Appendix A. 10% Conceptual Engineering Plans

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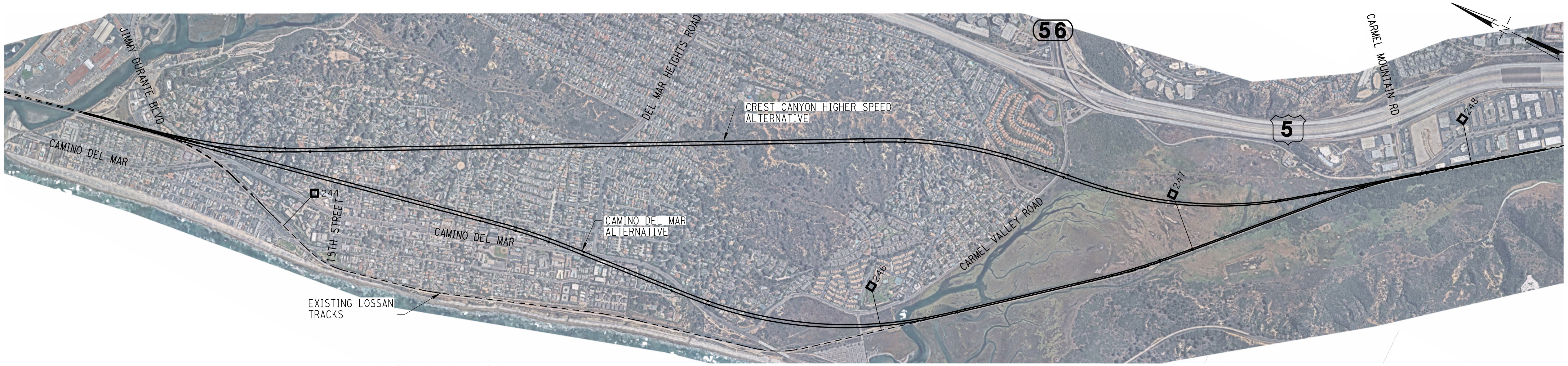
SAN DIEGO ASSOCIATION OF GOVERNMENTS PROJECT PLANS FOR THE SAN DIEGUITO TO SORRENTO VALLEY DOUBLE TRACK PROJECT

DEL MAR TUNNELS ALTERNATIVE ANALYSIS

SAN DIEGO REGIONAL RAIL CORRIDOR ALTERNATIVE ALIGNMENT AND IMPROVEMENTS CONCEPTUAL ENGINEERING STUDY

GENERAL NOTES:

1. THE AERIAL PHOTOGRAPHY AND CONTOURS SHOWN AS BACKGROUND ARE BASED ON DIGITAL MAPPING PROVIDED BY NEARMAP VENDOR DATED SEPTEMBER 11, 2019. THE BASIS OF ELEVATIONS IS THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)
2. PROPERTY LINE INFORMATION SHOWN IS BASED ON SANGIS MAPPING AND IS SCHEMATIC.
3. THE RAILROAD RIGHT-OF-WAY SHOWN IS BASED ON RECORD OF SURVEY MAPPING DEVELOPED AS PART OF OTHER PROJECTS.
4. THE PROJECT ASSUMES TIE IN TO THE PROPOSED SAN DIEGUITO RIVER BRIDGE REPLACEMENT DOUBLE TRACK AND SPECIAL EVENTS PLATFORM PROJECT CURRENTLY IN 90% DESIGN.



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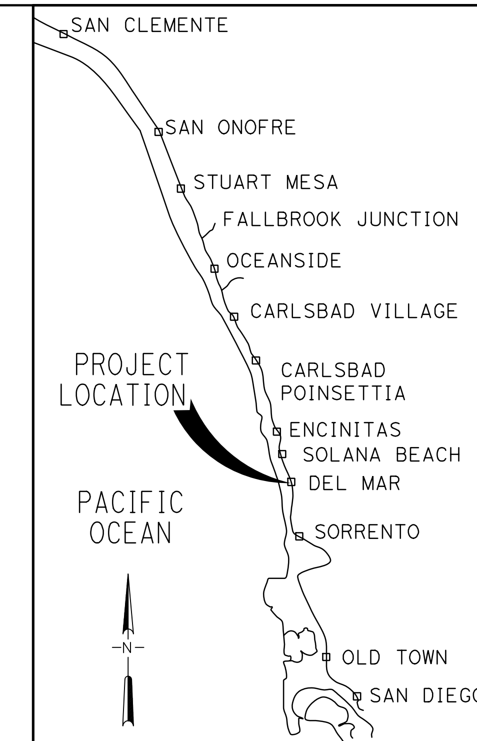

VICINITY MAP

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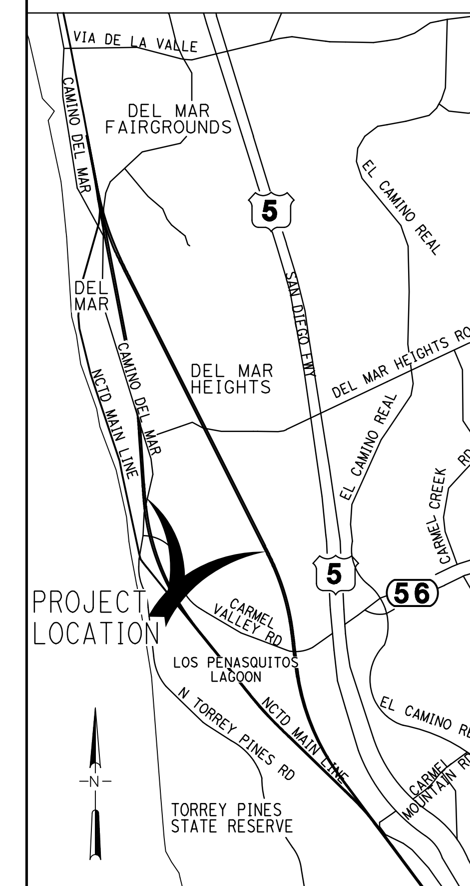

LOCATION MAP

NO SCALE

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DESIGNED BY K. MAGEE
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

TITLE SHEET, VICINITY MAP AND LOCATION MAP

CONTRACT NO.
DRAWING NO. G001
REVISION
SHEET NO. 1 of 35
SCALE AS NOTED

11/14/2022
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SHEET INDEX

DEL MAR TUNNEL

SHT NO	DWG NO	TITLE
		GENERAL
1	G001	TITLE SHEET, VICINITY MAP AND LOCATION MAP
2	G002	INDEX OF DRAWINGS
3	G003	LEGEND AND ABBREVIATIONS
4	SS001	BNSF, CPUC AND VEHICLE CLEARANCE ENVELOPE
5	SS002	TWIN BORE TUNNELS TYPICAL SECTION
6	SS003	CUT AND COVER TYPICAL SECTION - SHEET 1 OF 2
7	SS004	CUT AND COVER TYPICAL SECTION - SHEET 2 OF 2
8	SS005	U-STRUCTURE TYPICAL SECTION
9	SS006	CROSS PASSAGE TYPICAL SECTIONS
10	SU001	TWIN BORE TUNNELS EMERGENCY VENTILATION SCHEMATIC

CREST CANYON

SHT NO	DWG NO	TITLE
		RAIL
11	TS101	CREST CANYON HS TYPICAL SECTION - SHEET 1 OF 2
12	TS102	CREST CANYON HS TYPICAL SECTION - SHEET 2 OF 2
13	TR101	CREST CANYON HS TRACK PLAN AND PROFILE MT-1 STA 9+00 TO STA 70+00
14	TR102	CREST CANYON HS TRACK PLAN AND PROFILE MT-1 STA 70+00 TO STA 175+00
15	TR103	CREST CANYON HS TRACK PLAN AND PROFILE MT-1 STA 175+00 TO STA 257+42.39
		ROADWAY
16	GS101	CREST CANYON HS JIMMY DURANTE REALIGNMENT
		TUNNELS
17	SA101	CREST CANYON HS - TWIN BORE NORTH PORTAL CONSTRUCTION STAGING AREA
18	SA102	CREST CANYON HS - TWIN BORE SOUTH PORTAL CONSTRUCTION STAGING AREA
19	SA103	CREST CANYON HS - TWIN BORE NORTH PORTAL STRUCTURES AND PERMANENT FACILITIES
20	SA104	CREST CANYON HS - TWIN BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES
		BRIDGES
21	BR101	CREST CANYON HS - BRIDGE NO. 1 PLAN AND ELEVATION - MT-1 STA 173+06.80 TO STA 186+70.90
22	BR102	CREST CANYON HS - BRIDGE NO. 2 PLAN AND ELEVATION - MT-1 STA 231+20.49 TO STA 232+46.66

CAMINO DEL MAR

SHT NO	DWG NO	TITLE
		RAIL
23	TS201	CAMINO DEL MAR TYPICAL SECTION - SHEET 1 OF 2
24	TS202	CAMINO DEL MAR TYPICAL SECTION - SHEET 2 OF 2
25	TR201	CAMINO DEL MAR TRACK PLAN AND PROFILE MT-1 STA 9+00 TO STA 90+00
26	TR202	CAMINO DEL MAR TRACK PLAN AND PROFILE MT-1 STA 90+00 TO STA 185+00
27	TR203	CAMINO DEL MAR TRACK PLAN AND PROFILE MT-1 STA 185+00 TO STA 263+87.68
		ROADWAY
28	GS201	CAMINO DEL MAR JIMMY DURANTE REALIGNMENT
		TUNNELS
29	SA201	CAMINO DEL MAR - TWIN BORE NORTH PORTAL CONSTRUCTION STAGING AREA
30	SA202	CAMINO DEL MAR - TWIN BORE SOUTH PORTAL CONSTRUCTION STAGING AREA
31	SA203	CAMINO DEL MAR - TWIN BORE NORTH PORTAL STRUCTURES AND PERMANENT FACILITIES
32	SA204	CAMINO DEL MAR - TWIN BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES
		BRIDGES
33	BR201	CAMINO DEL MAR BRIDGE NO. 1 PLAN AND ELEVATION - MT-1 STA 131+48.41 TO STA 179+00.17
34	BR202	CAMINO DEL MAR BRIDGE NO. 2 PLAN AND ELEVATION - MT-1 STA 186+00 TO STA 198+00.17
35	BR203	CAMINO DEL MAR BRIDGE NO. 3 PLAN AND ELEVATION - MT-1 STA 237+64.90 TO STA 238+91.07

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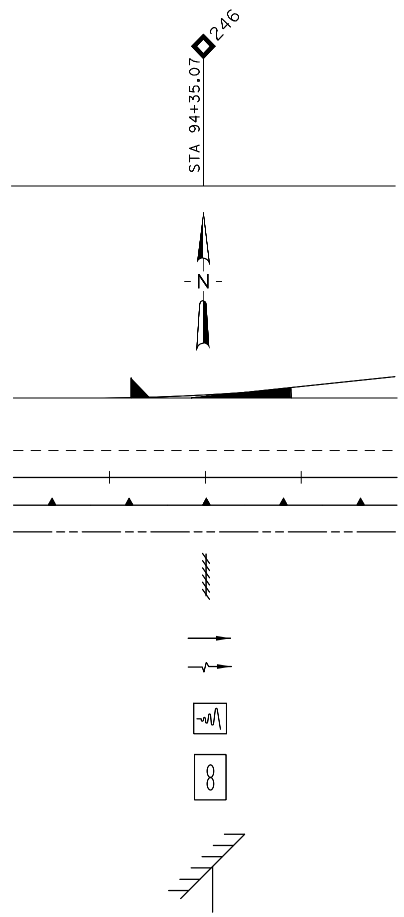
LEGEND



MILE POST
NORTH ARROW
TURNOUT - POWER OPERATED
EXISTING TRACK
MAIN TRACK
RETAINING WALL
RIGHT OF WAY
MOTORIZED DAMPER
SUPPLY AIRFLOW
EXHAUST OR RETURN AIRFLOW
ATTENUATOR
TUNNEL VENTILATION FAN
OUTSIDE AIR LOUVER

SYMBOLS

ø	DIAMETER
	DRAINAGE DIRECTION
	LIGHT POLE
	MANHOLE
	POTHOLE LOCATION
	SURVEY CONTROL POINT

ABBREVIATIONS

HORIZONTAL

℄	CENTERLINE
D_c	DEGREE OF CURVE
Δ	DEFLECTION ANGLE - CIRCULAR CURVE
I	DEFLECTION ANGLE - TOTAL CURVE
θ_s	DEFLECTION ANGLE - SPIRAL
E	EQUILIBRIUM SUPERELEVATION
E_a	ACTUAL SUPERELEVATION
E_u	UNBALANCED SUPERELEVATION
L_c	LENGTH OF CIRCULAR CURVE
L_s	LENGTH OF SPIRAL
L_T	LENGTH OF TOTAL CURVE
R	RADIUS
T	TANGENT
POB	POINT OF BEGINNING
PC	POINT OF CURVATURE
CS	POINT OF CIRCULAR CURVE TO SPIRAL
POE	POINT OF ENDING
PI	POINT OF INTERSECTION
PITO	POINT OF INTERSECTION OF TURNOUT
SC	POINT OF SPIRAL TO CIRCULAR CURVE
ST	POINT OF SPIRAL TO TANGENT
PS	POINT OF SWITCH
PT	POINT OF TANGENCY
TS	POINT OF TANGENT TO SPIRAL
POC	POINT ON CURVE
POS	POINT ON SPIRAL
POT	POINT ON TANGENT
PVC	POINT OF VERTICAL CURVE
PVI	POINT OF VERTICAL INTERSECTION
PVT	POINT OF VERTICAL TANGENT

AGENCIES

AASHTO	AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AMTRAK	NATIONAL RAILROAD PASSENGER CORPORATION
AREMA	AMERICAN RAILWAY ENGINEERING AND MAINTENANCE-OF-WAY ASSOCIATION
A.S.T.M.	AMERICAN SOCIETY FOR TESTING MATERIALS
AT&T	AMERICAN TELEPHONE AND TELEGRAPH COMPANY
BNSF	BURLINGTON NORTHERN SANTA FE RAILWAY
CALTRANS	CALIFORNIA DEPARTMENT OF TRANSPORTATION
CCC	CALIFORNIA COASTAL COMMISSION
CSD	CITY OF SAN DIEGO
EPA	ENVIRONMENTAL PROTECTION AGENCY
FRA	FEDERAL RAILROAD ADMINISTRATION
ICG	ICG TELECOM GROUP INC.
LRFD	LOAD AND RESISTANCE FACTOR DESIGN
MBC	MUNICIPAL BOND COMPANY
MCI	MICROWAVE COMMUNICATIONS, INC.
MTS	METROPOLITAN TRANSIT SYSTEM
NCTD	NORTH COUNTY TRANSIT DISTRICT
RWQCB	REGIONAL WATER QUALITY CONTROL BOARD
SANDAG	SAN DIEGO ASSOCIATION OF GOVERNMENTS
SANDWD	SAN DIEGO WATER DISTRICT
SBSD	SOLANA BEACH SANITATION DISTRICT
SCGC	SOUTHERN CALIFORNIA GAS COMPANY
SCTC	SOUTHERN CALIFORNIA TELEPHONE COMPANY
SDGE	SAN DIEGO GAS AND ELECTRIC COMPANY
SDRSD	SAN DIEGO REGIONAL STANDARDS DRAWINGS
SDID	SAN DIEGO IRRIGATION DISTRICT
SWCA	SOUTHWESTERN CABLE
TCG	TELEPORT COMMUNICATION GROUP
USACE	UNITED STATES ARMY CORPS OF ENGINEERS
USFWS	UNITED STATES FISH AND WILDLIFE SERVICE
UPRR	UNION PACIFIC RAILROAD

GENERAL

ABT	ABOUT
ABUT	ABUTMENT
AC	ASPHALT CEMENT CONCRETE
ACB	ARTICULATED CONCRETE BLOCK
AHD	AHEAD
APN	ASSESOR'S PARCEL NUMBER
APPROX.	APPROXIMATELY
AVE	AVENUE
AWW	ABSOLUTE WORK WINDOW
B/	BOTTOM OF
BK	BACK
BLVD	BOULEVARD
BM	BENCHMARK
BMP	BEST MANAGEMENT PRACTICES
BOR	BASE OF RAIL
BOT	BOTTOM
BR	BRIDGE
BTWN	BETWEEN
C/C	CENTER-TO-CENTER
CFM	CUBIC FEET PER MINUTE
CFS	CUBIC FEET PER SECOND
CIDH	CAST-IN-DRILLED-HOLE
CIP	CAST-IN-PLACE
CISS	CAST-IN-STEEL-SHELL
CJP	COMPLETE JOINT PENETRATION
CLR	CLEARANCE
CMP	CORRUGATED METAL PIPE
CMU	CONCRETE MASONRY UNIT
CO.	COMPANY
CONC	CONCRETE
CONSTR	CONSTRUCTION
CONT	CONTINUOUS
CONT'D	CONTINUED
CP	CONTROL POINT
CPT	CONE PENETRATION TEST
CT	COURT
CWR	CONTINUOUS WELDED RAIL
DIA	DIAMETER
DWG.	DRAWING
DR	DRIVE
DWY	DRIVEWAY
E	EXPANSION BEARING
EA	EACH
EB	END BRIDGE
EF	EACH FACE
E.G.	FOR EXAMPLE
EGL	ENERGY GRADE LINE
EIC	EMPLOYEE IN CHARGE
ELEC	ELECTRICAL
ELEV	ELEVATION
ERP	END RAILING POST
ESA	ENVIRONMENTALLY SENSITIVE AREA
EST	ESTIMATED
EQ	EQUAL
EX/EXIST	EXISTING
F	FIXED BEARING
FES	FLARED END SECTION
FID	FAN ISOLATION DAMPER
FG	FINISHED GRADE
FL	FLOWLINE
FPM	FEET PER MINUTE
FPS	FEET PER SECOND
FT	FEET
GA	GAUGE
GALV	GALVANIZED
GRND	GROUND
HDPE	HIGH DENSITY POLYETHYLENE
HGL	HYDRAULIC GRADE LINE

GENERAL CONT'D

HOR	HORIZONTAL
H.P.	HIGH POINT
ID	INSIDE DIAMETER/INNER DIAMETER
IN. W.G.	INCHES OF WATER GAGE
INC.	INCORPORATED
JT	JOINT
L	LENGTH
LB	POUND
LF	LINEAR FOOT
LHTO	LEFT HANDED TURN OUT
LOL	LAYOUT LINE
LT	LEFT
MAX	MAXIMUM
MD	MOTORIZED DAMPER
MH	MANHOLE
MHHW	MEAN HIGHER HIGH WATER
MIN	MINIMUM
MK	MARK
ML	MAINLINE
MP	MILE POST
MT-1	MAIN TRACK 1
MT-2	MAIN TRACK 2
MT-2-I	MAIN TRACK 2 INTERIM
mph	MILES PER HOUR
NB	NORTHBOUND
NIC	NOT IN CONTRACT
No.	NUMBER
NOI	NOTICE OF INTENT
NTS	NOT TO SCALE
O.C.	ON CENTER
OAL	OUTSIDE AIR LOUVER
OD	OUTSIDE DIAMETER
OTM	OTHER TRACK MATERIAL
PCC	PORTLAND CEMENT CONCRETE
PEHP	POLYETHYLENE HIGH PRESSURE
POTO	POWER OPERATED TURNOUT
PROP	PROPOSED
PSI	POUNDS PER SQUARE INCH
PVC	POLYVINYL CHLORIDE
Q100	100 YEAR FLOW DISCHARGE
RD	ROAD
RHTO	RIGHT HANDED TURN OUT
RW	RETAINING WALL
R/W	RIGHT-OF-WAY
RCB	REINFORCED CONCRETE BOX
RCP	REINFORCED CONCRETE PIPE
REINF	REINFORCEMENT
REQ'D	REQUIRED
REV	REVISION
RIM	TOP OF MANHOLE RIM
ROE	RIGHT OF ENTRY
RP	RAILING POST
RT	RIGHT
S	SLOPE
SA	SOUND ATTENUATOR
SB	STEEL BRACKET, SOUTHBOUND
SD	STORM DRAIN
SDDT	SAN DIEGUITO DOUBLE TRACK
SDRS	SAN DIEGO COUNTY REGIONAL STANDARD
SE	SUPERELEVATION
SGH	SUBGRADE HINGE
SHT	SHEET
SIG	SIGNAL
SPA	SPACE/SPACING
SS	SANITARY SEWER
SSPWC	STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION (GREEN BOOK)
ST	STREET

GENERAL CONT'D

STA	STATION
STD	STANDARD
STL	STEEL
STR	STRAIGHT
SWMDCMA	STORM WATER MANAGEMENT AND DISCHARGE CONTROL MAINTENANCE AGREEMENT
SWPPP	STORM WATER POLLUTION PREVENTION PLAN
T	THICKNESS
T/	TOP OF
T/R, T/RAIL	TOP OF RAIL
TC	TOP OF CURB
TCE	TEMPORARY CONSTRUCTION EASEMENT
TEMP	TEMPORARY
TO	TURNOUT
TRK	TRACK
TVF	TUNNEL VENTILATION FAN
TW	TOP OF WALL
TYP	TYPICAL
U.N.O.	UNLESS NOTED OTHERWISE
V	VELOCITY/SPEED
VAR	VARIABLE
VC	VITRIFIED CLAY
VCH	VELOCITY IN CHANNEL
VER	VERTICAL
VMAX	MAXIMUM VELOCITY
W	WIDTH
W/	WITH
WPCP	WATER POLLUTION CONTROL PLAN
WSE	WATER SURFACE ELEVATION
WT	WEIGHT
WW	WINGWALL
XING	CROSSING

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CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency
APPROVED: ____________ DATE: ________

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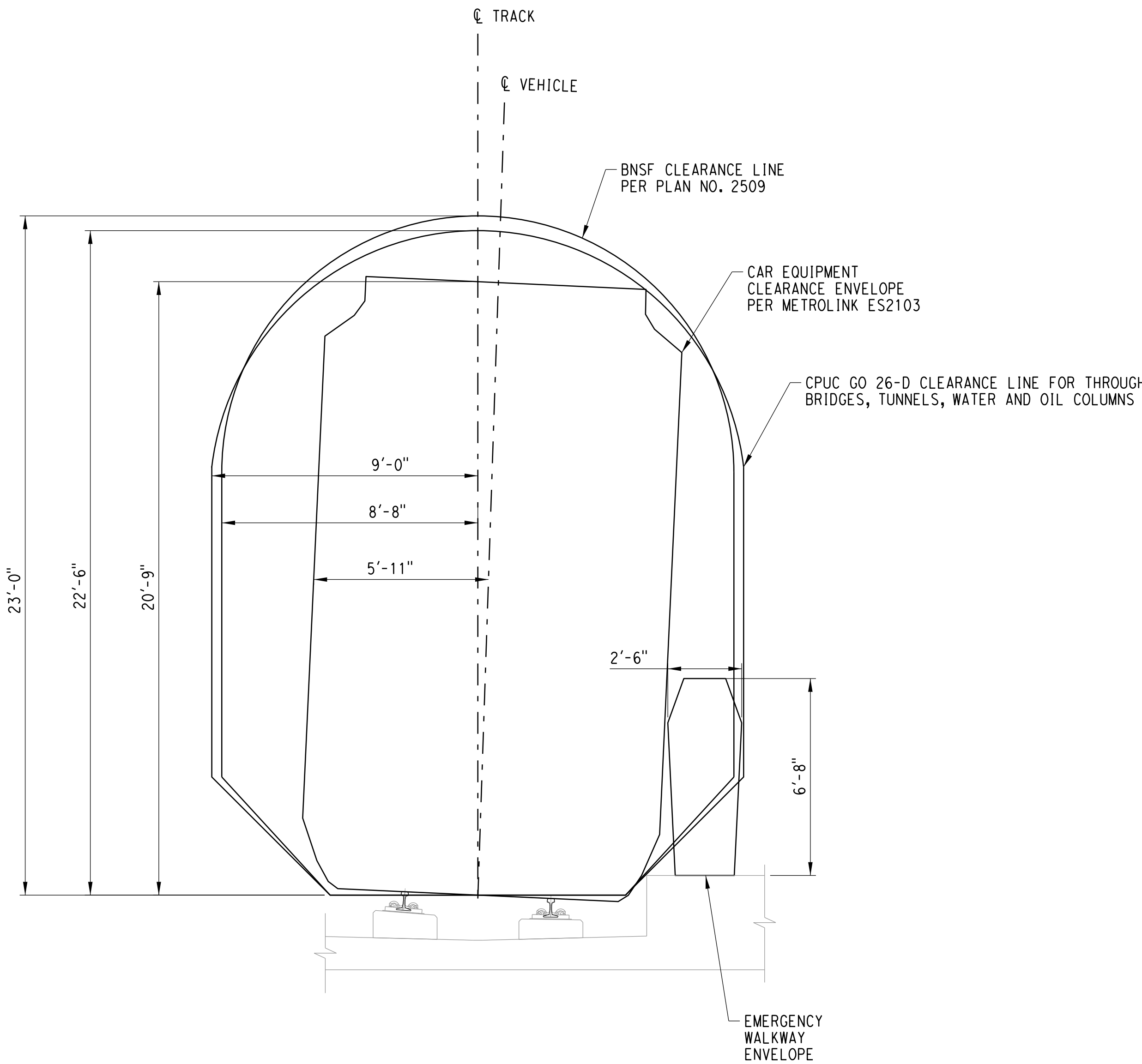
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
LEGEND AND ABBREVIATIONS

CONTRACT NO.
DRAWING NO. G003
REVISION
SHEET NO. 3 of 35
SCALE NTS

10/21/2022



BNSF, CPUC AND VEHICLE CLEARANCE ENVELOPES (WITH 2.75" SUPERELEVATION)

SCALE: 1" = 6'-0"

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DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

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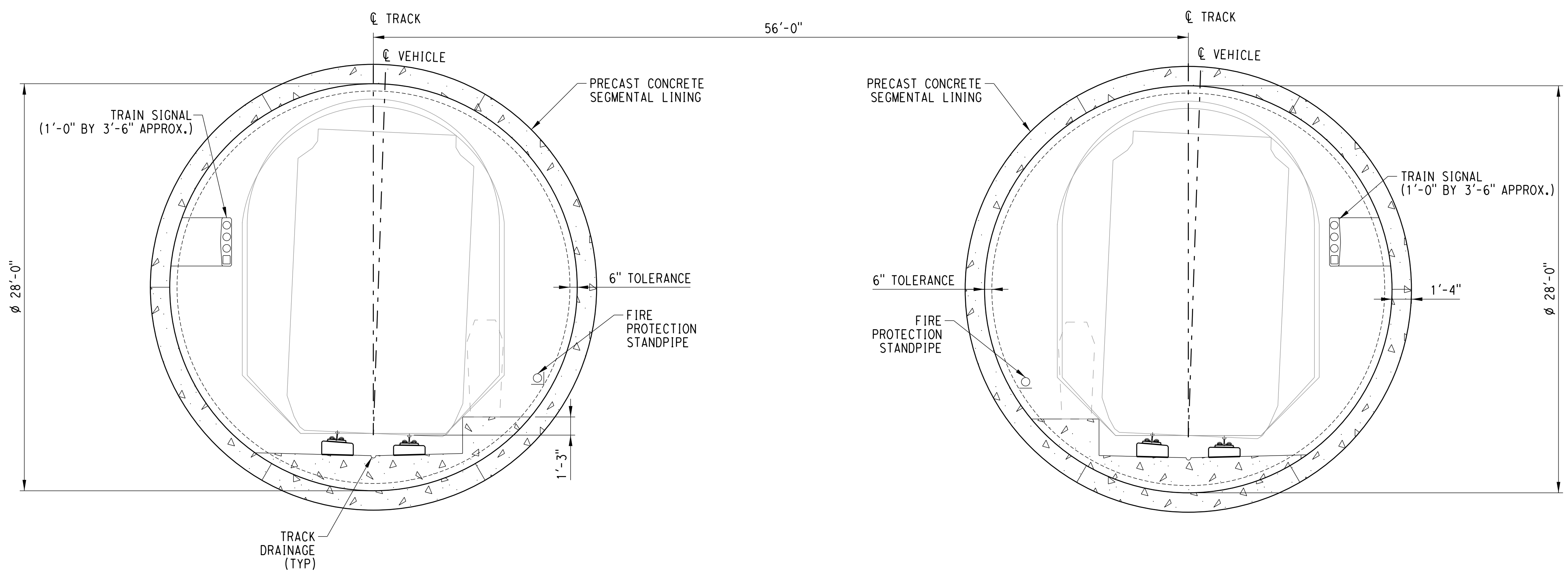
DEL MAR ALTERNATIVES ANALYSIS
BNSF, CPUC AND VEHICLE CLEARANCE ENVELOPES

CONTRACT NO.
DRAWING NO. SS001
REVISION
SHEET NO. 4 of 35
SCALE AS NOTED



TYPICAL TWIN BORE TUNNEL

SCALE 1"=10'-0"

NOTE

1. TUNNEL SPACING AT PORTALS TO BE FURTHER REFINED.
2. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001

5 0 5 10
SCALE IN FEET

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Tel: 619-881-0400

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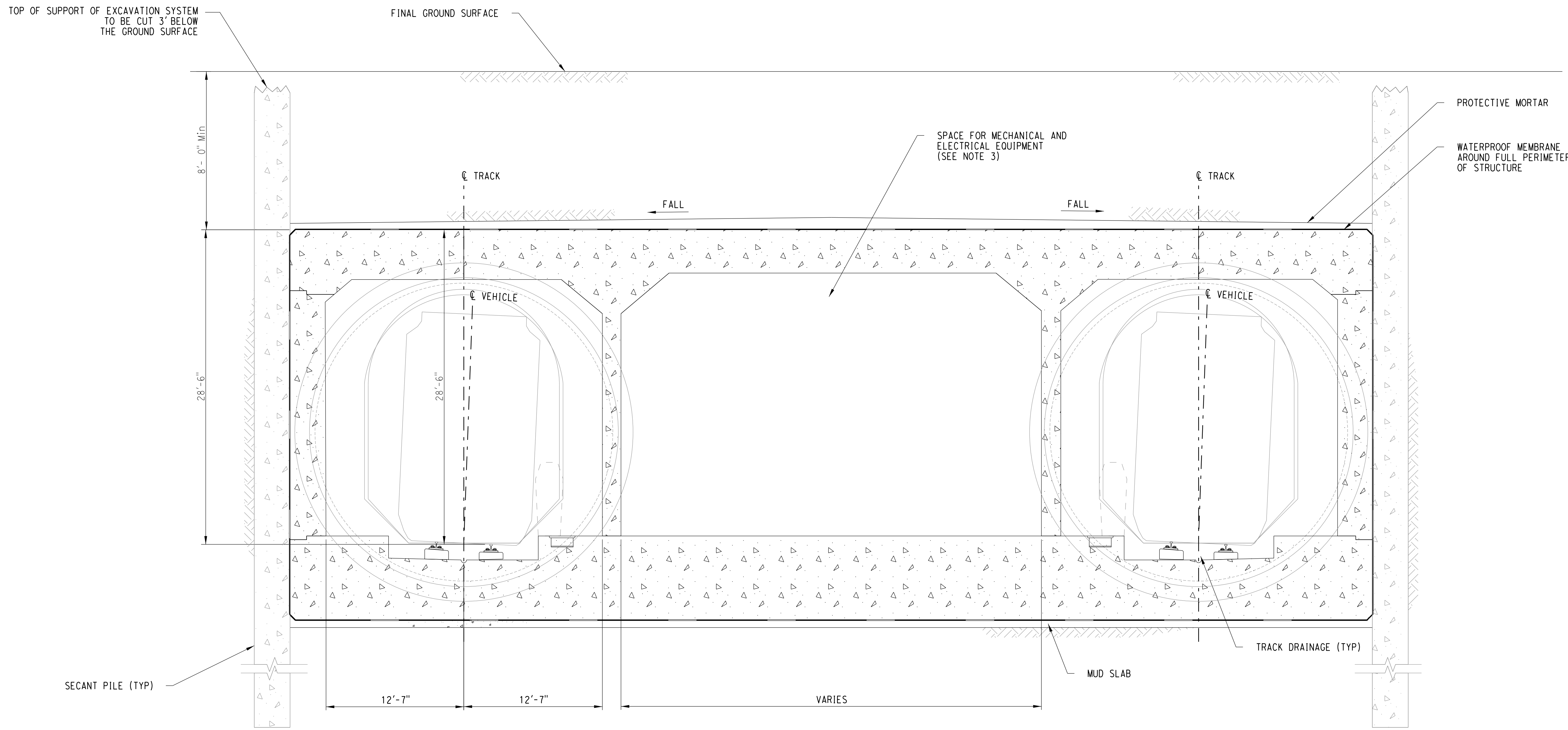
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

TWIN BORE TUNNELS
TYPICAL SECTION

CONTRACT NO.
DRAWING NO. SS002
REVISION
SHEET NO. 5 of 35
SCALE AS NOTED



CUT & COVER SECTION AT END OF BORED TUNNEL

SCALE: 1" = 10'-0"

NOTES:

1. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001.
2. DISTANCE BETWEEN TRACK CENTERLINE VARIES.

5 0 5 10
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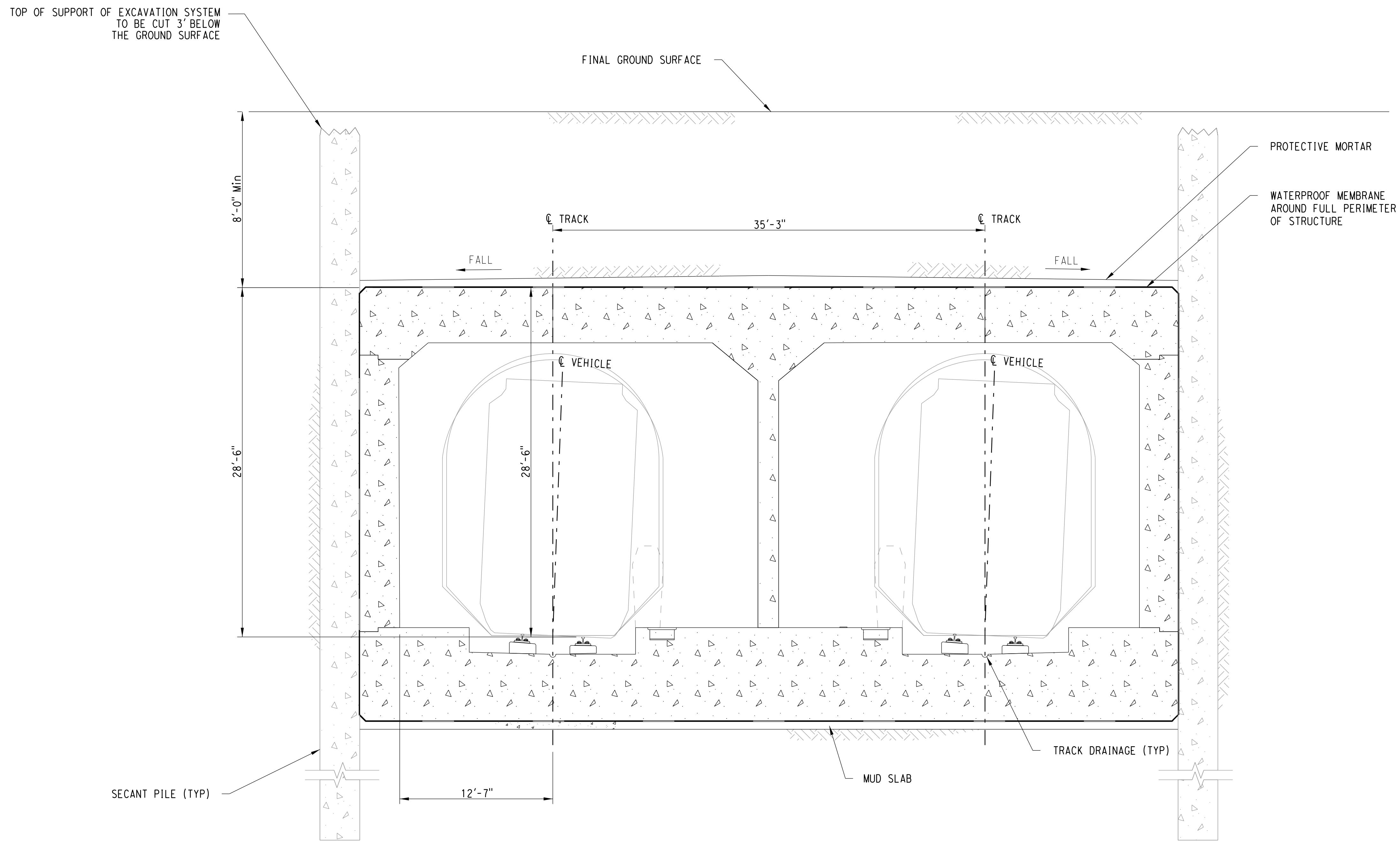
DEL MAR ALTERNATIVES ANALYSIS
CUT AND COVER TYPICAL SECTION SHEET 1 OF 2

CONTRACT NO.
DRAWING NO. SS003
REVISION
SHEET NO. 6 of 35
SCALE AS NOTED



CUT & COVER SECTION AT U-WALL INTERFACE

SCALE: 1" = 10'-0"

NOTES:

1. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001.
2. DISTANCE BETWEEN TRACK CENTERLINE VARIES.

5 0 5 10

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CUT AND COVER TYPICAL SECTION SHEET 2 OF 2

CONTRACT NO.
DRAWING NO. SS004
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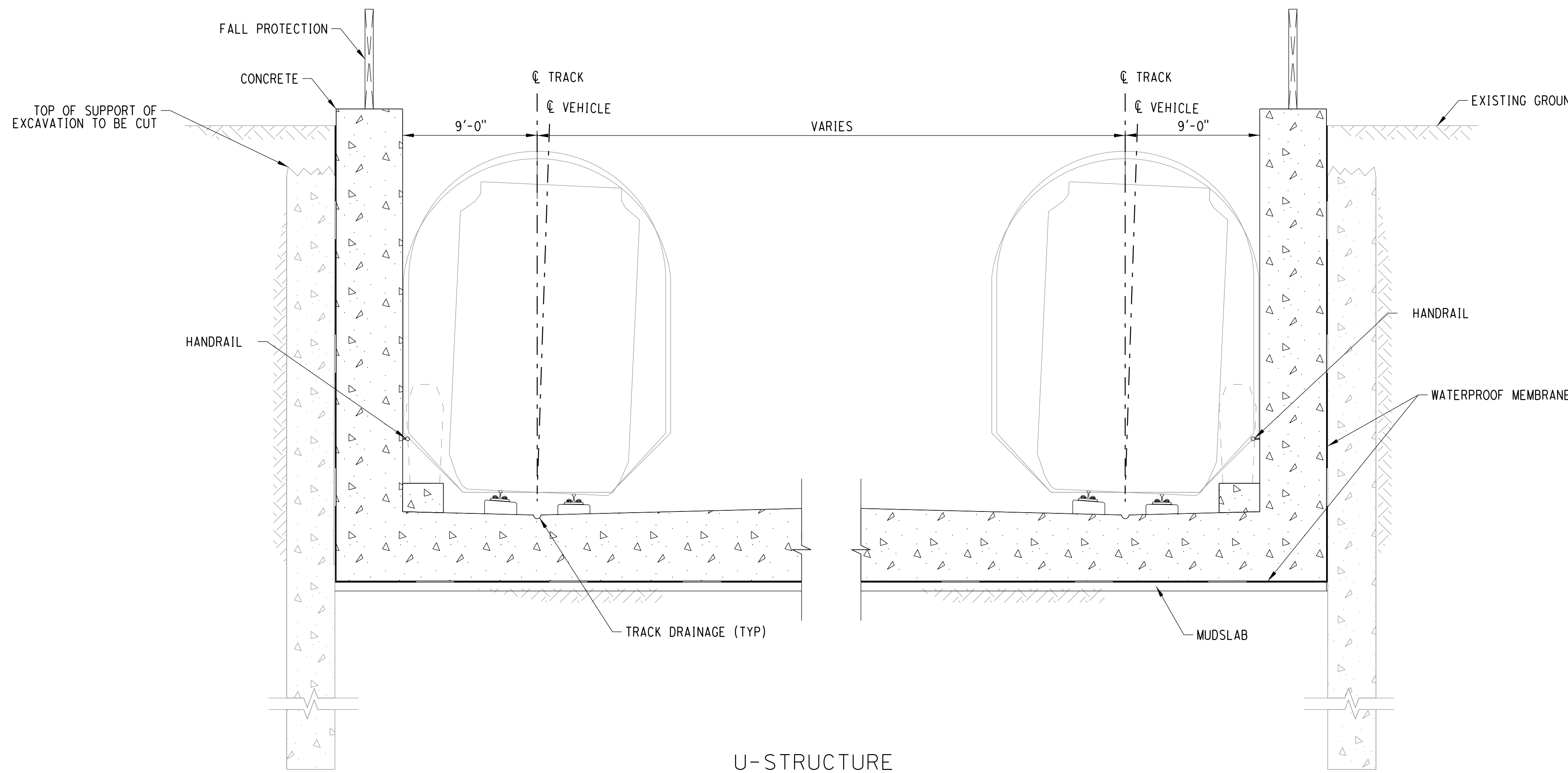
SHEET NO. 7 of 35
SCALE AS NOTED



U-STRUCTURE

SCALE: 1"=10'-0"

NOTE:

1. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001.

5 0 5 10

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DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street,
Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

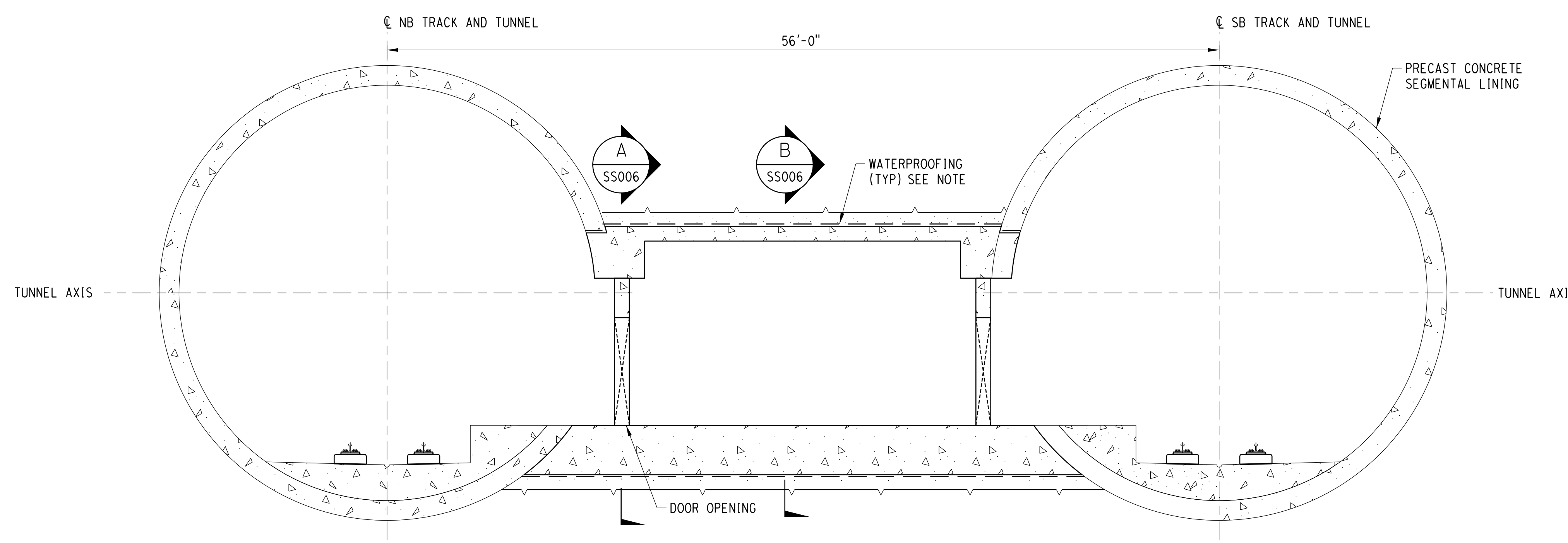
APPROVED: ____________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

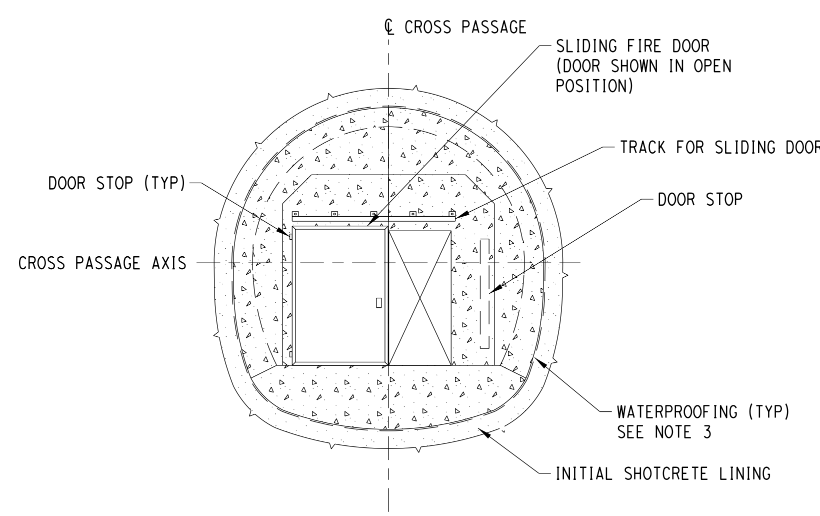
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

U-STRUCTURE TYPICAL SECTION

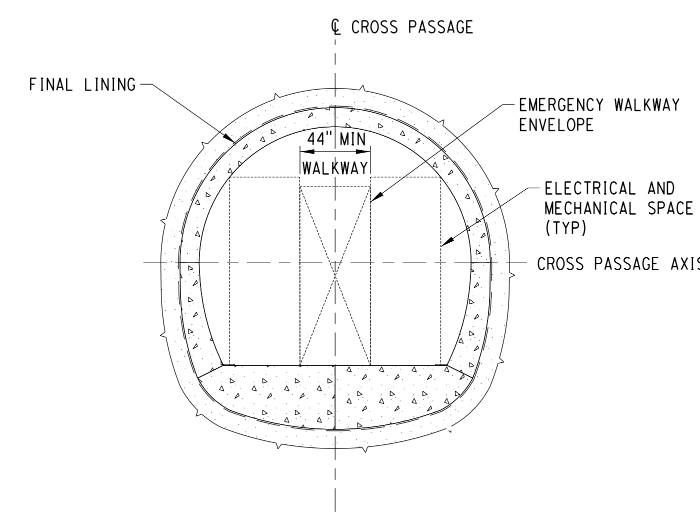
CONTRACT NO.
DRAWING NO. SS005
REVISION
SHEET NO. 8 of 35
SCALE AS NOTED



LONGITUDINAL SECTION
SCALE: 1"=10'-0"



A SS006 SECTION
SCALE: 1"=10'-0"



B SS006 SECTION
SCALE: 1"=10'-0"

NOTES:

1. TUNNEL SUPPORT SHOWN IS PRELIMINARY CONCEPT ONLY, ACTUAL REINFORCEMENT AND LINING DESIGN TBD.
2. CROSS PASSAGE SECTION TO BE HORSESHOE SHAPE FOR LINING SYSTEM SHOWN.
3. WATERPROOF MEMBRANE TYPE TO BE CONFIRMED. MEMBRANE TO BE EITHER HYDRO-CARBON RESISTENT (HCR) OR POLYVINYL CHLORIDE (PVC).
4. IN ORDER TO FACILITATE CROSS PASSAGE CONSTRUCTION GROUND IMPROVEMENT MAY BE REQUIRED.

5 0 5 10
SCALE IN FEET

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DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency
APPROVED: ______ DATE: ______

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CROSS PASSAGE TYPICAL SECTIONS

CONTRACT NO.
DRAWING NO. SS006
REVISION
SHEET NO. 9 of 35
SCALE AS NOTED

LEGEND

MOTORIZED DAMPER

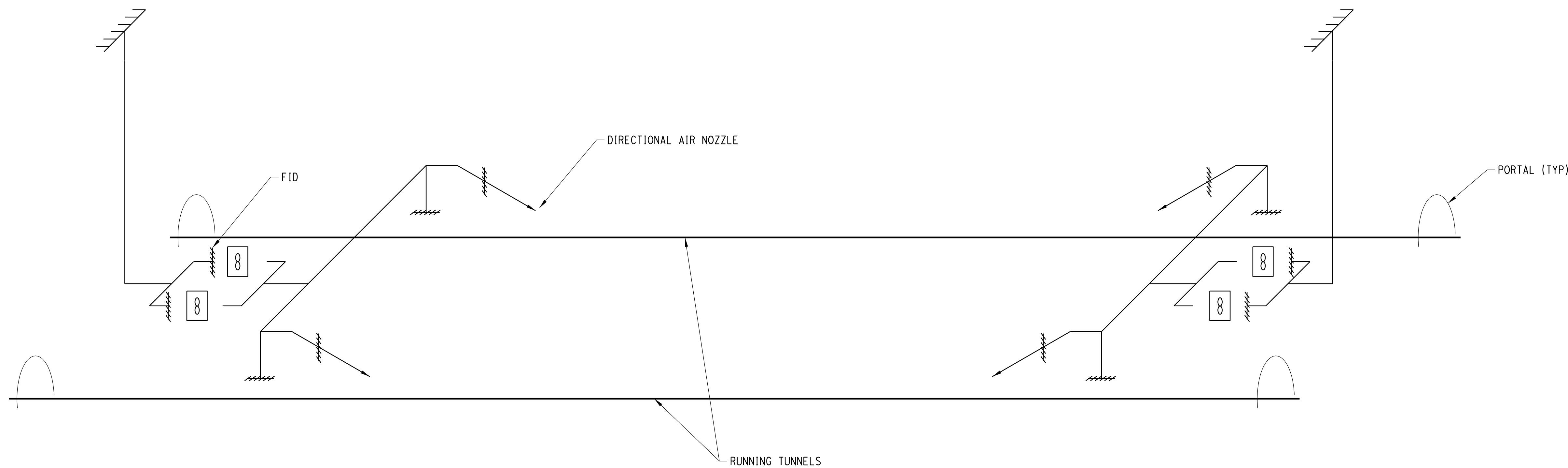
SOUND ATTENUATOR

TUNNEL VENTILATION FAN

OUTSIDE AIR LOUVER

ABBREVIATIONS

OAL	OUTSIDE AIR LOUVER
SA	SOUND ATTENUATOR
FID	FAN ISOLATION DAMPER
TVF	TUNNEL VENTILATION FAN



TWIN BORE AIRFLOW DIAGRAM
NTS

NOTE:
1. CONCEPT CAN BE ADAPTED TO SINGLE BORE ARRANGEMENTS.

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CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

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San Diego, CA 92101
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SANDAG
San Diego's Regional Planning Agency

APPROVED: ______ DATE: ______

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

TWIN BORE TUNNELS EMERGENCY VENTILATION SCHEMATIC

CONTRACT NO.
DRAWING NO. SU001
REVISION
SHEET NO. 10 of 35
SCALE AS NOTED

10/28/2022
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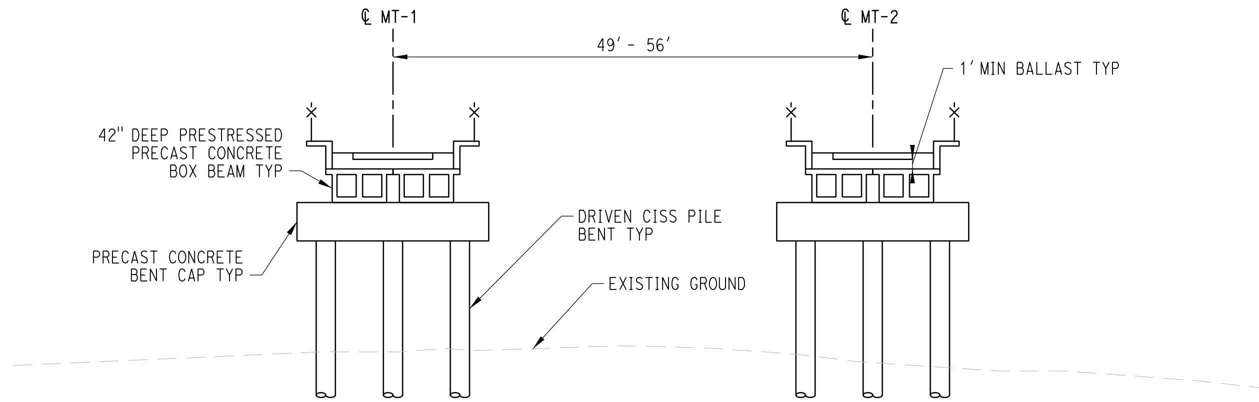
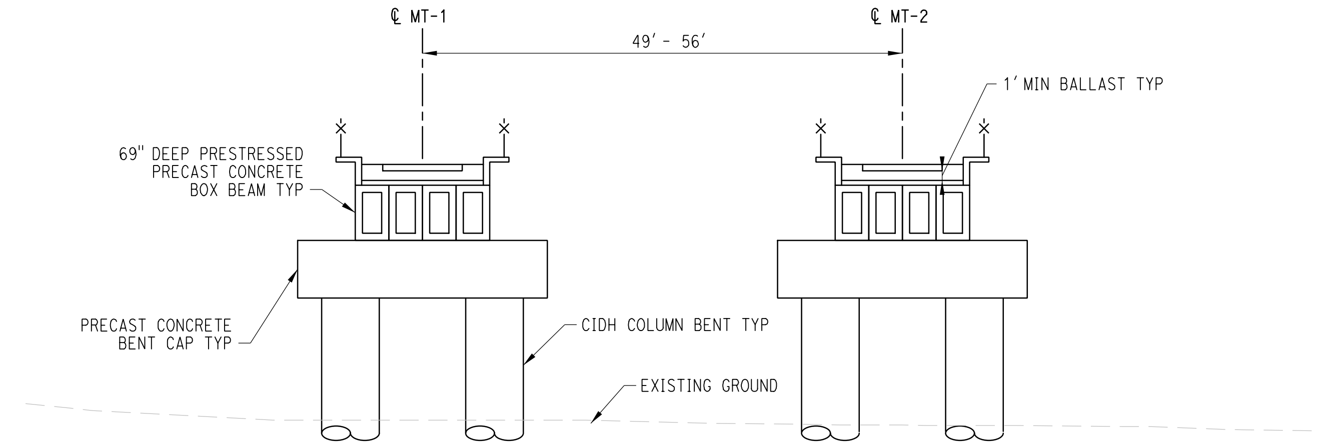
NOTES:

1. FOR TYPICAL SECTIONS AT CUT & COVER AND U-STRUCTURE LIMITS BETWEEN MT-1 STA 19+50 TO STA 30+25 AND MT-1 STA 166+75 TO STA 173+07, SEE STRUCTURAL PLANS.
2. FOR TYPICAL SECTIONS AT TUNNEL LIMITS BETWEEN MT-1 STA 30+25 TO STA 166+75, SEE STRUCTURAL PLANS.
3. REFER TO BRIDGE PLANS FOR ADDITIONAL DETAILS.
4. CONSTRUCTION OF SHOOFLY TRACK(S) AND OTHER TEMPORARY STRUCTURES ARE REQUIRED FOR THE CONSTRUCTION OF THIS PROJECT. TEMPORARY MEASURES TO BE DEVELOPED IN A SUBSEQUENT PHASE.



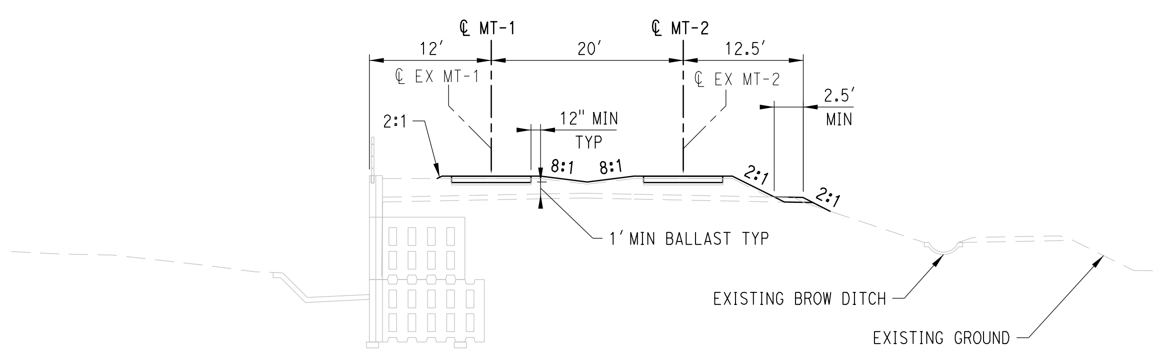
MT-1 STA 173+07 TO STA 173+97
MT-1 STA 174+67 TO STA 186+71
MT-1 STA 231+20 TO STA 232+47

NO SCALE



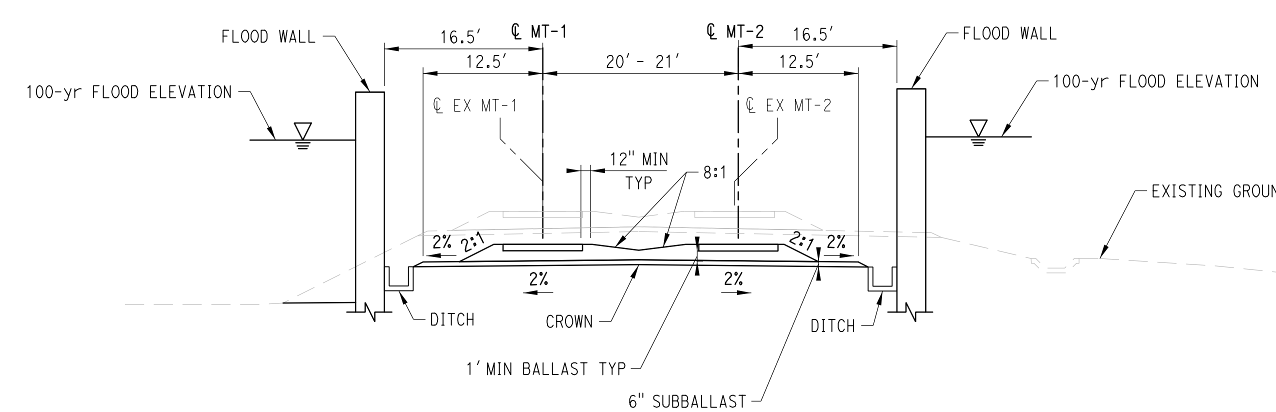
MT-1 STA 173+97 TO STA 174+67

NO SCALE



MT-1 STA 9+00 TO STA 12+00

NO SCALE



MT-1 STA 12+00 TO STA 19+50

NO SCALE

10/21/2022

REV.	DATE	DESCRIPTION	BY / SUB.	APP.

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DESIGNED BY J.K. ANDRES
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG San Diego's Regional Planning Agency

APPROVED: ______ DATE: ______

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS
TYPICAL SECTION
SHEET 1 OF 2

CONTRACT NO.
DRAWING NO. TS101
REVISION
SHEET NO. 11 of 35
SCALE NO SCALE

NOTES:

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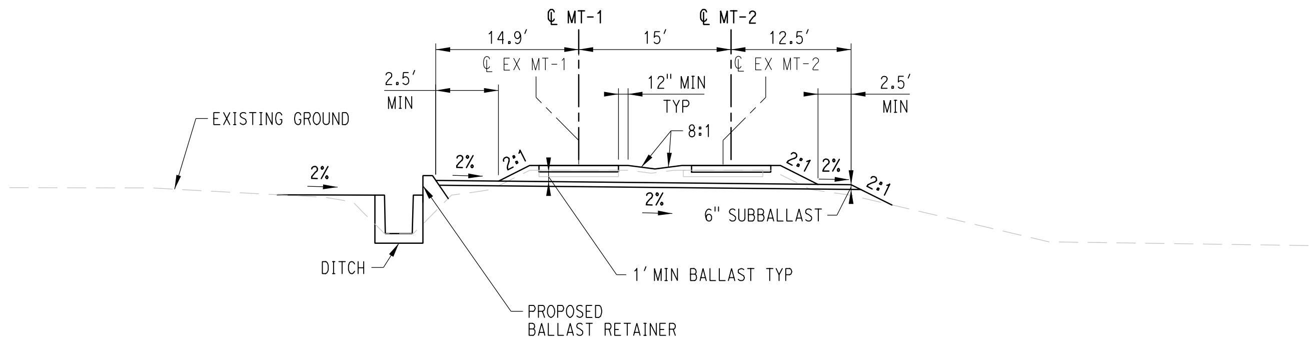
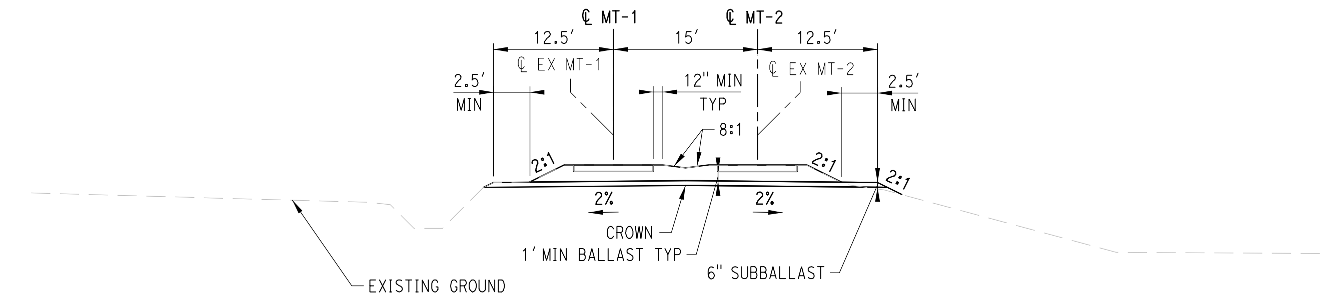


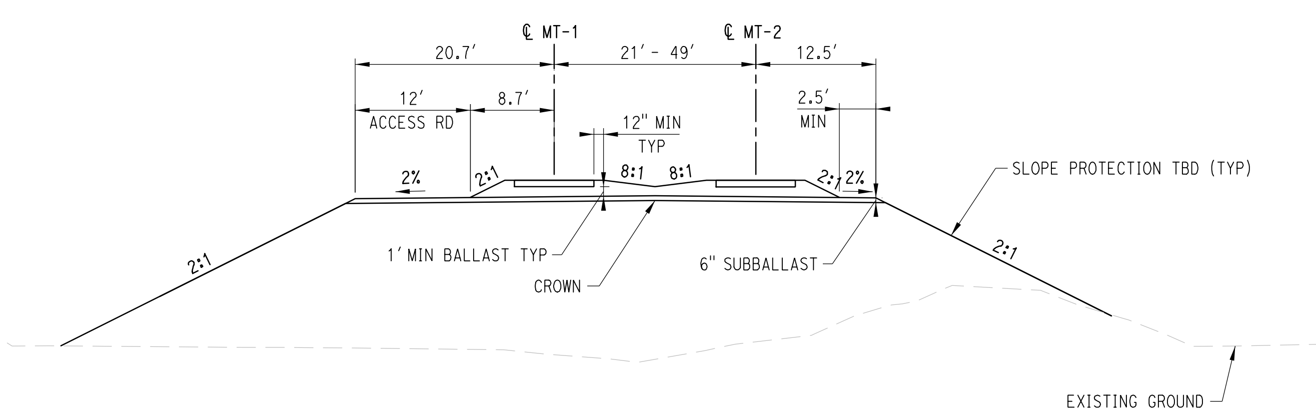
MT-1 STA 248+35 TO STA 252+35

NO SCALE



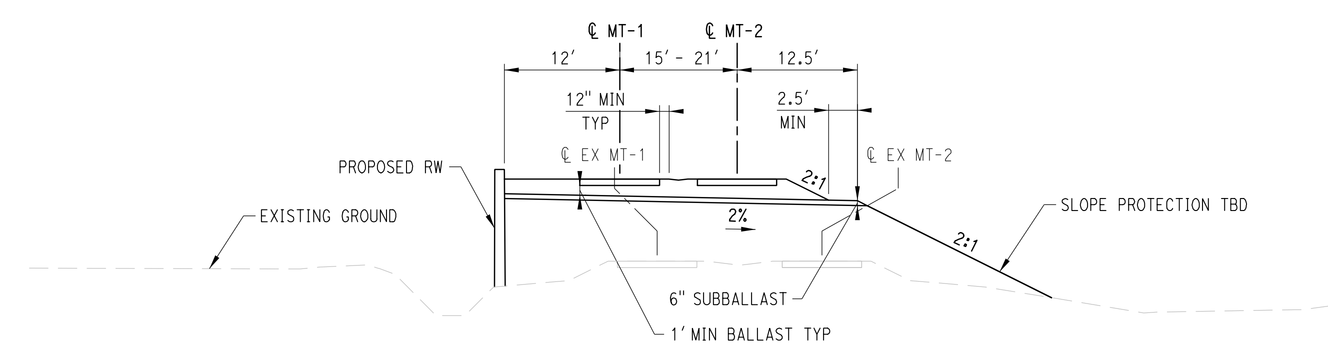
MT-1 STA 252+35 TO STA 257+42

NO SCALE



MT-1 STA 186+68 TO STA 231+20

NO SCALE



MT-1 STA 232+47 TO STA 248+35

NO SCALE

REV.	DATE	DESCRIPTION	BY / SUB.	APP.

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DESIGNED BY J.K. ANDRES
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ________ DATE: ________

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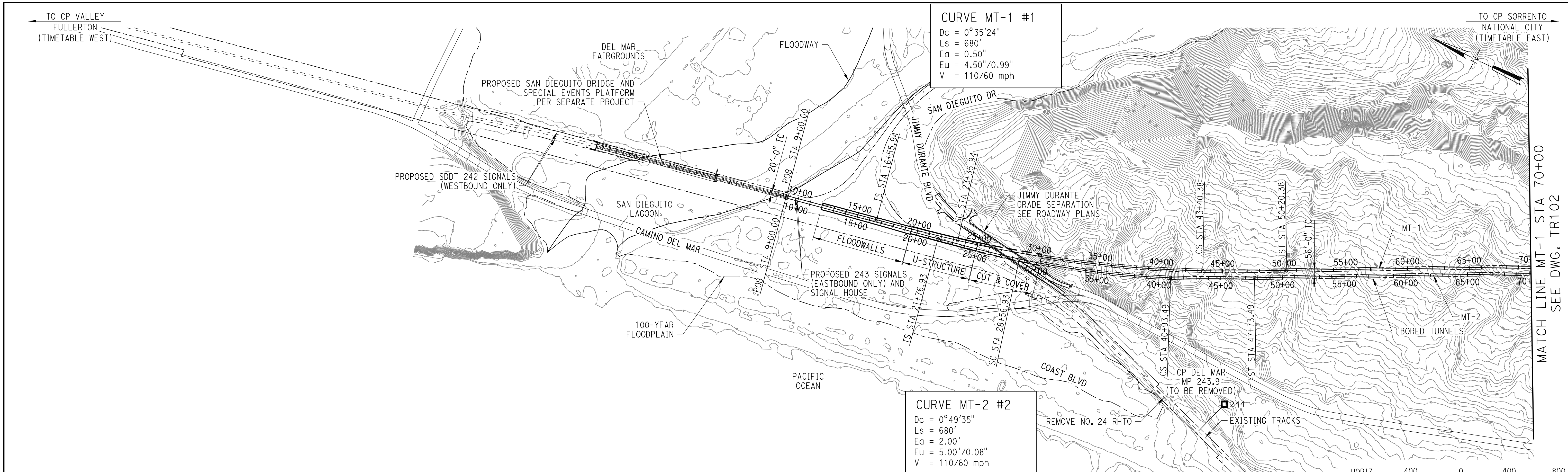
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS TYPICAL SECTION SHEET 2 OF 2

CONTRACT NO.
DRAWING NO. TS102
REVISION
SHEET NO. 12 of 35
SCALE NO SCALE

10/21/2022



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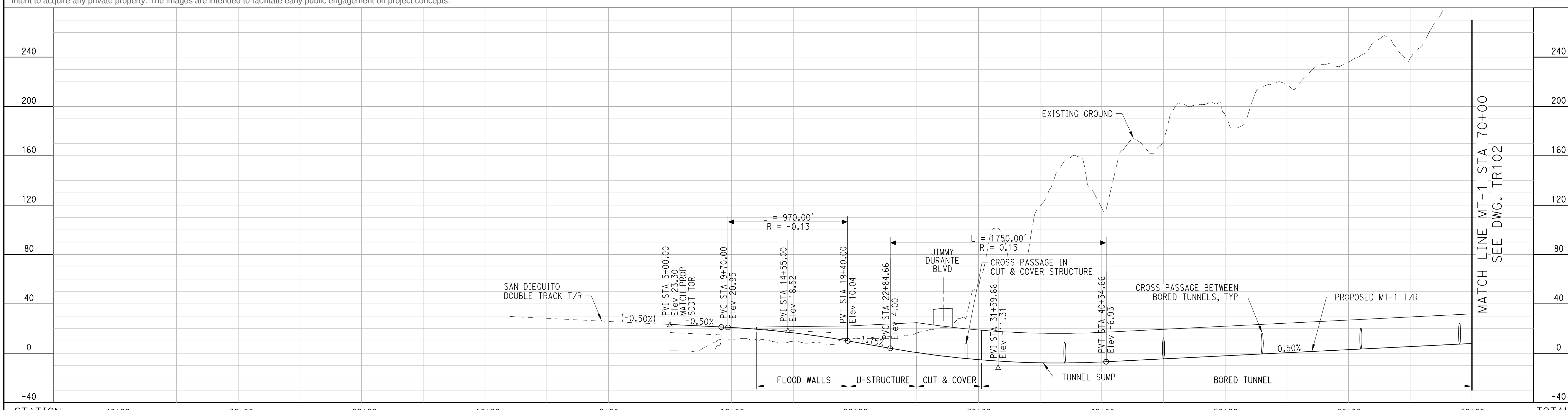
PLAN



REV.	DATE	DESCRIPTION	BY/SUB	APP.

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DESIGNED BY K. MAGEE
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ________ DATE: ________

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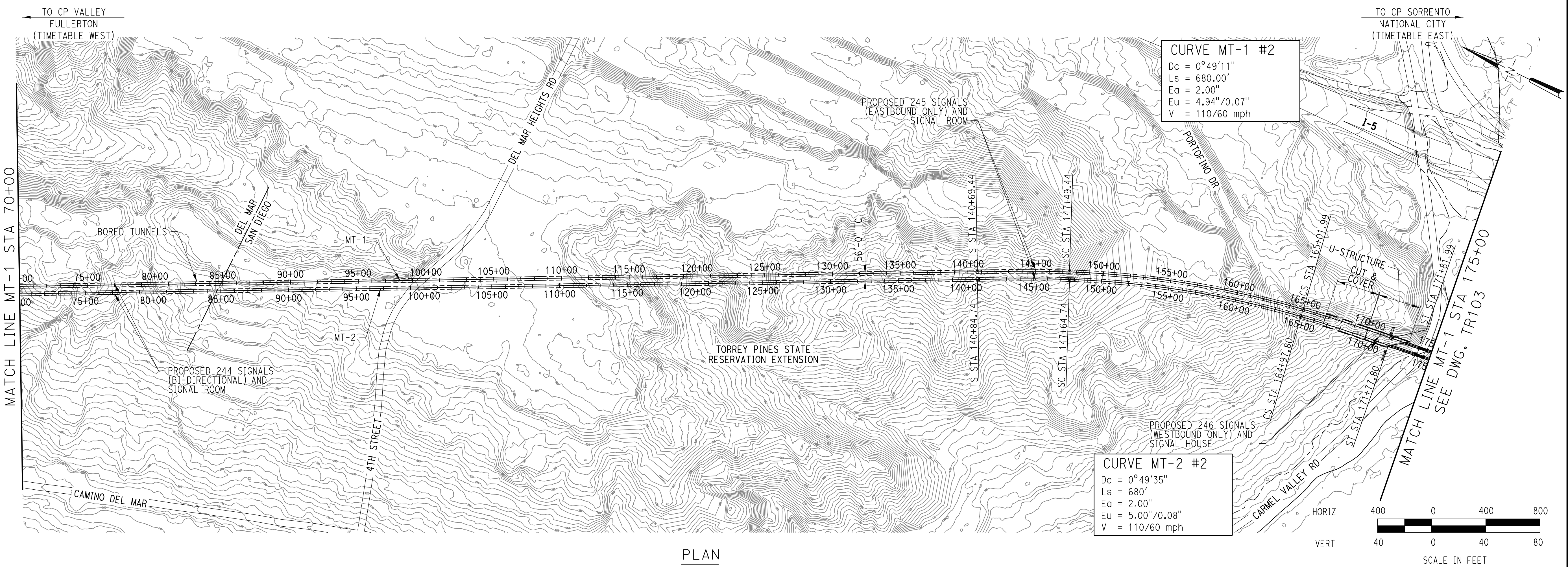
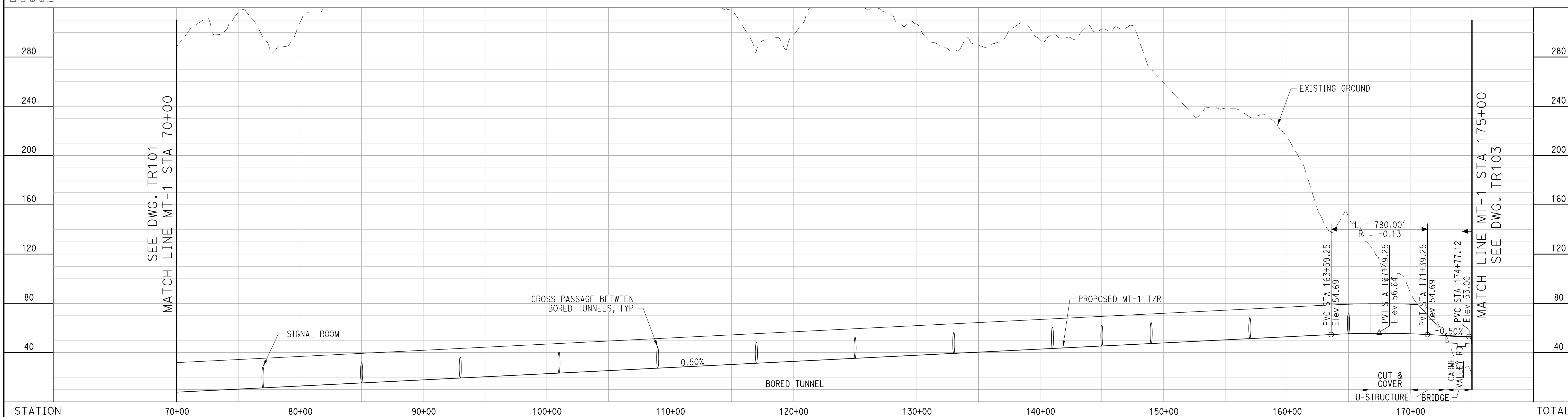
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS
TRACK PLAN AND PROFILE
MT-1 STA 9+00 TO STA 70+00

CONTRACT NO.
DRAWING NO. TR101
REVISION
SHEET NO. 13 of 35
SCALE VERT: 1"=80' HORIZ: 1"=800'

10/21/2022



PLAN



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DESIGNED BY K. MAGEE
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

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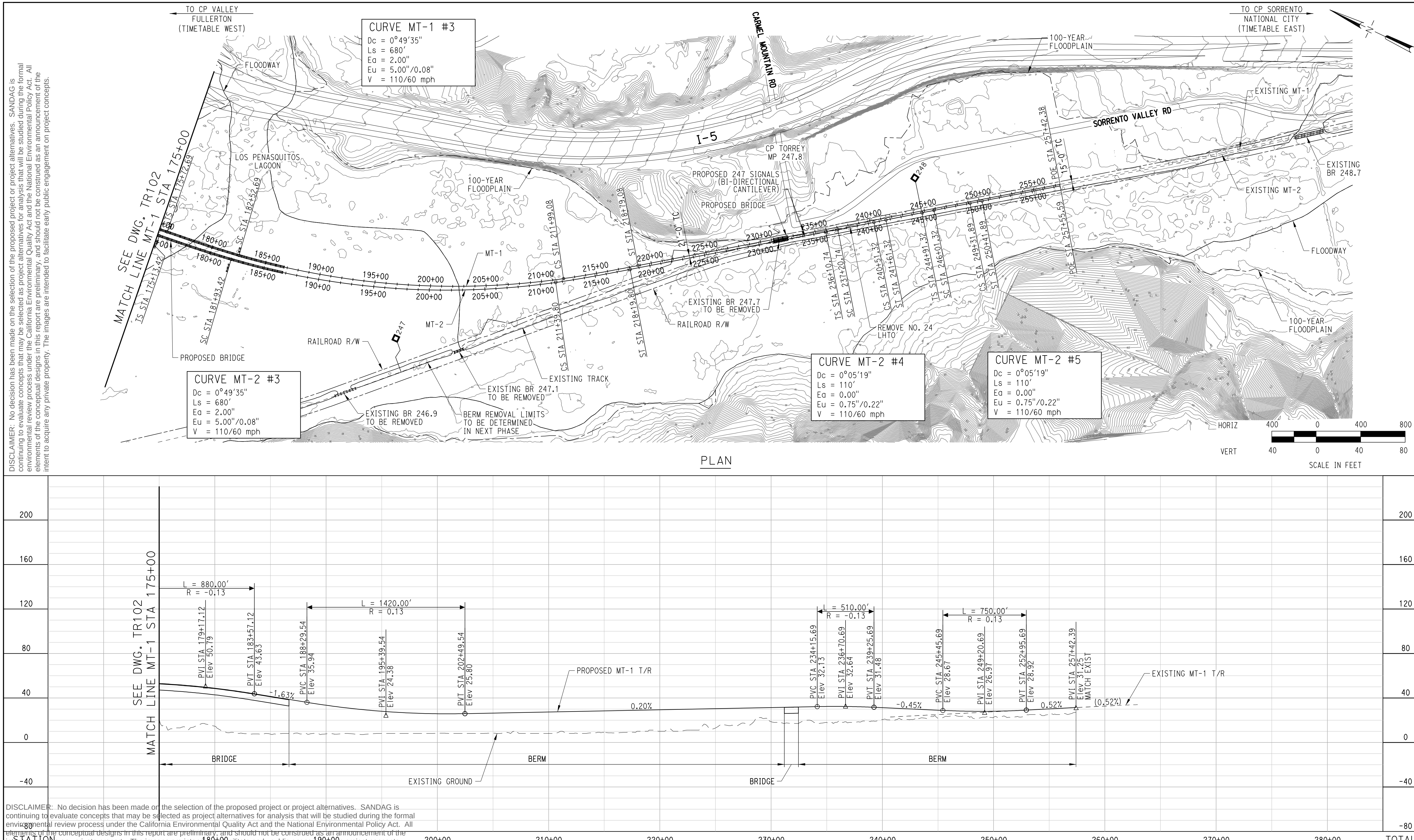
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS
TRACK PLAN AND PROFILE
MT-1 STA 70+00 TO STA 175+00

CONTRACT NO.
DRAWING NO. TR102
REVISION
SHEET NO. 14 of 35
SCALE VERT: 1"=80' HORIZ: 1"=800'

10/21/2022



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DESIGNED BY K. MAGEE
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS TRACK PLAN AND PROFILE MT-1 STA 175+00 TO STA 257+42.39

CONTRACT NO.
DRAWING NO. TR103
REVISION
SHEET NO. 15 of 35
SCALE VERT: 1"=80' HORIZ: 1"=800'

10/21/2022
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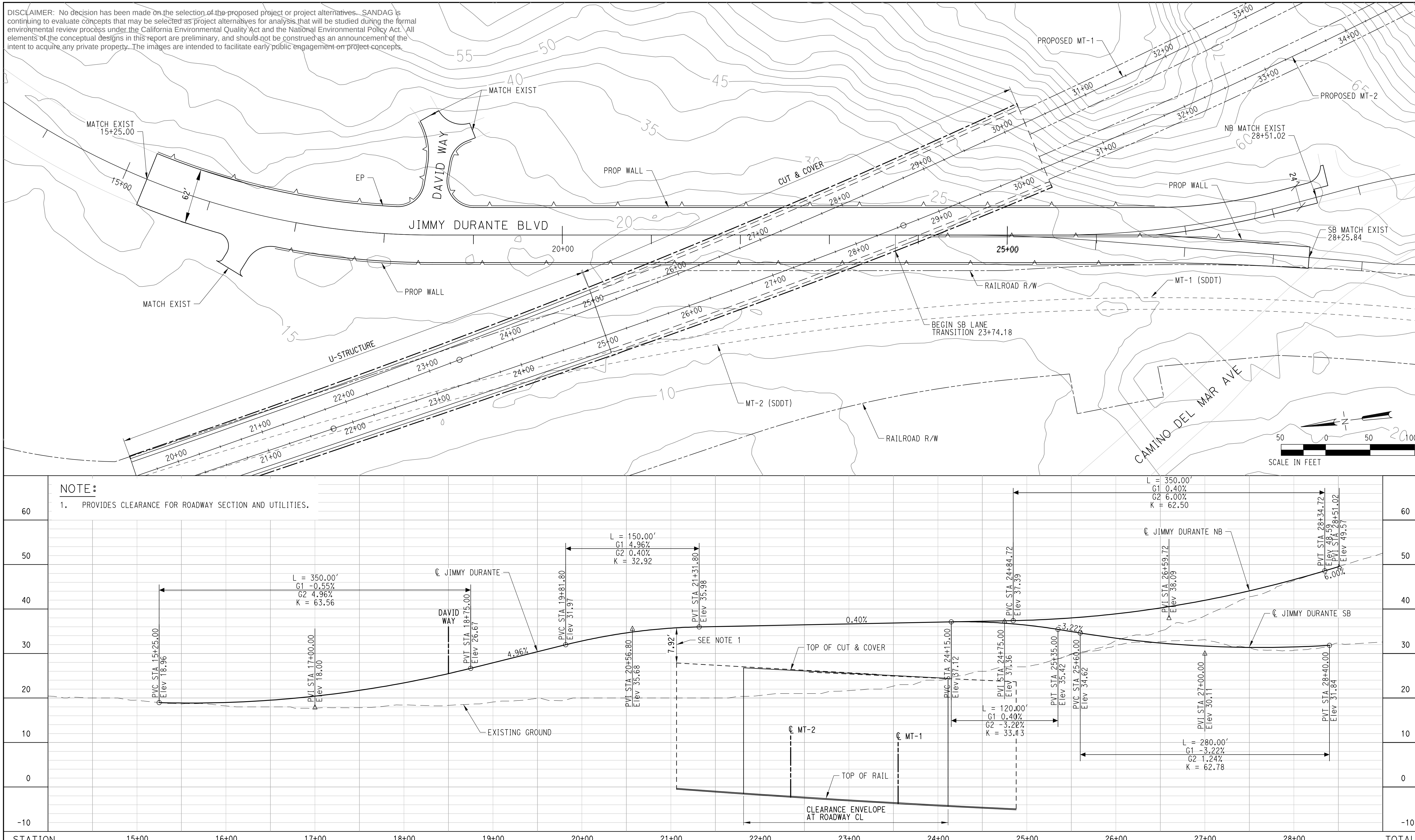

REV.	DATE	DESCRIPTION	BY / SUB	APP.

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DESIGNED BY K. WARFIELD
DRAWN BY M.R. GRANADO
CHECKED BY H. NGUYEN
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS
JIMMY DURANTE REALIGNMENT

CONTRACT NO.
DRAWING NO. GS101
REVISION
SHEET NO. 16 of 35
SCALE HORIZ: 1"=100′

10/20/2022
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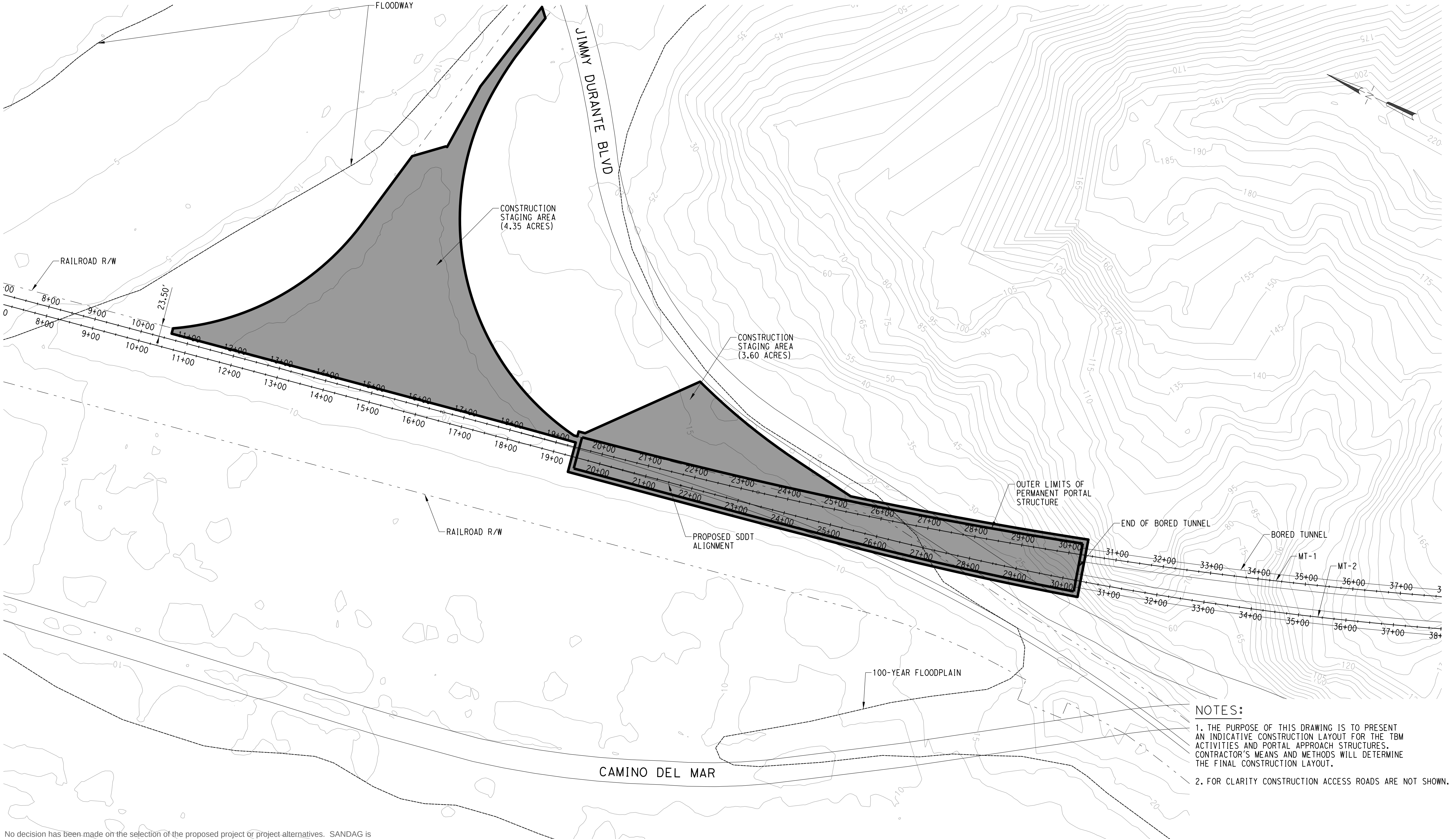

NOTES:

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT AN INDICATIVE CONSTRUCTION LAYOUT FOR THE TBM ACTIVITIES AND PORTAL APPROACH STRUCTURES. CONTRACTOR'S MEANS AND METHODS WILL DETERMINE THE FINAL CONSTRUCTION LAYOUT.
2. FOR CLARITY CONSTRUCTION ACCESS ROADS ARE NOT SHOWN.

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DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

APPROVED: ________ DATE: ________

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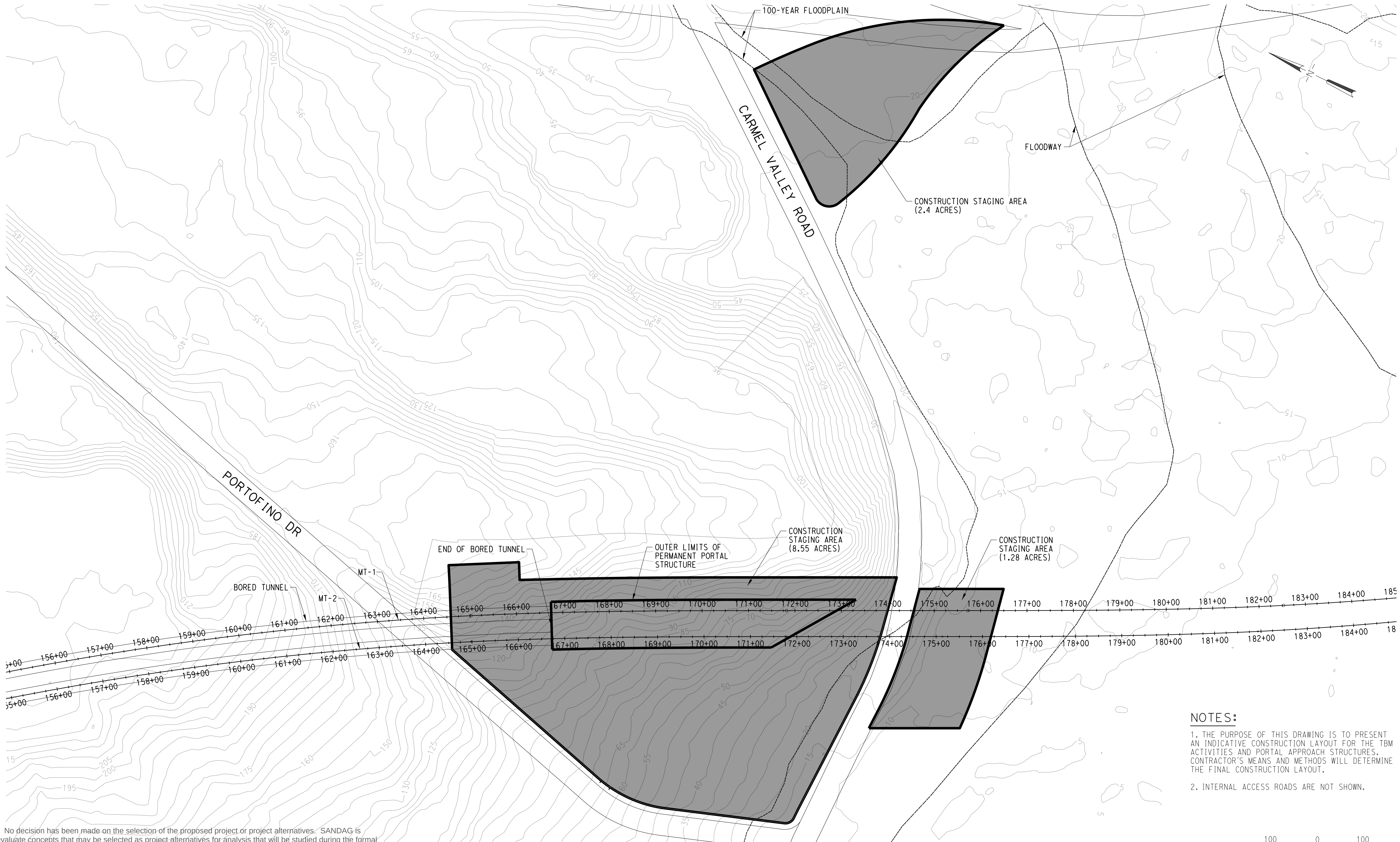
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE NORTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA101
REVISION
SHEET NO. 17 of 35
SCALE HORIZ: 1"=200'

10/28/2022



NOTES:

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT AN INDICATIVE CONSTRUCTION LAYOUT FOR THE TBM ACTIVITIES AND PORTAL APPROACH STRUCTURES. CONTRACTOR'S MEANS AND METHODS WILL DETERMINE THE FINAL CONSTRUCTION LAYOUT.

2. INTERNAL ACCESS ROADS ARE NOT SHOWN.

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10/28/2022
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APPROVED BY M. RAMSEY
DATE OCTOBER 2022

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750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ____________

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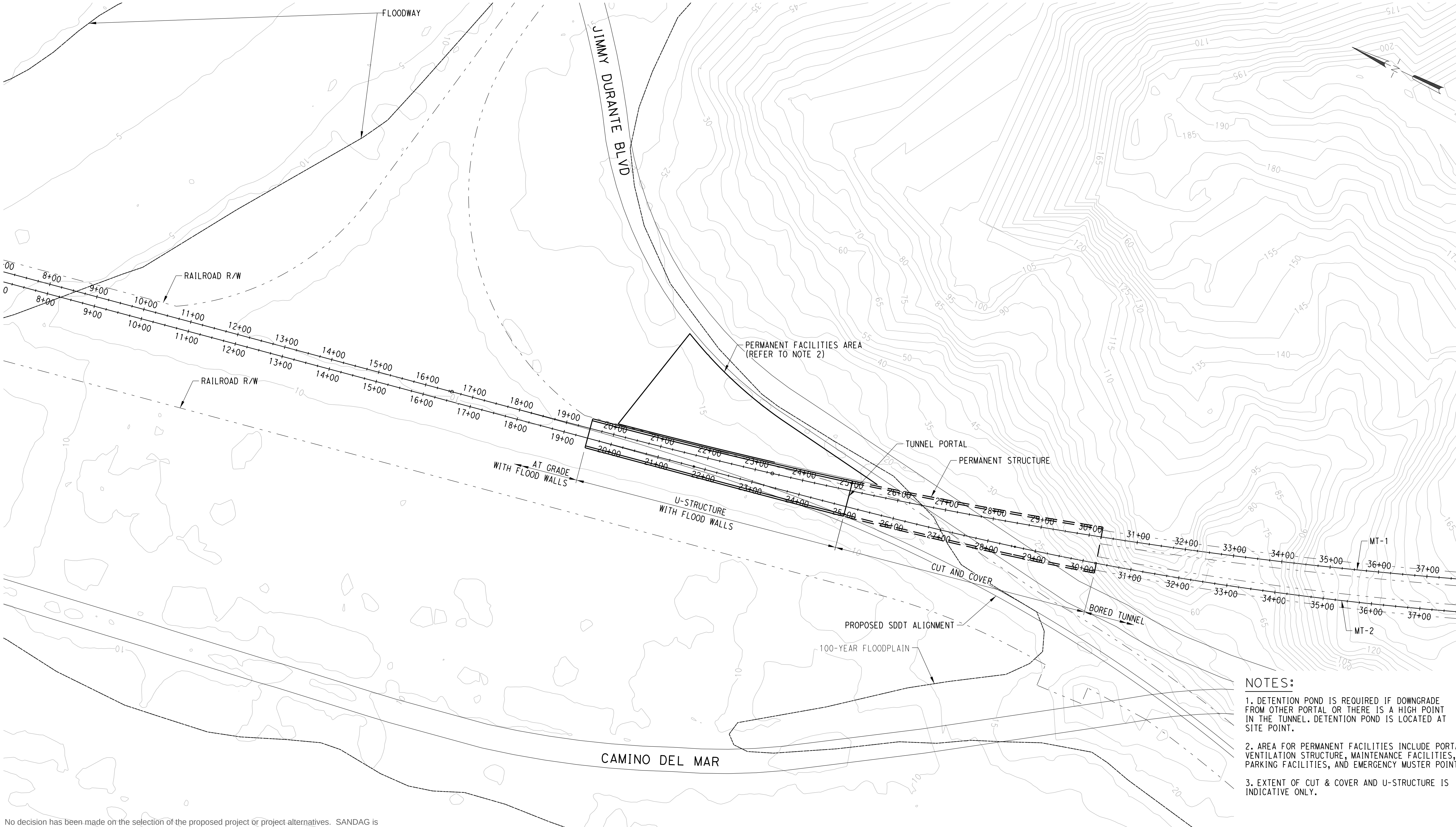
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE SOUTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA102
REVISION
SHEET NO. 18 of 35
SCALE HORIZ: 1"=200'



NOTES:

1. DETENTION POND IS REQUIRED IF DOWNGRADE FROM OTHER PORTAL OR THERE IS A HIGH POINT IN THE TUNNEL. DETENTION POND IS LOCATED AT SITE POINT.

