the project site (**LVOW 0460** – Category A proposed 2,700 ft of 7'x6' RCB in Vegas Drive between Torrey Pines Drive and Jones Boulevard; **LVOW 0360** – Category A proposed 2,700 ft of 8'x6' RCB in Vegas Drive between Jones Boulevard and Michael Way).

The storm drain system to be constructed with this project is a regional flood control facility (**LVOW 0241** – Category A proposed 8,600 ft of dual 8'x6' RCB in Vegas Drive between Michael Way and Rancho Drive). The existing downstream facility does not have the capacity to convey the flow that the 2002 MPU proposed dual 8'x6' facility would be able to convey; therefore the new storm drain design proposes to change this facility to a single storm drain. The proposed storm drain facility consists of approximately 520 feet of 6'x5' RCB, 1,030 feet of 7'x5' RCB, and 7,160 feet of 72" RCP in-order to perpetuate the existing downstream storm drain capacity along Vegas Drive between Shadow Mountain Place and Rancho Drive.

A future regional flood control facility, which is not displayed on Figure F-12, will be required in-order to convey a majority of the 100-year event flow rate as discussed in the previously mentioned Alternatives Study. The future regional flood control facility is assumed to convey approximately 690cfs of the total 850cfs 100-year event storm water runoff that originates west of the Vegas Drive/Decatur Boulevard intersection, north along Decatur Boulevard to Lake Mead Boulevard then east to the existing Carey-Lake Mead Detention Basin. The remaining 160cfs continues East of Decatur Blvd along Vegas Drive. The Decatur Boulevard facility between Vegas Drive and Lake Mead Boulevard was proposed in the Alternatives Study as 2,800 ft of 10'x7' RCB.

3.6 Hydrologic Analysis

The hydrologic methods used are in accordance with adopted Clark County Regional Flood Control District's Hydrologic Criteria and Drainage Design Manual (CCRFCD HCDDM), dated August 1999. The Soil Conservations Services (SCS) unit hydrograph method was used to model subbasin storm hydrographs, and the Muskingum-Cunge method was used for routing. The study area was divided into subbasins based on flow patterns, to allow for the identification of runoff flow rates at specific locations. Subbasins located east of Jones Boulevard were delineated utilizing 1 foot contours, as well as field observations. Subbasins located west of Jones Boulevard are referenced from the 2002 MPU and are not expected to be significantly altered in future Master Plan Updates. The total analyzed watershed area displayed on **Figure 3.2 - Drainage Sub-basin Map** is approximately 2.05 square miles.

3.6.1 Rainfall

The CCRFCD HCDDM was used to establish the point precipitation values and rainfall distribution. According to Figure 513 and Table 505 of the CCRFCD HCDDM, the project watershed is located within the McCarran Airport Rainfall Area and therefore the point precipitation value of 2.77" for the 100-year six-hour storm and 1.58" for the 10-year six-hour storm was used for all subbasins.

The point precipitation values are related to rainfall intensity at an isolated point. Since the rainfall occurs over an extensive area simultaneously, with the more intense rainfall occurring near the center of the storm, a reduction factor is used to adjust the point precipitation when modeling the rainfall decay characteristics over a large area. The rainfalls Depth Area Reduction Factors (DARF) were developed by the Nation Oceanic and Atmospheric Administration (NOAA) for the purpose of converting a point precipitation depth to an average precipitation over the entire storm area. The 10-year point precipitation value was combined with the DARF so that the 100-year point precipitation value may be used and converted using the adjusted DARF to obtain a 10-year flow rate. The adjusted DARF used to convert a 100-year flow rate to a 10-year flow rate was established by multiplying the original DARF by the ratio of the 10-year point precipitation (1.58") to the 100-year point precipitation (2.77") value. The DARF ratios used in this study to establish the design flow rates for the corresponding tributary area and return period are shown in Table 3.6.1.

Table 3.6.1 – Depth Area Reduction Factors

Tributary Area (sq mi)	DARF 10-year Discharge	DARF 100-Year Discharge
0.0 – 0.5	0.570	1.000
0.5 – 1.0	0.559	0.980
1.0 – 2.0	0.553	0.970
2.0 – 3.0	0.530	0.930

The percent of total depth tabulation recommended by the USACE, Los Angeles District in 1988, was adopted for rainfall/runoff prediction in the Las Vegas Valley. For those channels and design points were the tributary watershed is less than 8.0 square miles, the SDN3 storm distribution is used. The SDN5 storm distribution is used if the total watershed area exceeds 12.0 square miles and the SDN4 is for watershed areas between 8.0-12.0 square miles. The SDN3 storm distribution is used in this study.

3.6.2 Curve Number

Major precipitation losses consist of infiltration losses determined by the soil type and depression losses caused by land uses. Infiltration losses occur throughout the storm event, while depression losses only occur during the initial period of the storm. The difference between the total precipitation volume and the loss volume is the direct runoff volume. This method uses curve number (CN) index to relate precipitation losses to soil type, land use, and vegetative cover conditions.

The SCS methodology divides soils into four hydrological groups: Group A, Group B, Group C, and Group D, which were developed based on infiltration rates. SCD Group A soil has a high infiltration rate, such as for sands and gravels. SCS Group D soil has a low infiltration rate such as for clay soil or rock. The soils information for the project watershed was obtained from Figure H-12 of the 2002 MPU. Figure H-12 is included in Appendix D and shows the hydrologic soil group of soils identified from the *Soil Survey of Las Vegas Valley Area, Clark County, Nevada* (1985 Soil Conservation Services). The Natural Resources Conservation Service (NRCS) recently released the 2006 *Soil Survey of Clark County Area, Nevada.* The Vegas Drive project watershed is not addressed in the 2006 NRCS soil survey. Therefore, the 2006 NRCS soil survey does not provide additional soil information than that provided in the 2002 MPU Figure H-12 produced by PBS&J. According to Figure H-12 hydrologic soil group D is found throughout the project watershed.

The land use in the project area is mostly developed residential with the exception of the Fountain Park and Las Vegas Golf Club located on the southeast corner of Vegas Drive and Decatur Blvd. All subbasins are assumed to be completely developed with full street improvements throughout the watershed. Zoning throughout the watershed consists of a combination of commercial, open space, single family residential, and multi-family residential. City of Las Vegas *Future Land Use* and *Zoning* maps have been included in Appendix D. Weighted CN values were calculated for each subbasin using the *Curve Number Determination* spread sheet included in Appendix D and Table 602 of the CCRFCD HCDDM.

3.6.3 Lag Time

Lag times for each subbasin were calculated using the procedure outlined in the CCRFCD HCDDM. The following lag time (T_L) and time of concentration (T_C) equations were used for drainage basins with areas less than 1.0 square mile.

$$\text{Lag Time} = T_L = 0.6 * T_c \qquad T_c = T_i + T_t$$

$$T_i = 1.8*(1.1 - K)*L_i^{1/2} / S^{1/3} \qquad K = 0.0132*CN - 0.39$$

$$T_t = L_O/(60* V)$$

Where,
T_i = Initial overland flow time (minutes)
T_t = Travel time through the subbasin in the channel (minutes)
K = Flow resistance coefficient
L_i = Initial overland flow length (500 ft. maximum for rural areas, 300 ft. maximum for urban areas)
S = Average basin slope (percent)
CN = Curve Number
L_O = Overland flow length through the subbasin in the channel (ft)
V = Average Velocity (ft per second)

Average velocities for overland travel time were estimated using the equation $V = C*S^{1/2}$ derived from Manning's equation. Two sets of coefficients were utilized for developed and undeveloped areas. For developed areas, a coefficient of C=20.2 was used to represent street flow for the first 500 ft of travel, and C=30.6 represents curb height street flow for the remaining overland travel length. Coefficients for undeveloped areas were established to represent wide channel flow with Manning's n=0.040. Undeveloped areas used the coefficients C=14.8 for the first 500 ft and C=29.4 for the remaining overland travel distance. The time of concentration and lag time were calculated for each subbasin using the Standard Form 4 spreadsheet provided in Appendix D.

