R12

PRELIMINARY DRAFT DRAINAGE RELIEF STUDY

for

Reno Rendering Plant
Manogue High School
Evans Avenue-UPRR
Our Mother of Sorrows Cemetery

A Phase of the Paradise Pond Drainage Relief Program

RENO, NEVADA

November, 1989

prepared for:

CITY OF RENO NEVADA

Reno/Sparks, Nevada
Las Vegas, Nevada
Phoenix, Arizona

sea incorporated
CONSULTING ENGINEERS

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by

SEA, INCORPORATED
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950 Industrial Way
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I. INTRODUCTION

The drainage basin under study is part of the Paradise Pond Watershed. A master plan hydrologic analysis of the Paradise Pond Watershed was prepared for the City of Reno in August, 1986. The master plan addressed watershed problems and needs, and recommended improvement alternatives.

This study was undertaken under the authority of the City of Reno for the purpose of providing analysis and design services to address a subbasin within the Paradise Pond drainage area, specifically, the Manogue High School Union Pacific Railroad - Evans Avenue - Reno Rendering Plant drainage basin. The need for this study was prompted by the proposed replacement of Union Pacific Railroad's culverts above Manogue High School with larger pipes, which could intensify the critical flooding problems currently being experienced by Manogue High School. It is the City's intent that the entities affected by the various flooding problems within the study area cooperate in the construction of flood control improvements, with the purpose of this study being to determine the most cost effective improvements alternative to relieve the existing flooding problems.

The project scope includes providing a detailed study of the subject subbasin in order to formulate recommendations for flood control improvements. This involved the collection and review of existing information, hydrologic analysis of the basin using the U.S. Army Corps of Engineers HEC-1 Flood Hydrograph package computer program, selection of possible detention basin sites, soils investigations at the possible detention basin sites, and routing analysis along the flood route. Opinion of probable construction costs were developed for selected alternatives along with the analysis of the various retention and routing combinations in light of degree of protection, and constructability of the improvements.

II. EXISTING DRAINAGE PATTERNS

A review of the existing drainage basin hydrology, the existing flood control improvements, and the current land ownership, was made to form a basis for the flood control improvement needs for the study area and the feasibility of constructing such improvements.

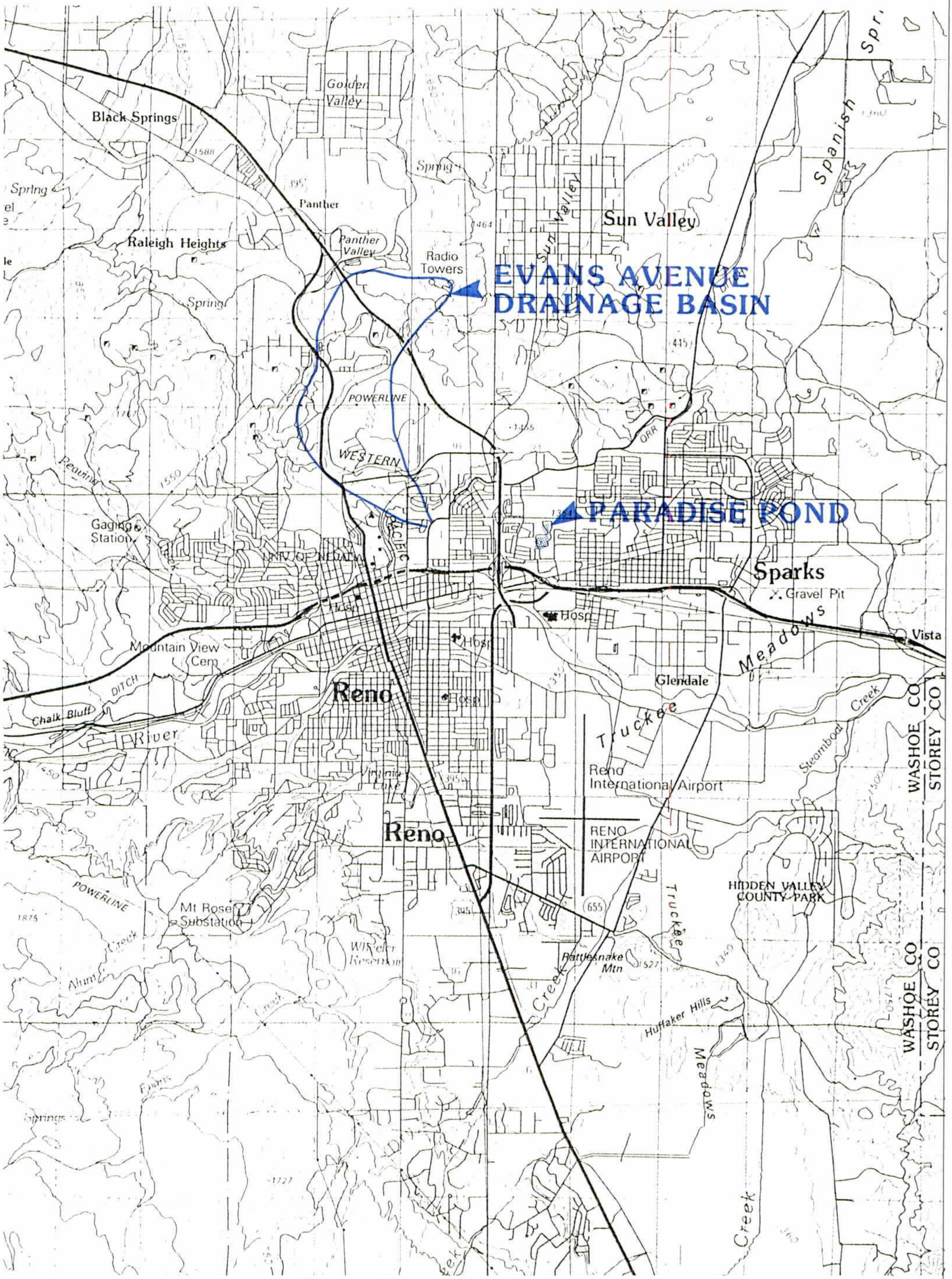
- **Study Area**

The Evans Avenue Drainage Basin is located in the northeastern portion of Reno, Washoe County, Nevada. The drainage basin covers 2.81 square miles in Sections 23-24, 26-27, 34-36, of T20N, R19E; Sections 1 and 2 of T19N, R19E, M.D.B.&M. (see Plate No. 1., "Vicinity Map")

The drainage basin generally slopes in a southeasterly direction. The elevation varies from 5,455 feet at the basin's upper reach to 4,500 feet at the Reno Rendering Plant and to a water surface elevation of approximately 4,438 feet at Paradise Pond.

Development within the basin varies from sparsely developed in the upper reaches north of our Mother of Sorrows Cemetery to highly developed with high density residential properties east of the Reno Rendering Plant and industrial development along Parr Boulevard.

The soils in the Evans Avenue Drainage Basin typically have a high percentage of clay. The Soil Conservation Service places this type of soil under its Hydrologic Soil Classification "D", which is soil with very low permeability or capacity to retain water and thus a high runoff rate. The vegetative cover in the undeveloped areas is predominately sparse to moderately dense sagebrush.



VICINITY MAP

PLATE 1

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- **Existing Facilities**

All drainage for the undeveloped upper reaches of the basin bordering Panther Valley is served by Nevada Department of Transportation (NDOT) designed culverts under Interstate 80 (I-80). NDOT design standards call for 100-year return period storm capacity.

Below I-80, the drainage is carried in natural open channels to the Parr Boulevard area where the existing drainage facilities are designed for a 5- or 10-year return period storm event. There are no current facilities to convey the 100-year storm event nor are there any existing detention facilities.

Below Parr Boulevard, the drainage is carried in a natural open channel east of and adjacent to U.S. 395 to Comstock Drive. A 48" diameter (ϕ) reinforced concrete pipe conveys the flows under Comstock Drive which has the capacity to convey approximately the 5-year storm event.

Below Comstock Drive, the drainage is carried in a natural open channel to McCarran Boulevard which is served by an 84" diameter corrugated metal pipe. This culvert has the capacity to convey approximately the 100-year storm event.

Below McCarran Boulevard, the drainage is carried in a natural open channel of moderate to steep slope to a 96" corrugated metal pipe under the University of Nevada, Reno (UNR) access road. This culvert has the capacity to carry approximately the 100-year storm event.

Below the UNR access road, the drainage is carried in a natural open channel to Evans Avenue. Evans Avenue is served by a 72"ϕ corrugated metal pipe. To convey the 100-year storm event, Evans Avenue acts as a detention dam with a storage of 26 acre-feet. It should be noted that the Evans Avenue embankment was not designed as a detention dam.

Below Evans Avenue, the drainage travels approximately 70 feet in a natural open channel to two 48"ϕ corrugated metal pipes under the Union Pacific Railroad. To convey the 100-year storm event, the Union Pacific Railroad embankment acts as a detention dam with a storage of 10 acre-feet. The railroad embankment also was not designed as a detention dam.

Immediately below the 48"ϕ culverts, the drainage is served by a 24"ϕ storm drain pipe under the Manogue High School athletic field. Due to the pipe's limited capacity (less than the drainage from a 5-year storm event), the storm water flows overland across the athletic field. The 24"ϕ storm drain pipe goes directly into a 72"ϕ reinforced concrete pipe crossing Valley Road. This pipe has the potential to carry the 100-year storm event if connected to the railroad embankment culverts.

From the outlet of the 72"ϕ storm drain under Valley Road, the drainage flows into a natural open channel to the Reno Rendering Plant. At the intersection of this channel with the Orr Ditch immediately below Valley Road, the drainage flows under the Orr Ditch via a 6'-9½"x4'0" reinforced concrete box inverted siphon.

In Oddie Boulevard below the Reno Rendering Plant, the drainage is conveyed in a 22"x36" corrugated metal arch pipe which has a capacity less than the 5-year runoff. At the northwest corner of Sutro Street and Oddie Boulevard, the culvert changes to a 36"ϕ storm drain which also has less than 5-year runoff capacity. At the northeast corner of Sutro Street and Oddie Boulevard, the culvert increases to a 42"ϕ reinforced concrete pipe storm drain. The 42"ϕ storm drain has the capacity to carry approximately one-half of the 5-year storm event. At Montello Street, the storm drain increases to a 48"ϕ reinforced concrete pipe which continues to Silverada Boulevard where it is upsized to a 54"ϕ reinforced concrete pipe. At its terminus at Paradise Pond, the pipe size is increased to a 66"ϕ reinforced concrete and corrugated metal pipe. Several storm drain laterals intersect the Oddie Boulevard system conveying local drainage from the area north of Oddie Boulevard.

A summary of the existing facility capacities is shown in Table 1. The Existing Facility Map is included in Appendix F. The Existing Facility Capacities Calculations are included as Appendix 3.

TABLE 1

EXISTING DRAINAGE FACILITY CAPACITY SUMMARY

FACILITY & LOCATION	APPROXIMATE CAPACITY CFS	RETURN PERIOD
48"ϕ RCP under Comstock Drive	120	< 5-yr.
84"ϕ CMP under McCarran Boulevard	650	100-yr.
96"ϕ CMP under UNR access road	650	100-yr.
72"ϕ CMP under Evans Avenue	586	100-yr.
2-48"ϕ CMP's under UPRR	558	100-yr.
72"ϕ RCP under Valley Road	670	100-yr.
42"ϕ RCP in Oddie Boulevard	110	< 5-yr.

- **Identified Flooding Problem Areas**

The following areas were identified as flooding problem areas:

1. Parr Boulevard - This area appears to have flooding potential but was not identified as a problem area at this time by the City of Reno Engineering Department.

2. Our Mother of Sorrows Cemetery - The Cemetery, which is located at the northeast corner of North Virginia Street and McCarran Boulevard, has experienced frequent flooding in the recent past. The Catholic Diocese of Reno desires flood protection for their cemetery plots and their lower access road. The Diocese also desires flood protection for their future expansion which includes additional cemetery plots.

3. <u>Evans Avenue and Union Pacific Railroad Embankments</u> - These embankments were not designed for flood water detention. The Union Pacific Railroad embankment failed in the recent past, prompting the railroad to construct twin 48"ϕ culverts under the embankment.

4. <u>Manogue High School</u> - The discharge from the Union Pacific Railroad culverts under their embankment exceeds the capacity of the 24"ϕ reinforced concrete pipe under the athletic field. This causes the excess runoff to flow overland across the athletic field, leaving flood debris on the field.

5. <u>Reno Rendering Plant</u> - The Reno Rendering Plant located northwest of the intersection of Oddie Boulevard and Wells Avenue has experienced flooding problems due to its proximity within the flood path of the subject basin.

6. <u>Residential Area</u> - Bordered by Oddie Boulevard to the north, U.S. 395 to the east, I-80 to the south, and Sutro Street to the west. All runoff which exceeds the Oddie Boulevard system capacity crosses Oddie Boulevard and flows south, unable to drain naturally to Paradise Pond. This condition, combined with poor drainage within the residential area, has caused flooding problems in this area.

- **Land Ownership**

The current assessor's parcel maps were used to identify parcels and property owners along the flood route below Parr Boulevard and above Reno Rendering Plant which were potential sites for a detention basin or routing improvements. The parcel owners were identified to assist in determining potential acquisition feasibility. The Land Ownership Map is included as Appendix L.

III. PROPOSED FLOOD CONTROL FACILITIES

Flood hydrographs were developed and then routed through several detention/conveyance alternatives to form the basis for flood control improvement recommendations.

• Methodology and Criteria

The methodology developed by the U.S. Army Corps of Engineers at the Hydrologic Engineering Center (known as the HEC-1 Flood Hydrograph Package) was selected to compute the peak storm water flows.

A complete discussion of the applied HEC-1 method may be found by reviewing the HEC-1 Flood Hydrograph Package Users Manual as published by the U.S. Army Corps of Engineers Water Resources Support Center, dated September 1981 (revised January 1985). The following is a general outline of the data used in the procedures:

1. Physical Characteristics of the Drainage Region

   a. Drainage Area
   b. Overland slope
   c. Soil type and land use

2. Drainage characteristics

   a. Lag time - the estimated time from the centroid of rainfall to peak discharge (hr.).

   b. SCS (Soil Conservation Service) Hydrology Curve Number (CN) - a number that reflects the percentage of storm rainfall that is converted to runoff; as the curve number increases, runoff increases.

3. Rainfall Quantities

   a. Rainfall Depth - the total depth of rainfall estimated to fall during a storm.

   b. Rainfall Temporal Distribution - the distribution of the total rainfall depth over the duration of the rainfall event.

The input data for the HEC-1 model was based upon the master plan report prepared for the City of Reno titled "A Hydrologic Analysis of the Paradise Pond Watershed" - August 1986. Upon reviewing the master plan, the following items were noted:

1. The master plan stated that the runoff from subareas R, N, O, and Q, flowed into the Orr Ditch and therefore did not contribute to the runoff flowing into Paradise Pond. It also stated that the runoff from Subareas C and H percolated into the ground, thereby not contributing runoff to the Paradise Pond Watershed. SEA believes that in the event of a severe storm, Subareas C, H and O will contribute runoff to the drainage basin being studied.

2. The master plan developed "present condition" Soil Conservation Service curve numbers and "future condition" curve numbers for the drainage areas, with the "present condition" curve numbers selected to be used. Based upon discussions with the City of Reno Engineering Department, it was decided to use the "future condition" curve numbers in this study.

3. In computing the subarea's time of concentration and lag time, the master plan uses velocities for the pipe reaches which are extremely high. The velocities were computed using Manning's formula based on full flow. In this study, it was assumed that during a severe storm event, the pipes would be unable to carry the runoff and overland flow would

predominate. Making this adjustment would tend to increase basin lag times.

4. The master plan routes the runoff from Subarea J into a 30"ϕ RCP which flows west down Enterprise Road into the Evans Avenue system. SEA believes that during a severe storm event the runoff will cross Enterprise Road and continue in a straight line manner down Valley Road.

5. In determining the routing travel times, the master plan used average velocities for overland flow. In areas of defined channel flow, SEA believes that it is appropriate to use Manning's equation for high-flow conditions to determine flow velocities. This adjustment tends to reduce travel time for routed hydrographs, causing a reduction in flood wave attenuation and an increase in peak flow downstream.

6. The master plan used the City of Reno Rainfall Curves developed by Winzler and Kelly Engineers in 1984. The 24-hour duration precipitation values were verified and used in this study.

7. The hydrologic soil group classifications used by the master plan for the various subareas were checked using the Soil Conservation Services' "Soil Survey of Washoe County". The soil groups used in the determination of curve numbers were verified.

8. The flood hydrograph program used in the master plan is not familiar to SEA. Instead, the U.S. Army Corps of Engineers HEC-1 Flood Hydrograph Package was selected for use in this study to provide a state-of-the-art analysis familiar to most agencies.

A summary of the drainage basin data and supporting calculations are included as Appendix 4.

The IBM XT 512K Version of the HEC-1 Flood Hydrograph Package that was used for the computer modeling has certain limitations. The maximum number of hydrograph ordinates that can be computed is 300. Due to this restriction, it was determined that the modeling of alternatives utilizing detention basins should be run with a 10 minute time interval to adequately analyze a 24 hour storm. This allowed for 50 hours of total time tabulation, which is necessary to provide sufficient time for the flood hydrograph to be routed through detention reservoirs. Using a 10 minute time interval for the computation and routing of flood hydrographs for a basin this small can contribute to small inaccuracies in peak runoff values. The base run for this study was, therefore, modeled using both a 5 and 10 minute time interval to provide a comparison of the computed peak flows. The variation in the resulting peak flows was negligible and it was determined that using a 10 minute time interval would be acceptable.

The 100-year return frequency, 24-hour duration storm was selected for the design storm for flood routing alternatives where possible. The 25-year and 5-year return frequency, 24-hour duration storms were also modeled and analyzed. The Probable Maximum Flood (PMF) and the 1/2 PMF events were considered in the modeling of detention basin spillways when the detention basin qualified as a dam under State of Nevada regulations. Two memoranda concerning requirements for review of a detention dam structure by the State of Nevada are included as Appendix 1.

The Evans Avenue embankment and the Union Pacific Railroad embankment do not have spillways. The embankments were not designed as detention basins and it is assumed that the 1/2 PMF event would breach both embankments.

- **Detention Basin Sites**

Upon analysis of the flood route, four potential detention sites were selected for modeling. These include:

1. Reno Rendering - The Reno Rendering detention site is a natural meadow located west of the Reno Rendering Plant. The Reno Rendering site has a natural slope to provide containment on the north side of the meadow. Detention structure embankments would need to be constructed on the east and south sides of the meadow. The height of the detention structure embankments used in the HEC-1 model vary depending upon the desired limit of impoundment, and thus, the amount of detention storage area that must be procured.

2. Evans Avenue/Union Pacific Railroad - These are two separate detention basin sites. Prior to the construction of Evans Avenue the railroad embankment of the Union Pacific Railroad northwest of the Manogue High School acted as a detention facility with only a 24"ϕ corrugated metal pipe as an outlet. In response to the failure of the railroad embankment during a flood, the Union Pacific Railroad installed twin 48" pipes through their embankment. The Union Pacific Railroad embankment was not constructed to detention structure standards. The construction of Evans Avenue immediately upstream of the railroad embankment drastically reduced the potential detention volume behind the railroad embankment. Evans Avenue was not designed as a detention structure because spillway capacity to provide PMF protection could not be readily provided. Instead, a 72"ϕ CMP culvert was installed under Evans Avenue. This culvert was designed to convey the 100-year storm runoff under the Evans Avenue embankment. As a result of the Evans Avenue construction, the UPRR is currently considering replacing the existing 48"ϕ culverts under their embankment with larger pipes that could pass the 100-year flood with zero headwater conditions.

3. Cemetery - The cemetery site is located in the draw north of Our Mother of Sorrows Cemetery, which is located on the northeast corner of McCarran Boulevard and North Virginia Street intersection. Within the cemetery site two potential dam sites were selected where it appeared that suitable abutments existed. The two potential dam sites, located approximately 400 feet apart, are referred to herein as the upper and lower cemetery sites. The Catholic Diocese of Reno has expressed an interest in utilizing the potential dam for irrigation storage.

4. George's Den - The George's Den site is located east of North Virginia Street and north of George's Den (a bar). The potential dam site is located upstream of the cemetery site in the same natural draw.

The potential dam sites are shown on the existing facility maps in Appendix F. It was determined upon analysis of the potential cemetery and George's Den dam sites, that only one detention dam could be used efficiently. Placing a second dam in the same area does not further reduce an already attenuated peak flow. The potential detention sites were field surveyed to obtain the stage-volume data used in the HEC-1 modeling.

All of the potential sites were modeled using the HEC-1 Flood Hydrograph package. The spillway crest for each dam was set at the maximum elevation possible for that site without allowing the flood pool to encroach upon restricted areas. The outlet was then sized to keep the 100-year flood pool elevation at or below the set spillway crest elevation. The outlet was sized assuming a free outlet at the toe of the dam. The Evans Avenue and Union Pacific Railroad embankments were modeled to represent the existing condition. As stated previously, these embankments were not designed to perform as detention structures. It was, therefore, considered desirable to select flood routing alternatives was to reduce the peak storage behind the Evans Avenue and Union Pacific Railroad embankments.

A preliminary soils investigation was conducted to determine the feasibility of the sites for construction of a dam. The existing Evans Avenue embankment was also investigated to determine the feasibility of retrofitting the embankment to dam standards. The soils investigation report is included as Appendix 2.

- **Detention Alternatives**

Several flood control alternatives were selected to analyze the effects of the various detention sites. These include detention and routing combinations above Reno Rendering Plant only (below Reno Rendering Plant the analysis becomes strictly a routing matter and is covered in the next section of this study.)

Alternative #1 The base run of existing conditions with no improvements.

Alternative #2 Utilizes the George's Den detention dam with a 48"ϕ RCP outlet.

Alternative #3 Utilizes the upper cemetery detention dam with a 42"ϕ RCP outlet.

Alternative #4 Utilizes the lower cemetery detention dam with a 30"ϕ RCP outlet.

Alternative #5 Utilizes the lower cemetery detention dam with a 30"ϕ RCP outlet. Subarea J is routed above Evans Avenue and the Manogue High School athletic field pipe is upsized to convey the outflow from the Union Pacific Railroad embankment.

Alternative #6 Utilizes the lower cemetery detention dam with a 30"ϕ RCP outlet. Subarea J is routed above Evans Avenue. The inlet to the 72"ϕ CMP under Evans Avenue is downsized to 48"ϕ with a 72"ϕ emergency spillway riser pipe set at the 100-year flood pool elevation. The Manogue High School athletic field pipe is upsized to convey the outflow from the Union Pacific Railroad embankment.

<u>Alternative #7</u> Utilizes the lower cemetery detention dam with a 30"ϕ RCP outlet. Subarea J is routed above Evans Avenue. The Manogue High School athletic field pipe is upsized to convey the outflow from the Union Pacific Railroad embankment. The Reno Rendering detention dam is utilized with a 36"ϕ RCP outlet and a high spillway crest elevation.

<u>Alternative #8</u> Utilizes the lower cemetery detention dam with a 30"ϕ RCP outlet. Subarea J is routed above Evans Avenue. The Manogue HIgh School athletic field pipe is upsized to convey the outflow from the Union Pacific Railroad embankment. The Reno Rendering detention dam is utilized with two 42"ϕ RCP outlets and a low spillway crest elevation.

<u>Alternative #9</u> Subarea J is routed above Evans Avenue. The Manogue High School athletic field pipe is upsized to convey the outflow from the Union Pacific Railroad embankment.

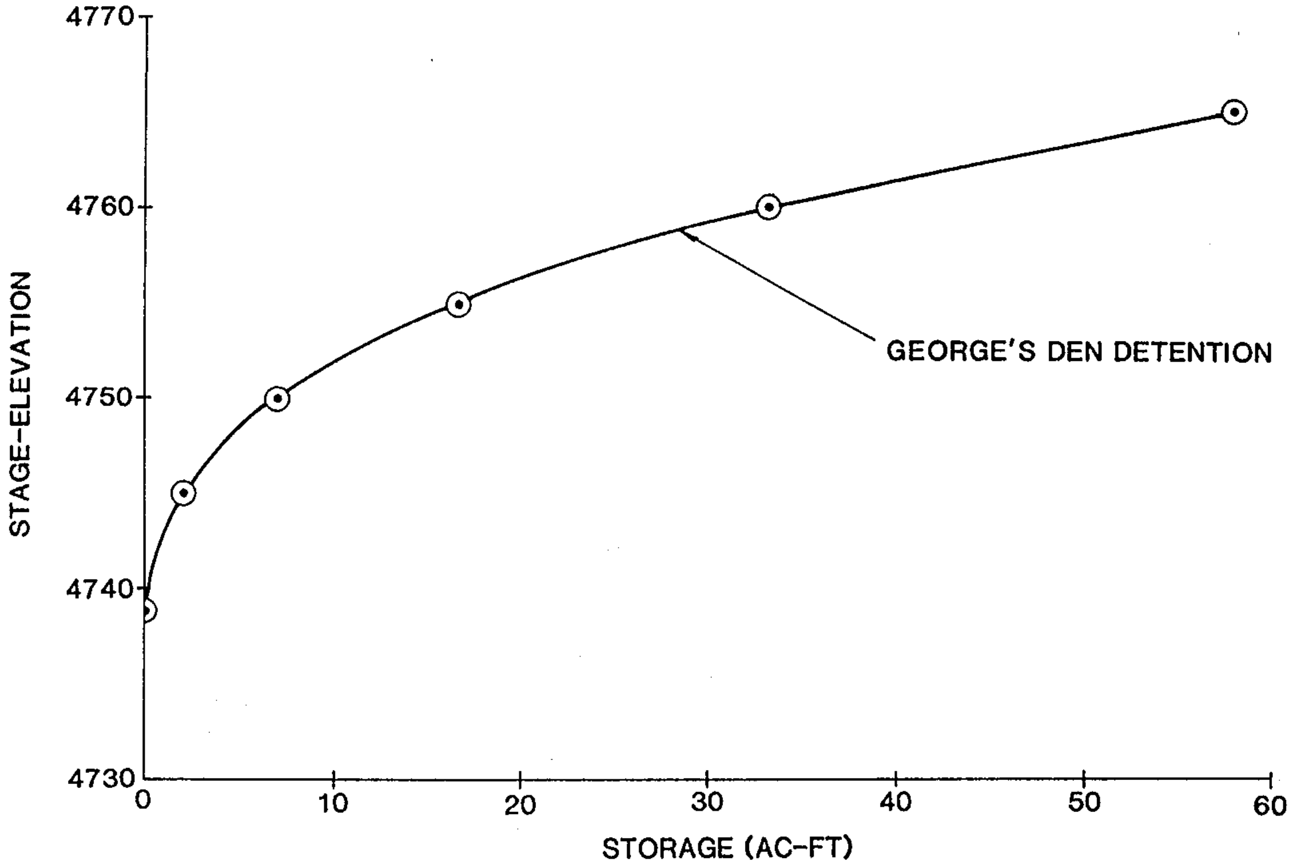
<u>Alternative #10</u> Utilizes the lower cemetery detention dam with a 36"ϕ RCP outlet. The outlet pipe was placed higher in the dam to provide 5.8 ac.ft. of storage for irrigation water. The outlet pipe size necessarily increased due to the reduced storage capacity of the detention dam. This alternative was analyzed only to illustrate the effect of reducing the detention capacity and does not represent an actual design of the irrigation storage requirements for the cemetery.

<u>Alternative #11</u> Utilizes the lower cemetery detention dam with a 30"ϕ RCP outlet. Subarea J is routed above Evans Avenue. The inlet to the 72"ϕ CMP under Evans Avenue is downsized to 48"ϕ.

The Manogue High School athletic field pipe is upsized to convey the outflow from the Union Pacific Railroad embankment. The Reno Rendering detention dam is utilized with a 36"ϕ RCP outlet and a high spillway crest elevation.

The above alternatives were modeled using the 100-year return frequency, 24-hour duration storm. The HEC-1 output for the 100-year storm is included as Appendix 8 for Alternatives 1 through 11.

Alternatives 13-16 are the same as Alternatives 5-8 using the 25-year return frequency, 24 hour duration storm. The HEC-1 output for the 25-year and 5-year storms are included as Appendix 9 for Alternatives 13 through 20. Alternatives 17-20 are the same as Alternatives 5-8 using the 5-year return frequency, 24-hour duration storm. The facility maps showing the flood routing alternatives are included in Appendix A. The stage-storage-curves for the detention dam alternatives are shown on Plates 2 through 4, and the calculations for the stage-storage curves are included as Appendix 5. Conceptual dam profiles for each site are shown on Plates 5 through 7. Tables 2 through 4 provide summaries of the peak discharges at each routing point for the alternatives listed above.