2. AREA FOR PERMANENT FACILITIES INCLUDE PORTAL VENTILATION STRUCTURE, MAINTENANCE FACILITIES, PARKING FACILITIES, AND EMERGENCY MUSTER POINTS.

3. EXTENT OF CUT & COVER AND U-STRUCTURE IS INDICATIVE ONLY.

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REV.	DATE	DESCRIPTION	BY / SUB.	APP.

Information confidential: all plans, drawings, specifications, and or information furnished herewith shall remain the property of the North County Transit District and shall be held confidential; and shall not be used for any purpose not provided for in agreements with the North County Transit District.

DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

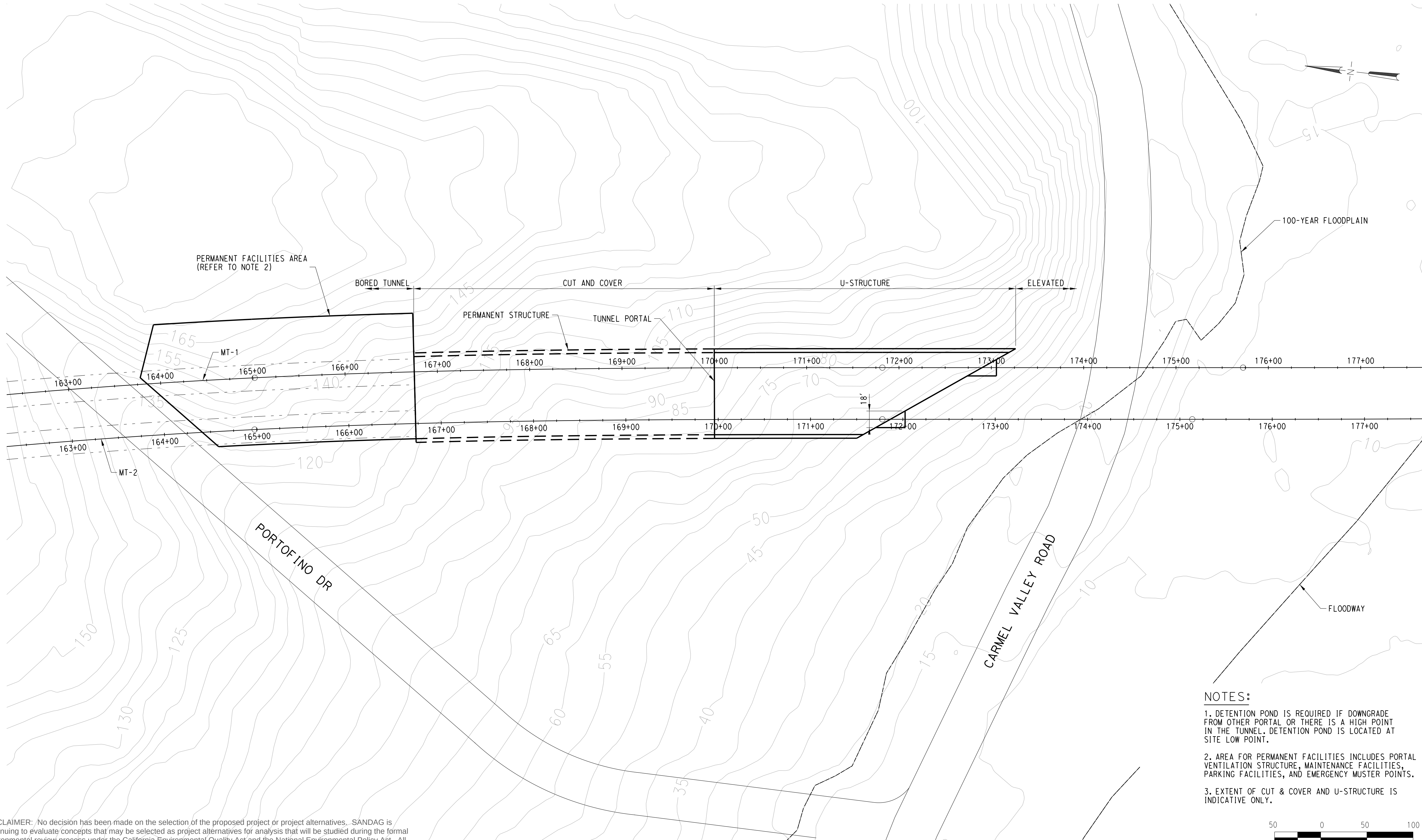
MOTT MACDONALD
750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency
APPROVED: ____________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE
NORTH PORTAL STRUCTURES
AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA103
REVISION
SHEET NO. 19 of 35
SCALE HORIZ: 1"=200'



NOTES:

1. DETENTION POND IS REQUIRED IF DOWNGRADE FROM OTHER PORTAL OR THERE IS A HIGH POINT IN THE TUNNEL. DETENTION POND IS LOCATED AT SITE LOW POINT.

2. AREA FOR PERMANENT FACILITIES INCLUDES PORTAL VENTILATION STRUCTURE, MAINTENANCE FACILITIES, PARKING FACILITIES, AND EMERGENCY MUSTER POINTS.

3. EXTENT OF CUT & COVER AND U-STRUCTURE IS INDICATIVE ONLY.

50 0 50 100
SCALE IN FEET

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10/28/2022
\\omapi-pwics03\iCS_working\15571\2014204_30\SDLOSSAN-08-MM-DT-SA104.dgn
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REV.	DATE	DESCRIPTION	BY / SUB.	APP.

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DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

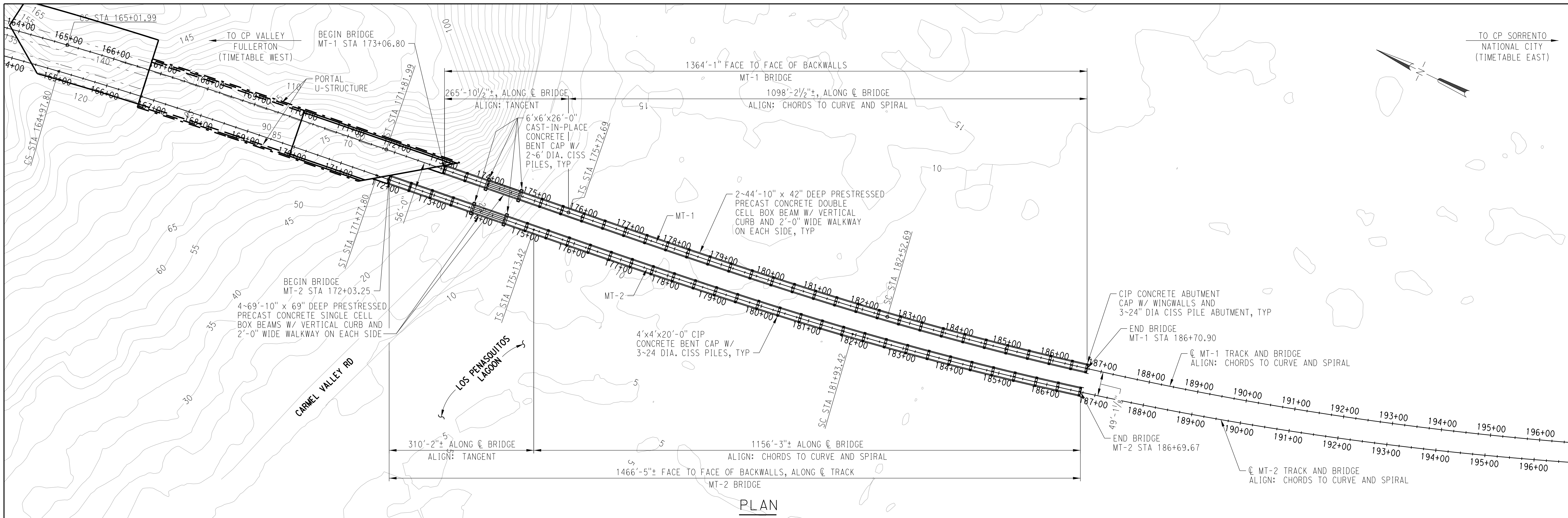
SANDAG
San Diego's Regional Planning Agency

APPROVED: ______ DATE: ______

10% SUBMITTAL
NOT FOR CONSTRUCTION

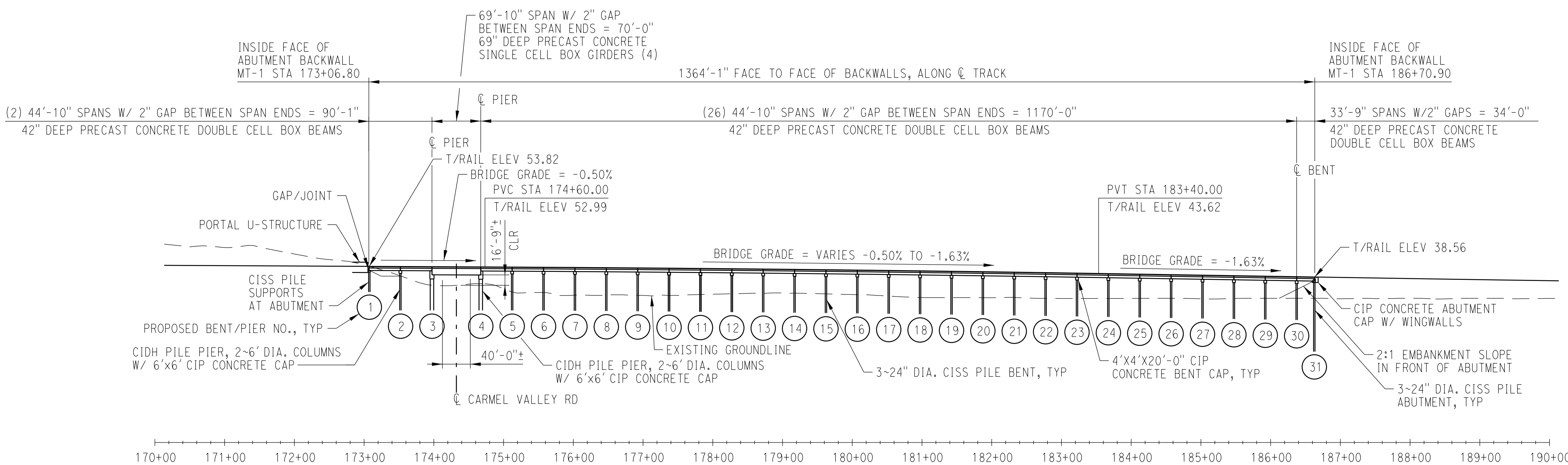
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA104
REVISION
SHEET NO. 20 of 35
SCALE HORIZ: 1"=100'



PLAN

SCALE: 1" = 200'



MT-1 ELEVATION

SCALE: 1" = 200'
MT-2 SIMILAR

GENERAL NOTES:

CAST-IN-STEEL-SHELL (CISS) PILE BENTS CONSIST OF 24" DIAMETER, 1" THICK DRIVEN STEEL PILES. INTERIOR OF PILES EXCAVATED AND FILLED WITH CONCRETE TO BOTTOM OF CIP CONCRETE BENT CAP.

CAST-IN-DRILLED HOLE (CIDH) PILE BENTS CONSIST OF CALTRANS TYPE II SHAFTS WITH REINFORCED CONCRETE AND TEMPORARY STEEL CASINGS FOR INSTALLATION.

DISTANCE FROM TOP OF RAIL TO TOP OF BRIDGE DECK = 2.375' AND ASSUMES 136 LB RAIL, CONCRETE TIES AND 12" BALLAST UNDER TIE.

WATERPROOFING ON BRIDGE DECK OVER ROADWAYS ONLY.

2'-0" WIDE WALKWAYS ON BOTH SIDES OF BRIDGES ATTACHED TO BALLAST CURB.

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10/20/2022
\\omapl-pwicsO3\ICS_working\14956\2014206.4\SDLOSSAN-05-HDR-DT-BR101.dgn
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C:\Program Files (x86)\Common Files\InterPlot\IPLOT\bin\plotdrvm.plt

REV.	DATE	DESCRIPTION	BY/SUB	APP.

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DESIGNED BY A. HAFNER
DRAWN BY J. MAY
CHECKED BY P. GRAFF
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

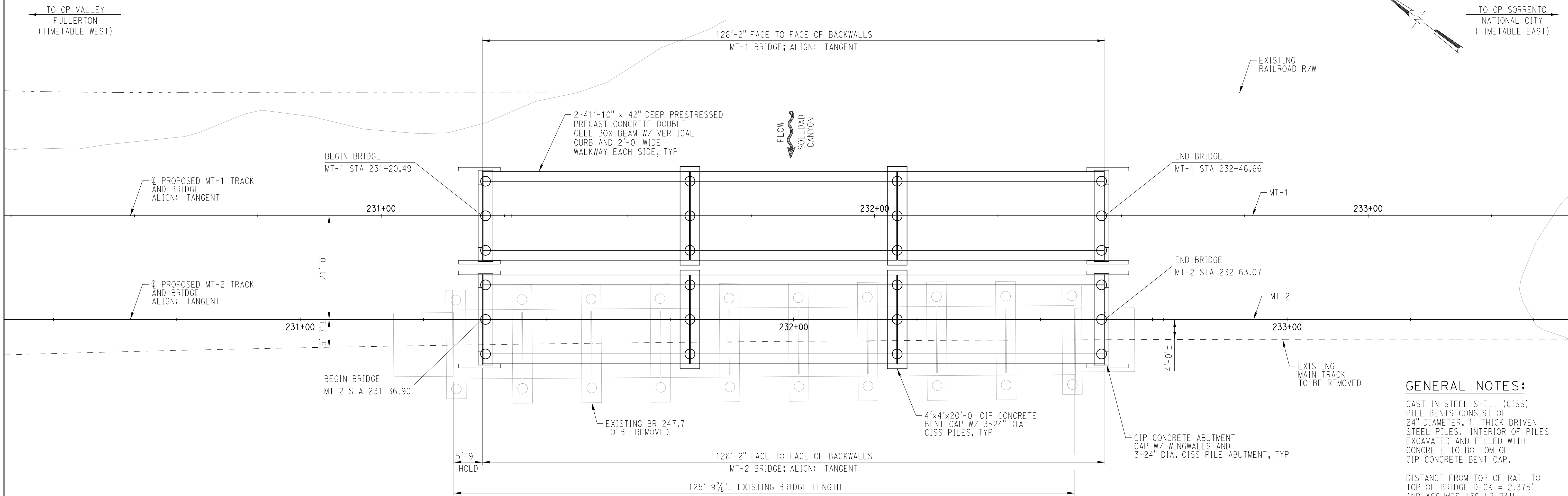
SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS
BRIDGE NO. 1 PLAN AND ELEVATION
MT-1 STA 173+06.80 TO STA 186+70.90

CONTRACT NO.
DRAWING NO. BR101
REVISION
SHEET NO. 21 of 35
SCALE 1" - 200'



PLAN

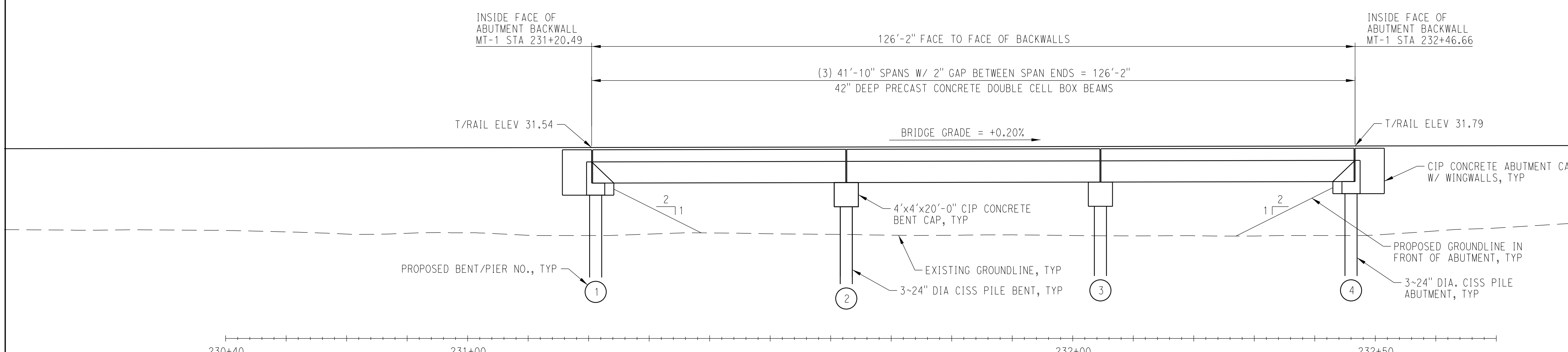
SCALE: 1" = 20'

GENERAL NOTES:

CAST-IN-STEEL-SHELL (CISS) PILE BENTS CONSIST OF 24" DIAMETER, 1" THICK DRIVEN STEEL PILES. INTERIOR OF PILES EXCAVATED AND FILLED WITH CONCRETE TO BOTTOM OF CIP CONCRETE BENT CAP.

DISTANCE FROM TOP OF RAIL TO TOP OF BRIDGE DECK = 2.375' AND ASSUMES 136 LB RAIL, CONCRETE TIES AND 12" BALLAST UNDER TIE.

2'-0" WIDE WALKWAYS ON BOTH SIDES OF BRIDGES ATTACHED TO BALLAST CURB.



MT-1 ELEVATION

SCALE: 1" = 20'
MT-2 SIMILAR
EXISTING BRIDGE NOT SHOWN FOR CLARITY

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intention to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

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DESIGNED BY A. HAFNER
DRAWN BY J. MAY
CHECKED BY P. GRAFF
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS
BRIDGE NO. 2 PLAN AND ELEVATION
MT-1 STA 231+20.49 TO STA 232+46.66

CONTRACT NO.
DRAWING NO. BR102
REVISION
SHEET NO. 22 of 35
SCALE 1" = 20'

10/7/2022

NOTES:

1. FOR TYPICAL SECTIONS AT CUT & COVER AND U-STRUCTURE LIMITS BETWEEN MT-1 STA 19+50 TO STA 31+50 AND MT-1 STA 127+50 TO STA 131+41, SEE STRUCTURAL PLANS.

2. FOR TYPICAL SECTIONS AT TUNNEL LIMITS BETWEEN MT-1 STA 31+50 TO STA 127+50, SEE STRUCTURAL PLANS.

3. REFER TO BRIDGE PLANS FOR ADDITIONAL DETAILS.

4. CONSTRUCTION OF SHOOFLY TRACK(S) AND OTHER TEMPORARY STRUCTURES ARE REQUIRED FOR THE CONSTRUCTION OF THIS PROJECT. TEMPORARY MEASURES TO BE DEVELOPED IN A SUBSEQUENT PHASE.

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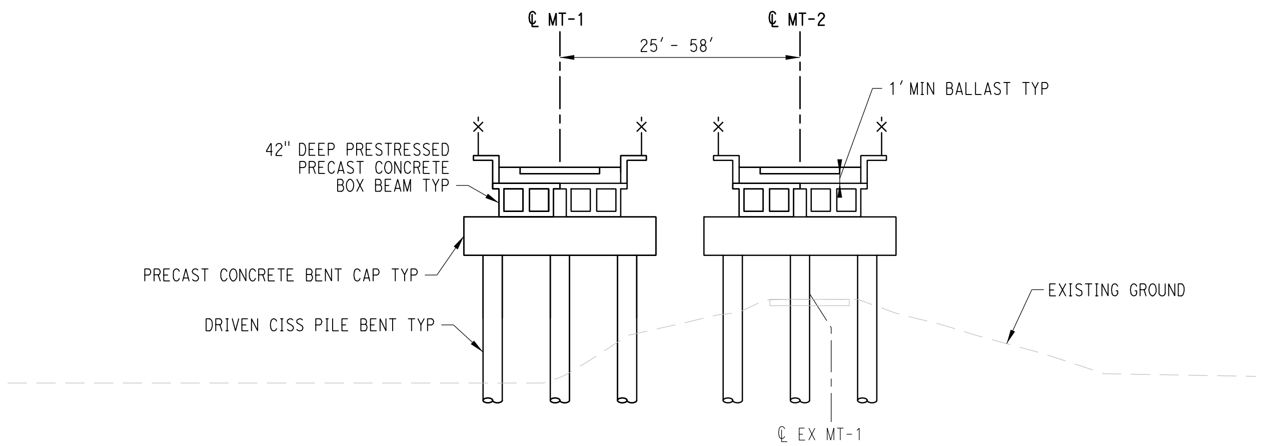
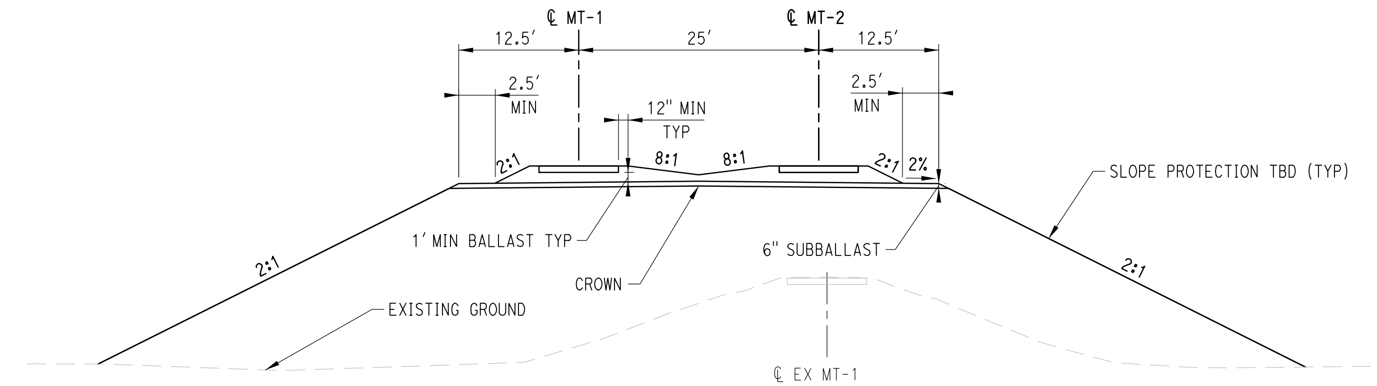


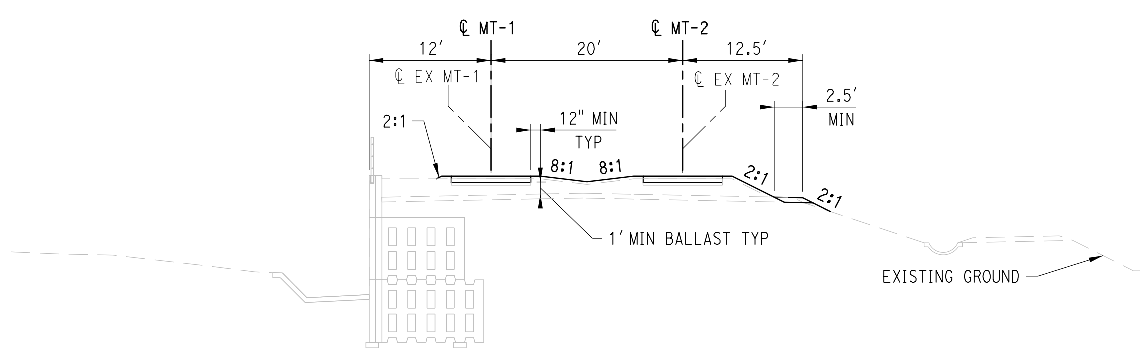
MT-1 STA 131+48 TO STA 179+00
MT-1 STA 186+00 TO STA 198+00
MT-1 STA 237+65 TO STA 238+91

NO SCALE



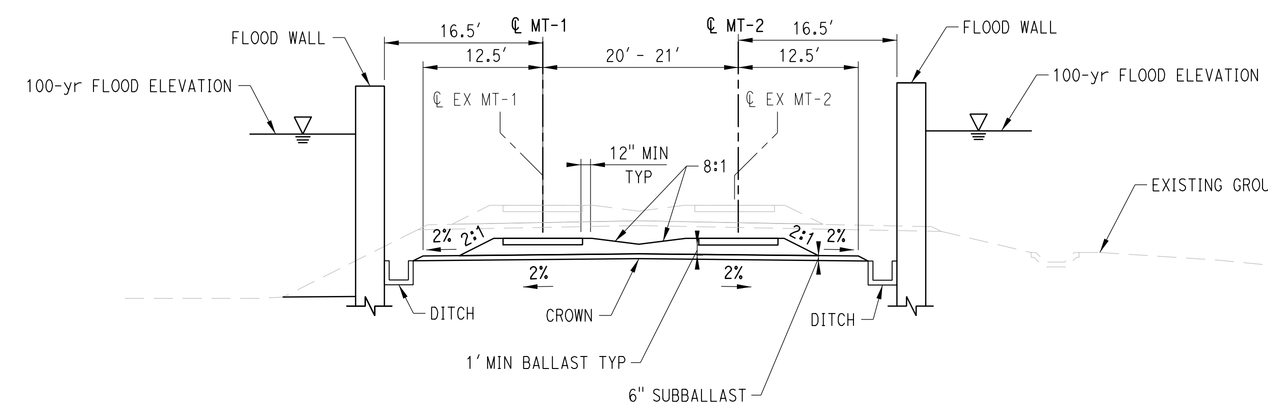
MT-1 STA 179+00 TO STA 186+00

NO SCALE



MT-1 STA 9+00 TO STA 12+00

NO SCALE



MT-1 STA 12+00 TO STA 19+50

NO SCALE

REV.	DATE	DESCRIPTION	BY / SUB	APP.

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DESIGNED BY J.K. ANDRES
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR
TYPICAL SECTION
SHEET 1 OF 2

CONTRACT NO.
DRAWING NO. TS201
REVISION
SHEET NO. 23 of 35
SCALE NO SCALE

10/21/2022

NOTES:

1. CONSTRUCTION OF SHOOFLY TRACK(S) AND OTHER TEMPORARY STRUCTURES ARE REQUIRED FOR THE CONSTRUCTION OF THIS PROJECT. TEMPORARY MEASURES TO BE DEVELOPED IN A SUBSEQUENT PHASE.

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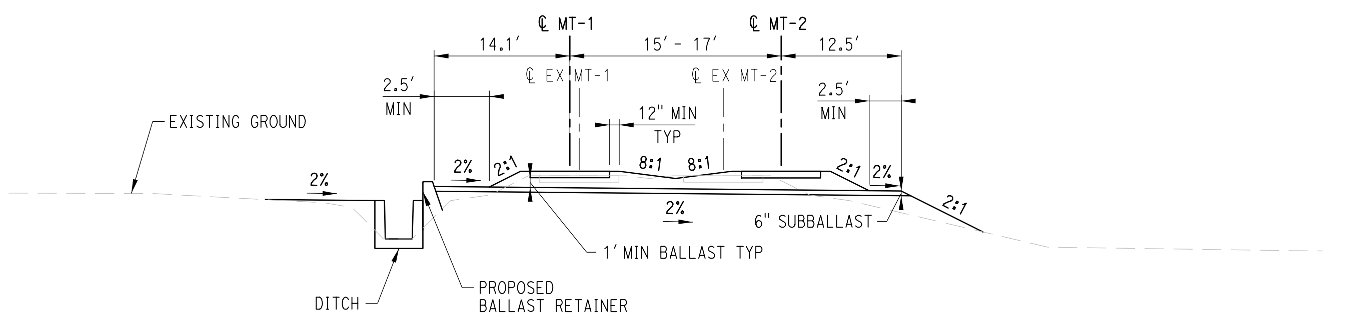
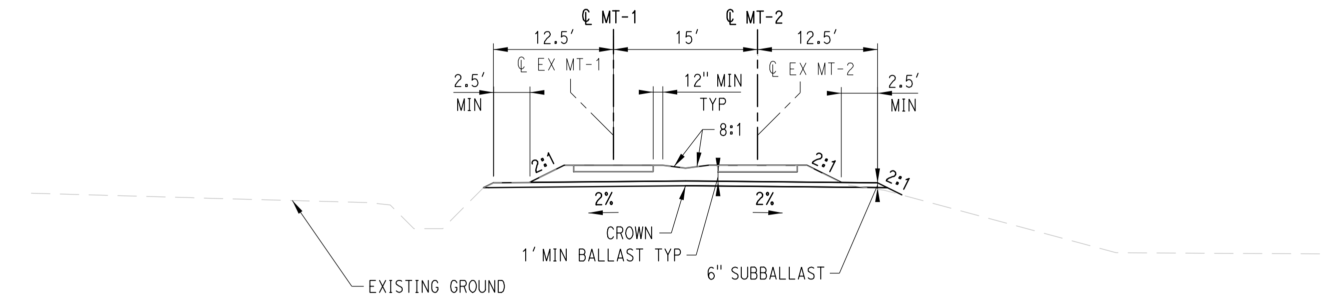


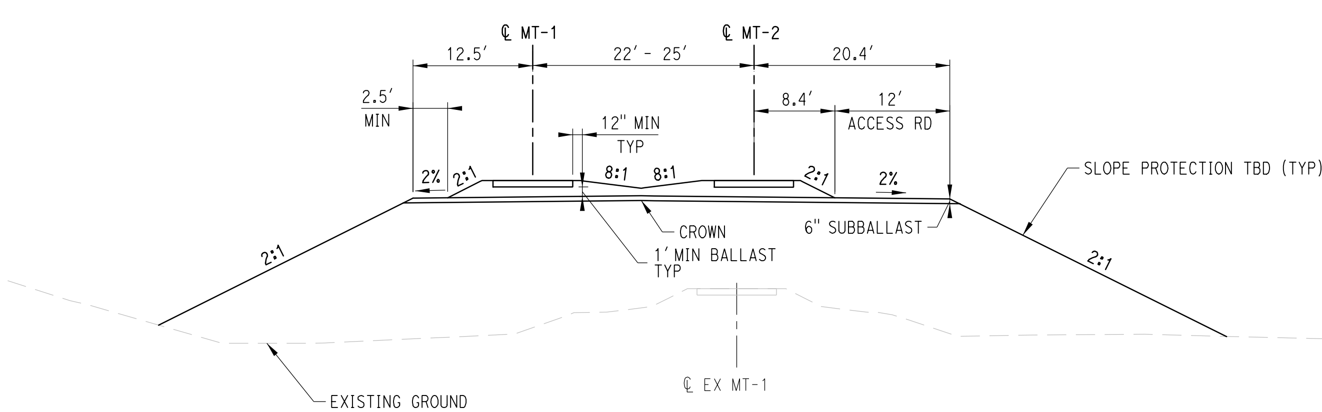
MT-1 STA 255+00 TO STA 259+00

NO SCALE



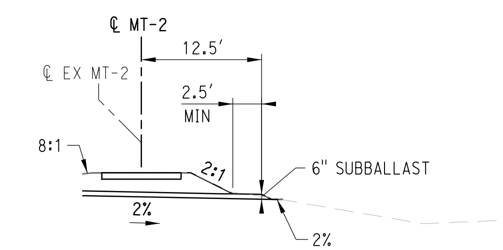
MT-1 STA 259+00 TO STA 263+88

NO SCALE



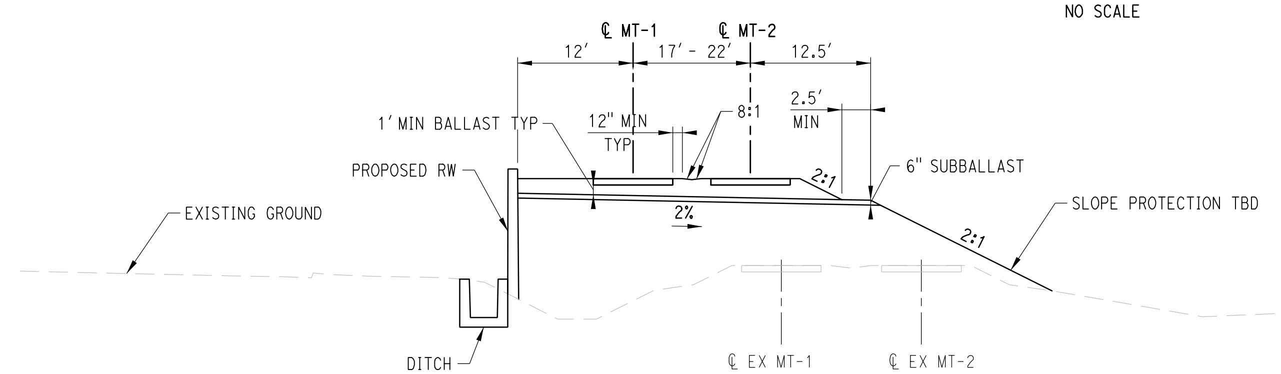
MT-1 STA 198+00 TO STA 237+65

NO SCALE



MT-1 STA 251+25 TO STA 255+00

NO SCALE



MT-1 STA 238+91 TO STA 255+00

NO SCALE

REV.	DATE	DESCRIPTION	BY / SUB	APP.

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DESIGNED BY J.K. ANDRES
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

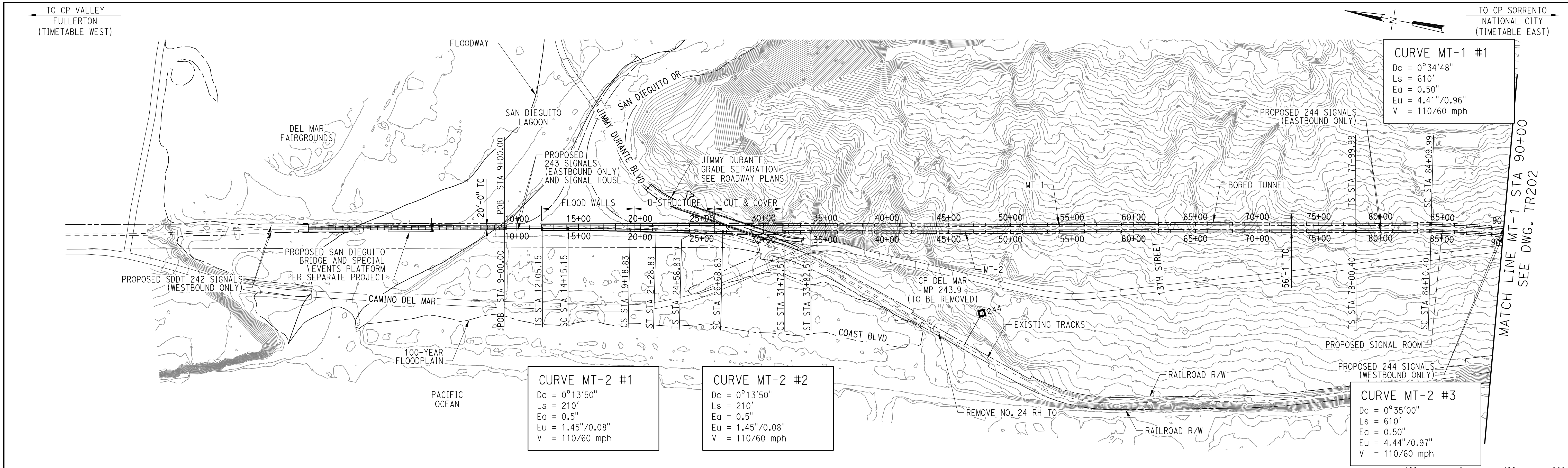
SANDAG San Diego's Regional Planning Agency

APPROVED: ______ DATE: ______

10% SUBMITTAL NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR TYPICAL SECTION SHEET 2 OF 2

CONTRACT NO.
DRAWING NO. TS202
REVISION
SHEET NO. 24 of 35
SCALE NO SCALE



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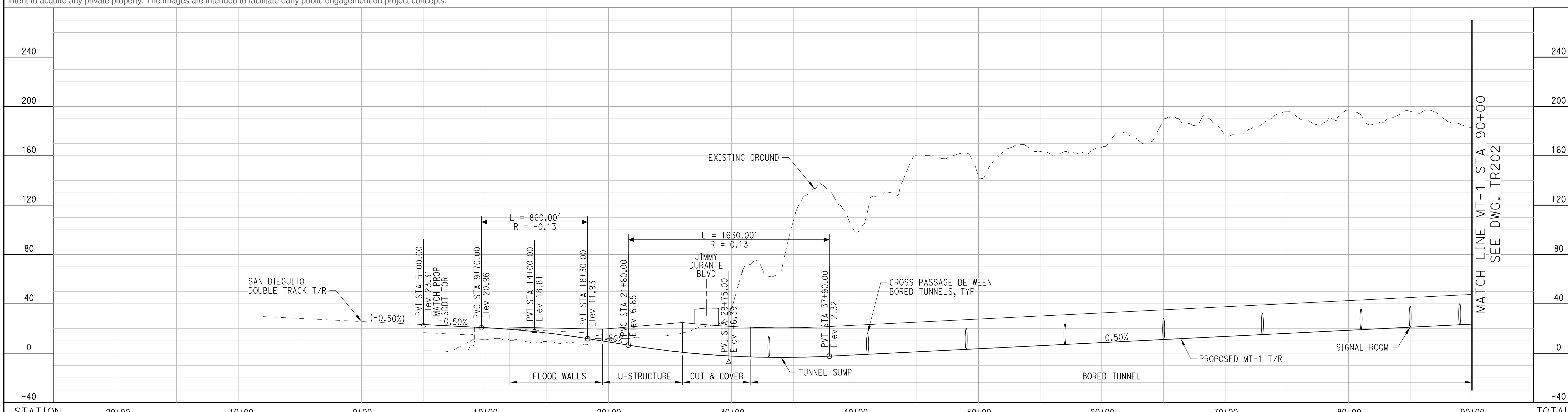
PLAN

HORIZ 400 0 400 800
VERT 40 0 40 80
SCALE IN FEET



REV.	DATE	DESCRIPTION	BY / SUB	APP.

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DESIGNED BY K. MAGEE
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

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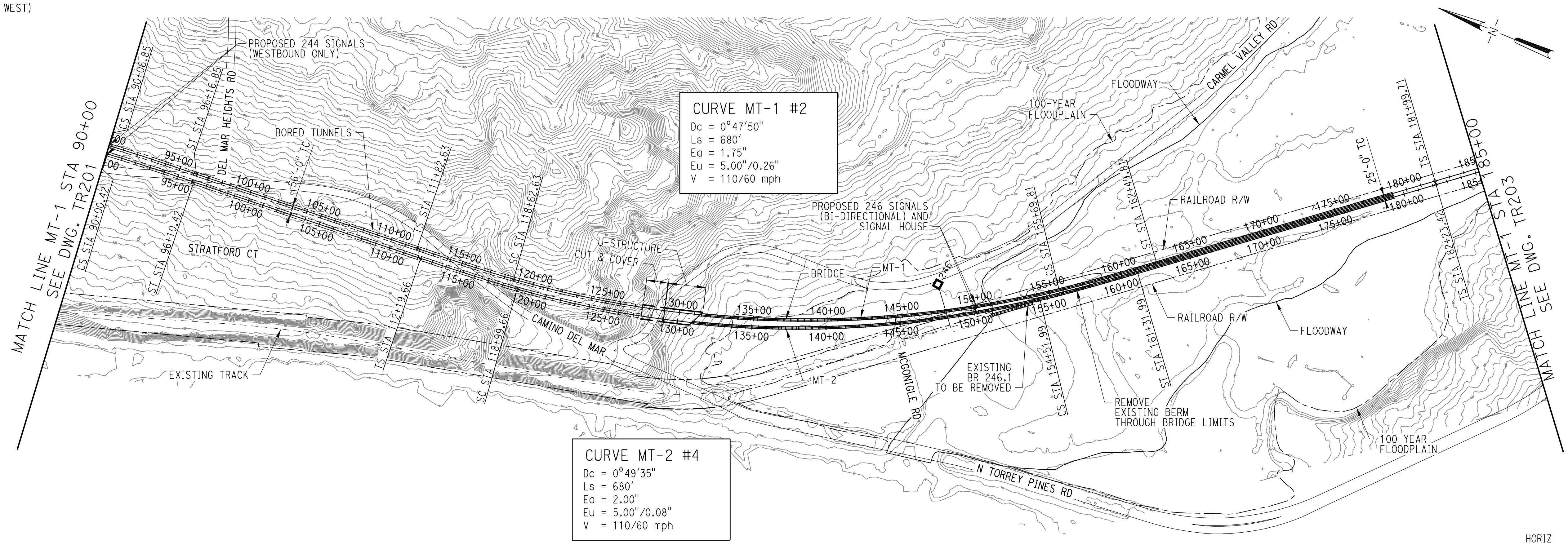
SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR
TRACK PLAN AND PROFILE
MT-1 STA 9+00 TO STA 90+00

CONTRACT NO.
DRAWING NO. TR201
REVISION
SHEET NO. 25 of 35
SCALE
VERT: 1'=80'
HORIZ: 1"=800'

TO CP VALLEY
FULLERTON
(TIMETABLE WEST)

TO CP SORRENTO
NATIONAL CITY
(TIMETABLE EAST)



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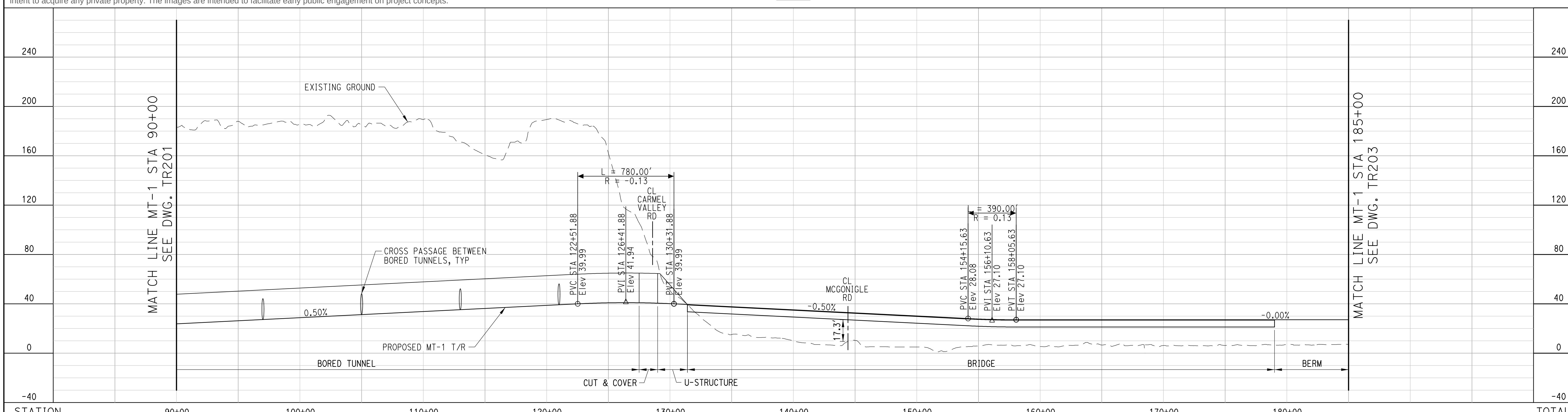
PLAN

HORIZ 400 0 400 800
VERT 40 0 40 80
SCALE IN FEET



REV.	DATE	DESCRIPTION	BY / SUB	APP.

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DESIGNED BY K. MAGEE
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR
TRACK PLAN AND PROFILE
MT-1 STA 90+00 TO STA 185+00

CONTRACT NO.
DRAWING NO. TR202
REVISION
SHEET NO. 26 of 35
SCALE
VERT: 1"=80'
HORIZ: 1"=800'

10/21/2022
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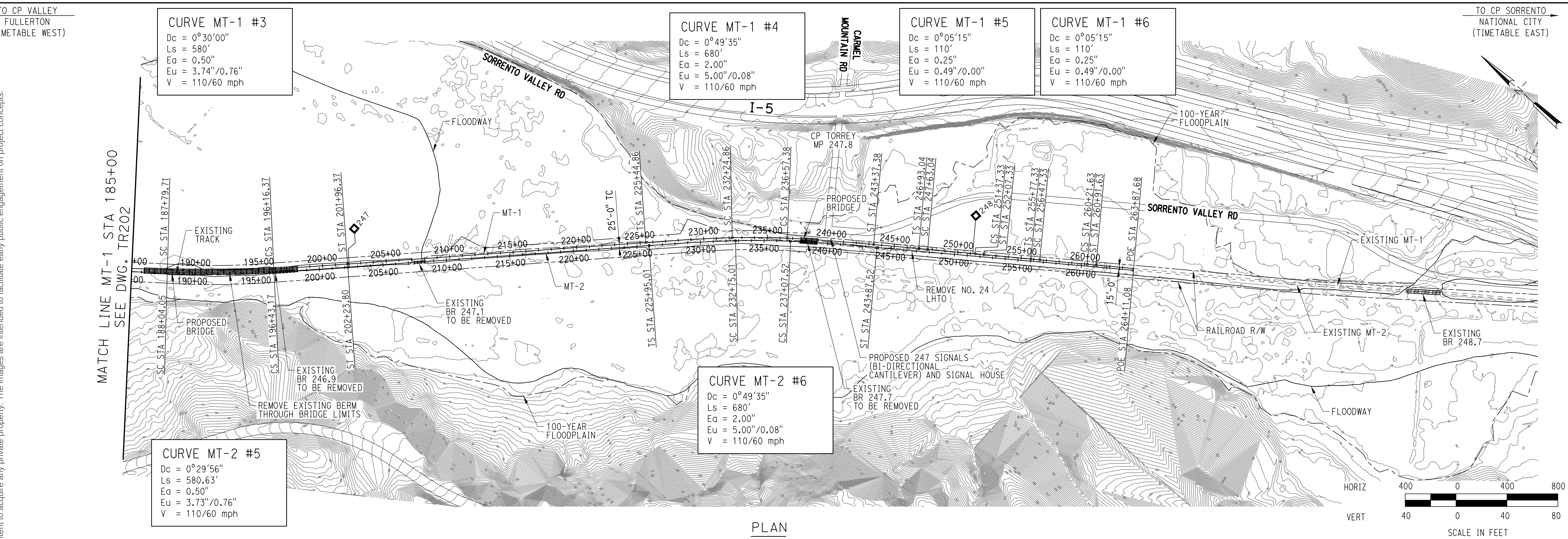

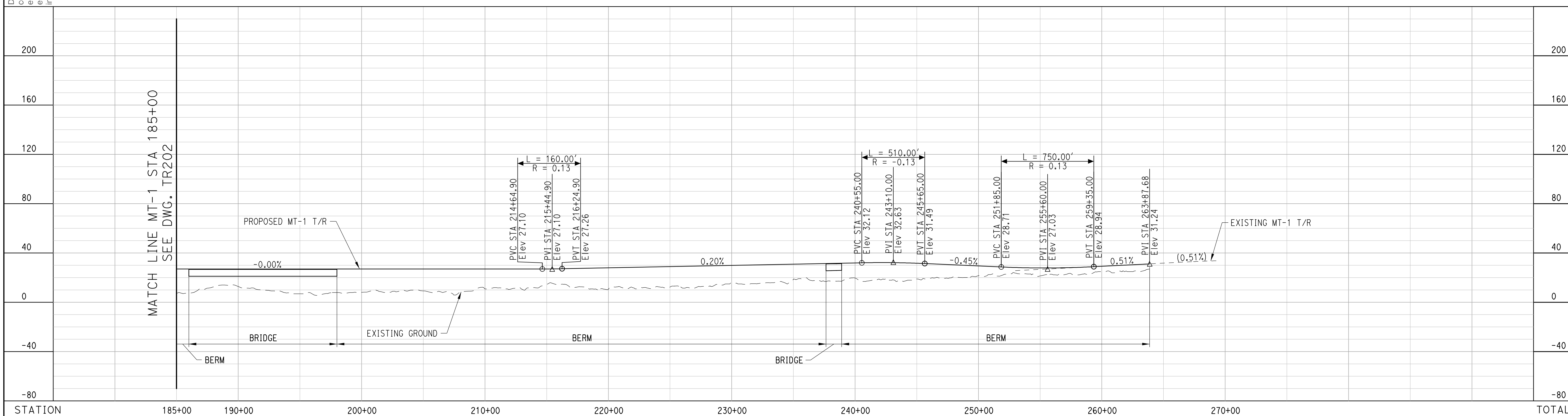

REV.	DATE	DESCRIPTION	BY SUB.	APP.

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DESIGNED BY K. MAGEE
DRAWN BY M.R. GRANADO
CHECKED BY A. RUBIO
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ____________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR
TRACK PLAN AND PROFILE
MT-1 STA 185+00 TO STA 263+87.68

CONTRACT NO.
DRAWING NO. TR203
REVISION
SHEET NO. 27 of 35
SCALE VERT: 1'=80' HORIZ: 1'=800'

10/21/2022
c:\pwworking\west01\d2014197\SDLOSSAN-03-HDR-DT-TR203.dgn
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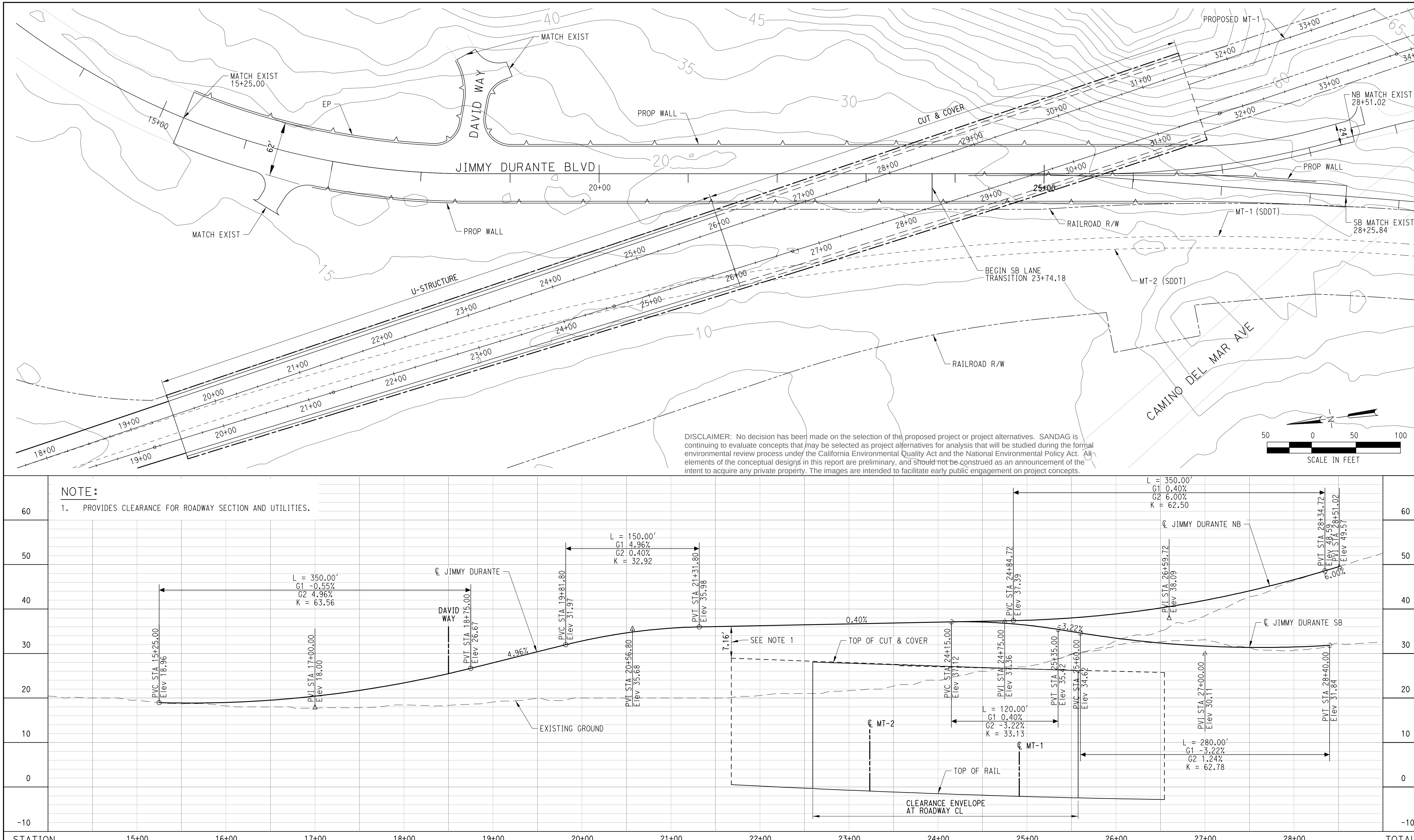

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DESIGNED BY K. WARFIELD
DRAWN BY M.R. GRANADO
CHECKED BY H. NGUYEN
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

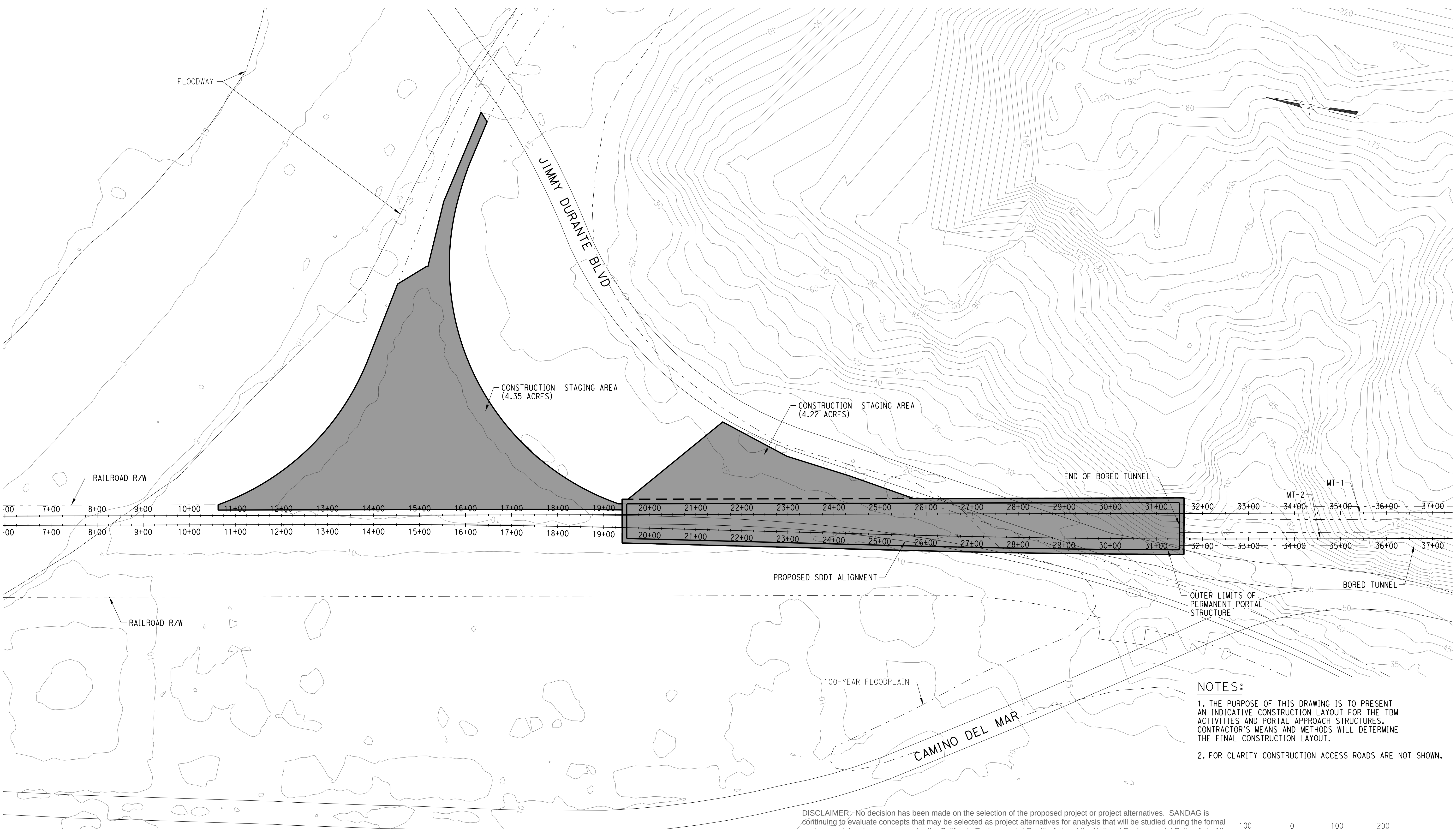
APPROVED: ____ DATE: ____

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR JIMMY DURANTE REALIGNMENT

CONTRACT NO.
DRAWING NO. GS201
REVISION

SCALE HORIZ: 1"=100'



NOTES:

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT AN INDICATIVE CONSTRUCTION LAYOUT FOR THE TBM ACTIVITIES AND PORTAL APPROACH STRUCTURES. CONTRACTOR'S MEANS AND METHODS WILL DETERMINE THE FINAL CONSTRUCTION LAYOUT.

2. FOR CLARITY CONSTRUCTION ACCESS ROADS ARE NOT SHOWN.

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REV. | DATE | DESCRIPTION | BY/SUB. | APP.

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DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street,
Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

CAMINO DEL MAR - TWIN BORE NORTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA201
REVISION
SHEET NO. 29 of 35
SCALE HORIZ: 1"=200'

10/28/2022
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NOTES:

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT AN INDICATIVE CONSTRUCTION LAYOUT FOR THE TBM ACTIVITIES AND PORTAL APPROACH STRUCTURES. CONTRACTOR'S MEANS AND METHODS WILL DETERMINE THE FINAL CONSTRUCTION LAYOUT.
2. FOR CLARITY CONSTRUCTION ACCESS ROADS ARE NOT SHOWN.

REV.	DATE	DESCRIPTION	BY / SUB.	APP.

Information confidential: all plans, drawings, specifications, and or information furnished herewith shall remain the property of the North County Transit District and shall be held confidential and shall not be used for any purpose not provided for in agreements with the North County Transit District.

DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency
APPROVED: DATE:

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR - TWIN BORE SOUTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA202
REVISION
SHEET NO. 30 of 35
SCALE HORIZ: 1"=100'

10/28/2022
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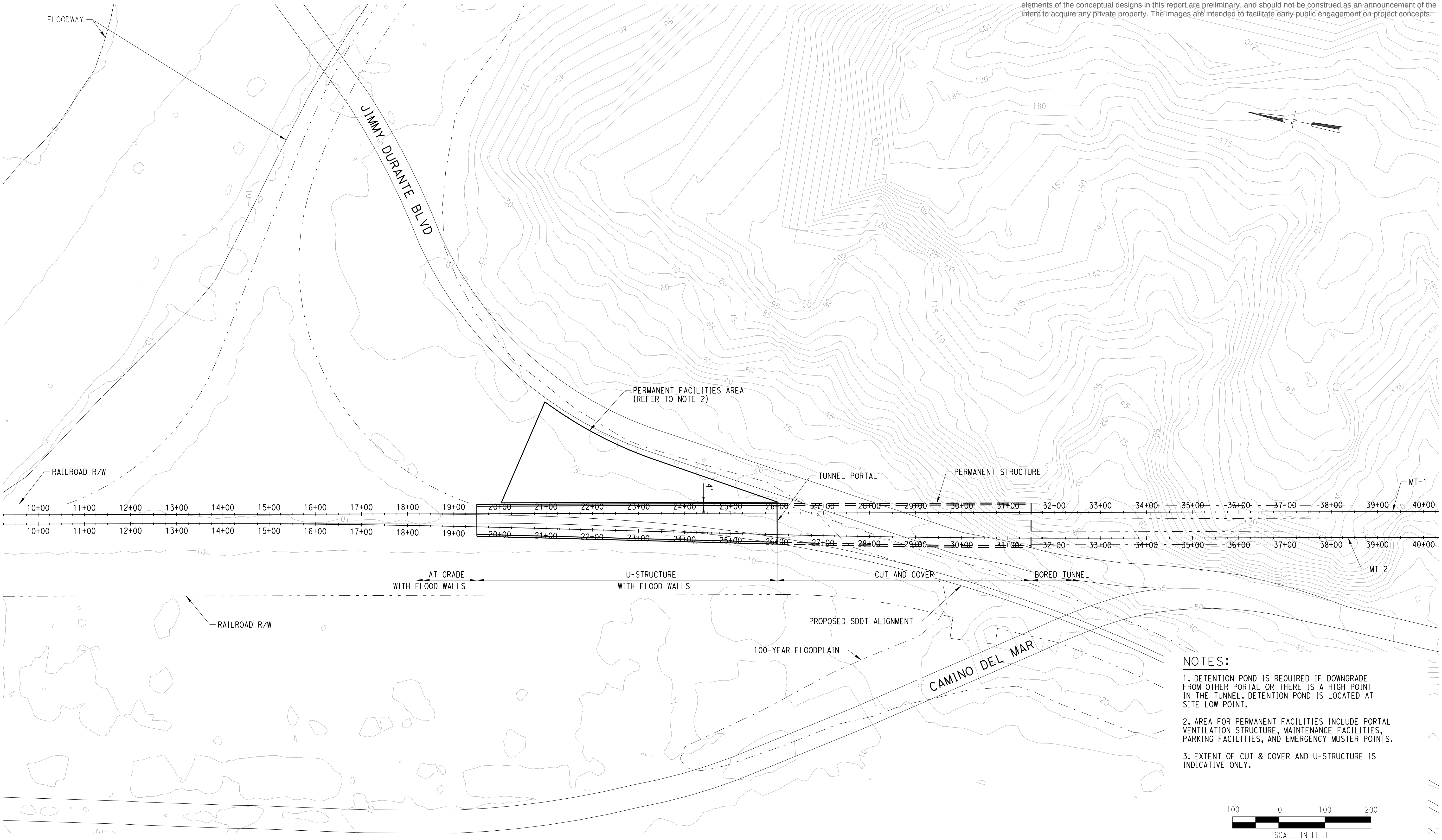

NOTES:

1. DETENTION POND IS REQUIRED IF DOWNGRADE FROM OTHER PORTAL OR THERE IS A HIGH POINT IN THE TUNNEL. DETENTION POND IS LOCATED AT SITE LOW POINT.

2. AREA FOR PERMANENT FACILITIES INCLUDE PORTAL VENTILATION STRUCTURE, MAINTENANCE FACILITIES, PARKING FACILITIES, AND EMERGENCY MUSTER POINTS.

3. EXTENT OF CUT & COVER AND U-STRUCTURE IS INDICATIVE ONLY.

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DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR TWIN BORE NORTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA203
REVISION
SHEET NO. 31 of 35
SCALE HORIZ: 1"=200'

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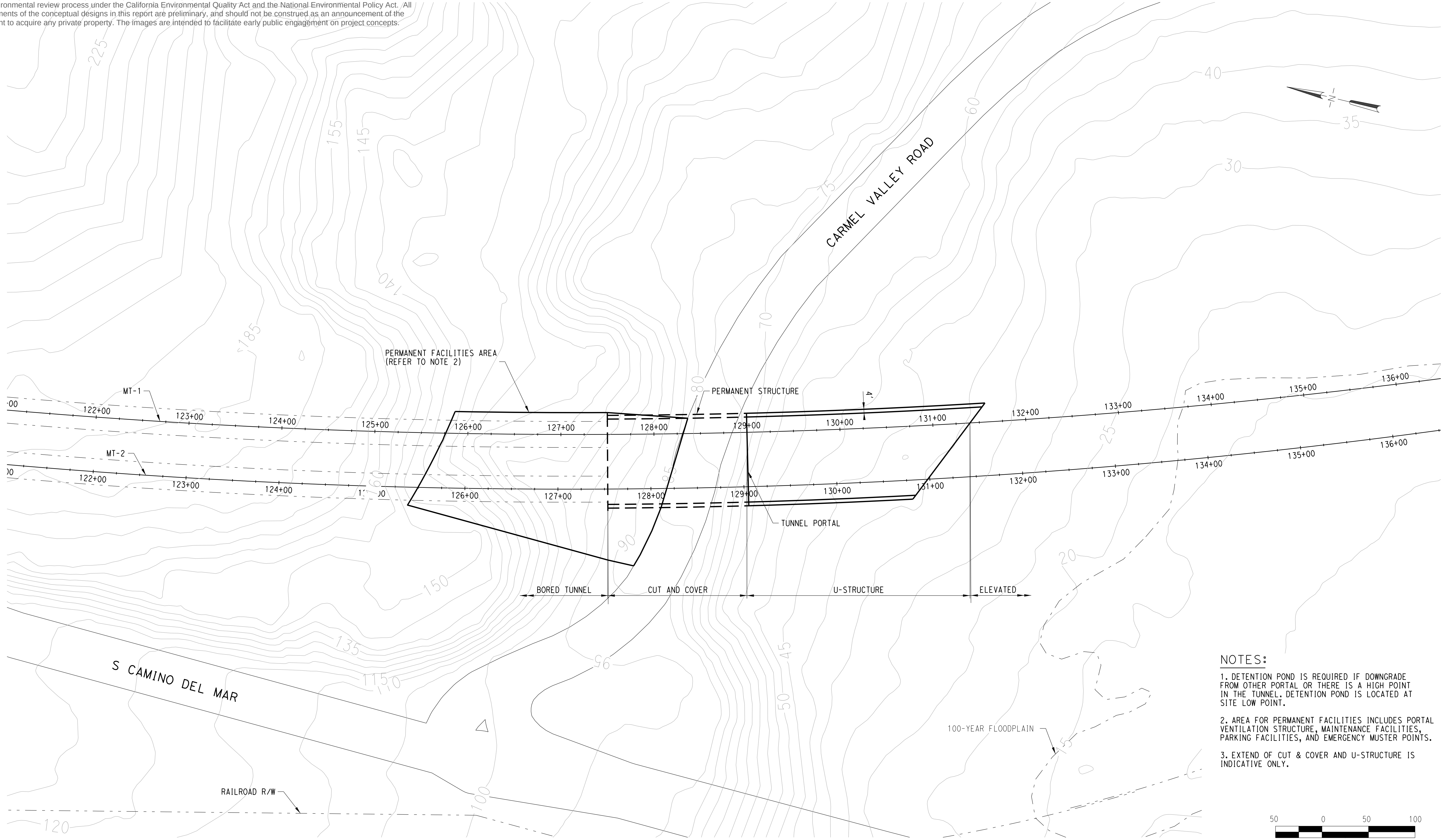

NOTES:

1. DETENTION POND IS REQUIRED IF DOWNGRADE FROM OTHER PORTAL OR THERE IS A HIGH POINT IN THE TUNNEL. DETENTION POND IS LOCATED AT SITE LOW POINT.

2. AREA FOR PERMANENT FACILITIES INCLUDES PORTAL VENTILATION STRUCTURE, MAINTENANCE FACILITIES, PARKING FACILITIES, AND EMERGENCY MUSTER POINTS.

3. EXTEND OF CUT & COVER AND U-STRUCTURE IS INDICATIVE ONLY.

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Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR TWIN BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA204
REVISION
SHEET NO. 32 of 35
SCALE HORIZ: 1"=100'

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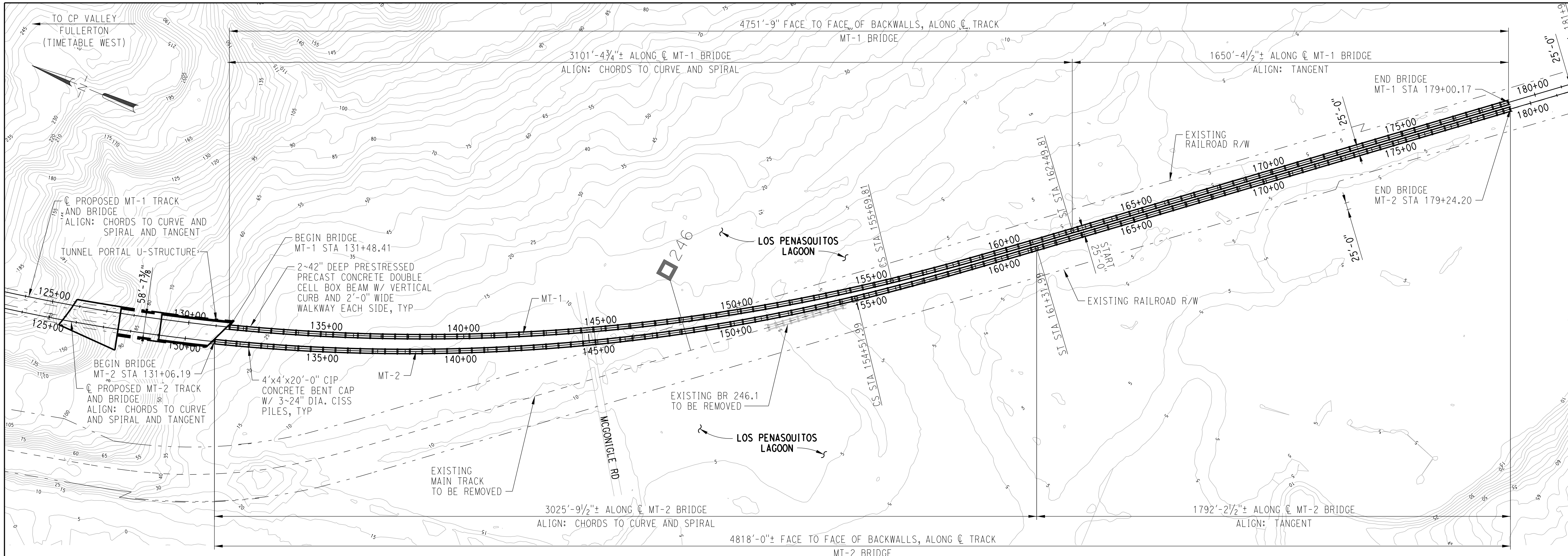


PLAN

SCALE: 1" = 400'

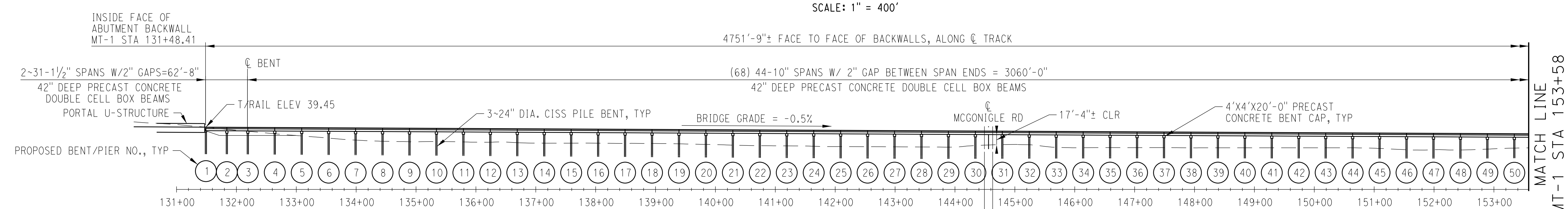
GENERAL NOTES:

CAST-IN-STEEL-SHELL (CISS) PILE BENTS CONSIST OF 24" DIAMETER, 1" THICK DRIVEN STEEL PILES. INTERIOR OF PILES EXCAVATED AND FILLED WITH CONCRETE TO BOTTOM OF CIP CONCRETE BENT CAP.

DISTANCE FROM TOP OF RAIL TO TOP OF BRIDGE DECK = 2.375' AND ASSUMES 136 LB RAIL, CONCRETE TIES AND 12" BALLAST UNDER TIE.

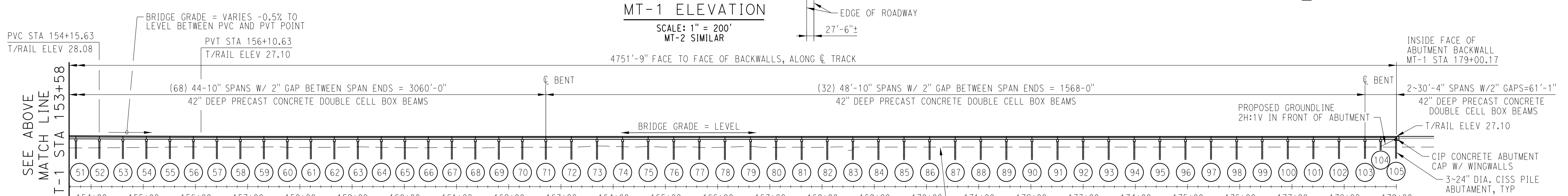
WATERPROOFING ON BRIDGE DECK OVER ROADWAYS ONLY.

2'-0" WIDE WALKWAYS ON BOTH SIDES OF BRIDGES ATTACHED TO BALLAST CURB.



MT-1 ELEVATION

SCALE: 1" = 200'
MT-2 SIMILAR



MT-1 ELEVATION

SCALE: 1" = 200'
MT-2 SIMILAR

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DESIGNED BY A. HAFNER
DRAWN BY J. MAY
CHECKED BY P. GRAFF
APPROVED BY R. KLOVSKY
DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency

APPROVED: ______ DATE: ______

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR
BRIDGE NO. 1 PLAN AND ELEVATION
MT-1 STA 131+48.41 TO STA 179+00.17

CONTRACT NO.
DRAWING NO. BR201
REVISION
SHEET NO. 33 of 35
SCALE AS NOTED

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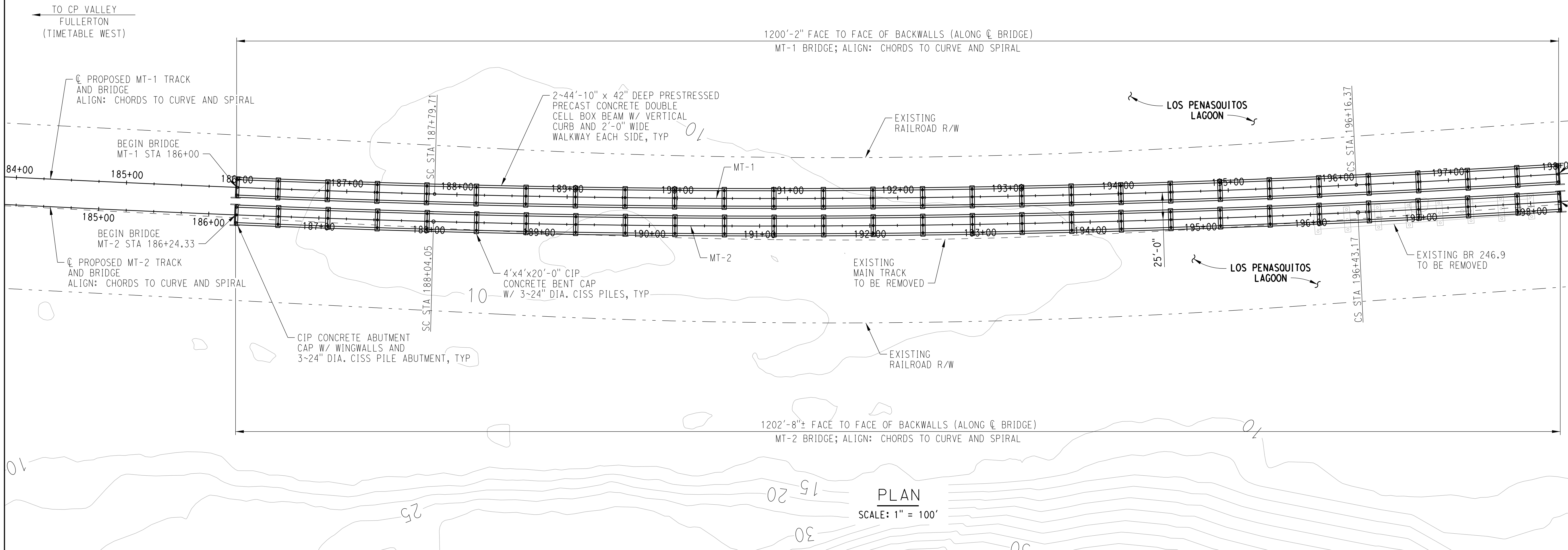


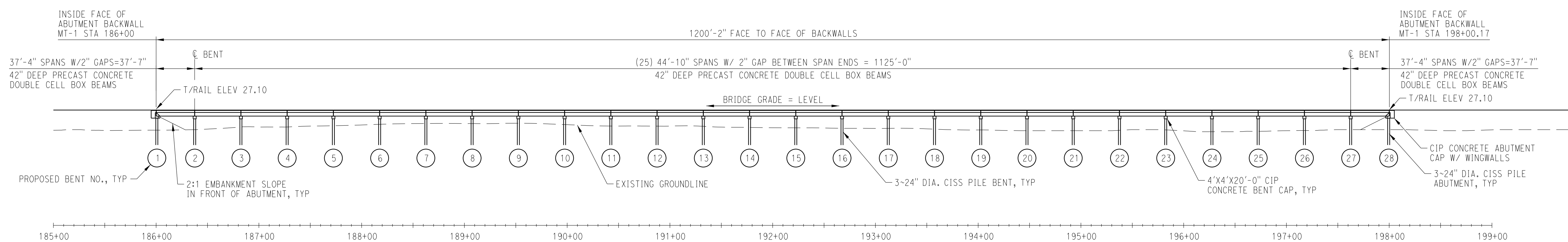
PLAN
SCALE: 1" = 100'

GENERAL NOTES:

CAST-IN-STEEL-SHELL (CISS) PILE BENTS CONSIST OF 24" DIAMETER, 1" THICK DRIVEN STEEL PILES. INTERIOR OF PILES EXCAVATED AND FILLED WITH CONCRETE TO BOTTOM OF CIP CONCRETE BENT CAP.

DISTANCE FROM TOP OF RAIL TO TOP OF BRIDGE DECK = 2.375' AND ASSUMES 136 LB RAIL, CONCRETE TIES AND 12" BALLAST UNDER TIE.

2'-0" WIDE WALKWAYS ON BOTH SIDES OF BRIDGES ATTACHED TO BALLAST CURB.



MT-1 ELEVATION
SCALE: 1" = 100'
MT-2 SIMILAR

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APPROVED: DATE:

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR
BRIDGE NO. 2 PLAN AND ELEVATION
MT-1 STA 186+00 TO STA 198+00.17

CONTRACT NO.
DRAWING NO. BR202
REVISION
SHEET NO. 34 of 35
SCALE 1'=100'

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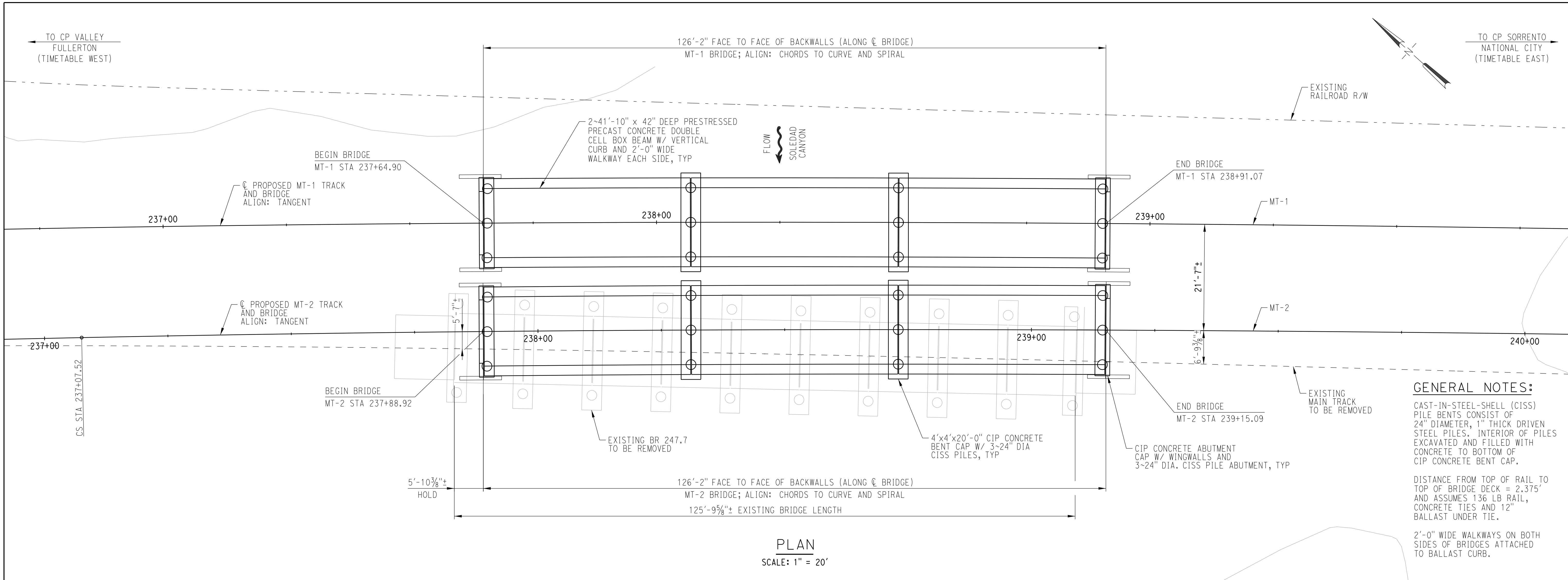

PLAN

SCALE: 1" = 20'

GENERAL NOTES:

CAST-IN-STEEL-SHELL (CISS) PILE BENTS CONSIST OF 24" DIAMETER, 1" THICK DRIVEN STEEL PILES. INTERIOR OF PILES EXCAVATED AND FILLED WITH CONCRETE TO BOTTOM OF CIP CONCRETE BENT CAP.

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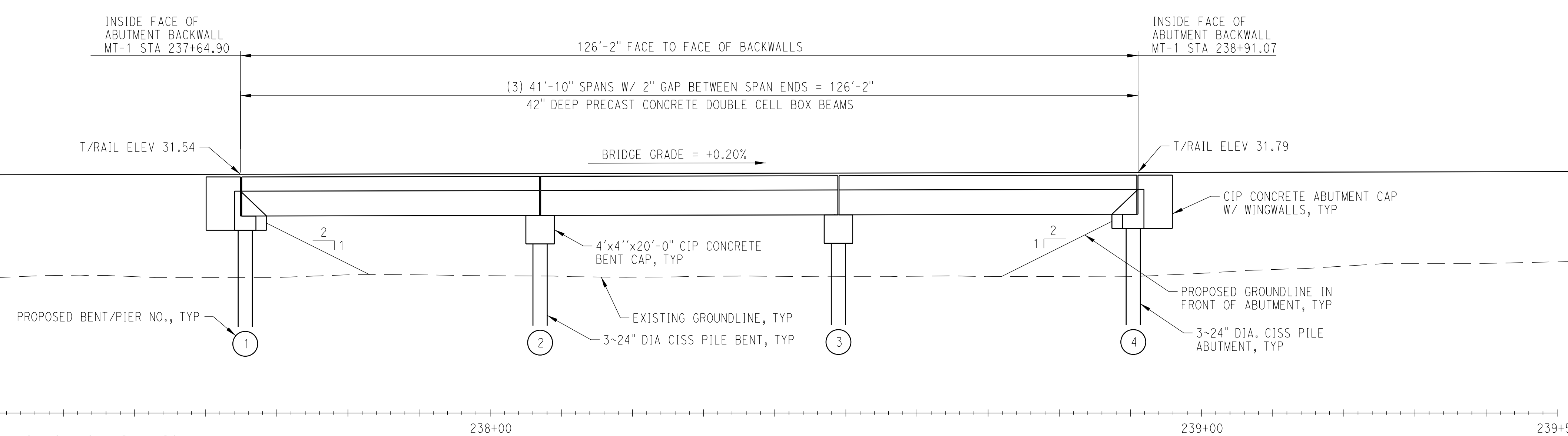

MT-1 ELEVATION

SCALE: 1" = 20'
MT-2 SIMILAR

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DATE OCTOBER 2022

HDR

SANDAG
San Diego's Regional Planning Agency
APPROVED: ______ DATE: ______

10% SUBMITTAL
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR BRIDGE NO. 3 PLAN AND ELEVATION MT-1 STA 237+64.90 TO STA 238+91.07

CONTRACT NO.
DRAWING NO. BR203
REVISION
SHEET NO. 35 of 35
SCALE 1'=20'

Appendix B. Environmental Approach White Paper

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San Dieguito to Sorrento Valley Double Track

Del Mar Tunnels Alternatives Analysis Environmental Approach White Paper

San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study

November 2022

Prepared by:

HDR B·R·G

Prepared for:

SANDAG

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619) 699-1900
www.sandag.org

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Contents

Figures

Acronyms

ACOE	United States Army Corps of Engineers
Caltrans	California Department of Transportation
CEQA	California Environmental Quality Act
EA	Environmental Assessment
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
FONSI	Finding of No Significant Impact
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
LOSSAN	Los Angeles-San Diego-San Luis Obispo
M(ND)	Mitigated Negative Declaration
ND	Negative Declaration
NEPA	National Environmental Policy Act
NWP	Nationwide Permits
ROW	right-of-way
SANDAG	San Diego Association of Governments
SDSVDT	San Dieguito to Sorrento Valley Double Track
STB	Surface Transportation Board
WOUS	waters of the United States

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Summary

The Los Angeles-San Diego-San Luis Obispo (LOSSAN) Rail Corridor passes through the City of Del Mar atop coastal bluffs, and the alignment consists of a single track in this location. An analysis is being prepared of alternatives for the relocation of the alignment from the bluffs to a tunneled double-track alignment between the south side of the San Dieguito Lagoon basin (near the Del Mar Fairgrounds) to the north end of Sorrento Valley. Two alternatives are being recommended for further design, the Revised Camino Del Mar alignment and the Revised Crest Canyon Higher Speed alignment. The purpose of this white paper is to provide a recommended environmental approach to allow the San Diego Association of Governments (SANDAG) to select a course of action for California Environmental Quality Act (CEQA) or National Environmental Policy Act (NEPA) compliance for the tunnel project. This white paper also presents a list of updated or new technical studies that are necessary for the recommended approaches.

In December 2006, the California Department of Transportation (Caltrans) and the Federal Railroad Administration (FRA) completed the Final Program Environmental Impact Report/Environmental Impact Statement (EIR/EIS) for LOSSAN rail corridor improvements. SANDAG was a responsible agency for the EIR. The EIR/EIS programmatically addressed improvements along the corridor, including a proposed tunnel under Del Mar. Any future environmental review will be a subsequent environmental document to this program EIR/EIS.

Three likely federal actions associated with the project would trigger the need to demonstrate compliance with the requirements of the NEPA (42 United States Code [USC] 4321, et. seq.) and the Act's implementing regulations (40 Code of Federal Regulations 1500 et. seq.). These actions are:

- Federal funding by the FRA and/or FTA;
- Certificate issued by the Surface Transportation Board [49 USC 10901 (c)]; and,
- Permit Issued by the U.S. Army Corps of Engineers (USACE) under Section 404 of the Clean Water Act and/or Section 10 of the Rivers and Harbors Act.

SANDAG has preliminarily determined an EIS is the appropriate document for compliance with NEPA. SANDAG should identify a federal lead agency as early as possible to confirm this approach.

The California Environmental Quality Act (CEQA; California Public Resources Code Section 2100 et. seq.) and the Guidelines for Implementation of the California Environmental Quality Act (California Code of Regulations Section 15000 et. seq.) apply to the Project. SANDAG would be the lead agency for compliance with CEQA. SANDAG has determined that an EIR is the appropriate document for compliance with CEQA.

There is one significant risk identified during preparation of this white paper; with regard to the Crest Canyon Alternative, the southern tunnel portal is planned for parcel number 301-341-04. That parcel was protected from development as a condition of Coastal Development Permit Number F8341. Any project approval by the Coastal Commission would likely require protections similar to those required of the prior Coastal Development Permit. For that reason, SANDAG should initiate negotiations with the Coastal Commission to determine the feasibility of using parcel 301-341-04 as a tunnel portal site.

Environmental technical reports/memos recommended to be prepared are listed in this white paper as are likely regulatory permits that would have to be obtained prior to project construction.

1 Introduction

The LOSSAN Rail Corridor passes through the City of Del Mar atop coastal bluffs, and the alignment consists of a single track in this location. The LOSSAN Corridor is a vital component of the San Diego region's transportation network. North County Transit District COASTER commuter service, Amtrak Pacific Surfliner intercity service, and BNSF Railway freight service rely on the corridor to move a combined 7.6 million passengers and $1 billion in goods each year. Bluff erosion and the threat of sea level rise due to climate change pose a threat to the continued safety and reliability of rail operations on the bluffs through Del Mar, and the single-track alignment restricts capacity to increase rail service. Therefore, SANDAG commissioned the San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study (SD-LOSSAN), which will determine a long-term safety and operations solution for the San Diego segment of the LOSSAN Rail Corridor.

The objectives of the project are the following:

- Relocate the tracks through Del Mar from the eroding coastal bluffs to a tunnel
- Encourage rail ridership on the LOSSAN Corridor to reduce vehicle miles traveled and associated greenhouse gas emissions by improving rail service through providing a double-track alignment that enables greater frequency of trains, operation at 110 miles per hour, and avoids delays caused by train meets in the segment
- Remove all or part of the existing railroad berm in Los Peñasquitos Lagoon if it is no longer needed and/or as mitigation for project-related impacts on waters of the U.S. (WOUS) and State of California/Coastal Wetlands

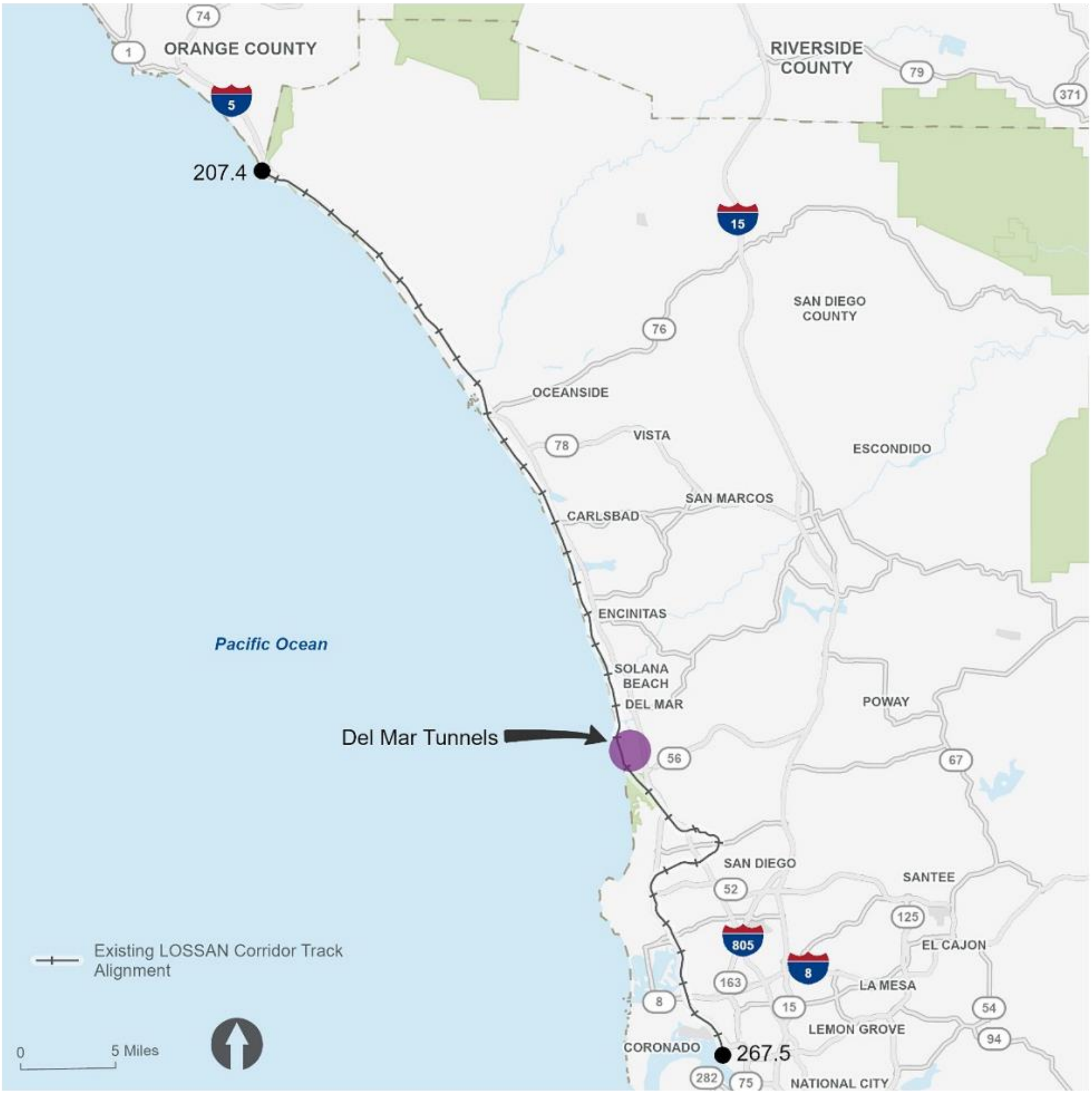
This report documents the analysis of alternatives for the relocation of the alignment from the bluffs to a double-track alignment between the south side of the San Dieguito Lagoon basin (near the Del Mar Fairgrounds) to the north end of Sorrento Valley. Figure 1 shows the regional location of the project.