3.6.4 Hydrograph Routing

Storm hydrographs generated from subbasins have to be conveyed downstream through a drainage network, which may consist of channels, detention basins, basin diversions, etc. Channel routing methods are numerical simulations of flood wave movements along a drainage network, and transform a hydrograph, adjusting for the travel time and channel storage volume.

In this study, the Muskingum-Cunge routing procedures were used to model storm water movements in the channel reaches. The Muskingum-Cunge method of routing is based on wave diffusion theory and is non-linear in nature. The amount of diffusion is matched to physical diffusion determined using physical channel characteristics. This method is recommended for flow modeling uniform defined channels. The parameters used for this method are the shape of the channel cross section, channel reach length, channel slope, and channel roughness.

3.6.5 Hydrologic Model

The *HEC-1 Flood Hydrograph Computer Model* was developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center in the late 1970's, and then adopted by the CCRFCD HCDDM in 1988 for rainfall and runoff predictions.

The model is used to simulate the surface runoff response of a watershed to a design rainfall event. Each subbasin in the watershed is described by a set of parameters that specify the subbasin drainage and storm characteristics. These basin parameters include precipitation, basin area, curve number, and lag time.

The drainage system in a watershed is converted into node-link type of system for use with the HEC-1 computer model. A node is placed at the subbasin outlet or at the design point along the water course. Links between nodes represent conveyance elements in the drainage system. Hydrographs are computed for each subbasin and are placed at the subbasin outlet node. These hydrographs are then routed and/or combined according to the drainage basin network configuration through links. The DARF is known at each design point and is used to determine the peak flow at the design point depending on the contributing area. Additional detailed explanation of the theory and background is presented in Section 4 of the SCS National Engineering Handbook and HEC-1 user manual.

The *NW3SC.dat* HEC-1 file developed by PBS&J for the 2002 MPU was used as a basis for developing the drainage models for this project site. The project watershed area was assumed to be completely developed with full street improvements throughout the watershed. This is considered a conservative assumption and was incorporated to simplify the hydrology models by using the same subbasin characteristics. Therefore the estimated amount of runoff flow for each subbasin is the same during the existing, interim, and ultimate conditions. A total of two HEC-1 models were created to establish the flows generated during the 10-year and 100-year, 6-hour storm events for the existing/interim (*VD3I.dat*) and ultimate conditions (*VD3U.dat*). The HEC-1 output for each model is included in the Appendix D. Table 3.6.2 is a summary of each subbasin's estimated peak runoff flow rates taken from the HEC-1 hydrology model output.

Table 3.6.2 Subbasin Peak Flow Rate Summary

Subbasin	Area (sq-mi)	10-Year Peak Flow Rate (cfs)	100-Year Peak Flow Rate (cfs)
OG011A-C	0.441	125	339
VG02-C	0.224	47	153
VG03-C	0.311	53	186
VG04	0.1121	61	137
VG05A	0.0113	8	18
VG05B	0.0161	11	25
VG05C	0.0236	16	35
VG05D	0.0044	3	7
VG05E	0.0094	6	14
VG05F	0.0047	4	8
VG05G	0.0781	42	95
VG05H	0.0115	8	17
VG05I	0.0046	3	7
VG05J	0.0035	4	7
VG05K	0.0063	6	12
VG05L	0.0060	6	12
VG05M	0.0135	12	24
VG22A	0.0077	7	14
VG06A	0.0135	11	22
VG06A	0.0135	11	22
VG22B	0.0071	6	12
VG06B	0.0106	8	17
VG06C	0.0575	10	36
VG22C	0.0066	5	10
VG06D	0.0971	17	58
VG22D	0.0116	6	15
VG22E	0.0279	13	32
VG22F	0.0071	6	12
VG22G	0.0109	7	16
VG22H	0.0213	12	29
VG22I	0.0050	3	8
LM09-C	0.2205	109	247
LM08	0.0581	40	87
VG20A	0.0623	42	87
VG20B	0.1284	66	147
VG20C	0.0140	12	24

3.7 Existing/Interim Conditions

The design flow rates for the existing and interim conditions were established using the same HEC-1 model, since the only differences would be in the channel routing cross-sections. Minor changes in the channel routing cross section were considered to generate insignificant differences in the estimated flow rates. Runoff flow collected in Vegas Drive is conveyed east as surface flow during the existing condition or during the interim condition as a combination of surface flow and within the proposed Vegas Drive storm drain system.

An existing 18" storm drain system was located in Vegas Drive between Jones Boulevard and Laurelhurst Drive (approximately 800 ft west of Decatur) where the system turns south and continues within Laurelhurst Drive. The capacity of the existing 18" storm drain system was considered to handle nuisance flows only. The existing 18" storm drain system will be utilized to continue conveying nuisance flows where it is not in conflict with the proposed Vegas Drive system. The existing 18" storm drain system is proposed to tie-in with the Vegas Drive system at multiple locations therefore abandoning (or removing as needed) portions that will no longer convey flow.

The interim condition analysis assumes the proposed storm drain facilities along Vegas Drive are fully constructed between Shadow Mountain Place and Rancho Drive. The intent is to extend the 455cfs capacity in the downstream 84" RCP (at Rancho Drive) west along Vegas Drive to Shadow Mountain Place. The proposed storm drain system can convey the 10-year event flow rates without surcharging the facility. During a 100-year event the storm water will be conveyed along Vegas Drive as a combination of surface flow and facility flow, with the proposed storm drain facility expected to be surcharged. The proposed storm drain system is discussed further in sections 3.9 of this report. Table 3.7.1 on the following page lists the peak design flow rates for the existing and interim conditions determined from the *VD3I.dat* HEC-1 model included in Appendix D.

Table 3.7.1 Existing/Interim Condition Design Point Flow Rates Summary

Design Point	Vegas Drive Cross Street Location Description	Tributary Area (sq-mi)	10-Year Peak Flow Rate (cfs)	100-Year Peak Flow Rate (cfs)
DP1	Saylor Way	1.09	236	718
DP2	Winwood Street	1.12	241	731
DP3	Shadow Mountain Place	1.14	246	744
DP4	Tequesta Road	1.15	248	749
DP5S	Laurelhurst Dr (from South of Vegas Dr)	0.09	47	107
DP5	Laurelhurst Drive	1.25	285	837
DP6	Decatur Boulevard	1.28	290	850
DP7	Approx. 400 ft east of Decatur Blvd	1.30	294	861
DP8	Yellow Rose Street	1.32	296	867
DP9	Stoney Drive	1.38	305	899
DP10	Valley Drive	1.38	306	902
DP11	Mountain Trail	1.52	328	978
DP12	Pyramid Drive	1.53	329	980
DP13	East Lane	1.56	335	999
DP14N	N Rancho Dr (from North of Vegas Dr)	0.48	210	234
DP14	North Rancho Drive	2.05	500	1138

The Vegas Drive proposed storm drain facility was designed to convey 455cfs based on the existing downstream facility design capacity.

3.8 Ultimate Condition

The ultimate condition analysis assumes all of the proposed CCRFCD master planned facilities affecting Vegas Drive are in-place, including the northerly storm drain branch along Decatur Blvd. This future Decatur Branch is assumed to convey approximately 690cfs of the total 850cfs at design point DP6 (see Table 3.8.1) north to the Lake Mead master planned storm drain system with the remaining 160cfs, labeled as design point DPT6E in the HEC-1 analysis, continuing East along Vegas Drive. The ultimate condition analysis also assumes the master planned facilities upstream of Shadow Mountain Place are online and conveying all of the flow at design point DP1. Table 3.8.1 on the following page lists the peak design flow rates for the ultimate condition determined from the *VD3U.dat* HEC-1 model included in Appendix D.

