APPENDIX B HYDROLOGY, DRAINAGE, AND CALCULATIONS

SURFACE WATER ANALYSIS

Surface Water Analysis and Peak Runoff Determination

The drainage control systems for the Pilgrim Rock Quarry site are designed to accommodate the anticipated volume of precipitation and resulting runoff generated from the four Pilgrim Rock Quarry sub-watershed areas during the peak 100-year, 24-hour rainfall event. The entire watershed area comprises of approximately 344 acres, in three separate flow paths eventually reaching Arkansas Creek to the west.

In calculating the surface hydrology, it is conservatively assumed that all of the precipitation impacting a particular sub-area is eventually diverted into designed drainage channels with no off-site drainage from disturbed areas. It is also assumed that the design storm event will have a duration that exceeds the time of concentration of overland flow.

The regional rainfall time distribution for the location of the Pilgrim Rock Quarry was classified as being Type I rainfall. The approximately 344 acres of natural topography were then divided into watersheds delineating potential runoff flow paths and concentration points (see Figure 12 of the Reclamation Plan). The times of concentration for sheet, channel, and shallow-concentrated flow regimes were then calculated for each sub-area along the anticipated flow paths, (see Appendix B). A runoff curve number (CN) was then determined for each sub-area based on the hydrologic soil group, cover type, soil treatment, and hydrologic condition. Appendix B of the CEQA Checklist provides the detailed Hydrology calculations.

Pilgrim Rock Quarry Sub-Watersheds

The four sub-watersheds were subdivided into 11 small sub-areas in order to estimate the culverts, v-ditches, swales, and downdrains required to convey water onsite from the proposed improvements. The Rational Method was used to calculate the peak runoff generated from the design 100-year, 24-hour storm event that is consistent with the Amador County Drainage requirements. It determines the peak flow by multiplying the weighted average runoff coefficient (C), rainfall intensity (in/hr), and area (acres). The design based on the peak flow generated from this storm event is anticipated to minimize the likelihood that the storm water flows will overtop V-ditches and plug culverts. This method is widely accepted for determining storm water runoff and was used for the 11 small sub areas. In calculating the surface hydrology, it is conservatively assumed that all the precipitation impacting a particular sub-area is eventually diverted into designed drainage channels. It is also assumed that the design storm event will have a duration that exceeds the time of concentration of overland flow. The surface water analysis was performed on the sub-areas as illustrated in Appendix B; Culvert and Channel Design, of the watershed and included run-on and proposed grading for one drainage swale culvert crossing and new surface water retention basins, V-ditches, downdrains, and the sizing of the sedimentation basins. Appendix B provides details for all the aforementioned site improvements.

SURFACE WATER ANALYSIS

The weighted runoff coefficient was estimated by using a runoff coefficient of 0.20 for undeveloped areas and 0.75 for developed quarry areas. The site is fairly to moderately vegetated with native and non-native grasses. The quarry site will contain various gravel surfaces based on the subsurface soil conditions, lined channels/ditches and road surfaces. Hence, the weighted runoff coefficient was estimated at 0.38 and used in our analysis in calculations detailed below.

Results of the Analyses

The design 100-year, 24-hour storm event will generate an estimated average peak flow of 79.6 cfs over the entire contributing project watershed area. Phases 1 through 5 (Figures 7 through 11) have sedimentation basins incorporated into the end of phase designs and will remain in place until the final floor grades have been achieved. The sedimentation basin for each phase will have sufficient capacity to collect and retain surface water runoff on site. All runoff will be contained on site and will be used for processing, dust suppression, and landscaping during the Pilgrim Rock Quarry operations. All runoff will be directed away from the Pilgrim Rock Quarry excavation area side slopes. The Pilgrim Rock Quarry is planned to maximize the use of access roads to control the runoff as well as provide appropriate side-slope stability.