Removing the tracks from the Del Mar bluffs and double tracking this segment of the corridor directly supports the objectives of SANDAG, the Coastal Commission, North County Transit District, Amtrak, and BNSF Railway by reducing travel times, enhancing safety, reliability and increasing capacity.

The purpose of this project is to provide a long-term solution to the continued safety and viability of the LOSSAN Corridor and the overall economic and environmental health of the San Diego region. As the bluffs recede, the corridor is becoming less viable and the costs to maintain and stabilize the bluffs will continue to increase. Since the year 2000, nearly $15 million (in year of expenditure) has been spent to maintain, stabilize, and repair damaged areas of the bluffs. Bluff stabilization is estimated to cost $100 million in the next decades. The design life of the current stabilization efforts is only 30 years, after that time period, additional stabilization effort will be needed, or the tracks need to be relocated off the bluff area.

If a catastrophic failure were to occur and there was a long-term shutdown, the cost to the region could be in the hundreds of millions of dollars in lost goods movement. It would also increase greenhouse gas output by increasing car and truck traffic on the already congested Interstate 5 highway.

Figure 1. Regional Location



The project's purpose aligns with the U.S. Department of Transportation's *Record of Decision for Los Angeles to San Diego, California (LOSSAN) Proposed Rail Corridor Improvements* issued in February 2009 supporting the Rail Improvements alternative proposed in the Final Program EIR/EIS for LOSSAN Rail Corridor Improvements issued by Caltrans in September 2007. The purpose of the proposed rail improvements to the LOSSAN corridor, as identified in the LOSSAN Program EIR/EIS, is to develop a faster, safer, and more reliable passenger rail system that provides added capacity in response to increased travel demand between Los Angeles, Orange, and San Diego Counties.

The project also supports overall objectives for establishing an integrated passenger rail system, described in the current (2018) California State Rail Plan as the 2040 Vision. The 2040 Vision will allow people to:

- Travel seamlessly across urban, suburban, and rural areas of the state with more trains, more often;
- Save time with significantly faster trips;
- Enjoy the journey on modern, safe, clean, and comfortable trains;
- Glide past traffic congestion on reliable trains and express buses in dedicated lanes;
- Transfer quickly and easily between high-speed, intercity, and regional trains, express buses, and transit at hub stations with coordinated arrivals and departures with significantly reduced wait times; and
- Plan entire door-to-door trips and purchase a single ticket using a streamlined trip-planning portal.

In addition to the policies set forth in the State Rail Plan, as noted in LOSSAN Program EIR/EIS, minimizing impacts on natural resources (e.g., wetlands, wildlife habitat) and human communities are also important objectives of the Caltrans regarding any improvement within the rail corridor.

Five alternatives were considered based on a SANDAG conceptual alignment study completed in 2017. Subsequently, SANDAG's 2021 Regional Plan identified a Del Mar tunnel as a major transportation network improvement with a horizon year of 2035. The current Alternatives Analysis Report recommends the Camino Del Mar and Crest Canyon Higher Speed alternatives, which were then advanced to 10 percent level conceptual engineering. Further analysis was conducted, including consideration of implications for right-of-way (ROW), utilities, grade separations, railroad systems, construction, and environmental impacts. These two alternative alignments are shown on Figure 2 and Figure 3 and are referred to as the Revised Camino Del Mar and Revised Crest Canyon Higher Speed alternatives, respectively.

Figure 2. Revised Camino Del Mar Alignment

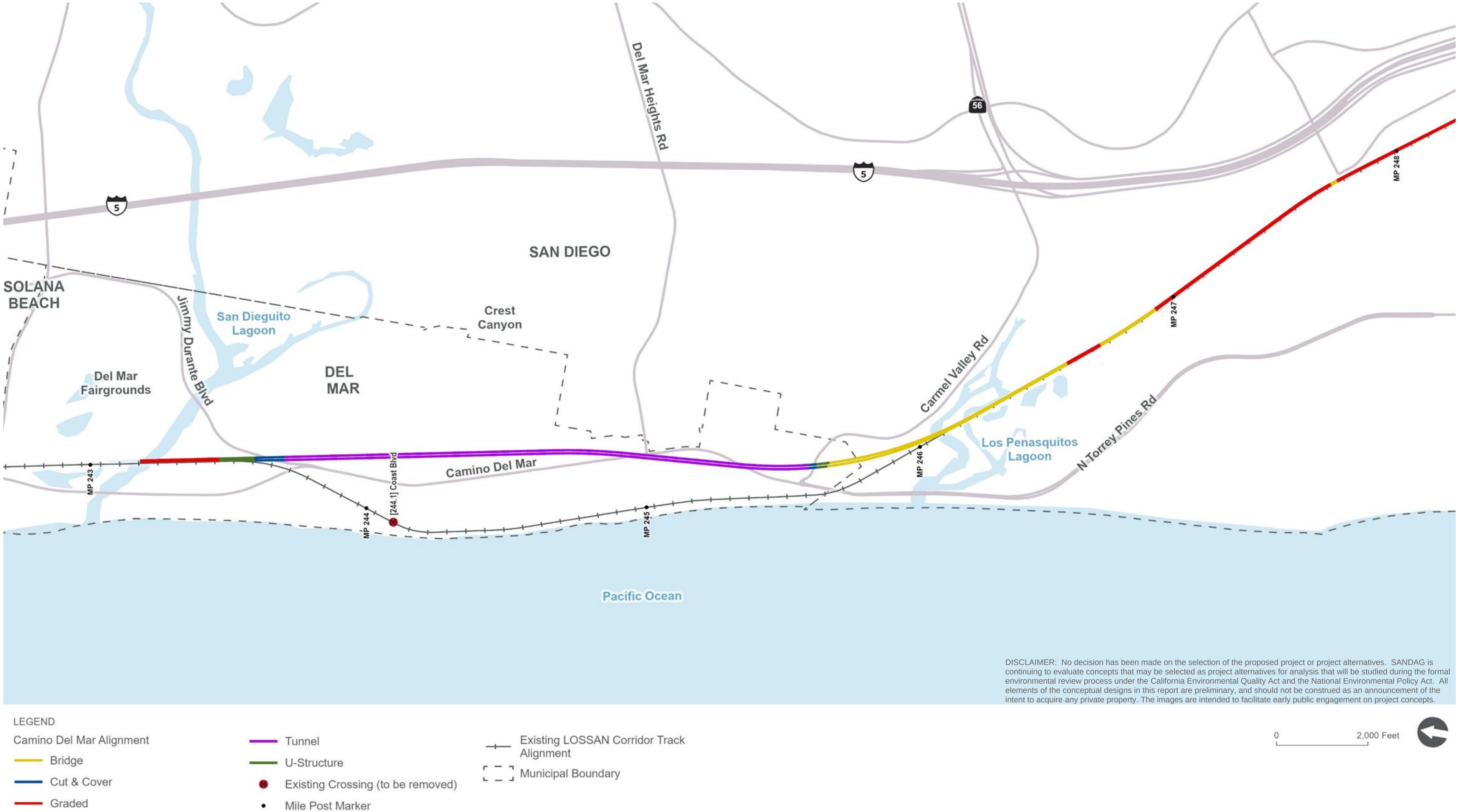
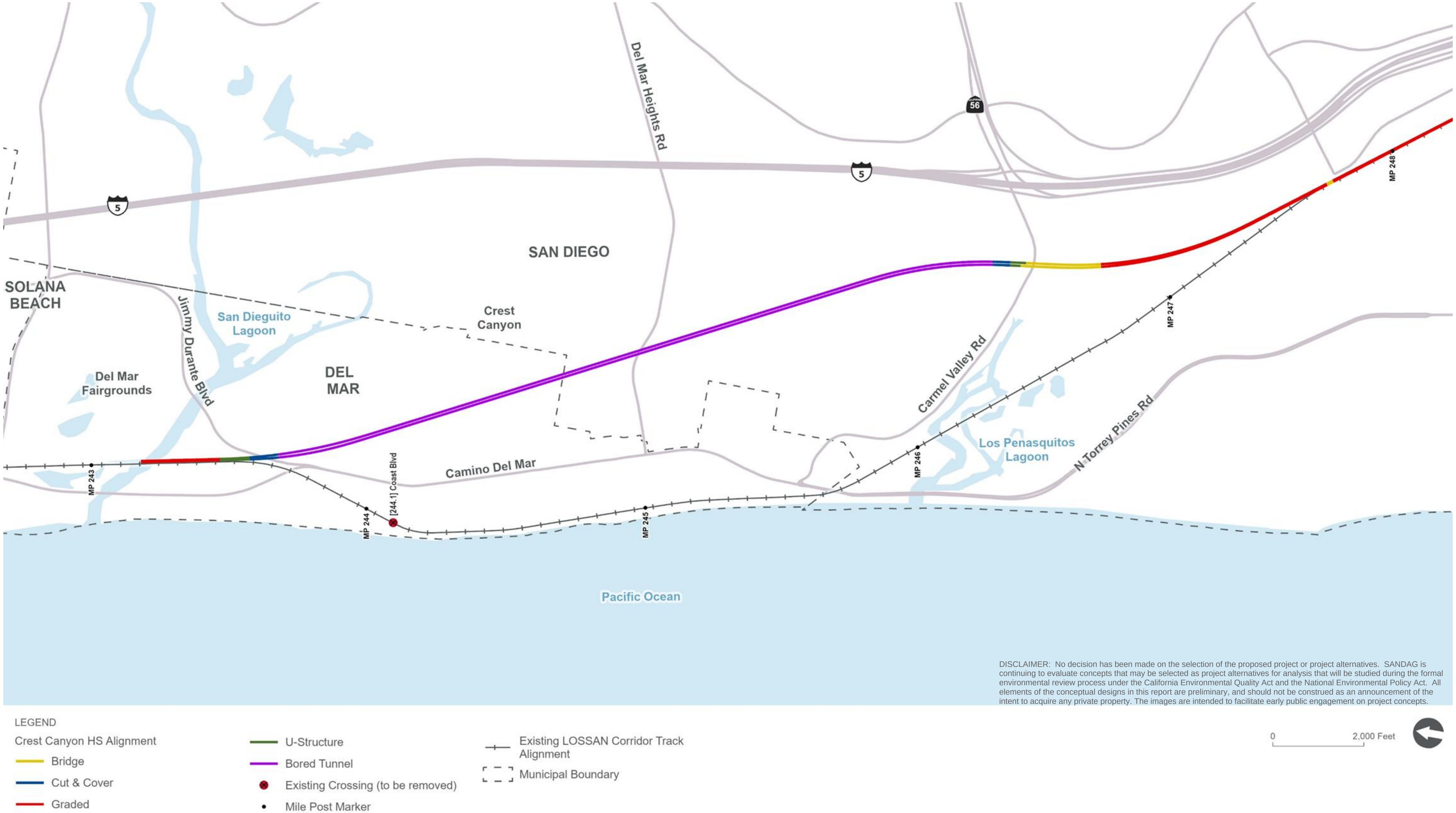


Figure 3. Revised Crest Canyon Higher Speed Alignment



It is recommended that both alignments be carried forward into preliminary environmental review for further evaluation and selection of a preferred alternative. As the alternatives are advanced beyond 10 percent conceptual design, potential cost savings, project delivery methods, and construction phasing should be analyzed further. The Revised Crest Canyon Higher Speed alternative southern portal parcel (301-341-04) was protected from development as a condition of Coastal Development Permit Number F8341. Any project approval by the Coastal Commission likely would require protections similar to those required of the prior Coastal Development Permit. It is recommended that the Revised Canyon Crest Higher Speed Alternative southern portal site and an alternative southern portal site be carried forward into Preliminary Environmental Review. The Alternative Southern Portal Site identified in Section 4.6 of the Alternatives Analysis Report is a good candidate to be carried forward.

The purpose of this white paper is to provide a recommended environmental approach for the narrowed list of tunnel alternatives to allow SANDAG to select a course of action for CEQA/NEPA compliance. This white paper also presents a list of updated or new technical studies that will be necessary for the recommended approaches.

1.1 Preliminary Environmental Recommendations

1.1.1 Previous Environmental Review

In December 2006, Caltrans and FRA completed the Final Program EIR/EIS for the Los Angeles to San Diego Rail Corridor Improvements. SANDAG was a responsible agency for the EIR. The EIR/EIS programmatically addressed improvements along the corridor, including a proposed tunnel under Del Mar. Two tunnel alternatives were considered, a tunnel under Camino Del Mar (Low-Build Rail Improvements Alternative) and a tunnel along Interstate 5 (High-Build Rail Improvements Alterative). FRA and Caltrans approved the Rail Improvements Alternatives as the preferred project/action. The current Alternatives Analysis Report is being prepared as the next step in implementing the Rail Improvements Alternative in the Del Mar area. Any future environmental review will be a subsequent environmental document to this program EIR/EIS.

1.1.2 National Environmental Policy Act

Three potential federal actions associated with the project would trigger the need to demonstrate compliance with the requirements of NEPA (42 U.S. Code [USC] 4321, et. seq.) and NEPA implementing regulations (40 Code of Federal Regulations 1500 et. seq.). These actions are:

- Federal funding by FRA and/or FTA;
- Certificate issued by the STB (49 USC 10901 (c)); and,
- Permit Issued by ACOE under Section 404 of the Clean Water Act and/or Section 10 of the Rivers and Harbors Act for impacts to WOUS.

1.1.3 Federal Railroad Administration and Federal Transit Administration

FRA and FTA recognize classes of actions that normally require an EIS. Actions likely associated with the SDSVDT project may fall under 23 USC 771.115 (a)(3): construction or extension of a fixed transit facility (e.g., rapid rail, light rail, commuter rail, bus rapid transit) that will not be located primarily within an existing transportation ROW.

The SDSVDT project may fall under 23 USC 771.116 (b) (12): minor rail line additions, including construction of side tracks, passing tracks, crossovers, short connections between existing rail lines, and new tracks within existing rail yards or ROW, provided that such additions are not inconsistent with existing zoning, do not involve acquisition of a significant amount of ROW, and do not significantly alter the traffic density characteristics of the existing rail lines or rail facilities.

From a federal perspective, the project could be deemed minor by FRA and/or FTA and a categorical exclusion with technical studies to demonstrate that the project would not have any significant environmental impacts would be required. With regard to consistency with existing zoning, it is beyond the scope of this white paper to perform a formal zoning analysis of potentially affected properties. It is recommended that consistency with local zoning be investigated during subsequent preliminary environmental review.

Surface Transportation Board

Similarly, the likely STB action appears to fall under 49 USC 1105.6 (a) and (b):

a. EISs will normally be prepared for rail construction proposals other than those described in paragraph (b)(1) of this section.
b. EAs will normally be prepared for the following proposed actions:
    (1) Construction of connecting track within existing rail rights-of-way, or on land owned by the connecting railroads.

Since the project would be outside of the existing rail ROW and the land is not owned by a railroad, it would appear an EIS is the appropriate document for NEPA compliance by STB.

In practice, STB recognizes build-in types of projects and that are considered minor actions. These actions require preparation of an EA. The SDSVDT project may be viewed as such a project by STB.

In the event the existing railroad ROW through Los Peñasquitos Lagoon is abandoned (fully or partially), STB regulations recognize abandonment actions as normally requiring EAs (49 USC 1105.6 (b)(2)).

Army Corps of Engineers

ACOE's NEPA guidance is given in the document *Permitting/Regulatory Guidance - Guide/Handbook: United States Army Corps of Engineers - Procedures for Implementing the National Environmental Policy Act* (ER 200-2-2). The ACOE NEPA procedures are included in the Regulatory and Permitting Information Desktop Toolkit maintained by the U.S. Department of Energy. They can be found at the following link: https://openei.org/wiki/RAPID/Roadmap/9-FD-k, which is current as of October 2022.

Section 9.6 of the Toolkit identifies that certain actions normally require the preparation of an EA but not necessarily an EIS. Such actions include regulatory actions (most permits will normally require only an EA).

An EA was the NEPA document prepared by ACOE for the Santa Margarita River Bridge Replacement and Second Track Project that had somewhat similar impacts to WOUS.

ACOE's Nationwide Permit Program and their Nationwide Permits (NWP) authorize certain activities under Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act of 1899 provided all conditions can be met by the project. ACOE complies with NEPA when adopting the

Nationwide Permit Program every 5 years; therefore, no further ACOE NEPA compliance would be required for projects that qualify for the NWP program.

There are 54 NWPs and number 14 (Linear Transportation Projects) and number 33 (Temporary Construction Access and Dewatering) are the two NWPS most applicable to the SDSVDT project where it discharges to WOUS. NWP 14 is only valid for a project that does not result in the loss of greater than 1/3 of an acre of tidal WOUS. The current proposal is to remove enough of the existing railroad berm from Los Peñasquitos Lagoon such that the net loss of tidal WOUS is less than zero. This approach has been successfully used by other LOSSAN projects that qualified under NWP 14, most recently including the San Dieguito Lagoon Double Track Project.

NEPA Recommendation

SANDAG has preliminarily determined an EIS is the appropriate document for compliance with NEPA. SANDAG should identify a federal lead agency as early as possible to confirm this approach.

1.1.4 California Environmental Quality Act

CEQA (California Public Resources Code Section 2100 et. seq.) and the Guidelines for Implementation of CEQA (California Code of Regulations Section 15000 et. seq.) apply to discretionary actions taken by the State of California and local governments in California.

CEQA applies to the SDSVDT project because it is a discretionary action that is considered a project under CEQA. The project likely would not qualify for a statutory or categorical exemption from CEQA. CEQA compliance would need to be obtained either through preparation of an (M)ND or an EIR.

CEQA Recommendation

The California Environmental Quality Act (CEQA; California Public Resources Code Section 2100 et. seq.) and the Guidelines for Implementation of the California Environmental Quality Act (California Code of Regulations Section 15000 et. seq.) apply to the Project. SANDAG would be the lead agency for compliance with CEQA. SANDAG has determined that an EIR is the appropriate document for compliance with CEQA.

1.2 Schedule (Assumes Joint Environmental Impact Report/ Environmental Impact Statement)

Technical Reports	Q1 2023 to Q2 2024
Notice of Preparation/Intent	Q2 2024
Draft EIR/EIS	Q2 2024 to Q2 2025
Public Review	Q3 2025
Final EIR/EIS	Q4 2026
Permitting	Q4 2025 to Q4 2027

1.3 Required Environmental Technical Reports

The scope of environmental review will depend ultimately on the project and alternatives to be evaluated under NEPA and/or CEQA. Both NEPA and CEQA require scoping early in the process to identify necessary areas of study and impact analysis. This white paper recommends SANDAG proceed with environmental technical report preparation while in discussion with potential federal action agencies. NEPA and CEQA required scoping should occur following identification of the NEPA lead agency and upon determination as to the applicability of CEQA.

It is recommended that none of the prior technical reports be updated. The analysis in the programmatic EIR/EIS is 15+ years old and was also programmatic in nature. For this reason, we recommend preparing new technical reports/memos.

The following technical reports or memorandums should be prepared to support a joint NEPA/CEQA document (list derived from the LOSSAN EIR/EIS environmental review topics and updated to include Social Justice and other CEQA initial study checklist topics):

- Air Quality
- Greenhouse Gas Impact Analysis
- Social and Environmental Justice
- Land Use and Planning, Communities and Neighborhoods, and Property
- Traffic and Circulation
- Travel Conditions
- Noise and Vibration
- Energy
- Public Utilities, Services and Facilities
- Public Health and Safety
- Biological Resources and Wetlands
- Hazardous Materials and Waste
- Hydrology and Water Resources
- Cultural and Paleontological Resources
- Aesthetics and Visual Resources
- Tribal Cultural Resources
- Prime and Unique Farmlands (Memo)
- Section 4(f) and 6(f) (Public Parks and Recreation)
- Mineral Resources (Memo)
- Geology and Soils
- Wildfire (Memo)
- Population/Housing (Memo)

The 2017 *Conceptual Engineering and Environmental Constraints for Double Track Alignment Alternatives Between Del Mar Fairgrounds and Sorrento Valley* (2017 Alternatives Study) was supported by one technical report for Cultural Resources. That report was not a full technical report as required by NEPA and CEQA. The report was based on a 2014 records search that is now seven your old. As such, a new record search and full technical report is required. A preliminary geotechnical evaluation was also prepared; however, it was not a full technical report either. No other technical reports were prepared for the 2017 Alternatives Study.

The current *Final Del Mar Tunnels Alternatives Analysis Report -San Dieguito to Sorrento Valley Double Trac*k is supported by a preliminary drainage report, geotechnical data and reconnaissance report and a noise and vibration technical report. These reports include useful information, but they were based on conceptual engineering and do not meet the requirements for project specific technical report.

With regard to the Canyon Crest Alternative, the southern tunnel portal is planned for parcel number 301-341-04. That parcel was protected from development as a condition of Coastal Development Permit No. F8341. Any project approval by the Coastal Commission likely would require protections similar to those required of the prior Coastal Development Permit. For that reason, SANDAG should initiate negotiations with the Coastal Commission to determine the feasibility of using parcel 301-341-04 as a tunnel portal site.

1.4 Regulatory Permitting

The following permits likely would be required because all project alternatives would involve placement of fill in water of the United States, occur in the Coastal Zone, have the potential to affect threatened and endangered species, and have the potential to affect important cultural resources:

- Clean Water Act Section 401 Water Quality Certification;
- Clean Water Act Section 404 Fill Permit;
- Rivers and Harbors Act Section 10 Permit (processed with the 404 permit);
- Coastal Zone Management Act Federal Consistency Certification;
- Endangered Species Act Section 7 Consultation;
- Magnuson-Stevens Fishery Conservation and Management Act Consultation; and
- National Historic Preservation Act Section 106 Consultation.

With regard to the Coastal Zone Management Act Federal Consistency Certification, SANDAG and Caltrans prepared a Public Works Plan/Transportation Resource Enhancement Plan for the North Coast Corridor. The NCC is a blueprint for implementing a $6-billion 40-year program of rail, highway, transit, bicycle, pedestrian, and coastal resource improvements that span 27 miles of the Northern San Diego County coastline, from La Jolla to Oceanside.

The Public Works Plan/Transportation Resource Enhancement Plan allows these improvements to be analyzed as an integrated system and to optimize the suite of improvements so that transportation goals are met in a manner that maintains and enhances public access to coastal resources and recreational facilities and sensitive coastal resources are protected and enhanced wherever feasible. The Public Works Plan/Transportation Resource Enhancement Plan also serves as the regulatory document that provides a comprehensive mechanism for conducting a federal consistency review under the Coastal Zone Management Act for all of the North Coast Corridor improvements, and for coastal development permitting and processing of applicable local coastal program amendments pursuant to the Coastal Act for those elements of the Public Works Plan/Transportation Resource Enhancement Plan subject to Public Works Plan requirements. In addition, the Public Works Plan/Transportation Resource Enhancement Plan links and identifies mitigation measures for project elements within lagoon areas that are subject solely to the Coastal Commission's coastal development permit review process.

Authority for a Public Works Plans is provided under California Code of Regulations Title 14, Chapter 7, Subchapter 2, Public Works Plans. The Del Mar tunnel Camino Del Mar and Interstate 5/Peñasquitos alignments are addressed in the Public Works Plans. The Public Works Plans provides as follows: *Given the program level of detail available for rail projects that the PWP/TREP indicates will be handled solely through federal consistency review, it is expected that federal consistency review for such rail improvements will be conducted in a phased manner. Similarly, rail projects that may be processed through the PWP may be subject to future PWP amendment and NOIDs to ensure consistency with the approved PWP; SANDAG/Caltrans may choose (in consultation with the Coastal Commission) to submit a coastal development permit application to the appropriate permitting agency.*

Appendix C. Utility Conflict Matrix

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Camino Del Mar - North
Utility Conflict Matrix

ITEM	UTILITY DESCRIPTION	UTILITY TYPE	UTILITY OWNER	DEPTH	LOCATION	STATION	DATA SOURCE	POTENTIAL CONFLICT	DISPOSITION	
									PIP/RELOCATE/ ENCASE	BY
1	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	JIMMY DURANTE BVD	15+25 TO 22+00 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
2	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	JIMMY DURANTE BVD	22+00 TO 26+00 (JDB)	SDGE ELECTRIC ASSET MAP	FILL OVER CUT & COVER SECTION	RELOCATE	UTILITY
3	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	LUZON AVE	28+50 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
4	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	JIMMY DURANTE BVD	22+75 (JDB)	SDGE ELECTRIC ASSET MAP	CUT & COVER SECTION	RELOCATE	UTILITY
5	UG ELECTRIC	ELECTRIC	SDGE	TBD	JIMMY DURANTE BVD	20+00 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
6	UG ELECTRIC	ELECTRIC	SDGE	TBD	JIMMY DURANTE BVD	21+25 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
7	OVERHEAD ELECTRIC	ELECTRIC	SDGE	TBD	DAVID WAY	18+50 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
8	AT&T CONDUIT	COMMUNICATIONS	AT&T	TBD	JIMMY DURANTE BLVD	15+25 TO 19+00 (JDB)	AT&T DISTRIBUTION INDEX MAP	FILL	RELOCATE	UTILITY
9	AT&T UNDERGROUND	COMMUNICATIONS	AT&T	TBD	JIMMY DURANTE BLVD	15+25 TO 19+00 (JDB)	AT&T DISTRIBUTION INDEX MAP	FILL	RELOCATE	UTILITY
10	AT&T UNDERGROUND	COMMUNICATIONS	AT&T	TBD	JIMMY DURANTE BLVD	18+50 (JDB)	AT&T DISTRIBUTION INDEX MAP	PERMANENT FACILITIES	RELOCATE	UTILITY
11	AT&T CONDUIT	COMMUNICATIONS	AT&T	TBD	SOUTH SIDE OF JIMMY DURANTE BLVD	23+50	AT&T DISTRIBUTION INDEX MAP	U-STRUCTURE	RELOCATE	UTILITY
12	AT&T UNDERGROUND	COMMUNICATIONS	AT&T	TBD	SOUTH SIDE OF JIMMY DURANTE BLVD	23+50	AT&T DISTRIBUTION INDEX MAP	U-STRUCTURE	RELOCATE	UTILITY
13	CHARTER AERIAL plus 3 LATERALS	COMMUNICATIONS	CHARTER SPECTRUM	AERIAL	JIMMY DURANTE BLVD	14+50 TO 22+00 (JDB)	CHARTER ASSET MAP	FILL	RELOCATE	UTILIITY
14	CHARTER AERIAL	COMMUNICATIONS	CHARTER SPECTRUM	AERIAL	JIMMY DURANTE BLVD	22+00 TO 26+00 (JDB)	CHARTER ASSET MAP	FILL OVER CUT & COVER SECTION	RELOCATE	UTILIITY
15	CHARTER AERIAL	COMMUNICATIONS	CHARTER SPECTRUM	AERIAL	JIMMY DURANTE BLVD	26+00 TO 28+50 (JDB)	CHARTER ASSET MAP	FILL	RELOCATE	UTILIITY
16	CHARTER UNDERGROUND	COMMUNICATIONS	CHARTER SPECTRUM	TBD	JIMMY DURANTE BLVD	18+50 (JDB)	CHARTER ASSET MAP	PERMANENT FACILITIES	RELOCATE	UTILITY
17	GAS (1 1/4" PE, 2" PE)	GAS	SDGE	TBD	JIMMY DURANTE BLVD	16+50 TO 20+50 (JDB)	SDGE GAS ASSET MAP	FILL	RELOCATE	UTILITY
18	GAS (2" PE)	GAS	SDGE	TBD	DAVID WAY	18+50 (JDB)	SDGE GAS ASSET MAP	FILL	RELOCATE	UTILITY
19	GAS (1 1/4" PE)	GAS	SDGE	TBD	JIMMY DURANTE BLVD	17+00 (JDB)	SDGE GAS ASSET MAP	PERMANENT FACILITIES	RELOCATE	UTILITY
20	GAS (1 1/4" PE)	GAS	SDGE	TBD	JIMMY DURANTE BLVD	25+25	SDGE GAS ASSET MAP	PERMANENT FACILITIES & U-STRUCTURE	RELOCATE	UTILITY
21	UG COMMUNICATIONS	COMMUNICATIONS	MCI-VERIZON	TBD	JIMMY DURANTE BLVD	15+25 TO 22+00 (JDB)	MCI ASSET MAP	FILL	RELOCATE	UTILITY
22	UG COMMUNICATIONS	COMMUNICATIONS	MCI-VERIZON	TBD	JIMMY DURANTE BLVD	22+00 TO 26+00 (JDB)	MCI ASSET MAP	FILL OVER CUT & COVER SECTION	RELOCATE	UTILITY
23	UG COMMUNICATIONS	COMMUNICATIONS	MCI-VERIZON	TBD	JIMMY DURANTE BLVD	26+00 TO 28+50 (JDB)	MCI ASSET MAP	FILL	PIP	UTILITY

Camino Del Mar - North
Utility Conflict Matrix

ITEM	UTILITY DESCRIPTION	UTILITY TYPE	UTILITY OWNER	DEPTH	LOCATION	STATION	DATA SOURCE	POTENTIAL CONFLICT	DISPOSITION	
									PIP/RELOCATE/ ENCASE	BY
24	30" RCP STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	5	JIMMY DURANTE BLVD	17+00 (JDB)	AS-BUILT (E-85-002-2)	FILL & PERMANENT FACILITIES	REPLACE	PROJECT
25	24" PVC STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	4'	LUZON AVE	27+00 TO 28+25 (JDB)	AS-BUILT (E-90-004-3)	FILL	PIP	PROJECT
26	24" RCP STORM DRAIN (CEMENT SLURRY BACKFILL)	STORM DRAIN	CITY OF DEL MAR	5	JIMMY DURANTE BLVD	15+25 TO 22+00 (JDB)	AS-BUILT (E-91-004-4)	FILL	PIP	PROJECT
27	24" RCP STORM DRAIN (CEMENT SLURRY BACKFILL)	STORM DRAIN	CITY OF DEL MAR	5	JIMMY DURANTE BLVD	22+00 TO 23+50 (JDB)	AS-BUILT (E-91-004-4)	FILL OVER CUT & COVER SECTION	RELOCATE	PROJECT
28	DRAINAGE CHANNEL	STORM DRAIN	NCTD	SURFACE	21ST STREET	26+50	CITY OF DEL MAR ASSET MAP	FILL OVER CUT & COVER SECTION	RELOCATE	PROJECT
29	12" VCP SEWER	SEWER	CITY OF DEL MAR	6' - 8'	JIMMY DURANTE BLVD	16+75 TO 20+00 (JDB)	AS-BUILT (E-80-004-01)	FILL & PERMANENT FACILITIES	TBD	PROJECT
30	6" SEWER	SEWER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	21+25 (JDB)	(E-78-003-02) WASTEWATER SYSTEMS MAP	FILL OVER CUT & COVER SECTION	TBD	PROJECT
31	12" PVC SEWER	SEWER	CITY OF DEL MAR	2'-8'	JIMMY DURANTE BLVD	24+75 (JDB)	AS-BUILT (E-86-001-09)	FILL OVER CUT & COVER SECTION	TBD	PROJECT
32	8" PVC SEWER	SEWER	CITY OF DEL MAR	2'-4'	JIMMY DURANTE BLVD	15+50 TO 20+00 (JDB)	AS-BUILT (E-15-005-25) (E-56-002-01)	FILL	TBD	PROJECT
33	8" SEWER	SEWER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	20+00 TO 22+00 (JDB)	(E-56-002-01) WASTEWATER SYSTEMS MAP	FILL	TBD	PROJECT
34	8" SEWER	SEWER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	22+00 TO 26+50 (JDB)	(E-60-009-02) WASTEWATER SYSTEMS MAP	FILL OVER CUT & COVER SECTION	TBD	PROJECT
35	8" VCP SEWER	SEWER	CITY OF DEL MAR	8'	DAVID WAY	18+50 (JDB)	AS BUILT (78-003-02)	FILL	RELOCATE	PROJECT
36	8" DIP & 6" PVC WATER	WATER	CITY OF DEL MAR	3'-4'	JIMMY DURANTE BLVD	15+25 TO 18+50 (JDB)	AS-BUILT (E-15-005-21)	FILL	TBD	PROJECT
37	12"FPVC WATER IN 20" STEEL CASING	WATER	CITY OF DEL MAR	3' MIN, TBD	JIMMY DURANTE BLVD	15+25 TO 22+00 (JDB)	AS-BUILT (E-15-005-21)	FILL	TBD	PROJECT
38	12"FPVC WATER IN 20" STEEL CASING	WATER	CITY OF DEL MAR	3' MIN, TBD	JIMMY DURANTE BLVD	22+00 TO 26+50 (JDB)	AS-BUILT (E-15-005-22/23)	FILL OVER CUT & COVER SECTION	TBD	PROJECT
39	12"FPVC WATER IN 20" STEEL CASING	WATER	CITY OF DEL MAR	3' MIN, TBD	JIMMY DURANTE BLVD	26+50 TO 28+00 (JDB)	AS-BUILT (E-15-005-22/23)	FILL	TBD	PROJECT
40	12" PVC WATER	WATER	CITY OF DEL MAR	4'	JIMMY DURANTE BLVD	28+00 TO 28+50 (JDB)	AS-BUILT (E-95-003-04)	FILL	TBD	PROJECT
41	6" ACP - ABANDONED	WATER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	20+00 TO 22+00 (JDB)	AS-BUILT (E-15-005-22)	FILL	ABANDON	PROJECT

Camino Del Mar - South
Utility Conflict Matrix

ITEM	UTILITY DESCRIPTION	UTILITY TYPE	UTILITY OWNER	DEPTH	LOCATION	STATION	DATA SOURCE	POTENTIAL CONFLICT	DISPOSITION	
									PIP/RELOCATE/ ENCASE	BY
1	10-INCH ACP WATER	WATER	CITY OF DEL MAR	3' min.	CARMEL VALLEY ROAD	128+25	AS-BUILT (E-80-001-04)	CUT & COVER SECTION	RELOCATE	PROJECT
2	6-INCH VCP SEWER (ABANDONED)	SEWER	CITY OF DEL MAR	12'-14'	CARMEL VALLEY ROAD	128+70	AS-BUILT (E-80-001-04)	CUT & COVER SECTION	REMOVE	PROJECT
3	8-INCH VCP SEWER	SEWER	CITY OF SAN DIEGO	6'	CARMEL VALLEY ROAD	128+30	AS-BUILT (E-80-001-04)-REFERENCED	CUT & COVER SECTION	RELOCATE	PROJECT
4	8-INCH VCP SEWER	SEWER	CITY OF SAN DIEGO	TBD	CARMEL VALLEY ROAD	128+60	CITY OF DEL MAR UTILITY ASSET MAP	CUT & COVER SECTION	RELOCATE	PROJECT
5	12-INCH PVC SEWER FORCE MAIN	SEWER	CITY OF DEL MAR	10'	CARMEL VALLEY ROAD	128+80	AS-BUILT (E-72-008-9)	CUT & COVER SECTION	RELOCATE	PROJECT
6	12-INCH RCP STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	5' TO DAYLIGHT	CARMEL VALLEY ROAD	130+00	AS-BUILT (E-80-001-04)	FILL OVER CUT & COVER SECTION	RELOCATE	PROJECT
7	24" RCP STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	4'-6.5'	CARMEL VALLEY ROAD	130+00	AS-BUILT (E-80-001-04)	FILL OVER CUT & COVER SECTION	RELOCATE	PROJECT
8	24" RCP STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	6'-9'	TORREY POINT RD CDS	126+00	AS-BUILT (E-80-001-07)	PERMANENT FACILITIES AREA	RELOCATE	PROJECT
9	CONDUIT (UG TELEPHONE)	COMMUNICATIONS	AT&T	TBD	CARMEL VALLEY ROAD	TBD	AT&T DISTRIBUTION INDEX MAP	CUT & COVER SECTION	RELOCATE	UTILITY
10	CONDUIT (UG TELEPHONE)	COMMUNICATIONS	AT&T	TBD	CARMEL VALLEY ROAD	130+50	AT&T DISTRIBUTION INDEX MAP	CONSTRUCTION STAGING AREA	PIP	PROJECT
11	4" GAS	GAS	SDGE	TBD	CARMEL VALLEY ROAD	TBD	SDGE GAS ASSET MAP (15600-119570)	CUT & COVER SECTION	RELOCATE	UTILITY
12	3/4" GAS	GAS	SDGE	TBD	CARMEL VALLEY ROAD	130+50	SDGE GAS ASSET MAP (15607-119570)	CONSTRUCTION STAGING AREA	PIP	PROJECT
13	PRIMARY UG ELECTRIC	ELECTRIC	SDGE	TBD	CARMEL VALLEY ROAD	130+50	SDGE GAS ASSET MAP (15607-119565)	CONSTRUCTION STAGING AREA	RELOCATE	UTILITY
14	SECONDARY UG ELECTRIC	ELECTRIC	SDGE	TBD	CARMEL VALLEY ROAD	130+50	SDGE GAS ASSET MAP (15607-119570)	CONSTRUCTION STAGING AREA	PIP	PROJECT
15	SECONDARY UG ELECTRIC	ELECTRIC	SDGE	TBD	TORREY POINT ROAD	126+00	SDGE GAS ASSET MAP (15607-119575)	CONSTRUCTION STAGING AREA	PIP	PROJECT

Crest Canyon Higher Speed - North
Utility Conflict Matrix

ITEM	UTILITY DESCRIPTION	UTILITY TYPE	UTILITY OWNER	DEPTH	LOCATION	STATION	DATA SOURCE	POTENTIAL CONFLICT	DISPOSITION	
									PIP/RELOCATE/ ENCASE	BY
1	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	JIMMY DURANTE BVD	15+25 TO 21+25 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
2	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	JIMMY DURANTE BVD	21+25 TO 24+50 (JBD)	SDGE ELECTRIC ASSET MAP	FILL OVER CUT & COVER SECTION	RELOCATE	UTILITY
3	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	JIMMY DURANTE BVD	24+50 TO 28+50 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
4	OVERHEAD ELECTRIC	ELECTRIC	SDGE	AERIAL	JIMMY DURANTE BVD	22+75 (JDB)	SDGE ELECTRIC ASSET MAP	CUT & COVER SECTION	RELOCATE	UTILITY
5	UG ELECTRIC	ELECTRIC	SDGE	TBD	JIMMY DURANTE BVD	20+00 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
6	UG ELECTRIC	ELECTRIC	SDGE	TBD	JIMMY DURANTE BVD	21+25 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
7	OVERHEAD ELECTRIC	ELECTRIC	SDGE	TBD	DAVID WAY	18+50 (JDB)	SDGE ELECTRIC ASSET MAP	FILL	RELOCATE	UTILITY
8	AT&T CONDUIT	COMMUNICATIONS	AT&T	TBD	JIMMY DURANTE BLVD	15+25 TO 19+00 (JDB)	AT&T DISTRIBUTION INDEX MAP	FILL	RELOCATE	UTILITY
9	AT&T UNDERGROUND	COMMUNICATIONS	AT&T	TBD	JIMMY DURANTE BLVD	15+25 TO 19+00 (JDB)	AT&T DISTRIBUTION INDEX MAP	FILL	RELOCATE	UTILITY
10	AT&T UNDERGROUND	COMMUNICATIONS	AT&T	TBD	JIMMY DURANTE BLVD	18+50 (JDB)	AT&T DISTRIBUTION INDEX MAP	PERMANENT FACILITIES	RELOCATE	UTILITY
11	AT&T CONDUIT	COMMUNICATIONS	AT&T	TBD	SOUTH SIDE OF JIMMY DURANTE BLVD	23+50	AT&T DISTRIBUTION INDEX MAP	U-STRUCTURE	RELOCATE	UTILITY
12	AT&T UNDERGROUND	COMMUNICATIONS	AT&T	TBD	SOUTH SIDE OF JIMMY DURANTE BLVD	23+50	AT&T DISTRIBUTION INDEX MAP	U-STRUCTURE	RELOCATE	UTILITY
13	CHARTER AERIAL plus 3 LATERALS	COMMUNICATIONS	CHARTER SPECTRUM	AERIAL	JIMMY DURANTE BLVD	15+25 TO 21+25 (JDB)	CHARTER ASSET MAP	FILL	RELOCATE	UTILIITY
14	CHARTER AERIAL plus 1 LATERAL	COMMUNICATIONS	CHARTER SPECTRUM	AERIAL	JIMMY DURANTE BLVD	21+25 TO 24+50 (JDB)	CHARTER ASSET MAP	FILL OVER CUT & COVER SECTION	RELOCATE	UTILIITY
15	CHARTER AERIAL plus 2 LATERALS	COMMUNICATIONS	CHARTER SPECTRUM	AERIAL	JIMMY DURANTE BLVD	24+50 TO 28+50 (JDB)	CHARTER ASSET MAP	FILL	RELOCATE	UTILIITY
16	CHARTER UNDERGROUND	COMMUNICATIONS	CHARTER SPECTRUM	TBD	JIMMY DURANTE BLVD	18+50 (JDB)	CHARTER ASSET MAP	PERMANENT FACILITIES	RELOCATE	UTILIITY
17	GAS (1 1/4" PE, 2" PE)	GAS	SDGE	TBD	JIMMY DURANTE BLVD	16+50 TO 20+50 (JDB)	SDGE GAS ASSET MAP	FILL	RELOCATE	UTILITY
18	GAS (2" PE)	GAS	SDGE	TBD	DAVID WAY	18+50 (JDB)	SDGE GAS ASSET MAP	FILL	RELOCATE	UTILITY
19	GAS (1 1/4" PE)	GAS	SDGE	TBD	JIMMY DURANTE BLVD	17+00 (JDB)	SDGE GAS ASSET MAP	PERMANENT FACILITIES	RELOCATE	UTILITY
20	GAS (1 1/4" PE)	GAS	SDGE	TBD	JIMMY DURANTE BLVD	25+25	SDGE GAS ASSET MAP	CUT & COVER SECTION	RELOCATE	UTILITY
21	MCI COMMUNICATIONS	COMMUNICATIONS	MCI	TBD	JIMMY DURANTE BLVD	15+25 TO 21+25 (JDB)	MCI ASSET MAP	FILL	RELOCATE	UTILIITY
22	MCI COMMUNICATIONS	COMMUNICATIONS	MCI	TBD	JIMMY DURANTE BLVD	21+25 TO 24+50 (JDB)	MCI ASSET MAP	FILL OVER CUT & COVER SECTION	RELOCATE	UTILIITY

Crest Canyon Higher Speed - North
Utility Conflict Matrix

ITEM	UTILITY DESCRIPTION	UTILITY TYPE	UTILITY OWNER	DEPTH	LOCATION	STATION	DATA SOURCE	POTENTIAL CONFLICT	DISPOSITION	
									PIP/RELOCATE/ ENCASE	BY
23	MCI COMMUNICATIONS	COMMUNICATIONS	MCI	TBD	JIMMY DURANTE BLVD	24+50 TO 28+50 (JDB)	MCI ASSET MAP	FILL	RELOCATE	UTILIITY
24	30" RCP STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	5	JIMMY DURANTE BLVD	17+00 (JDB)	AS-BUILT (E-85-002-2)	FILL & PERMANENT FACILITIES	REPLACE	PROJECT
25	24" RCP STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	4'	LUZON AVE	27+00 TO 28+25 (JDB)	AS-BUILT(E-85-005-5), (E-90-004-3)	FILL	PIP	PROJECT
26	24" RCP STORM DRAIN (CEMENT SLURRY BACKFILL)	STORM DRAIN	CITY OF DEL MAR	5	JIMMY DURANTE BLVD	15+25 TO 21+25 (JDB)	AS-BUILT (E-91-004-4)	FILL	PIP	PROJECT
27	24" RCP STORM DRAIN (CEMENT SLURRY BACKFILL)	STORM DRAIN	CITY OF DEL MAR	5	JIMMY DURANTE BLVD	21+25 TO 24+50 (JDB)	AS-BUILT (E-91-004-4)	FILL OVER CUT & COVER SECTION	RELOCATE	PROJECT
28	18" CMP STORM DRAIN	STORM DRAIN	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	28+00	AS-BUILT (E-91-004-4) - REFERENCED	FILL OVER CUT & COVER SECTION	RELOCATE	PROJECT
29	DRAINAGE CHANNEL	STORM DRAIN	NCTD	SURFACE	21ST STREET	26+50	CITY OF DEL MAR ASSET MAP	FILL OVER CUT & COVER SECTION	REPLACE	PROJECT
30	12" VCP SEWER	SEWER	CITY OF DEL MAR	6' - 8'	JIMMY DURANTE BLVD	16+75 TO 21+25 (JDB)	AS-BUILT (E-80-004-01) & (E85-002-1)	FILL & PERMANENT FACILITIES	TBD	PROJECT
31	6" VCP SEWER	SEWER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	21+25 (JDB)	(E-78-003-02) WASTEWATER SYSTEMS MAP	FILL OVER CUT & COVER SECTION	TBD	PROJECT
32	12" PVC SEWER	SEWER	CITY OF DEL MAR	2'-8'	JIMMY DURANTE BLVD	25+25 (JDB)	AS-BUILT (E-86-001-09)	FILL	TBD	PROJECT
33	8" VCP SEWER	SEWER	CITY OF DEL MAR	7'	JIMMY DURANTE BLVD	28+50 (JDB)	AS-BUILT (E-60-009-1)	FILL	TBD	PROJECT
34	8" PVC SEWER	SEWER	CITY OF DEL MAR	2'-4'	JIMMY DURANTE BLVD	15+50 TO 21+25 (JDB)	AS-BUILT (E-15-005-25) (E-56-002-01)	FILL	TBD	PROJECT
35	8" SEWER	SEWER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	21+25 TO 24+50 (JDB)	(E-56-002-01) WASTEWATER SYSTEMS MAP	FILL OVER CUT & COVER SECTION	TBD	PROJECT
36	8" SEWER	SEWER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	24+50 TO 28+25 (JDB)	(E-60-009-02) WASTEWATER SYSTEMS MAP	FILL	TBD	PROJECT
37	12" SEWER	SEWER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	25+00 (JDB)	(E-86-003-01) WASTEWATER SYSTEMS MAP	FILL OVER CUT & COVER SECTION	RELOCATE	PROJECT
38	12" VCP SEWER	SEWER	CITY OF DEL MAR	6' - 8'	JIMMY DURANTE BLVD	16+75 TO 21+25 (JDB)	AS-BUILT (E-80-004-01) (E-85-002-01)	FILL & PERMANENT FACILITIES	TBD	PROJECT
39	8" VCP SEWER	SEWER	CITY OF DEL MAR	8'	DAVID WAY	18+50 (JDB)	AS BUILT (78-003-02)	FILL	RELOCATE	PROJECT
40	8" DIP & 6" PVC WATER	WATER	CITY OF DEL MAR	3'-4'	JIMMY DURANTE BLVD	14+50 TO 18+50 (JDB)	AS-BUILT (E-15-005-21	FILL	TBD	PROJECT

Crest Canyon Higher Speed - North
Utility Conflict Matrix

ITEM	UTILITY DESCRIPTION	UTILITY TYPE	UTILITY OWNER	DEPTH	LOCATION	STATION	DATA SOURCE	POTENTIAL CONFLICT	DISPOSITION	
									PIP/RELOCATE/ ENCASE	BY
41	12"FPVC WATER IN 20" STEEL CASING	WATER	CITY OF DEL MAR	3' MIN, TBD	JIMMY DURANTE BLVD	14+50 TO 21+25 (JDB)	AS-BUILT (E-15-005-21)	FILL	TBD	PROJECT
42	12"FPVC WATER IN 20" STEEL CASING	WATER	CITY OF DEL MAR	3' MIN, TBD	JIMMY DURANTE BLVD	21+25 TO 24+50 (JDB)	AS-BUILT (E-15-005-22/23)	FILL OVER CUT & COVER SECTION	TBD	PROJECT
43	12"FPVC WATER IN 20" STEEL CASING	WATER	CITY OF DEL MAR	3' MIN, TBD	JIMMY DURANTE BLVD	24+50 TO 28+25 (JDB)	AS-BUILT (E-15-005-22/23)	FILL	TBD	PROJECT
44	6" ACP - ABANDONED	WATER	CITY OF DEL MAR	TBD	JIMMY DURANTE BLVD	20+00 TO 22+00 (JDB)	AS-BUILT (E-15-005-22)	FILL	ABANDON	PROJECT

Appendix D. Preliminary Drainage Report

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San Dieguito to Sorrento Double Track

Del Mar Tunnels Alternatives Analysis Preliminary Drainage Report

San Diego Regional Rail Corridor
Alternative Alignment and Improvements
Conceptual Engineering Study

October 2022

Prepared by:

HDR

Prepared for:

SANDAG

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619) 699-1900
www.sandag.org

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ABSTRACT

TITLE: San Dieguito to Sorrento Valley Double Track
Del Mar Alternatives Analysis Preliminary Drainage Report

AUTHOR: San Diego Association of Governments

DATE: October 2022

SOURCE OF COPIES: San Diego Association of Governments
401 B Street, Suite 800
San Diego, CA 92101
(619) 699-1900

NUMBER OF PAGES: 34

ABSTRACT: The Del Mar Tunnels Alternatives Analysis Drainage Report details design requirements and potential floodplain impacts associated with the two proposed alignment alternatives, as outlined in the *Del Mar Tunnels Alternatives Analysis Report* (HDR 2022). The Del Mar alignment alternatives begin and end in two separate watersheds, San Dieguito River and Los Peñasquitos, as well as their associated floodplains, and intersect areas potentially impacted by sea level rise. Potential flood risks influence proposed design features such as portal elevations, fill and bridge locations, and track alignment and profile. Recommended hydraulic design criteria for these features considered the San Diego Association of Governments (SANDAG) design criteria, proximity to the sea level rise area of influence, current Federal Emergency Management Agency (FEMA) flood elevations, and recent hydraulic analyses to limit impacts from and risk to project elements. In the areas potentially affected by sea level rise, recommendations are made to protect the tracks and portals against the 100-year flood event considering the effects of sea level rise through year 2100 per the LOSSAN Design Criteria. Recent hydraulic analyses were reviewed and used to assess the design elevation based on this standard. Tunnel portals are recommended to be above the 100-year flood plus sea level rise or the current 500-year flood, whichever is greater. These recommended criteria are considered, in addition to other design constraints that are not hydraulics related. This report also describes the current FEMA special flood hazard areas (floodplains) at San Dieguito and Los Peñasquitos Lagoons and the compliance requirements related to the National Flood Insurance Program. The proposed alignments should minimize impacts on the FEMA floodplains. A recommended approach to FEMA compliance, for each floodplain and alignment, is provided.

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Contents

Tables

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Acronyms

BFE	base flood elevation
CCC	California Coastal Commission
DCM	Design Criteria Manual
EGL	energy grade line
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
HEC-RAS	Hydrologic Engineering Center River Analysis System
I	Interstate
LOMR	Letter of Map Revision
LOSSAN	Los Angeles-San Diego-San Luis Obispo
MHHW	mean higher high water
NOAA	National Oceanic and Atmospheric Administration
project	San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study
ROW	right-of-way
SANDAG	San Diego Association of Governments
SD-LOSSAN	San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study

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1 Purpose

The purpose of this study is to analyze the potential floodplain impacts and design requirements associated with the proposed alternatives for removing the existing tracks from the Del Mar Bluffs and placing the tracks in a tunnel between the south side of San Dieguito Lagoon basin (near the Del Mar Fairgrounds) and the north end of Sorrento Valley. This report documents the hydraulic design support for the *Del Mar Alternatives Analysis Report* (HDR 2022). The hydraulic analysis includes evaluation of the alignments and design input for the track profile and drainage features for two proposed alternatives.

2 Background

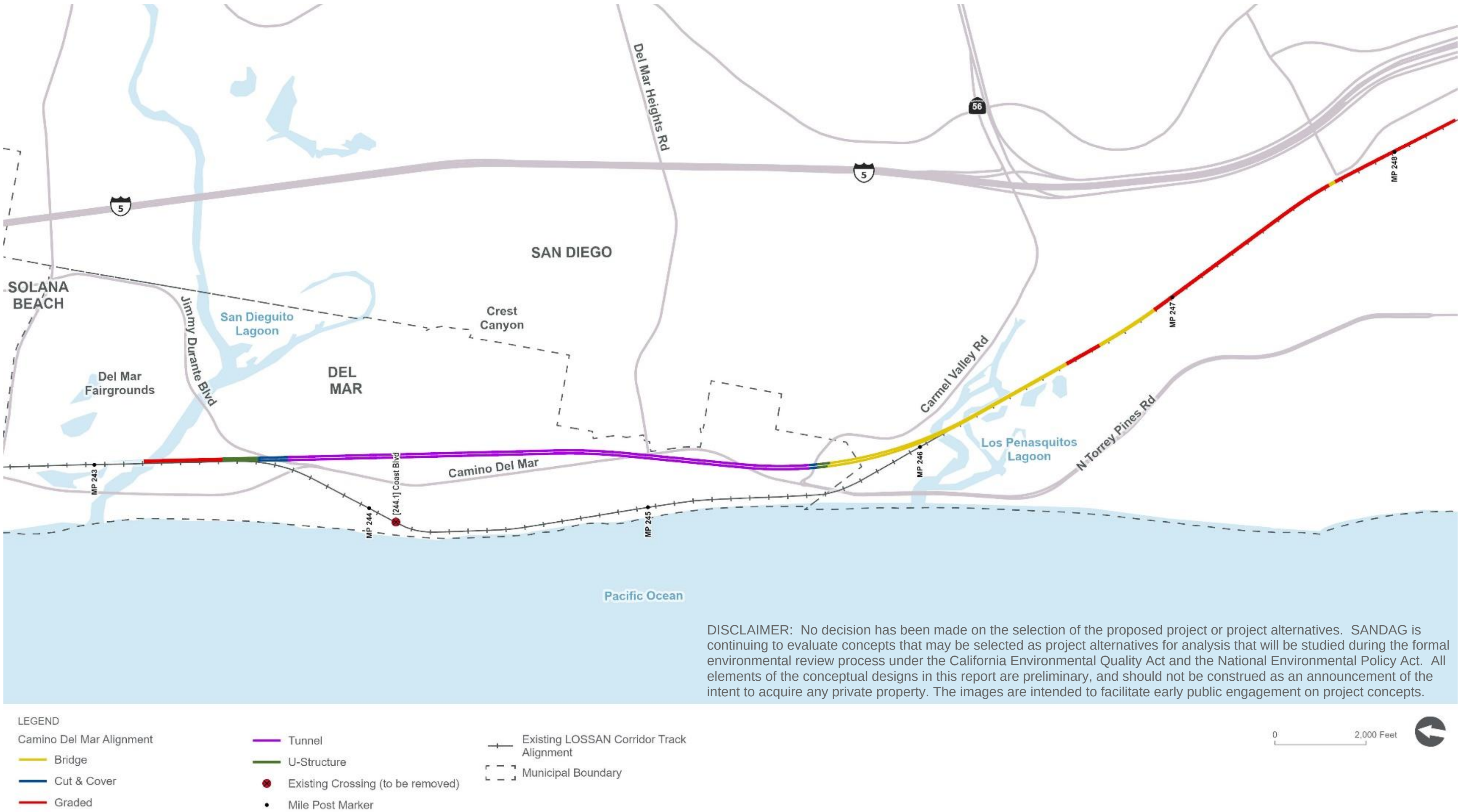
Ongoing bluff erosion and the threat of sea level rise due to climate change underscore the importance of moving the railroad tracks completely off the Del Mar Bluffs. As such, the San Diego Association of Governments (SANDAG) commissioned the San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study (SD-LOSSAN or project), which will determine a long-term safety and operations solution for the San Diego segment of the Los Angeles-San Diego-San Luis Obispo (LOSSAN) railroad tracks. The *Del Mar Alternatives Analysis Report* (HDR 2022) documented the analysis and selection of alternatives for relocating the existing single-track alignment of the LOSSAN rail corridor through the City of Del Mar, where the rail line runs along a terrace on the coastal bluffs, to a future double-tracked alignment between the south side of the San Dieguito Lagoon basin (near the Del Mar Fairgrounds) and the north end of Sorrento Valley in the City of San Diego. The alternatives analyzed would replace the existing LOSSAN rail corridor alignment along the coastal bluffs with a new alignment away from the bluffs, primarily located within tunnels through the coastal hill of Del Mar and on aerial structures, that would eliminate the risk of a rail corridor service outage caused by bluff erosion. The proposed alignment would provide greater track capacity and a higher operating speed for trains in the corridor, enabling projected increases in service. Five tunnel alignment alternatives were initially developed by SANDAG in a 2017 conceptual engineering and environmental analysis (HNTB Corporation 2017). For this project, the initial five alignment designs were refined to achieve higher operating speeds, then analyzed to determine their effectiveness in meeting the project's evaluation criteria. Alignments were evaluated based on defined planning, construction, post-construction/operation, and community acceptance considerations. At the conclusion of the alternatives analysis process, two alignments were carried forward for additional evaluation. A summary description of final two alternatives is provided below.

2.1.1 Camino Del Mar Alternative

The Camino Del Mar Alternative's north end begins south of the Del Mar Fairgrounds at the south end of the double-track bridge that crosses San Dieguito Lagoon. The bridge is proposed as part of the San Dieguito River Bridge Replacement Double Track and Special Events Platform. The alignment leaves the right-of-way (ROW) and crosses Jimmy Durante Boulevard, which would be realigned to cross over the tracks and enter the north portal of the tunnel in a U-structure and cut-and-cover box. The tunnel is a twin-bored configuration, which continues underground through the residential area of Del Mar near Camino Del Mar. The alignment exits the south portal with a cut-and-cover box under Carmel Valley Road near North Torrey Pines Road. It transitions to a U-structure proceeded by an aerial structure over McGonigle Road until it ties into the existing ROW in Los Peñasquitos Lagoon. Once within the existing ROW, the alignment transitions between the bridge structure and a berm until it ties into the existing tracks south of Bridge 247.7.

Figure 1 shows the proposed revised Camino Del Mar Alternative.

Figure 1. Camino Del Mar Alternative

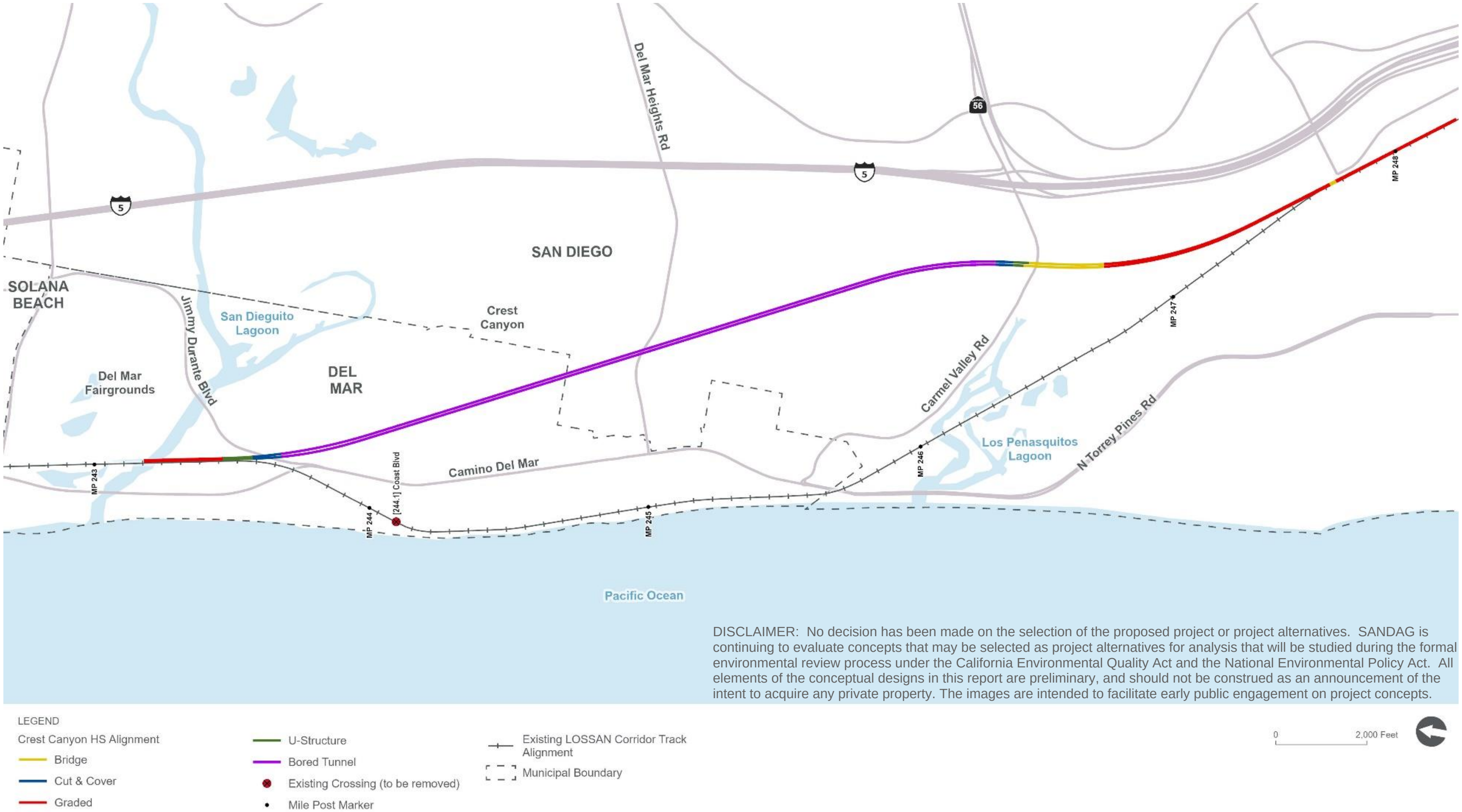


2.1.2 Crest Canyon Higher Speed Alternative

The Crest Canyon Higher Speed Alternative's north end begins south of the Del Mar Fairgrounds at the south end of the double-track bridge that crosses San Dieguito Lagoon. The bridge is proposed as part of the San Dieguito River Bridge Replacement Double Track and Special Events Platform. The alignment leaves the ROW and crosses Jimmy Durante Boulevard, which would be realigned to cross over the tracks and enters the north portal of the tunnel in a U-structure and cut-and-cover box. The tunnel is a twin-bored configuration, which continues underground through the residential area of Del Mar and the Torrey Pines Extension. The alignment exits the south portal with a cut-and-cover box and U-structure between Portofino Boulevard and Caminito Pointe Del Mar, where it transitions to an aerial structure over Carmel Valley Road and into Los Peñasquitos Lagoon. Once within the existing ROW, the alignment transitions between the bridge structure and a berm and rejoins the existing ROW at the north end of Sorrento Valley. The alignment ties into the existing tracks just south of Bridge 247.7.

Figure 2 shows the proposed revised Crest Canyon Higher Speed Alternative.

Figure 2. Crest Canyon Higher Speed Alternative



2.2 Major Drainages

The LOSSAN corridor through San Diego County passes through seven primary watersheds in the coastal region. The Del Mar Alternative Alignment begins and ends in two of these primary watersheds: San Dieguito River and Los Peñasquitos, as shown on Figure 3. An overview of each is provided in the subsequent sections.

2.2.1 San Dieguito River Watershed

The San Dieguito River Watershed drains an area of 346 square miles in the west to central part of San Diego County. The watershed includes two major surface water reservoirs: Sutherland Reservoir and Hodges Reservoir. The San Dieguito River is the primary drainage in the watershed with headwaters originating in the Witch Creek Basin. Flows from the Witch Creek and Sutherland Basins collect in the Sutherland Reservoir before discharging through Santa Ysabel Creek. Santa Ysabel Creek continues westward through San Pasqual Valley where it becomes the San Dieguito River. Below the Hodges Reservoir, multiple tributaries join the San Dieguito River and discharge into the Pacific Ocean via San Dieguito Lagoon. At the lagoon, crossings include Camino Del Mar, LOSSAN Bridge 243.0, and Jimmy Durante Boulevard.

In the early 2000s, Southern California Edison developed a coastal wetlands restoration plan for the San Dieguito Lagoon as a compensatory mitigation project for other power generation activities. The San Dieguito Wetlands Restoration Project included the creation of tidal and subtidal habitats, construction of berms to maintain sediment flows within the river and to the beach, and tidal inlet maintenance to promote regular tidal exchange through excavation of the river channel. The restoration project was completed in 2011.

2.2.2 Los Peñasquitos Watershed

The Los Peñasquitos Watershed is tributary to the Los Peñasquitos Lagoon, drains 94 square miles to the Pacific Ocean, and contains several subwatersheds, including Carmel Canyon Creek, Los Peñasquitos Creek, and Carroll Canyon Creek. Drainage for the Peñasquitos Watershed comes from as far east as Iron Mountain. The average annual precipitation within the Peñasquitos Watershed ranges from 9 inches at the coast to upwards of 21 inches within the eastern areas of the watershed. Los Peñasquitos Lagoon is approximately 565 acres of coastal estuary. In the Federal Emergency Management Agency (FEMA) mapping and documentation, Los Peñasquitos Lagoon and Sorrento Valley (toward the direction of Carroll Canyon) is referred to as Soledad Canyon.

As with all the coast estuaries within San Diego County, coastal railway alignments have impacted Los Peñasquitos Lagoon. The railway alignment constructed on an elevated, compacted fill berm runs through the center of the lagoon. Several of the historic tidal channels were cut off as a result of the berm, dividing the lagoon into an eastern and western basin. In addition, as urbanization of the watershed continued, the three main tributaries to the lagoon became perennial, contributing runoff flows into the lagoon. Hydrologic modification of Los Peñasquitos Creek has also resulted in the encroachment of fresh and brackish marsh at the southern portion of the lagoon.

Carmel Canyon Creek

Carmel Canyon Creek is the smallest of the three subwatersheds that drain to Los Peñasquitos Lagoon. The creek joins the lagoon midway, from a northeast direction. The creek passes through bridges at Carmel Creek Road, El Camino Real, and Interstate (I) 5 before meeting the lagoon. The creek channel through this reach is natural but constricted by encroachment from State Route 56. Well established vegetation reduces flow capacity through this reach. At the connection to the lagoon, dense stands of mature willow or mule fat scrub are present. Per the Los Peñasquitos Lagoon Enhancement Plan (ESA 2018), this area of riparian wetland has grown out of sediment trapped behind the original rail berm of 1883 (removed in 1998). There is no clearly defined channel or main conveyance area from the I-5 bridge into the lagoon area.

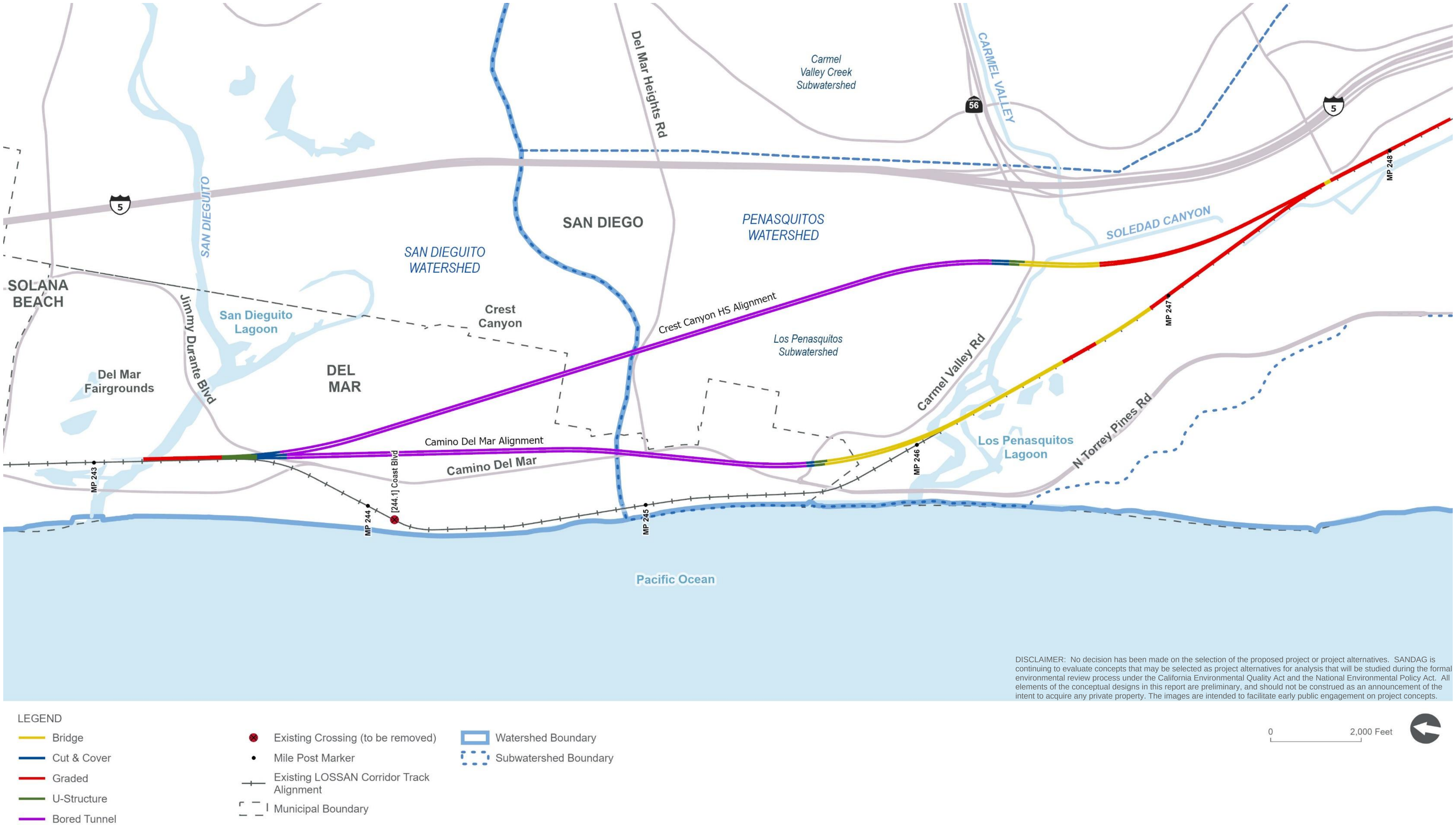
Los Peñasquitos Creek

Los Peñasquitos Creek is the largest of the three subwatersheds that drain to Los Peñasquitos Lagoon. Los Peñasquitos Creek drains an area approximately 58 square miles The creek joins the lagoon and its southernmost extent with the confluence of Carroll Canyon Creek. The creek passes through a pedestrian bridge at Sorrento Valley Road and LOSSAN Bridge 248.7 before meeting the lagoon. This area of the lagoon is also thickly vegetated, which has altered the drainage and flooding characteristics in the recent decades. A narrow channel has been intermittently maintained by the City of San Diego from Bridge 248.7 and the confluence of Carroll Canyon Creek, northwest into the lagoon. The lower reach of Los Peñasquitos Creek is sometimes referred to as Soledad Canyon.

Carroll Canyon

The Carroll Canyon sub-watershed is approximately 17 square miles. The creek joins the lagoon and its southern most extent with the confluence of Los Peñasquitos Creek. The creek passes through a pedestrian bridge at the North County Transit District Sorrento Valley Station and a bridge at Sorrento Valley Boulevard. The creek is channelized into a concrete trapezoidal channel in the same area but terminates just downstream of the pedestrian bridge. The final 1,400 feet is soft bottom but constricted by development on Roselle Street and the LOSSAN ROW. The creek also crosses below I-5, which is elevated considerably. Further upstream, the creek has a similar crossing under I-805. After crossing under I-805, the creek is sometimes referred to as Soledad Canyon.

Figure 3. Major Drainages



2.3 Federal Emergency Management Agency Floodplain Mapping

This section details the FEMA regulatory floodplains (also referred to as special flood hazard areas) that intersect the proposed alignment alternatives. These floodplains were established to help define areas of flood risk for the purposes of supporting the National Flood Insurance Program and do not necessary include all areas with flooding potential. According to the San Diego County Flood Insurance Study (FIS; FEMA 2019), the majority of the FEMA-designated floodplains were developed in the 1980s through hydrologic and hydraulic study. Some areas have been restudied and remapped for a variety of reasons but may not be reflected on the current Flood Insurance Rate Maps.

The floodplains along the LOSSAN corridor are either riverine floodplains, driven by river flood sources, or coastal flood hazard areas driven by oceanographic sources. Riverine detailed studies typically include the 10, 2, 1, and 0.2 percent annual chance exceedance discharge, or more commonly referred to as the 10-, 50-, 100-, and 500-year events, respectively. The 100-year event is the event that drives regulatory compliance and flood insurance requirements. The 10-year and 50-year event are primarily informational. The 500-year event is often mapped as a Flood Hazard Zone X but does not carry specific compliance measures. Table 1 provides a summary of flood source, flood hazard zone, discharge, and base flood elevations (BFE) at each intersection of the corridor and flood hazard zone. Figure 4 presents the FEMA flood hazard zones along the two alignment alternatives.

The 100-year riverine floodplain extent can be mapped into several types of special flood hazard areas, with the primary ones described below:

Zone AE: This includes areas subject to inundation by the 100-year flood event determined by detailed methods with BFEs shown. The community must review floodplain development on a case-by-case basis to ensure that increases in water surface elevations do not occur or are updated via a Letter of Map Revision (LOMR).

Zone AE with Regulatory Floodway: This zone is identical to a Zone AE, but with an established floodway. A Regulatory Floodway is the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a maximum of 1 foot (some communities may have lower limits). The intent of the floodway is to facilitate development in the Zone AE outside of the floodway without review.

Zone A: This includes areas subject to inundation by the 100-year flood event generally determined using approximate methodologies. Because detailed hydraulic analyses have not been performed, no BFEs or flood depths are shown. Development within a Zone A requires demonstration of cumulative impact on the upstream water surface elevation of 1 foot or less as compared with existing conditions.

Table 1. Los Angeles-San Diego-San Luis Obispo Federal Emergency Management Agency Special Flood Hazard Area

FEMA Flood Source and FIS Discharge Location	FEMA Special Flood Hazard Area	FEMA Effective Discharge (cubic feet per second)	Discussion
San Dieguito River – Upstream of Camino Del Mar Bridge	Zone AE – Floodway	10-year: 5,700 50-year: 31,400 100-year: 41,800 500-year: 90,000	Bridge 243.0 crosses San Dieguito River near the Pacific Ocean. The 100-year BFE at the bridge is approximately 14.7 feet and is approximately 17 feet where the rail leaves the southern edge of the floodplain. The 500-year flood profile elevation at the bridge is approximately 22 feet, with an interpolated elevation of 23.6 ft at the edge of the Zone X (500-year). Note: Per the FIS Floodway Data Table, the encroached floodway elevations match the published BFEs.
Los Peñasquitos Lagoon – Soledad Canyon at Mouth	Zone AE – Floodway	10-year: 5,000 50-year: 15,400 100-year: 23,000 500-year: 51,500	The lagoon BFE is approximately 14 feet for most areas. Toward Sorrento Valley, the BFEs begin to rise near Bridge 247.1. Toward Carmel Valley, BFEs do no increase until the I-5 bridge. The lagoon 500-year flood profile elevation is 20 feet for most of the lagoon.
Carmel Valley Creek – Above Confluence with Soledad Canyon	Zone AE – Floodway	10-year: 2,100 50-year: 6,500 100-year: 9,800 500-year: 21,300	The Carmel Valley streamline and floodplain begins right at the downstream side of the Old Sorrento Valley Road. However, the floodway connects to the Los Peñasquitos/Soledad Canyon floodway. The Carmel Valley BFEs start at 14.4 feet. The 500-year flood profile elevation is depicted as 18 feet, which is less than the Los Peñasquitos/Soledad Canyon profile. This is likely due to a modeling or mapping error.

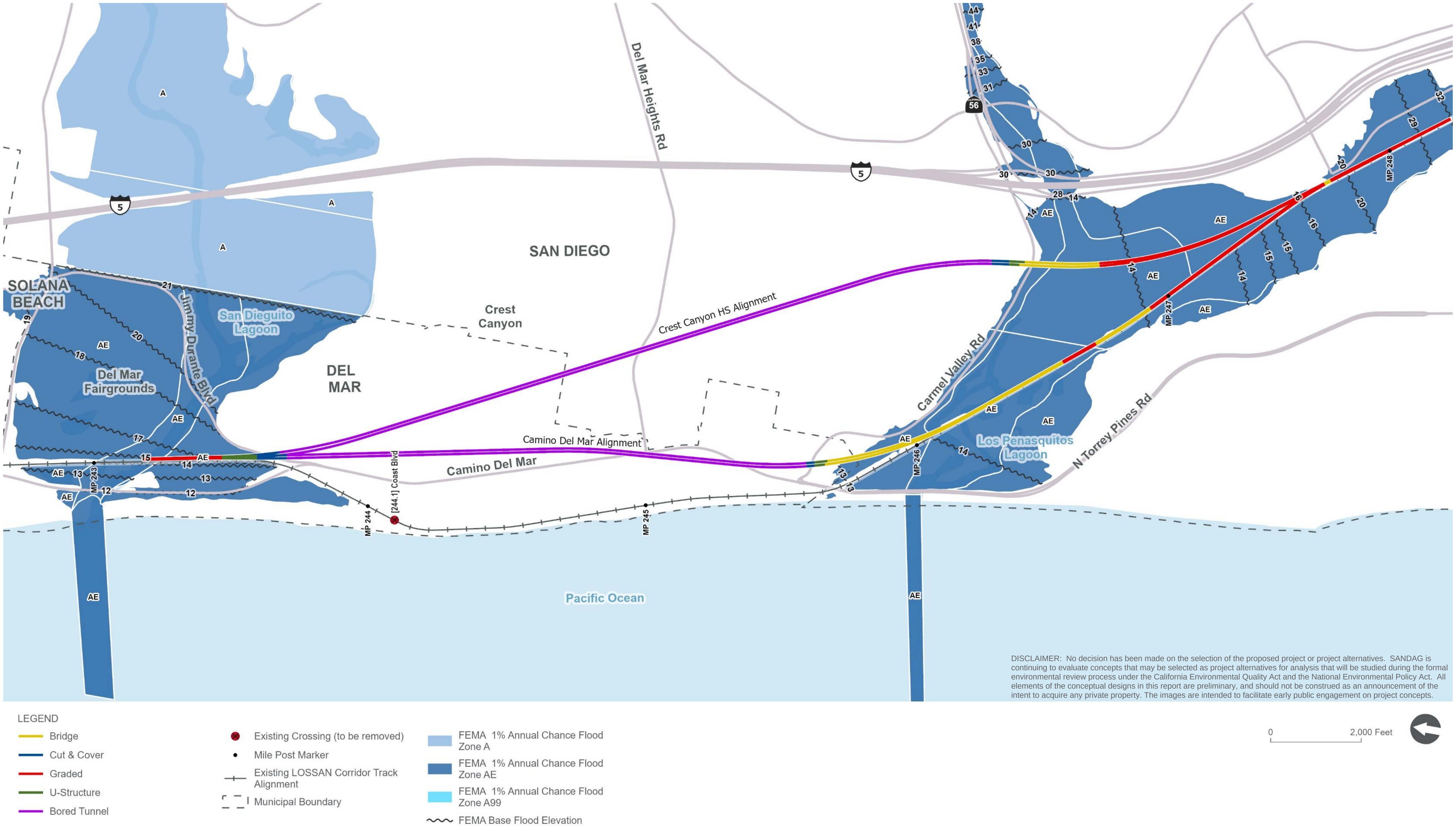
Sources: FEMA n.d., 2019

Notes:

All elevations have a vertical datum of North American Vertical Datum of 1988

BFE=base flood elevation; FEMA=Federal Emergency Management Agency; FIS=Flood Insurance Study; I=Interstate

Figure 4. Federal Emergency Management Agency Special Flood Hazard Areas



2.4 Previous Studies

2.4.1 Los Angeles-San Diego-San Luis Obispo Design Studies

San Dieguito Double Track Project

Moffat and Nichol completed a hydraulic analysis of the proposed San Dieguito River LOSSAN bridge (Bridge 243.0) for the San Dieguito Double Track Project in 2016. The purpose of the study was primarily to inform the design of the bridge structure. A one-dimensional, steady state model adapted from previous lagoon studies was applied to ensure the proposed bridge met SANDAG bridge hydraulic criteria. The FEMA effective model was not utilized for this study. River discharges from the FEMA FIS were used for the modeling. The study included a sensitivity analysis of potential sea level rise scenarios, from 1.4 to 5.5 feet. Sea level rise within this range was found to not impact hydraulics at the LOSSAN bridge, due to the controlling effects of the Camino Del Mar Bridge.

Los Peñasquitos Lagoon Bridge Replacement Project

HDR completed multiple, bridge-specific hydraulic analyses for the Los Peñasquitos Lagoon Bridge Replacement Project in 2013. These analyses were in support of the FEMA No-Rise Certificate and to inform design. The pertinent bridges included were Bridges 246.1, 246.9, and 247.1. Given the complex hydraulics of the lagoon, with overtopping flows, Bridges 246.9 and 247.1 were modeled with bridge specific models, utilizing conservative assumptions with regard to hydraulic gradient through the bridge. Bridge 246.1 was modeled using a revised version of the FEMA effective model, which included the addition of the North Torrey Pines Road Bridge and the existing (2013) Bridge 246.1. This model originated with a mean higher high water (MHHW) tailwater condition. No sea level rise scenario was evaluated. Similar to the Camino Del Mar Bridge at San Dieguito, the North Torrey Pines Road Bridge appears to constrict flood flows, suggesting impacts due to sea level rise might be muted.

Sorrento Valley Double Track Project

HDR completed hydraulic analysis and FEMA map revisions submittals for the Sorrento Valley Double Track Project. The Sorrento Valley Double Track Project extended from Bridge 247.7, which passes local drainage into Los Peñasquitos Lagoon, to Bridge 248.7, which crosses Los Peñasquitos Creek just upstream of the entrance into the lagoon. The FEMA Effective modeling for Soledad Canyon and Los Peñasquitos Creek was updated with new topographic data, vegetation, and structural obstructions. A LOMR was submitted to and approved by FEMA (LOMR 12-09-2141P). This LOMR updated the floodplain for the pre-project condition. A subsequent No-Rise Certificate for the proposed Bridge 248.7 was submitted and accepted by the City of San Diego. The modeling reach did not extend to the ocean but ended in the vicinity of Mile Post 247.3. At this location, current tidal conditions did not influence floodplain hydraulics.

Los Peñasquitos Lagoon Enhancement Plan

The 2018 Los Peñasquitos Lagoon Enhancement Plan was prepared by BRG Consulting, Inc. (2021) with regard to the alternative rail alignments in Los Peñasquitos Lagoon, and a number of observations as to environmental opportunities and constraints were identified. The original railway alignment (referred to as the 1888 railway alignment) and the existing railway alignment (referred to as the 1925 railway alignment) through the lagoon are frequently identified by the plan as physical constraints resulting in the following:

- Tidal prism reduction of 50 to 75 percent
- Inlet closures
- Increased silt deposition rates
- Freshwater conveyance reduction and flooding
- Vector presence

Any alternative that maintains the current (or similar) rail berm through the lagoon may not alleviate these conditions and would be a constraint to lagoon enhancement. Alternatives with removal or modification of the berm to address these constraints would be considered beneficial. Tunnel and bridge alternatives that would carry the tracks over or under the lagoon and its floodplain would be considered beneficial.

A number of goals are presented in the plan, but Goal 13 is most directly applicable to the current SD-LOSSAN.

New Goal 13: Remove, relocate, or modify existing infrastructure located along or within the lagoon to reduce or eliminate both direct and indirect impacts on lagoon resources and processes.

Objectives For Goal 13

1. Reduce impacts from existing infrastructure to ensure long-term environmental sustainability and address community and economic sustainability of the restoration.
2. Ensure the project identifies and considers the potential impact on the restoration project from future infrastructure (road, railroad, and utilities) projects and development.

Any railway alternative that maintains the current (or similar) rail berm through the lagoon would not appear to reduce impacts from existing infrastructure and would be a constraint to lagoon enhancement. The Camino Del Mar Alternative follows the 1925 railway alignment through the lagoon; this constraint applies to the Camino Del Mar Alternative.

Removal or modification of the berm to facilitate water passage through the railway alignment would be considered beneficial and presents an alignment opportunity. Tunnel and bridge alternatives that would carry the tracks over or under the lagoon and its floodplain would be considered beneficia.

2.4.2 City of Del Mar

Camino Del Mar Bridge Replacement Project

Chang Consultants conducted hydraulic analysis for replacement of the Camino Del Mar Bridge. This study utilized the same, or essentially the same, Hydrologic Engineering Center River Analysis System (HEC-RAS) one-dimensional, steady state model as the San Dieguito Double Track hydraulic analysis. However, this modeling effort looked at proposed configurations for Camino Del Mar but still included the existing Bridge 243.0 rail bridge structure. This is the opposite of the San Dieguito double-track analysis. A range of sea level rise options were considered up to 10 feet of sea level rise. The preferred bridge alternative reduces the tidal muting, which enables effects of sea level rise to be realized at the rail bridge. The results of this analysis indicated higher water surface elevations at the rail bridge, as compared with the San Dieguito Double Track study. However, these cannot be directly compared because of the different rail bridge configuration.

3 Hydraulic Design Criteria

The SANDAG LOSSAN Corridor Design Criteria Manual (DCM) Volume III (LOSSAN DCM) provides hydraulic design criteria for rail bridges and track roadbed adjacent to channels. From the LOSSAN DCM, the bridge hydraulic design criteria is as follows:

Section 8.3.1 Standard Design Criteria:

- The bridge opening will be sized so that the 50-year water surface for a low chord/soffit event will rise no higher than the lowest low chord of the bridge.
- The bridge opening will be sized so that the 100-year energy grade line (EGL) will not rise above the adjacent subgrade elevation unless engineering justification is provided.

The EGL is the elevation of the water surface elevation (hydraulic grade line) plus the velocity head. This approximates the total energy potential of flowing water and provides a theoretical maximum of flow runup potential. The EGL criteria helps differentiate slow-moving versus fast-moving hydraulic conditions in a way that a water surface elevation criterion would not.

The criteria for track roadbed elevation is provided in a separate section:

Section 8.4.1 Storm Frequency

- Track side drains shall be designed to accommodate a 100-year flood level below the bottom of the ballast in the channel and at culvert/storm drain entrances.

The criteria for trackside drainage are assumed to carry over to adjacent natural water courses. The nomenclature is not exact but applying a 100-year EGL standard to the bottom of ballast/top of subgrade would provide the same level of protection as at bridge structures.

The LOSSAN DCM criteria, as described above, was applied to the alternative concepts, utilizing the available information. The FEMA flood profiles for San Dieguito River, Soledad Canyon (Los Peñasquitos Lagoon), and Carroll Canyon provide an estimate of the 100-year water surface but not necessarily the EGL. An additional 1.5 foot was added to the 100-year water surface to account for velocities up to 10 feet per second.

3.1 Sea Level Rise Guidance

Potential sea level rise resulting from climate change is an important design consideration for future infrastructure located within the coastal area. Sea level rise impacts flood risk in two ways. First, rising ocean levels bring tidal areas and areas of wave attack risk further inland. Second, as the terminus for riverine flood water, higher ocean levels can raise anticipated BFEs in the upstream areas, depending on hydraulic characteristics of the floodplain. The National Oceanic and Atmospheric Administration (NOAA) provides a Sea Level Rise Viewer (NOAA n.d.), which allows users to view the current regular extent of the tidal range (MHHW), as well as sea level rise of up to 10 feet. Figure 5 and Figure 6 provide depictions of the San Dieguito Lagoon, under current MHHW conditions and a sea level rise of 7 feet, respectively. Similarly, MHHW and 7 feet of sea level rise are depicted for Los Peñasquitos Lagoon on Figure 7 and Figure 8. These figures demonstrate that the northern and southern portal and associated approaches have the potential to be influenced by changing sea level conditions.

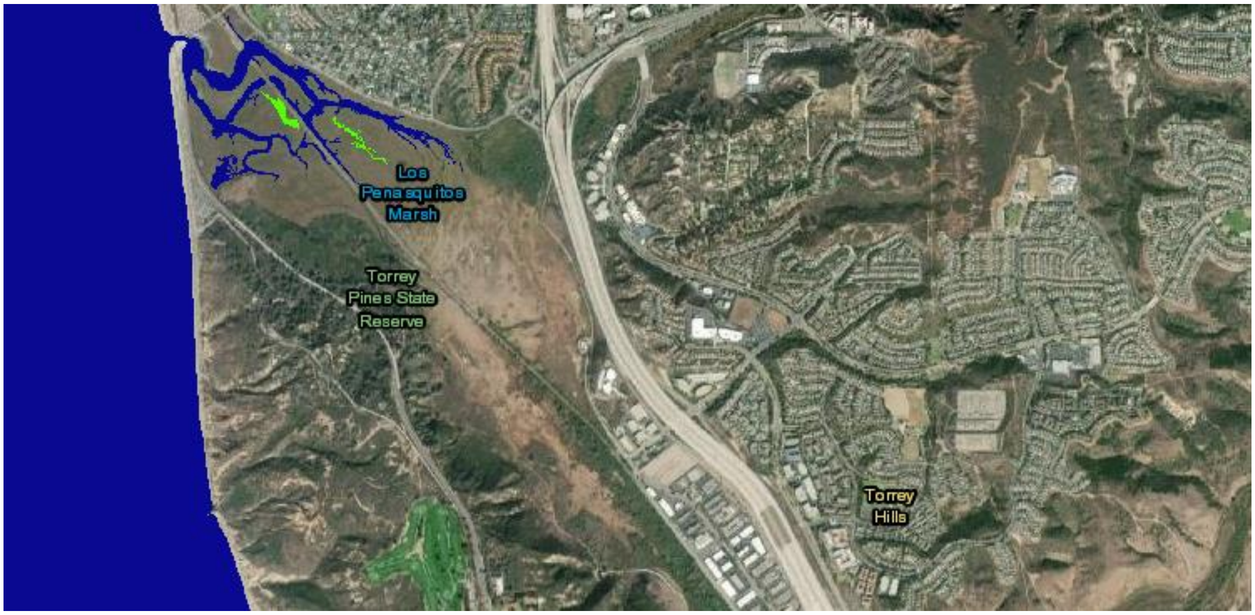
Figure 5. Current Mean Higher High Water San Dieguito Lagoon

Source: NOAA n.d.

Figure 6. Sea Level Rise (+7 feet) Area of Influence San Dieguito Lagoon

Source: NOAA n.d.

Figure 7. Current Mean Higher High Water Los Peñasquitos Lagoon



Source: NOAA n.d.

Figure 8. Sea Level Rise (+7 feet) Area of Influence Los Peñasquitos Lagoon

Source: NOAA n.d.

3.1.1 North County Transit District/San Diego Association of Governments Guidance

The LOSSAN DCM provides guidance related to sea level rise and design of new facilities. The LOSSAN DCM includes a table of estimated sea level range predictions, depending on greenhouse gas emission rates and scenarios (Figure 9). The LOSSAN DCM specifically indicates that alternative analyses for new projects should consider sea level rise between 1.4 and 5.5 feet, or "the latest guidance from State or Federal regulations, to determine project impacts associated with predicted sea level rise." The 2017 LOSSAN DCM predates the most recently updated *California Coastal Commission* (CCC) *Sea Level Rise Policy Guidance* (CCC 2018). Therefore, the CCC document governed.

Figure 9. Los Angeles-San Diego-San Luis Obispo Design Criteria Manual Sea Level Rise Predictions

Year	2000	2030	2050	2100
Lower Rate	0.0 ft.	0.3 ft.	0.4 ft.	1.4 ft.
Likely High	0.0 ft.	0.5 ft.	0.8 ft.	2.0 ft.
High Rate	0.0 ft.	1.0 ft.	2.0 ft.	5.5 ft.

Source: SANDAG 2017

3.1.2 California Coastal Commission Guidance

The CCC adopted the science update of *California Coastal Commission Sea Level Rise Policy Guidance* in 2018. This guidance provides 12 location specific sea level rise recommendations up and down the coast, based on NOAA tidal gage. The La Jolla Tidal Gage is the closest to the study area. Figure 10 provides a range of predicted sea level rise scenarios at the La Jolla Tidal Gage for 10-year increments, starting in 2030 and extending to 2150. Additionally, there are three risk-based estimates for each decade interval. The risk-based estimates are Low Risk Aversion, Medium-High Risk Aversion, and Extreme Risk Aversion, with the first two including an assigned exceedance probability.

Figure 10. California Coastal Commission Sea Level Rise Predictions, La Jolla

	Probabilistic Projections (in feet) (based on Kopp et al. 2014)		H++ Scenario (Sweet et al. 2017)
	Low Risk Aversion	**Medium-High Risk Aversion**	**Extreme Risk Aversion**
	Upper limit of "likely range" (~17% probability SLR exceeds…)	*1-in-200 chance (0.5% probability SLR exceeds…)*	*Single scenario (no associated probability)*
2030	0.6	0.9	1.1
2040	0.9	1.3	1.8
2050	1.2	2.0	2.8
2060	1.6	2.7	3.9
2070	2.0	3.6	5.2
2080	2.5	4.6	6.7
2090	3.0	5.7	8.3
2100	3.6	7.1	10.2
2110*	3.7	7.5	12.0
2120	4.3	8.8	14.3
2130	4.9	10.2	16.6
2140	5.4	11.7	19.2
2150	6.1	13.3	22.0

Source: CCC 2018

3.1.3 Study Sea Level Rise Approach

The selected sea level rise value for use in this study considered both the LOSSAN DCM and the CCC guidance. The LOSSAN DCM directed sea level rise scenarios based on year 2100. This target year was carried forward in selection of a target sea level rise value based on the CCC guidance, which was determined to be the most recent state guidance. The CCC Medium-High Risk Aversion value (7.1 feet) was selected for year 2100, as it was larger than the SANDAG recommendation of 5.5 feet and has an associated exceedance probability of 0.5 percent; there is 1 in 200 chance of sea level rise being higher than 7.1 feet. During preliminary engineering, these assumptions should be re-evaluated taking into consideration any new state sea level rise guidance adopted since the time of this report.