4770
4760
4750
4740
4730
STAGE-ELEVATION
GEORGE'S DEN DETENTION
0
10
20
30
40
50
60
STORAGE (AC-FT)

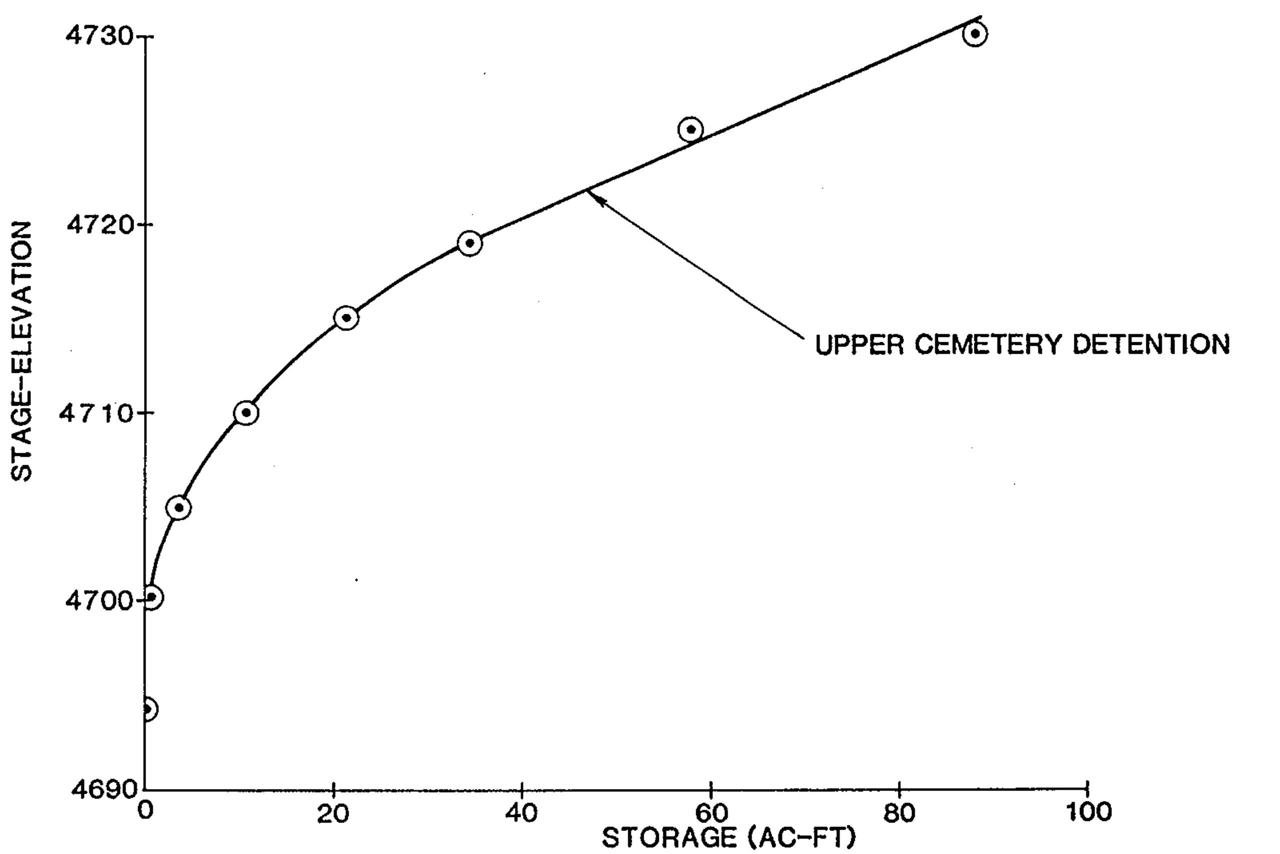
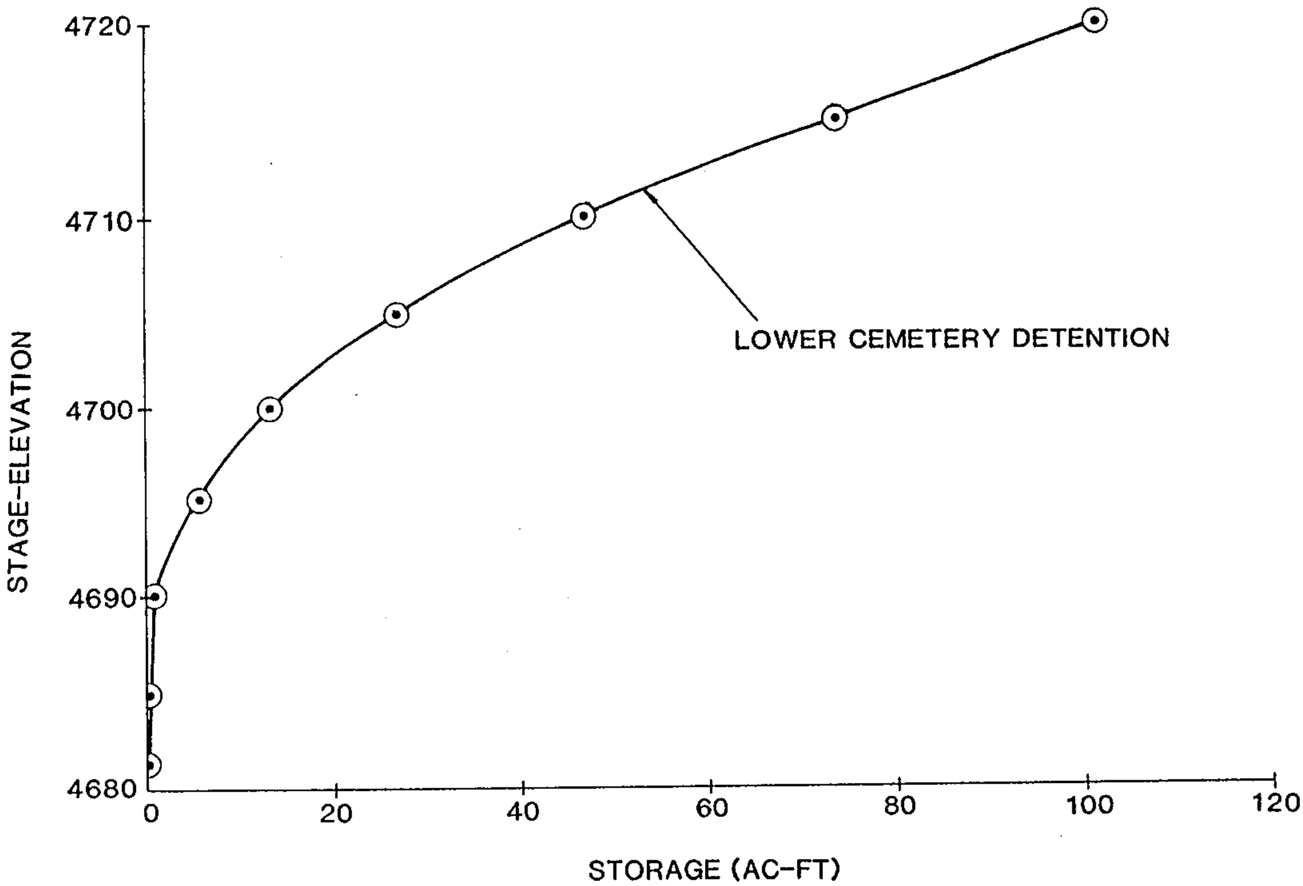

4730
4720
4710
4700
4690
STAGE-ELEVATION
UPPER CEMETERY DETENTION
0
20
40
60
80
100
STORAGE (AC-FT)

PLATE 2


4720
4710
4700
4690
4680
STAGE-ELEVATION
LOWER CEMETERY DETENTION
0
20
40
60
80
100
120
STORAGE (AC-FT)

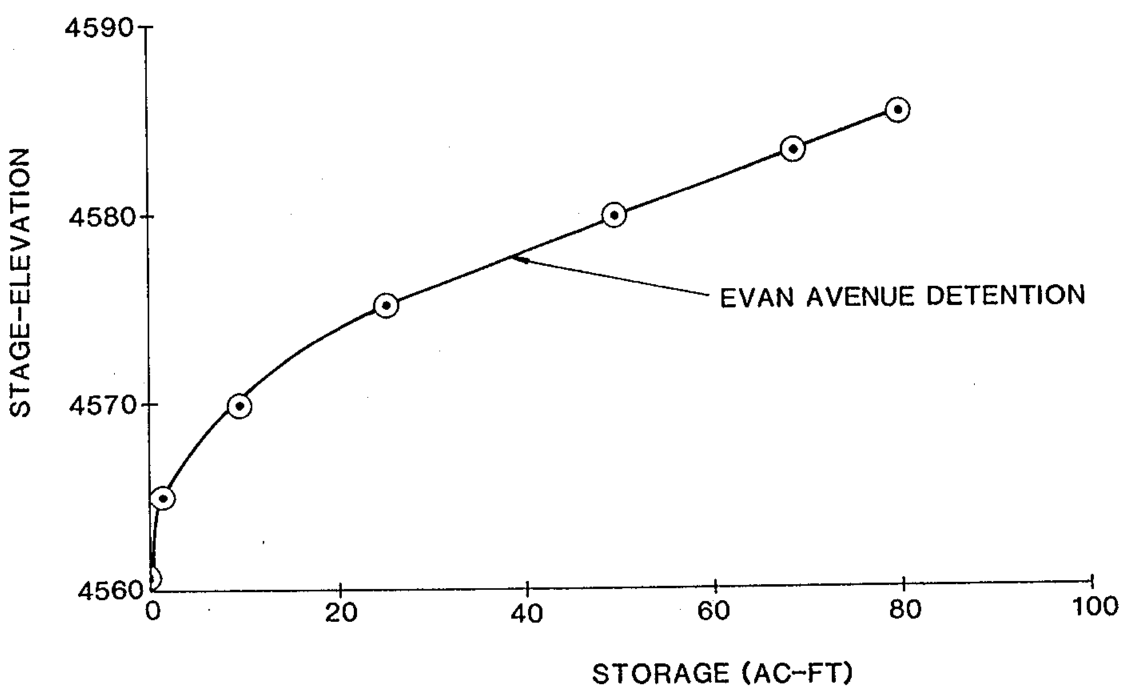
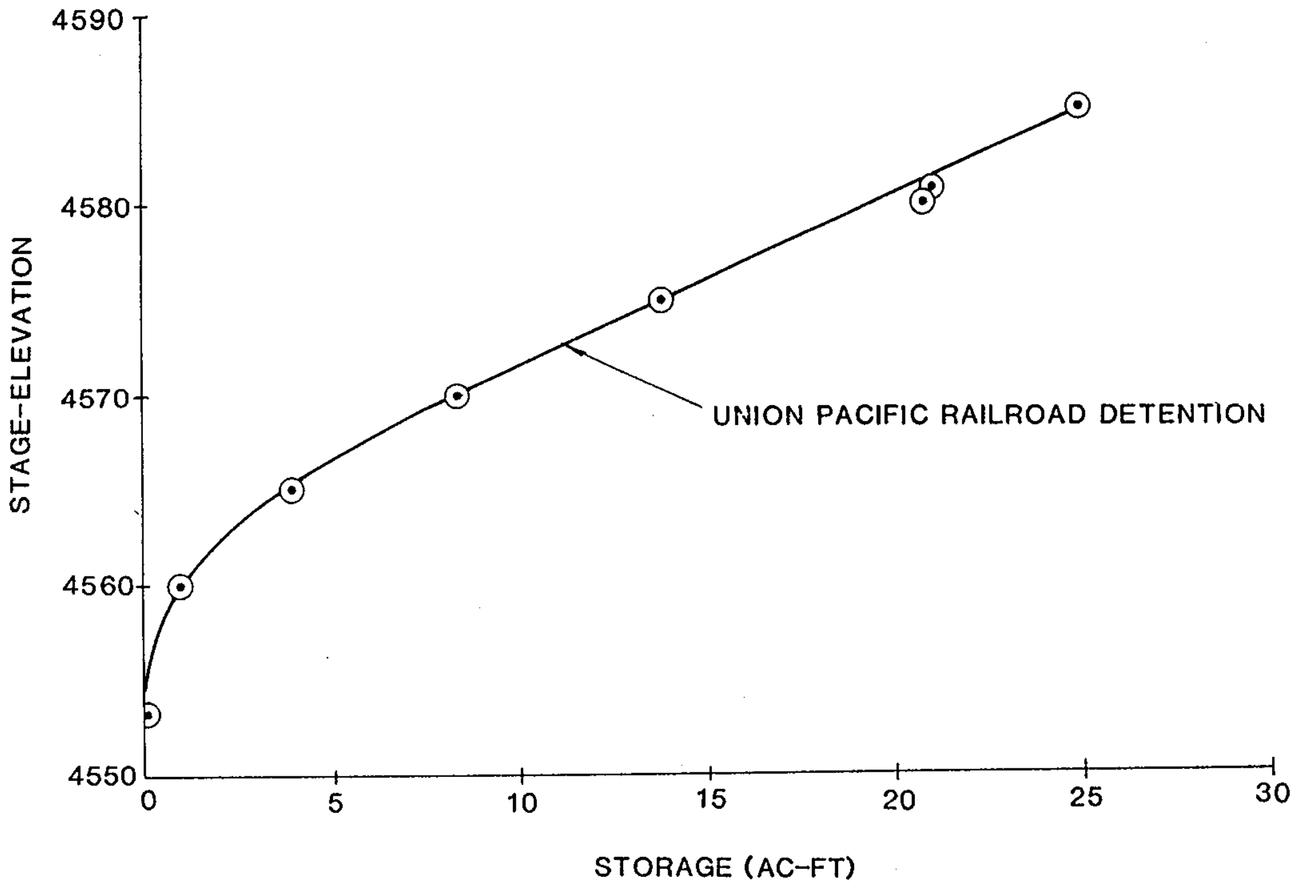

4590
4580
4570
4560
STAGE-ELEVATION
EVAN AVENUE DETENTION
0
20
40
60
80
100
STORAGE (AC-FT)

PLATE 3


4590
4580
4570
4560
4550
STAGE-ELEVATION
0
5
10
15
20
25
30
STORAGE (AC-FT)
UNION PACIFIC RAILROAD DETENTION

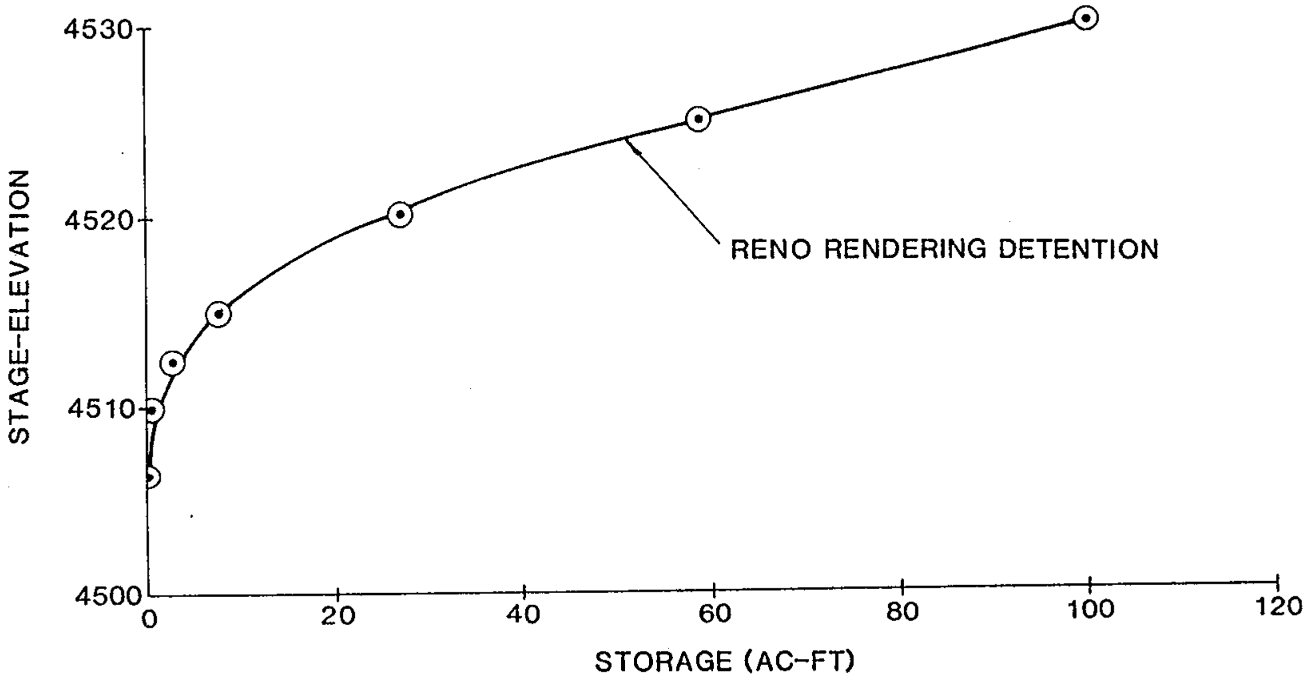
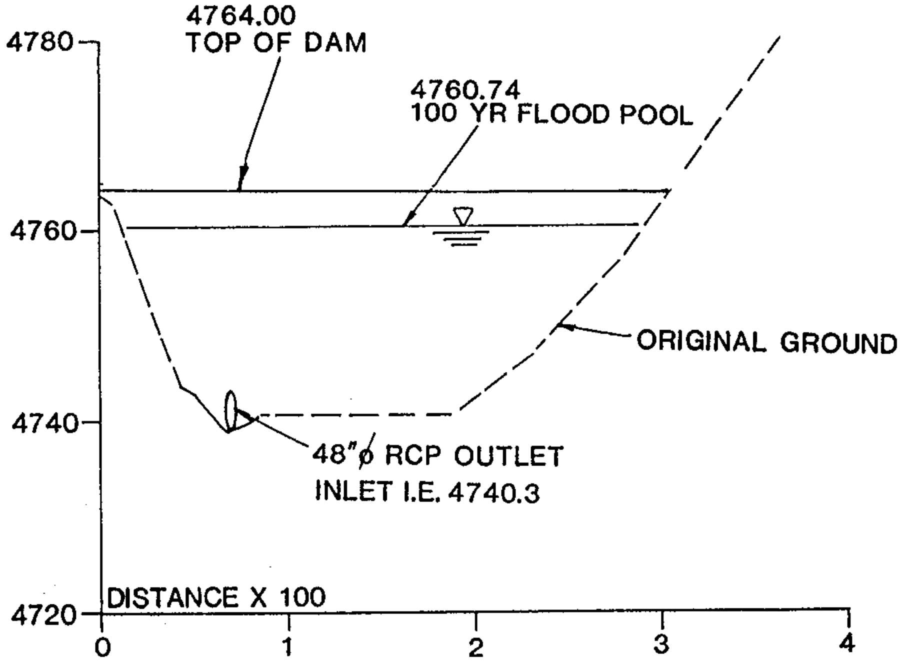
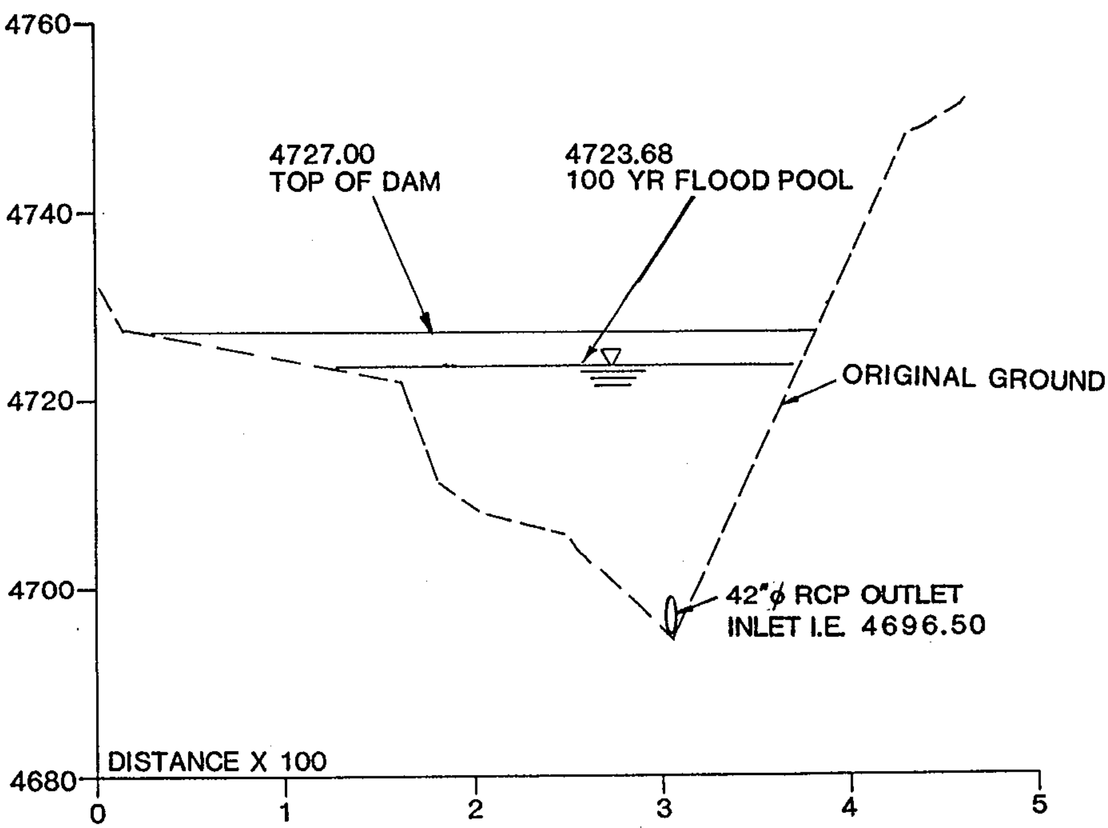

4530
4520
4510
4500
STAGE-ELEVATION
0
20
40
60
80
100
120
STORAGE (AC-FT)
RENO RENDERING DETENTION

PLATE 4



GEORGE'S DEN DAM



UPPER CEMETERY DAM

PLATE 5

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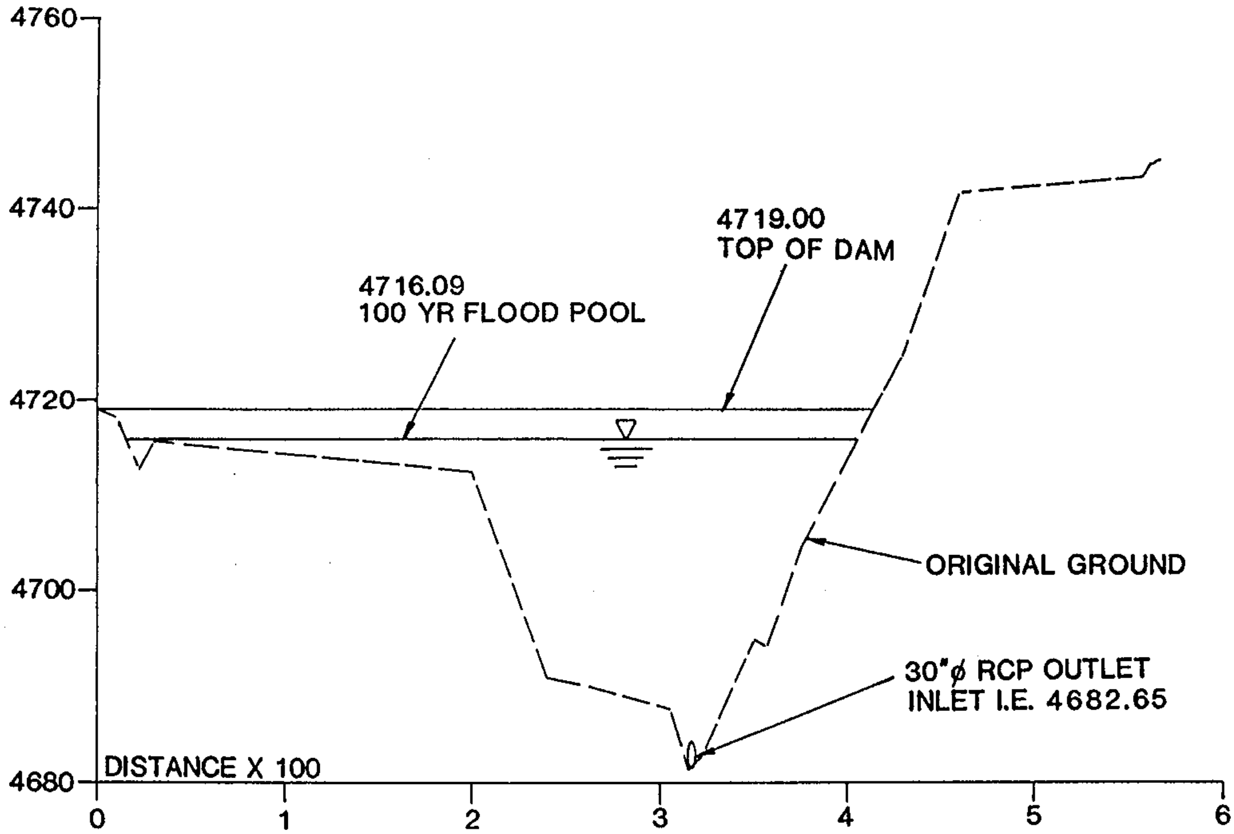
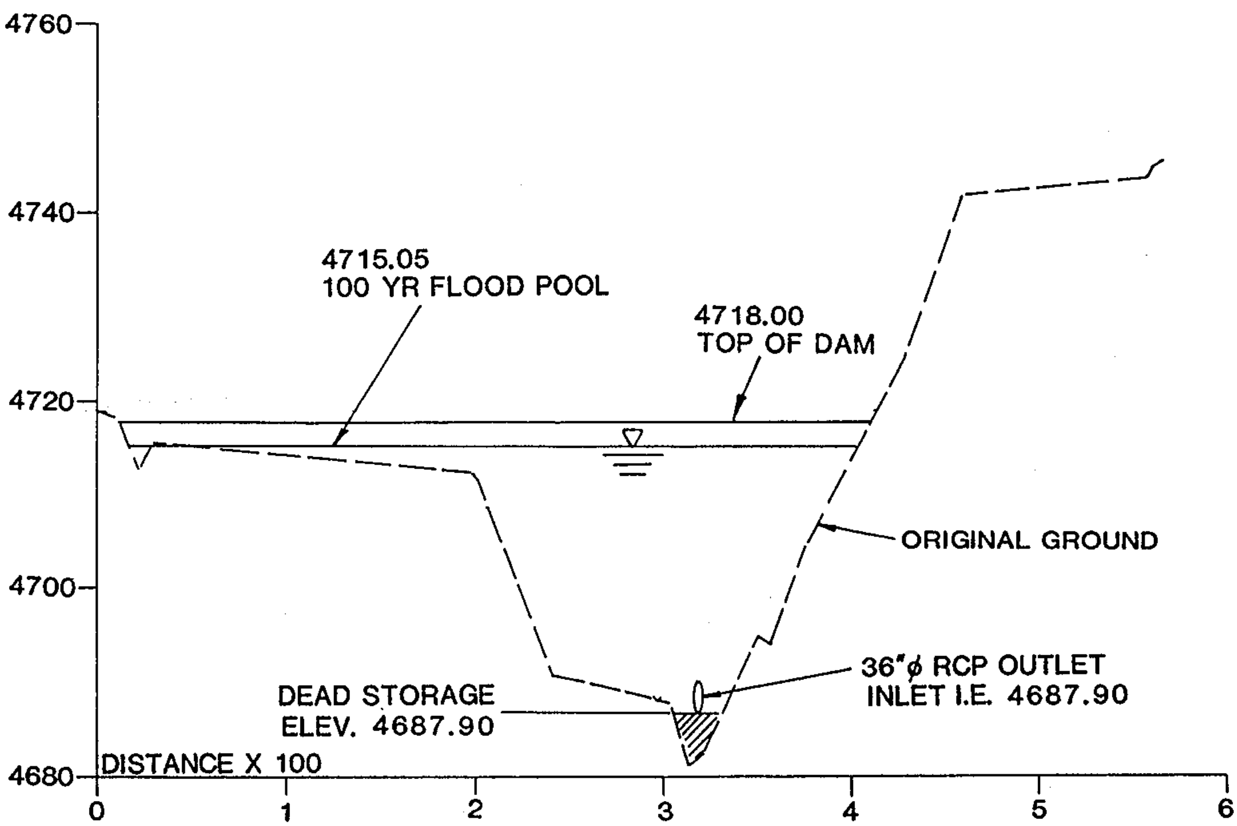


LOWER CEMETERY DAM



LOWER CEMETERY DAM W/IRRIGATION STORAGE

PLATE 6

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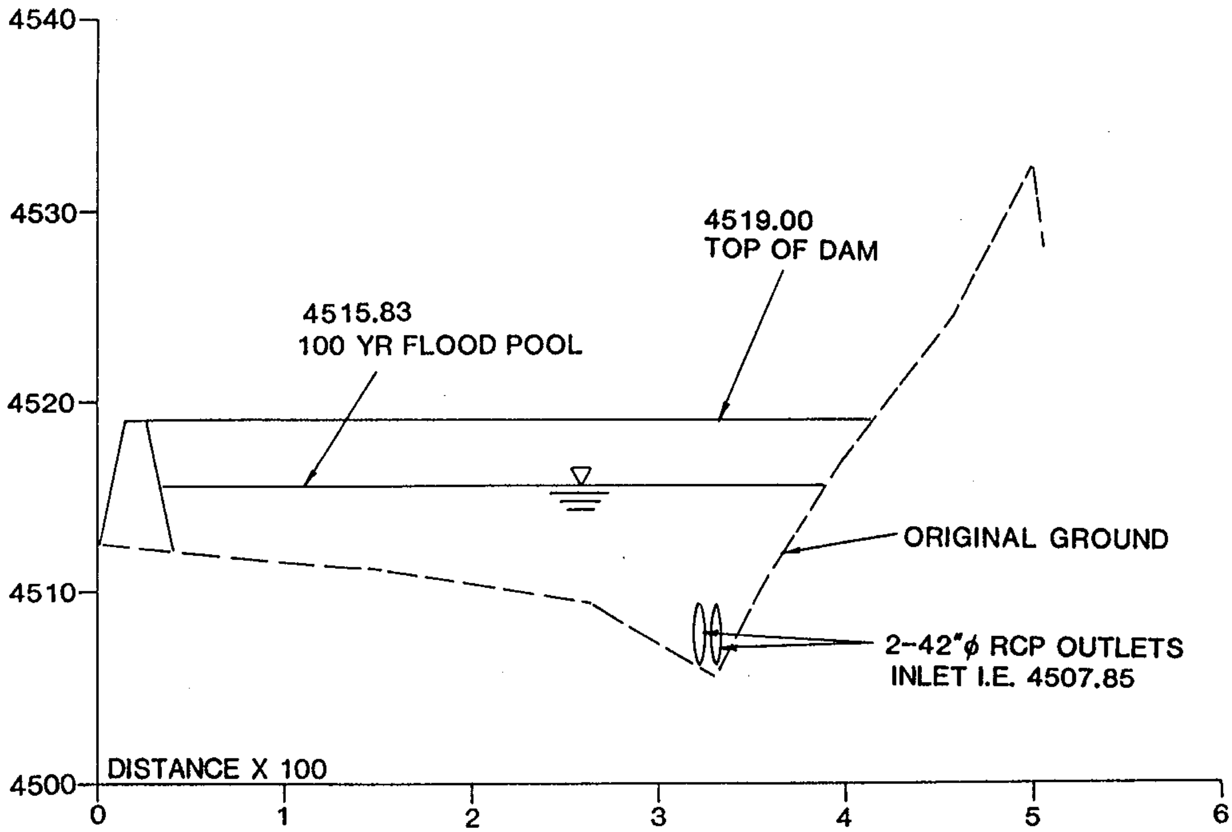
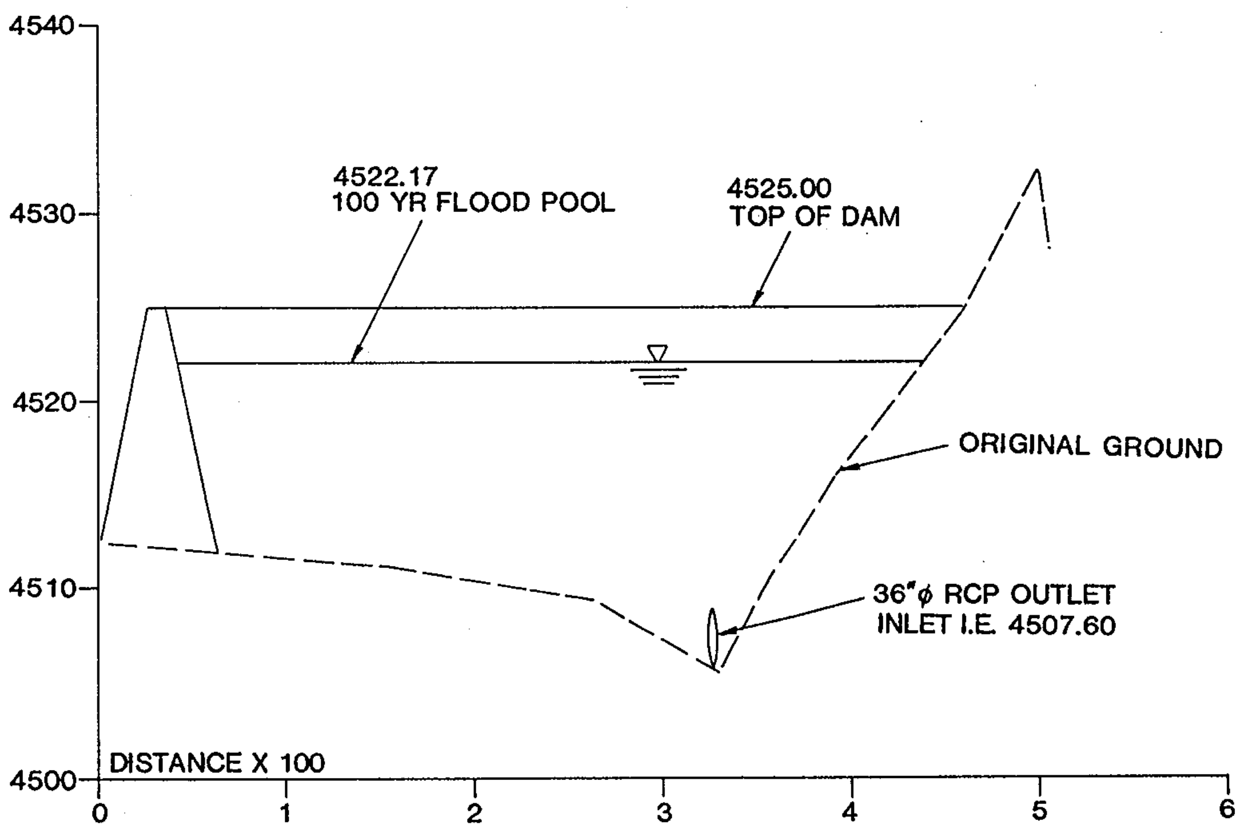


SMALL RENO RENDERING DAM



LARGE RENO RENDERING DAM

PLATE 7

sea

TABLE 2

ALTERNATIVE	100-YR PEAK DISCHARGE(cfs)/PEAK STORAGE (AC-FT) AT LOCATION:									
	114A	120A	GEORGES	116A	CEMETERY	117A	EVANS	UPRR	119A	RENDER
1. Base Run	540			650		847	586/26	558/10	585	
2. George's Detention (48"ϕ outlet)	540	605	322/37	351		572	441/10	426/4	554	
3. Upper Cemetery Detention (42"ϕ outlet)	540			650	250/55	436	368/6	348/2	518	
4. Lower Cemetery Detention (30"ϕ outlet)	540			650	126/82	340	289/4	286/1	480	
5. Same as #4 + Rte. Area "J" above Evans Ave.	540			650	126/82	523	399/8	379/2	423	
6. Same as #5 + add 48"ϕ restrictor on Evans culvert	540			650	126/82	523	227/20	227/1	280	
7. Same as #5 + add large Reno Rendering Detention (36"ϕ outlet)	540			650	126/82	523	399/8	379/2	423	155/41
8. Same as #5 + add small Reno Rendering Detention (2-42"ϕ outlets)	540			650	126/82	523	399/8	379/2	423	334/11
9. Route Area "J" above Evans Ave.	540			650		1017	623/34	591/12	602	
10. Lower Cemetery Detention w/irrigation storage (36"ϕ outlet)	540			650	179/69	389	324/5	317/1	505	
11. Same as #6 + add large Reno Rendering Detention (36"ϕ outlet)	540			650	126/82	523	227/20	227/1	280	154/40

TABLE 3

ALTERNATIVE	25-YR PEAK DISCHARGE(cfs)/PEAK STORAGE (AC-FT) AT LOCATION:									
	114A	120A	GEORGES	116A	CEMETERY	117A	EVANS	UPRR	119A	RENDER
12. Base Run	321			370		483	400/8	394/3	427	
13. Lower Cemetery Detention (30"ϕ outlet) + Rte. Area "J" below Evans Avenue	321			370	111/39	331	280/3	280/1	319	
14. Same as #13 + add 48"ϕ restrictor on Evans culvert	321			370	111/39	331	190/10	190/1	217	
15. Same as #13 + add large Reno Rendering Detention (36"ϕ outlet)	321			370	111/39	331	280/3	280/1	319	135/21
16. Same as #13 + add small Reno Rendering Detention (2-42"ϕ outlets)	321			370	111/39	331	280/3	280/1	319	278/3

TABLE 4

ALTERNATIVE	5-YR PEAK DISCHARGE(cfs)/PEAK STORAGE (AC-FT) AT LOCATION:									
	114A	120A	GEORGES	116A	CEMETERY	117A	EVANS	UPRR	119A	RENDER
17. Base Run	136			154		194	199/1	194/1	232	
18. Lower Cemetery Detention (30"ϕ outlet) + Rte. Area "J" above Evans Avenue	136			154	86/10	166	164/1	164/0	185	
19. Same as #18 + add 48"ϕ restrictor on Evans culvert	136			154	86/10	166	127/3	127/0	143	
20. Same as #18 + add large Reno Rendering Detention (36"ϕ outlet)	136			154	86/10	166	164/1	164/0	185	109/6
21. Same as #18 + add small Reno Rendering Detention (2-42"ϕ outlets)	136			154	86/10	166	164/1	164/0	185	184/0

- **Flood Routing Alternatives**

Of the detention alternatives analyzed above, Alternatives No. 5, 6, 7 and 8 were selected for further analysis. This analysis includes sizing the pipe in the Manogue High School athletic field, sizing drainage facilities for routing flows from Area J to either the Evans Avenue detention area or down Valley Road, and the routing of flows downstream of Reno Rendering Plant.

