

**OAKVALE ROAD REALIGNMENT AND
IMPROVEMENT PROJECT**

Final Initial Study and Mitigated Negative Declaration

APPENDICES B, C, AND D

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March 2015

Appendix B

30% Design Memorandum

(Selections have been omitted due to large volume)

DRAFT

OAKVALE ROAD REALIGNMENT

30% Design Memorandum

B&V PROJECT NO. 177740

PREPARED FOR

City of Escondido

20 MARCH 2014

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APPENDIX

- A. Preliminary Drawings
- B. Rock Stability Calculations
- C. Drainage Study
- D. County Permits

ACRONYMS AND ABBREVIATIONS

AB	aggregate base
AC	asphalt concrete
City	City of Escondido
Conc	concentrations
County	County of San Diego
EM	Engineering Manual
FERC	Federal Energy Regulatory Commission
H	horizontal
Max	maximum
Memo	memorandum
NAD	North American Datum
NGVD	National Geodetic Vertical Datum
PDR	Preliminary Design Report
RCC	Roller compacted concrete
RCP	reinforced concrete pipe
TBD	to be determined
V	vertical

LIST OF SYMBOLS

%	percent
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LIST OF UNITS

psi	pounds per square inch
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1. General

1.1 PURPOSE

The purpose of this memorandum is to describe the preliminary design and to serve as a step forward in completing the detailed design of the Oakvale Road Realignment Project. Information is included to describe the road alignment, rock cut stability, constructability, drainage, and permitting requirements.

1.2 PROJECT BACKGROUND

The Oakvale Road Realignment will be constructed to allow for the future replacement of Lake Wohlford Dam. Oakvale Road is a rural residential road with one 12 foot traffic lane each way with 2 ft paved shoulders.

1.3 PROJECT DESCRIPTION

The Oakvale Road realignment shifts the road further to the south, eliminating the tight "s-curve" currently in the road. Oakvale Road is owned by the County of San Diego (County). Therefore, the City of Escondido (City) will be required to coordinate with the County for realignment of the road. The proposed alignment will require significant rock cut slopes of approximately 0.75H:1V with slope heights up to approximately 110 feet. It is anticipated that the side slopes of the road cut will require slope stabilization as presented within this memo.

1.4 PROJECT DATUM

Historical documents for the construction of the road are based on the National Geodetic Vertical Datum 1929 (NGVD 29). The aerial survey has been completed for the project using NGVD 29. North American Datum 1983 (NAD 83) will be utilized for horizontal control. This project will use the same datum's during the design phase and construction phases.

- Vertical Datum: National Geodetic Vertical Datum 1929 (NGVD 29)
- Horizontal Datum: North American Datum 1983 (NAD 83)

The designers used the following survey to develop their basemaps:

- Aerial Survey – As prepared by Golden Triangle Land Survey, Inc. – December 2012
- Field Survey – As prepared by Golden Triangle Land Survey, Inc. – March 2013

2. Oakvale Road Realignment Analyses

2.1 INTRODUCTION

2.1.1 Scope

The following sections describe the road design, rock cut stability, constructability, drainage, and permitting requirements. The 30% design drawings are included in Appendix A.

2.1.2 Applicable Codes and Standards

- County of San Diego Road Standards
- San Diego County Drainage Design Standards
- Caltrans Highway Design Manual
- California Manual on Uniform Traffic Control Devices

2.1.3 References

2. Kleinfelder, “Geotechnical Data Report, Amendment No.1”, Wohlford Dam Replacement Project, prepared for the City of Escondido, 2013.
3. Caltrans Geotechnical Manual, “Rock Cut Slope Design”, 2013.
4. Black & Veatch, “Technical Memorandum – Spectral Matching, Revision 0”, 2014.
5. US Army Corps of Engineers, Rock Reinforcement, “EM 1110-1-2907”, 1980.
6. US Army Corps of Engineers, Rock Foundations, “EM 1110-1-2908”, 1994.
7. Hoek, E., “Practical Rock Engineering”, 2007.
8. Williams Form Engineering Corp., “Ground System Engineering, No. 112”, 2014.

2.2 ROAD DESIGN

2.2.1 Design Criteria

The roadway design will be developed to meet the requirements of the County of San Diego and City of Escondido. Design criteria for the new roadway section are presented in Table 2-1, below.

Table 2-1 Oakvale Road Design Criteria

DESIGN CRITERIA	REQUIREMENT	BASIS
Design width	28' surfacing width with 3' bench on the downhill side	Surfacing width per County of San Diego Rural Residential Road. 3' bench requested by County staff. Design Exception Request.
Design traffic load (e.g., traffic index or ESALs)	Traffic Index of 4.5	County of San Diego Residential Street
Roadway geometry	200' minimum radius 15% maximum grade	County of San Diego Rural Residential Road
Road Section	3"AC/6"AB	County of San Diego Residential Street minimum or as determined based on R-value test.
Drainage design		County of San Diego Design Standards
Pavement stripping	Double Yellow Centerline with White Edge Lines	California Manual on Uniform Traffic Control Devices
Required signage	TBD	County of San Diego Rural Residential Road

2.2.2 Road Alignment

The proposed Oakvale Road Alignment is located to the south of the existing road, outside of the area of work for the future dam replacement. The elevation of the road was set to allow for the expected future excavation for the dam to proceed without undermining the newly built road while minimizing the excavation to the south.

2.3 ROCK CUT STABILITY

The proposed alignment uses side slopes of approximately 0.75H:1V with slope heights up to approximately 110 feet. Based on the rock strength and discontinuity data collected from the closest geotechnical investigation location (i.e., boring KB-304), the side slopes of the road cut will require slope stabilization measures such as rock bolts with a wire mesh facing. Excavation at a shallower slope (e.g., 1H:1V) would result in less potential slope stabilization, but would require substantially more rock excavation for the road cut. The initial slope stabilization calculations are included in Appendix B.

A preliminary kinematic evaluation using the available data has been performed. This data includes geostructural mapping along Oakvale Road and the left abutment of the new Lake Wohlford Dam, rock cores obtained from a boring completed near the left abutment (i.e., KB-304), laboratory testing on rock cores and discontinuities from borings completed at Lake Wohlford Dam, and geostructural data from downhole televiewer surveys completed in boring KB-304. The kinematic evaluation considered a 0.75H:1V slope oriented with a dip azimuth of 0 and 54 degrees to consider the general conditions along the northwestern and southeastern portions of the road cut. Friction angles used to represent the shear strength along the discontinuities ranged from 30 to 40 degrees based on the results of direct shear tests on samples collected from the new Lake Wohlford Dam.

The kinematic evaluation indicates toppling failure of the rock slope is unlikely. Wedge failures are also less likely, but cannot be eliminated due to the range of potential dips observed in the discontinuities and the presence of a smaller number of unfavorable discontinuities that were mainly observed during geostructural mapping. Figure 2-1 presents an example of the kinematic wedge analysis that shows potential lines of discontinuity intersects (red dots) that are within or near the failure region (bold red polygon line) when a friction angle of 30 degrees is considered. Increasing the friction angle to 40 degrees (thin red polygon) decreases the likelihood for wedge failure significantly.

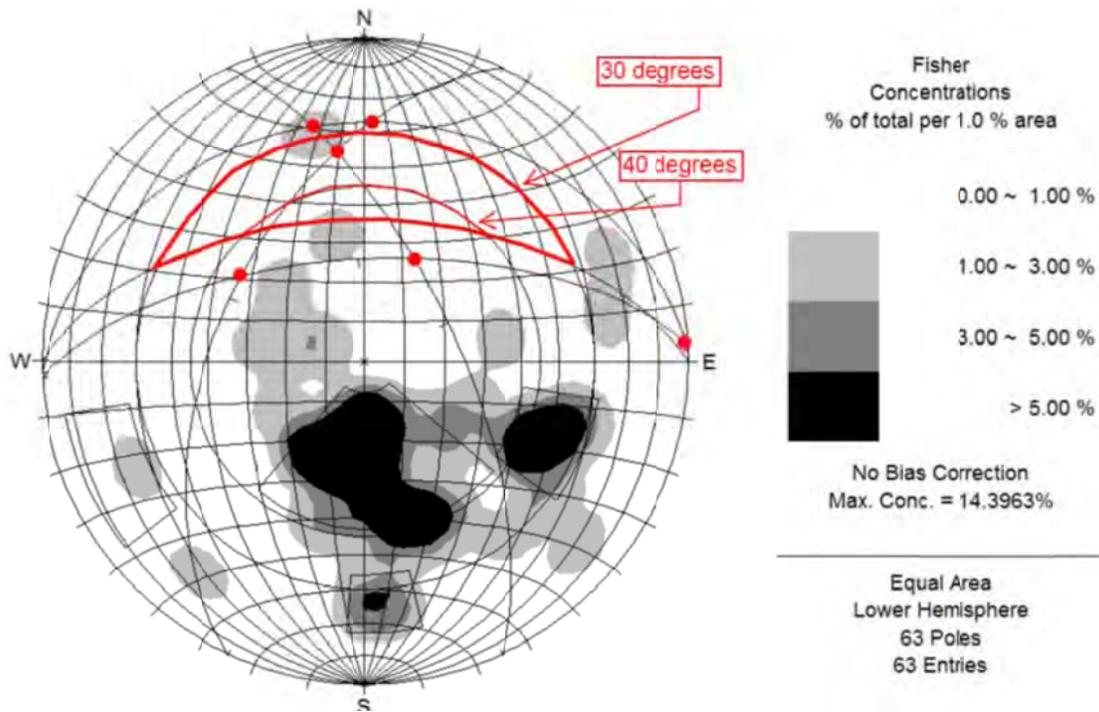


Figure 2-1. Wedge Failure Kinematic Analysis for Oakvale Road Cut Slope Considering Friction Angles of 30 and 40 Degrees along the Discontinuities

Planar failure of the rock slope is likely for friction angles of 30 and 40 degrees due to the presence of a large concentration of discontinuities with dip azimuths that daylight. Figure 2-2 presents an example of the kinematic planar failure analysis that shows a large concentration of discontinuities that may potentially daylight and result in planar failure of the road cut.

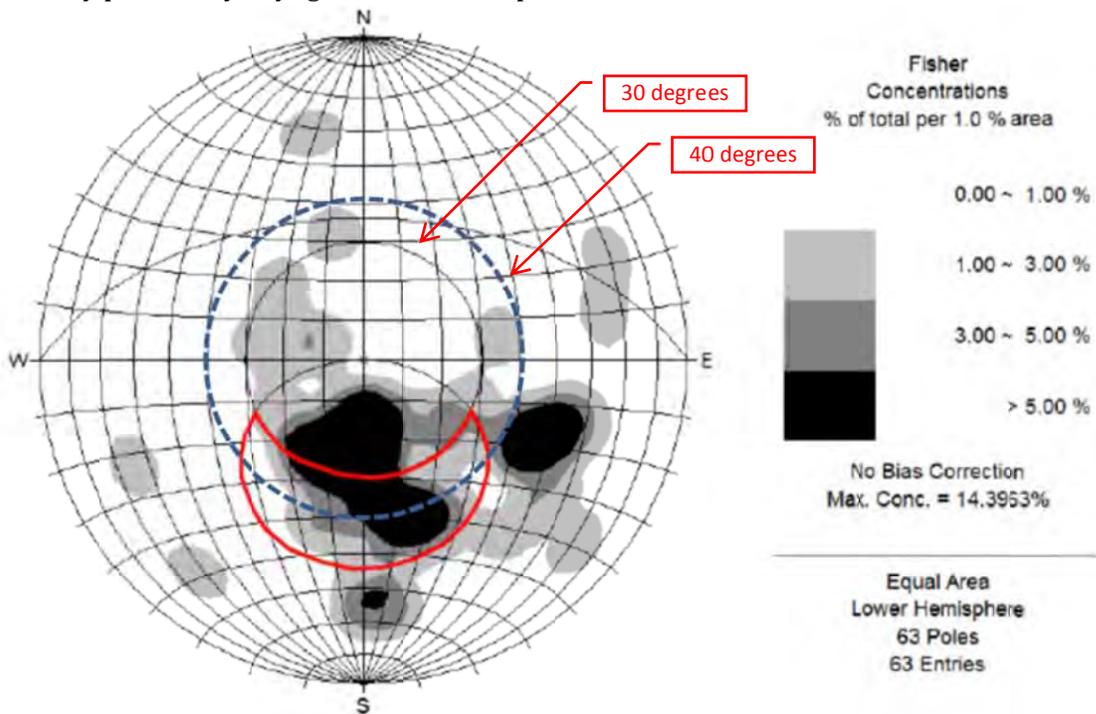


Figure 2-2. Planar Failure Kinematic Analysis for Oakvale Road Cut Slope Considering Friction Angles of 30 and 40 Degrees along the Discontinuities

Initial calculations of a factor of safety for the 0.75H:1V side slope indicate a value less than 1.5 for static conditions at the tallest portion of the slope (110 feet high). The factor of safety calculation followed the methods in U.S. Army Corps of Engineers Engineering Manual (EM) 1110-1-2907 with and without tension cracking. This factor of safety assumed a Mohr-Coulomb failure criterion where the average final values from the direct shear tests were used. A friction angle of 30 degrees was used with a shear intercept (cohesion) of 5 psi. Without tension cracking, the slope was considered unsaturated. The tension crack was assumed to be about 1/3 filled with water, when present, and has saturated the potential slip surface to represent unfavorable infiltration and drainage conditions. The low factor of safety led to a preliminary rock bolt design to provide about 137 kips of reinforcement to the 110 foot slope with a grid spacing of about 11.5 feet.

Rock bolts are used to stabilize the rock mass and prevent movement along discontinuities that may decrease the shear resistance in portions of the rock cut where the slope face orientation is unfavorable to the discontinuities. This preliminary rock bolt design followed the methods in U.S. Army Corps of Engineers EM 1110-1-2908 and considers potential failure of the rock mass (anchor depth), failure between the rock and grout/anchor (bond length), and failure of the tensioned reinforcement (strength of reinforcement bar or strands).

The high measured rock strength (minimum unconfined compressive strength of about 22,750 psi) results in a small required bond length that is less than the recommended minimum of 10 feet in EM 1110-1-2908. This minimum bond length prevents the grouted portion of the rock bolt from failing to reach the required anchor load. The anchor depth, which prevents failure of the rock mass under the required anchor load, is also less than 10 feet. The working capacity for each rock bolt was assumed to be 140 kips (rounded up from the required load of 137 kips). A 150 ksi all-thread-rod with a 1-3/8 inch diameter has an ultimate tensile strength of 237 kips. Sixty percent of this ultimate strength is 142 kips and will provide adequate strength for the working capacity.

Consideration of the road cut geometry indicates that up to 30 feet of the rock bolt will be in the failure mass (free-stress length) and that the total rock bolt length will be around 40 feet (free-stress length plus the bond length of 10 feet). This length will decrease as the rock bolts are installed further down the rock face due to a decrease in the free-stress length. Assuming slope heights over 30 feet tall require longer rock bolts and slope heights less than 30 feet tall require shorter rock bolts, a total of about 19,000 feet of rock bolts may be needed to stabilize the 0.75H:1V side slopes of the road cut.

2.3.1 Design Criteria

The rock cut slope design follows the CalTrans Geotechnical Manual. Preliminary design criteria and results are identified in this section based on extrapolation of the existing site data. The project contract documents for the rock cut stability will include a performance specification with detailed information for the Contractor to develop his design and modify the design based on conditions encountered in the field. Final design of the road cut by a contractor will consider completion of additional rock borings and pull-out tests to confirm or modify the preliminary design. Corrosion potential in the road cut will also be considered during final design.

2.4 CONSTRUCTABILITY

As described in Section 2.3 of this memo, rock excavation will be required for slopes of approximately 0.75H:1V with heights up to approximately 110 feet. A combination of blasting and

rippable excavation will be required; however, the amount of rippable rock is anticipated to be variable and limited to about the upper 10 to 40 feet of the excavation for a D9 tractor with a multi-or single shank based on compression wave velocity measurements in the area. Controlled blasting (pre-splitting or line drilling) should be used to excavate the final vertical face, control damage to the rock mass, and limit backbreak. Rock reinforcement (rock bolts) will be installed along the final excavation slopes as the excavation progresses to enable work to proceed below the high rock cuts. Drilling equipment will be required for controlled blasting and installation of rock reinforcement.

All excavated materials will be hauled offsite for disposal. In an effort to expedite the excavation, 10 hour (7 am to 5 pm) work days are anticipated. Work will be limited to a normal five day a week schedule, with weekend work allowed per City/County approval on a case by case basis. The traffic study is currently being completed and based upon its outcome additional requirements will be included within the project specifications.

Traffic Control on Oakvale Road is anticipated to be minimal, constrained to the transitions from the existing road to the new road. One lane of traffic will be open at all times with flaggers as needed. During the transition construction from the existing road to the new road, phasing will be required. Access to the residents located to the east on Oakvale Road will be maintained with one lane open at all times. Phasing details will be developed as part of the detailed design. Traffic control will be developed by the Contractor based on the California Manual on Uniform Traffic Control Devices.

2.5 DRAINAGE STUDY

A drainage study was prepared to support the construction plans for the re-alignment of Oakvale Road. Proposed improvements consist of 6-inch curb & gutter, asphalt concrete, an 18-inch storm drain, catch basin, headwall, rip-rap, and a brow ditch at the top of the proposed slope. Since the realignment section of the road remains the same from existing conditions there will be no net change in impervious surface. In addition, the drainage pattern for the proposed condition mimics the existing condition, in that ultimately all the stormwater flows towards Lake Wohlford in both conditions. The study was prepared to analyze the capacity of the existing culvert near the beginning of the improvements, to analyze the capacity of the proposed 18-inch RCP storm drain, and analyze the depth of flow in Oakvale Road during a 100 year storm event. See Appendix C for additional details.

2.6 PERMITTING REQUIREMENTS

The following permits are anticipated to be required for this project. The permit applications are included in Appendix D. The following permits are anticipated to be required for design approval:

- **Encroachment Permit:** The City will submit the application to encroach upon the County road.
- **Request for a Design Exception to a Road Standard:** The Request for a Design Exception will be submitted due to the reduced shoulder and parkway width. Based upon conversations with the County the proposed road realignment (12 ft lanes, 2 ft shoulders) will match the existing road.
- **Road Opening (Remandment):** The road opening notice will be submitted.
- **Grading Permit:** The grading plans and application will be submitted. The Contractor will obtain the permit during construction.

The following permits are anticipated to be required for Construction:

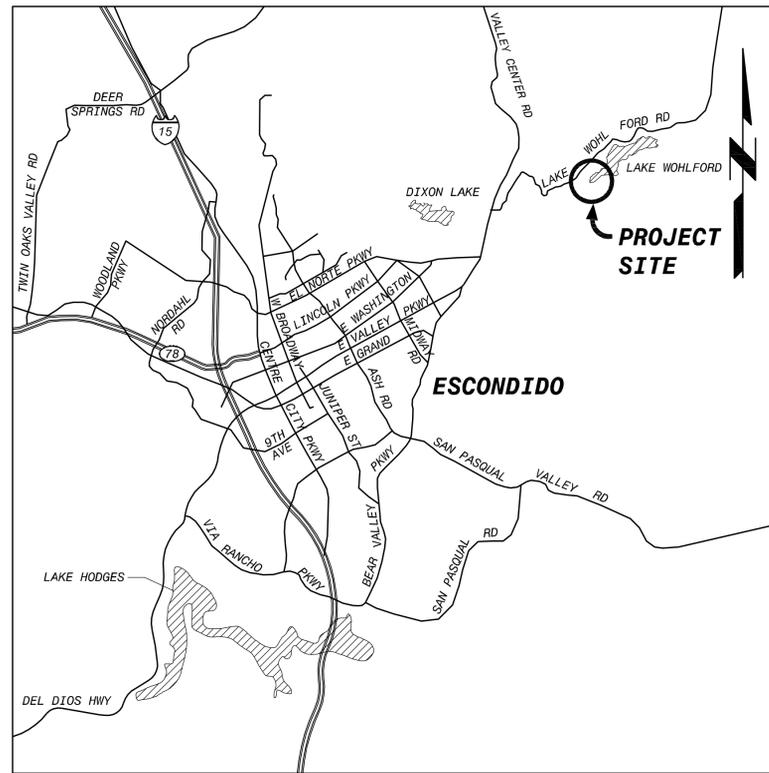
- **Traffic Control:** The Contractor will submit the application for the Traffic Control permit. Traffic Control Drawings will be developed as part of the project contract documents.
- **Grading Permit:** The Contractor will obtain the permit.

APPENDIX A – 30% DESIGN DRAWINGS

CITY OF ESCONDIDO

UTILITIES DEPARTMENT

OAKVALE ROAD REALIGNMENT PROJECT FOR PERMIT USE ONLY

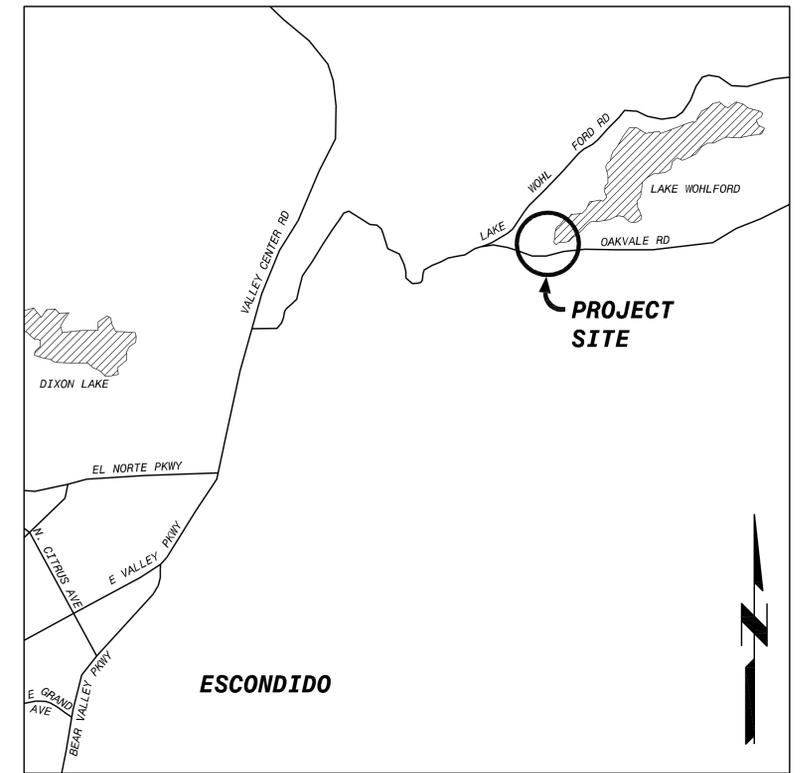


VICINITY MAP
NO SCALE



DRAWING INDEX

NO.	SHI	GENERAL
1	T-1	COVER SHEET, LOCATION MAPS, AND SHEET LIST
2	G-1	LEGEND AND ABBREVIATIONS
3	G-2	SURVEY AND GENERAL NOTES
CIVIL		
4	C-1	OVERALL SITE PLAN, STAGING AND SURVEY CONTROL
5	C-2	ROADWAY PLAN AND PROFILE
6	C-3	ROADWAY SECTIONS, DETAILS, AND NOTES
7	C-4	ROADWAY SECTIONS
8	C-5	ROADWAY SECTION AND DRAINAGE PROFILES
9	C-6	EROSION CONTROL PROFILES & REVEGETATION DETAILS
10	C-7	EROSION CONTROL & REVEGETATION
11	C-8	SLOPE STABILITY DETAILS



LOCATION MAP
NO SCALE

FOI# 28155 - 7/31/2014 2:33:44 PM
PW FLD# 177740 - 150-3140 Oakvale Rd Realignment

FOR PERMIT USE ONLY



OAKVALE ROAD
REALIGNMENT PROJECT

DRAWING NO.

T-1

CITY OF ESCONDIDO

GENERAL
COVER SHEET, LOCATION MAPS, AND SHEET LIST

Sheet 1 of 11



CONSTRUCTION RECORD	REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK	SCALE	Office	Designed By	Drawn By	Checked By	Submitted	
Contractor							NONUMENT 2179 NORTH 2005276.219 EAST 6531749.403 ELEVATION 1548.017 2" FROM P.T.R. WITH D.S.R. STAMPED "EGCS 1992 2179" ON TOP OF 4" DIRT SLAB ELEVATION 806.75 NVD 29 DATUM, US FEET	Horizontal NO SCALE	Filed	CTD	JEG	JWB	Plans Prepared Under Supervision Of	By
Inspector								Vertical NO SCALE	Traffic				Date	
Date Completed													R.C.E. No.	

Submitted _____
By _____
Deputy Director of Utilities

GENERAL LEGEND

-  SURFACED STREET, ROAD OR DRIVE
-  326 EXISTING GROUND CONTOUR
-  326 FINISH GRADE CONTOUR
-  NEW PIPING
-  R/W EXISTING RIGHT-OF-WAY LINE
-  OHE EXISTING OVERHEAD ELECTRICAL LINE
-  STREET LIGHT POLE
-  HEDGE, BRUSH, SHRUBS, WOODS
-  BROW DITCH
-  PVC STORM DRAIN
-  STORM DRAIN CATCH BASIN
-  CENTERLINE
-  SECTION NUMBER, DETAIL LETTER, OR DWG NO.
-  DRAWING NUMBER ON WHICH SECTION OR DETAIL APPEARS; OR WHERE SECTION IS CUT OR DETAIL IS NOTED
-  Ø DIAMETER
-  1676.0 SPOT ELEVATION
-  REMOVAL (PAVEMENT)
-  3.00% PROPOSED ROAD GRADE
-  (3.00%) EXISTING ROAD GRADE
-  ROCK CORE BORINGS
-  EXISTING ROCK CORE BORING
-  EXISTING SEISMIC REFRACTION

ABBREVIATIONS

- AB - AGGREGATE BASE
- AC - ASPHALT CEMENT
- AHD - AHEAD
- ASAP - AS SOON AS POSSIBLE
- BC - BEGIN CURVE
- BK - BACK
- BMP - BEST MANAGEMENT PRACTICES
- BVC - BEGIN VERTICAL CURVE
- CA - CALIFORNIA
- CB - CATCH BASIN
- CFS - CUBIC FEET PER SECOND
- CITY - CITY OF ESCONDIDO
- CL - CENTERLINE
- CLR - CLEAR
- CMP - CORRUGATED METAL PIPE
- CONC - CONCRETE
- CONN - CONNECTION
- CONT - CONTINUATION
- DBL - DOUBLE
- DEPT - DEPARTMENT
- DG - DISINTEGRATED GRANITE
- DI - DUCTILE IRON
- DIA - DIAMETER
- DWG, DWGS - DRAWING(S)
- E - EAST
- EA - EACH
- EC - END CURVE
- EF - EACH FACE
- EL - ELEVATION
- E/L - EASEMENT LINE
- EVC - END VERTICAL CURVE
- EW - EACH WAY
- EXP - EXPANSION
- EXIST, EX - EXISTING
- FF - FINISHED FLOOR
- FIN - FINISHED
- FL - FLOW LINE
- FPS - FEET PER SECOND MAX MAXIMUM
- FS - FINISHED SURFACE
- GB - GRADE BREAK
- GR - GROUND
- HMA - HOT MIX ASPHALT
- HORIZ - HORIZONTAL
- HP - HIGH POINT
- HPI - HORIZONTAL POINT OF INFLECTION
- HR - HOUR
- IE - INVERT ELEVATION
- IMP - IMPROVEMENTS
- L - LENGTH
- LF - LINEAR FEET
- LVC - LEFT VERTICAL CURVE
- MAX - MAXIMUM
- MIN - MINIMUM
- MMRP - MITIGATION MONITORING AND REPORTING PROGRAM
- MOD - MODIFICATION
- MTL - MATERIAL
- N - NORTH
- NO - NUMBER
- NPDES - NATIONAL POLLUTION DISCHARGE ELIMINATION SYSTEM
- NTS - NOT TO SCALE
- OC - ON CENTER
- OHE - OVERHEAD ELECTRIC
- OPR - OPERATOR
- PI - POINT OF INFLECTION
- PL - PLATE
- POR - PORTION
- PRECON - PRECONSTRUCTION
- PRV - PRESSURE REDUCING VALVE
- PVI - POINT OF VERTICAL INFLECTION
- PVMT - PAVEMENT
- R - RIGHT, RADIUS
- RCP - REINFORCED CONCRETE PIPE
- RD - ROAD
- RE - RESIDENT ENGINEER
- REC - RECORDED
- RECT - RECTANGULAR
- RED - REDUCER
- REQ'D - REQUIRED
- RET - RETAINING WALL
- RPM - RAISED PAVEMENT MARKER
- RS - ROAD SURVEY
- RT - RIGHT
- S - SLOPE
- SCH - SCHEDULE
- SD - STANDARD DRAWING, STORM DRAIN
- SDG&E - SAN DIEGO GAS & ELECTRIC COMPANY
- SDRSD - SAN DIEGO REGIONAL STANDARD DRAWINGS
- SF - SQUARE FEET
- SHT - SHEET
- SPEC - SPECIFICATION
- ST - STREET
- STA - STATION
- SSPWC - STANDARD SPECIFICATION FOR PUBLIC WORKS CONSTRUCTION
- STD - STANDARD
- STL - STEEL
- T, TEL - TELEPHONE, THICKNESS
- T/B - TOP AND BOTTOM
- TC - TOP OF CURB
- TEMP - TEMPORARY
- TG - TOP OF GRADE
- TI - TRAFFIC INDEX
- TOW, TW - TOP OF WALL
- TRANS - TRANSITION
- TYP - TYPICAL
- UNK - UNKNOWN
- UNO - UNLESS NOTED OTHERWISE
- VERT - VERTICAL
- VPI - VERTICAL POINT OF INFLECTION
- WJ - WITH

MATERIALS LEGEND

-  EXISTING EARTH OR GRADE
-  COMPACTED FILL OR NATIVE SOIL
-  EXISTING ROCK
-  CONCRETE
-  COMPACTED SUBGRADE
-  CLASS II AGGREGATE
-  AC PAVEMENT
-  RIP-RAP

REE01341_3/14/2014 6:45:05 AM
 PW FLDR: 17740...ISO-3007 - TempJates

FOR PERMIT USE ONLY



OAKVALE ROAD
REALIGNMENT PROJECT

DRAWING NO.
G-1

CITY OF ESCONDIDO



CONSTRUCTION RECORD		REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK	SCALE	Office	Designed By	Drawn By	Checked By	Submitted	
Contractor								MONUMENT 2179 NORTH 2005276.319 EAST 6531749.403 ELEVATION 1548.017 2" DRAW P/RE WITH DSDC STAMPED "EGCS 1992 2179" ON TOP OF 4" DIRT SLOPE ELEVATION 806.75 NVD 29 DATUM; US FEET	Horizontal	Filed	Plans Prepared Under Supervision Of			CTD TVL JWB	By _____
Inspector							Vertical		Traffic	Date		R.C.E. No.	Deputy Director of Utilities		
Date Completed							NO SCALE								

GENERAL
LEGEND AND ABBREVIATIONS

BASIS OF BEARINGS

NAD 83 (1992), CALIFORNIA COORDINATE SYSTEM, ZONE VI (CCS83.92)
 BASIS OF ELEVATIONS NGVD 29
 BEARING BETWEEN # 2213 TO # 2179 S 13° 32' 49" W; 3138.71 FEET
 SEE RECORD OF SURVEY 14236
 MONUMENT 2213
 N 2008327.605, E 6332483.674
 ELEVATION 1488.23
 2" IRON PIPE WITH DISC STAMPED "EGCS 1992 2213" AT EDGE OF ASPHALT ROAD

GENERAL NOTES:

1. THE PROJECT GENERAL CONDITIONS, SECTION A-00700, AND THE PROJECT SUPPLEMENTARY GENERAL CONDITIONS, SECTION A-00800, PROVIDE DETAILED CONTRACT CONDITIONS REGARDING THE CONTRACTOR'S AND THE CITY OF ESCONDIDO'S CONTRACT RESPONSIBILITIES.
2. ALL WORK TO BE PERFORMED IN ACCORDANCE WITH THE PROJECT PLANS, THE TECHNICAL SPECIFICATIONS, THE CITY OF ESCONDIDO DESIGN STANDARDS, AND CITY OF ESCONDIDO STANDARD DRAWINGS. THE SAN DIEGO REGIONAL STANDARD DRAWINGS, STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION (GREEN BOOK), CALTRANS STANDARD DRAWINGS, AND OTHER STANDARD DRAWINGS MAY ALSO BE REFERENCED. REFERENCE THE PROJECT GENERAL CONDITIONS ARTICLE 3 INTENT AND USE OF CONTRACT DOCUMENTS FOR DETAILS.
3. ALL CONTRACTORS WORKING IN THE CITY OF ESCONDIDO RIGHT-OF-WAY SHALL OBTAIN A SEPARATE CITY OF ESCONDIDO ENCROACHMENT PERMIT. REFERENCE THE PROJECT GENERAL CONDITIONS ARTICLE 6.16 PERMITS. FOR DETAILS, REFERENCE THE TECHNICAL SPECIFICATION 01530 TEMPORARY RIGHT-OF-WAY ENCROACHMENT. THE CITY OF ESCONDIDO ENCROACHMENT PERMIT REGULATES THE TEMPORARY REPAIR OF STREET SURFACES, AND OTHER ISSUES RELATED TO WORKING IN THE STREET.
4. THE PROJECT SUPPLEMENTARY GENERAL CONDITIONS LIST ADDITIONAL RIGHT-OF-WAY PERMITS THAT MAY BE REQUIRED FOR THIS PROJECT. THE CONDITIONS OF OTHER RIGHT-OF-WAY PERMITS APPLY TO THE CONTRACTOR. THE COUNTY OF SAN DIEGO CALLS THEM "EXCAVATION PERMITS" AND "TRAFFIC CONTROL PERMITS". BLASTING WILL REQUIRE A PERMIT FROM THE COUNTY OF SAN DIEGO.
5. CONTRACTOR SHALL NOTIFY THE UNDERGROUND SERVICE ALERT 811 AT LEAST 48 HOURS BEFORE STARTING WORK, BUT NO SOONER THAN 14 DAYS BEFORE STARTING WORK. REFERENCE PROJECT GENERAL CONDITIONS ARTICLES 4 & 6.28 FOR DETAILS. THE ENCROACHMENT PERMIT IS NOT VALID WITHOUT A DIG-ALERT TICKET NUMBER. IN OTHER WORDS: NO WORK INVOLVING ANY KIND OF DIGGING, OR EXCAVATION, MAY BE DONE WITHOUT A VALID DIG-ALERT TICKET NUMBER. THIS REGULATION IS PART OF THE STATE OF CALIFORNIA GOVERNMENT CODE (SECTION 4216), AND CANNOT BE WAIVED BY THE CITY OF ESCONDIDO.
6. THE CONTRACTOR SHALL PROVIDE CONTINUOUS ACCESS FOR EMERGENCY VEHICLES, RESIDENTS, AND UTILITY VEHICLES.
7. ALL WORK ON THIS PROJECT IS SUBJECT TO ENVIRONMENTAL REGULATION. REFERENCE THE CONTRACT SPECIFICATION SECTION 01560 TEMPORARY ENVIRONMENTAL CONTROLS & CONSTRAINTS. THIS SECTION REGULATES AIR POLLUTION INCLUDING DUST AND SMOKING EQUIPMENT. THE SECTION ALSO REGULATES STORM WATER POLLUTION PREVENTION PLANS (SWPPP), SWEEPING STREETS, TEMPORARY FIELD SANITATION, VISUAL POLLUTION, NOISE, STORAGE OF PRODUCTS ON THE JOB-SITE INCLUDING FUEL, CULTURAL RESOURCES, AND THE POTENTIAL FOR FINES AND CONTRACT SUSPENSION OR TERMINATION FOR NON-COMPLIANCE.
8. CONTACT NUMBERS:
 COUNTY OF SAN DIEGO DEPARTMENT OF PUBLIC WORKS (858) 694-2212
 COX CABLE TV (619) 262-1181
 AT&T TELEPHONE (866) 346-1168
 SAN DIEGO GAS & ELECTRICITY COMPANY (SDG&E) (800) 411-7343
9. TRAFFIC CONTROL PLANS WILL BE SUBMITTED TO AND REVIEWED BY THE COUNTY OF SAN DIEGO. THE PLANS SHALL CONFORM TO THE CALIFORNIA MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (CALIFORNIA MUTCD). CONTRACTOR DEVELOPED PLANS WILL NEED TO BE STAMPED BY A LICENSED CA ENGINEER. CONTRACTOR IS RESPONSIBLE FOR OBTAINING THE NECESSARY TRAFFIC CONTROL PERMITS FROM THE COUNTY OF SAN DIEGO.
10. EQUIPMENT STORAGE AND CONSTRUCTION STAGING SHALL BE LIMITED TO THE EXISTING RIGHT-OF-WAY AND THE LIMITS OF CLEANING AND GRUBBING AS SHOWN ON DWG C-1. ADDITIONAL STORAGE AND STAGING REQUIREMENTS SHALL BE COORDINATED BY THE CONTRACTOR.
11. CONSTRUCTION HAULING PROVISIONS ARE DESCRIBED IN THE WORK RESTRICTIONS & CONSTRAINTS SPECIFICATION.

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OAKVALE ROAD
 REALIGNMENT PROJECT

DRAWING NO.
G-2

CITY OF ESCONDIDO

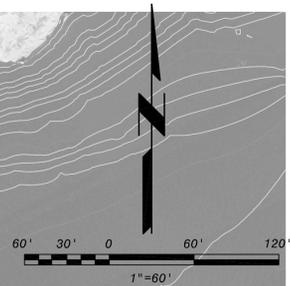
SURVEY AND GENERAL NOTES

Sheet 3 of 11

CONSTRUCTION RECORD	REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK	SCALE	Office	Designed By	Drawn By	Checked By
Contractor							MONUMENT 2179 NORTH 2005276.310 EAST 6331749.403 ELEVATION 1548.012	Horizontal	Filed	CTD	TVL	JWB
Inspector							2" IRON PIPE WITH DISC STAMPED "EGCS 1992 2179" ON TOP OF 4" DIRT SURF. ELEVATION 806.75	NO SCALE		Plans Prepared Under Supervision Of		
Date Completed							NDVD 29 (A) (M); US FEET	Vertical	Traffic			
								NO SCALE				

Submitted _____
 By _____
 Deputy Director of Utilities

Date _____
 R.C.E. No. _____



- NOTES:**
- FOR INFORMATION PURPOSES ONLY, THE CONTRACTOR CAN USE THE GEOTECHNICAL DATA REPORT (GDR) TO ESTABLISH PREVAILING GROUND CONDITIONS AS A BASELINE FOR THE ROCK BOLTING STABILIZATION DESIGN. BORING KB-304 AND SEISMIC REFRACTION LINE KSR3 ARE CLOSEST IN PROXIMITY TO THE ROAD EXCAVATION.
 - CONTRACTOR TO COORDINATE LOCATION OF NEW TELEPHONE POLES WITH AT&T. FINAL LOCATION TO BE DETERMINED.
 - ACCESS TO THE EXISTING DAM ACCESS ROAD AND DAM CREST SHALL BE MAINTAINED AT ALL TIMES.

SITE PLAN
1" = 60'-0"



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**OAKVALE ROAD
REALIGNMENT PROJECT**

DRAWING NO.
C-1

CITY OF ESCONDIDO

**CIVIL
OVERALL SITE PLAN, STAGING AND
SURVEY CONTROL**

Sheet 4 of 11

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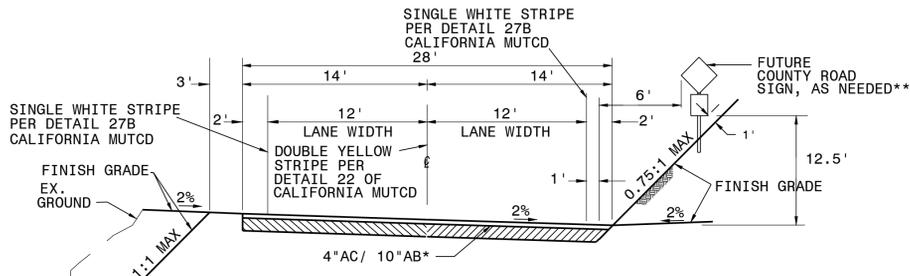
CONSTRUCTION RECORD	REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK
Contractor							NONUMENT 2179 NORTH 2005276.319 EAST 6531749.403 ELEVATION 1548.017 2" IRON PIPE WITH DISC STAMPED "EGCS 1992 2179" ON TOP OF 4" DIRT PIPE ELEVATION 806.75 NVD 29 DATUM, US FEET
Inspector							
Date Completed							

SCALE	Office	Designed By	Drawn By	Checked By
Horizontal 1"=60'	Filed	CTD	JEG	JWB
Vertical NO SCALE	Traffic	Plans Prepared Under Supervision Of		
			Date	R.C.E. No.

Submitted	
By	Deputy Director of Utilities

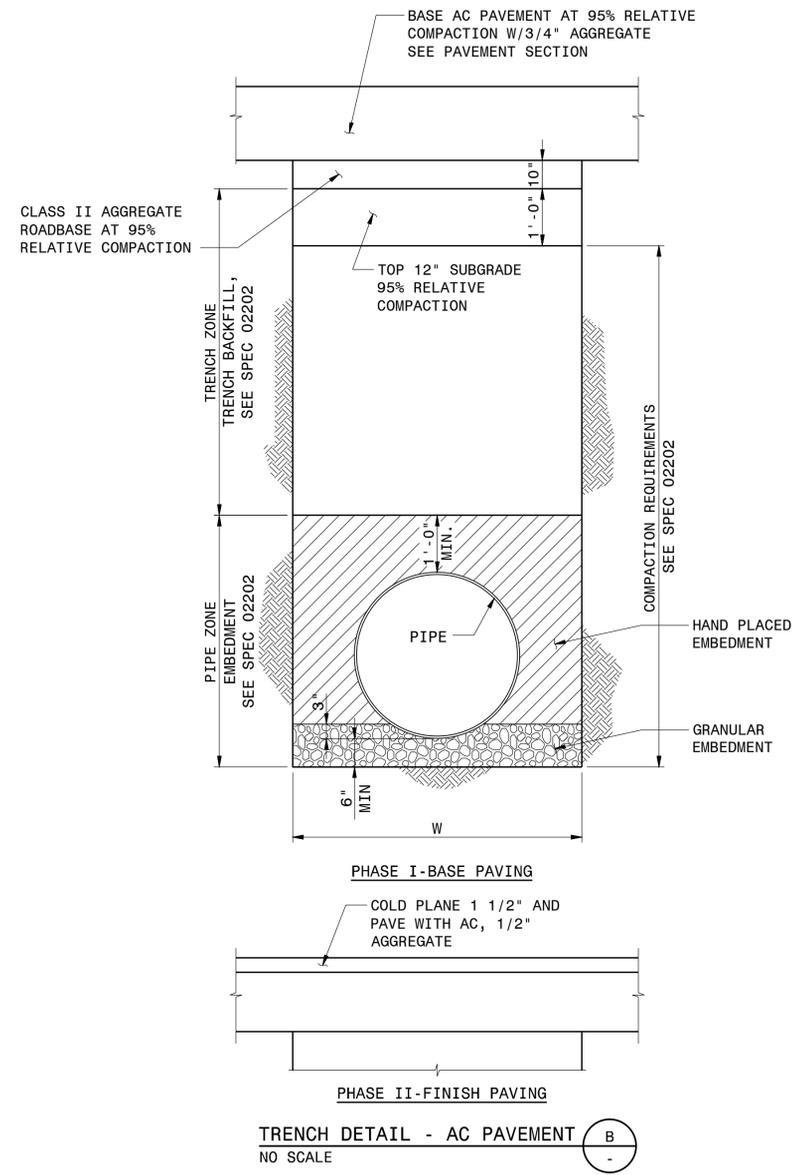
SIGNING AND STRIPING NOTES

1. THE CONTRACTOR IS RESPONSIBLE FOR INSTALLING ALL SIGNING AND STRIPING.
2. SIGNING, STRIPING AND PAVEMENT MARKINGS SHALL CONFORM TO THE CALIFORNIA MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (FHWA'S MUTCD 2003 REVISION 1, AS AMENDED FOR USE IN CALIFORNIA), CALTRANS STANDARD SPECIFICATIONS (JULY 2006 OR LATEST VERSION THEREOF), THESE PLANS AND THE SPECIAL PROVISIONS.
3. ALL SIGNING AND STRIPING IS SUBJECT TO THE APPROVAL OF THE CITY AND COUNTY PUBLIC WORKS DIRECTOR OR HIS REPRESENTATIVE, PRIOR TO INSTALLATION.
4. ANY DEVIATION FROM THESE SIGNING AND STRIPING PLANS SHALL BE APPROVED BY THE ENGINEER OF WORK AND CITY AND COUNTY PUBLIC WORKS DIRECTOR PRIOR TO ANY CHANGE IN THE FIELD.
5. ALL SIGNING AND STRIPING SHALL BE REFLECTIVE PER CALTRANS SPECIFICATIONS. STRIPING SHALL BE REPAINTED TWO WEEKS AFTER INITIAL PAINTING. SIGNING SHALL USE ENCAPSULATED LENS REFLECTIVE SHEETING (HIGH INTENSITY OR EQUAL).
6. PRIOR TO FINAL ACCEPTANCE OF STREET IMPROVEMENTS, ALL STREET STRIPING AND MARKINGS WITHIN A 500' PERIMETER OF THE CONSTRUCTION PROJECT WILL BE RESTORED TO A "LIKE NEW" CONDITION, IN A MANNER MEETING THE APPROVAL OF THE CITY INSPECTOR.
7. EXACT LOCATION OF STRIPING AND STOP LIMIT LINES SHALL BE APPROVED BY THE CITY AND COUNTY PUBLIC WORKS DIRECTOR OR HIS REPRESENTATIVE PRIOR TO INSTALLATION.
8. CONTRACTOR SHALL REMOVE ALL CONFLICTING PAINTED LINES, MARKINGS AND PAVEMENT LEGENDS BY GRINDING. DEBRIS SHALL BE PROMPTLY REMOVED BY THE CONTRACTOR.
9. ALL PAVEMENT LEGENDS SHALL BE THE LATEST VERSION OF THE CALTRANS METRIC STENCILS.
10. LIMIT LINES SHALL BE FIELD LOCATED.
11. ALL LIMIT LINES, PAVEMENT ARROWS AND PAVEMENT LEGENDS SHALL BE THERMOPLASTIC UNLESS OTHERWISE SPECIFIED.
12. ALL SIGNS SHALL BE STANDARD SIZE AS SHOWN IN THE CALIFORNIA MANUAL ON UNIFORM TRAFFIC CONTROL DEVICES (FHWA'S MUTCD 2003 REVISION 1, AS AMENDED FOR USE IN CALIFORNIA) UNLESS OTHERWISE SPECIFIED. SIGN TYPE, LOCATION & INSTALLATION OF SIGNS SHALL BE BY COUNTY.
13. SIGN POSTS SHALL BE SQUARE PERFORATED STEEL TUBING WITH BREAKAWAY BASE PER SAN DIEGO REGIONAL STANDARD DRAWING M-45.
14. WHEN A SIGN IS ATTACHED TO A POLE, IT SHALL BE MOUNTED USING A STANDARD COUNTY OF SAN DIEGO MOUNTING BRACKET WITH STRAPS.
15. EXISTING SIGNS REMOVED BY THE CONTRACTOR SHALL BE DELIVERED BY THE CONTRACTOR TO THE CITY OF ESCONDIDO.



TYPICAL SECTION OAKVALE RD.
NOT TO SCALE
TRAFFIC INDEX = 4.5

*THICKNESS SHALL BE VERIFIED BY THE R-VALUE TEST AND DETERMINED BY GEOTECHNICAL ENGINEER IN THE FIELD
**SIGN TYPE, LOCATION & INSTALLATION BY COUNTY



PHASE I - BASE PAVING
COLD PLANE 1 1/2" AND PAVE WITH AC, 1/2" AGGREGATE
PHASE II - FINISH PAVING
TRENCH DETAIL - AC PAVEMENT (B)
NO SCALE

NOTES:

1. UPON APPROVAL FROM THE CITY OF ESCONDIDO, THE CONTRACTOR SHALL PERFORM PHASE 1-BASE PAVING TO A THICKNESS EQUAL TO EXISTING PAVEMENT THICKNESS PLUS 1" AS SHOWN ON THE PLAN AND PROFILE DRAWINGS.
2. THE PAVEMENT REPAIR SHALL BE TRUE TO GRADE AND CROSS-SECTION AS DESCRIBED IN SECTION 302-5.6.2 SSPWC (GREEN BOOK).
3. THE CONTRACTOR SHALL RE-STRIP ANY LANE DELINEATION THAT HAS BEEN REMOVED OR DEGRADED BY THE WORK IMMEDIATELY OR SHALL INSTALL AND MAINTAIN TEMPORARY CHIP SEAL MARKERS UNTIL STRIPING IS PLACED. ALL RAISED PAVEMENT MARKERS (RPMs) SHALL BE REPLACED AFTER STRIPING.
4. COMPACTION TESTS ARE REQUIRED FOR ALL TRENCH BACKFILL, INCLUDING EMBEDMENT. COMPACTION FAILURE MUST BE CORRECTED IMMEDIATELY.
5. PAVEMENT MATERIAL AND SURFACING SHALL MATCH EXISTING.
6. PHASE II PAVING SHALL OCCUR AFTER FINAL TRANSITIONS ARE COMPLETED.
7. A TACK COAT OF ASPHALTIC EMULSION SHALL BE APPLIED TO THE EXISTING AC AT ALL CONTACT SURFACES TO PLACING NEW AC.
8. AC RESURFACING SHALL BE SEAL COATED WITH AN EMULSIFIED ASPHALT AND COVERED WITH SAND CHIP SEALING SHALL BE APPLIED AS REQUIRED BY THE COUNTY.