As discussed above, the tunnel portals, their approaches, and the proposed rail bridges in San Dieguito/Los Peñasquitos Lagoons are within the areas potentially affected by sea level rise. These features are also potentially impacted by riverine flooding, which needs to be assessed in conjunction with sea level rise, ideally with a numeric hydraulic model. In the absence of hydraulic modeling, conservative assumptions were made. Below is the recommended approach by lagoon.

San Dieguito Lagoon

San Dieguito Lagoon has been the subject of multiple hydraulic modeling studies, as detailed in Section 2.4. The San Dieguito Double Track Bridge Hydraulic Report concluded sea level rise would not impact flood hydraulics at the LOSSAN structure due to the muting effect of the existing Camino Del Mar Bridge. The Camino Del Mar Bridge serves as a hydraulic constriction during large riverine flood events, and due to the hydraulic condition at that bridge, it was concluded that the effects of a high tailwater condition due to sea level rise would not propagate beyond the road bridge. The study only considered 5.5 feet of sea level rise, and the modeling should be verified with 7.1 feet.

The Camino Del Mar Bridge replacement analysis concluded that the newly proposed road bridge will reduce this muting and allow the effects of sea level rise to influence flood hydraulics at Bridge 243.0. Neither of these studies considered the proposed bridge condition for both Camino Del Mar and Bridge 243.0. Therefore, the track profile was established considering the highest predicted 100-year results, which was a 100-year water surface elevation, interpolated for 7.1 feet of sea level rise. This was interpolated between model profiles assuming 5.5 and 8.8 feet of sea level rise. At the bridge, this is approximately 16.2 feet in elevation.

Los Peñasquitos Lagoon

There is no adequate hydraulic model for Los Peñasquitos Lagoon. The FEMA Effective HEC-RAS 1D model does not include the North Torrey Pines Road Bridge, nor does it include the three lagoon LOSSAN bridges. This model is the basis for the FEMA 100-year BFEs published on the Flood Insurance Rate Maps, as well as the 100-year and 500-year flood profiles in the San Diego County FIS. The model presumably assumes a starting water surface elevation, at the ocean, of MHHW. Sea level rise would presumably increase the general flood condition but might be muted by controlling features in the same manner as San Dieguito Lagoon but would need to be confirmed by proper hydraulic modeling. Without such modeling, 7.1 feet was added to the 100-year and 500-year flood profile elevation to serve as an estimate of flood level increase in conjunction with sea level rise at Los Peñasquitos Lagoon to establish the proposed track profiles.

3.2 Track and Tunnel Portal Profile Approach Summary

The track profiles of the alternatives considered SANDAG design criteria, proximity to the sea level rise area of influence, current FEMA flood elevations, and the recent hydraulic analysis. Additional, nonhydraulic-related design considerations influenced the final track profile and tunnel portal. Track profiles outside of the sea level rise area of influence were based on the FEMA 100-year BFE. For tracks inside the sea level rise area of influence, the profiles were based on the FEMA 100-year BFE plus sea level rise (+7.1 feet for Los Peñasquitos Lagoon). This was not applicable near San Dieguito Lagoon, as the alignment is only in the sea level rise area of influence.

The tunnel portals are a unique facility not covered in the LOSSAN DCM. Given their critical nature, an elevated standard of protection was assumed. For the Del Mar Tunnel portals, the minimum portal elevation was assumed to be either the FEMA 100-year BFE plus the anticipated sea level rise value or the FEMA 500-year flood elevation, whichever was greater. For the Los Peñasquitos Lagoon, this was the 100-yr BFE plus the assumed 7.1 feet of sea level rise. For San Dieguito Lagoon, this was the 500-year elevation. However, as discussed in Section 4.2, the portal elevations for both alternatives are set well below this elevation and other engineered features will be required for adequate flood protection. Table 2 provides a summary of hydraulic-related inputs for track and tunnel portal elevations, by location.

Table 2. Track and Portal Profile Approach Summary

Location	Track Subgrade Outside of Sea Level Rise Area of Influence	Track Subgrade Within Sea Level Rise Area of Influence	Tunnel Portal
San Dieguito Lagoon	—	Varies along the alignment[1]. At Bridge 243.0: Camino Del Mar study 100-year profile, interpolated for 7.1 feet sea level rise (16.2 feet elevation) At the southern extent of the FEMA floodplain: FEMA BFE of 17.0	FEMA 500-year flood elevation (23.6 feet elevation)
Los Peñasquitos Lagoon	FEMA 100-year BFE (varies)	FEMA 100-year BFE + 7.1 feet sea level rise (varies)	FEMA 100-year BFE + 7.1 feet sea level rise (21.3 feet elevation)

Notes:
1 The highest water surface elevation used from the SDDT, Camino Del Mar Bridge, and FIS modeling.
2 All elevations have a vertical datum of North American Vertical Datum of 1988
BFE=base flood elevation; FEMA=Federal Emergency Management Agency

4 Project Impacts and Considerations

There are two main project elements that could be impacted by and/or have an impact on the existing floodplains: track alignment and track profile. Both elements are discussed below for the two alternative alignments.

4.1 Track Alignment

4.1.1 Camino Del Mar Alternative

The proposed track alignment for the Camino Del Mar Alternative essentially follows the existing track alignment and, as such, would have little to no adverse impact on the existing Los Peñasquitos or San Dieguito floodplains. Through Los Peñasquitos Lagoon, this alternative is tied to the 1925 railway alignment, which currently serves to impede flows through the lagoon. This alignment has an opportunity for substantial improvement, by reducing fill segments and replacing with elevated segments. This can only improve the flood conditions through the lagoon and nearby areas.

4.1.2 Crest Canyon Higher Speed Alternative

Although the proposed track alignment for the Crest Canyon Higher Speed Alternative follows the existing track alignment within the San Dieguito floodplain, it traverses the Los Peñasquitos floodplain further inland and for a shorter distance. The proposed alignment also crosses the regulatory floodway at a different location, just downstream of the mouth of Carmel Valley Creek. This portion of the floodway is not defined by the Soledad Canyon hydraulic modeling or the Carmel Valley Creek hydraulic modeling. As such, it would be difficult to validate the proposed impact per FEMA requirements, based on the current model. This area of additional required fill would likely be offset by the removal of the existing alignment fill and result in a general improvement of the flood condition. The Crest Canyon Higher Speed Alternative presents an opportunity to remove the existing railroad berm, thereby alleviating the constraints identified with the existing alignment.

4.2 Track Profile

4.2.1 Camino Del Mar Alternative

Through the Los Peñasquitos Lagoon, the proposed track profile for the Camino Del Mar Alternative would be elevated above the FEMA floodplain (as discussed in Section 3.2). This elevated profile would provide a higher level of track protection to track elements and rail bridges, as compared with the current condition. For segments of fill, the higher track profile requires a larger, wider fill section. This is offset for the Camino Del Mar Alternative by lengthening the segments on elevated structures. This alternative would result in less fill in the Los Peñasquitos floodplain.

For the San Dieguito Lagoon, the track profile is informed by the hydraulic modeling conducted for the proposed bridge replacements at Camino Del Mar and NCTD Bridge 243.0, the highest of which was a water surface elevation of 16.2 feet. The elevation at the end of the bridge is 22.3 feet, which allows for the bridge a portion of the track south of the bridge to meet bridge and subgrade hydraulic design by keeping the subgrade elevation above the energy grade elevation. However, the track profile slopes to the portal elevation of approximately -3.0 feet. Therefore, the track will not meet subgrade criteria at approximately station 15+00. From this location to the tunnel portal, a flood wall should be included in the design to protect the track embankment and prevent overtopping during the 100-year event.

As stated, the north portal at San Dieguito is set too low to meet be protected from the FEMA 500-year water surface elevation. Therefore, some form of flood barrier or exclusion device is recommended at the portal which can be closed to prevent the portal from allowing flood flows into the tunnel. Presumably, rail operations would be halted prior to a 500-year flood event and the portal could be closed with an automated flood gate or similar device.

With the current concepts, the proposed track profile should have no adverse impact on the existing Los Peñasquitos or San Dieguito floodplains.

4.2.2 Crest Canyon Higher Speed Alternative

Through the Los Peñasquitos Lagoon, the proposed track profile for the Crest Canyon Higher Speed Alternative would be elevated above the FEMA floodplain (as discussed in Section 3.2) This elevated profile would provide a higher level of track protection to track elements and rail bridges, as compared with the current condition. For segments of fill, the higher track profile requires a larger, wider fill section. This is offset for the Crest Canyon Higher Speed Alternative by removal of the existing 1925 alignment berm and would result in significantly less fill in the Los Peñasquitos floodplain.

For the San Dieguito Lagoon, the track profile is informed by the hydraulic modeling conducted for the proposed bridge replacements at Camino Del Mar and NCTD Bridge 243.0, the highest of which was a water surface elevation of 16.2 feet. The elevation at the end of the bridge is 22.2 feet, which allows for a portion of the track south of the bridge to meet the bridge and subgrade hydraulic design by keeping the subgrade elevation above the energy-grade elevation. However, the track profile slopes to the portal elevation of approximately -5.0 feet. Therefore, the track will not meet subgrade criteria at approximately station 14+50. From this location to the tunnel portal, a flood wall should be included in the design to protect the track embankment and prevent overtopping during the 100-year event.

As stated, the north portal at San Dieguito is set too low to meet be protected from the FEMA 500-year water surface elevation. Therefore, some form of flood barrier or exclusion device is recommended at the portal which can be closed to prevent the portal from allowing flood flows into the tunnel. Presumably, rail operations would be halted prior to a 500-year flood event and the portal could be closed with an automated flood gate or similar device.

As such, the proposed track profile should have no adverse impact on the existing Los Peñasquitos or San Dieguito floodplains.

4.3 Federal Emergency Management Agency Compliance Strategy

Both alternatives will require meeting FEMA regulatory compliance requirements at San Dieguito and Los Peñasquitos Lagoons. Each floodplain currently includes a designated floodway, which requires no net rise in the 100-year BFE, of the encroached condition. This must be demonstrated with the FEMA effective model. At each of these floodplains, the recommended compliance strategy is similar for each alternative. However, the strategy is different at each floodplain. A recommended compliance approach is provided below.

4.3.1 San Dieguito Lagoon

The proposed features of both alternatives are outside of the regulatory floodway at San Dieguito. Per the floodway function, fill, obstruction, or other improvements within the floodway fringe are acceptable, as the floodway has been mapped assuming a fully encroached floodplain. No hydraulic analysis would be needed to obtain a No-Rise Certificate. Although not needed, the FEMA Effective model would adequately represent hydraulics through the lagoon and has served to inform the one-dimensional modeling used by others for the purposes of informing design.

4.3.2 Los Peñasquitos Lagoon

Both alternatives propose improvements and substantial changes within the regulatory floodway. Although both alternatives are likely to improve the lagoon hydraulics, demonstrating compliance with the FEMA Effective model is potentially problematic. The FEMA Effective model has several shortcomings:

1. It does not include the North Torrey Pines Road bridge, which potentially serves as a flow constriction.
2. It does not include any of the Los Peñasquitos Lagoon rail bridges (Bridges 246.1, 246.8, and 247.1). The effectiveness or ineffectiveness of these bridges are not accounted for.
3. A common water surface across the lagoon is assumed, even for areas where the existing track embankment bisects the lagoon. This may not be a realistic assumption.
4. Cross sections I and J do not span the full width of the floodplain, which suggests improper mapping or inadequate representation of the lagoon. This is also the area between which the Crest Canyon Higher Speed Alternative crosses the floodway, which is not defined by these cross sections or presented within the FIS Floodway Data Table.

Additionally, it is likely that the topographic and bathymetric data are not well represented in this older model. Given the significant level of work and change being proposed within the lagoon, the current FEMA model is not conducive to adequately modeling the proposed condition. It will also be difficult to use the FEMA Effective model to demonstrate a no-rise condition, given the modeling deficiencies.

It is recommended that an LOMR process be completed to remodel the Los Peñasquitos Lagoon floodplain (Soledad Canyon) for existing conditions. This would correct, update, and revise the FEMA floodplain to better represent the current flood extent and dynamics of the lagoon. This should be done ahead of the proposed project to de-couple these necessary changes with the project. Once approved by FEMA, proposed condition modeling could be conducted to demonstrate a no-rise based on this updated floodplain.

5 Conclusions and Recommendations

The Del Mar Alternatives Analysis Drainage Report details design requirements and potential floodplain impacts associated with the two proposed alignment alternatives, as outlined in the *Del Mar Tunnels Alternatives Analysis Report* (HDR 2022). Potential flood risks influence proposed design features, such as portal elevations, fill and bridge locations, and track alignment and profile. Recommended hydraulic design criteria for these features considered SANDAG design criteria, proximity to the sea level rise area of influence, current FEMA flood elevations, and recent hydraulic analyses to limit impacts and risk to project elements.

This report also describes the current FEMA special flood hazard areas (floodplains) at San Dieguito and Los Peñasquitos Lagoons and the compliance requirements related to the National Flood Insurance Program. The drainage design guidelines and recommendations presented are largely reliant on the FEMA floodplain mapping and BFEs, as identified in the current Flood Insurance Rate Maps. The accuracy of this mapping, however, is uncertain, outdated, and may not provide the requisite baseline information to accurately represent potential project impacts and benefits. Although this analysis is considered acceptable for an alternatives analysis effort, it is recommended that the floodplain analysis for Los Peñasquitos Lagoon be revised based on new topography and updated hydrology to inform design during subsequent phases. The impacts of future sea level rise for both Los Peñasquitos and San Dieguito floodplains should also be considered. The updated models can be used to confirm design for project features and evaluate potential project impacts and potential benefits.

It is recommended that the existing floodplain modeling for Los Peñasquitos Lagoon be updated and replaced via a formal LOMR process to better support FEMA compliance. At San Dieguito Lagoon, there is no need to replace the modeling for FEMA purposes, however detailed hydraulic modeling that includes the future Camino Del Mar Bridge and LOSSAN Bridge 243.0, combined with a range of sea level rise scenarios should be completed to inform water surface elevations for the track and tunnel designs and any flood protection features associated with these.

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6 References

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Appendix E. Geotechnical Data and Reconnaissance Reports

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San Dieguito to Sorrento Valley Double Track

Del Mar Tunnels Alternatives Analysis Geotechnical Data and Reconnaissance Reports

San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study

October 2022

Prepared by:

MOTT MACDONALD

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619)699-1900

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1 Introduction

The additional geotechnical efforts included as part of the Del Mar Tunnels Alternatives Analysis, included development of a Geotechnical Data Report and a Geological Reconnaissance Report. These efforts included review of previous and current studies within the project limits and additional borings to provide a better degree of confidence of the existing conditions.

This Appendix includes the two reports commissioned for the San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study (SD-LOSSAN)

1. Geotechnical Data Report (GDR), Los Angeles-San Diego-San Luis Obispo (LOSSAN) Rail Corridor San Diego Regional Rail Corridor Alternative Alignment and Improvement Project, Del Mar and San Diego, California. Earth Mechanics Inc (EMI), dated September 7, 2022.

2. Geologic Reconnaissance Report, Del Mar Alternative Tunnel Alignments Conceptual Engineering Study, San Diego Regional Rail Corridor Alternative Alignment and Improvements Project, Del Mar and San Diego, California. Leighton Associates. October 21, 2022.

2 Scope of Work

2.1 Geotechnical Data Report

The Consultant evaluated previous and current studies and alignments (1) by SANDAG for relocating the LOSSAN alignment through the City of Del Mar between Milepost (MP) 243 and MP 248. The Consultant completed an alternatives analysis (10% preliminary engineering) of each of these segments of the Corridor.

Geotechnical Analysis – The Consultant collected and summarized existing geotechnical explorations and studies, and coordinated borings along the alternatives to better define the ground conditions. Due to the limited access, a small number of borings were performed. The following activities were undertaken.

- Four (4) borings
- Two seismic P and S wave tests
- Two packer tests to determine rock mass permeability
- Two in-situ pressure meter tests
- Soil/Rock material type classification

The borings are near each portal, one boring along the alignment, and one boring near an anticipated fault.

The investigation data, boring logs laboratory and field results are compiled in the Geotechnical Data Report which includes:

- Site specific seismic design criteria.
- Preliminary geotechnical analysis and design, based on available data and field-testing results.

2.2 Geotechnical Reconnaissance Report

Engineering Geology – Consultant performed preliminary geologic assessment of rail alignment alternatives within Del Mar to aid in evaluating geologic constraints. The following activities were conducted:

- Led a one-day tour of the proposed alternatives to better understand the geologic setting and the relationship to the alternatives and the surrounding and adjacent existing structures.
- Performed literature review of readily available geotechnical and geologic maps and reports.
- Documented research to obtain previous geotechnical reports or boring logs along proposed alignments.
- Performed aerial photographic review of proposed alignments to investigate the signs of geologic displacement.
- Performed reconnaissance-level geologic field mapping along the proposed alignments.
- Prepared geologic maps using topographic maps and alternative alignments. Where identified, geologic maps include mapping of geologic discontinuities, mapped active, potentially active and inactive faulting.
- Ranking of geologic constraints.

The Geotechnical Reconnaissance Report includes discussion of geologic units, groundwater data, geologic engineering characteristics and geologic hazards.

3 Summary of Conclusions and Recommendations

The results of the additional geotechnical efforts are summarized within the Geotechnical Data and Reconnaissance Reports. In addition, construction considerations and future geotechnical investigation recommendations are also included in the Del Mar AA report, but are repeated within this memo for clarity.

3.1 Geotechnical Data Report

3.1.1 Preliminary Design Recommendations

The Geotechnical Data Report presents Preliminary Design Recommendations as:

- Preliminary Seismic Design
- Rock/Soil Design Parameters
- Earth Pressures at Portals

3.1.2 Future Geotechnical Investigations

For the next phase of design for the proposed San Diego Regional Rail project the geotechnical exploration investigation should address the following items:

- Determine the geologic subsurface conditions along the proposed tunnel alignment at depth.
- Collect additional groundwater data and provide quantitative data using piezometers along the proposed tunnel alignment to characterize hydrogeologic conditions.
- Collect additional data to document variability within subsurface geologic formations.
- Conduct additional investigation for the mapped fault crossing the proposed alignment to determine impacts on the proposed tunnel design and construction.
- Perform additional in-situ testing to further develop subsurface design parameters along the proposed tunnel alignment including soil modulus and permeability values.
- Collect additional shear wave velocity data for the purposes of characterizing the subsurface conditions for design and construction.

The next phase of investigation should be anticipated to consist of additional borings along the chosen alignment alternative. Soil borings should be anticipated to be drilled and sampled below the proposed tunnel invert with depths reaching up to 300 feet below existing grade or boring should be continued one diameter past the invert of the tunnel. Vibrating wire piezometers/pressure transducers (in combination with pump and/or packer tests) should be installed within borings along the proposed tunnel alignment to quantify groundwater impacts to the proposed tunnel design. Borings should include rock coring to provide samples to be characterized for strength testing, rock quality designation and in-situ testing including packer and pressuremeter testing. A geophysical seismic refraction investigation should be conducted to better define the actual location of the existing fault crossing and any sheared zones along the proposed alignment. Trenching to expose the fault may be required to determine fault orientation. Additional P & S wave suspension logs should also be conducted to provide additional data for tunnel construction and design, and development of detailed earthquake ground motion criteria.

3.1.3 Construction Considerations

3.1.3.1 Portals

Portals are currently anticipated to consist of U-structures transitioning to cut-and-cover or directly to the bored tunnel section. A support of excavation (SOE) system is anticipated to be required to be required to retain the existing ground on the sides of the portal. SOE could consist of a driven steel soldier pile and timber lagging system. From the limited borings drilled the present phase of design, both excavation and installation of SOE can be achieved with conventional construction equipment. Representative geologic materials and groundwater conditions will be discussed when additional subsurface investigations are performed in the future.

3.1.3.2 Bored Tunnel

The majority of the proposed tunnel alignment will be excavated within the sedimentary rock associated with both the Torrey Sandstone and Delmar Formations. Tunnel excavations should anticipate encountering soft rock conditions consisting of sandstone, siltstone and claystone associated with these formations. Based on the limited preliminary investigation and site assessment, the anticipated rock mass conditions should be considered rippable for a bored tunnel excavation.

Additional investigation will be needed as part of the next phase of design to better define the subsurface conditions from a geotechnical and geologic standpoint.

3.2 Geotechnical Reconnaissance Report

3.2.1 Corrosivity

With the proposed tunnel alignments crossing through marine deposits, their general proximity to the Pacific Ocean, and their potential susceptibility to encountering groundwater seepage conditions, we recommend that a corrosion engineer be retained during the design phase of the subject project. In addition, the tunnels should be designed and constructed in accordance with the guidance of the Services Life Design Guide for Corrosion Prevention of Concrete Structures in San Diego County (SANDAG, 2015).

3.2.2 Fault Classification

The proposed tunnel alignments are not located within a fault rupture hazard zone or within 1,000 feet of an active fault (15,000 years and younger); therefore, following Caltrans guidelines (Caltrans, 2017), further evaluation to investigate for surface fault rupture is not required.

3.2.3 Un-named Fault

Faulted, sheared bedrock, and seepage should be anticipated and accounted for where the unnamed fault crosses the tunnel alignments.

3.2.4 Shallow Ground Rupture

Ground rupture due to faulting is not considered a significant hazard in these areas although it should be considered as a possibility throughout San Diego County.

3.2.5 Liquefaction and Seismic Settlement

The proposed tunnel alignments are located within the mapped limits of the potentially liquefiable young alluvial and intertidal- estuarine deposits which are associated with the San Dieguito and Los Peñasquitos Lagoons.

3.2.6 Lateral Spreading

Due to the low potential for liquefaction and the deep nature of the proposed tunnel alignments, the potential for lateral spreading or flow failure is very low, except at the north portal locations where potential for lateral spreading exists

3.2.7 Tsunamis

The potential for damage due to a tsunami is low, except at the north portals which are located at the boundary of tsunami inundation areas.

Attachment 1: Preliminary Geotechnical Report

Attachment 2: Geologic Reconnaissance Report

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

September 27, 2022

EMI Project No. 20-134

HDR Engineering, Inc. (HDR)
591 Camino De La Reina, Suite 300
San Diego, CA 92108

Attention: Mrs. Kim Magee, PE

Subject: Preliminary Geotechnical Report (PGR)
Los Angeles-San Diego-San Luis Obispo (LOSSAN) Rail Corridor
San Diego Regional Rail Corridor Alternative Alignment and Improvement Project
Del Mar and San Diego, California

Dear Mrs. Magee:

Earth Mechanics, Inc. (EMI) is pleased to present this report summarizing the results of our geotechnical field investigation within the cities of Del Mar and San Diego, and providing preliminary design recommendations as part of the LOSSAN Rail Corridor Project in San Diego County, California. The project consists of developing and designing improvements to the 60-mile San Diego segment of the LOSSAN corridor. The geotechnical investigation is focused on obtaining geotechnical data to supplement the preliminary conceptual design of the proposed double track alignment alternatives between Del Mar Fair Grounds and Sorrento Valley that is currently being delivered by HDR Engineering, Inc. (HDR) under contract to the San Diego Association of Governments (SANDAG).

EMI prepared this PGDR in accordance with the requirements outlined in the project design criteria. The report documents the results of our subsurface explorations and laboratory testing for the project. Review comments received to-date have been incorporated in this version. We appreciate the opportunity to provide geotechnical design services for this project. If you have any questions, please call us at (714) 751-3826.

Sincerely,

EARTH MECHANICS, INC.

Patrick Wilson, PE 77983
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
PATRICK WILSON
NO. C77983
EXP. 6-30-2023
CIVIL
STATE OF CALIFORNIA

Thomas Feistel, PG 9590
Senior Staff Geologist

PROFESSIONAL GEOLOGIST
Thomas Benjamin Feistel
No. 9590
EXP. 02/29/2024
STATE OF CALIFORNIA

Hubert Law, RCE 55784
Principal

REGISTERED PROFESSIONAL ENGINEER
Hubert K. Law
No. C055784
12-31-2022
CIVIL
STATE OF CALIFORNIA

Michael Hoshiyama, CEG 2599
Senior Project Geologist

CERTIFIED ENGINEERING GEOLOGIST
MICHAEL Y. HOSHIYAMA
NO. 2599
EXP. 01-31-2024
STATE OF CALIFORNIA

PRELIMINARY GEOTECHNICAL REPORT
LOS ANGELES-SAN DIEGO-SAN LUIS OBISPO (LOSSAN) RAIL CORRIDOR
SAN DEIGO REGIONAL RAIL CORRIDOR ALTERNATIVE ALIGNMENT AND IMPROVEMENT PROJECT, DEL MAR AND SAN DIEGO, CALIFORNIA

Prepared for:

HDR Engineering, Inc.
591 Camino De La Reina, Suite 300
San Diego, CA 92108

Prepared by:

Earth Mechanics, Inc.
17800 Newhope Street, Suite B
Fountain Valley, California 92708

EMI Project No. 20-134

September 27, 2022

Earth Mechanics, Inc.
Geotechnical & Earthquake Engineering

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APPENDICES

1 INTRODUCTION

1.1 PROJECT DESCRIPTION

The LOSSAN Rail Corridor is a 351-mile stretch of track that connects the major metropolitan areas of Southern California and the Central Coast. The rail corridor extends between San Luis Obispo and San Diego with current train operations that include the Amtrak Pacific Surfliner, the Southern California Regional Rail Authority (SCRRA) Metrolink, North Coast Transit District's Coaster and Sprinter passenger rail services, and the Union Pacific and BNSF freight rail services.

SANDAG's LOSSAN corridor capital improvement program is focused on improving the 60-mile San Diego Subdivision of the LOSSAN Rail Corridor. The goal of the program is to significantly increase passenger and freight Level of Service (LOS) by proposing double tracking, bridge replacements and station improvements that will be needed in order to provide additional passenger rail service as an alternative to vehicular travel along Interstate 5.

Locally, the existing LOSSAN Rail Corridor between Del Mar Fairgrounds and Sorrento Valley is a single-track alignment that runs along the Del Mar Bluffs. Failures along the Del Mar Bluffs have been very common throughout the years and have been documented as far back as 1940 when a passenger train was derailed due to a bluff failure. Mitigation measures have been ongoing to help stabilize the bluffs. As a result of the continual issue related to bluff failures along the single track alignment, a new conceptual double track tunnel corridor is being proposed to be constructed through the City of Del Mar to divert the existing corridor away from the coastal bluffs.

1.2 PURPOSE AND SCOPE OF STUDY

1.2.1 PURPOSE

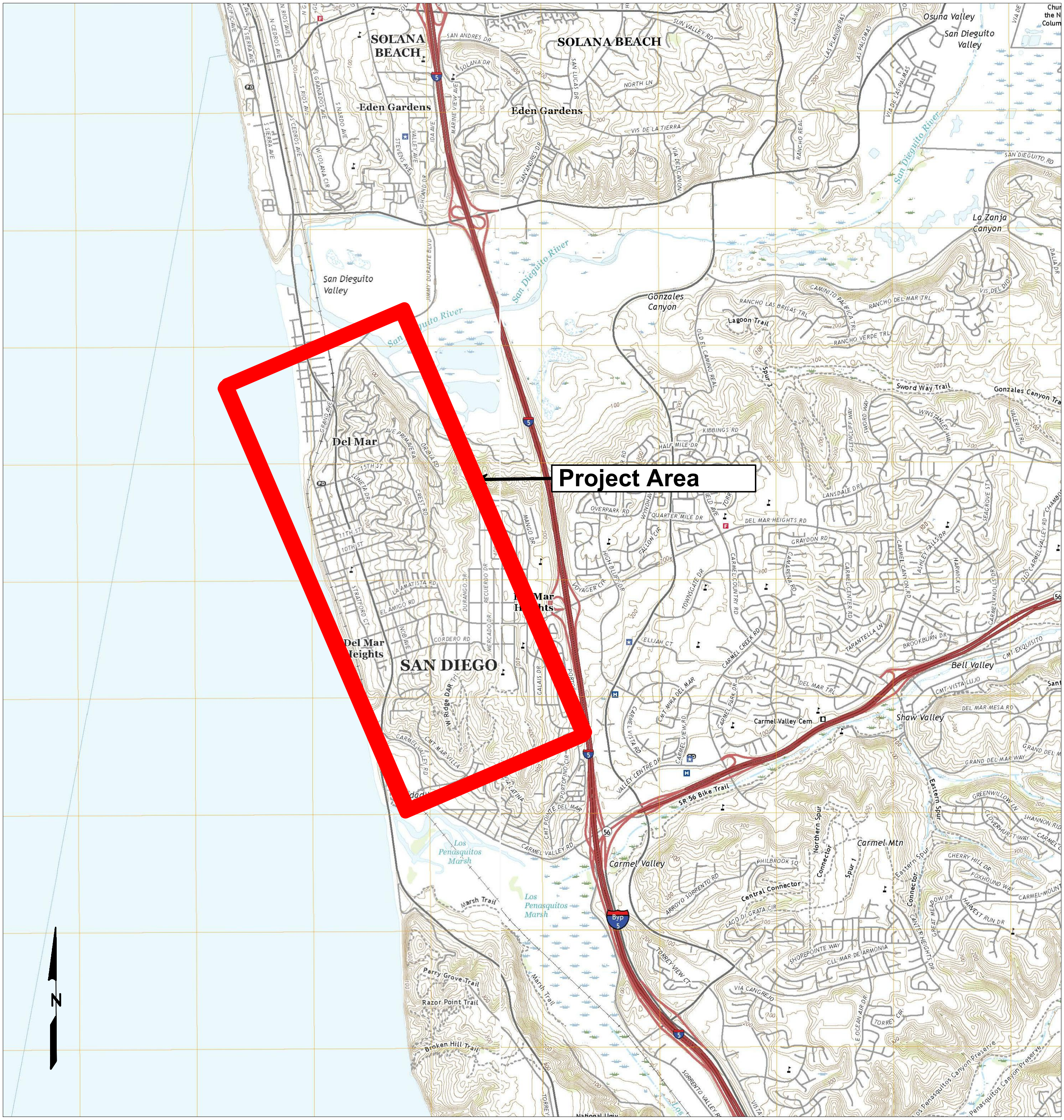
Specifically, the purpose of this Preliminary Geotechnical Design Report (PGDR) is to present the geotechnical data collected in the four borings drilled by Earth Mechanics, Inc. (EMI) on along the proposed alignments for a new double track tunnel to replace the existing single track between Del Mar Fairgrounds and Sorrento Valley. In addition to the above geotechnical data, this report also provides preliminary geotechnical design recommendations to assist HDR and SANDAG in the preliminary and conceptual design of the subject corridor. Further investigation with addition borings will be performed in the future phases of the project. There is a separate geology report prepared by Leighton providing a desktop study of geologic conditions complimentary to this report (Leighton, 2022).

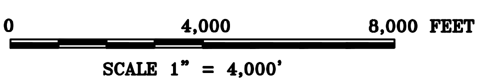
1.2.2 SCOPE

A geotechnical subsurface investigation was performed to obtain field and laboratory testing information on subsurface conditions to support the preliminary design. In order to satisfy the project requirements, the subsurface investigation performed by EMI included the following:

- Review of existing geotechnical, geological and seismological data;
- Geotechnical field investigation including drilling boreholes at four locations, as shown in Figures 4a-4d varying in depth from approximately 90 feet to 200 feet to characterize the subsurface stratigraphy;

- Collecting soil and bedrock samples from borings for visual classification and laboratory testing.
- Performing P-S wave downhole suspension logging to measure average shear wave velocity of subsurface materials near the proposed tunnel portals.
- Perform in-situ pressuremeter and packer testing to collect some engineering data on representative rock formations that are anticipated to be encountered within the proposed tunnel alignment
- Conducting laboratory testing on selected representative soil samples to assist in developing soil index and engineering parameters for preliminary geotechnical design.
- Provide preliminary seismic evaluation and design parameters.
- Preparation of a preliminary geotechnical design report to present EMI's current findings, preliminary conclusions, and recommendations.





REFERENCE: USGS Topographic 7.5 Minute Quadrangle Map - Del Mar OE W and Del Mar Quadrangle (2018)

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	**SD Regional Rail Corridor Alternative Alignment and Improvement Project**	Site Location Map
	Project No. 20-134 / Date: May 2021	Figure 1

2 PHYSIOGRAPHY AND GEOLOGIC SETTING

2.1 Regional Geology

The project site is located along the northwestern coast of San Diego within the greater Peninsular Ranges geomorphic province. The Peninsular Ranges geomorphic province extends nearly 900 miles from the Transverse Ranges and Los Angeles Basin in the north to the tip of Baja California in the south. The province is bounded on the east by the Colorado Desert, on the west by the Pacific Ocean, and ranges from approximately 30-100 miles in width. The region is characterized by uplifted terraces along the ocean segmented by drainages coming out from the mountainous region that makes up eastern San Diego. The mountain ranges in eastern San Diego County are predominantly made up of harder igneous and metamorphic rock assemblages while the coastal rocks tend to be comprised of softer sedimentary rocks (Kennedy and Tan, 2008). The San Andreas fault which trends northwest to southeast, is the dominate fault in the area. The other major faults in the area, including the Rose Canyon fault, tend to parallel the San Andreas fault.

In San Diego County, the Peninsular Ranges province is often subdivided into: a western coastal plain subzone (also known as the San Diego Embayment), a central mountain subzone, and an eastern desert subzone. The project corridor is located in the western coastal plain in the uplifted section between the San Dieguito Valley to the north and the Los Penasquitos Lagoon to the south.

2.2 Stratigraphy

The project area is underlain predominately by three rock units (Figure 2), Paralic Deposits, Torrey Sandstone formation and the Delmar Formation (Kennedy and Tan, 2008). Also, there is artificial fill underlying the homes and roads in the area. Descriptions of the major soil and rocks units within the site vicinity are below:

1. Artificial Fill (Af), Recent, fill is generally associated with existing developments and structures. Fills encountered at the project site generally consisted of reworked alluvial silty sands with some gravels.

2. Old Paralic Deposits (Qop) late to middle Pleistocene, unconsolidated silty to clayey sand and sandy clay
3. Very Old Paralic Deposits (Qvop) middle to early Pleistocene, moderately permeable, reddish-brown, inter-fingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone, and conglomerate.
4. Torrey Sandstone (Tt) middle Eocene, white to light brown, medium to coarse grained, moderately well indurated, massive and broadly cross-bedded, arkosic sandstone.
5. Delmar Formation (Td) middle Eocene, dusky yellowish green, sandy, claystone interbedded with medium to gray, coarse grained, clayey to silty sandstone.

Based on field investigation, artificial fill thicknesses across the project area site are generally on the order of 3 to 10 feet thick but may be thicker locally. The artificial fill is generally a re-working of one of one of the three rock units in the area. The artificial fill that was encountered consists mostly of medium-dense to dense silty sand that is moist and weakly cemented.

The very old paralic deposits were encountered in boring R-21-003 where they are a capping unit over the Torrey Sandstone. Generally, the unit is distinguishable from the Torrey Sandstone by its reddish brown tint. Geologic maps of the area break up the very old paralic deposit rock unit into multiple smaller units, as can be seen on the Regional Geologic Map (Figure 2). Besides at the portal locations the project is not likely to encounter any old and very old paralic deposits, even though they overly much of the project area.

The very old paralic deposits encountered during the investigation were generally fine to coarse grained sands with trace silts to sands with higher silt content. The deposits were generally moist and medium dense to dense. The very old paralic deposits unconformably overly the Torrey sandstone. The contact between the two units is believed to be largely flat to slightly undulating.

The Torrey Sandstone is predominately a lighter colored, white to yellowish white, coarse arkosic sandstone. Based on the samples recovered from the borings, the Torrey Sandstone was generally light in color, ranging from white to light brown to light gray, friable, moderately soft, and medium to coarse grained with few gravel layers.

For the majority of the project corridor, the Torrey Sandstone overlies the Delmar Formation. The Delmar Formation is predominately comprised of finer grained claystone and siltstone, with interbedded units of silty to clayey sandstone. The Delmar Formation was soft to moderately soft, darker in color than the other two formations and is generally a dark gray to olive. It should be noted that the contact between the Torrey Sandstone and Delmar Formation is transitional as the depositional environment was a marine transgression.

A subsurface cross section depicting the stratigraphy along the project corridor (Crest Canyon alignment) is presented in Appendix B.

2.3 STRUCTURE

The project site is located within a seismically active region of Southern California. The region includes multiple Holocene-active fault systems: the Newport-Inglewood-Rose Canyon fault zone, Coronado Bank and San Clemente fault zones off shore, and farther east the San Jacinto and Elsinore faults.

The region surrounding San Diego Bay, particularly offshore to the west, is transected by a series of long, predominantly northwest- trending, strike-slip fault systems. Most of the faults in the offshore region are poorly known but clusters of aligned earthquakes, displaced young strata, and geomorphology suggest that they are active. The faults that form these fault systems comprise a network of closely spaced branching and discontinuous features which together form major linear fault zones. In addition to faulting, these zones are commonly interconnected by zones of uplifting and folding and hence are sometimes called zones of deformation rather than fault zones.

The general structure of the coastal San Diego Area is predominately controlled by the Rose Canyon fault which is a right-lateral strike-slip fault. The movement along this fault, as well as the greater tectonic forces of the area, has created the alternating low valleys and costal cliffs along San Diego's western coastal areas. The Del Mar and corresponding project area is a northwest-trending uplifted marine terrace composed predominately of the Torrey Sandstone and Delmar

Formations. These formations are capped by the younger Paralic Deposits. The Paralic Deposits unconformably overlie the Torrey Sandstone, which is conformably in contact over the Delmar Formation. In this area the Torrey Sandstone and Delmar Formations gently dip to the southeast approximately 5 degrees, however, the northern portions of this area show dips more northward (Kennedy and Tan, 2008).

2.3.1 FAULTING

The project site is located within a seismically active region of Southern California. The region includes multiple Holocene-active fault systems: the Newport-Inglewood-Rose Canyon fault zone, Elsinore fault, San Felipe fault, and the San Jacinto fault. These faults are identified as Alquist-Priolo (AP) Earthquake Fault Zones defined by the Alquist-Priolo Earthquake Hazards Act of 1972 revised in 1994. The AP faults not only represent earthquake shaking hazards, but have a potential for surface ground rupture. The type and magnitude of the seismic hazard affecting the site are dependent on the distance to causative faults and the intensity and magnitude of the seismic event. Other potentially active faults, such as the Pt. Loma fault, may not be identified as AP Earthquake Fault Zones because their locations are not well-defined and/or they have not generated earthquakes within the last 11,000 years. The project corridor does not enter into any AP fault zones and does not cross any active fault traces.

Offshore Holocene-active faults include the Coronado Bank fault, the San Diego Trough fault zone, San Clemente fault and the Santa Cruz Catalina Ridge. All relevant active faults are shown on Figure 4. Locally, an unnamed fault that extends across the project area is shown on Figure 2.

Unnamed Fault

An unnamed mapped fault passes through the proposed rail alignments. Based on geologic reconnaissance by Leighton (2022), the fault is considered pre-Holocene and does not appear to offset younger overlying Quaternary deposits. The fault depicted is on the subsurface cross section in Appendix B as it offsets the Torrey Sandstone and Delmar Formation and an is also mapped on the Regional Geologic Map in Figure 2. The fault is not considered active and is classified as a Quaternary fault.

Newport-Inglewood-Rose Canyon Fault Zone

The Newport-Inglewood-Rose Canyon fault system is a complex zone of north- to northwest-trending right-lateral strike-slip fault segments extending from the Los Angeles basin to south of the US-Mexico border. Much of San Diego's topography, including Mount Soledad, Mission Bay, and San Diego Bay, is a result of large-scale uplifts and structural depressions due to the complex geometries of the fault system. The fault is located offshore, 2.5 miles west of the project site.

In the San Diego area, the Rose Canyon segment of the fault comes onshore at La Jolla and is characterized by zones of compression and extension associated with restraining and releasing bends in the faults. Locally, the fault zone is over 1 km in width and is composed of both dip-slip and strike-slip en echelon faults (Treiman, 2002). The onshore portion of the fault system extends from the eastern flank of Mount Soledad and continues southward to Mission Bay. Between Mission Bay and San Diego Bay, the Downtown Graben is a zone of north-trending faults within the Rose Canyon fault zone mapped in the East Village area of downtown San Diego (Treiman 2002). Within San Diego Bay, the fault system branches into three principal faults: the Spanish Bight, Coronado, and Silver Strand faults. To the south of San Diego Bay, the southern reach of the Silver Strand fault appears to step to the west to the Descanso fault, which is mapped offshore of Rosarito Beach, Mexico.

The fault slip rate of about 1.5 mm/year is based on detailed trenching along the main trace of the Rose Canyon Fault in Rose Creek (Lindvall and Rockwell 1995). Lindvall and Rockwell determined that at least three significant earthquakes, and possibly as many as six, occurred during the Holocene.

Pt. Loma Fault

The Pt. Loma fault is a Late Quaternary-aged normal fault. Trending north-northwest, the fault is located approximately 13.0 miles south of the project site along the east side of Point Loma Peninsula.

San Felipe Fault

Like the Elsinore fault, the 170 km long San Felipe fault is part of a network of northwest-trending dextral faults within the San Andreas fault system (Steely et al., 2009). The San Felipe fault zone has ~5.8 ± 2.8 km of right separation and is late Quaternary-aged. The fault is located approximately 78.0 miles east of the project site and is capable of producing a M6.3 earthquake.

San Jacinto Fault

The San Jacinto fault is a major strike-slip fault zone that runs through San Bernardino, Riverside, San Diego, and Imperial counties in Southern California. 244 km in length, the San Jacinto fault is a component of the larger San Andreas fault system. This is the most seismically active fault in southern California (Peterson et al., 1996), with significant earthquakes (larger than M5.5), including surface rupturing earthquakes in 1968 (M6.6 Borrego Mountain earthquake) and 1987 (M6.6 Superstition Hills and M6.2 Elmore Ranch earthquakes), and numerous smaller shocks within each of its main sections. Slip rates in the northern half of the fault system are around 12 mm/yr, but are only around 4 mm/yr for faults in the southern half where strands overlap or are sub-parallel. The fault is located 75 miles east of the project site.

Elsinore Fault

The nearest segment of the Elsinore fault (Julian segment) is located approximately 48 miles northeast of the project site. The Elsinore fault is a 250-km-long, right-lateral strike-slip fault and is a significant part of the San Andreas fault system. Trending northwest, the fault lies along the west side of the Salton Trough near the US-Mexico border north to Corona where it branches into the Whittier and Chino faults. The central portion of the fault includes the Glen Ivy, Temecula, Julian, and Coyote Mountain segments. The Julian segment of the fault consists of multiple strands and has a late Quaternary slip rate of 3 to 6 mm/yr (Vaughan and Rockwell, 1986). The slip rate for the entire Elsinore fault is estimated at about 5 mm/yr. Based on length and magnitude relationships, the fault is estimated to be capable of producing a M7.7 earthquake.

Offshore Faults

Several regional faults are located in the offshore continental borderland. The Coronado Bank, San Diego Trough, San Clemente fault, and Santa Cruz Catalina Ridge fault zones form a wide zone of northwest trending strike-slip faults that lie offshore, between 18 and 65 miles west of the project site. These faults are of significant length and are documented by marine geophysical methods to have offset either the shallowest seafloor sediments or the seafloor itself (Ryan and others, 2009). These faults have a slip rate in the range of 1 to 3 mm/yr and are capable of producing M7.4-7.5 earthquakes.

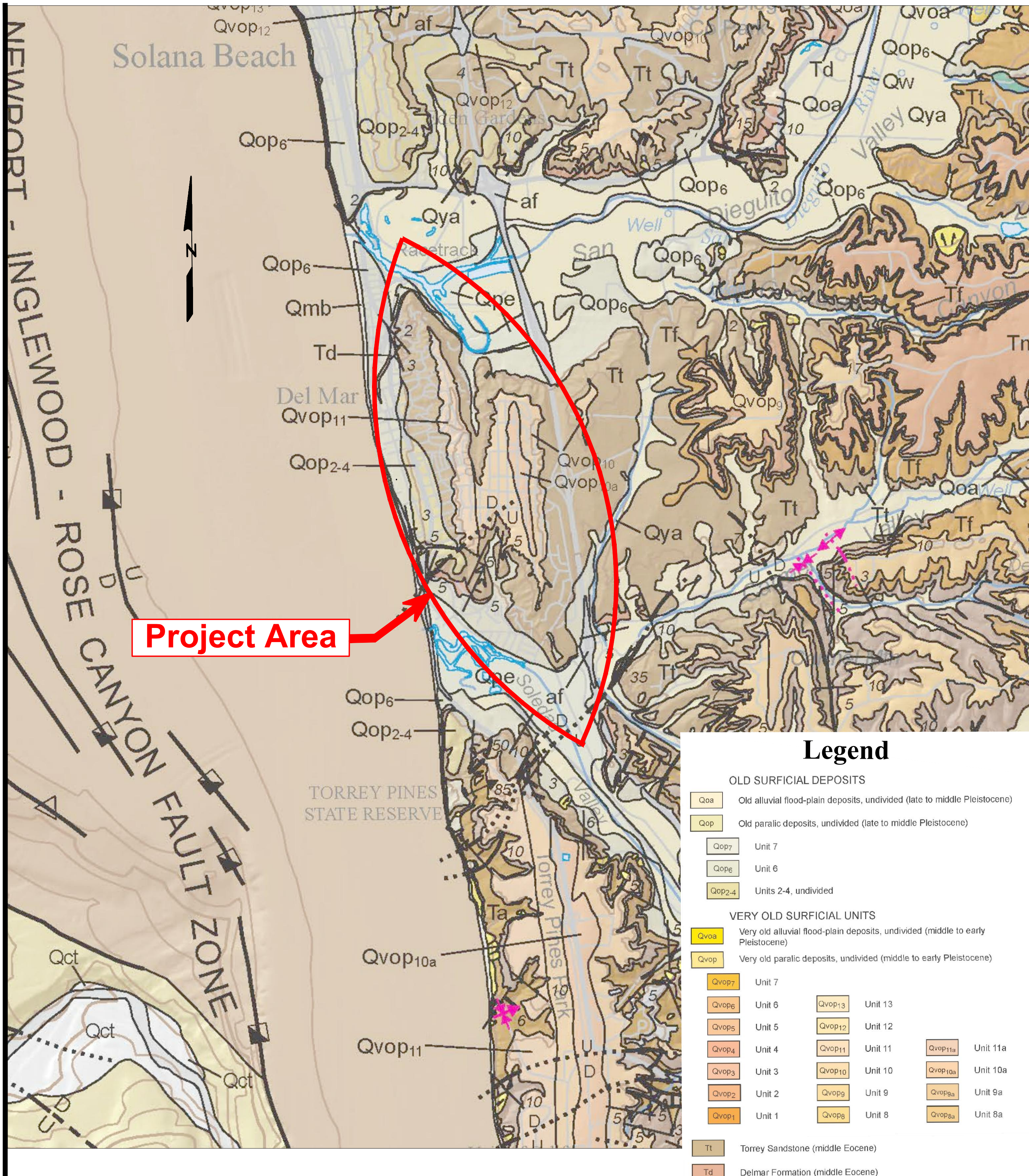
2.4 SEISMICITY

While San Diego does not have as great a frequency of historical earthquakes as other portions of southern California, historical epicenter maps show seismic activity throughout the region. The largest historical earthquakes in the San Diego region were the magnitude 6.5 1800 earthquake, which damaged the mission at San Juan Capistrano (then under construction) and adobe barracks at San Diego, the 1803 magnitude 5.0 earthquake and the 1862 magnitude 5.9 earthquake, believed to have occurred on either the Rose Canyon or Coronado Bank faults. While the project area is located in seismically active southern California, there is no clustering or alignment of earthquakes in proximity to the site. This apparent lack of earthquake activity suggests that the project area has been tectonically stable and suggests that there are no unrecognized active faults at the site.

2.5 GROUNDWATER

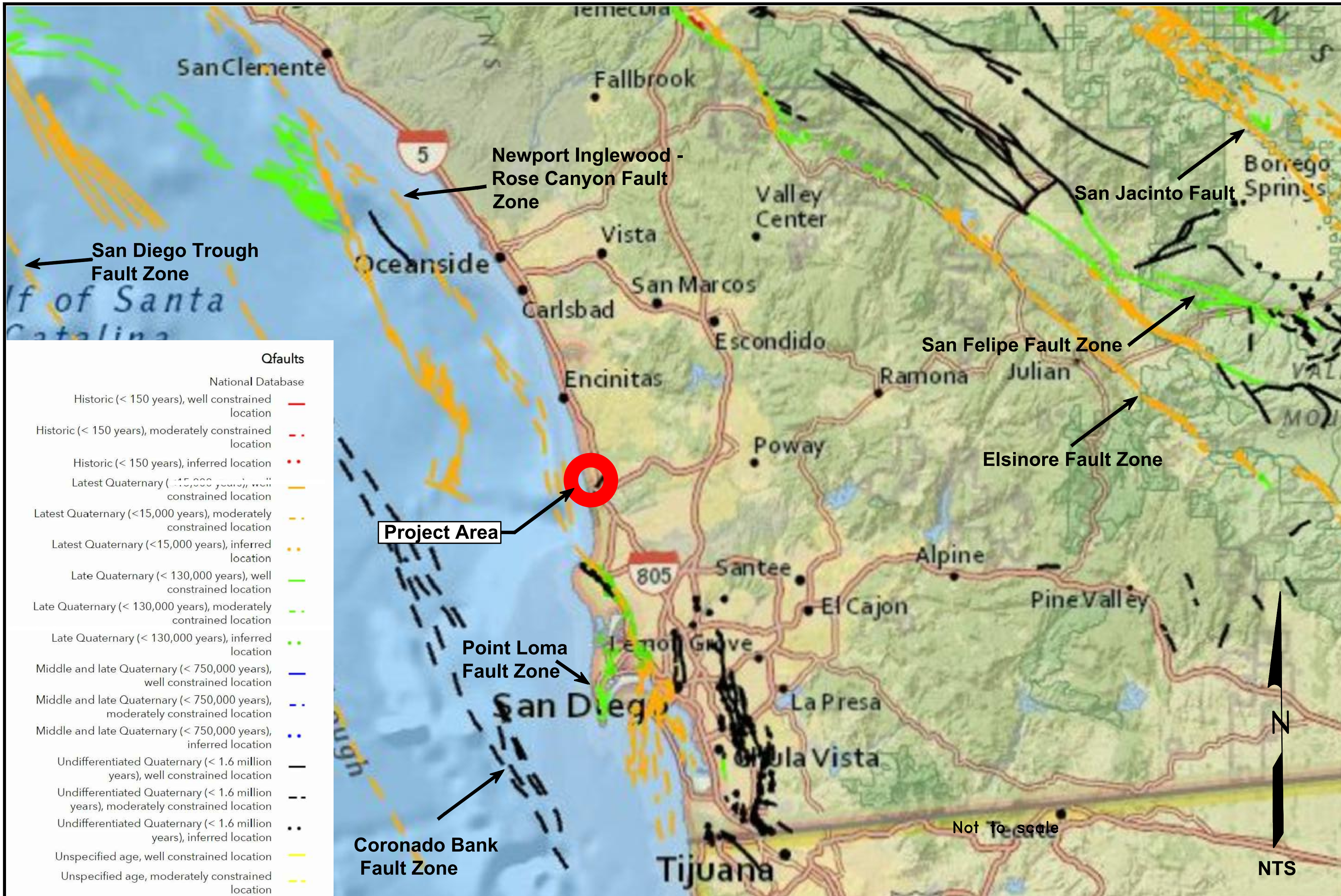
Perched groundwater/seepage was encountered at elevation +139.5 feet in boring R-21-002 and at elevation +91.1 feet in boring R-21-004. The GeoTracker website (http://geotracker.waterboards.ca.gov/) shows several groundwater monitoring wells at a site along Camino Del Mar and 9th Street conducted between 1996 and 1999. The wells indicate groundwater varying between elevations +112.4 feet and +125.4 feet. Based on the data, the groundwater readings are just above the contact with the Delmar Formation. Additionally, according to Gregg Drilling, they encountered groundwater near Torrey Pines Road and Carmel Valley Road at a depth of 27 feet (near elevation +0 feet MSL).

Perched groundwater should be anticipated along the contacts with the sedimentary bedrock formations. Static groundwater is anticipated to be closer to sea level while being subject to seasonal and tidal fluctuations. Due to the potential impacts of groundwater on the proposed tunnel design, a more detailed investigation should be conducted to determine groundwater levels and pressures across the project area. Installation of vibrating wire piezometers at the tunnel invert would allow for quantifying the impacts of both the static and perched groundwater on the proposed tunnel design.



REFERENCE: Kennedy, M.P., Tan, S.S, 2008, Geologic Map of the San Diego 30' x 60' Quadrangle, California, California Department of Conservation California Geological Survey, Regional Geologic Map Series, 1:1,000,000 Scale, Map No. 3.

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	SD Regional Rail Corridor Alternative Alignment and Improvement Project	Regional Geologic Map
	Project No. 20-134 / Date: May 2021	Figure 2



Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

SD Regional Rail Corridor Alternative Alignment and Improvement Project

Project No. 20-134

Date: May 2021

Regional Fault Map

Figure 3

3 GEOLOGIC AND SEISMIC HAZARDS

3.1 GEOLOGIC HAZARDS

3.1.1 Landsliding

Generally, landslides are downslope movements of conglomerations of soils or bedrock or combinations of both. Landslides can move in a translational or rotational motion. Landslides occur because of the loss of ability of earth materials to maintain their integrity at a specific gradient and as a result move into a less steep gradient or position of greater stability. The internal strength of the earth material is lost and the material settles into a form where the mass is centralized on the downhill side of motion. The earth mass is generally a cohesively connected unit that settles or moves as a unit. Landslides are usually associated with the presence or introduction of water; water increases the unit weight of the earth mass and decreases the shear strength of the earth materials. The chances of a landslide occurring are increased by: steeper slope gradients, decreased shear strength of earth materials, unfavorable bedding (out of slope), clay content of the soil or clay seams in bedrock, unfavorable slope orientation with existing fault boundaries, human disturbance of the earth mass or its boundaries, increased water content in the soil or bedrock, underground springs or rise in groundwater within the earth mass, kinematic forces due to earthquake shaking, and disturbance of lateral confining forces and/or the toe of a slope.

The project area is composed of elevated but predominately flat terrain in the northwestern part of San Diego. Mapping in the area shows the bedrock is dipping very shallowly at approximately 5 degrees to the east and southeast. There are a few landslides mapped on the northwestern portion of the project. According to Leighton (2022), a minor slope failure was observed along the coastal bluffs approximately 400 feet north of Jimmy Durante Blvd and Camino Del Mar. Landsliding is common within the Del Mar area as the local sandy bluffs are susceptible to failures particularly due to heavy rainfall and abundant groundwater seepage. The proposed tunnel portals may be susceptible to landsliding and seismically induced landsliding, thusfurther investigation and analysis will need to be conducted at the proposed portal locations during the design phase. For additional discussion of portal construction considerations, see Section 8.1.

3.1.1 EROSION

Terrestrial slope erosion is a process caused by gravitational failure abetted by water saturation of bluff materials and mass loading at the top of the bluff. Surface and groundwater hydrology combined with the geologic nature of the bluff control the effectiveness of terrestrial erosion. As discussed above, the existing bluff slopes within the Del Mar area are susceptible to failures and erosion due to water saturation by way of groundwater seepage from irrigation and precipitation. Perched water seems to accumulate at higher elevations and was observed within the borings at the site. Seepage is also evident based on the amount of existing vegetation along the existing slope faces around the project area. This does provide some added stability to the slope face by way of reducing soil saturation and added tensile strength to the surficial soil layer. The tunnel portal sites should anticipate erosion and potential for slope failures due to the existing slope geometry, bluff lithology, and groundwater seepage observed within the project site area.

3.1.2 Expansive Soils

Expansive soils swell or heave with increase in moisture content and shrink with decrease in moisture content. Montmorillonitic clays are most susceptible to expansion. The Delmar Formation consists of interbedded siltstone, claystone and sandstone. Within the formation, interbeds of high plasticity claystone were encountered at varying depths within the tunnel horizon zone. As a result, expansive soils should be anticipated to be encountered within the proposed tunnel alignment. Further investigation and testing should be conducted during the design phase.

3.1.1 Rippability

Rippability of rock is related to its hardness, joint spacing and compressional wave velocity. Based on the preliminary data from this investigation, the sedimentary rock at the site has seismic velocities ranging from 2,900 to 7,500 feet per second. The sedimentary rock at the site should be considered rippable with some of the more cemented units and layers categorized as marginally rippable. For additional discussion on tunnel construction considerations, see Section 8.2.

3.2 Seismic Hazards

3.2.1 Surface Fault Rupture

In general terms, an earthquake is caused when strain energy in rocks is suddenly released by movement along a plane of weakness. In some cases, fault movement propagates upward through the subsurface materials and causes displacement at the ground surface as a result of differential movement. Surface rupture usually occurs along traces of known or potentially active faults, although many historic events have occurred on faults not previously known to be active. Seismicity within this region is a result of the dominantly reverse-slip regime of the region.

The California Geological Survey (CGS) establishes criteria for faults as active, potentially active or inactive. Active faults are those that show evidence of surface displacement within the last 11,000 years (Holocene age). Potentially active faults are those that demonstrate displacement within the past 1.6 million years (Quaternary age). Faults showing no evidence of displacement within the last 1.6 million years may be considered inactive for most structures, except for critical or certain life structures. In 1972 the Alquist-Priolo Special Studies Zone Act (now known as the Alquist-Priolo Earthquake Fault Zone Act, 1994, or Alquist-Priolo Earthquake Hazards Act APEHA) was passed into law which requires studies within 500 feet of active or potentially active faults. The APEHA designs “active” and “potentially active” faults utilizing the same age criteria as that used by the CGS.

The project alignment does cross the mapped unnamed fault between Station 111+00 and 112+00 along the Crest Canyon alignment. This fault is not mapped as active but is mapped as potentially active. Though the fault offsets the Torrey Sandstone and Delmar Formations, there is no evidence that the fault displaces the capping Quaternary age very old paralic deposits. As a result, the potential for surface fault rupture within the proposed project corridor is considered low.

3.2.2 Seismic Shaking

The energy released during an earthquake propagates from its rupture surface in the form of seismic waves. The resulting strong ground motion from the seismic wave propagation can cause significant damage to structures. At any location, the intensity of the ground motion is a function of the distance to the fault rupture, the local soil/bedrock conditions, and the earthquake magnitude. Intensity is usually greater in areas underlain by unconsolidated earth material than in areas underlain by more competent rock.

Earthquakes are characterized by a moment magnitude, which is a quantitative measure of the strength of the earthquake based on strain energy released during the event. The magnitude is independent of the site, but is dependent on several factors including the type of fault, rock type, and stored energy. Moderate to severe ground shaking will be experienced in the project area if a large magnitude earthquake occurs on one of the nearby principal late Quaternary faults; moderate to severe ground shaking may cause structural damage to the on-site improvements.

Due to the proximity of the project area to numerous seismic sources (Figure 3), strong to moderate ground shaking should be anticipated within the project alignment in the event of a major earthquake from a nearby seismic source. All structures should be designed for site specific groundshaking demands in accordance with the latest applicable seismic design criteria.

3.2.3 Soil Liquefaction

Liquefaction is a phenomenon whereby saturated granular soils lose their inherent shear strength due to increased pore water pressures, which may be induced by cyclic loading such as that caused by an earthquake. Low relative density granular soils, shallow groundwater, and long duration and high acceleration seismic shaking are some of the factors favorable to cause liquefaction.

The majority of the tunnel alignments are anticipated to be founded within sedimentary rock formations. Only the north portal may encounter lower density granular soils at or below groundwater that may be susceptible to liquefaction. Further analysis will need to be done during the design phase of the project to assess the liquefaction at the north portal.

3.2.4 Lateral Spreading

Lateral spreading, closely related to liquefaction, occurs when soil mass slides laterally on a liquefied soil layer. Seismic shaking causes liquefaction of underlying saturated granular soil, and gravitational and inertial forces cause the liquefied layer and the overlying non-liquefied soil to move in a downslope direction. The magnitude of lateral displacement depends on earthquake magnitude, distance between the site and the seismic event, the peak ground acceleration, thickness of the liquefied layer, the ground slope or ratio of free-face height to distance between the free face and structure, fines content, average particle size of the soil comprising the liquefied layer, and the residual shear strength of the liquefiable soils.

Due to the nature of this project being a subsurface tunnel, lateral spreading is not likely and is not an anticipated concern.

4 GEOTECHNICAL INVESTIGATION PROGRAM

4.1 INTRODUCTION

Geotechnical borings were drilled along Luzon Avenue and Avenida Primavera in the City of Del Mar and along Durango Drive and Portofino Drive in the City of San Diego for the project. Before performing the investigation, previous geotechnical and geologic data was reviewed and incorporated into the planning.

4.2 EXPLORATORY BORINGS

The borings were drilled using mud rotary-wash drilling method (ASTM D5783) by Tri-County Drilling, Inc. using a Diedrich D-120 rotary wash rig. All borings were drilled with either rotary wash drilling using drag/tri-cone bits or triple barrel wireline HQ coring between March 1, 2021 and March 17, 2021 to depths ranging from about 90 to 200 feet below the ground surface. A total of 4 exploratory borings as shown in Appendix A were drilled. Locations and surface locations of the exploratory borings were measured in the field with a hand-held Global Positioning System (GPS) device with an estimated accuracy of about 10 feet. All of the boring location and elevation information is summarized in Table 1. Boring locations are also presented on the Boring Location Plan (Figure 4)

Our field representatives visually classified the soil cuttings and samples in accordance with Caltrans' Soil and Rock Logging Classification Manual (2010b) and maintained a detailed record of subsurface materials, changes during drilling, and groundwater conditions encountered in the exploratory borings. The boring logs show contacts/transitions between the differing soil layers based on changes in the soil cuttings and changes in the drilling operations (e.g., loss of drilling fluid, chatter of the drill rig, gauge pressure changes, etc.). All of these changes are noted in the field logs.

When subsurface conditions permitted, alternating relatively undisturbed soil sampling and Standard Penetration Test (SPT, ASTM D1586) were performed in the borings at 5-foot depth intervals. Relatively undisturbed drive samples were obtained using a Modified California split-spoon sampler (3.25-inch outer diameter) lined with brass rings. Each of these brass rings are 1-inch long with a 2.5-inch outside diameter. The Standard Penetration Test was performed with a SPT sampler (1.4-inch inside diameter) without liners. Both samplers, Modified California split-spoon and SPT, were driven into the ground using a 140-lb hammer free falling from a height of 30 inches. The number of blows to advance the samplers was recorded at every 6 inches of penetration, or until refusal. Only the total number of blows for the final 12 inches or less of driving the SPT and split-spoon samplers is shown on the LOTB sheet. The total blow counts required to drive the SPT and split-spoon sampler for the last 12 inches is referred as the Standard Penetration Resistance (N-value).

Because fluid is used while advancing a boring using rotary wash drilling methods, depth to groundwater may be difficult to identify. Notwithstanding this difficulty, groundwater was measured at the end of drilling after bailing out most, if not, all of the drilling mud from the boreholes. The driller waited until the water level stabilized before measuring. Groundwater depths varied between 58.5 and 68.9 feet below existing grade, though water readings are assumed to be perched water/seepage. A way to confirm whether groundwater is present during rotary wash drilling is to observe soil samples; if there is an increase in moisture or the soil sample is wet, groundwater may have been encountered.

After completion, the rotary-wash borings were backfilled with grout to about 5 feet from the surface. Soil cuttings were used to backfill the remainder of the borehole. If the boring was located in a paved area, the surface was topped with cold patched asphalt or lean concrete to match existing surface condition following the applicable City, County or Caltrans requirements. Remaining soil cuttings and drilling mud were collected in 55-gallon drums or similar containers and removed off-site.

Soil samples were sent to MTGL Inc. in San Diego, California for testing. Soil classifications in the field were verified by further examination in the laboratory and by test results. Final boring logs were prepared based on the field logs, examination of samples in the laboratory, and laboratory test results. The boring records and key to boring records, and other pertinent information are presented in Appendix A. It should be noted that the lines designating the interface between materials in the boring records generally represent approximate boundaries. The actual transition between subsurface materials is usually gradual.

Table 1. Current Geotechnical Exploration Information

Boring No.	Northing	Easting	Surface El. (feet MSL)	Depth (feet)	Bottom Elevation (feet MSL)	Groundwater Elevation (feet MSL)	Method of Exploration
R-21-001	1,932,645.362	6,250,477.845	+70	90.8	-20.8	NM	RW
R-21-002	1,931,382.766	6,251,075.054	+198	200	-2	Perched/Seepage at +139.5	RW
R-21-003	1,925,800.702	6,253,937.542	+365	120	+245	NM	RW
R-21-004	1,921,088.166	6,256,152.999	+160	110.4	+49.6	Perched/Seepage at +91.1	RW

Notes:

1. RW = Rotary Wash
2. NE = Not Encountered

4.3 Downhole P&S Wave Suspension Logging

4.3.1 Logging Procedures

Geophysical logging was performed on two borings, R-21-001 and R-21-004 (Table 2), by Geovision of Corona, California using the suspension method. The logging provided in-situ compressional (P, or primary) and shear (SH, or horizontal secondary) wave velocity measurements of the subsurface soil.

Geophysical logging was performed after completion of the borings using an OYO Model 170 Suspension Logging device to obtain in-situ horizontal shear and compressional wave velocity measurements at 1.6-foot intervals. The device consists of a logging recorder and a suspension logging probe that was lowered into the completed borehole. The probe (20 feet long) contains an impact source in the tip that generates an acoustic wave. The pressure transforms into P and S waves in the fluid-filled borehole through its walls into the surrounding soils. These waves propagating upward through the surrounding soils create detachable pressure waves in the fluid surrounding the receiver at the top of the borehole. This system directly determines the average wave velocity of the soil surrounding the borehole walls by measuring the elapsed time between arrivals of a wave propagating upward through the soil column.

Seismic velocity information could be used for a variety of purposes such as aiding the interpretation of stratigraphic information, characterization of ground response to earthquake motion, as well as development of ground stiffness for foundation design and tunnel deformation.

4.3.2 Seismic Velocity Test Results

As mentioned, seismic compression (P-) and shear (S-) wave velocities of the subsurface soils were measured in two (2) borings (R-21-001, and R-21-004) within the project corridor. Test locations and depths are summarized in Table 2. A summary report including the measured P- and S-wave velocity profiles versus depth is included in Appendix D. The P- and S-wave profiles are also shown on the subsurface cross section in Appendix B.

For boring R-21-001 at the north portal, the upper 40 feet of the subsurface soil/soft rock had a shear wave velocity ranging from 400 feet/second to 1,200 feet/second (1,000 feet/second to 4,000 feet/second for compressional wave velocity). Once within the Delmar Formation, the bottom 40 feet had shear wave velocities ranging from 1,600 feet/second to 2,600 feet/second (4,500 feet/second to 7,500 feet/second for compressional wave velocity). For boring R-21-004 at the south portal, the upper 80 feet within the Torrey Sandstone unit had a shear wave velocity ranging from 1,200 feet/second to 1,600 feet/second (2,900 feet/second to 4,200 feet/second for compressional wave velocity). Once within the Delmar Formation at the bottom 15-20 feet had shear wave velocities ranging from 1,600 feet/second to 2,400 feet/second (4,200 feet/second to 7,250 feet/second for compressional wave velocity).

Table 2. Summary of P-S Logging Test Depths

Boring No	Northing	Easting	Approx. Ground Surface El. (feet MSL)	Tested Depth Range (feet)
R-21-001	1,932,645.362	6,250,477.845	+70	6.9 to 76.4
R-21-004	1,921,088.166	6,256,152.999	+160	8.2 to 96.1

4.4 Pressuremeter Testing

Pressuremeter testing was performed by EMI engineers to measure stress-strain response of subsurface soil at planned depths within selected boreholes. A total of two (2) tests were performed in Borings R-21-001 and R-21-004 for the project using a TEXAM pressuremeter from RocTest Ltd.

4.4.1 SETUP

Each test was performed by inserting a probe consisting of a flexible membrane enclosed in a tubular stainless steel shield in the borehole and pressurizing the protected membrane hydraulically to deform the surrounding soil radially. The average radial expansion is measured by volume of the hydraulic fluid (water) injected into the probe by reading a volume counter. The hydraulic pressure applied into the probe is measured by a pressure gauge. The injected fluid pressure and volume provide the basis for the stress-strain relationship of the soil.

In the field, the pressuremeter probe was lowered into the drilled borehole by means of the drilling rods. The testing depth was determined based on the proposed tunnel invert depth and the observed soil/rock conditions during drilling as well as the limitation of equipment. Since the device is designed to fit in 3-inch diameter borehole, the driller used a 2-7/8 inch tri-cone drill bit to create the test hole which ultimately yields an approximately 3-inch hole for testing. After inserting the probe into the borehole, the fluid pressure was applied in several increments and volume changes were noted after the pressure was stabilized. Some loading/unloading cycles were performed as needed. The plots with the borehole pressures versus borehole deformation data are presented in Appendix E after calibrating the data to account for internal deformation of the pressuremeter system.

4.4.2 INTERPRETATION OF YOUNG'S AND SHEAR MODULI

Pressuremeter testing is based on a cavity expansion theory in an elastic medium, the radial expansion of a cylindrical cavity is related to the pressure by the equation:

$$\frac{\Delta r}{r} = \frac{(1+\nu)}{E}\Delta p$$

where r is radius and p is pressure of the cavity (in this case, borehole). The elastic constants in the above equation are Poisson's ratio (ν) and Young's modulus (E). Assuming that the value of Poisson's ratio is known, the Young's modulus (E) and the shear modulus (G) can be determined by the following expressions:

$$E = (1+\nu)\frac{\Delta p}{\Delta r/r}$$

and

$$G = \frac{E}{2(1+\nu)}$$

The applied pressure versus radial strain plots created from the processed data are presented in Appendix E. The Young's Moduli and at-rest earth pressure coefficients k_o interpreted from the plots are summarized in Table 3 above based on effective stress. The E_m is obtained from the linear gradient of the elastic range of the curve. The k_o value shown in Table 3 was obtained using the pressure at the lower end of the elastic range of the curve.

Table 3. Summary of Pressuremeter Test Results

Boring No	Depth	Rock Type	Original Loading E_m (ksf)	Loading/Reloading E_m (ksf) (avg)	k_o
R-21-001	48.0	Clayey Sandstone (Delmar Formation)	5,040	35,031	0.696
R-21-004	65.0	Sandstone (Torrey Sandstone Formation)	3,297	52,429	0.857

Please note that the E value from loading/reloading is considered for small strain deformation. The pressuremeter test results are highly influenced by sample disturbance and data interpretation; the results must be used with caution. Engineers are urged to compare the pressuremeter test results with typical elastic properties of similar soil/rock and to make engineering judgment before using in their design calculations.

4.5 Packer Testing

Packer testing is a common in-situ method of measuring hydraulic conductivity of soils and rock. The purpose of this testing was to provide estimates of in-situ transmissivity of the subsurface soils in the region of the tunnel bores, and to aid design of soil excavation, support, lining and dewatering systems for the proposed tunnels.

Packer testing was performed by EMI in three (3) borings (R-21-001, R-21-002, and R-21-003) at selected depth intervals. The depths and soil types tested are summarized in Table 4 and the field data can be found in Appendix F.

4.5.1 Testing Procedures

The testing procedures generally followed ASTM guidelines for the Constant Head Injection Test (ASTM D-4630–96). In each borehole, packer testing was performed after completion of drilling. Upon reaching the selected depth intervals, the borehole was flushed with clear water until the return water ran clear of any cuttings by visual observation. The test section was then internally sealed by two inflatable rubber packers at selected depths to isolate a test section. Water was then pumped through a hollow feed tube into the test section. The lengths of the test sections were typically 20 feet, or they were adjusted after inspection of the soil samples retrieved in that depth range. The water inflow quantity versus time was measured until steady-state flow was reached.

The constant water pressure was applied to the water in the sealed interval through the feed tube and the resulting flow rates were recorded over time. The constant pressures applied (up to three for each test) were less than the anticipated overburden stress at each test depth in order to avoid hydro-fracturing.

Testing details and measured test results are presented in Appendix F for each tested section with the applied pressures and measured water flow quantities.

4.5.2 PERMEABILITY

The equations used for calculation of the coefficient of permeability are as follows (Das, 1983):

$$k = \frac{q}{2\pi l h} \log \frac{l}{r} \qquad \text{for } l \geq 10r$$

and

$$k = \frac{q}{2\pi l h} \sinh^{-1} \frac{l}{2r} \qquad \text{for } 10r > l \geq r$$

where

k = coefficient of permeability of the soil,
q = constant rate of water injected into borehole,
l = length of test hole interval,
r = radius of borehole in test hole interval, and
h = differential head of groundwater in casing to ground surface.

The differential head of groundwater h is the total of injection pressure at the pressure gauge at the surface plus the gravity head of water (in the steel tubing) between the center of the packer test interval to the pressure gauge.

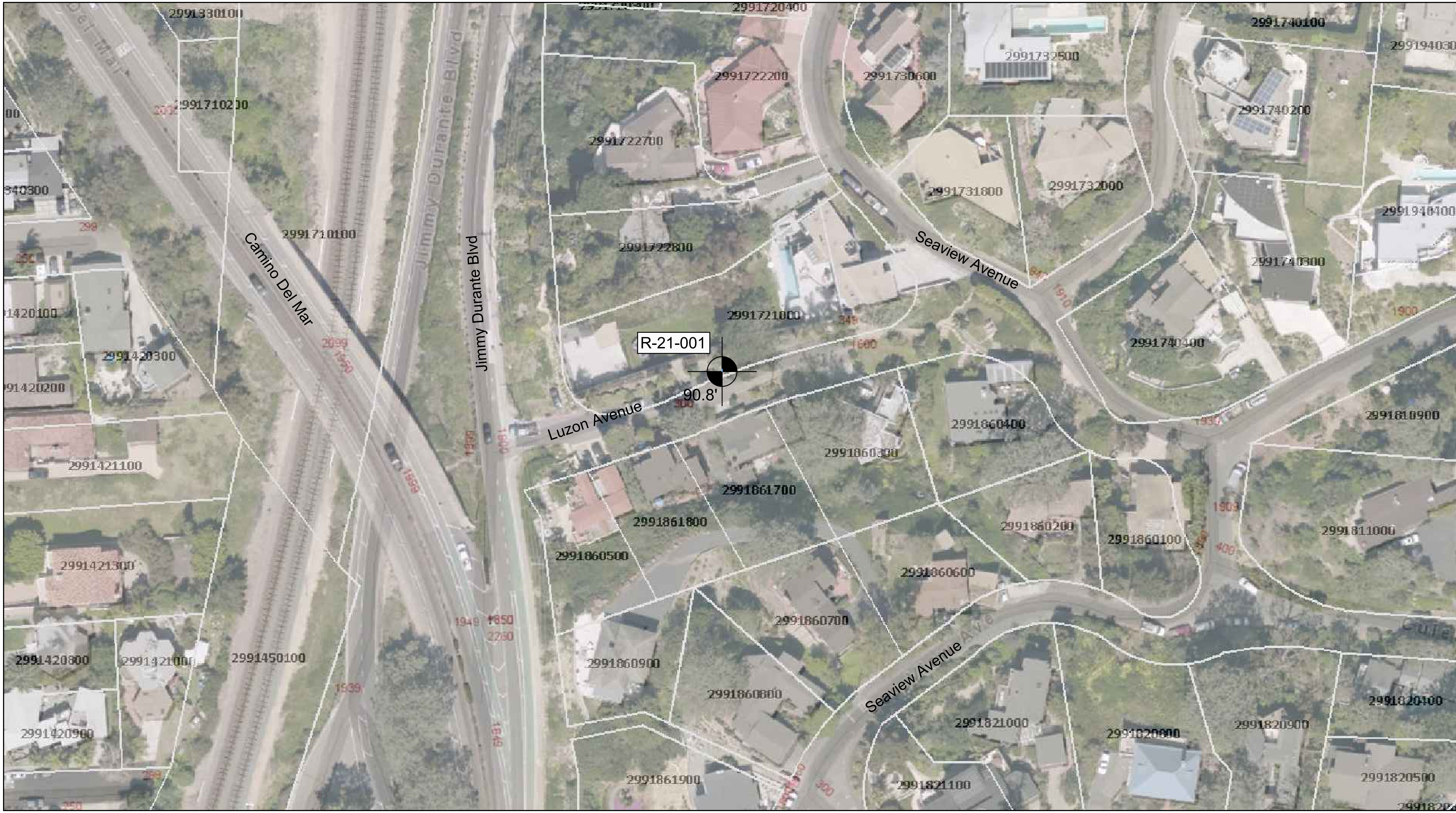
Calculated soil permeability ranges from these series of tests are summarized in Table 4. The data indicates that flow rates increase with increasing pressure. The k-values derived from the packer test represent only an estimate of soil permeability and should be cross-checked with other data and typical measurement results. That is because soil permeability could readily vary an order of magnitude even in a well-controlled laboratory environment.

Based on the packer test results, the tests within the Claystone and Clayey Sandstone of the Delmar Formation indicated permeability values ranging from 0 in/sec up to 0.35 in/sec. The lower bound data that ranges from 0 to 2.36 x 10^{-3} in/sec seem to be more representative of the actual rock mass permeability. The upper bound values ranging from 0.21 and 0.35 in/sec seem to indicate hydrojacking/hydrofracturing may be occurring within the rock mass during the packer test due to the poorly indurated nature of the rock formation. This may also indicate that water is leaking out the packer as well. No flow was recorded within the test section within the Torrey Sandstone yielding a k value of 0 in/sec. The k-values developed indirectly from the packer testing represent only an estimate of rock permeability and additional testing and analysis is recommended as part of the next phase of design to confirm the permeability values for the different geologic units anticipated to be encountered within the tunnel alignment.

Table 4. Summary of Packer Testing

Boring No	Boring Depth (ft)	Approx. GSE (ft)	Test Interval BGS (ft)	Center Depth (ft)	Soil/Rock Type	Approx. Coefficient of Permeability (in/sec)
R-21-001	91	+70	50 to 60	55	Interbedded Claystone/Clayey Sandstone (Delmar Formation)	0.22 to 0.24
R-21-002	200	+198	160 to 170	165	Clayey Sandstone (Delmar Formation)	0 to 2.36×10^{-3} (0.21 to 0.35)*
R-21-003	115	+365	95 to 105	100	Sandstone (Torrey Sandstone)	0 (practically no flow was recorded)

Notes:
USCS = Unified Soil Classification System, BGS = Below Ground Surface, GSE = Ground Surface Elevation
*Approx. permeability after 60psi due to possible packer leak/blowout or jacking/hydrofracturing



R-21-001

Location of Geotechnical Boring
"R" - Rotary Wash Boring

Drilled Depth

N

0 80 160 FEET

SCALE

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

Sheet 1 of 4

Earth Mechanics, Inc.
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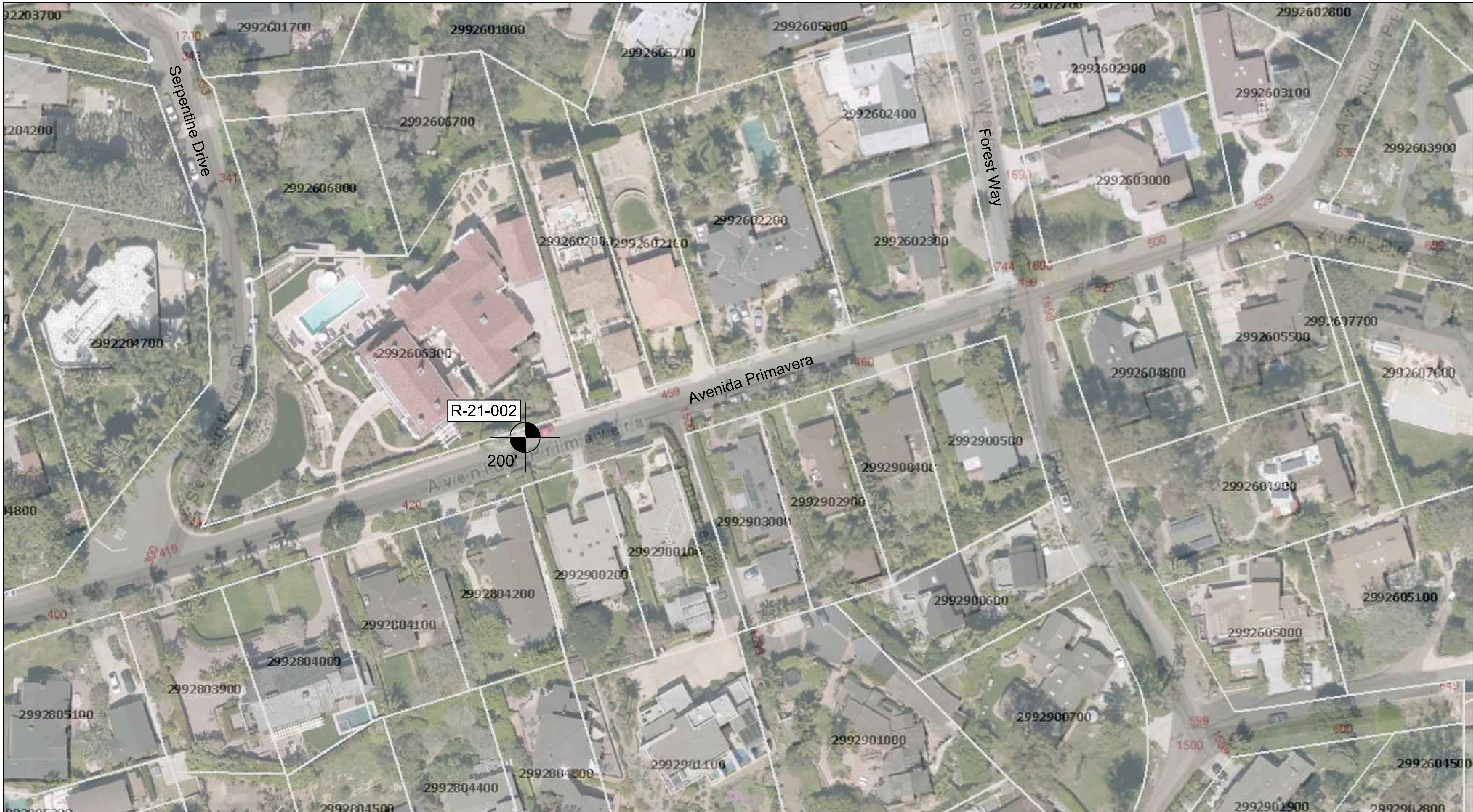
SD Regional Rail Corridor Alternative Alignment and Improvement Project
Boring Location Plan

Figure 4a



R-21-001

Drilled Depth

Location of Geotechnical Boring
"R" - Rotary Wash Boring

N

0 80 160 FEET
SCALE

Sheet 2 of 4

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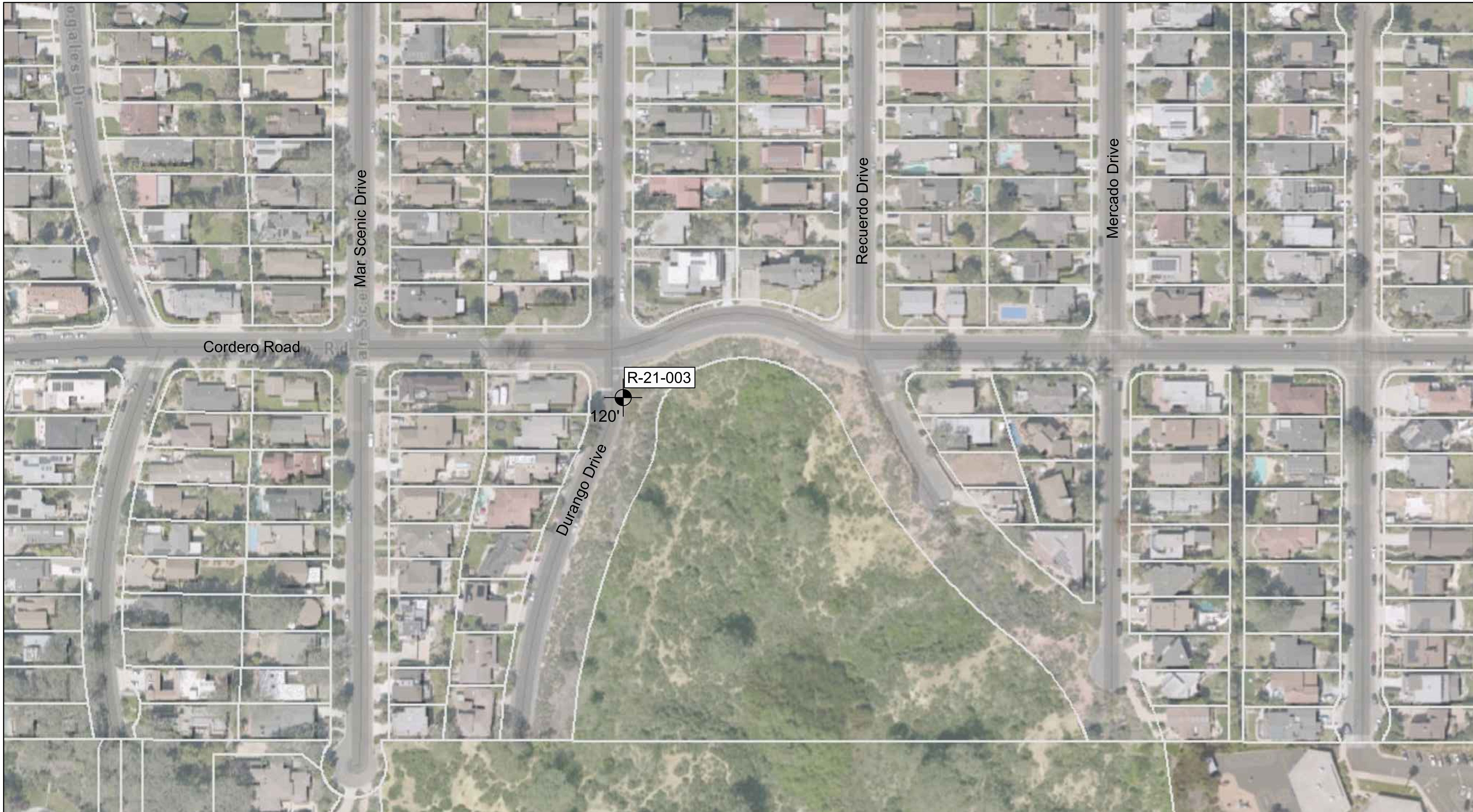
Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

SD Regional Rail Corridor Alternative Alignment and Improvement Project
Boring Location Plan

Figure 4b



R-21-001

Drilled Depth

Location of Geotechnical Boring
"R" - Rotary Wash Boring

N

0 150 300 FEET
SCALE

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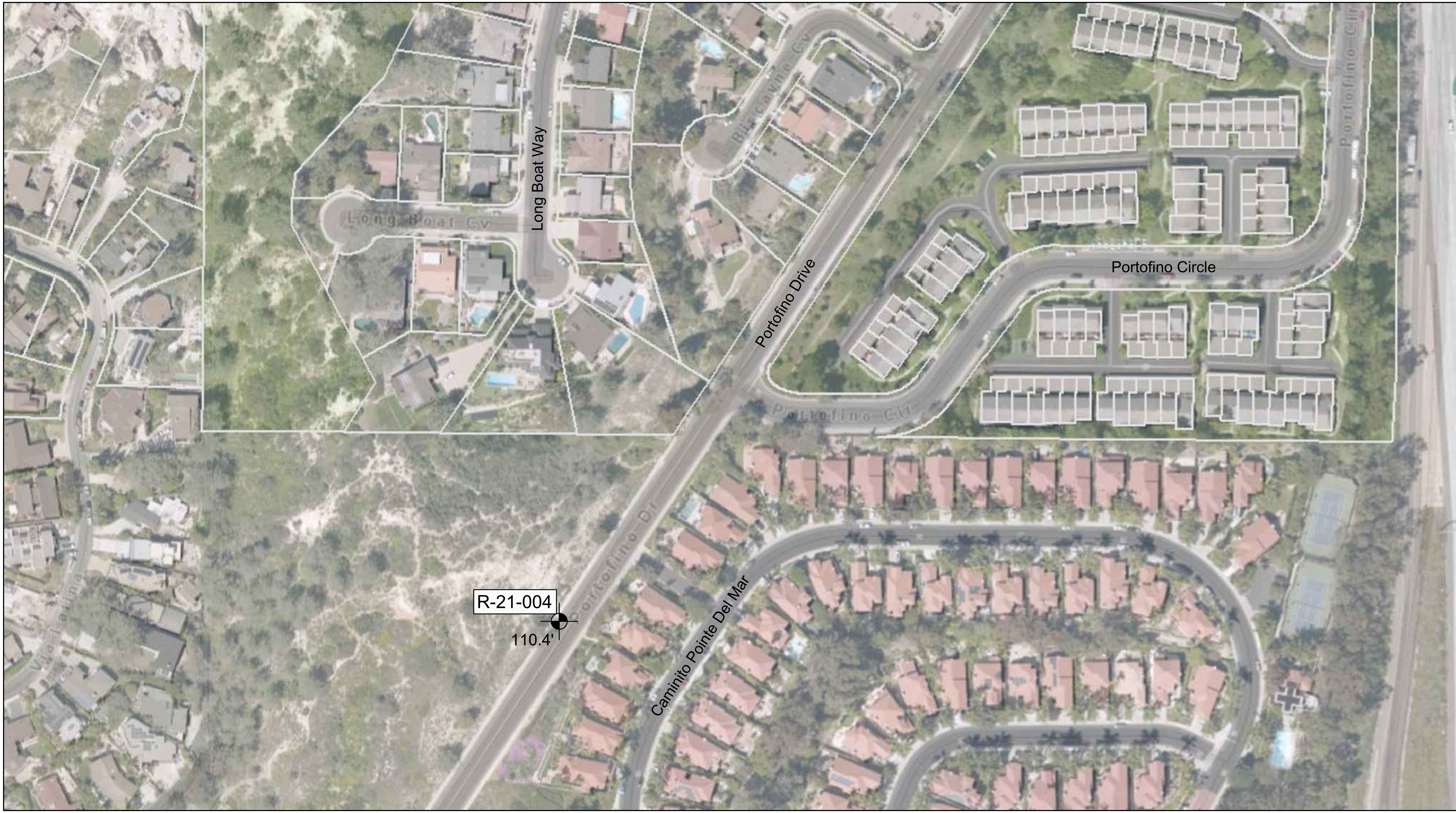
Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

SD Regional Rail Corridor Alternative Alignment and Improvement Project
Boring Location Plan

Figure 4c



R-21-001
Drilled Depth
Location of Geotechnical Boring
"R" - Rotary Wash Boring

N

0 150 300 FEET
SCALE

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

SD Regional Rail Corridor Alternative Alignment and Improvement Project
Boring Location Plan
Figure 4d

5 LABORATORY TESTING

Laboratory tests were performed on representative samples of soil and bedrock to determine or derive relevant physical and engineering properties. Selected samples were tested to determine soil classification, plasticity, shear strength parameters, and corrosion potential. A list of tests performed, the corresponding test methods, and purpose of testing is presented in Table 5. Laboratory tests were assigned by EMI and performed by MTGL, Inc. and Geo-Logic Associates under a subcontract to HDR. Results are summarized in Table 6. Detailed results are included in Appendix C.

Table 5. Explanation of Laboratory Testing Performed

Type of Test	Applicable Test Method	Purpose
Dry Density	ASTM D 2937	Estimate in-situ soil density
Moisture Content	ASTM D 2216	Estimate in-situ soil moisture content
No. 200 Wash	ASTM D 1140	Estimate percentage of gravel, sand, and fines content
Sieve Analysis	ASTM D 422	Estimate percentage of gravel, sand, and fines content
Specific Gravity	ASTM D 854	Estimate specific gravity of soil
Atterberg Limits	ASTM D 4318	Determine plasticity of soil
Direct Shear	ASTM D 3080	Estimate strength parameters of soil
Unconfined Compression Test	ASTM D 2166	Estimate strength parameters of soil
Unconsolidated Undrained Triaxial Text	ASTM D 2850	Measure stress-strain relationship of soil
Soil pH	CTM 643 & 532	Determine pH to assess corrosion potential of soil
Minimum Resistivity	CTM 643 & 532	Determine corrosion potential of soil
Sulfate Content	CTM 417	Determine sulfate content to assess corrosion potential of soil
Chloride Content	CTM 422	Determine chloride content to assess corrosion potential of soil

Notes:
1. ASTM = American Society for Testing and Materials.
2. CTM = California Test Method.

5.1 SOIL/ROCK CORROSIVITY

Soil/rock samples were tested for minimum resistivity, pH, soluble sulfate content, and soluble chloride content. Results are summarized in Table 6. Based on those results, the pH was determined to range between 7 and 7.6, the minimum resistivity varied from <500 to 3,000 ohm-cm, soluble chloride contents were between 139 and 314 parts per million (ppm) and soluble sulfate contents were between 251 and 847 ppm.

Based on the Caltrans Corrosion Guidelines (2003), soils are considered corrosive if the pH is 5.5 or less, the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm or greater. Based on the Caltrans criteria, some of the on-site soils are considered to be corrosive to bare metals and concrete.