The Manogue High School athletic field is served by a 24"ϕ pipe connected to a 72"ϕ pipe stubbed out from Valley Road to the east. Twin 48"ϕ CMP pipes enter the field from the west under the Union Pacific Railroad embankment. For the alternatives selected, a pipe to convey the flow from the Union Pacific Railroad culverts to the existing 72"ϕ pipe would need to be a 54" or 66" diameter RCP, depending upon the alternative. The size used for each alternative is shown on the alternative maps in Appendix A.

The flow from Area J will exceed the capacity of the existing culverts under McCarran Boulevard and spread out over a wide area where it crosses the low point in McCarran Boulevard just east of Evans Avenue. This flow would then sheet flow in a southerly direction to Enterprise Road. The existing channel is not sufficient to convey the entire 227 cfs that was calculated for the 100-year flow. One possible solution would be to provide an interception channel on the uphill side of Enterprise Road to collect and route the flow to the intersection of Evans Avenue and Enterprise Road. The flow would then pass under the intersection in a 60"ϕ pipe to convey the flow to the natural drainage above the Evans Avenue embankment. The intercepting channel could be a 60 foot wide asphalt-lined channel with 4% side slopes. A channel of this type could be compatible with future industrial-type development, typical of this area.

Excess flows below Reno Rendering have two directions of travel available due to the high point in the street profile where Oddie Boulevard and Wells Avenue meet directly below Reno Rendering Plant. Therefore, two directions of flood routing were investigated.

Flood routing in the Wells Avenue direction could be accomplished in two ways. One way would be to utilize the existing I-80 storm drain system by conveying flow to the intersection of I-80 and Wells, and penetrating the existing 54"ϕ storm drain. This storm drain parallels the south side of I-80 to I-580, then parallels the east side of I-580 to the point where I-580 intersects the Truckee River, where it ultimately discharges. The capacity of this pipe, according to NDOT calculations, is 77 cfs. A system designed to convey 70 cfs to this pipe would incorporate the following:

1. A 36"ϕ pipe from Reno Rendering Plant to Sadlier Lane.
2. A 2 foot flat bottom open channel ditch along the west side of Wells Avenue from Sadlier Lane to the north side of I-80. This ditch would be within the UNR farm property.
3. A 36"ϕ pipe passing under the I-80 overpass from the ditch to the 54"ϕ pipe on the south side of the interchange.

Street conveyance of the flow was considered for this alternative but was not included in this routing scenario because there are several existing residences on the west side of Wells Avenue north of Sadlier Lane that have basement garages below the gutter flowline elevation. The I-80 storm drain system was designed to provide 50-year return period flood protection for the depressed section of I-80 to the west of Wells Avenue. It is possible that the State would not allow additional drainage to be introduced into their system if there is any chance of a decrease in capacity of the system during the design storm. It should also be noted that the State operates and maintains the storm drain system, therefore, the City may not have control over factors affecting system capacity, such as the silting in of pipes. Further analysis of this routing option may be desirable after further consideration of these issues.

Another routing scenario down Wells Avenue would be to extend a pipe all the way to the Truckee River. A suitable alignment for such a pipe would be along Morrill Avenue from the south side of I-80 all the way to the river. It was discovered that there is only 7 feet of fall from the north side of I-80 to the 100-year flood elevation in the Truckee River at the point of outfall. The resulting slope of .0023 ft/ft requires very large pipes to convey any substantial flow. It was determined that comparable capacity could be obtained at a lower cost using the Oddie Boulevard alignment, therefore, this alternative was not investigated any further.

Routing in the direction of Oddie Boulevard presented two options, the first utilizing the existing Oddie system, and the second providing additional capacity by carrying a portion of the flow overland in the street section, and a portion of the flow underground in a new pipe system. The existing system capacity is approximately 110 cfs east of Sutro Avenue. The existing system consists of 42" diameter RCP beginning at Sutro, increasing to 48" diameter at Montello and increasing to 54" diameter at Silverada Boulevard. The outfall into Paradise Pond is a 66" diameter CMP. In order to utilize the 110 cfs capacity of the existing system east of Sutro Avenue, new pipe would need to be constructed from Reno Rendering Plant to the existing 42"ϕ pipe. By utilizing a 48"ϕ pipe, an additional 50 cfs could be carried to the east side of Sutro Avenue and then allowed to bubble up into the gutter using a suitable structure (such as a drop inlet). This would allow the additional 50 cfs to flow within the north half of the street section east of Sutro Avenue. A short flood wall would need to be constructed along the north right-of-way boundary of Oddie Boulevard to contain flood flows within the street section. Also, Montello Street would require modification to create a high point in the profile north of Oddie Boulevard to maintain containment.

In conversation with the City of Reno Engineering Department, it was learned that the proposed Clearacre storm drain project will be using a separate storm drain structure all the way to Paradise Pond and would not be utilizing any portion of the existing Oddie Boulevard storm drain system. Storm flows from the drainage area which will not be served by the Clearacre storm drain project is collected in local storm drain pipes north of Oddie Boulevard and connected to the existing Oddie Boulevard system. An analysis of the flood hydrographs from the local drainage at Sutro Avenue (a 24"ϕ pipe), and at Helena Way (a 36"ϕ pipe) revealed that the available Oddie Boulevard system capacity would be reduced by 7 cfs at the time the peak flow from the study area reaches the Oddie Boulevard system. At the time of peak discharge from the local drainage, the capacity of the existing system is exceeded. This period of excess flow would last for approximately 1.5 hours, with the excess flows continuing overland according to the existing drainage patterns. This occurrence does not affect the Reno Rendering Detention outfall capacity, however.

The option of constructing a second pipe system within Oddie Boulevard was also investigated. This system would consist of:

A. Pipe sizes equivalent to the existing system to carry approximately 110 cfs in Alternative 6.

B. Pipe sizes that would be upsized one pipe size from the existing system to carry 160 cfs in Alternative 8.

C. Pipe sizes that would be upsized two pipe sizes from the existing system to carry 220 cfs in Alternative 5.

A summary of the recommended flood routing below Reno Rendering for Alternatives 5 through 8 is included on Table 5. This table indicates a suggested construction phasing scenario for each alternative and the resulting reduced peak flows below Reno Rendering for the 5-year 25-year and 100-year return period storms. The pipe sizes used for each alternative are shown on the alternative maps in Appendix A. The hydraulic calculations supporting the flood routing alternative recommendations are included as Appendix 6.

TABLE 5

DETENTION AND CONVEYANCE ALTERNATIVE
FACILITIES CONSTRUCTION PHASING AND POST-CONSTRUCTION FLOW SUMMARY

Alternative-Phase	Facility	Excess Flow at Reno Rendering Q5	Q25	Q100
Base Flow	No Improvements	232	427	585
5-1	° Lower Cemetery Detention ° Route Area "J" above Evans ° 66"ϕ RCP across Manogue HS	185	319	423
5-2	° 42"ϕ/48"ϕ RCP from Reno Rendering to Sutro ° Oddie flood wall and Montello Modification	25	159	263
5-3	° 54"ϕ/60"ϕ/66"ϕ RCP from Reno Rendering to Paradise Pond	-0-	-0-	43
6-1	° Lower Cemetery Detention ° Route Area "J" above Evans ° 48"ϕ outlet for Evans detention ° 54"ϕ RCP across Manogue HS	143	217	280
6-2	° 42"ϕ/48"ϕ RCP from Reno Rendering to Sutro ° Oddie flood wall and Montello Modification	-0-	57	120
6-3	° 42"ϕ/48"ϕ/54"ϕ RCP from Reno Rendering to Paradise Pond	-0-	-0-	10
7-1	° Lower Cemetery Detention ° Route Area "J" above Evans ° 66"ϕ RCP across Manogue HS ° Large Reno Rendering Detention	109	135	155

7-2	° 42"ϕ/48"ϕ RCP from Reno Rendering to Sutro ° Oddie flood wall and Montello Modification	-0-	-0-	-0-
8-1	° Lower Cemetery Detention ° Route Area "J" above Evans ° 66"ϕ RCP across Manogue HS ° Small Reno Rendering Detention	184	278	334
8-2	° 42"ϕ/48"ϕ RCP from Reno Rendering to Sutro ° Oddie flood wall and Montello Modification ° 42"ϕ/48"ϕ/54"ϕ/60"ϕ RCP from Reno Rendering to Paradise Pond	-0-	-0-	14

IV. CONCLUSIONS AND RECOMMENDATIONS

The purpose of this study was to develop a single recommendation for construction of flood control improvements to relieve drainage problems in the study area. The subsequent recommended alternative must, therefore, satisfy the desired protection requirements, while simultaneously being economically feasible and consistent with the needs and desires of the property owners and future land use.

Due to the expected shortage of funds for construction of the improvements, any improvements that could be situated on the same property that is receiving flood protection from the proposed improvements would be desirable. For this reason, it is important to promote property owner participation in the drainage relief project.

- **Construction Costs and Feasibility Analysis**

The opinion of probable construction costs developed for each phase of each alternative are summarized in Tables 6 thru 9. These costs do not include engineering cost, land use or acquisition cost, utility adjustment or relocation costs, geotechnical investigation cost, and any nonessential construction costs such as recontouring of adjacent ground or relocation of existing access roads, etc. The costs assume that materials for the dams will be available on-site. It is noted in the preliminary soils investigation that some material may need to be imported for the dam embankment, however, the extent is not known at this time, nor is the potential source for borrow known at this time.

In addition to the probable construction cost, the relative benefit and construction feasibility for each alternative was analyzed by flood control improvement component. Many of the components are the same for each alternative or are similar, for instance, differing only in the pipe size. An analysis of each component and its relative effect upon the alternative follows:

1. **Lower Cemetery Detention** - It was found that any one of the three detention sites above the cemetery was needed by the cemetery for flood control. The Reno Catholic Diocese has been cooperative and willing to consider using their property for a detention site. The lower cemetery site was found to have the best detention features, which results in the smallest outlet pipe size of the three sites, and was also found to be the best suited for location of a dam embankment in the preliminary soils investigation. The lower site also results in a shorter outfall pipe length to Comstock Drive as a result of its proximity. Another advantage of the lower site is that it does not inundate the existing access road alignment adjacent to North Virginia Street for cemetery access into the upper Diocese property above George's Den. This feature was important to the Reno Catholic Diocese.

   The reduced peak flow resulting from the lower cemetery detention allows the retention of all existing downstream culverts without modification with the exception of the Manogue High School 24"ϕ pipe crossing the athletic field. In addition, the lower cemetery detention will allow flood flows from Area "J" to be routed above Evans Avenue, eliminating the need to construct a considerable length of large pipe within Valley Road from Enterprise to the meadow above the Reno Rendering Plant. The lower cemetery detention provides substantial reduction in peak flows at all points downstream for a very modest construction cost, and was incorporated into all alternatives.

2. **Route Area "J" Above Evans Avenue** - This feature provides the detention of flood flow from this large developed area. The construction cost of routing the flows over to Evans Avenue is considerably less expensive than piping the same flow down Valley Road. In addition to being more cost effective, this routing also provides a substantial reduction in the peak flow at points below Valley Road. The improvements required

for this component will not be located in areas receiving flood protection, however, the right-of-way requirements appear to be reasonable and the improvements appear to be consistent with future land use for this area. The additional flow onto the UNR property as a result of this routing is more than offset by the reduction in flow resulting from the lower cemetery detention. This component was also used for all alternatives.

3. **Pipe Across Manogue High School Athletic Field** - A pipe between the UPRR embankment culverts and the Valley Road culvert is required to relieve flooding and debris deposits on the athletic field. There is no other method available to relieve these problems without adversely affecting the use of the property. The lower cemetery detention will allow a smaller pipe size to be used than what would be required for the existing conditions. This component was also used for all alternatives.

4. **Evans Avenue Culvert Modification for 48" Diameter Outlet**- This modification does not require a permit from the State Water Resources Department. See Appendix 1 for memorandums concerning discussions with the State on the detention dam status of the Evans Avenue embankment.

   The peak stage and storage behind the Evans Avenue embankment increases with this modification over the values resulting from the lower cemetery detention scenario without any modification to the Evans Avenue culvert. However, this modification will still provide an improvement over the existing conditions when combined with the lower cemetery detention.

   It should be noted that modification of the outlet to reduce the size could pose a liability problem simply because the pipe is being restricted, even though the combined effect with the lower cemetery detention is to reduce peak stage and storage behind Evans Avenue.

The result of this modification is a substantial reduction in peak flow below the Evans Avenue embankment at very little cost. A few of the benefits include a reduction in the required pipe size for the pipe across Manogue High School athletic field and the attainment of zero head on the Union Pacific Railroad embankment culverts. This component would require very little right-of-way, if at all, which would be obtained from the University of Nevada, Reno. The increase in peak stage and storage behind Evans Avenue as a result of this modification is still more than offset by the lower cemetery detention, resulting in a net decrease in peak stage and storage behind Evans Avenue, an improvement for the University.

This component was only used in Alternative 6, however, analysis of the combination of this component with detention at Reno Rendering was performed with the result that the peak outflow from the Reno Rendering detention was not affected. Therefore, this component could be added to Alternative 7 or 8 without affecting the design peak flows below the Reno Rendering detention structure.

5. **Reno Rendering Detention** - The detention site for the small dam alternative lies entirely on an undeveloped parcel owned by the same property owner as the Reno Rendering Plant. As stated above, the Reno Rendering Plant is in need of flood control improvements, therefore, the construction of a detention dam on this property would directly benefit the property owner.

The detention site for the large dam alternative includes an additional undeveloped parcel immediately west of and adjacent to the site for the small dam. This property will not receive flood protection as a result of the dam, and, in fact, would be subject to a greater extent of flooding if this alternative is constructed. This would suggest that use or acquisition of this property could be expensive as the property owner would have no motivation to offer the property at a reasonable cost.

The large dam alternative eliminates the need for a second pipe in Oddie Boulevard from Reno Rendering to Paradise Pond, and is the only alternative to provide this feature. It is highly possible that the savings from the elimination of this pipe could offset the land use or acquisition cost.

6. **Pipe from Reno Rendering to Sutro Street** - Conveying flows from the meadow west of the Reno Rendering Plant to the existing 42"ϕ storm drain in Oddie Boulevard on the east side of Sutro Street allows the use of the full capacity of the existing storm drain system in Oddie Boulevard for a modest cost. The pipe can be upsized to convey an additional 50 CFS over the capacity of the Oddie system at a very minimal additional cost to utilize street section conveyance within Oddie Boulevard east of Sutro Street. See Item 7 below for details on this feature. This pipe would require an easement across parcels that will be receiving flood protection. Most of these parcels are undeveloped and a few have single family residences on them. This component was used for all alternatives.

7. **Oddie Boulevard Flood Wall and Montello Modification** - This component will convey a nominal flow through a highly flood prone area at a reasonable cost. This component would be less expensive than piping the same amount of flow and no right-of-way acquisition would need to be required, as all improvements could be constructed within the Oddie Boulevard and Montello right-of-way. It should be noted that if a sound wall along Oddie Boulevard should every be considered

for construction in the future, the sound wall could perform as a flood wall at a minimal cost. A field review of the Montello area revealed that the modification required to contain flood flows within Oddie Boulevard should not adversely affect the adjacent properties. This component was also used in all alternatives.

8. **Pipe From Reno Rendering Plant to Paradise Pond** - It is proposed that this pipe would parallel the existing pipe at approximately the same profile in order to avoid conflicts with other utilities. It is assumed that any necessary utility adjustments or relocations would have been performed during construction of the existing pipe system. This component would require easements across the same property noted above in Item 6, which properties are receiving flood protection and are mostly undeveloped. This pipe would be required for Alternatives 5, 6 and 8 to convey flood flows from Reno Rendering without causing flooding in the study area, but is expected to be very expensive.

TABLE 6

OPINION OF PROBABLE CONSTRUCTION COST

BY: GAS/PBE

DATE: November 6, 1989

ALTERNATIVE 5

Phase 1		
1)	Lower Cemetery Detention Dam	$ 412,625.00
2)	Route Area "J" Above Evans Avenue	130,100.00
3)	66"ϕ RCP Across Manogue High School	139,200.00
	Total Phase 1:	$ 681,925.00
Phase 2		
1)	42"ϕ/48"ϕ RCP from Reno Rendering to Sutro Street	$ 183,000.00
2)	Oddie Flood Wall and Montello Street Modification	169,000.00
	Total Phase 2:	$ 352,000.00
Phase 3		
1)	54"ϕ/60"ϕ/66"ϕ RCP from Reno Rendering to Paradise Pond	$1,187,100.00
	Total Phase 3:	$1,187,100.00
	TOTAL ALTERNATIVE 5:	$2,221,025.00

TABLE 7

OPINION OF PROBABLE CONSTRUCTION COST

BY: GAS/PBE

DATE: November 6, 1989

ALTERNATIVE 6

Phase 1		
1)	Lower Cemetery Detention Dam	$ 412,625.00
2)	Route Area "J" Above Evans Avenue	130,100.00
3)	48"ϕ outlet for Evans Detention	10,000.00
4)	54"ϕ RCP Across Manogue High School	111,600.00
	Total Phase 1:	$ 664,325.00
Phase 2		
1)	42"ϕ/48"ϕ RCP from Reno Rendering to Sutro Street	$ 183,000.00
2)	Oddie Flood Wall and Montello Street Modification	169,000.00
	Total Phase 2:	$ 352,000.00
Phase 3		
1)	42"ϕ/48"ϕ/54"ϕ RCP from Reno Rendering to Paradise Pond	$ 828,300.00
	Total Phase 3:	$ 828,300.00
	TOTAL ALTERNATIVE 6:	$1,844,625.00

TABLE 8

OPINION OF PROBABLE CONSTRUCTION COST

BY: GAS/PBE

DATE: November 6, 1989

ALTERNATIVE 7

Phase 1	
1) Lower Cemetery Detention Dam	$ 412,625.00
2) Route Area "J" Above Evans Avenue	130,100.00
3) 66"ϕ RCP Across Manogue High School	139,200.00
4) Large Reno Rendering Detention Dam	700,925.00
Total Phase 1:	$1,382,850.00
Phase 2	
1) 42"ϕ/48"ϕ RCP from Reno Rendering to Sutro Street	$ 183,000.00
2) Oddie Flood Wall and Montello Street Modification	169,000.00
Total Phase 2:	$ 352,000.00
TOTAL ALTERNATIVE 7:	$1,734,850.00

TABLE 9

OPINION OF PROBABLE CONSTRUCTION COST

BY: GAS/PBE

DATE: November 6, 1989

ALTERNATIVE 8

Phase 1	
1) Lower Cemetery Detention Dam	$ 412,625.00
2) Route Area "J" Above Evans Avenue	130,100.00
3) 66"ϕ RCP Across Manogue High School	139,200.00
4) Small Reno Rendering Detention Dam	525,750.00
Total Phase 1:	$1,207,675.00
Phase 2	
1) 42"ϕ/48"ϕ RCP from Reno Rendering to Sutro Street	$ 183,000.00
2) Oddie Flood Wall and Montello Street Modification	169,000.00
3) 42"ϕ/48"ϕ/54"ϕ/60"ϕ RCP from Reno Rendering to Paradise Pond	$ 996,500.00
Total Phase 2:	$1,348,500.00
TOTAL ALTERNATIVE 8:	$2,556,175.00

RECOMMENDATIONS

The analysis presented above suggests that there may be several "best" combinations of improvements depending upon the outcome of such variables as land acquisition cost, property owner cooperation, accepted level of liability risk, and availability of funds. Upon weighing the possible effects of each of these variables, the following assumptions were made.

1. Availability of funding is the most critical variable and therefore, the least expensive alternative would have the greatest chance of being constructed.

2. Property owner cooperation is the second most critical variable and, therefore, any flood control improvement components providing flood protection for the same property owner whose land is needed for the proposed facilities would also have the greatest chance of being constructed.

3. Land use or acquisition costs for Alternate 7 may eliminate the feasibility of this alternative, even though a portion of the land is already subject to flooding, the remaining area of the parcel could be resold for development, and the area reserved for the flood pool could be used by the City for recreation purposes.

4. The risk of liability exists not only for all flood control improvements but also for the lack of providing adequate improvements for flood protection while simultaneously allowing new development to increase the flooding potential downstream without mitigation. Due to the City's enthusiasm for Alternative 6, it is assumed that the increased liability risk for this alternative will not eliminate it from consideration.

Upon review of the probable construction cost, the feasibility of construction of the various components and the assumptions listed above, Alternatives 6 and 7 appear to be nearly equal in cost effectiveness and feasibility. If land acquisiton costs are reasonable for Alternative 7, the flood control improvements listed on Table 10 are recommended for construction. The improvements are listed in the recommended order of construction for phased construction.

TABLE 10

RECOMMENDED ALTERNATIVE:

ALTERNATIVE #7 + EVANS AVENUE CULVERT MODIFICATION

Proposed Improvement	Probable Construction Cost
1. Lower Cemetery Detention with outfall piped to Comstock Drive	$ 413,000
2. Evans Avenue culvert modification	10,000
3. Manogue High School athletic field pipe	112,000
4. Route Area "J" above Evans Avenue	130,000
5. Large Reno Rendering Detention with outfall piped to Sutro Street	884,000
6. Oddie Boulevard flood wall and Montello Modifications	169,000
Total	$1,718,000

Assuming funds will be very limited, it is anticipated that only small portions of the recommended alternative would be constructed at a time and that these improvements could be phased out over a long period of time. Therefore, the recommended alternative, Alternative 7, was modified to include the Evans Avenue culvert modification for a 48" diameter outlet. This will allow the reduction of peak flows during the interim between the time when the culvert modification is constructed and the time when Reno Rendering detention is constructed. It should be noted that the peak discharge from the Reno Rendering detention is not reduced by the construction of the culvert modification at Evans Avenue.

If land acquisition costs for Alternate 7 are prohibitive, then Alternate 6 is recommended for construction. Table 11 lists the improvements in the recommended order of construction.

TABLE 11

RECOMMENDED ALTERNATIVE:

ALTERNATIVE #6

Proposed Improvement	Probable Construction Cost
1. Lower Cemetery Detention with outfall piped to Comstock Drive	$ 413,000
2. Evans Avenue culvert modification	10,000
3. Manogue High School athletic field pipe	112,000
4. Route Area "J" above Evans Avenue	130,000
5. Pipe from Reno Rendering to Sutro Street	183,000
6. Oddie Boulevard flood wall and Montello Modifications	169,000
7. Second pipe from Reno Rendering to Paradise Pond	828,000
Total	$1,845,000

It should be noted that if the Evans Avenue culvert modification is omitted, the Manogue High School pipe and the second pipe from Reno Rendering to Paradise Pond must be upsized two pipe sizes at an additional cost of $376,000.

APPENDIX 1

PROJECT CONFERENCE MEMORANDUM

SEA Incorporated
CONSULTING ENGINEERS
950 Industrial Way
Sparks, NV 89431-6092
(702)358-6931
Fax. No.: 358-6954

Project Name: Evans Avenue Drainage Study

Project No.: 150-08-1

Date: September 7, 1989

Meeting Place:

Telephone Call: John Palm
Nevada Water Planning Div.
(885-4380)

Attending:

Discussion:

Requirements for review of a detention dam structure that qualifies as a dam were discussed. The following items were noted:

1. Prior to construction a permit will be required. The items required for obtaining a permit are as follows

    a. Three sets of plans and specifications.
    b. One copy of the design report containing geotechnical and hydrological information.
    c. The application form.
    d. $500 review fee.

2. The emergency spillway for a dam considered "high risk" (such as one above an urban area) is normally required to pass the probable maximum flood (PMF). The requirement for the spillway to pass the full PMF may be reduced if justified. Justification would involve proof that a dam break during the PMF would cause no greater safety risk downstream than would the PMF without a structure. Under these circumstances, the spillway capacity could be reduced to one-half of the PMF.

3. It was recommended that a meeting be held prior to the final design stage to discuss the proposed detention dam and spillway with the State. This meeting would be attended by John Palm and Mike Turnipseed of the Nevada Water Planning Division.

SEA, INCORPORATED

Guy A. Sharp

Guy A. Sharp, P.E.
Project Engineer

GAS:jk

cc: Bob Gottsacker - City of Reno
Richard Arden
Frank Alverson
Pete Etchart
File

PROJECT CONFERENCE MEMORANDUM

SEA incorporated
CONSULTING ENGINEERS
950 Industrial Way
Sparks, NV 89431-6092
(702)358-6931
Fax. No.: 358-6954

Project Name: Evans Avenue Drainage Study

Project No.: 150-08-1

Date: November 3, 1989

Meeting Place:

Telephone Call: John Palm
Nevada Water
Planning Div. (885-4380)

Attending:

Discussion:

The status of the existing Evans Avenue embankment as a possible detention structure was discussed. The following items were noted:

1. The State would consider the embankment to be a roadway and not a dam. The existing culvert underneath the embankment was designed to carry the 100-year flood at the time of construction. The fact that subsequent studies using future development show that a higher flow will cause some impoundment during the future 100-year flood would not be considered in qualifying it as a dam.

2. The State typically lets the owners of such structures take the risk of failure should they decide to make modifications to the embankment, therefore, the State would not require a permit for construction on this embankment and would not require the structure to be retrofitted to dam standards as a result of any changed hydraulic criteria.

SEA, INCORPORATED

Guy A. Sharp

Guy A. Sharp, P.E.

GAS:jk

cc: Bob Gottsacker
Richard Arden
Frank Alverson
Pete Etchart
File

APPENDIX 2

APPENDIX 2

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED DETENTION DAM SITES
MANOGUE HIGH SCHOOL/EVANS AVENUE DRAINAGE STUDY
RENO, NEVADA

INTRODUCTION

Presented herein are the results of our preliminary geotechnical investigation of the alternate sites for the proposed detention dam or dams to be located along the drainage channel which parallels North Virginia Street near Our Mother of Sorrows Cemetery and north of Reno Rendering, Reno, Nevada. The proposed dam sites are will be located within Sections 1 and 2, Township 19 North, Range 19 East; and Section 35, Township 20 North, Range 19 East.

The purpose of this study was to:

1. Determine the general soil and groundwater conditions at the alternate proposed detention dam sites (George's Den, Upper Cemetery, Lower Cemetery, and Reno Rendering.

2. Determine the general suitability of the proposed dam sites with regard to potential construction problems and availability of suitable borrow and slope protection.

3. Evaluate the suitability of the existing Evans Avenue roadway embankment as a detention dam.

4. Make general recommendations concerning additional geotechnical exploration necessary to provide information needed for final detention dam design.

The area covered by this investigation is shown on Plate 1 - Plot Plan. The investigation included field exploration, laboratory testing and engineering analysis to determine the physical and mechanical properties of the various foundation materials. Results of our field and laboratory testing programs are included in this report and form the basis for all conclusions and recommendations.

PROJECT DESCRIPTION

A drainage channel (intermittent stream) runs parallel to and just east of North Virginia Street between the roadway and the Union Pacific Railroad embankment. This channel continues beneath North McCarran Boulevard, Evans Avenue, the Union Pacific Railroad embankment, Manogue High School Football Field, and Valley Road through various drainage pipes and open channel sections before emptying into a meadow by Reno Rendering on Wells Avenue. The drainage from this channel is partially collected at the east end of this meadow into a 36-inch pipeline and then transported along Oddie Boulevard to Paradise Pond.