DETAIL- TRANSITION TO EXIST OAKVALE RD (A)
SCALE: 1"=20'

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OAKVALE ROAD REALIGNMENT PROJECT		DRAWING NO. C-3
CITY OF ESCONDIDO		



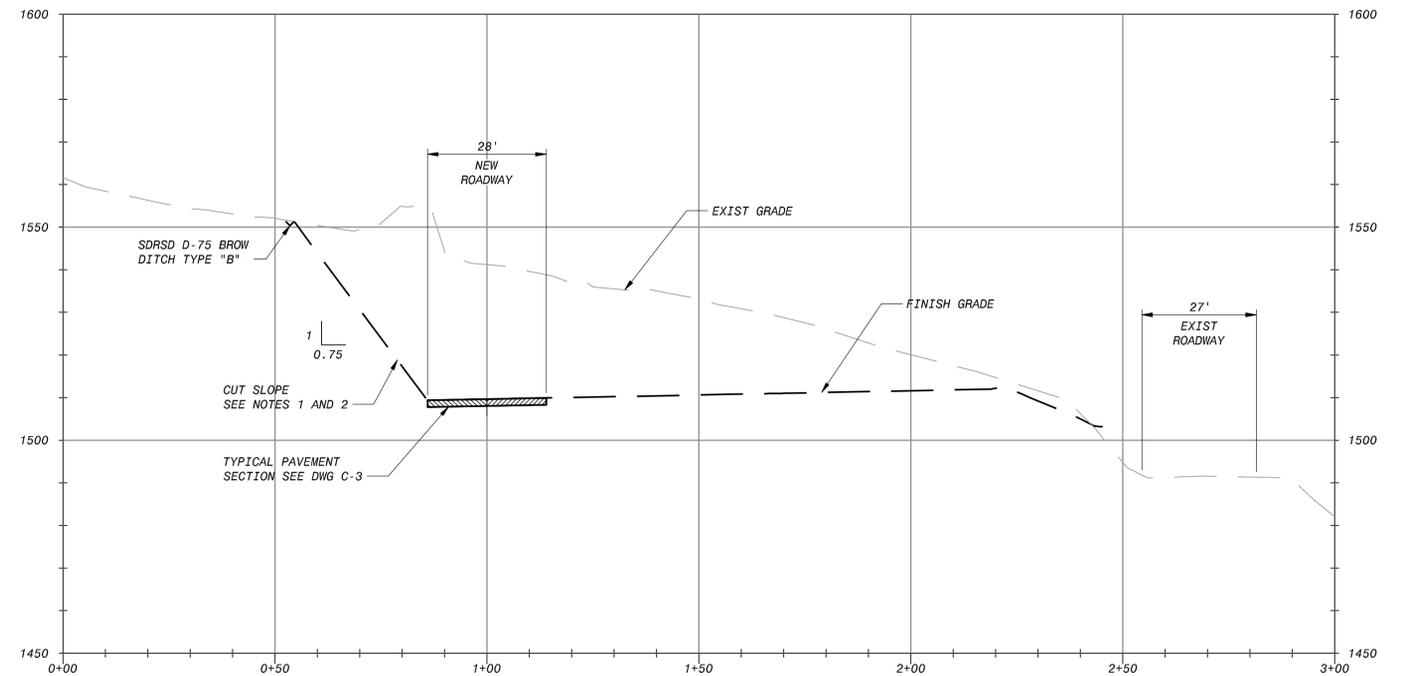
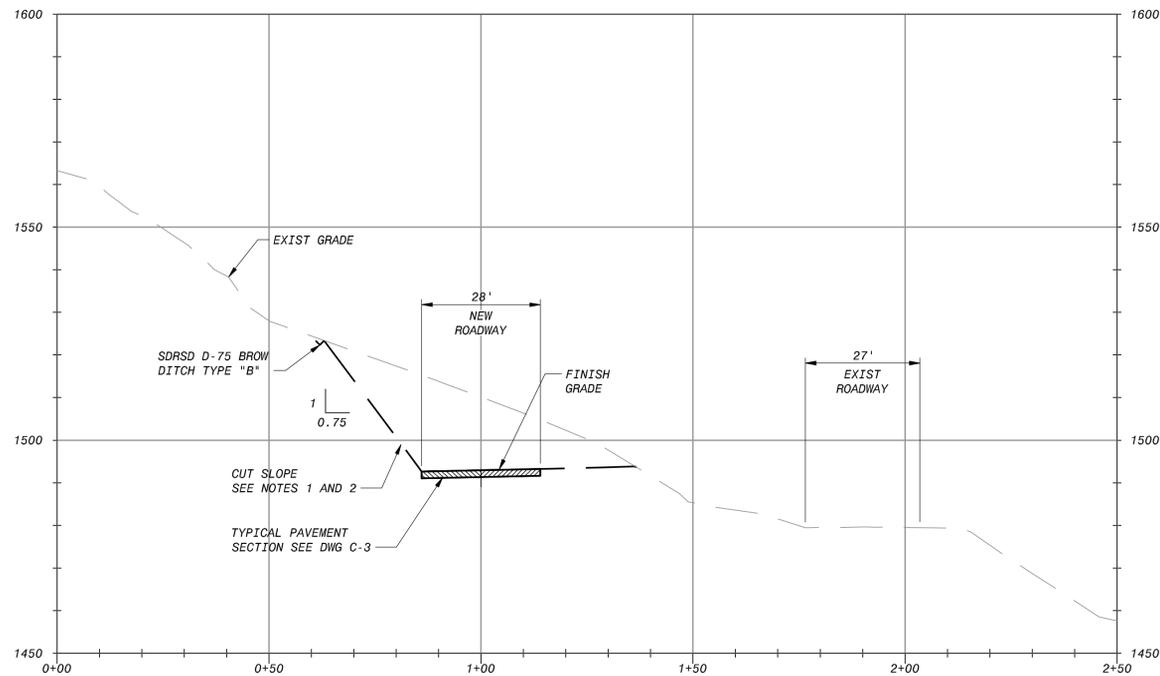
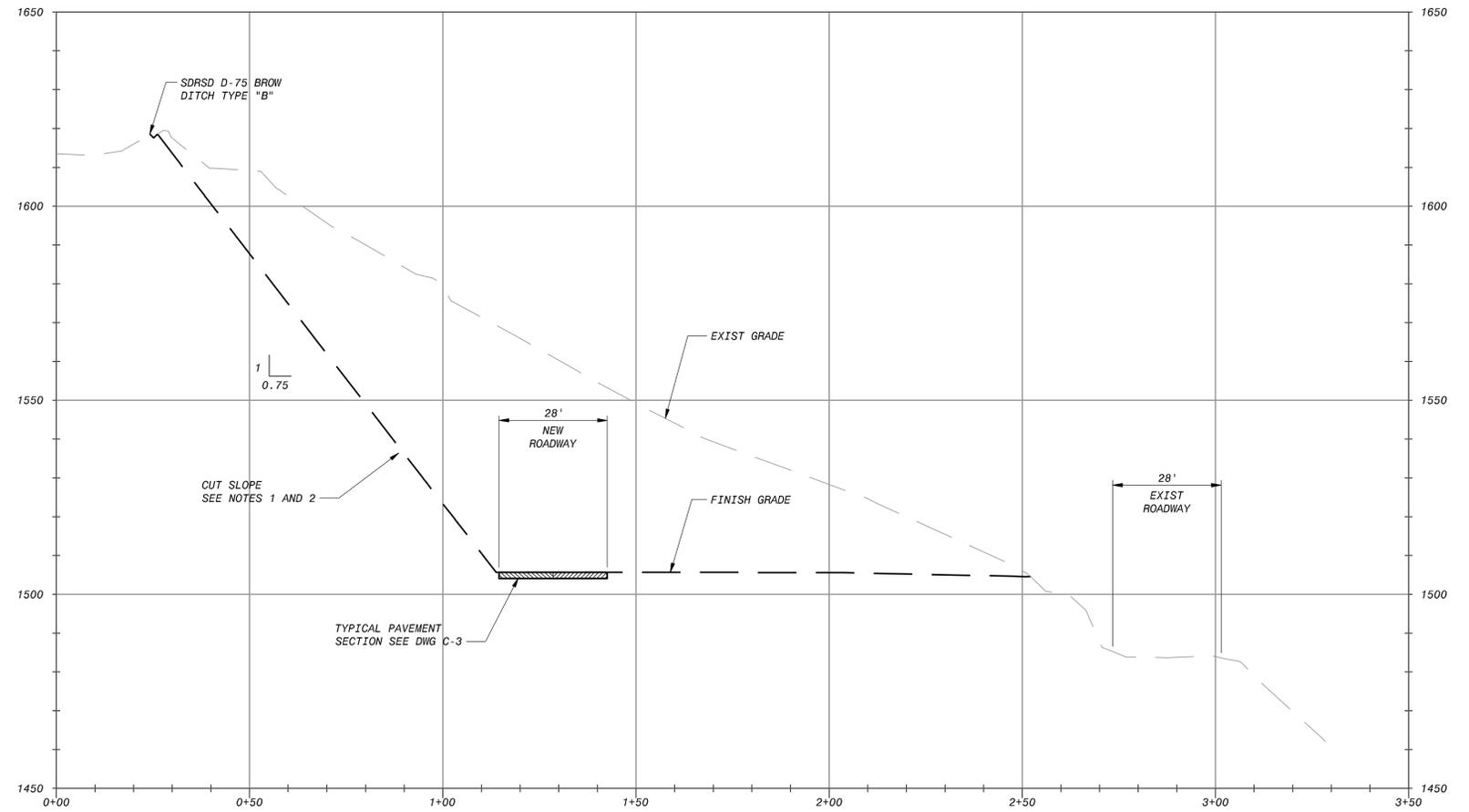
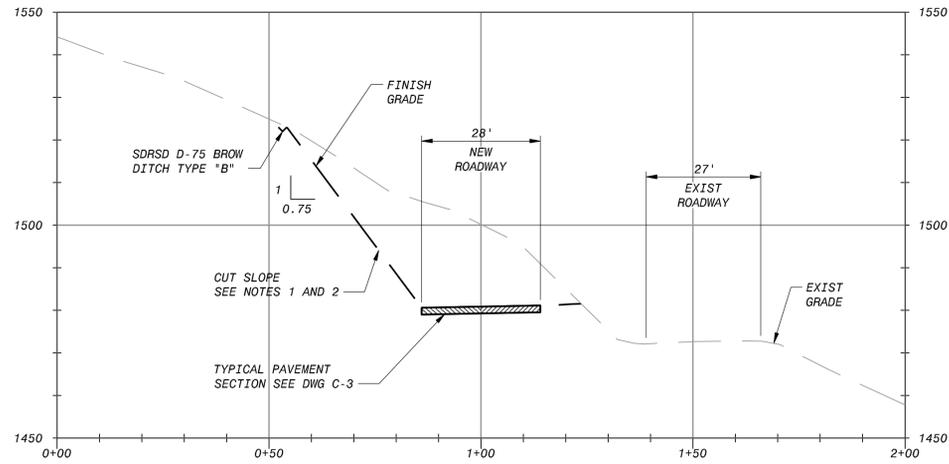
O'Day CONSULTANTS
2710 Loker Avenue West Suite 100
Coronado, California 92010
760-931-7700
Fax: 760-931-8680
OdayConsultants.com

Civil Engineering
Planning
Processing
Surveying

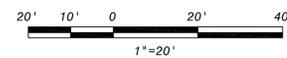
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Contractor								Horizontal	Filed	Plans Prepared Under Supervision Of	TG	KH	By
Inspector								AS SHOWN		Date			Deputy Director of Utilities
Date Completed								Vertical	Traffic	R.C.E. No.			

CIVIL ROADWAY SECTIONS, DETAILS, AND NOTES		Sheet 6 of 11
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LAKE WOHLFORD DAM
REPLACEMENT PROJECT

DRAWING NO.
C-4

CITY OF ESCONDIDO

CIVIL
ROADWAY SECTIONS

Sheet 7 of 11

CONSTRUCTION RECORD		REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK	SCALE	Office	Designed By	Drawn By	Checked By
Contractor								MONUMENT 2179 NORTH 2005276.319 EAST 6537749.403 ELEVATION 1548.017 2" IRON PIPE WITH 523C STAMPED "EGCS 1992 2179" ON TOP OF 4" DIRT SLOPE ELEVATION 806.75 NVD 29 DATUM; US FEET	Horizontal 1"=30' Vertical 1"=30'	Traffic	CTD	JEG	
Inspector							Plans Prepared Under Supervision Of				Date	R.C.E. No.	
Date Completed													

Submitted _____
 By _____
 Deputy Director of Utilities

EROSION CONTROL NOTES

- ALL BUILDING PADS TO BE DIKED AND THE DIKES MAINTAINED TO PREVENT WATER FROM FLOWING FROM THE PAD UNTIL THE STREETS AND DRIVEWAYS ARE PAVED AND WATER CAN FLOW FROM THE PADS WITHOUT CAUSING EROSION, OR CONSTRUCT DRAINAGE FACILITIES TO THE SATISFACTION OF THE CITY/COUNTY THAT WILL ALLOW WATER TO DRAIN FROM THE PAD WITHOUT CAUSING EROSION.
- TOPS OF ALL SLOPES TO BE DIKED OR TRENCHED TO PREVENT WATER FROM FLOWING OVER THE CREST OF SLOPES.
- MANUFACTURED SLOPES AND PADS SHALL BE ROUNDED VERTICALLY AND HORIZONTALLY AS APPROPRIATE TO BLEND WITH THE SURROUNDING TOPOGRAPHY.
- AS SOON AS CUTS OR EMBANKMENTS ARE COMPLETED, BUT NOT LATER THAN OCTOBER 1 ALL CUT AND FILL SLOPES SHALL BE STABILIZED WITH A HYDROMULCH MIXTURE OR AN EQUAL TREATMENT APPROVED BY THE CITY/COUNTY BETWEEN OCTOBER 1 AND APRIL 30. APPROVED SLOPE PROTECTION MEASURES SHALL PROCEED IMMEDIATELY BEHIND THE EXPOSURE OF CUT SLOPES AND/OR THE CREATION OF EMBANKMENT SLOPES.
- CATCH BASINS, DESILTING BASINS AND STORM DRAIN SYSTEM SHALL BE INSTALLED TO THE SATISFACTION OF THE CITY/COUNTY.
- GRAVEL BAG CHECK DAMS TO BE PLACED IN A MANNER APPROVED BY THE CITY/COUNTY IN UNPAVED STREETS WITH GRADIENTS IN EXCESS OF 2% AND ON OR IN OTHER GRADED OR EXCAVATED AREAS AS REQUIRED BY THE CITY/COUNTY.
- THE DEVELOPER TO MAINTAIN THE PLANTING AND EROSION CONTROL MEASURES DESCRIBED ABOVE UNTIL RELIEVED OF THE SAME BY THE CITY/COUNTY. THE DEVELOPER SHALL REMOVE ALL SOIL INTERCEPTED BY THE GRAVEL BAGS, CATCH BASINS AND DESILTING BASINS AND KEEP THESE FACILITIES CLEAN AND FREE OF SILT AND SAND AS DIRECTED BY THE CITY/COUNTY. THE DEVELOPER SHALL REPAIR ANY ERODED SLOPES AS DIRECTED BY THE CITY/COUNTY.

SILTATION AND SEDIMENT CONTROL MEASURES NOTES:

- THE SEDIMENT BASINS SHALL BE PROVIDED AT THE LOWER END OF EVERY DRAINAGE AREA PRODUCING SEDIMENT RUNOFF. THE BASINS SHALL BE MAINTAINED AND CLEANED TO DESIGN CONTOURS AFTER EVERY RUNOFF PRODUCING STORM. THE BASINS SHOULD BE SEMI-PERMANENT STRUCTURES THAT WOULD REMAIN UNTIL SOIL STABILIZING VEGETATION HAS BECOME WELL ESTABLISHED ON ALL ERODIBLE SLOPES.
- SEDIMENTATION BASINS MAY NOT BE REMOVED OR MADE INOPERATIVE WITHOUT PRIOR APPROVAL OF THE CITY/COUNTY.
- UTILITY TRENCHES THAT ARE CUT THROUGH BASIN DIKES OR BASIN INLET DIKES SHALL BE PLUGGED WITH GRAVELBAGS FROM TOP OF PIPE TO TOP OF DIKE.
- ALL UTILITY TRENCHES SHALL BE BLOCKED AT THE PRESCRIBED INTERVALS WITH A DOUBLE ROW OF GRAVELBAGS WITH A TOP ELEVATION TWO GRAVELBAGS BELOW THE GRADED SURFACE OF THE STREET. GRAVELBAGS ARE TO BE PLACED WITH LAPPED COURSES. THE INTERVALS PRESCRIBED BETWEEN GRAVELBAG BLOCKING SHALL DEPEND ON THE SLOPE OF THE GROUND SURFACE BUT NOT TO EXCEED THE FOLLOWING:

GRADE OF THE STREET	INTERVAL
LESS THAN 2%	200 FEET MAX
2% TO 4%	100 FEET
4% TO 10%	50 FEET
OVER 10%	25 FEET

- AFTER UTILITY TRENCHES ARE BACKFILLED AND COMPACTED, THE SURFACES OVER SUCH TRENCHES SHALL BE MOUNDING SLIGHTLY TO PREVENT CHANNELING OF WATER IN THE TRENCH AREA. CARE SHOULD BE EXERCISED TO PROVIDE FOR CROSS FLOW AT FREQUENT INTERVALS WHERE TRENCHES ARE NOT ON THE CENTERLINE OF A CROWNED STREET.

- ALL BUILDING PADS SHOULD BE SLOPED TOWARDS THE DRIVEWAYS AND VELOCITY CHECK DAMS PROVIDED AT THE BASE OF ALL DRIVEWAYS DRAINING INTO THE STREET.

- PROVIDE VELOCITY CHECK DAMS IN ALL UNPAVED GRADED CHANNELS AT THE INTERVALS INDICATED BELOW:

GRADE OF CHANNEL	INTERVALS BETWEEN CHECK DAMS
LESS THAN 3%	100 FEET
3% TO 6%	50 FEET
OVER 6%	25 FEET

- PROVIDE VELOCITY CHECK DAMS IN ALL PAVED STREET AREAS ACCORDING TO INTERVALS INDICATED BELOW. VELOCITY CHECK DAMS MAY BE CONSTRUCTED OF GRAVEL BAGS, TIMBER OR OTHER EROSION RESISTANT MATERIALS APPROVED BY THE CITY/COUNTY, AND SHALL EXTEND COMPLETELY ACROSS THE STREET OR CHANNEL AT RIGHT ANGLES TO THE CENTERLINE. VELOCITY CHECK DAMS MAY ALSO SERVE AS SEDIMENT TRAPS.

GRADE OF STREET	INTERVALS	NO. OF BAGS HIGH
LESS THAN 2%	AS REQUIRED. 200' MAX	1
2% TO 4%	100 FEET	1
4% TO 6%	50 FEET	1
6% TO 10%	50 FEET	2
MORE THAN 10%	25 FEET	2

- PROVIDE A GRAVELBAG SILT BASIN OR TRAP BY EVERY STORM DRAIN INLET TO PREVENT SEDIMENT FROM ENTERING DRAIN SYSTEM.

- GRAVELBAGS AND FILL MATERIAL SHALL BE STOCKPILED AT INTERVALS, READY FOR USE WHEN REQUIRED.

- ALL EROSION CONTROL DEVICES WITHIN THE DEVELOPMENT SHOULD BE MAINTAINED DURING AND AFTER EVERY RUNOFF PRODUCING STORM, IF POSSIBLE, MAINTENANCE CREWS WOULD BE REQUIRED TO HAVE ACCESS TO ALL AREAS.

- PROVIDE ROCK RIPRAP ON CURVES AND STEEP DROPS IN ALL EROSION PRONE DRAINAGE CHANNELS DOWNSTREAM FROM THE DEVELOPMENT. THIS PROTECTION WOULD REDUCE EROSION CAUSED BY THE INCREASED FLOWS THAT MAY BE ANTICIPATED FROM DENUDED SLOPES, OR FROM IMPERVIOUS SURFACES.

- ANY PROPOSED ALTERNATE CONTROL MEASURES MUST BE APPROVED IN ADVANCE BY ALL RESPONSIBLE AGENCIES: I.E., COUNTY ENGINEER, DEPARTMENT OF SANITATION AND FLOOD CONTROL, OFFICE OF ENVIRONMENTAL MANAGEMENT, ETC.

STORMWATER PROTECTION NOTES

- DURING THE RAINY SEASON THE AMOUNT OF EXPOSED SOIL ALLOWED AT ONE TIME SHALL NOT EXCEED THAT WHICH CAN BE ADEQUATELY PROTECTED BY THE PROPERTY OWNER IN THE EVENT OF A RAINSTORM. 125% OF ALL SUPPLIES NEEDED FOR BMP MEASURES SHALL BE RETAINED ON THE JOB SITE IN A MANNER THAT ALLOWS FULL DEPLOYMENT AND COMPLETE INSTALLATION IN 48 HOURS OR LESS OF A FORECAST RAIN.
- NO AREA BEING DISTURBED SHALL EXCEED 50 ACRES AT ANY GIVEN TIME WITHOUT DEMONSTRATING TO THE CITY/COUNTIES SATISFACTION THAT ADEQUATE EROSION AND SEDIMENT CONTROL CAN BE MAINTAINED. ANY DISTURBED AREA THAT IS NOT ACTIVELY GRADED FOR 10 DAYS MUST BE FULLY PROTECTED FROM EROSION. UNTIL ADEQUATE LONG TERM PROTECTIONS ARE INSTALLED, THE DISTURBED AREA SHALL BE INCLUDED WHEN CALCULATING THE ACTIVE DISTURBANCE AREA. ALL EROSION CONTROL MEASURES SHALL REMAIN INSTALLED AND MAINTAINED DURING ANY INACTIVE PERIOD.
- THE PROPERTY OWNER IS OBLIGATED TO INSURE COMPLIANCE WITH ALL APPLICABLE STORMWATER REGULATIONS AT ALL TIMES. THE BMP'S (BEST MANAGEMENT PRACTICES) THAT HAVE BEEN INCORPORATED INTO THIS PLAN SHALL BE IMPLEMENTED AND MAINTAINED TO EFFECTIVELY PREVENT THE POTENTIALLY NEGATIVE IMPACTS OF THIS PROJECT'S CONSTRUCTION ACTIVITIES ON STORMWATER QUALITY. THE MAINTENANCE OF THE BMP'S IS THE PERMITTEE'S RESPONSIBILITY, AND FAILURE TO PROPERLY INSTALL OR MAINTAIN THE BMP'S MAY RESULT IN ENFORCEMENT ACTION BY THE CITY/COUNTY OR OTHERS. IF INSTALLED BMP'S FAIL, THEY MUST BE REPAIRED OR REPLACED WITH AN ACCEPTABLE ALTERNATE WITHIN 24 HOURS, OR AS SOON AS SAFE TO DO SO.

STABILIZED FIBER MATRIX (SFM)

THE USE OF SFM IS SUBJECT TO THE FOLLOWING LIMITATIONS AND RESTRICTIONS:

- SFM MAY BE USED FOR TEMPORARY EROSION CONTROL FOR DISTURBED AREAS WITH A SLOPE RATIO OF 1 VERTICAL TO 2 HORIZONTAL OR SHALLOWER, INCLUDING PAD AND SEPTIC FIELD AREAS.
- THE SFM SHALL BE APPLIED AT LEAST 24 HOURS BEFORE OR AFTER RAINFALL AND SHALL BE APPLIED TO PROVIDE 100% COVERAGE (I.E. APPLIED FROM MULTIPLE DIRECTIONS AND ANGLES).
- THE APPLICATION AREA MUST BE PROTECTED BY BROW DITCHES AND/OR DIVERSION BERMS AT TOP OF SLOPES TO DIVERT FLOW FROM THE FACE OF THE PROTECTED SLOPE.
- FOR PERMANENT EROSION CONTROL PURPOSES, SFM MUST BE INSTALLED IN CONJUNCTION WITH SEEDED EROSION CONTROL VEGETATION OR HAND PLANTINGS. AS WITH ALL OTHER APPLICATIONS, SFM WILL NOT BE CONSIDERED PERMANENT UNTIL 70% VEGETATION ESTABLISHMENT.
- COVERAGE AND CONCENTRATION: FOR EACH ACRE COVERED, THE MINIMUM APPLICATION VOLUME SHALL BE 10 GALLONS NON-TOXIC WATER-PERMEABLE SOIL-STABILIZING LIQUID EMULSION WITH 3,000 LBS. OF HYDRAULIC MULCH. THE EMULSION MUST BE DESIGNED TO PROTECT SOIL, PREVENT EROSION, AND FLOCCULATE (CLUMP) SEDIMENT.
- A LETTER FROM THE HYDROSEED CONTRACTOR CERTIFYING THE SFM WAS INSTALLED IN ACCORDANCE WITH APPROVED APPLICATION RATES, COVERAGE, AND MANUFACTURER'S DILUTION RATIO SHALL BE SUBMITTED TO THE CITY/COUNTY FOR APPROVAL.

*** BOND FIBER MATRIX (BFM)**

THE USE OF BFM IS SUBJECT TO THE FOLLOWING LIMITATIONS AND RESTRICTIONS:

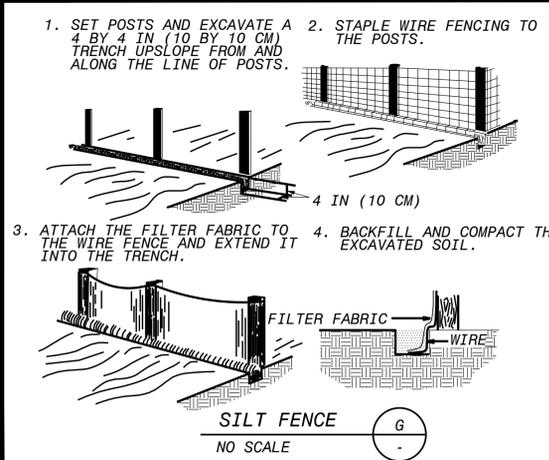
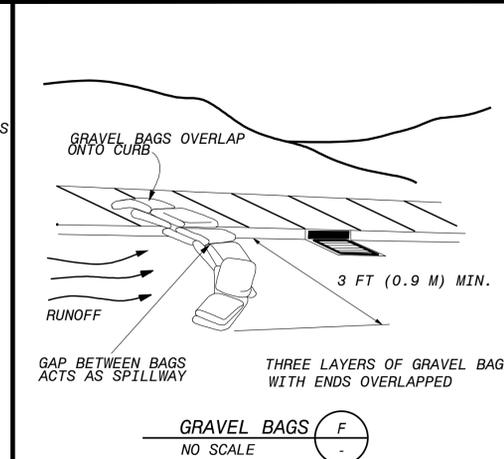
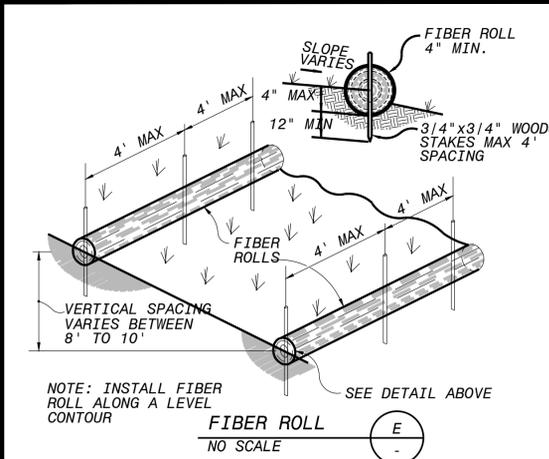
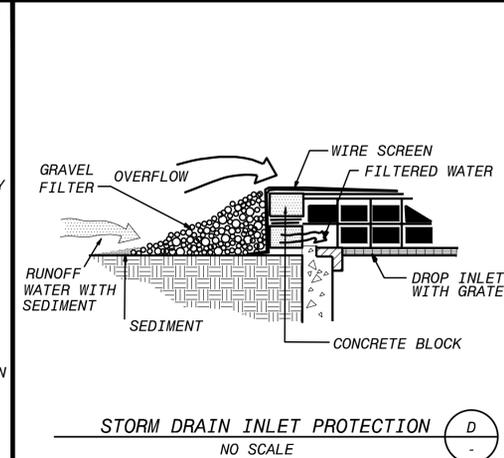
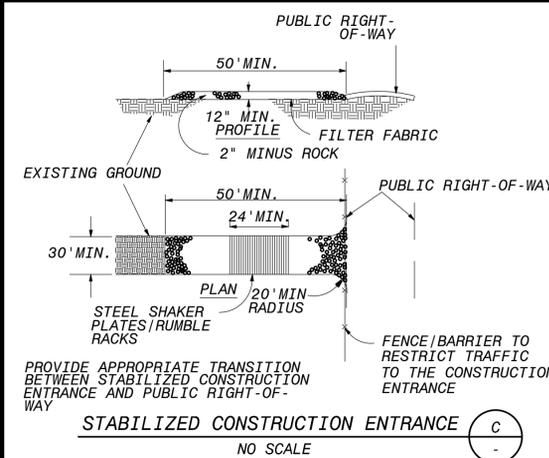
- APPLICATION RATES SHALL BE 3500 POUNDS PER ACRE MINIMUM FOR 2:1 OR SHALLOWER SLOPES AND 4000 POUNDS PER ACRE FOR SLOPES STEEPER THAN 2:1.
- BFM SHALL BE APPLIED AT LEAST 24 HOURS BEFORE OR AFTER RAINFALL.
- THE SITE MUST BE PROTECTED WITH BROW DITCHES AND/OR DIVERSION BERMS AT THE TOP OF SLOPES TO DIVERT FLOW FROM THE FACE OF THE SLOPE.
- BFM SHALL BE APPLIED TO PROVIDE 100% COVERAGE (I.E. APPLICATION FROM MULTIPLE ANGLES).
- FOR PERMANENT EROSION CONTROL PURPOSES, BFM MUST BE INSTALLED IN CONJUNCTION WITH SEEDED EROSION CONTROL VEGETATION.
- A LETTER FROM THE HYDROSEED CONTRACTOR CERTIFYING THAT THE BFM HAS BEEN INSTALLED IN ACCORDANCE WITH THE APPROVED APPLICATION RATES AND COVERAGE REQUIREMENTS SHALL BE SUBMITTED TO THE CITY/COUNTY INSPECTOR FOR APPROVAL.

HYDROSEED MIX

BOTANICAL NAME	COMMON NAME	RATE LBS./ACRE	MIN% PURE/GERM
ALYSSUM SPECIES	'ROYAL CARPET'	3	98/80
ALYSSUM SPECIES	'CARPET OF SNOW'	4	98/80
GAZANIA 'PIXIE DWARF'	DWARF CLUMPING GAZANIA	4	90/40
LITMONIUM PEREZI	SEA LAVENDER	3	65/5
VERBENA TENUISECTA	MOSS VERBENA	4	90/75
TOTAL POUNDS PER ACRE		18	

THE FOLLOWING MATERIALS SHALL BE IN THE MIX:

MULCH-CONWEB FIBER	2,200 LBS/ACRE
BINDER-TERRA TAC #3	150 LBS/ACRE
FERTILIZER 16-20-0	400 LBS/ACRE



- SET POSTS AND EXCAVATE A 4 BY 4 IN (10 BY 10 CM) TRENCH UPSLOPE FROM AND ALONG THE LINE OF POSTS.
- STAPLE WIRE FENCING TO THE POSTS.
- ATTACH THE FILTER FABRIC TO THE WIRE FENCE AND EXTEND IT INTO THE TRENCH.
- BACKFILL AND COMPACT THE EXCAVATED SOIL.



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OAKVALE ROAD REALIGNMENT PROJECT
CITY OF ESCONDIDO

DRAWING NO. C-6

CONSTRUCTION RECORD	REFERENCES	Date	By	REVISIONS	App'd	Date	BENCH MARK
Contractor							
Inspector							
Date Completed							

SCALE	Office	Designed By	Drawn By	Checked By
Horizontal	Filed	NF	TG	KH
Vertical	Traffic			

Submitted _____
By _____
Deputy Director of Utilities

CIVIL
EROSION CONTROL & REVEGETATION DETAILS

APPENDIX B – ROCK CUT STABILITY CALCULATIONS



Client Name City of Escondido Page 1 of ~~22~~ 30

Project Name Lake Wohlford Dam Project No. 177740

Calculation Title Oakvale Road Rock Slope Stability

Calculation No./File No. 58.4315.1001/51.1000

Verification Method: Check and Review Alternate Calculations

Objective: Determine the factor of safety for the rock slope near Oakvale Road. Provide preliminary rock bolt design for reinforcement (if needed).

Unverified Assumptions Requiring Subsequent Verification

No.	Assumption	Verified By	Date

Refer to Page _____ of this calculation for additional assumptions.

This Section Used for Software-Generated Calculations

Program Name/Number _____ Version _____

Standard B&V Application Used? Yes No

If no, list approved deviation permit number below and attach approved deviation permit.

Review and Approval

Rev	Prepared By	Date	Verified By	Date	Approved By	Date
0	B. Gomer <i>B/G</i>	3/4/2014	S. Cheema	3/7/2014		
1	B. Gomer <i>B/G</i>	6/12/14	S. Cheema	6/23/14		



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1.0 Objective

Determine the factor of safety for the rock slope near Oakvale Road. Provide preliminary rock bolt design for reinforcement (if needed) and estimate preliminary quantities.

2.0 References

1. Kleinfelder, Geotechnical Data Report, Amendment No. 1, Wohlford Dam Replacement Design Project, prepared for the City of Escondido, May 31, 2013.
2. CalTrans Geotechnical Manual, "Rock Cut Slope Design", June 2013
<http://www.dot.ca.gov/hq/esc/geotech/geo_manual/manual.html> (March 3, 2014).
3. Black & Veatch, Technical Memorandum – Spectral Matching, Revision 0, January 24, 2014.
4. Crystal Dirks <TaborCV@bv.com>, "RE: Oakvale RD 30% Design: Rock Cut Stability Text," February 27, 2014, office communication (March 3, 2014).
5. US Army Corps of Engineers, Rock Reinforcement, Engineering Manual 1110-1-2907, February 15, 1980.
6. US Army Corps of Engineers, Rock Foundations, Engineering Manual 1110-1-2908, November 30, 1994.
7. Hoek, E., "Practical Rock Engineering", 2007 <http://www.rocscience.com/hoek/pdf/Practical_Rock_Engineering.pdf> (January 31, 2013)
8. Williams Form Engineering Corp, Ground System Engineering, No. 112
<http://www.williamsform.com/Contact_Us/PDFs/Ground_Anchor_Systems.pdf> (March 3, 2014).

3.0 Design Basis

1. Reference 1 provides the results of laboratory tests on rock cores that include unconfined compressive strength (UCS) tests, measurements of dry unit weight (UW), and direct shear tests. Based on these tests, the minimum UCS of the rock is 22,500 psi, the average UW of the rock is 167 pcf, the shear intercept (c) along the discontinuity (likely due to surface roughness) is assumed to be about 5 psi based on the direct shear tests, and the average initial friction angle ($\varphi_{initial}$) is about 40 degrees and the average final friction angle (φ_{final}) is 30 degrees based on



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the direct shear tests. These values are similar to the values measured at boring KB-304 of $\varphi_{\text{initial}} = 51.7$ degrees and $\varphi_{\text{final}} = 29.5$ degrees. The shear intercept s from the direct shear tests ranged from 0 to 6 psi for the initial failure envelopes and 0 to 15.7 psi for the final failure envelopes. The average value for the final failure envelope of about 5 psi is used.

- Reference 1 provides the depth to groundwater at the closest boring (KB-304) to the proposed rock slope. The closest boring is near the toe of the proposed slope. The depth to groundwater is 42 feet and the rock slope is assumed to be fully drained.
- Reference 1 indicates the rock type is granite and that the rock is very hard with relatively high RQD value (90+ percent). Discontinuity orientations are based on the results of geotechnical data collected at the ground surface near the Oakvale Road and downhole televiewer data collected in boring KB-304 (Reference 1).
- The required FS for design is 1.5 for static conditions (Reference 2).
- The required FS for design is 1.0 for pseudo-static conditions (Reference 2).
- The design peak ground acceleration (PGA) at the site is 0.29 g (Reference 3)

3.1 Assumptions

- The data collected in boring KB-304 is representative of the conditions in the rock slope along Oakvale Road.
- The geotechnical data collected along Oakvale Road is representative of the conditions in the rock slope.
- A FS greater than 1.5 is not required due to the proximity of the rock cut to the Lake Wohlford Dam. The Reference 2 criteria apply.

4.0 Definition of Units & Constants

' = feet " = inch psi = pounds per square inch psf = pound per square foot (144*psi)
ft = feet lbs = pound in = inch T= ton (2000 pounds, 2 kips)
kips = 1000 pounds) FS = Factor of Safety



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1.0 Objective

Revision 1 makes minor modifications to Figures 5-8 and 5-9 to plot the 40 degree friction angle and not the 45 degree friction angle, and change the text in Section 5.4 to correctly state the ultimate strength of the R71 150 KSI All-Thread Bar is 237,000 psi. These changes are made based on comments received on the 30 percent design memorandum.

Revision 1 also adds a global stability analysis to this calculation that considers the cut slope with the road and dam excavation.

2.0 References

1. Same as Revision 0.

3.0 Design Basis

1. Same as Revision 0.

3.1 Assumptions

Same as Revision 0 with the addition that the rock bolts installed in the cut slope above the road will prevent a failure surface from forming in the reinforced portions of the slope.



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5.0 Analysis

This analysis considers two proposed rock slopes adjacent to the proposed Oakvale Road realignment for the Wohlford Dam Project. The two considered slopes are a 1:1 (horizontal:vertical) and 0.75:1 slopes.

5.1 Rock Slope Orientations

The rock slope orientations are based on Reference 4. Figure 5-1 presents the orientation of the road alignment, the 0.75:1 slope (H:V), and the estimated roadway orientations. The 0.75:1 slope produces an angle of 54 degrees below horizontal.

$$\text{dip} = \tan^{-1}(1/0.75) = 53.1^\circ \approx 54^\circ$$

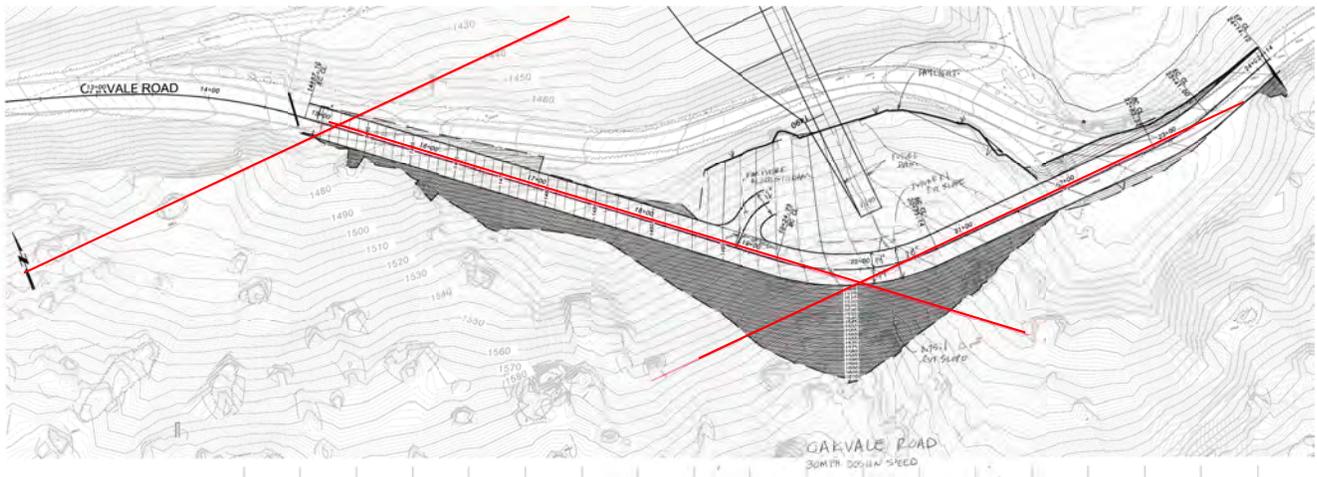


Figure 5-1 Road Orientations with 0.75:1 Road Cut

The considered road orientations are about 90 degrees from north (dip azimuth of 0 degrees) for the eastern portion of the road and about 132 degrees from north (dip azimuth of 54 degrees) for the western portion of the road. The road orientations are represented as red line in Figure 5-1, with the azimuth of the eastern portion copied and plotted next to the north arrow to provide a point of reference. At the intersection of the orientations for the eastern and western portions of the road, a maximum height (H) of about 110 feet is observed for the 0.75:1 road cut.

5.1 Kinematic Analyses of the Road Cuts

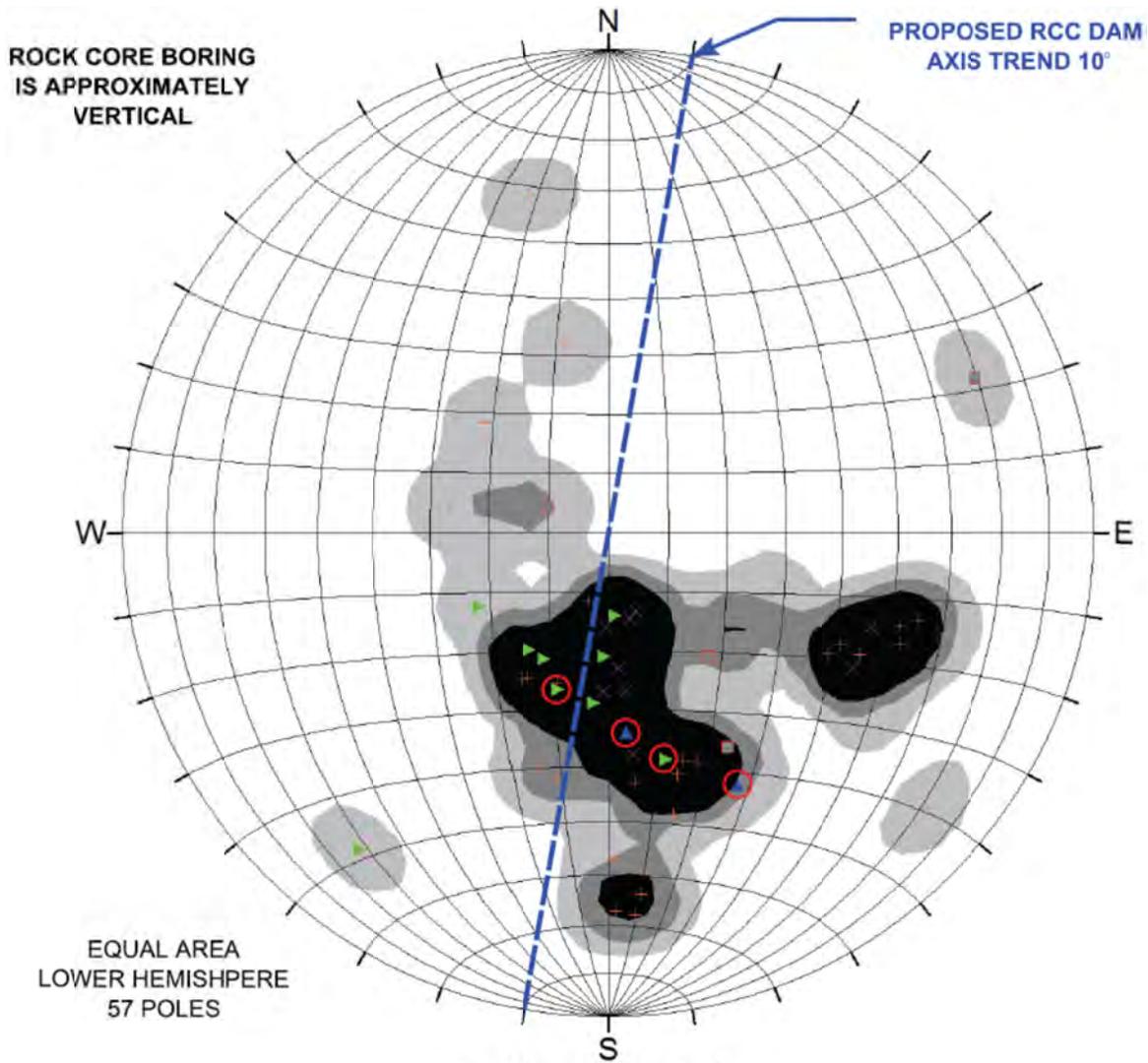
A kinematic analysis of the road cut is performed for both orientations. This analysis is completed on equal area stereonet to evaluate planar sliding, toppling, and wedge failure. The stereonet from Reference 1 for boring KB-304 and the Oakvale Road are presented below as Figure 5-2 and 5-3, respectively. Generally, the data show similar orientations for structures between the borings and surface geosstructural data. The main differences are the cluster of poles in Figure 5-3 in the southwest quadrant (dips near 80 degrees and dip azimuths near 70 degrees) and at the north and south poles (dips of near 90 degrees with dip azimuths near 0 and 180 degrees). The southwest quadrant shall be considered since several structural features were identified by all the surveys (GEI and Kleinfelder); however, the north and south poles shall not since only a few structures were identified and Kleinfelder did not identify any of these known structures along Oakvale Road.



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EXPLANATION

SYMBOLS

- < 20 feet depth [4]
- ▲ 20 - 40 feet depth [2]
- ▶ 40 - 60 feet depth [9]
- + 60 - 80 feet depth [27]
- x >80 feet depth [15]
- OPEN FEATURE

CONTOURING

FISHER CONCENTRATIONS
 % of total per 1.0% area

- 0 - 1%
- 1 - 3%
- 3 - 5%
- >5%

Figure 5-2 KB-304 Televiewer Data (Reference 1)



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Figure 5-4 presents the evaluation of toppling failure for the eastern portion of the road, and Figure 5-5 the evaluation of toppling failure for the western portion of the road. Both analyses assume the final friction angle of 30 degrees as a conservative evaluation, and a cone angle of 60 degrees for the toppling envelope. The region of potential toppling is outlined in red in both figures. The results indicate toppling failure is unlikely; however, potential near vertical discontinuities identified by earlier investigation – but not in the recent investigations – should be evaluated if they are identified during excavation or an additional field evaluation.

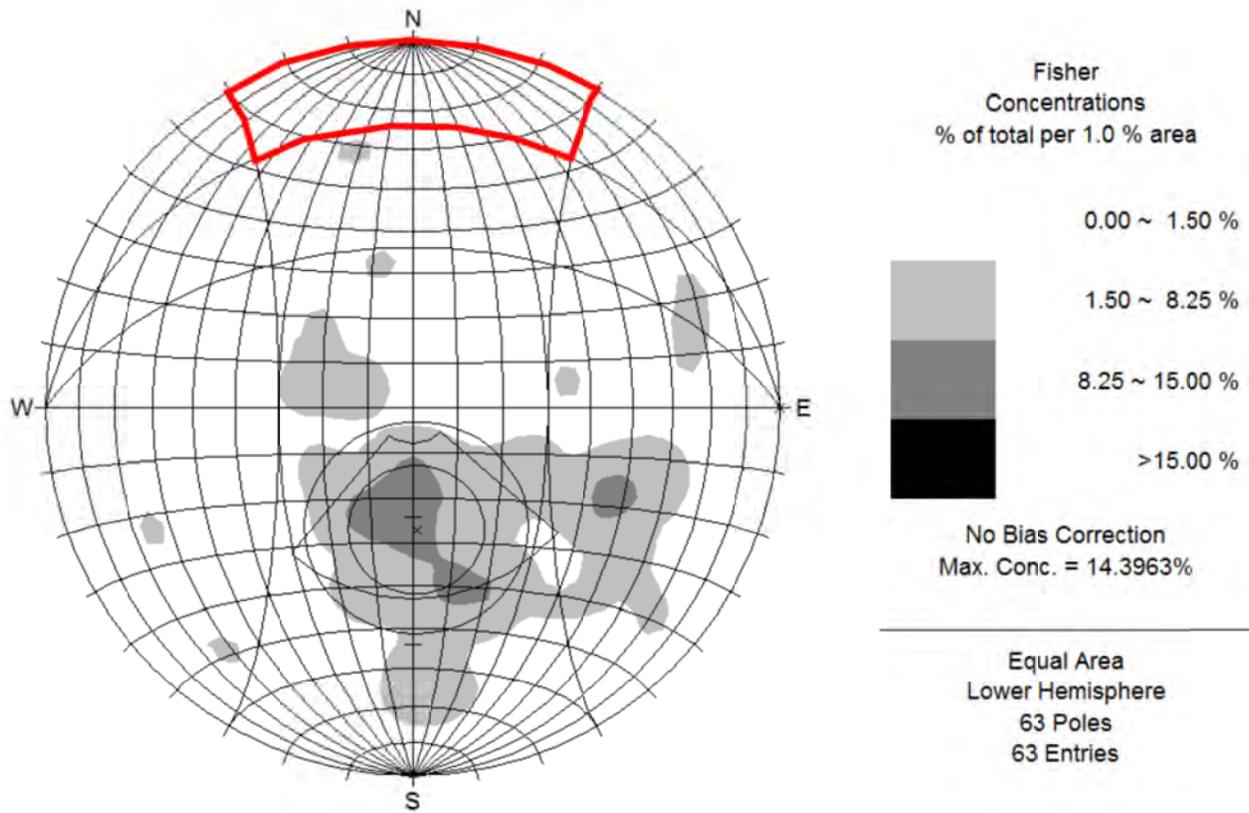


Figure 5-4 Toppling Evaluation for Eastern Portion of Road (Poles in Red Box would Potentially Fail)

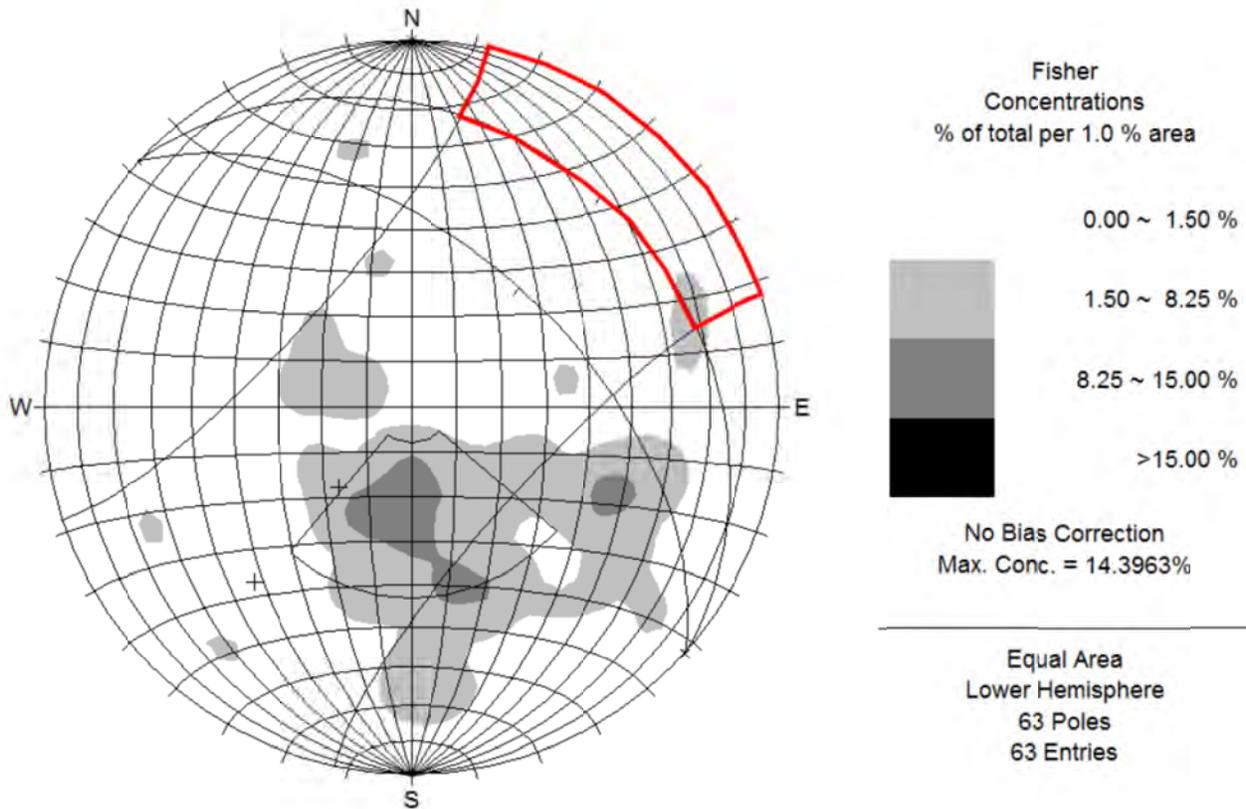


Figure 5-5 Toppling Evaluation for Western Portion of Road (Poles in Red Box would Potentially Fail)

Figure 5-6 presents the evaluation of planar failure for the eastern portion of the road, and Figure 5-7 the evaluation of planar failure for the western portion of the road. Both analyses assume the final friction angle of 30 degrees as a conservative evaluation (thin black line) and provide an additional estimate assuming the initial friction angle of 40 degrees (dashed blue line). The region of potential sliding is outlined in red in both figures for the final friction angle of 30 degrees. The results indicate sliding failure is likely. Sliding failure shall be considered for calculating a factor of safety and slope reinforcement.

Using the initial friction angle of 40 degrees reduces the number of poles that may potentially fail, but does not eliminate them. Changing the slope to 1:1 nearly eliminates the potential for sliding failure using the initial friction angle of 40 degrees for the eastern and western road. However, potential sliding failures remain for the final friction angle of 30 degrees.