Table 6. Summary of Laboratory Test Results

Boring No.	Sample Depth (ft)	USCS Symbol or Rock Type	Moisture Content (%)	Dry Density (pcf)	Total Density (pcf)	Grain Size (%)			Atterberg Limits (%)			Specific Gravity	Corrosion				Direct Shear (Peak Strength)		UC: Unconfined Compressive Strength (ksf)	UU: Undrained Shear Strength (ksf)
						Gravel	Sand	Fines	LL	PL	PI		pH	Minimum Resistivity (ohm-cm)	Soluble Sulfates (ppm)	Soluble Chlorides (ppm)	Friction Angle (deg)	Cohesion (psf)		
R-21-001	10.0	SP	12.3	106.2	119.3															
	15.0	SP-SM				0.0	88.0	12.0												
	20.0	SP-SM															38	48		
	30.0	SP-SM	5.2	114.1	120.0	0.0	92.0	8.0				2.61								
	40.0	SILTSTONE																		7.65
	45.0	SANDSTONE	20.2			0.0	61.0	39.0	29.5	20.7	8.8		7.3	690	407	215				
	55.0	CLAYSTONE	23.6					57.2	40.5	19.2	21.3		7.4	<500	625	205				
	65.0	CLAYSTONE	20.2			0.0	39.0	61.0	41.9	22.5	19.4		7.4	<500	374	232				
	70.0	CLAYSTONE	22.3			0.0	38.0	62.0												
	75.0	CLAYSTONE	18.1																	
	80.0	SANDSTONE	17.6					46.0	34.3	21.2	13.1									
	90.0	SILTSTONE	21.0																	
R-21-002	10.0	SM	14.0	125.6	143.1															
	20.0	SM	12.7	129.3	145.7							2.68								
	25.0	SANDSTONE				0.0	80.0	20.0												
	30.0	SANDSTONE															37	461		
	35.0	SANDSTONE				0.0	78.0	22.0												
	45.0	SANDSTONE	17.5					25.2												
	50.0	SANDSTONE	21.7																	
	70.0	SANDSTONE	19.0					28.7												
	80.0	SANDSTONE	19.1																	
	100.0	SANDSTONE	19.9					19.7												
	120.0	SANDSTONE	17.5					31.5												
	140.0	CLAYSTONE	18.3			0.0	48.0	52.0	47.2	22.6	24.6									
	145.0	CLAYSTONE	18.5			0.0	23.0	77.0	50.7	19.0	31.7									
	150.0	CLAYSTONE	20.5			0.0	37.0	63.0	47.6	21.7	25.9									
	150-153	SANDSTONE	15.6			0.0	79.0	21.0	36.2	20.2	16.0									8.04
	153.0	SANDSTONE																	2.10	
	157-161	SANDSTONE	14.7			0.0	60.0	40.0												26.08
	160.0	SANDSTONE																	6.54	
	160-165	SANDSTONE	15.3			0.0	85.0	15.0					7.0	740	847	257				24.98
	163.0	SANDSTONE																	7.19	
	167.0	SANDSTONE	14.3			0.0	87.0	13.0	NV	NP	NP								1.71	
	170.0	SANDSTONE																	9.22	
	170-175	SANDSTONE	13.4			0.0	67.0	33.0	36.6	19.5	17.1		7.2	640	251	244				8.47
	175-180	SANDSTONE	11.7			0.0	73.0	27.0	42.9	21.7	21.2		7.6	610	327	139				34.14
	185-190	SANDSTONE	11.6			0.0	59.0	41.0	39.2	21.6	17.6									
	191.0	SANDSTONE	15.7			0.0	67.0	33.0	35.1	19.2	15.9									
	195.0	SANDSTONE	11.8			0.0	74.0	26.0	25.8	19.9	5.9									
R-21-003	5-10	SANDSTONE	8.1			0.0	82.0	18.0												
	20-25	SANDSTONE	9.3			0.0	90.0	10.0	40.5	19.7	20.8									
	40-45	SANDSTONE	11.2			0.0	92.0	8.0	34.3	21.2	13.1		7.5	2100	460	314				

Table 6 (Continued). Summary of Laboratory Test Results

Boring No.	Sample Depth (ft)	USCS Symbol or Rock Type	Moisture Content (%)	Dry Density (pcf)	Total Density (pcf)	Grain Size (%)			Atterberg Limits (%)			Specific Gravity	Corrosion				Direct Shear (Peak Strength)		UC: Unconfined Compressive Strength (ksf)	UU: Undrained Shear Strength (ksf)
						Gravel	Sand	Fines	LL	PL	PI		pH	Minimum Resistivity (ohm-cm)	Soluble Sulfates (ppm)	Soluble Chlorides (ppm)	Friction Angle (deg)	Cohesion (psf)		
	60-65	SANDSTONE	14.3			0.0	96.0	4.0	NV	NP	NP									
	80-85	SANDSTONE	17.3			0.0	94.0	6.0	36.2	24.5	11.7									
	85-87	SANDSTONE																	1.56	
	97.5-99.5	SANDSTONE																		23.14
	100-105	SANDSTONE	14.3			0.0	93.0	7.0	NV	NP	NP									
R-21-004	5.0	SANDSTONE				0.0	88.0	12.0	42.9	21.4	21.5									
	10.0	SANDSTONE	11.5	115.9	129.2							2.61								
	20.0	SANDSTONE				0.0	86.0	14.0	39.2	21.2	18.0									
	45.0	SANDSTONE						7.6												
	50.0	SANDSTONE						9.3												
	60.0	SANDSTONE						10.8												
	75.0	SANDSTONE	21.9			0.0	90.0	10.0	35.1	19.2	15.9		7.3	3000	559	225				
	85.0	SANDSTONE	18.9			0.0	73.0	27.0												
	90.0	SANDSTONE	24.4																	
	100.0	CLAYSTONE	23.0			0.0	50.0	50.0	55.8	23.8	32.0									
	110.0	SANDSTONE						6.4												

6 PRELIMINARY DESIGN RECOMMENDATIONS

EMI understands that many of the details about the tunnel and portal structure configurations and anticipated means and methods of construction are being developed and not yet finalized. Preliminary design recommendations are provided in the following sections, as appropriate at the current stage of the project (less than 10% design) using very limited geotechnical data and design alternatives. These design recommendations will be updated as future geotechnical investigations and design progress.

6.1 PRELIMINARY SEISMIC DESIGN

Seismic design for railroad structures along the LOSSAN corridor generally follows the requirements outlined in Manual for Railway Engineering (AREMA), Chapter 9 Seismic Design for Railway Structures (2018). AREMA requires seismic performance of structures to be assessed for three levels of ground motion – Level I (Serviceability, 50- to 100-year return period), Level II (Ultimate, 200- to 475-year return period), and Level III (Survivability, 1000- to 2475-year return period). The design return period of each level of ground motion is based on the structure importance classification factor with consideration of on the immediate safety, immediate value and replacement value determined by the structure designers. For this stage of preliminary design, the maximum return period was conservatively adopted for each ground motion level.

Since the length of tunnel under consideration is roughly 14,000 feet, three locations (North Portal, Middle, South Portal) were selected to develop earthquake response spectra. The envelope of the three spectra is recommended as the preliminary design spectrum. The latitude and longitude of the three selected reference locations are listed in Table 7. Based on the P-S logging results listed in Appendix D, the range of shear wave velocities near the proposed tunnel alignment is roughly 900 to 2400 ft/s. The average value is estimated to be about 1700 ft/s (518 m/s), corresponding to a standard Site Class C.

Table 7. Seismic Hazard Analysis Parameters

Reference Site Location	Latitude (degrees)	Longitude (degrees)	Shear Wave Velocity (Vso) [1]
North Portal	32.9655	-117.2649	Approx. 900 to 2400 ft/s Avg = 1700 ft/s (518 m/s) (Site Class C)
Middle	32.9529	-117.2542	
South Portal	32.9340	-117.2458	

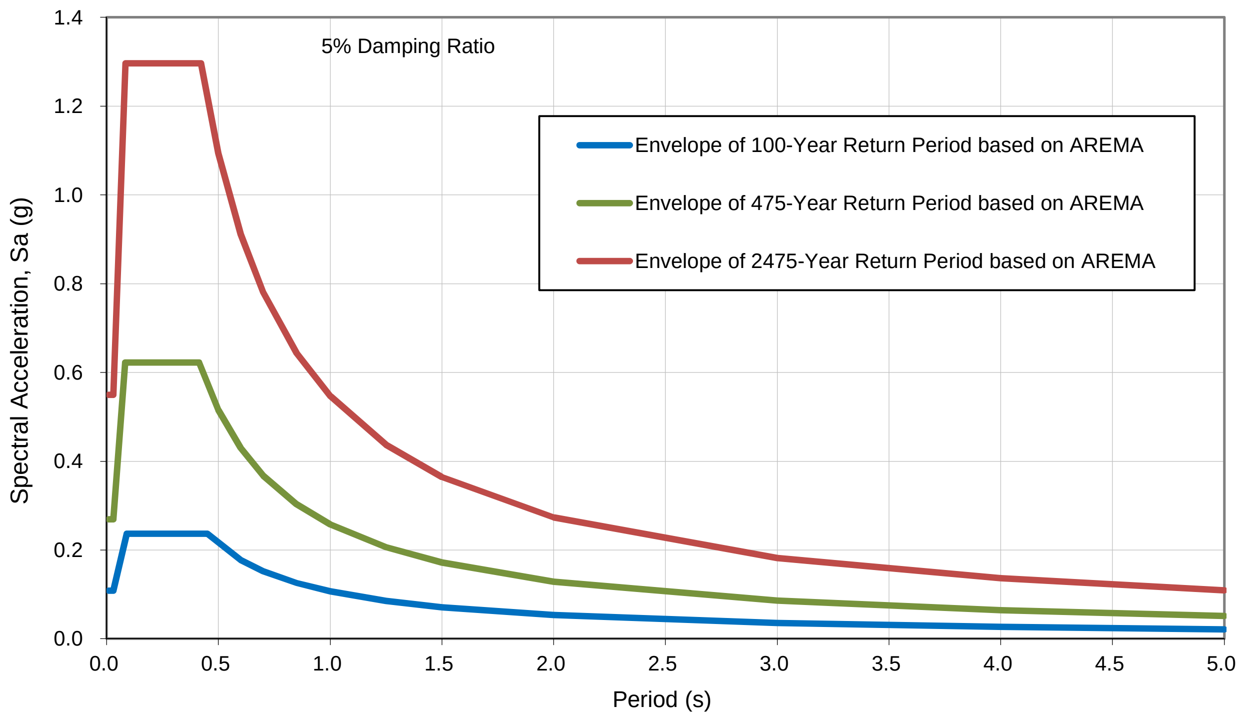
Notes:

1. Vso is the small-strain shear wave velocity.

6.1.1 Design Acceleration Response Spectra (ARS)

In order to characterize/quantify the design level ground motion demand for the project, EMI developed preliminary acceleration response spectra (ARS) following AREMA guidelines. According to Chapter 9 of the AREMA Manual, the latest version (Dynamic: Conterminous U.S. 2014 V4.2.0) of the USGS Unified Hazard tool (https://earthquake.usgs.gov/hazards/interactive) was used to develop the site-specific base acceleration coefficients (Sa at 0.0, 0.2, and 1.0 seconds) first for Site Class B (Vs= 760 m/s) for each event (100-year, 475-year and 2,475-year return periods). The resulting base acceleration coefficients were then modified for Site Class C to represent the ground conditions surrounding the tunnel and portals using the corresponding site amplification factors shown in AREMA. The recommended preliminary AREMA site-specific ARS curves at the North Portal, Middle, and South Portal are presented in Figure 5, along with the envelope of all three locations.

Based on deaggregation of the PSHA, seismic hazard at the site is primarily controlled by nearby events on the Rose Canyon fault (approximately M6.8 to M6.9 events at distances of 3.5 to 4.5 km). Events on nearby faults are known to have the potential for near-fault effects (for forward directivity scenarios) that can increase the ground shaking intensity, particularly in the longer period motion range (T > 0.5 seconds). AREMA does not currently require adjustments for near fault effects. Depending on the final adopted seismic design criteria for the project (e.g., if other than AREMA), near fault effects can be considered in a detailed site-specific probabilistic seismic hazard analysis (PSHA) at an appropriate stage of design.



100-Year Return Period					475-Year Return Period					2475-Year Return Period				
T (sec)	Spectral Acceleration (g)				T (sec)	Spectral Acceleration (g)				T (sec)	Spectral Acceleration (g)			
	North	Middle	South	Enve.		North	Middle	South	Enve.		North	Middle	South	Enve.
0.010	0.108	0.108	0.107	0.108	0.010	0.269	0.266	0.265	0.269	0.010	0.550	0.540	0.530	0.550
0.030	0.108	0.108	0.107	0.108	0.030	0.269	0.266	0.265	0.269	0.030	0.550	0.540	0.530	0.550
0.090	0.237	0.236	0.233	0.237	0.083	0.622	0.616	0.613	0.622	0.084	1.297	1.257	1.249	1.297
0.150	0.237	0.236	0.233	0.237	0.150	0.622	0.616	0.613	0.622	0.150	1.297	1.257	1.249	1.297
0.200	0.237	0.236	0.233	0.237	0.200	0.622	0.616	0.613	0.622	0.200	1.297	1.257	1.249	1.297
0.250	0.237	0.236	0.233	0.237	0.250	0.622	0.616	0.613	0.622	0.250	1.297	1.257	1.249	1.297
0.300	0.237	0.236	0.233	0.237	0.300	0.622	0.616	0.613	0.622	0.300	1.297	1.257	1.249	1.297
0.400	0.237	0.236	0.233	0.237	0.414	0.622	0.616	0.613	0.622	0.422	1.297	1.257	1.249	1.297
0.451	0.237	0.236	0.233	0.237	0.500	0.516	0.510	0.507	0.516	0.500	1.094	1.069	1.062	1.094
0.600	0.178	0.177	0.176	0.178	0.600	0.430	0.425	0.422	0.430	0.600	0.911	0.890	0.885	0.911
0.700	0.153	0.152	0.151	0.153	0.700	0.368	0.364	0.362	0.368	0.700	0.781	0.763	0.759	0.781
0.850	0.126	0.125	0.124	0.126	0.850	0.303	0.300	0.298	0.303	0.850	0.643	0.629	0.625	0.643
1.000	0.107	0.106	0.106	0.107	1.000	0.258	0.255	0.253	0.258	1.000	0.547	0.534	0.531	0.547
1.250	0.085	0.085	0.084	0.085	1.250	0.206	0.204	0.203	0.206	1.250	0.437	0.427	0.425	0.437
1.500	0.071	0.071	0.070	0.071	1.500	0.172	0.170	0.169	0.172	1.500	0.365	0.356	0.354	0.365
2.000	0.053	0.053	0.053	0.053	2.000	0.129	0.127	0.127	0.129	2.000	0.273	0.267	0.266	0.273
3.000	0.036	0.035	0.035	0.036	3.000	0.086	0.085	0.084	0.086	3.000	0.182	0.178	0.177	0.182
4.000	0.027	0.027	0.026	0.027	4.000	0.064	0.064	0.063	0.064	4.000	0.137	0.134	0.133	0.137
5.000	0.021	0.021	0.021	0.021	5.000	0.052	0.051	0.051	0.052	5.000	0.109	0.107	0.106	0.109

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Geotechnical and Earthquake Engineering

San Diego Regional Rail Corridor Alternative Alignment and Improvement Project

Project No.: 20-134 | **Date: May 2021**

Preliminary Design ARS

Figure 5

6.1.2 Seismic Ground Deformations for Tunnel Ovaling Demand

Seismic design of tunnels and other underground structures is generally based on a ground deformation or "ovaling/racking" approach (as opposed to using the ARS as for above-ground structures). Ground deformations are evaluated for final design by defining the design ground motions (time histories) at some elevation level below the tunnel invert, and then performing wave propagation (site response) analysis from that elevation level up through the ground surrounding the tunnel. Such a time history-based analysis approach is pre-matured at this preliminary stage when not enough details are known, but should be undertaken in subsequent design stages.

In the absence of time history-based analysis in the current preliminary design, ground deformations were estimated using a simplified approach. Free field ground strains were estimated as the peak ground velocity (PGV) divided by the shear wave velocity in the ground surrounding the tunnel as in the case of uniform elastic half space. Based on NCHRP 12-70 (2008), PGV is strongly correlated with the spectral acceleration at 1.0 second (S1) using the following equation:

$$PGV = 0.394 \times 10^{0.434C} \text{ (in/sec)}$$

$$C = 4.82 + 2.16 \log_{10}(S_1) + 0.013\, [2.3 \log_{10}(S_1) + 2.93]^2$$

The above correlation gives the mean plus one standard deviation PGV based on a regression analysis. The median can be estimated by dividing the above PGV result by 1.46. A range of PGV values was estimated for preliminary design based on the median to mean plus one results using the correlation. Site-specific values of S_1 were obtained for each event using the USGS Unified Hazard tool (https://earthquake.usgs.gov/hazards/interactive) for Site Class C (Vs= 537 m/s). The S_1 and PGV values are listed in Table 8. If required, strain compatible shear modulus G_s values can be estimated based on the elastic relationship $G_s = Vs^2\rho$ where the density (ρ) is provided in the following section.

The free field shear stain estimates should be considered preliminary for use in the current feasibility study project phase, based on the limited information available to EMI at this time. As mentioned above, time history-based site response analysis should be performed in order to perform a more refined seismic tunnel liner design at a later project stage.

Table 8. Summary of S_1, PGV and Preliminary Free Field Shear Strain around Tunnel

Parameters	100-yr Return Period			475-yr Return Period			2475-yr Return Period		
S_1 (g)	West	Middle	East	West	Middle	East	West	Middle	East
	0.089	0.089	0.089	0.219	0.221	0.219	0.528	0.532	0.528
Max S_1 (g)	0.089			0.221			0.532		
PGV (ft/s)[1]	0.3 to 0.4			0.7 to 1.0			1.7 to 2.4		
Vs/Vso[2]	0.97			0.93			0.84		
Vs (ft/s)[3]	873 to 2328			837 to 2232			756 to 2016		
Free Field Shear Strain (%)	0.01% to 0.05%			0.03% to 0.12%			0.08% to 0.32%		

Notes:

1. The range of PGV was estimated between mean value and mean value plus one standard deviation based on the NCHRP 12-70 correlation.
2. Reduction factors for strain compatible shear wave velocity (and associated shear modulus) were estimated based on Table 19.3-1 of ASCE 7-16.
3. The range of Vs was estimated based on the range of Vso listed in Table 7 multiplied by the Vs/Vso reduction factors.

6.2 SOIL/ROCK DESIGN PARAMETERS

Based on the cross sections shown in Appendix B, the tunnel alignment may cross through four geological units – Artificial Fill (Af), Old Paralic Deposits (Qop), Torrey Sandstone (Tt) and Delmar Formation (Td). Preliminary design parameters for these units are listed in Table 9. These preliminary parameters will need to be updated and refined as more subsurface data and design details become available. Soil and rock strength properties were derived from: correlations with SPT blowcounts in Appendix A (Lam/Martin, 1986); the laboratory results listed in Table 6 and Appendix C; the pressuremeter results listed in Section 4.6; P-S logging results in Appendix D; and AAHSTO LRFD Bridge Design Specifications Section 10.4.6.5.

Table 9. Preliminary Design Parameters

Geological Unit	Predominant Soil/Rock Type	Range of SPT-equivalent Blowcounts (blows/ft)	Total Density (pcf)	Soil		Rock			Poisson's Ratio (v)	Ko
				Friction Angle	Cohesion/ Undrained Shear Strength (ksf)	Unconf. Compr. Strength (ksf)	RQD	E_m [1] (ksf)		
Artificial Fill (Af)	Silty sand	7 to 21	110 to 120	29 to 34	0.2 to 0.05	-	-	-	0.25 to 0.35	0.44 to 0.51
Old Paralic Deposits (Qop)	Silty to clayey sand	12 to 35	110 to 120	32 to 38	0.2 to 0.05	-	-	-	0.25 to 0.35	0.38 to 0.47
Torrey Sandstone (Tt)	Sandstone	40 to >70	130 to 150	37 to 45	0.5 to 0.3	1.5 to 17	30 to 100	3,000 to 50,000	0.2 to 0.4	0.3 to 0.85 [2]
Delmar Formation (Td)	Claystone to sandstone	>70	130 to 150	-	4 to 10	-	-	5,000 to 35,000	0.25 to 0.45	0.3 to 0.7 [2]

Notes:

1. E_m is the elastic modulus of the rock mass. Lower value is based on initial loading and upper value is based on unloading/reloading from limited pressuremeter testing performed to-date, discussed in Section 4.4.
2. Upper value is based on limited pressuremeter testing performed to-date, discussed in Section 4.4.

6.3 EARTH PRESSURES AT PORTALS

It is currently anticipated that portals will consist of U-structures transitioning to cut-and-cover, or directly to the bored tunnel. Support of excavation (SOE) is expected to be required in order to construct the portals, such as a soldier pile and lagging system.

For flexible walls that are free to move laterally at the top retaining level ground, preliminary active earth pressures may be estimated based on an equivalent fluid pressure of 36 pcf. For rigid (non-yielding) walls that are restrained from movement, such as permanent U-structure walls, retaining level ground, preliminary at-rest earth pressures may be estimated based on an equivalent fluid pressure of 56 pcf. For braced/tie-back walls, preliminary apparent earth pressures may be estimated as a uniform pressure of 36H psf, where H is the retained height in feet.

Structures below the design groundwater level (to be established later based on further investigations as discussed in Section 7) should also be designed for hydrostatic pressures, or to relieve pressures by drainage or dewatering.

Preliminary seismic earth pressures can be evaluated for the portals using a seismic coefficient of one-third to one-half of the PGA values shown in Figure 5, depending on the type of structure, its dimensions, and anticipated ability of the walls to deflect/displace during the design earthquake events. More details about the structure configurations are required in order to evaluate seismic earth pressures for the three design earthquake events.

7 FUTURE GEOTECHNICAL INVESTIGATIONS

For the next phase of design for the proposed San Diego Regional Rail project the geotechnical exploration investigation should address the following items:

- Determine the geologic subsurface conditions along the proposed tunnel alignment at depth.
- Collect additional groundwater data and provide quantitative data using piezometers along the proposed tunnel alignment.
- Collect additional data to document variability within subsurface geologic formations.
- Conduct additional investigation for the mapped fault crossing the proposed alignment to determine impacts on the proposed tunnel design and construction.
- Perform additional in-situ testing to further develop subsurface design parameters along the proposed tunnel alignment including shrink/well testing for soil/rock, expansive soil potential, soil modulus and permeability values.
- Collect additional shear wave velocity data for the purposes of characterizing the subsurface conditions for design and construction.

The next phase of investigation should be anticipated to consist of additional borings along the chosen alignment alternative. Soil borings should be anticipated to be drilled and sampled below the proposed tunnel invert with depths reaching up to 300 feet below existing grade. Vibrating wire piezometers/pressure transducers should be installed within borings along the proposed tunnel alignment to quantify groundwater impacts to the proposed tunnel design. Borings should include rock coring to provide samples to be characterized for strength testing, rock quality designation and in-situ testing including packer and pressuremeter testing. A geophysical seismic refraction investigation should be conducted to better define the actual location of the existing fault crossing the proposed alignment. Additional P & S wave suspension logs should also be conducted to provide additional data for tunnel construction and design, and development of detailed earthquake ground motion criteria.

8 CONSTRUCTION CONSIDERATIONS

8.1 PORTALS

As mentioned previously, portals are currently anticipated to consist of U-structures transitioning to cut-and-cover or directly to the bored tunnel section. A support of excavation (SOE) system is anticipated to be required to be required to retain the existing ground on the sides of the portal. SOE could consist of a driven steel soldier pile and timber lagging system. Depending on the details and dimensions, tiebacks or internal bracings may be required, which will be evaluated at a later design stage. From the limited borings drilled the present phase of design, both excavation and installation of SOE can be achieved with conventional construction equipment.

As discussed previously, the proposed portal may be susceptible to landsliding/seismically induced landsliding due to the existing geologic lithology at the proposed portal locations. The sedimentary bedrock of the Torrey Sandstone and Del Mar Formation at the site are known to have landsliding potential particularly when the geologic structure is unfavorable. Adverse structure and jointing may create conditions for block/wedge failure loading for on the proposed portal structure. Understanding the potentially highly variable rock quality and structure will be critical to the design of the proposed portal. The geologic conditions will need to be properly characterized qualitatively and quantitatively in order to implement the best design for the portal structures. A more comprehensive discussion of representative geologic materials and groundwater conditions will be discussed when additional subsurface investigations are performed in the future.

8.2 BORED TUNNEL

The majority of the proposed tunnel alignment will be excavated within the sedimentary rock associated with both the Torrey Sandstone and Delmar Formations. Tunnel excavations should anticipate encountering soft rock conditions consisting of sandstone, siltstone and claystone associated with these formations. Based on the limited preliminary investigation and site assessment, the anticipated rock mass conditions should be considered rippable for a bored tunnel excavation.

The proposed tunnel Crest Canyon alignment is proposed to cross a mapped unnamed fault between Station 111+00 and 112+00, and thus highly weathered and sheared/faulted rock will be encountered. Weathered zones may be subject to spalling and caving of large wedges of rock from the tunnel roof, which can be mitigated with appropriate TBM selection and means and methods. Areas with fault gouge zones have the potential for ground squeezing and may require specialized tunnel support during construction and design. Groundwater will also be a concern at the fault zones as groundwater conditions will be highly variable due to the high variability of rock mass permeability and rock quality. Fault zones will likely require a TBM designed for highly variable subsurface conditions that range between strong to weak rock and even soil like materials that can be commonly found within a fault zone. This might include a hybrid TBM that can operate in open or closed mode with bentonite injection around the shield, tapered shield, auxiliary jacking and adjustable gage cutters. Additionally, high ground cover with high in-situ stress combined with weak rock conditions would also require a TBM with pressurized face capability in weak soil conditions below groundwater. An additional comprehensive geotechnical investigation will be needed as part of the next phase of design to better define the subsurface rock, soil and groundwater conditions from a geotechnical and geologic standpoint.

Tunneling experts on the team (HDR and Mott MacDonald) are evaluating feasibility of bored tunnel construction at the site. The sequential excavation method (SEM) is also being considered for cross passage construction. EMI is available to elaborate on the subsurface information and interpretations presented in this report to assist in those evaluations.

9 LIMITATIONS

This report is intended for use by the SANDAG and HDR for the preliminary conceptual design of Regional Rail Corridor project located in the vicinity of Del Mar and San Diego, California. This report is based on the project as described herein and the information obtained from the exploratory boreholes at the approximate locations shown on the attached plans. Findings contained herein are based on results of the field investigation and laboratory tests. Also, the earth materials and subsurface conditions encountered in the exploratory boreholes are presumed to be representative of the project site; however, subsurface conditions and characteristics of soils between exploratory boreholes can vary. EMI should be notified of any pertinent changes in the project plans or if subsurface conditions are found to vary from those described herein. Modifications to project plans or variations in subsurface conditions may require re-evaluation of the information contained in this report.

The data contained herein are applicable to the specific design elements and locations which are the subject of this report. It has no applicability to any other design elements or to any other locations, and any and all subsequent users accept any and all liability resulting from any use or reuse of the data without the prior written consent of EMI.

EMI is not responsible for construction means, methods, techniques, sequences, or procedures, or for safety precautions or programs in connection with the construction, for the acts or omissions of the Contractor, or any other person performing any construction, or for the failure of any worker to carry out the construction in accordance with the Final construction drawings and specifications.

Services performed by EMI were conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions. No other representation, expressed or implied, and no warranty or guarantee is included or intended.

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APPENDIX A

Boring Logs

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names	Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])
WA	Wash Analysis (ASTM D 1140-97)

SAMPLER GRAPHIC SYMBOLS

Standard Penetration Test (SPT)

Standard California Sampler

Modified California Sampler

Shelby Tube

Piston Sampler

NX Rock Core

HQ Rock Core

Bulk Sample

Other (see remarks)

DRILLING METHOD SYMBOLS

Auger Drilling

Rotary Drilling

Dynamic Cone or Hand Driven

Diamond Core

WATER LEVEL SYMBOLS

First Water Level Reading (during drilling)

Static Water Level Reading (short-term)

Static Water Level Reading (long-term)

Earth Mechanics, Inc.

Geotechnical and Earthquake Engineering

BORING RECORD LEGEND

San Diego Regional Rail Corridor

Project Number: 20-134

Date: 3-25-21

SHEET 1 of 3

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE:
This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010 Edition), Section 2, for tables of additional soil description components and discussion of soil description and identification.

REF = Refusal; During drilling seating interval (first 6-inch interval) is not achieved.

ROCK GRAPHIC SYMBOLS

- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

BEDDING SPACING

Descriptor	Thickness or Spacing
Massive	> 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	3-5/8 inches to 1 ft
Thinly bedded	1-1/4 to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		
Descriptor	Body of Rock	Fracture Surfaces		Texture	Solutioning	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation (refer to grain boundary conditions)	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Altered by chemical disintegration such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

Note: Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors should not be used where significant identifiable zones can be delineated. Only two adjacent descriptors shall be combined. "Very intensely weathered" is the combination descriptor for "decomposed to intensely weathered".

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 - 30,000
Strong	7,000 - 14,500
Medium Strong	3,500 - 7,000
Weak	700 - 3,500
Very Weak	150 - 700
Extremely Weak	< 150

CORE RECOVERY CALCULATION (%)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

RQD CALCULATION (%)

$$\frac{\Sigma \text{ Length of intact core pieces} > 4 \text{ in.}}{\text{Total length of core run (in.)}} \times 100$$

ROCK HARDNESS

Descriptor	Criteria
Extremely Hard	Specimen cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Specimen cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Specimen can be scratched with pocket knife or sharp pick with heavy pressure; heavy hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/6 in. with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Specimen can be grooved or gouged with pocket knife or sharp pick with light pressure, breaks with light to moderate hand pressure
Very Soft	Specimen can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

FRACTURE DENSITY

Descriptor	Criteria
Unfractured	No fractures
Very Slightly Fractured	Lengths greater 3 ft
Slightly Fractured	Lengths from 1 to 3 ft, few lengths outside that range
Moderately Fractured	Lengths mostly in range of 4 in. to 1 ft, with most lengths about 8 in.
Intensely Fractured	Lengths average from 1 in. to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very Intensely Fractured	Mostly chips and fragments with few scattered short core lengths

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF/MH	**3-1-21**	**3-2-21**	**N 1,932,645 E 6,250,478**	**R-21-001**
DRILLING CONTRACTOR **Tri-County**			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION **70.0 ft**
DRILLING METHOD **Rotary Wash**			DRILL RIG **Diedrich D-120**	BOREHOLE DIAMETER **4"**
SAMPLER TYPE(S) AND SIZE(S) (ID) **Mod Cal (2"), SPT (1.4")**			SPT HAMMER TYPE **Automatic 140lb, 30in drop**	HAMMER EFFICIENCY, ERi **82%**
BOREHOLE BACKFILL AND COMPLETION **Cement-Bentonite Grout**			GROUNDWATER READINGS DURING DRILLING **NM** AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **90.8 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT (Asphalt Concrete: 4 in.) (No Base).												
	1		SILTY SAND (SM); brown; moist; few subangular to subrounded GRAVEL, max. 1 in. dia.; mostly coarse to fine SAND; little nonplastic fines; weak cementation; (FILL).												
68.00	2														
	3														
66.00	4														
	5		Poorly graded SAND (SP); loose; brown; moist; mostly medium to fine SAND; few nonplastic fines; weak cementation.		1	4 4 3	7	56							
64.00	6														
	7														
62.00	8														
	9														
60.00	10		Medium dense.		2	5 8 9	17	61		12	106				
	11														
58.00	12														
	13														
56.00	14														
	15		Poorly graded SAND with SILT (SP-SM); medium dense; brown; moist; about 88% medium to fine SAND; about 12% nonplastic fines; weak cementation.		3	10 8 13	21	33							PA
54.00	16														
	17														
52.00	18														
	19														
50.00	20		Dense; mostly SAND; few fines.		4	15 23 29	52	67							DS
	21														
48.00	22														
	23														
46.00	24														
	25														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		Poorly graded SAND with SILT (SP-SM); dense; brown; moist; trace subangular to subrounded GRAVEL, max. 1/2 in. dia.; mostly medium to fine SAND; few nonplastic fines; weak cementation.		5	10 12 14	26	33							
44.00	26														
	27		Poorly graded SAND with SILT (SP-SM) *(continued)*.												
42.00	28														
	29														
40.00	30		About 92% medium SAND; about 8% nonplastic fines.		6	15 23 28	51	67		5	114				DS, PA, SG
	31														
38.00	32														
	33														
36.00	34														
	35		Poorly graded SAND with CLAY (SP-SC); medium dense; brown; moist; mostly coarse to fine SAND; few medium plasticity fines; weak cementation.		7	6 5 4	9	28							
34.00	36														
	37		SEDIMENTARY BEDROCK, Delmar Formation - Sandy SILTSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive.												Hard drilling at 37.5 feet
32.00	38														
	39														
30.00	40		Slight cemented.		8	30 50/5"		100							UU
	41														
28.00	42														
	43														
26.00	44														
	45		Clayey SANDSTONE, olive gray to olive brown; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; massive; friable. About 61% SAND; about 39% fines.		9	37 50/5"		100		20					CR, PA, PI
24.00	46														
	47														Hard drilling
22.00	48														Pressure Meter Test at 48 feet
	49														
20.00	50				10	50/2"		100							Packet Test between 50' and 60' depth
	51														
18.00	52														
	53														
16.00	54														
	55														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-001**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **2 of 4**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		SEDIMENTARY ROCK *(continued)*. Sandy CLAYSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive. About 43% SAND; about 57% fines.		11	28 43 50/3"		100		24					CR, PA, PI
14.00	56														
	57														Hard drilling
12.00	58														
	59														
10.00	60				12	50/3"		100							
	61														
8.00	62														
	63														
6.00	64														
	65		About 39% SAND; about 61% fines.		13	40 50/5.5"		78		20					CR, PA, PI
4.00	66														
	67														
2.00	68														
	69														
0.00	70		About 38% SAND; about 62% fines.		14	38 50/5"		82		22					PA
	71														
-2.00	72														
	73														
-4.00	74														
	75				15	45 50/4"		90		18					
-6.00	76														
	77														
-8.00	78														
	79														
-10.00	80		Clayey SANDSTONE, olive gray to olive brown; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; massive; friable. About 54% SAND; about 46% fines.		16	48 50/3"		100		18					PA, PI
	81														
-12.00	82														
	83														
-14.00	84														
	85														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-001**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **3 of 4**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/17/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		Sandy SILTSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive.		17	50/4"		100							
-16.00	86														
	87														
-18.00	88														
	89														
-20.00	90				18	43 50/4"		60		21					
	91		Bottom of borehole at 90.8 ft bgs Groundwater was not measured, P&S wave suspension logging was conducted after completion of drilling, Additionally packer testing was also conducted between depths of 50 and 60 feet. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.												
-22.00	92														
	93														
-24.00	94														
	95														
-26.00	96														
	97														
-28.00	98														
	99														
-30.00	100														
	101														
-32.00	102														
	103														
-34.00	104														
	105														
-36.00	106														
	107														
-38.00	108														
	109														
-40.00	110														
	111														
-42.00	112														
	113														
-44.00	114														
	115														

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE **BORING RECORD**				HOLE ID **R-21-001**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **4 of 4**

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF/MH	**3-3-21**	**3-8-21**	**N 1,931,383 E 6,251,075**	**R-21-002**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Tri-County		**198.0 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Rotary Wash	**Diedrich D-120**	**4"**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Mod Cal (2"), SPT (1.4")	**Automatic 140lb, 30in drop**	**82%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **58.5 (Seepage)**; AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING
Cement-Bentonite Grout		**200.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	0	ASPHALT (Asphalt Concrete: 5 in.) (No Base).									
	1	SILTY SAND (SM); brown; moist; mostly medium to fine SAND; little nonplastic fines; weak cementation; (FILL).									
196.00	2										
	3										
194.00	4										
	5	SILTY SAND (SM); very dense; brown; moist; mostly medium to fine SAND; little nonplastic fines; weak cementation; some coarse SAND.	1	18 19 18	37	100					
192.00	6										
	7										
190.00	8										
	9										
188.00	10	Medium SAND.	2	18 27 30	57	83		14	126		
	11										
186.00	12										
	13										
184.00	14										
	15	Dense.	3	8 12 11	23	83					
182.00	16										
	17										
180.00	18										
	19										
178.00	20	Very dense; coarse to medium SAND.	4	20 35 44	79	67		13	129		SG
	21										
176.00	22										
	23										
174.00	24										
	25										

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-002**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-25-21**	SHEET **1 of 7**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	25	SEDIMENTARY BEDROCK, Torrey Sandstone Formation - SANDSTONE; olive brown to light olive brown; soft to moderately soft; slightly weathered to moderately weathered; medium to fine grained; poorly indurated; friable. About 80% SAND; about 20% fines.	5	14 17 19	36	56					PA
172.00	26										
	27										
170.00	28										
	29										
168.00	30		6	21 30 35	65	67					DS
	31										
166.00	32										
	33										
164.00	34										
	35	About 78% SAND; about 22% fines.	7	8 16 36	52	67					PA
162.00	36										
	37										
160.00	38										
	39										
158.00	40	Light olive brown, slightly weathered to fresh.	8	50/2"		100					Hard drilling at 40 feet
	41										
156.00	42										
	43										
154.00	44										
	45	About 75% SAND; about 25% fines.	9	50/3"		100		18			PA
152.00	46										
	47										
150.00	48										
	49										
148.00	50	Light grayish brown.	10	50/4"		75		22			
	51										
146.00	52										
	53										
144.00	54										
	55										

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				R-21-002
DIST.	COUNTY	ROUTE	POSTMILE	EA 20-134
PROJECT OR BRIDGE NAME: San Diego Regional Rail Corridor				
BRIDGE NUMBER	PREPARED BY PSS		DATE 3-25-21	SHEET 2 of 7

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		*(continued)*. SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; friable.												
142.00	56														
	57														
140.00	58														
	59														
138.00	60				11	50/4"		25							
	61														
136.00	62														
	63														
134.00	64														
	65														
132.00	66														
	67														
130.00	68														
	69														
128.00	70		About 71% SAND; about 29% fines; light olive gray.		12	50/3"		100		19					PA
	71														
126.00	72														
	73														
124.00	74														
	75														
122.00	76														
	77														
120.00	78														
	79														
118.00	80		White to light gray.		13	50/4.5"		100		19					
	81														
116.00	82														
	83														
114.00	84														
	85														

(continued)

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; friable.												
112.00	86														
	87														
110.00	88														
	89														
108.00	90			X	14	50/5"		60							
	91														
106.00	92														
	93														
104.00	94														
	95														
102.00	96														
	97														
100.00	98														
	99														
98.00	100		About 80% SAND; about 20% fines.	X	15	50/3"		100		20					PA
	101														
96.00	102														
	103														
94.00	104														
	105														
92.00	106														
	107														
90.00	108														
	109														
88.00	110		Mottled yellowish brown.	X	16	50/3"		33							
	111														
86.00	112														
	113														
84.00	114														
	115														

(continued)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	115	SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; friable.									
82.00	116										
	117										
80.00	118										
	119										
78.00	120	About 68% SAND; about 32% fines.	17	48 50/5"		55		18			PA
	121										
76.00	122										
	123										
74.00	124										
	125	Gravelly to Cobbly SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; friable.									Rig chattering, some cobbles/gravel
72.00	126										
	127										
70.00	128										
	129										
68.00	130		18	50/3"		100					
	131										
66.00	132										
	133										
64.00	134										
	135										Rig chattering, some cobbles/gravel
62.00	136										
	137										
60.00	138	SEDIMENTARY BEDROCK, Delmar Formation - Sandy CLAYSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated.									Change to clay/silt at 138 feet
	139										
58.00	140	About 48% SAND; about 52% fines.	19	49 50/4"		90		18			PA, PI
	141										
56.00	142										
	143	Coarse SANDSTONE bed from 143 to 145 feet.									Hard drilling
54.00	144										
	145										

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				R-21-002
DIST.	COUNTY	ROUTE	POSTMILE	EA 20-134
PROJECT OR BRIDGE NAME: San Diego Regional Rail Corridor				
BRIDGE NUMBER	PREPARED BY: PSS		DATE: 3-25-21	SHEET: 5 of 7

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	145	About 23% SAND; about 77% fines; medium to high plasticity fines. *(continued)*.	20	49 50/3"		100		19			PA, PI
52.00	146										
	147										
50.00	148										
	149		21	47 50/5"		55					
48.00	150	About 37% SAND; about 63% fines.	1			93		21			PA, PI ,UU
	151										
46.00	152							16			
	153	Clayey SANDSTONE; olive gray to dark gray; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; massive; slightly fractured; friable. About 79% SAND; about 21% fines.	2			87					PA, PI ,UC
44.00	154										
	155		3			100					
42.00	156										
	157										UU
40.00	158										
	159	About 60% SAND; about 40% fines.						15			PA UC Packet Test between 160' and 170' depth
38.00	160		4			100					
	161	Mottled yellowish brown.									
36.00	162										
	163	About 85% SAND; about 15% fines.						15			CR, PA ,UC, UU
34.00	164										
	165		5			100					
32.00	166										
	167	2.5" black silty sand layer at 167 feet (organic odor). About 87% SAND; about 13% fines.						14			PA ,UC
30.00	168	Intensely fractured.									
	169										
28.00	170		6			90					UC
	171										
26.00	172										UU
	173	About 67% SAND; about 33% fines.						13			CR, PA, PI
24.00	174		7			100					
	175										

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-002**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-25-21**	SHEET **6 of 7**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	175	*(continued).*									
22.00	176		8			100					
	177										UU
20.00	178	About 73% SAND; about 27% fines. Gray to dark gray.						12			CR, PA, PI
	179										
18.00	180										
	181		9			100					
16.00	182										
	183										
14.00	184										
	185	Slightly fractured to moderately fractured.	10			100					
12.00	186										
	187										
10.00	188	About 59% SAND; about 41% fines.	11			100		12			PA, PI
	189										
8.00	190		12			100					
	191	CLAYSTONE; olive gray to dark gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive; intensely fractured; low to medium plasticity.						16			PA, PI
6.00	192	Clayey SANDSTONE; gray to dark gray; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; massive; slightly fractured; friable.									
	193										
4.00	194	About 67% SAND; about 33% fines.									
	195	About 74% SAND; about 26% fines.	13			100		12			PA, PI
2.00	196										
	197										
0.00	198	Sandy Silty CLAYSTONE; olive gray mottled reddish brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive; slightly fractured to intensely fractured; medium to high plasticity.									
	199										
-2.00	200	Bottom of borehole at 200.0 ft bgs. Groundwater seepage was observed at 58.5 feet. Packer testing was conducted between depths of 160 and 170 feet. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.									
	201										
-4.00	202										
	203										
-6.00	204										
	205										

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				R-21-002
DIST.	COUNTY	ROUTE	POSTMILE	EA 20-134
PROJECT OR BRIDGE NAME: San Diego Regional Rail Corridor				
BRIDGE NUMBER	PREPARED BY PSS		DATE 3-25-21	SHEET 7 of 7

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF	**3-16-21**	**3-17-21**	**N 1,925,801 E 6,253,938**	**R-21-003**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Tri-County		**365.0 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Rotary Wash	**Diedrich D-120**	**4"**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT (1.4")	**Automatic 140lb, 30in drop**	**82%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **NM**; AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING
Cement-Bentonite Grout		**120.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Moisture Content (%)	Remarks
	0	ASPHALT (Asphalt Concrete: 5.5 in.) (No Base).						
	1	SILTY SAND (SM); brown; moist; mostly medium to fine SAND; little nonplastic fines; weak cementation; (FILL).						
363.00	2							
	3							
361.00	4							
	5	SEDIMENTARY BEDROCK, Old Paralic Deposits - SANDSTONE; reddish brown to dark brown; very soft to soft; slightly weathered to fresh; medium to fine grained; poorly indurated; slightly fractured to intensely fractured; friable.	1	50/2"		50		
			1			100		
359.00	6							
	7							
357.00	8	About 82% SAND; about 18% fines.					8	PA
	9							
355.00	10		2			100		
	11							
353.00	12							
	13							
351.00	14							
	15		3			100		
349.00	16							
	17							
347.00	18							
	19							
345.00	20	Slight increase in grain size.	4			100		
	21							
343.00	22							
	23	About 90% SAND; about 10% fines.					9	PA, PI
341.00	24							
	25							

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		*(continued).*		5			40							No Recovery from 25 ft to 28 ft
339.00	26		Gravelly SANDSTONE; reddish brown to dark brown; soft to moderately soft; slightly weathered to fresh; medium to coarse grained; poorly indurated; massive; friable.												
	27														
337.00	28		Little GRAVEL, subrounded to rounded..												
	29														
335.00	30				6			40							No Recovery from 30 ft to 33 ft
	31														
333.00	32														
	33		Little GRAVEL, max 2 in. dia. sub-rounded..												
331.00	34		SEDIMENTARY BEDROCK, Torrey Sandstone Formation - SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; massive; friable; intensely to moderately fractured.												
	35				7			100							
329.00	36		Banding with pale yellow and light gray..												
	37														
327.00	38														
	39														
325.00	40		Unfractured to slightly fractured.		8			100							
	41														
323.00	42														
	43		About 92% SAND; about 8% fines.							11					CR, PA, PI
321.00	44														
	45				9			100							
319.00	46														
	47														
317.00	48														
	49														
315.00	50				10			100							
	51														
313.00	52														
	53														
311.00	54														
	55														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-003**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **2 of 5**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; unfractured to slightly fractured. *(continued)*.		11			100							
309.00	56														
	57														
307.00	58														
	59														
305.00	60				12			100							
	61														
303.00	62														
	63		About 96% SAND; about 4% fines.							14					PA, PI
301.00	64														
	65				13			100							
299.00	66														
	67														
297.00	68														
	69														
295.00	70				14			100							
	71														
293.00	72														
	73														
291.00	74														
	75				15			100							
289.00	76														
	77														
287.00	78														
	79														
285.00	80				16			100							
	81														
283.00	82														
	83		About 94% SAND; about 6% fines.							17					PA
281.00	84														
	85														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium grained; poorly indurated; unfractured to slightly fractured.		17			100							UC
279.00	86														
	87														
277.00	88														
	89														
275.00	90				18			100							
	91														
273.00	92														
	93		Medium to coarse grained SAND.												
271.00	94				19			61							
	95				20			100							Packet Test between 95' and 105' depth
269.00	96														
	97		Gravelly SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium to coarse grained; poorly indurated; some angular to subangular GRAVEL to 2" dia..												Rig chattering, gravelly
267.00	98														UU
	99		SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; fine to coarse grained; poorly indurated.												
265.00	100		Intensely fractured.		21			100							
	101														
263.00	102		Few angular to subangular GRAVEL. About 93% SAND; about 7% fines.							14					PA
	103														
261.00	104		Yellowish brown.												
	105		Unfractured to slightly fractured.		22			100							
259.00	106														
	107														
257.00	108														
	109														
255.00	110				23			100							
	111														
253.00	112														
	113														
251.00	114														
	115														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	115		SANDSTONE; white to yellowish brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; intensely fractured.		24			100							
249.00	116														
	117		Coarse grained SAND.												
247.00	118														
	119														
245.00	120		Bottom of borehole at 120.0 ft bgs Groundwater was not measured, Packer testing was conducted between depths of 95 and 105 feet. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.												
	121														
243.00	122														
	123														
241.00	124														
	125														
239.00	126														
	127														
237.00	128														
	129														
235.00	130														
	131														
233.00	132														
	133														
231.00	134														
	135														
229.00	136														
	137														
227.00	138														
	139														
225.00	140														
	141														
223.00	142														
	143														
221.00	144														
	145														

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-003**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **5 of 5**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF/MH	**3-12-21**	**3-15-21**	**N 1,921,088 E 6,256,153**	**R-21-004**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Tri-County		**160.0 ft**

DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Rotary Wash	**Diedrich D-120**	**4"**

SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Mod Cal (2"), SPT (1.4")	**Automatic 140lb, 30in drop**	**82%**

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Cement-Bentonite Grout		**68.9 (Seepage)**		**110.4 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0	ASPHALT (Asphalt Concrete: 4 in.) (No Base).												
	1	Poorly graded SAND with SILT (SP-SM); light brown to white; moist; mostly coarse to medium SAND; few nonplastic fines; weak cementation [FILL].												
158.00	2													
	3	SEDIMENTARY BEDROCK, Torrey Sandstone Formation - SANDSTONE; white to light brown; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; friable.												
156.00	4													
	5	About 88% SAND; about 12% fines.		1	33 41 50/5"		71							PA, PI
154.00	6													
	7													
152.00	8													
	9													
150.00	10			2	50/5"		100		12	116				SG
	11													
148.00	12													
	13													
146.00	14													
	15													
144.00	16													
	17													
142.00	18													
	19													
140.00	20	About 86% SAND; about 14% fines.		3	44 50/5"		91							PA, PI
	21													
138.00	22													
	23													
136.00	24													
	25													

(continued)

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		*(continued).*												
134.00	26														
	27														
132.00	28														
	29														
130.00	30				4	50/4"		50							
	31														
128.00	32														
	33														
126.00	34														
	35														
124.00	36														
	37														
122.00	38														
	39														
120.00	40				5	50/4"		50							
	41														
118.00	42														
	43														
116.00	44														
	45		About 92% SAND; about 8% fines.		6	50/4.5"		89							PA
114.00	46														
	47														
112.00	48														
	49														
110.00	50		About 91% SAND; about 9% fines.		7	50/5"		100							PA
	51														
108.00	52														
	53														
106.00	54														
	55														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE **BORING RECORD**				HOLE ID **R-21-004**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **2 of 4**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	55	SANDSTONE; white to light brown; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; friable. Cobble bed at 56 feet.	8	50/5"		80					
104.00	56										Rig chattering, cobbly and gravelly
	57										
102.00	58										
	59										
100.00	60	About 89% SAND; about 11% fines.	9	50/5"		100					PA
	61										
98.00	62										
	63										
96.00	64										
	65		10	50/5"		100					Pressure Meter Test at 65 feet
94.00	66										
	67										
92.00	68										
	69										
90.00	70		11	50/3"		33					No recovery
	71										
88.00	72										
	73										
86.00	74										
	75	About 90% SAND; about 10% fines; mottled reddish brown.	12	32 37 50/3"		80		22			CR, PA, PI
84.00	76										
	77										
82.00	78										
	79										
80.00	80		13	42 32 41	73	67					
	81										
78.00	82										
	83	SEDIMENTARY BEDROCK, Delmar Formation - Sandy SILTSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated.									Change to SILTSTONE at 83 feet
76.00	84										
	85										

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				R-21-004
DIST.	COUNTY	ROUTE	POSTMILE	EA 20-134
PROJECT OR BRIDGE NAME: San Diego Regional Rail Corridor				
BRIDGE NUMBER	PREPARED BY PSS		DATE 3-26-21	SHEET 3 of 4

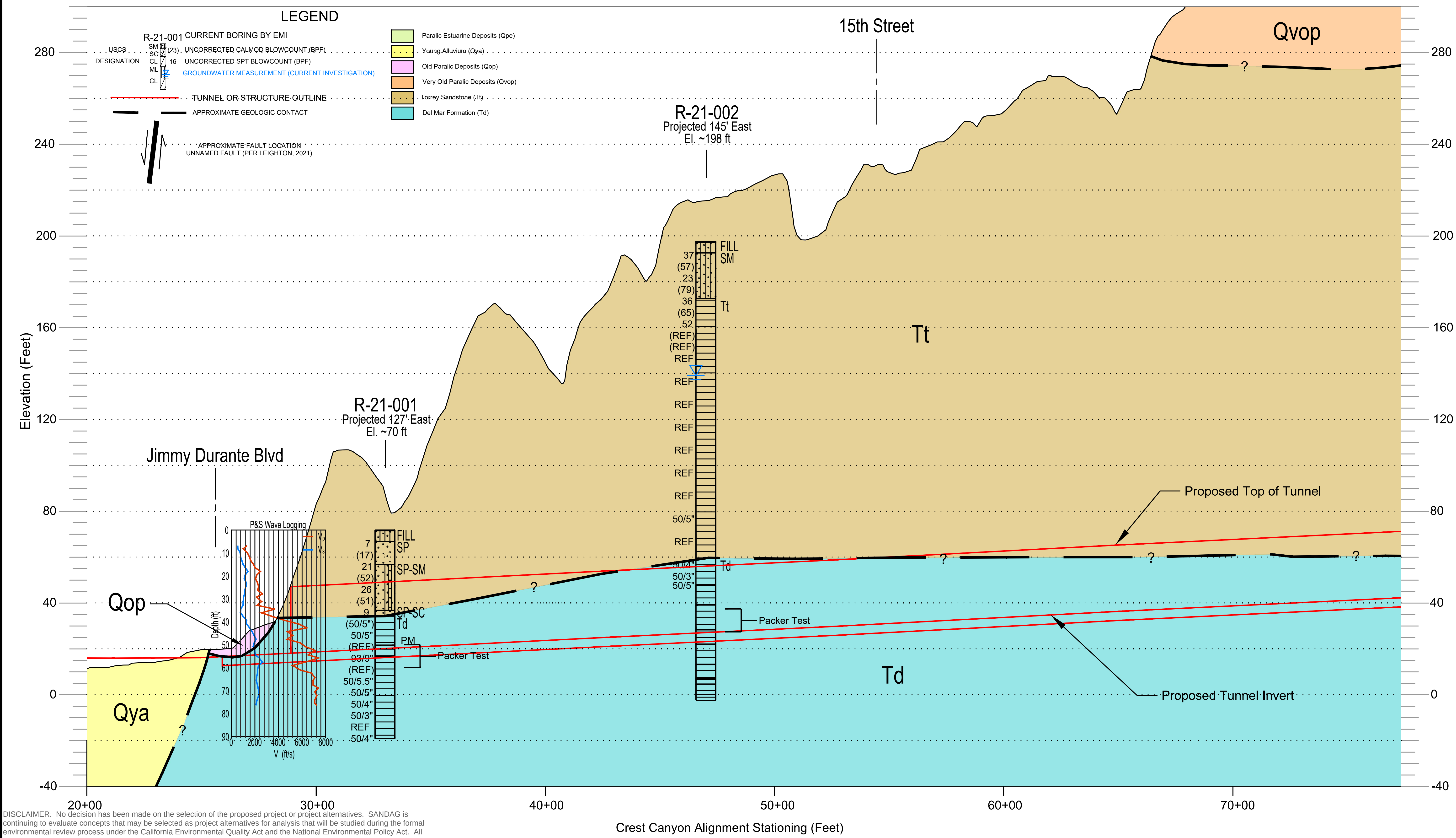
CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85	Clayey SANDSTONE, olive brown to olive gray; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; friable. About 73% SAND; about 27% fines.	X	14	42 50/5"		100		19					PA
74.00	86													
	87													
72.00	88													
	89													
70.00	90		X	15	50/3"		100		24					
	91													
68.00	92													
	93													
66.00	94													
	95													
64.00	96													
	97													
62.00	98													
	99													
60.00	100	Sandy CLAYSTONE; dark gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; medium to high plasticity. About 50% SAND; about 50% fines.	X	16	32 50/6"		100		23					PA, PI
	101													
58.00	102	Cobbles and gravels at 102 feet.												Rig chattering, cobbly and gravelly
	103													
56.00	104													
	105													
54.00	106													
	107													
52.00	108													
	109													
50.00	110	SANDSTONE, olive brown to olive gray; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; friable. About 94% SAND; about 6% fines.	X	17	50/5"		100							PA
	111													
48.00	112	Bottom of borehole at 110.4 ft bgs. Groundwater seepage was observed at 68.9 feet. P&S wave suspension logging was conducted after completion of drilling. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.												
	113													
46.00	114													
	115													

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

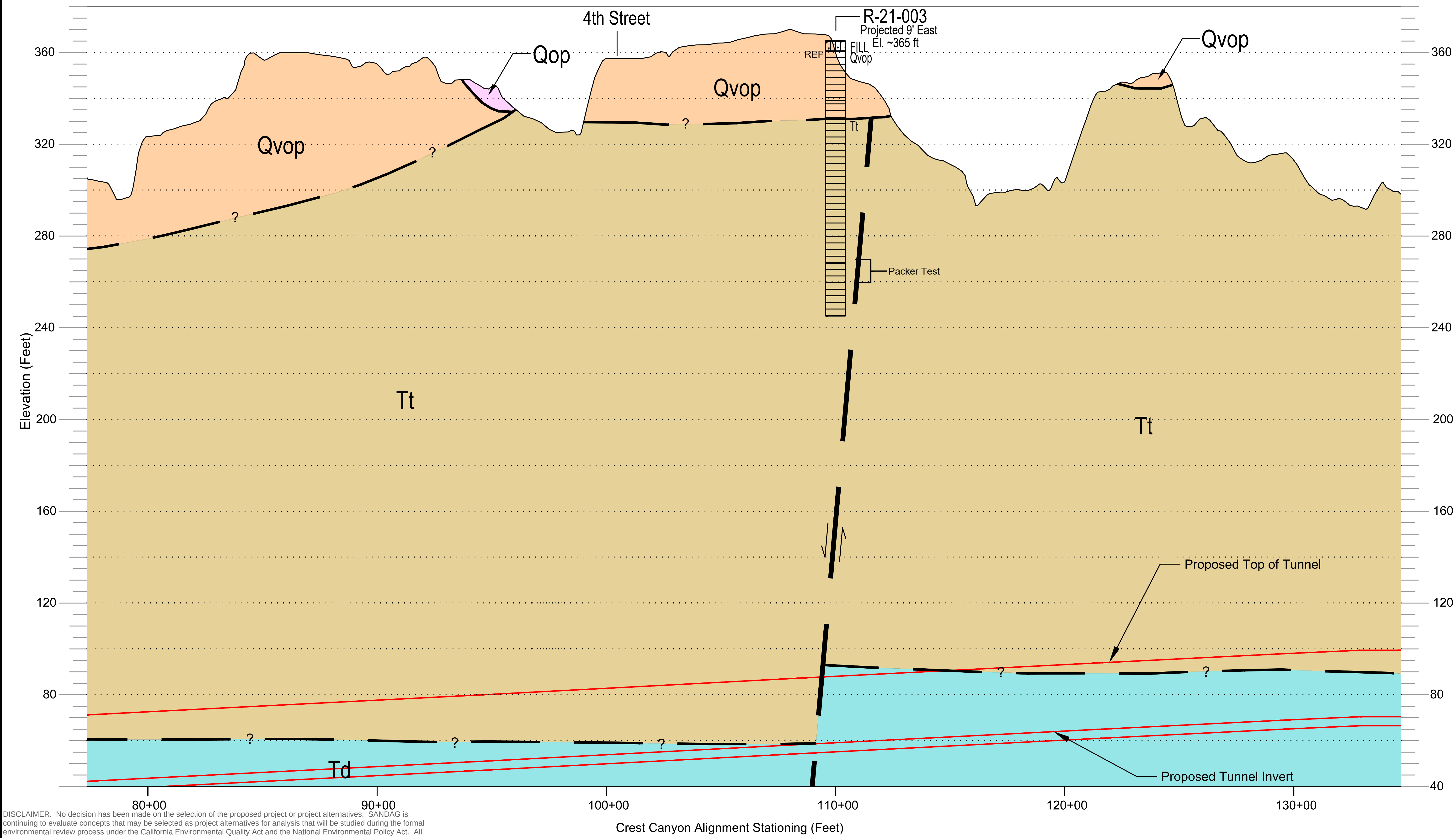
APPENDIX B

Subsurface Cross Section



DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	SD REGIONAL RAIL CORRIDOR ALTERNATIVE ALIGNMENT AND IMPROVEMENTS PROJECT	Subsurface Cross Section
	EMI Project No. 20-134 / Date: May 2021	1 of 3



DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

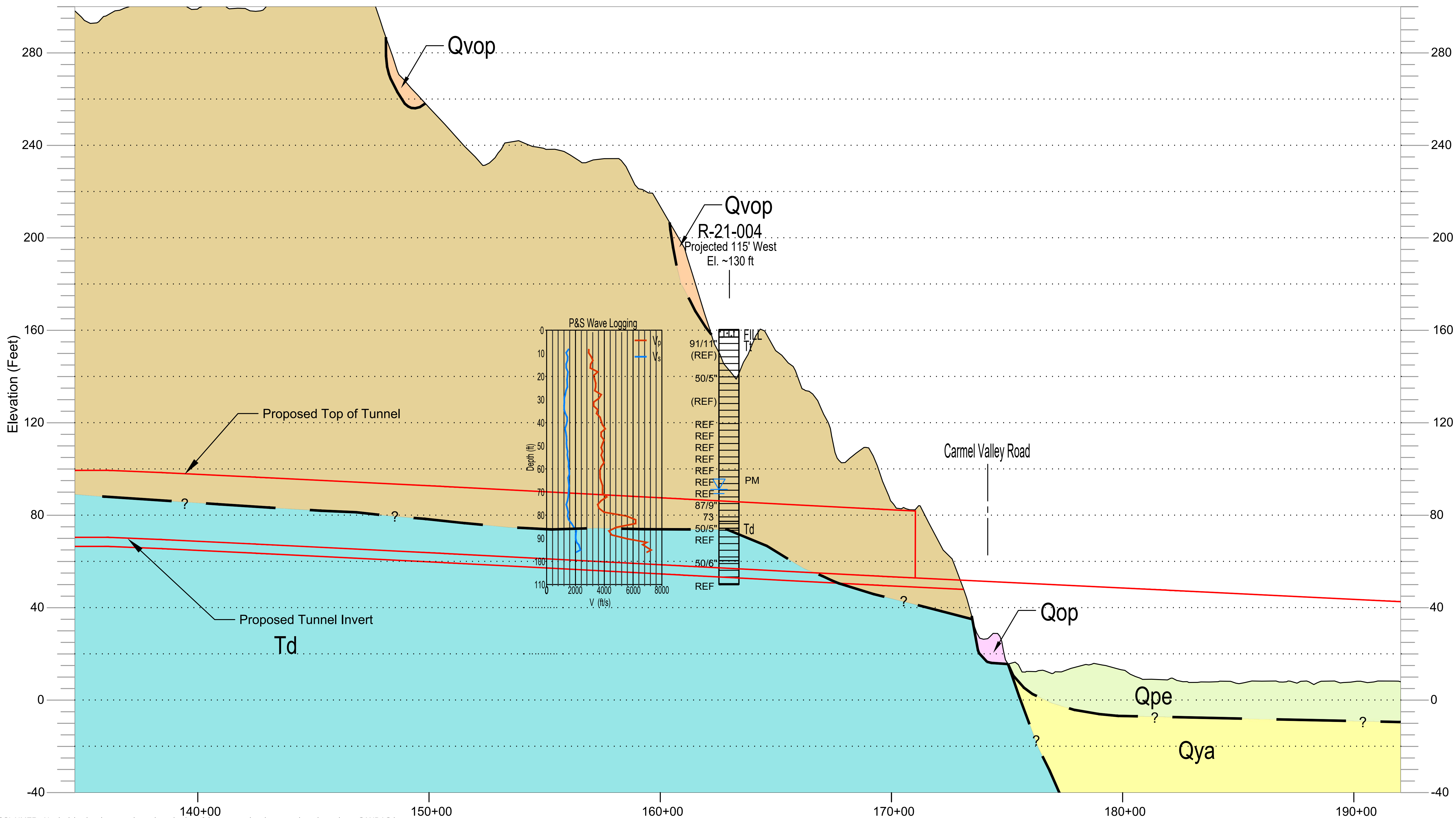
Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

SD REGIONAL RAIL CORRIDOR ALTERNATIVE ALIGNMENT AND IMPROVEMENTS PROJECT

EMI Project No. 20-134

Date: May 2021

Subsurface Cross Section



Crest Canyon Alignment Stationing (Feet)

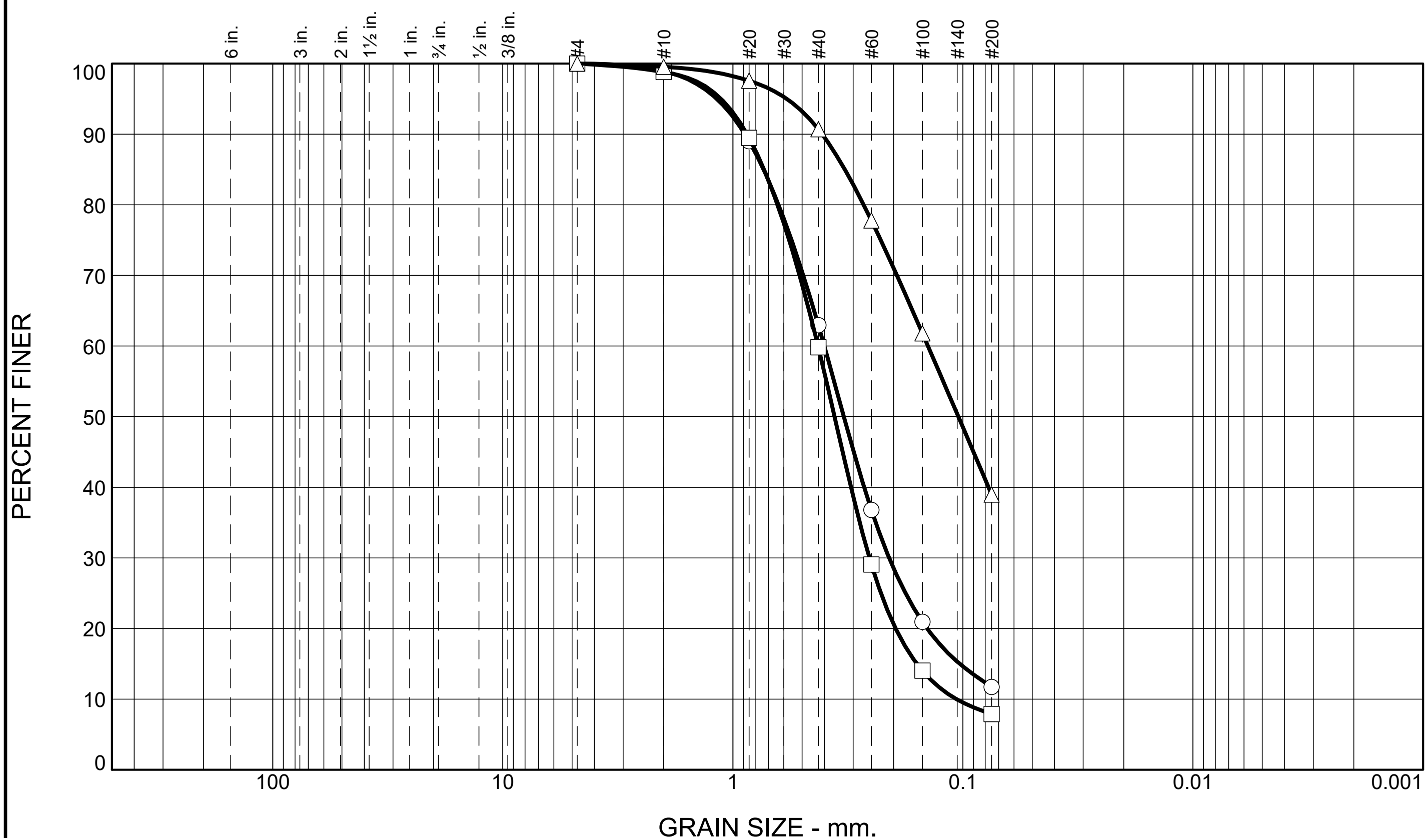
DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	SD REGIONAL RAIL CORRIDOR ALTERNATIVE ALIGNMENT AND IMPROVEMENTS PROJECT	Subsurface Cross Section
	EMI Project No. 20-134 / Date: May 2021	3 of 3

APPENDIX C

Laboratory Results

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
15ft	○	0	0	88	12					
30ft	□	0	0	92	8					
45ft	△	0	0	61	39		SC	29.5	20.7	8.8

SIEVE inches size	PERCENT FINER ○	□	△

	GRAIN SIZE		
D_{60}	0.4004	0.4263	0.1420
D_{30}	0.2093	0.2549	
D_{10}		0.1068	

	COEFFICIENTS		
C_c		1.43	
C_u		3.99	

SIEVE number size	PERCENT FINER ○	□	△
#4	100	100	100
#10	99	99	100
#20	89	89	98
#40	63	60	91
#60	37	29	78
#100	21	14	62
#200	12	7.9	39

Material Description
○
□
△

REMARKS:
○ LAB #116
□ LAB #116
△ LAB #116

○ Depth: 15 Sample Number: B1
□ Depth: 30 Sample Number: B1
△ Depth: 45 Sample Number: B1

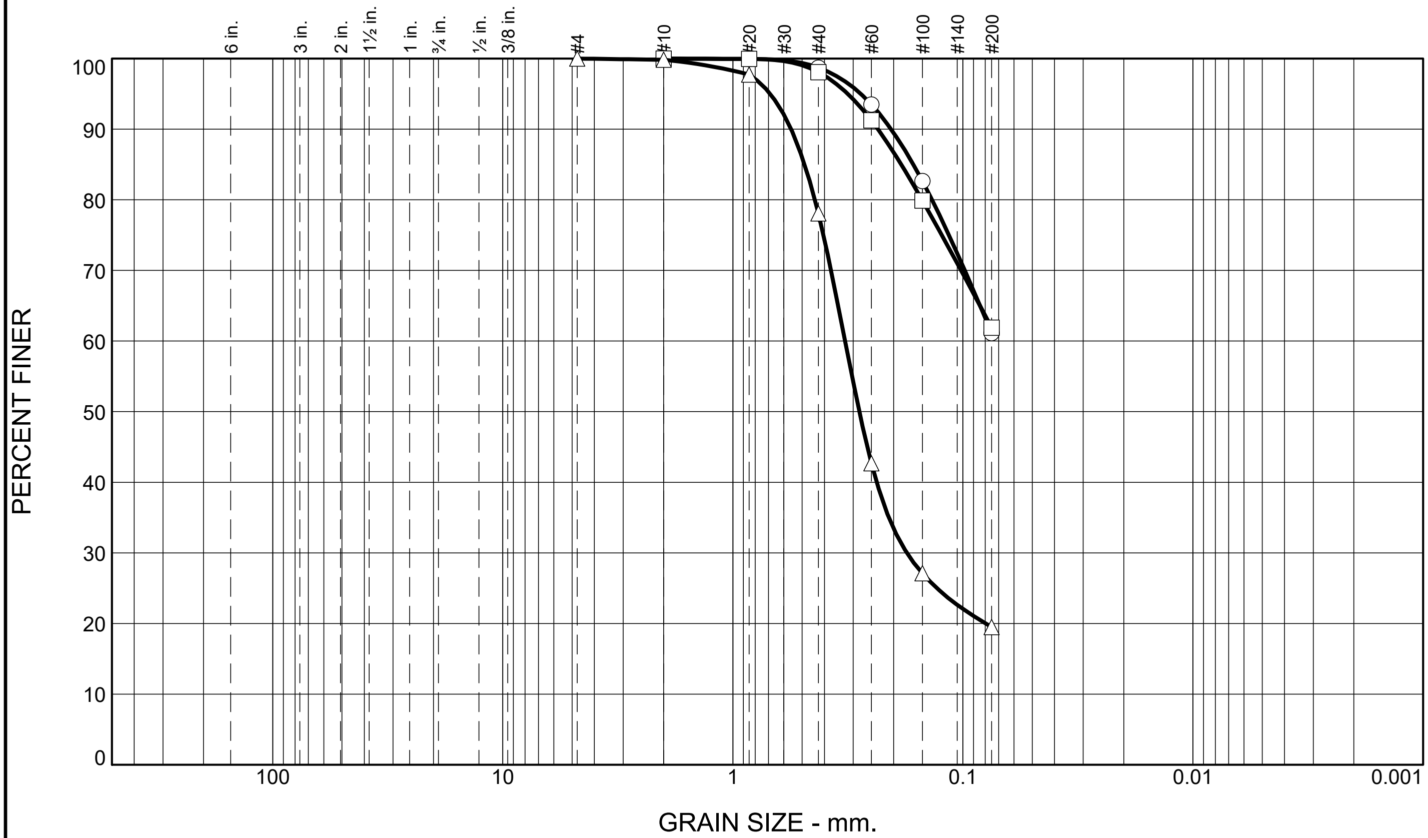
MTGL, Inc.

San Diego, CA

Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR
Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
65ft	○	0	0	39	61		CL	41.9	22.5	19.4
70ft	□	0	0	38	62					
20ft	△	0	0	80	20					

SIEVE inches size	PERCENT FINER ○	□	△
GRAIN SIZE			
D_{60}			0.3253
D_{30}			0.1754
D_{10}			
COEFFICIENTS			
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4			100
#10	100	100	100
#20	100	100	98
#40	99	98	78
#60	94	91	43
#100	83	80	27
#200	61	62	20

Material Description
○
□
△

REMARKS:
○ LAB #116
□ LAB #116
△ LAB #116

○ Depth: 65 Sample Number: B1
□ Depth: 70 Sample Number: B1
△ Depth: 25 Sample Number: B2

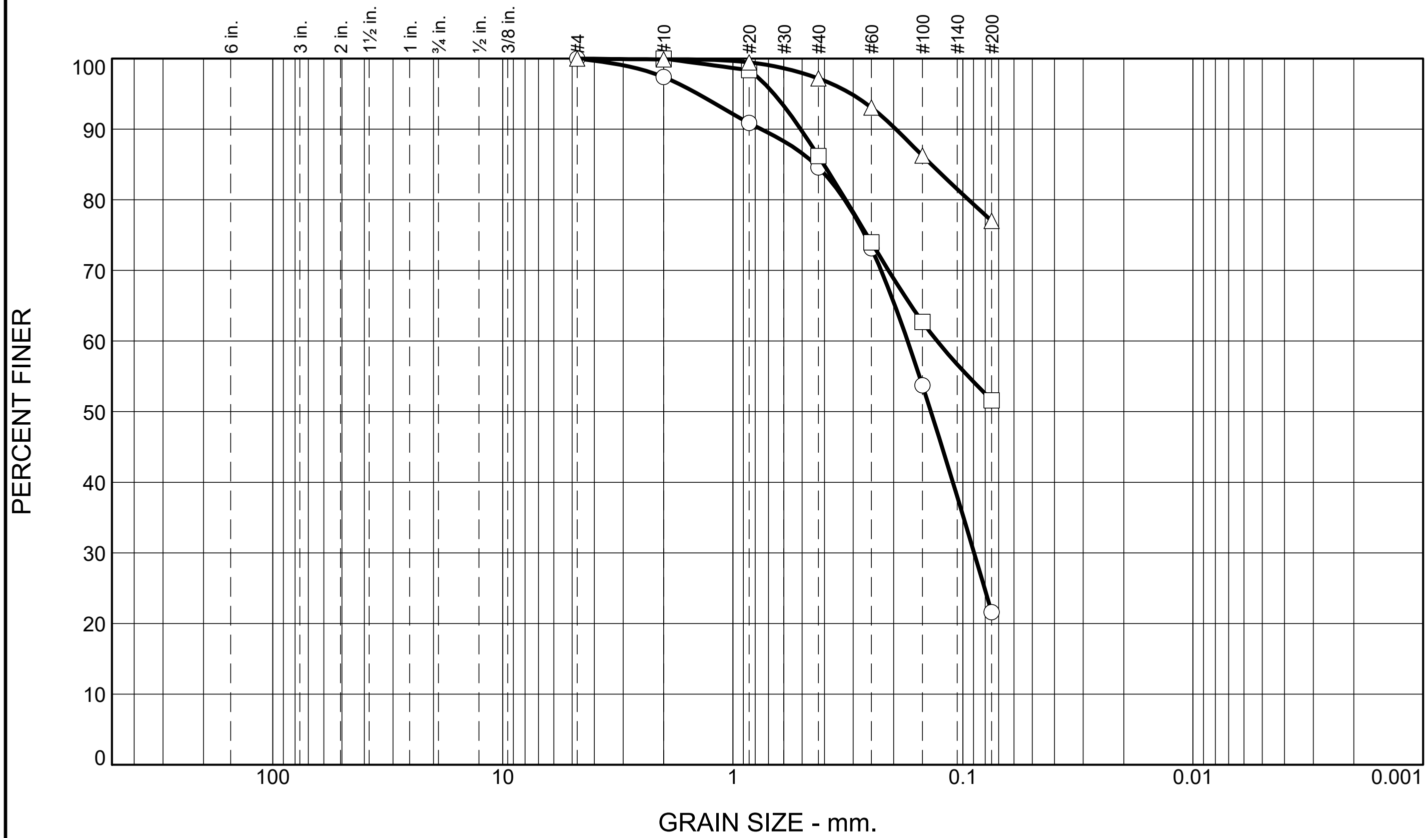
MTGL, Inc.

San Diego, CA

Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR
Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
35ft ○	0	0	78	22					
140ft □	0	0	48	52		CL	47.2	22.6	24.6
145ft △	0	0	23	77		CH	50.7	19.0	31.7

SIEVE inches size	PERCENT FINER ○	□	△
	GRAIN SIZE		
D_{60}	0.1740	0.1295	
D_{30}	0.0894		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	100		100
#10	97	100	100
#20	91	98	99
#40	85	86	97
#60	73	74	93
#100	54	63	86
#200	22	52	77

Material Description
○
□
△

REMARKS:
○ LAB #116
□ LAB #116
△ LAB #116

○ Depth: 35 Sample Number: B2
□ Depth: 140 Sample Number: B2
△ Depth: 145 Sample Number: B2

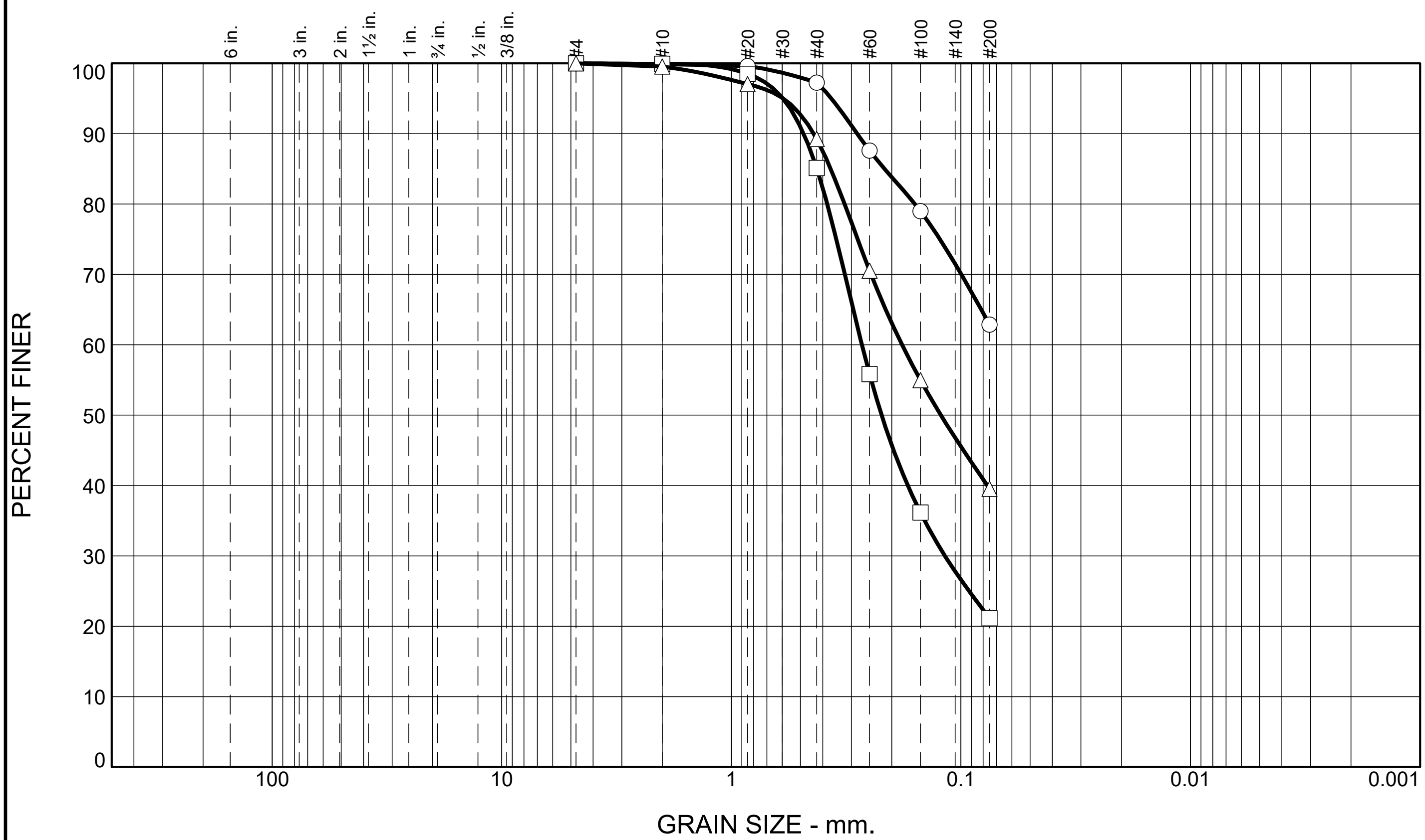
MTGL, Inc.

San Diego, CA

Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR
Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
150ft	○	0	0	37	63		CL	47.6	21.7	25.9
150-153ft	□	0	0	79	21		SC	36.2	20.2	16.0
157-161ft	△	0	0	60	40					

SIEVE inches size	PERCENT FINER ○	□	△

	GRAIN SIZE		
D_{60}		0.2698	0.1804
D_{30}		0.1174	
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	100	100	100
#10	100	100	100
#20	100	99	97
#40	97	85	89
#60	88	56	71
#100	79	36	55
#200	63	21	40

Material Description
○
□
△

REMARKS:
○ LAB #116
□ LAB #116
△ LAB #116

○ Depth: 150 Sample Number: B2
□ Depth: 150-153 Sample Number: B2
△ Depth: 157-161 Sample Number: B2

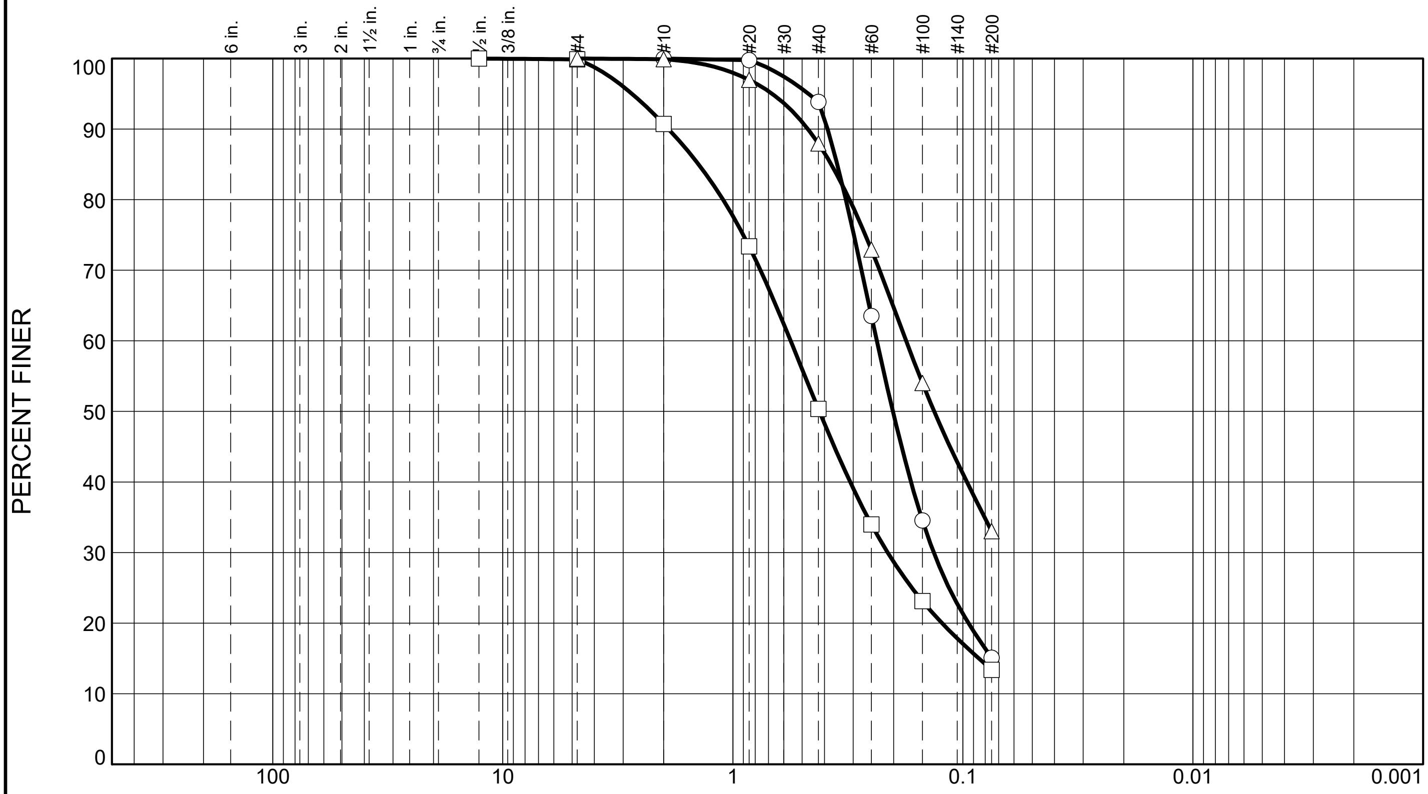
MTGL, Inc.

San Diego, CA

Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR
Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
160-165ft	○	0	0	85	15					
167ft	□	0	0	87	13		SM	NV	NP	NP
170-175ft	△	0	0	67	33		SC	36.6	19.5	17.1

SIEVE inches size	PERCENT FINER ○	□	△
.5		100	

	GRAIN SIZE		
D_{60}	0.2368	0.5610	0.1766
D_{30}	0.1339	0.2124	
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4		100	100
#10	100	91	100
#20	100	73	97
#40	94	50	88
#60	64	34	73
#100	35	23	54
#200	15	13	33

Material Description

○

□

△

REMARKS:

○ LAB #116

□ LAB #116

△ LAB #116

○ Depth: 160-165 Sample Number: B2

□ Depth: 167 Sample Number: B2

△ Depth: 170-175 Sample Number: B2

MTGL, Inc.

San Diego, CA

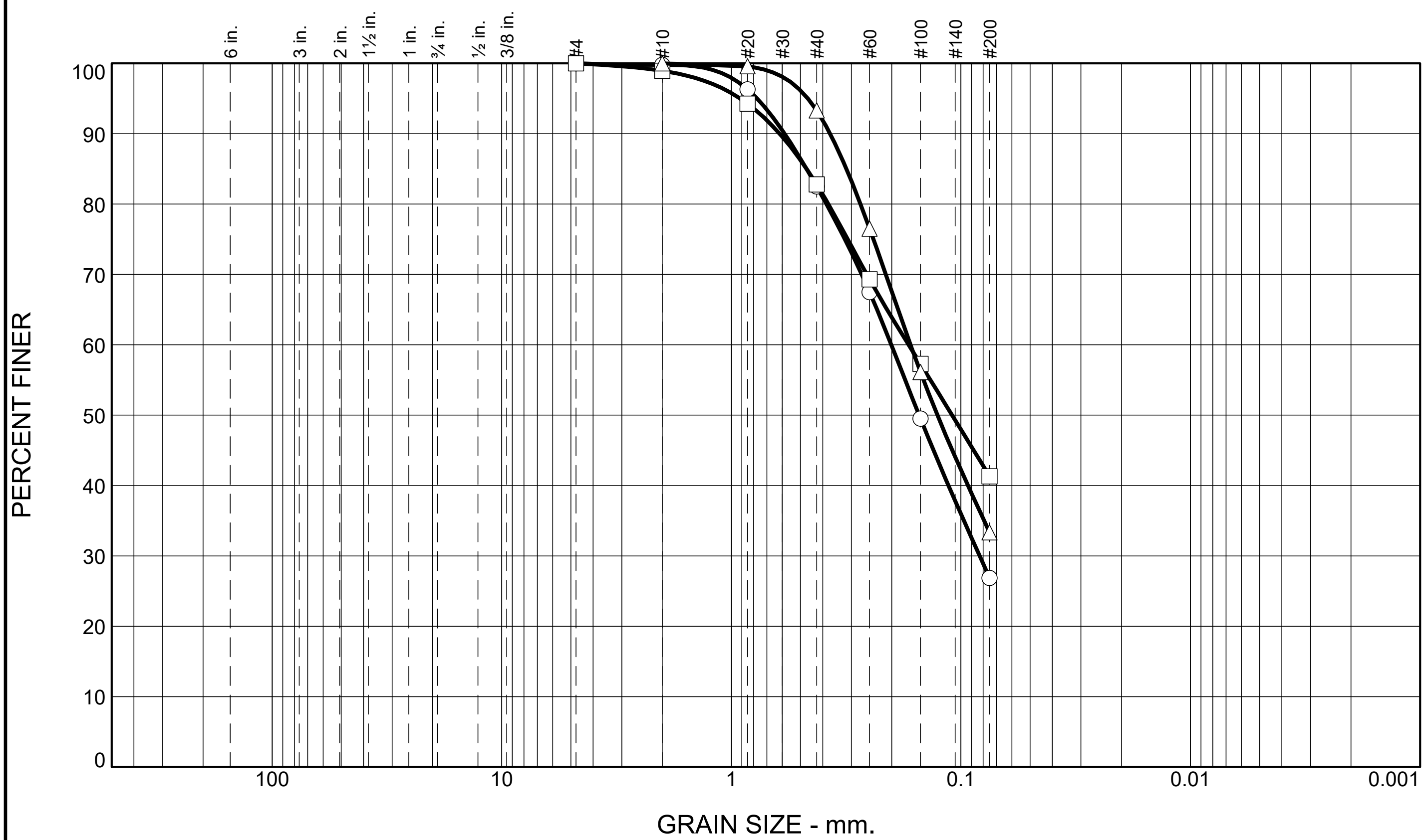
Client:

Project: SAN DIEGO REGIONAL RAIL CORRIDOR

Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
175-180ft	○	0	0	73	27		SC	42.9	21.7	21.2
185-190ft	□	0	0	59	41		SC	39.2	21.6	17.6
191ft	△	0	0	67	33		SC	35.1	19.2	15.9

SIEVE inches size	PERCENT FINER ○	□	△

	GRAIN SIZE ○	□	△
D_{60}	0.2010	0.1690	0.1658
D_{30}	0.0829		
D_{10}			

	COEFFICIENTS ○	□	△
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	100	100	
#10	100	99	100
#20	96	94	100
#40	83	83	93
#60	67	69	77
#100	50	57	56
#200	27	41	33

Material Description

○

□

△

REMARKS:

○ LAB #116

□ LAB #116

△ LAB #116

○ Depth: 175-180 Sample Number: B2

□ Depth: 185-190 Sample Number: B2

△ Depth: 191 Sample Number: B2

MTGL, Inc.

San Diego, CA

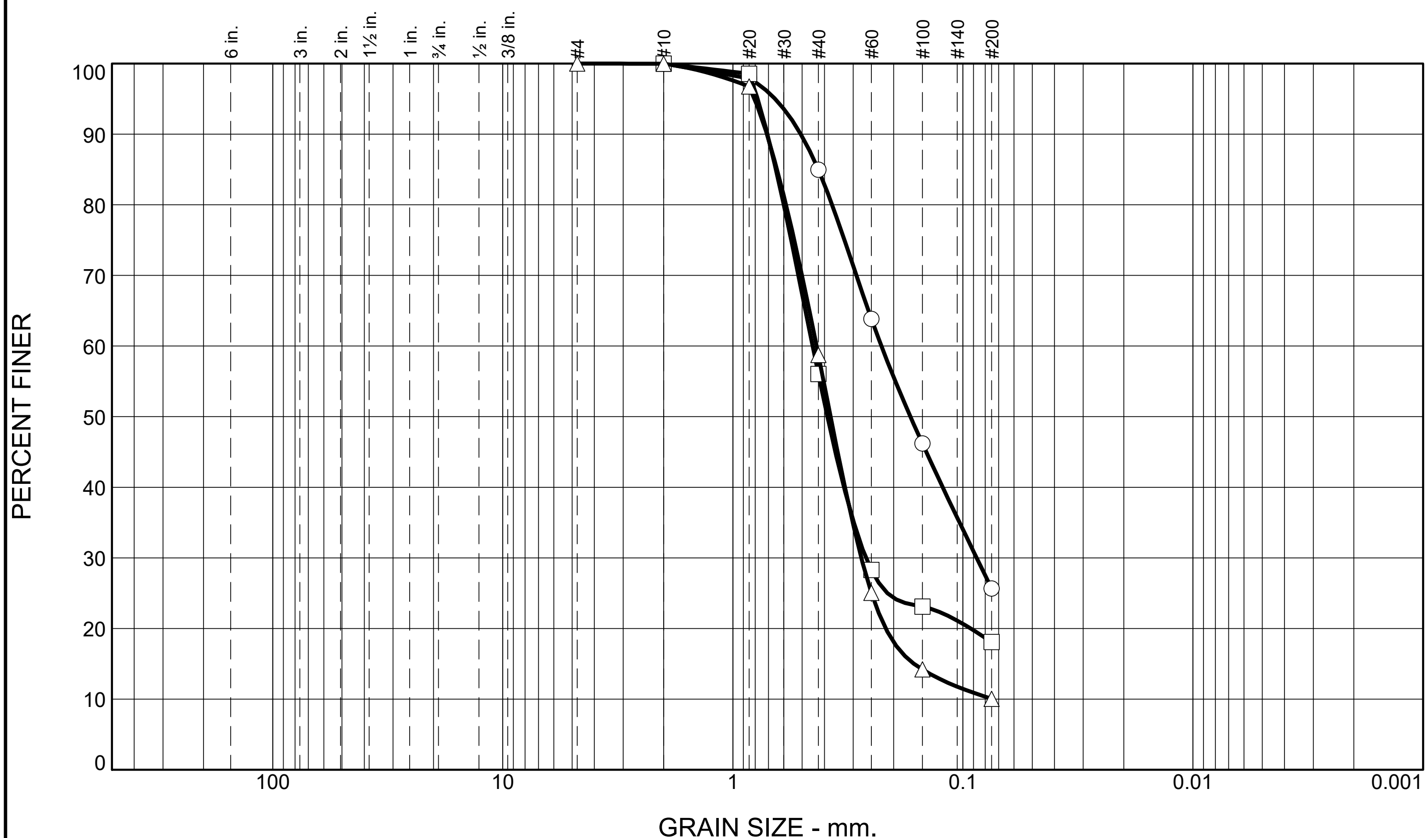
Client:

Project: SAN DIEGO REGIONAL RAIL CORRIDOR

Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
195ft	○	0	0	74	26		SC-SM	25.8	19.9	5.9
5-10ft	□	0	0	82	18					
20-25ft	△	0	0	90	10		SP-SC	40.5	19.7	20.8

SIEVE inches size	PERCENT FINER ○	□	△
	GRAIN SIZE		
D_{60}	0.2262	0.4501	0.4327
D_{30}	0.0872	0.2639	0.2765
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4			100
#10	100	100	100
#20	98	99	97
#40	85	56	59
#60	64	28	25
#100	46	23	14
#200	26	18	10

Material Description
○
□
△

REMARKS:
○ LAB #116
□ LAB #116
△ LAB #116

○ Depth: 195 Sample Number: B2
□ Depth: 5-10 Sample Number: B3
△ Depth: 20-25 Sample Number: B3

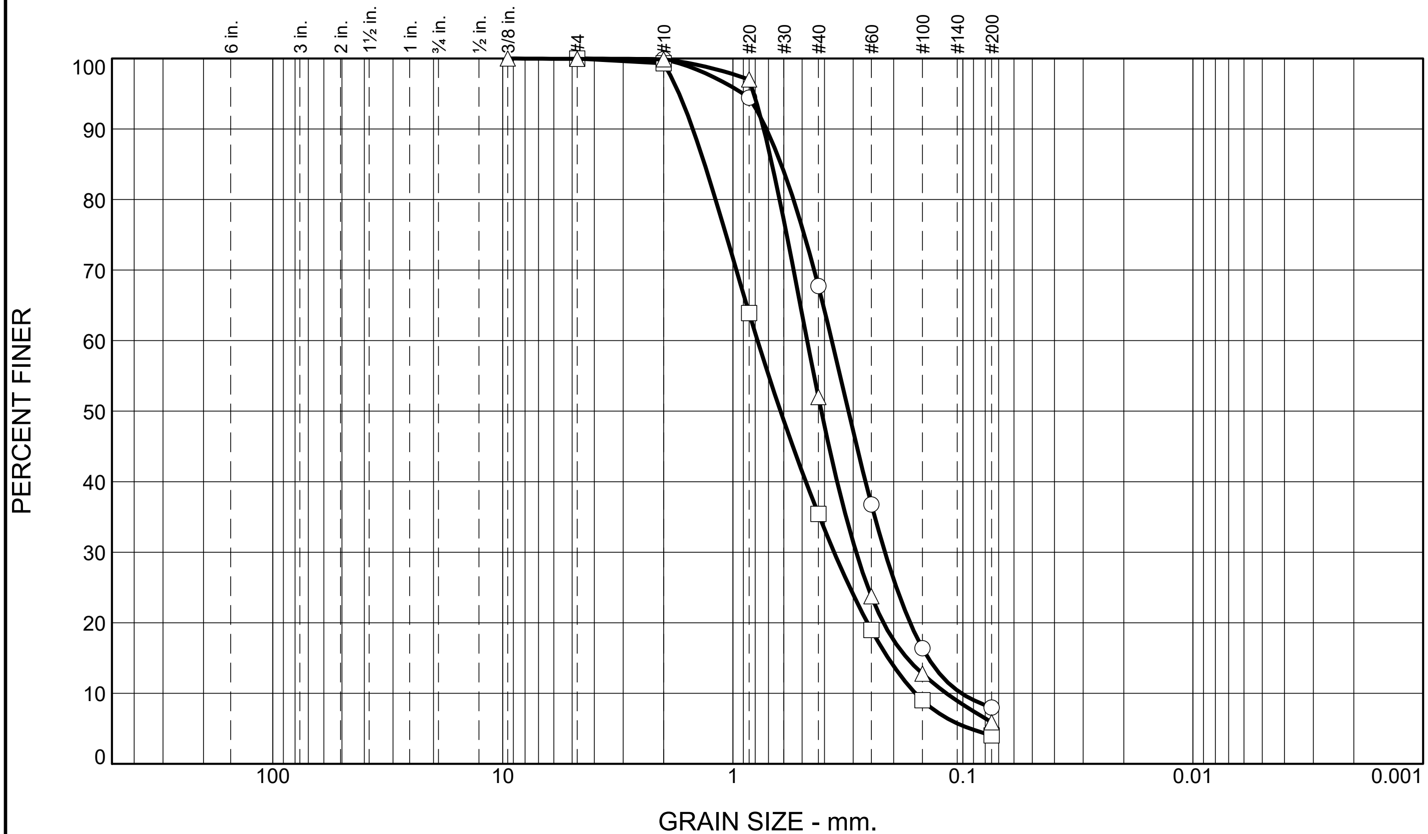
MTGL, Inc.

