable (e.g., if the water level during pumping of the well is lowered to the pump intake, and the pump cannot be lowered). Drawdown also increases pumping costs. The USGS has been monitoring water levels in communities throughout the basin for several years, and has estimated the drawdown caused by pumping of water from the N aquifer.

Figure 16 shows the simulated drawdown through 2002 for the top part of the N aquifer, using the base-case 3D model. Drawdown resulting from Peabody's pumping is greatest beneath the leasehold, and is very small within the unconfined area. The transition from confined to unconfined conditions greatly limits drawdown because of the much greater storage coefficient under unconfined conditions. Drawdown caused by pumping at the communities is also apparent. Community drawdown is most apparent at Shonto and Tuba City, where drawdown due to Peabody pumping is essentially non-existent, but occurs at other communities as well. The model-estimated drawdown caused by pumping at the end of 2002 is presented in Table 15. These wells were chosen because of their use by the USGS in the annual monitoring reports. The percentage of drawdown attributable to Peabody pumping was calculated from the base-case 3D modeling results, based on pumping simulations with and without Peabody pumping. Data on the depth of the N aquifer or uppermost open interval were obtained from USGS monitoring reports. The drawdown attributable to Peabody is subtracted from this depth to estimate the available water column remaining after incorporating the effect of Peabody's pumping. This thickness represents the drawdown available before the water level would be lowered to the top of the N aquifer or the top of the production interval in the well, if only Peabody had been pumping from the aquifer.

The greatest effects on water levels in 2002 are for Forest Lake, Chilchinibito, Rocky

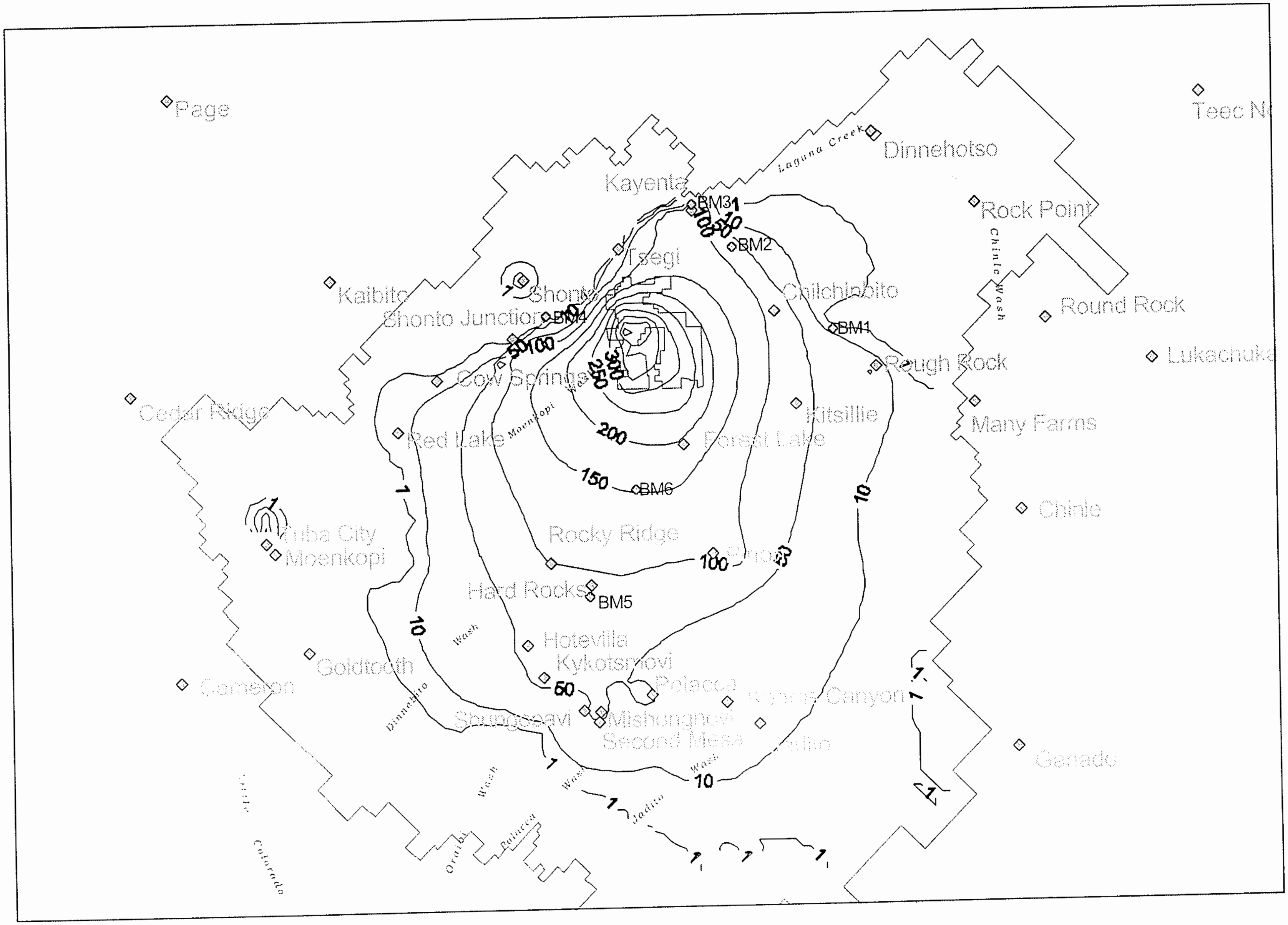


Figure 16. Simulated Drawdown in the N Aquifer, Scenario J and K in 2002 (50 ft interval)

Table 15

Effects of PWCC pumping on water levels in selected wells, end of 2002

Community	Well	Initial DTW (ft)	Simulated Drawdown (ft)	PWCC Allocation (%)	PWCC Allocation (ft)	Depth to N or Top of Open Interval	Remaining Excess Water Column (ft)
Chilchinibito	PM3	405	88	73%	64	1136	667
Forest Lake NTUA 1	4T-523	1096	200	92%	184	1870	590
Kayenta West	8T-541	227	119	27%	32	700	441
Keams Canyon	PM2	292.5	33	23%	7	900	600
Kykotsmovi	PM1	220	81	25%	20	880	640
Pinon	PM6	743.6	120	55%	67	1870	1060
Rocky Ridge	PM2	432	100	86%	86	1442	924
Rough Rock	10R-111	170	3	47%	2	210	38

Ridge and Pinon, where the estimated drawdown attributable to Peabody ranges from 64 to 184 feet. Elsewhere, the drawdown resulting from Peabody use of the water is 35 feet or less. At all locations except Rough Rock, more than 440 feet of water remains above the top of the aquifer as of the end of 2002. Pumping of the well itself will cause additional drawdown. Information on this local drawdown is not available, and it is assumed that the local drawdown is a few hundred feet or less. Thus these calculations indicate that Peabody's pumping, as of 2002, will not cause sufficient drawdown to reduce the production of the aquifer by dewatering. For Rough Rock (well 10R-111), the water column above the top of the aquifer was only 40 feet thick before any pumping, and Peabody's pumping reduces it by approximately 2 feet. Note that this well is east of the community of Rough Rock, where both PWCC and community-based drawdown is greater. At this location it is likely that the pump is already set below the top of the N aquifer, similar to wells in the unconfined area.

Figure 17 portrays the predicted drawdown in the N Aquifer at the end of 2007, due to PWCC and non-PWCC pumping. At this time Scenarios J and K are identical. Comparison of Figures 16 and 17 indicate that the majority of the drawdown due to Peabody's pumping has already occurred. The primary change from the 2002 results is at the leasehold, where drawdown has increased. Near the periphery of the drawdown cone, drawdown has increased only slightly.

Table 16 provides the estimated drawdowns due to PWCC pumping at the end of 2007. This represents the anticipated end of pumping from the N aquifer to supply the combined water needs of both mines and the coal-slurry pipeline. The drawdowns have increased slightly over those at the end of 2002, but the utility of the N aquifer has not been diminished.

Figures 18 and 19 show the drawdown predicted in 2028 for Scenarios J and K. Note that there is significant recovery of water levels near the leasehold, as a result of the reduction in pumping rates that started in 2008. Drawdown caused by the communities increases because of the projected increase in community water use. Recovery is greater for Scenario J; Scenario K assumes that the water needs at the Kayenta mine will still be met by pumping of the N Aquifer.

Table 17a and 17b presents information on the predicted drawdown at the end of 2028 and 2039, respectively, due to all pumping, and Peabody's pumping (Scenario J). PWCC's pumping is assumed to stop at the end of 2028, except for a continuing of 100 af/y to local residents. In most areas, there is predicted to be significant recovery of water levels, caused by the reduction of PWCC's pumping simulated as beginning in 2008. Drawdown caused

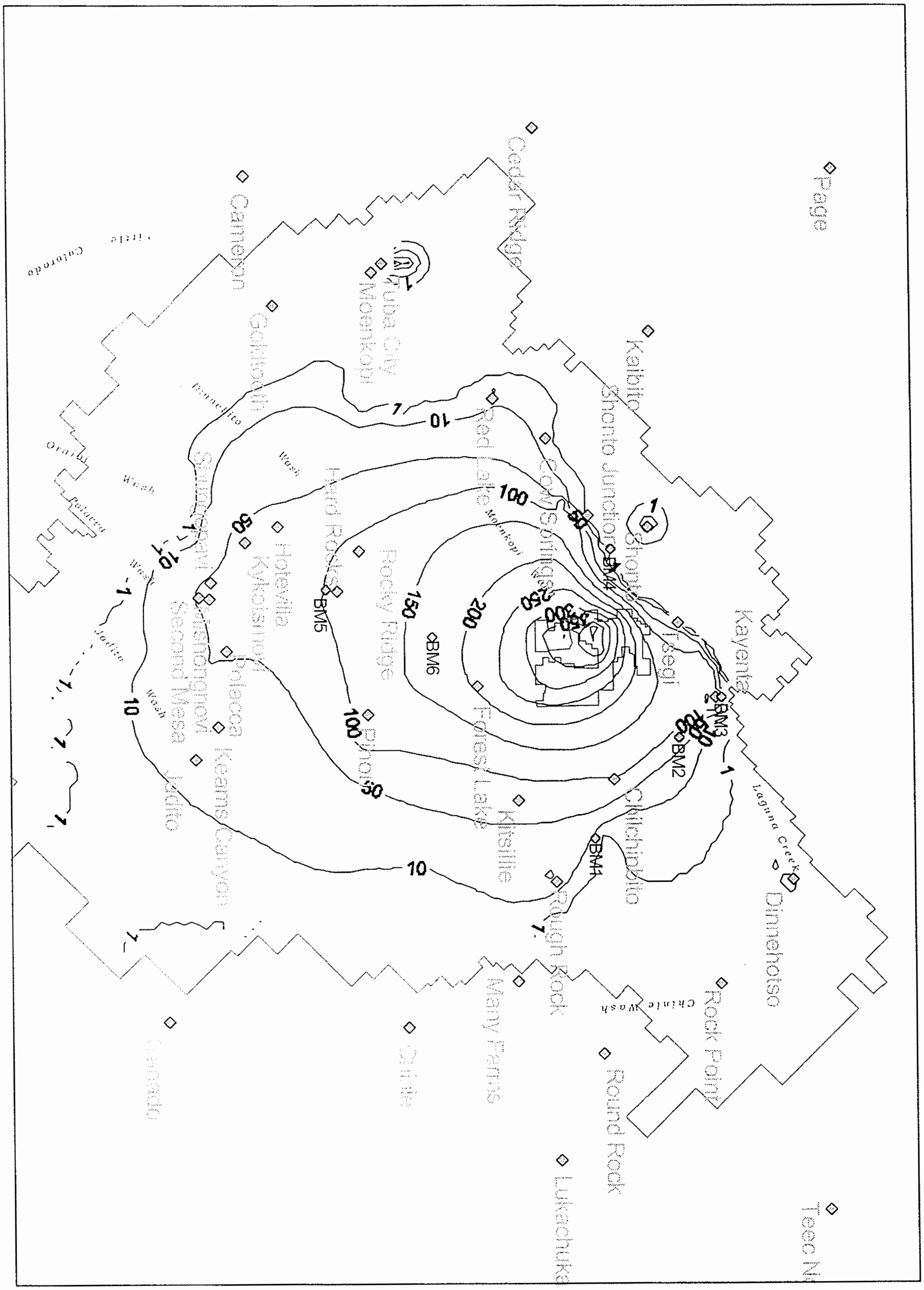


Figure 17. Simulated Drawdown in the N Aquifer, Scenario J and K in 2007 (50 ft interval)

Table 16

Effects of PWCC pumping on water levels in selected wells, end of 2007

Community	Well	Initial DTW (ft)	Simulated Drawdown (ft)	PWCC Allocation (%)	PWCC Allocation (ft)	Depth to N or Top of Open Interval	Remaining Excess Water Column (ft)
Chilchinibito	PM3	405	99	72%	71	1136	660
Forest Lake NTUA 1	4T-523	1096	219	91%	199	1870	575
Kayenta West	8T-541	227	133	26%	35	700	438
Keams Canyon	PM2	292.5	38	25%	9	900	598
Kykotsmovi	PM1	220	97	27%	26	880	634
Pinon	PM6	743.6	134	58%	77	1870	1049
Rocky Ridge	PM2	432	118	84%	100	1442	910
Rough Rock	10R-111	170	4	48%	2	210	38

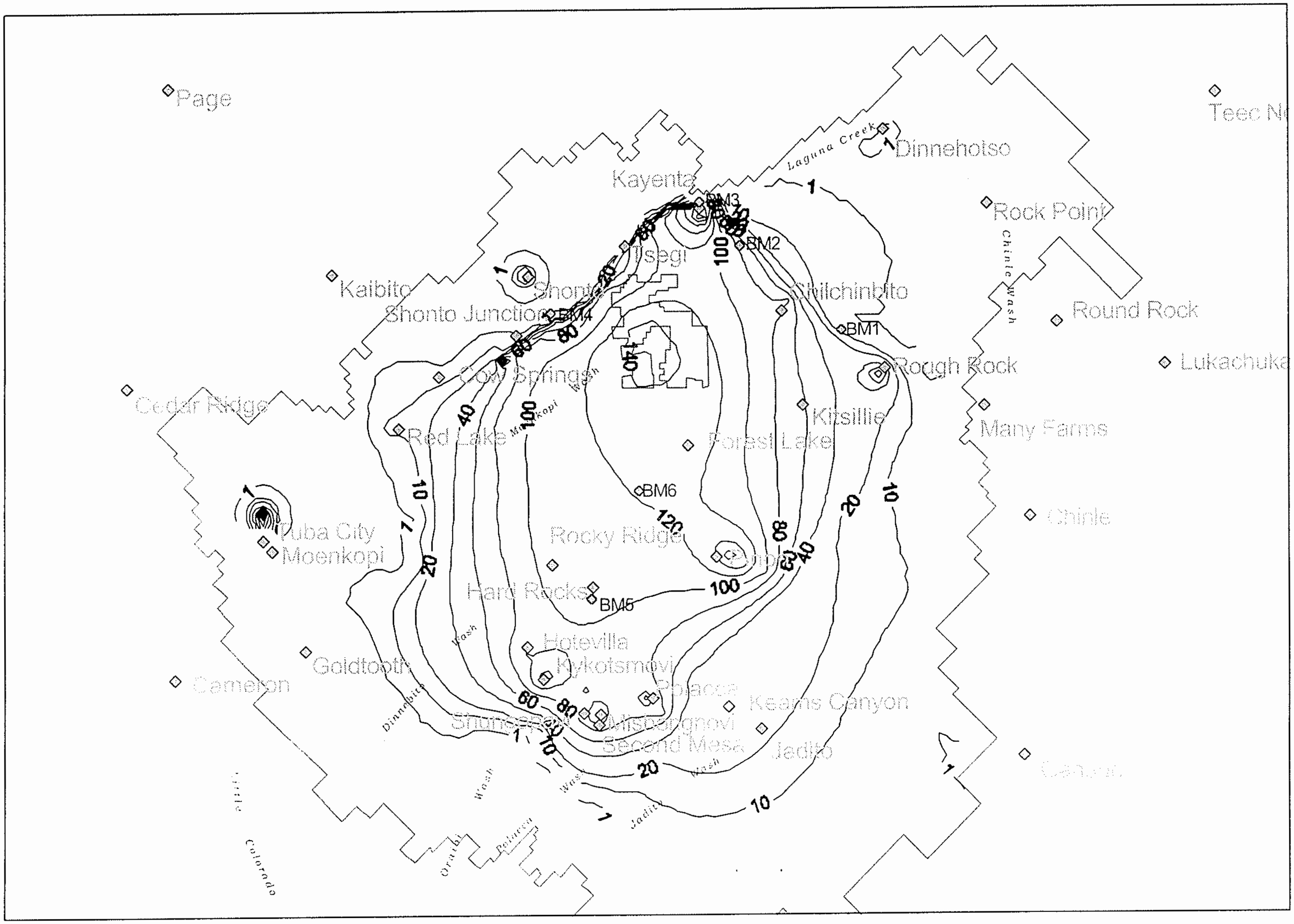


Figure 18. Simulated Drawdown in the N Aquifer, Scenario J in 2028 (20 ft interval)