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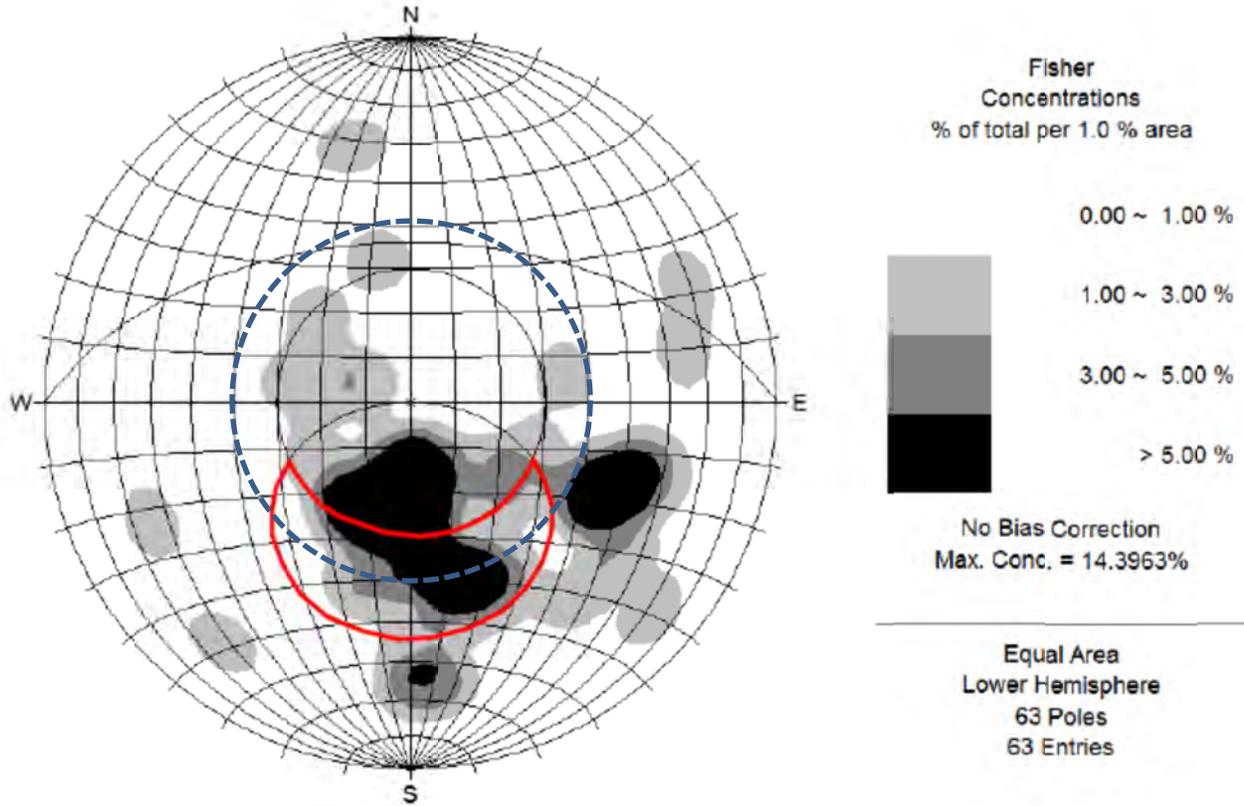


Figure 5-6 Planar Sliding Evaluation for Eastern Portion of Road (Poles in Red Box would Potentially Fail)

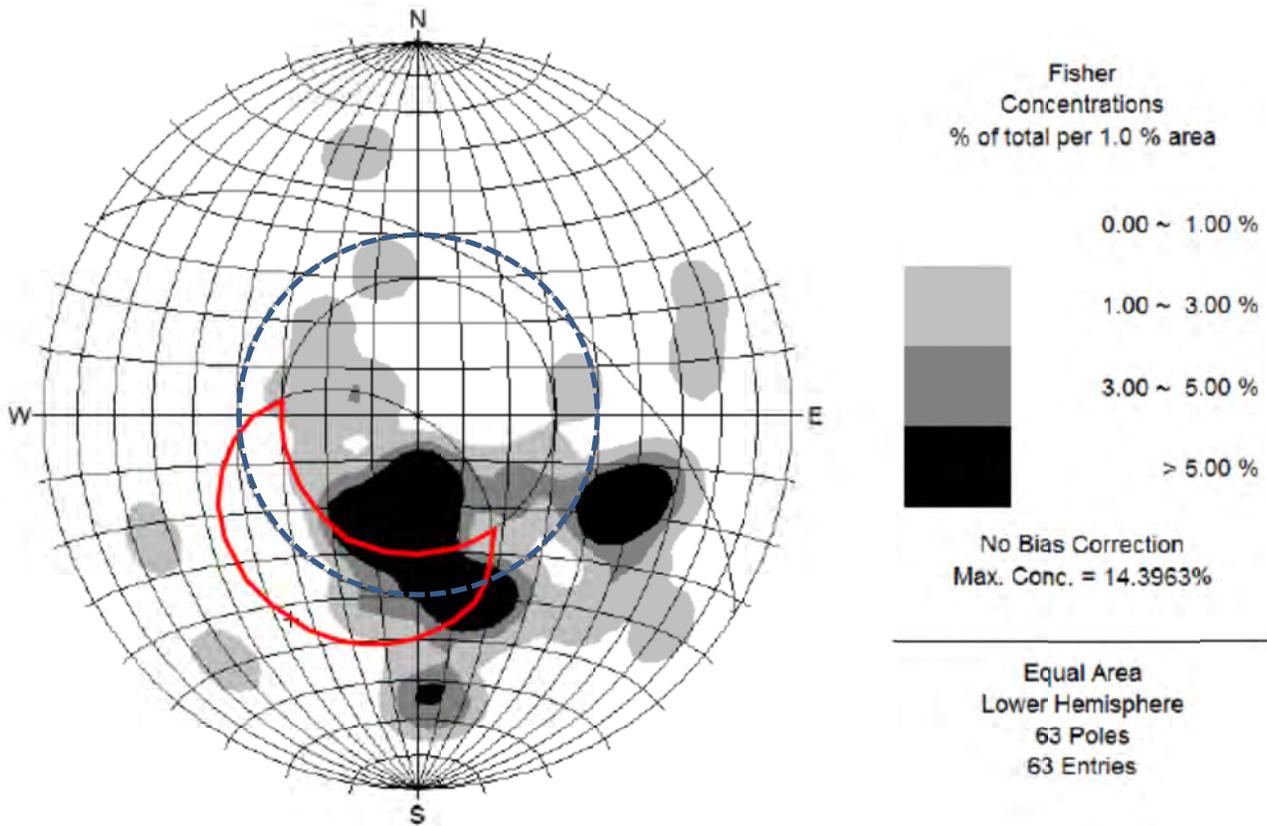


Figure 5-7 Planar Sliding Evaluation for Western Portion of Road (Poles in Red Box would Potentially Fail)

Figure 5-8 presents the evaluation of wedge failure for the eastern portion of the road, and Figure 5-9 the evaluation of wedge failure for the western portion of the road. Both analyses assume the final friction angle of 30 degrees as a conservative evaluation (thick red line) and provide an additional estimate assuming the initial friction angle of 40 degrees (thin red line). The results indicate wedge failure is possible for when the set in the southwest quadrant from the geostructural data in Reference 1 (Figure5-3) is included. Additionally, wedge failure is possible when steeper dipping discontinuities are considered in the main group of discontinuities (dips of about 10 to 50 degrees with dip azimuths of about 0 degrees).

Using the initial friction angle of 40 degrees essentially eliminates the potential for wedge failure. Additionally, changing the slope to 1:1 eliminates the potential for wedge failure using the initial friction angle of 40 degrees for the eastern and western road. However, potential sliding failures remain for the final friction angle of 30 degrees and a 1:1 slope. The wedge failures are not considered for stability analysis since they generally occur for conservative assumptions (final friction angle) and sets of discontinuities with lower concentrations in the stereonet plots.



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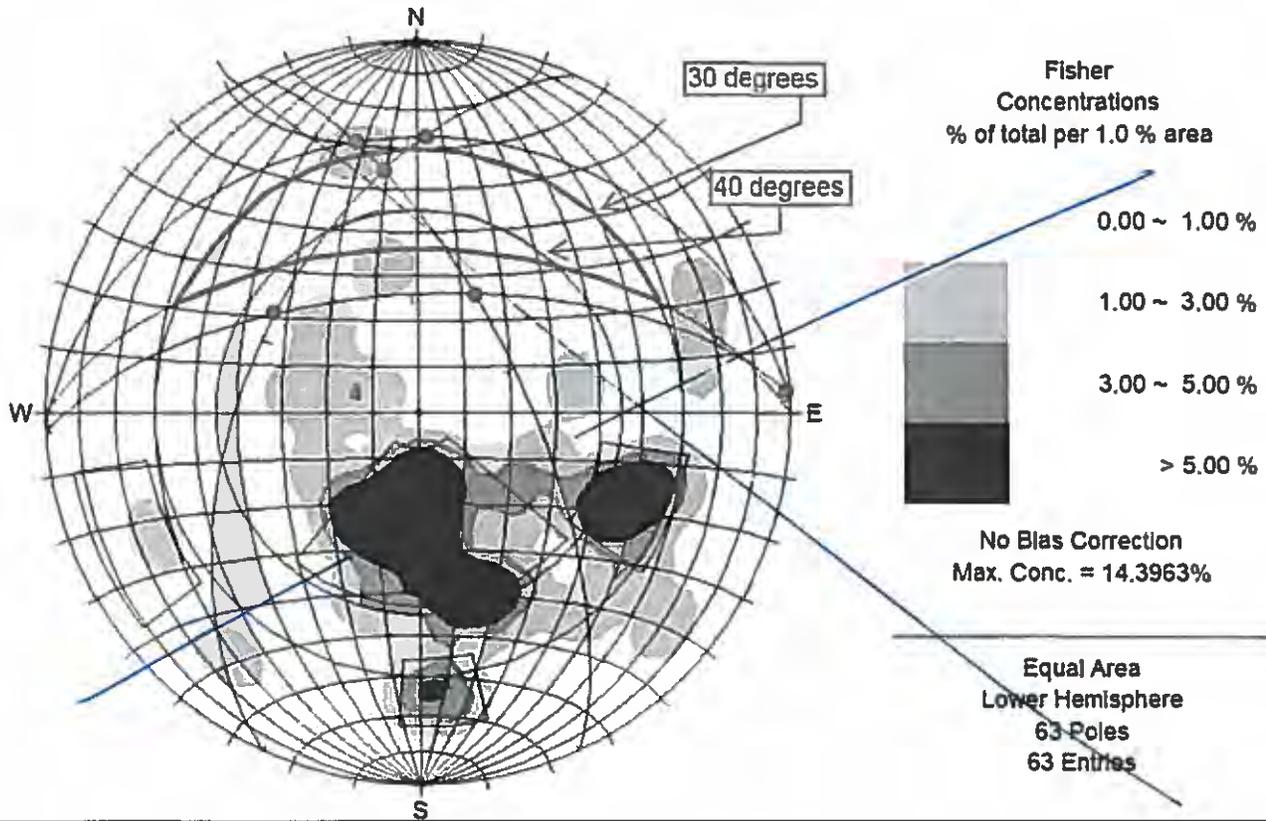


Figure 5-8 Wedge Sliding Evaluation for Eastern Portion of Road (Intersection in Red Box would Potentially Fail)

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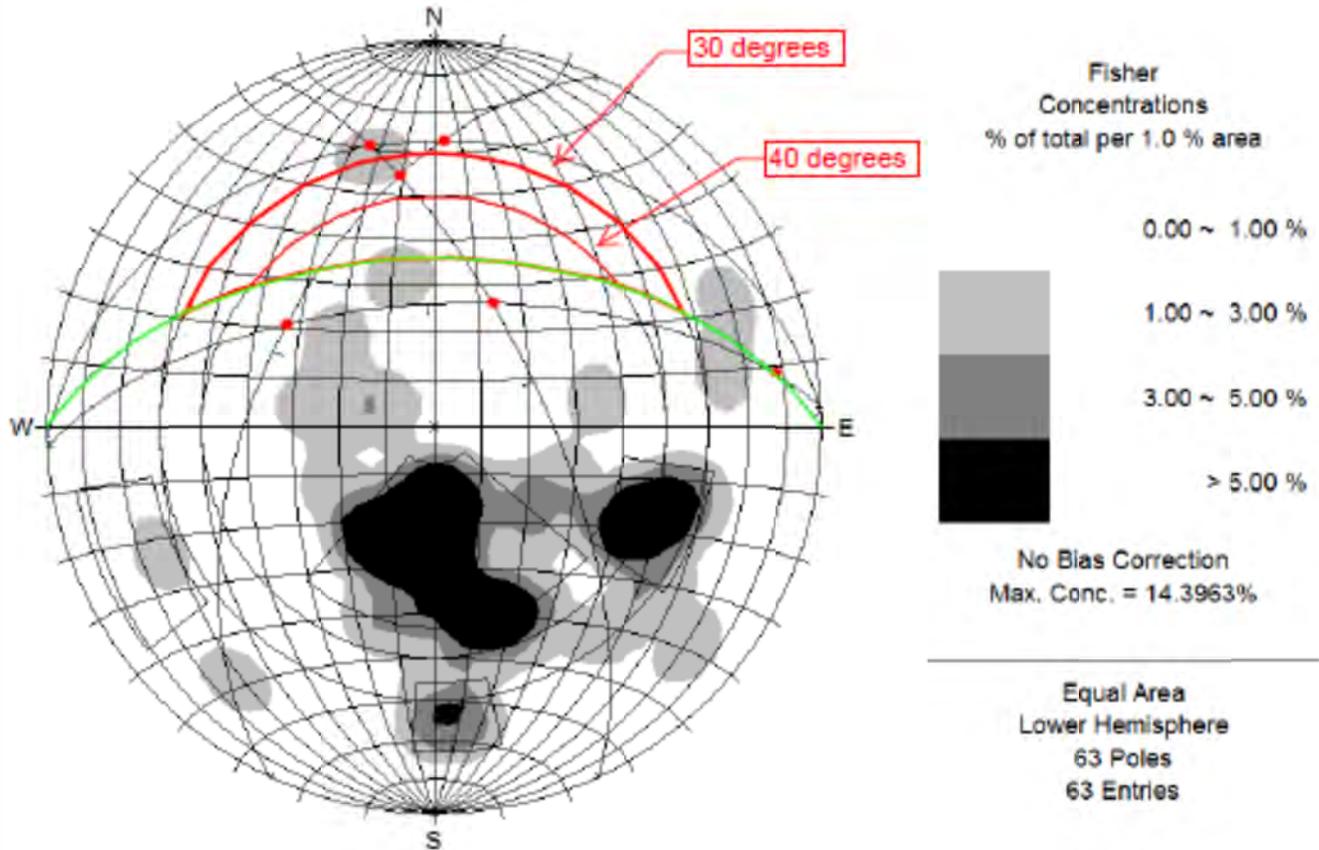


Figure 5-8 Wedge Sliding Evaluation for Eastern Portion of Road (Intersection in Red Box would Potentially Fail)

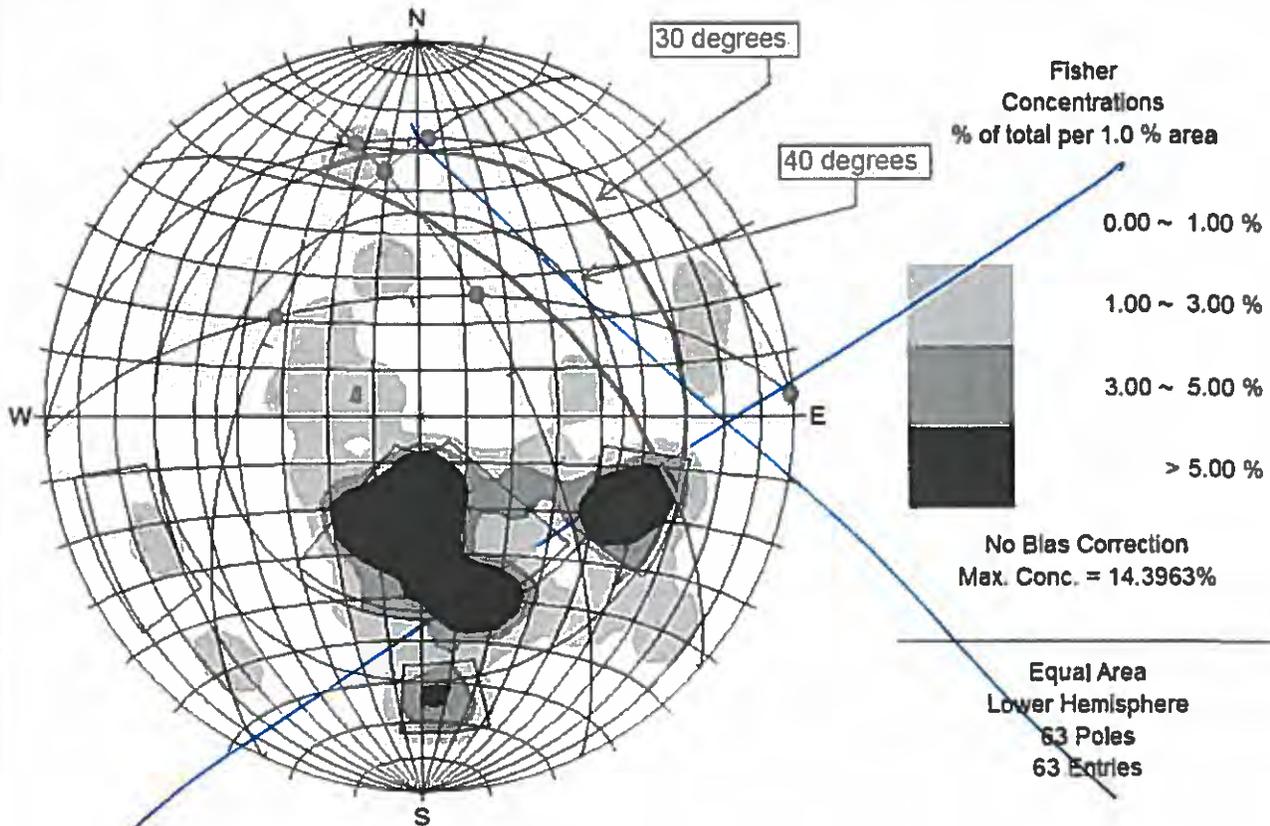


Figure 5-9 Wedge Sliding Evaluation for Western Portion of Road (Intersection in Red Box would Potentially Fail)

See page 12A for Figure 5-9

5.2 Static Planar Stability Analysis of the Road Cuts

Planar stability will be evaluated using the method described in References 5 and 7. The analysis considers an unsaturated slab without tension cracks and a saturated slab with tension cracking. The unsaturated condition is based on the observed groundwater level that is below the base of the rock slope and assumes no tension cracking, which reduces the volume and mass of the sliding block. The saturated condition assumes a tension crack has formed and allows water to infiltrate into the tension crack, flow down the potential failure surface, and drain out the toe of the potential failure surface. This is considered a worst case situation and assumes the water in the tension crack is about 15 feet below the ground surface.

Figure 5-10 presents the configuration of the analysis without tension cracking. Figures 5-11 and 5-12 present the results of the analyses for the situation without tension cracking. The results indicate a FS of 1.5 for planar failure with a rock slope of 1:1, and a FS of 1.5 for a rock slope of 0.75:1 with about 120,000 lbs of reinforcing per linear foot of rock cut. This results in a horizontal and vertical rock bolt spacing of about 14 and 13.5 feet on the rock slope face using a rock bolt working capacity of 140,000 lbs (See Section 5.4).



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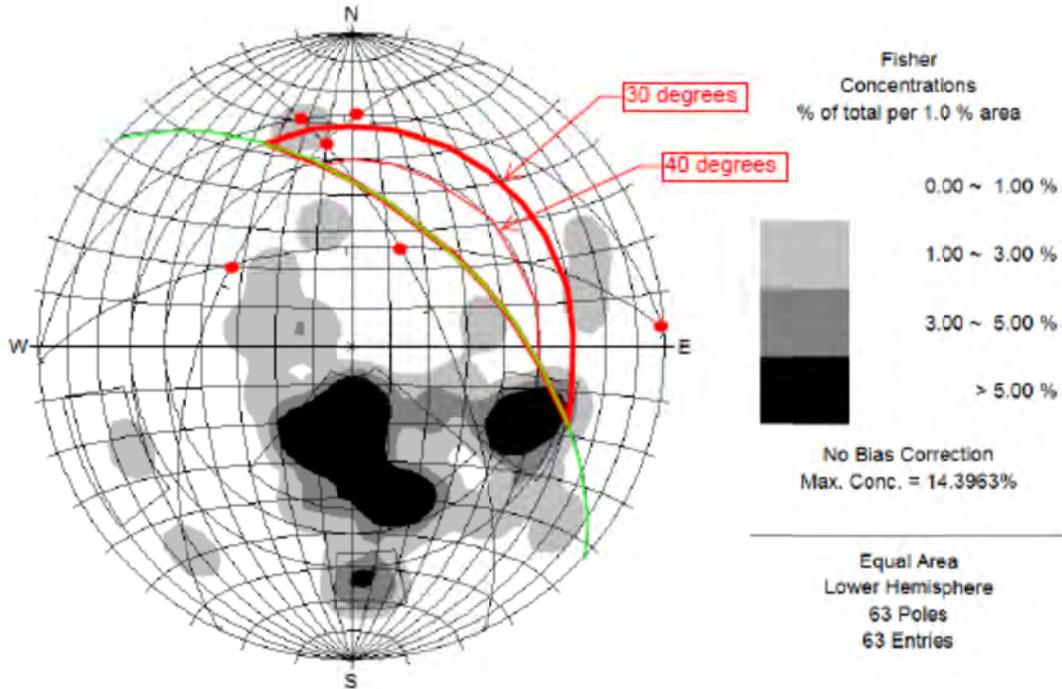


Figure 5-9 Wedge Sliding Evaluation for Western Portion of Road (Intersection in Red Box would Potentially Fail)



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Input Values

Height of Rock Slope

H 110 feet

Height of Groundwater

Z_w 0 feet

Dip of Rock Slope Face

ψ_f 54 degrees

Dip of Discontinuity

ψ_p 42 degrees

Shear Strength of Rock

φ_{initial} 40 degrees

φ_{final} 30 degrees

cohesion 5 psi

Unit Weight of Rock

γ_r 167 pcf

Unit Weight of Water

γ_w 62.4 pcf

Horizontal Earthquake Coefficient

α 0.00 g

Rock Bolt/Dowel Information

T 119,351 lbs/lft

θ 12 degrees (counterclockwise from normal)

θ_{optimal} 12 degrees (counterclockwise from normal)

Bolt Working Capacity

Bw 140000 lbs

Calculated Values

Base Area of Wedge

A 164.4 feet

Weight of Rock Wedge

W 388045.1 lbs/lft

Uplift Force from Water

U 0 lbs/lft

Factor of Safety

Resisting_{initial} 458296

Resisting_{final} 352257

Driving 234838

FS_{initial} 1.95

FS_{final} 1.50

Bolt Spacing on Discontinuity

Sd 13.9 feet

Bolt Spacing on Rock Slope Face (Law of sines)

α 12 degrees

γ 78 degrees

B 90 degrees

Sv 13.6 feet

Sh 13.9 feet

Figure 5-11 FS for 0.75:1 Rock Slope without Tension Cracking and Planar Failure



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Input Values

Height of Rock Slope
H 110 feet

Height of Groundwater
Z_w 0 feet

Dip of Rock Slope Face
 Ψ_r 45 degrees

Dip of Discontinuity
 Ψ_p 42 degrees

Shear Strength of Rock
 $\varphi_{initial}$ 40 degrees
 φ_{final} 30 degrees
cohesion 5 psi

Unit Weight of Rock
 γ_r 167 pcf

Unit Weight of Water
 γ_w 62.4 pcf

Horizontal Earthquake Coefficient
 α 0.00 g

Rock Bolt/Dowel Information
T - lbs/ft
 θ 12 degrees (counterclockwise from normal)
 $\theta_{optimal}$ 12 degrees (counterclockwise from normal)

Bolt Working Capacity
B_w 140000 lbs

Calculated Values

Base Area of Wedge
A 164.4 feet

Weight of Rock Wedge
W 111757.4 lbs/ft

Uplift Force from Water
U 0 lbs/ft

Factor of Safety
Resisting_{initial} 188051
Resisting_{final} 166313
Driving 74780.3
FS_{initial} 2.51
FS_{final} 2.22

Bolt Spacing on Discontinuity
S_d #DIV/0! feet

Bolt Spacing on Rock Slope Face (Law of sines)
 α 3 degrees
 γ 78 degrees
B 99 degrees
S_v #DIV/0! feet
S_h #DIV/0! feet

Figure 5-12 FS for 1:1 Rock Slope without Tension Cracking and Planar Failure

Figure 5-13 presents the configuration of the analysis with tension cracking and an assumed depth to water of 15 feet in the tension crack. Figures 5-14 and 5-15 present the results of the analyses for the situation with tension cracking. The results indicate a FS of 1.5 for planar failure with a rock slope of 1:1, and a FS of 1.5 for a rock slope of 0.75:1 with about 137,000 lbs of reinforcing per linear foot of rock cut. This results in a horizontal and vertical rock bolt spacing of about 11.5 feet on the rock slope face using a rock bolt working capacity of 140,000 lbs (See Section 5.4).

The required reinforcing load of about 137,000 lbs per linear foot of rock cut is the controlling condition for the static case. Therefore, the tension cracked analysis is used for design of the rock bolts.



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Input Values

Height of Rock Slope
H 110 feet

Height of Groundwater
Z_w 95 feet

Dip of Rock Slope Face
 Ψ_f 54 degrees

Dip of Discontinuity
 Ψ_p 42 degrees

Shear Strength of Rock
 $\phi_{initial}$ 40 degrees
 ϕ_{final} 30 degrees
cohesion 5 psi

Unit Weight of Rock
 γ_r 167 pcf

Unit Weight of Water
 γ_w 62.4 pcf

Horizontal Earthquake Coefficient
 α 0.00 g

Rock Bolt/Dowel Information
T 136,699 lbs
 Θ 12 degrees (counterclockwise from normal)
 $\Theta_{optimal}$ 12 degrees (counterclockwise from normal)

Bolt Working Capacity
Bw 140000 lbs

Calculated Values

Base Area of Wedge
A 133.0 feet

Weight of Rock Wedge
W 347030.2 lbs/ft

Height of Water in Tension Crack
z_w 6.0 feet

Uplift Force from Water on Discontinuity
U 25016.6 lbs/ft

Force from Water in Tension Crack
V 1134.6 lbs/ft

Depth of Tension Crack
z 21.0 feet

Distance of Tension Crack Behind Crest
b 18.9 feet

Factor of Safety
Resisting_{initial} 402701.2
Resisting_{final} 306945.5
Driving 204630.3
FS_{initial} 1.97
FS_{final} 1.50

Bolt Spacing on Discontinuity
Sd 11.7 feet

Bolt Spacing on Rock Slope Face (Law of sines)
 α 12.00 degrees
 γ 78 degrees
B 90.00 degrees
Sv 11.4 feet
Sh 11.7 feet

Figure 5-14 FS for 0.75:1 Rock Slope with Tension Cracking and Planar Failure



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Input Values

Height of Rock Slope
H 110 feet

Height of Groundwater
Z_w 95 feet

Dip of Rock Slope Face
 ψ_f 45 degrees

Dip of Discontinuity
 ψ_p 42 degrees

Shear Strength of Rock
 $\phi_{initial}$ 40 degrees
 ϕ_{final} 30 degrees
cohesion 5 psi

Unit Weight of Rock
 γ_r 167 pcf

Unit Weight of Water
 γ_w 62.4 pcf

Horizontal Earthquake Coefficient
 α 0.00 g

Rock Bolt/Dowel Information
T — lbs
 θ 12 degrees (counterclockwise from normal)
 $\theta_{optimal}$ 12 degrees (counterclockwise from normal)

Bolt Working Capacity
Bw 140000 lbs

Calculated Values

Base Area of Wedge
A 156.0 feet

Weight of Rock Wedge
W 108826.9 lbs/ft

Height of Water in Tension Crack
z_w 0.0 feet

Uplift Force from Water on Discontinuity
U 0.0 lbs/ft

Force from Water in Tension Crack
V 0.0 lbs/ft

Depth of Tension Crack
z 5.6 feet

Distance of Tension Crack Behind Crest
b 5.9 feet

Factor of Safety
Resisting_{initial} 180175.2
Resisting_{final} 159006.5
Driving 72819.39
FS_{initial} 2.47
FS_{final} 2.18

Bolt Spacing on Discontinuity
Sd #DIV/0! feet

Bolt Spacing on Rock Slope Face (Law of sines)
 α 3.00 degrees
 γ 78 degrees
B 99.00 degrees
Sv #DIV/0! feet
Sh #DIV/0! feet

Figure 5-15 FS for 1:1 Rock Slope with Tension Cracking and Planar Failure



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5.3 Seismic Planar Stability Analysis of the Road Cuts

The anticipated peak ground acceleration at the site is anticipated to be about 0.29 g based on a deterministic analysis for the Elsinore Fault (Reference 3). Using a factor of seismic coefficient (k_h) of 0.5 PGA (0.145 g) results in a pseudo-static FS greater than 1.1 with the same reinforcing required achieving a FS of 1.5 in the static analyses. Only the result from the controlling analysis with a 0.75:1 slope and tension cracking is shown below in Figure 5-16.

Input Values	Calculated Values
Height of Rock Slope H 110 feet	Base Area of Wedge A 133.0 feet
Height of Groundwater Z _w 95 feet	Weight of Rock Wedge W 347030.2 lbs/ft
Dip of Rock Slope Face ψ_r 54 degrees	Height of Water in Tension Crack z _w 6.0 feet
Dip of Discontinuity ψ_p 42 degrees	Uplift Force from Water on Discontinuity U 25016.6 lbs/ft
Shear Strength of Rock $\varphi_{initial}$ 40 degrees φ_{final} 30 degrees cohesion 5 psi	Force from Water in Tension Crack V 1134.6 lbs/ft
Unit Weight of Rock γ_r 167 pcf	Depth of Tension Crack z 21.0 feet
Unit Weight of Water γ_w 62.4 pcf	Distance of Tension Crack Behind Crest b 18.9 feet
Horizontal Earthquake Coefficient α 0.14 g	Factor of Safety Resisting _{initial} 375422.8 Resisting _{final} 288176.3 Driving 240735.4 FS _{initial} 1.56 FS _{final} 1.20
Rock Bolt/Dowel Information T 136,699 lbs θ 12 degrees (counterclockwise from normal) $\theta_{optimal}$ 12 degrees (counterclockwise from normal)	
Bolt Working Capacity Bw 140000 lbs	Bolt Spacing on Discontinuity Sd 11.7 feet
	Bolt Spacing on Rock Slope Face (Law of sines) $i\alpha$ 12.00 degrees γ 78 degrees B 90.00 degrees Sv 11.4 feet Sh 11.7 feet

Figure 5-16 Results from Pseudo-Static Analysis Considering a k_h of 0.14 g for the 0.75:1 Slope and Tension Cracking



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5.4 Preliminary Rock Bolt Evaluation

Analysis of rock bolt (tensioned) anchor systems are completed following Reference 6. The anchor load and spacing are based on the evaluation in Sections 5.2 and 5.3. The anchor load considered a FS of 1.5 for the slope under static conditions. The results indicate the working capacity for each rock bolt should be 140 kips (140,000 lbs).

Failure of the rock bolts considers failure within the rock mass, between the rock and grout/anchor, and between the grout and the rod. Yield of the top anchorage is not considered in this calculation.

Given the high rock strength (UCS=22,500 psi) the stressing limits of the reinforcing bar and anchorage are likely to control the design. Therefore, a design load of 140 kips (140,000 pounds) is assumed based on 60 percent of the ultimate strength of 150,000 psi for a 1-3/8 inch all-thread-bar (Reference 8). A table from Reference 8 is repeated below as Figure 5-17 with the strengths for the 150 ksi all-thread bar.

R71 150 KSI All-Thread-Bar - ASTM A722*

Nominal Bar Diameter & Pitch	Minimum Net Area Thru Threads	Minimum Ultimate Strength	Prestressing Force			Nominal Weight	Approx. Thread Major Dia.	Part Number
			0.80f pu A	0.70f pu A	0.60f pu A			
1" - 4 (26 mm)	0.85 in ² (549 mm ²)	128 kips (567 kN)	102 kips (454 kN)	89.3 kips (397 kN)	76.5 kips (340 kN)	3.09 lbs./ft. (4.6 Kg/M)	1-1/8" (28.6 mm)	R71-08
1-1/4" - 4 (32 mm)	1.25 in ² (807 mm ²)	188 kips (834 kN)	150 kips (667 kN)	131 kips (584 kN)	113 kips (506 kN)	4.51 lbs./ft. (6.71 Kg/M)	1-7/16" (36.5 mm)	R71-10
1-3/8" - 4 (36 mm)	1.58 in ² (1019 mm ²)	237 kips (1054 kN)	190 kips (843 kN)	166 kips (738 kN)	142 kips (633 kN)	5.71 lbs./ft. (8.50 Kg/M)	1-9/16" (39.7 mm)	R71-11
1-3/4" - 3-1/2 (46 mm)	2.60 in ² (1664 mm ²)	390 kips (1734 kN)	312 kips (1388 kN)	273 kips (1214 kN)	234 kips (1041 kN)	9.06 lbs./ft. (13.5 Kg/M)	2" (50.8 mm)	R71-14
2-1/4" - 3-1/2 (57 mm) *	4.08 in ² (2632 mm ²)	613 kips (2727 kN)	490 kips (2181 kN)	429 kips (1909 kN)	368 kips (1636 kN)	14.1 lbs./ft. (20.8 Kg/M)	2-1/2" (63.5 mm)	R71-18
2-1/2" - 3 (65 mm)	5.19 in ² (3350 mm ²)	778 kips (3457 kN)	622 kips (2766 kN)	545 kips (2422 kN)	467 kips (2074 kN)	18.2 lbs./ft. (27.1 Kg/M)	2-3/4" (69.9 mm)	R71-20
3" - 3 (75 mm) *	6.46 in ² (4169 mm ²)	969 kips (4311 kN)	775 kips (3448 kN)	678 kips (3018 kN)	581 kips (2587 kN)	22.3 lbs./ft. (32.7 Kg/M)	3-3/64" (77.4 mm)	R71-24

* ACI 355.1R section 3.2.5.1 indicates an ultimate strength in shear has a range of .6 to .7 of the ultimate tensile strength. Designers should provide adequate safety factors for safe shear strengths based on the condition of use.

- Per PTI recommendations for anchoring, anchors should be designed so that:
 - The design load is not more than 60% of the specified minimum tensile strength of the prestressing steel.
 - The lock-off load should not exceed 70% of the specified minimum tensile strength of the prestressing steel.
 - The maximum test load should not exceed 80% of the specified minimum tensile strength of the prestressing steel.

* The 2-1/4" and 3" diameter bars are not covered under ASTM A722.

Figure 5-17 Table of Properties for 150 ksi All-Thread-Bar (Reference 8)

The anchor depth – the anchor length below the potential failure surface necessary to develop the required anchor force for stability – depends on the type of rock and anchor pattern. Based on the analysis in Sections 5.2 and 5.3, multiple rows of anchors may be used. Based on Reference 1, the rock is considered competent and the following equation from References 6 and 8 is used to evaluate the anchor depth (D):

$$D = \frac{FS P}{\gamma l s} = \frac{1.5 \times 140,000 \text{ lbs}}{167 \text{ pcf} \times 11.5 \text{ ft} \times 11.5 \text{ ft}} = 9.5 \text{ ft}$$

Where: FS is the FS of 1.5 recommended in Reference XXX (USACE);

P Is the required load on the anchor (140,000 lbs), which is less than 60 percent of the ultimate strength of the 150,000 psi 1-3/8 inch all-thread-bar;

γ is the unit weight of the rock (167 pcf); and,

l and s are the required spacing of the rock bolts (assumed to be a square, or l = s)

Based on this estimate the anchor depth is 10 feet.

The anchor bonding between the anchor and grout, and grout and rock are now considered. Typically, the bond between the anchor and grout is about twice the bond between the grout and rock (Reference 6). Therefore, only the grout and rock



bond is considered. The length (L_B) of the grouted portion required to generate the required anchor load is calculated from the following relationship from References 6 and 8:

$$L_B = \frac{P}{\pi d \tau_w} = \frac{140,000 \text{ lbs}}{3.14 \times 3 \text{ inches} \times 300 \text{ psi}} = 50 \text{ inches} = 4.1 \text{ feet}$$

Where: P is the required load on the anchor;
 d is the effective diameter of the borehole; and,
 τ_w is the working bond strength (assumed to be half the ultimate bond strength, FS=2)

The borehole diameter is assumed to be 3 inches since the rod is assumed to be 1-3/8 inches. A larger borehole diameter decreases the required bond length. The working bond strength is typically taken as 10 percent of the rock unconfined compressive strength ($0.1 \times 22,500 \text{ psi} = 2,250 \text{ psi}$); however, maximum ultimate bond strength of 600 psi is recommended in References 6 and 8. Therefore, the working bond strength is 300 psi ($0.5 \times 600 \text{ psi} = 300 \text{ psi}$) and the anchor bonding length is 3 feet. A recommended minimum of 10 feet of bond length is used (Reference 6). This bond length controls the length of the rock bolt below the potential failure surface.

Use of 1-3/8 inch 150,000 psi all-thread-bar would appear to meet the required anchor load. Specifically, 150,000 psi all-thread-bar has a minimum ultimate strength of 237,000 lbs. The recommended design load for this material is not more than 60 percent of the minimum tensile strength of the prestressing steel (142,000 lbs) based on Reference 8. The required load of 140,000 lbs is less.

A bond length of 10 feet is required below the potential failure plane. As shown in Figure 5-18, the anticipated rock bolt length is based on the length of the line "ab" plus the bond length. The line "ab" is estimated using the "law of sines" to be

$$\text{length of } ab = 136 \text{ feet} \times \frac{\sin(54^\circ - 42^\circ)}{\sin(90^\circ)} = 28.3 \text{ feet} \approx 29 \text{ feet}$$

Where the value of 136 feet is the length of the rock face ($136 = \frac{110 \text{ feet}}{\cos(90^\circ - 54^\circ)}$).

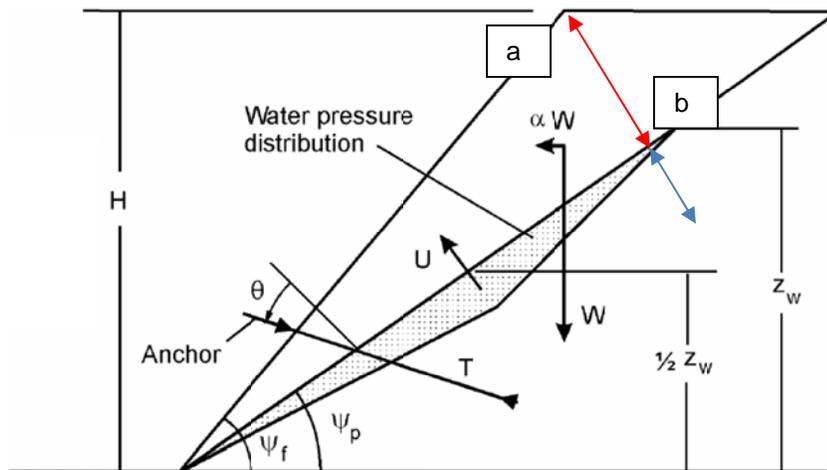


Figure 5-18 Total Anchor Length is the Free-Stress Length (red arrow) and the Bond Length (blue arrow)

The value of 29 feet is the free-stress length of the rock bolt. This results in a total rock bolt length of 39 feet (= free stress length + bond length = 29 feet + 10 feet). A total length of 40 feet will be used.



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5.6 Global Rock Slope Stability

The cut slope for the road is located above the proposed excavation for the Wohlford Dam replacement project. To assess the global stability of the entire slope, two potential cross sections are evaluated (Figure 5-19).

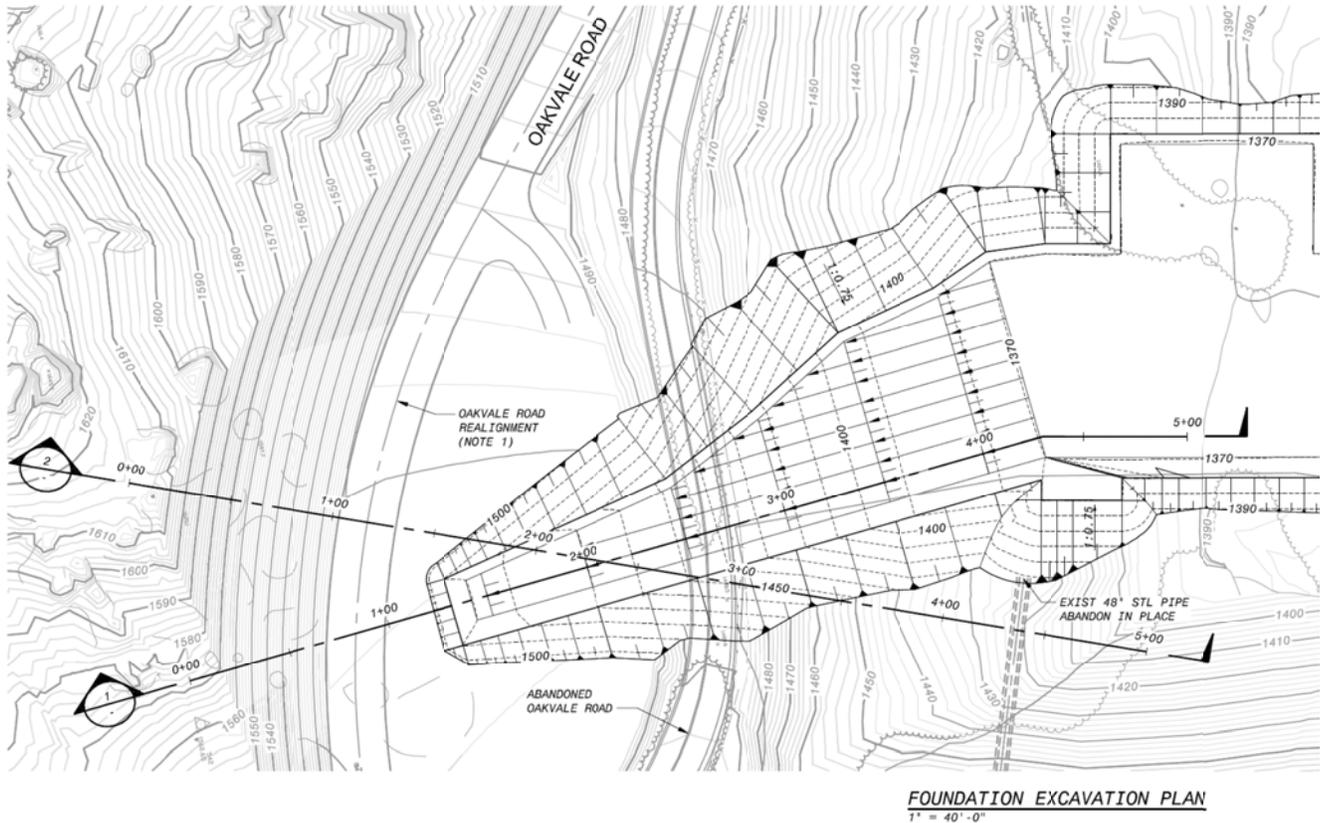


Figure 5-19 Cross Sections Considered for Global Stability

The two cross sections represent the steepest decent into the excavation (Section 1) and the steepest decent along the road cut (Section 2). Figures 5-20 and 5-21 present the cross sections for Section 1 and Section 2, respectively. The red lines on both figures show an average slope that will be used for the global stability analysis. This average slope is considered appropriate since the rock discontinuities, and not the rock strength of the mass, are anticipated to control stability and the excavation extends to bedrock.

The average slope for Section 1 (Figure 5-20) is in a dip direction near 0 degrees with a dip angle of about:

$$\text{dip} = \tan^{-1}(200 \text{ ft}/380 \text{ ft}) = 27.8^\circ \approx 28^\circ$$

The average slope for Section 2 (Figure 5-21) is in a dip direction near 25 degrees with a dip angle of about:

$$\text{dip} = \tan^{-1}(170 \text{ ft}/250 \text{ ft}) = 34.2^\circ \approx 34^\circ$$



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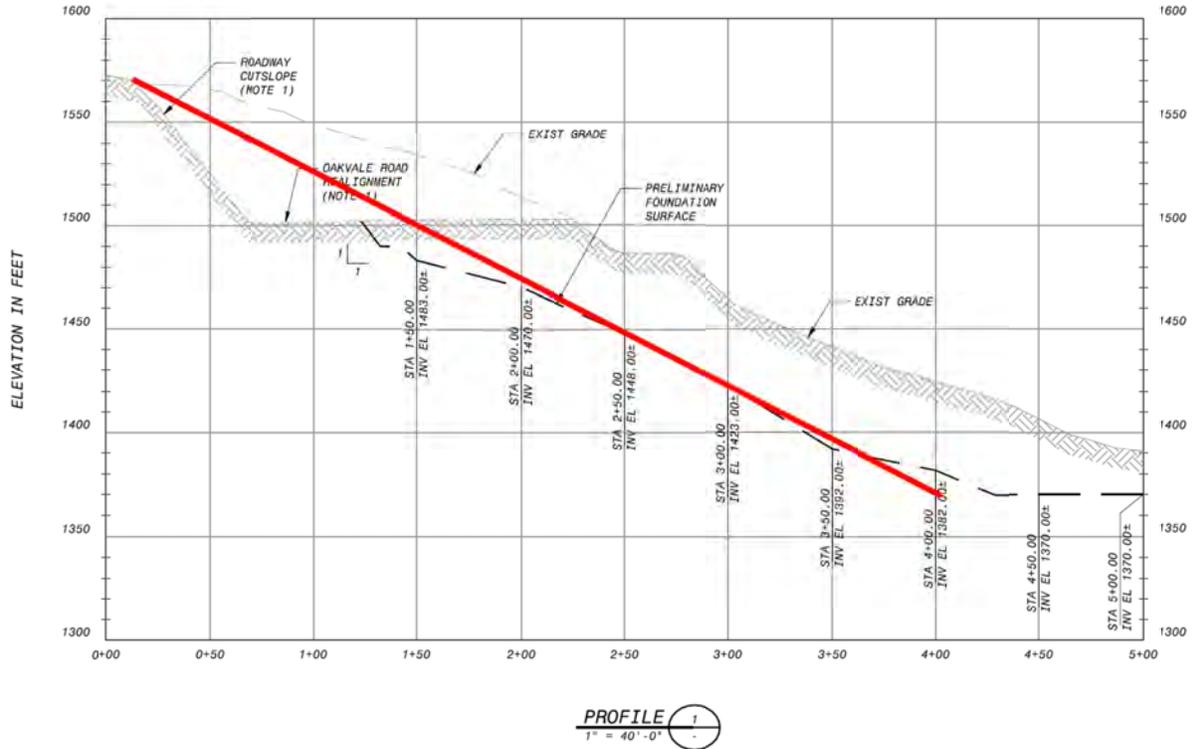


Figure 5-20 Cross Section 1 along Steepest Decent into the Excavation (Red line represents average slope)

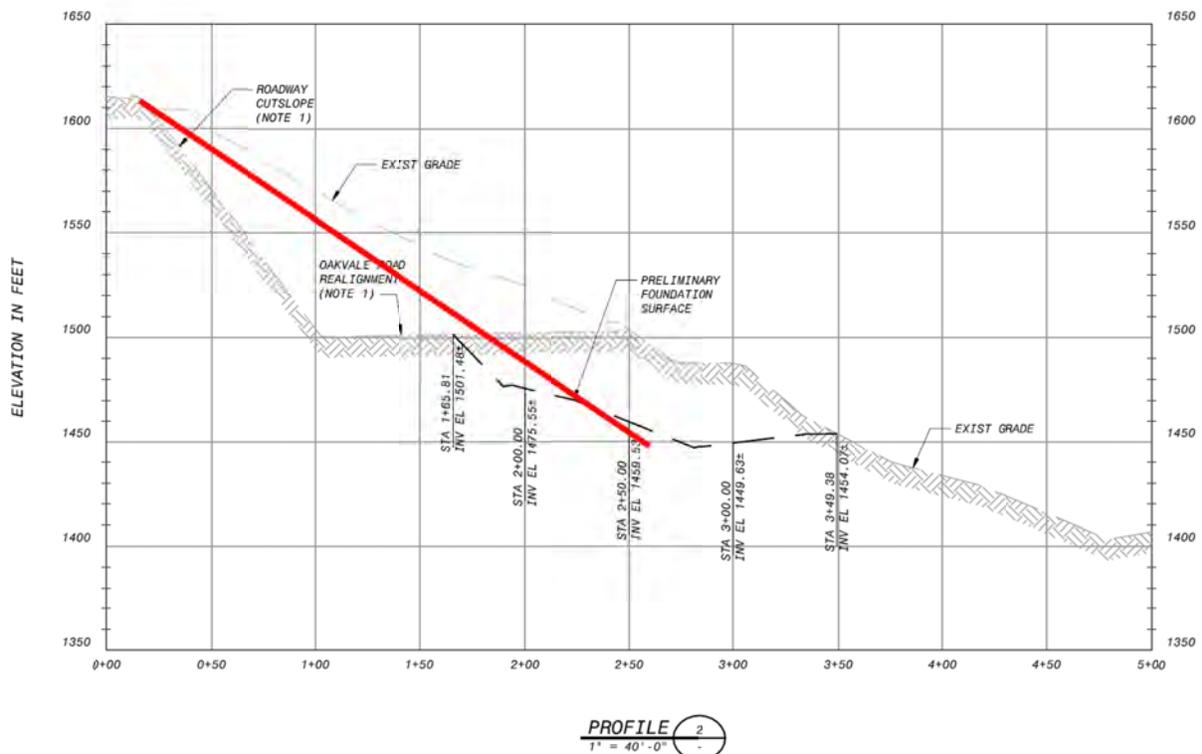


Figure 5-21 Cross Section 2 along Steepest Decent Along the Road Cut (Red line represents average slope)



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Kinematic evaluations of the rock stability indicate planar and wedge failure is similar to or less than the road cut evaluation. This reduction is mainly due to the fact the average dip is less than the dip of the road cut, which is reasonable given the fact the road forms a bench and the dam excavation is generally more horizontal. No failure envelope is present for Section 1 due to the shallow average dip, but a small potential failure envelope is present for Section 2 where discontinuities may daylight. Poles for discontinuities are present in the failure envelope for Section 2. Section 2 represents the steepest decent along the road cut into the dam excavation.

No additional evaluation of Section 1 is completed since the discontinuities will not daylight. Steeper portions of the slope above the roadway are reinforced and portions of the excavation below the roadway are either a shallower slope or immediately adjacent to the roadway (little or no rock above the failure plane).

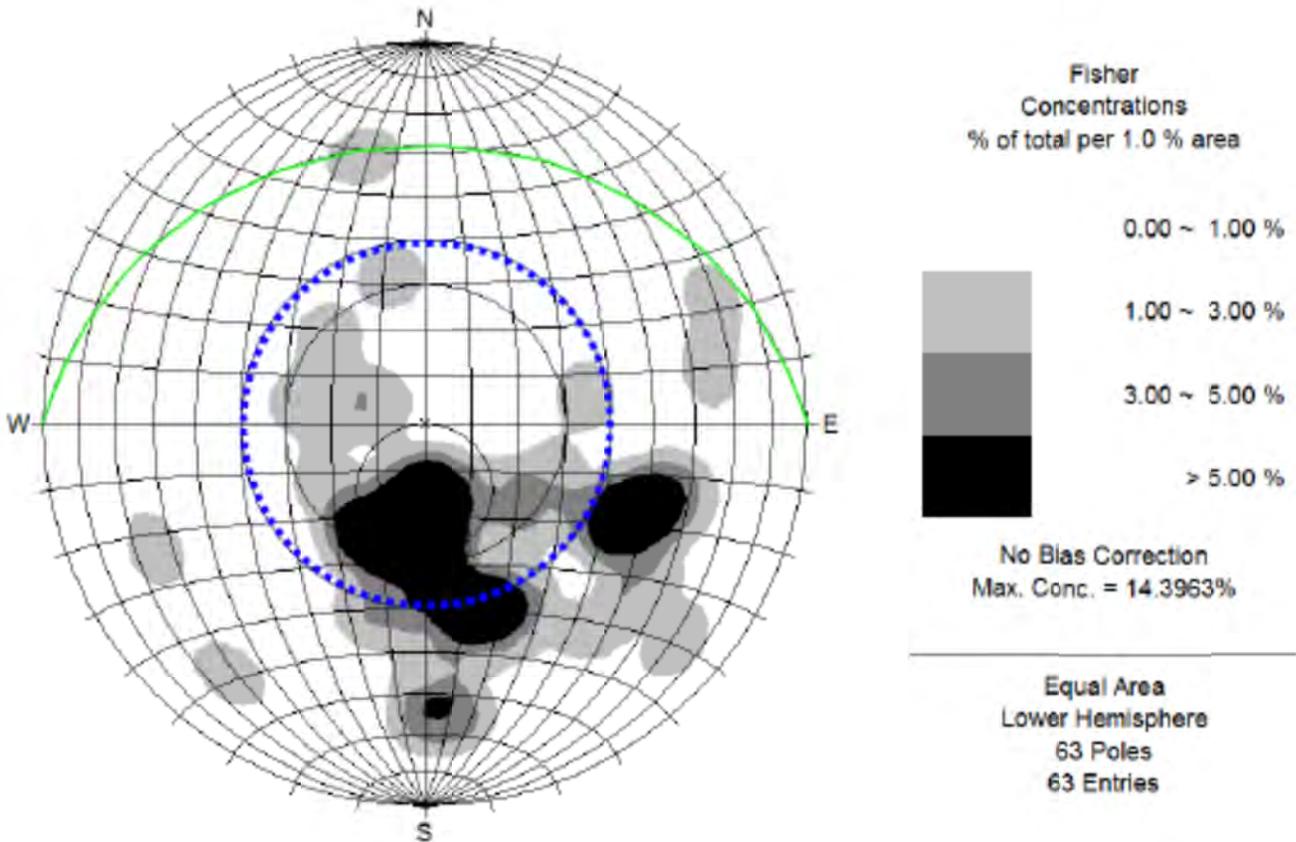


Figure 5-22 Planar Sliding Failure Evaluation for Section 1 of Global Failure (No potential failure region)

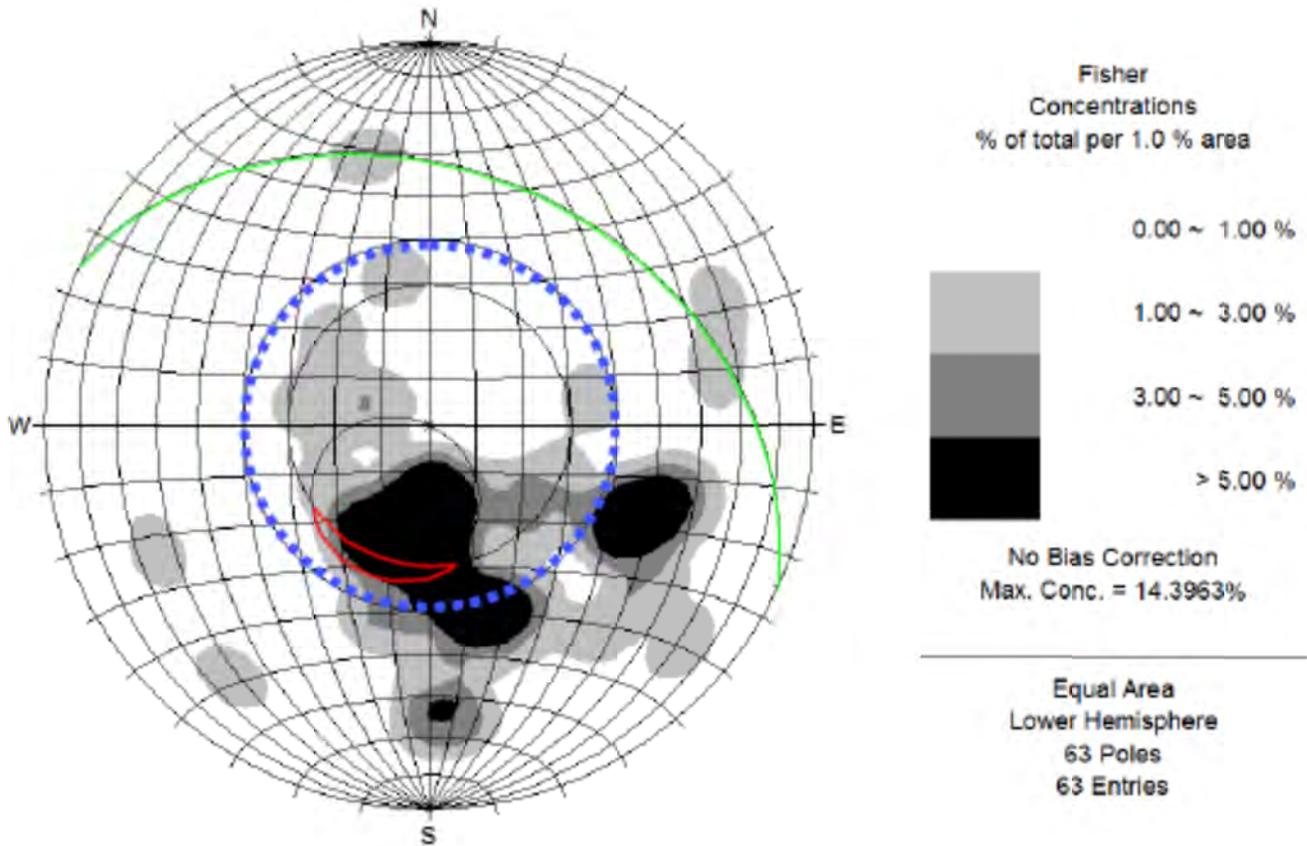


Figure 5-23 Planar Sliding Failure Evaluation for Section 2 of Global Failure (Poles in Red Box would potentially Fail)

Based on the presence of potential failure planes along Section 2, planar failure is evaluated following the procedure in Section 5.2 from reference 5 and 7. The results indicate the shallow planes that are near the potential failure envelope (about 30 degree dips) produce a factor of safety greater than 1.5 (1.60 and 1.95 in Figure 5-24). The water level is assumed to be at the road cut (about 60 feet up the 170 feet tall section), which is well above the observed water level from the subsurface investigation.