San Diego, CA

Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR
Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
40-45ft	○	0	0	92	8		SP-SC	34.3	21.2	13.1
60-65ft	□	0	0	96	4		SP	NV	NP	NP
80-85ft	△	0	0	94	6		SP-SC	36.2	24.5	11.7

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
.375			100

	GRAIN SIZE		
D_{60}	0.3714	0.7811	0.4753
D_{30}	0.2183	0.3630	0.2912
D_{10}	0.1016	0.1609	0.1179

	COEFFICIENTS		
C_c	1.26	1.05	1.51
C_u	3.66	4.85	4.03

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#4	100	100	100
#10	100	99	100
#20	94	64	97
#40	68	35	52
#60	37	19	24
#100	16	9	13
#200	8.0	4.0	5.9

Material Description

○

□

△

REMARKS:

○ LAB #116

□ LAB #116

△ LAB #116

○ Depth: 40-45 Sample Number: B3

□ Depth: 60-65 Sample Number: B3

△ Depth: 80-85 Sample Number: B3

MTGL, Inc.

San Diego, CA

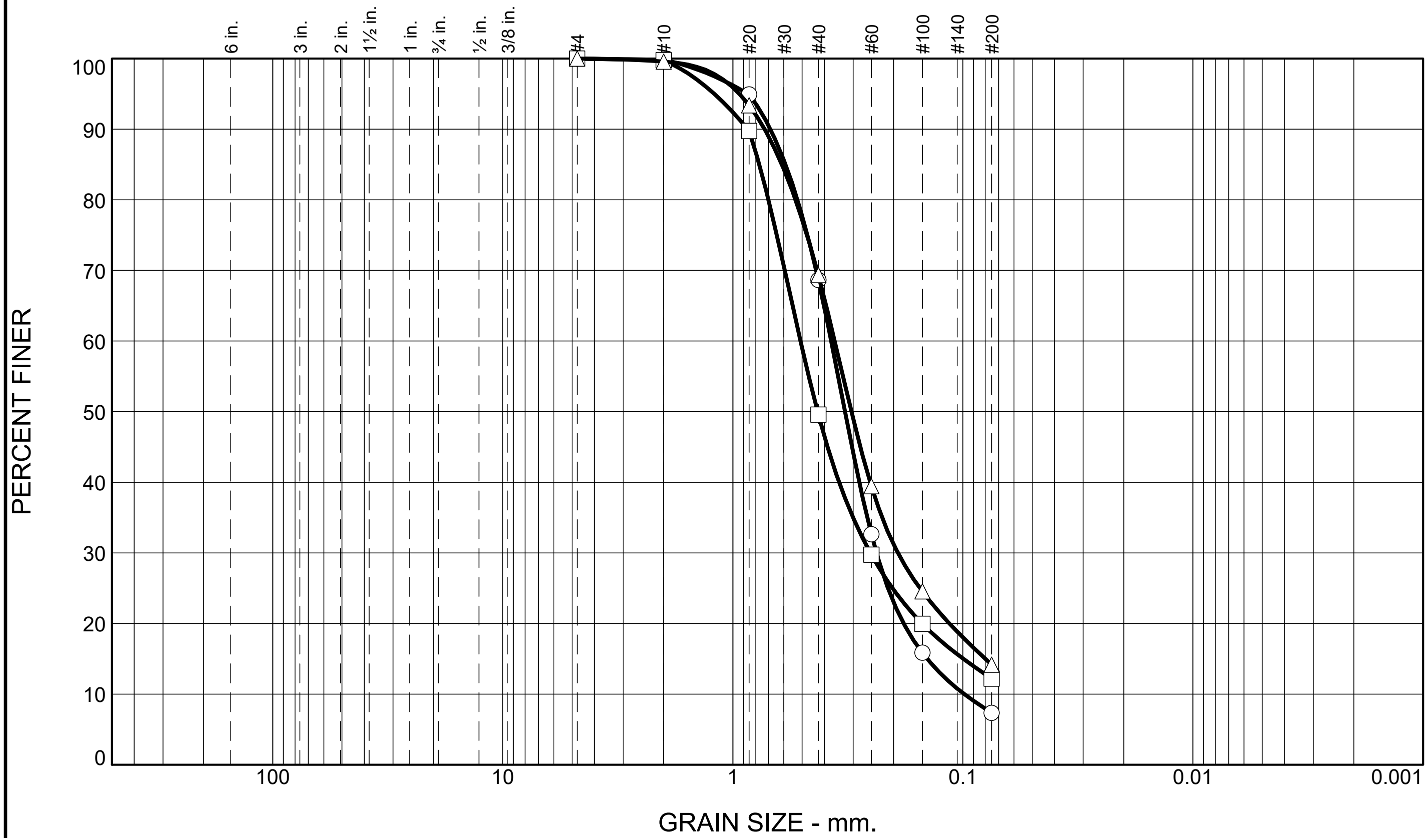
Client:

Project: SAN DIEGO REGIONAL RAIL CORRIDOR

Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



		+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
100-105ft	○	0	0	93	7		SP-SM	NV	NP	NP
5ft	□	0	0	88	12		SP-SC	42.9	21.4	21.5
20ft	△	0	0	86	14		SC	39.2	21.2	18.0

SIEVE inches size	PERCENT FINER ○	□	△
	GRAIN SIZE		
D_{60}	0.3738	0.5079	0.3609
D_{30}	0.2376	0.2525	0.1918
D_{10}	0.0986		
	COEFFICIENTS		
C_c	1.53		
C_u	3.79		

SIEVE number size	PERCENT FINER ○	□	△
#4	100	100	100
#10	100	100	100
#20	95	90	93
#40	69	50	69
#60	33	30	39
#100	16	20	25
#200	7.3	12	14

Material Description
○
□
△

REMARKS:
○ LAB #116
□ LAB #116
△ LAB #116

○ Depth: 100-105 Sample Number: B3
□ Depth: 5 Sample Number: B4
△ Depth: 20 Sample Number: B4

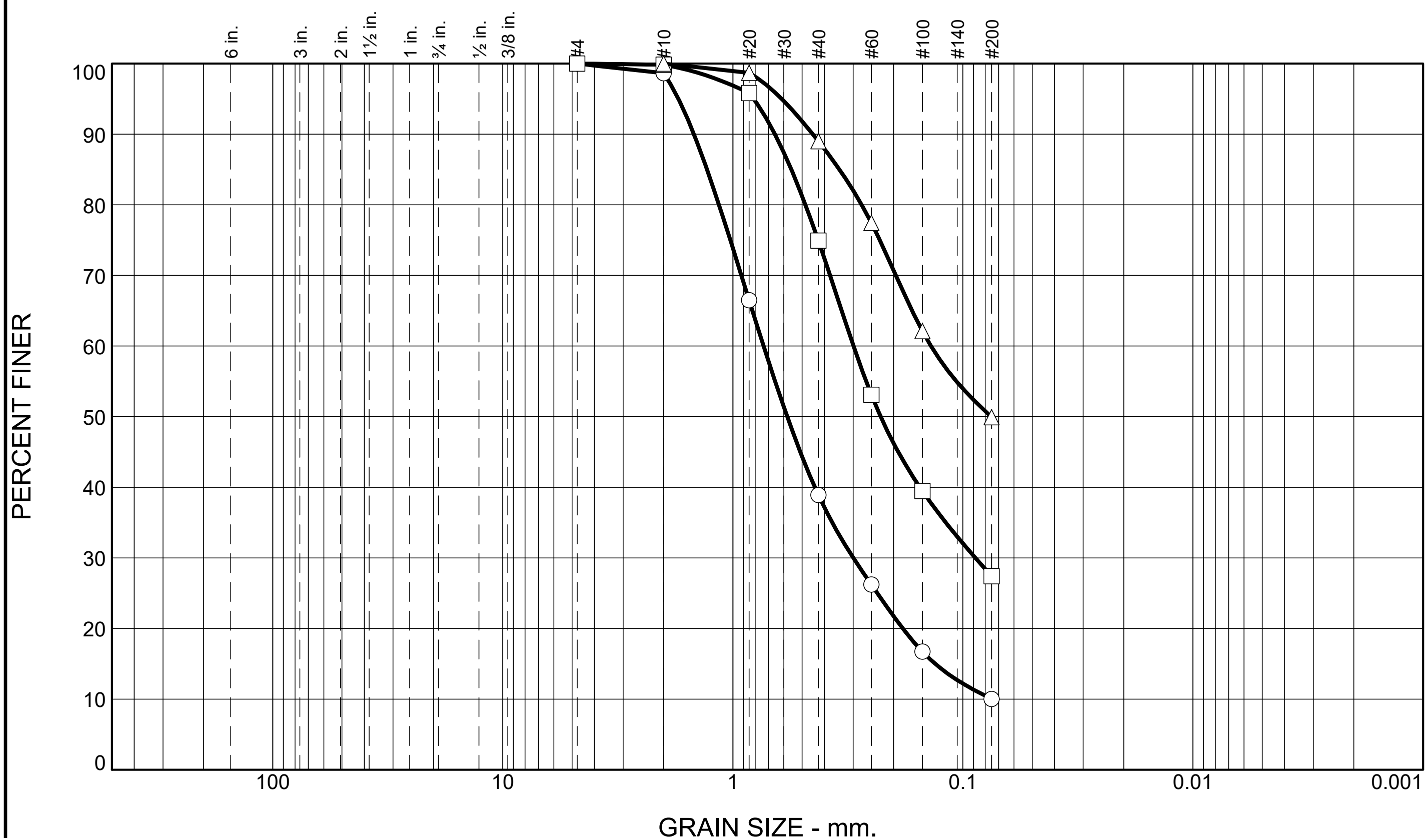
MTGL, Inc.

San Diego, CA

Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR
Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
75ft ○	0	0	90	10		SP-SC	35.1	19.2	15.9
85ft □	0	0	73	27					
100ft △	0	0	50	50		CH	55.8	23.8	32.0

SIEVE inches size	PERCENT FINER ○	□	△

	GRAIN SIZE ○	□	△
D_{60}	0.7352	0.2989	0.1376
D_{30}	0.2994	0.0883	
D_{10}			

	COEFFICIENTS ○	□	△
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	□	△
#4	100	100	
#10	99	100	100
#20	66	96	99
#40	39	75	89
#60	26	53	77
#100	17	39	62
#200	10	27	50

Material Description
○
□
△

REMARKS:
○ LAB #116
□ LAB #116
△ LAB #116

○ Depth: 75 Sample Number: B4
□ Depth: 85 Sample Number: B4
△ Depth: 100 Sample Number: B4

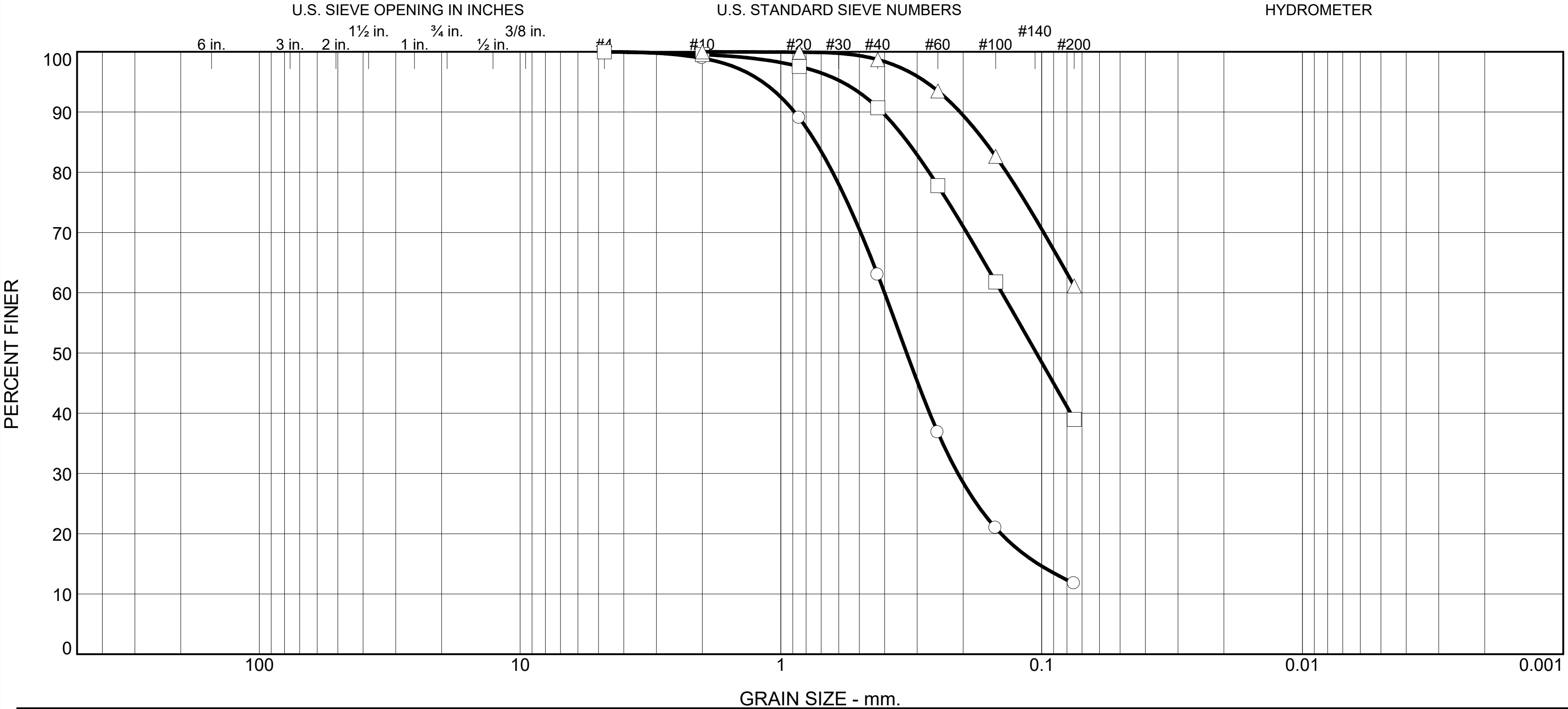
MTGL, Inc.

San Diego, CA

Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR
Project No.: 6320-A01 Figure

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0	0	0	1	36	51	12	
□	0	0	0	0	9	52	39	
△	0	0	0	0	1	38	61	

	Identification	Date Sampled	Date Received	Date Tested
○	Depth: 15 Sample Number: B1	3/2/21	4/1/21	4/8/21
□	Depth: 45 Sample Number: B1	3/2/21	4/1/21	4/8/21
△	Depth: 65 Sample Number: B1	3/2/21	4/1/21	4/8/21

Client

Project SAN DEIGO REGIONAL RAIL CORRIDOR

Project No. 6320-A01 Figure

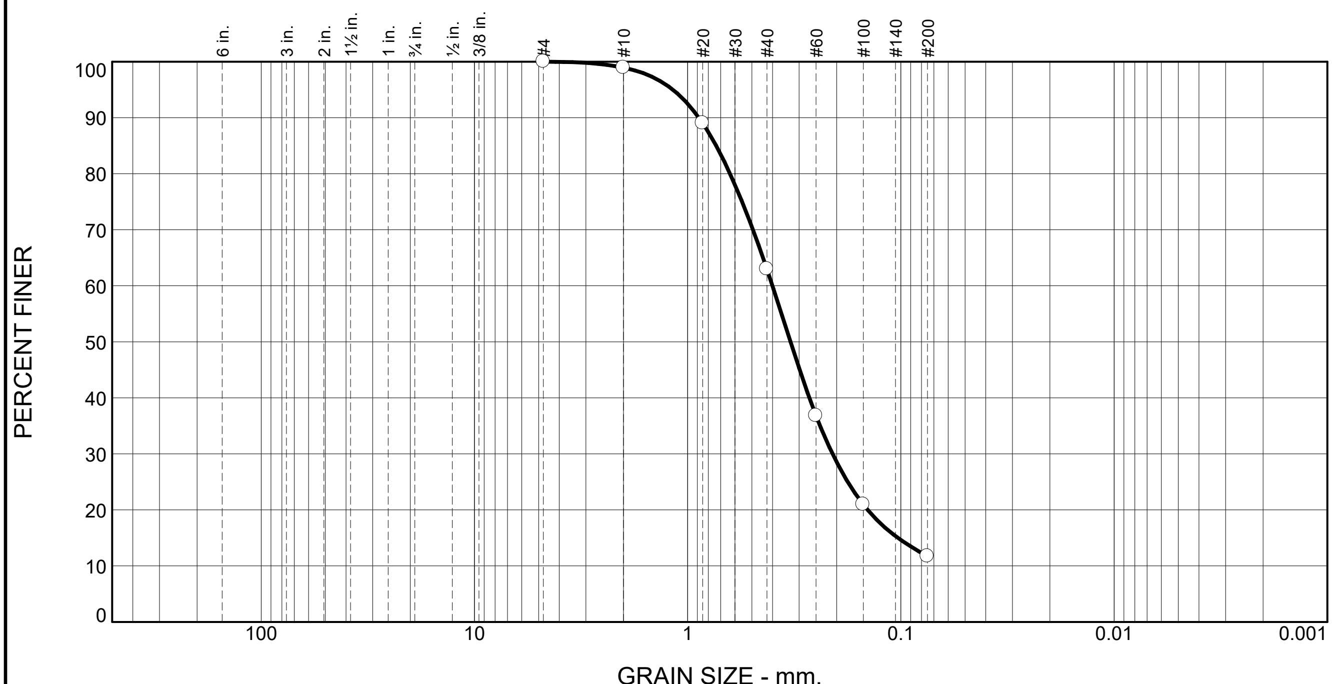
MTGL, Inc.

San Diego, CA

○ LAB #116
□ LAB #116
△ LAB #116

Tested By: JH **Checked By:** SC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	1	36	51	12	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	100		
#10	99		
#20	89		
#40	63		
#60	37		
#100	21		
#200	12		

* (no specification provided)

Material Description

Atterberg Limits (ASTM D 4318)

PL= **LL=** **PI=**

Classification

USCS (D 2487)= **AASHTO (M 145)=**

Coefficients

D_{90}= 0.8862 D_{85}= 0.7351 D_{60}= 0.4004
D_{50}= 0.3294 D_{30}= 0.2093 D_{15}= 0.1032
D_{10}= C_u= C_c=

Remarks

LAB #116

Date Received: 4/1/21 **Date Tested:** 4/8/21
Tested By: JH
Checked By: SC
Title: ENGINEER

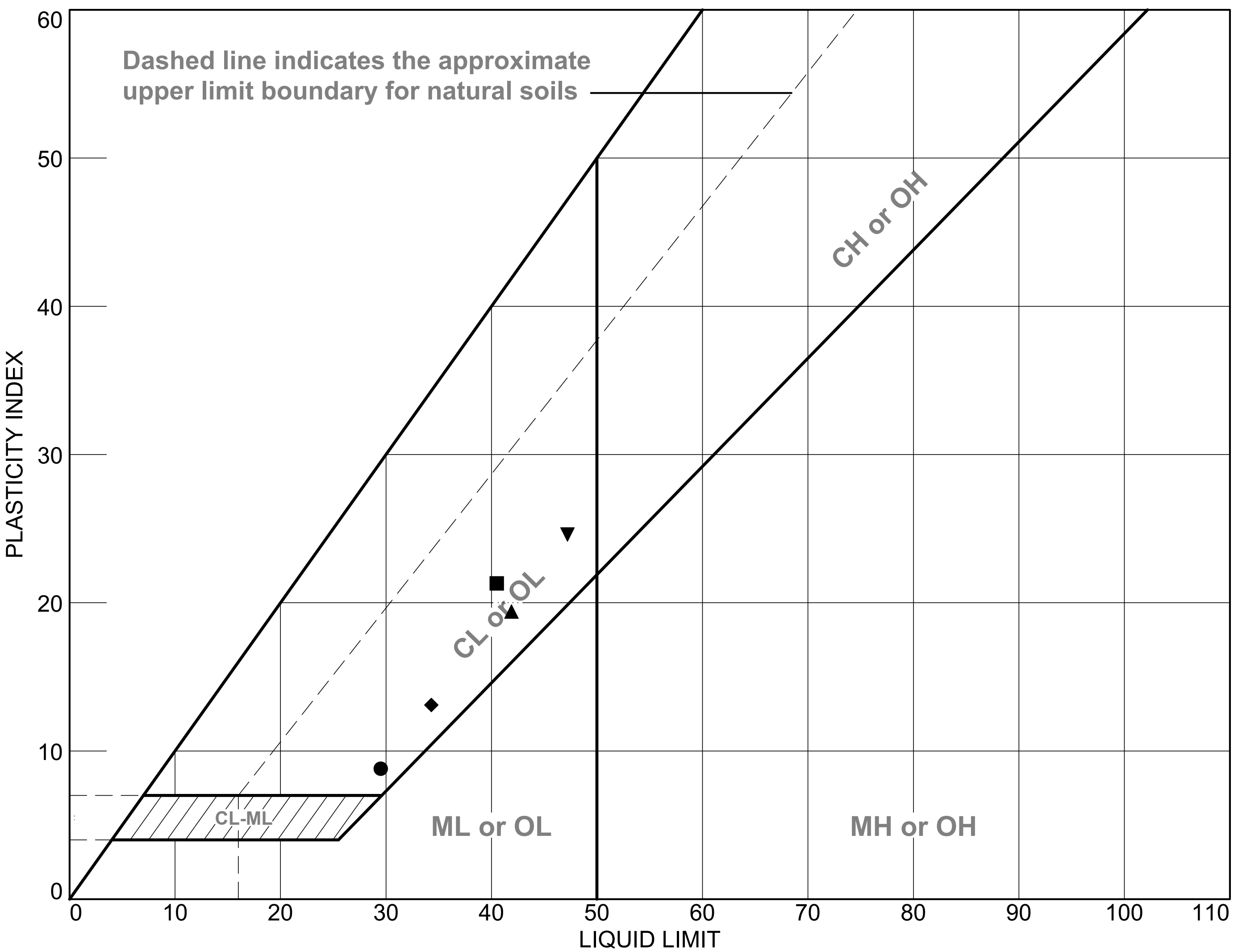
Sample Number: B1 **Depth:** 15 **Date Sampled:** 3/2/21

MTGL, Inc.

San Diego, CA

Client:
Project: SAN DEIGO REGIONAL RAIL CORRIDOR
Project No: 6320-A01 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		B1	45		20.7	29.5	8.8	SC
■		B1	55		19.2	40.5	21.3	
▲		B1	65		22.5	41.9	19.4	CL
◆		B1	80		21.2	34.3	13.1	
▼		B2	140		22.6	47.2	24.6	CL

MTGL, Inc.

San Diego, CA

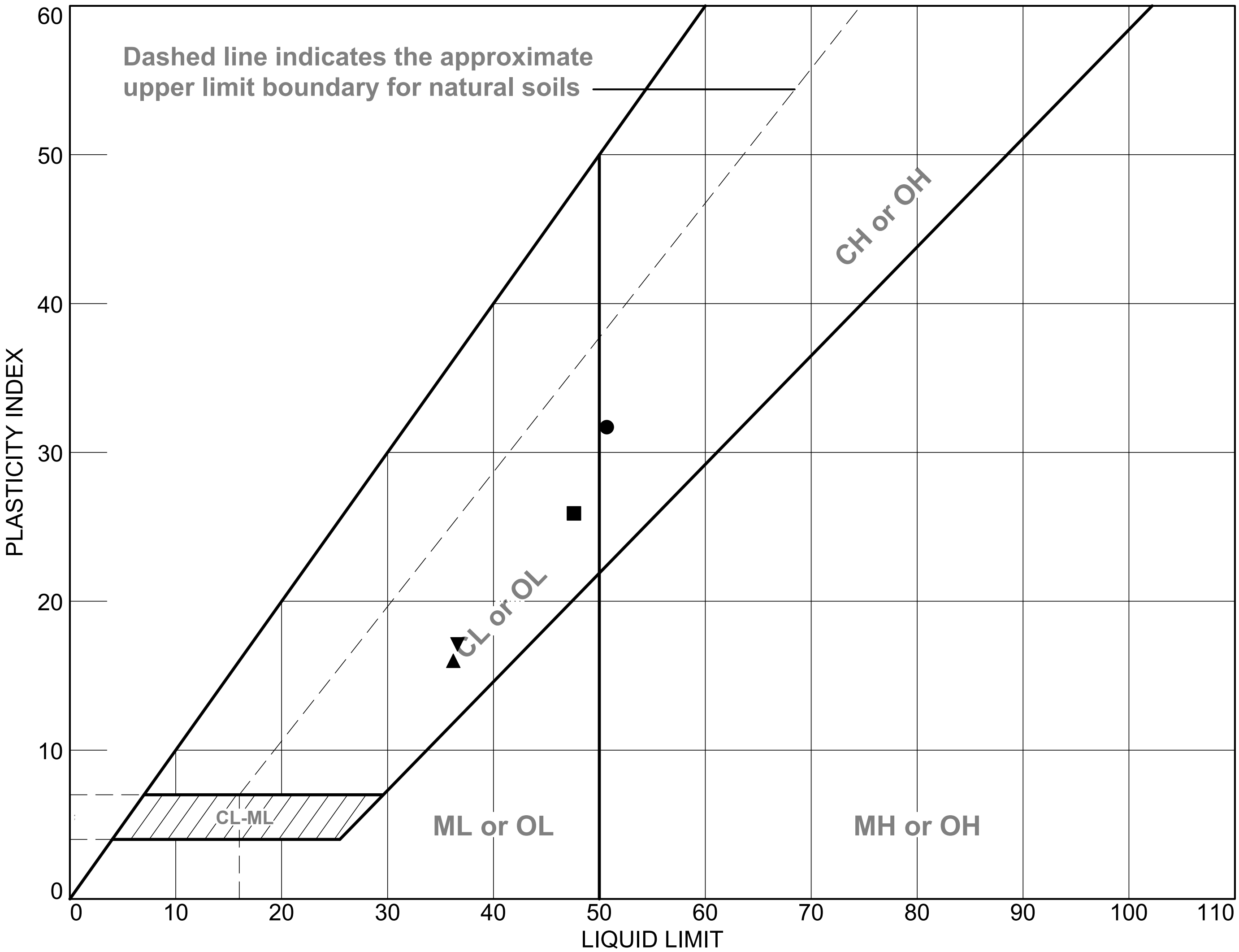
Client:

Project: SAN DIEGO REGIONAL RAIL CORRIDOR

Project No.: 6320-A01 **Figure**

Tested By: JH **Checked By:** SC

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		B2	145		19.0	50.7	31.7	CH
■		B2	150		21.7	47.6	25.9	CL
▲		B2	150-153		20.2	36.2	16.0	SC
◆		B2	167		NP	NV	NP	SM
▼		B2	170-175		19.5	36.6	17.1	SC

MTGL, Inc.

San Diego, CA

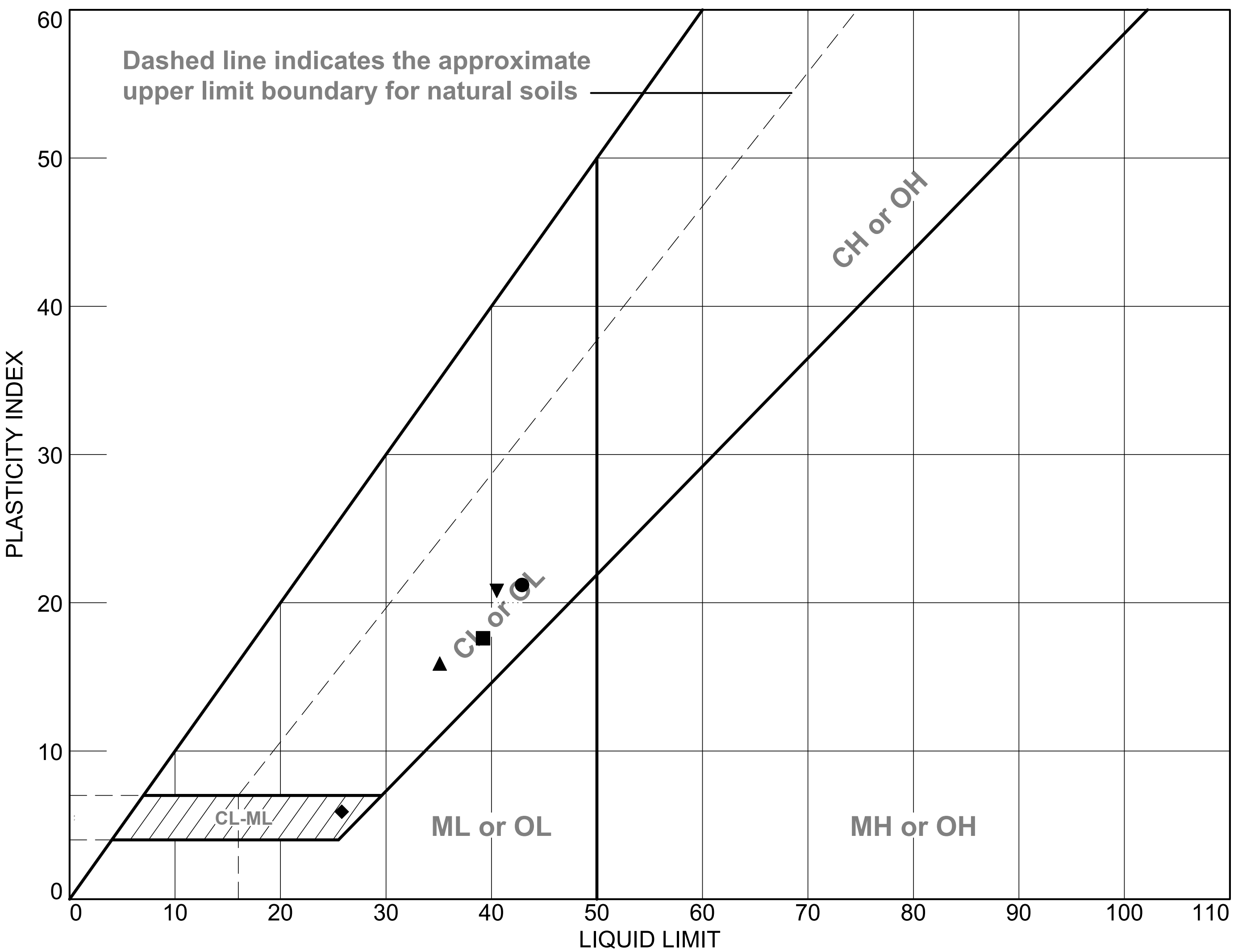
Client:

Project: SAN DIEGO REGIONAL RAIL CORRIDOR

Project No.: 6320-A01 **Figure**

Tested By: JH **Checked By:** SC

LIQUID AND PLASTIC LIMITS TEST REPORT

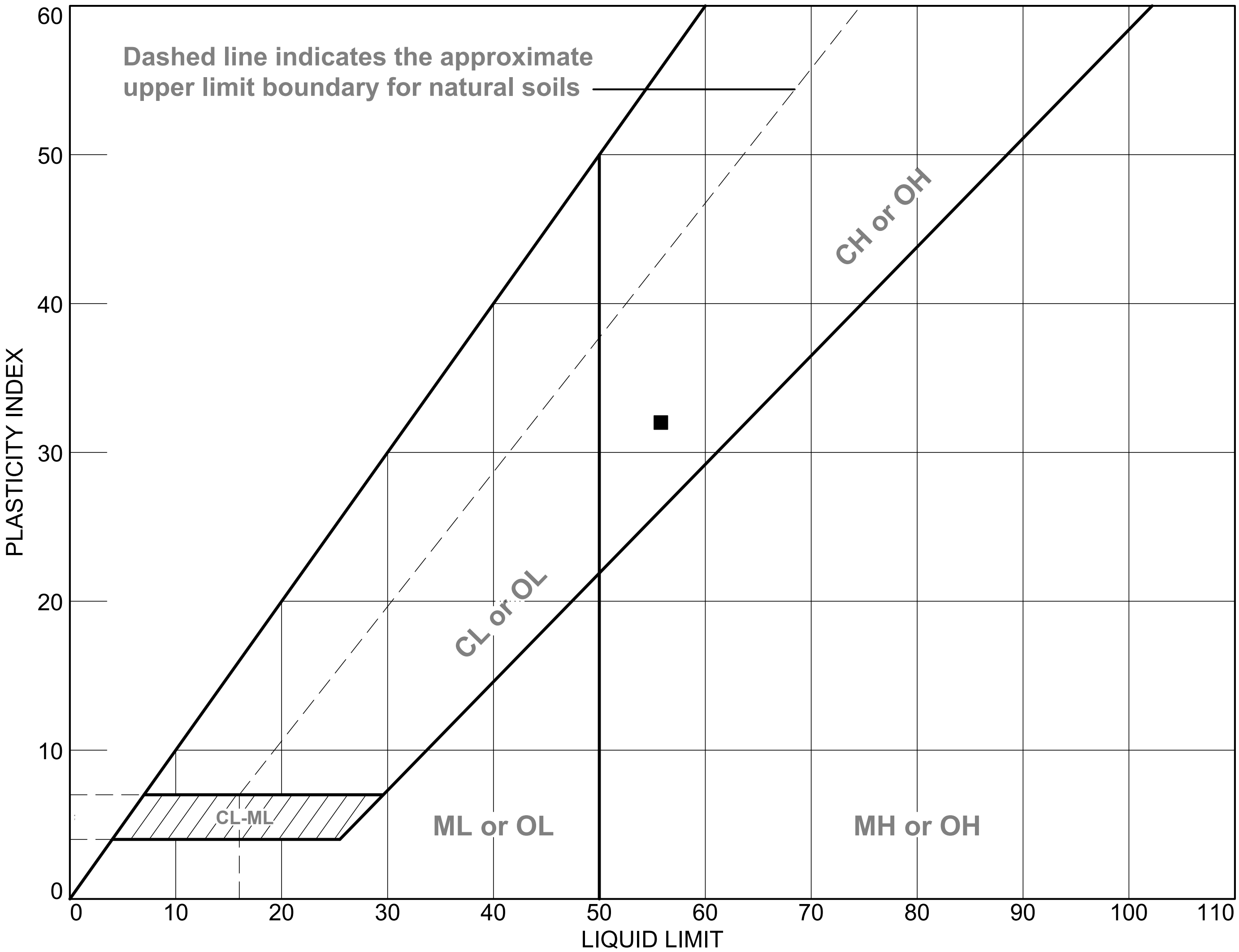


SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		B2	175-180		21.7	42.9	21.2	SC
■		B2	185-190		21.6	39.2	17.6	SC
▲		B2	191		19.2	35.1	15.9	SC
◆		B2	195		19.9	25.8	5.9	SC-SM
▼		B3	20-25		19.7	40.5	20.8	SP-SC

MTGL, Inc. **San Diego, CA**	**Client:** **Project:** SAN DIEGO REGIONAL RAIL CORRIDOR **Project No.:** 6320-A01 **Figure**

Tested By: JH **Checked By:** SC

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		B3	60-65		NP	NV	NP	SP
■		B4	100		23.8	55.8	32.0	CH

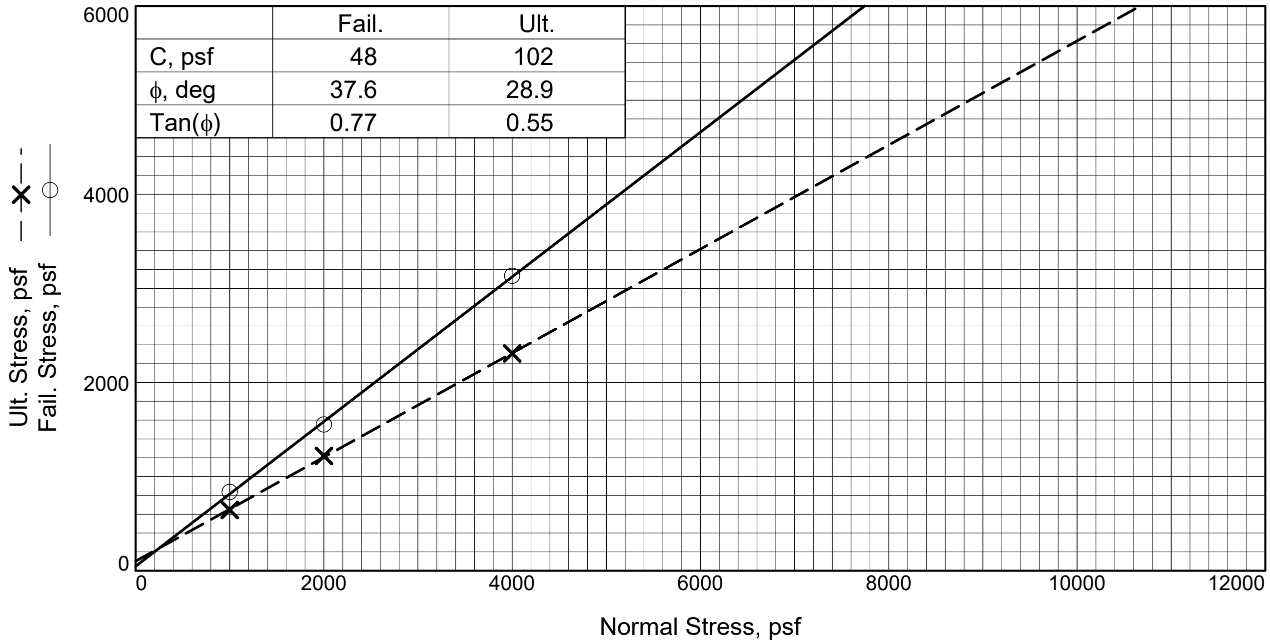
MTGL, Inc.

San Diego, CA

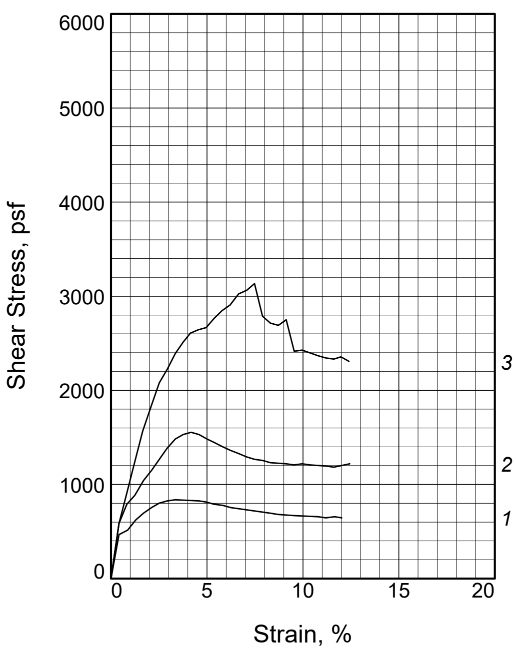
Client:
Project: SAN DIEGO REGIONAL RAIL CORRIDOR

Project No.: 6320-A01 **Figure**

Tested By: JH **Checked By:** SC



	Fail.	Ult.
C, psf	48	102
ϕ, deg	37.6	28.9
Tan(ϕ)	0.77	0.55



Sample No.		1	2	3
Initial	Water Content, %	6.7	6.5	6.4
	Dry Density, pcf	101.9	103.3	102.7
	Saturation, %	28.7	28.6	27.6
	Void Ratio	0.6239	0.6018	0.6108
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	22.4	21.0	19.6
	Dry Density, pcf	103.9	106.3	108.9
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.5924	0.5566	0.5195
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.98	0.97	0.94
Normal Stress, psf		1000	2000	4000
Fail. Stress, psf		837	1555	3134
Strain, %		3.3	4.2	7.5
Ult. Stress, psf		646	1220	2308
Strain, %		12.0	12.4	12.4
Strain rate, in./min.		0.010	0.010	0.010

Sample Type:
Description:

Specific Gravity= 2.65
Remarks: LAB #116

Figure ________

Client:

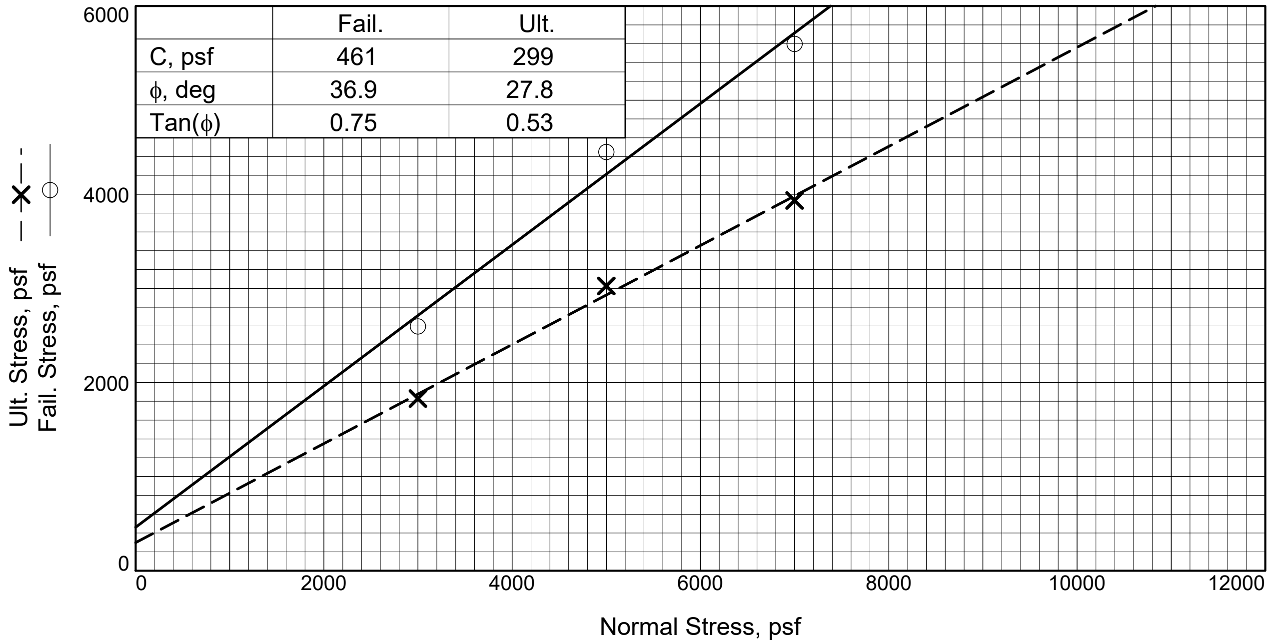
Project: SAN DIEGO REGIONAL RAIL CORRIDOR

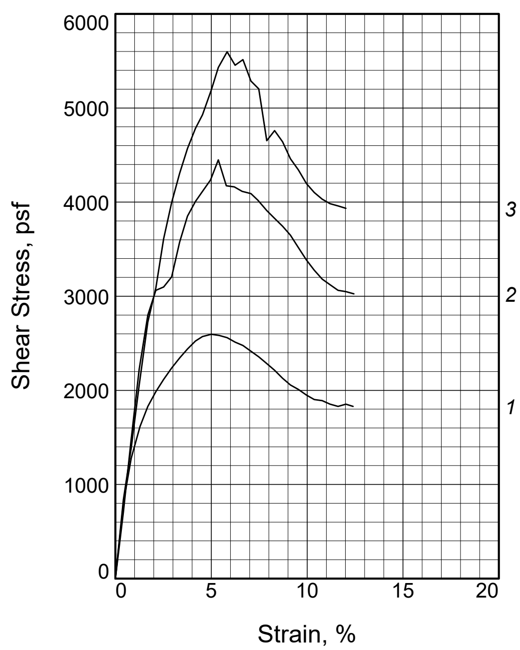
Sample Number: B1 **Depth:** 20

Proj. No.: 6320-A01 **Date Sampled:** 3/2/21

DIRECT SHEAR TEST REPORT
MTGL, Inc.
San Diego, CA

Tested By: JH **Checked By:** SC





Sample No.		1	2	3
Initial	Water Content, %	16.4	16.1	16.2
	Dry Density, pcf	109.1	109.7	109.4
	Saturation, %	84.1	83.8	83.8
	Void Ratio	0.5166	0.5086	0.5120
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.8	17.0	16.4
	Dry Density, pcf	112.3	114.1	115.3
	Saturation, %	100.0	100.0	100.0
	Void Ratio	0.4727	0.4505	0.4352
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.97	0.96	0.95
Normal Stress, psf		3000	5000	7000
Fail. Stress, psf		2595	4449	5597
Strain, %		5.0	5.4	5.8
Ult. Stress, psf		1830	3026	3935
Strain, %		12.4	12.4	12.0
Strain rate, in./min.		0.010	0.010	0.010

Sample Type:
Description:

Specific Gravity= 2.65
Remarks: LAB #116

Figure ________

Client:

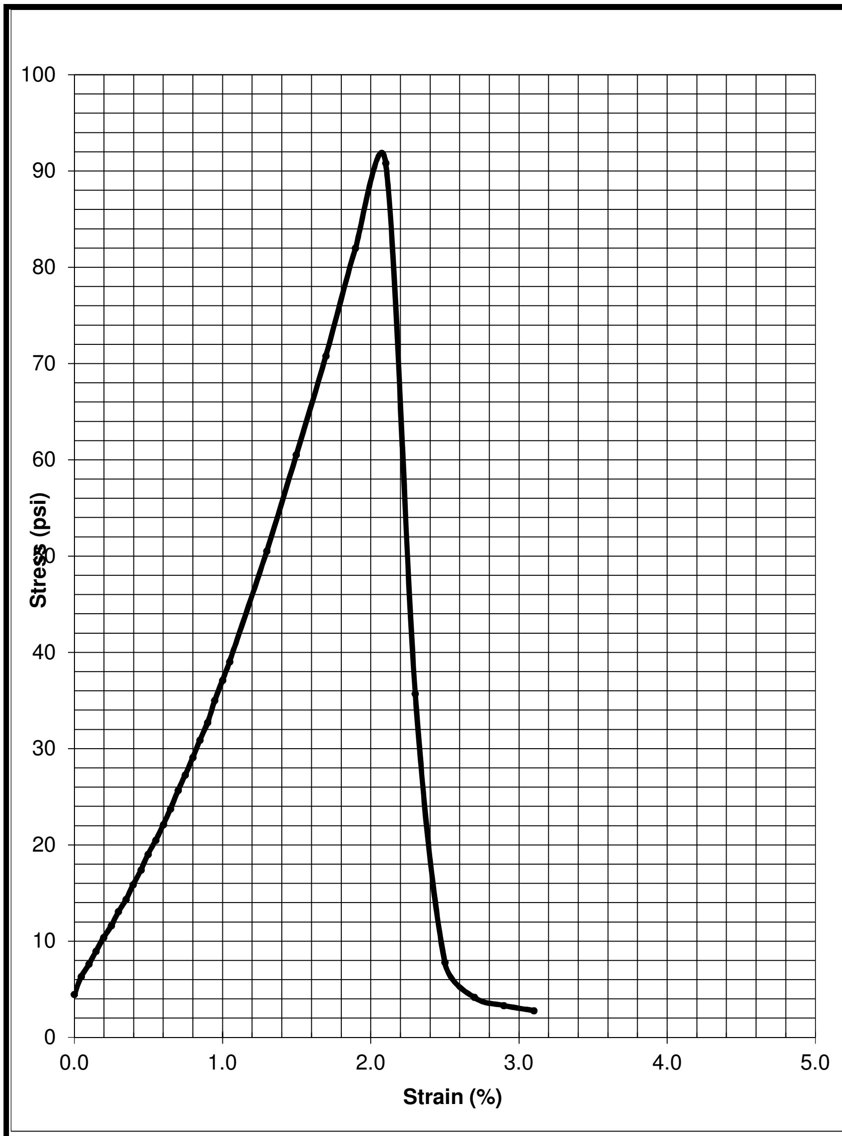
Project: SAN DIEGO REGIONAL RAIL CORRIDOR

Sample Number: B2 **Depth:** 30

Proj. No.: 6320-A01 **Date Sampled:** 3/8/21

DIRECT SHEAR TEST REPORT
MTGL, Inc.
San Diego, CA

Tested By: JH **Checked By:** SC



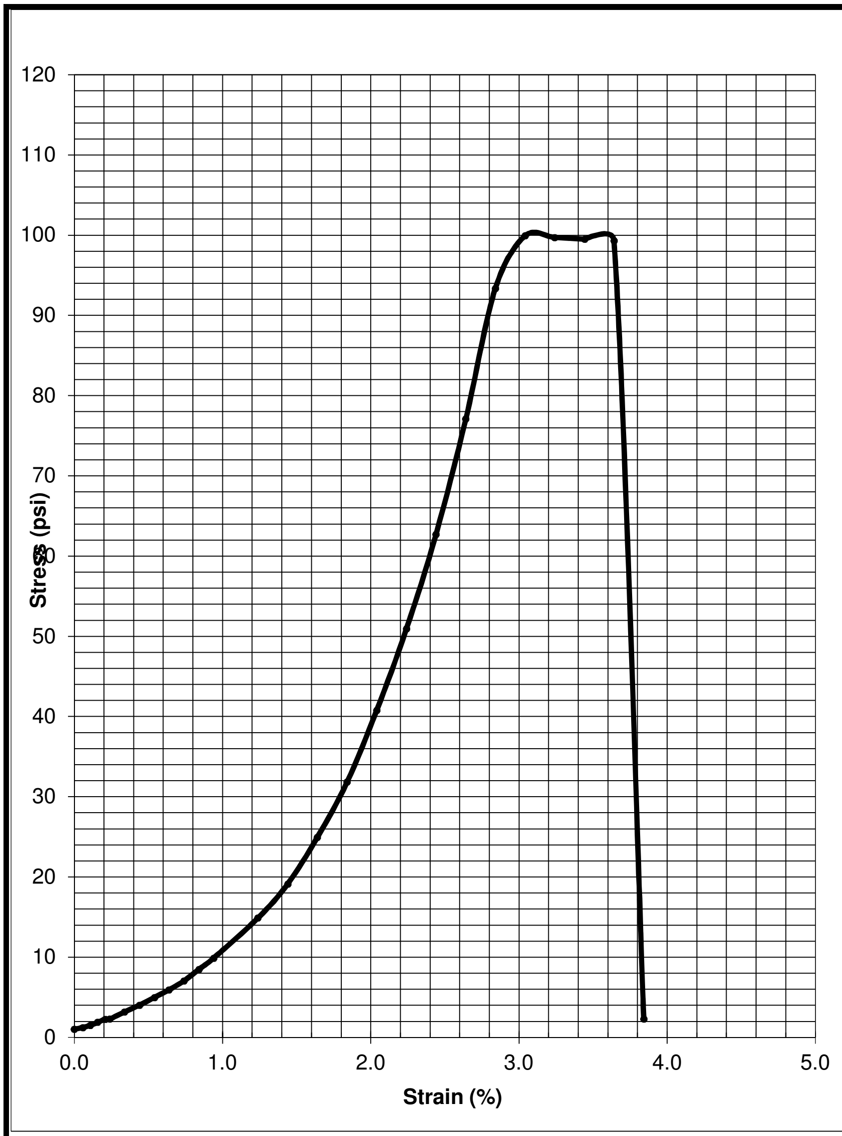
Soil Description: Brown, Siltsty Sandstone

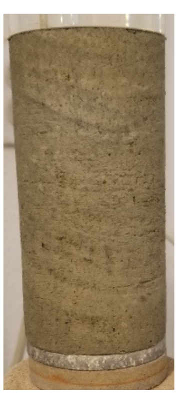
Type of Specimen: *Undisturbed*

	SPECIMEN		A	B	C
INITIAL	Wet Density (pcf)		***137.9***		
	Water Content (%)		***14.5***		
	Dry Density (pcf)		***120.5***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATIO N STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.010***		
FAILURE CONDITION	**Strain %**		***2.1***		
	$(\sigma_1 - \sigma_3)_f$	**psi**			
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**			
	σ_{1f}	**psi**	***90.8***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	5.0	in.

UNCONFINED COMPRESSION TEST - ASTM D-2166	Job Name MTGL # 6320A01	Date: 4-14-2021
	Job No. 2012-0123	Sample : B2 @ 160'

Geo**L**ogic **A**ssociates



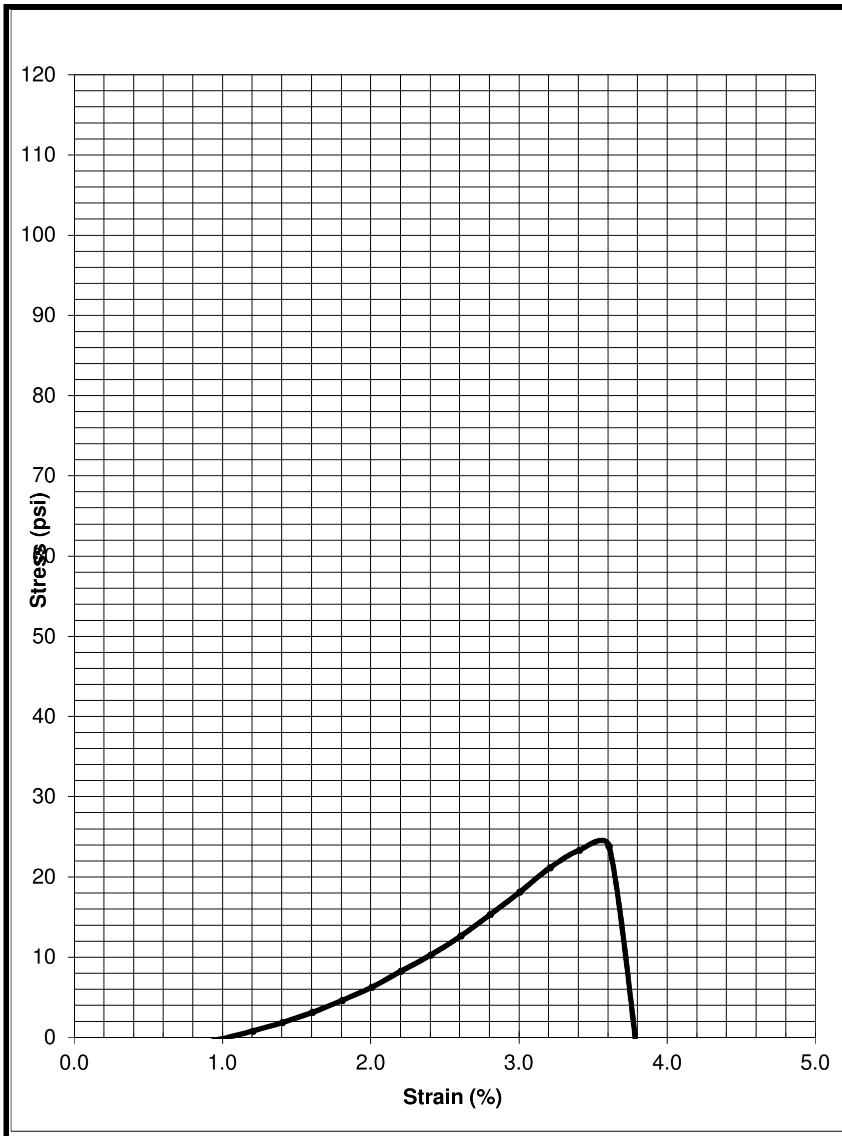


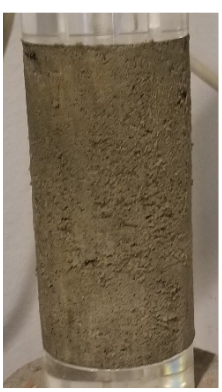
			A	B	C
Soil Description: Brown, Siltsty Sandstone					
Type of Specimen: *Undisturbed*					
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***135.2***		
	Water Content (%)		***17.0***		
	Dry Density (pcf)		***115.5***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATION STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.010***		
FAILURE CONDITION	**Strain %**		***3.0***		
	$(\sigma_1 - \sigma_3)_f$	**psi**			
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**			
	σ_{1f}	**psi**	***99.9***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	5.0	in.

UNCONFINED COMPRESSION TEST - ASTM D-2166

Job Name MTGL # 6320A01	Date: 4-14-2021
Job No. 2012-0123	Sample : B2 @ 163'

Geo**L**ogic **A**ssociates



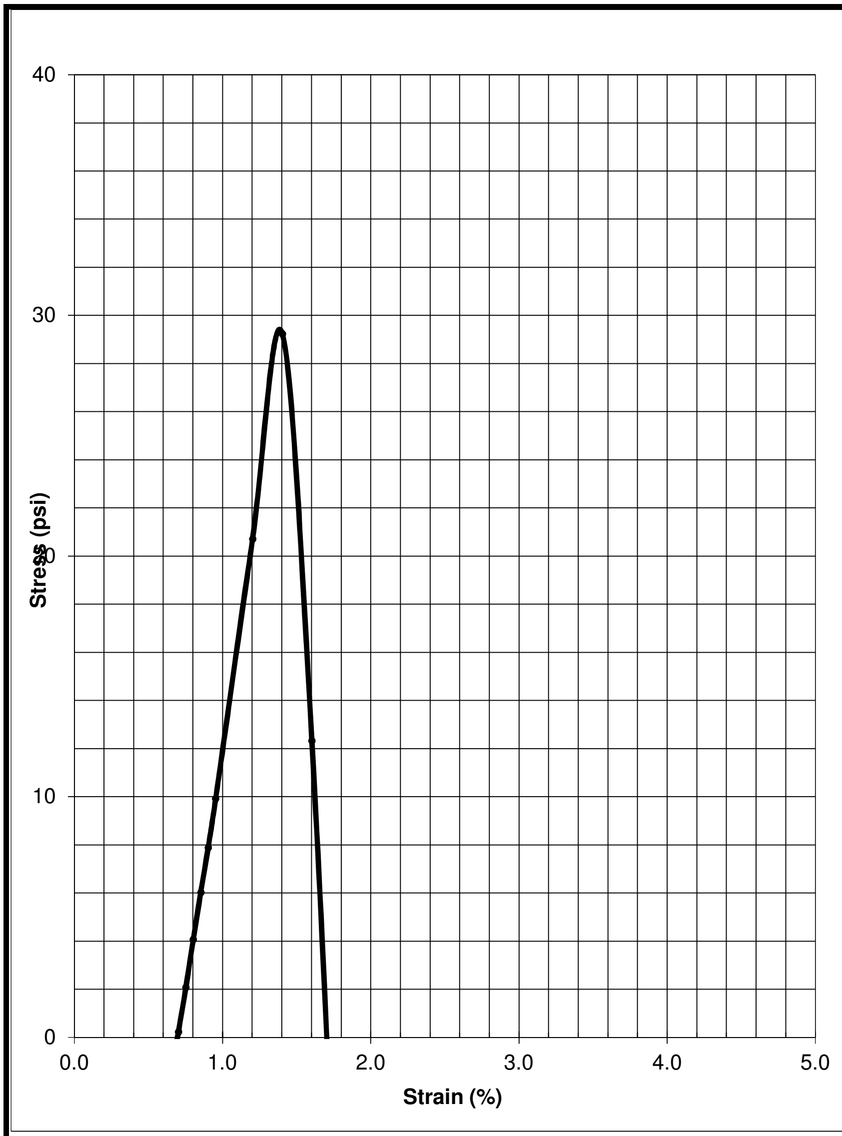


Soil Description: Brown, Siltsty Sandstone					
Type of Specimen: *Undisturbed*					
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***133.4***		
	Water Content (%)		***18.9***		
	Dry Density (pcf)		***112.2***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATIO N STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.010***		
FAILURE CONDITION	**Strain %**		***3.6***		
	$(\sigma_1 - \sigma_3)_f$	**psi**			
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**			
	σ_{1f}	**psi**	***23.8***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	5.0	in.

UNCONFINED COMPRESSION TEST - ASTM D-2166

Job Name MTGL # 6320A01	Date: 4-14-2021
Job No. 2012-0123	Sample : B2 @ 167'

Geo**L**ogic **A**ssociates

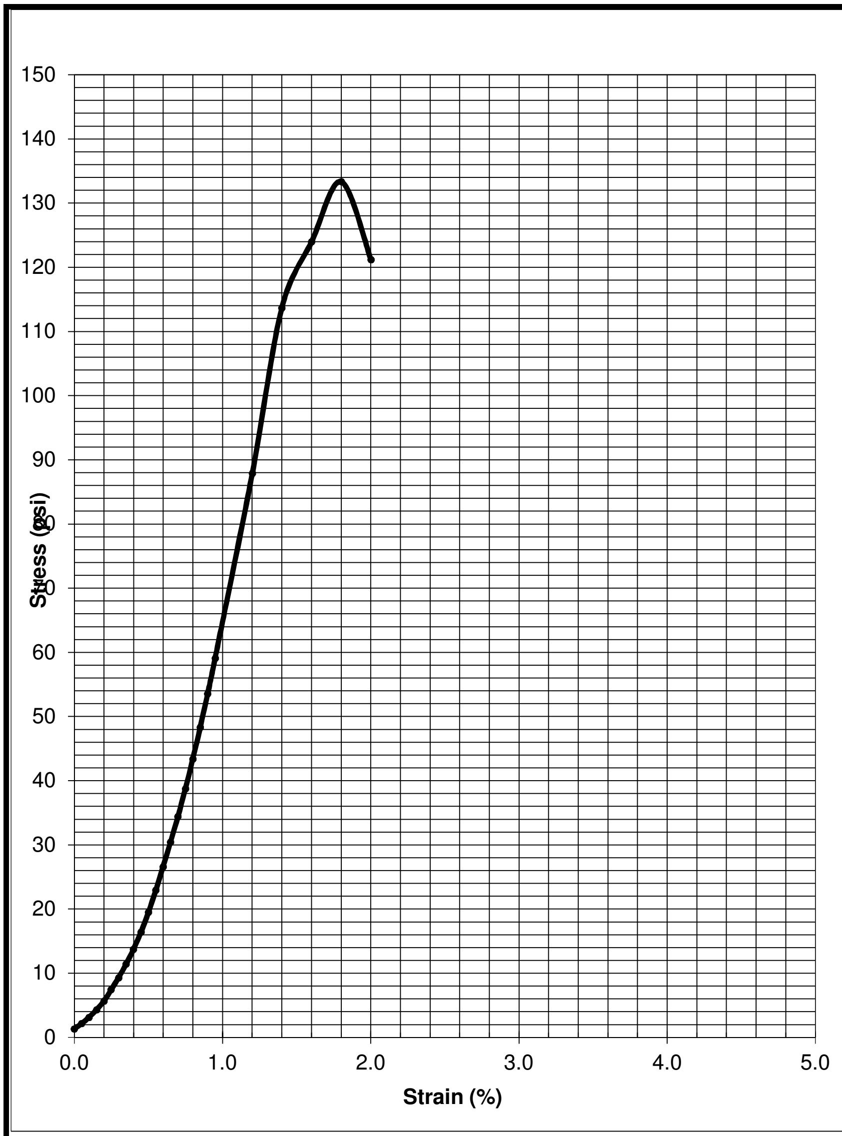


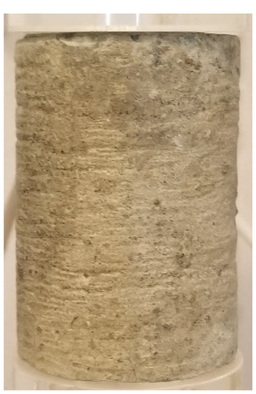
				A	B	C
Soil Description: Brown, Sandy Siltstone						
Type of Specimen: *Undisturbed*						
SPECIMEN				A	B	C
INITIAL	Wet Density (pcf)			***142.1***		
	Water Content (%)			***12.6***		
	Dry Density (pcf)			***126.2***		
FINAL	Wet Density (pcf)					
	Water Content (%)					
	Dry Density (pcf)					
SATURATION STAGE	Initial pwp	psi				
	Saturated pwp	psi				
	Final Cell pressure	psi				
	B value					
CONSOLIDATION STAGE	Cell Pressure	psi				
	Back Pressure	psi				
	Initial pwp	psi				
	Final pwp	psi				
COMPRESSION STAGE	Cell Pressure	psi				
	Initial pwp	psi				
	Initial σ'_3	psi				
	Strain Rate (in./min.)			***0.010***		
FAILURE CONDITION	**Strain %**			***1.4***		
	$(\sigma_1 - \sigma_3)_f$	**psi**				
	$(\sigma_1 / \sigma_3)_f$	**psi**				
	σ_3	**psi**				
	σ_{1f}	**psi**		***29.2***		
	c_v					
	m_v					
	k					
SAMPLE SIZE				D =	2.39	in.
				H =	5.0	in.

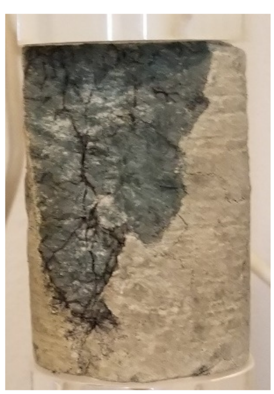
UNCONFINED COMPRESSION TEST - ASTM D-2166

Job Name MTGL # 6320A01	Date: 4-18-2021
Job No. 2012-0123	Sample : B2 @ 153'

Geo**L**ogic **A**ssociates





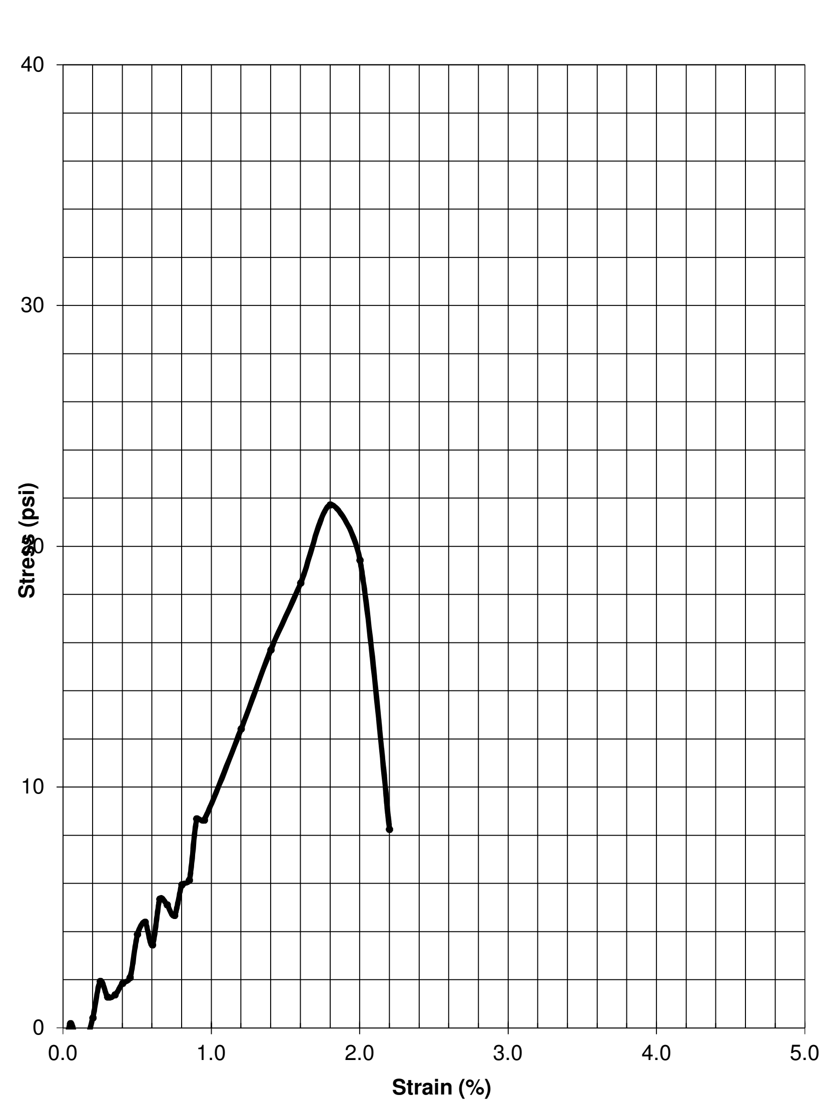


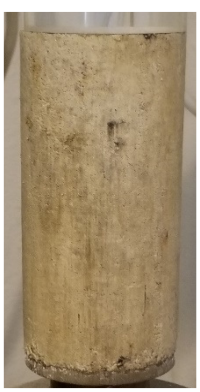
Soil Description: Brown, Sandy Siltstone					
Type of Specimen: *Undisturbed*					
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***136.3***		
	Water Content (%)		***14.4***		
	Dry Density (pcf)		***119.1***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATIO N STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.010***		
FAILURE CONDITION	**Strain %**		***1.8***		
	$(\sigma_1 - \sigma_3)_f$	**psi**			
	$(\sigma_1/\sigma_3)_f$	**psi**			
	σ_3	**psi**			
	σ_{1f}	**psi**	***133.4***		
	L/D		*1.5*		
	Correction Factor		*0.960*		
	σ_{1f} (corrected)		***128.1***		
SAMPLE SIZE			D =	2.32	in.
			H =	3.5	in.

UNCONFINED COMPRESSION TEST - ASTM D-2166

Job Name MTGL # 6320A01	Date: 4-18-2021
Job No. 2012-0123	Sample : B2 @ 170'

Geo**L**ogic **A**ssociates





Soil Description: Brown, Silty Sand

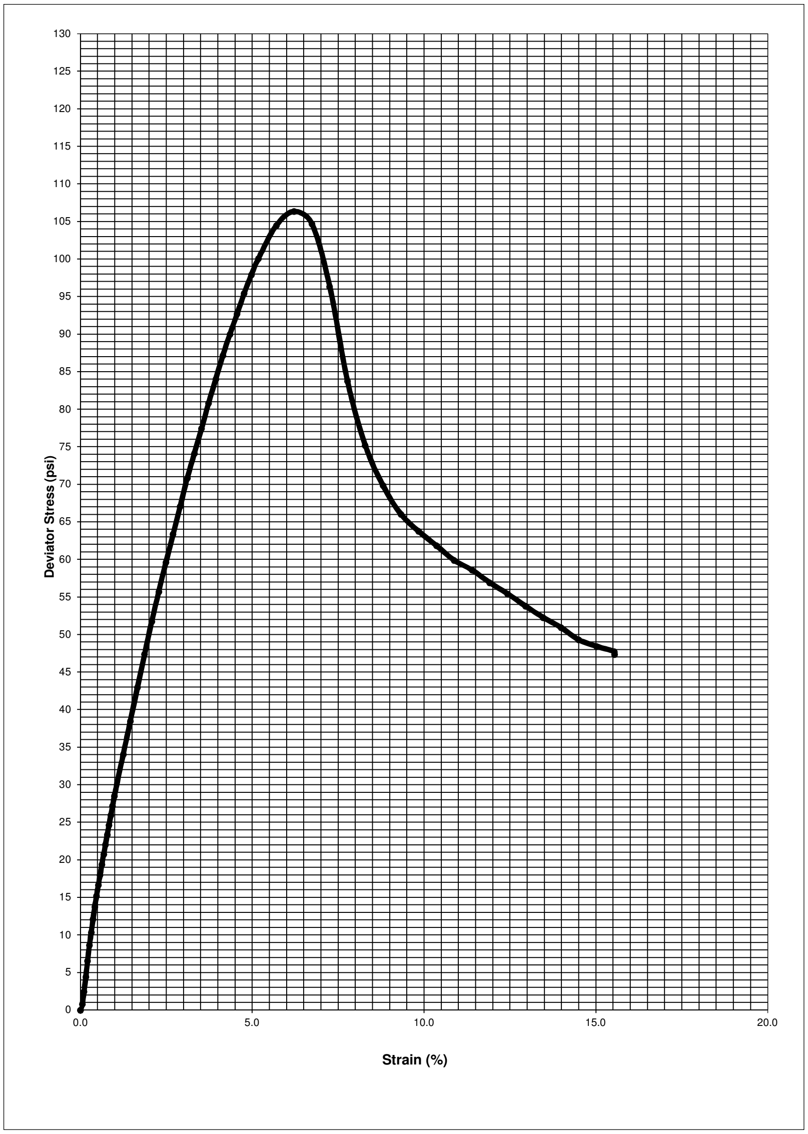
Type of Specimen: *Undisturbed*

SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***129.6***		
	Water Content (%)		***14.8***		
	Dry Density (pcf)		***112.9***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATION STAGE	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.010***		
FAILURE CONDITION	**Strain %**		***1.8***		
	$(\sigma_1 - \sigma_3)_f$	**psi**			
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**			
	σ_{1f}	**psi**	***21.7***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	5.0	in.

UNCONFINED COMPRESSION TEST - ASTM D-2166

Job Name MTGL # 6320A01	Date: 4-18-2021
Job No. 2012-0123	Sample : B3 @ 85'-87'

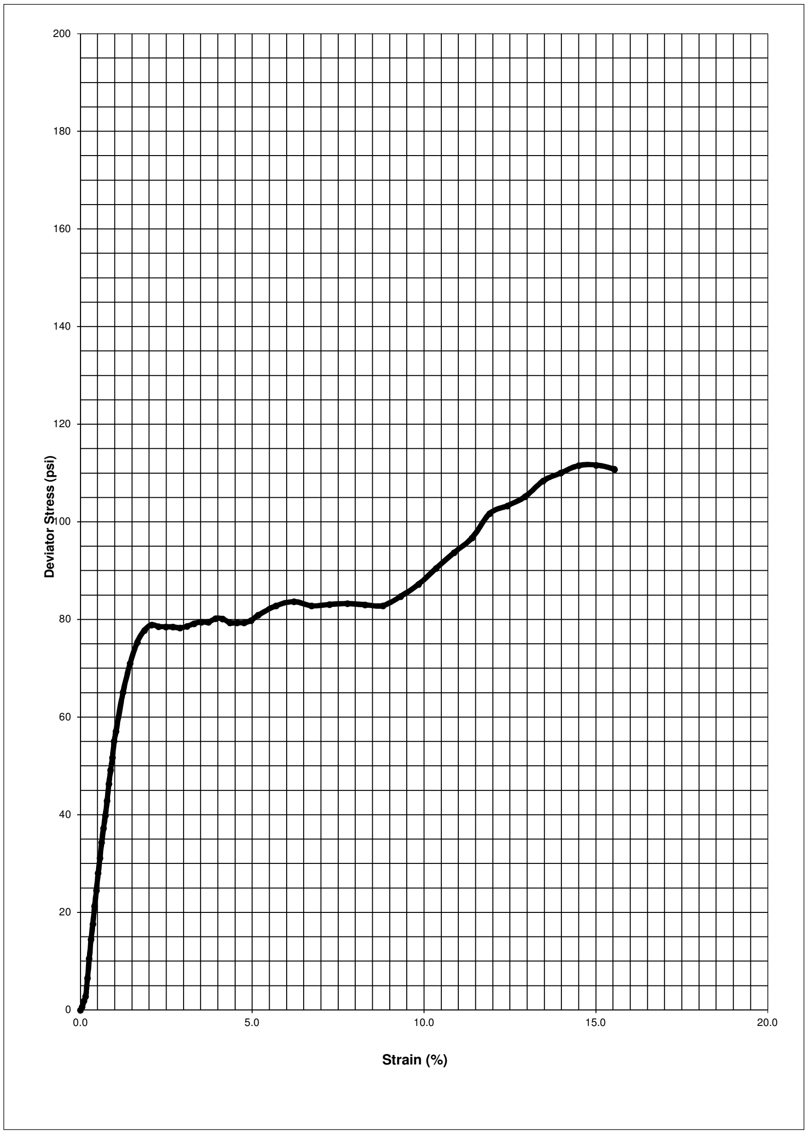
Geo**L**ogic **A**ssociates



Soil Description: Brown, Siltstone					
Type of Specimen: *Undisturbed*					
	SPECIMEN		A	B	C
INITIAL	Wet Density (pcf)		***132.3***		
	Water Content (%)		***19.9***		
	Dry Density (pcf)		***110.4***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATION	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***6.2***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***106.3***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***28.0***		
	σ_{1f}	**psi**	***134.3***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.39	in.
			H =	5.0	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name MTGL # 6320A01	Date: 4-10-2021
ASTM D-2850	Job No.	Sample : B1/D8 @ 40'

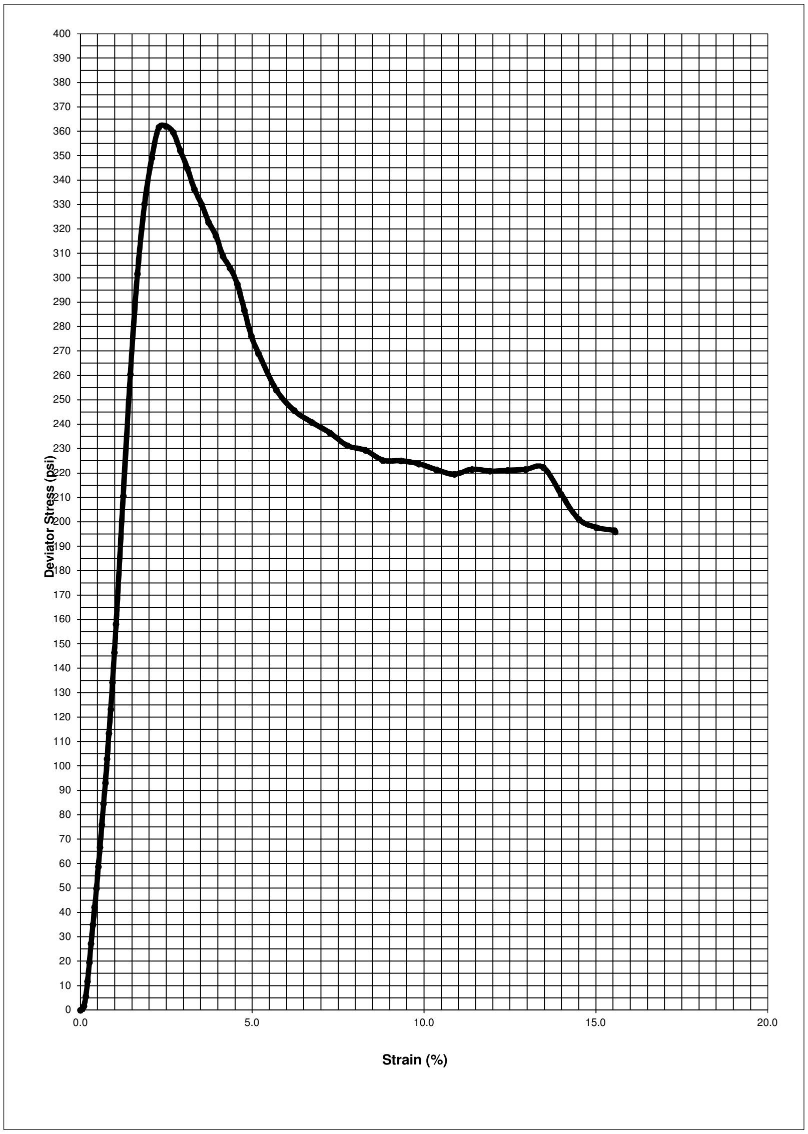
Geo**L**ogic **A**ssociates



				A	B	C
Soil Description: Brown, Siltstone						
Type of Specimen: *Undisturbed*						
SPECIMEN				A	B	C
INITIAL	Wet Density (pcf)			***134.9***		
	Water Content (%)			***16.5***		
	Dry Density (pcf)			***115.8***		
FINAL	Wet Density (pcf)					
	Water Content (%)					
	Dry Density (pcf)					
SATURATION STAGE	Initial pwp	psi				
	Saturated pwp	psi				
	Final Cell pressure	psi				
	B value					
CONSOLIDATION	Cell Pressure	psi				
	Back Pressure	psi				
	Initial pwp	psi				
	Final pwp	psi				
COMPRESSION STAGE	Cell Pressure	psi				
	Initial pwp	psi				
	Initial σ'_3	psi				
	Strain Rate (in./min.)			***0.005***		
FAILURE CONDITION	**Strain %**			***15.0***		
	$(\sigma_1 - \sigma_3)_f$	**psi**		***111.6***		
	$(\sigma_1 / \sigma_3)_f$	**psi**				
	σ_3	**psi**		***63.0***		
	σ_{1f}	**psi**		***174.6***		
	c_v					
	m_v					
	k					
SAMPLE SIZE				D =	2.42	in.
				H =	4.9	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name MTGL # 6320A01	Date: 4-10-2021
ASTM D-2850	Job No.	Sample : B2 @ 150'-153'

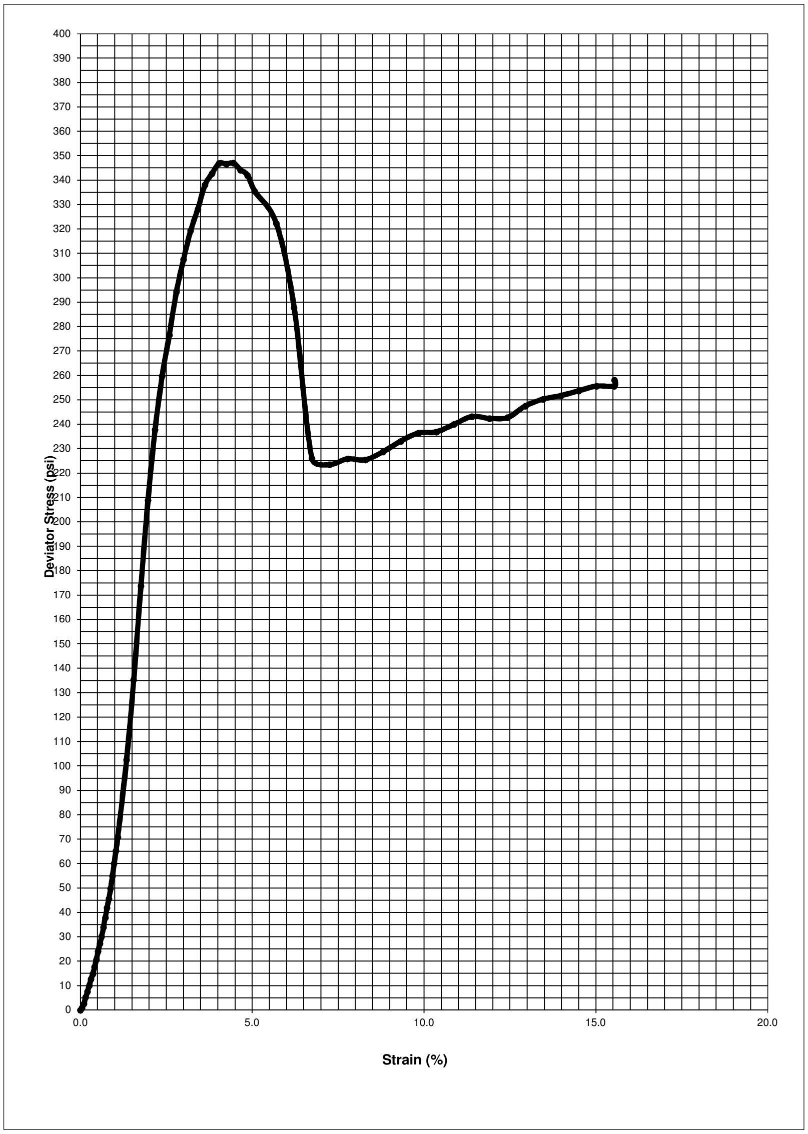
GeoLogic Associates



Soil Description: Brown, Sandstone					
Type of Specimen: *Undisturbed*					
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***142.9***		
	Water Content (%)		***8.0***		
	Dry Density (pcf)		***132.3***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATI ON	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***2.5***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***362.2***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***63.0***		
	σ_{1f}	**psi**	***425.2***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	4.9	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name MTGL # 6320A01	Date: 4-10-2021
ASTM D-2850	Job No.	Sample : B2 @ 157'-161'

Geo**L**ogic **A**ssociates



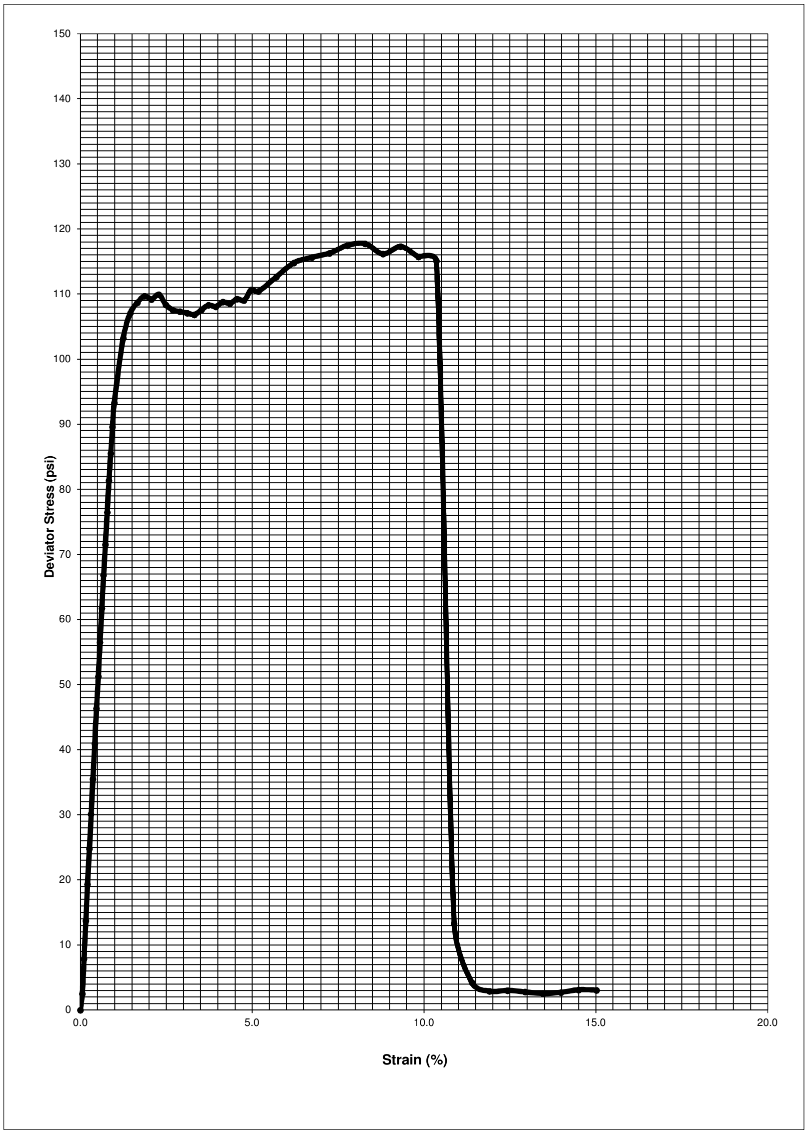
Soil Description: Brown, Sandstone

Type of Specimen: *Undisturbed*

SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***134.8***		
	Water Content (%)		***18.8***		
	Dry Density (pcf)		***113.5***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATION	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***4.0***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***346.9***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***63.0***		
	σ_{1f}	**psi**	***409.9***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	5.0	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name MTGL # 6320A01	Date: 4-10-2021
ASTM D-2850	Job No.	Sample : B2 @ 160' - 165'

GeoLogic Associates



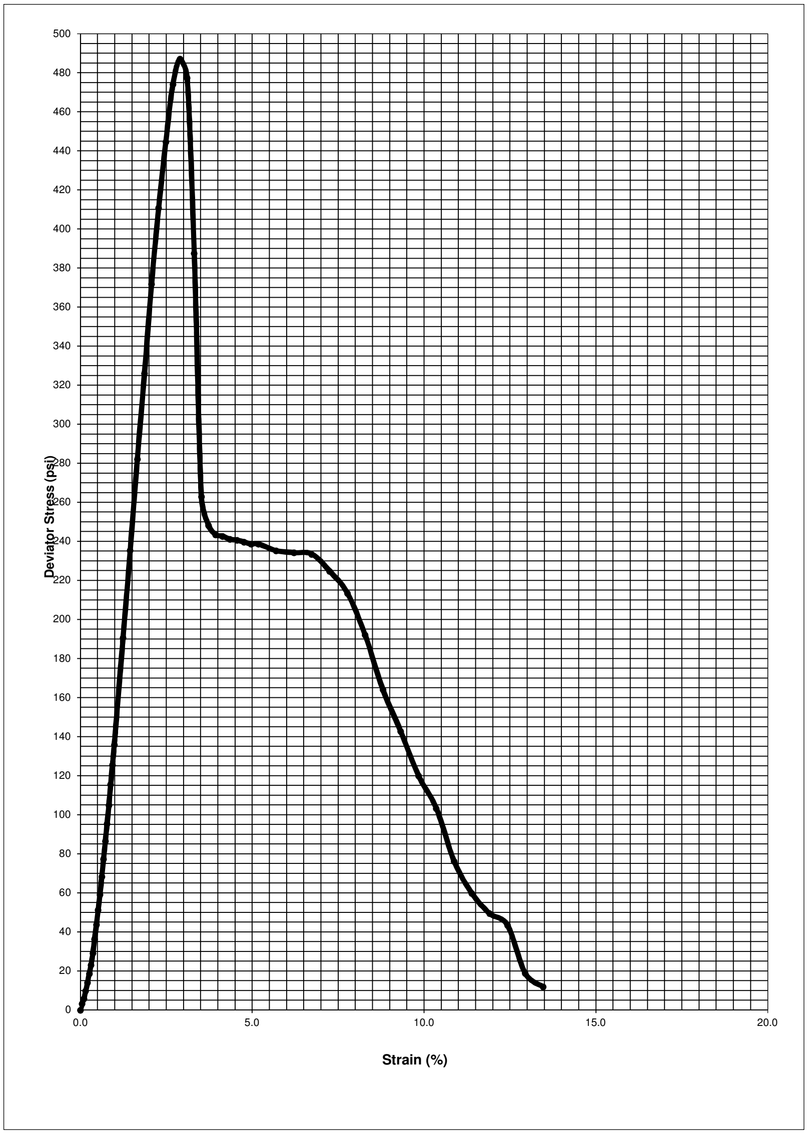
Soil Description: Brown, Sandy Siltstone

Type of Specimen: *Undisturbed*

SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***135.6***		
	Water Content (%)		***14.0***		
	Dry Density (pcf)		***119.0***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATION	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***8.3***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***117.7***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***69.0***		
	σ_{1f}	**psi**	***186.7***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	4.9	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name MTGL # 6320A01	Date: 4-10-2021
ASTM D-2850	Job No.	Sample : B2 @ 170' - 175'

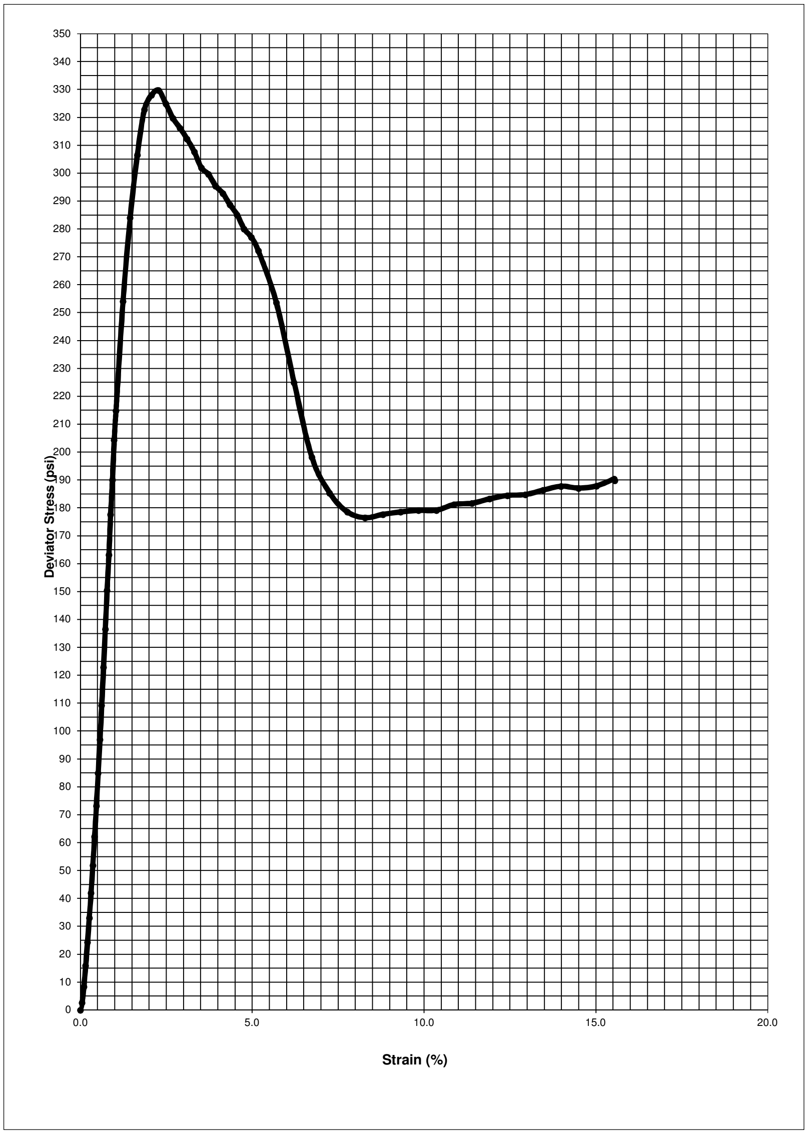
Geo**L**ogic **A**ssociates



Soil Description: Brown, Silt Clay					
Type of Specimen: *Undisturbed*					
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***140.3***		
	Water Content (%)		***10.7***		
	Dry Density (pcf)		***126.7***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp				
	Final Cell pressure				
	B value				
CONSOLIDATION	Cell Pressure	psi			
	Back Pressure				
	Initial pwp				
	Final pwp				
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp				
	Initial σ'_3				
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***2.7***		
	$(\sigma_1 - \sigma_3)_f$	psi	***474.2***		
	$(\sigma_1 / \sigma_3)_f$				
	σ_3		***76.0***		
	σ_{1f}		***550.2***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.34	in.
			H =	4.8	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name MTGL # 6320A01	Date: 4-10-2021
ASTM D-2850	Job No.	Sample : B2 @ 175' - 180'

Geo**L**ogic **A**ssociates



Soil Description: Brown, Sandstone					
Type of Specimen: *Undisturbed*					
SPECIMEN			A	B	C
INITIAL	Wet Density (pcf)		***127.0***		
	Water Content (%)		***12.3***		
	Dry Density (pcf)		***113.1***		
FINAL	Wet Density (pcf)				
	Water Content (%)				
	Dry Density (pcf)				
SATURATION STAGE	Initial pwp	psi			
	Saturated pwp	psi			
	Final Cell pressure	psi			
	B value				
CONSOLIDATI ON	Cell Pressure	psi			
	Back Pressure	psi			
	Initial pwp	psi			
	Final pwp	psi			
COMPRESSION STAGE	Cell Pressure	psi			
	Initial pwp	psi			
	Initial σ'_3	psi			
	Strain Rate (in./min.)		***0.005***		
FAILURE CONDITION	**Strain %**		***2.3***		
	$(\sigma_1 - \sigma_3)_f$	**psi**	***329.7***		
	$(\sigma_1 / \sigma_3)_f$	**psi**			
	σ_3	**psi**	***56.0***		
	σ_{1f}	**psi**	***385.7***		
	c_v				
	m_v				
	k				
SAMPLE SIZE			D =	2.41	in.
			H =	4.6	in.