For many years the Union Pacific Railroad embankment although not designed as a dam has acted as a detention structure for this drainage. However, the construction of Evans Avenue reduced the detention capacity by about one-half. Also, Evans Avenue was not designed as a detention facility, but instead was designed to pass the 100 year flood through a 72-inch diameter pipe. Following construction of Evans Avenue, the Union Pacific Railroad decided to protect their embankment by constructing discharge pipes through their embankment which will pass this 100 year design flood downstream without any detention. Consequently, areas that were already subject to flooding along Valley Road and in the meadow adjacent to Reno Rendering will receive these flood waters without the benefit of detention.

The City of Reno is therefore proposing to construct a detention dam to hold the 100 year design flood along the drainage channel somewhere north of McCarran Boulevard or alternatively employ Evans Avenue as a detention structure if feasible. They may also wish to construct another detention dam in the meadow adjacent to Reno Rendering above Wells Avenue.

Three alternate detention dam sites are being evaluated north of McCarran Boulevard (see Plot Plan): the George's Den site, the Upper Cemetery site and the Lower Cemetery site. An earthfill dam at the George's Den site would be approximately 26 feet high and 310 feet in length. Similar dams at the Upper and Lower Cemetery sites would be approximately 33 and 38 feet high and about 390 and 415 feet in length, respectively. Freeboard (included in the quoted approximate dam heights) would be about 3 feet. The Lower Cemetery Dam may also be required to store irrigation water in addition to the 100 year design flood. Two different dam configurations are also being considered in the meadow at Reno Rendering. One configuration would be 14 feet high and 790 feet in length while the other would be 20 feet high and 945 feet in length. Both of these latter two alternatives would be L-shaped structures.

SITE CONDITIONS

Four sites were explored by backhoe test pits for the proposed detention dams: George's Den, Upper Cemetery, Lower Cemetery, and Reno Rendering.

The Lower Cemetery site is characterized by more than 14 feet of fill at the western abutment; 5 to 7.5 feet of fill, litter and debris at streambed elevation; and exposed bedrock at the east abutment. The central portion of the site by the stream is well vegetated with trees, grass and shrubs. A 3-foot high embankment has been constructed at this location. The western portion of the dam site is covered by engineered fill placed to increase the size of Our Mother of Sorrows Cemetery. This fill is bisected by a drainage channel which services a culvert beneath Virginia Streets.

The Upper Cemetery site is characterized by shallow depths to bedrock and is well vegetated by small trees and brush near the streambed. Construction debris associated with the construction of the new mausoleum for Our Mother of Sorrows Cemetery has been dumped at this location. Sparse grass and sagebrush are located to each side of the heavily vegetated streambed area.

Hydrothermally altered, deeply weathered, volcanic bedrock is present at shallow depth at the George's Den site. The site is covered by vegetation in a manner similar to the other sites previously mentioned. Underground telephone cables and gas pipelines traverse the site.

The Reno Rendering site is a broad expanse of pasture land with thick grass cover. The Orr Ditch runs along the north edge of the meadow and is elevated relative to the meadow. Tall trees and wet ground near the streambed testify to considerable seepage from the drainage channel and probably Orr Ditch also. Relief is moderate at the site with a gentle slope towards the center of the meadow and eastwards. At the east end of the meadow the grass cover is absent where fill operations have occurred. Several fences traverse the property.

EXPLORATION

Exploration was performed in September, 1989 to determine the general soil and groundwater conditions at the proposed dam sites. The locations of the test pits are shown on Plate 1 - Plot Plan. Materials encountered during the exploration were logged by an engineering geologist. Representative samples were returned to our Sparks laboratory for testing and further classification. Test pit logs are presented on Plate 2, and a graphic soils classification chart explaining log symbols and terminology is presented as Plate 3.

LABORATORY TESTING

Representative samples of the soils encountered were tested in the laboratory to determine grain size distribution, moisture content and plasticity. Results of these gradation and plasticity tests are presented on Plates 4 and 5, and were used to verify field classifications of the soils in accordance with the Unified Soils Classification System, Plate 3. Classification in this manner is an indication of the soils' mechanical properties, and is used in part to evaluate soil bearing capacity, settlement characteristics and suitability as embankment material. All tests were conducted in accordance with ASTM standards.

GEOLOGIC AND GENERAL SOIL CONDITIONS

The cemetery and George's Den sites are underlain by andesitic, volcanic bedrock of the Tertiary Alta Formation. Much of the rock in this area has been variably hydrothermally altered and heavily weathered. As a result, the bedrock along the east side of the cemetery sites and the entirety of the George's Den site contains numerous clay filled fissures and grades into sandy clay. This sandy clay where present is generally highly plastic and expansive. The bedrock and/or the sandy clay veneer are covered by variable thicknesses of clayey sand, silty sand, sandy clay and sandy gravels derived from weathering of this and other nearby Tertiary formations.

The lower cemetery site has abundant uncontrolled fill with concrete, asphaltic concrete, wood and garbage debris which is largely confined to the area near the stream channel. The west end of this site lies in more than 14 feet of uncontrolled heterogeneous gravel-clay-sand fill.

The Reno Rendering site is overlain by Quaternary alluvial fan deposits of Peavine Mountain. These alluvial fan deposits are typically a clayey sandy gravel of great thickness overlain by 3 to 4 feet of highly plastic, gravelly sandy clay.

Shallow groundwater was encountered at all sites, but was confined to areas along the streambed and below Orr Ditch.

GEOLOGIC HAZARDS

The site as well as most of Washoe County, Nevada lies within Seismic Zone 3, a region with a high potential for major earthquake damage. The maximum credible earthquake for the Truckee Meadows is generally accepted as a magnitude 7.5 - 7.8 event. Available literature (Reno Folio Earthquake Hazards Map; Nevada Bureau of Mines and Geology; Bingler, E.C.; 1974) shows the closest potentially active faults are located on the lower slopes of the foothills of Peavine Mountain some one half mile to the west and southwest. The nearest mapped fault considered active is located some 4 miles to the east in Sparks near Pyramid Way and Greenbrae Drive.

By conventially accepted criteria, faults having movement within the last 11,000 years (Holocene) are defined as active, whereas those with activity occurring between 11,000 and 1,800,000 years ago (Pleistocene) are considered active or potentially active. Faults of this later age are generally classified as active only if field exploration or geomorphology indicates a long history of recurring activity.

A detailed liquefaction study is beyond the scope of this report. With shallow depth to bedrock at most of these dam sites and the presence of coarse gravel to an unknown depth at the Reno Rendering site, the potential for liquefaction at these sites is believed to be low to moderate.

There is some potential for dust generation during site grading and construction of the proposed dams.

DISCUSSIONS AND RECOMMENDATIONS

Geotechnical Suitability of Proposed Dam Sites

Cemetery and George's Den Sites

A dam could be constructed at any of these three sites which would have similar construction problems and similar problems of material availability. From the standpoint of potential site suitability for construction, the Lower Cemetery site appears slightly better than the other two proposed locations. The eastern side of the dam site is exposed bedrock which would not require excavation unlike the upstream locations. Of more significance, the western side of the site contains thick uncontrolled fills of clay-gravel-sand which could provide considerable material for embankment construction unlike the upstream dam sites with their generally thin soil cover. However, the western abutment fill at the Lower Cemetery site was only recently placed to expand Our Mother of Sorrows Cemetery, and the Diocese may prefer one of the upstream sites in order to preserve their investment in site improvements. Also, a considerable quantity of unusable debris fill must be removed from the drainage channel at the Lower Cemetery site. Clay core material for each of the proposed dams could probably be obtained from the thicker deposits in the stream channel bottom located at the dam site and upstream.

Specifically, volcanic bedrock is present at shallow depths of a few feet or is at the surface at these dam sites. This bedrock is highly weathered and contains numerous zones and partially filled cracks of highly expansive clay derived from hydrothermal alteration of the bedrock. Generally, the bedrock will provide a suitable foundation for a dam at each of the sites, but the weathered profile and possibly considerable cracking of the upper flows will require thorough subsurface exploration along the dam axis in the form of test pits, bedrock coring and packer pressure testing to establish the depth of the zone of alteration and cracking and the permeability of the dam foundation and abutments. A dam at any of these sites will require construction of a clay core keyed into the bedrock.

Where the bedrock is buried, the heavily weathered soil profile generally includes a zone of highly plastic sandy clay with occasional gravel and cobbles which is residual in nature or is derived from the weathering of nearby volcanic bedrock and subsequent transport. This clayey zone is typically 2 to 4 feet thick where present, and will provide good material for construction of the clay core. An extensive program of shallow test pit exploration will be necessary to locate sufficient quantities of this clay, and the thinness of this layer will probably require stripping over a large area in the bottom of the drainage channel to obtain sufficient quantities of this material.

The soil cover above the residual clay zone consists of clayey sand, sandy clay, clayey and silty sandy gravel and gravelly sands all of which can be used to construct the embankment outside of the clay core. However, except for the west side of the proposed dam at the Lower Cemetery site which is covered by a thick engineered fill, the soil cover including the clay zone is only a few feet thick and most of this cover is thought to be in the narrow bottom of the drainage channel. Consequently, a dam at either of the upper two sites will again probably require stripping over a large area to obtain sufficient material. After the extensive shallow test pit exploration to locate sufficient material has been conducted and available quantities have been computed, it may still be necessary to import some embankment and clay core material at each of these three proposed dam sites.

Finally, no material was observed in the preliminary test pit exploration which would provide a good source of embankment riprap. It is possible that excavation during the preparation of the volcanic bedrock foundation and abutments especially at the Lower Cemetery site might yield some suitable material for embankment slope protection, but generally this type of weathered bedrock does not provide suitable material.

Existing Evans Avenue Embankment

Reno Rendering Dam Sites

The meadow which is proposed as the location for each of two different dam configurations is broad and gently sloping below the Orr Ditch embankment. The soil profile in the meadow consists of a veneer of topsoil overlying a thin, gravelly sandy clay B horizon underlain by gravelly clayey sand and clayey sandy gravel alluvial fan deposits of unknown but believed to be great thickness.

The B horizon gravelly sandy clay and possibly the gravelly clayey sand might be suitable core material. The underlying sandy gravel and clayey sandy gravel would be suitable for embankment materials especially if the cleaner sandy gravels are mixed with some clay fines to provide binder. Excavation in the proposed impoundment area could provide a source for the embankment materials, and such an excavation would enable the construction of a lesser dam height Rock riprap would have to be imported at this location.

The proposed dams would be founded directly on the alluvial gravels. A grout curtain beneath the dam axis would probably be necessary to slow down the seepage losses and prevent embankment foundation instability and possible near quick conditions at the downstream face of the dam. Also, any excavated impoundment area would expose the flood waters to the underlying gravels. While seepage losses in temporary detention facilities are not inherently a concern it should be determined in this case if such seepage losses into the gravels could result in excessive seepage problems in the urban area down gradient. If such is the case, then the impoundment area should not be excavated and tests of the permeability of the B horizon material should be conducted, or the clay cover excavated to expose the underlying gravel embankment borrow should be replaced and compacted to seal off the gravels. In the latter case, all of the clay core material would have to be imported also.

Further Geotechnical Exploration

For any of the dam sites above McCarran Boulevard an extensive subsurface exploration program would be required along the dam axis consisting of test pits, soil borings, rock coring to penetrations of 20 feet or better into the rock and in situ hydraulic pressure packer testing to determine the thickness and condition of the heavily weathered bedrock zone and the permeability of this bedrock, especially in the abutments. Such a program in conjunction with the appropriate laboratory testing such as shear, consolidation and permeability tests would provide sufficient information for the design of a proper dam foundation and design of the dam embankments and core. An extensive shallow test pit exploration in the bottom of the drainage channel should also be conducted to determine the areal extent of both potential clay core and embankment material. The latter exploration for suitable borrow materials would not require sophisticated soil testing but generally just index and gradations to identify potential borrow materials. Of course, similar index and gradation testing would be necessary for any proposed imported borrow materials to determine their suitability.

For the proposed Reno Rendering meadow site, deep test pits excavated with a large track hoe along the dam axis, and more packer tests in the gravels would be required. Laboratory testing similar to that required for the dam sites north of McCarran would also be necessary to define embankment and foundation conditions and suitable imported embankment and clay core materials.

The exact scope and estimated cost of the necessary exploration and laboratory testing can best be determined when the specific sites for the proposed dams are known.

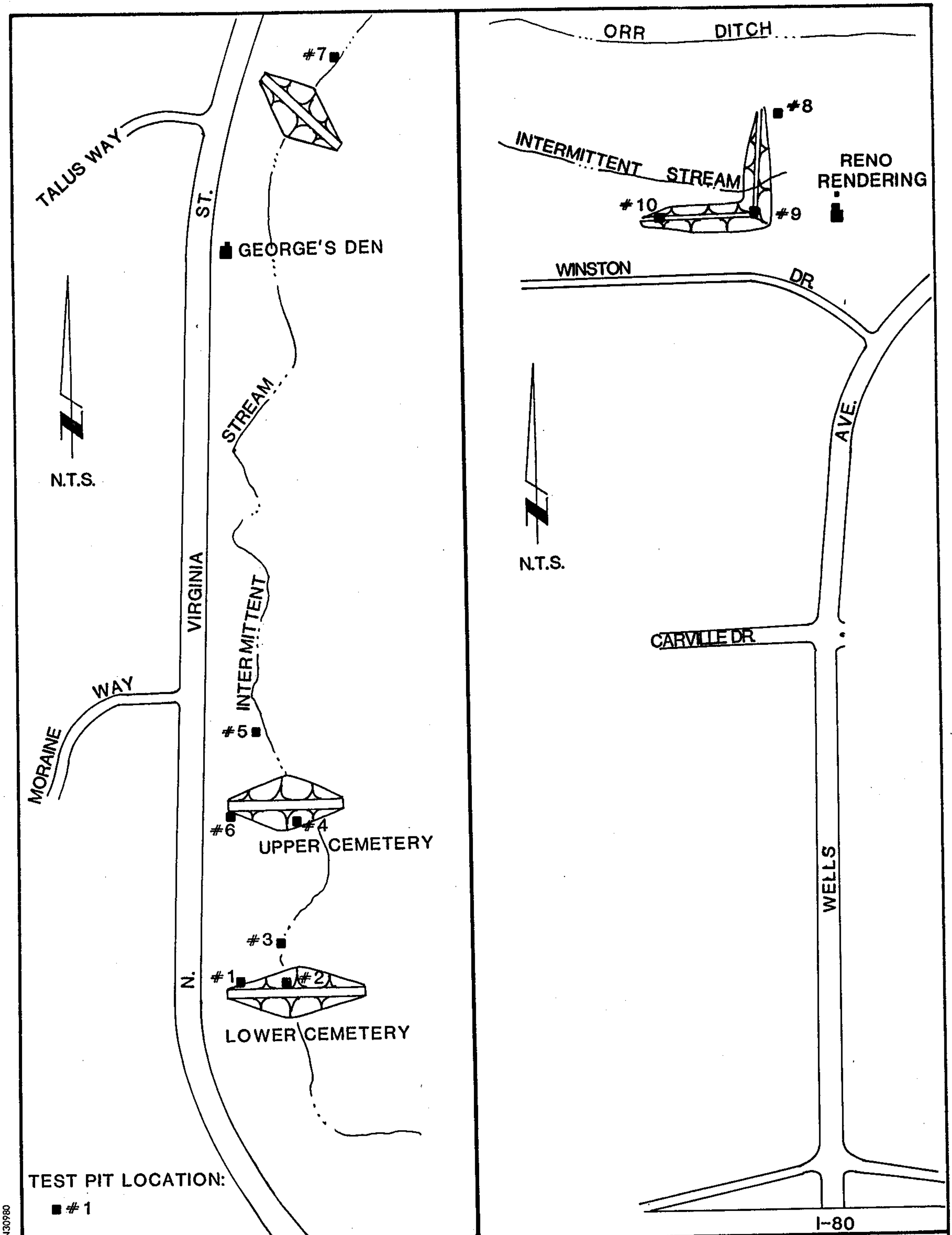
STANDARD LIMITATION CLAUSE

This report has been prepared in accordance with generally accepted engineering practices. The analyses and recommendations submitted are based upon field exploration performed at the location shown on Plate 1 - Plot Plan, of this report. This report does not reflect soils variations that may occur between these locations. The variations may become evident during the construction period, at which time re-evaluation of the recommendations may be necessary. We recommend our firm be retained to perform construction observation in all phases of the project related to geotechnical factors to insure compliance with our recommendations.

Equilibrium water level readings were made on the date shown on Plate 2 - Log of Borings or Test Pit Logs, of this report. Fluctuations in the water table may occur due to rainfall, temperature, seasonal runoff, or adjacent irrigation practices. Construction planning should be based o assumptions of possible variations.

This report has been prepared to provide information allowing the Architect or Engineer to design the project. In the event of changes in the design or location of the project form the time of this report, recommendations should be reviewed and possibly modified by the Geotechnical Engineer. If the Geotechnical Engineer is not accorded the privilege of making this recommended review, he can assume no responsibility for misinterpretation or misapplication of his recommendations or their validity in the event changes have been made in the original design concept without his prior review. The Geotechnical Engineer makes no other warranties, either expressed or implied, as to the professional advice provided under the terms of this agreement and included in this report.

PLATES



sea CONSULTING ENGINEERS

RENO/SPARKS, NEVADA
LAS VEGAS, NEVADA
PHOENIX, ARIZONA

PROPOSED DRAINAGE
RELIEF DAMS
CITY OF RENO

Project No. 150-08-1

PLATE 1

TEST PIT LOG

TEST PIT NO. 1 Lower Cemetery
LOGGED BY Trygve Loken
DATE 9-28-89

GROUND ELEVATION
GROUND WATER DEPTH Not Encountered
DATE MEASURED

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG	DESCRIPTION
					0 - 2.0 Moist, compact, brown Gravelly Clayey Sand Fill with 30% low to medium plastic fines, 50% very fine to coarse sand, 20% gravel to 3" diameter.
			2		
	1A				2.0 - 3.0 Moist, compact, brown, Clayey Gravelly Sand Fill with 25% medium plastic fines, 40% very fine to coarse sand, 35% gravel to 3" diameter. Cobbles to 6" diameter comprise 10%.
			4		
			6		
	1B	18			3.0 - 14.0 Moist to very moist, compact, brown Gravelly Clayey Sand Fill with 40% medium plastic fines, 50% very fine to coarse sand, 10% gravel to 3" diameter. Very little variation
			8		
			10		
			12		

TEST PIT NO. 2 Lower Cemetery
LOGGED BY Trygve Loken
DATE 9/28/89

GROUND ELEVATION
GROUND WATER DEPTH 5.5'
DATE MEASURED

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG	DESCRIPTION
					0 - 1.5 Slightly moist, compact, brown Silty Gravelly Sand Fill with 20% slightly plastic fines, 50% very fine to coarse sand, 30% gravel to 3" dia.
			2		
			4		1.5 - 7.5 Debris - asphaltic concrete chunks, tree roots, clay, sand, and gravel.
			6		7.5 - 9.5 Wet, stiff, dark purplish brown, Sandy Clay with 65% high plastic fines, 35% very fine to medium sand. Trace coarse sand and gravel to 1/4" diameter.
			8		
	2B	38			
	2C	27	10		9.5 - 10.2 Grades into wet, compact, brown-tan Sandy Clay with 55% medium plastic fines, 45% very fine to coarse sand. Trace gravel to 1/2" diameter.
			12		

Description: Describe soil type by unified soil classification system with emphasis on in-place or natural condition.

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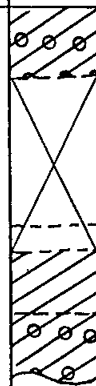
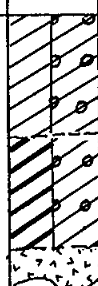
TEST PIT LOG

TEST PIT NO. 3 Lower Cemetery
LOGGED BY Trygve Loken
DATE 9-28-89
GROUND ELEVATION
GROUND WATER DEPTH 5.0
DATE MEASURED 9-28-89

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG	DESCRIPTION
					0 - 1.5 Slightly moist, slightly compact, browr Clayey Gravelly Sand Fill with 30% low plastic fines, 40% very fine to coarse sand, 30% gravel to 3" diameter.
			2		1.5 - 4.5 Debris, garbage, plastic, headlights, pots, pipe, wiring, etc.
			4		
	3B	28	6		4.5 - 5.0 Wood - deadfall tree limbs
p=0.5 tsf	3C		8		5.0 - 6.5 Wet, stiff gray Sandy Clay with 55% medium plastic fines, 45% very fine to coarse sand, 5% gravel to 1/2" diameter.
			10		6.5 - 8.0 Grades into wet, soft to firm, gray-tan Clayey Sand with 40% medium plastic fines, 60% very fine to coarse sand, 5% gravel to 1/2" diameter
			12		

TEST PIT NO. 4 Upper Cemetery
LOGGED BY Trygve Loken
DATE 9-28-89
GROUND ELEVATION
GROUND WATER DEPTH 5.5
DATE MEASURED 9-28-89

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG	DESCRIPTION
					0 - 2.5 Slightly moist, compact, brown Clayey Sand to Sandy Clay with 50% medium to high plastic fines 50% very fine to coarse sand. Trace gravel to 3/4" diameter.
			2		
			4		2.5 - 5.0 Moist, stiff, greenish gray Sandy Clay to Clayey Sand with 50% high plastic fines, 50% very fine to coarse sand. Deeply weathered bedrock.
			6		5.0 - 5.5 Fractured, weathered, gray volcanic bedrock, Near refusal.
			8		
			10		
			12		

Description: Describe soil type by unified soil classification system with emphasis on in-place or natural condition.
p- hand penetrometer value (tsf)

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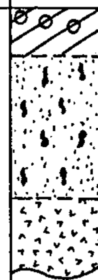
TEST PIT LOG

TEST PIT NO. 5 Upper Cemetery
LOGGED BY Trygve Loken
DATE 9-28-89

GROUND ELEVATION
GROUND WATER DEPTH 4.0
DATE MEASURED 9-28-89

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG
	5A	13	2	
			4	
			6	
			8	
			10	
			12	



DESCRIPTION

0 - 1.0
Moist, slightly compact, brown Clayey Gravelly Sand Fill with 30% low plastic fines, 40% very fine to coarse sand, 30% gravel to 3" diameter. Cobbles and boulders to 24" diameter comprise 5% (Truckee river, round boulders).

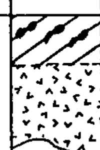
1.0 - 4.0
Moist, compact, brown Sandy Gravel with 5% low to medium plastic fines, 25% very fine to coarse sand, 70% gravel to 3" diameter. Cobbles to 5" diameter comprise 10% of mass.

4.0 - 5.5
Wet, soft to hard, fractured weathered gray volcanic Bedrock.

TEST PIT NO. 6 Upper Cemetery
LOGGED BY Trygve Loken
DATE 9-28-89

GROUND ELEVATION
GROUND WATER DEPTH Not Encountered
DATE MEASURED

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG
	6A	8		
			2	
			4	
			6	
			8	
			10	
			12	



DESCRIPTION

0 - 1.0
Moist, compact, tan Clayey Sandy Gravel with 10% slightly to low plastic fines, 30% very fine to coarse sand, 60% gravel to 3" diameter. (Graded weathered bedrock)

1.0 - 2.5
hard, fractured, weakly weathered tan volcanic Bedrock.

Description: Describe soil type by unified soil classification system with emphasis on in-place or natural condition.

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TEST PIT LOG

TEST PIT NO. 7 Georges Den Site
LOGGED BY Trygve Loken
DATE 9-28-89

GROUND ELEVATION
GROUND WATER DEPTH 2.5
DATE MEASURED 9-28-89

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG	DESCRIPTION
	7A	24			0 - 2.0 Moist, compact, brown Clayey Sand with 30% low plastic fines, 70% very fine to coarse sand. Trace gravel to 3/4" diameter.
			2		2.0 - 3.0 Wet, loose, gray Clayey Sandy Gravel with 10% low plastic fines, 30% very fine to coarse sand 60% gravel to 3/4" diameter.
			4		
			6		3.0 - 7.0 Very moist, slightly stiff, yellow-orange Sandy Clay with 70% high plastic fines 30% very fine to coarse sand. Lower 2 feet with numerous soft purple cobbles.
			8		
			10		7.0 - 12.0 Moist, soft, purple, weathered volcanic Bedrock.
			12		

TEST PIT NO. 8 Reno Rendering Site
LOGGED BY Trygve Loken
DATE 9-29-89

GROUND ELEVATION
GROUND WATER DEPTH 2.0
DATE MEASURED 9-29-89

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH	LOG	DESCRIPTION
	8A				0 - 1.0 Wet, slightly compact, brown Silty Gravelly Sand with 20% slightly plastic fines, 50% very fine to coarse sand, 30% gravel to 3/4" diameter
	8B		2		
p=1.25 tsf	8C	29			1.0 - 2.0 Wet, stiff, light brown, Gravelly Sandy Clay with 40% medium plastic fines, 40% very fine to coarse sand, 20% gravel to 3/8" diameter.
			4		
	8D	14	6		2.0 - 4.0 Wet, stiff, gray Gravelly Clayey Sand with 45% high plastic fines, 45% very fine to coarse sand, 10% gravel to 3/4" diameter.
			8		
			10		4.0 - 9.0 Wet, loose, brown Sandy Gravel with 5% slightly low plastic fines, 30% very fine to coarse sand, 65% gravel to 3" diameter. Trace cobbles to 5" diameter.
			12		

Description: Describe soil type by unified soil classification system with emphasis on in-place or natural condition.
p- hand penetrometer value (tsf)

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TEST PIT LOG

TEST PIT NO. 9 Reno Rendering Site
LOGGED BY Trygve Loken
DATE 9-29-89
GROUND ELEVATION
GROUND WATER DEPTH Not Encountered
DATE MEASURED

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH
	9A		
			2
	9B	21	
			4
			6
	9C		
			8
			10
			12

DESCRIPTION

0 - 1.0
Moist, slightly compact, brown Gravelly Silty Sand Fill with 20% slightly plastic fines, 60% very fine to coarse sand, 20% gravel to 3/4" diameter.

1.0 - 4.0
Moist, stiff, light brown Gravelly Sandy Clay with 35% high plastic fines, 35% very fine to coarse sand, 30% gravel to 1" diameter.

4.0 - 11.5
Moist, compact, light brown Clayey Sandy Gravel with 20% low plastic fine, 30% very fine to coarse sand, 50% gravel to 3" diameter.

TEST PIT NO. 10 Reno Rendering Site
LOGGED BY Trygve Loken
DATE 9-29-89
GROUND ELEVATION
GROUND WATER DEPTH Not Encountered
DATE MEASURED

NOTES	SAMPLE NUMBER	MOISTURE PERCENT	DEPTH
p=0.75 tsf	10A		
			2
			4
	10B	12.4	
			6
			8
			10
			12

DESCRIPTION

0 - 3.0
Moist, firm, brown Gravelly Sandy Clay with 40% medium to high plastic fines, 40% medium to high plastic fines, 40% very fine to coarse sand, 20% gravel to 1/2" diameter. Top 0.3" darker, organic rich with roots.

3.0 - 12.5
Moist, compact, light brown Clayey Sandy Gravel with 10% low plastic fines, 25% very fine to coarse sand, 65% gravel to 3" diameter. Trace cobbles to 6". Becomes slightly less gravelly with depth.