The potential contributions from the rock bolts above the roadway are not included in the planar failure analysis since the failure surface would cut beneath them. For this same reason, tension cracking is not considered since the tension crack would need to form through the reinforcement above the roadway.

Application of a horizontal earthquake coefficient equivalent to 0.145g results in a FS of 1.21 and 1.46, which are both above the required a pseudo-static FS of 1.1 (Figure 5-25).

Based on these results the slope is considered globally stable for planar failure along rock discontinuities.



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Input Values

Height of Rock Slope

H 170 feet

Height of Groundwater

Z_w 60 feet

Dip of Rock Slope Face

ψ_f 34 degrees

Dip of Discontinuity

ψ_p 30 degrees

Shear Strength of Rock

$\varphi_{initial}$ 40 degrees

φ_{final} 30 degrees

cohesion 5 psi

Unit Weight of Rock

γ_r 167 pcf

Unit Weight of Water

γ_w 62.4 pcf

Horizontal Earthquake Coefficient

α 0.00 g

Rock Bolt/Dowel Information

T 119,351 lbs/ft

θ 12 degrees (counterclockwise from normal)

$\theta_{optimal}$ 0 degrees (counterclockwise from normal)

Bolt Working Capacity

B_w 140000 lbs

Calculated Values

Base Area of Wedge

A 340.0 feet

Weight of Rock Wedge

W 602056.4 lbs/ft

Uplift Force from Water

U 112320 lbs/ft

Factor of Safety

Resisting_{initial} 588,056

Resisting_{final} 480,980

Driving 301,028

FS_{initial} 1.95

FS_{final} 1.60

Bolt Spacing on Discontinuity

S_d #DIV/0! feet

Bolt Spacing on Rock Slope Face (Law of sines)

α 4 degrees

γ 78 degrees

B 98 degrees

S_v #DIV/0! feet

S_h #DIV/0! feet

Figure 5-24 FS for Global Section 2 Rock Slope without Tension Cracking for Planar Failure



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Input Values	Calculated Values
Height of Rock Slope	Base Area of Wedge
H 170 feet	A 340.0 feet
Height of Groundwater	Weight of Rock Wedge
Z _w 60 feet	W 602056.4 lbs/ft
Dip of Rock Slope Face	Uplift Force from Water
ψ_f 34 degrees	U 112320 lbs/ft
Dip of Discontinuity	Factor of Safety
ψ_p 30 degrees	Resisting _{initial} 551,430
Shear Strength of Rock	Resisting _{final} 455,779
$\phi_{initial}$ 40 degrees	Driving 376,631
ϕ_{final} 30 degrees	FS _{initial} 1.46
cohesion 5 psi	FS _{final} 1.21
Unit Weight of Rock	Bolt Spacing on Discontinuity
γ_r 167 pcf	S _d #DIV/0! feet
Unit Weight of Water	Bolt Spacing on Rock Slope Face (Law of sines)
γ_w 62.4 pcf	α 4 degrees
Horizontal Earthquake Coefficient	γ 78 degrees
α 0.15 g	B 98 degrees
Rock Bolt/Dowel Information	S _v #DIV/0! feet
T - lbs/ft	S _h #DIV/0! feet
θ 12 degrees (counterclockwise from normal)	
$\theta_{optimal}$ 0 degrees (counterclockwise from normal)	
Bolt Working Capacity	
B _w 140000 lbs	

Figure 5-25 Results from Pseudo-Static Analysis Considering a k_h of 0.145g for Global Section 2 without Tension Cracking for Planar Failure

A kinematic evaluation is also completed for wedge failure considers the same discontinuities presented in Figures 5-8 and 5-9. Figure 5-26 and 5-27 present the kinematic evaluations for Section 1 and Section 2, respectively, for global stability using the average slope.



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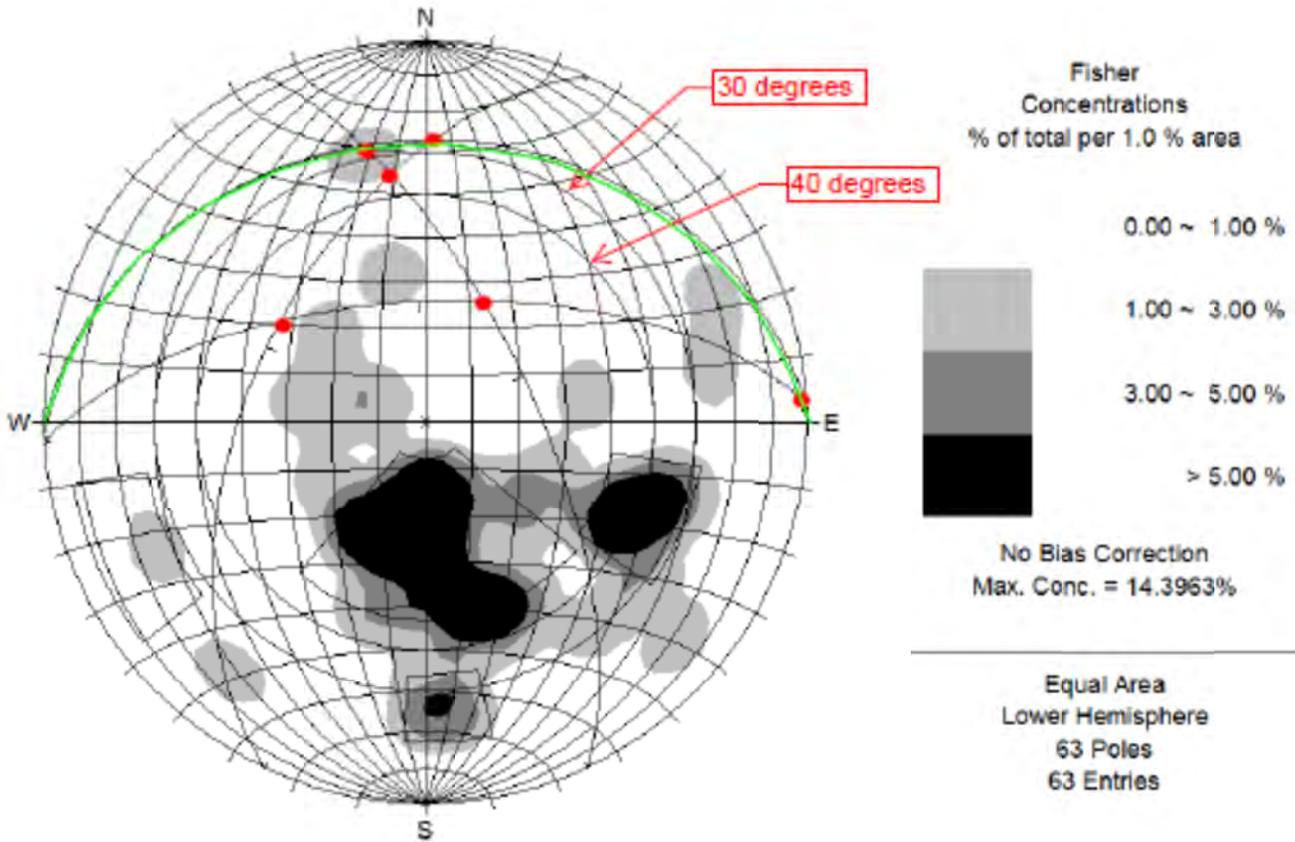


Figure 5-26 Wedge Sliding Evaluation for Global Section 1 (No potential failure region)

Since no failure region is present for Section 1, no additional evaluation is completed. For Section 2 a small potential failure region is present; however, no planes intersect within the potential failure region.

As an additional evaluation, a FS for the wedge failure is calculated using SWEDGE for Section 2. The analysis produces a FS of 2.21 and 2.62 for the two different strength parameters. These FS includes 50 percent saturation of the joints and no reinforcement, and require very persistent discontinuities that extend over hundreds of feet.

Again, the assumption for global failure is that the failure plane extends under or behind the reinforcement (i.e., rock bolts) in the road cut and does not fail through the reinforcement. Therefore, the reinforcement from the rock bolts is not included in the analyses. Attachment D provides the output from the SWEDGE analysis.



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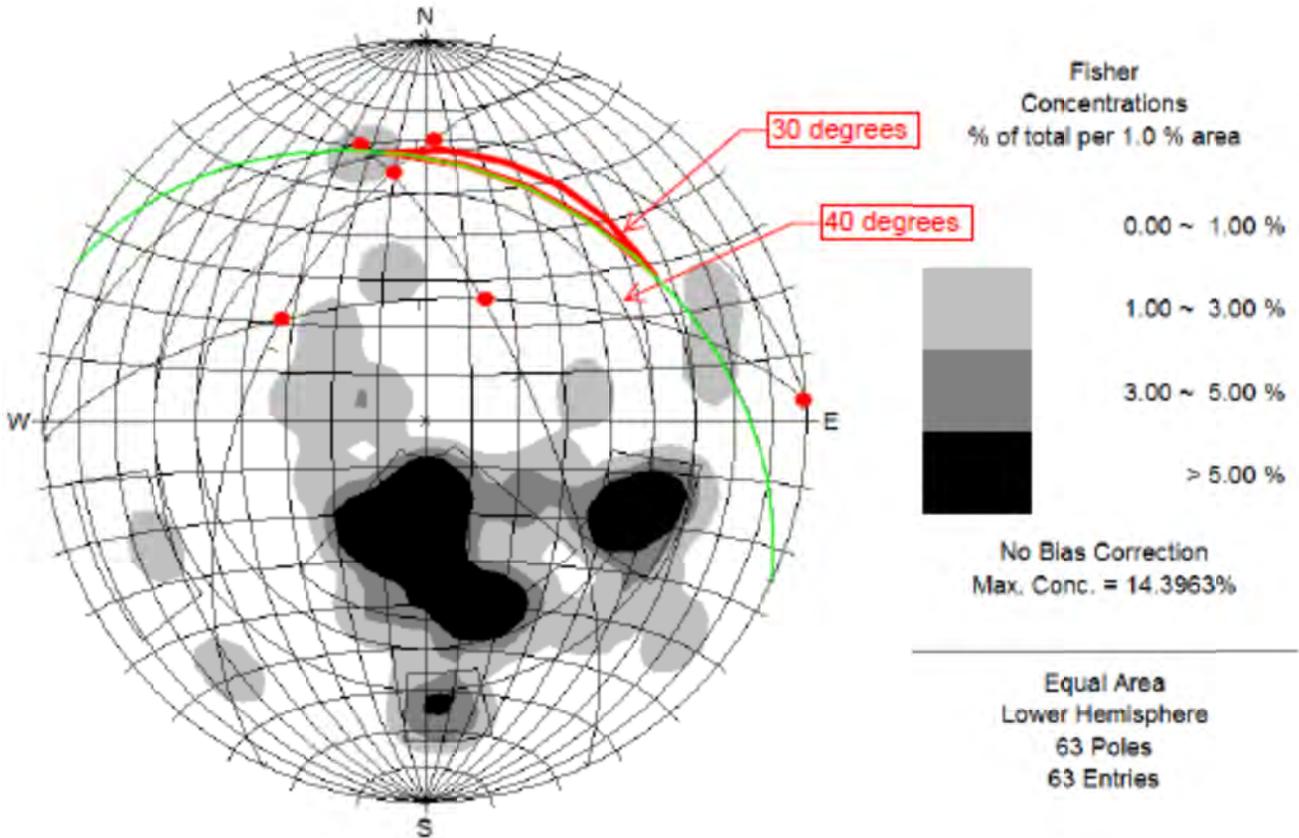


Figure 5-27 Wedge Sliding Evaluation for Global Section 2 (Small potential failure region)



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6.0 Conclusions

The analyses evaluated the FS for unsaturated rock mass failure with no tension cracking, saturated rock mass failure with tension cracking (collection of surface infiltration due to tension cracking to a depth of 15 feet below the ground surface), and the pseudo-static seismic case with a k_h value of 0.14 g. Slopes of 0.75:1 and 1:1 were considered. The controlling case is the saturated rock mass failure with tension cracking (collection of surface infiltration due to tension cracking to a depth of 15 feet below the ground surface). To achieve a FS of 1.5, about 137,000 lbs of reinforcing per linear foot of rock cut is required. This results in a rectangular grjunk id spacing of about 11.5 feet in the horizontal and vertical directions on the rock slope face.

If a total of 40 feet is needed every 11.5 feet in a grid pattern to reinforce an approximately 900 foot long slope, then the grid pattern would include about 80 bolts in each horizontal row and 10 vertical rows. The total number of bolts would be 800 which results in about 32,000 feet (=800 bolts x 40 feet/bolt) of rock bolts to reinforce the slope to a FS of 1.5 for static conditions. The actual bolt spacing would likely be refined to place more bolts at or above the slope cut (possibly 50 percent or more) and place bolts along the slope face to reinforce specific blocks and aid in placing a facing on the slope. Additionally, most of the slopes are much less than 110 feet high. Rock bolts on these slopes would be needed to reinforce the undercut portion of the slope.

As a refined estimate, the length of the road cut is divided into two portions: 1) portions where the cut height is greater than 30 feet and 2) portions where the cut height is less than 30 feet. It's estimated that about 250 feet of the cut slope is less than 30 feet in height and about 500 feet is greater than 30 feet. For the 500 feet that has a cut slope height greater than 30 feet, the assumption is a horizontal row of 50 bolts with 10 vertical rows for a total of 500 bolts. Half the bolts are assumed to be 40 feet long and half are assumed to be 30 feet long due to a reduced free-stress length on lower portions of the slope. The total rock bolt length is then 17,500 linear feet (=250 bolts * 40 feet + 250 bolts * 30 feet) for the cut slopes greater than 30 feet tall. For the cut slopes less than 30 feet tall the bolt pattern is assumed to be a 15 foot grid, or 17 bolts in a horizontal row and 3 vertical rows for a total of 51 bolts. These bolts are assumed to be 30 feet long since they will be placed on the lower portions of the slope. The total rock bolt quantity is then estimated to be about 19,000 linear feet of rock bolts (=17,500 + 51 bolts * 30 feet).

The rock bolt design should be refined following pull-out tests and the type of rock bolt anchor should be selected considering the potential for corrosion at the site. During excavation, potential discontinuities that may lead to wedge failure or small block planar failure should be identified, evaluated, and reinforced.

Global stability is confirmed in Revision 1 for the slope with the road & Dam excavation.

7.0 List of Attachments

- Attachment A – Select Pages from Reference 1 (99 pages)
- Attachment B - Reference 3 (28 pages)
- Attachment C - Reference 4 (4 pages)
- Attachment D - SWEDGE Output (11 pages)*

APPENDIX C – DRAINAGE STUDY

DRAINAGE STUDY

For

Oakvale Road Realignment

March 19, 2014

JN 121032

Prepared For:

City of Escondido

Prepared By:
O'DAY CONSULTANTS
2710 Loker Avenue West, Suite 100
Carlsbad, CA 92010



Nichole Fine
Exp. 9/30/14

C 70035

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Appendix A- 100 Year Storm Hydrologic Calculations	5
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Basin 2	12-16
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	19-26
Appendix B- Hydraulic Calculations	

EXHIBITS

Exhibit A	-	Vicinity Map
Exhibit B	-	Table 3-1 Runoff Coefficients For Urban Areas (Rational Method)
Exhibit C	-	Soil Group Map
Exhibit D	-	100-year 6-hour precipitation map
Exhibit E	-	100-year 24-hour precipitation map
Exhibit F	-	Figure 3-3 Urban Areas Overland Time of Flow Nomograph
Exhibit G	-	Figure 3-1 Intensity Duration Design Chart
Exhibit H	-	Table 3-2 Maximum Overland Flow Length and Initial Tc
Exhibit I	-	Figure 3-4 Nomograph for Tc Natural Watersheds
Exhibit J	-	40-Scale Proposed Conditions Hydrologic Map (Map Pocket)

Declaration of Responsible Charge

I hereby declare that I am the Engineer of Work for this project, that I have exercised responsible charge over the design of this project as defined in section 6703 of the Business and Professions Code, and that the design is consistent with current standards.

I understand that the check of project drawings and specifications by the County of San Diego is confined to a review only and does not relieve me, as the Engineer of Work, of my responsibilities for the project design.

O'Day Consultants, Inc.
2710 Loker Avenue West, Suite 100
Carlsbad, CA 92010
(760) 931-7700



Date: 3/19/2014

Nichole A. Fine
C 70035 Exp. 09/30/14



Introduction and Description

This drainage study was prepared to support the construction plans for the re-alignment of Oakvale Road. Oakvale Road is a rural road that runs in the east-west direction along the southwestern flank of Lake Wohlford, in the County of San Diego. See Exhibit A for a Vicinity Map.

The re-alignment of Oakvale Road is necessary because of the future replacement of Lake Wohlford Dam as required by the Federal Energy Regulatory Commission (FERC) as well as the City of Escondido. Realignment of Oakvale Road will also eliminate some of the tight existing curves. Proposed improvements consist of 6" curb & gutter, AC, an 18" storm drain, catch basin, headwall, rip-rap, and a brow ditch at the top of the proposed slope.

Existing Conditions:

Oakvale Road was constructed per San Diego County Road Survey 660. Per the as-built drawings there are 3~18" CMP culverts along the length of Oakvale Road that will be re-aligned. The culverts drain storm water from the steep southerly slopes to the north towards Lake Wohlford.

Proposed Conditions:

Approximately 925 linear feet of Oakvale Road shall be re-aligned and constructed approximately 175 feet southerly of the existing road. The cross-slope of the road is 2.0% towards the south. The longitudinal slope of the proposed road ranges from 2% to 15%. The proposed road is cut into the existing slope and has 0.75:1 slope max.

Since the section of the road remains the same from existing conditions there will be no net change in impervious surface. In addition the drainage pattern for the proposed condition mimics the existing condition, in that ultimately all the stormwater flows towards Lake Wohlford in both conditions.

This study was prepared to analyze the capacity of the existing culvert near the beginning of the improvements, to analyze the capacity of the proposed 18" RCP storm drain, and analyze the depth of flow in Oakvale Road during a 100 year storm event. See Appendix A for the proposed 100 year hydrologic calculations and Exhibit J for the drainage map.

A summary of the 100 year flowrates are as follows:

Drainage Facility	100 Year Flowrate
Existing 18" CMP near Sta 14+80	11.9 CFS
Proposed Oakvale Road (Curb & Gutter)	4.00 CFS
Proposed 18" RCP Storm Drain	15.6 CFS
Proposed Earth Swale	1.5 CFS

Per Appendix B, the maximum capacity of the D-75 Brow Ditch at 2.0% is 13.9 cfs which exceeds the maximum 100 year flowrate of 8.4 cfs in the brow ditch flowing to Node 201.

Hydrologic Calculations

The Hydrologic calculations were performed utilizing the San Diego County Rational Method as described in the San Diego County Hydrology Manual, June 2003. Pertinent exhibits from the San Diego County Hydrology Manual are enclosed for reference, as follows:

- Exhibit B - Table 3-1 Runoff Coefficients For Urban Areas (Rational Method)
- Exhibit C - Soil Group Map
- Exhibit D - 100-year 6-hour precipitation map
- Exhibit E - 100-year 24-hour precipitation map
- Exhibit F - Figure 3-3 Urban Areas Overland Time of Flow Nomograph
- Exhibit G - Figure 3-1 Intensity Duration Design Chart
- Exhibit H - Table 3-2 Maximum Overland Flow Length and Initial Tc
- Exhibit I - Figure 3-4 Nomograph for Tc Natural Watersheds
- Exhibit J - 40-Scale Proposed Conditions Hydrologic Map (Map Pocket)

APPENDIX A
100 Year Hydrologic Calculations

(See Exhibit 'J')

BASIN 1 - See Exhibit 'I' for Basin Delineation

San Diego County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software, (c)1991-2006 Version 7.7

Rational method hydrology program based on
San Diego County Flood Control Division 2003 hydrology manual
Rational Hydrology Study Date: 03/19/14

100 Year Storm Hydrologic Calculations
Oakvale Road Realignment **Basin 1**
JN 121032 Calcs By NF 3/19/14

***** Hydrology Study Control Information *****

Program License Serial Number 6218

Rational hydrology study storm event year is 100.0
English (in-lb) input data Units used

Map data precipitation entered:
6 hour, precipitation(inches) = 3.500
24 hour precipitation(inches) = 8.000
P6/P24 = 43.8%
Adjusted 6 hour precipitation (inches) = 3.600
Adjusted P6/P24 = 45.0%
San Diego hydrology manual 'C' values used

+++++
Process from Point/Station 100.000 to Point/Station 101.000
**** INITIAL AREA EVALUATION ****

Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 0.000
Decimal fraction soil group D = 1.000
[UNDISTURBED NATURAL TERRAIN]
(Permanent Open Space)
Impervious value, Ai = 0.000
Sub-Area C Value = 0.350
Initial subarea total flow distance = 220.000(Ft.)
Highest elevation = 1644.800(Ft.)
Lowest elevation = 1558.000(Ft.)
Elevation difference = 86.800(Ft.) Slope = 39.455 %
Top of Initial Area Slope adjusted by User to 1.000 %
INITIAL AREA TIME OF CONCENTRATION CALCULATIONS:
The maximum overland flow distance is 70.00 (Ft)

for the top area slope value of 1.00 %, in a development type of
Permanent Open Space

In Accordance With Figure 3-3

Initial Area Time of Concentration = 11.29 minutes

$TC = [1.8 * (1.1 - C) * distance(Ft.)^{.5}] / (\% slope^{(1/3)})$

$TC = [1.8 * (1.1 - 0.3500) * (70.000^{.5})] / (1.000^{(1/3)}) = 11.29$

Rainfall intensity (I) = 5.607(In/Hr) for a 100.0 year storm

Effective runoff coefficient used for area (Q=KCIA) is C = 0.350

Subarea runoff = 0.373(CFS)

Total initial stream area = 0.190(Ac.)

++++
Process from Point/Station 101.000 to Point/Station 102.000
**** IMPROVED CHANNEL TRAVEL TIME ****

Upstream point elevation = 1558.000(Ft.)
Downstream point elevation = 1453.100(Ft.)
Channel length thru subarea = 366.000(Ft.)
Channel base width = 2.000(Ft.)
Slope or 'Z' of left channel bank = 1.000
Slope or 'Z' of right channel bank = 1.000
Estimated mean flow rate at midpoint of channel = 4.348(CFS)
Manning's 'N' = 0.015
Maximum depth of channel = 1.000(Ft.)
Flow(q) thru subarea = 4.348(CFS)
Depth of flow = 0.148(Ft.), Average velocity = 13.698(Ft/s)
Channel flow top width = 2.296(Ft.)
Flow Velocity = 13.70(Ft/s)
Travel time = 0.45 min.
Time of concentration = 11.74 min.
Critical depth = 0.484(Ft.)

Adding area flow to channel
Rainfall intensity (I) = 5.469(In/Hr) for a 100.0 year storm
Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 0.000
Decimal fraction soil group D = 1.000
[UNDISTURBED NATURAL TERRAIN]
(Permanent Open Space)
Impervious value, Ai = 0.000
Sub-Area C Value = 0.350
Rainfall intensity = 5.469(In/Hr) for a 100.0 year storm
Effective runoff coefficient used for total area
(Q=KCIA) is C = 0.350 CA = 1.512
Subarea runoff = 7.897(CFS) for 4.130(Ac.)
Total runoff = 8.270(CFS) Total area = 4.320(Ac.)
Depth of flow = 0.217(Ft.), Average velocity = 17.171(Ft/s)
Critical depth = 0.711(Ft.)

++++
Process from Point/Station 101.000 to Point/Station 102.000
**** CONFLUENCE OF MAIN STREAMS ****

The following data inside Main Stream is listed:

In Main Stream number: 1
 Stream flow area = 4.320(Ac.)
 Runoff from this stream = 8.270(CFS)
 Time of concentration = 11.74 min.
 Rainfall intensity = 5.469(In/Hr)
 Program is now starting with Main Stream No. 2

++++
 Process from Point/Station 103.000 to Point/Station 104.000
 **** INITIAL AREA EVALUATION ****

Decimal fraction soil group A = 0.000
 Decimal fraction soil group B = 0.000
 Decimal fraction soil group C = 0.000
 Decimal fraction soil group D = 1.000
 [MEDIUM DENSITY RESIDENTIAL]
 (7.3 DU/A or Less)
 Impervious value, Ai = 0.400
 Sub-Area C Value = 0.570
 Initial subarea total flow distance = 260.000(Ft.)
 Highest elevation = 1506.200(Ft.)
 Lowest elevation = 1500.800(Ft.)
 Elevation difference = 5.400(Ft.) Slope = 2.077 %
 Top of Initial Area Slope adjusted by User to 2.000 %
 Bottom of Initial Area Slope adjusted by User to 2.242 %
 INITIAL AREA TIME OF CONCENTRATION CALCULATIONS:
 The maximum overland flow distance is 80.00 (Ft)
 for the top area slope value of 2.00 %, in a development type of
 7.3 DU/A or Less
 In Accordance With Figure 3-3
 Initial Area Time of Concentration = 6.77 minutes
 $TC = [1.8 * (1.1 - C) * \text{distance}(\text{Ft.})^{.5}] / (\% \text{ slope}^{(1/3)})]$
 $TC = [1.8 * (1.1 - 0.5700) * (80.000^{.5})] / (2.000^{(1/3)}) = 6.77$
 The initial area total distance of 260.00 (Ft.) entered leaves a
 remaining distance of 180.00 (Ft.)
 Using Figure 3-4, the travel time for this distance is 1.84 minutes
 for a distance of 180.00 (Ft.) and a slope of 2.24 %
 with an elevation difference of 4.04(Ft.) from the end of the top area
 $Tt = [11.9 * \text{length}(\text{Mi})^3] / (\text{elevation change}(\text{Ft.}))^{.385} * 60(\text{min/hr})$
 $= 1.837 \text{ Minutes}$
 $Tt = [(11.9 * 0.0341^3) / (4.04)]^{.385} = 1.84$
 Total initial area Ti = 6.77 minutes from Figure 3-3 formula plus
 1.84 minutes from the Figure 3-4 formula = 8.61 minutes
 Rainfall intensity (I) = 6.680(In/Hr) for a 100.0 year storm
 Effective runoff coefficient used for area (Q=KCIA) is C = 0.570
 Subarea runoff = 1.333(CFS)
 Total initial stream area = 0.350(Ac.)

++++
 Process from Point/Station 104.000 to Point/Station 102.000

**** STREET FLOW TRAVEL TIME + SUBAREA FLOW ADDITION ****

Top of street segment elevation = 1500.800(Ft.)
 End of street segment elevation = 1453.100(Ft.)
 Length of street segment = 510.000(Ft.)
 Height of curb above gutter flowline = 6.0(In.)
 Width of half street (curb to crown) = 14.000(Ft.)
 Distance from crown to crossfall grade break = 12.500(Ft.)
 Slope from gutter to grade break (v/hz) = 0.090
 Slope from grade break to crown (v/hz) = 0.020
 Street flow is on [1] side(s) of the street
 Distance from curb to property line = 0.500(Ft.)
 Slope from curb to property line (v/hz) = 0.020
 Gutter width = 1.500(Ft.)
 Gutter hike from flowline = 1.500(In.)
 Manning's N in gutter = 0.0150
 Manning's N from gutter to grade break = 0.0150
 Manning's N from grade break to crown = 0.0150
 Estimated mean flow rate at midpoint of street = 2.689(CFS)
 Depth of flow = 0.227(Ft.), Average velocity = 5.339(Ft/s)
 Streetflow hydraulics at midpoint of street travel:
 Halfstreet flow width = 6.575(Ft.)
 Flow velocity = 5.34(Ft/s)
 Travel time = 1.59 min. TC = 10.20 min.
 Adding area flow to street
 Rainfall intensity (I) = 5.988(In/Hr) for a 100.0 year storm
 User specified 'C' value of 0.560 given for subarea
 Rainfall intensity = 5.988(In/Hr) for a 100.0 year storm
 Effective runoff coefficient used for total area
 (Q=KCIA) is C = 0.563 CA = 0.664
 Subarea runoff = 2.645(CFS) for 0.830(Ac.)
 Total runoff = 3.978(CFS) Total area = 1.180(Ac.)
 Street flow at end of street = 3.978(CFS)
 Half street flow at end of street = 3.978(CFS)
 Depth of flow = 0.251(Ft.), Average velocity = 5.833(Ft/s)
 Flow width (from curb towards crown)= 7.815(Ft.)

++++++
 Process from Point/Station 104.000 to Point/Station 102.000
 **** CONFLUENCE OF MAIN STREAMS ****

The following data inside Main Stream is listed:

In Main Stream number: 2
 Stream flow area = 1.180(Ac.)
 Runoff from this stream = 3.978(CFS)
 Time of concentration = 10.20 min.
 Rainfall intensity = 5.988(In/Hr)
 Summary of stream data:

Stream No.	Flow rate (CFS)	TC (min)	Rainfall Intensity (In/Hr)
2	3.978	10.20	5.988

1	8.270	11.74	5.469	
2	3.978	10.20	5.988	
Qmax(1) =				
	1.000 *	1.000 *	8.270) +	
	0.913 *	1.000 *	3.978) + =	11.903
Qmax(2) =				
	1.000 *	0.869 *	8.270) +	
	1.000 *	1.000 *	3.978) + =	11.164

Total of 2 main streams to confluence:

Flow rates before confluence point:

8.270 3.978

Maximum flow rates at confluence using above data:

11.903 11.164

Area of streams before confluence:

4.320 1.180

Results of confluence:

Total flow rate = 11.903(CFS)

Time of concentration = 11.740 min.

Effective stream area after confluence = 5.500(Ac.)

End of computations, total study area = 5.500 (Ac.)

BASIN 2 - See Exhibit 'I' for Basin Delineation

San Diego County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software, (c)1991-2006 Version 7.7

Rational method hydrology program based on
San Diego County Flood Control Division 2003 hydrology manual
Rational Hydrology Study Date: 03/19/14

100 Year Storm Hydrologic Calculations
Oakvale Road Realignment Basin 2
JN 121032 Calcs By NF 3/19/14

***** Hydrology Study Control Information *****

Program License Serial Number 6218

Rational hydrology study storm event year is 100.0
English (in-lb) input data Units used

Map data precipitation entered:
6 hour, precipitation(inches) = 3.500
24 hour precipitation(inches) = 8.000
P6/P24 = 43.8%
Adjusted 6 hour precipitation (inches) = 3.600
Adjusted P6/P24 = 45.0%
San Diego hydrology manual 'C' values used

+++++
Process from Point/Station 200.000 to Point/Station 201.000
**** INITIAL AREA EVALUATION ****

Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 0.000
Decimal fraction soil group D = 1.000
[UNDISTURBED NATURAL TERRAIN]
(Permanent Open Space)
Impervious value, Ai = 0.000
Sub-Area C Value = 0.350
Initial subarea total flow distance = 500.000(Ft.)
Highest elevation = 1650.400(Ft.)
Lowest elevation = 1504.200(Ft.)
Elevation difference = 146.200(Ft.) Slope = 29.240 %
Top of Initial Area Slope adjusted by User to 2.000 %
Bottom of Initial Area Slope adjusted by User to 28.447 %
INITIAL AREA TIME OF CONCENTRATION CALCULATIONS:

The maximum overland flow distance is 85.00 (Ft)
for the top area slope value of 2.00 %, in a development type of
Permanent Open Space

In Accordance With Figure 3-3

Initial Area Time of Concentration = 9.88 minutes

$TC = [1.8 * (1.1 - C) * distance(Ft.)^{.5}] / (\% slope^{(1/3)})$

$TC = [1.8 * (1.1 - 0.3500) * (85.000^{.5})] / (2.000^{(1/3)}) = 9.88$

The initial area total distance of 500.00 (Ft.) entered leaves a
remaining distance of 415.00 (Ft.)

Using Figure 3-4, the travel time for this distance is 1.31 minutes
for a distance of 415.00 (Ft.) and a slope of 28.45 %

with an elevation difference of 118.06(Ft.) from the end of the top area

$Tt = [11.9 * length(Mi)^3] / (elevation\ change(Ft.))^{.385} * 60 (min/hr)$

= 1.314 Minutes

$Tt = [(11.9 * 0.0786^3) / (118.06)]^{.385} = 1.31$

Total initial area $Ti = 9.88$ minutes from Figure 3-3 formula plus

1.31 minutes from the Figure 3-4 formula = 11.19 minutes

Rainfall intensity (I) = 5.640(In/Hr) for a 100.0 year storm

Effective runoff coefficient used for area (Q=KCIA) is C = 0.350

Subarea runoff = 2.329(CFS)

Total initial stream area = 1.180(Ac.)

Process from Point/Station 200.000 to Point/Station 201.000
**** CONFLUENCE OF MAIN STREAMS ****

The following data inside Main Stream is listed:

In Main Stream number: 1

Stream flow area = 1.180(Ac.)

Runoff from this stream = 2.329(CFS)

Time of concentration = 11.19 min.

Rainfall intensity = 5.640(In/Hr)

Program is now starting with Main Stream No. 2

Process from Point/Station 202.000 to Point/Station 203.000
**** INITIAL AREA EVALUATION ****

Decimal fraction soil group A = 0.000

Decimal fraction soil group B = 0.000

Decimal fraction soil group C = 0.000

Decimal fraction soil group D = 1.000

[UNDISTURBED NATURAL TERRAIN]

(Permanent Open Space)

Impervious value, $A_i = 0.000$

Sub-Area C Value = 0.350

Initial subarea total flow distance = 295.000(Ft.)

Highest elevation = 1618.600(Ft.)

Lowest elevation = 1506.800(Ft.)

Elevation difference = 111.800(Ft.) Slope = 37.898 %

Top of Initial Area Slope adjusted by User to 30.000 %

Bottom of Initial Area Slope adjusted by User to 30.000 %

INITIAL AREA TIME OF CONCENTRATION CALCULATIONS:

The maximum overland flow distance is 100.00 (Ft)
for the top area slope value of 30.00 %, in a development type of
Permanent Open Space

In Accordance With Figure 3-3

Initial Area Time of Concentration = 4.34 minutes

$TC = [1.8*(1.1-C)*distance(Ft.)^0.5]/(%\ slope^{(1/3)})]$

$TC = [1.8*(1.1-0.3500)*(100.000^0.5)/(30.000^{(1/3)})] = 4.34$

The initial area total distance of 295.00 (Ft.) entered leaves a
remaining distance of 195.00 (Ft.)

Using Figure 3-4, the travel time for this distance is 0.72 minutes
for a distance of 195.00 (Ft.) and a slope of 30.00 %

with an elevation difference of 58.50(Ft.) from the end of the top area

$Tt = [11.9*length(Mi)^3/(elevation\ change(Ft.))]^{.385} *60(min/hr)$
= 0.720 Minutes

$Tt=[(11.9*0.0369^3)/(58.50)]^{.385} = 0.72$

Total initial area $Ti = 4.34$ minutes from Figure 3-3 formula plus
0.72 minutes from the Figure 3-4 formula = 5.06 minutes

Rainfall intensity (I) = 9.407(In/Hr) for a 100.0 year storm

Effective runoff coefficient used for area (Q=KCIA) is C = 0.350

Subarea runoff = 2.667(CFS)

Total initial stream area = 0.810(Ac.)

+++++
Process from Point/Station 203.000 to Point/Station 201.000
**** IMPROVED CHANNEL TRAVEL TIME ****

Upstream point elevation = 1506.800(Ft.)

Downstream point elevation = 1504.200(Ft.)

Channel length thru subarea = 136.000(Ft.)

Channel base width = 0.000(Ft.)

Slope or 'Z' of left channel bank = 1.000

Slope or 'Z' of right channel bank = 1.000

Estimated mean flow rate at midpoint of channel = 6.894(CFS)

Manning's 'N' = 0.015

Maximum depth of channel = 2.000(Ft.)

Flow(q) thru subarea = 6.894(CFS)

Depth of flow = 1.002(Ft.), Average velocity = 6.860(Ft/s)

Channel flow top width = 2.005(Ft.)

Flow Velocity = 6.86(Ft/s)

Travel time = 0.33 min.

Time of concentration = 5.39 min.

Critical depth = 1.242(Ft.)

Adding area flow to channel

Rainfall intensity (I) = 9.031(In/Hr) for a 100.0 year storm

Decimal fraction soil group A = 0.000

Decimal fraction soil group B = 0.000

Decimal fraction soil group C = 0.000

Decimal fraction soil group D = 1.000

[UNDISTURBED NATURAL TERRAIN]

(Permanent Open Space)

Impervious value, $A_i = 0.000$

Sub-Area C Value = 0.350

Rainfall intensity = 9.031(In/Hr) for a 100.0 year storm
 Effective runoff coefficient used for total area
 (Q=KCIA) is C = 0.350 CA = 1.225
 Subarea runoff = 8.396(CFS) for 2.690(Ac.)
 Total runoff = 11.063(CFS) Total area = 3.500(Ac.)
 Depth of flow = 1.197(Ft.), Average velocity = 7.721(Ft/s)
 Critical depth = 1.500(Ft.)

++++
 Process from Point/Station 203.000 to Point/Station 201.000
 **** CONFLUENCE OF MAIN STREAMS ****

The following data inside Main Stream is listed:

In Main Stream number: 2
 Stream flow area = 3.500(Ac.)
 Runoff from this stream = 11.063(CFS)
 Time of concentration = 5.39 min.
 Rainfall intensity = 9.031(In/Hr)
 Summary of stream data:

Stream No.	Flow rate (CFS)	TC (min)	Rainfall Intensity (In/Hr)
1	2.329	11.19	5.640
2	11.063	5.39	9.031
Qmax(1) =			
	1.000 *	1.000 *	2.329) +
	0.625 *	1.000 *	11.063) + = 9.239
Qmax(2) =			
	1.000 *	0.482 *	2.329) +
	1.000 *	1.000 *	11.063) + = 12.186

Total of 2 main streams to confluence:
 Flow rates before confluence point:
 2.329 11.063
 Maximum flow rates at confluence using above data:
 9.239 12.186
 Area of streams before confluence:
 1.180 3.500

Results of confluence:
 Total flow rate = 12.186(CFS)
 Time of concentration = 5.395 min.
 Effective stream area after confluence = 4.680(Ac.)

++++
 Process from Point/Station 201.000 to Point/Station 201.000
 **** SUBAREA FLOW ADDITION ****

Rainfall intensity (I) = 9.031(In/Hr) for a 100.0 year storm

User specified 'C' value of 0.900 given for subarea
Time of concentration = 5.39 min.
Rainfall intensity = 9.031(In/Hr) for a 100.0 year storm
Effective runoff coefficient used for total area
(Q=KCIA) is C = 0.362 CA = 1.728
Subarea runoff = 3.420(CFS) for 0.100(Ac.)
Total runoff = 15.606(CFS) Total area = 4.780(Ac.)

++++
Process from Point/Station 201.000 to Point/Station 204.000
**** PIPEFLOW TRAVEL TIME (Program estimated size) ****

Upstream point/station elevation = 1502.600(Ft.)
Downstream point/station elevation = 1500.000(Ft.)
Pipe length = 52.20(Ft.) Slope = 0.0498 Manning's N = 0.015
No. of pipes = 1 Required pipe flow = 15.606(CFS)
Nearest computed pipe diameter = 18.00(In.)
Calculated individual pipe flow = 15.606(CFS)
Normal flow depth in pipe = 11.82(In.)
Flow top width inside pipe = 17.09(In.)
Critical Depth = 17.00(In.)
Pipe flow velocity = 12.68(Ft/s)
Travel time through pipe = 0.07 min.
Time of concentration (TC) = 5.46 min.
End of computations, total study area = 4.780 (Ac.)

BASIN 3 - See Exhibit 'I' for Basin Delineation

San Diego County Rational Hydrology Program

CIVILCADD/CIVILDESIGN Engineering Software, (c)1991-2006 Version 7.7

Rational method hydrology program based on
San Diego County Flood Control Division 2003 hydrology manual
Rational Hydrology Study Date: 03/19/14

100 Year Storm Hydrologic Calculations
Oakvale Road Realignment Basin 3
JN 121032 calcs by NF 3/19/2014

***** Hydrology Study Control Information *****

Program License Serial Number 6218

Rational hydrology study storm event year is 100.0
English (in-lb) input data Units used

Map data precipitation entered:
6 hour, precipitation(inches) = 3.500
24 hour precipitation(inches) = 8.000
P6/P24 = 43.8%
Adjusted 6 hour precipitation (inches) = 3.600
Adjusted P6/P24 = 45.0%
San Diego hydrology manual 'C' values used

+++++
Process from Point/Station 300.000 to Point/Station 301.000
**** INITIAL AREA EVALUATION ****

Decimal fraction soil group A = 0.000
Decimal fraction soil group B = 0.000
Decimal fraction soil group C = 0.000
Decimal fraction soil group D = 1.000
[UNDISTURBED NATURAL TERRAIN]
(Permanent Open Space)
Impervious value, Ai = 0.000
Sub-Area C Value = 0.350
Initial subarea total flow distance = 100.000(Ft.)
Highest elevation = 1506.000(Ft.)
Lowest elevation = 1503.000(Ft.)
Elevation difference = 3.000(Ft.) Slope = 3.000 %
INITIAL AREA TIME OF CONCENTRATION CALCULATIONS:
The maximum overland flow distance is 100.00 (Ft)
for the top area slope value of 3.00 %, in a development type of

Permanent Open Space
 In Accordance With Figure 3-3
 Initial Area Time of Concentration = 9.36 minutes
 $TC = [1.8*(1.1-C)*distance(Ft.)^{.5}/(\% slope^{(1/3)})]$
 $TC = [1.8*(1.1-0.3500)*(100.000^{.5})/(3.000^{(1/3)})] = 9.36$
 Rainfall intensity (I) = 6.330(In/Hr) for a 100.0 year storm
 Effective runoff coefficient used for area (Q=KCIA) is C = 0.350
 Subarea runoff = 0.222(CFS)
 Total initial stream area = 0.100(Ac.)

+++++
 Process from Point/Station 301.000 to Point/Station 302.000
 **** IMPROVED CHANNEL TRAVEL TIME ****

Upstream point elevation = 1503.000(Ft.)
 Downstream point elevation = 1492.000(Ft.)
 Channel length thru subarea = 265.000(Ft.)
 Channel base width = 0.000(Ft.)
 Slope or 'Z' of left channel bank = 2.000
 Slope or 'Z' of right channel bank = 2.000
 Estimated mean flow rate at midpoint of channel = 0.901(CFS)
 Manning's 'N' = 0.023
 Maximum depth of channel = 0.500(Ft.)
 Flow(q) thru subarea = 0.901(CFS)
 Depth of flow = 0.345(Ft.), Average velocity = 3.786(Ft/s)
 Channel flow top width = 1.380(Ft.)
 Flow Velocity = 3.79(Ft/s)
 Travel time = 1.17 min.
 Time of concentration = 10.53 min.
 Critical depth = 0.418(Ft.)
 Adding area flow to channel
 Rainfall intensity (I) = 5.868(In/Hr) for a 100.0 year storm
 Decimal fraction soil group A = 0.000
 Decimal fraction soil group B = 0.000
 Decimal fraction soil group C = 0.000
 Decimal fraction soil group D = 1.000
 [UNDISTURBED NATURAL TERRAIN]
 (Permanent Open Space)
 Impervious value, Ai = 0.000
 Sub-Area C Value = 0.350
 Rainfall intensity = 5.868(In/Hr) for a 100.0 year storm
 Effective runoff coefficient used for total area
 (Q=KCIA) is C = 0.350 CA = 0.259
 Subarea runoff = 1.298(CFS) for 0.640(Ac.)
 Total runoff = 1.520(CFS) Total area = 0.740(Ac.)
 Depth of flow = 0.420(Ft.), Average velocity = 4.315(Ft/s)
 Critical depth = 0.512(Ft.)
 End of computations, total study area = 0.740 (Ac.)

APPENDIX B
Hydraulic Calculations

Channel Report

Exist. 18 inch CMP at Sta 14+80

Circular

Diameter (ft) = 1.50

Invert Elev (ft) = 0.10

Slope (%) = 10.00

N-Value = 0.015

Calculations

Compute by: Known Q

Known Q (cfs) = 11.90

Highlighted

Depth (ft) = 0.68

Q (cfs) = 11.90

Area (sqft) = 0.78

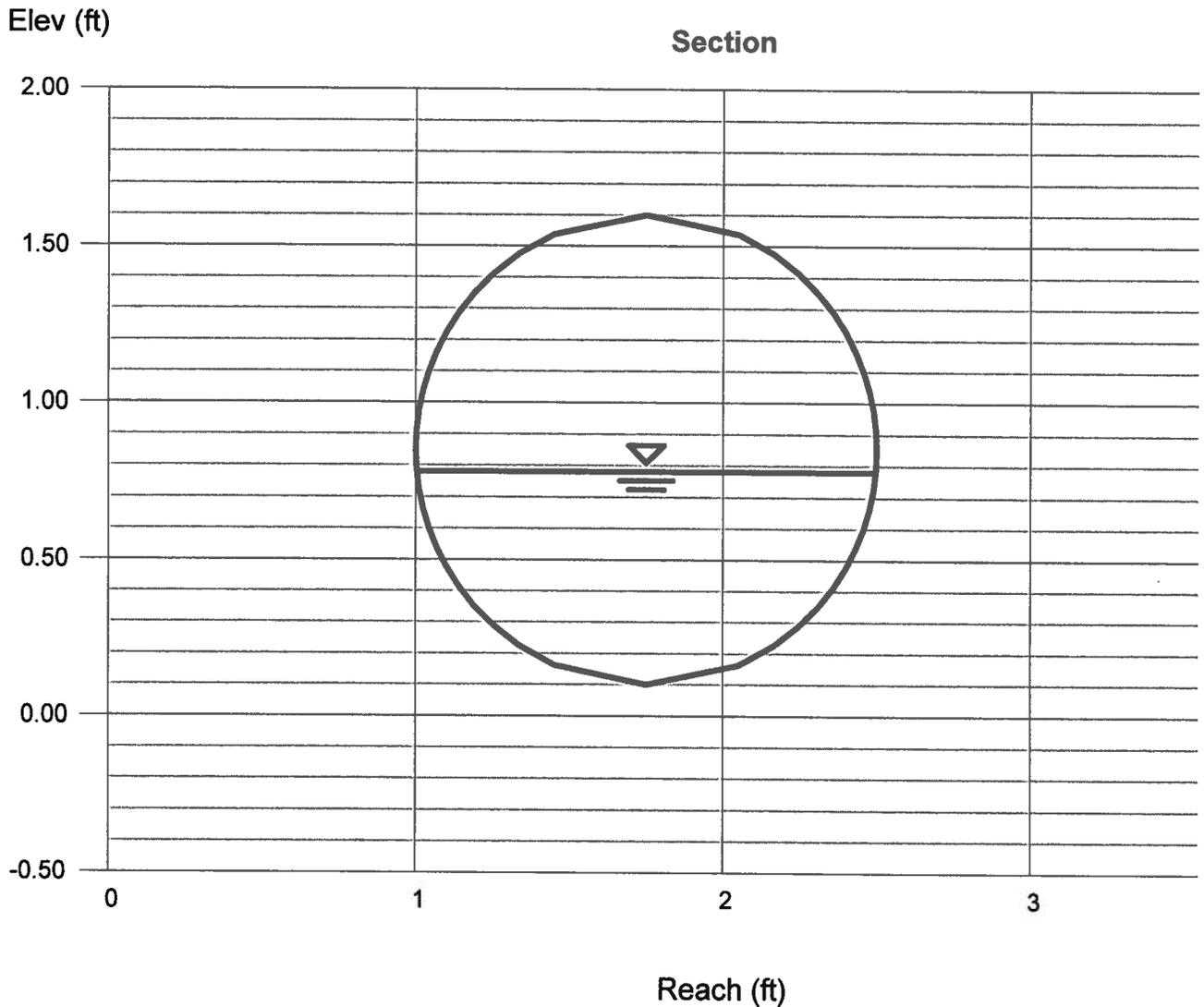
Velocity (ft/s) = 15.22

Wetted Perim (ft) = 2.22

Crit Depth, Y_c (ft) = 1.31

Top Width (ft) = 1.49

EGL (ft) = 4.28



Channel Report

Oakvale Road at 2% Section- 100 Year Storm

User-defined

Invert Elev (ft) = 0.13
 Slope (%) = 2.00
 N-Value = 0.015

Calculations

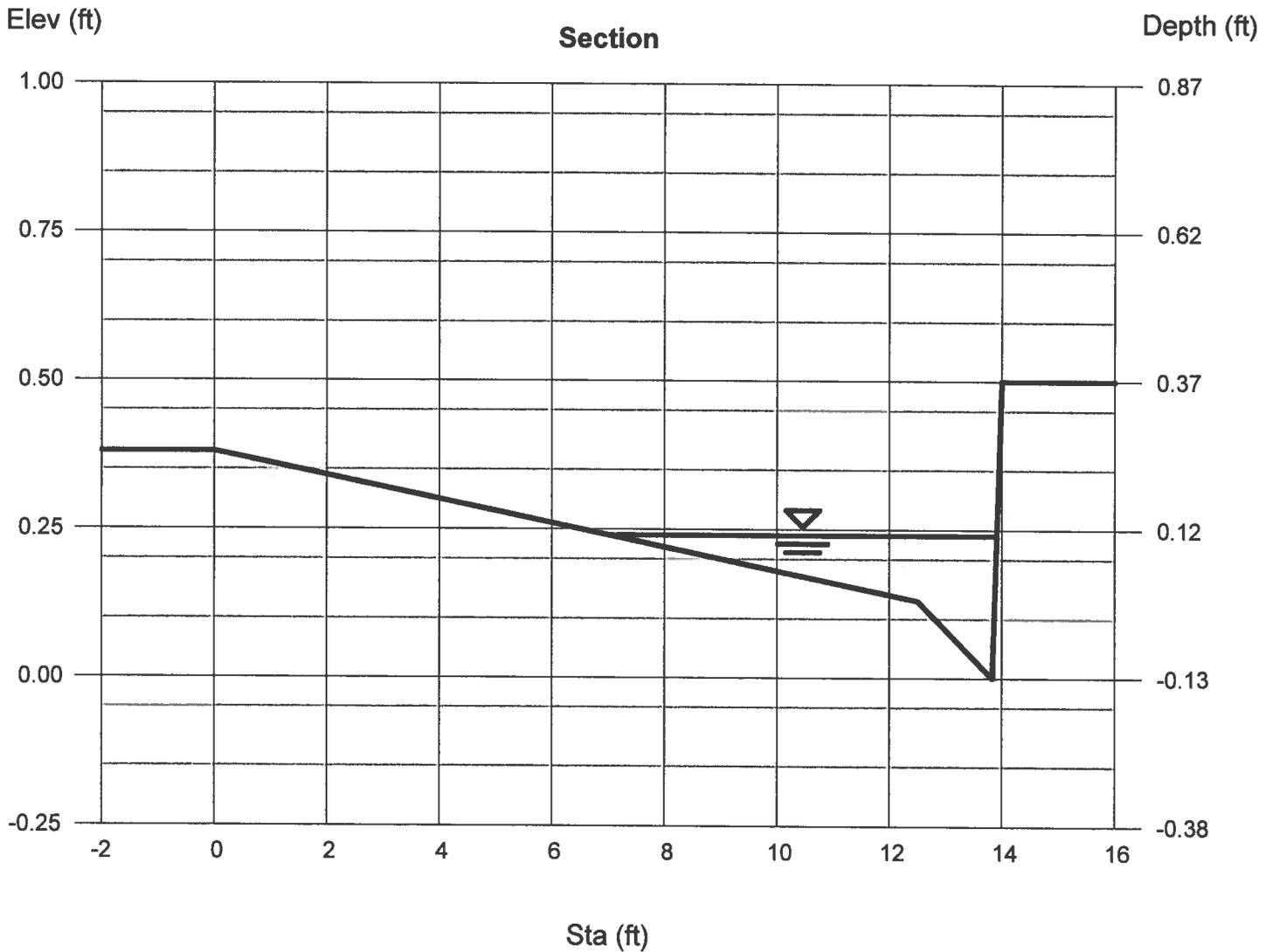
Compute by: Known Q
 Known Q (cfs) = 1.33

Highlighted

Depth (ft) = 0.11
 Q (cfs) = 1.330
 Area (sqft) = 0.55
 Velocity (ft/s) = 2.44
 Wetted Perim (ft) = 7.09
 Crit Depth, Yc (ft) = 0.14
 Top Width (ft) = 6.91
 EGL (ft) = 0.20

(Sta, El, n)-(Sta, El, n)...