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST	Job Name MTGL # 6320A01	Date: 4-10-2021
ASTM D-2850	Job No.	Sample : B3 @ 97.5-99.5'

GeoLogic Associates

APPENDIX D

P&S Wave Suspension Logging Report

GEOVision
geophysical services

PS SUSPENSION VELOCITIES

DEL MAR, CALIFORNIA

Prepared for

Earth Mechanics, Inc.
17800 Newhope Street, Suite B
Fountain Valley, California 92708
(714) 751 - 3826

Prepared by

GEO*Vision* Geophysical Services
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

April 9, 2021
Report 21048-01 rev 1

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APPENDIX B	**GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS**

INTRODUCTION

Borehole geophysical measurements were collected in two boreholes at a site in Del Mar, California. Data acquisition was performed on March 2nd and 15th, 2021. Data analysis and report were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer.

SCOPE OF WORK

This report presents the results of borehole geophysical measurements collected in two boreholes as detailed in Table 1.

The OYO Suspension PS Logging System (Suspension System) was used to obtain in-situ horizontal shear (S_H) and compressional (P) wave velocity measurements in one uncased borehole at 1.6 foot intervals. Measurements followed **GEO***Vision* Procedure for PS Suspension Seismic Velocity Logging, revision 1.5. Acquired data were analyzed and a profile of velocity versus depth was produced for both S_H and P waves.

A detailed reference for the suspension PS velocity measurement techniques used in this study is:

> Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

INSTRUMENTATION

Suspension Velocity Instrumentation

Suspension velocity measurements were performed using the suspension PS logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geo (RG). This system directly determines the average velocity of a 3.3-foot high segment of the soil column surrounding the boring of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the boring producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source (S_H) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in these surveys is approximately 22 feet, with the center point of the receiver pair 12.5 feet above the bottom end of the probe.

The probe receives control signals from, and sends the digitized receiver signals to, instrumentation on the surface via an armored conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data using a sheave of known circumference fitted with a digital rotary encoder.

The entire probe is suspended in the boring by the cable, therefore, source motion is not coupled directly to the boring walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the boring and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it passes through the casing and grout annulus and impinges upon the wall of the boring. These waves propagate through the soil and rock surrounding the boring, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. Orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 6.3 foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver.
4. In saturated soils, the received P-wave signal is typically of much higher frequency than the received S_H-wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in fluid is significantly greater than the dimension of the fluid annulus surrounding the probe (feet versus inches scale), preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.
3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The Suspension PS system has six channels (two simultaneous recording channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale. Data are stored on disk for further processing.

Review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), and sample rate to optimize the quality of the data before recording. Verification of the calibration of the Suspension PS digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

MEASUREMENT PROCEDURES

Suspension Velocity

Two boreholes were logged with the PS Suspension tool. Measurements followed the **GEO***Vision* Procedure for PS Suspension Seismic Velocity Logging, revision 1.5. Prior to logging, the probe was positioned with the top of the probe even with a stationary reference point. The electronic depth counter was set to the distance between the mid-point of the receiver and the top of the probe, minus the height of the stationary reference point, if any, verified with a tape measure, and recorded on the field logs. The probe was lowered to the bottom of the boring, stopping at 1.6 foot intervals to collect data, as summarized in Table 2.

At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed, and the gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and recorded to disk before moving to the next depth.

Upon completion of the measurements, the probe zero depth indication at the depth reference point was verified prior to removal from the boring.

DATA ANALYSIS

Suspension Velocity

Using the proprietary OYO program PSLOG.EXE version 1.0, the recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 1.0 meter segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into a Microsoft Excel® template to complete the velocity calculations based on the arrival time picks made in PSLOG. The Microsoft Excel® analysis files were previously delivered. Due to the longevity of this project, results were delivered at intervals as requested.

The P-wave velocity over the 6.3-foot interval from source to receiver 1 (S-R1) was also picked using PSLOG, and calculated and plotted in Microsoft Excel®, for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 4.8 feet to correspond to the mid-point of the 6.3-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting 0.35 milliseconds, the calculated and experimentally verified delay from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

As with the P-wave records, the recorded digital waveforms were analyzed to locate clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital Fast Fourier Transform – Inverse Fast Fourier Transform (FFT – IFFT) lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 600 Hz in the slowest zones to 4000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source or by boring inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuations.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 6.3-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 4.8 feet to correspond to the mid-point of the 6.3-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting 0.35 milliseconds, the calculated and experimentally verified delay from the beginning of the record at the source trigger pulse to source impact.

Poisson’s Ratio, **ν**, was calculated using the following formula:

$$\boldsymbol{\nu} = \frac{\left(\frac{V_s}{V_p}\right)^2 - 0.5}{\left(\frac{V_s}{V_p}\right)^2 - 1.0}$$

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 2, the time difference over the 3.3 foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering of the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by residual P-wave signal.

Data and analyses were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer as a component of the in-house data validation program.

RESULTS

Suspension Velocity

Suspension R1-R2 P- and S_H-wave velocities for boreholes R-20-004 and R-21-001 are presented in Figures 4 and 5, respectively. The suspension velocity data presented in this figure are also presented in Tables 3 and 4, respectively. The Microsoft Excel® analysis files are delivered separately.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figure A-1 and A-2 for boreholes R-20-004 and R-21-001, to aid in visual comparison. It should be noted that R1-R2 data are an average velocity over a 3.3-foot segment of the soil column; S-R1 data are an average over 6.3 feet, creating a significant smoothing relative to the R1-R2 plots. The S-R1 velocity data displayed in this figure is also presented in Table A-1 and A-2 respectively, and included in the Microsoft Excel® analysis files delivered separately. The Microsoft Excel® analysis files include Poisson's Ratio calculations, tabulated data, and plots.

SUMMARY

Discussion of Suspension Velocity Results

Suspension PS velocity data are ideally collected in uncased fluid-filled boreholes, drilled with rotary mud (rotary wash) methods, as was the case for these boreholes.

Suspension PS velocity data quality is judged based upon 5 criteria.

	Criteria	R-20-004	R-21-001
1	Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.	Yes	Yes
2	Consistency between data from adjacent depth intervals.	Yes	Yes
3	Consistent relationship between P-wave and S_H -wave (excluding transition to saturated soils)	Yes, saturation occurs around 80 ft. There may be a slight perch zone around 86-88 ft	Yes, saturation occurs around 40 ft. There may be a slight perch zone around 60ft
4	Clarity of P-wave and S_H-wave onset, as well as damping of later oscillations.	Clear good data set	
5	Consistency of profile between adjacent borings, if available.	Yes, velocity profiles appear consistent at depth.	

These data indicate good consistency between R1-R2 and S-R1 velocities, and consistency between adjacent depths. All arrival picks are unambiguous, and the relationship between P-wave and S_H-wave are reasonable.

Suspension Velocity Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.3-foot interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable with estimated precision of +/- 5%. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analyses. All work was performed under **GEO***Vision* quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO***Vision* California Professional Geophysicist or Engineer.

Prepared by:

04/09/2021

Andrew T McNab — Date

GEO*Vision* Geophysical Services

Reviewed and approved by

04/09/2021

Victor M Gonzalez — Date

California Professional Geophysicist PGp 1074

GEO*Vision* Geophysical Services

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist or Engineer using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing, interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

Table 1. Borehole Logging Dates and Locations

BOREHOLE NUMBER	DATE LOGGED	COORDINATES[1]		
		LATITUDE	LONGITUDE	ELEVATION (FEET MSL)
R-20-004	3/15/2021	TBA	TBA	TBA
R-21-001	3/2/2021	TBA	TBA	TBA

[1]Awaiting coordinates from client

Table 2. Logging Tools, Depth Ranges and Sample Intervals

BOREHOLE NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (FEET)	SAMPLE INTERVAL (FEET)	LOGGING DATE(S)
R-20-004	SUSPENSION DOWN01	8.2 – 96.1	1.6	3/15/2021
R-21-001	SUSPENSION DOWN01	6.9 – 76.4	1.6	3/2/2021

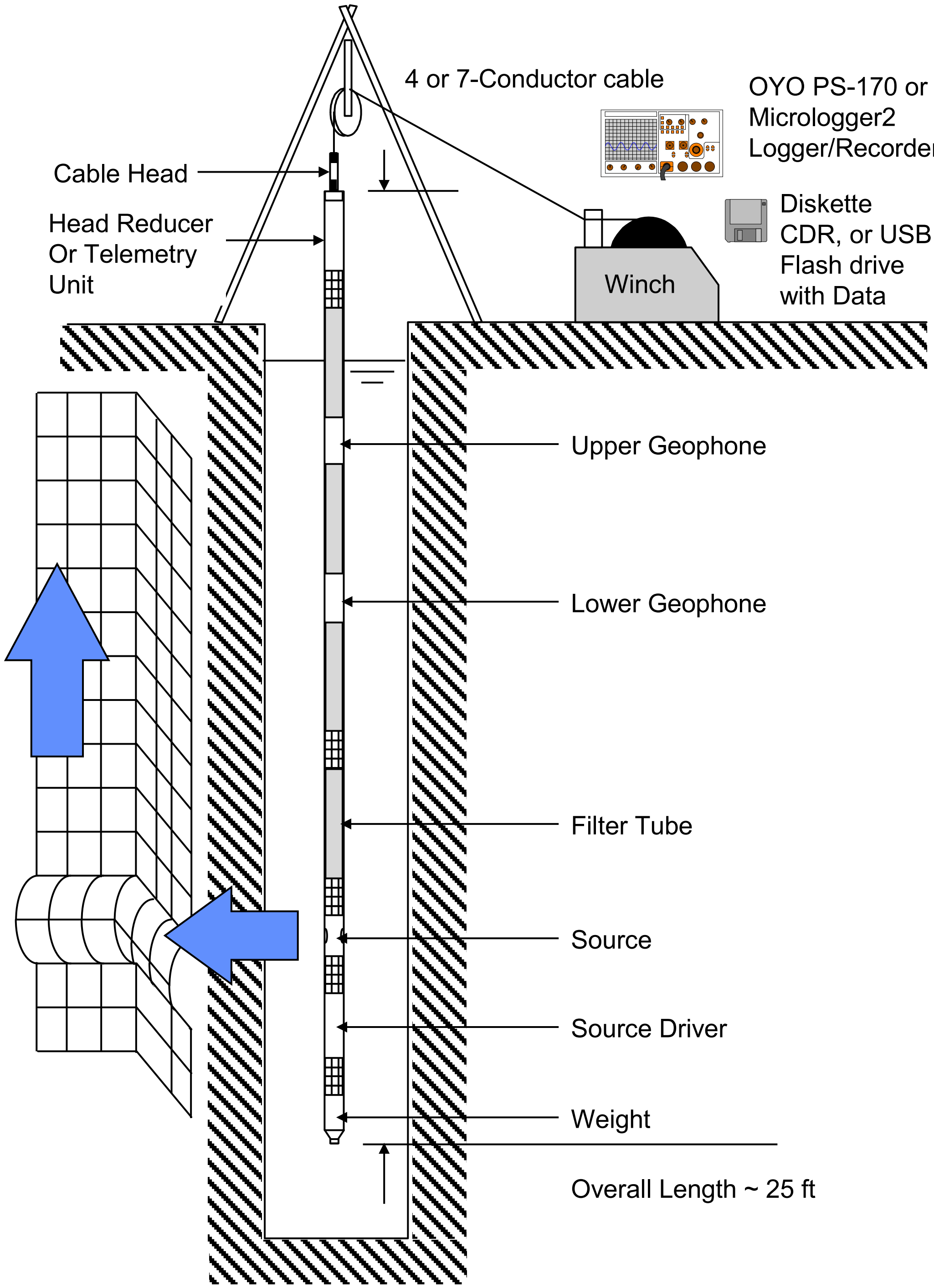


Figure 1: Concept illustration of P-S logging system

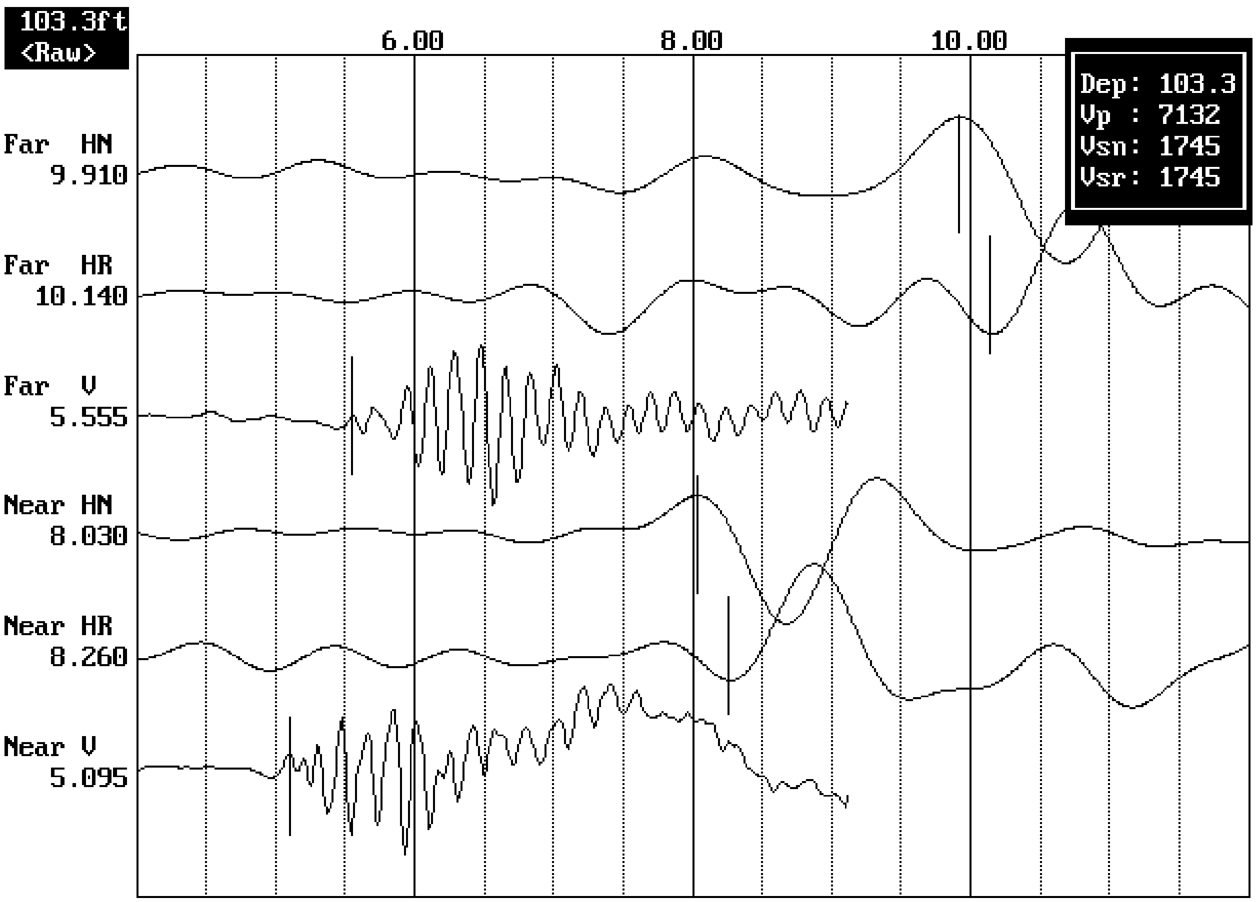


Figure 2: Example of filtered (1400 Hz lowpass) suspension record

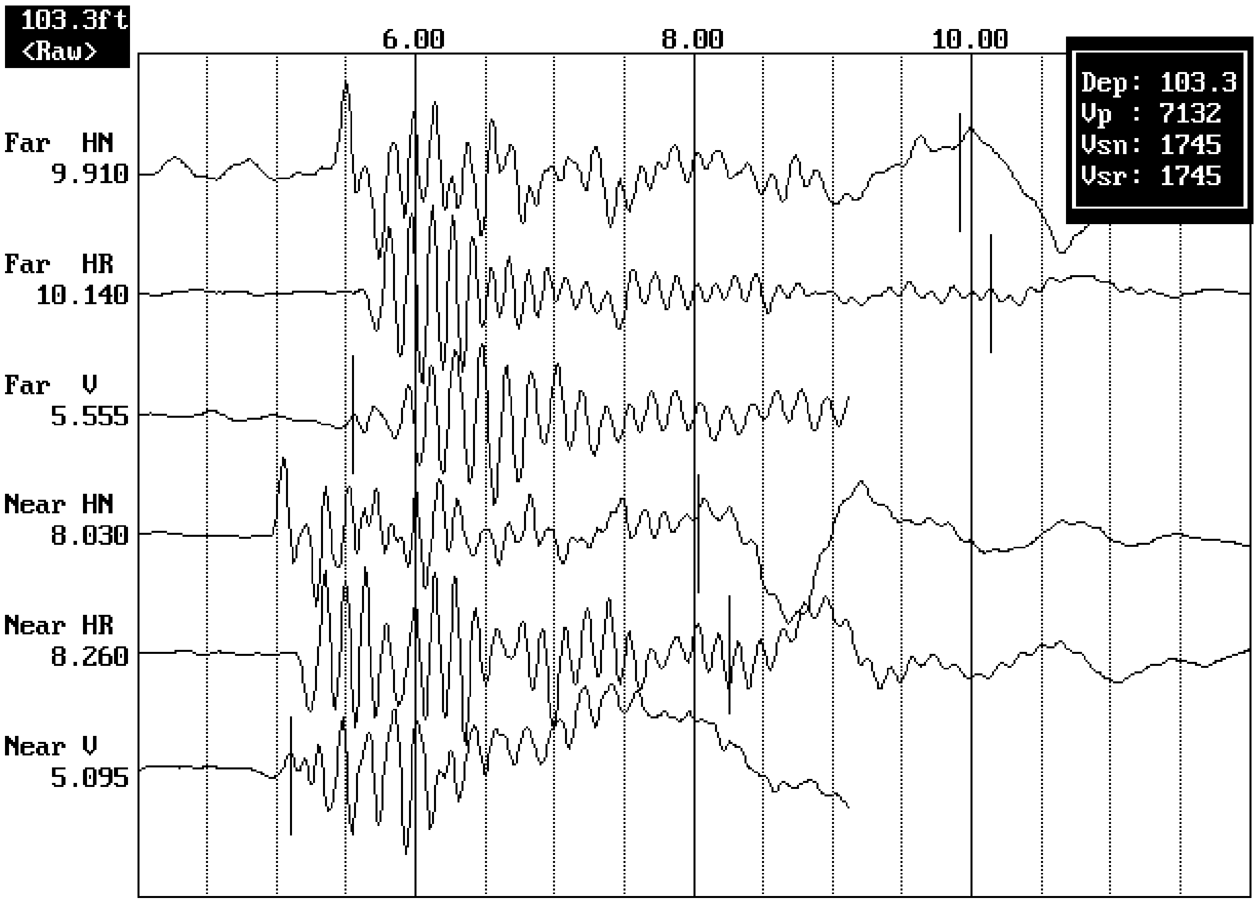


Figure 3: Example of unfiltered suspension record

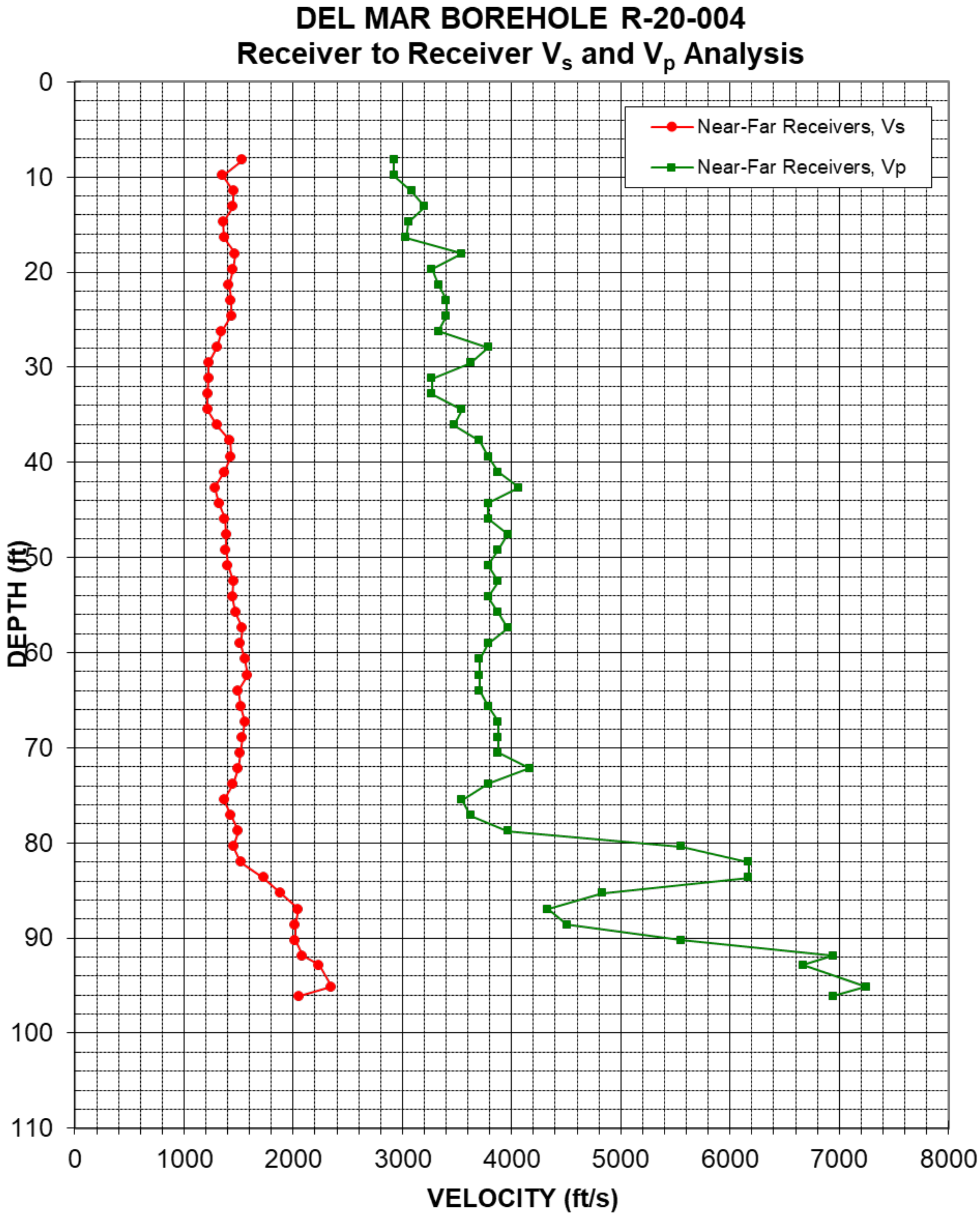


Figure 4: Borehole R-20-004, Suspension R1-R2 P- and SH-wave velocities

Table 3. Borehole R-20-004, Suspension R1-R2 depths and P- and S_H-wave velocities

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole R-20-004

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
8.2	1540	2920	0.31	2.5	470	890	0.31
9.8	1360	2920	0.36	3.0	410	890	0.36
11.5	1460	3090	0.36	3.5	440	940	0.36
13.1	1450	3210	0.37	4.0	440	980	0.37
14.8	1360	3060	0.38	4.5	410	930	0.38
16.4	1370	3030	0.37	5.0	420	920	0.37
18.0	1470	3550	0.40	5.5	450	1080	0.40
19.7	1450	3270	0.38	6.0	440	1000	0.38
21.3	1410	3330	0.39	6.5	430	1020	0.39
23.0	1430	3400	0.39	7.0	440	1040	0.39
24.6	1440	3400	0.39	7.5	440	1040	0.39
26.3	1340	3330	0.40	8.0	410	1020	0.40
27.9	1300	3790	0.43	8.5	400	1150	0.43
29.5	1230	3620	0.44	9.0	370	1100	0.44
31.2	1230	3270	0.42	9.5	370	1000	0.42
32.8	1220	3270	0.42	10.0	370	1000	0.42
34.5	1220	3550	0.43	10.5	370	1080	0.43
36.1	1300	3470	0.42	11.0	400	1060	0.42
37.7	1420	3700	0.41	11.5	430	1130	0.41
39.4	1430	3790	0.42	12.0	440	1150	0.42
41.0	1370	3880	0.43	12.5	420	1180	0.43
42.7	1280	4070	0.44	13.0	390	1240	0.44
44.3	1320	3790	0.43	13.5	400	1150	0.43
45.9	1370	3790	0.42	14.0	420	1150	0.42
47.6	1390	3970	0.43	14.5	420	1210	0.43
49.2	1380	3880	0.43	15.0	420	1180	0.43
50.9	1400	3790	0.42	15.5	430	1150	0.42
52.5	1460	3880	0.42	16.0	440	1180	0.42
54.1	1440	3790	0.42	16.5	440	1150	0.42
55.8	1470	3880	0.42	17.0	450	1180	0.42
57.4	1540	3970	0.41	17.5	470	1210	0.41
59.1	1520	3790	0.40	18.0	460	1150	0.40
60.7	1560	3700	0.39	18.5	470	1130	0.39
62.3	1580	3700	0.39	19.0	480	1130	0.39
64.0	1490	3700	0.40	19.5	460	1130	0.40
65.6	1520	3790	0.40	20.0	460	1150	0.40

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole R-20-004

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
67.3	1560	3880	0.40	20.5	470	1180	0.40
68.9	1540	3880	0.41	21.0	470	1180	0.41
70.5	1520	3880	0.41	21.5	460	1180	0.41
72.2	1490	4170	0.43	22.0	460	1270	0.43
73.8	1450	3790	0.41	22.5	440	1150	0.41
75.5	1370	3550	0.41	23.0	420	1080	0.41
77.1	1430	3620	0.41	23.5	440	1100	0.41
78.7	1490	3970	0.42	24.0	460	1210	0.42
80.4	1460	5560	0.46	24.5	440	1690	0.46
82.0	1520	6170	0.47	25.0	460	1880	0.47
83.7	1730	6170	0.46	25.5	530	1880	0.46
85.3	1880	4830	0.41	26.0	570	1470	0.41
86.9	2040	4330	0.36	26.5	620	1320	0.36
88.6	2020	4500	0.37	27.0	620	1370	0.37
90.2	2020	5560	0.42	27.5	620	1690	0.42
91.9	2080	6940	0.45	28.0	640	2120	0.45
92.9	2240	6670	0.44	28.3	680	2030	0.44
95.1	2350	7250	0.44	29.0	720	2210	0.44
96.1	2060	6940	0.45	29.3	630	2120	0.45

Notes: "-" means no data available at that depth.

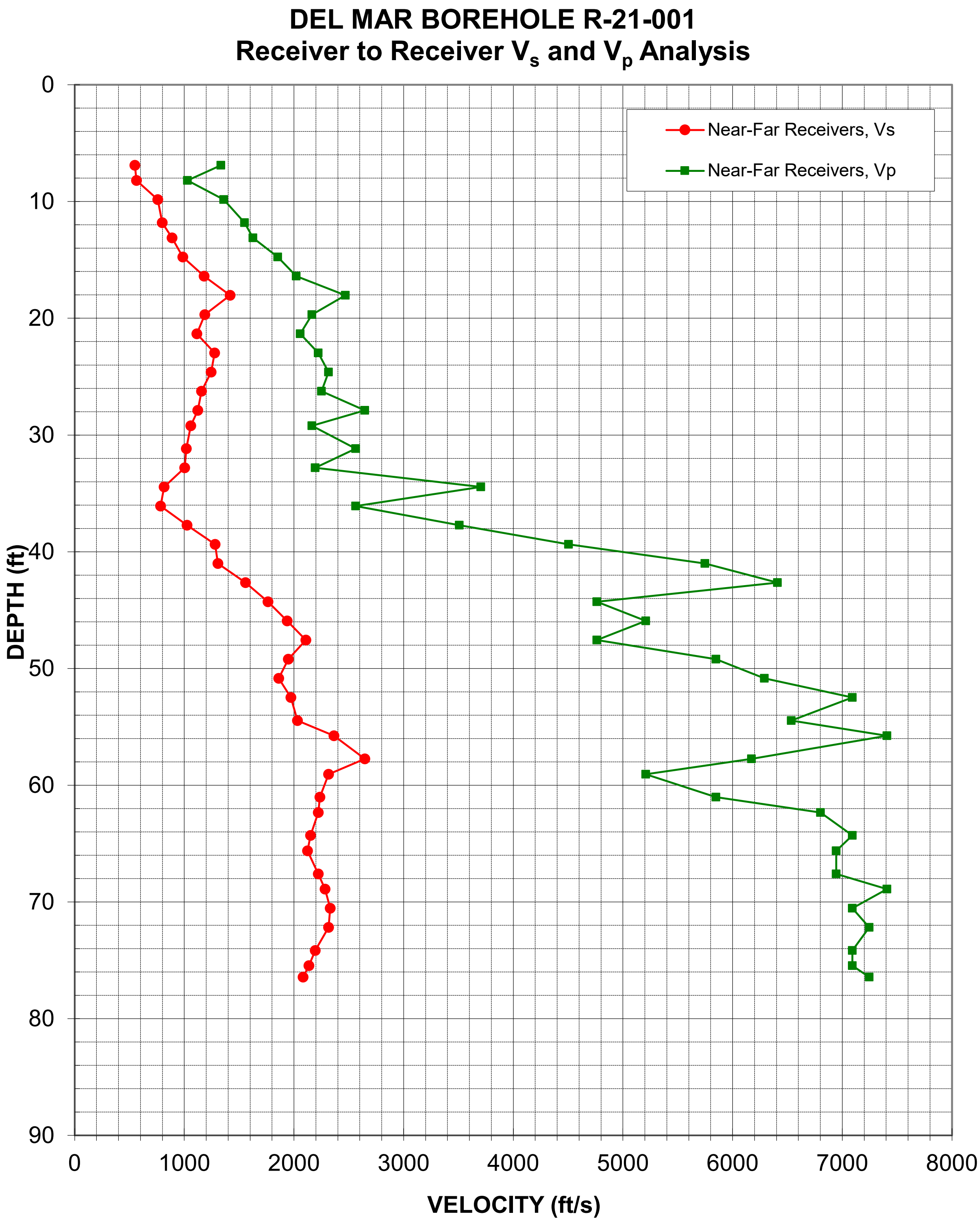


Figure 5: Borehole R-20-001, Suspension R1-R2 P- and S_H-wave velocities

Table 4. Borehole R-21-001, Suspension R1-R2 depths and P- and S_H-wave velocities

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole R-21-001

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
6.9	550	1330	0.40
8.2	560	1030	0.28
9.8	760	1360	0.28
11.8	800	1550	0.32
13.1	890	1630	0.29
14.8	990	1850	0.30
16.4	1180	2020	0.24
18.0	1420	2470	0.25
19.7	1190	2160	0.29
21.3	1110	2060	0.29
23.0	1280	2220	0.25
24.6	1240	2310	0.30
26.3	1160	2250	0.32
27.9	1120	2650	0.39
29.2	1060	2160	0.34
31.2	1020	2560	0.41
32.8	1000	2190	0.37
34.5	820	3700	0.47
36.1	780	2560	0.45
37.7	1030	3510	0.45
39.4	1280	4500	0.46
41.0	1310	5750	0.47
42.7	1560	6410	0.47
44.3	1760	4760	0.42
45.9	1940	5210	0.42
47.6	2110	4760	0.38
49.2	1950	5850	0.44
50.9	1860	6290	0.45
52.5	1970	7090	0.46
54.5	2030	6540	0.45
55.8	2360	7410	0.44
57.7	2650	6170	0.39
59.1	2310	5210	0.38
61.0	2240	5850	0.41
62.3	2220	6800	0.44
64.3	2150	7090	0.45
65.6	2120	6940	0.45
67.6	2220	6940	0.44

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
2.1	170	410	0.40
2.5	170	310	0.28
3.0	230	410	0.28
3.6	240	470	0.32
4.0	270	500	0.29
4.5	300	560	0.30
5.0	360	620	0.24
5.5	430	750	0.25
6.0	360	660	0.29
6.5	340	630	0.29
7.0	390	680	0.25
7.5	380	710	0.30
8.0	350	690	0.32
8.5	340	810	0.39
8.9	320	660	0.34
9.5	310	780	0.41
10.0	310	670	0.37
10.5	250	1130	0.47
11.0	240	780	0.45
11.5	310	1070	0.45
12.0	390	1370	0.46
12.5	400	1750	0.47
13.0	470	1950	0.47
13.5	540	1450	0.42
14.0	590	1590	0.42
14.5	640	1450	0.38
15.0	590	1780	0.44
15.5	570	1920	0.45
16.0	600	2160	0.46
16.6	620	1990	0.45
17.0	720	2260	0.44
17.6	810	1880	0.39
18.0	710	1590	0.38
18.6	680	1780	0.41
19.0	680	2070	0.44
19.6	660	2160	0.45
20.0	650	2120	0.45
20.6	680	2120	0.44

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole R-21-001

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
68.9	2280	7410	0.45	21.0	700	2260	0.45
70.5	2330	7090	0.44	21.5	710	2160	0.44
72.2	2310	7250	0.44	22.0	710	2210	0.44
74.2	2190	7090	0.45	22.6	670	2160	0.45
75.5	2140	7090	0.45	23.0	650	2160	0.45
76.4	2080	7250	0.45	23.3	640	2210	0.45

Notes: "-" means no data available at that depth.

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

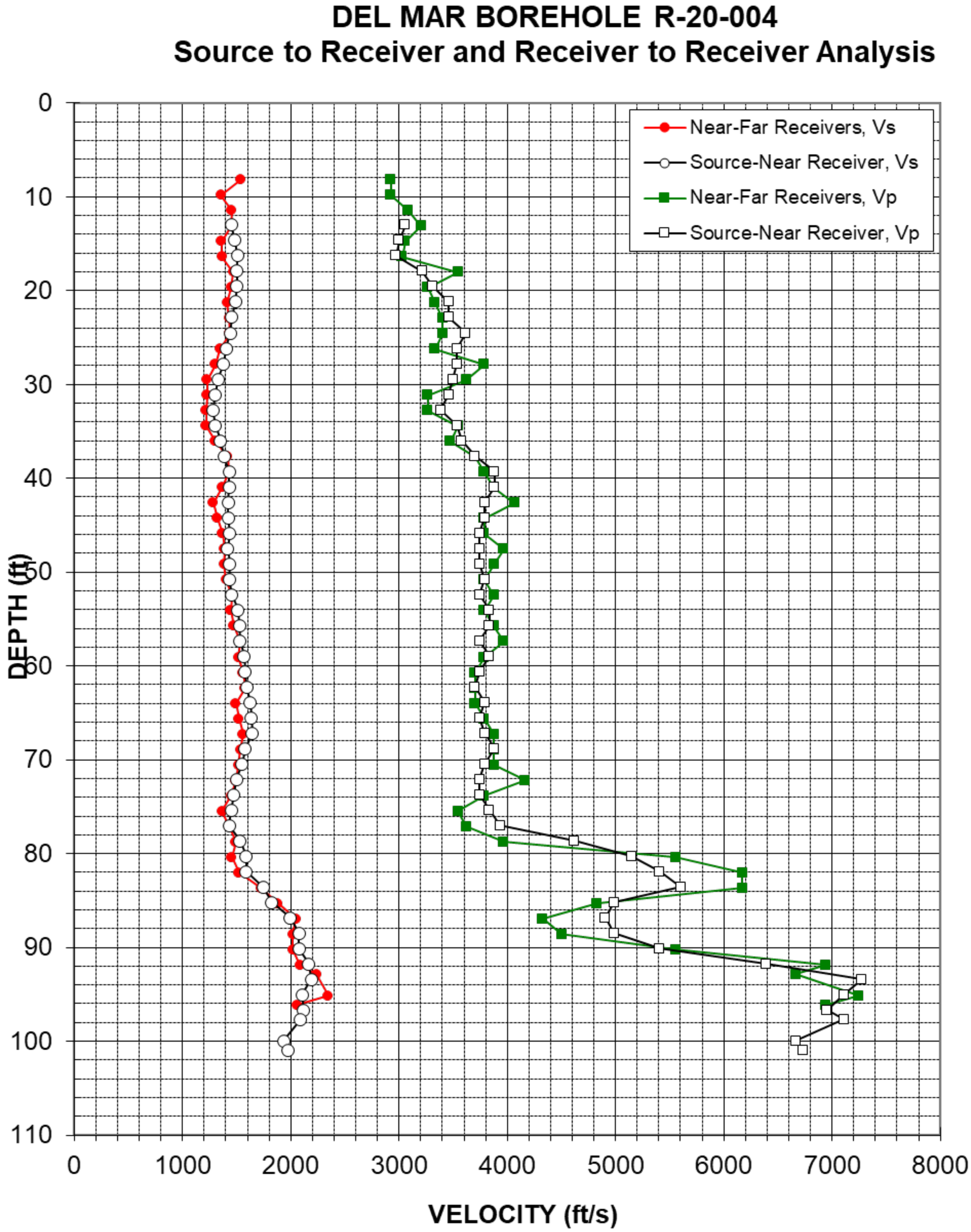


Figure A-1: Borehole R-20-004, Suspension S-R1 P- and S_H-wave velocities

Table A-1. Borehole R-20-004, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole R-20-004

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
13.0	1450	3060	0.36	4.0	440	930	0.36
14.7	1480	3000	0.34	4.5	450	910	0.34
16.3	1510	2970	0.33	5.0	460	910	0.33
18.0	1500	3210	0.36	5.5	460	980	0.36
19.6	1500	3310	0.37	6.0	460	1010	0.37
21.2	1490	3460	0.39	6.5	450	1050	0.39
22.9	1460	3460	0.39	7.0	440	1050	0.39
24.5	1440	3620	0.41	7.5	440	1100	0.41
26.2	1400	3540	0.41	8.0	430	1080	0.41
27.8	1380	3540	0.41	8.5	420	1080	0.41
29.4	1330	3500	0.42	9.0	400	1070	0.42
31.1	1310	3460	0.42	9.5	400	1050	0.42
32.7	1280	3390	0.42	10.0	390	1030	0.42
34.4	1310	3540	0.42	10.5	400	1080	0.42
36.0	1350	3580	0.42	11.0	410	1090	0.42
37.6	1390	3700	0.42	11.5	420	1130	0.42
39.3	1430	3880	0.42	12.0	440	1180	0.42
40.9	1430	3880	0.42	12.5	440	1180	0.42
42.6	1420	3790	0.42	13.0	430	1160	0.42
44.2	1420	3790	0.42	13.5	430	1160	0.42
45.8	1440	3750	0.41	14.0	440	1140	0.41
47.5	1420	3750	0.42	14.5	430	1140	0.42
49.1	1440	3750	0.41	15.0	440	1140	0.41
50.8	1430	3790	0.42	15.5	440	1160	0.42
52.4	1460	3750	0.41	16.0	440	1140	0.41
54.0	1510	3840	0.41	16.5	460	1170	0.41
55.7	1530	3840	0.41	17.0	460	1170	0.41
57.3	1530	3750	0.40	17.5	470	1140	0.40
59.0	1570	3840	0.40	18.0	480	1170	0.40
60.6	1580	3750	0.39	18.5	480	1140	0.39
62.2	1590	3700	0.39	19.0	490	1130	0.39
63.9	1630	3790	0.39	19.5	500	1160	0.39
65.5	1640	3750	0.38	20.0	500	1140	0.38
67.2	1640	3790	0.38	20.5	500	1160	0.38
68.8	1580	3880	0.40	21.0	480	1180	0.40
70.5	1550	3790	0.40	21.5	470	1160	0.40
72.1	1500	3750	0.41	22.0	460	1140	0.41

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole R-20-004

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
73.7	1480	3750	0.41	22.5	450	1140	0.41
75.4	1450	3840	0.42	23.0	440	1170	0.42
77.0	1430	3930	0.42	23.5	440	1200	0.42
78.7	1530	4620	0.44	24.0	470	1410	0.44
80.3	1590	5150	0.45	24.5	480	1570	0.45
81.9	1590	5410	0.45	25.0	480	1650	0.45
83.6	1740	5600	0.45	25.5	530	1710	0.45
85.2	1820	4980	0.42	26.0	560	1520	0.42
86.9	2000	4910	0.40	26.5	610	1500	0.40
88.5	2080	4980	0.40	27.0	630	1520	0.40
90.1	2080	5410	0.41	27.5	630	1650	0.41
91.8	2170	6390	0.44	28.0	660	1950	0.44
93.4	2190	7280	0.45	28.5	670	2220	0.45
95.1	2100	7110	0.45	29.0	640	2170	0.45
96.7	2120	6960	0.45	29.5	650	2120	0.45
97.7	2090	7110	0.45	29.8	640	2170	0.45
100.0	1940	6660	0.45	30.5	590	2030	0.45
101.0	1970	6730	0.45	30.8	600	2050	0.45

Notes: "-" means no data available at that depth.

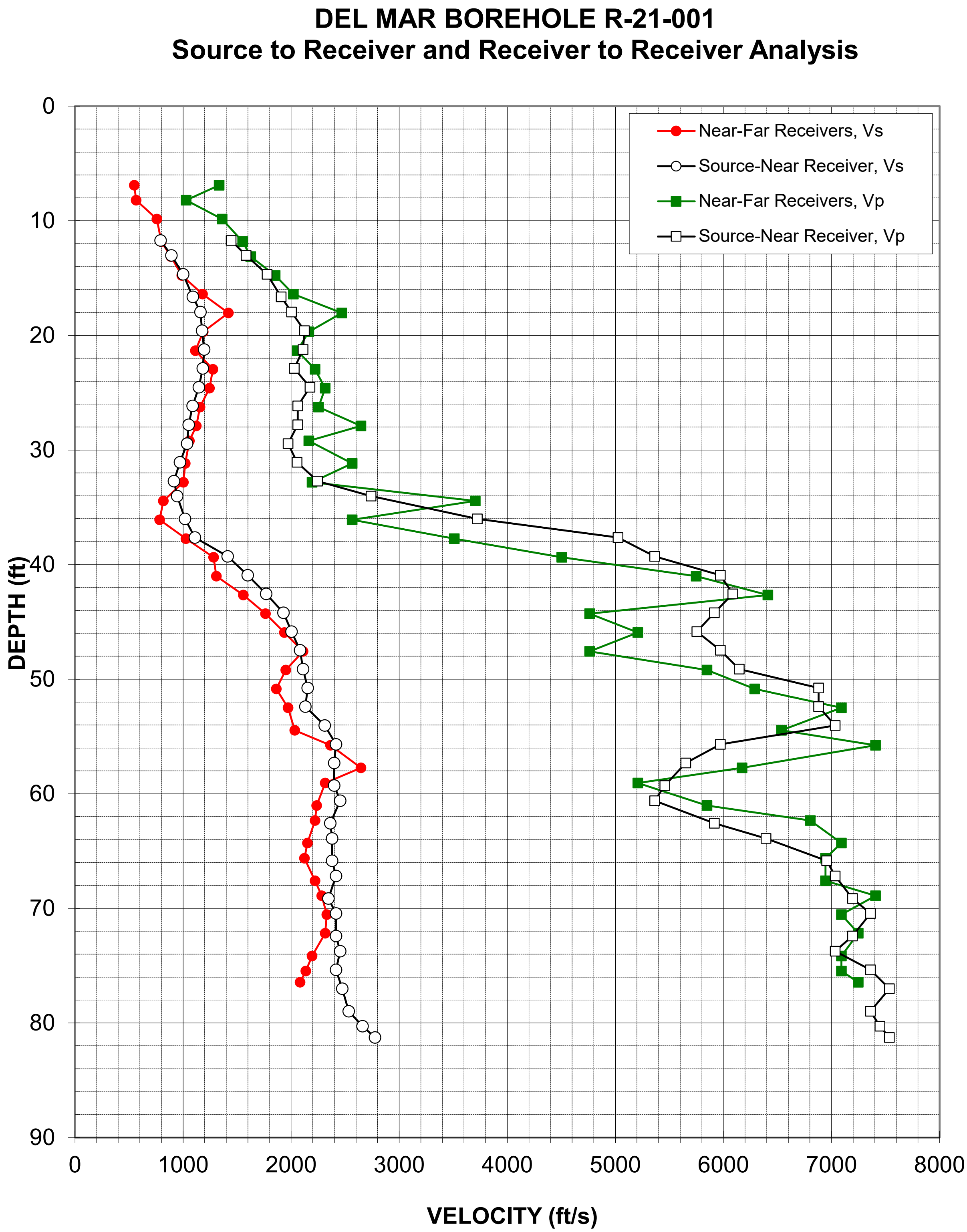


Figure A-2: Borehole A-21-001, Suspension S-R1 P- and S_H-wave velocities

Table A-2. Borehole R-21-001, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole R-21-001

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
11.7	790	1450	0.29	3.6	240	440	0.29
13.0	890	1580	0.27	4.0	270	480	0.27
14.7	1000	1780	0.27	4.5	310	540	0.27
16.6	1090	1910	0.26	5.1	330	580	0.26
18.0	1160	2000	0.25	5.5	350	610	0.25
19.6	1180	2120	0.28	6.0	360	650	0.28
21.2	1190	2110	0.26	6.5	360	640	0.26
22.9	1180	2030	0.24	7.0	360	620	0.24
24.5	1150	2180	0.31	7.5	350	660	0.31
26.2	1090	2060	0.31	8.0	330	630	0.31
27.8	1050	2060	0.32	8.5	320	630	0.32
29.4	1040	1970	0.31	9.0	320	600	0.31
31.1	970	2060	0.36	9.5	300	630	0.36
32.7	910	2240	0.40	10.0	280	680	0.40
34.0	940	2740	0.43	10.4	290	840	0.43
36.0	1020	3720	0.46	11.0	310	1130	0.46
37.6	1110	5020	0.47	11.5	340	1530	0.47
39.3	1410	5360	0.46	12.0	430	1640	0.46
40.9	1600	5970	0.46	12.5	490	1820	0.46
42.6	1770	6090	0.45	13.0	540	1860	0.45
44.2	1930	5920	0.44	13.5	590	1800	0.44
45.8	2000	5750	0.43	14.0	610	1750	0.43
47.5	2080	5970	0.43	14.5	630	1820	0.43
49.1	2110	6150	0.43	15.0	640	1870	0.43
50.8	2150	6880	0.45	15.5	660	2100	0.45
52.4	2130	6880	0.45	16.0	650	2100	0.45
54.0	2310	7030	0.44	16.5	700	2140	0.44
55.7	2420	5970	0.40	17.0	740	1820	0.40
57.3	2400	5650	0.39	17.5	730	1720	0.39
59.3	2400	5460	0.38	18.1	730	1660	0.38
60.6	2450	5360	0.37	18.5	750	1640	0.37
62.6	2360	5920	0.41	19.1	720	1800	0.41
63.9	2380	6390	0.42	19.5	730	1950	0.42
65.9	2380	6960	0.43	20.1	730	2120	0.43
67.2	2420	7030	0.43	20.5	740	2140	0.43
69.1	2340	7190	0.44	21.1	710	2190	0.44

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole R-21-001

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
70.5	2420	7360	0.44	21.5	740	2240	0.44
72.4	2420	7190	0.44	22.1	740	2190	0.44
73.7	2450	7030	0.43	22.5	750	2140	0.43
75.4	2420	7360	0.44	23.0	740	2240	0.44
77.0	2470	7540	0.44	23.5	750	2300	0.44
79.0	2530	7360	0.43	24.1	770	2240	0.43
80.3	2660	7450	0.43	24.5	810	2270	0.43
81.3	2780	7540	0.42	24.8	850	2300	0.42

Notes: "-" means no data available at that particular interval of depth.

APPENDIX B

BORING GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

ACCREDITED
ISO/IEC 17025
CALIBRATION LABORATORY
AC-1969.03

Certificate of Calibration

Date: Nov 11, 2020 **Cert No. 551220083929148**

Customer:
GEOVISION

1124 OLYMPIC DRIVE
CORONA CA 92881

		Work Order #:	LA-90048480
		Purchase Order #:	19401-201023-01
MPC Control #:	AM6768	Serial Number:	160024
Asset ID:	160024	Department:	N/A
Gage Type:	LOGGER	Performed By:	TYLER MCKEEN
Manufacturer:	OYO	Received Condition:	IN TOLERANCE
Model Number:	3403	Returned Condition:	IN TOLERANCE
Size:	N/A	Cal. Date:	October 27, 2020
Temp/RH:	26.7°C / 41.2%	Cal. Interval:	12 MONTHS
Location:	Calibration performed at MPC facility	Cal. Due Date:	October 27, 2021

Calibration Notes:
See attached data sheet for calculations. (1 Page)
Calibrated IAW customer supplied data form Rev 2.1
Frequency measurement uncertainty = 0.0005 Hz
Unit calibrated with Laptop Panasonic Model CF-29, s/n: 6AKSB01291 and RG Micrologger II Serial No. 5772
Calibrated To 4:1 Accuracy Ratio

Calibration performed in accordance with approved GEOVision calibration procedures included in work Instruction No. 13
Software: ML PS 4.00 Suspension Logger, GVLog.jar (2004) and pslog.exe ver 1.00 software.

Standards Used to Calibrate Equipment

I.D.	Description.	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
DB8748	GPS TIME AND FREQUENCY RECEIVER	58503A	3625A01225	HEWLETT PACKARD	Apr 30, 2021	551220083021224
BD7715	UNIVERSAL COUNTER	53131A	3416A05377	HEWLETT PACKARD	Apr 30, 2021	551220082934517
LAS0018	ARB / FUNC GENERATOR	33250A	US40001522	AGILENT	Apr 30, 2021	551220083580408

Calibrating Technician: QC Approval:

Ilya Vaks

TYLER MCKEEN ILYA VAKS

STATEMENTS OF PASS OR FAIL CONFORMANCE: The uncertainty of measurement has been taken into account when determining compliance with specification. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCSL Z540.3-2006 and in case without guard banded the probability of false-accept depending on test uncertainty ratio.

THE CALIBRATION REPORT STATUS:
PASS- Term used when compliance statement is given, and the measurement result is PASS.
PASS^z- Term used when compliance statement is given, and the measurement result is conditional passed or PASS^z.
FAIL- Term used when compliance statement is given, and the measurement result is FAIL.
FAIL^z- Term used when compliance statement is given, and the measurement result is conditional failed or FAIL^z.
REPORT OF VALUE - Term used when reported measurement is not requiring compliance statement in report.
ADJUSTED- When adjustments are made to an instrument which changes the value of measurement from what was measured as found to new value as left.
LIMITED - When an instrument fails calibration but is still functional in a limited manner.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCSL Z540.3. Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. The information on this report pertains only to the instrument identified, this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

Certificate of Calibration

Date: Nov 11, 2020 **Cert No. 551220083929148**

Procedures Used in this Event

Procedure Name	Description
GEOVISION SEISMIC Rev. 2.1	Seismic Logger/Recorder Calibration Procedure, Rev. 2.1

Calibrating Technician: QC Approval:

TYLER MCKEEN ILYA VAKS

STATEMENTS OF PASS OR FAIL CONFORMANCE: The uncertainty of measurement has been taken into account when determining compliance with specification. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCSL Z540.3-2006 and in case without guard banded the probability of false-accept depending on test uncertainty ratio.

THE CALIBRATION REPORT STATUS:
PASS- Term used when compliance statement is given, and the measurement result is PASS.
PASS^Z- Term used when compliance statement is given, and the measurement result is conditional passed or PASS^Z.
FAIL- Term used when compliance statement is given, and the measurement result is FAIL.
FAIL^Z- Term used when compliance statement is given, and the measurement result is conditional failed or FAIL^Z.
REPORT OF VALUE - Term used when reported measurement is not requiring compliance statement in report.
ADJUSTED- When adjustments are made to an instrument which changes the value of measurement from what was measured as found to new value as left.
LIMITED - When an instrument fails calibration but is still functional in a limited manner.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCSL Z540.3. Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. The information on this report pertains only to the instrument identified, this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.

GEOVision geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.:	OYO	Model no.:	3403
Serial no.:	160024	Calibration date:	10/27/2020
By:	Micro Precision	Due date:	10/27/2021
Counter mfg.:	Hewlett Packard	Model no.:	53131A
Serial no.:	3416A 05377	Calibration date:	04/23/2020
By:	Micro Precision	Due date:	04/30/2021
Signal generator mfg.:	Agilent	Model no.:	33250A
Serial no.:	US40001522	Calibration date:	04/02/2020
By:	Micro Precision	Due date:	04/30/2021
Laptop controller mfg.:	Panasonic	Model no.:	CF-29
Serial no.:	6AKSB01291	Calibration date:	N/A

SYSTEM SETTINGS:

Gain:	0 (2 all / lowest)
Filter	10kHz low pass
Range:	200 to 5 microsec
Delay:	0
Stack (1 std)	1
System date = correct date and time	Yes

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Software ML PS 4.00
PS Log.exe V.1.00
GVLog.jar ver. 2004

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found 0.12% As left 0.12%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name C024	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	001	180.2	49.94	180	50.00	180.2	49.94
100.0	100.0	100	002	89.9	100.1	90	100.0	89.9	100.1
200.0	200.0	50	003	45	200.0	45.05	199.8	45	200.0
500.0	500.0	20	004	18	500.0	18	500.0	18	500.0
1000	1000	10	005	9	1000	9.01	998.9	9	1000
2000	2000	5	006	4.495	2002	4.495	2002	4.505	1998

Calibrated by: Name Date 10/29/20 Signature Tyler McKean

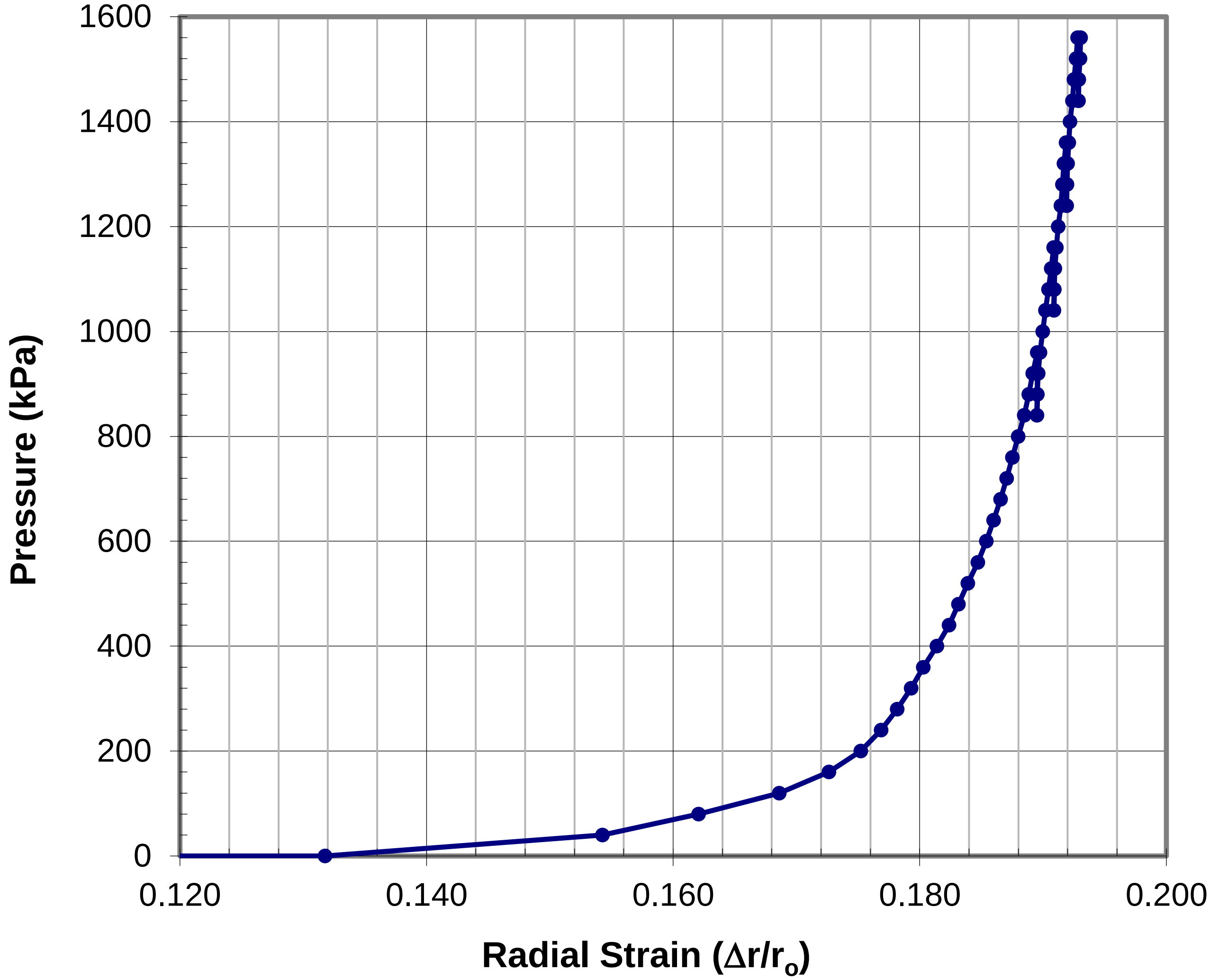
Witnessed by: Emily Feldman Name 10/27/20 Date Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.1 February 7, 2012

APPENDIX E

Pressuremeter Testing Results

Pressuremeter Test Result



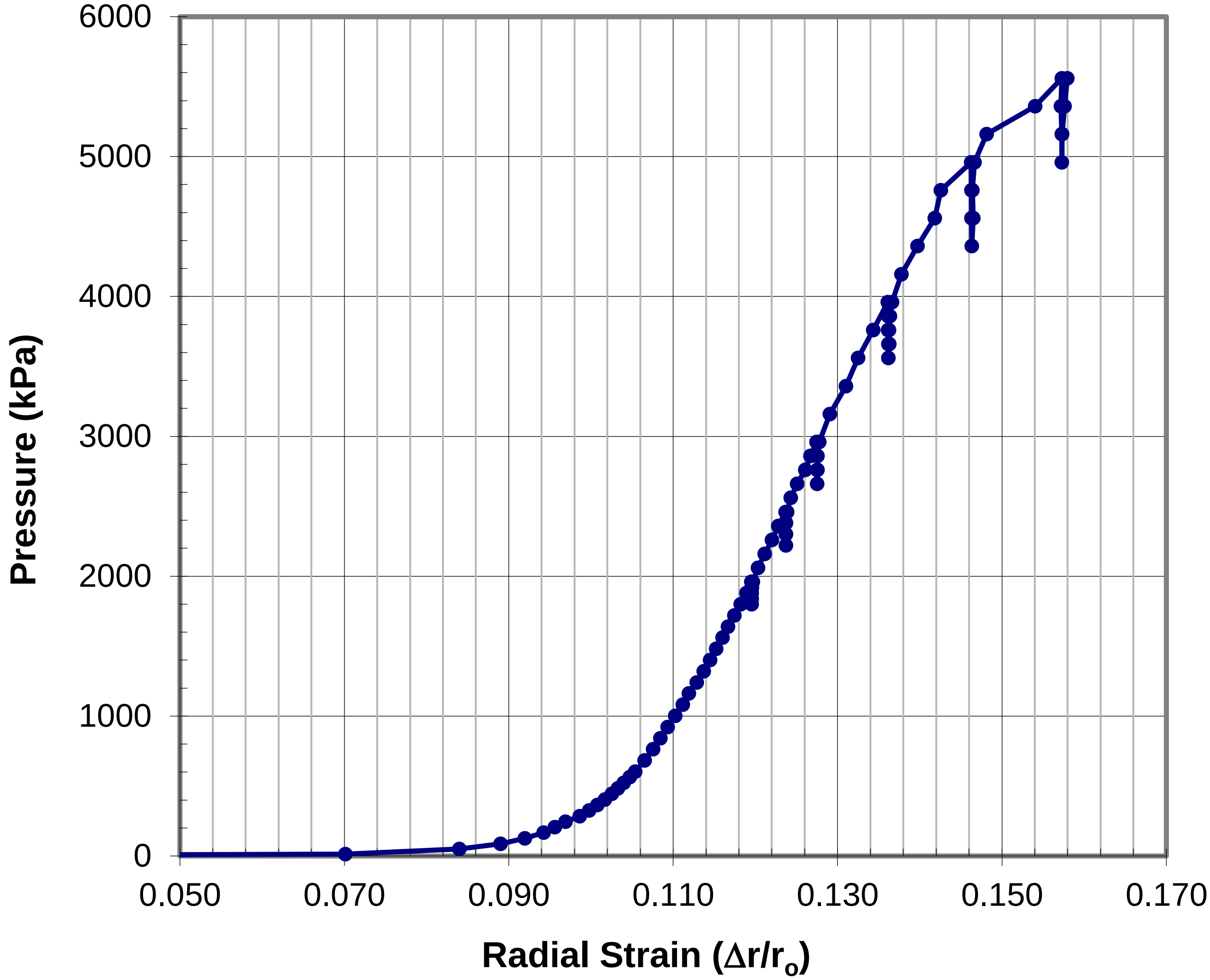
Boring No.:	**R-21-001**
Location:	**Luzon Avenue**
	Del Mar, CA
Test Date:	**3/1/2021**
Test Depth:	**48 feet**
Soil Type:	**Clayey Sandstone**

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

Regional Rail Corridor

Project No :	20-134	Date :	5/12/2021

Pressuremeter Test Result



Boring No.:	**R-21-004**
Location:	**Portofino Drive**
	Del Mar, CA
Test Date:	**3/1/2021**
Test Depth:	**65 feet**
Soil Type:	**Sandstone**

Earth Mechanics, Inc. Geotechnical and Earthquake Engineering	**Regional Rail Corridor**
	Project No : 20-134 Date : 5/12/2021

APPENDIX F

Packer Testing Results

Packer Test Results

Project:	Regional Rail Corridor		
Borehole ID	R-21-001		
Test Date:	3/2/2021		
By:	TBF/PSS		
Depth to Top of Packer:	50.0 ft	**Borehole Diameter:**	4.1 inches
Depth to Bottom of Packer:	60.0 ft	**Borehole Dip:**	Vertical
Test Interval:	10.0 ft	**Total Depth of Boring**	91 ft
GW Depth:	NM ft		

Rock Description

Depth	Soil Description
50'-60'	Interbedded Sandy Claystone/SANDSTONE (Del Mar Formation)

Packer Pressure (psi)	**300**		**300**		**300**	
Constant Water Pressure (psi)	15		25		35	
Starting Time						
Time (min)	**Water Flow**		**Water Flow**		**Water Flow**	
	Flow (gal)	**Flow Rate (gpm)**	**Flow (gal)**	**Flow Rate (gpm)**	**Flow (gal)**	**Flow Rate (gpm)**
0	941570.0	-	941845.0	-	942018.0	-
1	941580.0	10.0	941856.0	11.0	942032.0	14.0
2	941590.0	10.0	941866.5	10.5	942045.0	13.0
3	941600.0	10.0	941876.7	10.2	942057.5	12.5
4	941612.5	12.5	941887.0	10.3	942070.0	12.5
5	941622.5	10.0	941897.0	10.0	942082.2	12.2
6	941633.0	10.5	941907.0	10.0	942094.0	11.8
7	941643.5	10.5	941917.0	10.0	942105.7	11.7
8	941653.0	9.5	941927.0	10.0	942117.5	11.8
9	941662.8	9.8	941936.5	9.5	942129.5	12.0
10	941672.3	9.5	941946.2	9.7	942140.5	11.0
11	941682.0	9.7	941955.7	9.5	942152.0	11.5
12	941691.7	9.7	941965.0	9.3	942163.0	11.0
13	941701.0	9.3	941974.2	9.2	942174.5	11.5
14	941710.2	9.2	941983.3	9.1	942185.0	10.5
15	941719.1	8.9	941992.4	9.1	942196.0	11.0
16	941728.5	9.4	942001.4	9.0	942207.0	11.0
17	941736.7	8.2	942010.5	9.1	942218.0	11.0
18	941745.0	8.3			942229.0	11.0
19	941753.0	8.0			942239.5	10.5
20	941761.2	8.2			942250.5	11.0
21	941769.2	8.0				

Packer Test Results

Project: Regional Rail Corridor
Borehole ID R-21-002
Test Date: 3/2/2021
By: TBF/PSS

Depth to Top of Packer: 160.0 ft		**Borehole Diameter:** 3.8 inches	
Depth to Bottom of Packer: 170.0 ft		**Borehole Dip:** Vertical	
Test Interval: 10.0 ft		**Total Depth of Boring** 200 ft	
GW Depth: 58.5 ft			

Rock Description

Depth	Soil Description
160'-170'	Clayey Sandstone (Del Mar Formation)

Packer Pressure (psi)	300		300		300	
Constant Water Pressure (psi)	15		30		45	
Starting Time						
Time (min)	Water Flow		Water Flow		Water Flow	
	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)
0	942236.9	-	942237.5	-	942239.3	0.2
1	942236.9	0.00	942237.5	0.0	942239.7	0.3
2	942236.9	0.00	942237.5	0.0	942239.8	0.2
3	942237.0	0.05	942237.7	0.2	942240.0	0.2
4	942237.0	0.00	942237.8	0.1	942240.2	0.2
5	942237.0	0.00	942238.0	0.2	942240.4	0.2
6			942238.2	0.2	942240.6	0.2

Packer Pressure (psi)	300		300		300	
Constant Water Pressure (psi)	60		45 (Step Down)		30 (Step Down)	
Starting Time						
Time (min)	Water Flow		Water Flow		Water Flow	
	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)
0	942240.9	-	942460.0	-	942630.0	-
1	942241.6	0.7	942491.0	31.0	942645.0	15.0
2	942243.0	1.4	942510.0	19.0	942661.0	16.0
3	942246.1	3.1	942534.0	24.0	942676.0	15.0
4	942255.0	8.9	942555.0	21.0	942692.0	16.0
5	942268.0	13.0	942576.5	21.5	942708.0	16.0
6	942296.0	28.0			942724.0	16.0
7	942326.0	30.0			942740.0	16.0
8	942364.0	38.0			942756.0	16.0
9						

Packer Test Results

Project: Regional Rail Corridor
Borehole ID R-21-003
Test Date: 3/17/2021
By: TBF/PSS
Depth to Top of Packer: 95.0 ft
Depth to Bottom of Packer: 105.0 ft
Test Interval: 10.0 ft
GW Depth: 58.5 ft
Borehole Diameter: 3.8 inches
Borehole Dip: Vertical
Total Depth of Boring 115 ft

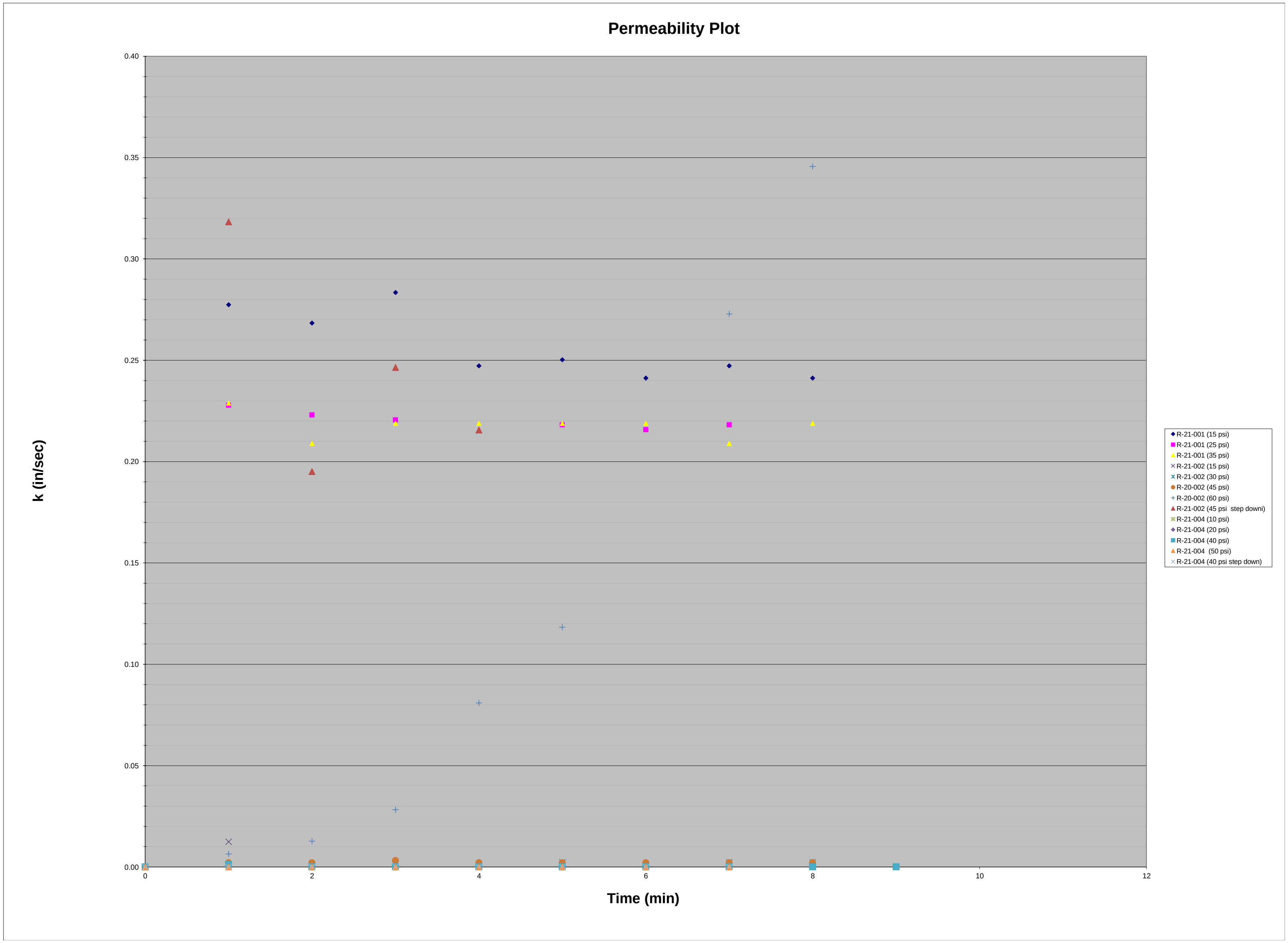
Rock Description	
Depth	Soil Description
95'-105'	Sandstone (Torrey Sandstone)

Packer Pressure (psi)	300		300		300	
Constant Water Pressure (psi)	10		20		40	
Starting Time	13:00		13:20		13:29	
Time (min)	Water Flow		Water Flow		Water Flow	
	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)
0	942773.0	-	942773.0	-	942775.3	-
1	942773.0	0.00	942773.0	0.0	942775.4	0.1
2	942773.0	0.00	942773.0	0.0	942775.4	0.0
3	942773.0	0.00	942773.0	0.0	942775.4	0.0
4	942773.0	0.00	942773.0	0.0	942775.4	0.0
5	942773.0	0.00	942773.0	0.0	942775.4	0.0
6						

Packer Pressure (psi)	300		300			
Constant Water Pressure (psi)	50		40 (Step Down)			
Starting Time	13:42		13:58			
Time (min)	Water Flow		Water Flow		Water Flow	
	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)	Flow (gal)	Flow Rate (gpm)
0	942775.5	-	942775.5	-		
1	942775.5	0.0	942775.5	0.0		
2	942775.5	0.0	942775.5	0.0		
3	942775.5	0.0	942775.5	0.0		
4	942775.5	0.0	942775.5	0.0		
5	942775.5	0.0	942775.5	0.0		
6	942775.5	0.0				
7	942775.5	0.0				
8	942775.5	0.0				
9						

	Time	Boring Number:											
		R-21-001	**R-21-001**	**R-21-001**	**R-21-002**	**R-21-002**	**R-21-002**	**R-21-002**	**R-21-002**	**R-21-002**	**R-21-003**	**R-21-003**	**R-21-003**	**R-21-003**
		15psi	25psi	35psi	15psi	30psi	45psi	60psi	45psi	30psi	10psi	20psi	40psi	50psi
Permeability	0	-	-	-	-	-	-	-	-	-	-	-	-	-
	1	2.77E-01	2.28E-01	2.29E-01	1.24E-02	0.00E+00	2.05E-03	6.37E-03	3.18E-01	1.77E-01	0.00E+00	0.00E+00	1.20E-03	0.00E+00
	2	2.68E-01	2.23E-01	2.09E-01	0.00E+00	0.00E+00	2.05E-03	1.27E-02	1.95E-01	1.89E-01	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	3	2.83E-01	2.21E-01	2.19E-01	0.00E+00	0.00E+00	3.08E-03	2.82E-02	2.46E-01	1.77E-01	0.00E+00	0.00E+00	0.00E+00	0.00E+00
	4	2.47E-01	5.99E-01	2.19E-01	6.92E-04	0.00E+00	2.05E-03	8.10E-02	2.16E-01	1.89E-01			0.00E+00	0.00E+00
	5	2.50E-01	2.18E-01	2.19E-01	0.00E+00	2.36E-03	2.05E-03	1.18E-01	2.21E-01	1.89E-01			0.00E+00	0.00E+00
	6	2.41E-01	2.16E-01	2.19E-01	0.00E+00	1.18E-03	2.05E-03	5.58E-01		1.89E-01			0.00E+00	0.00E+00
	7	2.47E-01	2.18E-01	2.09E-01		2.36E-03	2.05E-03	2.73E-01		1.89E-01			0.00E+00	0.00E+00
	8	2.41E-01		2.19E-01		2.36E-03	2.05E-03	3.46E-01		1.89E-01			0.00E+00	
	9												0.00E+00	
	10													

	Time	Boring Number:			
		R-21-004			
		40psi			
Permeability	0	0			
	1	0.00E+00			
	2	0.00E+00			
	3	0.00E+00			
	4	0.00E+00			
	5	0.00E+00			
	6	0.00E+00			
	7	0.00E+00			
	8				



Permeability Plot
k (in/sec)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
2
4
6
8
10
12
Time (min)
R-21-001 (15 psi)
R-21-001 (25 psi)
R-21-001 (35 psi)
R-21-002 (15 psi)
R-21-002 (30 psi)
R-20-002 (45 psi)
R-20-002 (60 psi)
R-21-002 (45 psi step downi)
R-21-004 (10 psi)
R-21-004 (20 psi)
R-21-004 (40 psi)
R-21-004 (50 psi)
R-21-004 (40 psi step down)

GEOLOGIC RECONNAISSANCE REPORT
DEL MAR ALTERNATIVE TUNNEL ALIGNMENTS
CONCEPTUAL ENGINEERING STUDY
SAN DIEGO REGIONAL RAIL CORRIDOR ALTERNATIVE ALIGNMENT AND IMPROVEMENTS PROJECT
DEL MAR AND SAN DIEGO, CALIFORNIA

Prepared For

HDR ENGINEERING, INC.
401 B STREET, SUITE 1110
SAN DIEGO, CALIFORNIA 92101

Prepared By

LEIGHTON CONSULTING, INC.
3934 MURPHY CANYON RD, SUITE B-205
SAN DIEGO, CALIFORNIA 92123

Project Number 13682.001

October 21, 2022

Leighton 60th Anniversary

Leighton Consulting, Inc.
A Leighton Group Company

October 21, 2022

Project No. 13682.001

HDR Engineering, Inc.
401 B Street, Suite 1110
San Diego, California 92101

Attention: Mr. Rob Klovsky

Subject: Geologic Reconnaissance Report
Del Mar Alternative Tunnel Alignments
Conceptual Engineering Study
San Diego Regional Rail Corridor Alternative Alignment and Improvements Project
Del Mar and San Diego, California

In accordance with your request and authorization, Leighton Consulting, Inc. (Leighton) is pleased to submit this geologic reconnaissance report for the selected Del Mar Alternative Tunnel Alignments as part of the San Diego Regional Rail Corridor Alternative Alignment and Improvements Project. This report includes a planning-level engineering geologic assessment along the proposed Crest Canyon High Speed and Camino Del Mar tunnel alignments located in the Cities of Del Mar and San Diego, California.

If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

Registered Professional Engineer, Carlos V. Amante, GE 2724, Expires 6/30/2024, Geotechnical, State of California

Professional Geologist, Robert C. Stroh, No. 2099, Certified Engineering Geologist, Exp. 10/31/23, State of California

Carlos V. Amante, GE 2724
Principal Engineer
Extension 1682, camante@leightongroup.com

Robert C. Stroh, CEG 2099
Principal Engineering Geologist
Extension 4090, rstroh@leightongroup.com

Professional Geologist, Roy N. Butz, No. 2752, Certified Engineering Geologist, Exp. 3/31/23, State of California

Roy N. Butz, CEG 2752
Senior Project Geologist
Extension: 8489, rbutz@leightongroup.com

Distribution: (1) Addressee via email

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1.0 EXECUTIVE SUMMARY

In accordance with the request and authorization of the San Diego Association of Governments (SANDAG), this report presents the results of Leighton's geologic reconnaissance of the selected Del Mar Tunnel Alignments as part of the Conceptual Engineering Study for the San Diego Regional Rail Corridor Alternative Alignment and Improvements Project. The study was performed to provide a preliminary engineering geologic assessment of the proposed Crest Canyon High Speed and Camino Del Mar tunnel alignments which will extend through the City of Del Mar and northern coastal limits of San Diego, California. As part of our evaluation, we reviewed previous documents and reports, conducted limited reconnaissance-level geologic mapping, performed generalized geologic hazard analyses, and prepared this summary report.

In order to provide an assessment of various geologic engineering factors affecting the tunnel portions of the proposed Del Mar rail alignment alternatives, summary tables of Material Engineering Characteristics, and Stability Elements and Constraints were developed. They provide descriptions and relative rankings with respect to geologic conditions, excavation stability, groundwater, liquefaction/dynamic settlement, and construction impacts to adjacent developments. The major factors influencing excavation stability include exposure of cohesionless materials, potentially expansive sedimentary rock materials, adverse geologic structure, such as faulted and sheared bedrock, high groundwater and/or seepage.

2.0 INTRODUCTION

2.1 Purpose and Scope

In accordance with the request and authorization of SANDAG in conjunction with HDR Engineering, Inc., this report presents the results of Leighton's geologic reconnaissance study of the selected Del Mar Tunnel Alignments as part of the San Diego Regional Rail Corridor Alternative Alignment and Improvements Project. The limits of our study included two alternatives with a combined length of over 26,600 linear feet of proposed tunnel alignments which traverse beneath portions of the City of Del Mar and the northern coastal limits of San Diego, California (Figure 1). This geologic reconnaissance study was completed to evaluate whether the proposed Del Mar Alternative Tunnel Alignments are within geologic materials with known (or a potential for) geologic hazards based on existing site features and review of readily available geologic documents. Leighton's scope of work consisted of:

- Researching in-house and published geotechnical, geologic, topographic, and seismic reports and maps of the area (see Appendix A for the list of references used in this report);
- Research to obtain previous geologic, geotechnical reports, and boring logs (see Appendix A);
- Stereoscopic analysis of aerial photos to assist in the geologic interpretation and identification of faults and other potential hazard-related features (see Appendix A);
- Limited geologic field mapping (reconnaissance-level) along the proposed tunnel alignments to observe surface features and site geologic conditions;
- Preparation of a *Geologic and Fault Map (*see Plate 1). The proposed centerlines of the Main Track (MT-1) tunnel alignments, provided by HDR Engineering, Inc., were used as the reference features for the geologic map;
- Preparation of two *Generalized Geologic Cross-Sections* (see Plate 2). The generalized geologic cross-sections were prepared by using topographic and track-grade profiles provided by HDR Engineering, Inc.;
- Ranking of potential geotechnical constraints along the proposed tunnel alignments. Values, generally between 0 and 4, are shown on the Generalized Geologic Cross-Sections and identify potential geotechnical constraints. In addition, a summary of the constraints is presented in the table of *Stability*

Elements and Constraints (presented in Appendix B). These values can be utilized to prioritize areas of highest concern by geologic condition;

- Discussion of formational materials anticipated along the proposed tunnel alignments. The geologic map units are discussed in Section 4.0. Presentation of the generalized engineering characteristics of the materials is included in the table of *Material Engineering Characteristics* (presented in Appendix B);
- Characterization of the faulting and seismic setting along the proposed tunnel alignments. Discussion is included in Section 5.0; and
- Preparation of this report presenting our findings, a description of the engineering geologic characteristics of the earth materials, and the identification of potential geologic hazards along the proposed tunnel alignments.

2.2 Project Description

The Conceptual Engineering Study for the proposed Del Mar tunnel alignments will assess the current corridor conditions along the San Diego Subdivision rail corridor in order to develop route alternatives to improve serviceability. The San Diego Subdivision is the southernmost portion of the Los Angeles-San Diego-San Luis Obispo (LOSSAN) rail corridor and extends approximately 60 miles from the Orange County/San Diego County line to downtown San Diego. The general purpose of the Conceptual Engineering Study is to develop a program of improvements in order to reduce travel time for commuter and intercity passenger rail service while still being competitive with automobile travel times.

As part of the San Diego Regional Rail Corridor Alternative Alignment and Improvements Project, SANDAG has selected two potential tunnel alignments which will traverse through the City of Del Mar and the northern coastal limits of San Diego, California. The purpose of these tunnels is to move the existing rail system off the coastal bluffs in the Del Mar area where marine erosion, subaerial erosion, and slope instabilities have created costly maintenance and repairs over the years. The two selected Del Mar Tunnel Alignments' Alternatives are described in further detail below:

2.2.1 Crest Canyon High Speed Alternative

The proposed Crest Canyon High Speed tunnel alignment trends in a north-northwest direction and is approximately 15,400 linear feet in length (approximate Station Number 19+00 to 173+00). The proposed northern tunnel portal U-structure is approximately 1,100 feet in length and located north of the intersection of Jimmy Durante Blvd and Camino Del Mar in the

City of Del Mar, California and the southern tunnel portal is located approximately 600 feet southeast of the intersection of Portofino Drive and Carmel Valley Road in the City of San Diego, California.

Overall, surface topography along the proposed Crest Canyon High Speed tunnel alignment generally consists of uplifted gently westward to eastward sloping landforms, including terraces and hillsides which have been subdued by erosional processes and human development, with surface elevations ranging from approximately 30 to 350 feet North American Vertical Datum of 1988 (NAVD 88). The proposed tunnel track-grade elevations range from approximately 50 feet at the south portal to 12 feet at the north portal, with a low point at Station 36+00 with an elevation of approximately -10 feet and a high point at Station 170+00 with an elevation of approximately 58 feet. The location of the proposed Crest Canyon High Speed tunnel alignment is depicted on Figure 1 and Plate 1.

2.2.2 Camino Del Mar Alternative

The proposed Camino Del Mar tunnel alignment trends in a south to north direction, is approximately 11,200 linear feet in length (approximate Station Number 19+00 to 131+00), and generally runs north to south below the City of Del Mar, similar to the overall orientation of Camino Del Mar. The proposed northern U-structure portal is approximately 1,200 feet in length and located north of the intersection of Jimmy Durante Blvd and Camino Del Mar in the City of Del Mar, California and the southern portal is located approximately 450 feet southeast of the intersection of Camino Del Mar and Carmel Valley Road in the City of San Diego, California.

Overall, surface topography along the proposed Camino Del Mar tunnel alignment generally consists of uplifted gently westward to eastward sloping landforms, including terraces and hillsides which have been subdued by erosional processes and human development, with surface elevations ranging from approximately 30 to 200 feet (NAVD 88). The proposed tunnel track-grade elevations range from approximately 35 feet at the south portal to 12 feet at the north portal, with a low point at Station 34+00 with an elevation of approximately -2 feet and a high point at Station 126+00 with an elevation of approximately 42 feet. The location of the proposed Camino Del Mar tunnel alignment is depicted on Figure 1 and Plate 1.

3.0 SUBSURFACE EXPLORATION

3.1 Geotechnical Borings

As part of the Conceptual Engineering Study, Earth Mechanics Inc. (EMI) has performed a preliminary geotechnical investigation (EMI, 2022) for the Del Mar tunnel alternatives. EMI's limited geotechnical investigation generally consisted of excavation of four small diameter mud rotary borings to depths between approximately 91 and 200 feet below the existing ground surface (bgs). The approximate locations of EMI's borings are shown on the *Geologic and Fault Map* (Plate 1) and the Generalized Geologic Cross-Sections (Plate 2). EMI's borings are presented in Appendix C.

3.2 Previous Geotechnical Studies

As part of our study, we performed document research to obtain previous geologic, geotechnical reports, and boring logs which are located in the general vicinity of the proposed tunnel alignments. As a result of our research, we were able to obtain and review the following geotechnical reports in addition to EMI's Preliminary Geotechnical Report (2022) that are pertinent to the project site.

- Leighton Consulting, Inc., 2020, San Dieguito Bridge Replacement, Double Track and Del Mar Fairgrounds Special Events Platform (Milepost 242 to Milepost 244) 90% Design, Draft Geotechnical Design Report, Project Number 11860.007, dated January 31, 2020.
- Ninyo and Moore, 2013, Update Geotechnical Evaluation, North County Transit District, Bridge 246.1 Replacement Project, Los Penasquitos Lagoon, San Diego, California, Project Number 105991020, dated August 29, 2013.

Specifically, we were able to find five small-diameter geotechnical borings that were performed for the San Dieguito Bridge Replacement Project that are generally located within the valley beyond the location of the north tunnel portals. These geotechnical borings extend to depths between approximately 21½ and 56½ feet bgs. In addition, we found one small-diameter geotechnical boring that was performed for the Bridge 246.1 Replacement Project which is generally located within the valley south of the south tunnel portals. This geotechnical boring extends to a depth of approximately 120½ feet bgs. The approximate location of the geotechnical borings is shown on the Geologic and Fault Map (Plate 1) and a copy of the boring logs are provided in Appendix C of this report.

4.0 GEOLOGY

4.1 Geologic Setting

The proposed Del Mar Alternative Tunnel Alignments are situated in the Peninsular Range province, a California Geomorphic province with a long and active geologic history. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic-aged igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous-age, Tertiary-age, and Quaternary-age sedimentary units. Most of the coastal region of the County of San Diego, including along the proposed tunnel alignments, is located within this coastal region and are underlain by sedimentary units. Specifically, the proposed tunnel alignments are located within the coastal plain section of the Peninsular Range Geomorphic Province of California, which generally consists of subdued landforms underlain by Tertiary-age sedimentary formational units consisting of the Torrey Sandstone and Delmar Formation which is overlain by younger Paralic Deposits. The aerial distribution of the geologic units is depicted on Plate 1 and brief descriptions of these units, as described in the cited literature and as observed during our geologic mapping, are presented below.

4.1.1 Artificial Fill (Af)

Based on our mapping and document research, areas of artificial fill soils were observed in a number of places overlying the proposed tunnel alignments. Locally, artificial fill soils may be located at the proposed tunnel portal locations. These fill soils are generally associated with the existing developments and improvements in the area and can range from 1 to 25 or more feet in depth. In general, these materials are expected to be of variable density and predominantly consist of reworked versions of the surrounding materials, and may present a settlement concern at the portal locations.

4.1.2 Quaternary-aged Paralic Estuarine Deposits (Qpe)

Based on our mapping and document research, intertidal-estuarine deposits associated with the San Dieguito and Soledad Valley are present near the proposed tunnel portal locations. These estuarine materials generally consist of interbedded loose silty sand and sandy silt and very soft to firm clay with organics. Although the estuarine deposits are not expected to be encountered during tunnel excavation operations, these materials are compressible and can present a settlement concern.

4.1.3 Quaternary-aged Young Alluvial Flood Plain Deposits (Qya)

Based on our mapping and document research, thick young alluvial deposits associated with the San Dieguito River drainage are present at the northern proposed tunnel portal locations. In addition, alluvial deposits associated with the Los Penasquitos River drainage are present within the valley located beyond the southern proposed tunnel portal locations. In general, these poorly consolidated and potentially compressible soils predominantly consist of light to medium gray and dark brown clays, silts and sands with occasional pebble and cobble lenses. Where these alluvial deposits are expected to be encountered during tunnel and/or portal excavation operations (northern portal locations), these materials can present a settlement concern where additional loading and/or dewatering operations are proposed.

4.1.4 Quaternary-aged Old Paralic Deposits (Qop_6 and $Qop_{2\text{-}4}$)

Mapping by Kennedy and Tan (2008), and corroborated during our mapping indicate Quaternary-aged Old Paralic Deposits are deposited on now elevated and relatively level wave cut platforms which overlie the underlying sedimentary formational units. Based on our experience and site mapping, these materials generally consist of medium dense to dense, moderately permeable, reddish brown to brown, interfingered strandline, beach, estuarine, and colluvial deposits composed of silty to clayey sand with interbedded layers of gravels and cobbles. The Quaternary-aged Old Paralic Deposits outcrop primarily along the coastal bluffs in the Del Mar area; however, we anticipate that these deposits will be encountered along the flanks of the hills located near or at the southern proposed tunnel portal locations (Plate 1).

4.1.5 Quaternary-aged Very Old Paralic Deposits ($Qvop_{11}$, $Qvop_{10}$, and $Qvop_{10a}$)

Mapping by Kennedy and Tan (2008), and corroborated during our mapping indicate Quaternary-aged Very Old Paralic Deposits have been largely derived from the erosion and redeposition of older sedimentary rocks located within the San Diego embayment. Based on our experience and site reconnaissance, these materials generally consist of dense to very dense, fine- to medium-grained, moderately permeable, silty to clayey sandstone with interbedded regions of sandy cobble conglomerate. In addition, the stratigraphic unit identified as $Qvop_{10a}$ is an ancient dune and back beach "beach ridge" deposit which generally consists of very dense, fine-grained, reddish-brown, moderately permeable, cross-bedded, silty sandstone (Kennedy and Tan, 2008). The ridge has a conspicuous linear topographic high that was formed along a strand line and is closely related to stratigraphic unit $Qvop_{10}$. The Quaternary-aged Very Old Paralic Deposits primarily outcrop along the higher elevations overlying the proposed tunnel alignments and are not anticipated to be encountered during tunnel excavation operations.