Description: Describe soil type by unified soil classification system with emphasis on in-place or natural condition.
p- hand penetrometer value (tsf)

sea • RENO/SPARKS • LAS VEGAS GEOTECHNICAL DIVISION PROJECT NO. 150-08-1 PLATE 2e

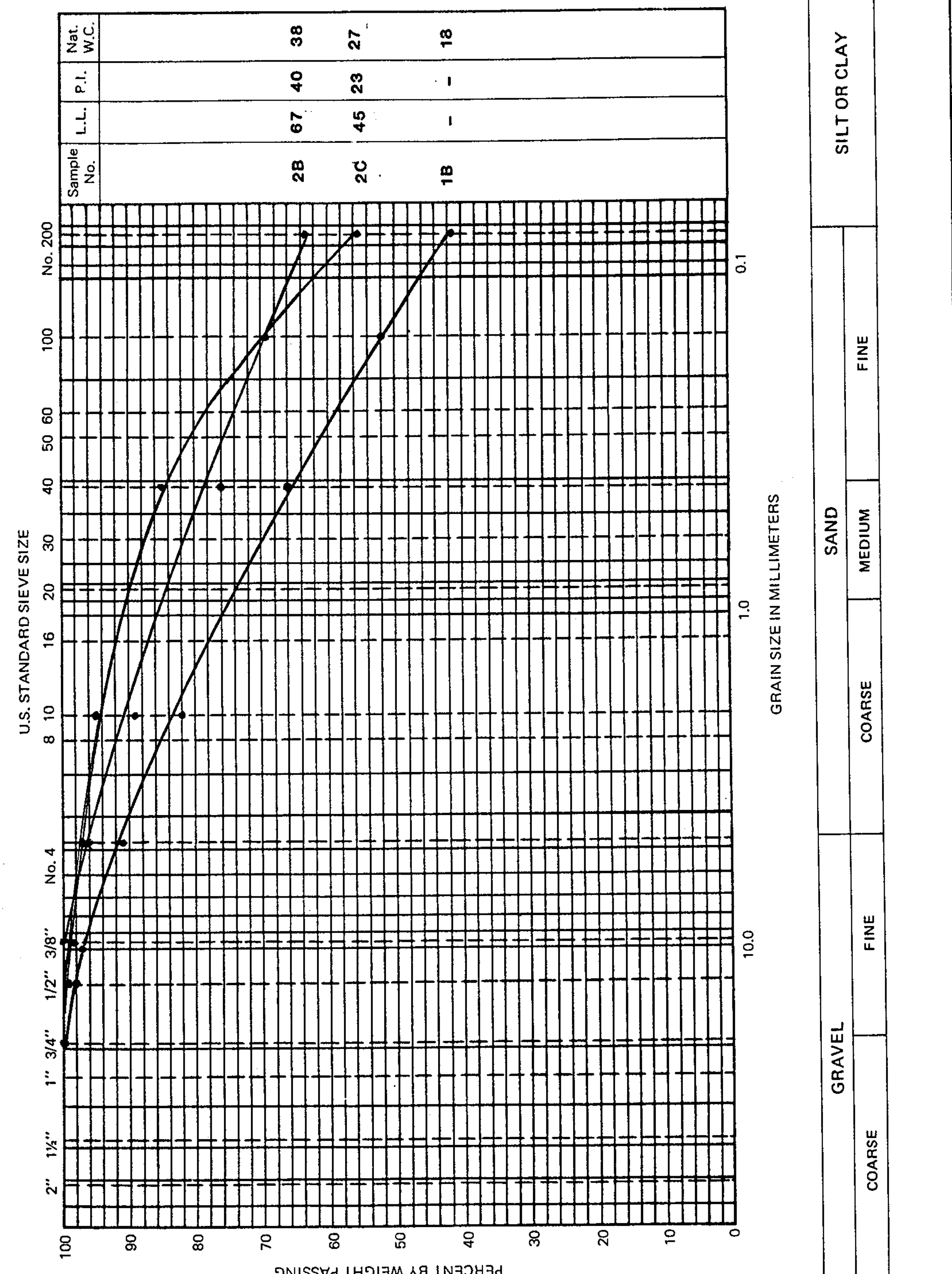
GRAPHIC SOIL CLASSIFICATION CHART

COARSE GRAIN SOILS — LESS THAN 50% PASSING NO. 200 SIEVE	GRAVELS	LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY- GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		APPRECIABLE AMOUNT OF FINES	GM	SILTY GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS	CLEAN NO FINES	SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		APPRECIABLE AMOUNT OF FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURE
FINE GRAIN SOILS — MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANNIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS
			CL	SANDY CLAYS, SILTY CLAYS, LEAN CLAYS.
			OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICTY.
		LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

MATERIAL CHANGE

ESTIMATED MATERIAL CHANGE

GRADATION CURVES



FORM 160

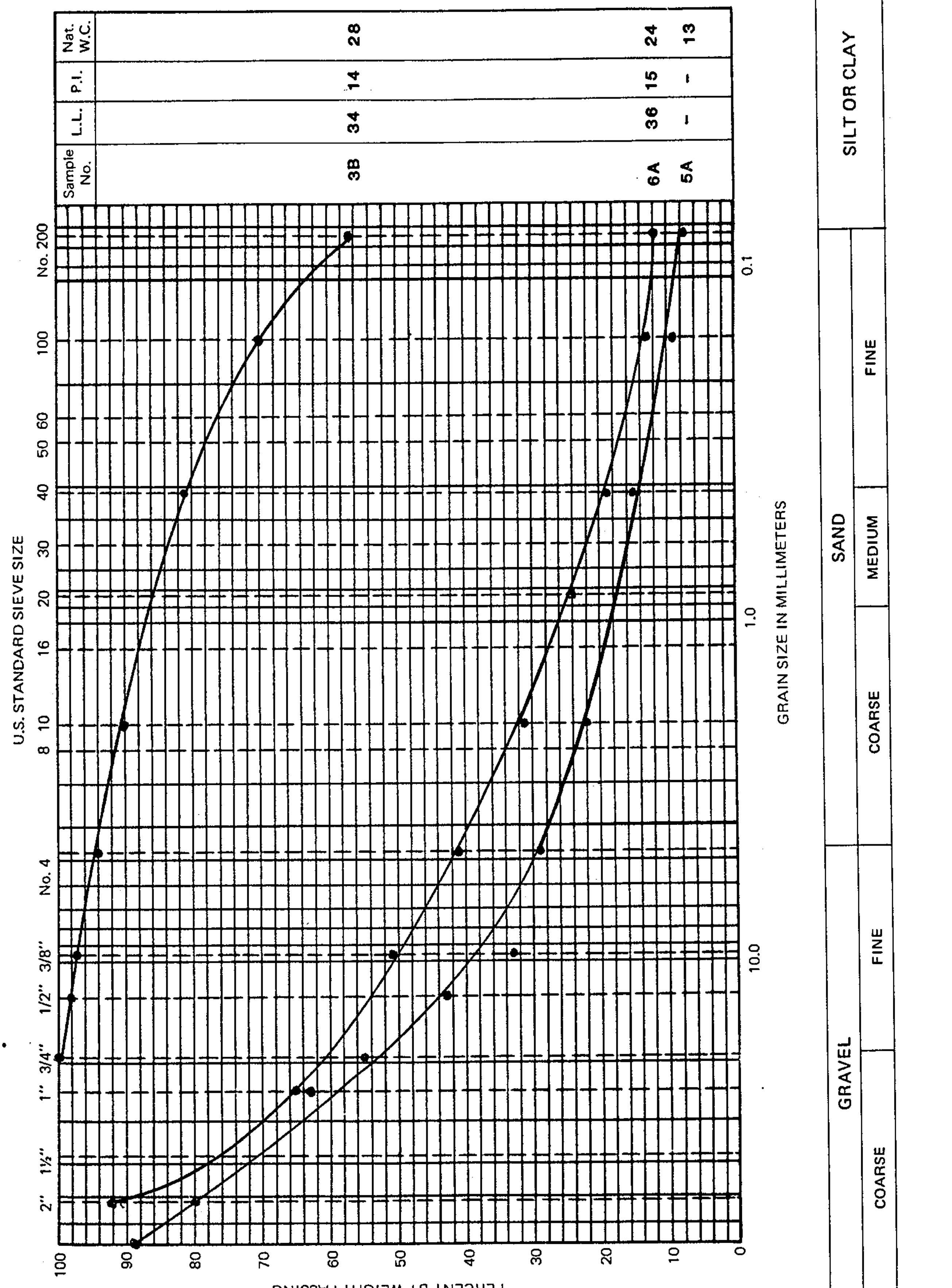
sea FOUNDATION DIVISION

PRELIMINARY STUDY
RENO DRAINAGE RELIEF

SPARKS, NEVADA • SEATTLE, WASHINGTON • LAS VEGAS, NEVADA

PROJECT NO. 150-08-1
PLATE 4a

GRADATION CURVES



Sample No.	L.L.	P.I.	Nat. W.C.
3B	34	14	28
6A	36	15	24
5A	-	-	13

sea FOUNDATION DIVISION

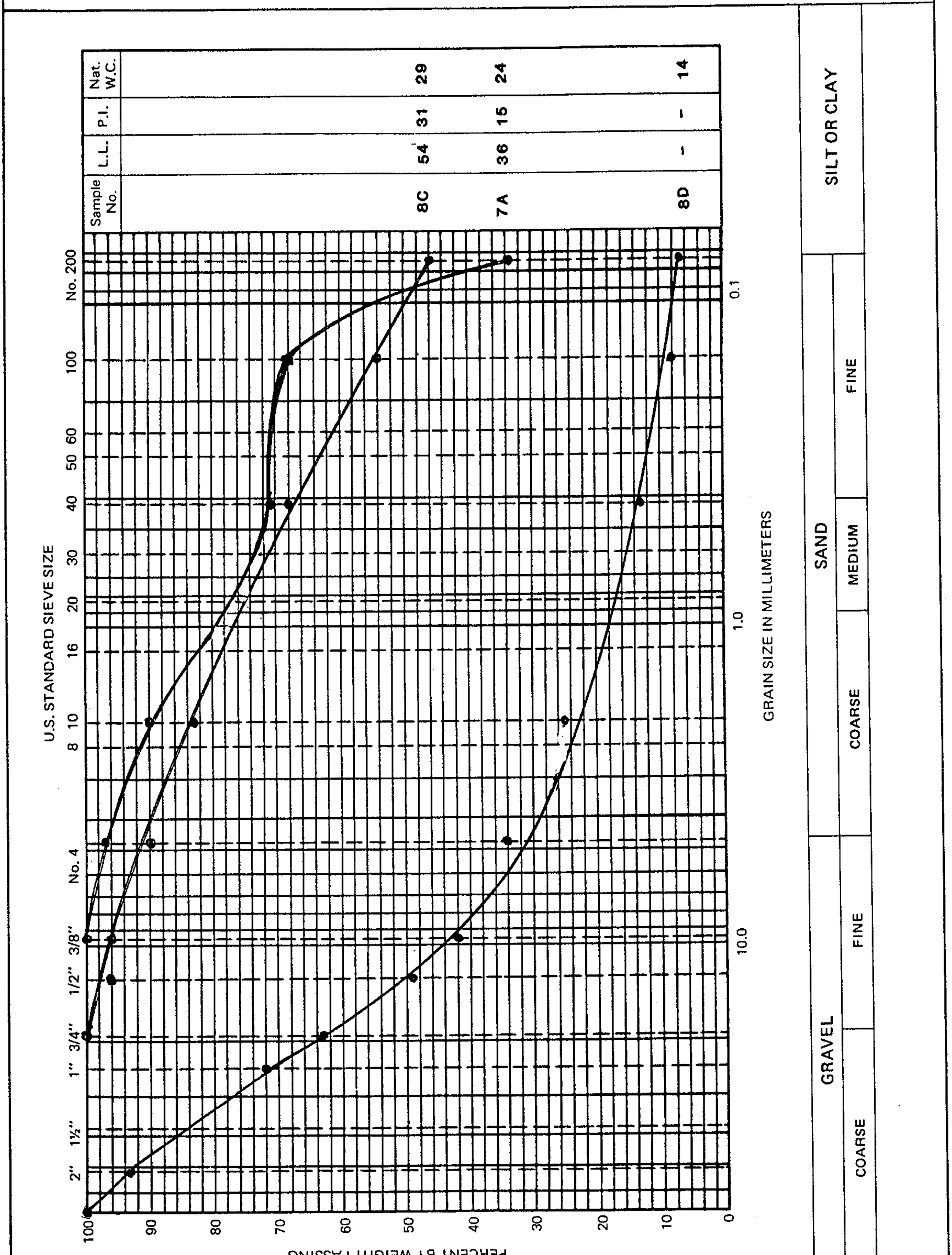
PRELIMINARY STUDY
RENO DRAINAGE RELIEF

SPARKS, NEVADA • SEATTLE, WASHINGTON • LAS VEGAS, NEVADA

PROJECT NO. 150-08-1
PLATE 4b

FORM 160

GRADATION CURVES



FORM 160

FOUNDATION DIVISION

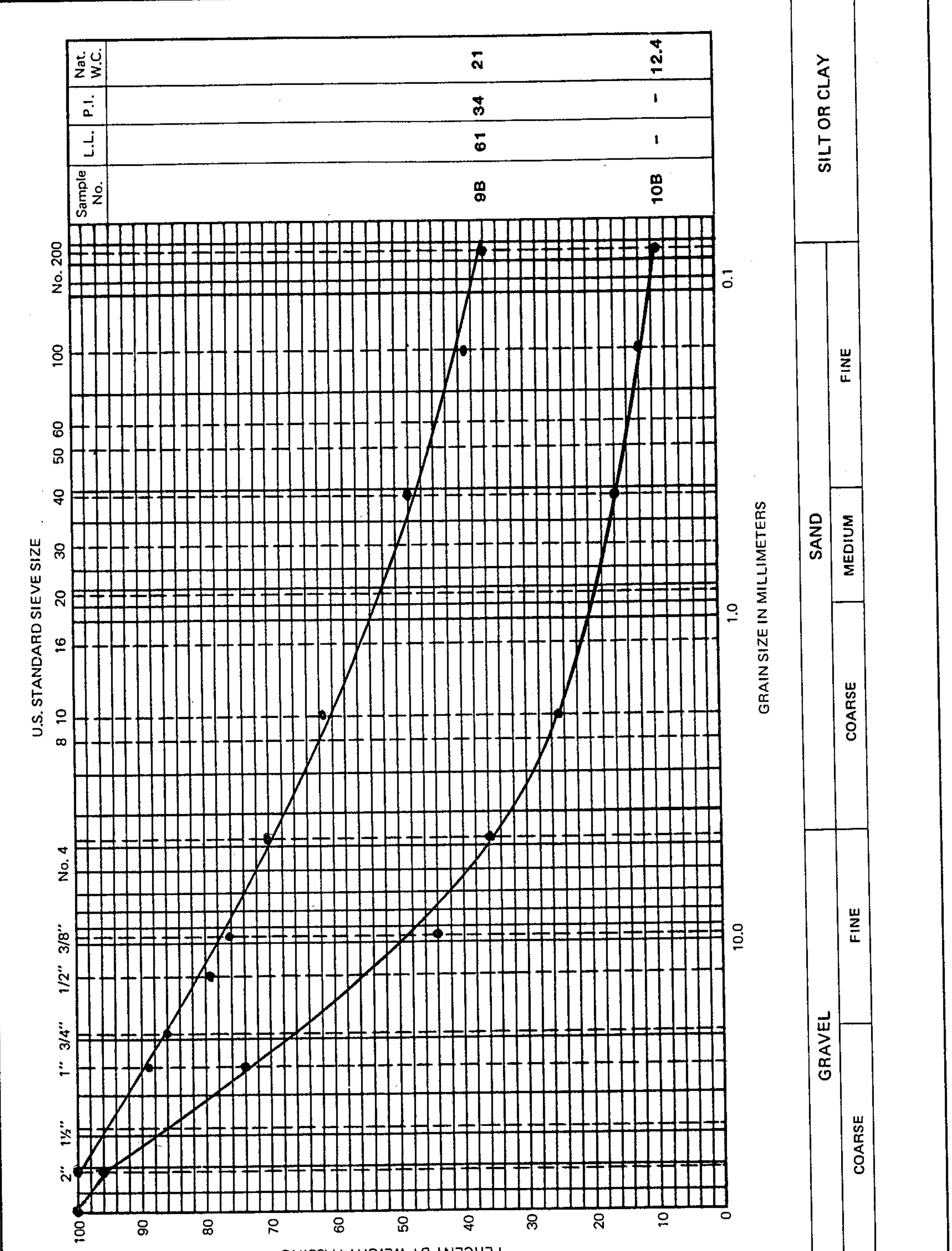
PRELIMINARY STUDY
RENO DRAINAGE RELIEF

SPARKS, NEVADA • SEATTLE, WASHINGTON • LAS VEGAS, NEVADA

PROJECT NO. 150-08-1
PLATE 4c

GRADATION CURVES

Sample No.	L.L.	P.I.	Nat. W.C.
9B	61	34	21
10B	-	-	12.4



TEST RESULTS

MECHANICAL ANALYSIS

Sample Number	1B	2B	2C	3B	5A	6A
Sieve Size	**Percent By Weight Passing**					
3 Inch					89	
2 Inch					80	92
1 Inch					63	65
3/4 Inch	100		100	100	55	64
1/2 Inch	98		99	98	43	56
3/8 Inch	97	100	98	97	33	51
No. 4	91	96	97	94	29	41
No. 10	82	89	95	90	22	31
No. 40	66	76	85	81	15	19
No. 100	52	69	69	70	9	13
No. 200	41.7	63.4	54.8	56.5	7.5	10.2
Liquid Limit	-	67	45	34	-	-
Plastic Index	-	40	23	14	-	-
Moisture Content	18	38	27	28	13	8

MECHANICAL ANALYSIS

Sample Number	7A	8C	8D	9B	10B
Sieve Size	**Percent By Weight Passing**				
3 Inch			100		100
2 Inch			93	100	96
1 Inch			72	89	74
3/4 Inch		100	62	86	65
1/2 Inch		96	49	79	54
3/8 Inch	100	96	42	76	44
No. 4	97	90	34	70	35
No. 10	90	83	25	61	25
No. 40	70	68	13	48	16
No. 100	68	54	8	39	12
No. 200	33.7	45.9	6.6	35.9	9.5
Liquid Limit	36	54	-	61	-
Plastic Index	15	31	-	34	-
Moisture Content	24	29	14	21	12.4

APPENDIX 3

sea incorporated
CONSULTING ENGINEERS

CONSULTING ENGINEERS

☒ **Reno/Sparks**
950 Industrial Way
Sparks, NV 89431-6092
(702) 358-6931

☐ **Las Vegas**
1405 Arville Street
Las Vegas, NV 89102
(702) 877-3000

☐ **Phoenix**
2920 N. 24th Ave., #6
Phoenix, AZ 85015-5948
(602) 257-4699

JOB NO. 150-08-1

PROJECT City of Reno Drainage Study DATE 10/26/87

SUBJECT Existing Pipe Capacities DESIGNED PBE/GS CHECKED

- UNR Access Road

96" ø CMP - based on entrance control with no head; discharge ≅ 400 cfs

To convey the approx. 100-yr. flow of 650 cfs you would need HW/D = 1.35

1.35(96") = 129.6" = 10.8' which is 2.8' above top of pipe.

∴ The 96" ø CMP can convey approx. the 100-yr. storm event.

- Comstock Drive

48" Ø RCP HW/D = 13/9 = 1.44 Q = 120 cfs - without flooding access road.

This is less than the 5-yr storm event of approx. 154 cfs.

- Oddie Blvd.

42" RCP (NE cor. Sutro & Oddie) s = .0120, n = .013

Q = 110 cfs

This is less than the 5-yr storm event flow coming out of Reno Rendering (232 cfs)

The downstream pipes convey the same or greater amount.

48" Ø RCP, s = 0.006% => 110 cfs

54" Ø RCP, s = 0.0044% => 130 cfs

CONSULTING ENGINEERS

SEA incorporated CONSULTING ENGINEERS

☒ Reno/Sparks
950 Industrial Way
Sparks, NV 89431-6092
(702) 358-6931

☐ Las Vegas
1405 Arville Street
Las Vegas, NV 89102
(702) 877-3000

☐ Phoenix
2920 N. 24th Ave., #6
Phoenix, AZ 85015-5948
(602) 257-4699

JOB NO. #0150-08-1

SHEET 2 OF 2

PROJECT City of Reno Drainage Study DATE 10/26/89

SUBJECT Existing Pipe Capacities DESIGNED PBE CHECKED

- McCarran Blvd

84" ϕ CMP HW/D = 50/7 = 7.14 Q ≅ 950 cfs based upon entrance control which is greater than the 100-yr. storm event (≅ 650 cfs)

Req'd. headwater to pass Q_{100}:

@ Q = 650 cfs , HW/D = 2.9

$$HW = \frac{84}{12}(2.9) = 20.3'$$

- Valley Road

72" ϕ RCP S = .025 '/' , n = .013

Capacity = 670 cfs flowing full

FORM 230

APPENDIX 4

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION

U.S. DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

HYDROMETEOROLOGICAL REPORT NO. 49

Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages

Prepared by
E. Marshall Hansen,
Francis K. Schwarz, and John T. Riedel
Hydrometeorological Branch
Office of Hydrology
National Weather Service

Silver Spring, Md.
September 1977

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C. 20402
Stock No. 003-017-00408-2

Table 6.3A.—Local-storm PMP computation, Colorado River, Great Basin and California drainages. For drainage average depth PMP. Go to table 6.3B if areal variation is required.

Drainage Sorrows Det. Dam Area 1.99± mi^2 (km^2)
Latitude ______ Longitude ______ Minimum Elevation 4681.2 ft (m)

Steps correspond to those in sec. 6.3A.

1. Average 1-hr 1-mi^2 (2.6-km^2) PMP for drainage [fig. 4.5]. 8.5 in. (mm)

2. a. Reduction for elevation. [No adjustment for elevations up to 5,000 feet (1,524 m): 5% decrease per 1,000 feet (305 m) above 5,000 feet (1,524 m)]. 100 %

   b. Multiply step 1 by step 2a. 8.5 in. (mm)

3. Average 6/1-hr ratio for drainage [fig. 4.7]. 1.64

	Duration (hr) 1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio of step 3 [table 4.4].	43	70	87	100	125	140	150	157	164	%
5. 1-mi^2 (2.6-km^2) PMP for indicated durations [step 2b X step 4].	3.6	6.0	7.4	8.5	10.6	11.9	12.8	13.3	13.9	in. (mm)
6. Areal reduction [fig. 4.9].	94.5	96	96	96.5	97	97.5	98	98.5	99	%
✱ 7. Areal reduced PMP [steps 5 X 6].	3.4	5.8	7.1	8.2	10.3	11.6	12.5	13.1	13.8	in. (mm)

8. Incremental PMP [successive subtraction in step 7]. ___ ___ ___ ___ ___ ___ in. (mm)

   ___ ___ ___ ___ } 15-min. increments

9. Time sequence of incremental PMP according to:

   Hourly increments [table 4.7]. ___ ___ ___ ___ ___ ___ in. (mm)

   Four largest 15-min. increments [table 4.8]. ___ ___ ___ ___ in. (mm) ratios than storms with high 3/1-hr ratios. The geographical distribution of 15-min to 1-hr ratios also were inversely correlated with magnitudes of the 6/1-hr ratios of figure 4.7. For example, Los Angeles and San Diego (high 6/1-hr ratios) have low 15-min to 1-hr ratios (approximately 0.60) whereas the 15-min to 1-hr ratios in Arizona and Utah (low 6/1-hr ratios) were generally higher (approximately 0.75).

Depth-duration relations for durations less than 1 hour were then smoothed to provide a family of curves consistent with the relations determined for 1 to 6 hours, as shown in figure 4.3. Adjustment was necessary to some of the curves to provide smoother relations through the common point at 1 hour.

We believe we were justified in reducing the number of the curves shown in figure 4.3 for durations less than 1 hour, letting one curve apply to a range of 6/1-hr ratios. The corresponding curves have been indicated by letter designators, A-D, on figure 4.3. As an example, for any 6-hr amount between 115% and 135% of 1-hr, 1-mi^2 (2.6-km^2) PMP, the associated values for durations less than 1 hour are obtained from the curve designated as "B".

Table 4.4 lists durational variations in percent of 1-hr PMP for selected 6/1-hr rain ratios. These values were interpolated from figure 4.3.

To determine 6-hr PMP for a basin, use figure 4.3 (or table 4.4) and the geographical distribution of 6/1-hr ratios given in figure 4.7.

Table 4.4.--Durational variation of 1-mi^2 (2.6-km^2) local-storm PMP in percent of 1-hr PMP (see figure 4.3)

6/1-hr ratio	Duration (hr) 1/4	1/2	3/4	1	2	3	4	5	6
1.1	86	93	97	100	107	109	110	110	110
1.2	74	89	95	100	110	115	118	119	120
1.3	74	89	95	100	114	121	125	128	130
1.4	63	83	93	100	118	126	132	137	140
1.5	63	83	93	100	121	132	140	145	150
1.6	63	83	93	100	124 125	138 140	147 150	154 157	160 164
1.64 1.8	43	70	87	100	130	149	161	171	180
2.0	43	70	87	100	137	161	175	188	200

4.5 Depth-Area Relation

We have thus far developed local-storm PMP for an area of 1 mi^2 (2.6 km^2). To apply PMP to a basin, we need to determine how 1-mi^2 (2.6-km^2) PMP should decrease with increasing area. We have adopted depth-area relations based on rainfalls in the Southwest and from consideration of a model thunderstorm.

Red is right

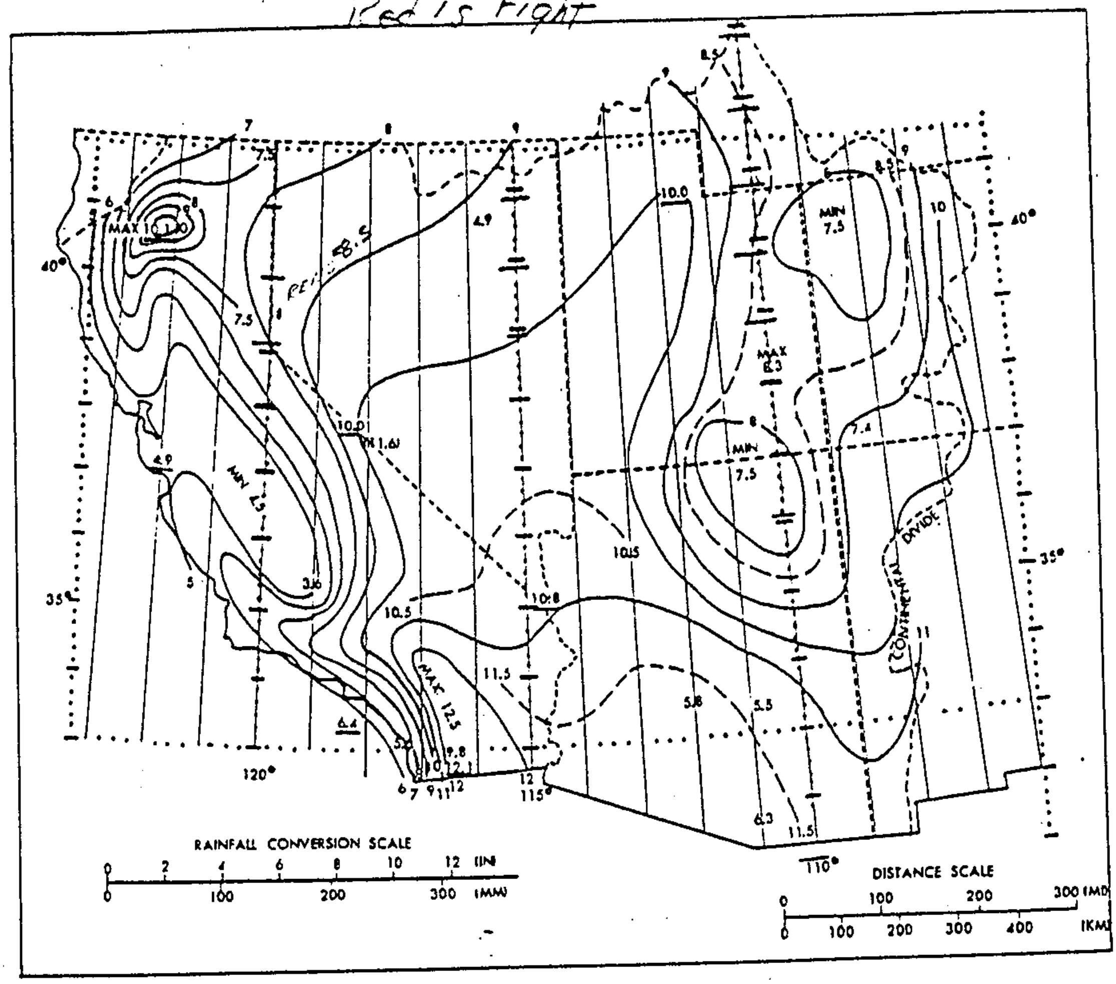


Figure 4.5--Local-storm PMP for 1 mi^2 (2.6 km^2) 1 hr. Directly applicable for locations between sea level and 5000 ft (1524 m). Elevation adjustment must be applied for locations above 5000 ft.

events. In contrast to figure 4.4, figure 4.5 maintains a maximum between these two locations. There is no known meteorological basis for a different solution. The analysis suggests that in the northern portion of the region maximum PMP occurs between the Sierra Nevada on the west and the Wasatch range on the east.

A discrete maximum (> 10 inches, 254 mm) occurs at the north end of the Sacramento Valley in northern California because the northward-flowing moist air is increasingly channeled and forced upslope. Support for this PMP center comes from the Newton, Kennett, and Red Bluff storms (fig. 4.1). Although the analysis in this region appears to be an extension of the broad maximum through the center of the Southwestern Region, it does not indicate the direction of moist inflow. The pattern has evolved primarily as a result of attempts to tie plotted maxima into a reasonable picture while considering inflow directions, terrain effects, and moisture potential.

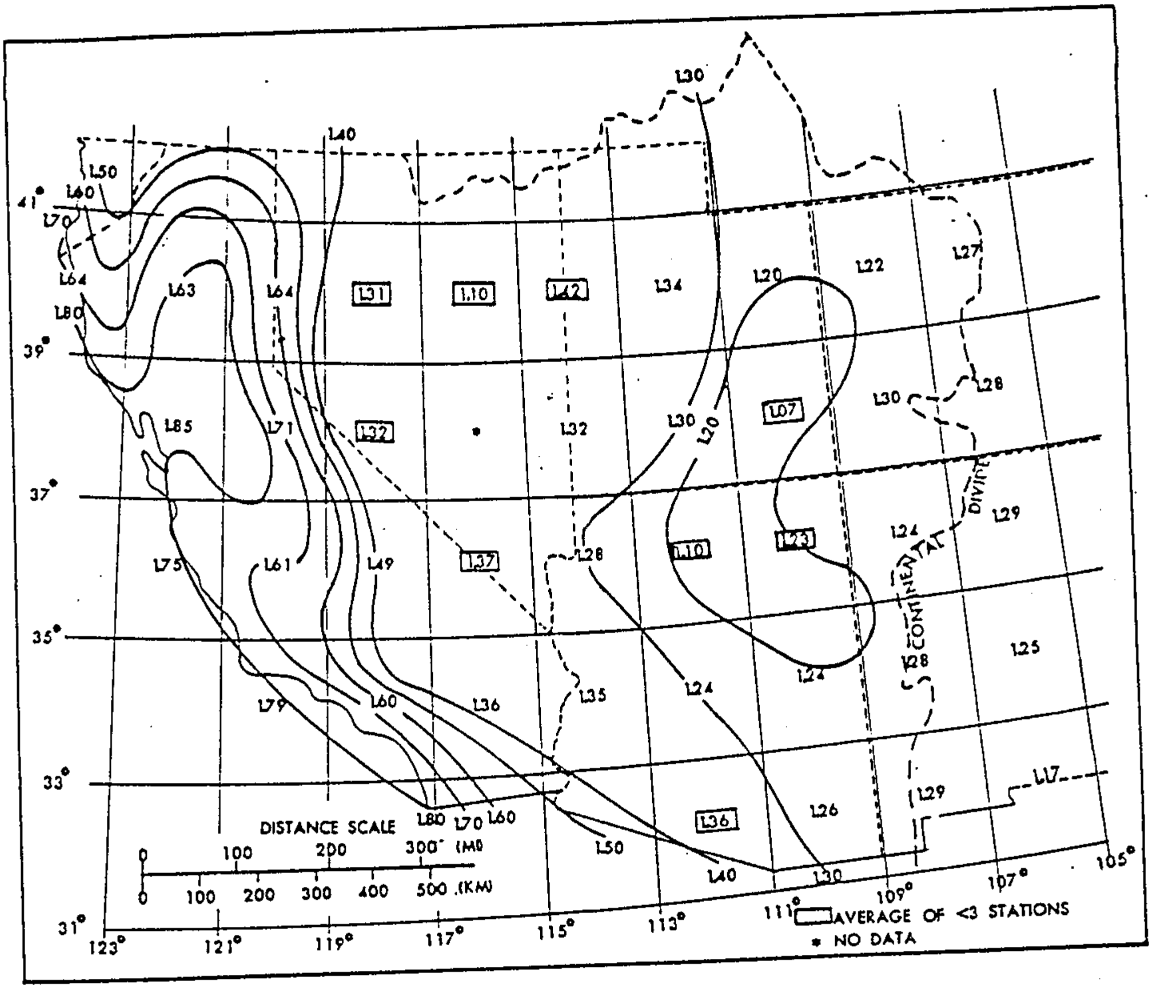


Figure 4.7.--Analysis of 6/1-hr ratios of averaged maximum station data (plotted at midpoints of a 2° latitude-longitude grid).

establish the basic depth-duration curve, then structure a variable set of depth-duration curves to cover the range of 6/1-hr ratios that are needed.

Three sets of data were considered for obtaining a base relation (see table 4.3 for depth-duration data):

a. An average of depth-duration relations from each of 17 greatest 3-hr rains from summer storms (1940-49) in Utah (U. S. Weather Bureau 1951b) and in unpublished tabulations for Nevada and Arizona (1940-63). The 3-hr amounts ranged from 1 to 3 inches (25 to 76 mm) in these events.

b. An average depth-duration relation from 14 of the most extreme short-duration storms listed in Storm Rainfall (U. S. Army, Corps of Engineers 1945-). These storms come from Eastern and Central States and have 3-hr amounts of 5 to 22 inches (127 to 559 mm).