(0.00, 0.38)-(12.50, 0.13, 0.015)-(14.00, 0.50, 0.015)



Channel Report

Oakvale Road at 15% Section- 100 Year Storm

User-defined

Invert Elev (ft) = 0.13
 Slope (%) = 15.00
 N-Value = 0.015

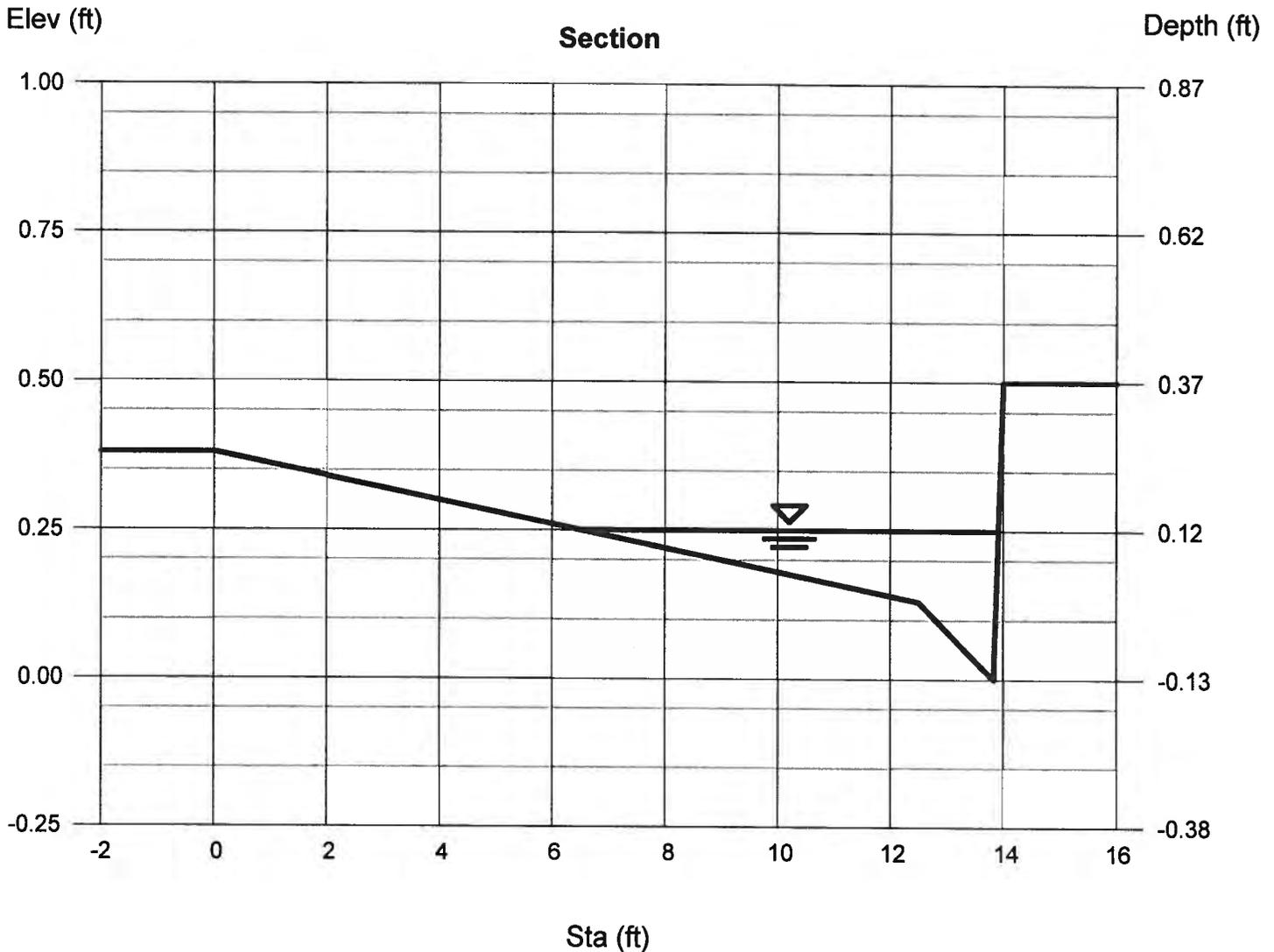
Calculations

Compute by: Known Q
 Known Q (cfs) = 4.00

Highlighted

Depth (ft) = 0.12
 Q (cfs) = 4.000
 Area (sqft) = 0.62
 Velocity (ft/s) = 6.49
 Wetted Perim (ft) = 7.60
 Crit Depth, Yc (ft) = 0.25
 Top Width (ft) = 7.41
 EGL (ft) = 0.77

(Sta, El, n)-(Sta, El, n)...
 (0.00, 0.38)-(12.50, 0.13, 0.015)-(14.00, 0.50, 0.015)



Channel Report

Proposed 18 inch RCP near STA 22+00

Circular

Diameter (ft) = 1.50

Invert Elev (ft) = 0.10

Slope (%) = 5.00

N-Value = 0.015

Calculations

Compute by: Known Q

Known Q (cfs) = 15.60

Highlighted

Depth (ft) = 0.99

Q (cfs) = 15.60

Area (sqft) = 1.24

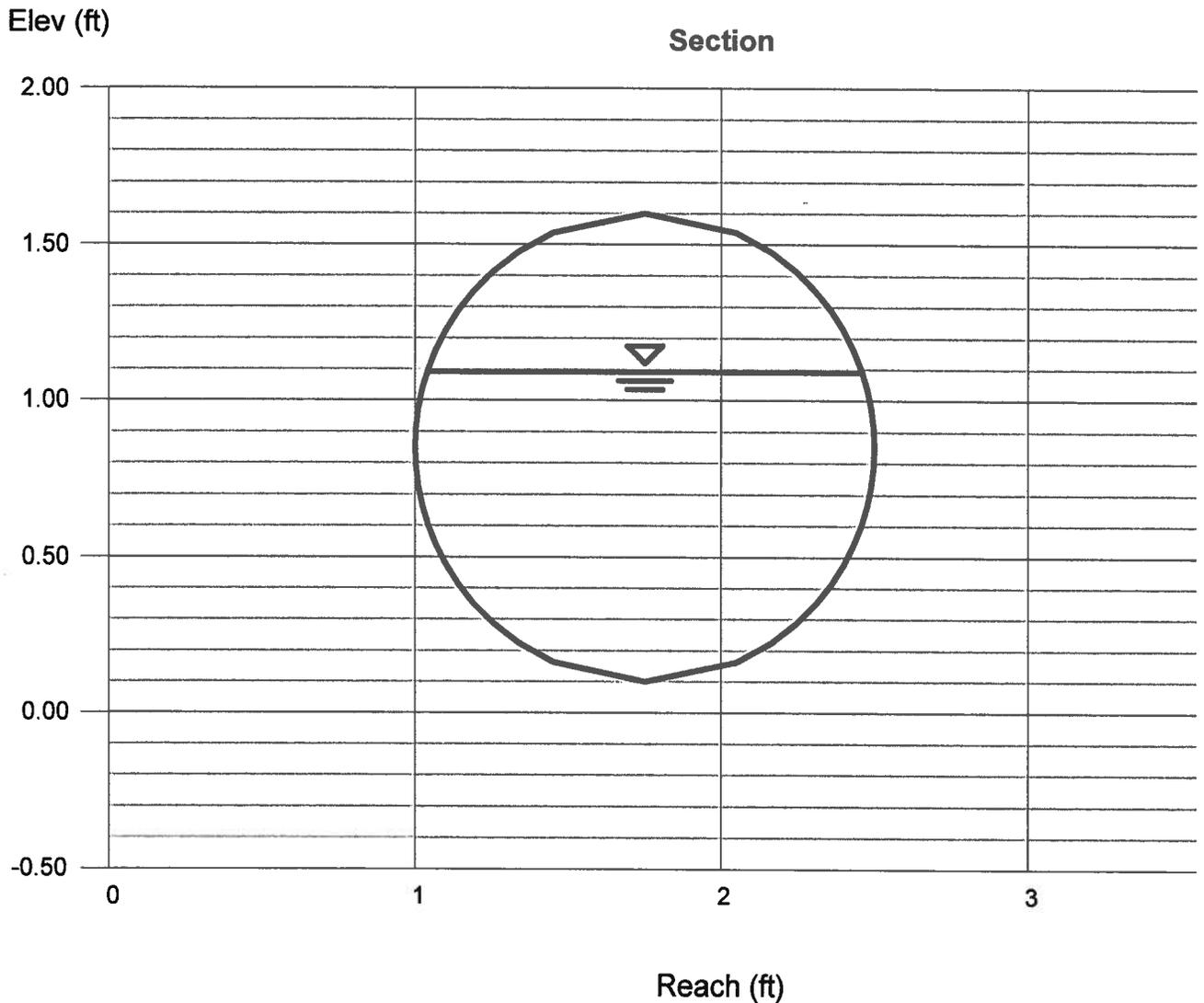
Velocity (ft/s) = 12.57

Wetted Perim (ft) = 2.85

Crit Depth, Yc (ft) = 1.42

Top Width (ft) = 1.42

EGL (ft) = 3.45



Channel Report

Temp. Earth Swale N. Side of Oakvale Road

Triangular

Side Slopes (z:1) = 2.00, 2.00
Total Depth (ft) = 1.00

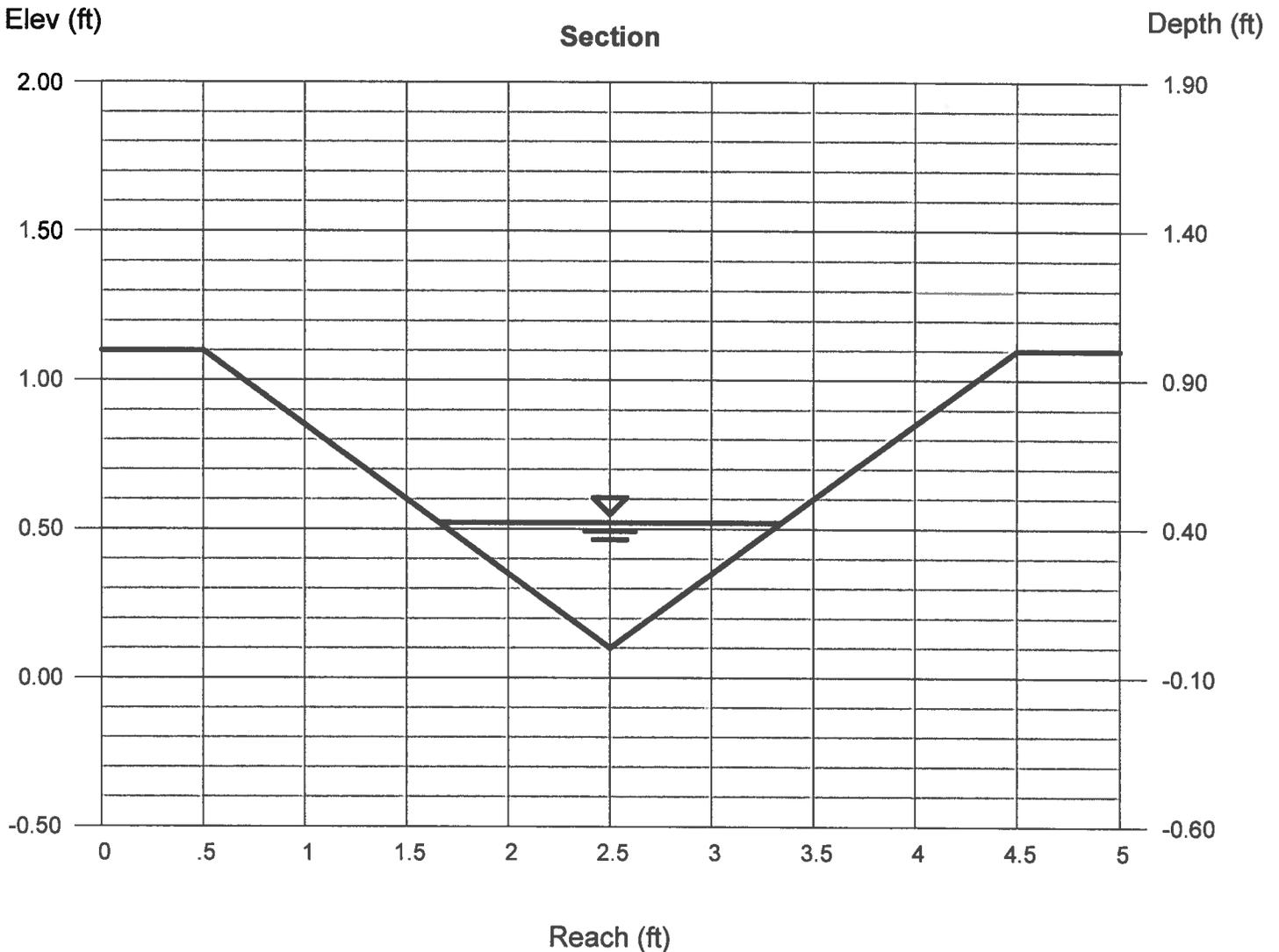
Invert Elev (ft) = 0.10
Slope (%) = 4.20
N-Value = 0.023

Calculations

Compute by: Known Q
Known Q (cfs) = 1.52

Highlighted

Depth (ft) = 0.42
Q (cfs) = 1.520
Area (sqft) = 0.35
Velocity (ft/s) = 4.31
Wetted Perim (ft) = 1.88
Crit Depth, Yc (ft) = 0.52
Top Width (ft) = 1.68
EGL (ft) = 0.71



Channel Report

Capacity of D-75 Brow Ditch

Circular

Diameter (ft) = 2.00

Invert Elev (ft) = 0.10

Slope (%) = 2.00

N-Value = 0.015

Calculations

Compute by: Known Q

Known Q (cfs) = 13.90

Highlighted

Depth (ft) = 1.00

Q (cfs) = 13.90

Area (sqft) = 1.58

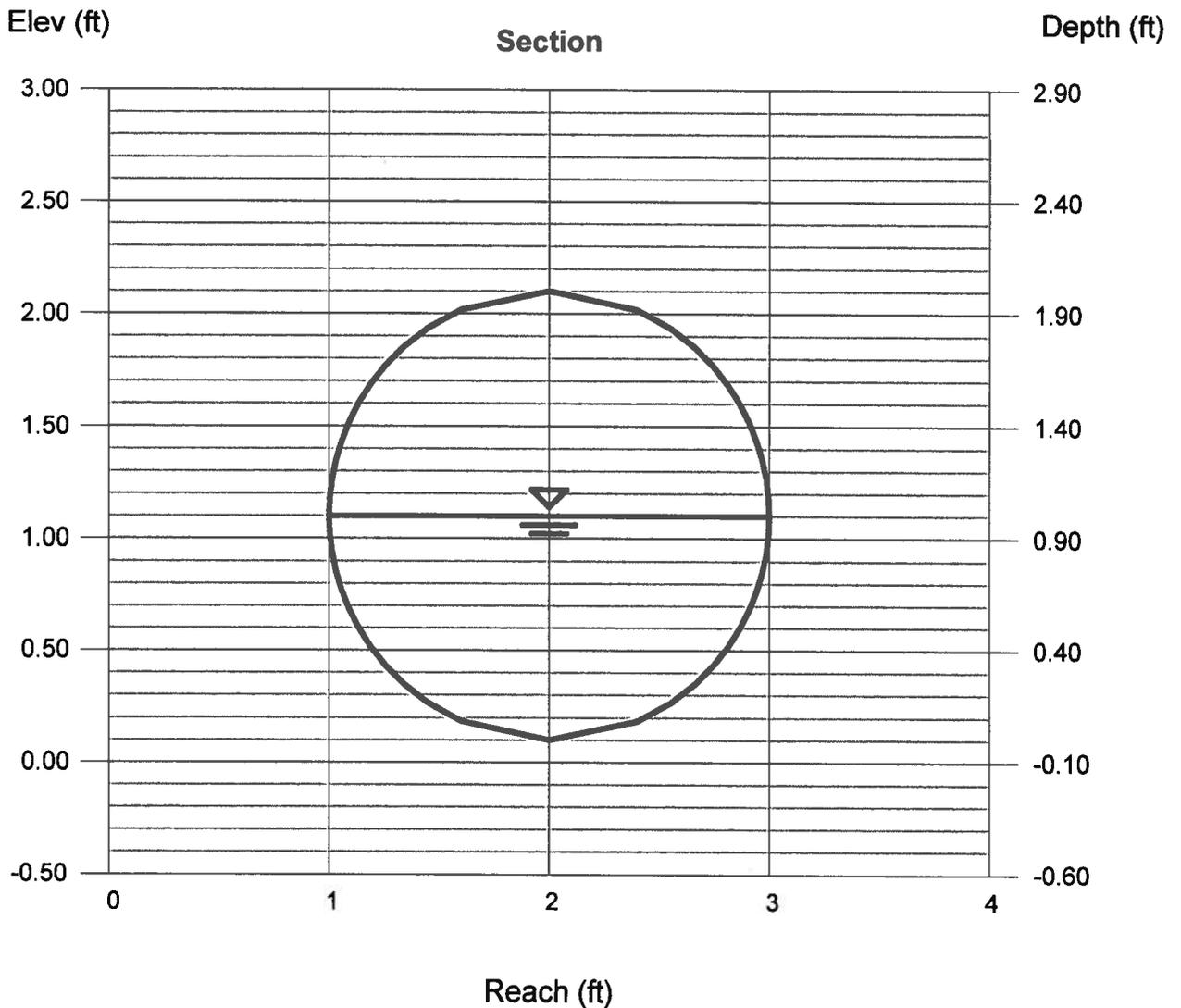
Velocity (ft/s) = 8.80

Wetted Perim (ft) = 3.15

Crit Depth, Y_c (ft) = 1.35

Top Width (ft) = 2.00

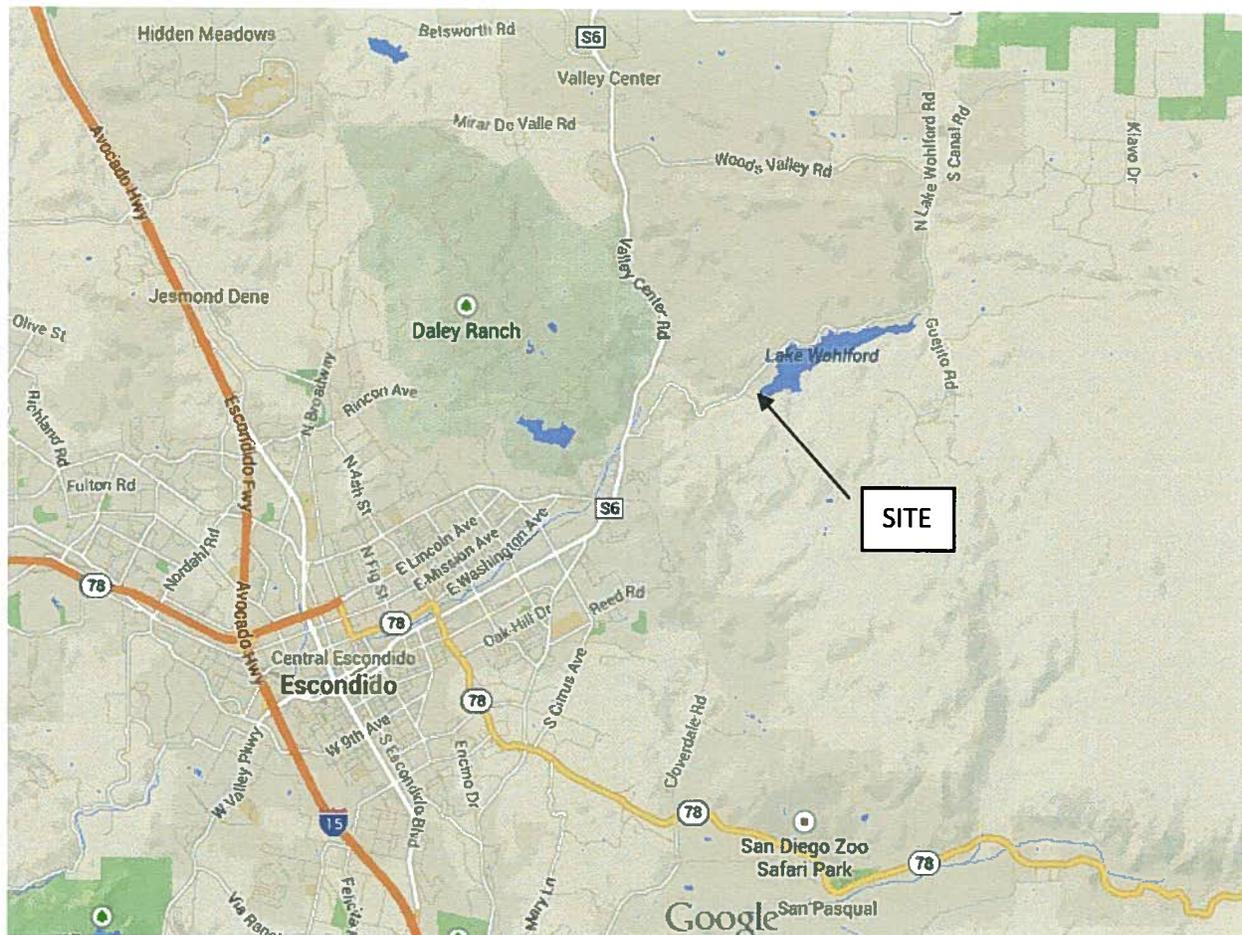
EGL (ft) = 2.20



EXHIBITS

Exhibit A

Vicinity Map



**Table 3-1
 RUNOFF COEFFICIENTS FOR URBAN AREAS**

NRCS Elements	County Elements	% IMPER.	Runoff Coefficient "C"			
			A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

NRCS = National Resources Conservation Service

**** FOR OAKVALE ROAD RUN-OFF COEFFICIENT CALLS, SEE ATTACHED**

Run-Off Coefficient Calculations for Oakvale Road Sub-Basin

Portion of Basin 1 (that includes Oakvale Road)

Total Area= 0.83ac+0.35 ac= 1.18 ac

Impervious Portion of Street= 0.44 ac

Pervious Portion of Sub-Basin= 1.18-0.44 = 0.74 ac

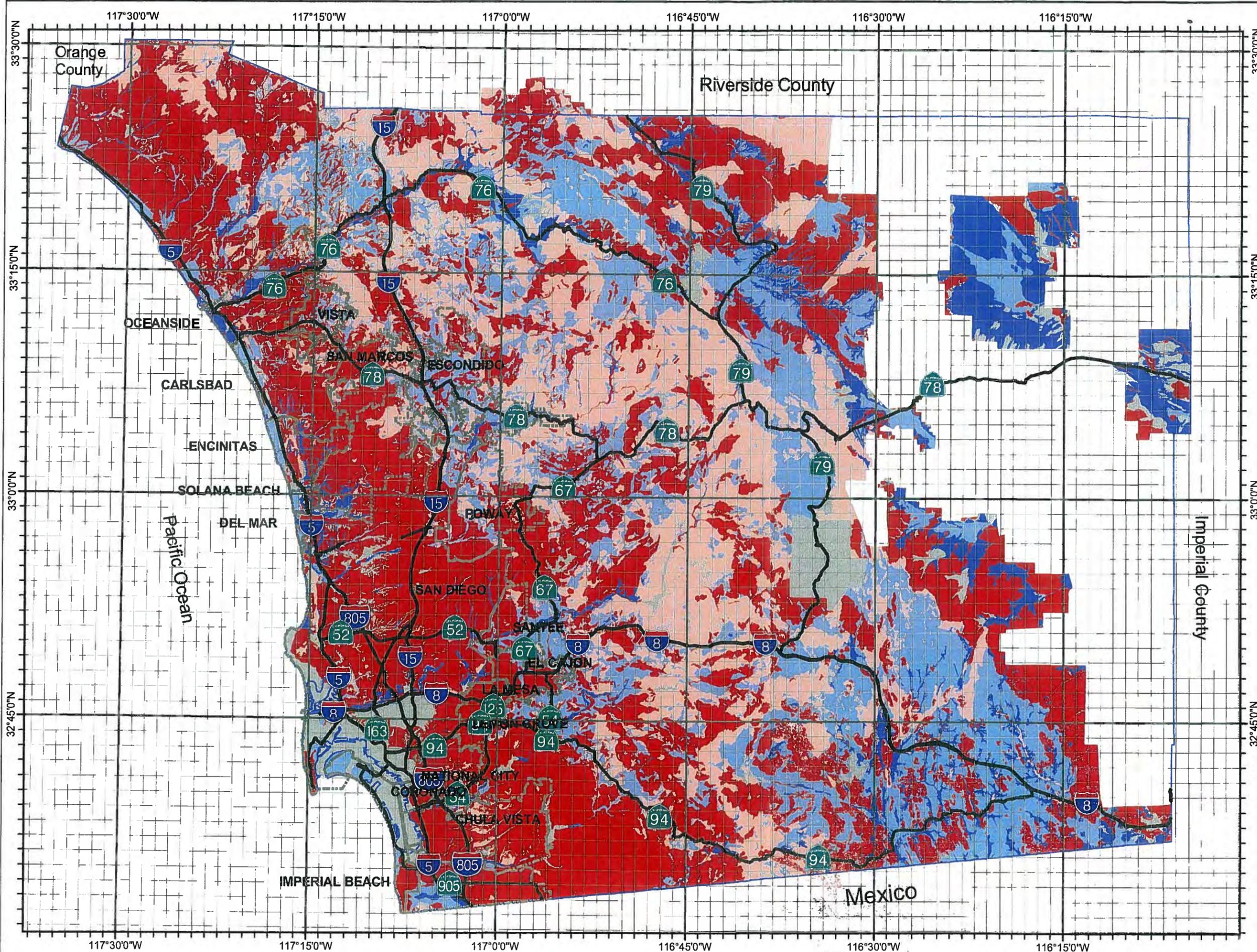
$$C = \frac{(0.44 * 0.90) + (0.74 * 0.35)}{1.18 \text{ ac}} = 0.56$$

EXHIBIT C

County of San Diego

Hydrology Manual

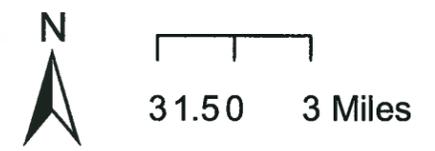
Soil Hydrologic Group



Legend

- Major Roads
- ▭ Incorporated City Bdy
- HYDROLOGIC SOIL GROUP**
- ▭ Hydrologic Group Undefined
- ▭ Hydrologic Group A
- ▭ Hydrologic Group B
- ▭ Hydrologic Group C
- ▭ Hydrologic Group D
- ▭ No Soil Data

Note: Soil Data Source
 USDA/NRCS
 SSURGO Soils 2007



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County of San Diego Hydrology Manual



Rainfall Isoplethals

100 Year Rainfall Event - 6 Hours

..... Isopleth (inches)

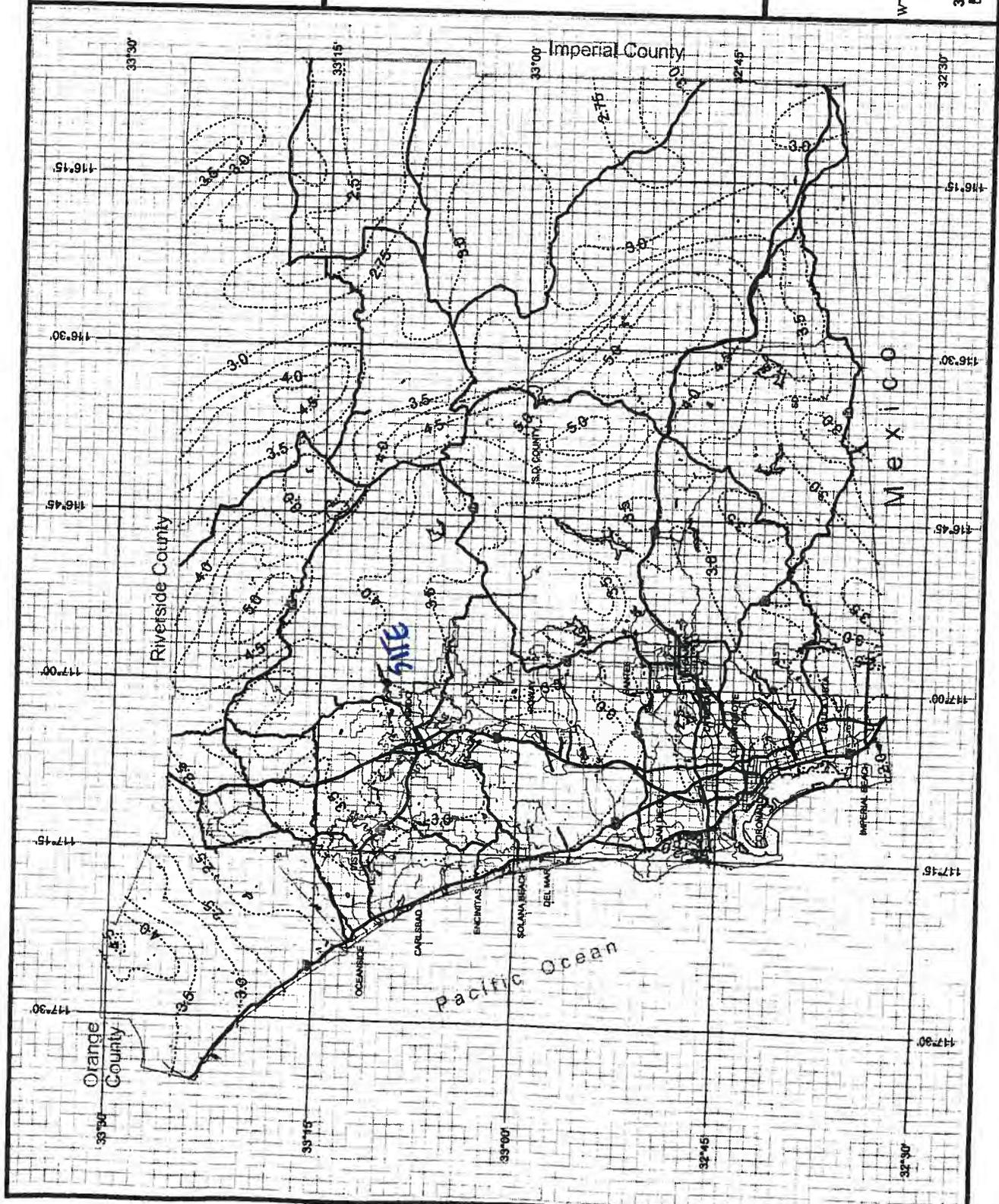
$P_6 = 3.5$



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EXHIBIT D



County of San Diego Hydrology Manual



Rainfall Isoplethials

100 Year Rainfall Event - 24 Hours



P24 = 8.0



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EXHIBIT E

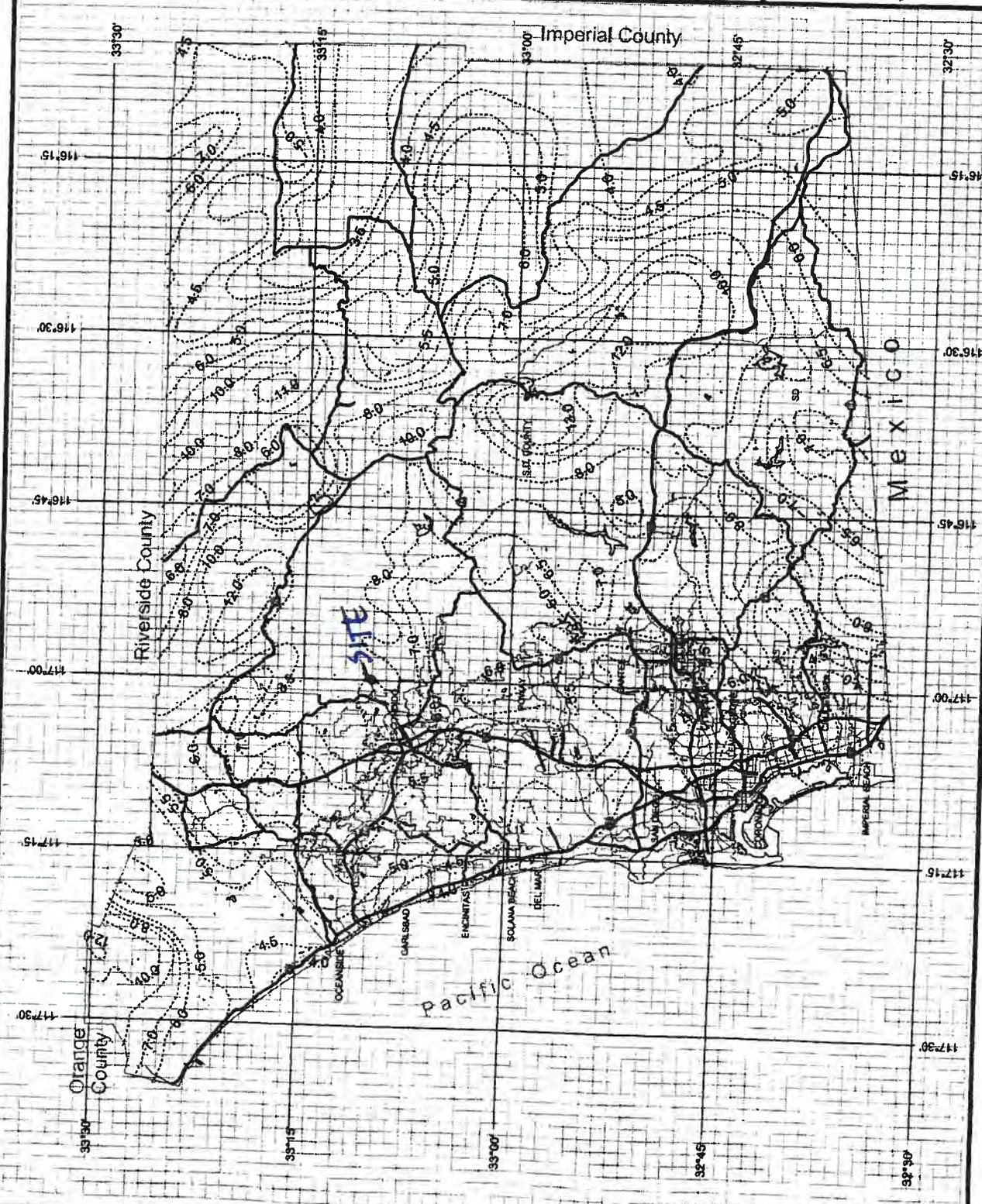
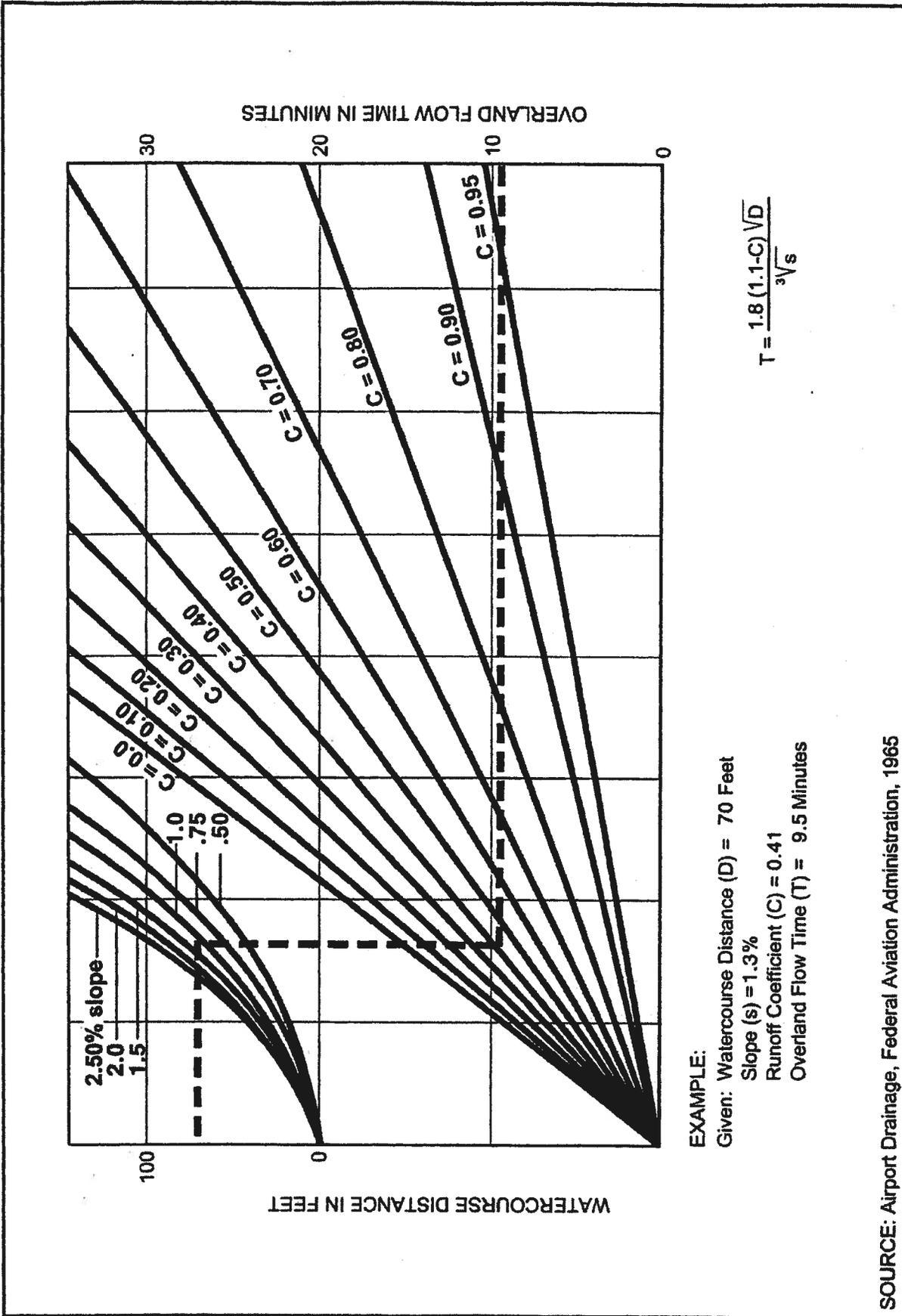


EXHIBIT F



SOURCE: Airport Drainage, Federal Aviation Administration, 1965

FIGURE

3-3

Rational Formula - Overland Time of Flow Nomograph

FIGURE
3-1

Intensity-Duration Design Chart - Template

Directions for Application:

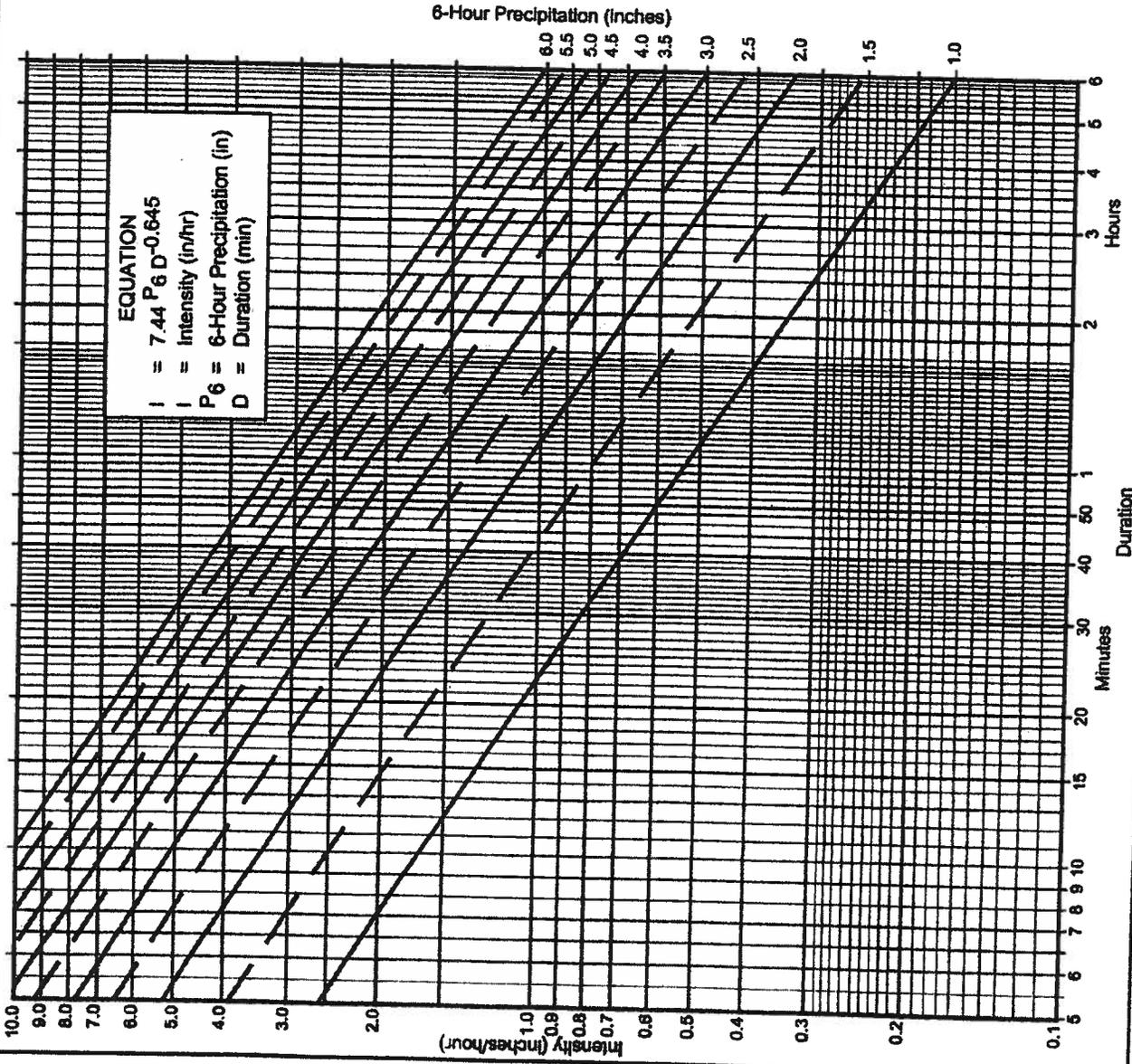
- (1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).
- (2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicable to Desert).
- (3) Plot 6 hr precipitation on the right side of the chart.
- (4) Draw a line through the point parallel to the plotted lines.
- (5) This line is the intensity-duration curve for the location being analyzed.

Application Form:

- (a) Selected frequency _____ year
- (b) $P_6 =$ _____ in., $P_{24} =$ _____, $\frac{P_6}{P_{24}} =$ _____ %⁽²⁾
- (c) Adjusted $P_6^{(2)} =$ _____ in.
- (d) $i_x =$ _____ min.
- (e) $I =$ _____ in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

P6	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.06	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.68	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
80	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
100	0.34	0.51	0.68	0.85	1.02	1.19	1.38	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00



Note that the Initial Time of Concentration should be reflective of the general land-use at the upstream end of a drainage basin. A single lot with an area of two or less acres does not have a significant effect where the drainage basin area is 20 to 600 acres.

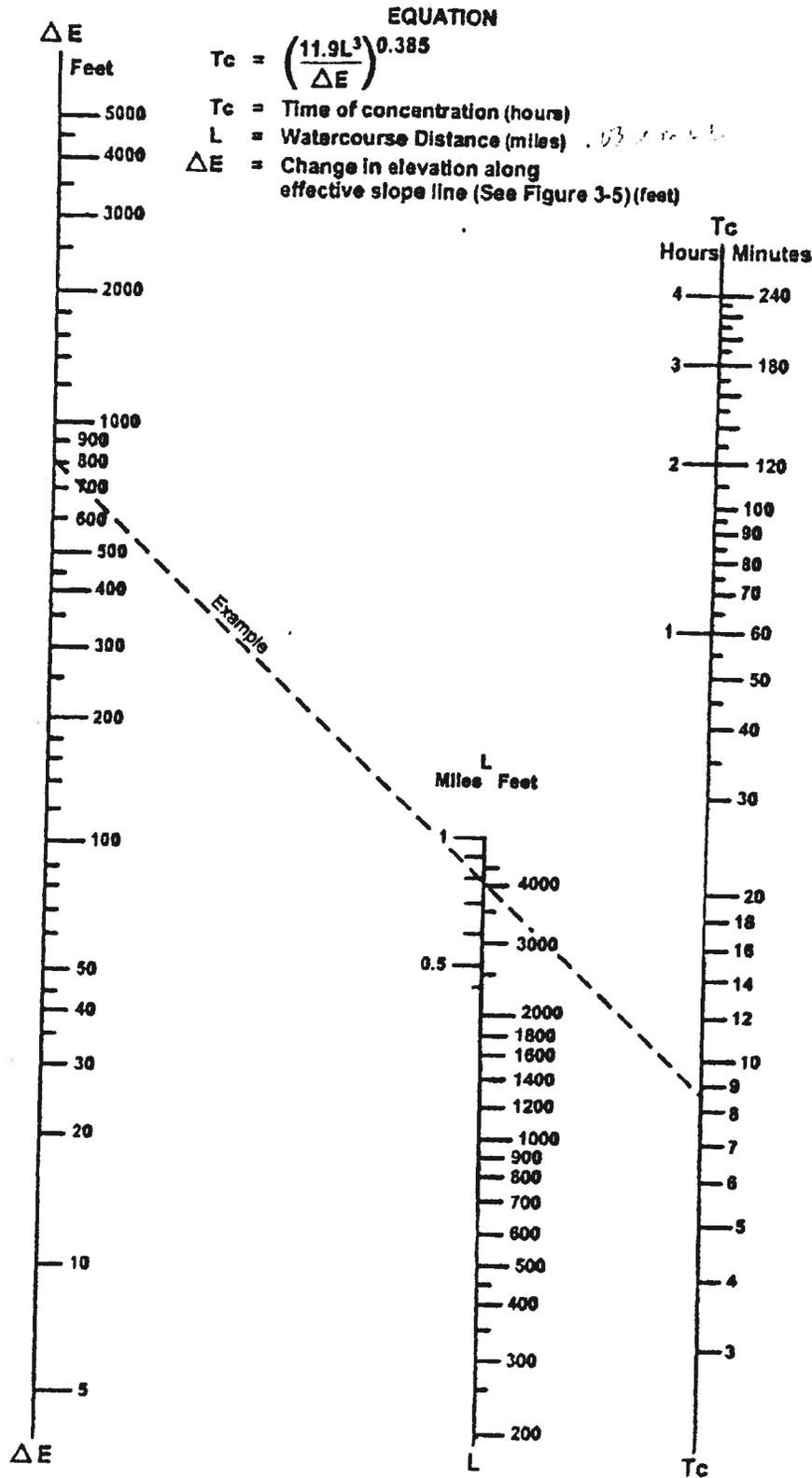
Table 3-2 provides limits of the length (Maximum Length (L_M)) of sheet flow to be used in hydrology studies. Initial T_i values based on average C values for the Land Use Element are also included. These values can be used in planning and design applications as described below. Exceptions may be approved by the "Regulating Agency" when submitted with a detailed study.

Table 3-2

**MAXIMUM OVERLAND FLOW LENGTH (L_M)
& INITIAL TIME OF CONCENTRATION (T_i)**

Element*	DU/ Acre	.5%		1%		2%		3%		5%		10%	
		L_M	T_i										
Natural		50	13.2	70	12.5	85	10.9	100	10.3	100	8.7	100	6.9
LDR	1	50	12.2	70	11.5	85	10.0	100	9.5	100	8.0	100	6.4
LDR	2	50	11.3	70	10.5	85	9.2	100	8.8	100	7.4	100	5.8
LDR	2.9	50	10.7	70	10.0	85	8.8	95	8.1	100	7.0	100	5.6
MDR	4.3	50	10.2	70	9.6	80	8.1	95	7.8	100	6.7	100	5.3
MDR	7.3	50	9.2	65	8.4	80	7.4	95	7.0	100	6.0	100	4.8
MDR	10.9	50	8.7	65	7.9	80	6.9	90	6.4	100	5.7	100	4.5
MDR	14.5	50	8.2	65	7.4	80	6.5	90	6.0	100	5.4	100	4.3
HDR	24	50	6.7	65	6.1	75	5.1	90	4.9	95	4.3	100	3.5
HDR	43	50	5.3	65	4.7	75	4.0	85	3.8	95	3.4	100	2.7
N. Com		50	5.3	60	4.5	75	4.0	85	3.8	95	3.4	100	2.7
G. Com		50	4.7	60	4.1	75	3.6	85	3.4	90	2.9	100	2.4
O.P./Com		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
Limited I.		50	4.2	60	3.7	70	3.1	80	2.9	90	2.6	100	2.2
General I.		50	3.7	60	3.2	70	2.7	80	2.6	90	2.3	100	1.9

*See Table 3-1 for more detailed description



SOURCE: California Division of Highways (1941) and Kirpich (1940)

Nomograph for Determination of
Time of Concentration (T_c) or Travel Time (T_t) for Natural Watersheds

FIGURE

3-4

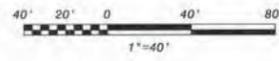
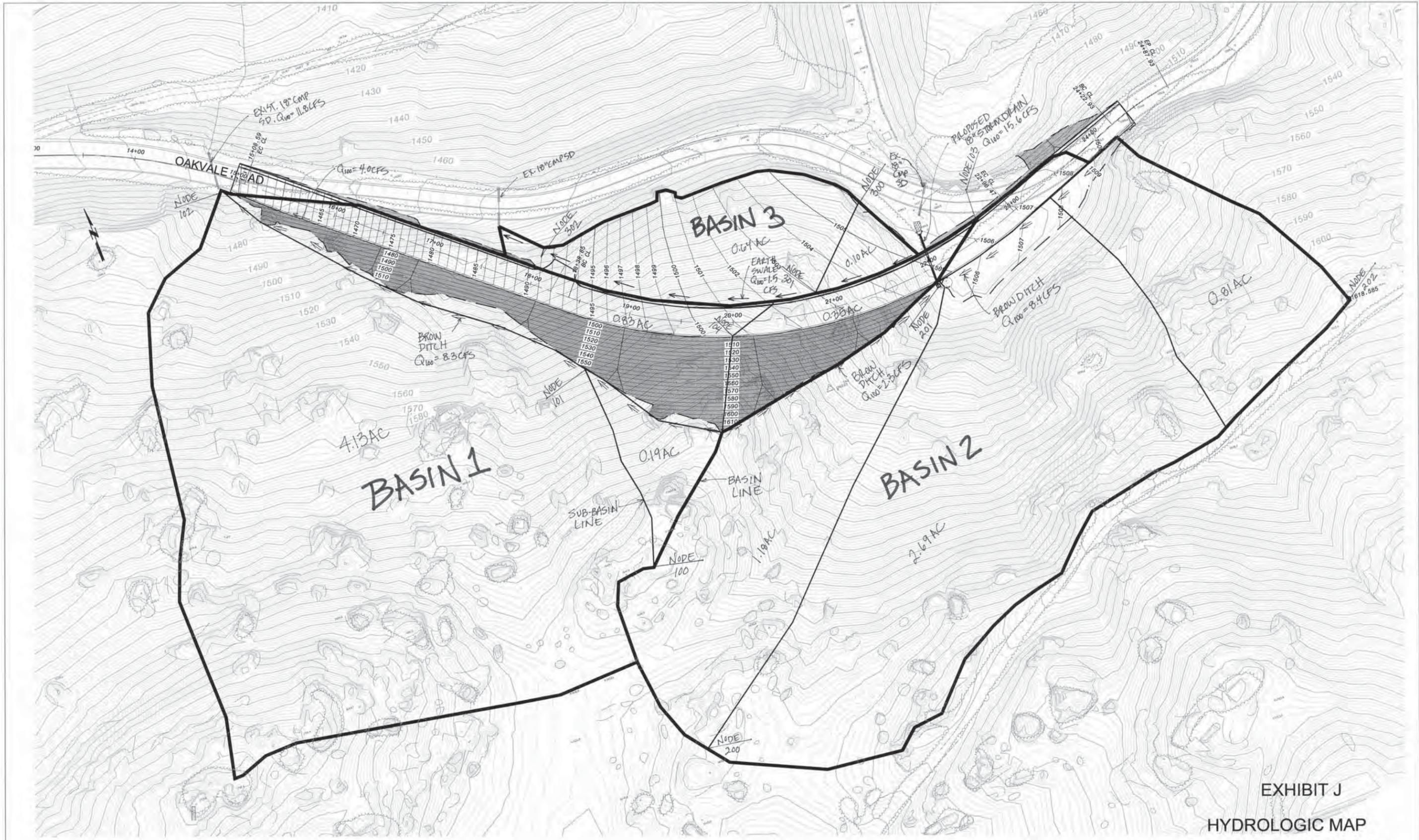


EXHIBIT J
HYDROLOGIC MAP
PROPOSED CONDITIONS
OAKVALE ROAD

APPENDIX D – COUNTY PERMITS

Appendix C

Traffic Impact Analysis

TRAFFIC IMPACT ANALYSIS
OAKVALE ROAD REALIGNMENT
Escondido, California
July 29, 2014

LLG Ref. 3-14-2324

**Linscott, Law &
Greenspan, Engineers**

4542 Ruffner Street
Suite 100

San Diego, CA 92111

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EXECUTIVE SUMMARY

The Oakvale Road Realignment project (“Project”), proposes to realign and straighten an approximately 1,200 foot stretch of Oakvale Road near the Lake Wohlford Dam. The Project site is located on Oakvale Road, approximately 750 feet east of its intersection with Lake Wohlford Road in an unincorporated area of San Diego County, just east of the City of Escondido. The Project will result in the removal of 56,000 cubic yards (cy) of material via truck to quarry locations outside of the area. The project study area includes four (4) intersections and two (2) roadway street segments. The traffic analyses for the project were conducted in accordance with the *City of Escondido’s Traffic Impact Study Guidelines*. The following scenarios are evaluated in this report:

- Existing
- Existing + Project
- Existing + Cumulative Projects
- Existing + Project + Cumulative Projects

The Project traffic generation calculations were conducted using a site-specific trip generation methodology described in detail in *Section 5.0*. Based on the number of employees, truck trips, and other Project characteristics, the Project is calculated to generate 497 daily trips with 59 trips (32 inbound/ 27 outbound) in the AM peak hour and 86 trips (30 inbound/ 56 outbound) during the PM peak hour. The duration of construction traffic will be four (4) months.