Table 3.8.1 Ultimate Condition Design Point Flow Rates Summary

Design Point	Vegas Drive Cross Street Location Description	Tributary Area (sq-mi)	10-Year Peak Flow Rate (cfs)	100-Year Peak Flow Rate (cfs)
DP1	Saylor Way	1.09	236	718
DP2	Winwood Street	1.12	241	731
DP3	Shadow Mountain Place	1.14	246	744
DP4	Tequesta Road	1.15	248	749
DP5S	Laurelhurst Dr (from South of Vegas Dr)	0.09	47	107
DP5	Laurelhurst Drive	1.25	285	837
* DP6	* Decatur Boulevard	1.28	* 290	* 850
*DPT6E	* Continues East in Vegas Dr	0.00	* 160	* 160
DP7	Approx. 400 ft east of Decatur Blvd	0.02	178	196
DP8	Yellow Rose Street	0.04	186	225
DP9	Stoney Drive	0.10	193	247
DP10	Valley Drive	0.10	196	256
DP11	Mountain Trail	0.24	233	341
DP12	Pyramid Drive	0.25	226	348
DP13	East Lane	0.28	243	395
DP14N	N Rancho Dr (from North of Vegas Dr)	0.48	210	234
DP14	North Rancho Drive	0.77	445	618

The Vegas Drive proposed storm drain facility was designed to convey 455cfs based on the existing downstream facility design capacity.

* During the Ultimate Condition, a flow split occurs at DP6 that assumes a maximum of 160 cfs (DPT6E) continues East along Vegas Dr while the remaining majority of flow is conveyed north along Decatur Blvd in a future connecting CCRFCD master planned facility.

3.9 Hydraulic Analysis and Facility Design

The proposed Vegas Drive storm drain facilities were divided into two separate storm drain system alignments. The "LVOW" and "SD3" storm drain systems are discussed in detail below. The hydraulic analysis for the "LVOW" and "SD3" storm sewer systems were performed using the Los Angeles County Flood Control District's WSPGW software. The WSPGW output for the "LVOW" and "SD3" storm drain facilities are included in Appendix E.

3.9.1 "LVOW" System

The proposed "LVOW" interim facility between Shadow Mountain Place and Rancho Drive is designed to convey 455cfs. This facility is proposed as a 6'x5' RCB for approximately 520 feet at the upstream end and transitions over 10 feet to a 72" RCP for 2,290 feet to west of Decatur Blvd. The "LVOW" facility transitions near Decatur Blvd to a 7'x5' RCB for 1,060 feet so to avoid sanitary sewer lateral conflicts by remaining above the laterals. Approximately 800 feet east of Decatur Blvd the storm drain transitions back into a 72" RCP and dives below the sanitary sewer laterals where needed for the remaining 4,835 feet to connect into the existing 84" RCP west of Rancho Drive. A total of 5 (4-along "LVOW" and 1-along storm drain lateral #4A) locking manhole lids are called out on the storm drain plans in locations where the storm drain has shallow cover and is expected to be surcharged during a 100-year event. A hydraulic jump is expected to occur in the 72" RCP west of Rancho Drive due to the backwater effects caused by the shallow slope of the existing 8'x5' RCB crossing Rancho Drive. The storm drain invert is located approximately 15 feet below the ground surface near Rancho Drive, while the hydraulic jump is expected to surcharge the storm drain to approximately 9 feet when conveying approximately 455cfs.

According to the Geotechnical Report prepared for the Vegas Drive Improvements by Terracon Consulting Engineers and Scientists, high ground water will likely be encountered along the project boundary prompting the need for temporary dewatering. Attempts were made to minimize the depth of the proposed storm drain system to reduce the potential groundwater impacts where practical. Impacts to the installation procedures for the proposed storm drain due to the expected high ground water will be addressed in the Special Provisions provided with a following submittal. Refer to the Geotechnical Report provided in the Vegas Drive Improvements Preliminary Design Report dated July 13, 2007 for additional discussion of the geotechnical analysis.

3.9.2 "SD3" System

The "SD3" system is proposed on Shadow Mountain Place north of Vegas Drive. This system is intended to resolve a current flooding problem by draining the low-point along Shadow Mountain Place, located at the intersection of Shadow Mountain Place and Bloomdale Way (approximately 750' north of Vegas Drive). A total of 3 NDOT type 2 drop inlets are proposed to drain this low point and convey approximately 35 cfs in a 36" RCP to the "LVOW" storm drain facility.

3.10 Street Capacity Analysis

Street capacity calculations were performed in order to maintain the 12-foot dry lane criteria for the minor storm. A flow depth can be determined that would maintain one 12-foot dry lane in each direction. This flow depth was input into a spreadsheet that utilizes Manning's Equation to calculate the flow capacity in a street section. An allowable flow was calculated for each segment of Vegas Drive, utilizing the minimum slope of the vertical profile for that segment. The product of the velocity and flow depth was calculated to be less than 6 for the 10-year event. Table 3.10.1 summarizes the 10-year event street flow capacities previously provided in

Appendix E of the *Drainage Study Update to the Preliminary Design Report, Vegas Drive – Shadow Mountain to Rancho Drive,* dated December 2007 (*Update to the Preliminary Design Report*). See **Figure 3.3 – Proposed Peak Flow Summary** for the street cross section locations and a summary of the peak flow rate locations.

Table 3.10.1 Vegas Drive 10-Year Event Street Flow Capacities

"VD" Station to Station	Min. Street Slope (%)	VxD_{10}	Allow. ½ Street Q_{10} (cfs)	Existing Condition North or South Side Q_{10} (cfs)	Interim Condition		Ultimate Condition	
					North Side Q_{10} (cfs)	South Side Q_{10} (cfs)	North Side Q_{10} (cfs)	South Side Q_{10} (cfs)
6+00 to 9+00	1.55	1.71	10.8	**120.5**	**76.5**	**76.5**	1.4	2.3
9+00 to 11+00	0.76	1.20	7.6	**123.0**	**42.0**	**42.0**	0.1	0.2
11+00 to 18+50	1.28	1.55	9.9	**123.0**	**48.0**	**45.0**	6.1	3.2
18+50 to 22+50	1.38	1.61	10.2	**124.0**	**48.0**	**37.1**	6.1	1.4
22+50 to 28+50	1.12	1.45	9.2	**124.0**	**43.0**	**41.1**	6.4	5.4
28+50 to 35+50	1.61	1.74	11.1	**142.5**	**39.2**	6.8	7.7	6.8
35+50 to 40+50	0.73	1.17	7.4	**145.0**	7.0	1.2	7.0	1.2
40+50 to 44+50	0.81	1.24	7.8	**147.0**	4.1	2.0	4.1	2.0
44+50 to 47+00	0.60	1.06	6.7	**147.0**	2.1	0.4	2.1	0.4
47+00 to 52+00	1.54	1.70	10.8	**147.0**	8.1	8.4	8.1	8.4
52+00 to 54+00	1.74	1.81	11.5	**148.0**	3.5	3.7	3.5	3.7
54+00 to 56+50	1.27	1.55	9.8	**148.0**	3.5	8.4	3.5	8.4
56+50 to 60+50	0.80	1.23	7.8	**152.5**	3.5	4.1	3.5	4.1
60+50 to 66+00	1.35	1.59	10.1	**152.5**	6.6	9.8	6.6	9.8
66+00 to 70+00	0.60	1.06	6.7	**153.0**	3.8	4.5	3.8	4.5
70+00 to 72+50	1.83	1.86	11.8	**153.0**	9.8	10.2	9.8	10.2
72+50 to 78+00	1.71	1.79	11.4	**164.0**	4.7	10.5	4.7	10.5
78+00 to 81+50	0.89	1.29	8.2	**164.0**	6.0	2.7	6.0	2.7
81+50 to 84+50	2.34	2.10	13.3	**164.0**	13.0	8.7	13.0	8.7
84+50 to 89+50	2.73	2.27	14.4	**164.5**	5.1	4.1	5.1	4.1
89+50 to 94+00	1.03	1.39	8.8	**167.5**	5.5	4.1	5.5	4.1