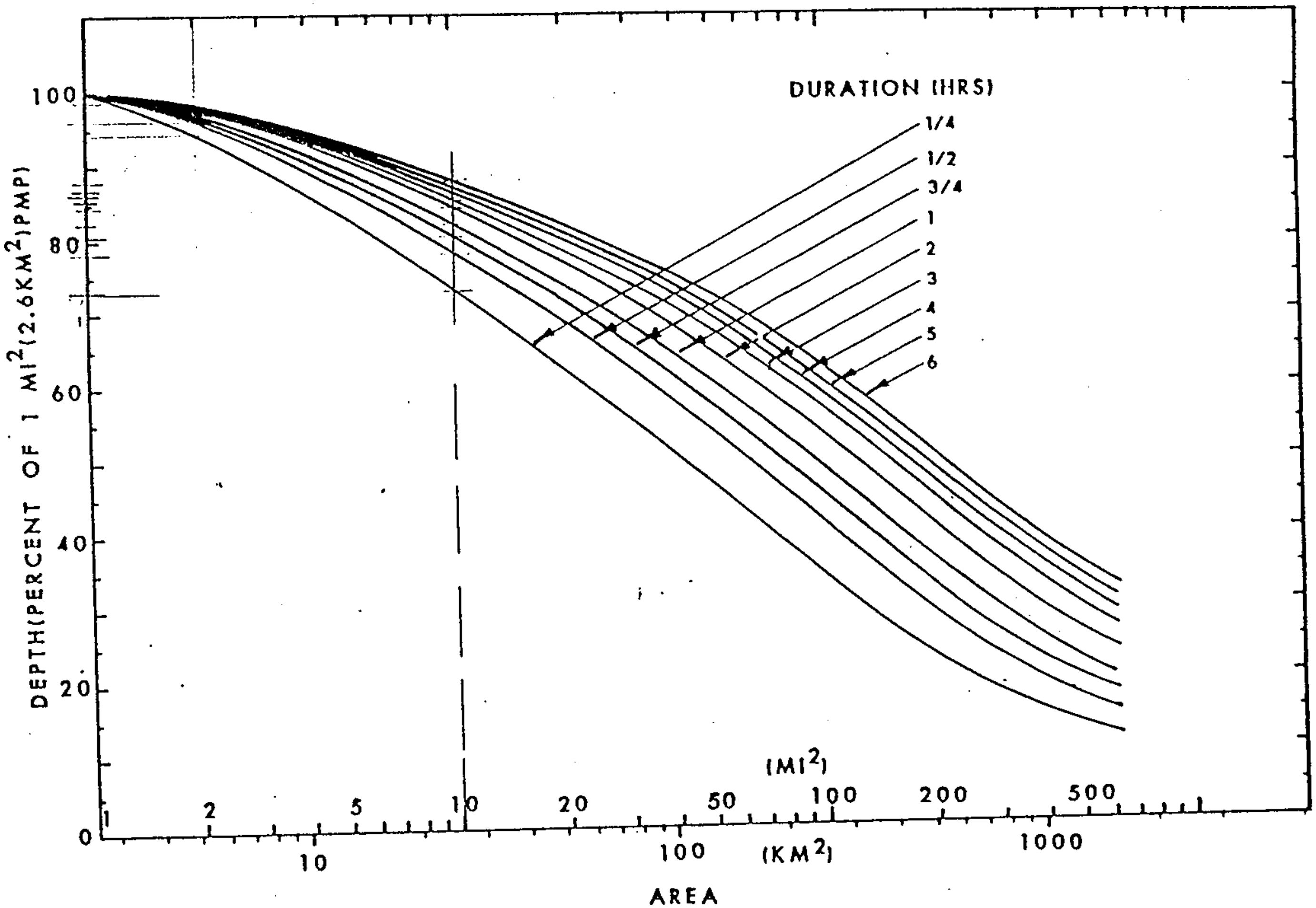


Figure 4.9.--Adopted depth-area relations for local-storm PMP.

Subarea "H" Present Condition

Time of Concentration

using nearly bare ground Fig 3-1 TR-55

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 Velocity	Time (t_c) sec.
A to B	Overland (poor Cond.)	22%	1950'	4.6 Ft/sec.	424
B to C	Overland (poor Cond.)	5%	2320'	2.25 Ft/sec.	1031
C to D	~~48" Pipe~~ Overland	~~3%~~ 5%	~~1270'~~ 1270'	① ~~17.2 Ft/sec~~ 2.25	~~74~~ 564
					1529 sec. 2019

$t_c = \frac{2019 \; \cancel{1529}}{3600} = 0.56 \; \cancel{0.42}$ hrs.

$L = 0.6\,(t_c)$

$L = 0.6\,(0.56 \; \cancel{0.42}) = \boxed{0.34 \; \cancel{0.25}}$ hrs.

$$V = \frac{1.49}{N}\left(\frac{D}{4}\right)^{2/3}(S)^{1/2}$$

$$① \; V = \frac{1.49}{0.015}\left(\frac{4}{4}\right)^{2/3}(0.03)^{1/2} = 17.2 \text{ Ft/sec}$$

Avg. Slope = 0.22(1950)
0.05(2320)
+ ~~0.03~~ (1270)

~~583.10~~ 608.5

$$= \frac{608.5 \; \cancel{583.10}}{(1950+2320+1270)} = 0.110 \; \cancel{0.105}$$

$$L = \frac{L^{0.8}(S+1)^{0.7}}{1900\,(Y)^{0.5}}$$

$$L = \frac{(1950+2320+1270)^{0.8}\left(\frac{1000}{84 \; \cancel{90}} - 10\right)+}{1900 \cdot (11.0 \; \cancel{10.5})^{0.5}}$$

$L = 0.33 \; \cancel{0.27}$ hrs.

using CN = 84

$L = 0.34$

Subarea "B" Present Condition Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 Velocity	Time sec.
A to B	Overland (poor cond.)	18 ~~4~~%	1350	4.25 Ft/sec. ✓	318
B to C	overland (poor cond.)	4%	1580	2 Ft/sec. ✓	790
C to D	~~Pipe 30"~~ Overland	4%	380	* ~~14.5 Ft/sec.~~ 2 f/s	~~26~~ 190
D to E	overland (short weeds)	3%	1890	1.2 Ft/sec ✓	1575
					~~2709~~ sec 2873

$t_c = \frac{2873}{3600}$ ~~$\frac{2709}{3600}$~~ $= 0.80$ ~~0.75~~ hrs.

Avg. Slope

0.18 (1350)
0.04 (1580)
0.04 (380)
0.03 (1890)

378.10

$= \frac{378.10}{5200}$

$= 7.27\%$

~~$L = 0.6\,t_c$~~

$L = \boxed{0.48}$ ~~0.45~~ hrs.

$$L = \frac{(1350+1580+380+1890)^{0.8}\left[\left(\frac{1000}{89.81}-10\right)+1\right]^{0.7}}{1900\,(7.272)^{0.5}}$$

$L = 0.$~~32~~ 37 hrs.

Using CN=84

$L = 0.39$ hrs

$* \; V = \frac{1.49}{0.015}\left(\frac{2.5}{4}\right)^{2/3}(0.04)^{1/2} =$ ~~14.5 Ft/sec~~

Subarea "C" Present Condition Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft)	Table 3-1 Velocity	Time sec.
A to B	Overland (poor)	20%	1150	4.5 Ft/sec	256
B to C	Overland (poor)	3.3%	300	1.8 Ft/sec.	167
					423 sec.

$t_c = \frac{423}{3600} = 0.12$ hrs.

Avg. Slope

0.20 (1150)
0.033 (300)
239.9

$= \frac{239.9}{1450} = 16.5\%$

$L = 0.6\, t_c$

$L = 0.6(0.12) = 0.07$ hrs.

$$L = \frac{(1150+300)^{0.8}\left[\left(\frac{1000}{\cancel{.88}\;81}-10\right)+1\right]^{0.7}}{1900\,(16.5\%)^{0.5}}$$

$L = \cancel{0.08}\ 0.10$ hrs.

Using CN = 81

$L = 0.10$

Subarea "D" Present Condition Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 Velocity	Time sec.
A to B	overland (poor cond.)	8%	270	2.8 Ft/sec ✓	96
B to C	Asphalt Ditched	4%	850	4.0 Ft/sec. ✓	212
C to D	grass Ditched (bare)	6.5%	650	2.5 Ft/sec ✓	260
D to E	~~18" Pipe~~ Overland	4%	960	2 f/s ~~10.3 Ft/sec.~~	~~93~~ 480
E to F	~~24" Pipe~~ Overland	1.5%	1240	1.25 f/s ~~7.7 Ft/sec~~	~~161~~ 992
F to G	~~48" Pipe~~ Overland	1.0%	420	1.0 f/s ~~9.9 Ft/sec.~~	~~42~~ 420
G to H	Ditched grass	1.0%	190	1.5 Ft/sec. ✓	127
H to I	~~48" Pipe~~ Overland	2.5%	100	~~15.7 Ft/sec.~~ 1.6 f/s	~~6~~ 62.5
			4680		~~997 sec.~~ 2649.5

$$t_c = \frac{\cancel{997}\ 2649.5}{3600} = \cancel{0.28}\ 0.74 \text{ hrs.}$$

$L = 0.6\, t_c$ + AREA C

$\boxed{L = \cancel{0.17}\ 0.44 \text{ hrs.}}$ $+0.07$ $\boxed{0.51}$

$$V_1 = \frac{1.49}{0.015}\left(\frac{1.5}{4}\right)^{2/3}(0.04)^{1/2} = 10.3 \text{ Ft/sec}$$

$$V_2 = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.015)^{1/2} = 7.7 \text{ Ft/sec}$$

$$V_3 = \frac{1.49}{0.015}\left(\frac{4}{4}\right)^{2/3}(0.01)^{1/2} = 9.9 \text{ Ft/sec}$$

$$V_4 = \frac{1.49}{0.015}\left(\frac{4}{4}\right)^{2/3}(0.025)^{1/2} = 15.7 \text{ Ft/sec}$$

$$L = \frac{(4680)^{0.8}\left[\left(\frac{1000}{90} - 10\right) + 1\right]^{0.7}}{1900\ (3.5\%)^{0.5}}$$

$= 0.41$ hrs.

Avg. Slope

0.08	270
0.04	850
0.065	650
0.04	960
0.015	1240
0.01	420
0.01	190
0.025	100

163.45 ÷ 4680 = 3.52

196.81 15.7

Subarea "E" Present Condition Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 velocity	Time sec.
A to B	Open Ditch (Bare)	1	1450	1 Ft/sec	1450
B to C	36" Pipe overland	1	280	~~8.2 Ft/sec~~ 1 Ft/s	~~34~~ 280
C to D	Open Ditch (Bare)	1	980	1 Ft/sec	980
D to E	24" PIPE overland	2	130	~~8.8 Ft/sec.~~ 1.4 Ft/s	~~15~~ 93
E to F	Open Ditch	1.5	650	1.2 Ft/sec	542
F to G	36" PIPE overland	~~5~~ 2	~~50~~ 120	~~18.3 Ft/sec~~ 1.4 Ft/s	~~7~~ 86
G to H	Open Ditch	1.5	700	1.2 Ft/sec	583
? H to I	36" PIPE overland	1.5	60	~~10.0 Ft/sec~~ ~~8.2~~ 1.25 Ft/s	~~6~~ 48
? I to J	Open Ditch	10	300	3.2 Ft/sec.	94
			4670		~~3711~~ 4156 sec

$$t_c = \frac{4156}{3600} \left(\cancel{\frac{3711}{3600}}\right) = 1.15 \ (\cancel{1.03}) \text{ hrs.}$$

$$L = 0.6\, t_c$$

$$L = 0.6 (1.15)(\cancel{1.03}) = \boxed{0.69\ (\cancel{0.62})\text{ hrs.}}$$

$$V_1 = \frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.01)^{1/2} = 8.2\ \text{Ft/sec}$$

$$V_2 = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.02)^{1/2} = 8.8\ \text{Ft/sec}$$

$$V_3 = \frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.05)^{1/2} = 18.3\ \text{Ft/sec}$$

$$V_4 = \frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.015)^{1/2} = 10.0\ \text{Ft/sec}$$

Avg. Slope

0.01 (1450)
0.01 (280)
0.01 (980)
0.02 (130)
0.015 (650)
0.05 (120)
0.015 (700)
0.015 (60)
0.10 (300)

$$\frac{86.85}{4670} = 1.867$$

$$L = \frac{(4670)^{0.8}\left[\left(\frac{1000}{9293}-10\right)+1\right]^{0.7}}{1900\,(1.86)^{0.5}}$$

$$L = \cancel{0.52}\ 0.49 \text{ hrs.}$$

pipe velocities high, doesn't account for overland flow if pipes are at capacity

Subarea "F" Present Cond. Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 Velocity	Time sec.
A to B	Curb & Gutter flow	2.5	500	3.2 Ft/sec.	156
B to C	12" PIPE steel	6.5	500	5.1 Ft/s ~~10.0~~ Ft/sec.	50.98
C to D	18" PIPE steel	5.0	1060	4.5 ~~11.5~~ Ft/sec	~~92~~ 236
D to E	24" PIPE steel	6.0	2070	4.8 ~~15.3~~ Ft/sec	~~135~~ 431
			4130		~~433~~ sec 921

$t_c = \frac{921 \ \cancel{433}}{3600} = 0.26 \ \cancel{0.12} \text{ hrs.}$

$L = 0.6\, t_c$

$L = 0.6(0.12) = 0.15 \ \cancel{0.07} \text{ hrs.}$

Avg. Slope

$$\begin{array}{rl} 0.025 & (500) \\ 0.065 & (500) \\ 0.05 & (1060) \\ 0.06 & (2070) \\ \hline & 222.2 \end{array}$$

$\frac{222.2}{4130} = 5.4\%$

$$L = \frac{(4130)^{0.8}\left[\left(\frac{1000}{89.86} - 10\right) + 1\right]^{0.7}}{1900\,(5.4)^{0.5}}$$

$L = 0.34 \ \cancel{0.31} \text{ hrs.}$

$V_1 = \frac{1.49}{0.015}\left(\frac{1}{4}\right)^{2/3}(0.065)^{1/2} = \cancel{10.0} \text{ Ft/sec}$

$V_2 = \frac{1.49}{0.015}\left(\frac{1.5}{4}\right)^{2/3}(0.05)^{1/2} = \cancel{11.5} \text{ Ft/sec}$

$V_3 = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.06)^{1/2} = \cancel{15.3} \text{ Ft/sec}$

pipe velocities high / doesn't account for pipes being at capacity and overland flow

Subarea "G" Present Cond. Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft)	Table 3-1 Velocity	Time sec.
A to B	Open land	12.5%	1420	3.5 Ft/sec	406
B to C	30" PIPE should handle flows	3%	120	12.6 Ft/sec	10
C to D	Open channel	7%	450	2.6 Ft/sec.	173
D to E	Open channel	1.5%	4480	~~0.85~~ 5 Ft/sec	~~5271~~ 896
			6470		~~5860~~ 1485

$t_c = \frac{\cancel{5860}\ 1485}{3600} = \underline{\cancel{1.63}\ 0.41\ hrs.}$

$L = 0.6\ t_c$

$L = 0.6(\cancel{1.63}\ 0.41) = \boxed{\cancel{0.98\ hrs.}}$ $\boxed{0.25\ hrs}$

Avg Slope

0.125(1420)
0.03(120)
0.07(450)
0.015(4480)

279.80

$\frac{279.80}{6470} = 4.3\%$

$V_1 = \frac{1.49}{0.015}\left(\frac{2.5}{4}\right)^{2/3}(0.03)^{1/2} = 12.6\ Ft/sec$

$L = \frac{(6470)^{0.8}\left(\left(\frac{1000}{89}-10\right)+1\right)^{0.7}}{1900\ (4.3)^{0.5}}$

pipe velocity high

$L = 0.50\ hrs.$

with Georges Rd (UPPER)
time = 406+10+173+1000/5
= 789

789/3600 = 0.22
L = 0.6(0.22) = 0.13

LOWER
696/3600 = 0.19
L = 0.6(0.19) = ?

Subarea "H" Present Condition Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 Velocity	Time sec.
A to B	over land	16%	1530'	4 Ft/sec.	383

$$L = \frac{(1530)^{0.8}\left[\left(\frac{1000}{89} - 10\right) + 1\right]^{0.7}}{1900\ (16)^{0.5}}$$

$L = 0.08$ hrs.

$t_c = \frac{383}{3600} = 0.11$ hrs.

$L = 0.6\ t_c = 0.6\,(0.11)$

$(L = 0.06$ hrs.$)$

Subarea "I" Present Condition Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 Velocity	Time sec.
A to B	Sheet flow	5%	880	4.5 Ft/sec.	196
B to C	~~12" PIPE~~ paved road	6.0	820	~~9.6~~ 4.8	~~85~~ 171
C to D	~~15" PIPE~~ paved road	7.0	610	~~12.1~~ 5.3	~~50~~ 115
D to E	~~24" PIPE~~ paved road	8.5%	1550	~~18.2~~ 5.8	~~85~~ 267
E to F	Overland	10.0	610	3.2 Ft/sec	191
F to G	~~36" PIPE~~ overland	2.0	220	~~12.2~~ 2.7	~~18~~ 81.5
G to H	Open channel	1%	40'	1 Ft/sec.	40
H to I	~~24" PIPE~~ overland	2.0	30'	~~8.8~~ 2.7	~~3~~ 11
I to J	Overland Channel	5%	400'	2.3 Ft/sec	174
J to K	~~30" PIPE~~ overland	~~4~~ 4.0	50'	~~8.9~~ 4.0	~~6~~ 12.5
K to L ~~L to M~~	Overland channel	1.5%	1400'	1.2 Ft/sec.	1167
			6610		2015 sec 2426

$t_c = \frac{2426 \; \cancel{2015}}{3600} = \frac{0.67}{\cancel{0.56}}$ hrs.

$L = 0.6 \; (\overset{0.67}{\cancel{0.56}}) \; \boxed{\frac{0.40}{\cancel{0.34}}}$ hrs.

pipe velocities
Are high.
Does not account
for overland flow
when pipes are
at capacity

$V_1 = \frac{1.49}{0.015}\left(\frac{1}{4}\right)^{2/3}(0.06)^{1/2} = 9.6$

$V_2 = \frac{1.49}{0.015}\left(\frac{1.25}{4}\right)^{2/3}(0.07)^{1/2} = 12.1$

$V_3 = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.085)^{1/2} = 18.2$

$V_4 = \frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.02)^{1/2} = 12.2$

$V_5 = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.02)^{1/2} = 8.8$

$V_6 = \frac{1.49}{0.015}\left(\frac{2.5}{4}\right)^{2/3}(0.015)^{1/2} = 8.9$

Avg. Slope

0.05 (880)
0.06 (820)
0.07 (610)
0.085 (1550)
0.10 (610)
0.02 (220)
0.01 (40)
0.02 (30)
0.05 (400)
0.04 (50)
0.015 (1400)

377.05

$\frac{377.05}{6610} = 5.7\%$

$$L = \frac{(6610)^{0.8}\left[\left(\frac{1000}{88.86}-10\right)+1\right]^{0.7}}{1900 \; (5.7)^{0.5}}$$

$L = \overset{0.49}{\cancel{0.46}}$ hrs.

Subarea "J" Present Condition   Time of Concentration

Reach	Descrip. of flow		Slope (%)	Length (Ft)	Table 3-1 velocity	Time sec.
A to B	Asphalt		7.0%	300	5.3 Ft/sec ✓	57
B to C	12" Pipe	paved	6.3%	2470	~~9.9 Ft/sec~~ 5 ft/s	~~250~~ 494
C to D	18" Pipe	paved	9.8%	1180	~~16.2 Ft/sec~~ 6.2 ft/s	~~73~~ 189
D to E	21" Pipe	paved	10.5	170	~~18.5 Ft/sec~~ 6.4 ft/s	~~9~~ 27
E to F	24" Pipe	paved	9.2%	400	~~19.0 Ft/sec~~ 6 ft/s	~~21~~ 66
F to G	~~30" Pipe~~	paved	8.0%	180	~~20.5 Ft/sec~~ 5.6 ft/s	~~9~~ 32
G to H	Open Ditch		1.5%	320'	1.2 Ft/sec	267
H to I	~~36" Pipe~~	overland	~~1.2~~ 1%	~~900~~ 50'	~~8.2 Ft/sec~~ 3.2 ft/s	~~110~~ 16
				5920 / 5070		796 sec / 1148

$t_c = \frac{\cancel{796}\ 1148}{3600} = \cancel{0.22}\ 0.32$ hrs.

$L = 0.6\,(\cancel{0.22}\ 0.32)$
$\boxed{= \cancel{0.13}\ 0.19 \text{ hrs}}$

$V_1 = \frac{1.49}{0.015}\left(\frac{1}{4}\right)^{2/3}(0.063)^{1/2} = 9.9$ Ft/sec.

$V_2 = \frac{1.49}{0.015}\left(\frac{1.5}{4}\right)^{2/3}(0.098)^{1/2} = 16.2$

$V_3 = \frac{1.49}{0.015}\left(\frac{1.75}{4}\right)^{2/3}(0.105)^{1/2} = 18.5$

$V_4 = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.092)^{1/2} = 19.0$

$V_5 = \frac{1.49}{0.015}\left(\frac{2.5}{4}\right)^{2/3}(0.08)^{1/2} = 20.5$

$V_6 = \frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.01)^{1/2} = 8.2$

Avg. Slope

0.07 (300)
0.063 (2470)
0.098 (1180)
0.105 (170)
0.092 (400)
0.08 (180)
0.015 (320)
0.110 ~~0.01 (900)~~ 50
375.10

$\frac{375.10}{5920} = \cancel{6.34}\ 7.32\%$

$L = \frac{(\cancel{5920}\ 5070)^{0.8}\left[\left(\frac{1000}{8.87} - 10\right) + 1\right]^{0.7}}{1900\ (\cancel{6.34}\ 7.32)^{0.5}}$

$L = \cancel{0.40}\ 0.31$ hrs.

pipe velocity high
Are extremely
Does not account
for overland flow
if pipes are
beyond capacity

Subarea "O" Present Condition Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft.)	Table 3-1 Velocity	Time sec.
A to B	15" Pipe	3.3%	1360'	8.3 Ft/sec	164 ✓
B to C	Asphalt (sheet)	2%	150	2.8 Ft/sec ✓	54 ✓
C to D	Overland	6%	200	2.4 Ft/sec ✓	83 ✓
			1710		301 sec.

$t_c = \frac{301}{3600} = 0.08\ hrs.$ ✓

$\boxed{L = 0.6(0.08) = 0.05\ hrs.}$ ✓

Avg. Slope

0.033 (1360)
0.02 (150)
0.06 (200)

59.98 ÷ 1710

= 0.035

$V_1 = \frac{1.49}{0.015}\left(\frac{1.25}{4}\right)^{2/3}(0.033)^{1/2} = 8.3$

$L = \frac{(1710)^{0.8}\left[\left(\frac{1000}{59}-10\right)+1\right]^{0.7}}{1900\,(3.5)^{0.5}}$

$L = 0.19\ hrs.$

Subarea 'P'. Present Condition

Time of Concentration

Reach	Descrip. of flow	Slope (%)	Length (Ft)	Table 3-1 velocity	Time sec.
A to B	Overland flow	1	800'	1 Ft/sec.	800
B to C	~~18"~~ Pipe overland	3.5%	550'	~~9.7 Ft/sec~~ 2.5 ft/s	~~57~~ 220
	Overland Ditch	5%	50'	2.3 Ft/sec.	22
	~~18" Pipe~~ overland portion	5%	200'	~~11.6 Ft/sec~~ 3.2 ft/s	~~17~~ 63
	Over Asphalt	7.5	600'	5.5 Ft/sec.	109
	~~72" Pipe~~ overland portion	3%	60'	~~22.5 Ft/sec~~ 2.4 ft/s	~~3~~ 25
	Overland Ditch	5%	160'	1.6 Ft/sec.	100
	~~30" Pipe~~ overland	2.5	200'	~~11.5 Ft/sec~~ 2.25 ft/s	~~17~~ 89
	Overland Ditch	1.5%	1300'	0.85 Ft/sec.	1529
	~~36" Pipe~~	~~1.5%~~	~~1400'~~	~~10.0/10.0 Ft/sec~~	~~140~~
			~~5320'~~ 3920		~~2794~~ sec. 2957

$t_c = \frac{2957\ \cancel{2794}}{3600} = 0.82\ \cancel{0.78}\ hrs.$

$L = 0.6\,(0.82\ \cancel{0.78}) = \boxed{0.49\ \cancel{0.47}}\ hrs.$

pipe velocities are high doesn't account for overland flow in pipes are at capacity

$V_1 = \frac{1.49}{0.015}\left(\frac{1.5}{4}\right)^{2/3}(0.035)^{1/2} = 9.7$

$V_2 = \frac{1.49}{0.015}\left(\frac{1.5}{4}\right)^{2/3}(0.05)^{1/2} = 11.6$

$V_3 = \frac{1.49}{0.015}\left(\frac{6}{4}\right)^{2/3}(0.03)^{1/2} = 22.5$

$V_4 = \frac{1.49}{0.015}\left(\frac{2.5}{4}\right)^{2/3}(0.025)^{1/2} = 11.5$

$V_5 = \frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.015)^{1/2} = 10.0$

Avg. Slope

0.01 (800)
0.035 (550)
0.05 (50)
0.05 (200)
0.075 (600)
0.03 (60)
0.05 (160)
0.025 (200)
0.015 (1300)
0.015 (1400) = 140.05 ÷ 5320 = 2.6%

$L = \frac{(3920\ \cancel{5320})^{0.8}\left[\left(\frac{1000}{80} - 10\right) + 1\right]^{0.7}}{1900\ (2.6)^{0.5}}$

$= 0.45\ \cancel{0.57}\ hrs.$

TR55

Subarea "U" Present Condition

Time of Concentration

Reach	Discrip. of flow	Slope (%)	Length (Ft)	Table 3-1 Velocity	Time sec.
A to B	Asphalt flow	6.0	700	5.0 Ft/sec.	140
B to C	Open Ditch	1.0% ~~2.2%~~	980	1.0 Ft/sec	880
C to D	Asphalt flow	15%	100	8.7 Ft/sec	12
D to E	18" Pipe	0.5%	360	3.6 Ft/sec	100
E to F	24" Pipe	0.5	650	4.4 Ft/sec	148
F to G	30" Pipe	0.5	300	5.1 Ft/sec	59
G to H	33" Pipe	0.5	400	5.5 Ft/sec.	73
H to I	36" Pipe	0.5	270	5.8 Ft/sec	47
~~I to J~~	~~42" Pipe~~	~~0.5~~	2280'	7.0 Ft/sec	326
J to K	~~54" Pipe~~	~~0.5~~	630'	7.6 Ft/sec	83
K to L	~~60" Pipe~~	0.5%	220'	8.2 Ft/sec	27
			~~6790'~~		~~1895~~ 1459

$t_c = \frac{1459\ \cancel{1895}}{3600} = 0.405\ \cancel{0.53}\ hrs.$

$L = 0.6(0.405\ \cancel{0.53})$
$= 0.24\ \cancel{0.32}\ hrs.$

Avg. Slope

0.06(700)
0.01(980)
0.15(100)
0.005(5110)

91.35 ÷ 6790
= 1.35%

$$L = \frac{(6790)^{0.8}\left[\left(\frac{1000}{87}-10\right)+1\right]^{0.7}}{1900\ (1.35)^{0.5}}$$

~~L = 0~~

L = 1.00 hrs.

DJ | PARADISE POND

Routing subarea "A" through Subarea "E"

1.) 24"x30" square culvert

$L = 50'$
$S = 2\%$

$V = \frac{1.49}{0.015}\left(\frac{2(2.5)}{2+2+2.5+2.5}\right)^{2/3}(0.02)^{1/2} = 9.5\ Ft/sec.$ $V = 1.4'/s$ overland

$t = 50/\cancel{9.5}\ 1.4 = \cancel{5}\ 36\ sec$

~~2.)~~

2.) OPEN DITCH

$L = 20'$
$S = \cancel{6\%}\ 40\%$

$V = \cancel{1.49}\ 6.5\ Ft/sec.$

$t = 20/6.5 = 3\ sec.$

3.) 36" pipe

$L = 50'$
$S = 2\%$

$V = \frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.02)^{1/2} = 11.6\ Ft/sec.$ $V = 1.4'/s$ overland

$t = 50/\cancel{11.6}\ 1.4 = \cancel{4}\ 36\ sec.$

4.) OPEN DITCH

$L = 580'$
$S = 1\%$

$V = 1\ Ft/sec$

$t = 580/1 = 580\ sec.$

Routing "A" through "E" Con't

5.) see time of Concentration for Subarea "E"

~~D to E~~ B to C	~~34~~ 280
C to D	980
D to E	~~18~~ 93
E to F	542
F to G	~~7~~ 86
G to H	583
? H to I	~~6~~ 48
I to J	94
	~~2261~~ sec total 2706

total time (K)

$$\cancel{5}^{36}+3+\cancel{4}^{36}+580+\cancel{2261}^{2706} = \cancel{2853}^{3361} \text{ sec.}$$

$$\frac{\cancel{2853}^{3361}}{3600} = \cancel{0.79}^{0.93} \text{ hrs.} = K$$

$x = 0.30$ try

Routing subarea "B" through subarea "D"

1.) 18" pipe

$L = 930'$
$S = 2.5\%$

$\cancel{V \approx \frac{1.49}{0.015}\left(\frac{1.5}{4}\right)^{2/3}(0.025)^{1/2} = 8.2 \text{ Ft/sec}}$ $V = 1.6'/s$ overland

$t = 930/\cancel{8.2}\ 1.6 = \cancel{113}\ 581$ sec.

2) Open channel

$L = 40'$
$S = 2.5\%$

$V = 0.75$ Ft/sec

$t = 40/0.75 = 53$ sec.

3.) 24" pipe

$L = 390'$
$S = 3.5\%$

$\cancel{V = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.035)^{1/2} = 11.7 \text{ Ft/sec}}$ $V = 1.85$ overland

$t = 390/\cancel{11.7}\ 1.85 = \cancel{33}\ 211$ sec.

4.) Open Channel

$L = 10'$
$S = 10\%$

$V = 1.5$ Ft/sec

$t = 10/1.5 = 7$ sec. ✓

Routing "B" through "D"

5.) 48" PIPE

$L = 420'$
$S = 1.0\%$
$\cancel{V = \frac{1.49}{0.015}\left(\frac{4}{4}\right)^{2/3}(0.01)^{1/2} = 9.9 \text{ ft/sec}}$ $V = 1.4$ '/s
overland; pavement
$t = 420/\cancel{9.9}\ 1.4 = \cancel{42}\ 300$ sec.

6.) Open channel

$L = 190'$
$S = 1.0\%$

$V = 1.5$ ft/sec.
$t = 190/1.5 = 127$ sec. ✓

7.) 48" Pipe

$L = 100'$
$S = 2.5\%$
$\cancel{V = \frac{1.49}{0.015}\left(\frac{4}{4}\right)^{2/3}(0.025)^{1/2} = 15.7 \text{ ft/sec}}$ $V = 1.6$ '/s
overland
$t = 100/\cancel{15.7}\ 1.6 = \cancel{6 \text{ sec.}}\ 62.5$ sec.

time total (K)
$\cancel{113}\ 581 + 53 + \cancel{35}\ 211 + 7 + \cancel{42}\ 300 + 127 + \cancel{6}\ 62.5 = \cancel{381}\ 1341.5$ sec.
$\frac{\cancel{381}\ 1341.5}{3600} = \cancel{0.11}\ 0.37$ hrs. $= K$

$x = 0.30$ try

Routing Subareas "B" & "D" through Subarea "E"

1.) 48" PIPE

$L = 850'$
$S = 2.5\%$

$V = \frac{1.49}{0.015}\left(\frac{4}{4}\right)^{2/3}(0.025)^{1/2} = 15.7$ ft/sec. $V = 1.6$ f/s
overland

$t = 850/\cancel{15.7}$ 1.6 $= \cancel{54}$ sec. 531 sec

total time (K)

$K = \frac{531 \; \cancel{54}}{3600} = 0.15 \; \cancel{0.015}$ hrs.

$x = 0.30$ try

Routing Subarea "F" through Subarea "G"

1.) 24" PIPE

$L = 320'$
$S = 13\%$
$~~V = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{0.666}(0.13)^{0.5} = 22.6~ft/sec.~~$ $V_{street} = 4.8'/s$

$t = \cancel{14}~sec$ 67

2.) Open channel

$L = 4410'$
$S = 1.5\%$

$~~V = 0.85~ft/sec~~ = 5'/s$

$t = \cancel{5188}~sec.$ 882 sec

total time (K)

$K = \frac{\cancel{5188+14}~882+67}{3600} = \cancel{1.45}~0.26~hrs.$

$x = 0.30$ try

Routing Subareas "B, "D", "E" "A" through Subarea "G"

1.) Open channel

$L = 1120'$
$s = 2\%$

~~V = 1 ft/sec.~~ V = 6 '/s

~~t = 1120 sec.~~ 186 sec.