Project traffic was distributed via identified truck routes. The distribution was developed with respect to the Project’s location relative to local destinations and regional access via I-15 and the roadway characteristics and existing traffic patterns on the truck routes.

Cumulative projects were accounted for based on research conducted by LLG within the City of Escondido and County of San Diego, and an interpolated growth rate based on Year 2035 volumes and existing traffic counts was applied.

A Horizon Year (2035) analysis was not conducted since there will be no traffic volumes on the street system once the realignment of Oakvale Road is complete.

Based on the City of Escondido significance criteria, ***no significant impacts were identified***. Therefore no mitigation measures are proposed.

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- B. Intersection Methodology and Analysis Sheets
- C. City of Escondido Roadway Classification Table

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TRAFFIC IMPACT ANALYSIS
OAKVALE ROAD REALIGNMENT
Escondido, California
July 29, 2014

1.0 INTRODUCTION

Linscott, Law and Greenspan, Engineers (LLG) have prepared the following traffic impact analysis to assess the impacts to the street system as a result of the Oakvale Road Realignment project (“Project”), which proposes to realign and straighten an approximately 1,200 foot stretch of Oakvale Road near the Lake Wohlford Dam. The Project site is located on Oakvale Road, approximately 750 feet east of its intersection with Lake Wohlford Road in an unincorporated area of San Diego County, just east of the City of Escondido.

Figure 1–1 shows the Project vicinity and *Figure 1–2* illustrates, in more detail, the site location.

The traffic analysis presented in this report includes the following:

- Project Description
- Existing Conditions
- Analysis Approach and Methodology
- Significance Criteria
- Analysis of Existing Conditions
- Project Trip Generation/Distribution/Assignment
- Cumulative Projects
- Analysis of Near-term Scenarios
- Significance of Impacts and Mitigation Measures

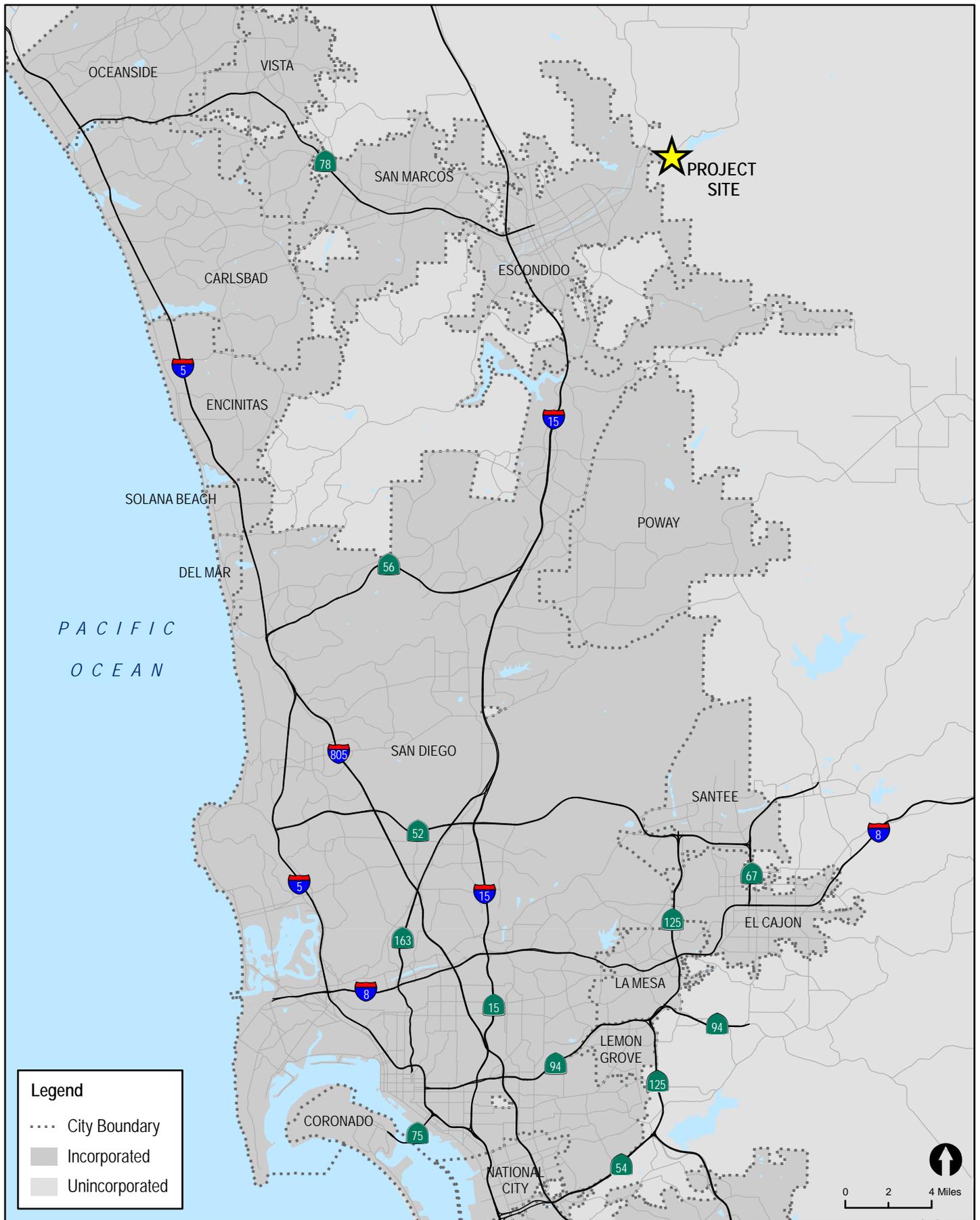


Figure 1-1

Vicinity Map

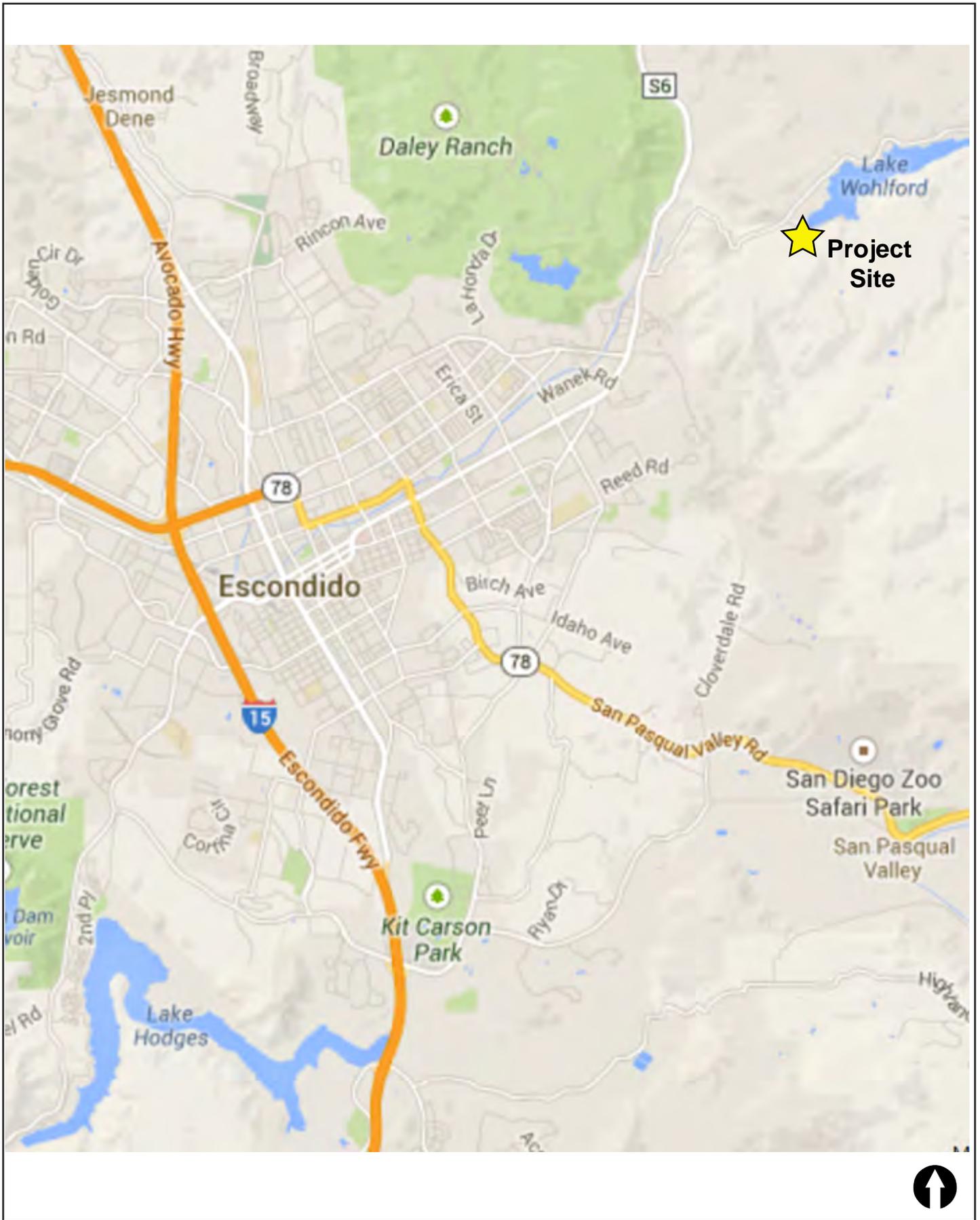


Figure 1-2

Project Area Map

2.0 PROJECT DESCRIPTION

2.1 Project Description

The City of Escondido (City) plans to realign an approximately 1,200 foot stretch of Oakvale Road near the Lake Wohlford Dam, straightening the road and moving it toward the south. This will be accomplished by excavating into the adjacent slope to remove approximately 56,000 cubic yards of rock and earth and create space for realigning the road, which would be shifted a maximum distance of approximately 150 feet to the south of the existing alignment.

Once the rock and earth is removed from the hillside, it would be stockpiled on site and loaded into 10-cubic-yard (cy) dump trucks for hauling down the mountain to a nearby quarry. For planning purposes, project engineers have indicated that the hauling phase would entail approximately 70 trips per day.

Oakvale Road would remain open to traffic during project construction, though the eastbound lane may be periodically closed to enable more room for construction vehicle access or construction staging. A traffic control plan would be prepared that would identify measures to maintain traffic safety and ensure emergency access throughout project construction.

Roadway excavation is scheduled to begin in May 2015. The hauling phase of the project is expected to last approximately four months. Final construction completion is expected in December 2015.

2.2 Project Location

The project is located on Oakvale Road, approximately 750 feet east of its intersection with Lake Wohlford Road and immediately south of Lake Wohlford Dam, in an unincorporated area of the County of San Diego east of the City of Escondido. Oakvale Road is a County of San Diego roadway. The surrounding land, including the Lake Wohlford dam and reservoir, is owned by the City of Escondido. With the exception of the Lake Wohlford Dam, the project setting is one of rural, natural open space and vegetation with dense chaparral and oak trees. The area is characterized by steep and rocky topography. Oakvale Road provides access to a small community of single-family residences located south of the dam, approximately 0.5 mile east of the project site.

3.0 EXISTING CONDITIONS

3.1 Study Area

The study area was determined in accordance with the City of Escondido's published *Traffic Impact Analysis Requirement Guidelines (2014)*. Further details on the City's guidelines for developing the study area can be found in *Section 4.0*. The study area includes the following four (4) existing public intersections and two (2) street segments.

Intersections:

1. Lake Wohlford Road / Oakvale Road
2. Lake Wohlford Road / Valley Center Road
3. Valley Parkway / El Norte Parkway
4. Valley Parkway / Bear Valley Parkway

Segments:

Lake Wohlford Road

Valley Center Road to Oakvale Road

Valley Parkway

El Norte Parkway to Lake Wohlford Road

3.2 Existing Transportation Conditions

The following is a brief description of the streets in the Project area. Roadway classifications are taken from the City of Escondido's *General Plan Mobility Element (2011)*.

Lake Wohlford Road is a generally east/west facility with portions in both the City of Escondido and the unincorporated area of the County of San Diego. The majority of the roadway in the Project vicinity is within the City of Escondido where it is classified as a Local Collector. It is currently constructed as a two-lane undivided roadway with narrow shoulders and no passing lanes, through generally steep and mountainous terrain. The posted speed limit is 50 mph.

Valley Parkway is a north/south roadway within the City of Escondido and is classified as a Prime Arterial in the vicinity of the Project. From Bear Valley Parkway to Beven Drive, Valley Parkway is currently built as a five-lane divided roadway. From Beven Drive to Lake Wohlford Road, Valley Parkway transitions to a two-lane roadway with a two-way left-turn lane median. The posted speed limit is 45 mph.

Bear Valley Parkway is a north/south facility in the City of Escondido with varying classifications. In the Project study area, from Valley Parkway to Boyle Avenue it is currently constructed as a four-lane divided roadway and classified as a Major Road. The posted speed limit is 45 mph north of Boyle Avenue. Curbside parking is prohibited. Bear Valley Parkway provides Class II bicycle lanes from Valley Parkway to Boyle Avenue.

El Norte Parkway is currently built as a Four-Lane Collector west of Valley Parkway to Washington Avenue. Bike lanes and bus stops are provided on El Norte Parkway in the study area.

Figure 3–1 shows an existing conditions diagram, including signalized/unsignalized intersections and lane configurations.

3.3 Existing Traffic Volumes

Table 3–1 is a summary of the most recent available ADTs from LLG counts commissioned in February and March 2014 when schools were in session. Manual hand counts at the study area intersections were also conducted in February and March 2014.

**TABLE 3–1
EXISTING TRAFFIC VOLUMES**

Street Segment	ADT ^a
Lake Wohlford Road Valley Center Road to Oakvale Road	4,680
Valley Parkway El Norte Parkway to Lake Wohlford Road	29,700 ^b

Footnotes:

- a. Average Daily Traffic Volume counts conducted in February and March 2014 by LLG Engineers.
- b. Volume based on Escondido General Plan Update Traffic Impact Analysis, December 5, 2011

Figure 3–2 shows the Existing Traffic Volumes. *Appendix A* contains the manual count sheets.

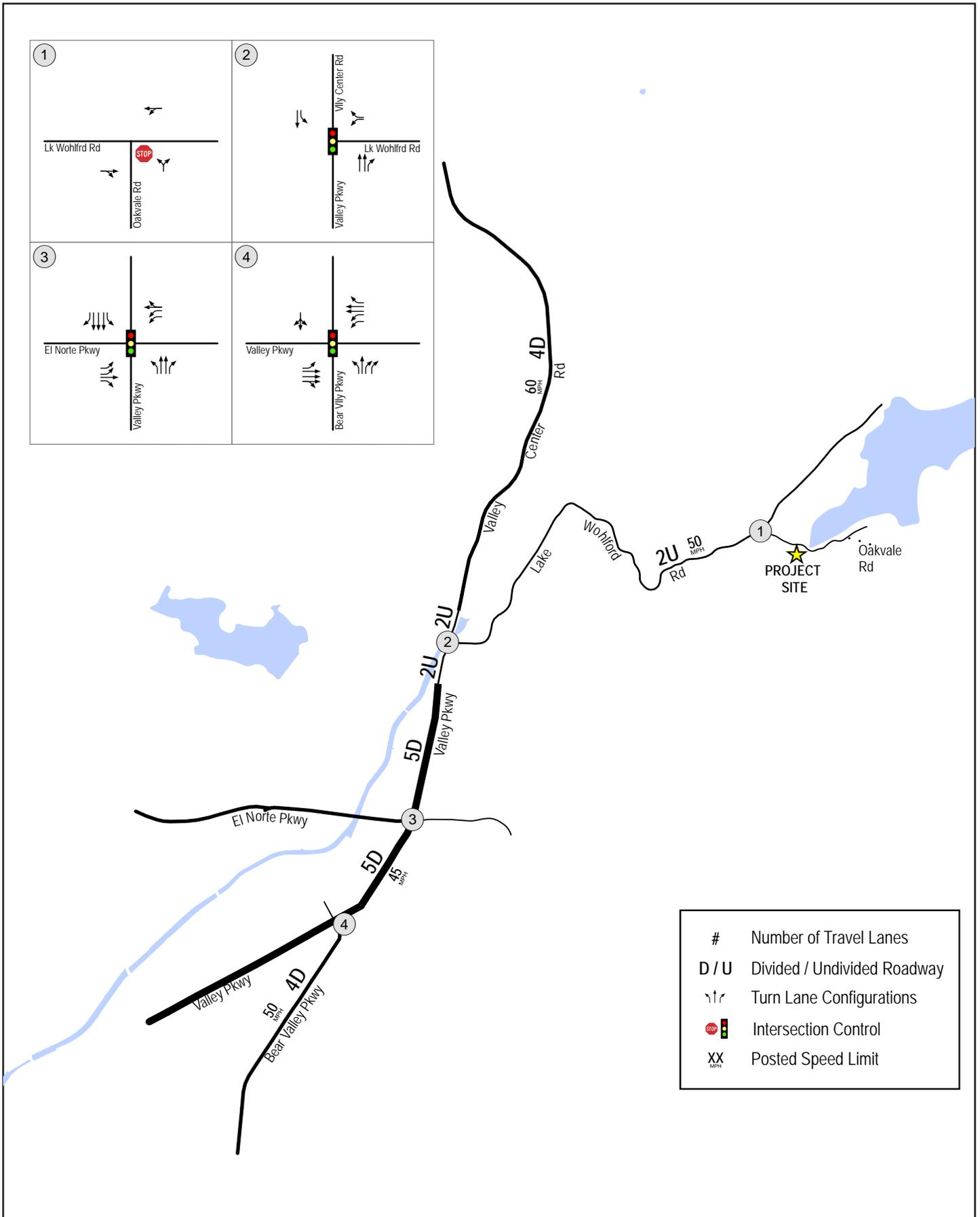


Figure 3-1

Existing Conditions Diagram

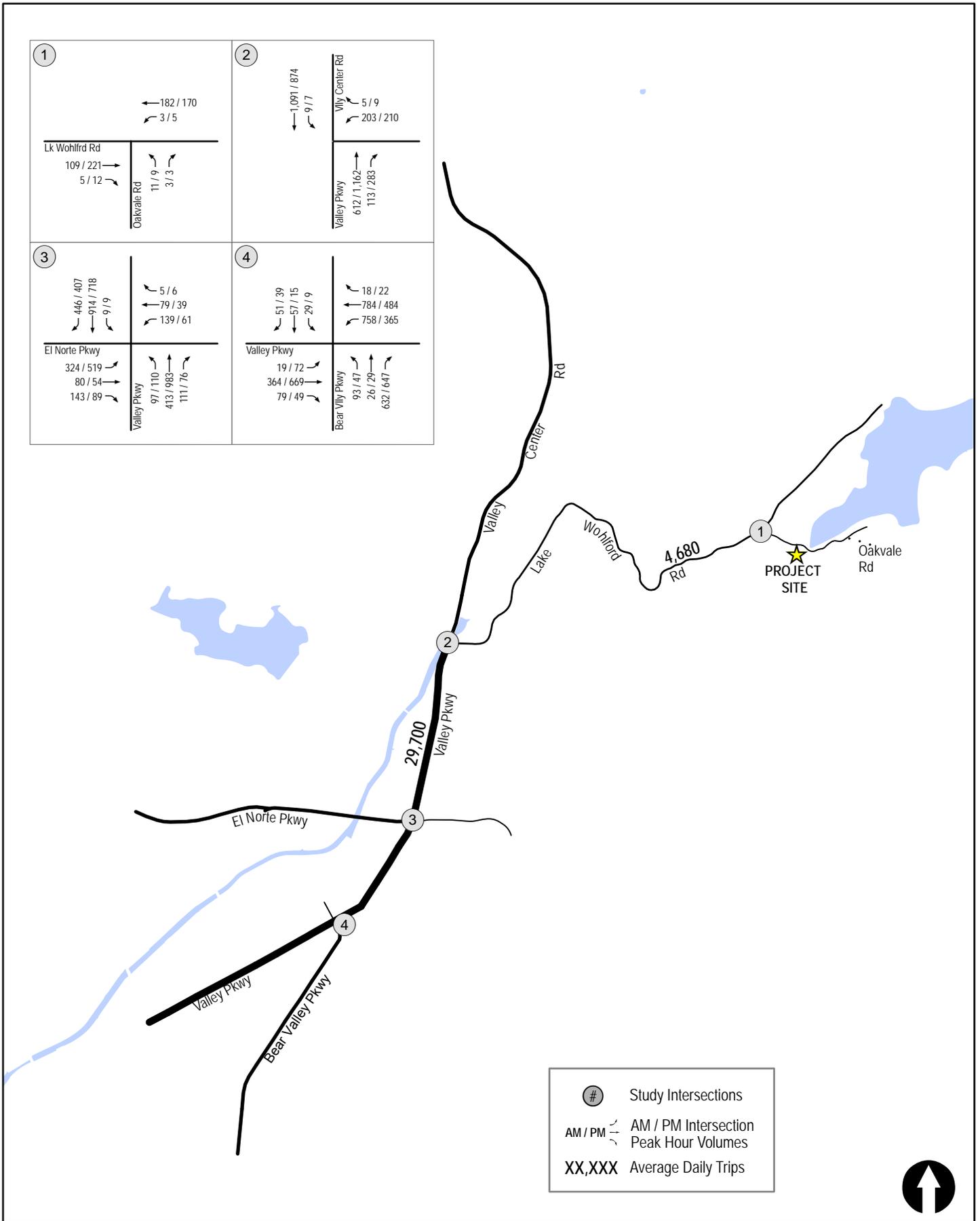


Figure 3-2

Existing Traffic Volumes

4.0 ANALYSIS APPROACH AND METHODOLOGY

Level of service (LOS) is the term used to denote the different operating conditions which occur on a given roadway segment under various traffic volume loads. It is a qualitative measure used to describe a quantitative analysis taking into account factors such as roadway geometries, signal phasing, speed, travel delay, freedom to maneuver, and safety. Level of service provides an index to the operational qualities of a roadway segment or an intersection. Level of service designations range from A to F, with LOS A representing the best operating conditions and LOS F representing the worst operating conditions. Level of service designation is reported differently for signalized intersections, unsignalized intersections and roadway segments.

The City of Escondido's recently published Traffic Impact Analysis Guidelines provide the following direction on report approach and methodology:

1. The traffic study should include a SANDAG prepared Select Zone Assignment for the project to determine the project traffic distribution.
2. The traffic study should utilize the Brief Guide of Vehicular Traffic Generation Rates for the San Diego Region (April 2002) published by SANDAG, to determine the project traffic volume.
3. Traffic should utilize the following scenarios to determine project traffic impacts at intersections and along roadway segments.
 - a. Existing Condition (based on new traffic counts)
 - b. Existing + Project Traffic Condition
 - c. Existing + Cumulative Projects Traffic Condition
 - d. Existing + Cumulative Projects + Project Traffic Condition
 - e. Year 2035 Traffic Condition
4. Highway Capacity Manual (Year 2010) should be utilized to determine level of service for intersections.
5. The study area should include at least all site access points and major intersections (signalized and un-signalized) adjacent to the site. The tables below contain the trigger-points to identify if a roadway segment or intersection should be included in the Traffic Impact Analysis.

Table 4-1 below contains the trigger-points for roadway segments within the City of Escondido for different street classifications based on Average Daily Traffic (ADT) added to the segment. **Table 4-2** below contains the trigger-points for intersections based on peak hour volumes.

TABLE 4-1
TRAFFIC IMPACT ANALYSIS ADT THRESHOLDS FOR ROADWAY SEGMENTS

Street Classification	Lanes	Cross Sections (ft.)	TIA Trigger-Points (ADT generation)
Prime Arterial	(8 lanes)	116/136 (NP)	900
	(6 lanes)	106/126 (NP)	800
Major Road	(6 lanes)	90/110 (NP)	700
	(4 lanes)	82/102 (NP)	500
Collector	(4 lanes)	64/84 (NP)	500
	(4 lanes)	(WP)	250
Local Collector and all other	(2 lanes)	42/66 (NP)	200
		(WP)	

TABLE 4-2
TRAFFIC IMPACT ANALYSIS ADT THRESHOLDS FOR INTERSECTIONS

Intersection Classification (Minor leg of the intersection)	TIA Trigger-Points (AM or PM peak hour trips added to any leg)
Prime Arterial	50
Major Road	40
Collector	30
Local Collector	20

Signalized intersections were analyzed under AM and PM peak hour conditions. Average vehicle delay was determined utilizing the methodology found in Chapter 18 of the *2010 Highway Capacity Manual (HCM)*, with the assistance of the *Synchro* (version 8) computer software. The delay values (represented in seconds) were qualified with a corresponding intersection LOS. Signalized intersection calculation worksheets and a more detailed explanation of the methodology are attached in *Appendix B*.

Unsignalized intersections were analyzed under AM and PM peak hour conditions. Average vehicle delay and LOS was determined based upon the procedures found in Chapters 19 and 20 of the *2010 Highway Capacity Manual (HCM)*, with the assistance of the *Synchro* (version 8) computer software. Unsignalized intersection calculation worksheets and a more detailed explanation of the methodology are attached in *Appendix B*.

Street segment analysis is based upon the comparison of daily traffic volumes (ADTs) to the City of Escondido *Roadway Classification, Level of Service, and ADT Table*. This table provides segment capacities for different street classifications, based on traffic volumes and roadway characteristics. The City of Escondido *Roadway Classification, Level of Service, and ADT Table* is attached in ***Appendix C***.

5.0 SIGNIFICANCE CRITERIA

The project study area includes locations that lie exclusively within the City of Escondido. The following is a summary of the City’s published significance criteria.

5.1 City of Escondido

In accordance with “SANTEC/ITE Guidelines for Traffic Impact Studies in the San Diego Region”, the following thresholds shall be used to identify if a project is of significant traffic impact under any scenario. Based on SANTEC/ITE guidelines, if now or in the future, the project’s traffic impact causes the values in **Table 5-1** below to be exceeded in a roadway segment or an intersection that is operating at LOS D or worse, it is determined to be a significant impact and the project shall identify mitigation measures.

**TABLE 5-1
CITY OF ESCONDIDO TRAFFIC IMPACT SIGNIFICANCE THRESHOLDS**

Level of Service With Project	Allowable Change due to Project Impact		
	Roadway Segments		Intersections
	V/C	Speed Reduction (mph)	Delay (sec.)
D, E, or F	0.02	1	2

6.0 ANALYSIS OF EXISTING CONDITIONS

6.1 Peak Hour Intersection Levels of Service

Table 6–1 summarizes the existing peak hour intersection operations. As shown, all the study area intersections are calculated to currently operate at service levels of LOS C or better during both the AM and PM peak hours.

**TABLE 6–1
EXISTING INTERSECTION OPERATIONS**

Intersection	Control Type	Peak Hour	Existing	
			Delay ^a	LOS ^b
1. Lake Wohlford Road / Oakvale Road	MSSC ^c	AM	10.4	B
		PM	11.5	B
2. Lake Wohlford Road / Valley Center Road	Signal	AM	15.5	B
		PM	10.7	B
3. Valley Parkway / El Norte Parkway	Signal	AM	22.8	C
		PM	26.6	C
4. Valley Parkway / Bear Valley Parkway	Signal	AM	24.9	C
		PM	20.3	C

Footnotes:

- a. Average delay expressed in seconds per vehicle.
- b. Level of Service.
- c. Minor Street Stop Controlled intersection, minor street left-turn delay is reported.

SIGNALIZED		UNSIGNALIZED	
DELAY/LOS THRESHOLDS		DELAY/LOS THRESHOLDS	
Delay	LOS	Delay	LOS
0.0 ≤ 10.0	A	0.0 ≤ 10.0	A
10.1 to 20.0	B	10.1 to 15.0	B
20.1 to 35.0	C	15.1 to 25.0	C
35.1 to 55.0	D	25.1 to 35.0	D
55.1 to 80.0	E	35.1 to 50.0	E
≥ 80.1	F	≥ 50.1	F

6.2 Daily Street Segment Levels of Service

Table 6–2 summarizes the existing segment operations along the key study area roadways. As shown, all roadway segments currently operate at LOS C or better.

**TABLE 6-2
EXISTING STREET SEGMENT OPERATIONS**

Street Segment	Existing Classification	Capacity (LOS E) ^a	ADT ^b	LOS ^c	V/C ^d
Lake Wohlford Road Valley Center Road to Oakvale Road	Local Collector	9,700 ^e	4,680	A	0.482
Valley Parkway El Norte Parkway to Lake Wohlford Road	5-Lane Major ^f	43,500	29,700	C	0.683

Footnotes:

- a. Capacities based on the *City of Escondido Roadway Classification* (see Appendix C).
- b. Average Daily Traffic Volumes
- c. Level of Service
- d. Volume to Capacity
- e. County of San Diego “Light Collector w/Reduced Shoulder (2.2F)” capacity used.
- f. Capacity based on average of 4-Lane Major and 6-Lane Major

7.0 TRIP GENERATION, DISTRIBUTION, AND ASSIGNMENT

7.1 Trip Generation

Based on direct coordination with the client, LLG determined the following values and assumptions to be used in calculating Project trip generation.

There are expected to be 35 construction workers on site on a regular basis. A trip rate of 2.2 per employee was used to account for daily commuting to and from the Project site and occasional extra trips taken by employees during the workday. This rate also includes trips by other miscellaneous staff that may be on site on an intermittent basis, including surveyors, materials testing, inspection, or environmental monitoring. The AM and PM peak hour percentages were determined assuming an 8-hour workday from 7 AM to 4 PM. Thus, most employees will be making the inbound trip to the Project site prior to the AM peak hour. By the same logic, most employees are expected to make the outbound trip from the Project site during the PM peak hour.

The Project is expected to generate 70 round-trip truck trips per day. Thus a rate of 2.0 accounts for both inbound and outbound truck trips. A Passenger Car Equivalent (PCE) of 3.0 is applied to account for the differing speed, acceleration, and maneuverability of heavy trucks compared to typical passenger vehicles. The AM and PM peak hour percentages were determined by assuming 100% of truck trips take place within and are evenly dispersed over an 8-hour workday. This yields an hourly share of 12.5% of truck trips, which is rounded up to 13% in the calculation below.

Table 7-1 shows a summary of the Project traffic generation. As tabulated the proposed Project is calculated to generate the equivalent of 497 daily trips with 59 trips (32 inbound/27 outbound) in AM peak hour and 86 trips (30 inbound/56 outbound) during PM peak hour.

**TABLE 7-1
PROJECT TRIP GENERATION**

Type	Amount	Rate ^a	PCE ^b	ADT	AM Peak Hour				PM Peak Hour					
					% of ADT	In:Out		Volume		% of ADT	In:Out		Volume	
						Split	In	Out	Split		In	Out		
Employee	35	2.2	1.0	77	5%	9:1	4	0	40%	1:9	3	28		
Truck	70	2.0	3.0	420	13%	5:5	28	27	13%	5:5	27	28		
Total				497			32	27			30	56		

Footnotes:

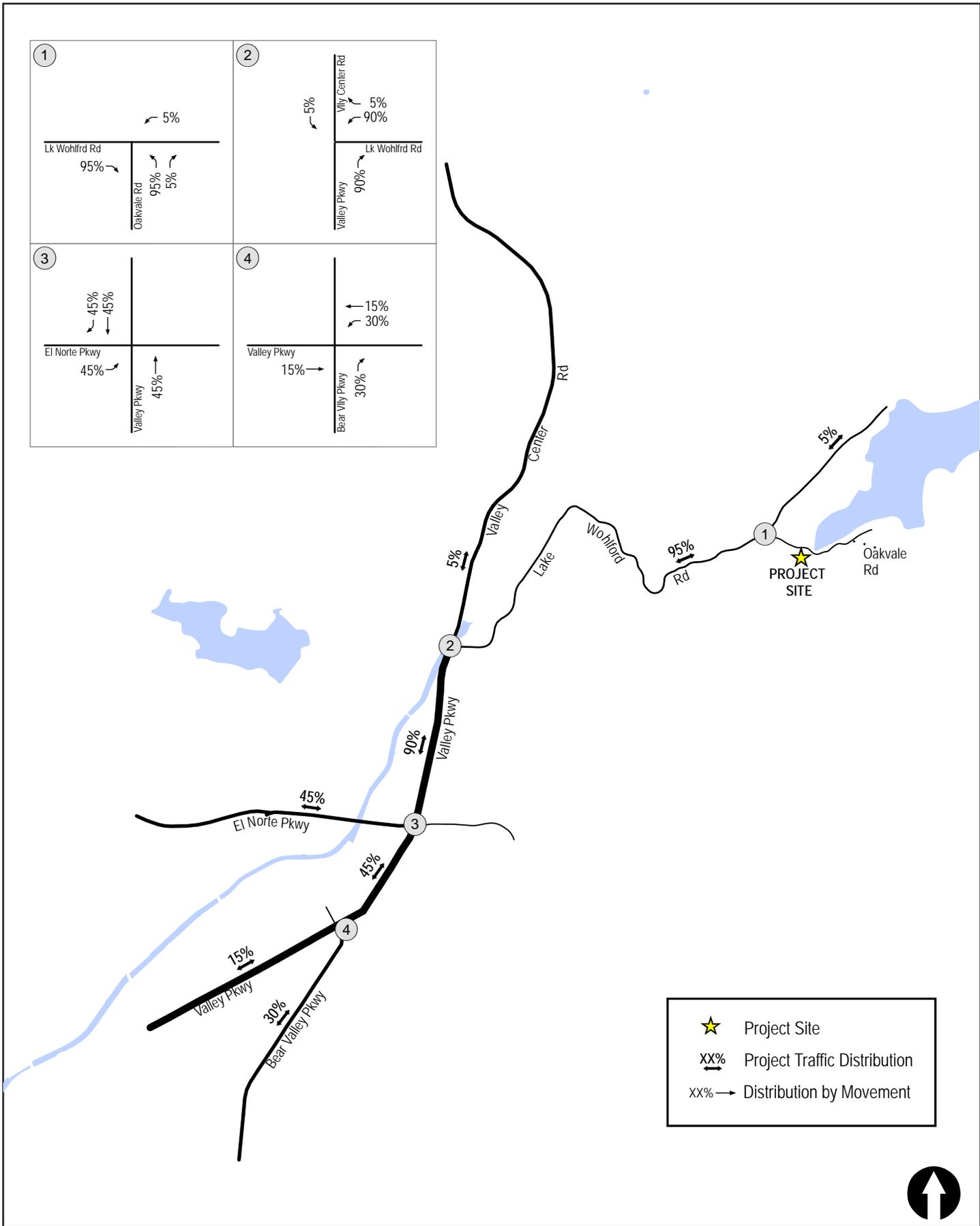
- a. Rate is based on site-specific trip generation factors.
- b. Passenger Car Equivalent

7.2 Trip Distribution & Assignment

Project trips were distributed regionally based on potential destinations for material hauling from construction activity. A small number of trips were distributed via Lake Wohlford Road and Valley Center Road to possible local destinations in Valley Center. The rest of the trips are distributed to regional destinations via the City of Escondido's identified truck routes, ultimately reaching I-15 for

regional access. Trips were split among these existing truck routes based on orientation to northbound or southbound destinations and roadway characteristics including size of the roadway, existing congestion patterns, surrounding land uses, and spacing of signalized intersections.

Figure 7-1 shows the Project trip distribution percentages. **Figure 7-2** shows the AM/PM peak hour Project traffic volumes. **Figure 7-3** shows Existing + Project traffic volumes. **Figure 7-4** shows truck routes within the City of Escondido as identified in the City's *Mobility and Infrastructure Element*.



★ Project Site
 XX% → Project Traffic Distribution
 XX% → Distribution by Movement



Figure 7-1

Project Trip Distribution

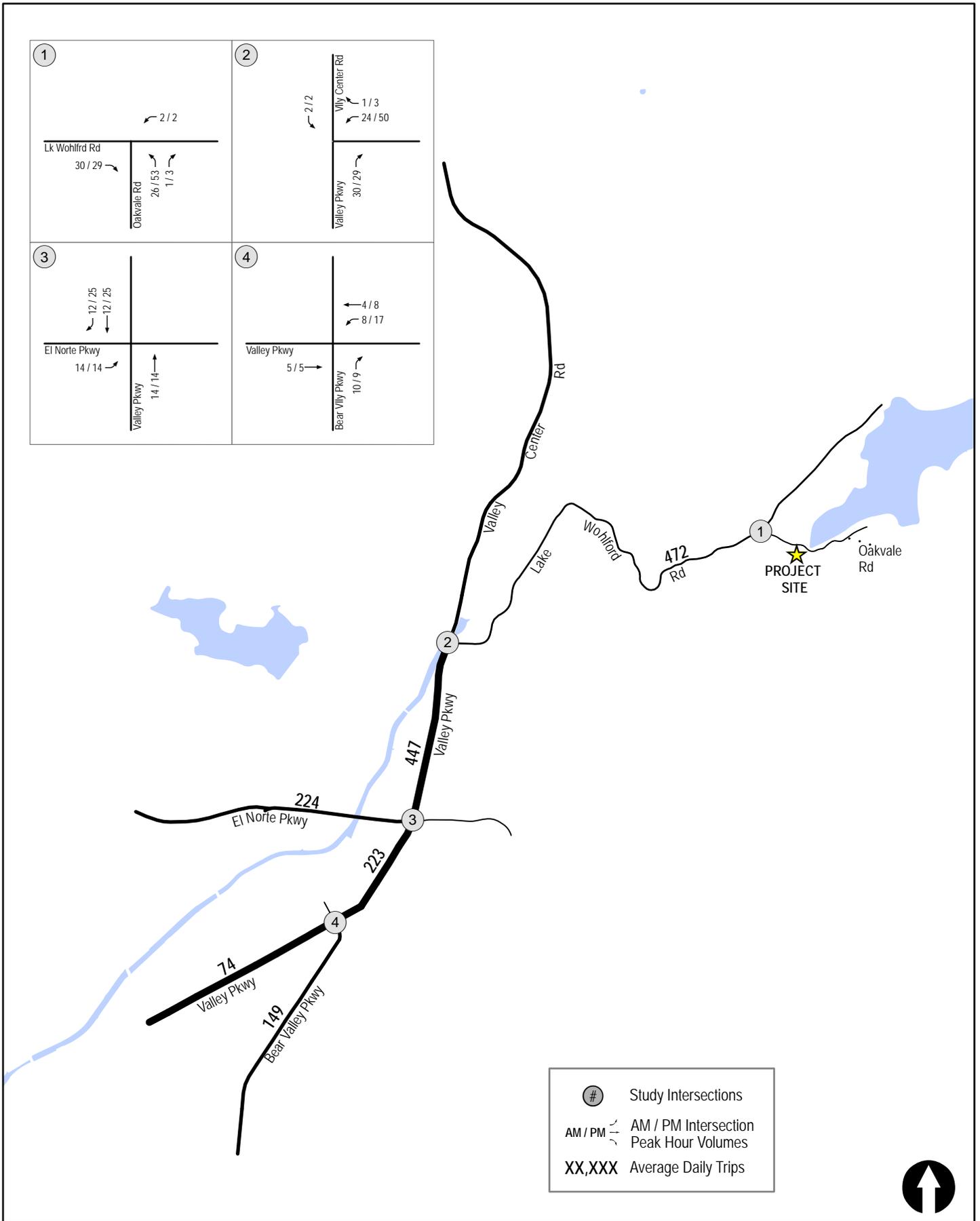


Figure 7-2

Project Traffic Volumes

OAKVALE ROAD REALIGNMENT

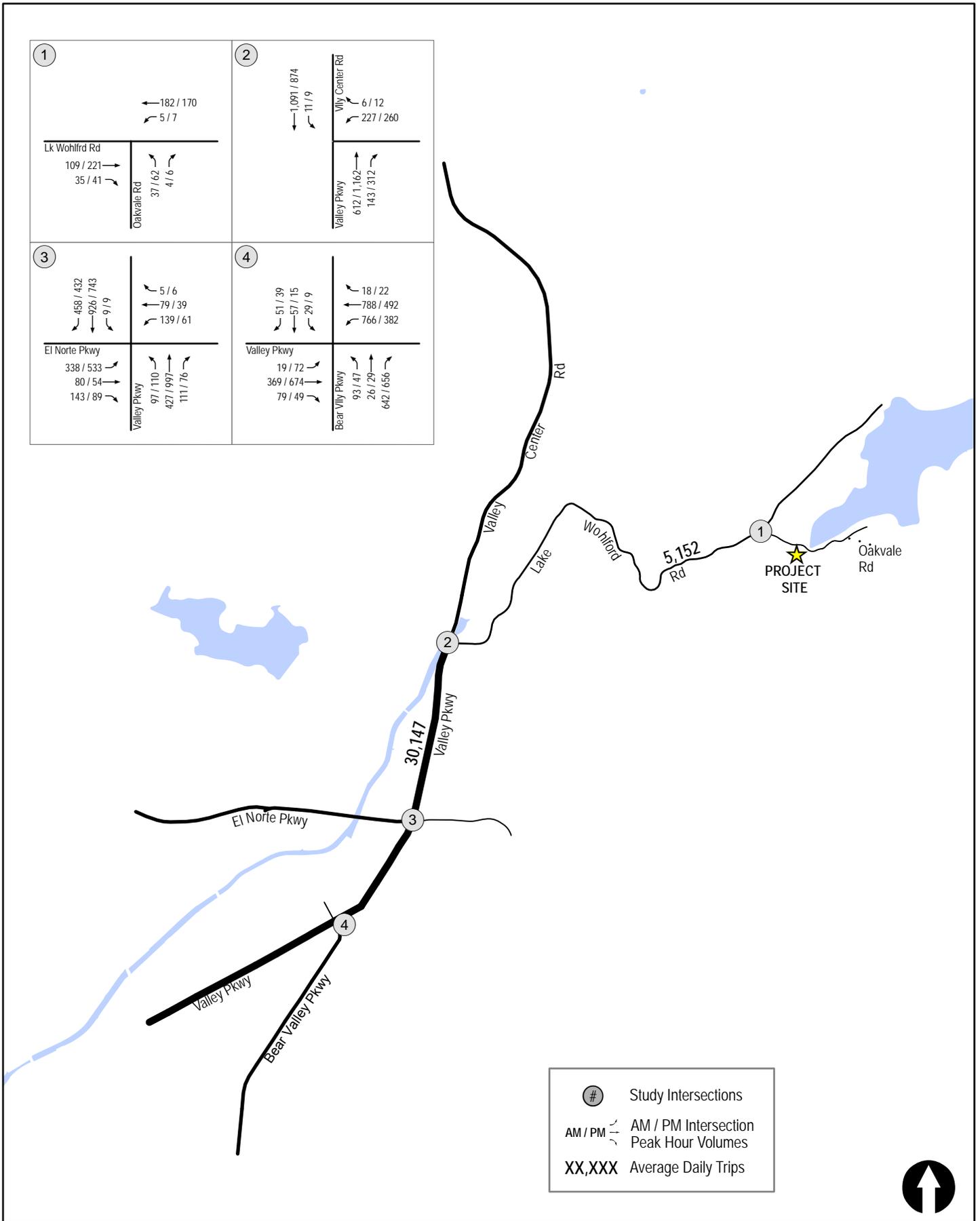
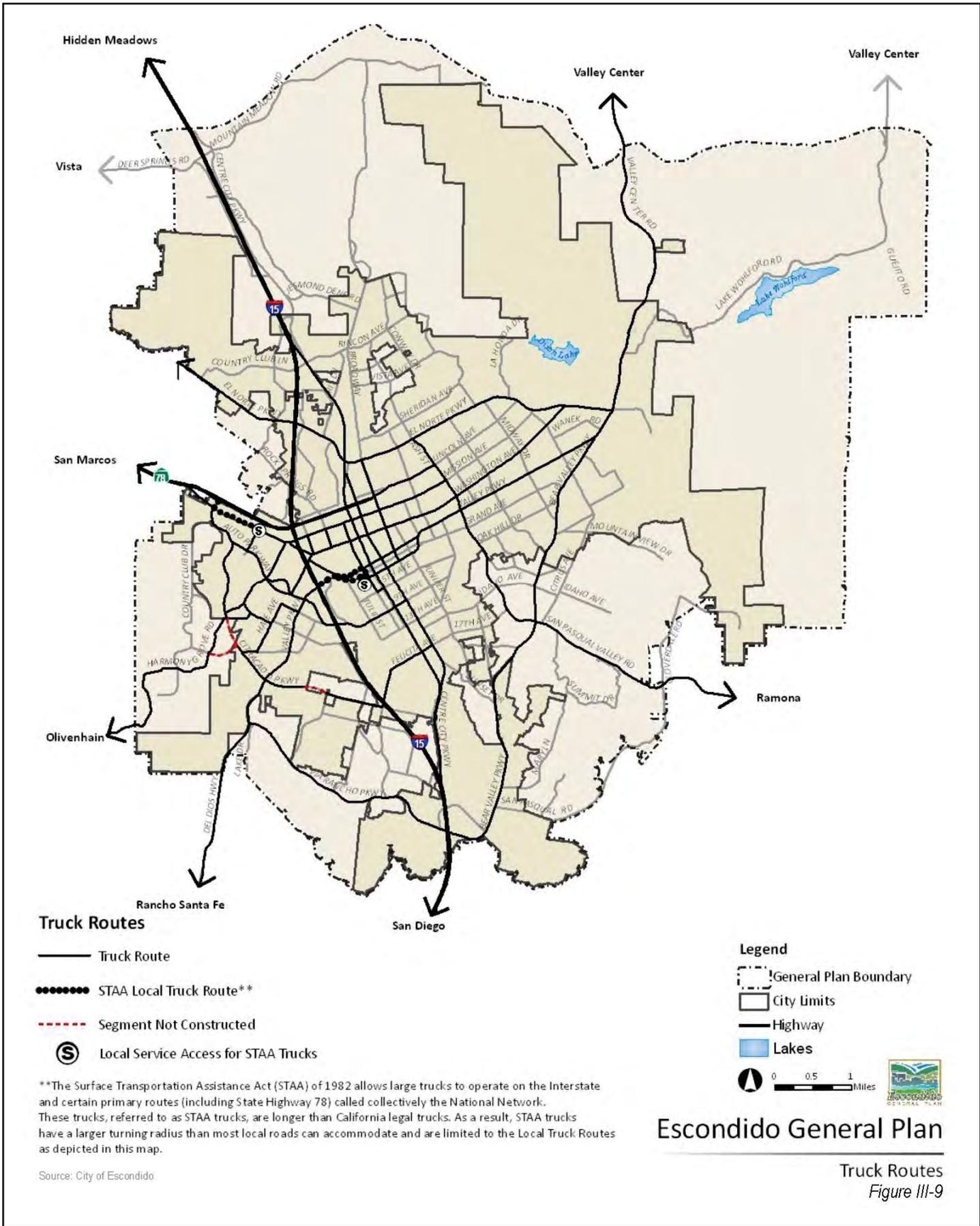


Figure 7-3

Existing + Project Traffic Volumes



8.0 CUMULATIVE GROWTH TRAFFIC VOLUMES

Cumulative projects are other projects in the study area that will add traffic to the local circulation system in the near future. LLG coordinated directly with City of Escondido and County of San Diego staff to determine and obtain cumulative projects' traffic volume information to be included for analysis.

LLG has recently completed traffic studies for two projects in the vicinity which may add traffic to the roadway system analyzed. However, due to uncertain completion dates of these projects a cumulative growth approach was used. In order to forecast near-term cumulative traffic conditions LLG used interpolated growth between Year 2035 and existing Year 2014 traffic counts. Typical annual growth ranged between 2-5% at study area intersections and segments. Location-specific growth factors were applied to each study area location for a period of one (1) year to arrive at near-term baseline conditions which are analyzed in this report as "+ Cumulative Growth" scenarios.

Figure 8-1 shows the total cumulative growth traffic volumes. *Figure 8-2* shows the existing + cumulative growth traffic volumes. *Figure 8-3* shows the existing + project + cumulative growth traffic volumes.