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DESIGN STORM VOLUME CALCULATIONS

Time of Concentration to Peak (Tp) Tp = 0.6*Tc

	A	B	C	D	
Tp	0.121	0.118	0.125	0.031	hours

Time Base (Tb) Tb = 2.67*Tp

Tb	0.32	0.32	0.33	0.08	inches/hr

Design Storm Volume (Vd) Vd = 1800*(Qd*Tb)

Vd (100)	35,884	30,489	84,184	9,488	cf
Vd (25)	28,707	24,392	67,347	7,591	cf
Vd (1)	26,554	22,562	62,296	7,021	cf

Vd (100)	0.82	0.70	1.93	0.22	Ac-ft
Vd (25)	0.66	0.56	1.55	0.17	Ac-ft
Vd (1)	0.61	0.52	1.43	0.16	Ac-ft

(1) Storm Volumes evaluated using the Amador County Erosion & Sediment Control Guidelines for Developing Areas of the Sierras (High Sierra RC&D Council, November 1981).

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Culvert and Channel Design

Hydraulic Design of Open Channels

Open Channel Flow, Max. Allowable Velocity: 5 feet per second for Coarse Gravel

Open Channel Flow, Max. Allowable Velocity: 10 feet per second for Fully Lined

Manning's Eqn. Q = (1.49*A*R2/3*Sf1/2) / η

Manning's Roughness Coefficient = 0.03 (Natural Channels, smooth, recently constructed)

0.04 (Rip Rap and/or Natural Channels with Stones and Weeds.

Channel No.	Sub Areas (1)	Channel Type	Roughness Coefficient η	Peak Flow Q_p (cfs)	Channel Slope Sf = So	Cross Sectional Area A (sf)	Wetted Perimeter P (ft)	Hydraulic Radius R (ft)	Channel Side Slope (H:1) (ft) (4,5) Left (ft)	Right (ft)	Channel Depth d (ft)	Q (Solved) Capacity (cfs)	Q (Design) Required (cfs)	Percent Full	Velocity V (fps)	Maximum Allowable V_{MAX} (fps)
V1	Access Road	V-ditch	0.040	61.8	0.010	18.0	13.416	1.342	2	2	3.00	81.6	61.8	0.76	4.5	10
V2	Access Road	V-ditch	0.040	61.8	0.020	15.1	12.298	1.230	2	2	2.75	91.5	61.8	0.68	6.0	10
V3	Access Road	V-ditch	0.040	61.8	0.040	12.5	11.180	1.118	2	2	2.50	100.3	61.8	0.62	8.0	10
V4	Access Road	V-ditch	0.040	61.8	0.060	10.1	10.062	1.006	2	2	2.25	92.8	61.8	0.67	9.2	10
V5	Access Road	V-ditch	0.040	61.8	0.080	8.0	8.944	0.894	2	2	2.00	78.2	61.8	0.79	9.8	10
V6	Access Road	V-ditch	0.040	61.8	0.090	6.8	8.273	0.827	2	2	1.85	67.4	61.8	0.92	9.8	10
V7	Access Road	V-ditch	0.040	61.8	0.100	7.7	11.068	0.692	3	2	1.75	70.5	61.8	0.88	9.2	10
V8	Mine Road (see Note 7)	V-ditch	0.040	***14.0***	0.100	4.5	6.708	0.671	2	2	1.50	***40.6***	***14.0***	0.34	9.0	10
V9	Mine Pond Road (see Note 6)	V-ditch	0.040	***34.9***	0.083	5.1	6.364	0.795	1	1	2.25	***46.7***	***34.9***	0.75	9.2	10
V10	Mine Floor	Swale	0.040	139.6	0.010	40.0	40.200	0.995	10	10	2.00	148.5	139.6	0.94	3.7	10
V11	Mine Bench (see Note 7)	V-ditch	0.040	***14.0***	0.010	2.3	0.671	3.388	0.5	50	0.30	19.1	14.0	0.73	8.4	10