As noticed in Table 3.10.1, the maximum allowable 10-year event street capacity is exceeded during the existing condition for the length of the project and during the interim condition between stations 6+00 and 28+50 for both the north and south sides of Vegas Drive and on the north half of Vegas Drive between stations 28+50 and 35+50. Additional drop inlets would be required at the upstream end of the proposed Vegas Drive storm drain system to satisfy the 12-ft dry lane criteria during the interim condition. The additional drop inlets would not be required during the ultimate condition and therefore would be a throw away cost. Although the 12-ft dry lane criteria may not be met at all locations during the interim condition it is considered an improvement from the existing condition where in effect all of the runoff flow is conveyed as surface flow in Vegas Drive. Therefore PDG considers this an acceptable design. Additional analyses were performed to calculate the actual dry lane width at the locations where the 12-ft criteria is not met. The additional calculations were provided in Appendix E of the *Update to the Preliminary Design Report* and summarized in Table 3.10.2.

Table 3.10.2 Actual Vegas Drive 10-Year Event Street Flows (Interim Condition)

VD Station to Station	Portion of Vegas Drive	Depth (ft)	Velocity (fps)	$VxD_{(10)}$	Actual ½ Street $Q_{(10)}$ (cfs)	Actual Dry Lane Width (ft)
6+00 to 9+00	North & South Side	0.82	5.59	4.60	76.50	0.00
9+00 to 11+00	North & South Side	0.77	3.66	2.80	42.00	0.00
11+00 to 18+50	North Side	0.74	4.58	3.38	48.00	0.00
11+00 to 18+50	South Side	0.72	4.50	3.26	45.00	0.00
18+50 to 22+50	North Side	0.73	4.71	3.44	48.00	0.00
18+50 to 22+50	South Side	0.68	4.38	2.97	37.10	1.88
22+50 to 28+50	North Side	0.73	4.24	3.09	43.00	0.00
22+50 to 28+50	South Side	0.72	4.18	3.01	41.10	0.00
28+50 to 35+50	North Side	0.68	4.70	3.17	39.20	2.08

Street capacity calculations were also performed for the 100-year storm event. The product of the velocity and flow depth was the governing factor in determining the major storm event street capacity. An allowable street flow was calculated for each segment of Vegas Drive by setting the V*D factor equal to the maximum value of 8.0. During the interim condition 100-year storm event the street capacities along Vegas Drive are expected to be exceeded for the majority of the project length. Calculations for the 100-year storm street flow capacities were provided in Appendix E of the *Update to the Preliminary Design Report* and summarized in Table 3.10.3 below.

Table 3.10.3 Vegas Drive 100-Year Event Street Flow Capacities

VD Station to Station	Min. Street Slope (%)	$VxD_{(100)}$	Allow. ½ Street $Q_{(100)}$ (cfs)	Existing Condition	Interim Condition		Ultimate Condition	
				North or South Side $Q_{(100)}$ (cfs)	North Side $Q_{(100)}$ (cfs)	South Side $Q_{(100)}$ (cfs)	North Side $Q_{(100)}$ (cfs)	South Side $Q_{(100)}$ (cfs)
6+00 to 9+00	1.55	8.00	171.9	**365.5**	**283.5**	**283.5**	4.7	8.1
9+00 to 11+00	0.76	8.00	194.5	**372.0**	**216.0**	**216.0**	0.6	1.3
11+00 to 18+50	1.28	8.00	178.2	**372.0**	**216.0**	**216.0**	14.6	8.3
18+50 to 22+50	1.38	8.00	178.2	**374.5**	**203.5**	**203.5**	14.6	5.1
22+50 to 28+50	1.12	8.00	182.5	**474.5**	**196.0**	**196.0**	17.1	13.1
28+50 to 35+50	1.61	8.00	170.6	**418.5**	**181.5**	**181.5**	19.2	28.1
35+50 to 40+50	0.73	8.00	203.6	**425.0**	188.0	188.0	14.0	7.1

Table 3.10.3 Vegas Drive 100-Year Event Street Flow Capacities (continued)

VD Station to Station	Min Street Slope (%)	$V \times D_{100}$	Allow ½ Street Q_{100} (cfs)	Existing Condition	Interim Condition		Ultimate Condition	
				North or South Side Q_{100} (cfs)	North Side Q_{100} (cfs)	South Side Q_{100} (cfs)	North Side Q_{10} (cfs)	South Side Q_{10} (cfs)
40+50 to 44+50	0.81	8.00	192.6	**430.5**	186.5	186.5	9.6	9.7
44+50 to 47+00	0.60	8.00	201.6	**430.5**	186.5	186.5	6.1	4.2
47+00 to 52+00	1.54	8.00	172.1	**430.5**	**186.5**	**186.5**	18.1	21.2
52+00 to 54+00	1.74	8.00	171.9	**433.5**	**187.5**	**187.5**	10.1	12.3
54+00 to 56+50	1.27	8.00	178.5	**433.5**	**187.5**	**187.5**	10.1	21.2
56+50 to 60+50	0.80	8.00	193.0	**449.5**	**202.5**	**202.5**	10.1	26.7
60+50 to 66+00	1.35	8.00	181.4	**449.5**	**202.5**	**202.5**	16.5	46.0
66+00 to 70+00	0.60	8.00	201.6	**451.0**	201.0	201.0	11.7	32.4
70+00 to 72+50	1.83	8.00	178.5	**451.0**	**201.0**	**201.0**	21.7	51.7
72+50 to 78+00	1.71	8.00	168.6	**489.0**	**229.0**	**229.0**	12.9	57.2
78+00 to 81+50	0.89	8.00	189.7	**489.0**	**219.0**	**219.0**	27.7	33.1
81+50 to 84+50	2.34	8.00	157.9	**489.0**	**219.0**	**219.0**	35.7	45.1
84+50 to 89+50	2.73	8.00	178.5	**490.0**	**205.0**	**205.0**	19.7	32.5
89+50 to 94+00	1.03	8.00	189.7	**499.5**	**194.5**	**194.5**	17.7	32.5

3.11 Drop Inlets

Drop inlets along Vegas Drive were sized to capture approximately 70-100% of the 10-year ultimate condition flows and 100% of the flow at the Vegas Drive major intersections with Michael Way and Decatur Blvd. The length of the combination curb opening and grate drop inlets were determined based on the FHWA HEC-22 method and as specified by the CCRFCD HCDDM. The drop inlets were also analyzed for the 100-year ultimate condition flow rates and for the interim condition 10-year and 100-year flow rates. A total of 36 drop inlets are proposed with this project. Calculations for the drop inlets were performed for the 10-year and 100-year design storms during the interim and ultimate conditions. All drop inlet calculations have been provided in Appendix E.

3.12 Storm Drain Laterals

Storm drain lateral HGL's were calculated using a Modified Standard Form 6 worksheet. The Modified Standard Form 6 was adjusted to calculate critical depth within the lateral to a maximum depth equal to the top of pipe. The calculated downstream (D/S) HGL elevation is determined using the appropriate D/S connecting pipe's upstream (U/S) HGL elevation or the HGL elevation within the Vegas Drive trunk line calculated by the *VD455.wsw* design WSPG model. The controlling D/S HGL elevation is then taken as the greater of the calculated D/S HGL elevation or the D/S lateral invert elevation plus critical depth. Rather than assuming the lateral is always pressurized, this step allows the lateral to be analyzed as a partially full (non-pressurized) or full (pressurized) storm drain pipe. The computed headlosses are then added to the controlling D/S HGL elevation to obtain the U/S HGL elevation assuming an outlet controlled culvert. The lateral is also checked for inlet control using the nomograph calculator for the FHWA HDS-5. The controlling U/S HGL elevation is taken as the greater of the outlet or inlet controlled calculations. The Modified Standard Form 6 hydraulic analysis for the storm drain laterals has been provided in Appendix E of this study.