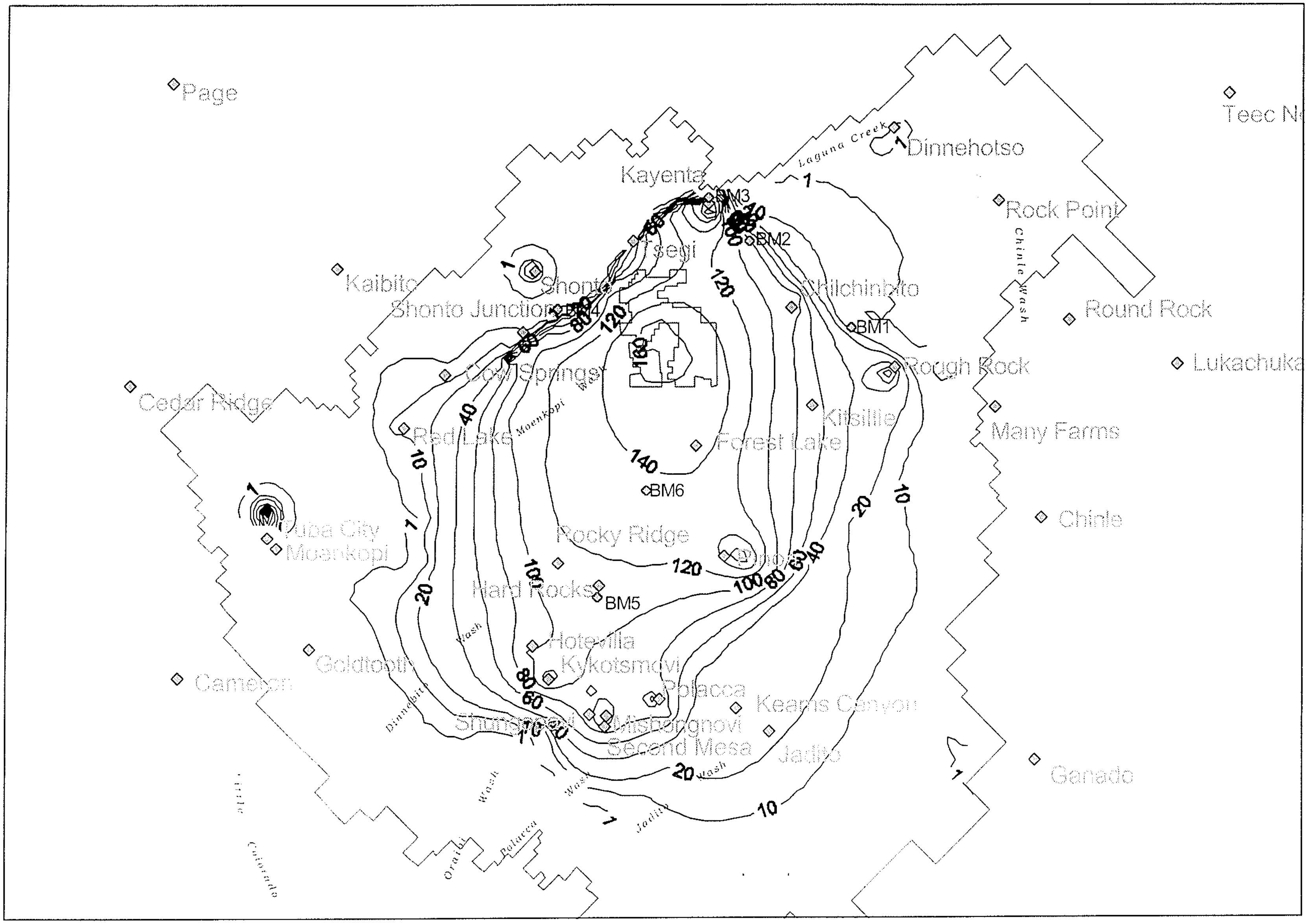


Figure 19. Simulated Drawdown in the N Aquifer, Scenario K in 2028 (20 ft interval)

Table 17

Effects of PWCC pumping on water levels in selected wells, Scenario J

Community	Well	Initial DTW (ft)	Simulated Drawdown (ft)	PWCC Allocation (%)	PWCC Allocation (ft)	Depth to N or Top of Open Interval	Remaining Excess Water Column (ft)
Chilchinibito	PM3	405	85	47%	40	1136	691
Forest Lake NTUA 1	4T-523	1096	140	76%	106	1870	668
Kayenta West	8T-541	227	169	13%	22	700	451
Keams Canyon	PM2	292.5	59	20%	12	900	596
Kykotsmovi	PM1	220	152	20%	31	880	629
Pinon	PM6	743.6	155	41%	64	1870	1062
Rocky Ridge	PM2	432	111	70%	78	1442	932
Rough Rock	10R-111	170	6	37%	2	210	38

a. End of 2028

Community	Well	Initial DTW (ft)	Simulated Drawdown (ft)	PWCC Allocation (%)	PWCC Allocation (ft)	Depth to N or Top of Open Interval	Remaining Excess Water Column (ft)
Chilchinibito	PM3	405	87	31%	27	1136	704
Forest Lake NTUA 1	4T-523	1096	113	60%	68	1870	706
Kayenta West	8T-541	227	197	9%	18	700	455
Keams Canyon	PM2	292.5	74	17%	13	900	595
Kykotsmovi	PM1	220	191	14%	26	880	634
Pinon	PM6	743.6	174	28%	49	1870	1077
Rocky Ridge	PM2	432	104	56%	59	1442	951
Rough Rock	10R-111	170	7	28%	2	210	38

b. End of 2039

by pumping in the indicated community wells will further reduce the water column thickness while the local pumping is occurring. For nearly all of these wells, the remaining water column is hundreds of feet thick, indicating that the N aquifer will be able to continue to supply water at previous rates. The sole exception is possibly well 10R-111 near Rough Rock. As previously discussed, this well only had a water column of 40 feet above the top of the aquifer before pumping. Peabody's predicted reduction is approximately 2 feet, but local pumping from the well would be expected to have a greater impact. If the drawdown due to pumping from 10R-111 is more than 38 feet, dewatering of the aquifer in the vicinity of the well may occur. Here, the N aquifer is approximately 600 ft thick, so that local dewatering, if it occurs, will have only a minor impact on aquifer productivity.

The differences between Scenarios J and K are minor (Figures 18 and 19, and Tables 17 and 18). In Scenario K, there will also be recovery in nearly all areas between 2029 and 2039 (Tables 18a and 18b). In areas near the leasehold, recovery of water levels is predicted to continue to occur at a moderate pace. In areas where there has been limited PWCC-induced drawdown, drawdown will continue to increase at a slow rate as the system equilibrates.

Impacts on stream baseflow and spring discharge rates. Because of the limited drawdown due to Peabody in unconfined areas and concentrated nature of discharge areas, the effects of Peabody's pumping on stream baseflow and spring discharge rates are expected to be small. Two-dimensional simulations previously performed by the USGS (Eychaner, 1983; Brown and Eychaner, 1988) and by GeoTrans (1987) provided results consistent with this expectation.

Table 19 presents the predicted effects of pumping on discharge into streams at the end of 2007. This and following tables were constructed by performing simulations in which there is both PWCC and non-PWCC pumping, and tabulating the results (columns labeled "All"). The simulations were repeated, but with Peabody's pumping removed. The difference is the effect of Peabody's use of N-Aquifer water. The column labeled "% Total PWCC" is the percentage reduction of the pre-pumping streamflow caused by Peabody's pumping from the beginning of Peabody's pumping in 1965 through the end of the year indicated in the table, 2007.

For example, the simulated pre-pumping discharge into Moenkopi Wash was 4305.1 acre-feet per year (af/y). At the end of 2007, the simulated discharge is predicted to be reduced

Table 18

Effects of PWCC pumping on water levels in selected wells, Scenario K

Community	Well	Initial DTW (ft)	Simulated Drawdown (ft)	PWCC Allocation (%)	PWCC Allocation (ft)	Depth to N or Top of Open Interval	Remaining Excess Water Column (ft)
Chilchinibito	PM3	405	94	52%	48	1136	683
Forest Lake NTUA 1	4T-523	1096	158	78%	124	1870	650
Kayenta West	8T-541	227	171	14%	24	700	449
Keams Canyon	PM2	292.5	59	21%	12	900	595
Kykotsmovi	PM1	220	153	21%	33	880	627
Pinon	PM6	743.6	161	44%	71	1870	1056
Rocky Ridge	PM2	432	119	72%	86	1442	924
Rough Rock	10R-111	170	6	39%	2	210	38

a. End of 2028

Community	Well	Initial DTW (ft)	Simulated Drawdown (ft)	PWCC Allocation (%)	PWCC Allocation (ft)	Depth to N or Top of Open Interval	Remaining Excess Water Column (ft)
Chilchinibito	PM3	405	91	34%	31	1136	700
Forest Lake NTUA 1	4T-523	1096	122	63%	76	1870	698
Kayenta West	8T-541	227	198	10%	20	700	453
Keams Canyon	PM2	292.5	75	18%	14	900	594
Kykotsmovi	PM1	220	193	15%	29	880	631
Pinon	PM6	743.6	180	31%	55	1870	1071
Rocky Ridge	PM2	432	111	59%	66	1442	944
Rough Rock	10R-111	170	7	30%	2	210	38

b. End of 2039

Table 19

Simulated reductions in discharge (acre feet per year) to streams, Scenarios J and K, 2007.

	1955	2007		Change due to Pumping			%	
Pumping	None	All	Non-PWCC	All	Non-PWCC	PWCC	All	PWCC
Chinle Wash	498.9	498.8	498.8	0.0	0.0	0.0	0.01	0.00
Laguna Creek	2535.7	2429.0	2438.3	106.7	97.4	9.3	4.21	0.37
Pasture Canyon	426.8	384.1	384.1	42.6	42.6	0.0	9.99	0.00
Moenkopi Wash	4305.1	4281.6	4302.4	23.4	2.6	20.8	0.54	0.48
Dinebito Wash	515.6	514.9	515.3	0.7	0.3	0.4	0.13	0.07
Oraibi Wash	458.1	455.2	455.7	2.9	2.4	0.5	0.64	0.11
Polacca Wash	440.6	430.2	431.2	10.4	9.4	1.1	2.37	0.25
Jaidito Wash	2027.4	2013.7	2017.3	13.7	10.1	3.6	0.68	0.18
Cow Springs	2178.0	2167.8	2177.2	10.2	0.8	9.4	0.47	0.43

by 23.4 af/y, of which 2.6 af/y is caused by non-PWCC pumping, and 20.8 af/y by PWCC pumping. The percentage reduction due to Peabody's pumping is estimated to be 0.48%. The percentage reductions for all discharge areas are estimated to range from 0.0 to nearly 0.5%.

The predicted effects on streams for Scenario J pumping in 2029 and 2039 are provided in Table 20a and 20b, respectively. The largest change from 2007 is the increase in effect at Cow Springs (1.15% in 2029, and 1.46% in 2039. This predicted decrease would not be measurable. The pumping is greater in Scenario K (Table 21a and Table 21b), and the effects on streamflow are slightly greater than in Scenario J. The magnitudes of these predicted reductions in discharge are also too small to be measurable.

In contrast with the regionally significant discharge areas, the models did not specifically evaluate the effect of pumping on individual springs in non-wash settings (1) because of the difficulty of accurately simulating these impacts considering the topographic relief and constraints on grid spacing, and (2) because of the limited drawdown in unconfined areas caused by distant pumping. The locations of many of the smaller springs are determined by the geometric relationships between beds of different hydraulic properties, and by locations of fracture zones. Many of the smaller springs discharge from formations, such as those in the D aquifer, that contain low hydraulic conductivity beds. These lower conductivity beds, which are responsible for the occurrence of the springs, will tend to isolate the springs from the effects of pumping of the N aquifer.

Further, the discharge rates of these springs are likely to be more sensitive to changes in local recharge than to drawdown caused by distant pumping. These springs are typically located near recharge areas, and temporal changes in their discharge rates caused by short-term changes in local recharge rates would be expected. Observations of springs discharging from the Wepo formation on the leasehold confirm the temporal variability of these smaller springs. Tree-ring studies performed throughout the southwestern U.S. document the variability of precipitation on the scale of decades (see, for example, Stahle and others, 2000). Even if good spring flow data were available, this variability would make calibration to these data difficult. Because of the character of these springs and of the groundwater system, the effects of Peabody's pumping are expected to be undetectable, and very small regardless. Measurement of pumping effects on these springs will be difficult because of the expected small magnitude of these effects, seasonal changes of precipitation and evapotranspiration rates, and longer term changes in local precipitation rates.

Table 20

Simulated reductions in discharge (acre feet per year) to streams, Scenario J, 2029 and 2039

	1955	2029		Change due to Pumping			%	
Pumping	None	All	Non-PWCC	All	Non-PWCC	PWCC	All	PWCC
Chinle Wash	498.9	498.8	498.8	0.1	0.1	0.0	0.03	0.00
Laguna Creek	2535.7	2372.2	2382.0	163.5	153.7	9.8	6.45	0.39
Pasture Canyon	426.8	318.6	318.6	108.1	108.1	0.0	25.33	0.00
Moenkopi Wash	4305.1	4273.0	4298.8	32.0	6.3	25.8	0.74	0.60
Dinebito Wash	515.6	514.0	514.9	1.6	0.8	0.9	0.32	0.17
Oraibi Wash	458.1	451.7	453.1	6.5	5.1	1.4	1.41	0.31
Polacca Wash	440.6	420.9	422.9	19.7	17.7	2.0	4.47	0.45
Jaidito Wash	2027.4	1996.0	2005.7	31.4	21.7	9.7	1.55	0.48
Cow Springs	2178.0	2149.5	2174.6	28.5	3.4	25.1	1.31	1.15

a. 2029

	1955	2039		Change due to Pumping			%	
Pumping	None	All	Non-PWCC	All	Non-PWCC	PWCC	All	PWCC
Chinle Wash	498.9	498.7	498.7	0.2	0.2	0.0	0.04	0.00
Laguna Creek	2535.7	2330.9	2341.8	204.8	193.9	10.9	8.08	0.43
Pasture Canyon	426.8	290.4	290.4	136.3	136.3	0.0	31.94	0.00
Moenkopi Wash	4305.1	4272.0	4296.6	33.1	8.5	24.7	0.77	0.57
Dinebito Wash	515.6	513.5	514.5	2.1	1.1	1.0	0.41	0.20
Oraibi Wash	458.1	449.6	451.3	8.5	6.9	1.6	1.86	0.36
Polacca Wash	440.6	417.2	418.6	23.4	22.0	1.4	5.31	0.32
Jaidito Wash	2027.4	1986.5	1997.9	40.9	29.5	11.5	2.02	0.57
Cow Springs	2178.0	2140.8	2172.5	37.2	5.5	31.7	1.71	1.46

b. 2039

Table 21

Simulated reductions in discharge (acre feet per year) to streams, Scenario K, 2029 and 2039.