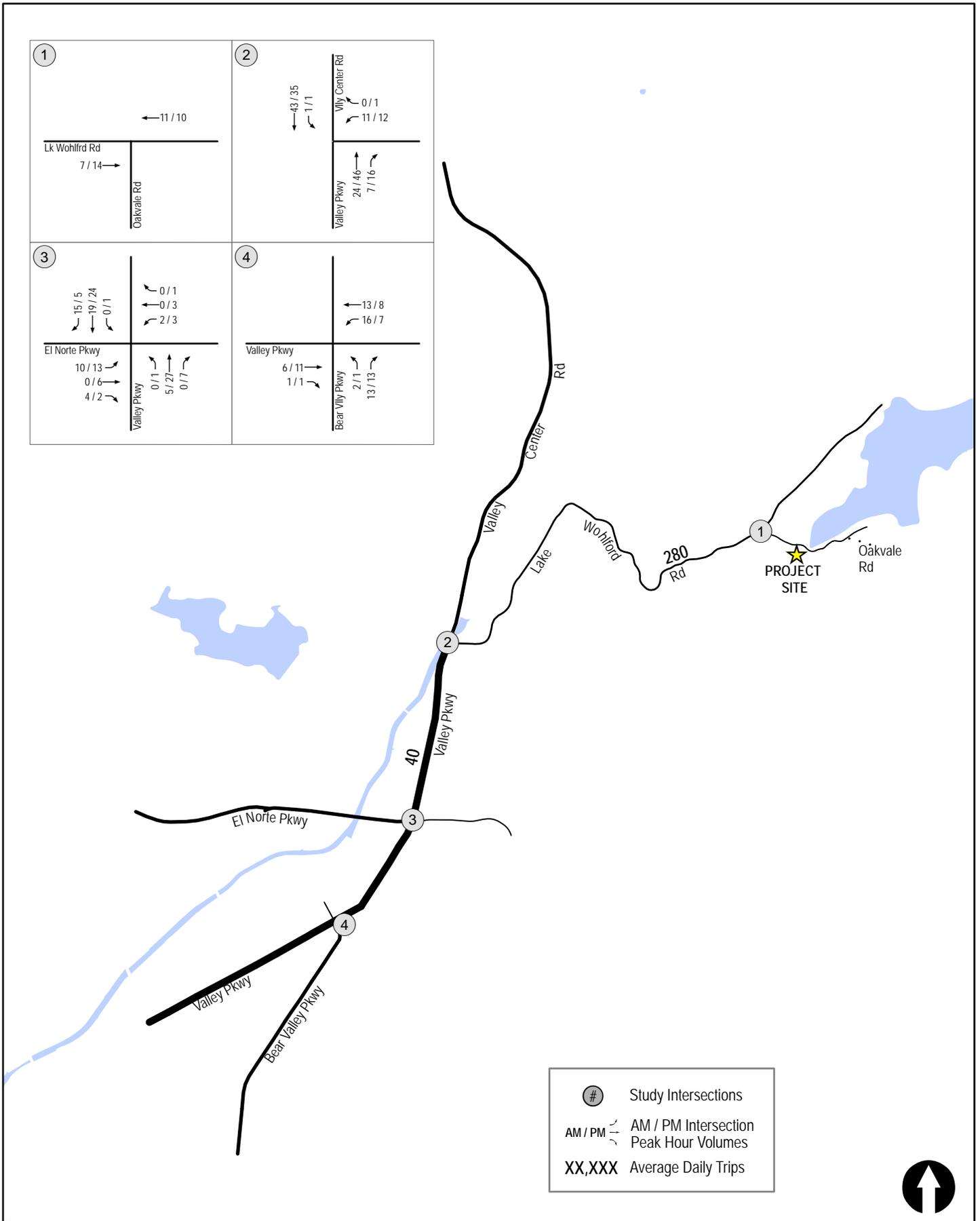
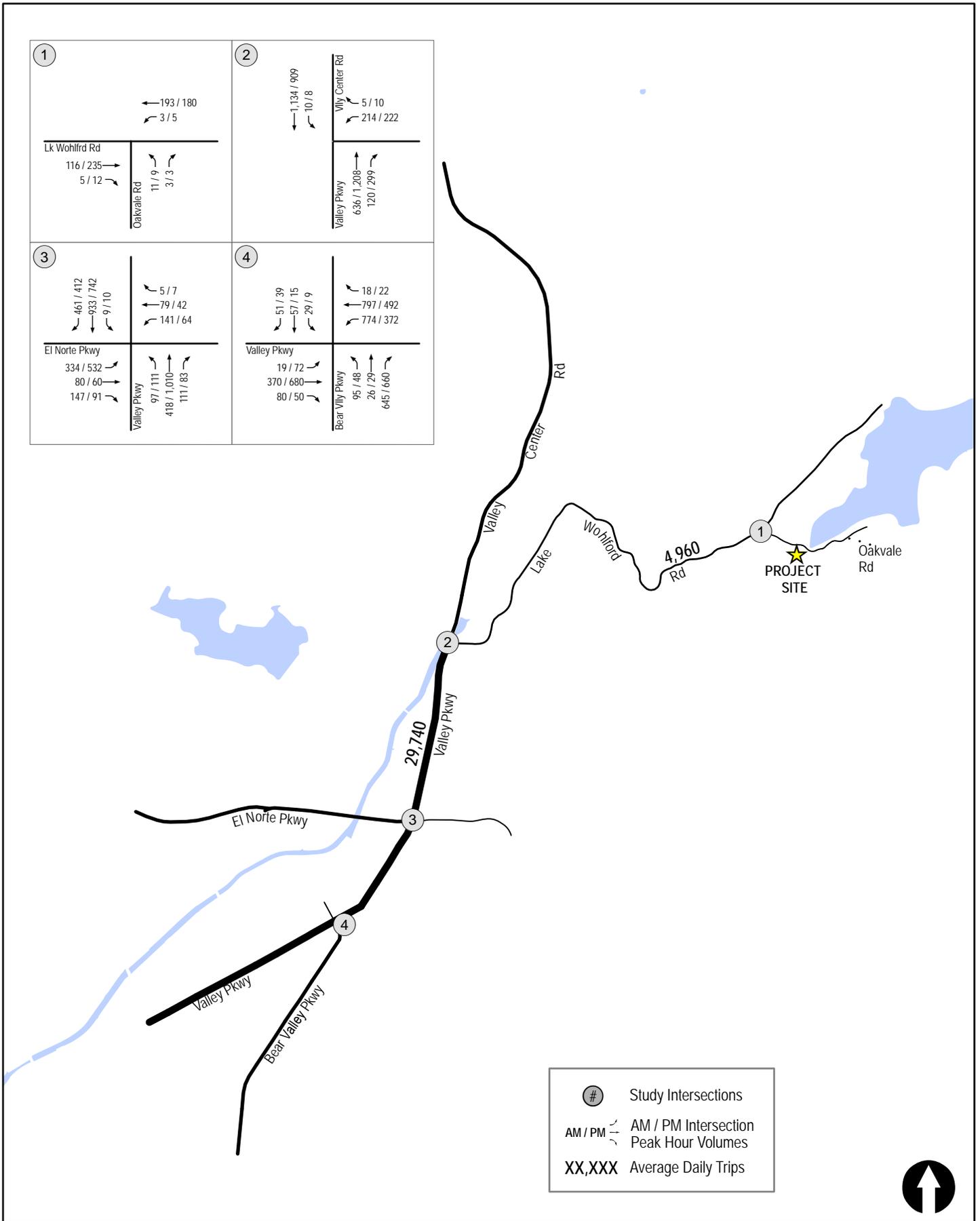


Figure 8-1

Cumulative Growth Traffic Volumes

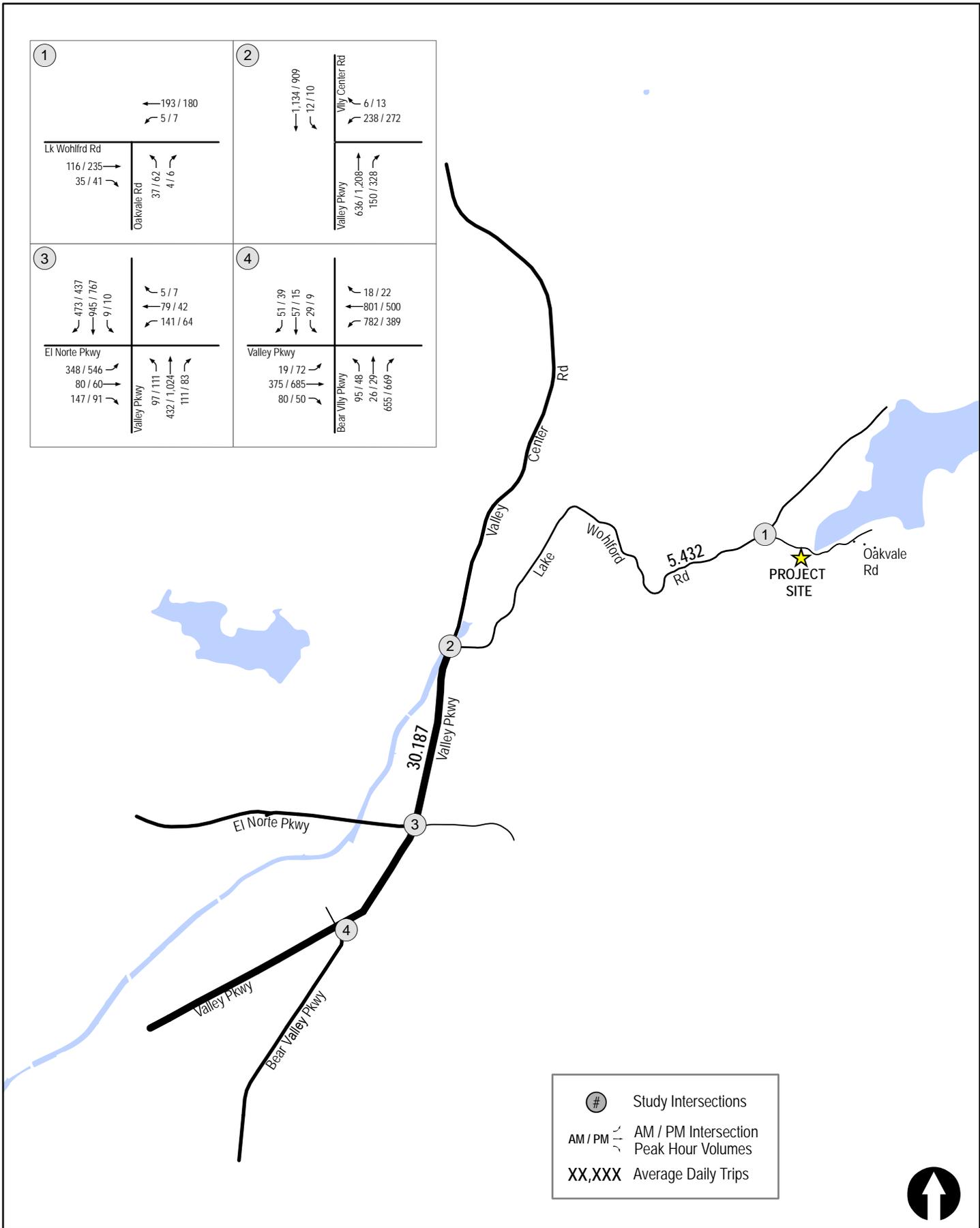
OAKVALE ROAD REALIGNMENT



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Existing + Cumulative Growth Traffic Volumes

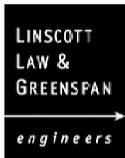
Figure 8-2



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Figure 8-3

Existing + Project + Cumulative Growth Traffic Volumes



9.0 ANALYSIS OF NEAR-TERM SCENARIOS

The following is a summary of the operational analyses for the various street-system components for the near-term traffic scenarios. Potentially significant impacts are **bolded** and underlined.

9.1 Existing + Project

9.1.1 Intersection Analysis

Table 9-1 summarizes the peak hour intersection operations with the addition of Project traffic. *Table 9-1* shows that all the study area intersections are calculated to continue to operate at LOS C or better with the addition of Project traffic.

9.1.2 Segment Operations

Table 9-2 summarizes the roadway segment operations with the addition of Project traffic. As seen in *Table 9-2*, study area segments are calculated to continue to operate at LOS C or better.

9.2 Existing + Cumulative Growth

9.2.1 Intersection Analysis

Table 9-1 summarizes the peak hour intersection operations with the addition of cumulative growth traffic. *Table 9-1* shows that the study area intersections are calculated to operate at LOS C or better.

9.2.2 Segment Operations

Table 9-2 summarizes the roadway segment operations with the addition of cumulative growth traffic. As seen in *Table 9-2*, all study area segments are calculated to continue to operate at LOS C or better.

9.3 Existing + Project + Cumulative Growth

9.3.1 Intersection Analysis

Table 9-1 summarizes the peak hour intersection operations for Existing + Cumulative Growth + Project conditions. *Table 9-1* shows that all study area intersections are calculated to operate at LOS C or better with the addition of Project traffic.

9.3.2 Segment Operations

Table 9-2 summarizes the roadway segment operations for the Existing + Cumulative Growth + Project conditions. As seen in *Table 9-2*, all study area segments are calculated to continue to operate at LOS C or better.

**TABLE 9-1
NEAR-TERM INTERSECTION OPERATIONS**

Intersection	Control Type	Peak Hour	Existing		Existing + Project			Existing + Cumulative Growth		Existing + Cumulative Growth + Project			Significant Impact?
			Delay ^a	LOS ^b	Delay	LOS	Δ ^c	Delay	LOS	Delay	LOS	Δ ^c	
1. Lake Wohlford Rd / Oakvale Rd	MSSC ^d	AM	10.4	B	11.1	B	0.7	10.5	B	11.3	B	0.8	No
		PM	11.5	B	12.9	B	1.4	11.7	B	13.3	B	1.6	No
2. Lake Wohlford Rd / Valley Center Rd	Signal	AM	15.5	B	17.5	B	2.0	18.3	B	20.8	C	2.5	No
		PM	10.7	B	13.6	B	2.9	11.5	B	14.6	B	3.1	No
3. Valley Parkway / El Norte Parkway	Signal	AM	22.8	C	23.1	C	0.3	23.1	C	23.4	C	0.3	No
		PM	26.6	C	27.9	C	1.3	28.3	C	29.6	C	1.3	No
4. Valley Parkway / Bear Valley Parkway	Signal	AM	24.9	C	25.1	C	0.2	25.2	C	25.4	C	0.2	No
		PM	20.3	C	20.4	C	0.1	20.4	C	20.6	C	0.2	No

Footnotes:

- a. Average delay expressed in seconds per vehicle.
- b. Level of Service.
- c. Δ denotes an increase in delay due to project.
- d. Minor Street Stop Controlled intersection. Minor street left turn delay is reported.

SIGNALIZED		UNSIGNALIZED	
DELAY/LOS THRESHOLDS		DELAY/LOS THRESHOLDS	
Delay	LOS	Delay	LOS
0.0 ≤ 10.0	A	0.0 ≤ 10.0	A
10.1 to 20.0	B	10.1 to 15.0	B
20.1 to 35.0	C	15.1 to 25.0	C
35.1 to 55.0	D	25.1 to 35.0	D
55.1 to 80.0	E	35.1 to 50.0	E
≥ 80.1	F	≥ 50.1	F

**TABLE 9-2
NEAR-TERM STREET SEGMENT OPERATIONS**

Street Segment	Capacity (LOS E) ^a	Existing			Existing + Project				Existing + Cumulative Projects			Existing + Cumulative Projects + Project				Sig Impact?
		ADT ^b	LOS ^c	V/C ^d	ADT	LOS	V/C	Δ ^e	ADT	LOS	V/C	ADT	LOS	V/C	Δ ^e	
Lake Wohlford Road Valley Center Rd to Oakvale Rd	9,700	4,680	A	0.482	5,152	A	0.531	0.049	4,960	A	0.511	5,432	A	0.560	0.049	No
Valley Parkway El Norte Pkwy to Lake Wohlford Rd	43,500	29,700	C	0.683	30,147	C	0.693	0.010	29,740	C	0.684	30,187	C	0.694	0.010	No

Footnotes:

- a. Capacities based on the *City of Escondido Roadway Classification* (See Appendix C).
- b. Average Daily Traffic
- c. Level of Service
- d. Volume to Capacity ratio
- e. Δ denotes a Project-induced increase in the Volume to Capacity (V/C) ratio.
- f. County of San Diego “Light Collector w/Reduced Shoulder (2.2F)” capacity used.

10.0 SIGNIFICANCE OF IMPACTS AND MITIGATION MEASURES

10.1 Significance of Impacts

The traffic impacts of the proposed 497-ADT Oakvale Road Realignment Project do not exceed the significance thresholds published by the City of Escondido. *No significant impacts are calculated.*

10.2 Mitigation Measures

The Project does not result in any significant impacts according to City of Escondido criteria. *No mitigation measures are required.*

End of Report

TECHNICAL APPENDICES
OAKVALE ROAD REALIGNMENT
Escondido, California
July 29, 2014

LLG Ref. 3-14-2324

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APPENDIX A
INTERSECTION AND SEGMENT MANUAL COUNT SHEETS

Turn Count Summary

Accurate Video Counts Inc
info@accuratevideocounts.com
(619) 987-5136



Location: Oakvale Rd @ Lake Wohlford Rd

Date of Count: Tuesday, March 04, 2014

Analysts: LV/CD

Weather: Sunny

AVC Proj No: 14-0174





Location: Oakvale Rd @ Lake Wohlford Rd

AM Period (7:00 AM - 9:00 AM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
7:00 AM	0	0	0	0	42	2	0	0	3	1	11	0	59
7:15 AM	0	0	0	0	42	0	3	0	4	1	22	0	72
7:30 AM	0	0	0	0	42	0	1	0	4	2	13	0	62
7:45 AM	0	0	0	0	51	1	2	0	1	0	26	0	81
8:00 AM	0	0	0	0	42	0	1	0	3	0	23	0	69
8:15 AM	0	0	0	0	39	2	0	0	3	4	24	0	72
8:30 AM	0	0	0	0	50	0	0	0	4	1	36	0	91
8:45 AM	0	0	0	0	31	1	0	0	0	1	28	0	61
Total	0	0	0	0	339	6	7	0	22	10	183	0	567

AM Intersection Peak Hour : 7:45 AM - 8:45 AM

Intersection PHF : 0.86

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	0	0	0	0	182	3	3	0	11	5	109	0	313
PHF	#####	#####	#####	#####	0.89	0.38	0.38	#####	0.69	0.31	0.76	#####	0.86
Movement PHF	#DIV/0!			0.89			0.88			0.77			0.86

PM Period (4:00 PM - 6:00 PM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
4:00 PM	0	0	0	0	25	0	0	0	2	4	58	0	89
4:15 PM	0	0	0	0	44	2	0	0	3	3	52	0	104
4:30 PM	0	0	0	0	30	1	1	0	2	2	69	0	105
4:45 PM	0	0	0	0	42	1	1	0	2	2	53	0	101
5:00 PM	0	0	0	0	54	1	1	0	2	5	47	0	110
5:15 PM	0	0	0	0	35	0	1	0	1	5	49	0	91
5:30 PM	0	0	0	0	38	1	1	0	1	3	64	0	108
5:45 PM	0	0	0	0	37	1	1	0	4	3	47	0	93
Total	0	0	0	0	305	7	6	0	17	27	439	0	801

PM Intersection Peak Hour : 4:15 PM - 5:15 PM

Intersection PHF : 0.95

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	0	0	0	0	170	5	3	0	9	12	221	0	420
PHF	#####	#####	#####	#####	0.787	0.625	0.75	#####	0.75	0.6	0.801	#####	0.95
Movement PHF	#DIV/0!			0.80			1.00			0.82			0.95

Turn Count Summary

Accurate Video Counts Inc
info@accuratevideocounts.com
(619) 987-5136



Location: Lake Wohlford Rd @ Valley Center Rd

Date of Count: Tuesday, March 04, 2014

Analysts: LV/CD

Weather: Sunny

AVC Proj No: 14-0174





Location: Lake Wohlford Rd @ Valley Center Rd

AM Period (7:00 AM - 9:00 AM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
7:00 AM	0	261	0	2	0	48	17	140	0	0	0	0	468
7:15 AM	0	306	0	1	0	54	26	140	0	0	0	0	527
7:30 AM	0	291	2	0	0	49	17	152	0	0	0	0	511
7:45 AM	0	247	3	2	0	59	36	152	0	0	0	0	499
8:00 AM	0	247	4	2	0	41	34	168	0	0	0	0	496
8:15 AM	0	205	6	2	0	55	31	103	0	0	0	0	402
8:30 AM	0	243	1	0	0	56	39	153	0	0	0	0	492
8:45 AM	0	276	3	0	0	44	33	129	0	0	0	0	485
Total	0	2,076	19	9	0	406	233	1,137	0	0	0	0	3,880

AM Intersection Peak Hour : **7:15 AM - 8:15 AM**

Intersection PHF : **0.96**

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	0	1,091	9	5	0	203	113	612	0	0	0	0	2,033
PHF	#####	0.89	0.56	0.63	#####	0.86	0.78	0.91	#####	#####	#####	#####	0.96
Movement PHF		0.90			0.85			0.90		#DIV/0!			0.96

PM Period (4:00 PM - 6:00 PM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
4:00 PM	0	204	3	2	0	32	68	297	0	0	0	0	606
4:15 PM	0	207	3	3	0	57	67	289	0	0	0	0	626
4:30 PM	0	228	2	2	0	42	84	271	0	0	0	0	629
4:45 PM	0	224	1	2	0	50	51	305	0	0	0	0	633
5:00 PM	0	215	1	2	0	61	81	297	0	0	0	0	657
5:15 PM	0	206	4	0	0	36	61	285	0	0	0	0	592
5:30 PM	0	200	1	1	0	49	67	280	0	0	0	0	598
5:45 PM	0	181	0	1	0	48	64	268	0	0	0	0	562
Total	0	1665	15	13	0	375	543	2,292	0	0	0	0	4,903

PM Intersection Peak Hour : **4:15 PM - 5:15 PM**

Intersection PHF : **0.97**

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	0	874	7	9	0	210	283	1162	0	0	0	0	2545
PHF	#####	0.958	0.583	0.75	#####	0.861	0.842	0.952	#####	#####	#####	#####	0.97
Movement PHF		0.96			0.87			0.96		#DIV/0!			0.97

Turn Count Summary

Accurate Video Counts Inc
info@accuratevideocounts.com
(619) 987-5136



Location: El Norte Pkwy @ Valley Center Rd

Date of Count: Tuesday, March 04, 2014

Analysts: LV/CD

Weather: Sunny

AVC Proj No: 14-0174



Vehicular Count

Accurate Video Counts Inc
info@accuratevideocounts.com
(619) 987-5136



Location: El Norte Pkwy @ Valley Center Rd

AM Period (7:00 AM - 9:00 AM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
7:00 AM	116	218	1	0	7	25	7	87	12	23	5	69	570
7:15 AM	135	263	0	0	16	31	7	82	19	48	7	70	678
7:30 AM	122	245	2	2	12	34	19	105	21	53	7	89	711
7:45 AM	103	215	4	1	22	27	20	118	25	31	25	87	678
8:00 AM	86	191	3	2	29	47	65	108	32	11	41	78	693
8:15 AM	115	225	1	0	23	42	19	77	15	18	19	54	608
8:30 AM	119	225	0	1	11	20	19	141	16	12	7	71	642
8:45 AM	95	142	0	1	5	14	7	104	9	12	4	58	451
Total	891	1,724	11	7	125	240	163	822	149	208	115	576	5,031

AM Intersection Peak Hour : 7:15 AM - 8:15 AM

Intersection PHF : 0.97

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	446	914	9	5	79	139	111	413	97	143	80	324	2,760
PHF	0.83	0.87	0.56	0.63	0.68	0.74	0.43	0.88	0.76	0.67	0.49	0.91	0.97
Movement PHF	0.86			0.71			0.76			0.92			0.97

PM Period (4:00 PM - 6:00 PM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
4:00 PM	83	158	0	2	8	7	20	245	25	19	12	107	686
4:15 PM	90	174	2	2	15	18	16	226	28	23	9	133	736
4:30 PM	87	186	2	1	6	14	17	252	29	26	11	139	770
4:45 PM	106	174	5	1	12	14	23	261	23	16	14	105	754
5:00 PM	124	184	0	2	6	15	20	244	30	24	20	142	811
5:15 PM	109	150	2	1	11	14	24	229	21	23	13	115	712
5:30 PM	106	163	3	0	9	18	29	219	27	16	9	137	736
5:45 PM	64	172	1	1	14	6	42	230	23	24	18	121	716
Total	769	1361	15	10	81	106	191	1,906	206	171	106	999	5,921

PM Intersection Peak Hour : 4:15 PM - 5:15 PM

Intersection PHF : 0.95

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	407	718	9	6	39	61	76	983	110	89	54	519	3071
PHF	0.82	0.965	0.45	0.75	0.65	0.847	0.826	0.942	0.917	0.856	0.675	0.914	0.95
Movement PHF	0.92			0.76			0.95			0.89			0.95

Turn Count Summary

Accurate Video Counts Inc
info@accuratevideocounts.com
(619) 987-5136



Location: Valley Center Rd @ Bear Valley Pwky

Date of Count: Tuesday, March 04, 2014

Analysts: LV/CD

Weather: Sunny

AVC Proj No: 14-0174



Vehicular Count

Accurate Video Counts Inc
info@accuratevideocounts.com
(619) 987-5136



Location: Valley Center Rd @ Bear Valley Pwky

AM Period (7:00 AM - 9:00 AM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
7:00 AM	17	13	6	2	111	139	71	1	17	5	33	6	421
7:15 AM	10	24	2	1	166	188	109	4	15	20	72	4	615
7:30 AM	17	15	7	2	197	194	173	12	34	43	91	4	789
7:45 AM	13	14	20	5	203	182	213	5	27	14	107	7	810
8:00 AM	11	4	0	10	218	194	137	5	17	2	94	4	696
8:15 AM	8	4	0	0	179	145	84	3	9	2	58	6	498
8:30 AM	16	3	0	3	147	130	109	3	5	3	61	6	486
8:45 AM	5	4	1	2	117	93	64	1	6	4	64	11	372
Total	97	81	36	25	1,338	1,265	960	34	130	93	580	48	4,687

AM Intersection Peak Hour : **7:15 AM - 8:15 AM**

Intersection PHF : **0.90**

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	51	57	29	18	784	758	632	26	93	79	364	19	2,910
PHF	0.75	0.59	0.36	0.45	0.90	0.98	0.74	0.54	0.68	0.46	0.85	0.68	0.90
Movement PHF	0.73			0.92			0.77			0.84			0.90

PM Period (4:00 PM - 6:00 PM)													
	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
4:00 PM	4	2	5	3	120	80	123	7	11	7	160	13	535
4:15 PM	7	4	1	4	113	101	149	6	8	8	172	13	586
4:30 PM	14	3	2	3	107	90	147	7	14	7	144	22	560
4:45 PM	14	5	2	4	116	106	169	5	13	10	163	12	619
5:00 PM	10	3	0	4	137	84	157	5	8	4	184	18	614
5:15 PM	12	1	1	6	106	93	140	5	10	19	150	14	557
5:30 PM	9	4	2	3	117	102	173	8	15	15	154	16	618
5:45 PM	8	7	6	9	124	86	177	11	14	11	181	24	658
Total	78	29	19	36	940	742	1,235	54	93	81	1,308	132	4,747

PM Intersection Peak Hour : **5:00 PM - 6:00 PM**

Intersection PHF : **0.93**

	Southbound			Westbound			Northbound			Eastbound			TOTAL
	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	Right	Thru	Left	
Volume	39	15	9	22	484	365	647	29	47	49	669	72	2447
PHF	0.81	0.536	0.375	0.611	0.883	0.895	0.914	0.659	0.783	0.645	0.909	0.75	0.93
Movement PHF	0.75			0.97			0.89			0.91			0.93



24 Hour Segment Count

Accurate Video Counts Inc
 info@accuratevideocounts.com
 (619) 987-5136



Location: 1. Lake Wohlford Rd btw Valley Center Rd to Oakvale Rd
Orientation: East-West
Date of Count: Tuesday, March 04, 2014
Analysts: DASH
Weather: Sunny
AVC Proj. No: 14-0174

24 Hour Segment Volume					4,684				
Time	Hourly Volume			Time	Hourly Volume				
	EB	WB	Total		EB	WB	Total		
12:00 AM - 1:00 AM	20	29	49	12:00 PM - 1:00 PM	118	126	244		
1:00 AM - 2:00 AM	10	26	36	1:00 PM - 2:00 PM	127	117	244		
2:00 AM - 3:00 AM	10	20	30	2:00 PM - 3:00 PM	154	161	315		
3:00 AM - 4:00 AM	6	20	26	3:00 PM - 4:00 PM	207	166	373		
4:00 AM - 5:00 AM	13	34	47	4:00 PM - 5:00 PM	222	139	361		
5:00 AM - 6:00 AM	40	77	117	5:00 PM - 6:00 PM	211	142	353		
6:00 AM - 7:00 AM	63	151	214	6:00 PM - 7:00 PM	165	146	311		
7:00 AM - 8:00 AM	78	181	259	7:00 PM - 8:00 PM	153	70	223		
8:00 AM - 9:00 AM	118	165	283	8:00 PM - 9:00 PM	104	42	146		
9:00 AM - 10:00 AM	88	126	214	9:00 PM - 10:00 PM	110	53	163		
10:00 AM - 11:00 AM	92	144	236	10:00 PM - 11:00 PM	61	59	120		
11:00 AM - 12:00 PM	122	126	248	11:00 PM - 12:00 AM	34	38	72		
Total	660	1,099	1,759	Total	1,666	1,259	2,925		

24-Hour EB Volume 2,326 **24-Hour WB Volume 2,358**

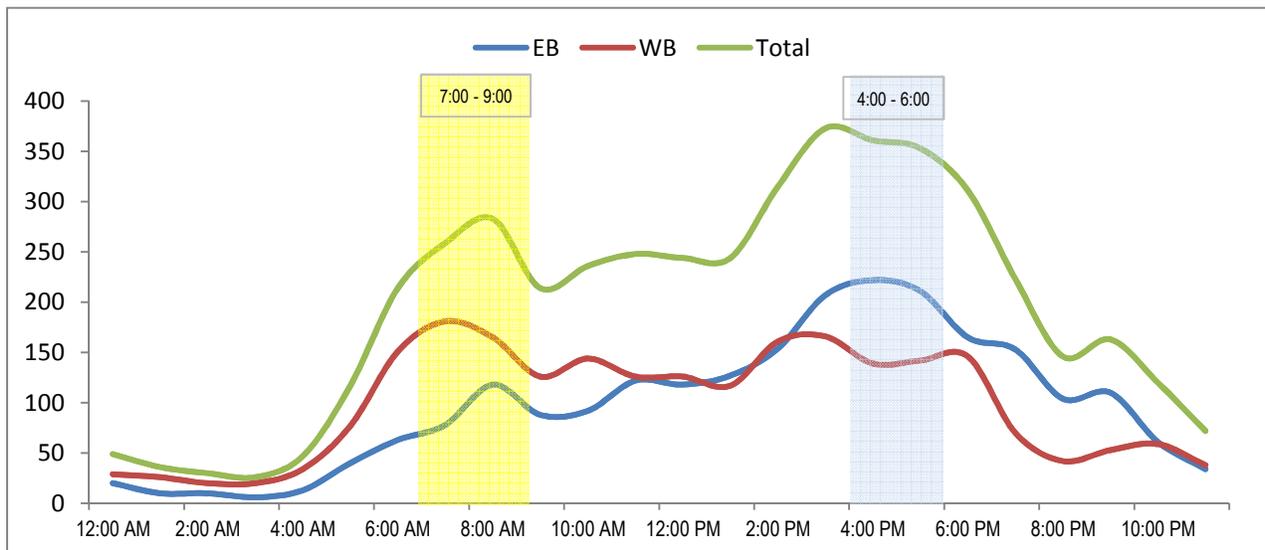


TABLE 22-1
NORTHEAST QUADRANT
EXISTING STREET SEGMENT OPERATIONS

Street Segment	Currently Built As ^a	Existing Capacity (LOS E) ^a	Existing		
			ADT ^b	LOS ^c	V/C ^d
Escondido Boulevard					
El Norte Pkwy to Lincoln Ave	2-Ln Local Collector	19,000 ^f	9,700	B	0.51
Fig Street					
El Norte Pkwy to Lincoln Ave	2-Ln Local Collector	15,000	3,800	A	0.25
Jesmond Dene Road					
Centre City Pkwy to Ivy Dell Ln	2-Ln Local Collector	15,000	2,100	A	0.14
Ivy Dell Ln to Broadway	2-Ln Local Collector	15,000	2,900	A	0.19
Midway Drive					
El Norte Pkwy to Lincoln Ave	2-Ln Local Collector	10,000	4,600	B	0.46
Morning View Road					
El Norte Pkwy to Lincoln Ave	2-Ln Local Collector	15,000	8,200	B	0.55
North Iris Lane					
Country Club Rd to Centre City Pkwy	2-Ln Local Collector	15,000	5,300	B	0.35
Rose Street					
El Norte Pkwy to Lincoln Ave	2-Ln Local Collector	10,000	3,100	A	0.31
Seven Oaks Road					
El Norte Pkwy to Borden Rd	2-Ln Local Collector	15,000	3,400	A	0.23
Borden Rd to Rock Springs Rd	2-Ln Local Collector	15,000	2,100	A	0.14
Valley Center Road					
El Norte Pkwy to Lake Wohlford Rd	4-Ln Major	43,500 ^e	29,700	C	0.68
North of Lake Wohlford Rd	4-Ln Major	37,000	21,300	C	0.58
Vista Verde Way					
Vista Ave to El Norte Pkwy	2-Ln Local Collector	10,000	2,400	A	0.24

Footnotes:

- a. Capacities based on City of Escondido Roadway Classification & LOS table (See Table 3-1).
- b. Average Daily Traffic.
- c. Level of Service.
- d. Volume to Capacity ratio.
- e. Roadway currently built as a five-lane roadway. Average of 4-Ln Major and 6-Ln Super Major used in analysis.
- f. Escondido Boulevard has a two-way center turn lane allowing for an increased capacity of 19,000 ADT.

LOS	V/C Ratio
A	0.00 ≥ 0.34
B	0.35 ≥ 0.54
C	0.55 ≥ 0.74
D	0.75 ≥ 0.89
E	0.90 ≥ 1.00
F	>1.00

APPENDIX B

PEAK HOUR INTERSECTION ANALYSIS WORKSHEETS

EXISTING ANALYSIS WORKSHEETS

Intersection

Int Delay, s/veh 0.5

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	109	5	3	182	11	3
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	77	77	89	89	88	88
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	142	6	3	204	12	3

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	148	356
Stage 1	-	-	145
Stage 2	-	-	211
Critical Hdwy	-	4.12	6.42
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	2.218	3.518
Pot Cap-1 Maneuver	-	1434	642
Stage 1	-	-	882
Stage 2	-	-	824
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	1434	641
Mov Cap-2 Maneuver	-	-	641
Stage 1	-	-	882
Stage 2	-	-	822

Approach	EB	WB	NB
HCM Control Delay, s	0	0.1	10.4
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	683	-	-	1434	-
HCM Lane V/C Ratio	0.023	-	-	0.002	-
HCM Control Delay (s)	10.4	-	-	7.5	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.1	-	-	0	-

HCM 2010 Signalized Intersection Summary
 2: Valley Parkway/Valley Center Rd & Lake Wohlford Rd

Existing AM
 4/29/2014

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	203	5	612	113	9	1091		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	239	6	680	126	10	1212		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.85	0.85	0.90	0.90	0.90	0.90		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	283	253	2381	1065	18	1370		
Arrive On Green	0.16	0.16	0.67	0.67	0.01	0.74		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	239	6	680	126	10	1212		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	10.0	0.2	5.9	2.2	0.4	37.6		
Cycle Q Clear(g_c), s	10.0	0.2	5.9	2.2	0.4	37.6		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	283	253	2381	1065	18	1370		
V/C Ratio(X)	0.84	0.02	0.29	0.12	0.56	0.88		
Avail Cap(c_a), veh/h	373	333	2381	1065	93	1370		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.1	27.0	5.0	4.4	37.5	7.6		
Incr Delay (d2), s/veh	12.7	0.0	0.3	0.2	25.0	8.7		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	5.8	0.1	2.9	1.0	0.3	21.8		
LnGrp Delay(d),s/veh	43.8	27.0	5.4	4.7	62.5	16.3		
LnGrp LOS	D	C	A	A	E	B		
Approach Vol, veh/h	245		806			1222		
Approach Delay, s/veh	43.4		5.2			16.7		
Approach LOS	D		A			B		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	4.8	55.2				60.0		16.2
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.4	7.9				39.6		12.0
Green Ext Time (p_c), s	0.0	25.5				13.1		0.3
Intersection Summary								
HCM 2010 Ctrl Delay			15.5					
HCM 2010 LOS			B					

HCM 2010 Signalized Intersection Summary
 3: Valley Parkway & El Norte Parkway

Existing AM
 4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	324	80	143	139	79	5	97	413	111	9	914	446
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	352	87	155	196	111	7	128	543	146	10	1063	519
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.92	0.92	0.92	0.71	0.71	0.71	0.76	0.76	0.76	0.86	0.86	0.86
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	447	292	248	277	186	12	161	1791	801	18	2161	878
Arrive On Green	0.13	0.16	0.16	0.08	0.11	0.11	0.09	0.51	0.51	0.01	0.42	0.42
Sat Flow, veh/h	3442	1863	1583	3442	1734	109	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	352	87	155	196	0	118	128	543	146	10	1063	519
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1843	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	8.0	3.3	7.4	4.5	0.0	4.9	5.7	7.3	4.1	0.5	12.3	17.6
Cycle Q Clear(g_c), s	8.0	3.3	7.4	4.5	0.0	4.9	5.7	7.3	4.1	0.5	12.3	17.6
Prop In Lane	1.00		1.00	1.00		0.06	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	447	292	248	277	0	198	161	1791	801	18	2161	878
V/C Ratio(X)	0.79	0.30	0.62	0.71	0.00	0.60	0.79	0.30	0.18	0.57	0.49	0.59
Avail Cap(c_a), veh/h	595	1081	919	340	0	933	241	1791	801	88	2161	878
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	34.2	30.2	31.9	36.3	0.0	34.5	36.1	11.7	10.9	39.9	16.9	11.9
Incr Delay (d2), s/veh	5.1	0.6	2.6	5.1	0.0	2.9	10.2	0.4	0.5	25.5	0.8	2.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.1	1.8	3.4	2.3	0.0	2.7	3.3	3.7	1.9	0.3	5.9	8.4
LnGrp Delay(d),s/veh	39.3	30.8	34.5	41.4	0.0	37.4	46.3	12.1	11.4	65.4	17.7	14.9
LnGrp LOS	D	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		594			314			817			1592	
Approach Delay, s/veh		36.8			39.9			17.3			17.1	
Approach LOS		D			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.8	46.0	11.5	17.7	12.4	39.4	15.5	13.7				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	9.3	6.5	9.4	7.7	19.6	10.0	6.9				
Green Ext Time (p_c), s	0.0	18.8	0.1	1.8	0.1	10.8	0.5	1.7				
Intersection Summary												
HCM 2010 Ctrl Delay			22.8									
HCM 2010 LOS			C									

HCM 2010 Signalized Intersection Summary
4: Bear Valley Parkway & Valley Parkway

Existing AM
4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	19	364	79	758	784	18	93	26	632	29	57	51
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	23	433	94	824	852	20	121	34	821	40	78	70
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.84	0.84	0.84	0.92	0.92	0.92	0.77	0.77	0.77	0.73	0.73	0.73
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	36	752	159	954	1542	690	364	382	1344	49	96	86
Arrive On Green	0.02	0.18	0.18	0.28	0.44	0.44	0.21	0.21	0.21	0.13	0.13	0.13
Sat Flow, veh/h	1774	4208	888	3442	3539	1583	1774	1863	2787	368	718	644
Grp Volume(v), veh/h	23	346	181	824	852	20	121	34	821	188	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1706	1721	1770	1583	1774	1863	1393	1731	0	0
Q Serve(g_s), s	1.0	7.3	7.6	17.8	14.0	0.6	4.5	1.2	16.0	8.2	0.0	0.0
Cycle Q Clear(g_c), s	1.0	7.3	7.6	17.8	14.0	0.6	4.5	1.2	16.0	8.2	0.0	0.0
Prop In Lane	1.00		0.52	1.00		1.00	1.00		1.00	0.21		0.37
Lane Grp Cap(c), veh/h	36	606	305	954	1542	690	364	382	1344	232	0	0
V/C Ratio(X)	0.64	0.57	0.59	0.86	0.55	0.03	0.33	0.09	0.61	0.81	0.00	0.00
Avail Cap(c_a), veh/h	114	695	350	1147	1678	751	364	382	1344	355	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	38.0	29.3	29.4	26.8	16.4	12.6	26.5	25.1	14.8	32.8	0.0	0.0
Incr Delay (d2), s/veh	17.8	0.9	2.0	6.1	0.3	0.0	2.4	0.5	2.1	8.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.7	3.5	3.7	9.2	6.8	0.2	2.5	0.6	6.8	4.4	0.0	0.0
LnGrp Delay(d),s/veh	55.7	30.2	31.5	32.9	16.7	12.6	28.9	25.6	16.9	40.8	0.0	0.0
LnGrp LOS	E	C	C	C	B	B	C	C	B	D		
Approach Vol, veh/h		550			1696			976			188	
Approach Delay, s/veh		31.7			24.5			18.7			40.8	
Approach LOS		C			C			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	25.6	17.9		14.5	5.6	38.0				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		18.0	19.8	9.6		10.2	3.0	16.0				
Green Ext Time (p_c), s		0.0	1.9	4.4		0.4	0.0	10.0				
Intersection Summary												
HCM 2010 Ctrl Delay			24.9									
HCM 2010 LOS			C									

Intersection

Int Delay, s/veh 0.4

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	221	12	5	170	9	3
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	82	82	80	80	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	270	15	6	212	9	3

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	0	277
Stage 1	-	-	277
Stage 2	-	-	225
Critical Hdwy	-	4.12	6.42
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	2.218	3.518
Pot Cap-1 Maneuver	-	1278	762
Stage 1	-	-	770
Stage 2	-	-	812
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	1278	762
Mov Cap-2 Maneuver	-	-	526
Stage 1	-	-	770
Stage 2	-	-	808

Approach	EB	WB	NB
HCM Control Delay, s	0	0.2	11.5
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	570	-	-	1278	-
HCM Lane V/C Ratio	0.021	-	-	0.005	-
HCM Control Delay (s)	11.5	-	-	7.8	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.1	-	-	0	-

HCM 2010 Signalized Intersection Summary
 2: Valley Parkway/Valley Center Rd & Lake Wohlford Rd

Existing PM
 4/29/2014

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	210	9	1162	283	7	874		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	241	10	1210	295	7	910		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.87	0.87	0.96	0.96	0.96	0.96		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	285	255	2387	1068	13	1368		
Arrive On Green	0.16	0.16	0.67	0.67	0.01	0.73		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	241	10	1210	295	7	910		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	10.1	0.4	12.9	5.7	0.3	19.4		
Cycle Q Clear(g_c), s	10.1	0.4	12.9	5.7	0.3	19.4		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	285	255	2387	1068	13	1368		
V/C Ratio(X)	0.84	0.04	0.51	0.28	0.55	0.67		
Avail Cap(c_a), veh/h	372	332	2387	1068	93	1368		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.1	27.0	6.1	5.0	37.7	5.3		
Incr Delay (d2), s/veh	12.9	0.1	0.8	0.6	31.6	2.6		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	5.9	0.2	6.5	2.6	0.3	10.6		
LnGrp Delay(d),s/veh	44.0	27.1	6.9	5.6	69.3	7.8		
LnGrp LOS	D	C	A	A	E	A		
Approach Vol, veh/h	251		1505			917		
Approach Delay, s/veh	43.3		6.7			8.3		
Approach LOS	D		A			A		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	4.6	55.4				60.0		16.3
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.3	14.9				21.4		12.1
Green Ext Time (p_c), s	0.0	23.6				24.3		0.3
Intersection Summary								
HCM 2010 Ctrl Delay			10.7					
HCM 2010 LOS			B					

HCM 2010 Signalized Intersection Summary
 3: Valley Parkway & El Norte Parkway

Existing PM
 4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	519	54	89	61	39	6	110	983	76	9	718	407
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	583	61	100	80	51	8	116	1035	80	10	780	442
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.89	0.89	0.89	0.76	0.76	0.76	0.95	0.95	0.95	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	592	371	315	141	107	17	148	1783	798	18	2190	954
Arrive On Green	0.17	0.20	0.20	0.04	0.07	0.07	0.08	0.50	0.50	0.01	0.43	0.43
Sat Flow, veh/h	3442	1863	1583	3442	1573	247	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	583	61	100	80	0	59	116	1035	80	10	780	442
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1819	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	13.7	2.2	4.4	1.9	0.0	2.5	5.2	16.7	2.1	0.5	8.4	12.5
Cycle Q Clear(g_c), s	13.7	2.2	4.4	1.9	0.0	2.5	5.2	16.7	2.1	0.5	8.4	12.5
Prop In Lane	1.00		1.00	1.00		0.14	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	592	371	315	141	0	124	148	1783	798	18	2190	954
V/C Ratio(X)	0.98	0.16	0.32	0.57	0.00	0.47	0.79	0.58	0.10	0.57	0.36	0.46
Avail Cap(c_a), veh/h	592	1076	915	338	0	917	240	1783	798	87	2190	954
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	33.6	27.0	27.8	38.3	0.0	36.5	36.6	14.2	10.5	40.1	15.6	8.9
Incr Delay (d2), s/veh	32.9	0.2	0.6	3.5	0.0	2.8	8.8	1.4	0.3	25.5	0.5	1.6
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	9.2	1.2	2.0	0.9	0.0	1.4	2.9	8.4	1.0	0.3	4.0	5.9
LnGrp Delay(d),s/veh	66.5	27.2	28.4	41.8	0.0	39.3	45.4	15.5	10.8	65.6	16.0	10.5
LnGrp LOS	E	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		744			139			1231			1232	
Approach Delay, s/veh		58.2			40.7			18.0			14.5	
Approach LOS		E			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.8	46.0	8.3	21.2	11.8	40.0	19.0	10.6				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	18.7	3.9	6.4	7.2	14.5	15.7	4.5				
Green Ext Time (p_c), s	0.0	15.9	0.1	1.0	0.1	14.3	0.0	1.0				
Intersection Summary												
HCM 2010 Ctrl Delay			26.6									
HCM 2010 LOS			C									

HCM 2010 Signalized Intersection Summary
 4: Bear Valley Parkway & Valley Parkway

Existing PM
 4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 		 	 				 		 	
Volume (veh/h)	72	669	49	365	484	22	47	29	647	9	15	39
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	79	735	54	376	499	23	53	33	727	12	20	52
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.91	0.91	0.91	0.97	0.97	0.97	0.89	0.89	0.89	0.75	0.75	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	100	1141	83	543	1193	534	480	504	1194	16	26	67
Arrive On Green	0.06	0.24	0.24	0.16	0.34	0.34	0.27	0.27	0.27	0.07	0.07	0.07
Sat Flow, veh/h	1774	4837	354	3442	3539	1583	1774	1863	2787	238	397	1033
Grp Volume(v), veh/h	79	514	275	376	499	23	53	33	727	84	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1800	1721	1770	1583	1774	1863	1393	1669	0	0
Q Serve(g_s), s	2.6	8.1	8.1	6.1	6.4	0.6	1.3	0.8	11.9	2.9	0.0	0.0
Cycle Q Clear(g_c), s	2.6	8.1	8.1	6.1	6.4	0.6	1.3	0.8	11.9	2.9	0.0	0.0
Prop In Lane	1.00		0.20	1.00		1.00	1.00		1.00	0.14		0.62
Lane Grp Cap(c), veh/h	100	799	425	543	1193	534	480	504	1194	109	0	0
V/C Ratio(X)	0.79	0.64	0.65	0.69	0.42	0.04	0.11	0.07	0.61	0.77	0.00	0.00
Avail Cap(c_a), veh/h	150	918	487	1514	2215	991	480	504	1194	452	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	27.5	20.3	20.4	23.5	15.1	13.2	16.2	16.0	13.1	27.2	0.0	0.0
Incr Delay (d2), s/veh	14.8	1.2	2.4	1.6	0.2	0.0	0.5	0.2	2.3	11.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.7	3.9	4.3	3.0	3.2	0.3	0.7	0.4	4.9	1.7	0.0	0.0
LnGrp Delay(d),s/veh	42.3	21.6	22.8	25.1	15.4	13.2	16.7	16.3	15.4	38.3	0.0	0.0
LnGrp LOS	D	C	C	C	B	B	B	B	B	D		
Approach Vol, veh/h		868			898			813			84	
Approach Delay, s/veh		23.9			19.4			15.5			38.3	
Approach LOS		C			B			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	13.3	17.9		7.8	7.3	23.9				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		13.9	8.1	10.1		4.9	4.6	8.4				
Green Ext Time (p_c), s		0.8	1.2	3.8		0.2	0.0	10.3				
Intersection Summary												
HCM 2010 Ctrl Delay			20.3									
HCM 2010 LOS			C									

EXISTING + PROJECT ANALYSIS

Intersection

Int Delay, s/veh 1.3

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	109	35	5	182	37	4
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	77	77	89	89	88	88
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	142	45	6	204	42	5

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	187	380
Stage 1	-	-	164
Stage 2	-	-	216
Critical Hdwy	-	4.12	6.42
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	2.218	3.518
Pot Cap-1 Maneuver	-	1387	622
Stage 1	-	-	865
Stage 2	-	-	820
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	1387	619
Mov Cap-2 Maneuver	-	-	619
Stage 1	-	-	865
Stage 2	-	-	816

Approach	EB	WB	NB
HCM Control Delay, s	0	0.2	11.1
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	637	-	-	1387	-
HCM Lane V/C Ratio	0.073	-	-	0.004	-
HCM Control Delay (s)	11.1	-	-	7.6	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.2	-	-	0	-

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	227	6	612	143	11	1091		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	267	7	680	159	12	1212		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.85	0.85	0.90	0.90	0.90	0.90		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	309	276	2332	1043	21	1346		
Arrive On Green	0.17	0.17	0.66	0.66	0.01	0.72		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	267	7	680	159	12	1212		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	11.3	0.3	6.3	3.0	0.5	40.1		
Cycle Q Clear(g_c), s	11.3	0.3	6.3	3.0	0.5	40.1		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	309	276	2332	1043	21	1346		
V/C Ratio(X)	0.86	0.03	0.29	0.15	0.58	0.90		
Avail Cap(c_a), veh/h	366	327	2332	1043	92	1346		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.1	26.5	5.6	5.0	38.1	8.6		
Incr Delay (d2), s/veh	16.6	0.0	0.3	0.3	22.6	9.9		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	7.0	0.1	3.1	1.4	0.4	23.6		
LnGrp Delay(d),s/veh	47.7	26.6	5.9	5.3	60.7	18.5		
LnGrp LOS	D	C	A	A	E	B		
Approach Vol, veh/h	274		839			1224		
Approach Delay, s/veh	47.1		5.8			18.9		
Approach LOS	D		A			B		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	4.9	55.1				60.0		17.5
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.5	8.3				42.1		13.3
Green Ext Time (p_c), s	0.0	25.6				11.4		0.2
Intersection Summary								
HCM 2010 Ctrl Delay			17.5					
HCM 2010 LOS			B					

Existing + Proj AM
3: Valley Parkway & El Norte Parkway

Oakvale Road
6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	338	80	143	139	79	5	97	427	111	9	926	458
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	367	87	155	196	111	7	128	562	146	10	1077	533
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.92	0.92	0.92	0.71	0.71	0.71	0.76	0.76	0.76	0.86	0.86	0.86
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	460	299	254	276	186	12	161	1782	797	18	2149	881
Arrive On Green	0.13	0.16	0.16	0.08	0.11	0.11	0.09	0.50	0.50	0.01	0.42	0.42
Sat Flow, veh/h	3442	1863	1583	3442	1734	109	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	367	87	155	196	0	118	128	562	146	10	1077	533
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1843	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	8.4	3.3	7.4	4.5	0.0	5.0	5.8	7.6	4.1	0.5	12.6	18.3
Cycle Q Clear(g_c), s	8.4	3.3	7.4	4.5	0.0	5.0	5.8	7.6	4.1	0.5	12.6	18.3
Prop In Lane	1.00		1.00	1.00		0.06	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	460	299	254	276	0	197	161	1782	797	18	2149	881
V/C Ratio(X)	0.80	0.29	0.61	0.71	0.00	0.60	0.79	0.32	0.18	0.57	0.50	0.61
Avail Cap(c_a), veh/h	592	1075	914	338	0	928	240	1782	797	87	2149	881
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	34.2	30.1	31.8	36.5	0.0	34.7	36.3	11.9	11.1	40.1	17.2	12.1
Incr Delay (d2), s/veh	5.8	0.5	2.4	5.2	0.0	2.9	10.4	0.5	0.5	25.5	0.8	3.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.3	1.8	3.4	2.4	0.0	2.7	3.3	3.8	1.9	0.3	6.1	8.6
LnGrp Delay(d),s/veh	40.0	30.6	34.2	41.7	0.0	37.6	46.6	12.4	11.6	65.6	18.1	15.2
LnGrp LOS	D	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		609			314			836			1620	
Approach Delay, s/veh		37.2			40.2			17.5			17.4	
Approach LOS		D			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.8	46.0	11.5	18.1	12.4	39.4	15.9	13.7				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	9.6	6.5	9.4	7.8	20.3	10.4	7.0				
Green Ext Time (p_c), s	0.0	19.1	0.1	1.8	0.1	10.5	0.5	1.7				
Intersection Summary												
HCM 2010 Ctrl Delay			23.1									
HCM 2010 LOS			C									