3.13 Pipe Class Determination

Storm drain pipe class was determined using the fill height tables from the American Concrete Pipe Association provided in Appendix E. Pipe Class III is the minimum RCP pipe class recommended for roadway projects. All proposed storm drain pipes were determine to require Class III or less, therefore all storm drain pipes for this project are identified as Class III RCP. A pipe class determination worksheet is included in Appendix E.

3.14 Conclusions and Recommendations

1) The purpose of this project is to provide improved storm water drainage capacity along the Vegas Drive corridor.
2) The project site is not located within a FEMA-designated Special Flood Hazard Area (SFHA).
3) The site is a proposed CCRFCD master planned facility labeled LVOW 0241 (8,600 ft of dual 8'x6' RCB). This project proposes to modify this to a single barrel facility consisting of 520 feet of 6'x5' RCB, 1,060 feet of 7'x5' RCB, and 7,125 feet of 72" RCP.
4) The proposed storm drain facility in Vegas Drive is designed to extend the current capacity of 455cfs in the existing downstream facility located at Rancho Drive, upstream along Vegas Drive to the start of the proposed improvements thereby improving the current storm water drainage in this area.
5) A future storm drain facility will be required to convey a majority of the flow North along Decatur Blvd so to reduce the 100-year event flow rate at Rancho Dr as discussed in the Alternatives Study.
6) The proposed drainage facilities are shown in the 70% Storm Drain Plan and Profile sheets provided in the separate Vegas Drive Improvements 70% Plan set.
7) The methods used to calculate storm runoff and various hydraulic calculations for this project are in compliance with the CCRFCD HCDDM.
8) The proposed drainage facilities will not negatively impact any upstream or downstream properties.

3.15 References

1) Clark County Regional Flood Control District, *Hydraulic Criteria and Drainage Design Manual*, August 1999.
2) PBS&J, *Clark County Regional Flood Control District, 2002 Las Vegas Valley Master Plan Update*, 2002.
3) PBS&J, *City of Las Vegas Central Neighborhood Flood Control Master Plan*, March 2005.
4) PBS&J, *City of Las Vegas City Wide Hydrology Analysis*, 1997.
5) U.S. Army Corps of Engineers, HEC-1 *Flood Hydrograph Package*, September 1981, Revised January 1985.
6) U.S. Department of Agriculture, Soil Conservation Services, *Soil Survey of Las Vegas Valley Area, Nevada*, July 1985.
7) FHWA, *Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22* (HEC-22), Second Edition, August 2001.
8) PDG, *Final Regional Drainage Alternatives Study, Vegas Drive – Shadow Mountain Place to Rancho Drive*, May 2007.
9) Kimley-Horn and Associates, Inc., *Final Storm Drain Design Report for Vegas Drive / Owens Avenue Rancho Drive to Interstate 15*, June 2001.

4.0 COST ESTIMATE

The unit costs utilized for the following cost estimates were based upon an analysis of several recent bid tabulations from City of Las Vegas projects and Clark County Area projects of similar size and scope.

More weight was given to prevailing bids and to current bids. Unit costs were adjusted to account for inflation from the date of the bid and no project more than three years old was considered. Competitor's unit costs were analyzed to avoid giving undo weight to a prevailing bid which may have been unbalanced. Unit costs which were more than 15 percent over the average of all other bids were disregarded in favor of the average of the competitor's bids.

In an effort to limit the number of line items, certain ancillary line items were combined together; for example, prime coat and fog seal costs were included with asphaltic concrete.

Table 4.1 provides the Engineer's Opinion of Probable Construction Cost for the 70% design proposed interim storm drain facility. Table 4.2 provides the estimated construction cost for a conceptual 10-year storm drain facility. The estimated cost to construct the proposed interim storm drain facility is $10,125,000. The estimated cost for a conceptual 10-year storm drain facility is $8,185,000. The difference in cost between the interim facility and the conceptual 10-year facility is $1,940,000.

TABLE 4.1 - ENGINEER'S OPINION OF CONSTRUCTION COST - INTERIM STORM DRAIN FACILITY

Item	Description	Est. Quantity		Unit Price	Total Price
1	ASSOCIATED AC PAVEMENT REMOVAL & REPLACEMENT	20,000	SY	$17.00	$340,000
2	MISC. REMOVALS	1	LS	$50,000.00	$50,000
3	5'x2' RCB	82	LF	$350.00	$28,525
4	6'x5' RCB	555	LF	$740.00	$410,700
5	7'x5' RCB	1,060	LF	$800.00	$848,000
6	TRANSITION STRUCTURE ("LVOW" STA 21+00 TO 21+10)	1	LS	$10,000.00	$10,000
7	TRANSITION STRUCTURE ("SD3" STA 19+13.89 TO 19+18.89)	1	LS	$5,500.00	$5,500
8	TRANSITION STRUCTURE ("LVOW" STA 44+00 TO 44+05)	1	LS	$6,000.00	$6,000
9	TRANSITION STRUCTURE ("LVOW" STA 54+65 TO 54+75)	1	LS	$10,000.00	$10,000
10	6'x5' RCB CAP	1	LS	$600.00	$600
11	18" RCP	490	LF	$120.00	$58,800
12	24" RCP	455	LF	$150.00	$68,250
13	30" RCP	40	LF	$200.00	$8,000
14	36" RCP	1,205	LF	$250.00	$301,250
15	42" RCP	140	LF	$310.00	$43,400
16	72" RCP	7,100	LF	$600.00	$4,260,000
17	19"x30" ELLIPTICAL RCP	50	LF	$195.00	$9,750
18	24"x38" ELLIPTICAL RCP	130	LF	$260.00	$33,800
19	29"x45" ELLIPTICAL RCP	130	LF	$325.00	$42,250
20	34"x53" ELLIPTICAL RCP	255	LF	$400.00	$102,000
21	TYPE "CM" OR "DM" DROP INLET L = 20'	9	EA	$20,000.00	$180,000
22	TYPE "DM2" DROP INLET L =15'	7	EA	$18,000.00	$126,000
23	TYPE "CM" OR "DM" DROP INLET L =10'	10	EA	$15,000.00	$150,000
24	TYPE "CM" OR "DM" DROP INLET L = 5'	8	EA	$10,000.00	$80,000
25	NDOT TYPE 2 DROP INLET	3	EA	$3,000.00	$9,000
26	TYPE I STORM DRAIN MANHOLE	10	EA	$3,000.00	$30,000
27	TYPE III STORM DRAIN MANHOLE	4	EA	$3,500.00	$14,000
28	NDOT TYPE 4 MANHOLE	21	EA	$10,000.00	$210,000
29	STORM DRAIN MANHOLE RISER FOR RCB	9	EA	$2,000.00	$18,000
30	CONSTRUCTION SURVEYING	1	LS	$200,000.00	$200,000
31	TRAFFIC CONTROL AND MAINTENANCE	1	LS	$400,000.00	$400,000
32	VERTICALLY ADJUST WATER VALVE BOX	50	EA	$500.00	$25,000
33	6" WATERLINE RELOCATION - "VD" STA 35+10	1	LS	$7,000.00	$7,000
34	8" WATERLINE RELOCATION - "VD" STA 6+70	1	LS	$10,000.00	$10,000
35	8" WATERLINE RELOCATION - "VD" STA 10+10	1	LS	$10,000.00	$10,000
36	8" WATERLINE RELOCATION - "VD" STA 28+15	1	LS	$10,000.00	$10,000
37	8" WATERLINE RELOCATION - "VD" STA 36+75	1	LS	$15,000.00	$15,000
38	12" WATERLINE RELOCATION - "VD" STA 37+50	1	LS	$50,000.00	$50,000
39	12" WATERLINE RELOCATION - "VD" STA 42+35	1	LS	$15,000.00	$15,000
40	16" WATERLINE RELOCATION - "VD" STA 89+00	1	LS	$40,000.00	$40,000
41	8" C900 PVC WATERLINE	541	LF	$120.00	$64,920
42	8" WATERLINE RELOCATION - "VD" STA 13+00	1	LS	$10,000.00	$10,000
43	8" WATERLINE RELOCATION - "VD" STA 29+25	1	LS	$10,000.00	$10,000
44	12" C900 PVC WATERLINE	165	LF	$150.00	$24,750
45	12" WATERLINE RELOCATION - "VD" STA 40+90	1	LS	$15,000.00	$15,000
46	12" WATERLINE RELOCATION - "VD" STA 43+80	1	LS	$16,000.00	$16,000
47	12" WATERLINE RELOCATION - "VD" STA 44+50	1	LS	$15,000.00	$15,000
48	2" WATERLINE RELOCATION - "VD" STA 43+85	1	LS	$4,000.00	$4,000
49	16" WATERLINE RELOCATION - "VD" STA 85+00	1	LS	$38,000.00	$38,000
50	ABANDON 8" SANITARY SEWER LINE "VD" 10+00	1	LS	$3,000.00	$3,000
				SUB TOTAL	**$8,436,495**
				CONTINGENCIES (20%)	**$1,687,299**
				GRAND TOTAL	**$10,123,794**