	1955	2029		Change due to Pumping			%	
Pumping	None	All	Non-PWCC	All	Non-PWCC	PWCC	All	PWCC
Chinle Wash	498.9	498.8	498.8	0.1	0.1	0.0	0.03	0.00
Laguna Creek	2535.7	2371.0	2382.0	164.7	153.7	11.0	6.50	0.43
Pasture Canyon	426.8	318.6	318.6	108.1	108.1	0.0	25.33	0.00
Moenkopi Wash	4305.1	4267.8	4298.8	37.3	6.3	31.0	0.87	0.72
Dinebito Wash	515.6	513.9	514.9	1.7	0.8	0.9	0.33	0.18
Oraibi Wash	458.1	451.6	453.1	6.5	5.1	1.4	1.42	0.31
Polacca Wash	440.6	420.8	422.9	19.8	17.7	2.1	4.49	0.47
Jaidito Wash	2027.4	1995.7	2005.7	31.7	21.7	10.0	1.57	0.50
Cow Springs	2178.0	2148.4	2174.6	29.6	3.4	26.2	1.36	1.21

a. 2029

	1955	2039		Change due to Pumping			%	
Pumping	None	All	Non-PWCC	All	Non-PWCC	PWCC	All	PWCC
Chinle Wash	498.9	498.7	498.7	0.2	0.2	0.0	0.04	0.00
Laguna Creek	2535.7	2329.7	2341.8	206.0	193.9	12.1	8.12	0.48
Pasture Canyon	426.8	290.4	290.4	136.3	136.3	0.0	31.94	0.00
Moenkopi Wash	4305.1	4265.5	4296.6	39.6	8.5	31.1	0.92	0.72
Dinebito Wash	515.6	513.4	514.5	2.2	1.1	1.1	0.43	0.22
Oraibi Wash	458.1	449.5	451.3	8.6	6.9	1.8	1.88	0.38
Polacca Wash	440.6	417.0	418.6	23.6	22.0	1.6	5.35	0.36
Jaidito Wash	2027.4	1985.3	1997.9	42.1	29.5	12.6	2.08	0.62
Cow Springs	2178.0	2137.8	2172.5	40.1	5.5	34.6	1.84	1.59

b. 2039

In summary, groundwater models are the best tools available for evaluating the contributions of different pumping stresses on the observed or measured effects (i.e., water levels and stream flows). Models of the N Aquifer flow system have been developed by both the USGS and by Peabody since the 1980's, with each successive effort improving on the previous. As additional data have been collected and improved computational tools made available, the models have incorporated more knowledge of the groundwater system. The models have varied in detail; however, they were each based on the data available at the time of the model's development and incorporate the major components of the N Aquifer flow system. Further, each model has been subjected to a calibration process whereby the ability of the model to simulate historical measurements is demonstrated. Peabody's 3D model has been used to evaluate the effects of uncertainty in the recharge rate. Importantly, the several models are consistent with respect to their predictions of the impacts from pumping on the N Aquifer flow system. They predict that water levels in the confined part of the N aquifer will be reduced by pumping but that they will remain well above the top of the N aquifer. The effect of Peabody's pumping on discharge to streams has been and will continue to be minimal.

Effect on the Structural Integrity of the N Aquifer. Lowering of water levels by pumping has allowed compaction of unconsolidated sediments in areas of the western U.S. (e.g., Las Vegas valley, Nevada; Antelope Valley, California; San Joquin Valley, California). The U.S. Geological Survey (Galloway and others, 1999) recently published a Circular documenting examples of aquifer compaction and related land subsidence associated with reduction of water pressures, oxidation of organic deposits, and formation of sinkholes in carbonate terranes. It states (p. 8-9):

> REVERSIBLE DEFORMATION OCCURS IN ALL AQUIFER SYSTEMS
>
> The relation between changes in ground-water levels and compression of the aquifer system is based on the principle of effective stress first proposed by Karl Terzaghi (Terzaghi, 1925). By this principle, when the support provided by fluid pressure is reduced, such as when ground-water levels are lowered, support previously provided by the pore-fluid pressure is transferred to the skeleton of the aquifer system, which compresses to a degree. Conversely, when the pore-fluid pressure is increased, such as when ground water recharges the aquifer system, support previously provided by the skeleton is transferred to the fluid and the skeleton expands. In this way, the skeleton alternately undergoes compression and expansion as the pore-fluid pressure fluctuates with aquifer-system discharge and recharge. *When the load on the skeleton remains less than any previous*

maximum load, the fluctuations create only a small elastic deformation of the aquifer system and small displacement of land surface. [Emphasis added] This fully recoverable deformation occurs in all aquifer systems, commonly resulting in seasonal, reversible displacements in land surface of up to 1 inch or more in response to the seasonal changes in ground-water pumpage.

The USGS circular was primarily addressing basin fill materials of relatively young age. The rocks of the N aquifer are more than 135 million years old, have been buried to sufficient depth to cause pressure welding of the quartz grains, and exhumed. Thus, it is unlikely that production of water from the N aquifer will cause the load on the skeleton to exceed the previous maximum load or produce sufficient compaction to be of concern.

To provide information with which to calculate the amounts of compaction that might occur, rock mechanics studies were performed (GeoTrans, 1993; Peabody, 1994). Because cores of the Navajo Sandstone beneath the Peabody leasehold were not available, samples were collected from outcrop areas. These samples had been subjected to near-surface weathering processes that would remove calcite cement, and thus the testing results are believed to overestimate the effect of drawdown on the material properties. Reduction of water pressure (by pumping, for example) removes some of the support that helps maintain the thickness of the aquifer, and thus allows the rock or aquifer to compact. The laboratory tests were designed to measure this compaction process and its effect on the porosity and hydraulic conductivity of the rock samples. These were performed by placing the samples in a test cell in which the pressure was increased to simulate the pressures at the depth of the aquifer in the deepest parts of the basin. The resulting changes in the samples' porosities and their hydraulic conductivity were measured.

Five samples were placed under effective stresses of up to 2,000 psi, which is approximately equivalent to a depth of burial of 3,000 feet and a depth to water of 600 feet. This is greater than the actual stress conditions near the deepest part of the basin. Measurements of the reduction in porosity of these outcrop samples as the effective stress was increased (water pressure decreased) indicate that the compressibility of the sandstone is about 4×10^{-6}/psi, which is higher than expected for many, un-weathered sandstones. This value is consistent with the weathered nature of the samples. The data also indicate that the samples had previously been subjected to higher pressures than in the outcrop setting, consistent with the geologic history of the area and microscopic observations that the sand grains had been pressure welded. Derivation of compressibility from specific storage measurements for the aquifer (based on model-based interpretations of the observed drawdown caused by Peabody's use of the aquifer) yield numbers approximately one-tenth of the laboratory compressibility measurements. This observation suggests that the compressibility of the weathered rock is approximately 10 times that of the un-weathered rock. Thus, the laboratory compressibility measurements should not be used to characterize the specific storage of the aquifer, but they do provide insight into the maximum changes in the porosity and hydraulic conductivity as water levels change as a result of pumping.

Calculations based on these laboratory compressibility measurements indicate that there could be as much as 1.5 feet reduction in the thickness of the aquifer by 2007. This is approximately a 0.12 percent decrease in thickness. Using compressibility values that are more representative of un-weathered sandstone, the decrease in thickness would be approximately one order of magnitude smaller, or 0.15 feet. The reduction in hydraulic conductivity as a result of the drawdown-induced compaction was also measured on the samples. These measurements indicate that the reduction would be approximately 5% in the immediate vicinity of the Peabody water-supply wells. If un-weathered samples had been tested, the measured reduction would have been considerably less.

Peabody has run video logs in its water-supply wells to evaluate the condition of well screens and the amount of scale that might clog the screen openings. If compaction of the N aquifer sufficient to cause concern were occurring, buckling of the screens would be expected. Many of the wells were logged in the early 1980's, after the majority of drawdown at the wells had occurred; no damage attributable to compaction has been observed. The most recent video log was run in June, 2001 in NAV 8, and no evidence of compaction effects was found. If compaction is not significant at these wells where drawdown and overburden stress are greatest, then compaction in other areas of the aquifer will also be negligible.

In summary, the data indicate that there is no risk of damage to the structural integrity of the aquifer resulting from projected drawdown. Similarly, compaction has been and will be insignificant, and any compaction is expected to be recoverable.

Effects of Induced Leakage of Poorer Quality Water from the Overlying D-Aquifer System on N-Aquifer Water Quality. In the vicinity of the leasehold, water levels in the D aquifer are 100 to 250 feet higher than in the N aquifer. Thus, there is natural downward movement of water from the D to the N aquifer. The large difference in water levels suggests that hydraulic conductivity of the Carmel is low, and therefore that the rate of movement is slow. Drawdown in the N aquifer caused by pumping of water from the N aquifer will increase the rate of water movement in proportion to the increase in water level change. Thus, several hundred feet of drawdown in the N aquifer could increase the leakage rate several fold. Whether this is important depends on the magnitude of leakage prior to any pumping. If the pre-pumping leakage rate were very small, increasing it several fold would still produce a small leakage rate.

The most direct means to evaluate the impact on N aquifer water chemistry is to evaluate water-chemistry data. Water samples have been collected from well 4T-402, a windmill which is completed in the D aquifer near the center of the leasehold. Water from this well has a high TDS, with concentrations of major ions as shown in Table 22. The chemistry of this water is distinct from that of the N aquifer. Wells in the Peabody wellfield have been routinely sampled since approximately 1981; results have been provided to OSM in annual monitoring reports. Until the mid 1980's, laboratory problems produced data of uncertain quality. These problems have since been resolved, and the analytical results over the last fifteen years show only occasional "noise" and no clear temporal trends.

Four of the wells (NAV 4, NAV 5, NAV 7, and NAV 8) in the wellfield are completed in both the N and D aquifers. Based on the chemical data, the contribution to the wells pumpage from the D aquifer is small. Table 22 presents average concentrations of major ions for D aquifer well 4T-402 and the Peabody production wells. The percentage of water derived from the D aquifer is also presented, based on the mixing equation for chloride:

$$X\ Cl_{Daq} + (1-X)\ Cl_{Naq} = Cl_{sample}$$

where X is the proportion of water from the D aquifer, Cl_{Daq}, Cl_{Naq}, and Cl_{sample} are the chloride concentrations in the D aquifer, N aquifer, and the water sample, respectively. Even in the wells that are partially completed in the D aquifer, the chloride-based values are less than 2% contribution from the D aquifer, even after more than 30 years of pumping. The chloride data indicate that the percent of D aquifer-derived water is approximately 0.2% or less. The lack of a significant trend of increasing concentrations suggests that these concentrations are largely determined by pre-pumping N aquifer chemistry. The sulfate values suggest a greater contribution from the D aquifer, but may be affected by gypsum particles deposited with the quartz and other mineral grains.

Using the base-case 3D model, a prediction was made of the increase in leakance from the D to the N aquifer, assuming that pumping would occur as described in Scenarios J and K. The program ZONEBDGT (Harbaugh, 1990) was used to calculate the flow within the N aquifer within a small block encompassing the Peabody wellfield. These calculations indicate that the leakage from the D to the N aquifer within this block would increase by a factor of 1.8 between the pre-pumping period and 2007 (this factor will decrease in later years as N

Table 22

Average Concentrations of Major Ions from D and N Aquifer Wells on or near the PWCC Leasehold, and Calculated Contribution from the D Aquifer Based on Chloride Concentrations

Well	Ca (mg/l)	Na (mg/l)	Alkalinity as CaCO3 (mg/l)	Cl (mg/l)	SO4 (mg/l)	%D Aquifer (Cl)
4T-402	7.1	540	401	200.	554	100.0
NAV 2	9.5	28.5	80.3	2.0	10.5	0.25
NAV 3	4.5	37.8	82.8	1.8	5.0	0.15
NAV 4	5.2	44.2	86.5	3.6	11.4	1.06
NAV 5	3.1	61.1	107.6	4.0	20.3	1.26
NAV 6	3.9	38.5	83.6	1.5	5.4	0.00
NAV 7	4.0	48.8	86.8	3.3	17.4	0.91
NAV 8	25.1	69.2	96.8	5.2	120.6	1.86
NAV 9	4.1	33.5	71.5	1.8	4.6	0.15

Aquifer pumping is reduced). They also indicate that lateral flow into the block from surrounding N aquifer rocks would increase by a factor of about 20. Thus, the chemistry of the water pumped from the wellfield would primarily be determined from chemistry of the water in the N aquifer in areas surrounding the wellfield. The small component of D aquifer water in the N aquifer water (Table 22), even if assumed to be entirely representative of pre-pumping conditions in the N aquifer, indicates that the effect of pumping on the water quality is insignificant. This results because of (1) the limited leakage rate under non-pumping conditions (evidenced by the present water chemistry), (2) the limited increase in leakage rate (factor of 1.8), and (3) the flow dynamics produced by pumping water primarily from the N aquifer.

Based on ZONEBDGT calculations, and mixing equations, the change in sulfate concentrations in several different areas within the N aquifer basin was calculated. The results for Scenarios J and K are shown in Table 23a and 23b, respectively, and reflect the cumulative effect of pumping by PWCC between 1956 and 2038. Because of the small amount of leakage through the Carmel under natural conditions (indicated by the low TDS levels in the N aquifer after leakage from the D aquifer for thousands of years), the increase in leakage due to pumping is predicted to cause very minor changes in the chemistry of the N Aquifer water. Where natural leakage is believed to be higher (in the eastern part of the basin) based on water chemistry data), nearly 75 years of pumping is predicted to cause an increase in sulfate concentrations of about 1%. In all other areas, the increase is predicted to be less than 0.1 percent.

<u>Impact of Wash Plant Refuse Disposal on Ground Water Flow and Quality.</u> The Black Mesa Mine plans to construct and operate a coal wash plant facility. This wash plant will be used to refine the separation of coal and mine waste materials. It is estimated that the coal washing facility will produce approximately 1.38 million tons of refuse per year, comprised of a mixture of coarse (plus 100-mesh) and fine (minus 100-mesh) materials. The location and configuration of the wash plant can be found on Drawing No. 85480, Sheet 1A, Black Mesa Mine Facilities, in Volume 22 of the PAP.

Attachment 3 to this PHC contains the report entitled "Wash Plant Refuse Disposal Hydrologic Impact Evaluation Report" (Water Waste and Land, 2003). This report provides a thorough analysis of the potential impacts to the hydrologic balance that may result from disposal of wash plant refuse in two coal resource areas. The report concludes that the disposal of wash plant refuse for a short 3-year period in the final N6 pit, followed by long-term disposal of wash plant refuse in the J23 pit will result in negligible and likely unmeasurable impacts to the hydrologic balance.

Table 23

Maximum predicted sulfate concentrations (mg/L) resulting from PWCC pumping, 1956-2039.

Subarea	Initial Concentration (mg/L)		Final Concentration (mg/L)	Change
	D Aquifer	Navajo sandstone	Navajo sandstone	
Northeast	250	100	100.045	0.0448%
East	850	50	50.511	1.0229%
Hopi Buttes	360	100	100.091	0.0914%
Forest Lake	1000	30	30.060	0.2006%
Kitsillie	75	5	5.003	0.0601%
Pinon	200	10	10.006	0.0578%
Rocky Ridge	250	10	10.011	0.1149%
Preston Mesa	400	30	30.000	0.0001%
Leasehold	400	20	20.017	0.0869%
Pinon to Kitsillie	1000	45	45.034	0.0746%
Surrounding leasehold	100	50	50.001	0.0029%
Red Lake to Tuba City	400	35	35.012	0.0347%
Hotevilla to Kabito	200	140	140.002	0.0014%
Pinon to Rocky Ridge	210	70	70.006	0.0086%

a. Scenario J

Subarea	Initial Concentration (mg/L)		Final Concentration (mg/L)	Change
	D Aquifer	Navajo sandstone	Navajo sandstone	
Northeast	250	100	100.047	0.0468%
East	850	50	50.524	1.0472%
Hopi Buttes	360	100	100.093	0.0933%
Forest Lake	1000	30	30.064	0.2139%
Kitsillie	75	5	5.003	0.0630%
Pinon	200	10	10.006	0.0600%
Rocky Ridge	250	10	10.012	0.1197%
Preston Mesa	400	30	30.000	0.0001%
Leasehold	400	20	20.019	0.0937%
Pinon to Kitsillie	1000	45	45.035	0.0783%
Surrounding leasehold	100	50	50.002	0.0031%
Red Lake to Tuba City	400	35	35.013	0.0365%
Hotevilla to Kabito	200	140	140.002	0.0015%
Pinon to Rocky Ridge	210	70	70.006	0.0090%

b. Scenario K

Surface Water

Effects of Dams, Sediment Ponds and Permanent Internal Impoundments on Runoff and Channel Characteristics. Ten major dams have or will be constructed on principal tributaries confluent to Moenkopi Wash during the life of the mining operation. Portions of the drainages above as well as below the dams will be affected. The reach immediately above a dam will gradually aggrade headward as more and more water is impounded until a pool level is reached that is in equilibrium with water gains and losses. Channel reaches below the dams will become incised by smaller active meandering channels whose widths are a function of drastically reduced runoff potential, channel gradients and sediment load particle size ranges. Vegetation will begin encroaching on the edges of the new active channels as there will be insufficient runoff to remove it.