2.) Open Channel

$L = 4480'$
$S = 1.5\%$

~~V = 0.85 ft/sec.~~ V = 5 '/s

t = ~~5271~~ sec 896 s

Total time (K)

$$K = \frac{186 + 896 \quad \cancel{5271 + 1120}}{3600} = 0.30 \text{ hrs.} \quad \cancel{1.78} \text{ hrs.}$$

x = 0.30 try

Routing above subareas through Subarea "I"

1.) 36" Pipe

$L = 910'$
$S = 1.7\%$
$V = \cancel{\frac{1.49}{0.015}\left(\frac{3}{4}\right)^{2/3}(0.017)^{1/2} = 10.7\ Ft/sec.}$ $V = 5'/s$ overland

$t = \frac{910}{\cancel{10.7}\ 5} = \cancel{85}\ 182$ sec.

2.) Open channel

$L = 30'$
$S = 2\%$

$V = \cancel{1.4\ Ft/sec.}\ 5'/s$
$t = \frac{30}{\cancel{1.4}\ 5} = \cancel{21}\ 6$ sec.

3.) 48" PIPE

$L = 100'$
$S = 2\%$
$V = \frac{1.49}{0.015}\left(\frac{4}{4}\right)^{2/3}(0.02)^{1/2} = \cancel{14.0\ Ft/sec.}$ $V = 5'/s$ overland
$t = \frac{100}{\cancel{14}\ 5} = \cancel{7}\ 20$ sec.

4.) Oppen Channel

$L = 310'$
$S = 3\%$

$\cancel{V = 0.8\ Ft/sec}$ $V = 5'/s$

$t = \frac{310}{\cancel{0.8}\ 5} = \cancel{388}\ 62$ sec.

Routeing subareas through Subarea "I" Con't

5.) 78" Pipe

$L = 300'$
$S = 6.5\%$

$\cancel{V = \frac{1.49}{0.015}\left(\frac{6.5}{4}\right)^{2/3}(0.065)^{1/2} = 35.0 \text{ Ft/sec}}$ $V = 22'/s$
Q≈2400
D/d 53.0?

$t = 300/\cancel{35}\ 22 = \cancel{9}\ 14 \text{ sec.}$

6.) Open Channel

$L = 960'$
$S = 3\%$

$V = 1.2 \text{ Ft/sec.}$

$t = 960/\cancel{1.2}\ 8 = \cancel{800 \text{ sec.}}\ 120$

7.) 96" Pipe

$L = 100'$
$S = 10\%$

$\cancel{V = \frac{1.49}{0.015}\left(\frac{8}{4}\right)^{2/3}(0.10)^{1/2} = 49.9 \text{ Ft/sec.}}$ $V = 26'/s$
Q≈900
D/d: 34.5

$t = 100/\cancel{49.9}\ 26 = \cancel{2}\ 4 \text{ sec.}$

8.) Open channel

$L = \cancel{2100}\ 1900$
$S = 2\%$

$V = \cancel{1.4}\ 7 \text{ Ft/sec}$

$t = \cancel{2100}\ 1900/\cancel{1.4}\ 7 = \cancel{1500 \text{ sec.}}\ 271$

total time (K)

$$K = \frac{\cancel{95}\,182 + \cancel{21}\,6 + \cancel{7}\,20 + \cancel{308}\,62 + \cancel{9}\,14 + \cancel{800}\,120 + \cancel{2}\,4 + \cancel{1500}\,271}{3600} = \cancel{0.78}\ 0.19 \text{ hrs.}$$

$x = 0.30$ try

TDU

Routing Subarea "J" through Subarea "~~H~~" P

1.) Rip - Rap Ditch

$L = 450'$
$s = 14\%$

$V = 5.7$ ft/sec

$t = 450/5.7 = 79$ sec.

Overland
$L = 500'$
$S = 5\%$
$V = 2.2'/s$
$t = 500/2.2 = 227$ sec

2.) Open Ditch

$L = 610'$
$s = 2\%$

$V = 1.4$ ft/sec.

$t = 610/1.4 = 436$ sec.

paved
$L = 1200'$
$S = 6\%$
$V = 4.8'/s$
$t = 1200/4.8 = 250$ sec

Overland
$L = 1400'$
$S = 3.5'/s$
$V = 1.8'/s$
$t = 1400/1.8 = 778$ sec

total time (K)

$$K = \frac{\begin{array}{c}227+250+778\\ \cancel{436+79}\end{array}}{3600} = \begin{array}{c}0.135\\ \cancel{0.14}\ hrs.\end{array}$$

$x = 0.30$ try

Routing "I" through "P"

See subarea "P"

~~for time of 1789~~ sec.

future

Add ~~24"~~ 2~48" φ CMP

$L = 850'$
$S = 1.5\%$

$V = \frac{1.49}{0.015}\left(\frac{2}{4}\right)^{2/3}(0.015)^{1/2} = 7.7$ ft/sec. $V = 7.6$ ft/s

$t = 850/\cancel{7.7}$ 7.6 $= \cancel{111}$ sec. 112 sec

existing
$L = 850 @ 2'/s$
$L = 1600 @ 6'/s$
$t = 425 + 267 = 692$
total time 0.19

350 2'
1600

overland
$L = 1600'$
$S = 2.5\%$
$V = 6$
$t = 1600/6 = 267$ sec

total time (K)

$\frac{112 + 267 \quad \cancel{1789 + 111}}{3600} = 0.11 \; \cancel{0.53}$ hrs. $= K$

$x = 0.30$ try

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JOB NO. #0150-08-1

SHEET ______ OF ______

PROJECT City of Reno Drainage Study DATE 10/26/89

SUBJECT ______ DESIGNED PBE CHECKED ______

Routing from Reno Rendering to Sutro

- 42"Ø RCP @ 2.4% $L = 700'$ $V_{full} = 16.1'/s$

Travel time = 43.5 sec = .012 hrs

- 48"Ø RCP @ 1.2% $L = 850'$ $V_{full} = 12.5'/s$

Travel time = 68 sec = .019 hrs.

Total travel time = 0.03 hrs.

Routing from Sutro to Helena

- 42"Ø RCP @ 1.2% $L = 990'$ $V_{full} = 11.4'/s$

Travel time = 86.8 sec. = 0.024 hrs.

- 48"Ø RCP @ 0.6% $L = 990'$ $V_{full} = 8.8'/s$

Travel time = 112.5 sec = 0.031 hrs.

- 48"Ø RCP @ 1.0% $L = 180'$ $V_{full} = 11.4'/s$

Travel time = 15.8 sec. = 0.004 hrs

Total travel time = 0.06 hrs.

FORM 230

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JOB NO. 150-03-1

SHEET 1 OF 1

PROJECT City of Reno DATE

SUBJECT Muskingum - # of subreaches DESIGNED PBE CHECKED

Routing	K	# of subreaches $\Delta t=5$	$\Delta t=10$
111 to 114	0.93	5	3
112 to 113	0.37	2	1
113 to 114	0.15	1	1
114 to 116	0.30	2	1
115 to 116	0.26	2	1
116 to 117	0.19	1	1
117 to 119	0.11	1	1
118 to 119	0.35	2	1
		5 min.	10 min.

Routing	K	# of subreaches	
114 to 120	0.10	1	1
115 to 120	0.08	1	1
120 to 116	0.19	1	1
118 to 117	0.35	2	1

$$\frac{1}{2(1-X)} \le \frac{AMSKK}{NSTPS * \Delta t} \le \frac{1}{2X}$$

AMSKK = travel time through the reach in hrs
NSTPS = # of routing steps
Δt = time interval in hours

1 reach

$$\frac{1}{2(1-.2)} \le \frac{AMSKK}{1 * .0833} \le \frac{1}{2(.2)}$$
$.1667$

0.052 hrs ≤ AMSKK ≤ 0.208 hrs. ← 5 min
0.1042 0.4168 ← 10 min

2 reaches

0.104 hrs ≤ AMSKK ≤ 0.417 hrs.
0.208 0.834

3 reaches

0.156 hrs. ≤ AMSKK ≤ 0.625 hrs.
0.313 1.250

4 reaches

0.208 hrs. ≤ AMSKK ≤ 0.833 hrs.
1.67

5 reaches

0.260 hrs ≤ AMSKK ≤ 1.042
2.08

FORM 230

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JOB NO. ______

SHEET ______ OF ______

PROJECT: City of Reno Drainage Study DATE: 10/26/89

SUBJECT: lag times DESIGNED: PBE CHECKED: ______

Lag thru remainder of Area P

Assume flow in street

750' @ ≈1% = 2'/s = 375 sec

1400' @ ≈5.7% = 4.8'/s = 292 sec.

500' @ ≈0.6% = 1.5'/s = 333 sec

400' @ ≈1% = 2'/s = 200 sec

900' @ ≈15% overland = 3.25'/s = 277 sec.

1477 sec. = 0.41 hrs

FORM 230

Subarea "U" Present Condition

TR 55

Land Use	D %	D CN	D Product	B %	B CN	B Product	%	CN	Product
Open Space	~~5.5~~ 9.5	82	779 ~~451~~	~~1.5~~ 3.5	63 ~~76~~	220.5 ~~94.5~~			
~~Residental 1/8 acre~~	~~2.5~~	~~92~~	~~230~~	~~9.5~~	~~85~~	~~807.5~~			
Roadway ~~w/ C&G~~	4 ~~0.5~~	98	392 49						
~~Roadway w/ open Ditch~~	~~2~~	~~94~~	~~188~~	~~1~~	~~89~~	~~89~~			
Grass area	~~1~~	~~80~~	~~80~~						
Residental 1/4 acre	~~34~~ 72	87	6264 ~~2958~~	~~4~~ 11	75	825 ~~300~~			
Interchange				~~6~~	69	~~414~~			
~~Commercial~~	~~8.5~~	95	~~807.5~~	24	92	~~2208~~			
			~~4763.5~~ 7435			~~3913~~ 1045.5			

Weighted CN = $\frac{\cancel{4763.5 + 3913}}{100} = \frac{8481}{\cancel{8676.5}}$

= 87 85

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JOB NO. 0150-08-1

SHEET 1 OF 1

PROJECT City of Reno Drainage Study DATE 8/22/89

SUBJECT Georges Den Detention Site DESIGNED PBE CHECKED

ELEV.	STORAGE		
740.	45 CY	=	0.03 ac-ft
745.	3140 CY	=	1.95 ac-ft
750.	11,265 CY	=	6.98 ac-ft
755.	27,126 CY	=	16.81 ac-ft
760.	53,750 CY	=	33.32 ac-ft
765.	93,673 CY	=	58.06 ac-ft
770.	147,006 CY	=	91.12 ac-ft

low level outlet ≈ 738.8

Routing

114 to 120 ≈ 2000'
L = 1120' S = 2% V = 6'/s t = 187 s
L = 880' S = 1.5% V = 5'/s t = 176 s

total time = 187+176/3600 = $\boxed{0.10\text{ hrs}}$

115 to 120 ≈ 1000'
L = 320' S = 13% V = 4.8'/s t = 67 s
L = 680' S = 1.5% V = 5'/s t = 136 s

total time = 203+67/3600 = $\boxed{0.08\text{ hrs}}$

120 to 116

114 to 116 Total time = 0.30 hrs
− 0.10 hrs (114 to 120)
0.20 hrs

115 to 116 Total time = 0.26 hrs
− 0.08 hrs (115 to 120)
0.18 hrs

$\boxed{0.19\text{ hrs}}$

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JOB NO. ______

SHEET ______ OF ______

PROJECT ______ DATE 9/6/89

SUBJECT Upper Sorrows Detention Site DESIGNED PBE CHECKED ______

minimum elevation 4694.1

Elev.	Storage (CY)	Storage (AC-ft)
4694.1	0	0
4700.	736	0.46
4705.	5634	3.49
4710.	16,891	10.47
4715.	34,865	21.61
4719.	55,200	34.21
4725.	93,837	58.16
4730.	142,244	88.17

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JOB NO. 0150-0[illegible]

SHEET 1 OF

PROJECT City of Reno Drainage Study DATE 8/18/89

SUBJECT L. Sorrows Detention Site DESIGNED CHECKED

minimum elevation 4681.2

Elev.	Storage		
4685	148. CY	=	0.09 ac-ft
4690	1570. CY	=	0.97 ac-ft
4695	9366. CY	=	5.81 ac-ft
4700	21,941. CY	=	13.60 ac-ft
4705	42,661. CY	=	26.44 ac-ft
4710	75,640. CY	=	46.88 ac-ft
4715	119,314 CY	=	73.96 ac-ft
4719	163,919 CY	=	101.60 ac-ft
4721	≈ 188,561 CY	=	116.88 ac-ft

SORROWS (24"Ø Outlet)
100 YR. PEAK STAGE - 718.17
SORROWS (30"Ø Outlet)
100 YR. PEAK STAGE - 715.60

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JOB NO. 0150-08-1

SHEET 1 OF 1

PROJECT City of Reno Drainage Study DATE 8/22/89

SUBJECT Reno Rendering Detention Site DESIGNED PBE CHECKED

4509.44 - Top W. end 36" RCP

ELEV.	STORAGE		
4510	667 CY	=	0.41 ac-ft
4512.5	4344 CY	=	2.69 ac-ft
4515	11,880 CY	=	7.36 ac-ft
4520	43,443 CY	=	26.93 ac-ft
4525	94,832 CY	=	58.78 ac-ft
4530	161,772 CY	=	100.27 ac-ft

Low level outlet (existing)

36"Ø IE = 4506.11

Area = 7.07 sq.ft, C = 0.68, e = 0.5

Top of dam ≈ 4530, L ≈ 510', C = 3.1, e = 1.5

Spillway elev. = 4528, L = 50, C = 3.1, e = 1.5

FORM 230

APPENDIX 5

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JOB NO. 150-08-1

SHEET 1 OF

PROJECT Evans Ave. D.S. DATE 11/3/89

SUBJECT Rough cost estimate DESIGNED [initials] CHECKED

1. Lower Cemetery Detention outfall

a. 36" ϕ RCP	– 420 LF	@	60	=	25,200
b. 42" ϕ RCP	– 420 LF	@	80	=	33,600
c. MH	– 5 EA	@	2500	=	12,500
	Alt. #5,6,7 & 8		Total	=	71,300

2. Manogue H.S. pipe

a. 54" ϕ RCP	– 690 LF	@	140	=	96,600
b. MH	– 2 EA	@	2500	=	5,000
c. Spec. MH	– 1 EA	@	10,000	=	10,000
	Alt. #6		Total	=	111,600
a. 66" ϕ RCP	– 690 LF	@	180	=	124,200
b. MH	– 2 EA	@	2500	=	5,000
c. Spec. MH	– 1 EA	@	10,000	=	10,000
	Alt. #5,7 & 8		Total	=	139,200

3. Evans Ave modifications

a. Inlet structure – LS @ $10,000 = $10,000

Alt. #6 Total =

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JOB NO. ______

SHEET 2 OF ______

PROJECT ______ DATE ______

SUBJECT ______ DESIGNED ______ CHECKED ______

4. Pipe from Reno Render. to Sutro (160 cfs capacity)

a. 42" ϕ RCP	- 700 LF @	80	=	56,000
b. 48" ϕ RCP	- 850 LF @	120	=	102,000
c. Hdwl.	- 1 EA @	5000	=	5,000
d. MH	- 6 EA @	2500	=	15,000
e. Spec. D.I.	- 1 EA @	5000	=	5,000
Alt. #5,6,7&8		Total	=	183,000

5. Pipe from Reno Render. to Paradise Pond (220 cfs cap.)

a. 54" ϕ RCP	- 2540 LF @	150	=	381,000
b. 60" ϕ RCP	- 3360 LF @	170	=	571,200
c. 66" ϕ RCP	- 960 LF @	190	=	182,400
d. Hdwl.	- 2 EA @	5000	=	10,000
e. MH	- 17 EA @	2500	=	42,500
Alt. #5		Total	=	1,187,100

FORM 230

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SHEET 3 OF

PROJECT DATE

SUBJECT DESIGNED CHECKED

6. Pipe from Reno Render. to Paradise Pond (160 cfs cap.)

a.	42" φ RCP	- 700 LF @		80	=	56,000
b.	48" φ RCP	- 1840 LF @		120	=	220,800
c.	54" φ RCP	- 3360 LF @		150	=	504,000
d.	60" φ RCP	- 960 LF @		170	=	163,200
e.	Hdwl.	- 2 EA @		5000	=	10,000
f.	MH	- 17 EA @		2500	=	42,500
			Alt. #8	Total	=	996,500

7. Pipe from Reno Render. to Paradise Pond (110 cfs cap.)

a.	42" φ RCP	- 2540 LF @		90	=	228,600
b.	48" φ RCP	- 3360 LF @		120	=	403,200
c.	54" φ RCP	- 960 LF @		150	=	144,000
d.	Hdwl.	- 2 EA @		5000	=	10,000
e.	MH	- 17 EA @		2500	=	42,500
			Alt. #6		=	828,300

8. Route Area 'J' to above Evans Ave.

a.	60" φ RCP	- 450 LF @		160	=	72,000
b.	Hdwl.	- 2 EA @		5000	=	10,000
c.	MH	- 1 EA @		2500	=	2500
d.	AC swale	- 5,700 SY @		8	=	45,600
			Alt. #5,6,7&8	Total	=	130,100

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SHEET 4 OF ______

PROJECT ______ DATE ______

SUBJECT ______ DESIGNED ______ CHECKED ______

9. Floodwall and Montello modification

a. Flood wall	-	2400 LF @	$ 65	=	$156,000
b. C&G	-	200 LF @	9	=	1,800
c. Sidewalk	-	200 LF @	14	=	2,800
d. Plantmix surf.	-	600 SY @	14	=	8,400
		Alt. #5,6,7&8	Total	=	$169,000

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JOB NO. 150-08-1

PROJECT Evans Ave. Drainage Study DATE 11/2/89

SUBJECT Quantities DESIGNED PBE CHECKED

Georges Den Dam

$$\left(\frac{0+21}{2}\right)25' + \left(\frac{21+1256}{2}\right)36' + \left(\frac{1256+1616}{2}\right)42'$$

$$+\left(\frac{1616+1616}{2}\right)104' + \left(\frac{1616+0}{2}\right)118' = 347{,}000\ CF$$

$$= \underline{12{,}900\ CY} \quad \underline{\text{neat line}}$$

Upper Cemetery Dam

$$\left(\frac{0+131}{2}\right)144' + \left(\frac{131+800}{2}\right)20' + \left(\frac{800+1313}{2}\right)76'$$

$$+\left(\frac{1313+3053}{2}\right)47' + \left(\frac{3053+0}{2}\right)85' = 331{,}400\ CF$$

$$= \underline{12{,}300\ CY} \quad \underline{\text{neat line}}$$

Lower Cemetery Dam

$$\left(\frac{0+150}{2}\right)19' + \left(\frac{150+96}{2}\right)7' + \left(\frac{96+193}{2}\right)169'$$

$$+\left(\frac{193+2240}{2}\right)40' + \left(\frac{2240+2713}{2}\right)65' + \left(\frac{2713+3891}{2}\right)10'$$

$$+\left(\frac{3891+1813}{2}\right)42' + \left(\frac{1813+0}{2}\right)58' = 441{,}720\ CF$$

$$= \underline{16{,}400\ CY} \quad \underline{\text{neat line}}$$

FORM 230

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JOB NO. 150-08-1

SHEET 2 OF 5

PROJECT Evans Ave. Drainage Study DATE 11/2/89

SUBJECT Quantities DESIGNED PBE CHECKED

Small Reno Rendering Dam

$$\left(\frac{0+193}{2}\right)15' + \left(\frac{193+240}{2}\right)110' + \left(\frac{240+350}{2}\right)113' + \left(\frac{350+591}{2}\right)64'$$

$$\left.\begin{array}{l} + \left(\frac{591+0}{2}\right)86' = 114{,}150\ CF \\ \left(\frac{193+0}{2}\right)400' = 38{,}600\ CF \end{array}\right\} \underline{5700\ CY}\ \underline{\text{neat line}}$$

Large Reno Rendering Dam

$$\left(\frac{0+553}{2}\right)13' + \left(\frac{553+630}{2}\right)90' + \left(\frac{630+800}{2}\right)112' + \left(\frac{800+1146}{2}\right)64'$$

$$\left.\begin{array}{l} + \left(\frac{1146+0}{2}\right)131' = 274{,}250\ CF \\ \left(\frac{553+0}{2}\right)520' = 143{,}780\ CF \end{array}\right\} \underline{15{,}500\ CY}\ \underline{\text{neat line}}$$

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JOB NO. 150-02-1

PROJECT Evans Ave. Drainage Study DATE 11/2/89

SUBJECT Quantities DESIGNED PBE CHECKED

Pipe lengths

Lower Cemetery Dam (30"Ø RCP)

(33' x 2.5) 2 + 10' + 10' = 185'

Small Reno Rendering Dam (2-42"Ø RCP)

(13.5 x 2.5) 2 + 10' + 10' = 88'

Large Reno Rendering Dam (36"Ø RCP)

(19.5 x 2.5) 2 + 10' + 10' = 118'

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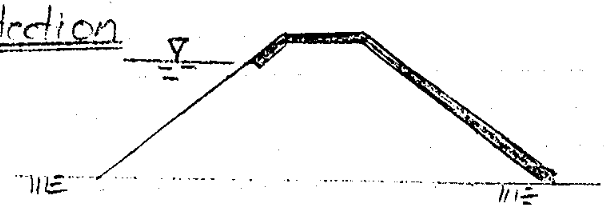
JOB NO. 150.08.1

PROJECT Evans Ave. Drainage Study DATE 11/3/89

SUBJECT Quantities DESIGNED PBE CHECKED

Downstream slope protection

3' thick Soil Cement



Lower Cemetery

$$\left(\frac{0+17}{2}\right)19' + \left(\frac{17+13}{2}\right)7' + \left(\frac{13+20}{2}\right)169' + \left(\frac{20+79}{2}\right)40'$$

$$+\left(\frac{79+88}{2}\right)65' + \left(\frac{88+106}{2}\right)10' + \left(\frac{106+71}{2}\right)42' + \left(\frac{71+0}{2}\right)58'$$

$$+(410')(10') + (410')(8') = 24{,}600\ SF\,(3')/27 = 2750\ CY$$

Small Reno Rendering Dam

$$\left(\frac{0+19}{2}\right)15' + \left(\frac{19+22}{2}\right)110' + \left(\frac{22+27}{2}\right)113' + \left(\frac{27+36.5}{2}\right)64'$$

$$+\left(\frac{36.5+0}{2}\right)86' + \left(\frac{19+0}{2}\right)400' + (790')(10') + (790)(8')$$

$$= 26{,}800\ SF\,(3')/27 = 3000\ CY$$

Large Reno Rendering Dam

$$\left(\frac{0+35}{2}\right)13' + \left(\frac{35+38}{2}\right)90' + \left(\frac{38+43}{2}\right)112' + \left(\frac{43+52.5}{2}\right)64'$$

$$+\left(\frac{52.5+0}{2}\right)131' + \left(\frac{35+0}{2}\right)520' + (945')(10') + (945')(8')$$

$$= 40{,}700\ SF\,(3')/27 = 4550\ CY$$

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JOB NO. 150-05-1

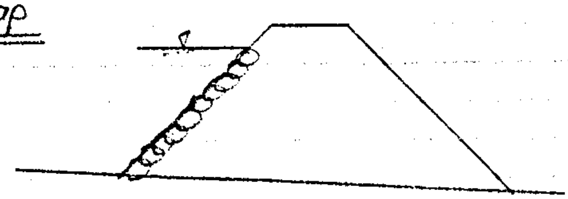
SHEET 5 OF 5

PROJECT Evans Ave. Drainage Study DATE 11/3/89

SUBJECT Quantities DESIGNED PBE CHECKED

Upstream Dam Face Rip-rap

3' min. thickness



Lower Cemetery Dam

$$\left(\frac{0+8}{2}\right)19' + \left(\frac{8+4}{2}\right)7' + \left(\frac{4+11}{2}\right)169' + \left(\frac{11+67}{2}\right)40'$$

$$+\left(\frac{67+75}{2}\right)65' + \left(\frac{75+93}{2}\right)10' + \left(\frac{93+59}{2}\right)42' + \left(\frac{59+0}{2}\right)58'$$

$$= 13,300\ SF\ (3') = 39,900\ CF = \underline{\underline{1500\ CY}}$$

Small Reno Rendering Dam

$$\left(\frac{0+11}{2}\right)15' + \left(\frac{11+13.5}{2}\right)110' + \left(\frac{13.5+19}{2}\right)113' + \left(\frac{19+28}{2}\right)64'$$

$$+\left(\frac{28+0}{2}\right)86' + \left(\frac{0+11}{2}\right)400' = 8200\ SF(3')/27 = \underline{\underline{950\ CY}}$$

Large Reno Rendering Dam

$$\left(\frac{0+27}{2}\right)13' + \left(\frac{27+30}{2}\right)90' + \left(\frac{30+35}{2}\right)112' + \left(\frac{35+44.5}{2}\right)64'$$

$$+\left(\frac{44.5+0}{2}\right)131' + \left(\frac{27+0}{2}\right)520' = \frac{18,900\ SF\ (3')}{27} = \underline{\underline{2100\ CY}}$$

FORM 230

APPENDIX 6

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JOB NO. ____

SHEET 1 OF ____

PROJECT ____ DATE ____

SUBJECT ____ DESIGNED ____ CHECKED ____

Routing scenarios :

Oddie system cap. (42"ϕ RCP @ S = .0115 ′/′) = 110 cfs

I-80 system cap. (54"ϕ RCP @ S = .0012 ′/′) = 70 cfs

Alt. #3 – upper cemetery detention w/ 42"ϕ outlet

Q_{100} = 518 cfs @ Reno Rendering

Routing: Oddie system = 110 + 110 + 50 = 270 cfs
I-80 system = 70 cfs
Excess = 178 cfs

Alt. #4 – lower cemetery detention w/ 30"ϕ outlet

Q_{100} = 480 cfs @ Reno Rendering

Routing: Oddie system = 110 + 110 + 50 = 270 cfs
I-80 system = 70 cfs
Excess = 140 cfs

Alt. #5 – #4 w/ 'J' routed above Evans — Q_{25}

Q_{100} = 423 cfs @ Reno Rendering — 319

Routing: Ex. Oddie system = 110 + 50 = 160 cfs
New Oddie system = 220 cfs — 49
Excess = 43 cfs — ~~0~~

Alt. #6 – #5 w/ 48"ϕ outlet @ Evans — Q_{25}

Q_{100} = 280 cfs @ Reno Rendering — 217

Routing: Ex. Oddie system = 110 + 50 = 160 cfs
New Oddie system = 110 cfs — 57
Excess = 10 cfs — ~~0~~

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JOB NO. ______

SHEET 2 OF ______

PROJECT ______ DATE ______

SUBJECT ______ DESIGNED ______ CHECKED ______

Alt. #7 — #5 + lg. Reno Rend. Det. w/ 36"ϕ outlet — Q_{25}

Q_{100} = 155 cfs @ Reno Rendering — 135

Routing: Ex. Oddie system = 110 + 45 = 155 cfs
Excess = $\cancel{0}$ — $\cancel{0}$

Alt. #8 — #7 w/ 2-42"ϕ outlets (sm. dam) — Q_{25}

Q_{100} = 334 cfs @ Reno Rendering — 275

Routing: Ex. Oddie system = +110+50 = 160cfs
New Oddie system = 160 cfs — 8
Excess = 14 cfs — $\cancel{0}$

Alt. #11 — #6 + Reno Rend. Det. w/ 36"ϕ outlet — =

Q_{100} = 154 cfs @ Reno Rendering

Routing: Oddie system = 110 + 44 = 154 cfs
Excess = $\cancel{0}$

Base run — no improvements upstream

Q_{100} = 585 cfs @ Reno Rendering

Routing: Oddie system = 110 + 110 + 50 = 270 cfs
I-80 system = 70 cfs
Excess = 245 cfs

Alt. #10 — #4 w/ irrig. storage, requires 36" outlet

Q_{100} = 505 cfs @ Reno Rendering

Routing: Oddie system = 110 + 110 + 50 = 270 cfs
I-80 system = 70 cfs
Excess =

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PROJECT ______ DATE ______

SUBJECT ______ DESIGNED ______ CHECKED ______

Alternatives for routing excess flows:

A. Add'l. pipe in Oddie Blvd.

- assume same size & parallel to exist. system will fit, match from Sutro to Paradise Pond

Cap. = 110 cfs

(See "E." for pipe from Reno Rendering to Sutro)

B. New pipe from I-80 to Truckee R. (from Wells ditch)

length = 400' + 400' + 2300' = 3100'

U/S HGL elev = 4479

D/S HGL elev = 4472 (100-yr. flood from FIRM)

Δ ELEV = 7 ft

HGL slope = $\frac{7}{3100}$ = .0023 '/' , using n = .013 :

Cap.	48" φ RCP	=	70 cfs
"	54" φ "	=	96 cfs
"	60" φ "	=	127 cfs
"	66" φ "	=	165 cfs
"	72" φ "	=	210 cfs

C. Pipe & ditch from Reno Rendering to I-80

1. Pipe to Sadlier – max. Q = 70 cfs

Total head = 105.5 – 85.0 + 1.5 = 22'

Total length = 450 + 200 + 1000 = 1650 LF

HGL S = $\frac{22}{1650}$ = .0133 '/' , n = .013

cap.	36" φ RCP	=	78 cfs
"	42" φ "	=	120 cfs
"	48" φ "	=	170 cfs
"	54" φ "	=	230 cfs

FORM 230

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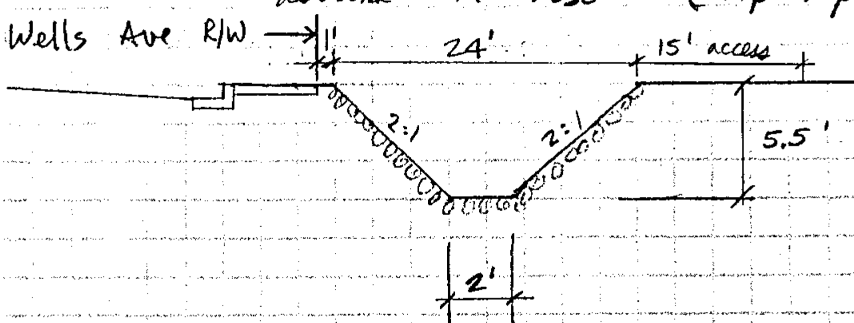
C. (cont.)