TABLE 4.2 - CONCEPTUAL ESTIMATED CONSTRUCTION COST FOR 10-YR STORM DRAIN FACILITY

Item	Description	Est. Quantity		Unit Price	Total Price
1	ASSOCIATED AC PAVEMENT REMOVAL & REPLACEMENT	14,000	SY	$17.00	$238,000
2	MISC. REMOVALS	1	LS	$50,000.00	$50,000
3	TRANSITION STRUCTURE	1	LS	$6,000.00	$6,000
4	66" RCP CAP	1	LS	$500.00	$500
5	18" RCP	1,000	LF	$120.00	$120,000
6	24" RCP	200	LF	$150.00	$30,000
7	30" RCP	1,280	LF	$200.00	$256,000
8	36" RCP	400	LF	$250.00	$100,000
9	60" RCP	3,000	LF	$470.00	$1,410,000
10	66" RCP	5,700	LF	$530.00	$3,021,000
11	3' TYPE "D" DROP INLET	3	EA	$2,000.00	$6,000
12	TYPE "CM" OR "DM" DROP INLET L =15'	9	EA	$18,000.00	$162,000
13	TYPE "CM" OR "DM"DROP INLET L =10'	6	EA	$15,000.00	$90,000
14	TYPE "CM" OR "DM" DROP INLET L = 5'	17	EA	$10,000.00	$170,000
15	NDOT TYPE 2 DROP INLET	3	EA	$3,000.00	$9,000
16	TYPE I STORM DRAIN MANHOLE	2	EA	$3,000.00	$6,000
17	TYPE III STORM DRAIN MANHOLE	3	EA	$3,500.00	$10,500
18	NDOT TYPE 4 MANHOLE	27	EA	$10,000.00	$270,000
19	CONSTRUCTION SURVEYING	1	LS	$160,850.00	$160,850
20	TRAFFIC CONTROL AND MAINTENANCE	1	LS	$321,700.00	$321,700
21	VERTICALLY ADJUST WATER VALVE BOX	50	EA	$500.00	$25,000
22	6" WATERLINE RELOCATION - "VD" STA 35+10	1	LS	$7,000.00	$7,000
23	8" WATERLINE RELOCATION - "VD" STA 6+70	1	LS	$10,000.00	$10,000
24	8" WATERLINE RELOCATION - "VD" STA 10+10	1	LS	$10,000.00	$10,000
25	8" WATERLINE RELOCATION - "VD" STA 28+15	1	LS	$10,000.00	$10,000
26	8" WATERLINE RELOCATION - "VD" STA 36+75	1	LS	$15,000.00	$15,000
27	12" WATERLINE RELOCATION - "VD" STA 37+50	1	LS	$50,000.00	$50,000
28	12" WATERLINE RELOCATION - "VD" STA 42+35	1	LS	$15,000.00	$15,000
29	16" WATERLINE RELOCATION - "VD" STA 89+00	1	LS	$40,000.00	$40,000
30	8" C900 PVC WATERLINE	541	LF	$120.00	$64,920
31	8" WATERLINE RELOCATION - "VD" STA 13+00	1	LS	$10,000.00	$10,000
32	8" WATERLINE RELOCATION - "VD" STA 29+25	1	LS	$10,000.00	$10,000
33	12" C900 PVC WATERLINE	165	LF	$150.00	$24,750
34	12" WATERLINE RELOCATION - "VD" STA 40+90	1	LS	$15,000.00	$15,000
35	12" WATERLINE RELOCATION - "VD" STA 43+80	1	LS	$16,000.00	$16,000
36	12" WATERLINE RELOCATION - "VD" STA 44+50	1	LS	$15,000.00	$15,000
37	2" WATERLINE RELOCATION - "VD" STA 43+85	1	LS	$4,000.00	$4,000
38	16" WATERLINE RELOCATION - "VD" STA 85+00	1	LS	$38,000.00	$38,000
39	ABANDON 8" SANITARY SEWER LINE "VD" 10+00	1	LS	$3,000.00	$3,000
				SUB TOTAL	**$6,820,220**
				CONTINGENCIES (20%)	**$1,364,044**
				GRAND TOTAL	**$8,184,264**

5.0 SPECIAL PROVISIONS TECHNICAL SPECIFICATIONS

VEGAS DRIVE IMPROVEMENTS
SHADOW MOUNTAIN PLACE TO RANCHO DRIVE

OUTLINE

107 Legal Relations and Responsibilities to the Public
- 107.07 Traffic and Access
  - 107.07.01 Traffic Control Regulations
  - 107.07.02 Traffic Control Requirements
  - 107.07.03 Traffic Control and Barricade Plan
  - 107.07.04 Traffic Control Plan for Highway Work Zones
  - 107.07.05 Traffic Control Measurement and Payment
- 107.16 Contractor's Responsibility for the Work and Materials
- 107.18 Furnishing Right-of-Way
- 107.23 Public Relations and Notification

108 Prosecution and Progress
- 108.03 Prosecution and Progress
- 108.06 Temporary Suspension of Work
- 108.08 Determination and Extension of Contract Time
- 108.09 Failure to Complete the Work on Time
- 108.14 Contract Close-Out Procedure
- 108.15 Warranty Inspection

109 Measurement and Payment
- 109.01 Measurement of Quantities
- 109.05 Historical Owner Caused Delay Allowance
- 109.06 Partial Payments
- 109.08 Construction Conflicts and Additional Work
- 109.09 Extension of Quantities
- 109.10 Unsettled Claims

110 Wages, Hours, and Conditions of Employment
- 110.01 Wages, Hours and Employment Practices

200 Mobilization and Demolition
- 200.01.01 General
- 200.05.01 Payment

202 Removal of Structures and Obstructions
- 202.03.01 General
- 202.03.07 Cold Planing
- 202.04.01 Measurement
- 202.05.01 Payment

203 Excavation and Embankment
- 203.01.01 General
- 203.02.01 Roadway Excavation
- 203.04.01 Measurement
- 203.05.01 Payment

206 Structure Excavation
- 206.03.03 Safety Requirements and Regulations

208 Trench Excavation and Backfill

208.01.01 General
ADD THE FOLLOWING TO THIS SUBSECTION
Due to the likelihood of encountering groundwater during excavation, monitoring wells should be installed along the project alignment prior to construction. The newly installed monitoring wells and existing wells can aid in the identification of how groundwater will affect construction of the project. The Geotechnical Engineer should be notified if variations are noted during or prior to construction.