The effects of sediment ponds and permanent internal impoundments on runoff and channel characteristics will be minimal on an individual basis, but comparable to the effects of dams when considered in total. It is estimated that more than 320 sediment ponds and several permanent internal impoundments have been or will be constructed during the life of the mining operation. The internal impoundments are typically small, excepting PIIs like N2-RA, N7-D and the one impoundment proposed for the J-19 coal resource area, and most have been built on pre-law lands. Channel effects will be similar to those described for dams. Since most of the sediment ponds are on very small side tributaries, there will not be any up-drainage impacts of any significance. Because of the number of ponds and their wide range of locations, the downstream effects (active channel narrowing and vegetative encroachment) will be manifested over longer channel distances.

In addition to the permanent internal impoundments, 31 sediment control structures (see Chapter 6, Table 9) are proposed for consideration as permanent impoundments that will remain as permanent features of the postmining landscape. The total drainage area that these 31 permanent impoundments will encompass amounts to only 0.5 percent and 2.0 percent respectively of the entire Dinnebito and Moenkopi watersheds (down to each confluence with the Little Colorado River).

The impacts of the sediment ponds and dams will be of little significance as there are no local users of water for flood irrigation (see Alluvial Valley Floor section of Chapter 17). Following removal of the dams and sediment ponds, there will be certain short-term impacts to the channel reaches immediately below these structures. Sediment loads will temporarily increase as the active channel widens in response to the increased runoff potential. The increased channel bank vegetation should provide some stability during this active channel readjustment period. The potential for flood flows overtopping the channels will be negligible as the typical channel banks are 15 to 20 plus feet high above the active channel. The frequency of the larger runoff events will dictate how fast the channels reestablish themselves in quasi-equilibrium with the environmental conditions.

<u>Effects of Dams, Sediment Ponds and Permanent Internal Impoundments on Downstream Users</u>. As of January 2002, the total Dinnebito and Moenkopi watershed areas to the leasehold boundary draining to PWCC dams, ponds and impoundments are 4.08 and 63.01 square miles, respectively. There are numerous large, significant tributaries to both washes between the leasehold and the Little Colorado River. Comparing the above impounded drainage areas to the total drainage areas for both washes (2,605.3 and 812.8 square miles, respectively) suggests that this loss of runoff is of little significance at the points where the runoff water has any potential for being used for flood irrigation. As of January 2002, the impounded drainage areas on the leasehold amounted to only 0.5 percent and 2.42 percent of the total Dinnebito and Moenkopi watersheds, respectively.

Busby (1966) developed estimates of average annual runoff in the counterminous United States, including Northeastern Arizona. Based on these average annual estimates, runoff was calculated for the total watershed areas of both Dinnebito and Moenkopi washes to their respective confluences with the Little Colorado River. Average annual runoff for each basin was determined by summing the calculated runoff for partial areas defined as the watershed area lying between each pair of average annual runoff isopleths that transect the basin. The average annual runoff isopleths shown for the Black Mesa region on the Hydrologic Investigation Atlas HA-212 were used. Therefore, the lower portions of each basin were assigned an average annual runoff value of 0.1 inches, and the upper portions of each basin, including those portions in which PWCC's leasehold are situated, were assigned much higher average annual runoff numbers (1.25 to 1.75 inches). Based on Busby's empirical estimates, the average annual runoff for the entire Dinnebito basin was calculated to be 17,242 acre-feet, and 57,022 acre-feet of average annual runoff for the entire Moenkopi basin was determined.

Table 24 presents combined annual runoff measured from 1987 through 2002 at continuous flow monitoring sites SW155, SW25, and SW26, as well as annual runoff measured for the same period at the USGS Streamflow-gaging station (09401260) located on Moenkopi Wash at Moenkopi, Arizona. The runoff values are presented as acre-feet and inches of runoff. The inches of runoff for the PWCC sites were calculated by dividing the total runoff in acre-feet by the combined drainage area (in acres) above all three monitoring sites that was not controlled by PWCC dams, ponds and impoundments for each year shown (e.g., 190.25

Table 24

Measured Annual Runoff at PWCC's Continuous Flow Monitoring Sites and at the USGS Streamflow-Gaging Station 09401260, Moenkopi Wash at Moenkopi, Arizona

	PWCC Sites[1] Total		USGS Station 09401260		
Calendar Year	Total Runoff (acre-ft)	Runoff[3] (in.)	Total Runoff (acre-ft)	Adjusted Total Runoff[4] (acre-ft)	Runoff[5] (in.)
1987	3,307.2	0.31	10,030	9,230	0.11
1988	3,387.7	0.32	8,970	7,990	0.10
1989	1,475.4	0.14	3,270	2,480	0.03
1990	1,899.0	0.18	7,610	6,680	0.08
1991	276.2	0.03	1,750	1,000	0.01
1992	1,864.2	0.18	3,820	3,110	0.04
1993	414.4	0.04	8,000	7,050	0.08
1994	124.1	0.01	1,370	410	0.005
1995	1,092.7	0.11	2,720	1,790	0.02
1996	374.9	0.04	1,610	730	0.01
1997	2,860.7	0.28	8,520	7,620	0.09
1998	548.8	0.05	1,650	610	0.01
1999	1,618.1	0.16	13,810	12,870	0.15
2000	210.9	0.02	3,430	2,370	0.03
2001	800.1	0.08	14,739	13,974	0.17
2002	920.4	0.09	9,026	8,215	0.10
	Avg.	0.13		Avg.	0.06

1 - Combined Measured Annual Runoff from Sites SW155, SW25, and SW26 (PWCC Annual Hydrology Reports, 1987 - 2002)

2 - USGS records (NWISWeb, 2003)

3 - Based on the combined drainage area for all three sites (253.27 square miles) less total PWCC-impounded area during each calendar year

4 - Runoff numbers adjusted to remove groundwater baseflow component and reflect only snowmelt and rainfall runoff

5 - Based on the total drainage area for USGS Station 09401260 (1629 square miles) less total PWCC-impounded area during each calendar year

square miles in 2000) and multiplied by 12. Similarly, the inches of runoff for the USGS Moenkopi gage was calculated by first subtracting baseflow contributions from ground water discharge from each year's total measured runoff, then dividing the adjusted total runoff (acre-feet) by the total drainage area (in acres) above the gage that was not controlled by PWCC impoundments (e.g., 1565.99 square miles in 2000). The inches of runoff presented for both locations represent runoff generated from precipitation events.

For the sixteen-year period presented in Table 24, the upper sites (SW155, SW25, and SW26) averaged 0.13 inches of runoff, and the USGS gage at Moenkopi averaged 0.06 inches of runoff. The average annual runoff in inches determined from the 16-year record at the USGS gage at Moenkopi (0.06 inches) was used to estimate the average annual runoff (in acre feet) for the entire watersheds of both the Dinnebito and Moenkopi basins, and are presented on Table 25. Comparing these values (Table 25) with the average annual runoff estimated for both basins using Busby's estimates (17,242 acre-feet for Dinnebito; 57,022 acre-feet for Moenkopi), it is obvious that Busby's empirical estimates of average annual runoff for the Black Mesa region are extremely high and unrealistic compared to average annual runoff calculations that are based on local stream flow measurements.

Table 25 also presents drainage areas and average annual runoff estimates for the watershed areas draining PWCC dams, ponds and impoundments (impounded areas) within both Dinnebito and Moenkopi washes as of November 2003 and for the proposed life of mining. Impounded areas are based on summing designed drainage areas for the existing impoundments (November 2003) and those proposed for the life of mining (see Drawing 85406, Volume 22). Table 25 shows the November 2003 impounded area is 0.5 percent and 2.4 percent respectively of the total drainage areas for the Dinnebito and Moenkopi basins, and for the life of mining, the total impounded area increases slightly to 0.8 percent and 2.8 percent respectively of the total Dinnebito and Moenkopi drainage areas.

The 16-year average measured runoff at the three PWCC sites (0.13 inches, Table 24) was used to estimate average annual runoff for the November 2003 and life of mining impounded areas. The estimates of average annual runoff for the November 2003 impounded area on the leasehold is 1.1 and 5.2 percent respectively of the average annual runoff calculated for the entire Dinnebito and Moenkopi basins. Table 25 shows average annual runoff for the life of mining impounded area on the leasehold will increase slightly to 1.7 percent and 6.1 percent respectively of the average annual runoff calculated for the entire Dinnebito and Moenkopi basins. Additional impounding area for the life of mining will include construction of several temporary sediment structures proposed for the J23, J19 West, and

Table 25

Drainage Areas and Estimates of Annual Runoff

	Moenkopi Wash Basin		Dinnebito Wash Basin	
	Total Area (mi^2)	Runoff (ac-ft)	Total Area (mi^2)	Runoff (ac-ft)
Totals without PWCC Ponds	2,605.3	8,337.0[1]	812.8	2,601.0[1]
PWCC Dams, Ponds, and PII's - November 2003	62.84	435.7[2]	4.19	29.1[2]
PWCC Dams, Ponds, and PII's - Life of Mine[3,4]	73.51	509.7[2]	6.27	43.5[2]
Post-mining Permanent Impoundments[5]	57.50	398.6[2]	3.84	26.6[2]

1 - Based on 16-year average annual runoff measured at USGS Station 09401260.

2 - Based on 16-year average annual runoff measured at PWCC gages SW155, SW25, and SW26.

3 - Maximum PWCC-impounded area within Moenkopi Wash Basin occurs in 2020.

4 - Maximum PWCC-impounded area within Dinnebito Wash Basin begins in 2020.

5 - See Table 9, Chapter 6, Facilities.

N99 mining areas, and three proposed permanent impoundments in the J19, J21, and N10 reclaimed landscapes (see Chapter 6, Facilities).

Table 25 also presents the total impounded area of permanent impoundments proposed to remain in the post-mining landscape in both the Dinnebito and Moenkopi basins (see Chapter 6, Facilities, and Chapter 14, Land Use). Following final reclamation of all mining areas, PWCC's proposed permanent impoundments will comprise 0.47 and 2.21 percent respectively of the total Dinnebito and Moenkopi drainage areas. Using the annual average runoff of 0.13 inches determined from 16 years of stream flow measurements collected at the three PWCC gages, the permanent impoundments could impound about 1.0 and 4.8 percent of the average annual runoff at the lower ends of the Dinnebito and Moenkopi basins, respectively.

Based on percentages of impounded drainage areas presented in Table 24 for the November 2003, life of mining, and permanent impoundments with the total basin areas of Dinnebito and Moenkopi washes, loss of runoff in each basin is of little significance at downstream points where runoff water has any potential for being used. An alluvial farm plot and phreatophyte survey performed by Intermountain Soils, Inc. in June, 1985 documented that there is no evidence that flood irrigation was ever practiced in the past or that it is presently being practiced along the major washes and tributaries within the leasehold. All agricultural plots inspected were located on high terraces and were planted with shallow rooting cultivars, which are solely reliant on rainfall infiltration. Inspection of regional reservation land use maps indicates that flood irrigation is not practiced below the leasehold along lower Dinnebito and Moenkopi Washes other than some 70 miles below the leasehold at the town of Moenkopi. PWCC is not aware of any other diversions immediately downstream of, or further downstream for approximately 70 miles in either Dinnebito or Moenkopi Washes. Runoff from precipitation events in both washes typically occurs as flash floods, with rapidly rising water levels, high velocities, and very high concentrations of suspended solids. The channel beds and banks of both channels are subject to significant changes in width and depth as a result of runoff events, often changing appreciably during each event, which can create significant problems regarding the construction and maintenance of water diversion structures.

Comparisons of average annual runoff estimates indicate the impounded areas for the life of mining have the potential to, on average, reduce average annual runoff in the Dinnebito basin by no more than 1.7 percent, and in the Moenkopi basin by no more than 6.1 percent.

Total runoff in the basins is greater than estimated in Table 25 because of depression storage, channel transmission losses and evapotranspiration. Channel transmission losses along the sand-bed channel bottoms within the leasehold have been estimated to be quite high, potentially resulting in more than a 50 percent reduction of flow volumes during runoff events that occur along the major channels within the leasehold (see Chapter 15, Hydrologic Description).

Review of historical daily records from both the three upper PWCC sites (PWCC Annual Hydrology Reports, 1997 through 2002, see Preface to Chapter 15, Hydrologic Description) and the USGS Moenkopi gage (NWISWeb, 2002) indicate significant loss of runoff from the upper basin area can occur. From August 7 through August 8, 1987, 1,328.7 acre-feet of runoff was measured at the three PWCC gages. One large event was measured at SW155 on August 8, featuring a peak discharge of 10,100 cfs and a total runoff volume of 638.7 acre-feet. Total runoff volume measured at the USGS gage from August 8 through 9, 1987 was 668.7 acre-feet, suggesting almost 50 percent of the total runoff (1,328.7 acre-feet) from the three upper sites was lost downstream if these were the sole source of runoff recorded at Moenkopi. On August 16, 1989, summer thunderstorms generated moderate-sized flash floods at all three gages at about 1600 hours, resulting in a total runoff volume of 524.8 acre-feet. No runoff had occurred at any of the three sites for at least 6 days prior. Runoff at the USGS Moenkopi gage was only 1.3 acre-feet on the same day, and only 117 acre-feet was measured on August 17, 1998. The record comparison indicates about 77 percent of the 524.8 acre-feet of runoff generated from this portion of the basin was lost. On July 27, 1998, a flash flood passed by SW25 at a peak flow of 1,650 cfs resulting in a total runoff volume of 206.7 acre-feet. This one event was more than 37 percent of the total runoff measured at the three PWCC gages in 1998. The USGS gage measured only 14 acre-feet of runoff from July 27 through 29, 1998, indicating a loss of more than 93 percent of the 206.7 acre-feet. It is likely the 14 acre-feet measured at the USGS gage was comprised of return flow from bank storage from the upstream, 70-mile channel reach, and that the entire volume of the 200-plus acre-feet runoff event from the upper basin was lost in the channel. It should be pointed out that these comparisons assume no additional inflows to Moenkopi Wash below the leasehold occurred. This is an unlikely assumption considering that the entire basin above the USGS gage is large, and summer thunderstorms in the region often move great distances while maintaining high rainfall amounts and intensities, even though the areal extent of individual storm cells may be relatively small.

Table 24 indicates actual runoff is highly variable from year to year in both the upper and lower portions of the Moenkopi basin. Runoff variability is closely related to the highly variable climatic differences typical in this semi-arid environment, and the limited areal extent and varying intensities of the storms that do occur. From 1987 through 2002, measured annual runoff at the three PWCC gages has ranged from 124.1 acre-feet in 1994 to a high of 3,787.7 acre-feet in 1988. For the same 14-year period, measured runoff at the USGS Moenkopi gage was also lowest in 1994, but the highest annual runoff was 13,974 acre-feet in 2001. Total measured runoff at the three PWCC gages in 1988 was greatly influenced by one extremely large runoff event measured at SW25 on August 26, 1988. The peak discharge was estimated at 25,000 cfs for a total runoff volume of 1,836 acre-feet. This one event accounted for more than 50 percent of the total runoff measured at the three PWCC gages in 1988. The total runoff measured at the three PWCC gages from August 25 through August 27, 1988 was 2,624.5 acre-feet, about 69 percent of the annual total measured in 1988. For the same period, the USGS gage measured 2,945.5 acre-feet, indicating that this extreme event fell on other portions of the Moenkopi basin and contributed additional runoff to the gage some 70 miles downstream.