2. Open-channel – max. Q = 70 cfs

Slope = $\frac{(85.9 - 79.4)}{1720}$ = 0.0038 '/'

assume n = .030 (rip-rap sides + bottom)



for b = 2', z = 2, n = .030, S = .004 '/'

Q = 70 cfs: d_n = 2.6' (V = 3.7 fps), d_c = 1.9' (flow is subcritical)
Q = 50 cfs: d_n = 2.2' (V = 3.6 fps), d_c = 1.6' (flow is subcritical)

3. Pipe to I-80 – max. Q = 70 cfs

S = $\frac{(73.5 - 65.0)}{400}$ = 0.021 '/'

try 36" ϕ RCP, min. S = .011 '/'

Ck. entrance control:

Req'd HW/D = 1.7, HW = 3(1.7) = 5.1' OK

try 42" RCP, min. S = .0046 '/' (allows shallower installation)

Req'd HW/D = 1.2, HW = 3(1.2) = 3.6'

For Q = 50 cfs – try 30" ϕ RCP, min. S = .014 '/'

Req'd. HW/D = 2.0, HW = 2.5 (2.0) = 5.0' OK

FORM 230

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SHEET 5 OF ____

PROJECT ____ DATE ____
SUBJECT ____ DESIGNED ____ CHECKED ____

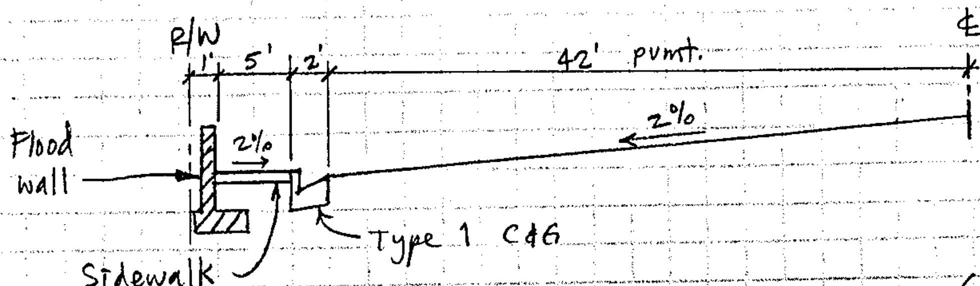
D. Oddie Blvd - overland

Slope: (Sutro to Montello) = 1.2%

(Montello to Fwy.) = 0.6%

R/W : 100'

RTC ultimate improvements: 4 lanes + flares



Assumed Conveyance Section (North ½ of road)

X -	0.0	0.99	1.0	6.46	6.5	8.0	50.0
Y -	10.00	10.00	9.64	9.54	9.04	9.16	10.00

use: (8.0) n = .012 / (50.) n = .015', s = .006 '/1, inc = .04'

*081HYMOIN

Capacity - see HYMO output

use Q = 50 cfs

Note:

This alternative assumes that the flow will be piped from Reno Rendering to the east side of Sutro. An appropriate structure will be required to connect to the existing 42"ϕ storm drain in Oddie Blvd. and allow excess flows to "bubble up" into to street section.

Appendix 6

SE&A, INC.

HYMO - HYDROLOGIC MODELING PROGRAM BY JIMMY R. WILLIAMS AND ROY W. HANN, JR.

* IMPLEMENTED ON HP-1000 A-SERIES 3/83 BY COMPENG COMPUTER SYSTEMS, LTD.

* INPUT AND OUTPUT ENHANCEMENTS AND SCS METHOD HYDROGRAPH GENERATING CAPABILITY ADDED 4/84 BY SEA, INC.

INPUT FILE = *081HYMOIN

JOB NUMBER: 0150-08-1 EVANS AVE. D. S. - STREET CAPACITIES
9:35 AM THU., 19 OCT., 1989

START TIME= 0.

Oddie Blvd - I-580 to Montello

COMPUTE RATING CURVE ID= 1 VS= 1 NSEG= 2 MIN EL= 9.04 FT INC= -.04 FT

CHSLOPE= .006 FPSLOPE= .006

N= .012 DIST= 8. FT N= .015 DIST= 50. FT

DIST(FT)	ELEV(FT)	DIST(FT)	ELEV(FT)
0.	10.	.99	10.
1.	9.64	6.46	9.54
6.5	9.04	8.	9.16
50.	10.		

RATING CURVE VALLEY SECTION 1.0

WATER SURFACE ELEV	FLOW AREA SQ FT	FLOW RATE CFS
9.04	0.0	0.0
9.08	.0	.0
9.12	.0	.0
9.16	.1	.1
9.20	.2	.3
9.24	.4	.6
9.28	.6	1.2
9.32	1.0	2.0
9.36	1.4	3.0
9.40	1.9	4.4
9.44	2.5	6.1
9.48	3.1	8.3
9.52	3.9	10.8
9.56	4.7	13.1
9.60	5.7	16.0
9.64	6.9	20.0
9.68	8.1	25.4
9.72	9.5	31.5
9.76	10.9	38.4
9.80	12.5	46.1

FINISH

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JOB NO. ______

SHEET 6 OF ______

PROJECT ______ DATE ______

SUBJECT ______ DESIGNED ______ CHECKED ______

E. Pipe from Reno Rendering to Sutro

1. Pipe to connect to existing system

- assume design flow includes capacity of existing 42"ϕ storm drain plus the overland flow capacity of the street section

$Q = 110 + 50 = \underline{160 \text{ cfs}}$

HGL Slope

U/S IE = 4505.5
D/S IE = 4478.4

Δ ELEV = 27.1

Length = 250 + 1300 = 1550 ft

$S = \frac{27.1}{1550} = .0175 \text{ ′/′}$

- need 48"ϕ RCP, min. S = .0120′/′

- since min. slope is less than exist. slope in Oddie, try smaller pipe in upper end

D/S IE = 4478.4 + (.012 × 850′) = 4488.6

U/S IE = 4505.5
D/S IE = 4488.6

Δ ELEV = 16.9 ft

$S = \frac{16.9}{700} = .0241 \text{ ′/′}$

- use 42"ϕ RCP, min. S = .024′/′ (upper 700′)

- use 48"ϕ RCP, min. S = .012′/′ (lower 850′)

FORM 230

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JOB NO.

SHEET 7 OF

PROJECT DATE

SUBJECT DESIGNED CHECKED

E. (cont.)

2. Pipe from Reno Rendering to Sutro to connect to new Oddie system

Alt. #5 Q = 220 cfs

Alt. #6 Q = 110 cfs

Alt. #8 Q = 160 cfs

Alt. #8 - use 42"φ RCP for upper 700' & 48"φ RCP for lower 850' per E.1. (Sht. 6)

Alt. #5 - need min. 54"φ RCP @ S = .0120'/' for lower 850'

- for entrance control, 54"φ RCP requires $HW/D = 2.0$, $HW = \frac{54}{12}(2) = 9'$ OK

Alt. #6 - need min. 42"φ RCP @ S = .0120'/' for lower 850'

- for entrance control, 42"φ RCP requires $HW/D = 1.85$, $HW = \frac{42}{12}(1.85) = 6.5'$ OK

FORM 230

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JOB NO. ______

SHEET 8 OF ______

PROJECT ______ DATE ______

F.
SUBJECT Cost comparison - Oddie vs. Wells outfall DESIGNED ______ CHECKED ______

Possible upsizing of Oddie system:

- match outfall pipe IE (66"ϕ), assume top of pipe profile same as for exist. system
- cap. 66"ϕ RCP @ S = .0044% = 230 cfs
- size upstream pipes for: 220 cfs — 160 cfs

for S = .0044%, use	66"ϕ RCP	(1700 LF)	60"ϕ RCP
for S = .0070% +, use	60"ϕ RCP	(2620 LF)	54"ϕ RCP
for S = .0120%, use	54"ϕ RCP	(990 LF)	48"ϕ RCP
for S = .0240%, use	48"ϕ RCP		42"ϕ RCP

Equivalent capacity using Wells Ave. route: (Q = 210 cfs)

Reno Render. to Sadlier (S = .0133%) - 54"ϕ RCP (1650 LF)

or { Sadlier to I-80 (S = .004%) - ' F.B. ditch (1600 LF)
" " " (S = .0038%) - 66"ϕ RCP (1600 LF) }

I-80 to Truckee (S = .0023%) - 72"ϕ RCP (3100 LF)

Conclusion

- by inspection, Wells Ave. route would cost more than Oddie route:

Pipe ϕ	Wells	Oddie	Difference
54"	1650'	990'	(670' less)
66"	1600'	1700'	(100' more)
60"	-0-	2620'	(480' less & 2 sizes smaller,
72"	3100'	-0-	+ probably shallower excavation)

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JOB NO. ______

PROJECT ______ DATE ______

SUBJECT G. Existing Oddie system analysis DESIGNED ______ CHECKED ______

Ex. Pipe inventory - Paradise Pond to Sutro

66"ϕ RCP/CMP	- channel to MH-1 (outfall)		= 300 LF ±
54"ϕ RCP	- MH-1 to MH-2	(S=.0044'/')	= 480 LF
"	- MH-2 to MH-3	(")	= 180 LF
48"ϕ RCP	- MH-3 to MH-4	(S=.0060'/')	= 570 LF
"	- MH-4 to MH-5	(")	= 480 LF
"	- MH-5 to MH-6	(")	= 420 LF
"	- MH-6 to MH-7	(S=.0093'/')	= 540 LF
"	- MH-7 to MH-8	(S=.0100'/')	= 360 LF
"	- MH-8 to MH-10	(S=.0060'/')	= 990 LF
42"ϕ RCP	- MH-10 to MH-12	(S=.0120'/')	= 990 LF

Pipe Totals - 54"ϕ RCP - 660 LF

48"ϕ RCP - 3360 LF

42"ϕ RCP - 990 LF

Ex. Capacities -

cap. 54"ϕ RCP @ S=.0044'/' = 130 cfs

" 48"ϕ RCP @ S=.0060'/' = 110 cfs

" 42"ϕ RCP @ S=.0120'/' = 110 cfs

use 110 cfs

FORM 230

Analysis of local drainage effect on existing Oddie Blvd. system:

Peak discharge - Q_{100}, Att. #7

Area "P" - Sutro (24"ϕ) = 76 cfs (exceeds pipe capacity)

Area "U" - Helena (36"ϕ) = 88 cfs (exceeds pipe capacity)

Render @ Helena or Sutro = 155 cfs @ 16:20

Render @ Sutro = 110 cfs @ 12:10 (before peak)

Comb. @ Sutro = 189 cfs @ 12:10

Render @ Helena = 98 cfs @ 12:00 (before peak)

Comb. @ Helena = 251 cfs @ 12:00

Comb. @ Helena = 162 cfs @ 16:40 (past peak)

Conclusions

1. Discharge from the Sutro & Helena storm drains will overload the existing Oddie system from approx. 11:50 to 13:20. The concurrent discharge from the Reno Rendering Dam outfall uses up the existing pipe capacity and may cause flow to back up in the Sutro & Helena systems (Q_{100} = 79 cfs @ 11:50 & Q_{100} = 145 cfs @ 13:10 from Reno Rendering outfall).

2. Excess flows up to 30 cfs will cross Oddie during a 1½ hr. period & flow south along Sutro.

3. Excess flow in Helena will not cross Oddie ... [illegible]

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SHEET 11 OF ______

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SUBJECT ______ DESIGNED ______ CHECKED ______

Flood routing

1. a. Route area "J" to east of Evans ($Q_{100} = 227$ cfs)

- assume future development of parcel (commercial)
- assume sheet flow across McCarran

Options: a) Collect flows along McCarran w/ ditch & pipe across parcel, down Enterprise, & across Evans

b) Collect flows along Enterprise & pipe under Evans

Pipe size req'd. (assume inlet control):

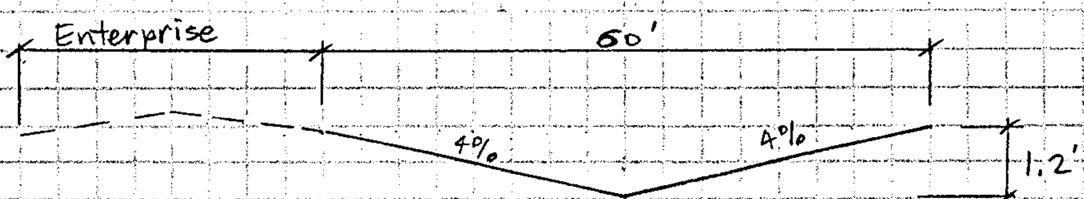
use 60" ϕ RCP @ $HW/D = 1.6 \pm$, $HW = 1.6(5) = 8.0'$

Conveyance along Enterprise:

$S = 1.0\%$, $Q_{100} = 227$ cfs

- assume 4% X-slope V-ditch, AC lined ($z = 25$)

for $n = .015$, $d_n = \underline{1.15'}$ & $d_c = 1.4'$



Req'd. Paved Channel

b. Route area 'J' down Valley in pipe

- using same entrance control as above, use 60" ϕ RCP

FORM 230

2. Pipe across Manogue High School athletic field
   - assume pressure flow conditions w/ suitable structure at terminus of twin 48"ø UPRR culverts
   - capacity assumed to be limited by friction slope = average available slope

<u>Δ ELEV</u>

  47.5 (IE 48" UPRR culvert outlet)
− 34.8 (IE 72" storm drain inlet)

  12.7 ft

<u>SLOPE</u> $S = \frac{12.7'}{690'} = 0.0184$ '/'

(slope of exist. pipe = 0.025 '/', therefore use average slope computed above)

<u>CAPACITY</u>

− for n = .013 & S = .0184 '/'

Q = 580 cfs

<u>Ck. potential transition head loss:</u> (Q = 580 cfs)

$A_{2\text{-}48"} = 25.1\ ft^2$, V = 23.1 fps

$A_{72"} = 28.3\ ft^2$, V = 20.5 fps

ΔV = 23.1 − 20.5 = 2.6 fps

$V^2/2g$ = 0.10' <u>negligible</u>

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SHEET 13 OF

PROJECT DATE

SUBJECT DESIGNED CHECKED

-estimate pipe sizes for $S = .018'/1$, $n = .013$

Alt. No.	Q_{100}	Pipe size	
4	286	60"φ	(54"φ for S = .020'/1)
5,7,8	379	66"φ	(60"φ for S = .020'/1)
6,11	227	54"φ	
Base	560	72"φ	
10	317	60"φ	
12	591	72"φ	

FORM 230

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42" RCP s = .0120 = 110 cfs

downstream ≥

54" RCP S = .0044'/' = 130 cfs

48" RCP S = 0.006 '/' = 110 cfs

Cemetery (toe of dam to Comstock 48"φ RCP)

- upper dam site Q_{100} = 250 cfs, 48"φ @ S = .029 '/'

- lower " " " 125 cfs, 42"φ @ S = .015 '/'
36"φ @ S = .034 '/'

- cap. of ex. 48" RCP: HW/D = 13/9 = 1.44, Q = 120 cfs (w/o flooding access road)

S = 65 - 6 = 59 FG -6'
- 47 IE (48")
12 ÷ 640 = 1.9% (lower cemetery only)

S = 73 IE (36")
- 47 IE (48")
26 ÷ 970 = 2.7% (36" inlet to 48" inlet ave.)

APPENDIX 7

ALTERNATIVE 5, PHASE 1

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE: 11/8/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	LOWER CEMETERY DETENTION DAM			
a)	CLEARING AND GRUBBING	1.0 LS	$20,000.00	$20,000.00
b)	ROCK SLOPE PROTECTION	1500.0 CY	$15.00	$22,500.00
c)	NATIVE CLAY FILL	17900.0 CY	$5.00	$89,500.00
d)	3' IMPERVIOUS BLANKET	3000.0 CY	$4.00	$12,000.00
e)	EXCAVATION CUTOFF TRENCH	2000.0 CY	$4.00	$8,000.00
f)	30" RCP w/ CUTOFF COLLARS	185.0 LF	$120.00	$22,200.00
g)	HEADWALLS & APPURTENANCES	1.0 LS	$10,000.00	$10,000.00
h)	CHAIN LINK FENCE AND GATE	1.0 LS	15000.00	15,000.00
i)	INLET MANHOLE & APPURT.	1.0 LS	5000.00	5,000.00
j)	SOIL EROSION CONTROL	1.0 LS	10000.00	10,000.00
k)	SOIL CEMENT	2750.0 CY	37.50	103,125.00
l)	ROCK LINED CHANNEL	400.0 LF	10.00	4,000.00
m)	DEWATERING	1.0 LS	20000.00	20,000.00
n)	36" RCP	420.0 LF	60.00	25,200.00
o)	42" RCP	420.0 LF	80.00	33,600.00
p)	MANHOLE	5.0 EA	2500.00	12,500.00
			SUBTOTAL:	$412,625.00
2)	ROUTE AREA "J" ABOVE EVANS AVE.			
a)	60" RCP	450.0 LF	160.00	72,000.00
b)	HEADWALL	2.0 EA	5000.00	10,000.00
c)	MANHOLE	1.0 EA	2500.00	2,500.00
d)	AC SWALE	5700.0 SY	8.00	45,600.00
			SUBTOTAL:	$130,100.00
3)	66" RCP ACROSS MANOGUE HIGH SCHOOL			
a)	66" RCP	690.0 LF	180.00	124,200.00
b)	MANHOLE	2.0 EA	2500.00	5,000.00
c)	SPECIAL MANHOLE	1.0 EA	10000.00	10,000.00
			SUBTOTAL:	$139,200.00

ALTERNATIVE 5, PHASE 1 TOTAL: $681,925.00

ALTERNATIVE 5, PHASE 2

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE 11/6/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	42" RCP/48" RCP FROM RENO RENDERING TO SUTRO ST.			
a)	42" RCP	700.0 LF	80.00	56,000.00
b)	48" RCP	850.0 LF	120.00	102,000.00
c)	HEADWALL	1.0 EA	5000.00	5,000.00
d)	MANHOLE	6.0 EA	2500.00	15,000.00
e)	SPECIAL DROP INLET	1.0 EA	5000.00	5,000.00
			SUBTOTAL:	$183,000.00
2)	ODDIE BLVD. FLOOD WALL AND MONTELLO ST. MODIFICATION			
a)	FLOOD WALL	2400.0 LF	65.00	156,000.00
b)	CURB AND GUTTER	200.0 LF	9.00	1,800.00
c)	SIDEWALK	200.0 LF	14.00	2,800.00
d)	PLANTMIX SURFACE	600.0 SY	14.00	8,400.00
			SUBTOTAL:	$169,000.00

ALTERNATIVE 5, PHASE 2 TOTAL $352,000.00

ALTERNATIVE 5, PHASE 3

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE 11/6/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	54" RCP/60" RCP/66" RCP FROM RENO RENDERING TO PARADISE POND			
a)	54" RCP	2540.0 LF	150.00	381,000.00
b)	60" RCP	3360.0 LF	170.00	571,200.00
c)	66" RCP	960.0 LF	190.00	182,400.00
d)	HEADWALL	2.0 EA	5000.00	10,000.00
e)	MANHOLE	17.0 EA	2500.00	42,500.00
			SUBTOTAL:	$1,187,100.00

ALTERNATIVE 5, PHASE 3 TOTAL $1,187,100.00

ALTERNATIVE 6, PHASE 1

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE: 11/7/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	LOWER CEMETERY DETENTION DAM			
a)	CLEARING AND GRUBBING	1.0 LS	$20,000.00	$20,000.00
b)	ROCK SLOPE PROTECTION	1500.0 CY	$15.00	$22,500.00
c)	NATIVE CLAY FILL	17900.0 CY	$5.00	$89,500.00
d)	3' IMPERVIOUS BLANKET	3000.0 CY	$4.00	$12,000.00
e)	EXCAVATION CUTOFF TRENCH	2000.0 CY	$4.00	$8,000.00
f)	30" RCP w/ CUTOFF COLLARS	185.0 LF	$120.00	$22,200.00
g)	HEADWALLS & APPURTENANCES	1.0 LS	$10,000.00	$10,000.00
h)	CHAIN LINK FENCE AND GATE	1.0 LS	15000.00	15,000.00
i)	INLET MANHOLE & APPURT.	1.0 LS	5000.00	5,000.00
j)	SOIL EROSION CONTROL	1.0 LS	10000.00	10,000.00
k)	SOIL CEMENT	2750.0 CY	37.50	103,125.00
l)	ROCK LINED CHANNEL	400.0 LF	10.00	4,000.00
m)	DEWATERING	1.0 LS	20000.00	20,000.00
n)	36" RCP	420.0 LF	60.00	25,200.00
o)	42" RCP	420.0 LF	80.00	33,600.00
p)	MANHOLE	5.0 EA	2500.00	12,500.00
			SUBTOTAL:	$412,625.00
2)	ROUTE AREA "J" ABOVE EVANS AVE.			
a)	60" RCP	450.0 LF	160.00	72,000.00
b)	HEADWALL	2.0 EA	5000.00	10,000.00
c)	MANHOLE	1.0 EA	2500.00	2,500.00
d)	AC SWALE	5700.0 SY	8.00	45,600.00
			SUBTOTAL:	$130,100.00
3)	48" OUTLET FOR EVANS AVE. DETENTION			
a)	INLET STRUCTURE	1.0 LS	10000.00	10,000.00
			SUBTOTAL:	$10,000.00
4)	54" RCP ACROSS MANOGUE H.S.			
a)	54" RCP	690.0 LF	140.00	96,600.00
b)	MANHOLE	2.0 EA	2500.00	5,000.00
c)	SPECIAL MANHOLE	1.0 EA	10000.00	10,000.00
			SUBTOTAL:	$111,600.00

ALTERNATIVE 6, PHASE 1 TOTAL: $664,325.00

ALTERNATIVE 6, PHASE 2

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE 11/7/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	42" RCP/48" RCP FROM RENO RENDERING TO SUTRO ST.			
a)	42" RCP	700.0 LF	80.00	56,000.00
b)	48" RCP	850.0 LF	120.00	102,000.00
c)	HEADWALL	1.0 EA	5000.00	5,000.00
d)	MANHOLE	6.0 EA	2500.00	15,000.00
e)	SPECIAL DROP INLET	1.0 EA	5000.00	5,000.00
			SUBTOTAL:	$183,000.00
2)	ODDIE BLVD. FLOOD WALL AND MONTELLO ST. MODIFICATION			
a)	FLOOD WALL	2400.0 LF	65.00	156,000.00
b)	CURB AND GUTTER	200.0 LF	9.00	1,800.00
c)	SIDEWALK	200.0 LF	14.00	2,800.00
d)	PLANTMIX SURFACE	600.0 SY	14.00	8,400.00
			SUBTOTAL:	$169,000.00
			ALTERNATIVE 6, PHASE 2 TOTAL	$352,000.00

ALTERNATIVE 6, PHASE 3

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE 11/7/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	42" RCP/48" RCP/54" RCP FROM RENO RENDERING TO PARADISE POND			
a)	42" RCP	2540.0 LF	90.00	228,600.00
b)	48" RCP	3360.0 LF	120.00	403,200.00
c)	54" RCP	960.0 LF	150.00	144,000.00
d)	HEADWALL	2.0 EA	5000.00	10,000.00
e)	MANHOLE	17.0 EA	2500.00	42,500.00
			SUBTOTAL:	$828,300.00
			ALTERNATIVE 6, PHASE 3 TOTAL	$828,300.00

ALTERNATIVE 7, PHASE 1

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE: 11/7/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	LOWER CEMETERY DETENTION DAM			
a)	CLEARING AND GRUBBING	1.0 LS	$20,000.00	$20,000.00
b)	ROCK SLOPE PROTECTION	1500.0 CY	$15.00	$22,500.00
c)	NATIVE CLAY FILL	17900.0 CY	$5.00	$89,500.00
d)	3' IMPERVIOUS BLANKET	3000.0 CY	$4.00	$12,000.00
e)	EXCAVATION CUTOFF TRENCH	2000.0 CY	$4.00	$8,000.00
f)	30" RCP w/ CUTOFF COLLARS	185.0 LF	$120.00	$22,200.00
g)	HEADWALLS & APPURTENANCES	1.0 LS	$10,000.00	$10,000.00
h)	CHAIN LINK FENCE AND GATE	1.0 LS	15000.00	15,000.00
i)	INLET MANHOLE & APPURT.	1.0 LS	5000.00	5,000.00
j)	SOIL EROSION CONTROL	1.0 LS	10000.00	10,000.00
k)	SOIL CEMENT	2750.0 CY	37.50	103,125.00
l)	ROCK LINED CHANNEL	400.0 LF	10.00	4,000.00
m)	DEWATERING	1.0 LS	20000.00	20,000.00
n)	36" RCP	420.0 LF	60.00	25,200.00
o)	42" RCP	420.0 LF	80.00	33,600.00
p)	MANHOLE	5.0 EA	2500.00	12,500.00
			SUBTOTAL:	$412,625.00
2)	ROUTE AREA "J" ABOVE EVANS AVE.			
a)	60" RCP	450.0 LF	160.00	72,000.00
b)	HEADWALL	2.0 EA	5000.00	10,000.00
c)	MANHOLE	1.0 EA	2500.00	2,500.00
d)	AC SWALE	5700.0 SY	8.00	45,600.00
			SUBTOTAL:	$130,100.00
3)	66" RCP ACROSS MANOGUE H.S.			
a)	66" RCP	690.0 LF	180.00	124,200.00
b)	MANHOLE	2.0 EA	2500.00	5,000.00
c)	SPECIAL MANHOLE	1.0 EA	10000.00	10,000.00
			SUBTOTAL:	$139,200.00
4)	LARGE RENO RENDERING DETENTION DAM			
a)	CLEARING AND GRUBBING	1.0 LS	15000.00	15,000.00
b)	ROCK SLOPE PROTECTION	2100.0 CY	15.00	31,500.00
c)	CLAY FILL	18000.0 CY	10.00	180,000.00
d)	3' IMPERVIOUS BLANKET	3000.0 CY	8.00	24,000.00
e)	SLURRY TRENCH	1.0 LS	200000.00	200,000.00
f)	36" RCP w/ CUTOFF COLLARS	120.0 LF	140.00	16,800.00
g)	HEADWALLS & APPURTENANCES	1.0 LS	10000.00	10,000.00
h)	CHAIN LINK FENCE & GATE	1.0 LS	15000.00	15,000.00
i)	TYPE IV MANHOLE & APPURT.	1.0 LS	5000.00	5,000.00
j)	SOIL EROSION CONTROL	1.0 LS	10000.00	10,000.00
k)	SOIL CEMENT	4550.0 CY	37.50	170,625.00
l)	ROCK LINED CHANNEL	300.0 LF	10.00	3,000.00
m)	DEWATERING	1.0 LS	20000.00	20,000.00
			SUBTOTAL:	$700,925.00

ALTERNATIVE 7, PHASE 1 TOTAL: $1,382,850.00

ALTERNATIVE 7, PHASE 2

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE 11/7/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	42" RCP/48" RCP FROM RENO RENDERING TO SUTRO ST.			
a)	42" RCP	700.0 LF	80.00	56,000.00
b)	48" RCP	850.0 LF	120.00	102,000.00
c)	HEADWALL	1.0 EA	5000.00	5,000.00
d)	MANHOLE	6.0 EA	2500.00	15,000.00
e)	SPECIAL DROP INLET	1.0 EA	5000.00	5,000.00
			SUBTOTAL:	$183,000.00
2)	ODDIE BLVD. FLOOD WALL AND MONTELLO ST. MODIFICATION			
a)	FLOOD WALL	2400.0 LF	65.00	156,000.00
b)	CURB AND GUTTER	200.0 LF	9.00	1,800.00
c)	SIDEWALK	200.0 LF	14.00	2,800.00
d)	PLANTMIX SURFACE	600.0 SY	14.00	8,400.00
			SUBTOTAL:	$169,000.00

ALTERNATIVE 7, PHASE 2 TOTAL: $352,000.00

ALTERNATIVE 8, PHASE 1

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE: 11/8/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	LOWER CEMETERY DETENTION DAM			
a)	CLEARING AND GRUBBING	1.0 LS	$20,000.00	$20,000.00
b)	ROCK SLOPE PROTECTION	1500.0 CY	$15.00	$22,500.00
c)	NATIVE CLAY FILL	17900.0 CY	$5.00	$89,500.00
d)	3' IMPERVIOUS BLANKET	3000.0 CY	$4.00	$12,000.00
e)	EXCAVATION CUTOFF TRENCH	2000.0 CY	$4.00	$8,000.00
f)	30" RCP w/ CUTOFF COLLARS	185.0 LF	$120.00	$22,200.00
g)	HEADWALLS & APPURTENANCES	1.0 LS	$10,000.00	$10,000.00
h)	CHAIN LINK FENCE AND GATE	1.0 LS	15000.00	15,000.00
i)	INLET MANHOLE & APPURT.	1.0 LS	5000.00	5,000.00
j)	SOIL EROSION CONTROL	1.0 LS	10000.00	10,000.00
k)	SOIL CEMENT	2750.0 CY	37.50	103,125.00
l)	ROCK LINED CHANNEL	400.0 LF	10.00	4,000.00
m)	DEWATERING	1.0 LS	20000.00	20,000.00
n)	36" RCP	420.0 LF	60.00	25,200.00
o)	42" RCP	420.0 LF	80.00	33,600.00
p)	MANHOLE	5.0 EA	2500.00	12,500.00
			SUBTOTAL:	$412,625.00
2)	ROUTE AREA "J" ABOVE EVANS AVE.			
a)	60" RCP	450.0 LF	160.00	72,000.00
b)	HEADWALL	2.0 EA	5000.00	10,000.00
c)	MANHOLE	1.0 EA	2500.00	2,500.00
d)	AC SWALE	5700.0 SY	8.00	45,600.00
			SUBTOTAL:	$130,100.00
3)	66" RCP ACROSS MANOGUE H.S.			
a)	66" RCP	690.0 LF	180.00	124,200.00
b)	MANHOLE	2.0 EA	2500.00	5,000.00
c)	SPECIAL MANHOLE	1.0 EA	10000.00	10,000.00
			SUBTOTAL:	$139,200.00
4)	SMALL RENO RENDERING DETENTION DAM			
a)	CLEARING AND GRUBBING	1.0 LS	10000.00	10,000.00
b)	ROCK SLOPE PROTECTION	950.0 CY	15.00	14,250.00
c)	CLAY FILL	7700.0 CY	10.00	77,000.00
d)	3' IMPERVIOUS BLANKET	2000.0 CY	8.00	16,000.00
e)	SLURRY TRENCH	1.0 LS	200000.00	200,000.00
f)	2-42" RCP w/ CUTOFF COL.'S	90.0 LF	300.00	27,000.00
g)	HEADWALLS & APPURTENANCES	1.0 LS	12000.00	12,000.00
h)	CHAIN LINK FENCE & GATE	1.0 LS	15000.00	15,000.00
i)	TYPE IV MANHOLE & APPURT.	1.0 LS	10000.00	10,000.00
j)	SOIL EROSION CONTROL	1.0 LS	10000.00	10,000.00
k)	SOIL CEMENT	3000.0 CY	37.50	112,500.00
l)	ROCK LINED CHANNEL	200.0 LF	10.00	2,000.00
m)	DEWATERING	1.0 LS	20000.00	20,000.00
			SUBTOTAL:	$525,750.00

ALTERNATIVE 8, PHASE 1 TOTAL: $1,207,675.00

ALTERNATIVE 8, PHASE 2

OPINION OF PROBABLE CONSTRUCTION COST
BY: GS/PBE
DATE 11/7/89

ITEM	DESCRIPTION	QUANTITY	UNIT COST	EXTENSION
1)	42" RCP/48" RCP FROM RENO RENDERING TO SUTRO ST.			
a)	42" RCP	700.0 LF	80.00	56,000.00
b)	48" RCP	850.0 LF	120.00	102,000.00
c)	HEADWALL	1.0 EA	5000.00	5,000.00
d)	MANHOLE	6.0 EA	2500.00	15,000.00
e)	SPECIAL DROP INLET	1.0 EA	5000.00	5,000.00
			SUBTOTAL:	$183,000.00
2)	ODDIE BLVD. FLOOD WALL AND MONTELLO ST. MODIFICATION			
a)	FLOOD WALL	2400.0 LF	65.00	156,000.00
b)	CURB AND GUTTER	200.0 LF	9.00	1,800.00
c)	SIDEWALK	200.0 LF	14.00	2,800.00
d)	PLANTMIX SURFACE	600.0 SY	14.00	8,400.00
			SUBTOTAL:	$169,000.00
3)	42"/48"/54"/60" RCP FROM RENO RENDERING TO PARADISE POND			
a)	42" RCP	700.0 LF	80.00	56,000.00
b)	48" RCP	1840.0 LF	120.00	220,800.00
c)	54" RCP	3360.0 LF	150.00	504,000.00
d)	60" RCP	960.0 LF	170.00	163,200.00
e)	HEADWALL	2.0 EA	5000.00	10,000.00
f)	MANHOLE	17.0 EA	2500.00	42,500.00
			SUBTOTAL:	$996,500.00

ALTERNATIVE 8, PHASE 2 TOTAL: $1,348,500.00