Notes:

1. The sub areas are shown on Figure for Hydrologic and Drainage Analysis.

2. Steady state, uniform flow conditions assumed.

3. Not Used.

4. The maximum channel side slope is 2H to 1V except for pond access and mine bench.

5. Depths shown represent top of lined material/finish grade and not top of subgrade.

6. Used 25% of the peak flow (% capacity) to design channels that is representative of contributing area above.

7. Used 10% of the peak flow (% capacity) to design benches and road that is representative of contributing area above.

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Culvert and Channel Design

Open Channel Flow, Max. Allowable Velocity: 5 feet per second for Coarse Gravel

Hydraulic Design of Culverts and Downdrains

Location [1,2]	HW/D	Diameter (D)		Flow (Q)	Headwater (HW)	Pipe Material[5]	Ke	Ko	Length (L)
	(FT)	(FT)	(IN)	(CFS)	(FT)				(ft)
CULERTS									
C1 (A, use 2 Culverts)[3]	1.7	3.0	36	61.8	5.1	CMP	0.5	1	150*2
C2 (B, use 2 Culverts)[3]	1.5	3.0	36	53.6	4.5	CMP	0.5	1	104*2
C5A (C, not referenced)	3.0	3.0	36	140.0	9.0	CMP	0.5	1	UNK
C5B (C, use 2 Culverts)[3]	2.0	3.0	36	70.0	6.0	CMP	0.5	1	UNK
C3[4]	0.85	1.5	18	5.0	1.3	CMP	0.5	1	120
C4[5]	0.85	2.0	24	10.0	1.7	CMP	0.5	1	74
C6	1.1	2.0	24	15.0	2.2	CMP	0.5	1	UNK
DOWNDRAINS									
D1 (Ph1&2, North as needed)	1.4	2.0	24	20.0	2.8	CMP	0.5	1	175 each
D 2 (Ph 5, East)	1.0	2.0	24	11.1	2.0	CMP	0.5	1	260
D3 (Typ. SS DD)	0.8	1.3	15	2.6	1.0	CMP	0.5	1	3782

Notes:

1. Location points for culverts are shown on the Hydrologic Plan drawing and Grading Plans.

2. Culverts are designated as "C" and downdrains "D".

3. Use 2 - 36" diameter culverts.

4. Peak flow of 5 cfs was used based on reduced contributing area above site providing 1 cfs maximum.

5. Peak flow of 10 cfs was used based on reduced contributing area above site providing 5 cfs maximum.

6. Culverts and Downdrains are designed using corrugated metal pipe (CMP) and FHWA nomograph, Chart 2, Inlet Control

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PEAK FLOW CALCULATIONS

	A	B	C	D		
L	0.568	0.432	0.732	0.225	miles	From Figure 1
H	140	65	272	293	feet	Mean Annual Precipitation 21 inches
Tc	**0.201**	**0.197**	**0.209**	**0.052**	hours	
	12.1	11.8	12.5	3.1	minutes	
C	0.36	0.35	0.39	0.38	unitless	
I (100)	2.50	2.50	2.50	2.50	in/hr (1)	
A	69.52	62.03	145.05	66.93	Acres	
Q (100)	**61.8**	**53.6**	**139.6**	**63.4**	cfs	
Q (25)	**49.4**	**42.9**	**111.7**	**50.7**	cfs	I (25) = 2 in / hr
Q (10)	**45.7**	**39.7**	**103.3**	**46.9**	cfs	I (10) = 1.85 in / hr

(1) Average rainfall intensity (inches per hour) for a given frequency and for a duration equal to the time of concentration Tc (see Fig 2).

(2) Mean Annual Precipitation depth is 21 inches from Figure 1.

(3) Peak flow evaluated using the rational method per Amador County Erosion & Sediment Control Guidelines for Developing Areas of the Sierras (High Sierra RC&D Council, November 1981).