By contrast, the total runoff measured at the USGS Moenkopi gage in 1988 was only the fifth highest of the sixteen years presented for this gage (see Table 24). Combined total measured runoff at the three PWCC gages as a percentage of the USGS Moenkopi gage ranged widely from 5.7 percent in 2001 to 90.0 percent in 1998, illustrating the considerable variability in runoff within the basin. In fact, total measured runoff from the upper part of the basin (PWCC gages) in 2001 was only 5.7 percent of the highest annual measured runoff at the USGS Moenkopi gage (13,974 acre-feet).

Review of the measured daily records at both the three PWCC gages and USGS Moenkopi gage and the annual measured runoff shown in Table 24 suggests that 1) considerable amounts of runoff generated in the upper basin can be lost before reaching downstream locations, ranging from 50 percent of runoff events in excess of 1,000 acre-feet upwards to 100 percent for smaller events (200 acre-feet); 2) areal and temporal variability of runoff within both Dinnebito and Moenkopi basins is high; 3) channel transmission losses can significantly reduce annual runoff contributed from the upper portions of both basins; and 4) the impact of PWCC impounded areas in the upper part of both the Dinnebito and Moenkopi basins is minimal.

Peabody has monitored annual water levels and volumes in the MSHA size dams since construction, beginning with J7-DAM in August 1978. Estimates of water volumes in all ponds based on quarterly and monthly inspections were compiled for the years 1989, 1990, and 1996 through 2002. Table 26a is a compilation of the results of the above-referenced monitoring and water volume estimates. The values listed in each column are the volumes of water in acre-feet measured or estimated in the ponds and MSHA dams for each year or period presented.

Table 26a shows a 699 acre-foot increase in the amount of water impounded from 1996 to 1997, and a 465 acre-foot increase from 1998 to 1999. Assuming the increases shown for these two periods represent only surface water runoff, dividing both amounts by the total impounded area present during each period yields values of annual runoff in inches of 0.20 for 1997 and 0.13 for 1999. These values compare reasonably well with the inches of runoff measured at the three PWCC gages in 1997 (0.28) and 1999 (0.16). The 1999 annual runoff measured at the PWCC gages was only 12.6 percent of the 1999 annual runoff measured some 70 miles downstream at the USGS Moenkopi gage. Considering the variability in measured annual runoff from year to year at the upper portion of the Moenkopi basin at PWCC's leasehold compared to measurements made further downstream at the USGS gage at Moenkopi, impounded runoff in PWCC's dams, ponds and impoundments appear to have had a minimal effect on downstream runoff.

Based on the pond and dam monitoring information presented in Table 26a, the following analysis was performed to further assess the potential impact of the dams and ponds on flow volumes at the town of Moenkopi. The analysis considers whether the amount of water captured by the impoundments in a year would reach the town of Moenkopi if the total amount was due to a single, large storm at the leasehold. Further review of Table 26a indicates that one of the years with significant increases in water impounded from the previous year was 1983-1984. Five hundred eighty-seven acre-feet of additional water was impounded from overland runoff, Navajo well pumpage and pit pumpage. The latter two water sources were not considered to be a significant part of the total and were thus ignored. In Table 26a, 60 new acre-feet of water was assumed to be impounded by all the non-MSHA sized sediment ponds combined for each of the years 1978 through 1986. This 60 acre-feet added to the 1983-1984 increase in water impounded by MSHA structures yields a total of 647 acre feet of new water for that year.

The analysis approach employed moving a flow volume equal to 644 acre feet down a 70 mile length of Moenkopi Wash in a channel with a constant 80 foot flat bottom width (based on a cross section of Moenkopi Wash that is being measured and monitored within the leasehold for indirect flow calculations) as shown in Figure 20. Although flow loss to the channel banks is significant, infiltration loss through the channel bottom was the only one considered. An hourly loss rate of 1 inch per hour was used and is the lowest loss rate

TABLE 26a

Summary of Maximum Impounded Surface Runoff in

MSHA Dams and Sediment Ponds by Year

(Acre-feet)

Year	J2-A	J-7	J7-JR	J16-A	J16-L	N14-D	N14-E	N14-F	N14-G	N14-H	All Other Ponds[1]	Total
8/78-8/79		137										
8/79-8/80		117										
8/80-8/81		37										
8/81-8/82		182		**		8	**	0.5	5		60	196
8/82-8/83		180		**		80	**	2	6		60	180
8/83-8/84		425		13	220	153	**	4	40		60	855
8/84-8/85		305		4	***	150	**	4	26		60	489
8/85-8/86	*	335		10	65	153	**	4	13	2	60	582
1989-1990	42	300		50	69	107	0.1	6	35	38	305	952
1996	24	100		3	36	29	2	1	1.5	19	88	314
1997	47	338		38	82	96	**	2.7	30	50	329	1013
1998	36	140		7.5	44	53	**	0.4	15	39	295	630
1999	23.4	293		63	235	123	0.5	5.6	43	73	236	1095
2000	14.4	165		15.4	137	70	**	3.1	33	59	158	637
2001	14.2	116.2	*	44	114.6	33.9	**	2.6	21.3	30.4	233.4	611
2002	29.9	90.5	4.3	32	100.9	24.3	**	1.2	15.4	22.7	172.3	494

* Pond under construction

** Negligible amount of water impounded

*** Pond drained for repair

[1] Assumed 60 additional acre-feet impounded each year between 8/81 and 8/86

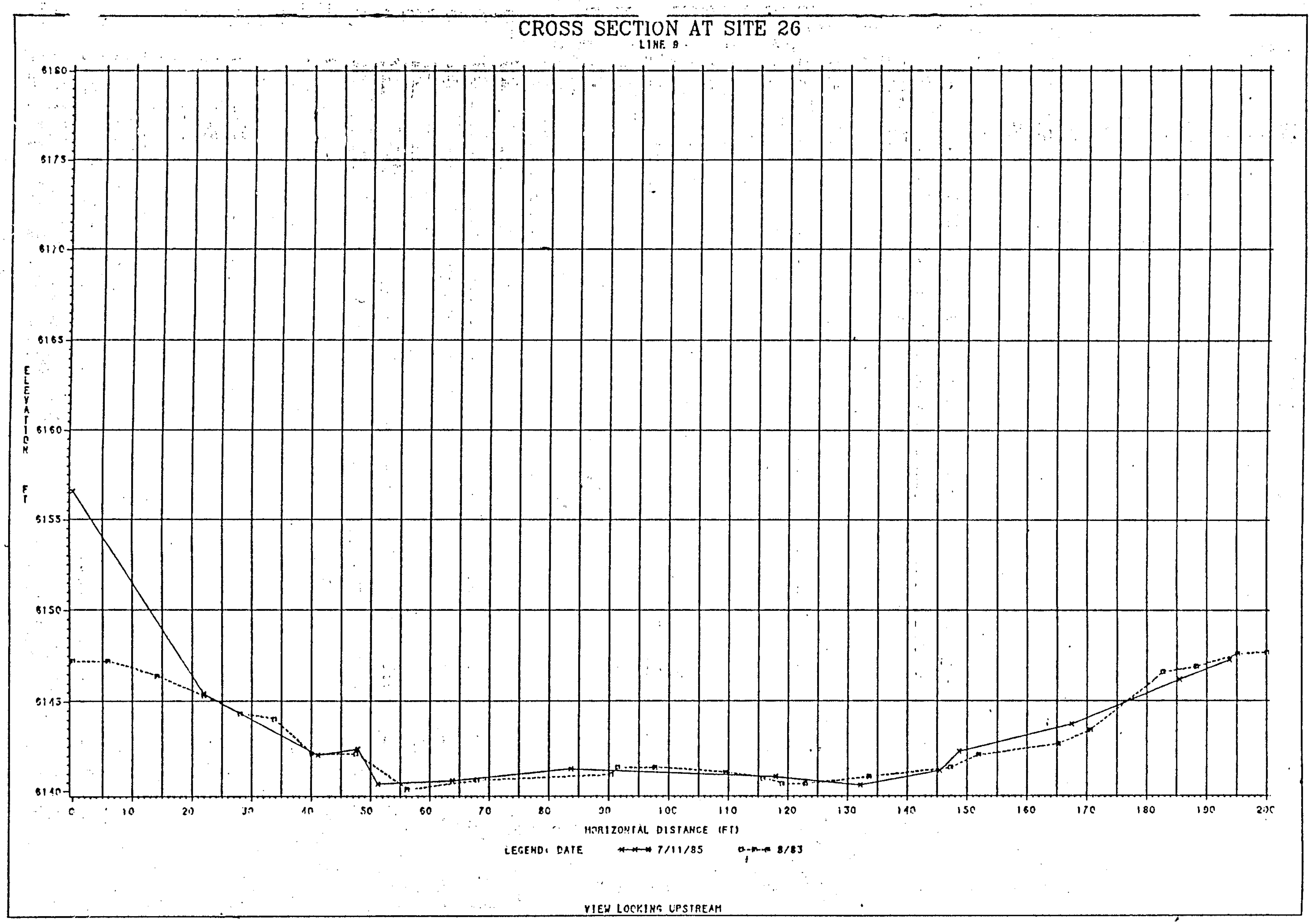


FIGURE 20 CHANNEL CROSS SECTION FOR MOENKOPI WASH ON LEASEHOLD USED FOR FLOW LOSS COMPUTATIONS DOWN TO THE TOWN OF MOENKOPI

determined from particle size analyses of bed material from the principal channels transgressing the leasehold (see Table 12, Chapter 15).

A storm runoff flow with a total flow volume of approximately 644 acre feet was computed using SEDIMOT II for a portion of Moenkopi Wash within the leasehold. Trial and error 24-hour precipitation inputs were tried until a total flow volume as close to 647 acre feet as possible was achieved. The duration of this flow hydrograph (18.4 hours, refer to Table 26b) was used to determine the minimum amount of time that an infiltration loss of 1 inch per hour would occur over each square foot of the channel bottom between Moenkopi Wash on the leasehold and Moenkopi Wash at the town of Moenkopi (a distance of at least 70 miles). Table 26c shows the infiltration loss in acre feet (14.5) for each mile that a flow with an 18.4 hour duration moves towards the town of Moenkopi. At a rate of 14.5 acre feet per mile, the entire 644 acre foot flow generated on the leasehold would be lost to channel bed infiltration before the flow had moved 45 of the 70 miles towards the town of Moenkopi.

TABLE 26c

Channel Bed Infiltration Loss for Each Hour of Flow Over the Channel Bed Area Between the Leasehold and the Town of Moenkopi

Channel Bottom Area for Each Lineal Foot in Acres	Infiltration Rate in feet/hour	Acre Feet of Flow Loss for Each Mile of Flow with an 18.4 Hour Duration
.0018	.083	14.5

The above analysis was performed using very conservative numbers. Average channel bottom widths from the leasehold to the town of Moenkopi are considerably larger than 80 feet and would account for larger infiltration losses per mile than were used. Channel bed infiltration rates are considerably higher than the 1 inch per hour rate that was used. This rate is probably more indicative of saturated flow infiltration rates. The flow duration would increase as the flow hydrograph peak lowers and the flow rate slows in the downstream direction. The 18.4 hours is the shortest time span during which flow losses over each square foot of the channel would occur. Finally the total flow volume used (644 acre feet) is extreme and is an accumulation of runoff from many storms. Individual storm volume totals lost due to the impoundments would be considerably smaller and totally lost

TABLE 26b

Discharge Hydrograph Output From SEDIMOT II Run

for 644 Acre Foot Flow Volume on Moenkopi Wash

Time (hrs)	Discharge (cfs)	Time (hrs)	Discharge (cfs)	Time (hrs)	Discharge (cfs)	Time (hrs)	Discharge (cfs)	Time (hrs)	Discharge (cfs)
1170	0.134	1460	1302.409	1750	605.403	2040	374.685	2330	279.027
1180	1.615	1470	1287.579	1760	589.178	2050	373.363	2340	277.233
1190	6.186	1480	1263.996	1770	573.179	2060	371.934	2350	275.672
1200	15.553	1490	1232.567	1780	557.571	2070	370.334	2360	274.297
1210	31.760	1500	1195.552	1790	542.437	2080	368.501	2370	273.069
1220	54.974	1510	1155.723	1800	527.854	2090	366.421	2380	271.960
1230	88.993	1520	1115.680	1810	513.939	2100	364.215	2390	270.949
1240	138.810	1530	1077.302	1820	500.777	2110	361.935	2400	270.021
1250	205.400	1540	1041.274	1830	488.194	2120	359.478	2410	269.148
1260	281.526	1550	1007.689	1840	476.169	2130	356.731	2420	268.192
1270	361.065	1560	976.513	1850	464.747	2140	353.617	2430	267.129
1280	438.975	1570	947.754	1860	453.973	2150	350.093	2440	265.948
1290	515.344	1580	921.268	1870	443.887	2160	346.190	2450	264.557
1300	600.635	1590	896.752	1880	434.526	2170	342.010	2460	262.719
1310	701.142	1600	873.816	1890	425.950	2180	337.645	2470	260.319
1320	810.924	1610	852.136	1900	418.221	2190	333.144	2480	257.228
1330	920.040	1620	831.417	1910	411.375	2200	328.525	2490	253.426
1340	1018.324	1630	811.516	1920	405.418	2210	323.828	2500	249.172
1350	1098.921	1640	792.390	1930	400.316	2220	319.122	2510	244.594
1360	1160.101	1650	773.931	1940	395.991	2230	314.440	2520	239.480
1370	1205.486	1660	755.867	1950	392.336	2240	309.839	2530	233.614
1380	1239.773	1670	738.026	1960	389.232	2250	305.361	2540	226.834
1390	1265.835	1680	720.297	1970	386.572	2260	301.042	2550	219.062
1400	1284.288	1690	702.753	1980	384.264	2270	296.924	2560	210.368
1410	1296.290	1700	685.754	1990	382.244	2280	293.063	2570	200.974
1420	1304.311	1710	669.443	2000	380.466	2290	289.519	2580	191.094
1430	1308.856	1720	653.512	2010	378.884	2300	286.337	2590	180.841
1440	1310.865	1730	637.632	2020	377.411	2310	283.536	2600	170.265
1450	1309.468	1740	621.607	2030	376.016	2320	281.111	2610	159.467

TABLE 26b (Cont.)

Discharge Hydrograph Output From SEDIMOT II Run

for 644 Acre Foot Flow Volume on Moenkopi Wash

Time (hrs)	Discharge (cfs)	Time (hrs)	Discharge (cfs)
2620	148.600	2840	18.492
2630	137.759	2850	16.234
2640	127.075	2860	14.264
2650	116.645	2870	12.497
2660	106.548	2880	10.924
2670	96.878	2890	9.513
2680	87.759	2900	8.230
2690	79.320	2910	7.067
2700	71.659	2920	6.037
2710	64.817	2930	5.050
2720	58.777	2940	4.199
2730	53.468	2950	3.483
2740	48.778	2960	2.899
2750	44.591	2970	2.404
2760	40.804	2980	2.024
2770	37.338	2990	1.717
2780	34.134	3000	1.456
2790	31.145	3010	1.228
2800	28.343		
2810	25.708		
2820	23.231		
2830	20.935		

as channel bed infiltration in shorter distances from the leasehold. Considering watershed areas, estimates of annual runoff, comparisons of daily stream flow measurements and measured annual runoff, and runoff volumes impounded, the sediment ponds and dams on the leasehold do not have any measurable impact on surface water use at the town of Moenkopi.

Effects of Dams, Sediment Ponds and Permanent Internal Impoundments on Stream-Water Quality. The effects of pond and dam discharges on stream-water quality will be negligible, because all sediment ponds and dams are designed to contain the 10-year, 24-hour runoff volumes plus sediment. Pond and dam discharges resulting from storm runoff have and should continue to be infrequent. In the event of their occurrence, PWCC will make all efforts to comply with the effluent limits and monitoring requirements of the NPDES permit (No. AZ0022179, Attachment 3, Chapter 16, Hydrologic Monitoring Program). The disposal of sediment removed from sediment ponds is conducted in a manner that protects stream water quality and is described in the section entitled "Design Methodology" of Chapter 6, Facilities.