208.02.05 Type II Aggregate Base Backfill
ADD THE FOLLOWING TO THIS SUBSECTION
Type II aggregate base backfill shall be 10 to 15 percent fines (material passing the No. 200 sieve or other material that is not susceptible to piping).

208.03.01 Trench Excavation, General
ADD THE FOLLOWING TO THIS SUBSECTION
Trenching and shoring operations shall be conducted in accordance with Section 10 Nos. 1926.650 through 1926.652 of the State of Nevada Occupational Safety and Health Standards for the Construction Industry (with amendments as of August, 1991) and in accordance with 29 CFR Part 1926, Occupational Safety and Health Standards – Excavations; Final Rule (October 31, 1989). Safety of construction personnel is the responsibility of the contractor.

In cases where groundwater is encountered of relatively small amounts seepage in excavations can be controlled with pumps. In cases of significant groundwater flows or in locations where excavations penetrate groundwater to significant depth, dewatering techniques such as well points may be required to control the ground water. Dewatering methods will be required as necessary, so that excavation and placement of the box culvert, box culvert bedding and backfill materials are completed in the dry. The dewatering system should be designed, installed and operated by contractors skilled and experienced in this field. The dewatering system should be capable of maintaining the water level at least 3 feet below trench base level during excavation, box culvert installation/rehabilitation, and backfill of the trench. Dewatering should be continuous until the groundwater can be allowed to rise in accordance with the following:

- No box culvert placed in water and the water is not allowed to rise over them until any concrete or mortar has set for a minimum of 8 hours:
- Groundwater is not allowed to rise unequally against any walls until design strength is achieved, or for a period of 28 days, whichever occurs first; and
- Groundwater is not allowed to rise in box culvert trenches or drained excavations until box culvert or structures are backfilled or restrained to prevent flotation.

208.03.09 Trench Backfill
ADD THE FOLLOWING TO THIS SUBSECTION
Type II aggregate base backfill shall be used along the entire portions of the project alignment where groundwater is encountered. The use of Type II backfill material can be terminated at an elevation corresponding to 2 feet above the highest observed groundwater level.

208.03.16 Drain Backfill

ADD THE FOLLOWING TO THIS SUBSECTION

In the section of the trenches where ground water occurs higher than the bottom of the trench, cross dams will be required at the bottom of the trench at intervals not exceeding 500 feet. The dams should be constructed of unreinforced concrete with a minimum thickness of 6 inches, and should extend the full width of the trench. The bottom of the dams should be established below the box culvert bedding material, and the top should be to a height equal to the established ground water level or the invert of the box culvert, whichever is greater.

208.04.01 Measurement

208.05.01 Payment

302 Aggregate Base Courses

302.03.01 Subgrade Preparation
302.05.01 Payment

401 Plantmix Bituminous Pavement – General

401.02.01 Composition of Mixtures
401.02.05 Field Compaction and Mix Design Correlation (type 2 mix designs only)
401.02.06 Asphalt Pavement Analyzer
401.03.12 Acceptance Sampling and Testing of IN-Place Bituminous Mixture

402 Plantmix Bituminous Surface

402.01.01 General
402.03.02 Spreading and Finishing
402.03.03 Surface Tolerances
402.04.01 Measurement
402.05.01 Payment

403-2 Plantmix Bituminous Gap-Graded Surface

403-2.01 General
403-2.02 General (Materials)
403-2.02.01 Composition of Gap-Graded (UTACS) Mixture
403-2.02.02 Polymer Modified Membrane
403-2.03.01 General (Construction)
403-2.03.02 Gap-Graded UTACS Paving Equipment
403-2.03.03 Application of Gap-Graded UTACS Surface
403-2.03.04 Surface Preparation for UTACS
403-2.03.05 Joints
403-2.03.06 Quality Control Aspects
403-2.03.07 Surface Tolerances for UTACS
403-2.03.08 UTACS Pavement Repairs
403-2.04.01 Measurement
403-2.05.01 Payment

405 Tack Coat

405.03.04 Application of Asphaltic Emulsion
405.05.01 Payment

406 Prime Coat

406.02.01 Bituminous Material
406.03.04 Application of Bituminous Material
406.05.01 Payment

407 Seal Coat

407.01.01 General
407.03.04 Application of Bituminous Material
407.05.01 Payment

502 Concrete Structures

502.02.01 General
502.03.20 Precast Concrete Box Culverts
502.04.01 Measurement
502.05.01 Payment

505 Reinforcing Steel

505.04.01 Measurement
505.05.01 Payment

603 Reinforced Concrete Pipe

603.03.03 Laying Culvert Pipe
603.03.04 Rubber Gasketed Joints
603.04.01 Measurement
603.05.01 Payment

605 Thermoplastic Pipe Culverts

605.03.01 General
605.05.01 Payment

609 Catch Basins, Manholes, and Inlets

609.02.01 General
609.03.02 Adjusting Catch Basin, Manhole and Inlet Covers
609.04.01 Measurement
609.05.01 Payment

613 Concrete Curb, Walk, Gutters, Driveways and Alley Intersections

613.02.01 General
613.03.01 General (Construction)
613.04.01 Measurement
613.05.01 Payment

622 Construction Surveying By the Contractor

622.01.01 General
622.03.01 General (Construction)
622.03.02 Final Accuracy
622.04.01 Measurement
622.05.01 Payment

623 Traffic Signals and Street Lighting

- 623G.01.01 General
- 623G.01.02 Regulations and Code
- 623G.02.01 Conduit
- 623G.02.03 Expansion Fittings
- 623G.02.04 Conductors and Cable
- 623G.03.01 Maintenance of Existing and Temporary Electrical Systems
- 623G.03.06 Foundations
- 623G.03.07 Wiring
- 623G.03.08 Service
- 623T.02.01 Cabinets Enclosure
- 623T.02.02 Cabinet Equipment
- 623T.02.03 Traffic Signal Controllers
- 623T.02.04 Loop Detectors
- 623T.02.05 Vehicle Signal Faces
- 623T.02.07 Pedestrian Signal Faces
- 623T.02.09 Standards, Steel Pedestals, and Posts
- 623T.02.13 Traffic Signal Video Image Detection Systems
- 623T.03.01 Painting
- 623L.02.01 Materials, Standards and Posts
- 623L.02.03 Lighting Luminaries
- 623.04.01 Measurement
- 623.05.01 Payment

624 Accommodations for Public Traffic

- 624.01.01 General
- 624.01.02 Traffic Control Plan (TCP)
- 624.03.01 Special Detours
- 624.03.02 Flaggers
- 624.03.06 Temporary Striping
- 624.03.08 Pipe Laying Operations
- 624.03.09 Traffic Control Devises
- 624.03.10 Traffic Control Supervisor
- 624.03.11 Required Protection for Open Excavation Duriing Working and Non-Working Hours
- 624.03.12 Special Traffic Control or Access Issues
- 624.04.01 Measurement
- 624.05.01 Payment

625 Construction Signs

- 625.02.03 Retro-Reflectivity

626 Final Cleanup

- 626.04.01 Measurement
- 626.05.01 Payment

627 Permanent Signs

- 627.01.01 General
- 627.04.01 Measurement
- 627.05.01 Payment

628 Painting Traffic Striping, Pavement Markings, and Curb Markings
- 628.01.01 General
- 628.03.09 Installation of Retro-Reflective Preformed Pavement Markings
- 628.04.01 Measurement
- 628.05.01 Payment