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PEAK FLOW CALCULATIONS

Soil Properties (per NRCS and Amador County)

	A	B	C	D	
AsD	0	0	53	56.5	
Mn	14.1	3.5	0	0.0	
RbD	2.7	0	0	0.0	
Sa	50.7	42.4	110.2	10.4	
Total	**67.5**	**45.9**	**163.2**	**66.9**	Acres

Coefficient of Runoff- C

Area A	Unimproved	Improved	Total
Area	53.5	14	67.5
Slope	0.11		
Surface Perm	0.1		
Vegetation	0.07		
Surface	0.09		
Total	0.37	0.30	
C*A	19.80	4.20	
Weighted CA	**0.36**		

Area D	Unimproved	Improved	Total
Area	28.3	38.6	66.9
Slope	0.11		
Surface Perm	0.1		
Vegetation	0.05		
Surface	0.09		
Total	0.35	0.40	
C*A	9.92	15.44	
Weighted CA	**0.38**		

Area B	Unimproved	Improved	Total
Area	29.9	16	45.9
Slope	0.11		
Surface Perm	0.1		
Vegetation	0.07		
Surface	0.09		
Total	0.37	0.30	
C*A	11.06	4.80	
Weighted CA	**0.35**		

Area C	Unimproved	Improved	Total
Area	81.2	82	163.2
Slope	0.11		
Surface Perm	0.1		
Vegetation	0.07		
Surface	0.09		
Total	0.37	0.40	
C*A	30.04	32.80	
Weighted CA	**0.39**		

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Sedimentation Basin Capacity

NRCS Eqn. for shallow concentrated travel time, Ti =(L/3600*V); Vi=16.1345(S0.5) and for sheet flow, Ti =0.007·(nL)0.8/ [P20.5 S0.4]

Peak Flow and Basin Volume

Sedimentation Basin Structure [1]	Intensity	Runoff Coefficient	Drainage Area	Peak Flow (Q=CiA)	Required Volume[2]	Req'd Volume Freeboard	Actual Volume[4]
	(in/hr)		(acres)	(cfs)	(cf)	(cf)	(cf)
Phase 1 (Build out)	2.50	0.39	145.1	139.6	84,184	105,230	117,670
Process Area	2.50	0.36	69.5	61.8	35,884	44,855	64,179
Phase 4 & 5	2.50	0.38	66.9	63.4	9,488	11,296	180,411

Notes

1. The sedimentation basin structures are designed to accommodate a 100-yr, 24-hr event in accordance with Amador County Standards.

2. The required volume is based on the design storm volume per the Amador County Standards.

3. Not Used.

4. Volume of a trapazoid is: $Vol_{Trap} = L*(B_{bot}+(B_{top}-B_{bot}t)*(H_{water\ level}/H_{top})+B_{bot})*H_{water\ level}/2$ Actual volume parameters are shown in table below.

Side slope: 1V to H:	2.0	ft/ft
Freeboard:	2	feet

Sedimentation Basin Design Parameters

Storage Volume [2]	Dimension (ft) [1], [3]				Length-to-Width Ratio		Elevation at bottom of Sed Basin (ft msl) [5]
	Length (L)	Top Width (W_{top})	Bottom Width (W_{bot})	Depth ($H_{to}p=H_{water\ level}$) (D)[4]	L	W_{top}	
Phase 1 (Build out)	205	103	51	8	2.0	to 1	***~190***
Process Area	218	56	24	8	3.9	to 1	***~239***
Phase 4 & 5	220	110	55	10.5	2.0	to 1	***~177***

Notes

1. The storage capacity was determined using a 100-year 24-hour storm event.

2. Not Used

3. The sedimentation basin side slope is 2H to 1V.

4. Basin capacity includes free board.

5. Bottom elevation of basin (maximum elevation) is based on deepest basin depth in table and shall be backfilled to a minimim elevation of 175 ft. msl at end of use.