The NPDES Permit allows pond dewatering as a means of providing sufficient detention time and storage to help ensure discharge effluent limits are met and there are no significant water quality impacts to the streams. Pond to pond pumping is also periodically employed. Seepage from dam embankments or around the sides of embankments is also presently being monitored in accordance with the NPDES Permit to document this form of pond discharge poses no significant threat to the receiving stream water quality.

Runoff discharges from the permanent internal impoundments are extremely unlikely. Should they occur, impacts to the stream-water quality will be negligible. Table 27 shows average concentrations for select chemical constituents measured in permanent internal impoundments from 1986 through 2002. Almost all the impoundments selected contain surface water runoff and have no appreciable ground-water contribution from resaturated spoil, with the exception of Pond N2-RA. Table 28 shows average concentrations for the same chemical constituents measured in stream flows generated by rainfall runoff at stream monitoring sites for the same period. Excepting pond N2-RA, water quality documented in the permanent internal impoundments is similar to slightly lower in range and magnitude compared to stream flows.

TABLE 27

Mean Concentrations of Selected Chemical Parameters Measured In

Permanent Internal Impoundments on Reclaimed Areas on Black Mesa

(1986-2002)

Monitoring Site

Parameter	116	124	118*	N1-RA	122*	123*	112*	113*	119*	N7-D	N2-RA	N2-RB	N2-RC	N8-RA
pH	8.2	7.8	8.6	9.5	8.0	7.5	7.8	7.9	7.9	8.1	8.5	8.1	8.6	8.0
TDS	459	205	144	424	143	177	281	603	165	939	11944	566	227	133
Alk	84	100	105	145	96	102	109	205	116	74	301	113	97	56
SO_4	225	68	16	180	15	21	98	252	25	595	8280	297	79	34
Ca	63	44	24	34	25	26	24	46	28	155	451	108	44	26
Mg	25	13	11	23	9	9	12	21	12	56	549	34	12	4
Na	29	4	5	69	4	7	44	117	9	41	2414	12	6	2
Cl	10	3	5	7	5	6	4	8	2	20	54	6	4	4

*Pre-law area ponds

Revised 11/21/03

TABLE 28

Mean Concentrations of Selected Chemical Parameters

Measured at Stream Station Sites on Black Mesa

During Rainfall Runoff Events

(1986 - 2002)

Stream Monitoring Site

	Dinnebito Wash		Reed Valley Wash	Yellow Water Wash				Coal Mine Wash		R.P. Valley Wash[1]		Moenkopi Wash	
	34	78	37*	50	15	157	16	18**	25	14	155	35	26
Parameter													
pH	8.1	8.0	8.0	8.0	8.0	8.2	8.1	8.0	8.0	8.3	8.3	8.1	8.0
TDS	1170	1489	1485	755	686	231	471	1335	1538	268	316	292	1109
Alk	91	87	121	86	85	111	80	123	119	92	88	68	107
SO_4	740	937	694	437	398	122	242	810	977	109	128	118	660
Ca	166	194	162	125	127	50	87	165	168	46	43	52	152
Mg	70	98	105	44	34	8	19	80	97	12	12	11	66
Na	75	98	100	19	16	4	13	104	141	15	31	5	83
Cl	17	22	213	17	10	3	8	26	20	10	11	4	38

Notes:

1 Red Peak Valley Wash

* Excludes chemical data for two samples that were influenced by magnesium chloride spills, upgradient of this monitoring site.

** Includes chemical data from sub-sites FLUM18 and CG18.

Annual Hydrology Reports (AHR's) present comparisons of recent and historical pond and stream water quality data with recommended numeric limits for livestock drinking water and other uses. Sources of the livestock drinking water limits used in the AHR's include (among others) the Navajo EPA (1999), Hopi Tribe (1998), National Academy of Science (1974), and the USEPA (1995). In the March 5, 2001 Hydrologic Monitoring Program Permit Revision package, PWCC attached the document entitled "Justification of Monitor and Monitoring Frequency Reductions at the Black Mesa and Kayenta Mines, Arizona" (PWCC, 2001). The document presents a thorough evaluation of summary statistics, water types, trend analyses, and comparisons of historical stream water quality with livestock and other use limits. Based on the livestock limit comparisons presented in the document that used total recoverable metal analyses, all stream flow generated by storm runoff is not suitable for livestock drinking water. The document also mentions, if only dissolved analyses are used for comparison purposes, most of the stream water quality is suitable for livestock drinking.

The Navajo Nation's water quality standards (NNEPA, 1999) establish livestock drinking water limits using dissolved metal analyses, with the exception of mercury (total recoverable). Using these standards, in addition to other tribal standards (Hopi, 1998) and limits recommended by the National Academy of Science for nitrate, nitrite and TDS (NAS, 1974), and by the USEPA for fluoride (USEPA, 1995), comparisons were made between permanent internal impoundment and stream flow water quality collected from 1986 through 2002. Table 29 lists the comparison results for the permanent internal impoundments, and Table 30 shows the comparison results for the stream monitoring sites. Table 29 shows that, excepting the high pH values measured in PII N1-RA and the high TDS values at pond N2-RA, the permanent impoundment water quality is suitable for use as livestock drinking water. Table 30 also indicates most of the stream flow generated by rainfall runoff is suitable for livestock drinking water, excepting very infrequent measurements of a select few parameters at Sites 16, 18, 25, 26, 34, 37 and 50. The high pH values documented in Pond N1-RA would likely be reduced by contact with soil and channel bed materials if a discharge occurs. An unlikely discharge from either Pond N1-RA or N2-RA would be diluted when mixing with the larger volumes of stream flow runoff. Due to the similarity in water quality between permanent internal impoundments and stream flows, discharges from permanent internal impoundments would not significantly affect stream-water quality, and would not change the potential stream water use.

<u>Effects of Stream Channel Diversions on Channel Characteristics and Runoff Water Quality</u>. Six channel diversions affecting approximately 6.0 miles of channel in tributaries to

Table 29

Exceedences of Livestock Drinking Water

Limits at Permanent Internal Impoundments

(1986-2002)

Analyte	Standard	No. Sites	Sites	Frequency	Exceedence Date Range	Exceedence Value Range	Exceedence Median
Aluminum, Dissolved	0.0000 - 5.0000	0	none				
Arsenic, Dissolved	0.0000 - 200.0000	0	none				
Boron, Dissolved	0.0000 - 5000.0000	0	none				
Cadmium, Dissolved	0.0000 - 50.0000	0	none				
Chromium, Dissolved	0.0000 - 1000.0000	0	none				
Copper, Dissolved	0.0000 - 500.0000	0	none				
Fluoride	0.0000 - 2.0000	0	none				
Lead, Dissolved	0.0000 - 100.0000	1	N2-RA-P	1/0/2/17	09/20/1990-10/16/1995	200.0000 - 200.0000	200.0000
						(<) 200.0000 - 400.0000	300.0000
Nitrate Nitrogen_N	0.0000 - 100.0000	0	none				
Nitrite Nitrogen_N	0.0000 - 10.0000	0	none				
Field Ph	6.5000 - 9.0000	4	N1-RA-P	11/0/0/13	01/28/1992-09/18/2000	9.1200 - 10.6100	9.4600
			N2-RB-P	1/0/0/4	01/28/1999-01/28/1999	9.1000 - 9.1000	9.1000
			PII116-P	1/0/0/2	07/28/1998-07/28/1998	9.0200 - 9.0200	9.0200
			PII118-P	1/0/0/6	05/27/1993-05/27/1993	9.4200 - 9.4200	9.4200
Ph At 25 Deg. Cent.	6.5000 9.0000	2	N1-RA-P	9/0/0/19	05/28/1987-09/18/2000	9.1000 - 10.6000	9.6000
			N7-D-P	1/0/0/10	03/04/1998-03/04/1998	6.4000 - 6.4000	6.4000
Selenium, Dissolved	0.0000 50.0000	0	none				
Solids, Dissolved	0.0000 6999.0000	1	N2-RA-P	19/0/0/22	03/05/1986-04/10/2000	7832.0000 - 18100.0000	12708.0000
Total Recoverable Hg	0.0000 10.0000	0	none				
Vanadium, Dissolved	0.0000 100.0000	1	N2-RA-P	0/0/1/18	03/05/1986-03/05/1986	(<) 500.0000 - 500.0000	500.0000
Zinc, Dissolved	0.0000 - 25.0000	0	none				

Frequency = uncensored/between MDL&PQL/censored/no. samples, (B) = Between MDL&PQL range, (<) = Censored range

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Table 30

Exceedences of Livestock Drinking Water Limits

at Stream Monitoring Sites - Rainfall Runoff

(1986-2002)

Analyte	Standard	No. Sites	Sites	Frequency	Exceedence Date Range	Exceedence Value Range	Exceedence Median
Aluminum, Dissolved	0.0000 - 5.0000	0	none				
Arsenic, Dissolved	0.0000 - 200.0000	0	none				
Boron, Dissolved	0.0000 - 5000.0000	0	none				
Cadmium, Dissolved	0.0000 - 50.0000	0	none				
Chromium, Dissolved	0.0000 - 1000.0000	0	none				
Copper, Dissolved	0.0000 - 500.0000	0	none				
Fluoride	0.0000 - 2.0000	2	37	1/0/0/25	05/07/1992-05/07/1992	2.6000 - 2.6000	2.6000
			18	1/0/0/39	09/05/1990-09/05/1990	2.4000 - 2.4000	2.4000
Lead, Dissolved	0.0000 - 100.0000	0	none				
Nitrate Nitrogen_N	0.0000 - 100.0000	0	none				
Nitrite Nitrogen_N	0.0000 - 10.0000	0	none				
Field Ph	6.5000 - 9.0000	0	none				
Ph At 25 Deg. Cent.	6.5000 - 9.0000	2	37	1/0/0/25	08/28/1999-08/28/1999	6.4000 - 6.4000	6.4000
			16	1/0/0/39	10/06/1993-10/06/1993	6.2000 - 6.2000	6.2000
Selenium, Dissolved	0.0000 - 50.0000	1	37	0/0/1/14	05/07/1992-05/07/1992	(<) 200.0000 - 200.0000	200.0000
Solids, Dissolved	0.0000 - 6999.0000	2	37	2/0/0/25	05/07/1992-11/01/1995	7600.0000 - 10170.0000	8885.0000
			25	1/0/0/29	07/21/1998-07/21/1998	7750.0000 - 7750.0000	7750.0000
Total Recoverable Hg	0.0000 - 10.0000	3	34	1/0/0/24	07/08/1998-07/08/1998	13.0000 - 13.0000	13.0000
			26	1/0/0/24	07/24/1991-07/24/1991	12.0000 - 12.0000	12.0000
			50	1/0/0/20	08/06/1991-08/06/1991	20.0000 - 20.0000	20.0000
Vanadium, Dissolved	0.0000 - 100.0000	0	none				
Zinc, Dissolved	0.0000 - 25.0000	0	none				

Frequency = uncensored/between MDL&PQL/censored/no. samples, (B) = Between MDL&PQL range, (<) = Censored range

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Moenkopi Wash have or will be constructed during the life of the mining operations. The effects of channel diversions on channel characteristics and stability will be minor for the following reasons. All diversion channels will be at least as wide as the existing channel, which should eliminate the potential for flow constrictions and excessive lateral erosion. All diversion channel slopes will approximate original channel slopes so that comparable flow velocity ranges will be maintained. Energy dissipators will be constructed at the entrance and exit points of each diversion to provide an additional control on flow velocities and erosion potential at these points. The only anticipated channel effects from the diversions would be the channel's natural tendency to reestablish meanders. This will cause some minor erosion on alternating sides of the diversion where the meandering thalweg intersects side slopes. The stability of the channel diversions will be no less than the stability of the natural channels.

The diversion channel construction activity and the natural meandering tendency of the active channel thalweg will expose fresh alluvial surfaces to weathering and erosion. This will result in additional amounts of sediment and dissolved chemicals being contributed to the streamflows. Several years of monitoring downstream from the Coal Mine Wash and Yazzie Wash channel changes indicates that natural background levels of sediment are so high that these minor additions are negligible (Chapter 15). Dissolved chemical loads have been historically quite variable. Stream water chemistry appears to be significantly affected by the portion of the watershed the flow originates in and the magnitude of the sediment load being transported by the flow. The cation exchange capacity of the sediment is high, and this does affect the flow chemistry. It is concluded that the water chemistry effects of channel diversions are minimal as they cannot be distinguished from natural fluctuations.

<u>Effects of Culverts at Road Crossings on Stream Runoff and Water Quality</u>. The effects of culverts on stream runoff and water quality will be minimal for the following reasons. All culverts or combinations of culverts are designed to pass the 10-year 6-hour flow with at least 1 foot of freeboard. If culvert exit velocities exceed six feet per second, riprapped energy dissipators will be employed to reduce the velocities. If exit velocities are between four to six feet per second, culverts will be inspected periodically for evidence of accelerated erosion immediately below their outfalls. If accelerated erosion is occurring, riprapped energy dissipators will be constructed at these points. Finally, these structures involve such minor areas of disturbance that chemical and sediment changes in the flows will be undetectable.

Removal of Pre-existing Surface Water Structures. One pre-existing surface water structure (DM-1) will be removed as a result of constructing the Reed Valley Wash channel diversion. One pre-existing structure (DM-7) was disturbed as a result of upgrading the original embankment for sediment control (K-P pond). The K-P pond will be reclaimed after permit approval. It is a redundant pond as a result of the completion of Wild Ram Valley Dam (J2-A pond) downstream. One pre-existing structure (DM-9) was impacted by construction of the main J-1/N-6 haul road. A portion of the pre-existing watershed was truncated as a result of the haul road alignment. The pre-existing watershed will not be restored because the haul road will most probably be retained as part of the postmining land use plan.

The probable hydrologic consequences of mining and related activities on 22 actual or suspected pre-existing surface water structures will be nill or inconsequential. This conclusion is reached for one or more of the following reasons: 1) minimal or no direct or indirect physical disturbance will occur at several of the pond sites or in impounding watersheds during the life-of-mine activities; 2) several sites do not actually exist; 3) several structures are non-functional due to structural failure; and 4) several structures are not applicable to this permitting action.

Interim impacts caused by the loss of the three structures previously discussed have been or will be mitigated by providing alternate water sources (N-aquifer public water standpipes and existing and proposed sediment control structures). The three structures will be replaced with one of vastly superior structural design following the completion of mining and reclamation in the affected areas.

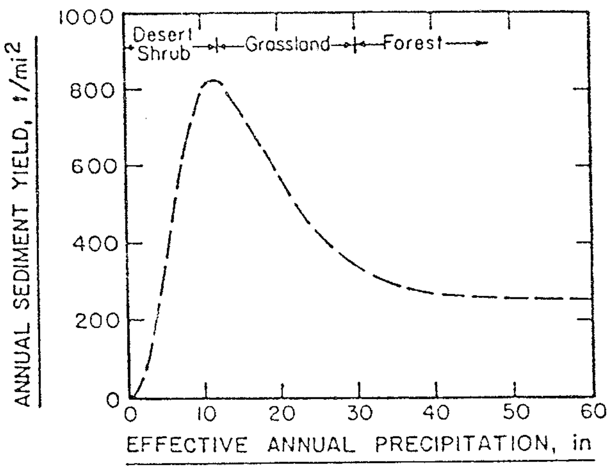
The loss of structure DM-7 will be mitigated by the retention of the J2-A pond as a permanent impoundment. The loss of DM-9 will be mitigated by the retention of several pre-law internally draining ponds in reclaimed portions of the J-1/N-6 or J-3 coal resource areas, or the retention of Ponds J3-D or J3-E as permanent impoundments. The loss of structure DM-1 will be mitigated by the retention of the J16-L sediment control structure (Reed Valley Dam) as a permanent impoundment. All the proposed permanent impoundments currently meet, or will be upgraded to meet the permanent performance standards (see Chapter 6 for design information). All proposed permanent impoundments and pre-law internally draining ponds have been demonstrated to have superior persistence capabilities and water quality (see Chapters 6 and 15 and Appendix E to Permit AZ-0001D and the 1/17/94 cover letter response, including Appendices 1 and 2, to technical Deficiency Number 3 to Chapter 16, Permit AZ-0001D).