629 Water Distribution Facilities
- 629.01.02 Standards
- 629.03.01 General
- 629.03.17 Test Station Relocation
- 629.04.01 Measurement
- 629.05.01 Payment

630 Sanitary Sewer
- 630.03.16 Manhole Grade Ring Adjustment
- 630.04.01 Measurement
- 630.05.01 Payment

631 Street Name Signs
- 631.01.01 General
- 631.04.01 Measurement
- 631.05.01 Payment

633 Pavement Markers
- 633.02.02 Reflective Pavement Markers
- 633.04.01 Measurement
- 633.05.01 Payment

637 Pollution Control
- 637.01.01 General
- 637.04.01 Measurement
- 637.05.01 Payment

670 Construction Conflicts and Contingencies

703 Bituminous Materials
- 703.03.02 Asphalt Cements
  - 703.03.02.01 Superpave (performance Graded) Asphalt Binder
  - 703.03.02.02 In-Place Asphalt Cement Requirements
  - 703.03.04.01 Polymer Modified Emulsion Membrane (PMM)

705 Aggregates for Bituminous Courses
- 705.03.02 Plantmix Roadmix Asphalt Concrete Surface Course Types "S1 through S3"

714 Paint and Pavement Markings
- 714.03.06 Thermoplastic Paint and Pavement Markings

6.0 EXISTING UTILITIES

All major existing utilities found along the Vegas Drive corridor are shown in the Storm Drain Plan & Profile sheets as determined from research of existing record drawings and field survey by the PDG survey crew. Major underground existing utilities crossing the proposed storm drain facilities have been potholed. The pothole results are shown in Table 6.1 – Pothole Schedule and have been incorporated in the Storm Drain Plan and Profile sheets. A detailed utility conflict schedule was prepared to identify expected conflicts & resolutions with proposed storm drain improvements. See **Figure 6.1 – Vegas Drive Improvements Utility Conflict Schedule**, following Table 6.1.

Table 6.1 – Pothole Schedule

No.	Test Hole No.	"VD" Station	Left (ft)	Right (ft)	Utility Depth (ft)	Surface Elevation	Utility Top Elevation	Utility
1	1	10+09.47		25.82	2.91	2241.72	**2238.81**	*Water Line 8"*
2	2	10+19.23		26.48	9.83	2241.66	**2231.83**	*Water Line 39"*
3	3	32+48.96		23.34	2.69	2210.44	**2207.75**	*Electric 3 - 4"*
4	4	35+88.77		24.01	2.35	2203.74	**2201.39**	*Gas Line 4"*
5	5	36+79.65		23.42	3.80	2202.94	**2199.14**	*Water Line 8"*
6	6	37+19.38		21.76	1.87	2202.78	**2200.91**	*Traffic Signal / Street Light 3 - 2"*
7	7	36+83.41	26.48		4.99	2203.06	**2198.07**	*Water Line 12"*
8	8	36+83.68	30.84		2.95	2203.00	**2200.05**	*Gas Line 4"*
9	9	43+47.97		20.2	3.80	2197.79	**2193.99**	*Water Line 12"*
10	11	44+38.95		20.12	4.28	2197.20	**2192.92**	*Sanitary Sewer 4"*
11	12	51+57.86		20.23	3.93	2187.41	**2183.48**	*Water Line 6"*
12	13	84+88.83		15.42	5.06	2137.47	**2132.41**	*Water Line 16"*
13	14	86+68.83	17.05		3.77	2131.93	**2128.16**	*Water Line 16"*
14	15	86+54.15		5.49	3.26	2132.89	**2129.63**	*Gas Line 4"*
15	2-1	76+63.70		12.14	3.10	2151.56	**2148.46**	*Gas Line 4"*
16	2-3	77+19.27		27.04	5.38	2149.93	**2144.55**	*Gas Line 8"*
17	2-4	83+43.91		12.04	3.38	2141.25	**2137.87**	*Gas Line 4"*
18	2-5	83+51.39		26.75	5.20	2140.82	**2135.62**	*Gas Line 8"*
19	2-6	85+21.02		11.52	2.33	2136.54	**2134.21**	*Gas Line 2"*
20	2-8	87+62.00		27.17	4.85	2128.81	**2123.96**	*Gas Line 8"*
21	2-9	88+44.32		5.9	3.21	2126.05	**2122.84**	*Gas Line 4"*
22	2-10	89+68.44	18.5		3.91	2122.30	**2118.39**	*Water Line 16"*
23	2-11	90+15.81	24.99		4.19	2121.47	**2117.28**	*Water Line 8"*
24	2-12	90+97.85	18.95		4.00	2120.81	**2116.81**	*Water Line 16"*
25	Az-1	6+6.89	13.68		3.10	2247.08	**2243.98**	*Gas Line 4"*
26	Az-2	6+05.95	24.7		3.70	2246.83	**2243.13**	*Gas Line 4"*
27	Az-3	6+15		25±	2.30	2246.8±	**2244.5±**	*Water Line 8"*
28	Az-4	9+75.51	32.75		2.50	2241.38	**2238.88**	*Gas Line 4"*

Table 6.1 – Pothole Schedule (continued)

No	Test Hole No.	"VD" Station	Left (ft)	Right (ft)	Utility Depth (ft)	Surface Elevation	Utility Top Elevation	Utility
29	Az-5	10+24.11		21.1	4.00	2241.51	**2237.51**	*Gas Line 4"*
30	Az-6	12+82.11	21.99		2.90	2237.95	**2235.05**	*Gas Line 4"*
31	Az-7	12+95.20		5.93	3.20	2238.20	**2235.00**	*Water Line 8"*
32	Az-8	27+88.74		59.48	3.15	2217.27	**2214.12**	*Gas Line 2"*
33	Az-9	28+16.46		63.85	3.80	2217.09	**2213.29**	*Water Line 8"*
34	Az-10	29+32.19	20.1		3.00	2215.74	**2212.74**	*Gas Line 4"*
35	Az-11	29+30.92		4.59	3.50	2215.86	**2212.36**	*Water Line 8"*
36	Az-12	40+90.19	26.03		3.70	2199.75	**2196.05**	*Water Line 12"*
37	Az-13	40+90.13	16.26		3.40	2200.06	**2196.66**	*Gas Line 4"*
38	Az-14	40+89.48		26.95	4.30	2199.88	**2195.58**	*Gas Line 8"*
39	Az-15	51+97.18	26.17		3.60	2186.40	**2182.80**	*Water Line 12"*
40	Az-16	51+97.78	16.56		2.70	2186.64	**2183.94**	*Gas Line 4"*
41	Az-17	51+96.74		28.11	5.85	2186.36	**2180.51**	*Gas Line 8"*
42	Az-18	72+45.20	25.36		3.60	2159.23	**2155.63**	*Water Line 12"*
43	Az-19	72+44.91	16.66		3.30	2159.46	**2156.16**	*Gas Line 4"*
44	Az-20	84+44.24	14.57		4.70	2138.21	**2133.51**	*Water Line 12"*
45	Az-21	6+51.24	628.07		3.10	2242.32	**2239.22**	*Water Line 8"*
46	Az-22	35+90.04		182.93	1.70	2204.58	**2202.88**	*Gas Line 4"*
47	Az-23	8+10.33		40.3	3.10	2244.71	**2241.61**	*Sanitary Sewer 4"*
48	Az-26	12+60.14	22.26		4.20	2238.14	**2233.94**	*Concrete Encased Water Line 6"*
49	Az-27	72+45.63		27.16	3.80	2159.16	**2155.36**	*Gas Line 8"*
50	Az-28	35+55.34		4.68	4.40	2204.43	**2200.03**	*Water Line 8"*
51	Az-29	36+15.72		22.37	13.00	2203.33	**2190.33**	*Water Line 30"*
52	Az-30	36+93.76		14.45	7.10	2203.18	**2196.08**	*Water Line 12"*