Existing + Proj AM
4: Bear Valley Parkway & Valley Parkway

Oakvale Road
6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	19	369	79	766	788	18	93	26	642	29	57	51
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	23	439	94	833	857	20	121	34	834	40	78	70
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.84	0.84	0.84	0.92	0.92	0.92	0.77	0.77	0.77	0.73	0.73	0.73
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	36	754	157	961	1550	693	362	380	1347	49	96	86
Arrive On Green	0.02	0.18	0.18	0.28	0.44	0.44	0.20	0.20	0.20	0.13	0.13	0.13
Sat Flow, veh/h	1774	4219	879	3442	3539	1583	1774	1863	2787	368	718	644
Grp Volume(v), veh/h	23	350	183	833	857	20	121	34	834	188	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1708	1721	1770	1583	1774	1863	1393	1731	0	0
Q Serve(g_s), s	1.0	7.4	7.7	18.0	14.1	0.6	4.6	1.2	16.0	8.3	0.0	0.0
Cycle Q Clear(g_c), s	1.0	7.4	7.7	18.0	14.1	0.6	4.6	1.2	16.0	8.3	0.0	0.0
Prop In Lane	1.00		0.51	1.00		1.00	1.00		1.00	0.21		0.37
Lane Grp Cap(c), veh/h	36	606	305	961	1550	693	362	380	1347	232	0	0
V/C Ratio(X)	0.64	0.58	0.60	0.87	0.55	0.03	0.33	0.09	0.62	0.81	0.00	0.00
Avail Cap(c_a), veh/h	113	692	348	1141	1670	747	362	380	1347	353	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	38.1	29.5	29.6	26.9	16.3	12.5	26.7	25.3	14.9	33.0	0.0	0.0
Incr Delay (d2), s/veh	17.8	0.9	2.2	6.4	0.3	0.0	2.5	0.5	2.1	8.2	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.7	3.5	3.8	9.4	6.9	0.2	2.5	0.6	6.9	4.5	0.0	0.0
LnGrp Delay(d),s/veh	56.0	30.4	31.8	33.2	16.7	12.6	29.1	25.8	17.1	41.1	0.0	0.0
LnGrp LOS	E	C	C	C	B	B	C	C	B	D		
Approach Vol, veh/h		556			1710			989			188	
Approach Delay, s/veh		31.9			24.7			18.9			41.1	
Approach LOS		C			C			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	25.9	18.0		14.5	5.6	38.3				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		18.0	20.0	9.7		10.3	3.0	16.1				
Green Ext Time (p_c), s		0.0	1.8	4.3		0.4	0.0	10.1				
Intersection Summary												
HCM 2010 Ctrl Delay			25.1									
HCM 2010 LOS			C									

Intersection

Int Delay, s/veh 1.5

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	221	41	7	170	62	6
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	82	82	80	80	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	270	50	9	212	62	6

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	0	320
Stage 1	-	-	295
Stage 2	-	-	230
Critical Hdwy	-	-	4.12
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	-	2.218
Pot Cap-1 Maneuver	-	-	1240
Stage 1	-	-	755
Stage 2	-	-	808
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	-	1240
Mov Cap-2 Maneuver	-	-	509
Stage 1	-	-	755
Stage 2	-	-	802

Approach	EB	WB	NB
HCM Control Delay, s	0	0.3	12.9
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	524	-	-	1240	-
HCM Lane V/C Ratio	0.13	-	-	0.007	-
HCM Control Delay (s)	12.9	-	-	7.9	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.4	-	-	0	-

HCM 2010 Signalized Intersection Summary
 2: Valley Parkway/Valley Center Rd & Lake Wohlford Rd

6/25/2014

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	260	12	1162	312	9	874		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	299	14	1210	325	9	910		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.87	0.87	0.96	0.96	0.96	0.96		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	338	302	2295	1027	16	1319		
Arrive On Green	0.19	0.19	0.65	0.65	0.01	0.71		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	299	14	1210	325	9	910		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	13.0	0.6	14.4	7.2	0.4	22.0		
Cycle Q Clear(g_c), s	13.0	0.6	14.4	7.2	0.4	22.0		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	338	302	2295	1027	16	1319		
V/C Ratio(X)	0.88	0.05	0.53	0.32	0.56	0.69		
Avail Cap(c_a), veh/h	359	320	2295	1027	90	1319		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.2	26.1	7.4	6.1	39.0	6.6		
Incr Delay (d2), s/veh	21.3	0.1	0.9	0.8	27.0	3.0		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	8.3	0.3	7.2	3.3	0.3	12.2		
LnGrp Delay(d),s/veh	52.5	26.2	8.3	7.0	66.0	9.6		
LnGrp LOS	D	C	A	A	E	A		
Approach Vol, veh/h	313		1535			919		
Approach Delay, s/veh	51.3		8.0			10.1		
Approach LOS	D		A			B		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	4.7	55.3				60.0		19.1
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.4	16.4				24.0		15.0
Green Ext Time (p_c), s	0.0	22.9				23.1		0.1
Intersection Summary								
HCM 2010 Ctrl Delay			13.6					
HCM 2010 LOS			B					

HCM 2010 Signalized Intersection Summary

3: Valley Parkway & El Norte Parkway

6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	533	54	89	61	39	6	110	997	76	9	743	432
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	599	61	100	80	51	8	116	1049	80	10	808	470
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.89	0.89	0.89	0.76	0.76	0.76	0.95	0.95	0.95	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	592	371	315	141	107	17	148	1783	798	18	2190	954
Arrive On Green	0.17	0.20	0.20	0.04	0.07	0.07	0.08	0.50	0.50	0.01	0.43	0.43
Sat Flow, veh/h	3442	1863	1583	3442	1573	247	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	599	61	100	80	0	59	116	1049	80	10	808	470
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1819	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	14.0	2.2	4.4	1.9	0.0	2.5	5.2	17.0	2.1	0.5	8.8	13.6
Cycle Q Clear(g_c), s	14.0	2.2	4.4	1.9	0.0	2.5	5.2	17.0	2.1	0.5	8.8	13.6
Prop In Lane	1.00		1.00	1.00		0.14	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	592	371	315	141	0	124	148	1783	798	18	2190	954
V/C Ratio(X)	1.01	0.16	0.32	0.57	0.00	0.47	0.79	0.59	0.10	0.57	0.37	0.49
Avail Cap(c_a), veh/h	592	1076	915	338	0	917	240	1783	798	87	2190	954
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	33.7	27.0	27.8	38.3	0.0	36.5	36.6	14.2	10.5	40.1	15.7	9.1
Incr Delay (d2), s/veh	39.9	0.2	0.6	3.5	0.0	2.8	8.8	1.4	0.3	25.5	0.5	1.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	9.9	1.2	2.0	0.9	0.0	1.4	2.9	8.5	1.0	0.3	4.1	6.4
LnGrp Delay(d),s/veh	73.6	27.2	28.4	41.8	0.0	39.3	45.4	15.7	10.8	65.6	16.2	10.9
LnGrp LOS	F	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		760			139			1245			1288	
Approach Delay, s/veh		63.9			40.7			18.1			14.6	
Approach LOS		E			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.8	46.0	8.3	21.2	11.8	40.0	19.0	10.6				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	19.0	3.9	6.4	7.2	15.6	16.0	4.5				
Green Ext Time (p_c), s	0.0	16.1	0.1	1.0	0.1	14.0	0.0	1.0				
Intersection Summary												
HCM 2010 Ctrl Delay			27.9									
HCM 2010 LOS			C									

HCM 2010 Signalized Intersection Summary

4: Bear Valley Parkway & Valley Parkway

6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	72	674	49	382	492	22	47	29	656	9	15	39
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	79	741	54	394	507	23	53	33	737	12	20	52
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.91	0.91	0.91	0.97	0.97	0.97	0.89	0.89	0.89	0.75	0.75	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	100	1139	83	562	1210	541	476	499	1202	16	26	67
Arrive On Green	0.06	0.24	0.24	0.16	0.34	0.34	0.27	0.27	0.27	0.07	0.07	0.07
Sat Flow, veh/h	1774	4840	351	3442	3539	1583	1774	1863	2787	238	397	1033
Grp Volume(v), veh/h	79	518	277	394	507	23	53	33	737	84	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1801	1721	1770	1583	1774	1863	1393	1669	0	0
Q Serve(g_s), s	2.6	8.2	8.3	6.5	6.6	0.6	1.3	0.8	12.2	3.0	0.0	0.0
Cycle Q Clear(g_c), s	2.6	8.2	8.3	6.5	6.6	0.6	1.3	0.8	12.2	3.0	0.0	0.0
Prop In Lane	1.00		0.19	1.00		1.00	1.00		1.00	0.14		0.62
Lane Grp Cap(c), veh/h	100	798	424	562	1210	541	476	499	1202	109	0	0
V/C Ratio(X)	0.79	0.65	0.65	0.70	0.42	0.04	0.11	0.07	0.61	0.77	0.00	0.00
Avail Cap(c_a), veh/h	149	909	483	1500	2195	982	476	499	1202	447	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	27.8	20.6	20.6	23.6	15.1	13.1	16.5	16.3	13.1	27.5	0.0	0.0
Incr Delay (d2), s/veh	15.1	1.3	2.6	1.6	0.2	0.0	0.5	0.3	2.3	11.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.7	4.0	4.4	3.2	3.2	0.3	0.7	0.4	5.0	1.7	0.0	0.0
LnGrp Delay(d),s/veh	42.9	21.9	23.2	25.2	15.3	13.1	16.9	16.5	15.4	38.5	0.0	0.0
LnGrp LOS	D	C	C	C	B	B	B	B	B	D		
Approach Vol, veh/h		874			924			823			84	
Approach Delay, s/veh		24.2			19.5			15.6			38.5	
Approach LOS		C			B			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	13.7	18.0		7.9	7.4	24.4				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		14.2	8.5	10.3		5.0	4.6	8.6				
Green Ext Time (p_c), s		0.7	1.3	3.7		0.2	0.0	10.5				
Intersection Summary												
HCM 2010 Ctrl Delay			20.4									
HCM 2010 LOS			C									

EXISTING + CUMULATIVE ANALYSIS WORKSHEETS

Intersection

Int Delay, s/veh 0.5

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	116	5	3	193	11	3
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	77	77	89	89	88	88
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	151	6	3	217	12	3

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	157	378
Stage 1	-	-	154
Stage 2	-	-	224
Critical Hdwy	-	4.12	6.42
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	2.218	3.518
Pot Cap-1 Maneuver	-	1423	624
Stage 1	-	-	874
Stage 2	-	-	813
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	1423	623
Mov Cap-2 Maneuver	-	-	623
Stage 1	-	-	874
Stage 2	-	-	811

Approach	EB	WB	NB
HCM Control Delay, s	0	0.1	10.5
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	666	-	-	1423	-
HCM Lane V/C Ratio	0.024	-	-	0.002	-
HCM Control Delay (s)	10.5	-	-	7.5	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.1	-	-	0	-

HCM 2010 Signalized Intersection Summary
 2: Valley Parkway/Valley Center Rd & Lake Wohlford Rd

Existing + Cumulative AM
 4/29/2014

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	214	5	636	120	10	1134		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	252	6	707	133	11	1260		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.85	0.85	0.90	0.90	0.90	0.90		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	296	264	2358	1055	19	1358		
Arrive On Green	0.17	0.17	0.67	0.67	0.01	0.73		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	252	6	707	133	11	1260		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	10.6	0.2	6.4	2.4	0.5	43.5		
Cycle Q Clear(g_c), s	10.6	0.2	6.4	2.4	0.5	43.5		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	296	264	2358	1055	19	1358		
V/C Ratio(X)	0.85	0.02	0.30	0.13	0.57	0.93		
Avail Cap(c_a), veh/h	370	330	2358	1055	92	1358		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.1	26.8	5.3	4.7	37.8	8.7		
Incr Delay (d2), s/veh	14.5	0.0	0.3	0.2	23.7	12.3		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	6.4	0.1	3.2	1.1	0.4	26.3		
LnGrp Delay(d),s/veh	45.6	26.8	5.7	4.9	61.5	21.0		
LnGrp LOS	D	C	A	A	E	C		
Approach Vol, veh/h	258		840			1271		
Approach Delay, s/veh	45.1		5.6			21.4		
Approach LOS	D		A			C		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	4.8	55.2				60.0		16.8
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.5	8.4				45.5		12.6
Green Ext Time (p_c), s	0.0	27.0				9.2		0.3
Intersection Summary								
HCM 2010 Ctrl Delay			18.3					
HCM 2010 LOS			B					

HCM 2010 Signalized Intersection Summary
3: Valley Parkway & El Norte Parkway

Existing + Cumulative AM
4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	334	80	147	141	79	5	97	418	111	9	933	461
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	363	87	160	199	111	7	128	550	146	10	1085	536
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.92	0.92	0.92	0.71	0.71	0.71	0.76	0.76	0.76	0.86	0.86	0.86
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	457	296	252	279	186	12	161	1784	798	18	2151	880
Arrive On Green	0.13	0.16	0.16	0.08	0.11	0.11	0.09	0.50	0.50	0.01	0.42	0.42
Sat Flow, veh/h	3442	1863	1583	3442	1734	109	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	363	87	160	199	0	118	128	550	146	10	1085	536
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1843	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	8.3	3.4	7.7	4.6	0.0	5.0	5.7	7.4	4.1	0.5	12.7	18.5
Cycle Q Clear(g_c), s	8.3	3.4	7.7	4.6	0.0	5.0	5.7	7.4	4.1	0.5	12.7	18.5
Prop In Lane	1.00		1.00	1.00		0.06	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	457	296	252	279	0	198	161	1784	798	18	2151	880
V/C Ratio(X)	0.79	0.29	0.64	0.71	0.00	0.60	0.79	0.31	0.18	0.57	0.50	0.61
Avail Cap(c_a), veh/h	592	1076	915	339	0	929	240	1784	798	87	2151	880
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	34.2	30.2	32.0	36.4	0.0	34.6	36.2	11.8	11.0	40.1	17.2	12.1
Incr Delay (d2), s/veh	5.6	0.5	2.7	5.4	0.0	2.9	10.3	0.4	0.5	25.5	0.8	3.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.3	1.8	3.5	2.4	0.0	2.7	3.3	3.7	1.9	0.3	6.1	8.7
LnGrp Delay(d),s/veh	39.8	30.7	34.7	41.9	0.0	37.5	46.5	12.3	11.5	65.6	18.1	15.3
LnGrp LOS	D	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		610			317			824			1631	
Approach Delay, s/veh		37.2			40.2			17.5			17.4	
Approach LOS		D			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.8	46.0	11.6	17.9	12.4	39.4	15.8	13.7				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	9.4	6.6	9.7	7.7	20.5	10.3	7.0				
Green Ext Time (p_c), s	0.0	19.2	0.1	1.8	0.1	10.4	0.5	1.8				
Intersection Summary												
HCM 2010 Ctrl Delay			23.1									
HCM 2010 LOS			C									

HCM 2010 Signalized Intersection Summary
4: Bear Valley Parkway & Valley Parkway

Existing + Cumulative AM

4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 		 	 			 	 		 	
Volume (veh/h)	19	370	80	774	797	18	95	26	645	29	57	51
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	23	440	95	841	866	20	123	34	838	40	78	70
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.84	0.84	0.84	0.92	0.92	0.92	0.77	0.77	0.77	0.73	0.73	0.73
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	36	753	158	967	1556	696	361	379	1349	49	96	86
Arrive On Green	0.02	0.18	0.18	0.28	0.44	0.44	0.20	0.20	0.20	0.13	0.13	0.13
Sat Flow, veh/h	1774	4212	885	3442	3539	1583	1774	1863	2787	368	718	644
Grp Volume(v), veh/h	23	352	183	841	866	20	123	34	838	188	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1707	1721	1770	1583	1774	1863	1393	1731	0	0
Q Serve(g_s), s	1.0	7.5	7.8	18.3	14.3	0.6	4.7	1.2	16.0	8.3	0.0	0.0
Cycle Q Clear(g_c), s	1.0	7.5	7.8	18.3	14.3	0.6	4.7	1.2	16.0	8.3	0.0	0.0
Prop In Lane	1.00		0.52	1.00		1.00	1.00		1.00	0.21		0.37
Lane Grp Cap(c), veh/h	36	606	305	967	1556	696	361	379	1349	231	0	0
V/C Ratio(X)	0.65	0.58	0.60	0.87	0.56	0.03	0.34	0.09	0.62	0.81	0.00	0.00
Avail Cap(c_a), veh/h	113	689	347	1137	1664	744	361	379	1349	352	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	38.3	29.6	29.7	26.9	16.4	12.5	26.8	25.4	15.0	33.1	0.0	0.0
Incr Delay (d2), s/veh	17.9	1.0	2.3	6.6	0.4	0.0	2.6	0.5	2.2	8.3	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.7	3.6	3.9	9.5	7.1	0.2	2.5	0.6	7.1	4.5	0.0	0.0
LnGrp Delay(d),s/veh	56.2	30.6	32.0	33.5	16.7	12.5	29.4	25.9	17.1	41.4	0.0	0.0
LnGrp LOS	E	C	C	C	B	B	C	C	B	D		
Approach Vol, veh/h		558			1727			995			188	
Approach Delay, s/veh		32.1			24.9			18.9			41.4	
Approach LOS		C			C			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	26.1	18.1		14.5	5.6	38.6				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		18.0	20.3	9.8		10.3	3.0	16.3				
Green Ext Time (p_c), s		0.0	1.8	4.3		0.4	0.0	10.1				
Intersection Summary												
HCM 2010 Ctrl Delay			25.2									
HCM 2010 LOS			C									

Intersection

Int Delay, s/veh 0.3

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	235	12	5	180	9	3
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	82	82	80	80	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	287	15	6	225	9	3

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	0	294
Stage 1	-	-	294
Stage 2	-	-	238
Critical Hdwy	-	4.12	6.22
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	2.218	3.318
Pot Cap-1 Maneuver	-	1260	745
Stage 1	-	-	756
Stage 2	-	-	802
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	1260	745
Mov Cap-2 Maneuver	-	-	505
Stage 1	-	-	756
Stage 2	-	-	798

Approach	EB	WB	NB
HCM Control Delay, s	0	0.2	11.7
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	549	-	-	1260	-
HCM Lane V/C Ratio	0.022	-	-	0.005	-
HCM Control Delay (s)	11.7	-	-	7.9	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.1	-	-	0	-

HCM 2010 Signalized Intersection Summary
 2: Valley Parkway/Valley Center Rd & Lake Wohlford Rd

Existing + Cumulative PM
 4/29/2014

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	222	10	1208	299	8	909		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	255	11	1258	311	8	947		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.87	0.87	0.96	0.96	0.96	0.96		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	299	266	2363	1057	14	1356		
Arrive On Green	0.17	0.17	0.67	0.67	0.01	0.73		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	255	11	1258	311	8	947		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	10.7	0.4	14.1	6.3	0.3	21.7		
Cycle Q Clear(g_c), s	10.7	0.4	14.1	6.3	0.3	21.7		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	299	266	2363	1057	14	1356		
V/C Ratio(X)	0.85	0.04	0.53	0.29	0.55	0.70		
Avail Cap(c_a), veh/h	369	329	2363	1057	92	1356		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.1	26.8	6.6	5.3	38.0	5.8		
Incr Delay (d2), s/veh	14.8	0.1	0.9	0.7	28.9	3.0		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	6.5	0.2	7.1	2.9	0.3	11.9		
LnGrp Delay(d),s/veh	45.9	26.9	7.5	6.0	66.9	8.8		
LnGrp LOS	D	C	A	A	E	A		
Approach Vol, veh/h	266		1569			955		
Approach Delay, s/veh	45.1		7.2			9.3		
Approach LOS	D		A			A		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	4.6	55.4				60.0		17.0
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.3	16.1				23.7		12.7
Green Ext Time (p_c), s	0.0	23.9				24.2		0.3
Intersection Summary								
HCM 2010 Ctrl Delay			11.5					
HCM 2010 LOS			B					

HCM 2010 Signalized Intersection Summary
 3: Valley Parkway & El Norte Parkway

Existing + Cumulative PM

4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	532	60	91	64	42	7	111	1010	83	10	742	412
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	598	67	102	84	55	9	117	1063	87	11	807	448
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.89	0.89	0.89	0.76	0.76	0.76	0.95	0.95	0.95	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	589	375	318	144	112	18	149	1775	794	19	2179	950
Arrive On Green	0.17	0.20	0.20	0.04	0.07	0.07	0.08	0.50	0.50	0.01	0.43	0.43
Sat Flow, veh/h	3442	1863	1583	3442	1562	256	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	598	67	102	84	0	64	117	1063	87	11	807	448
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1818	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	14.0	2.4	4.5	2.0	0.0	2.8	5.3	17.5	2.4	0.5	8.8	12.9
Cycle Q Clear(g_c), s	14.0	2.4	4.5	2.0	0.0	2.8	5.3	17.5	2.4	0.5	8.8	12.9
Prop In Lane	1.00		1.00	1.00		0.14	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	589	375	318	144	0	130	149	1775	794	19	2179	950
V/C Ratio(X)	1.01	0.18	0.32	0.58	0.00	0.49	0.79	0.60	0.11	0.57	0.37	0.47
Avail Cap(c_a), veh/h	589	1071	910	337	0	912	239	1775	794	87	2179	950
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	33.9	27.1	27.9	38.5	0.0	36.5	36.7	14.5	10.7	40.2	15.9	9.1
Incr Delay (d2), s/veh	40.8	0.2	0.6	3.7	0.0	2.8	8.8	1.5	0.3	24.2	0.5	1.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	9.9	1.3	2.0	1.0	0.0	1.5	3.0	8.8	1.1	0.4	4.2	6.0
LnGrp Delay(d),s/veh	74.6	27.3	28.5	42.1	0.0	39.4	45.6	16.0	11.0	64.4	16.4	10.8
LnGrp LOS	F	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		767			148			1267			1266	
Approach Delay, s/veh		64.4			40.9			18.4			14.8	
Approach LOS		E			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.9	46.0	8.4	21.4	11.9	40.0	19.0	10.9				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	19.5	4.0	6.5	7.3	14.9	16.0	4.8				
Green Ext Time (p_c), s	0.0	15.8	0.1	1.1	0.1	14.4	0.0	1.1				
Intersection Summary												
HCM 2010 Ctrl Delay			28.3									
HCM 2010 LOS			C									

HCM 2010 Signalized Intersection Summary
4: Bear Valley Parkway & Valley Parkway

Existing + Cumulative PM

4/29/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	72	680	50	372	492	22	48	29	660	9	15	39
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	79	747	55	384	507	23	54	33	742	12	20	52
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.91	0.91	0.91	0.97	0.97	0.97	0.89	0.89	0.89	0.75	0.75	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	100	1144	84	551	1204	539	477	501	1196	16	26	67
Arrive On Green	0.06	0.24	0.24	0.16	0.34	0.34	0.27	0.27	0.27	0.07	0.07	0.07
Sat Flow, veh/h	1774	4836	354	3442	3539	1583	1774	1863	2787	238	397	1033
Grp Volume(v), veh/h	79	523	279	384	507	23	54	33	742	84	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1800	1721	1770	1583	1774	1863	1393	1669	0	0
Q Serve(g_s), s	2.6	8.3	8.3	6.3	6.6	0.6	1.4	0.8	12.3	2.9	0.0	0.0
Cycle Q Clear(g_c), s	2.6	8.3	8.3	6.3	6.6	0.6	1.4	0.8	12.3	2.9	0.0	0.0
Prop In Lane	1.00		0.20	1.00		1.00	1.00		1.00	0.14		0.62
Lane Grp Cap(c), veh/h	100	802	426	551	1204	539	477	501	1196	109	0	0
V/C Ratio(X)	0.79	0.65	0.66	0.70	0.42	0.04	0.11	0.07	0.62	0.77	0.00	0.00
Avail Cap(c_a), veh/h	149	912	484	1505	2202	985	477	501	1196	449	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	27.7	20.5	20.5	23.6	15.1	13.1	16.4	16.2	13.2	27.4	0.0	0.0
Incr Delay (d2), s/veh	15.0	1.4	2.7	1.6	0.2	0.0	0.5	0.3	2.4	11.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.7	4.0	4.4	3.1	3.2	0.3	0.7	0.4	5.0	1.7	0.0	0.0
LnGrp Delay(d),s/veh	42.7	21.9	23.2	25.2	15.3	13.2	16.9	16.4	15.6	38.4	0.0	0.0
LnGrp LOS	D	C	C	C	B	B	B	B	B	D		
Approach Vol, veh/h		881			914			829			84	
Approach Delay, s/veh		24.1			19.4			15.7			38.4	
Approach LOS		C			B			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	13.5	18.1		7.9	7.4	24.2				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		14.3	8.3	10.3		4.9	4.6	8.6				
Green Ext Time (p_c), s		0.7	1.3	3.7		0.2	0.0	10.5				
Intersection Summary												
HCM 2010 Ctrl Delay			20.4									
HCM 2010 LOS			C									

EXISTING + PROJECT + CUMULATIVE ANALYSIS WORKSHEETS

Intersection

Int Delay, s/veh 1.2

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	116	35	5	193	37	4
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	77	77	89	89	88	88
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	151	45	6	217	42	5

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	196	401
Stage 1	-	-	173
Stage 2	-	-	228
Critical Hdwy	-	4.12	6.42
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	2.218	3.518
Pot Cap-1 Maneuver	-	1377	605
Stage 1	-	-	857
Stage 2	-	-	810
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	1377	602
Mov Cap-2 Maneuver	-	-	602
Stage 1	-	-	857
Stage 2	-	-	806

Approach	EB	WB	NB
HCM Control Delay, s	0	0.2	11.3
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	621	-	-	1377	-
HCM Lane V/C Ratio	0.075	-	-	0.004	-
HCM Control Delay (s)	11.3	-	-	7.6	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.2	-	-	0	-

Existing + Proj AM
 2: Valley Parkway/Valley Center Rd & Lake Wohlford Rd

Oakvale Road
 6/25/2014

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	238	6	636	150	12	1134		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	280	7	707	167	13	1260		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.85	0.85	0.90	0.90	0.90	0.90		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	321	287	2310	1034	22	1335		
Arrive On Green	0.18	0.18	0.65	0.65	0.01	0.72		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	280	7	707	167	13	1260		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	12.0	0.3	6.8	3.2	0.6	46.3		
Cycle Q Clear(g_c), s	12.0	0.3	6.8	3.2	0.6	46.3		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	321	287	2310	1034	22	1335		
V/C Ratio(X)	0.87	0.02	0.31	0.16	0.58	0.94		
Avail Cap(c_a), veh/h	363	324	2310	1034	91	1335		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.1	26.3	5.9	5.3	38.4	9.7		
Incr Delay (d2), s/veh	18.5	0.0	0.3	0.3	21.8	14.4		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	7.5	0.1	3.4	1.5	0.4	28.5		
LnGrp Delay(d),s/veh	49.6	26.4	6.2	5.6	60.1	24.1		
LnGrp LOS	D	C	A	A	E	C		
Approach Vol, veh/h	287		874			1273		
Approach Delay, s/veh	49.0		6.1			24.5		
Approach LOS	D		A			C		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	5.0	55.0				60.0		18.2
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.6	8.8				48.3		14.0
Green Ext Time (p_c), s	0.0	27.0				6.9		0.2
Intersection Summary								
HCM 2010 Ctrl Delay			20.8					
HCM 2010 LOS			C					

Existing + Proj AM
3: Valley Parkway & El Norte Parkway

Oakvale Road
6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	348	80	147	141	79	5	97	432	111	9	945	473
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	378	87	160	199	111	7	128	568	146	10	1099	550
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.92	0.92	0.92	0.71	0.71	0.71	0.76	0.76	0.76	0.86	0.86	0.86
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	470	303	258	279	186	12	161	1775	794	18	2139	882
Arrive On Green	0.14	0.16	0.16	0.08	0.11	0.11	0.09	0.50	0.50	0.01	0.42	0.42
Sat Flow, veh/h	3442	1863	1583	3442	1734	109	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	378	87	160	199	0	118	128	568	146	10	1099	550
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1843	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	8.7	3.4	7.7	4.6	0.0	5.0	5.8	7.8	4.1	0.5	13.1	19.3
Cycle Q Clear(g_c), s	8.7	3.4	7.7	4.6	0.0	5.0	5.8	7.8	4.1	0.5	13.1	19.3
Prop In Lane	1.00		1.00	1.00		0.06	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	470	303	258	279	0	197	161	1775	794	18	2139	882
V/C Ratio(X)	0.80	0.29	0.62	0.71	0.00	0.60	0.79	0.32	0.18	0.57	0.51	0.62
Avail Cap(c_a), veh/h	589	1071	910	337	0	925	239	1775	794	87	2139	882
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	34.2	30.1	31.9	36.6	0.0	34.8	36.4	12.1	11.2	40.3	17.5	12.3
Incr Delay (d2), s/veh	6.4	0.5	2.4	5.5	0.0	2.9	10.5	0.5	0.5	25.5	0.9	3.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.6	1.8	3.5	2.4	0.0	2.7	3.3	3.9	1.9	0.3	6.3	9.1
LnGrp Delay(d),s/veh	40.6	30.6	34.3	42.1	0.0	37.7	46.9	12.6	11.7	65.8	18.4	15.6
LnGrp LOS	D	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		625			317			842			1659	
Approach Delay, s/veh		37.6			40.5			17.6			17.7	
Approach LOS		D			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.8	46.0	11.6	18.3	12.4	39.4	16.2	13.8				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	9.8	6.6	9.7	7.8	21.3	10.7	7.0				
Green Ext Time (p_c), s	0.0	19.5	0.1	1.8	0.1	10.0	0.5	1.8				
Intersection Summary												
HCM 2010 Ctrl Delay			23.4									
HCM 2010 LOS			C									

Existing + Proj AM
4: Bear Valley Parkway & Valley Parkway

Oakvale Road
6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		  		 	 				 		 	
Volume (veh/h)	19	375	80	782	801	18	95	26	655	29	57	51
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	23	446	95	850	871	20	123	34	851	40	78	70
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.84	0.84	0.84	0.92	0.92	0.92	0.77	0.77	0.77	0.73	0.73	0.73
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	36	755	157	973	1563	699	359	377	1352	49	96	86
Arrive On Green	0.02	0.18	0.18	0.28	0.44	0.44	0.20	0.20	0.20	0.13	0.13	0.13
Sat Flow, veh/h	1774	4223	876	3442	3539	1583	1774	1863	2787	368	718	644
Grp Volume(v), veh/h	23	356	185	850	871	20	123	34	851	188	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1708	1721	1770	1583	1774	1863	1393	1731	0	0
Q Serve(g_s), s	1.0	7.6	7.9	18.6	14.4	0.6	4.7	1.2	16.0	8.3	0.0	0.0
Cycle Q Clear(g_c), s	1.0	7.6	7.9	18.6	14.4	0.6	4.7	1.2	16.0	8.3	0.0	0.0
Prop In Lane	1.00		0.51	1.00		1.00	1.00		1.00	0.21		0.37
Lane Grp Cap(c), veh/h	36	606	306	973	1563	699	359	377	1352	231	0	0
V/C Ratio(X)	0.65	0.59	0.61	0.87	0.56	0.03	0.34	0.09	0.63	0.81	0.00	0.00
Avail Cap(c_a), veh/h	112	686	346	1132	1656	741	359	377	1352	350	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	38.5	29.8	29.9	27.0	16.4	12.5	27.0	25.6	15.1	33.3	0.0	0.0
Incr Delay (d2), s/veh	17.9	1.0	2.4	6.9	0.4	0.0	2.6	0.5	2.2	8.4	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.7	3.6	3.9	9.8	7.1	0.2	2.5	0.7	7.2	4.5	0.0	0.0
LnGrp Delay(d),s/veh	56.4	30.8	32.3	33.9	16.7	12.5	29.6	26.1	17.3	41.7	0.0	0.0
LnGrp LOS	E	C	C	C	B	B	C	C	B	D		
Approach Vol, veh/h		564			1741			1008			188	
Approach Delay, s/veh		32.4			25.1			19.1			41.7	
Approach LOS		C			C			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	26.4	18.1		14.6	5.6	38.9				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		18.0	20.6	9.9		10.3	3.0	16.4				
Green Ext Time (p_c), s		0.0	1.8	4.2		0.4	0.0	10.2				
Intersection Summary												
HCM 2010 Ctrl Delay			25.4									
HCM 2010 LOS			C									

Intersection

Int Delay, s/veh 1.5

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Vol, veh/h	235	41	7	180	62	6
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Free	Free	Free	Free	Stop	Stop
RT Channelized	-	None	-	None	-	None
Storage Length	-	-	-	-	0	-
Veh in Median Storage, #	0	-	-	0	0	-
Grade, %	0	-	-	0	0	-
Peak Hour Factor	82	82	80	80	100	100
Heavy Vehicles, %	2	2	2	2	2	2
Mvmt Flow	287	50	9	225	62	6

Major/Minor	Major1	Major2	Minor1
Conflicting Flow All	0	0	337
Stage 1	-	-	312
Stage 2	-	-	243
Critical Hdwy	-	-	4.12
Critical Hdwy Stg 1	-	-	5.42
Critical Hdwy Stg 2	-	-	5.42
Follow-up Hdwy	-	-	2.218
Pot Cap-1 Maneuver	-	-	1222
Stage 1	-	-	742
Stage 2	-	-	797
Platoon blocked, %	-	-	-
Mov Cap-1 Maneuver	-	-	1222
Mov Cap-2 Maneuver	-	-	489
Stage 1	-	-	742
Stage 2	-	-	791

Approach	EB	WB	NB
HCM Control Delay, s	0	0.3	13.3
HCM LOS			B

Minor Lane/Major Mvmt	NBLn1	EBT	EBR	WBL	WBT
Capacity (veh/h)	504	-	-	1222	-
HCM Lane V/C Ratio	0.135	-	-	0.007	-
HCM Control Delay (s)	13.3	-	-	8	0
HCM Lane LOS	B	-	-	A	A
HCM 95th %tile Q(veh)	0.5	-	-	0	-

HCM 2010 Signalized Intersection Summary
 2: Valley Parkway/Valley Center Rd & Lake Wohlford Rd

6/25/2014

								
Movement	WBL	WBR	NBT	NBR	SBL	SBT		
Lane Configurations								
Volume (veh/h)	272	13	1208	328	10	909		
Number	3	18	2	12	1	6		
Initial Q (Qb), veh	0	0	0	0	0	0		
Ped-Bike Adj(A_pbT)	1.00	1.00		1.00	1.00			
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00		
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1863		
Adj Flow Rate, veh/h	313	15	1258	342	10	947		
Adj No. of Lanes	1	1	2	1	1	1		
Peak Hour Factor	0.87	0.87	0.96	0.96	0.96	0.96		
Percent Heavy Veh, %	2	2	2	2	2	2		
Cap, veh/h	350	312	2273	1017	18	1308		
Arrive On Green	0.20	0.20	0.64	0.64	0.01	0.70		
Sat Flow, veh/h	1774	1583	3632	1583	1774	1863		
Grp Volume(v), veh/h	313	15	1258	342	10	947		
Grp Sat Flow(s),veh/h/ln	1774	1583	1770	1583	1774	1863		
Q Serve(g_s), s	13.7	0.6	15.7	7.9	0.4	24.5		
Cycle Q Clear(g_c), s	13.7	0.6	15.7	7.9	0.4	24.5		
Prop In Lane	1.00	1.00		1.00	1.00			
Lane Grp Cap(c), veh/h	350	312	2273	1017	18	1308		
V/C Ratio(X)	0.89	0.05	0.55	0.34	0.57	0.72		
Avail Cap(c_a), veh/h	356	318	2273	1017	89	1308		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00		
Uniform Delay (d), s/veh	31.2	25.9	7.9	6.5	39.3	7.2		
Incr Delay (d2), s/veh	23.5	0.1	1.0	0.9	25.3	3.5		
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0		
%ile BackOfQ(50%),veh/ln	9.0	0.3	7.8	3.7	0.3	13.6		
LnGrp Delay(d),s/veh	54.7	26.0	8.9	7.4	64.6	10.7		
LnGrp LOS	D	C	A	A	E	B		
Approach Vol, veh/h	328		1600			957		
Approach Delay, s/veh	53.4		8.6			11.3		
Approach LOS	D		A			B		
Timer	1	2	3	4	5	6	7	8
Assigned Phs	1	2				6		8
Phs Duration (G+Y+Rc), s	4.8	55.2				60.0		19.7
Change Period (Y+Rc), s	4.0	4.0				4.0		4.0
Max Green Setting (Gmax), s	4.0	48.0				56.0		16.0
Max Q Clear Time (g_c+I1), s	2.4	17.7				26.5		15.7
Green Ext Time (p_c), s	0.0	23.1				22.6		0.0
Intersection Summary								
HCM 2010 Ctrl Delay			14.6					
HCM 2010 LOS			B					

HCM 2010 Signalized Intersection Summary
 3: Valley Parkway & El Norte Parkway

6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	546	60	91	64	42	7	111	1024	83	10	767	437
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1863	1863	1863	1900	1863	1863	1863	1863	1863	1863
Adj Flow Rate, veh/h	613	67	102	84	55	9	117	1078	87	11	834	475
Adj No. of Lanes	2	1	1	2	1	0	1	2	1	1	3	1
Peak Hour Factor	0.89	0.89	0.89	0.76	0.76	0.76	0.95	0.95	0.95	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	589	375	318	144	112	18	149	1775	794	19	2179	950
Arrive On Green	0.17	0.20	0.20	0.04	0.07	0.07	0.08	0.50	0.50	0.01	0.43	0.43
Sat Flow, veh/h	3442	1863	1583	3442	1562	256	1774	3539	1583	1774	5085	1583
Grp Volume(v), veh/h	613	67	102	84	0	64	117	1078	87	11	834	475
Grp Sat Flow(s),veh/h/ln	1721	1863	1583	1721	0	1818	1774	1770	1583	1774	1695	1583
Q Serve(g_s), s	14.0	2.4	4.5	2.0	0.0	2.8	5.3	17.8	2.4	0.5	9.2	14.0
Cycle Q Clear(g_c), s	14.0	2.4	4.5	2.0	0.0	2.8	5.3	17.8	2.4	0.5	9.2	14.0
Prop In Lane	1.00		1.00	1.00		0.14	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	589	375	318	144	0	130	149	1775	794	19	2179	950
V/C Ratio(X)	1.04	0.18	0.32	0.58	0.00	0.49	0.79	0.61	0.11	0.57	0.38	0.50
Avail Cap(c_a), veh/h	589	1071	910	337	0	912	239	1775	794	87	2179	950
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	33.9	27.1	27.9	38.5	0.0	36.5	36.7	14.6	10.7	40.2	16.0	9.4
Incr Delay (d2), s/veh	47.9	0.2	0.6	3.7	0.0	2.8	8.8	1.6	0.3	24.2	0.5	1.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	10.5	1.3	2.0	1.0	0.0	1.5	3.0	9.1	1.1	0.4	4.4	6.5
LnGrp Delay(d),s/veh	81.7	27.3	28.5	42.1	0.0	39.4	45.6	16.2	11.0	64.4	16.5	11.2
LnGrp LOS	F	C	C	D		D	D	B	B	E	B	B
Approach Vol, veh/h		782			148			1282			1320	
Approach Delay, s/veh		70.1			40.9			18.5			15.0	
Approach LOS		E			D			B			B	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	5.9	46.0	8.4	21.4	11.9	40.0	19.0	10.9				
Change Period (Y+Rc), s	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0				
Max Green Setting (Gmax), s	4.0	41.0	8.0	47.0	11.0	34.0	14.0	41.0				
Max Q Clear Time (g_c+I1), s	2.5	19.8	4.0	6.5	7.3	16.0	16.0	4.8				
Green Ext Time (p_c), s	0.0	16.0	0.1	1.1	0.1	14.0	0.0	1.1				
Intersection Summary												
HCM 2010 Ctrl Delay			29.6									
HCM 2010 LOS			C									

HCM 2010 Signalized Intersection Summary
 4: Bear Valley Parkway & Valley Parkway

6/25/2014

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	72	685	50	389	500	22	48	29	669	9	15	39
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1863	1863	1900	1863	1863	1863	1863	1863	1863	1900	1863	1900
Adj Flow Rate, veh/h	79	753	55	401	515	23	54	33	752	12	20	52
Adj No. of Lanes	1	3	0	2	2	1	1	1	2	0	1	0
Peak Hour Factor	0.91	0.91	0.91	0.97	0.97	0.97	0.89	0.89	0.89	0.75	0.75	0.75
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	100	1143	83	569	1221	546	473	497	1204	16	26	67
Arrive On Green	0.06	0.24	0.24	0.17	0.34	0.34	0.27	0.27	0.27	0.07	0.07	0.07
Sat Flow, veh/h	1774	4839	352	3442	3539	1583	1774	1863	2787	238	397	1033
Grp Volume(v), veh/h	79	526	282	401	515	23	54	33	752	84	0	0
Grp Sat Flow(s),veh/h/ln	1774	1695	1801	1721	1770	1583	1774	1863	1393	1669	0	0
Q Serve(g_s), s	2.6	8.4	8.5	6.6	6.7	0.6	1.4	0.8	12.6	3.0	0.0	0.0
Cycle Q Clear(g_c), s	2.6	8.4	8.5	6.6	6.7	0.6	1.4	0.8	12.6	3.0	0.0	0.0
Prop In Lane	1.00		0.20	1.00		1.00	1.00		1.00	0.14		0.62
Lane Grp Cap(c), veh/h	100	801	425	569	1221	546	473	497	1204	109	0	0
V/C Ratio(X)	0.79	0.66	0.66	0.70	0.42	0.04	0.11	0.07	0.62	0.77	0.00	0.00
Avail Cap(c_a), veh/h	148	904	480	1492	2183	976	473	497	1204	445	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	27.9	20.7	20.7	23.7	15.1	13.1	16.6	16.4	13.3	27.6	0.0	0.0
Incr Delay (d2), s/veh	15.3	1.5	2.9	1.6	0.2	0.0	0.5	0.3	2.5	11.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.7	4.1	4.6	3.2	3.3	0.3	0.7	0.4	5.2	1.7	0.0	0.0
LnGrp Delay(d),s/veh	43.3	22.2	23.6	25.3	15.3	13.1	17.1	16.7	15.7	38.6	0.0	0.0
LnGrp LOS	D	C	C	C	B	B	B	B	B	D		
Approach Vol, veh/h		887			939			839			84	
Approach Delay, s/veh		24.5			19.5			15.8			38.6	
Approach LOS		C			B			B			D	
Timer	1	2	3	4	5	6	7	8				
Assigned Phs		2	3	4		6	7	8				
Phs Duration (G+Y+Rc), s		20.0	13.9	18.2		7.9	7.4	24.7				
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0				
Max Green Setting (Gmax), s		16.0	26.0	16.0		16.0	5.0	37.0				
Max Q Clear Time (g_c+I1), s		14.6	8.6	10.5		5.0	4.6	8.7				
Green Ext Time (p_c), s		0.6	1.3	3.7		0.2	0.0	10.7				
Intersection Summary												
HCM 2010 Ctrl Delay			20.6									
HCM 2010 LOS			C									

APPENDIX C

CITY OF ESCONDIDO ROADWAY CLASSIFICATION TABLE

Table

CITY OF ESCONDIDO PROPOSED LEVEL OF SERVICE STANDARDS

STREET SEGMENT AVERAGE DAILY VEHICLE TRIP THRESHOLDS

Street Classification	Lanes	Cross Sections	Level of Service				
			A	B	C	D	E
Prime Arterial	(8lanes)	116/136 (NP)	23,800	37,800	51,800	62,300	70,000
	(6lanes)	106/126 (NP)	20,400	32,400	44,400	53,400	60,000
Major Road	(6lanes)	90/110 (NP)	17,000	27,000	37,000	44,500	50,000
	(4lanes)	82/102 (NP)	12,600	20,000	27,400	32,900	37,000
Collector	(4lanes)	64/84 (NP)	11,600	18,500	25,300	30,400	34,200
	(4lanes)	(WP)	6,800	10,800	14,800	17,800	20,000
Local Collector	(2lanes)	42/66 (NP)	5,100	8,100	11,100	13,400	15,000
		(WP)	3,400	5,400	7,400	8,900	10,000
Rural collector	(2lanes)						

(NP) No Parking

(WP) With Parking

The following V/C Ratios were utilized for determining Existing and Future Level of Service.

Level of Service

Volume/Capacity (V/C) Ratio

A-	Less than or Equal to	0.00	to	0.34
B-	Less than or Equal to	0.35	to	0.54
C-	Less than or Equal to	0.55	to	0.74
D-	Less than or Equal to	0.75	to	0.89
E-	Less than or Equal to	0.90	to	1.00

v/c 0.74

Appendix D

Air Quality Modeling Data

Urbemis 2007 Version 9.2.4

Combined Summer Emissions Reports (Pounds/Day)

File Name: C:\a PROJECTS\oakvale Road MND AQ Escondido\URB.urb924

Project Name: Oakvale Road Realignment

Project Location: Orange County

On-Road Vehicle Emissions Based on: Version : Emfac2007 V2.3 Nov 1 2006

Off-Road Vehicle Emissions Based on: OFFROAD2007

Summary Report:

CONSTRUCTION EMISSION ESTIMATES

	<u>ROG</u>	<u>NOx</u>	<u>CO</u>	<u>SO2</u>	<u>PM10 Dust</u>	<u>PM10 Exhaust</u>	<u>PM10</u>	<u>PM2.5 Dust</u>	<u>PM2.5 Exhaust</u>	<u>PM2.5</u>	<u>CO2</u>
2015 TOTALS (lbs/day unmitigated)	4.72	33.06	24.92	0.01	6.34	2.00	7.42	1.32	1.84	2.32	4,498.42

Construction Unmitigated Detail Report:

CONSTRUCTION EMISSION ESTIMATES Summer Pounds Per Day, Unmitigated

	<u>ROG</u>	<u>NOx</u>	<u>CO</u>	<u>SO2</u>	<u>PM10 Dust</u>	<u>PM10 Exhaust</u>	<u>PM10</u>	<u>PM2.5 Dust</u>	<u>PM2.5 Exhaust</u>	<u>PM2.5</u>	<u>CO2</u>
Time Slice 4/1/2015-7/31/2015 Active Days: 88	2.75	21.19	14.44	0.01	6.34	1.08	7.42	1.32	1.00	2.32	3,423.66
Demolition 04/01/2015-07/31/2015	2.75	21.19	14.44	0.01	6.34	1.08	7.42	1.32	1.00	2.32	3,423.66
Fugitive Dust	0.00	0.00	0.00	0.00	6.30	0.00	6.30	1.31	0.00	1.31	0.00
Demo Off Road Diesel	2.43	17.76	11.95	0.00	0.00	0.95	0.95	0.00	0.88	0.88	2,323.03
Demo On Road Diesel	0.29	3.37	1.28	0.01	0.03	0.12	0.15	0.01	0.11	0.12	883.00
Demo Worker Trips	0.03	0.06	1.21	0.00	0.01	0.01	0.02	0.00	0.00	0.01	217.63

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Time Slice 8/3/2015-8/14/2015 Active Days: 10	<u>4.72</u>	<u>33.06</u>	<u>24.92</u>	0.00	1.02	<u>2.00</u>	3.02	0.22	<u>1.84</u>	2.05	<u>4,498.42</u>
Mass Grading 08/01/2015-08/14/2015	4.72	33.06	24.92	0.00	1.02	2.00	3.02	0.22	1.84	2.05	4,498.42
Mass Grading Dust	0.00	0.00	0.00	0.00	1.00	0.00	1.00	0.21	0.00	0.21	0.00
Mass Grading Off Road Diesel	4.65	32.93	22.51	0.00	0.00	1.99	1.99	0.00	1.83	1.83	4,063.16
Mass Grading On Road Diesel	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Mass Grading Worker Trips	0.07	0.13	2.41	0.00	0.02	0.01	0.03	0.01	0.01	0.02	435.26
Time Slice 8/17/2015-9/1/2015 Active Days: 12	2.27	17.54	11.08	0.00	1.01	0.81	1.82	0.21	0.75	0.96	2,371.68
Fine Grading 08/15/2015-09/01/2015	2.27	17.54	11.08	0.00	1.01	0.81	1.82	0.21	0.75	0.96	2,371.68
Fine Grading Dust	0.00	0.00	0.00	0.00	1.00	0.00	1.00	0.21	0.00	0.21	0.00
Fine Grading Off Road Diesel	2.26	17.50	10.40	0.00	0.00	0.81	0.81	0.00	0.74	0.74	2,247.32
Fine Grading On Road Diesel	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Fine Grading Worker Trips	0.02	0.04	0.69	0.00	0.01	0.00	0.01	0.00	0.00	0.00	124.36
Time Slice 9/2/2015-9/9/2015 Active Days: 6	1.49	9.46	8.08	0.00	0.01	0.73	0.74	0.01	0.67	0.68	1,323.64
Asphalt 09/02/2015-09/09/2015	1.49	9.46	8.08	0.00	0.01	0.73	0.74	0.01	0.67	0.68	1,323.64
Paving Off-Gas	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Paving Off Road Diesel	1.41	8.91	6.69	0.00	0.00	0.71	0.71	0.00	0.65	0.65	979.23
Paving On Road Diesel	0.04	0.48	0.18	0.00	0.00	0.02	0.02	0.00	0.02	0.02	126.79
Paving Worker Trips	0.03	0.06	1.21	0.00	0.01	0.01	0.02	0.00	0.00	0.01	217.63

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Time Slice 12/1/2015-12/15/2015	1.95	14.20	10.03	0.00	1.27	0.69	1.96	0.27	0.63	0.90	2,125.78
Active Days: 11											
Demolition 12/01/2015-12/15/2015	1.95	14.20	10.03	0.00	1.27	0.69	1.96	0.27	0.63	0.90	2,125.78
Fugitive Dust	0.00	0.00	0.00	0.00	1.26	0.00	1.26	0.26	0.00	0.26	0.00
Demo Off Road Diesel	1.88	13.69	8.66	0.00	0.00	0.66	0.66	0.00	0.61	0.61	1,790.42
Demo On Road Diesel	0.04	0.45	0.17	0.00	0.00	0.02	0.02	0.00	0.01	0.02	117.73
Demo Worker Trips	0.03	0.06	1.21	0.00	0.01	0.01	0.02	0.00	0.00	0.01	217.63

Phase Assumptions

Phase: Demolition 12/1/2015 - 12/15/2015 - Roadway Demolition

Building Volume Total (cubic feet): 22560

Building Volume Daily (cubic feet): 3000

On Road Truck Travel (VMT): 27.78

Off-Road Equipment:

- 1 Concrete/Industrial Saws (10 hp) operating at a 0.73 load factor for 8 hours per day
- 1 Crawler Tractors (147 hp) operating at a 0.64 load factor for 8 hours per day
- 1 Rubber Tired Dozers (357 hp) operating at a 0.59 load factor for 1 hours per day
- 2 Signal Boards (15 hp) operating at a 0.78 load factor for 8 hours per day
- 1 Tractors/Loaders/Backhoes (108 hp) operating at a 0.55 load factor for 6 hours per day
- 1 Water Trucks (189 hp) operating at a 0.5 load factor for 8 hours per day

Phase: Demolition 4/1/2015 - 7/31/2015 - Excavation Description

Building Volume Total (cubic feet): 1500000

Building Volume Daily (cubic feet): 15000

On Road Truck Travel (VMT): 208.33

Off-Road Equipment:

- 1 Concrete/Industrial Saws (10 hp) operating at a 0.73 load factor for 8 hours per day
- 1 Crawler Tractors (147 hp) operating at a 0.64 load factor for 8 hours per day
- 1 Rubber Tired Dozers (357 hp) operating at a 0.59 load factor for 1 hours per day

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- 1 Rubber Tired Loaders (164 hp) operating at a 0.54 load factor for 8 hours per day
- 2 Tractors/Loaders/Backhoes (108 hp) operating at a 0.55 load factor for 6 hours per day
- 1 Water Trucks (189 hp) operating at a 0.5 load factor for 8 hours per day

Phase: Fine Grading 8/15/2015 - 9/1/2015 - Roadway construction - Fine Grading

Total Acres Disturbed: 0.5

Maximum Daily Acreage Disturbed: 0.05

Fugitive Dust Level of Detail: Default

20 lbs per acre-day

On Road Truck Travel (VMT): 0

Off-Road Equipment:

- 1 Graders (174 hp) operating at a 0.61 load factor for 6 hours per day
- 1 Rubber Tired Dozers (357 hp) operating at a 0.59 load factor for 6 hours per day
- 1 Tractors/Loaders/Backhoes (108 hp) operating at a 0.55 load factor for 7 hours per day
- 1 Water Trucks (189 hp) operating at a 0.5 load factor for 8 hours per day

Phase: Mass Grading 8/1/2015 - 8/14/2015 - Roadway Construction - rough grading

Total Acres Disturbed: 0.5

Maximum Daily Acreage Disturbed: 0.05

Fugitive Dust Level of Detail: Default

20 lbs per acre-day

On Road Truck Travel (VMT): 0

Off-Road Equipment:

- 1 Forklifts (145 hp) operating at a 0.3 load factor for 8 hours per day
- 1 Graders (174 hp) operating at a 0.61 load factor for 6 hours per day
- 1 Paving Equipment (104 hp) operating at a 0.53 load factor for 8 hours per day
- 2 Plate Compactors (8 hp) operating at a 0.43 load factor for 8 hours per day
- 2 Rollers (95 hp) operating at a 0.56 load factor for 8 hours per day
- 1 Rubber Tired Dozers (357 hp) operating at a 0.59 load factor for 6 hours per day
- 2 Signal Boards (15 hp) operating at a 0.78 load factor for 8 hours per day

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3 Tractors/Loaders/Backhoes (108 hp) operating at a 0.55 load factor for 7 hours per day

1 Water Trucks (189 hp) operating at a 0.5 load factor for 8 hours per day

Phase: Paving 9/2/2015 - 9/9/2015 - Roadway Construction - Paving

Acres to be Paved: 0.5

Off-Road Equipment:

4 Cement and Mortar Mixers (10 hp) operating at a 0.56 load factor for 6 hours per day

1 Pavers (100 hp) operating at a 0.62 load factor for 7 hours per day

1 Rollers (95 hp) operating at a 0.56 load factor for 7 hours per day

1 Tractors/Loaders/Backhoes (108 hp) operating at a 0.55 load factor for 7 hours per day