Effects of Runoff From Reclaimed Areas on the Quantity and Quality of Streamflow. Considering the natural physiographic region in which Peabody is reclaiming lands disturbed by mining, and criteria imposed by regulatory authorities for evaluating reclamation efforts with regard to bond release, probable hydrologic consequences of runoff from post-law reclaimed areas is addressed in the following sections. Bond release criteria include the successful establishment of vegetative cover, topsoil stabilization, and the effects of runoff from reclaimed areas on the quantity and quality of waters in the receiving streams. Runoff from reclaimed areas will flow into receiving streams following the removal of sediment structures at the time of bond release.

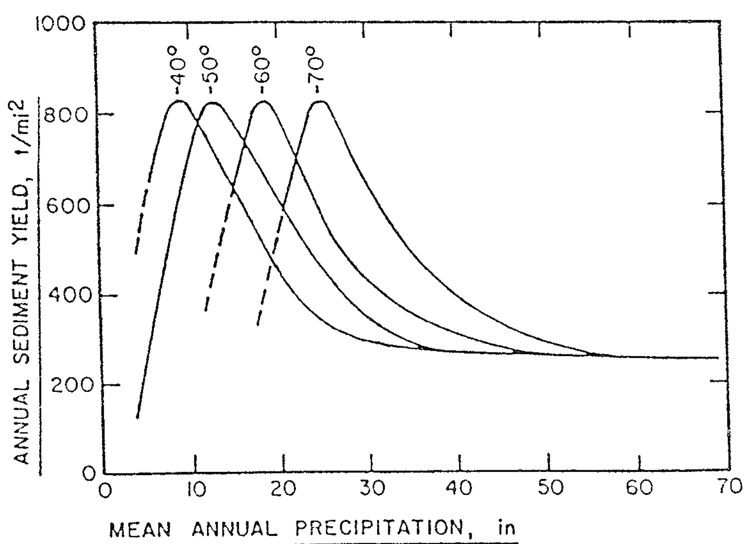
Reclamation efforts undertaken by Peabody in post-law coal resource areas on the leasehold occur in a physiographic region typified by a mild mean annual temperature (48F) and a low mean annual precipitation (10 inches). Mean annual precipitation is based on nonheated recording rain gauges. Including the contributions from snow, the mean effective precipitation on the leasehold is about twelve inches. Typical basin morphologies in the region include highly eroded landscapes of moderate to high relief, with entrenched sandbed channels and headward-cutting arroyos.

In this arid climate, intense summer thunderstorms produce flash-flooding in ephemeral channels resulting in high concentrations of sediment loads (10^5 mg/l). The highly erodible natural soils provide a significant contribution to the sediment yields produced in this climate. The limited vegetative cover in this region due to climatic and grazing conditions contributes to the flashy response of ephemeral channels from intense storms. Figure 21.a. shows a relationship among effective annual precipitation (EAP), climate and annual sediment yield (Langbein and Schumm 1958). Considering this diagram, EAP and climate on Black Mesa correlate to the highest annual sediment yields. Figure 21.b. shows the same relationship as Figure 21.a., including the effect of mean annual temperature (MAT) (Schumm 1977). MAT on Black Mesa, in combination with EAP and climate, correlate to extreme annual sediment yields. Estimates of annual sediment yields (tons/mi^2) on the leasehold, incorporating site-specific parameters into the USLE, range between 4,666 tons/mi^2 and 14,477 tons/mi^2. These estimates were made taking into account the factors that affect erosion in the region, including the typical sparse cover and highly erodable soils (see Annual Sediment Yield Estimates, Chapter 15).

Reclaimed areas created by Peabody on Black Mesa will have topography characterized by long slopes no greater than 3:1 (h:v). Topsoil material used to cover regraded spoil



a. *Variation of sediment yield with climate in the United States (from Langbein and Schumm, 1958).*



b. *The effect of mean annual temperature (°F) on the sediment yield-- climate relationship (after Schumm, 1977, p. 44).*

FIGURE 21 Climate and Sediment Yield

material will be spread to a minimum depth of twelve inches. Spoil material will be compacted to some degree during regrading, as it contains higher clay contents than topsoil material. The only suitable topsoil materials available are highly erosive due to their overall fine-sandy texture and lack of organic material, and are typical of those forming regionally under arid conditions. The "K" value assigned to topsoil material used for reclaimed areas by Intermountain Soils, Inc. personnel is .43 (Chapter 8), which confirms the high erosion potential of the topsoil.

Topsoiled reclaimed areas will feature vegetation established sufficiently to support the stabilization of topsoil material and the postmining land use of livestock grazing. Vegetative ground cover in the reclaimed areas will be similar to the native vegetation. For a discussion of vegetative ground cover and success standards for cover see Chapters 23 and 26, Permit AZ-0001D.

Discharge. The effects of runoff from reclaimed areas on the quantity and quality of waters in receiving streams will be minimal. Receiving streams on Black Mesa (Moenkopi, Coal Mine, Yellow Water, Dinnebito, Yucca Flat and Red Peak Washes) commonly yield discharges characterized by hydrographs with sharp peaks, short time to peaks, and short durations. These hydrograph characteristics become somewhat dampened downstream, as channel slopes lessen and cross section geometries increase.

Runoff from reclaimed areas should largely occur as overland flow, typified by hydrographs of gentle peaks and longer durations. With the controlled topography in reclaimed areas (slopes less than 3:1) and the modified drainage system, runoff times of concentration will be longer, resulting in reduced flow peaks and longer hydrograph durations than typical hydrographs of runoff from natural undisturbed basins on Black Mesa. External drainages will be established as part of the final reclamation, along with networks.

Runoff volumes and discharges from reclaimed areas should result in localized decreases in runoff to receiving streams. Reclaimed coal resource areas will contribute less runoff to receiving streams for similar storms than those same areas did prior to mining. Computations using SEDIMOT II to predict runoff and sediment differences from areas in the Coal Mine Wash drainage before mining and following reclamation show reductions in peak discharges and runoff volumes for an identical storm input (see Coal Mine Wash Pre- and Postmining Sediment Yield Estimates, Chapter 15, PAP). In watersheds with large portions of mined and reclaimed areas, magnitudes of the predicted decreases in peak flows range between 2 and 24 percent. Reductions in predicted runoff volumes range between 5 and 21 percent.

Topography, soils and vegetation modeled in the Coal Mine Wash drainage are typical of final reclamation that will be established in all mined coal resource areas on the Black Mesa leasehold. Based on SEDIMOT II predictions, watersheds established in reclaimed coal resource areas will typically yield reduced peak flows and runoff volumes compared to runoff from the areas before mining activities commenced. The impact of these reductions in runoff from reclaimed areas to receiving streams will be local. SEDIMOT II predictions of peak discharge and runoff volume from the entire Coal Mine Wash watershed under postmining conditions at Site 18 (includes junctions I-XIV) were only slightly less than the runoff generated under premining conditions. Predicted peak discharge and runoff volumes were reduced by only 2 percent and 3 percent respectively. Considering the order of magnitude of flows for which predicted runoff parameters were determined by SEDIMOT II up to junction XIV (10^3), these reductions are not significant. Also, junction XIV was established only a short distance downstream from these largely reclaimed watersheds in which runoff reductions were estimated at more than 20 percent.

The prediction results for modeling Coal Mine Wash drainage under pre- and postmining conditions suggest that, for a 24-hour duration storm of uniform distribution over the entire watershed, runoff reductions from reclaimed areas will be local and will result in insignificant reductions of runoff in the main channels. As runoff in the main channel systems progresses downstream, encountering additional lateral inflow from undisturbed basins, localized runoff reductions will become less pronounced and unmeasurable.

Generally, an increase in total drainage area is accompanied by an increase in watershed discharge. Reclaimed areas on Black Mesa that will drain into the Moenkopi watershed comprise only two percent of the total Moenkopi watershed above its confluence with the Little Colorado River. Slight reductions in runoff from reclaimed areas will not affect the overall runoff from this watershed area; however, runoff from the large drainage areas above the village of Moenkopi near Tuba City has been utilized for flood irrigation purposes. Reductions in runoff discharge in Moenkopi Wash from reclaimed areas on the leasehold will not be detected some 70 miles downstream in the vicinity of Moenkopi.

Busby (1966) mentions that approximately 50 percent of the runoff produced in tributaries of the Little Colorado River is lost in transmission before reaching this major channel. Channel transmission and evapotranspiration losses of this magnitude would completely mask any runoff reductions from the small, reclaimed areas on the leasehold to receiving streams.

Sediment. Sediment concentrations measured in receiving streams as part of monitoring efforts by Peabody personnel commonly range from 10^4 to 10^5 mg/l (see Peabody Sediment Monitoring, Chapter 15). Sediment yields (tons/day) have been determined on a storm basis from measured discharges and sediment concentrations made at automated stream station sites on the leasehold. Measured sediment yields range from 10^2 to 10^3 tons per day for low discharges, and up to 10^5 tons per day in higher discharges (Automated Site Sediment Yield Analyses, Chapter 15, PAP).

Channel contributions to measured sediment yields were estimated using SEDIMOT II computations (see Coal Mine Wash Pre- and Postmining Sediment Yield Estimates, Chapter 15, PAP). Using a range of storms, peak discharge and sediment concentrations were predicted for the entire Coal Mine Wash drainage above the location of Stream Station 16. These predicted values were converted to tons per day and plotted on the sediment rating curve developed from data collected at Site 16 (Figure 22). Regression lines defining the relationships among the measured and predicted values were determined and are labeled on Figure 22. Comparisons of the regression lines at various discharges suggest that sediment contributions from the channel sides and bed to the main channel sediment load could be as high as 45 percent at discharges in the range of 3,000 cfs. It can be concluded that the main channels of the principal drainages that disect the Black Mesa leasehold could contribute up to 45 percent of the total sediment load discharge during large flow events.

Due to the likelihood of intense summer thunderstorms occurring on reclaimed areas, and the highly erosive nature of topsoil material, sediment concentrations of runoff from reclaimed areas could approach concentrations comparable to receiving streams. For purposes of comparing premining conditions (undisturbed) with postmining conditions (reclaimed coal resource areas), sedimentation estimates in runoff from Coal Mine Wash have been made using SEDIMOT II (see Coal Mine Wash Pre- and Postmining Sediment Yield Estimates, Chapter 15, PAP). The drainage area above the location at which these estimates were made comprised almost 43 square miles. Sediment yield calculations were

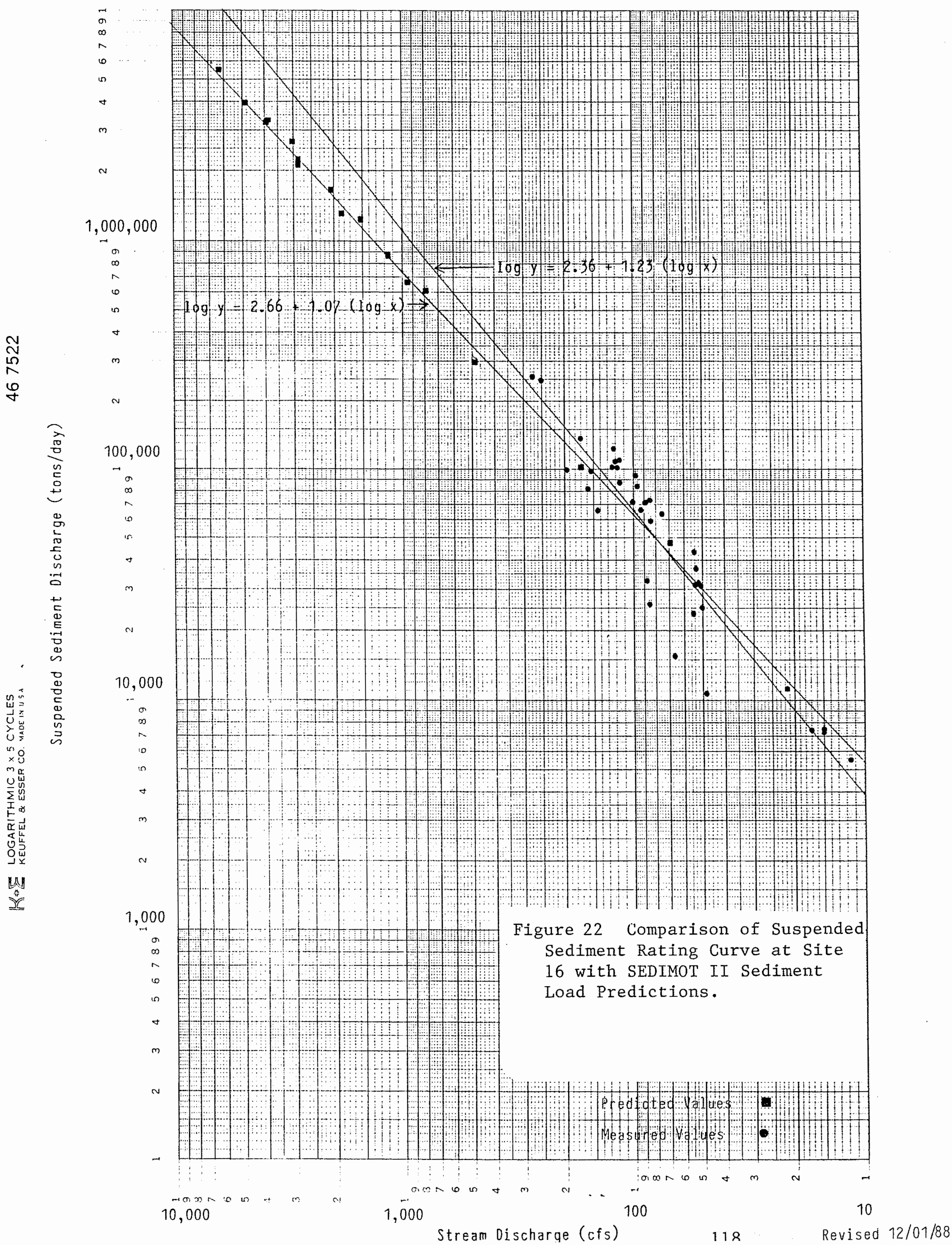


Figure 22 Comparison of Suspended Sediment Rating Curve at Site 16 with SEDIMOT II Sediment Load Predictions.

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made assuming that the outlet of this drainage area is located about one mile downstream from the N-1 reclaimed area at Stream Station 18. Results (Chapter 15) show decreased sediment concentrations (1 to 23 percent) and sediment yields (4 to 34 percent) in streamflow due to discharge from modeled watersheds within the Coal Mine Wash watershed largely comprised of reclaimed areas.

Again, reclaimed topography, soils and vegetation modeled in the Coal Mine Wash drainage are typical of final reclamation to be established in all mined coal resource areas. Watersheds established in reclaimed coal resource areas will typically yield reduced peak sediment concentrations and sediment yields compared to premining conditions. The effect of decreased sediment concentrations and yields in receiving stream runoff resulting from reclaimed area runoff will be local. Generally, as discharges increase in receiving streams, reduced sediment contributions from watersheds largely composed of reclaimed areas become less pronounced. Model predictions for the entire Coal Mine Wash watershed at Site 18 show a reduction in sediment yield (5 percent) and a 1 percent increase in peak sediment concentration for postmining conditions. The order of magnitude for both predicted parameters is 10^5, which diminishes the significance of the difference in these parameters between premining and postmining conditions.

As flow in receiving streams proceeds downstream, lateral inflow from undisturbed watersheds will contribute to sediment loads in the main channels. These additional contributions will tend to mask the localized decreases in sediment loads resulting from watersheds comprised mainly of reclaimed areas. Finally, sediment yield contributions from channel beds and sides may be as high as 40 percent, which will offset the predicted reductions in sediment loads from reclaimed areas. Channel contributions to sediment loads are predicted to completely mask the localized effects of reclaimed area contributions in the downstream direction.

Water Quality. Receiving stream-water quality has been monitored since 1981 at stream station sites on the leasehold (see Stream Water Quality Section, Chapter 15). Permanent internal impoundments (PII) established in both pre-law and post-law reclaimed areas on Peabody's leasehold have also been sampled for water quality. Previously introduced tables 27 and 28 are summaries of sample means for selected major chemical parameters. Table 27 presents mean parameter values measured in PII's from 1986 through 2000 that were constructed in both pre-law and post-law areas, and Table 28 presents mean parameter values measured at stream station sites for the same period.

Generally, PII's created in pre-law areas have water quality similar to post-law areas. Runoff flowing into PII's in pre-law areas occurs on regraded spoil material. Although post-law areas were topsoiled, comparisons using mean parameter values from post-law and pre-law PII's indicate no significant differences in the quality of water flowing over spoil material versus topsoil material.

Mean chemical parameter values from PII's are similar to but slightly lower in range and magnitude compared with stream flows, with the exception of PII's N1-RA and N2-RA. Mean pH measured in PII's range between 7.5 and 8.6 (except PII N1-RA), while stream pH values range similarly between 8.0 and 8.3. Excepting PII N2-RA, which receives a significant amount of high-TDS water from resaturated spoil in addition to runoff from reclaimed areas, mean TDS in PII's (133 to 939 mg/l) range lower than rainfall runoff measured in receiving streams (231 to 1489 mg/l). Although the mean values presented in Tables 27 and 28 indicate variability among PII's and stream flows, generally, TDS, sulfate, calcium, magnesium, sodium, and chloride are slightly lower in PII's compared with stream flows.

Tables 29 and 30 (previously discussed) indicate that water quality in most PII's and streams fall within the livestock drinking water limits (based largely on dissolved analyses of trace metals) recommended by Tribal agencies (NNEPA, 1999; Hopi, 1998), National Academy of Science (1974) and the USEPA (1995). Limited exceptions include high pH values in PII N1-RA, high TDS values in PII N2-RA, and infrequent exceedences of a limited number of the livestock drinking water limits at several stream sites.

Runoff water quality from reclaimed areas (including pre-law areas not topsoiled) will not significantly alter receiving stream water quality, nor change the potential use of receiving stream flows. Mixing of any infrequent pond discharge from PII's with the larger volumes of stream flow runoff will provide a slight diluting effect, rendering any potential impact on receiving stream water quality insignificant.

<u>The Impact of the Reclamation Plan on the Stability of Reclaimed Areas</u>. Reclamation of coal resource areas on PCC's Black Mesa leasehold occurs in a semi-arid climate. Common products of this climatic regime include flash floods in ephemeral channels resulting from very intense summer thunderstorms. Drainages exhibit high degrees of drainage densities, severly eroded landscapes of moderate to high relief, entrenched sandbed channels and the continual evolution of rills and gullies in the upslope portions of drainage basins.

No physical measurement guidelines have been found that provide distinctions between rills and gullies. Generally, gullies are classified as large rills. Quantification of the processes that form rills and gullies has not yielded conclusive results. Gullies have been classified as continuous or discontinuous (Leopold and Miller, 1956). Continuous gullies begin their downstream course with many small rills, while discontinuous gullies start with an abrupt head cut (Heede, 1975). Most rills and gullies that form naturally on Black Mesa are continuous, as abrupt head cuts in these systems are not commonplace, occurring only where lithologic controls predominate.

Several key factors contribute to the formation of rills and gullies in the semi-arid southwest. Intense thunderstorms commonly generate large raindrops that impact soil surfaces with high degrees of kinetic energy. The raindrop impacts detach soil particles, which are then entrained by overland flow. The kinetic energy imparted by very intense rainfall tends to seal some soil surfaces rapidly, concentrating overland runoff. The disruption of the soil surface and concentration of overland flow during a storm event creates an opportunity for the establishment of small rills.

Another major influence is the vegetative canopy covering the soil surface. The vegetative canopy intercepts a portion of the total rainfall volume reducing the potential for rapid runoff. The vegetative cover tends to reduce the energy of the raindrop impacts, thereby lessening the degree to which the soil surface is impacted and the quantity of detached soil particles.

The tendency of a soil to erode (detachment, also affects the degree to which rilling occurs. Sandy textured soils have a higher susceptibility for detachment than soils high in clay content. The presence of organic matter tends to provide soil cohesiveness, reducing the possibility of soil detachment. Topsoil material present on the leasehold tends to have a sandy texture and be low in organic matter and clay content.

Morphologic factors such as slope steepness, length, shape and drainage density affect the rilling process. The tractive force, a measure of detachment potential of flow, increases with slope steepness (Meyer, Foster and Romkens, 1975). Runoff increases with distance from the tops of slopes, as the contributing drainage area above increases. As the length of slopes increase so does the potential for rill and gully development. The shape of an irregular slope will affect the development of rills depending on the interrelationships of slopes and slope lengths. Natural basins will establish drainage networks of a sufficient density to carry excess runoff to the basin outlet. Although rills and gullies are small in comparison to main channels, they are an integral part of a basin's drainage network.

Many theories and concepts have been developed in the literature that explain the development of rills in gullies in semi-arid environments. Schumm and Hadley (1957) proposed a model of semi-arid erosion in which channels (including rills and gullies) adjust, by either aggrading or downcutting, to variations in sediment loads and discharge. Bergstrom and Schumm (1981) discuss a model based on the episodic behavior of a drainage basin, in which distinct zones of a watershed adjust channel characteristics in response to episodic changes in flow and sediment with time. The concept of equilibrium is discussed at length by Schumm (1977), and involves the complex process-response concept of a fluvial system.

Regardless of whether the drainage systems on Black Mesa are in quasi-equilibrium, or whether their development over time may be explained by a model, several factors influencing the development of rills and gullies in these drainages and in reclaimed areas remain constant. Intense summer thunderstorms occurring on Black Mesa generate high-energy raindrops that result in considerable soil detachment. Also, the vegetation canopy cover to be successfully established in coal resource areas will be similar to canopy covers found in the natural surrounding landscape. Topsoil material used as plant-growth media in reclaimed areas has the same erosive texture as soils found in the surrounding highly eroded landscape. Natural drainages on Black Mesa exhibit a high degree of density, naturally forming rills and entrenched gullies in the upland areas. Regardless of the extent of vegetal cover or the flatness of the regraded slopes, rills are going to form in the reclaimed areas as the basins adjust drainage to convey excess runoff. Summer thunderstorms are intense and localized resulting in overland flow that rapidly concentrates and scours in relatively short distances.

Peabody has developed a plan for insuring the stability of reclaimed areas (see Chapter 26). The key to the plan is to control those components of the surface runoff process to the extent that the potential for erosion is greatly minimized. By controlling the erosive nature of the surface runoff the degree of rilling and gullying will be minimized such that sufficient landform stability can be achieved and a successful vegetative cover can be developed that will promote the postmining land use of livestock grazing and wildlife habitat.

An important component of the plan (see Chapter 26) is to construct gradient terraces with slight positive drainage (no greater than 2 percent) on reclaimed slopes (greater than 10 percent) that have high potentials for excessive erosion and uncontrolled drainage development (rills and gullies). These terraces will break up slope lengths, limiting the upslope area contributions to overland flow. Distances over which tractive forces increase will be controlled, which will limit the scouring action of concentrated runoff in the downstream direction. By establishing limited drainage areas between the contour terraces, the size and density of rills that occur will be minimized.

Primary surface manipulations include: 1) deep ripping on all slopes ; and 2) contour furrowing using an offset disk unit that will promote infiltration and reduce excess runoff. The retopsoiled areas, including contour terraces, will be mulched with a cover crop or anchored straw or hay mulch, and then revegetated with the permanent seed mixes (see Chapter 26). Revegetation and mulching will promote soil cohesiveness as vegetation becomes established, providing further resistance to rilling.

In addition to the creation of gradient terraces and the surface treatments, a network of downdrains and main channels will be constructed. Downdrains will be established at specific intervals across the slopes for connecting the contour terraces to the main channel. Downdrains will enhance the stability and integrity of the contour terraces, as they will convey runoff from the inter-terrace areas to the main channel without promoting failure of the terraces. An important feature of the plan is the sizing and lengths of the terraces between the downdrains. Terrace embankment heights and lengths will be maximized to insure the containment of concentrated overland runoff and to increase the time of concentration of flow to the downdrains, respectively. This should greatly reduce the potential for extreme downcutting in the downdrains.

The downdrain systems will be constructed in some instances after topsoil has been replaced. Under these circumstances, topsoil will be removed at a minimum width of 45 feet to prevent topsoil loss. Ripping and disking will be implemented across the downdrain system creating a surface roughness perpendicular to flow. This will provide some resistance to scour in the downdrain. In addition, the non-topsoiled drains will contain a significant percentage of rock fragments further increasing the surface roughness.

The main channels will be engineered to convey the appropriate discharge contributed by the watershed areas drained. The main channels will range in width from approximately 45 to 135 feet which includes a fifteen foot apron on each side of the channel. The main channels and aprons will not be topsoiled to prevent topsoil loss. Application of the seed mixes will be used to revegetate and further stabilize the non-topsoiled areas.

The establishment of the drainage network outlined above will increase the overall time of concentration of flows and reduce peak flows from the reclaimed area basins. Flow velocities will be controlled, as surface manipulations, including those performed in downdrains and the main channels, provide roughness and resistance to scour. Thus, drainage development in reclaimed areas will be planned and controlled, thereby minimizing the number and size of rills. Landform stability and vegetative development supportive of the post-mining land use can be achieved, because the reclaimed area drainage development will have been controlled and reasonably stabilized rather than in a state of quasi-equilibrium between storms of large return periods as in the natural drainage system.

Summary

This chapter has presented a discussion of probable hydrologic consequences of the proposed life-of-mine mining plan. Table 30 summarizes the discussion by listing the probable hydrologic consequences and the results of the analysis of each. As can be seen, all the probable impacts have been determined to have either no impact or no short or long term significant impacts.

TABLE 31

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Ground Water		
1. Interruptions of ground-water flow and drawdown in the Wepo aquifer	Maximum 10% reduction in water levels in 2 wells partially completed in Wepo Formation	No short or long term significant impacts
2. Removal or elimination of local wells and springs	Two local wells completed in the Toreva aquifer and one spring will be removed by mining. Alternate water supply is being provided until the wells and spring are replaced	Impact during the life of the pit. Following reclamation, Peabody will replace the wells and spring. No short or long term significant impacts
3. Containment and discharge of pit inflow pumpage	Pumpage can be treated with settling basins so that discharge meets applicable standards	No short or long term significant impacts

TABLE 31 (Cont.)

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Ground Water		
4. Impact of replaced spoil material on ground-water flow and recharge	Resaturation will take from a few to as many as 100 years. Water levels will recover to near premining levels. Water is not currently used to support land use activities due to quality and yield. Alternate water supply is available.	No short or long term significant impacts
5. Impact of replaced spoil on ground-water quality	Increased levels of Ca, Mg, Na, SO_4, HCO_3, and TDS in the resaturated portion of Wepo aquifer within mining areas only. Potential for acid formation and trace element migration is minimal. Water not currently used to support land use activities due to quality and yield. Alternative water supply available. Water use category will remain unchanged.	No short or long term significant impacts

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TABLE 31 (Cont.)

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Ground Water		
6. Interruptions of Wepo recharge to the alluvial aquifer	0-20 foot localized (time and space) declines in portions of the alluvial aquifer near N-14, J-16 and J-19/20. No local use of alluvial aquifer on leasehold and water does not support critical habitat or species. Impact is transient.	No short or long term significant impact
7. Truncation of alluvial aquifers by dams	No observed impact on existing alluvial water levels since dams are mainly in small tributaries and Wepo discharges to alluvium.	No short or long term significant impact
8. Recharge of alluvial aquifer from resaturated spoil in Wepo formation	Low transmissivity in Wepo so this source has less impact than other sources of recharge (rainfall and snowmelt).	No short or long term significant impact

TABLE 31 (Cont.)

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Ground Water		
8. (Cont.)	No local use of alluvial aquifer on leasehold and water does not support critical habitats or plant species. Impact is transient.	
9. Interruptions of spring flows (Wepo or alluvial)	No Wepo or alluvial springs expected to be impacted by remaining mining operations. One spring at N-14 removed by mining has been mitigated by alternative water sources.	No short or long term significant impacts
10. Peabody wellfield pumpage reducing regional water levels and stream and spring flows	PWCC wellfield pumping will lower confined water levels basin-wide. The majority of drawdown has already occurred. Predicted drawdowns at surrounding communities are not large enough to affect aquifer productivity. At Rough Rock, where pre-pumping head was about 40 feet, PWCC-caused drawdown predicted to be only 2 feet. Water levels near the leasehold will begin to recover following the reduction or	No short or long term significant impact

TABLE 31 (Cont.)

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Ground Water		
10. (Cont.)	cessation of mining. No risk of structural damage to the aquifer. Maximum predicted reduction in baseflow of regional streams (all PWCC pumping) is 0.72 percent or less, except at Cow Springs, where a reduction of up to 1.59 percent is predicted. This assumes a low recharge rate, and is still insignificant.	
11. Impact of induced leakage from D-aquifer to N-aquifer	No evidence suggesting impacts to N-aquifer due to leakage from D-aquifer.	No short or long term significant impact
12. Impact of wash plant refuse disposal	Leachate from the refuse expected to be of similar or better quality than the Wepo aquifer. Little drainage from refuse expected.	No short or long term significant impact
Surface Water		
1. Impact of dams, ponds or impoundments on runoff and channel characteristics	Minor headward aggradation above embankments in stream. Minor incising of streams below dams.	No short or long term significant impact

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TABLE 31 (Cont.)

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Surface Water		
1. (Cont.)	Vegetation encroachment on new channels. Most ponds and dams temporary structures. Small percentage of drainage impounded and structure to be dewatered. Following removal sediment loads will temporarily increase. Channels will reestablish.	
2. Impact of dams, ponds or impoundments on downstream water users	No flood irrigation practice on or downstream of leasehold for several miles. Only 0.8 percent and 2.8 percent of total Dinnebito and Moenkopi watersheds to be dammed for life of mining. Record review does not indicate significant impacts have or will occur downstream.	No short or long term significant impacts
3. Impact of dams, ponds or impoundments on stream water quality	Infrequent discharges will meet applicable NPDES effluent limits. Discharge from permanent internal impoundments unlikely.	No short or long term significant impacts

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TABLE 31 (Cont.)

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Surface Water		
4. Impact of stream channel diversion on channel characteristics and water quality	Diversion as wide as actual channels. Slopes approximate natural slopes. Energy dissipation when needed. Construction and reclamation will temporarily increase sediment loads. Downstream monitoring shows no effect.	No short or long term significant impacts
5. Effects of culverts at road crossings on stream runoff and water quality	Proper engineering design and use of energy dissipators minimize erosion and allow adequate discharge.	No short or long term significant impacts
6. Removal of pre-existing surface water structures	Three pre-existing surface water structures will be removed by mining. Alternate water supply is being provided until the structures are replaced by permanent impoundments	No short or long term significant impacts
7. Runoff from reclaimed areas to streams	Reshaping of regraded spoils, revegetation and soil reconstruction activities result in localized decreases in peak discharge, runoff volumes, peak sediment concentrations, sediment yield and chemical	No short or long term significant impacts

TABLE 31 (Cont.)

Summary of Probable Hydrologic Consequences of the Life-of-Mine Mining Plan for Black Mesa and Kayenta Mines

Probable Hydrologic Consequences	Analysis Results	Significance
Surface Water 7. (Cont.)	constituents. However, effects will be minor compared to total flow and quality of receiving streams. Original premining conditions will likely be approximated with time following reclamation. Total disturbed area small in comparison to total watersheds.	No short or long term significant impacts
8. Impact of the Reclamation Plan on the Stability of Reclaimed Areas	Development of contour terraces, downdrains and main channels in reclaimed areas with engineering design to insure a controlled drainage development. Sediment yields and flow rates and volumes from reclaimed areas should be lower. Some maintenance may be required, particularly in pre-plan reclaimed areas.	No short or long term significant impacts

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