4.1.6 Tertiary-aged Torrey Sandstone (Tt)

The Tertiary-aged Torrey Sandstone was deposited along a submerging coast on an arcuate barrier beach that enclosed and then later transgressed over the older Delmar Formation. Its deposition ceased when submergence slowed and the shoreline retreated. Based on our experience and site mapping, the Torrey Sandstone is primarily composed of arkosic sandstone which is white to light brown, dense to very dense, medium- to coarse-grained, subangular, and moderately well indurated. It is also massive and broadly cross-bedded. It should be noted that the geologic contact between the Torrey Sandstone and the underlying Delmar Formation is gradational, highly interfingered, and should be anticipated along the proposed tunnel alignments.

4.1.7 Tertiary-aged Delmar Formation (Td)

The Tertiary-aged Delmar Formation is an ancient lagoonal deposit and is the dominant subsurface basal unit that is mapped along the sea cliffs in the Del Mar area. Based on our experience and site reconnaissance, the Delmar Formation primarily consists of weakly bedded claystone and siltstone and to a lesser extent silty sandstone. The siltstone and claystone generally are olive-green to gray, moist to very moist, stiff to hard, moderately weathered (when close to the surface), fractured, and sheared. The sandstone generally consists of off-white to gray and mottled yellow to orange-brown, damp to moist, dense to very dense, silty, well indurated, fine- to medium-grained sandstone. The sandstone is typically massive to faintly bedded (probably cross-bedding), micaceous, iron oxide stained with very scattered pebble to small cobble lenses and claystone rip-up clasts (Abbott, 1985). The sandstone is found in both lenses (channel infills) and in beds of varying thickness. It is anticipated that a majority of the tunnel alignments will be founded completely within the Delmar Formation.

4.2 Geologic Structure

Based on our experience, geologic reconnaissance-level mapping, and review of published geologic literature (Appendix A), the Tertiary-aged (Early to Middle Eocene) Torrey Sandstone and Delmar Formation are the primary geologic units anticipated to be encountered along the proposed tunnel alignments. The Torrey Sandstone is massive and broadly cross-bedded. The Delmar Formation contains occasional randomly orientated fissures, shearing, and jointing. The mapped bedding within these formational materials generally exhibits variable dips typically horizontal to less than 5 degrees in a generally southeasterly to northeasterly direction. Mapped geologic structure is considered in our assessments given on the Table of Stability Elements and Constraints (Appendix B).

The Torrey Sandstone and Delmar Formation are shoreline deposits related to time-transgressive units that were deposited in an interface zone between marine and terrestrial environments. These two deposits represent shallow shelf marine formational units that interfinger extensively generally over a vertical distance of roughly 30 feet, therefore strata that are clearly of Delmar character can and do appear in the Torrey Sandstone. The Torrey Sandstone's depositional environment consists of nearshore sand dunes and outwash tidal channels, while the Delmar Formation's depositional environment consists of lagoon and estuarine deposits, tidal flats, and sublittoral tidal channels (Wilson, 1972). Therefore, the contact between the Torrey Sandstone and the Delmar Formation should not be considered distinct, but rather highly gradational. Based on our experience, geologic reconnaissance-level mapping, and review of published geologic literature (Appendix A), a transitional facies change between the Torrey Sandstone and the Delmar Formation may be present over a 30-foot sequence with an overall slight dip to the east.

4.3 Review of Aerial Photographs and Topographic Maps

We performed a review of topographic maps covering the site area, with the oldest being from the National Oceanic and Atmospheric Administration (NOAA) from the 1890's. In general, the maps showed relatively uniform topographic contours, several canyon areas that have incised the formational units overlying the proposed tunnel alignments, and there was no indication of geomorphic topographic features characteristic of faulting or other potential geologic hazard-related features. Also, we performed stereoscopic analysis of aerial photographs for assisting in the geologic interpretation. With regard to topographic and aerial photograph morphology, we did not observe linear topographic expressions that are characteristic of fault or other potential geologic hazard-related features at the project alignment. The lack of topographic relief at the known fault locations indicates that the age of faulting predates the deposition of Quaternary-aged formations along the tunnel alignments.

4.4 Material Engineering Characteristics

The geologic units described above have unique engineering characteristics including erodibility, expansion potential, corrosivity, excavation difficulty, and slope stability. These characteristics are briefly discussed below and presented in the table of Material Engineering Characteristics in Appendix B for various locations within the proposed tunnel alignments.

4.4.1 Erodibility

Erodibility of the geologic units is a function of cohesion, cementation, moisture content, and degree of weathering of the material where exposed at the surface. In general, due to the depth of the tunnel alignments, the formational materials which are anticipated along the proposed tunnel alignments are considered to be very resistant to erodibility. However, the surficial formational units located near the southern proposed tunnel portals and the alluvial materials underlying the proposed northern tunnel portals may be very sandy, contain locally friable zones, are moderately weathered, and can be highly erodible if left unprotected. In addition, occasional randomly orientated fissures and jointing zones which are susceptible to fracturing as a result of tensile stresses may be encountered within the Delmar Formation. Where the tunnel alignments cross through the Delmar Formation, potential failures as a result of existing sheared claystone, fissures, and jointing should be accounted for during the design phase of the subject project.

4.4.2 Expansion Potential

Based on our experience with sites located adjacent to the proposed tunnel alignments, the formational units identified as Old Paralic Deposits, Very Old Paralic Deposits, and the Torrey Sandstone are generally considered to be granular in nature and are anticipated to have a very low to low expansion potential. However, the Delmar Formation is derived from ancient marine, estuarine, and lagoonal deposits which are generally fine-grained in nature and can be highly expansive. Where the proposed tunnel alignments cross through the Delmar Formation, the potential for adverse effects from expansive soils should be accounted for during the design phase of the subject project.

4.4.3 Corrosivity

The active tectonics of southern California resulted in uplift of coastal land. Therefore, the marine deposits in these coastal lands contain chloride and sulfate ions, and the proposed tunnels can be exposed directly to these marine deposits. Chloride and sulfate ions can leach out of the marine deposits and into the groundwater, and the groundwater can also expose the tunnels to these ions.

With the proposed tunnel alignments crossing through marine deposits, their general proximity to the Pacific Ocean, and their potential susceptibility to encountering groundwater seepage and/or static groundwater table conditions, we recommend that a corrosion engineer be retained during the design phase to provide corrosion assessment of the subject project. In addition, the tunnels should be designed and constructed in accordance with the guidance of the Services Life Design Guide for Corrosion Prevention of Concrete Structures in San Diego County (SANDAG, 2015).

4.4.4 Compressible Soils

Based on our mapping and document research, thick young alluvial deposits associated with the San Dieguito River drainage are present at the northern proposed tunnel portal locations. Specifically, these young alluvial deposits are located below the proposed Crest Canyon and Camino Del Mar High Speed tunnel alignments from approximately Station 19+00 to 27+00. These young alluvial deposits are generally weak in structure and are considered to be potentially compressible in their natural state. Structural loads imposed on compressible soil materials could result in adverse settlement. The alluvial deposits located within the limits of the proposed tunnel alignments should be further evaluated with additional detailed geotechnical studies including subsurface exploration.

4.4.5 Excavation Difficulty

The proposed tunnel alignments are located within sedimentary rock which is generally very dense and hard in nature. In addition, variations in excavatability exist, resulting from localized cementation (concretions) of the sandstone, siltstone, claystone, or conglomerate units. The generalized excavation difficulty is given in the table of Material Engineering Characteristics (Appendix B). As previously discussed, the geologic contact between the Torrey Sandstone and the Delmar Formation should not be considered to be vertically distinct, but rather highly gradational and non-uniform. Therefore, variation in tunnel excavation rates at this transitional depositional facies change between the Torrey Sandstone and Delmar Formation should be anticipated. In addition, groundwater seepage should be anticipated within this transitional facies change, creating the potential for caving and/or sloughing conditions.

4.4.6 Stability

The earth materials found along the tunnel alignments will display variation in natural stability resulting from a variety of factors, including the in-situ shear strength of the material, the moisture content or presence of subsurface seepage, and geologic structure including localized fracture or bedding surfaces. The generalized stability characteristics for the above earth materials are provided in the table of Material Engineering Characteristics (Appendix B). These general characteristics, along with local variables, are also incorporated into our excavation stability assessments given in the table of Stability Elements and Constraints (Appendix B).

4.4.7 Landslides

During our site reconnaissance, we observed a minor slope failure along a western-facing coastal bluff located approximately 400 feet north of the intersection of Jimmy Durante Blvd and Camino Del Mar. The block failure appears to be located within the Delmar Formation and may have occurred where irrigation and/or storm water has infiltrated into unprotected fissures or joints. Based on our experience and review of published geologic literature (Appendix A), this surficial slope instability is localized and strongly influenced by the presence of very steep slope surfaces. Deep global instability is not considered a hazard at this location, although structural stabilization measures including the construction of a soldier pile retaining wall have been recently performed in the area of the block failure. As-built plans for the soldier pile retaining wall should be reviewed to verify that the new structural improvements do not conflict with the design and construction of the proposed tunnel alignments.

5.0 SEISMIC SETTING

5.1 Faulting

The site is located within the Peninsular Ranges Geomorphic Province, which is traversed by several major Holocene-active faults. The Whittier-Elsinore, San Jacinto, and the San Andreas faults are major Holocene-active fault systems located east of the proposed tunnel alignments, and the Rose Canyon, Newport-Inglewood (offshore), and Coronado Bank are Holocene-active faults located west to southwest of the proposed tunnel alignments (Figure 2, Regional Fault Map). The primary seismic risks to the project area is the Rose Canyon Fault Zone located approximately 1.5 miles west of the subject project (USGS, 2008).

5.1.1 Fault Classification

The State Mining and Geology Board (SMGB) has defined a Holocene-active fault as a fault which had surface displacement within Holocene time (about the last 11,700 years). In addition, a pre-Holocene fault is a fault that has not shown displacement in the past 11,700 years, and does not meet the criteria of “Holocene-active fault” as defined in the Alquist-Priolo Earthquake Fault Zoning (AP) Act and SMGB regulations.

These definitions are used in delineating Special Studies Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972. The intent of this law is to assure that unwise urban development does not occur across the traces of Holocene-active faults. As background, Special Publication (SP) 42 was provided for guidance with regard to informing reviewers and practitioners on the locations of the fault rupture hazard zones in California, and was subsequently revised several times between 1976 and 2007. The most recently adopted revision of the document was completed in 2018 and resulted in a significant change from previous versions, as it now provides guidelines (previously included as supplements for SP 42) for both reviewers and practitioners working in Earthquake Fault Zones (EFZ). Based on our review of the State of California updated EFZ maps for San Diego in 2021 for the Point Loma and La Jolla Quadrangles, the site is not located within a EFZ. In addition, the proposed tunnel alignments are not located within a fault rupture hazard zone or within 1,000 feet of an active fault (15,000 years and younger); therefore, following Caltrans guidelines (Caltrans, 2017), further evaluation to investigate for surface fault rupture is not required. No changes that would affect the activity of faulting at the subject site are proposed.

5.1.2 Rose Canyon Fault Zone

The Rose Canyon fault zone consists predominantly of right-lateral strike-slip faults that extend south-southeast bisecting the San Diego metropolitan area. Various fault strands display strike-slip, normal, oblique, or reverse components of displacement. The Rose Canyon fault zone extends offshore at La Jolla and continues north-northwest subparallel to the coastline. The offshore segments are poorly constrained regarding location and character. South of downtown, the fault zone splits into several splays that underlie San Diego Bay, Coronado, and the ocean floor south of Coronado (Treiman, 1993 and 2000; Kennedy and Clarke, 1999). Portions of the fault zone in the Mount Soledad, Rose Canyon, and downtown San Diego areas have been designated by the State of California (CGS, 2003) as being Earthquake Fault Zones.

5.1.3 Unnamed Fault

As depicted on Plates 1 and 2, a northeast trending pre-Holocene fault crosses the Camino Del Mar tunnel alignment at approximately Station 125+00 to 126+00 and crosses the Crest Canyon High Speed at approximately Station 111+00 to 112+00. Based on regional geologic mapping and our geologic reconnaissance, this unnamed fault generally indicates an average trend of N40E, dipping approximately 65 degrees to the northwest, and does not appear to transect the younger overlying Quaternary-age deposits. Therefore, this unnamed fault is considered to be pre-Holocene in age and not active. Faulted, sheared bedrock, and seepage should be anticipated and accounted for where the unnamed fault crosses the tunnel alignments.

5.2 Seismic Hazards

Severe ground shaking is most likely to occur during an earthquake on one of the regional Holocene-active faults in Southern California. The effect of seismic shaking may be mitigated by adhering to the applicable design codes and state-of-the-art seismic design practices. Secondary effects associated with severe seismic ground shaking which may affect the site includes shallow ground rupture, soil liquefaction and dynamic settlement, seiches, and tsunamis. These secondary effects of seismic shaking are discussed below.

5.2.1 Shallow Ground Rupture

For the proposed tunnel alignments, no Holocene-active faults are mapped crossing the tunnels and the alignments are not located within a mapped

Alquist-Priolo EFZ (CGS, 2021). Ground rupture due to faulting is not considered a significant hazard in these areas, although it may be considered a hazard throughout San Diego County.

5.2.2 Liquefaction and Seismic Settlement

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confining pressure, depth to groundwater, and the intensity and duration of the seismic ground shaking. The cohesionless soils most susceptible to liquefaction are loose, saturated sands and non-plastic silts.

It is anticipated that a majority of the proposed tunnel alignments will be founded in sedimentary rock units which are considered to be very dense to hard in nature. It is our opinion that the potential for liquefaction and seismic settlement below the tunnel alignments within the sedimentary rock units can be considered to be very low. However, based on our reconnaissance-level mapping and our review of the Geologic and Fault Map (Plate 1), the proposed northern tunnel portal alignments are located within the mapped limits of potentially liquefiable young alluvial materials which are associated with the San Dieguito River drainage. Specifically, these young alluvial deposits are located below the proposed Crest Canyon and Camino Del Mar High Speed tunnel alignments from approximately Station 19+00 to 27+00. It is anticipated that these young alluvial materials located below the static groundwater table are potentially subject to localized liquefaction and seismic settlement and should be further evaluated with additional detailed geotechnical studies including subsurface exploration.

5.2.3 Lateral Spreading

Lateral spreading can occur when saturated alluvial materials overlain by sloping ground liquefy and cause a reduction of lateral resisting force. Lateral spreading is manifested by lateral displacement and slumping of the embankment. Empirical relationships have been derived (Youd et al., 1999) to estimate the magnitude of lateral spread due to liquefaction. These relationships include parameters such as earthquake magnitude, distance of the earthquake from the site, slope height and angle, the thickness of liquefiable soil, and gradation characteristics of the soil.

Due to the deep nature of the proposed tunnel alignments and the plan for a majority of the tunnel alternative alignments to be founded in sedimentary rock units which are considered to have a low potential for liquefaction, the potential for lateral spreading or flow failure is considered to be very low in these areas. Although, based on our reconnaissance-level mapping and our review of the Geologic and Fault Map (Plate 1), the proposed northern tunnel portal alignments are located within mapped limits of potentially liquefiable young alluvial materials which are associated with the San Dieguito River drainage. Specifically, these young alluvial deposits are located below the proposed Crest Canyon and Camino Del Mar High Speed tunnel alignments from approximately Station 19+00 to 27+00. It is anticipated that these young alluvial materials are potentially subject to lateral spreading or flow failure and should be further evaluated with additional detailed geotechnical studies including subsurface exploration.

5.2.4 Tsunamis

A tsunami is a sea wave generated by submarine earthquakes, landslides or volcanic activity that displaces a relatively large volume of water in a very short period. Several factors at the originating point such as earthquake magnitude, type of fault, depth of earthquake, focus, water depth, and the ocean bottom profile all contribute to the size and momentum of a tsunami (Iida, 1969). Factors such as the distance away from the originating point, coastline profile (including width of the continental shelf), and angle at which the tsunami approaches also affect the size and severity of a tsunami.

Southern California is not only favorably oriented (i.e., not directly in line with any of the major originating tsunami zones), it has a relatively wide (about 140 miles) and rugged continental shelf or borderland, which acts as a diffuser and reflector of remotely, generated tsunami wave energy (Joy, 1968). In addition, the existing geologic and seismic hazard conditions (such as the abundance of strike-slip faults, and the scarcity of large submarine earthquakes) along the coastline also tend to minimize the likelihood of a localized tsunami.

Based on experience with remotely generated tsunami and the favorable geologic and seismic conditions along the coastline, there is little potential for catastrophic damage along the San Diego County coastline. Based on our review of Tsunami Inundation Map (CGS, 2022) of the Del Mar Quadrangle, the southern proposed tunnel portals are not mapped in inundation areas. In addition, the southern portals are mapped as being founded in competent sedimentary bedrock that is protected from coastal influences. Therefore, the potential for damage due to a tsunami along the southern proposed tunnel portals is considered to be low. The northern proposed tunnel portals are located slightly within the mapped tsunami inundation areas. In addition, the northern proposed tunnel portals are located within the limits of mapped young alluvial materials which are generally considered not to be well protected from coastal influences and are considered to be potentially subject to damage from a tsunami.

6.0 GEOLOGIC EVALUATION

6.1 Stability Elements and Constraints

The following issues are quantified along the proposed tunnel alignments, as shown along the Generalized Geologic Cross-Sections (Plate 2). The table of Stability Elements and Constraints presented in Appendix B provides description of conditions for correlation. A series of values, ranging between 0 and 4, with a corresponding description of the expected condition, are assigned to each constraint category. In general, a value of 0 indicates negligible concern, while a higher value is assigned when more adverse conditions are expected.

It should be understood that the ranking of the individual site factors is largely based on our professional engineering and geologic judgements of the conditions inferred from review of available geotechnical literature and limited site observations. Detailed geotechnical investigations of the proposed tunnel alignments and/or improvements should be performed as part of future studies.

6.1.1 Excavation Stability Formational Materials

There is always a risk of excavation failures during any tunnel project. However, based on our experience and reconnaissance level-mapping, the sedimentary formational materials located along the proposed tunnel alignments may generally be accomplished with conventional heavy-duty earthwork equipment. We anticipate that the highest risk of temporary instability will likely be at the tunnel portals and where the pre-Holocene unnamed fault crosses the tunnel alignments. In addition, where the Torrey Sandstone and Delmar Formation may have localized zones of concretions, gravel/cobble, and friable sands. These zones may require specialized shoring and/or tunnel excavation techniques. In addition, as previously discussed, both proposed tunnel alignments will be located near the geologic contact between the Torrey Sandstone and the Delmar Formation. As previously discussed, the geologic contact between the Torrey Sandstone and the Delmar Formation should not be considered to be distinct, but rather highly gradational and non-uniform. Therefore, variation in tunnel excavation rates at this transitional facies change between the Torrey Sandstone and Delmar Formation should be anticipated. In addition, groundwater seepage within this transitional facies change may be encountered creating potential for caving and/or sloughing conditions.

6.1.2 Excavation Stability Alluvium

We anticipate that the highest risk of temporary instability will likely be encountered during the excavation of the northern portal u-structures which will likely utilize conventional open cut excavation techniques. The portal structures are planned within poorly consolidated and potentially compressible young alluvial deposits with shallow groundwater conditions. Excavation activities within these alluvial deposits will require specialized dewatering and shoring techniques that can mitigate potential basal heave and flowing sand conditions. The effects of dewatering to induce settlement of nearby improvements will also need to be considered in excavation design.

6.1.3 Groundwater and Seepage

Two types of groundwater are expected along the proposed tunnel alignments are addressed in this report. The first includes perched water (seepage), commonly found within 10 to 30 feet of the existing ground surface. This groundwater (i.e., resulting from irrigation and precipitation) infiltrating through the sandy terraces, then becomes perched on or within the less porous and denser underlying sedimentary formational units. Perched groundwater is generally expected along the geologic contact between the Quaternary-aged Paralic Deposits and the Tertiary-aged Torrey Sandstone. Also, perched groundwater is generally expected along the geologic contact between the Quaternary-aged Old Paralic Deposits and the Tertiary-aged Delmar Formation near the tunnel portals and within the transitional facies change between the Torrey Sandstone and Delmar Formation, which was previously discussed in Section 4.1. Perched groundwater should be anticipated within faults that intercept the tunnel alignments.

The second variety of groundwater includes the static groundwater table which is generally correlated to fluctuating sea level elevations. Based on the results of our geologic reconnaissance and our experience in the site area, groundwater elevation is anticipated to be approximately +5 feet above mean sea level (msl). In addition, to account for tidal influences and potential sea level rise, design consultants should consider a range of sea level rise from 1.4 to 5.5 feet, or the latest guidance from State or Federal regulations, based on the LOSSAN Corridor Design Criteria Volume III (SANDAG, 2017). Also, shallow groundwater is anticipated within the alluvial deposits present within the San Dieguito River drainage located at the proposed northern tunnel portals. The level of groundwater within the alluvial deposits is anticipated to vary seasonally and may be assumed to be at or above the ground surface during flooding events.

6.1.4 Liquefaction/Dynamic Settlement

It is anticipated that a majority of the proposed tunnel alignments will be founded in sedimentary rock units which are considered to be very dense to hard in nature. It is our opinion that the potential for liquefaction and seismic settlement below the tunnel alignments that cross through the sedimentary rock units can be considered to be very low. Although, based on our reconnaissance-level mapping and our review of the Geologic and Fault Map (Plate 1), the proposed northern tunnel portal alignments are located within mapped limits of potentially liquefiable young alluvial materials which have a shallow groundwater table associated with the San Dieguito River drainage. Specifically, these young alluvial deposits are located below the proposed Crest Canyon and Camino Del Mar High Speed tunnel alignments from approximately Station 19+00 to 27+00. It is anticipated that these young alluvial materials located below the static groundwater table are potentially subject to localized liquefaction and seismic settlement and should be further evaluated with additional detailed geotechnical studies including subsurface exploration.

6.1.5 Adjacent Property Impacts

In general, the proposed tunnel alignments and portals are expected to impact existing properties. In areas where temporary excavations could impact offsite areas, special recommendations should be provided to mitigate for potential instability. As with all excavations, the stability of the offsite area cannot be guaranteed, nor one's maintenance of existing structures are always feasible. However, with proper engineering, the risk and area of impact can be reduced to tolerable levels.

Structural stabilization measures that have been previously constructed within adjacent properties may conflict with the design and construction of the proposed tunnel alignments. In addition, deep excavations and specialized dewatering systems may be needed where the tunnel alignment and/or portals cross through the poorly consolidated and potentially settlement sensitive young alluvial deposits. In general, further site specific geotechnical investigations are required to properly assess potentially unstable offsite areas such as those underlain by weak bedrock materials, young alluvium, and construction related issues. The installation of monitoring devices such as slope inclinometers may be appropriate where excavations are proposed adjacent to and below existing improvements.

7.0 CONCLUSIONS

Based on our geologic reconnaissance study of the site, it is our opinion that development of the proposed tunnel alignments is feasible from a geologic and geotechnical standpoint. However, detailed geotechnical studies including subsurface exploration and laboratory testing will be needed to provide more specific design-level recommendations. The following is a summary of the significant geological factors that should be considered during the feasibility analysis and/or design of proposed project.

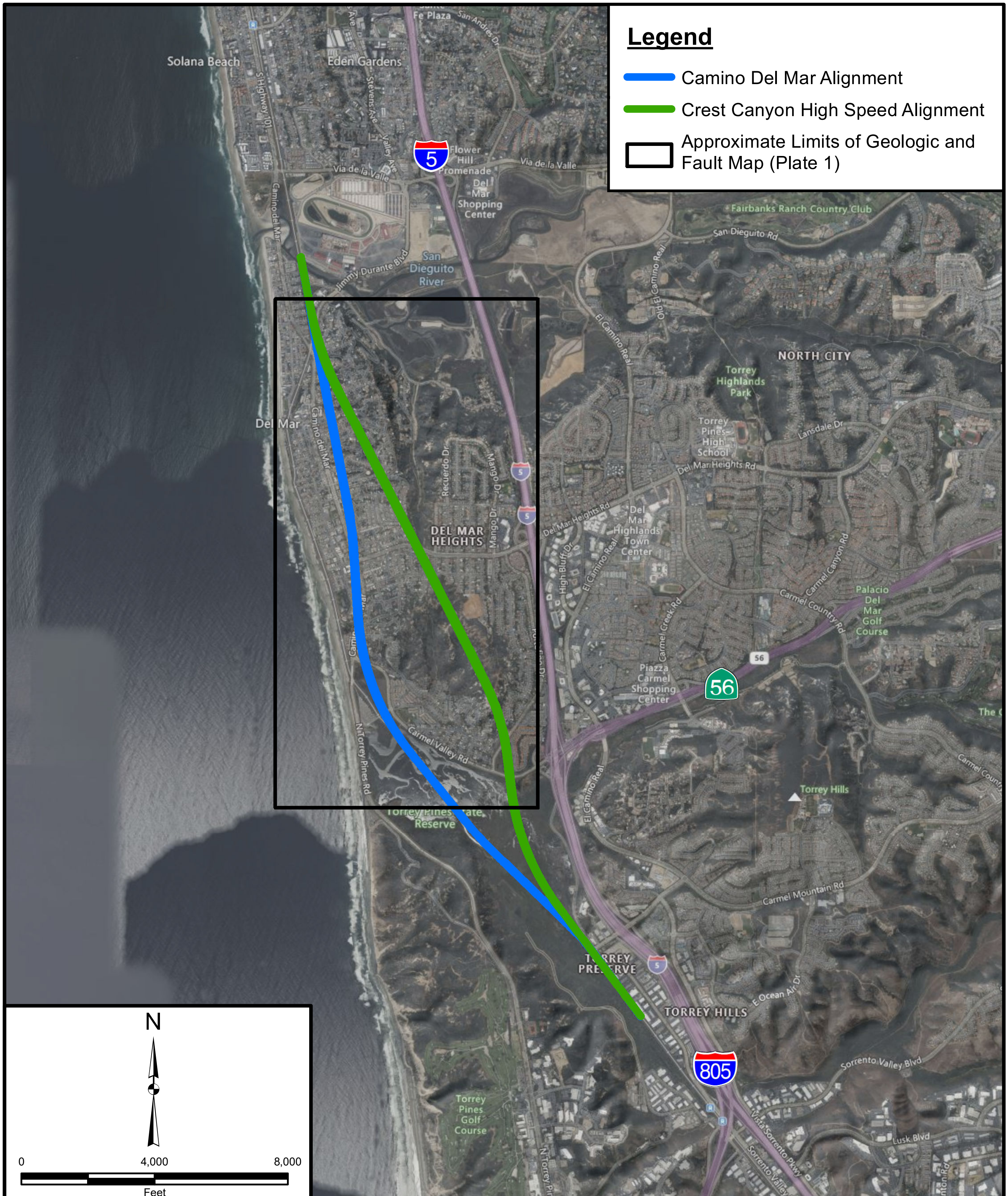
- As the project site is located in the seismically active southern California area, all structures should be designed to tolerate the dynamic loading resulting from seismic ground motions.
- The proposed tunnel alignments are not located within a fault rupture zone or within 1,000 feet of an active fault (15,000 years and younger). Therefore, following Caltrans guidelines (Caltrans, 2017), further evaluation to investigate for surface fault rupture is not required.
- Both the Camino Del Mar and Crest Canyon High Speed tunnel alignments are transected by a northeast trending pre-Holocene fault. Faulted and sheared sedimentary formational units should be anticipated and accounted for where this fault crosses the tunnel alignments.
- Based on our experience and geologic reconnaissance, the sedimentary formational units along the proposed tunnel alignments should be generally excavatable with conventional heavy-duty earthwork equipment. However, localized zones of concretions, gravel/cobble, and friable sands may require specialized shoring and/or tunnel excavation techniques.
- Based on the results of our geologic reconnaissance, the Camino Del Mar and Crest Canyon High Speed tunnel alignments are located directly along the highly gradational and interfingered geologic contact between the Torrey Sandstone and Delmar Formation. Therefore, tunnel design should account for transecting variable sandstone, siltstone, and claystone facies during excavation operations.
- The young alluvial deposits and existing undocumented fill materials located below the proposed Crest Canyon and Camino Del Mar High Speed tunnel alignments from approximately Station 19+00 to 27+00 are generally weak and considered potentially compressible in their natural state. Structural loads imposed on compressible soils could result in adverse settlement. The alluvial deposits located within the limits of the proposed tunnel alignments should be further evaluated with additional detailed geotechnical studies including subsurface exploration.

- The young alluvial deposits associated with the San Dieguito River drainage are located below the proposed Crest Canyon and Camino Del Mar High Speed tunnel alignments from approximately Station 19+00 to 27+00. It is anticipated that the young alluvial materials located below the static groundwater table are potentially subject to localized liquefaction, seismic settlement, and lateral spreading and should be further evaluated with additional detailed geotechnical studies including subsurface exploration.
- The northern proposed tunnel portals are located slightly within the boundary of the mapped tsunami inundation areas. In addition, the northern proposed tunnel portals are located within the limits of mapped young alluvial materials which are generally considered not to be well protected from coastal influences and are considered to be potentially subject to damage from a tsunami.
- It is anticipated that the static groundwater table will be encountered during tunnel excavation activities from Station 19+00 to 70+00 along both proposed tunnel alignments. In addition, localized seepage along dense zones and sand lenses within the Old Paralic Deposits or at the geologic contact with the Delmar Formation is anticipated. At the proposed tunnel elevations, the geologic contact between the Torrey Sandstone and Delmar Formation may have seepage conditions, and seepage is possible at the location of the unnamed fault which crosses both tunnel alignments.

8.0 LIMITATIONS

The geologic data provided in this report is by necessity incomplete. The information presented is primarily intended for planning purposes is not meant to be definitive data for specific sites. Detailed geotechnical studies including further subsurface exploration, laboratory testing, stability analyses, are appropriate for specific, detailed alignment designs. It should be understood that the ranking of the individual site factors is in a large part based on our engineering and geologic judgements based on limited information, acquired during or review of geologic literature including available geologic maps and aerial photographs, and limited field mapping.

FIGURES AND PLATES



Project: 13682.001	Eng/Geol: CA/RNB
Scale: 1 " = 4,000 '	Date: October 2022
Base Map: Bing Maps 2022	
Author: Leighton Geomatics (mmurphy)	

SITE LOCATION MAP

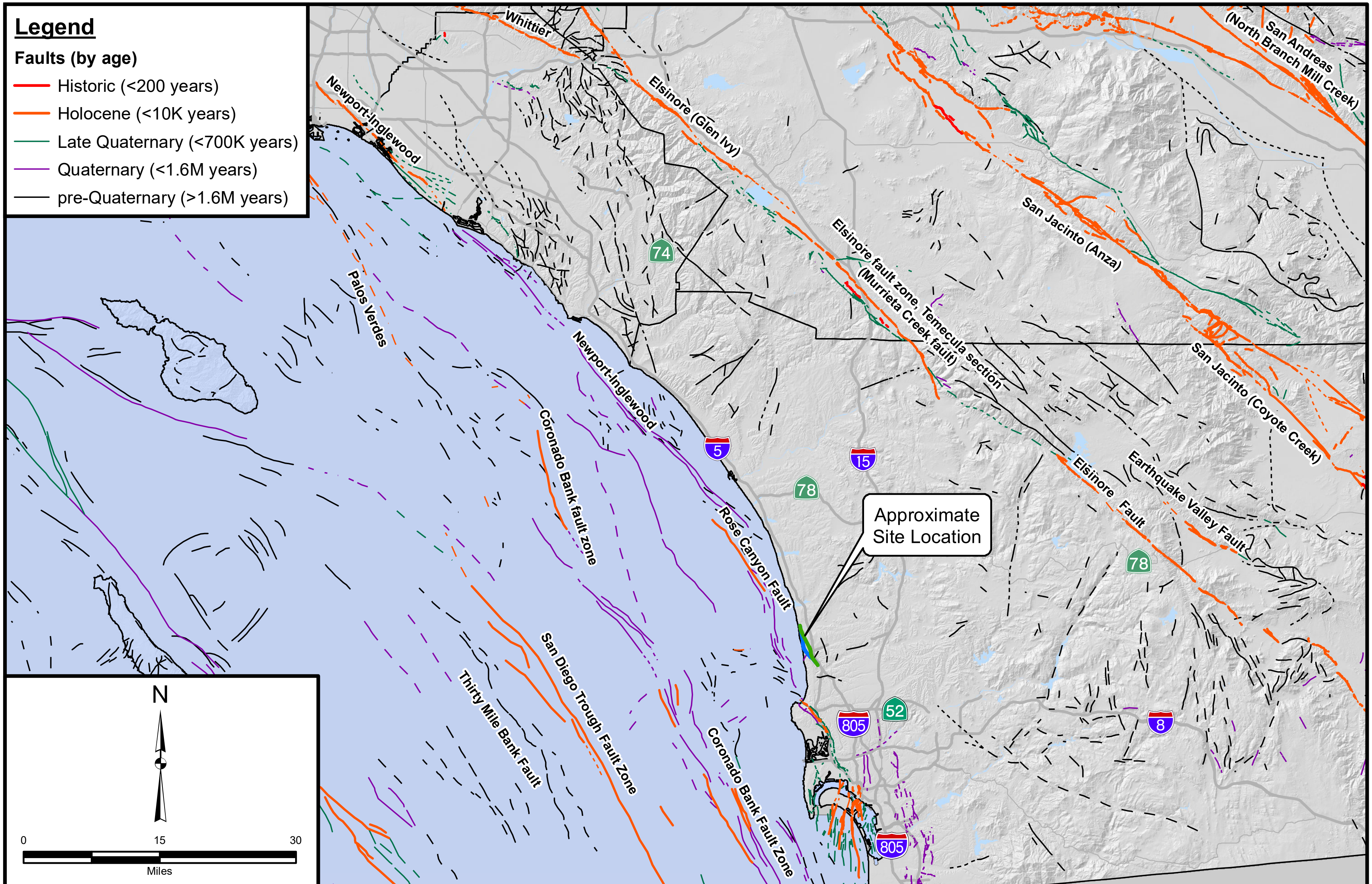
Del Mar Alternative Tunnel Alignments
Conceptual Engineering Study
San Diego Regional Rail Corridor Alternative Alignment and Improvements Project
San Diego County, California

FIGURE 1

Leighton

Map Saved as V:\Drafting\13682\001\Maps\Delmar AA\13682-001_F01_SLM_2022-10-10.mxd on 10/10/2022 12:15:48 PM

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.



Project: 13682.001	Eng/Geol: CA/RNB
Scale: 1 " = 15 miles	Date: October 2022

Faults: Bryant, Bryant CGS 2010

Author: Leighton Geomatics (mmurphy)

REGIONAL FAULT MAP

Del Mar Alternative Tunnel Alignments
Conceptual Engineering Study
San Diego Regional Rail Corridor Alternative
Alignment and Improvements Project
San Diego County, California

FIGURE 2

Leighton

Map Saved as V:\Drafting\13682\001\Maps\Delmar AA\13682-001_F02_RFM_2022-10-10.mxd on 10/10/2022 3:16:24 PM

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

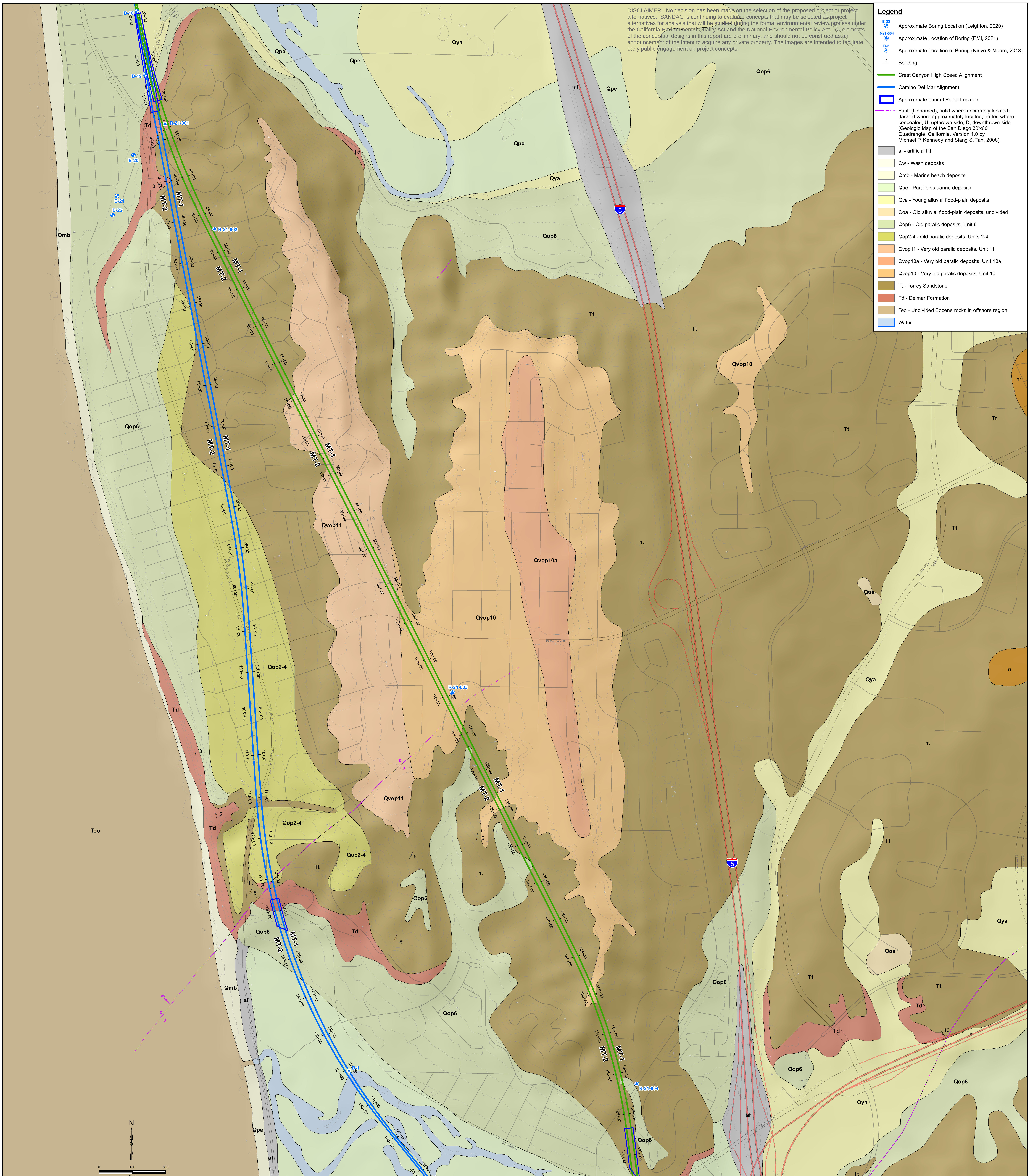


PLATE 1	GEOLOGIC AND FAULT MAP
Scale: 1 " = 400 feet	Del Mar Alternative Tunnel Alignments Conceptual Engineering Study San Diego Regional Rail Corridor Alternative Alignment and Improvements Project San Diego County, California
Date: October 2022	
Proj: 13682.001	Leighton
Eng/Geol: CA/RNB	

Base Map: Base map: Kennedy, M.P. and Tan, S.S, 2008, Geologic Map of the San Diego 30' X 60' Quadrangle, California Compiled by Michael P. Digital Preparation by Kelly R. Bovard, Anne G. Garcia and Diane Burns, California Geological Survey.

CAMINO DEL MAR ALIGNMENT (MT-1)

CREST CANYON HIGH SPEED ALIGNMENT (MT-1)

Legend

B-20 T.D.=21.5'	Approximate Boring Location Showing Total Depth (Leighton, 2020)
R-21-004 T.D.=120'	Approximate Boring Location Showing Total Depth (EMI, 2022)
B-1 T.D.=120.5'	Approximate Boring Location Showing Total Depth (Ninyo & Moore, 2013)
	Proposed Camino Del Mar Track-Grade along Alignment (MT-1)
	Proposed Crest Canyon High Speed Track-Grade along Alignment (MT-1)
	Approximate Regional Groundwater Table
—?—	Approximate Geologic Contact (dashed where queried)
	Approximate Location of Unnamed Fault
Camino Del Mar Subsurface Tunnel Alignment Station 125+00	Text Box Stability Elements and Constraints are Detailed in Appendix B

GEOLOGIC UNITS

Qpe	Quaternary-aged Paralic Estuarine Deposits
Qya	Quaternary-aged Young Alluvium
Qop_6	Quaternary-aged Old Paralic Deposits (Unit 6)
Qop_{2-4}	Quaternary-aged Old Paralic Deposits (Units 2-4)
$Qvop_{11}$	Quaternary-aged Very Old Paralic Deposits (Unit 11)
$Qvop_{10}$	Quaternary-aged Very Old Paralic Deposits (Unit 10)
Tt	Tertiary-aged Torrey Sandstone
Td	Tertiary-aged Delmar Formation

80 ft
VERTICAL
0 ft
0 ft 400 ft
HORIZONTAL
APPROXIMATE GRAPHIC SCALE

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

GENERALIZED GEOLOGIC CROSS-SECTIONS	PLATE 2
Del Mar Alternative Tunnel Alignments Conceptual Engineering Study San Diego Regional Rail Corridor Alternative Alignment and Improvements Project San Diego County, California	Scale: H: 1"=400' V: 1"=80'
	Date: October 2022
Leighton	Proj: 13682.001
	Eng/Geol: CA/RNB
Base Map:	

APPENDIX A
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REFERENCES

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Aerial Photographs

Photo Number	Date	Source	Scale
AXN 8M 7, 8, 9, and 10	4/11/1953	USDA	1:24,000

APPENDIX B
TABLES TO ACCOMPANY GENERALIZED GEOLOGIC CROSS-SECTIONS

APPENDIX B

TABLES TO ACCOMPANY GENERALIZED GEOLOGIC CROSS-SECTIONS

Material Engineering Characteristics								
Map Symbol(s)		**Geologic Map Unit**	**Erodibility**	**Expansion Potential**	**Corrosivity**	**Excavation Difficulty**	**Slope Stability**
Soil Units	Qya	Young Alluvium	Moderate	Low (Locally Medium to High)	Negligible	Easy	Poor
Sedimentary Formational Units	Qop_6, Qop_{2-4}	Old Paralic Deposits	Moderate	Very Low to Low	Negligible	Easy to Moderate	Poor to Fair (Where Consolidated)
	$Qvop_{10}$, $Qvop_{11}$	Very Old Paralic Deposits	Moderate to Very Resistant	Low to Very Low (Locally Very High)	Negligible	Moderate to Difficult	Fair to Very Good
	Tt	Torrey Sandstone	Moderate to Resistant	Low (Locally Medium to High)	Negligible	Moderate to Difficult	Good to Very Good
	Td	Delmar Formation	Moderate	Medium to High	Negligible to Severe	Moderate	Fair to Very Good

APPENDIX B (CONTINUED)

Stability Elements and Constraints		
Geotechnical Constraint	**Value**	**Description of Expected Condition**
Soil Characteristics	0	Adverse soil conditions are not expected. Potential for expansive soils from the tunnel excavation soil cuttings is considered negligible.
	1	Localized zones within the sedimentary formational units may contain soil cutting materials that are considered to be potentially expansive material.
	2	The sedimentary formational units are considered to contain soil cutting materials that have a high to very high expansion potential.
	3	The alluvial materials are considered to be weak and potentially compressible in their natural state and may also contain soil materials that have a high to very high expansion potential.
Temporary Excavation Stability	0	No significant subsurface excavation is proposed.
	1	The proposed excavation is expected to be within competent, cohesive materials. Adverse geologic structure or groundwater/seepage is not expected.
	2	Potentially adverse geologic structure and/or seepage may contribute to temporary instability. Mitigative measures may be required.
	3	Adverse geologic structure will necessitate in-construction measures to improve temporary stability. Excavation into faulted bedrock material exposes fractures and shears which contribute to excavation instability as well as destabilized surrounding areas without mitigative measures.
	4	High groundwater, or heavy seepage and saturated materials are expected within the proposed excavation, contributing to excavation instability.

APPENDIX B (CONTINUED)

Stability Elements and Constraints (Continued)		
Geotechnical Constraint	**Value**	**Description of Expected Condition**
Groundwater	0	Groundwater conditions are not expected to adversely impact the proposed design.
	1	Minor to moderate perched groundwater is possible. Drainage measures beyond conventional designed mitigative measures may be necessary.
	2	Moderate to heavy perched groundwater is possible. Drainage measures beyond conventional designed mitigative measures may be necessary.
	3	Locally high groundwater is expected. Dewatering measures may be required.
Liquefaction	0	Liquefiable materials are not expected beneath the proposed tunnel alignments.
	1	Potentially liquefiable materials may underlie the proposed tunnel alignments. Further investigation or mitigation is recommended.
Adjacent Property Impacts	0	The proposed portals and tunnel alignments are located outside of the influence of existing structures.
	1	The proposed portals and tunnel alignments abuts existing development. However, construction of the proposed design is expected to be feasible with minor impacts to the adjacent development.
	2	The proposed portals and tunnel alignments abuts existing development. Construction of the proposed design is expected to impose on the existing structures within the planned excavation footprint.
	3	The proposed portals and tunnel alignments abuts existing development, which is located upon potentially unstable geology. Construction of the proposed design, including remedial grading measures is expected to impose on these existing structures. Monitoring and other safeguards, including relocation, may be appropriate.

APPENDIX C
GEOTECHNICAL BORING LOGS

EMI, 2022

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names	Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])
WA	Wash Analysis (ASTM D 1140-97)

SAMPLER GRAPHIC SYMBOLS

Standard Penetration Test (SPT)

Standard California Sampler

Modified California Sampler

Shelby Tube — Piston Sampler

NX Rock Core — HQ Rock Core

Bulk Sample — Other (see remarks)

DRILLING METHOD SYMBOLS

Auger Drilling — Rotary Drilling — Dynamic Cone or Hand Driven — Diamond Core

WATER LEVEL SYMBOLS

First Water Level Reading (during drilling)

Static Water Level Reading (short-term)

Static Water Level Reading (long-term)

Earth Mechanics, Inc.

Geotechnical and Earthquake Engineering

BORING RECORD LEGEND

San Diego Regional Rail Corridor

Project Number: 20-134	Date: 3-25-21	SHEET 1 of 3

CONSISTENCY OF COHESIVE SOILS				
Descriptor	**Unconfined Compressive Strength (tsf)**	**Pocket Penetrometer (tsf)**	**Torvane (tsf)**	**Field Approximation**
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS	
Descriptor	**SPT N_{60} - Value (blows / foot)**
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE	
Descriptor	**Criteria**
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS	
Descriptor	**Criteria**
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE		
Descriptor		**Size**
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS	
Descriptor	**Criteria**
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION	
Descriptor	**Criteria**
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE:
This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010 Edition), Section 2, for tables of additional soil description components and discussion of soil description and identification.

REF = Refusal; During drilling seating interval (first 6-inch interval) is not achieved.

ROCK GRAPHIC SYMBOLS

- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

BEDDING SPACING

Descriptor	Thickness or Spacing
Massive	> 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	3-5/8 inches to 1 ft
Thinly bedded	1-1/4 to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		
Descriptor	Body of Rock	Fracture Surfaces		Texture	Solutioning	General Characteristics
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation (refer to grain boundary conditions)	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Altered by chemical disintegration such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

Note: Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors should not be used where significant identifiable zones can be delineated. Only two adjacent descriptors shall be combined. "Very intensely weathered" is the combination descriptor for "decomposed to intensely weathered".

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 - 30,000
Strong	7,000 - 14,500
Medium Strong	3,500 - 7,000
Weak	700 - 3,500
Very Weak	150 - 700
Extremely Weak	< 150

ROCK HARDNESS

Descriptor	Criteria
Extremely Hard	Specimen cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Specimen cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Specimen can be scratched with pocket knife or sharp pick with heavy pressure; heavy hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/6 in. with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Specimen can be grooved or gouged with pocket knife or sharp pick with light pressure, breaks with light to moderate hand pressure
Very Soft	Specimen can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

CORE RECOVERY CALCULATION (%)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

RQD CALCULATION (%)

$$\frac{\Sigma \text{ Length of intact core pieces} > 4 \text{ in.}}{\text{Total length of core run (in.)}} \times 100$$

FRACTURE DENSITY

Descriptor	Criteria
Unfractured	No fractures
Very Slightly Fractured	Lengths greater 3 ft
Slightly Fractured	Lengths from 1 to 3 ft, few lengths outside that range
Moderately Fractured	Lengths mostly in range of 4 in. to 1 ft, with most lengths about 8 in.
Intensely Fractured	Lengths average from 1 in. to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very Intensely Fractured	Mostly chips and fragments with few scattered short core lengths

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF/MH	**3-1-21**	**3-2-21**	**N 1,932,645 E 6,250,478**	**R-21-001**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Tri-County		**70.0 ft**
DRILLING METHOD	**DRILL RIG**	**BOREHOLE DIAMETER**
Rotary Wash	**Diedrich D-120**	**4"**
SAMPLER TYPE(S) AND SIZE(S) (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
Mod Cal (2"), SPT (1.4")	**Automatic 140lb, 30in drop**	**82%**
BOREHOLE BACKFILL AND COMPLETION	**GROUNDWATER READINGS — DURING DRILLING: NM — AFTER DRILLING (DATE):**	**TOTAL DEPTH OF BORING**
Cement-Bentonite Grout		**90.8 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	0	ASPHALT (Asphalt Concrete: 4 in.) (No Base).									
	1	SILTY SAND (SM); brown; moist; few subangular to subrounded GRAVEL, max. 1 in. dia.; mostly coarse to fine SAND; little nonplastic fines; weak cementation; (FILL).									
68.00	2										
	3										
66.00	4										
	5	Poorly graded SAND (SP); loose; brown; moist; mostly medium to fine SAND; few nonplastic fines; weak cementation.	1	4, 4, 3	7	56					
64.00	6										
	7										
62.00	8										
	9										
60.00	10	Medium dense.	2	5, 8, 9	17	61		12	106		
	11										
58.00	12										
	13										
56.00	14										
	15	Poorly graded SAND with SILT (SP-SM); medium dense; brown; moist; about 88% medium to fine SAND; about 12% nonplastic fines; weak cementation.	3	10, 8, 13	21	33					PA
54.00	16										
	17										
52.00	18										
	19										
50.00	20	Dense; mostly SAND; few fines.	4	15, 23, 29	52	67					DS
	21										
48.00	22										
	23										
46.00	24										
	25										

(continued)

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	25	Poorly graded SAND with SILT (SP-SM); dense; brown; moist; trace subangular to subrounded GRAVEL, max. 1/2 in. dia.; mostly medium to fine SAND; few nonplastic fines; weak cementation.	5	10 12 14	26	33					
44.00	26	Poorly graded SAND with SILT (SP-SM) *(continued)*.									
	27										
42.00	28										
	29										
40.00	30	About 92% medium SAND; about 8% nonplastic fines.	6	15 23 28	51	67		5	114		DS, PA, SG
	31										
38.00	32										
	33										
36.00	34										
	35	Poorly graded SAND with CLAY (SP-SC); medium dense; brown; moist; mostly coarse to fine SAND; few medium plasticity fines; weak cementation.	7	6 5 4	9	28					
34.00	36										
	37	SEDIMENTARY BEDROCK, Delmar Formation - Sandy SILTSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive.									Hard drilling at 37.5 feet
32.00	38										
	39										
30.00	40	Slight cemented.	8	30 50/5"		100					UU
	41										
28.00	42										
	43										
26.00	44										
	45	Clayey SANDSTONE, olive gray to olive brown; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; massive; friable. About 61% SAND; about 39% fines.	9	37 50/5"		100		20			CR, PA, PI
24.00	46										
	47										Hard drilling
22.00	48										Pressure Meter Test at 48 feet
	49										
20.00	50		10	50/2"		100					Packet Test between 50' and 60' depth
	51										
18.00	52										
	53										
16.00	54										
	55										

(continued)

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		SEDIMENTARY ROCK *(continued)*. Sandy CLAYSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive. About 43% SAND; about 57% fines.		11	28 43 50/3"		100		24					CR, PA, PI
14.00	56														
	57														Hard drilling
12.00	58														
	59														
10.00	60				12	50/3"		100							
	61														
8.00	62														
	63														
6.00	64														
	65		About 39% SAND; about 61% fines.		13	40 50/5.5"		78		20					CR, PA, PI
4.00	66														
	67														
2.00	68														
	69														
0.00	70		About 38% SAND; about 62% fines.		14	38 50/5"		82		22					PA
	71														
-2.00	72														
	73														
-4.00	74														
	75				15	45 50/4"		90		18					
-6.00	76														
	77														
-8.00	78														
	79														
-10.00	80		Clayey SANDSTONE, olive gray to olive brown; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; massive; friable. About 54% SAND; about 46% fines.		16	48 50/3"		100		18					PA, PI
	81														
-12.00	82														
	83														
-14.00	84														
	85														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-001**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **3 of 4**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/17/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		Sandy SILTSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive.		17	50/4"		100							
-16.00	86														
	87														
-18.00	88														
	89														
-20.00	90				18	43 50/4"		60		21					
	91		Bottom of borehole at 90.8 ft bgs Groundwater was not measured, P&S wave suspension logging was conducted after completion of drilling, Additionally packer testing was also conducted between depths of 50 and 60 feet. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.												
-22.00	92														
	93														
-24.00	94														
	95														
-26.00	96														
	97														
-28.00	98														
	99														
-30.00	100														
	101														
-32.00	102														
	103														
-34.00	104														
	105														
-36.00	106														
	107														
-38.00	108														
	109														
-40.00	110														
	111														
-42.00	112														
	113														
-44.00	114														
	115														

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				R-21-001
DIST.	COUNTY	ROUTE	POSTMILE	EA 20-134
PROJECT OR BRIDGE NAME: San Diego Regional Rail Corridor				
BRIDGE NUMBER	PREPARED BY: PSS		DATE: 3-26-21	SHEET: 4 of 4

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF/MH	**3-3-21**	**3-8-21**	**N 1,931,383 E 6,251,075**	**R-21-002**
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Tri-County				**198.0 ft**
DRILLING METHOD			DRILL RIG	BOREHOLE DIAMETER
Rotary Wash			**Diedrich D-120**	**4"**
SAMPLER TYPE(S) AND SIZE(S) (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Mod Cal (2"), SPT (1.4")			**Automatic 140lb, 30in drop**	**82%**
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: **58.5 (Seepage)**; AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING
Cement-Bentonite Grout				**200.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	0	ASPHALT (Asphalt Concrete: 5 in.) (No Base).									
	1	SILTY SAND (SM); brown; moist; mostly medium to fine SAND; little nonplastic fines; weak cementation; (FILL).									
196.00	2										
	3										
194.00	4										
	5	SILTY SAND (SM); very dense; brown; moist; mostly medium to fine SAND; little nonplastic fines; weak cementation; some coarse SAND.	1	18 19 18	37	100					
192.00	6										
	7										
190.00	8										
	9										
188.00	10	Medium SAND.	2	18 27 30	57	83		14	126		
	11										
186.00	12										
	13										
184.00	14										
	15	Dense.	3	8 12 11	23	83					
182.00	16										
	17										
180.00	18										
	19										
178.00	20	Very dense; coarse to medium SAND.	4	20 35 44	79	67		13	129		SG
	21										
176.00	22										
	23										
174.00	24										
	25										

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Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-002**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME: **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-25-21**	SHEET **1 of 7**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		SEDIMENTARY BEDROCK, Torrey Sandstone Formation - SANDSTONE; olive brown to light olive brown; soft to moderately soft; slightly weathered to moderately weathered; medium to fine grained; poorly indurated; friable. About 80% SAND; about 20% fines.		5	14 17 19	36	56							PA
172.00	26														
	27														
170.00	28														
	29														
168.00	30				6	21 30 35	65	67							DS
	31														
166.00	32														
	33														
164.00	34														
	35		About 78% SAND; about 22% fines.		7	8 16 36	52	67							PA
162.00	36														
	37														
160.00	38														
	39														
158.00	40		Light olive brown, slightly weathered to fresh.		8	50/2"		100							Hard drilling at 40 feet
	41														
156.00	42														
	43														
154.00	44														
	45		About 75% SAND; about 25% fines.		9	50/3"		100		18					PA
152.00	46														
	47														
150.00	48														
	49														
148.00	50		Light grayish brown.		10	50/4"		75		22					
	51														
146.00	52														
	53														
144.00	54														
	55														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE					HOLE ID
BORING RECORD					**R-21-002**
DIST.	COUNTY	ROUTE	POSTMILE		EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**					
BRIDGE NUMBER	PREPARED BY **PSS**			DATE **3-25-21**	SHEET **2 of 7**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		*(continued)*. SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; friable.												
142.00	56														
	57														
140.00	58														
	59														
138.00	60			X	11	50/4"		25							
	61														
136.00	62														
	63														
134.00	64														
	65														
132.00	66														
	67														
130.00	68														
	69														
128.00	70		About 71% SAND; about 29% fines; light olive gray.	X	12	50/3"		100		19					PA
	71														
126.00	72														
	73														
124.00	74														
	75														
122.00	76														
	77														
120.00	78														
	79														
118.00	80		White to light gray.	X	13	50/4.5"		100		19					
	81														
116.00	82														
	83														
114.00	84														
	85														

(continued)

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; friable.												
112.00	86														
	87														
110.00	88														
	89														
108.00	90				14	50/5"		60							
	91														
106.00	92														
	93														
104.00	94														
	95														
102.00	96														
	97														
100.00	98														
	99														
98.00	100		About 80% SAND; about 20% fines.		15	50/3"		100		20					PA
	101														
96.00	102														
	103														
94.00	104														
	105														
92.00	106														
	107														
90.00	108														
	109														
88.00	110		Mottled yellowish brown.		16	50/3"		33							
	111														
86.00	112														
	113														
84.00	114														
	115														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				R-21-002
DIST.	COUNTY	ROUTE	POSTMILE	EA 20-134
PROJECT OR BRIDGE NAME: San Diego Regional Rail Corridor				
BRIDGE NUMBER	PREPARED BY PSS		DATE 3-25-21	SHEET 4 of 7

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	115	SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; friable.												
82.00	116													
	117													
80.00	118													
	119													
78.00	120	About 68% SAND; about 32% fines.		17	48 50/5"		55		18					PA
	121													
76.00	122													
	123													
74.00	124													
	125	Gravelly to Cobbly SANDSTONE; light olive brown; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; friable.												Rig chattering, some cobbles/gravel
72.00	126													
	127													
70.00	128													
	129													
68.00	130			18	50/3"		100							
	131													
66.00	132													
	133													
64.00	134													
	135													Rig chattering, some cobbles/gravel
62.00	136													
	137													
60.00	138	SEDIMENTARY BEDROCK, Delmar Formation - Sandy CLAYSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated.												Change to clay/silt at 138 feet
	139													
58.00	140	About 48% SAND; about 52% fines.		19	49 50/4"		90		18					PA, PI
	141													
56.00	142													
	143	Coarse SANDSTONE bed from 143 to 145 feet.												Hard drilling
54.00	144													
	145													

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE **BORING RECORD**				HOLE ID **R-21-002**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-25-21**	SHEET **5 of 7**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	145		About 23% SAND; about 77% fines; medium to high plasticity fines. *(continued)*.		20	49 50/3"		100		19					PA, PI
52.00	146														
	147														
50.00	148														
	149				21	47 50/5"		55							
48.00	150		About 37% SAND; about 63% fines.		1			93		21					PA, PI ,UU
	151														
46.00	152									16					
	153		Clayey SANDSTONE; olive gray to dark gray; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; massive; slightly fractured; friable. About 79% SAND; about 21% fines.		2			87							PA, PI ,UC
44.00	154														
	155				3			100							
42.00	156														
	157														UU
40.00	158														
	159		About 60% SAND; about 40% fines.							15					PA UC Packet Test between 160' and 170' depth
38.00	160				4			100							
	161		Mottled yellowish brown.												
36.00	162														
	163		About 85% SAND; about 15% fines.							15					CR, PA ,UC, UU
34.00	164														
	165				5			100							
32.00	166														
	167		2.5" black silty sand layer at 167 feet (organic odor). About 87% SAND; about 13% fines.							14					PA ,UC
30.00	168														
	169		Intensely fractured.												
28.00	170				6			90							UC
	171														
26.00	172														UU
	173		About 67% SAND; about 33% fines.							13					CR, PA, PI
24.00	174				7			100							
	175														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-002**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-25-21**	SHEET **6 of 7**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	175		*(continued).*		8			100							
22.00	176														
	177														UU
20.00	178		About 73% SAND; about 27% fines. Gray to dark gray.							12					CR, PA, PI
	179														
18.00	180				9			100							
	181														
16.00	182														
	183														
14.00	184		Slightly fractured to moderately fractured.		10			100							
	185														
12.00	186														
	187														
10.00	188		About 59% SAND; about 41% fines.		11			100		12					PA, PI
	189														
8.00	190		CLAYSTONE; olive gray to dark gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive; intensely fractured; low to medium plasticity.		12			100							
	191									16					PA, PI
6.00	192		Clayey SANDSTONE; gray to dark gray; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; massive; slightly fractured; friable.												
	193														
4.00	194		About 67% SAND; about 33% fines.												
	195		About 74% SAND; about 26% fines.		13			100		12					PA, PI
2.00	196														
	197														
0.00	198		Sandy Silty CLAYSTONE; olive gray mottled reddish brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; massive; slightly fractured to intensely fractured; medium to high plasticity.												
	199														
-2.00	200		Bottom of borehole at 200.0 ft bgs Groundwater seepage was observed at 58.5 feet. Packer testing was conducted between depths of 160 and 170 feet. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.												
	201														
-4.00	202														
	203														
-6.00	204														
	205														

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE **BORING RECORD**				HOLE ID **R-21-002**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-25-21**	SHEET **7 of 7**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF	**3-16-21**	**3-17-21**	**N 1,925,801 E 6,253,938**	**R-21-003**
DRILLING CONTRACTOR **Tri-County**			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION **365.0 ft**
DRILLING METHOD **Rotary Wash**			DRILL RIG **Diedrich D-120**	BOREHOLE DIAMETER **4"**
SAMPLER TYPE(S) AND SIZE(S) (ID) **SPT (1.4")**			SPT HAMMER TYPE **Automatic 140lb, 30in drop**	HAMMER EFFICIENCY, ERi **82%**
BOREHOLE BACKFILL AND COMPLETION **Cement-Bentonite Grout**			GROUNDWATER READINGS DURING DRILLING **NM** AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **120.0 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Remarks
	0	ASPHALT (Asphalt Concrete: 5.5 in.) (No Base).									
	1	SILTY SAND (SM); brown; moist; mostly medium to fine SAND; little nonplastic fines; weak cementation; (FILL).									
363.00	2										
	3										
361.00	4										
	5	SEDIMENTARY BEDROCK, Old Paralic Deposits - SANDSTONE; reddish brown to dark brown; very soft to soft; slightly weathered to fresh; medium to fine grained; poorly indurated; slightly fractured to intensely fractured; friable.	1	50/2"		50					
			1			100					
359.00	6										
	7										
357.00	8	About 82% SAND; about 18% fines.						8			PA
	9										
355.00	10		2			100					
	11										
353.00	12										
	13										
351.00	14										
	15		3			100					
349.00	16										
	17										
347.00	18										
	19										
345.00	20	Slight increase in grain size.	4			100					
	21										
343.00	22										
	23	About 90% SAND; about 10% fines.						9			PA, PI
341.00	24										
	25										

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		*(continued)*.		5			40							No Recovery from 25 ft to 28 ft
339.00	26		Gravelly SANDSTONE; reddish brown to dark brown; soft to moderately soft; slightly weathered to fresh; medium to coarse grained; poorly indurated; massive; friable.												
	27														
337.00	28		Little GRAVEL, subrounded to rounded..												
	29														
335.00	30				6			40							No Recovery from 30 ft to 33 ft
	31														
333.00	32														
	33		Little GRAVEL, max 2 in. dia. sub-rounded..												
331.00	34		SEDIMENTARY BEDROCK, Torrey Sandstone Formation - SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; massive; friable; intensely to moderately fractured.												
	35				7			100							
329.00	36		Banding with pale yellow and light gray..												
	37														
327.00	38														
	39														
325.00	40		Unfractured to slightly fractured.		8			100							
	41														
323.00	42														
	43		About 92% SAND; about 8% fines.							11					CR, PA, PI
321.00	44														
	45				9			100							
319.00	46														
	47														
317.00	48														
	49														
315.00	50				10			100							
	51														
313.00	52														
	53														
311.00	54														
	55														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-003**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **2 of 5**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; unfractured to slightly fractured. *(continued)*.		11			100							
309.00	56														
	57														
307.00	58														
	59														
305.00	60				12			100							
	61														
303.00	62														
	63		About 96% SAND; about 4% fines.							14					PA, PI
301.00	64														
	65				13			100							
299.00	66														
	67														
297.00	68														
	69														
295.00	70				14			100							
	71														
293.00	72														
	73														
291.00	74														
	75				15			100							
289.00	76														
	77														
287.00	78														
	79														
285.00	80				16			100							
	81														
283.00	82														
	83		About 94% SAND; about 6% fines.							17					PA
281.00	84														
	85														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				**R-21-003**
DIST.	COUNTY	ROUTE	POSTMILE	EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**				
BRIDGE NUMBER	PREPARED BY **PSS**		DATE **3-26-21**	SHEET **3 of 5**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium grained; poorly indurated; unfractured to slightly fractured.		17			100							UC
279.00	86														
	87														
277.00	88														
	89														
275.00	90				18			100							
	91														
273.00	92														
	93		Medium to coarse grained SAND.												
271.00	94				19			61							
	95				20			100							Packet Test between 95' and 105' depth
269.00	96														
	97		Gravelly SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; medium to coarse grained; poorly indurated; some angular to subangular GRAVEL to 2" dia..												Rig chattering, gravelly
267.00	98														UU
	99		SANDSTONE; white to light olive brown; soft to moderately soft; slightly weathered to fresh; fine to coarse grained; poorly indurated.												
265.00	100		Intensely fractured.		21			100							
	101														
263.00	102		Few angular to subangular GRAVEL. About 93% SAND; about 7% fines.							14					PA
	103														
261.00	104		Yellowish brown.												
	105		Unfractured to slightly fractured.		22			100							
259.00	106														
	107														
257.00	108														
	109														
255.00	110				23			100							
	111														
253.00	112														
	113														
251.00	114														
	115														

(continued)

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE					HOLE ID
BORING RECORD					**R-21-003**
DIST.	COUNTY	ROUTE	POSTMILE		EA **20-134**
PROJECT OR BRIDGE NAME **San Diego Regional Rail Corridor**					
BRIDGE NUMBER	PREPARED BY **PSS**			DATE **3-26-21**	SHEET **4 of 5**

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	115		SANDSTONE; white to yellowish brown; soft to moderately soft; slightly weathered to fresh; medium to fine grained; poorly indurated; intensely fractured.		24			100							
249.00	116														
	117		Coarse grained SAND.												
247.00	118														
	119														
245.00	120		Bottom of borehole at 120.0 ft bgs Groundwater was not measured, Packer testing was conducted between depths of 95 and 105 feet. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.												
	121														
243.00	122														
	123														
241.00	124														
	125														
239.00	126														
	127														
237.00	128														
	129														
235.00	130														
	131														
233.00	132														
	133														
231.00	134														
	135														
229.00	136														
	137														
227.00	138														
	139														
225.00	140														
	141														
223.00	142														
	143														
221.00	144														
	145														

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
PSS/TBF/MH	**3-12-21**	**3-15-21**	**N 1,921,088 E 6,256,153**	**R-21-004**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Tri-County		**160.0 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Rotary Wash	**Diedrich D-120**	**4"**
SAMPLER TYPE(S) AND SIZE(S) (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
Mod Cal (2"), SPT (1.4")	**Automatic 140lb, 30in drop**	**82%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **68.9 (Seepage)**; AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING
Cement-Bentonite Grout		**110.4 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	0		ASPHALT (Asphalt Concrete: 4 in.) (No Base).												
	1		Poorly graded SAND with SILT (SP-SM); light brown to white; moist; mostly coarse to medium SAND; few nonplastic fines; weak cementation [FILL].												
158.00	2														
	3		SEDIMENTARY BEDROCK, Torrey Sandstone Formation - SANDSTONE; white to light brown; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; friable.												
156.00	4														
	5		About 88% SAND; about 12% fines.		1	33 41 50/5"		71							PA, PI
154.00	6														
	7														
152.00	8														
	9														
150.00	10				2	50/5"		100		12	116				SG
	11														
148.00	12														
	13														
146.00	14														
	15														
144.00	16														
	17														
142.00	18														
	19														
140.00	20		About 86% SAND; about 14% fines.		3	44 50/5"		91							PA, PI
	21														
138.00	22														
	23														
136.00	24														
	25														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	25		*(continued).*												
134.00	26														
	27														
132.00	28														
	29														
130.00	30				4	50/4"		50							
	31														
128.00	32														
	33														
126.00	34														
	35														
124.00	36														
	37														
122.00	38														
	39														
120.00	40				5	50/4"		50							
	41														
118.00	42														
	43														
116.00	44														
	45		About 92% SAND; about 8% fines.		6	50/4.5"		89							PA
114.00	46														
	47														
112.00	48														
	49														
110.00	50		About 91% SAND; about 9% fines.		7	50/5"		100							PA
	51														
108.00	52														
	53														
106.00	54														
	55														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	55		SANDSTONE; white to light brown; soft to moderately soft; slightly weathered to fresh; coarse to medium grained; poorly indurated; friable. Cobble bed at 56 feet.		8	50/5"		80							
104.00	56														Rig chattering, cobbly and gravelly
	57														
102.00	58														
	59														
100.00	60		About 89% SAND; about 11% fines.		9	50/5"		100							PA
	61														
98.00	62														
	63														
96.00	64														
	65				10	50/5"		100							Pressure Meter Test at 65 feet
94.00	66														
	67														
92.00	68														
	69														
90.00	70				11	50/3"		33							No recovery
	71														
88.00	72														
	73														
86.00	74														
	75		About 90% SAND; about 10% fines; mottled reddish brown.		12	32 37 50/3"		80		22					CR, PA, PI
84.00	76														
	77														
82.00	78														
	79														
80.00	80				13	42 32 41	73	67							
	81														
78.00	82														
	83		SEDIMENTARY BEDROCK, Delmar Formation - Sandy SILTSTONE; olive gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated.												Change to SILTSTONE at 83 feet
76.00	84														
	85														

(continued)

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	Gas Data	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (tsf)	Drilling Method	Casing Depth	Remarks
	85		Clayey SANDSTONE, olive brown to olive gray; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; friable. About 73% SAND; about 27% fines.		14	42 50/5"		100		19					PA
74.00	86														
	87														
72.00	88														
	89														
70.00	90				15	50/3"		100		24					
	91														
68.00	92														
	93														
66.00	94														
	95														
64.00	96														
	97														
62.00	98														
	99														
60.00	100		Sandy CLAYSTONE; dark gray mottled brown; soft to moderately soft; slightly weathered to fresh; poorly indurated; medium to high plasticity. About 50% SAND; about 50% fines.		16	32 50/6"		100		23					PA, PI
	101														
58.00	102		Cobbles and gravels at 102 feet.												Rig chattering, cobbly and gravelly
	103														
56.00	104														
	105														
54.00	106														
	107														
52.00	108														
	109														
50.00	110		SANDSTONE, olive brown to olive gray; soft to moderately soft; slightly weathered to fresh; fine to medium grained; poorly indurated; friable. About 94% SAND; about 6% fines.		17	50/5"		100							PA
	111														
48.00	112		Bottom of borehole at 110.4 ft bgs. Groundwater seepage was observed at 68.9 feet. P&S wave suspension logging was conducted after completion of drilling. After completion boring was tremie grouted with cement-bentonite grout and borehole was patched with rapid-set concrete and black dye.												
	113														
46.00	114														
	115														

Earth Mechanics, Inc.
Geotechnical and Earthquake Engineering

REPORT TITLE				HOLE ID
BORING RECORD				R-21-004
DIST.	COUNTY	ROUTE	POSTMILE	EA 20-134
PROJECT OR BRIDGE NAME: San Diego Regional Rail Corridor				
BRIDGE NUMBER	PREPARED BY PSS		DATE 3-26-21	SHEET 4 of 4

CALTRANS BORING RECORD MET+ENG FIXED 20-134 HDR, SAN DIEGO REGIONAL RAIL CORRIDOR - GEOLOGYUPDATED.GPJ EMI CALTRANS 2013 V2.0.GLB 5/13/21

LCI, 2020

GEOTECHNICAL BORING LOG B-18

Project No.	10147-010	**Date Drilled**	5-29-15
Project	San Dieguito Double Track Project	**Logged By**	RNB
Drilling Co.	Pacific Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	12'
Location	See Plate 4: Geotechnical Map	**Sampled By**	RNB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0							SC	UNDOCUMENTED ARTIFICIAL FILL (Afu)	
10				B-1 1'-5'					@ 1': Clayey SAND, loose, dark brown, moist, trace gravel	
									QUATERNARY ALLUVIUM (Qya)	CR
	5			R-1	1 2 2			SM	@ 5': Silty SAND, loose, dark gray, moist, fine-grained (*no recovery at 5')	
5										
	10			S-1	2 3 4				@ 10': Silty SAND, loose, dark gray, moist, fine-grained	
0										
									@ 13': Groundwater encountered at 13 feet below ground surface	
	15			S-2	3 3 4					SA, H
-5										
	20			S-3	2 8 16		24	SP-SM	@ 20': Poorly graded SAND with SILT, medium dense, gray, wet, fine-grained	-200
-10										
	25			S-4	6 17 27				@ 25': Poorly graded SAND with SILT, dense, gray, wet, fine-grained	-200
-15										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG B-18

Project No.	10147-010	**Date Drilled**	5-29-15
Project	San Dieguito Double Track Project	**Logged By**	RNB
Drilling Co.	Pacific Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	12'
Location	See Plate 4: Geotechnical Map	**Sampled By**	RNB

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-5	3 7 11		26	SM	@ 30': Silty SAND, medium dense, light brown, fine-grained	-200
-20										
	35			S-6	2 5 11				@ 35': Silty SAND, medium dense, brown, wet, fine-grained, trace oxidation staining	-200
-25										
	40			S-7	2 10 18		28		@ 40': Silty SAND, medium dense, brown, wet, fine-grained	-200
-30										
	45			S-8	4 10 22				@ 45': Silty SAND, dense, light brown, wet, fine-grained	-200
-35										
	50			S-9	2 10 14		29		@ 50': Silty SAND, medium dense, light brown, wet, fine-grained	-200
-40										
	55			S-10	2 8 14			CL	@ 55': Lean CLAY, medium dense, light brown, wet, fine-grained	-200
-45									**Total Depth = 56.5 Feet** **Groundwater encountered at 13 feet at time of drilling** **Backfilled with bentonite/cement grout on 5/29/15**	
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG B-19

Project No.	10147-010	**Date Drilled**	5-29-15
Project	San Dieguito Double Track Project	**Logged By**	RNB
Drilling Co.	Pacific Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	17'
Location	See Plate 4: Geotechnical Map	**Sampled By**	RNB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SM	UNDOCUMENTED ARTIFICIAL FILL (Afu)	
15				B-1 1'-5'					@ 1': Silty SAND, loose, dark brown, moist, trace gravel	RV
								SM	QUATERNARY ALLUVIUM (Qya)	
	5			R-1	3 4 7	106	12		@ 5': Silty SAND, medium dense, brown, moist, fine-grained	DS
10										
	10			R-2	3 2 2				@ 10': Silty SAND, loose, brown, moist, fine-grained (*no recovery)	
5										
	15			S-1	3 6 12		25	SP-SM	@ 15': Poorly gradede SAND with SILT, medium dense, gray, moist, fine-grained	SA, H
0									@ 17': Groundwater encountered at 17 feet below ground surface	
	20			S-2	1 1 1			CH	@ 20': Fat CLAY, very soft, black, wet, high plasticity, strong organic smell	AL
-5										
	25			S-3	5 10 11			SM	@ 25': Silty SAND, medium dense, gray, wet, fine-grained	-200
-10										
	30									

SAMPLE TYPES:
B BULK SAMPLE
C CORE SAMPLE
G GRAB SAMPLE
R RING SAMPLE
S SPLIT SPOON SAMPLE
T TUBE SAMPLE

TYPE OF TESTS:
-200 % FINES PASSING
AL ATTERBERG LIMITS
CN CONSOLIDATION
CO COLLAPSE
CR CORROSION
CU UNDRAINED TRIAXIAL
DS DIRECT SHEAR
EI EXPANSION INDEX
H HYDROMETER
MD MAXIMUM DENSITY
PP POCKET PENETROMETER
RV R VALUE
SA SIEVE ANALYSIS
SE SAND EQUIVALENT
SG SPECIFIC GRAVITY
UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG B-19

Project No.	10147-010	**Date Drilled**	5-29-15
Project	San Dieguito Double Track Project	**Logged By**	RNB
Drilling Co.	Pacific Drilling	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	17'
Location	See Plate 4: Geotechnical Map	**Sampled By**	RNB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-4	8 12 17			SM	@ 30': Silty SAND, medium dense, brown, wet, fine-grained	-200
-15									**Total Depth = 31.5 Feet** **Groundwater encountered at 17 feet at time of drilling** **Backfilled with bentonite/cement grout on 5/29/15**	
	35									
-20										
	40									
-25										
	45									
-30										
	50									
-35										
	55									
-40										
	60									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG B-20

Project No.	10147-010	**Date Drilled**	5-3-15
Project	San Dieguito Double Track Project	**Logged By**	BSS
Drilling Co.	Pacific Drilling	**Hole Diameter**	6"
Drilling Method	Solid Stem Auger - 140lb - Manualhammer - 30" Drop	**Ground Elevation**	21'
Location	See Plate 5: Geotechnical Map	**Sampled By**	BSS

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							SM	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0.5': Silty SAND, medium dense, light brown, fine, moist, trace mica	
20										
	5			R-1	4 7 11	100	10		@ 5': Silty SAND, medium dense, brown, moist, fine-grained, poorly-graded	DS
15										
	10			R-2	10 16 19	105	2		@ 10': Silty SAND, medium dense, brown, moist, fine-grained, poorly-graded	
10										
	15			R-3	10 10 18	107	5		@ 15': Silty SAND, medium dense, light reddish brown, moist, fine-grained, poorly-graded	
5										
									QUATERNARY OLD PARALIC DEPOSITS (Qop)	
	20			S-4	7 16 19			SC	@ 20': Clayey SAND, dense, grayish brown, saturated, some gravel	
0										
									Total Depth = 21.5 Feet **Groundwater encountered at 19 feet at time of drilling** **Backfilled with bentonite/cement grout on 5/3/15**	
	25									
-5										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG B-21

Project No.	10147-010	**Date Drilled**	5-3-15
Project	San Dieguito Double Track Project	**Logged By**	BSS
Drilling Co.	Pacific Drilling	**Hole Diameter**	6"
Drilling Method	Solid Stem Auger - 140lb - Manualhammer - 30" Drop	**Ground Elevation**	30'
Location	See Plate 1: Geotechnical Map	**Sampled By**	BSS

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
30	0			B-1 1'-6'				SC	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0.5': Clayey SAND, dense, reddish brown, moist, some gravel	
25	5			R-1	4 4 7	103	7	SM	@ 5': Silty SAND, medium dense, light brown, moist, fine-grained, micaceous, few gravel	DS
20	10			R-2	5 5 11	112	9		@ 10': Silty SAND, medium dense, reddish brown, moist, fine, trace clay, poorly-graded @ 12': Trace cobbles encountered	DS
15	15			R-3	6 7 9	103	8		@ 15': Silty SAND, medium dense, light reddish brown, moist, fine to medium, trace clay, poorly-graded	
10	20			R-4	8 12 15	112	8	CL SP	@ 20': Sandy CLAY, medium dense, grayish brown, moist, fine to medium QUATERNARY OLD PARALIC DEPOSITS (Qop) @ 21': SAND, medium dense, grayish brown, moist, poorly-graded, fine-grained	
5	25			S-5	6 6 11			SP-SM	@ 25': Poorly graded SAND with SILT, medium dense, grayish brown, wet, fine to medium, poorly-graded	SA, H
0	30								**Total Depth = 26.5 Feet** **Groundwater encountered at 22.5 feet at time of drilling** **Backfilled with bentonite/cement grout on 5/3/15**	

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG B-22

Project No.	10147-010	**Date Drilled**	5-3-15
Project	San Dieguito Double Track Project	**Logged By**	BSS
Drilling Co.	Pacific Drilling	**Hole Diameter**	6"
Drilling Method	Solid Stem Auger - 140lb - Manualhammer - 30" Drop	**Ground Elevation**	32'
Location	See Plate 1: Geotechnical Map	**Sampled By**	BSS

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0							SM	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0.5': Silty SAND, loose, light brown, moist to wet, some clay, fine	
30										
	5			R-1	P P 2				@ 5': No recovery, very loose	
25										
	10			S-2 B-1 10'-20'	3 2 2				@ 10': Silty SAND, loose, reddish brown, moist, fine to medium, some clay, poorly-graded	RV
20										
	15			R-3	10 10 14	113	8		@ 15': Silty SAND, medium dense, light reddish brown, moist to wet, fine, poorly-graded	
15										
	20			R-4	6 7 9	102	2		@ 20': Silty SAND, medium dense, light brown, moist, fine, some clay, poorly-graded	
10										
									QUATERNARY OLD PARALIC DEPOSITS (Qop)	
	25			R-5	12 23 31	104	6	SM	@ 25': Silty SAND, dense, olive-brown, wet, fine-grained, some gravel, micaceous, trace calcium-carbonate blebs, poorly-graded, poorly indurated	
5									**Total Depth = 26.5 Feet** **Groundwater encountered at 25 feet at time of drilling** **Backfilled with bentonite/cement grout on 5/3/15**	
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

N&M, 2013

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								FILL: Track ballast.
							SM	Reddish brown, wet, medium dense, silty fine to medium SAND; trace gray clay; few ballast.
5			18	8.4	101.5			Damp.
10			8					Moist to wet.
			16	16.1	114.6			
							SM	ALLUVIUM: Gray, saturated, dense, silty fine SAND; micaceous.
15			27					
20								

DATE DRILLED 4/26/08 - 4/28/08 BORING NO. B-1

GROUND ELEVATION 18' ± (MSL) SHEET 1 OF 7

METHOD OF DRILLING 4" Mud Rotary (NWJ - Gregg)

DRIVE WEIGHT 140 Lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

Ninyo & Moore

BORING LOG

NCTD BRIDGE 246.1 REPLACEMENT PROJECT
LOS PEÑASQUITOS LAGOON, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991020	8/13	A-1

DATE DRILLED 4/26/08 - 4/28/08 BORING NO. B-1

GROUND ELEVATION 18' ± (MSL) SHEET 2 OF 7

METHOD OF DRILLING 4" Mud Rotary (NWJ - Gregg)

DRIVE WEIGHT 140 Lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							SM	ALLUVIUM: (Continued) Gray, saturated, medium dense, silty fine SAND; micaceous.
25			13					Many shell fragments.
							SP-SM	Gray, saturated, medium dense, poorly graded SAND with silt; little clay; micaceous.
30			36	27.1	96.6			
35								
40								

Ninyo & Moore

BORING LOG

NCTD BRIDGE 246.1 REPLACEMENT PROJECT
LOS PEÑASQUITOS LAGOON, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991020	8/13	A-2

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		■	13				SP	ALLUVIUM: (Continued) Dark gray, saturated, medium dense, poorly graded SAND; micaceous.
45								
							SC	Light gray, saturated, dense, clayey, fine to medium SAND; trace silt; shell fragments in shoe.
50		■	28					
55								
60								

DATE DRILLED 4/26/08 - 4/28/08 BORING NO. B-1

GROUND ELEVATION 18' ± (MSL) SHEET 3 OF 7

METHOD OF DRILLING 4" Mud Rotary (NWJ - Gregg)

DRIVE WEIGHT 140 Lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

Ninyo & Moore

BORING LOG

NCTD BRIDGE 246.1 REPLACEMENT PROJECT
LOS PEÑASQUITOS LAGOON, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991020	8/13	A-3

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
60		■	99/11"				SC	ALLUVIUM: (Continued) Gray, saturated, very dense, clayey, fine to coarse SAND; many shell fragments; some organic debris; strong odor.
65								
70		■	9				SM	Dark gray, saturated, medium dense, silty SAND.
75								
80								

DATE DRILLED 4/26/08 - 4/28/08 BORING NO. B-1

GROUND ELEVATION 18' ± (MSL) SHEET 4 OF 7

METHOD OF DRILLING 4" Mud Rotary (NWJ - Gregg)

DRIVE WEIGHT 140 Lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

Ninyo & Moore

BORING LOG

NCTD BRIDGE 246.1 REPLACEMENT PROJECT
LOS PEÑASQUITOS LAGOON, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991020	8/13	A-4

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
80		■	13				SM	ALLUVIUM: (Continued) Dark gray, saturated, medium dense, silty SAND; micaceous.
85								
90								
95								
100								

DATE DRILLED 4/26/08 - 4/28/08 BORING NO. B-1

GROUND ELEVATION 18' ± (MSL) SHEET 5 OF 7

METHOD OF DRILLING 4" Mud Rotary (NWJ - Gregg)

DRIVE WEIGHT 140 Lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 4/26/08 - 4/28/08 BORING NO. B-1

GROUND ELEVATION 18' ± (MSL) SHEET 6 OF 7

METHOD OF DRILLING 4" Mud Rotary (NWJ - Gregg)

DRIVE WEIGHT 140 Lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

DEPTH (feet)	BLOWS/FOOT	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
100		SM	ALLUVIUM: (Continued) Dark gray, saturated, medium dense, silty SAND; micaceous.
		GM	Gray brown, saturated, very dense, coarse sandy silty GRAVEL.
			Lost circulation.
105	38		
110			
115	111/8"		No recovery.
120			

Ninyo & Moore

BORING LOG

NCTD BRIDGE 246.1 REPLACEMENT PROJECT
LOS PEÑASQUITOS LAGOON, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991020	8/13	A-6

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
120		■	120/6"					
								Total Depth = 120.5 feet. Groundwater was measured at a depth of approximately 14 feet in the borehole during drilling. Drilled on 4/26/08 through 4/27/08 to 104 feet in depth; backfilled with approximately 10 cubic feet of bentonite grout on 4/27/08. Redrilled on 4/28/08 to 120.5 feet in depth; backfilled with approximately 11 cubic feet of bentonite grout on 4/28/08. <u>Note:</u> Groundwater may rise to a level higher than that measured in the borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
125								
130								
135								
140								

DATE DRILLED 4/26/08 - 4/28/08 BORING NO. B-1

GROUND ELEVATION 18' ± (MSL) SHEET 7 OF 7

METHOD OF DRILLING 4" Mud Rotary (NWJ - Gregg)

DRIVE WEIGHT 140 Lbs. (Auto-Trip) DROP 30"

SAMPLED BY MAH LOGGED BY MAH REVIEWED BY RI

Ninyo & Moore

BORING LOG

NCTD BRIDGE 246.1 REPLACEMENT PROJECT
LOS PEÑASQUITOS LAGOON, SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991020	8/13	A-7

Appendix F. Noise and Vibration Technical Memorandum

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San Dieguito to Sorrento Valley Double Track

Del Mar Tunnels Alternatives Analysis Noise and Vibration Technical Memorandum

San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study

August 2023

Prepared by:

Entech Consulting Group
Planning • Environmental • Construction Management

Prepared for:

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619) 699-1900
www.sandag.org

Contents

Figures

Tables

Acronyms

dB	decibels
dBA	A-weighted decibels
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
Leq(h)	hourly equivalent sound level
Ldn	Day-Night Sound Level
Lmax	maximum sound level
LOSSAN	Los Angeles-San Diego-San Luis Obispo
SANDAG	San Diego Association of Governments
SEL	sound exposure level
VdB	vibration decibels

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1 Purpose of Report

The existing Los Angeles-San Diego-San Luis Obispo (LOSSAN) railroad corridor evaluates two conceptual project alternatives within the City of Del Mar and San Diego. The purpose of a tunnel alternative would be to relocate the existing tracks off the eroding coastal bluffs in Del Mar. The only feasible approach to realign the existing tracks through the densely developed areas of the City of Del Mar or the City of San Diego Torrey Pines Community would require tunneling. The two proposed conceptual alternatives are the Crest Canyon Higher Speed alternative and the Camino Del Mar alternative. The purpose of this report is to evaluate and provide information about the potential noise and vibration impacts of these two conceptual project alternatives. This preliminary assessment of potential noise and vibration impacts is qualitative and focuses on the potential for project-related impacts that could affect the surrounding community. The existing environment is characterized to determine dominant noise sources, sensitive land uses, and current noise and vibration levels. Applicable regulatory requirements are presented, and a comparison of these standards with existing noise and vibration levels is made. Specific design features of each conceptual alternative is reviewed to determine their potential effects, without mitigation, on the surrounding noise and vibration environment. Recommendations are made where necessary to incorporate design features to mitigate and reduce potential project-related noise and vibration impacts.

2 Project Description of Conceptual Alternatives

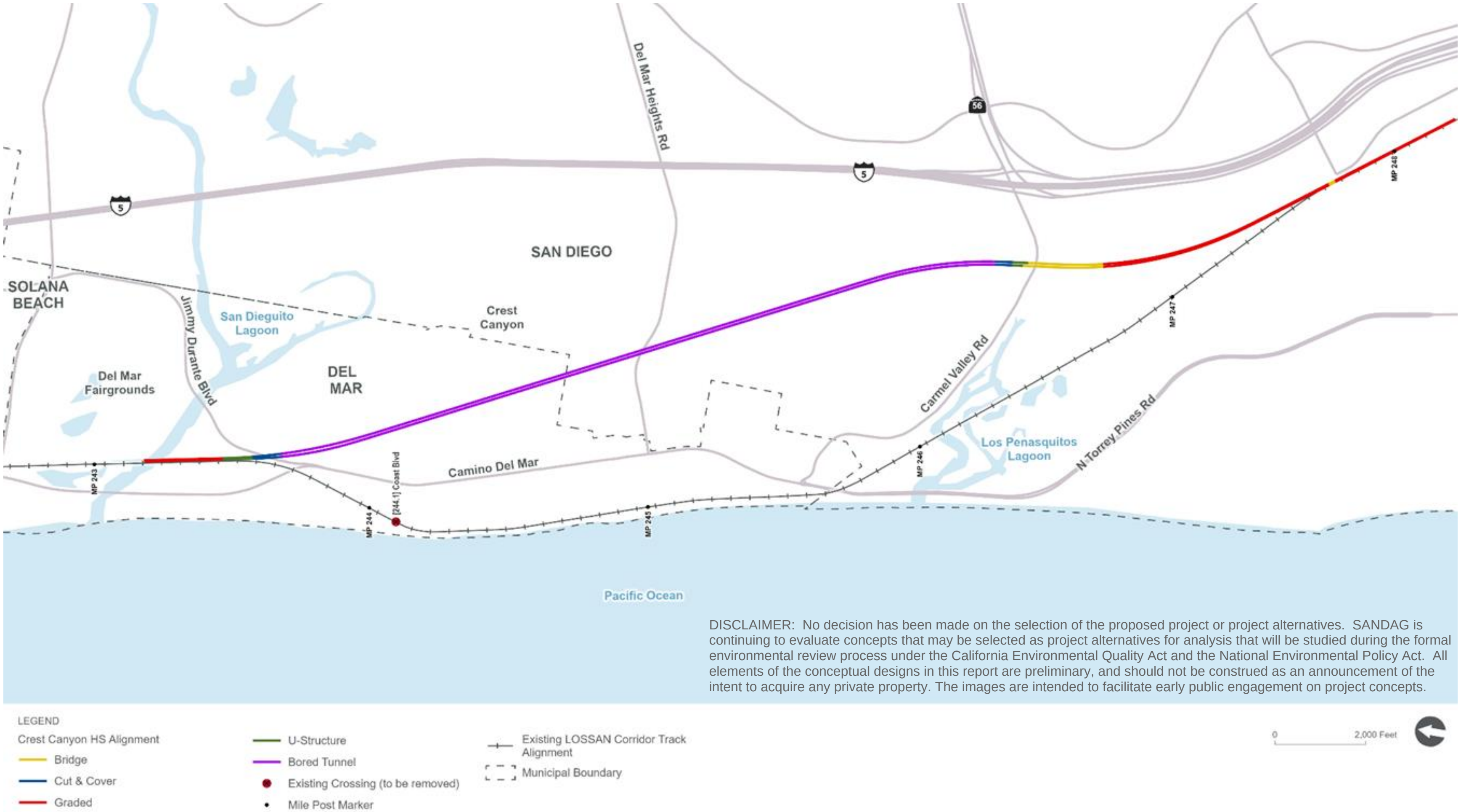
The following section describes the two conceptual project alternatives presented within the City of Del Mar and the Community of Torrey Pines, as shown on Figure 1.

Figure 1. Conceptual Project Alternatives

2.1 Crest Canyon Higher Speed

As shown on Figure 2, the Crest Canyon Higher Speed conceptual alternative's north end begins south of the Del Mar Fairgrounds at the south end of the future double-track bridge that crosses the San Dieguito Lagoon. This segment is proposed as part of the San Dieguito River Bridge Replacement Double Track and Special Events Platform. The trains would enter the north portal of the tunnel, just north of Jimmy Durante Boulevard, which would be realigned to cross over the tracks. The tunnel is a twin-bored tunnel configuration with a U-structure and cut-and-cover section at the entrance. The tunnel continues underground through the residential area of Del Mar and Torrey Pines Extension. Trains emerge from the south portal between Portofino Boulevard and Caminito Pointe Del Mar and transfers onto an aerial structure across Los Peñasquitos Lagoon. The southern end then transitions to a berm section near Sorrento Valley Road and ties into the existing tracks south of Bridge 247.7. The aerial structure would be constructed of concrete, with the substructure consisting of exposed steel pipe piles filled with concrete. The steel acts as a permanent casing with additional sacrificial thickness for corrosion, allowing for the placement of the piling in the soft soils.

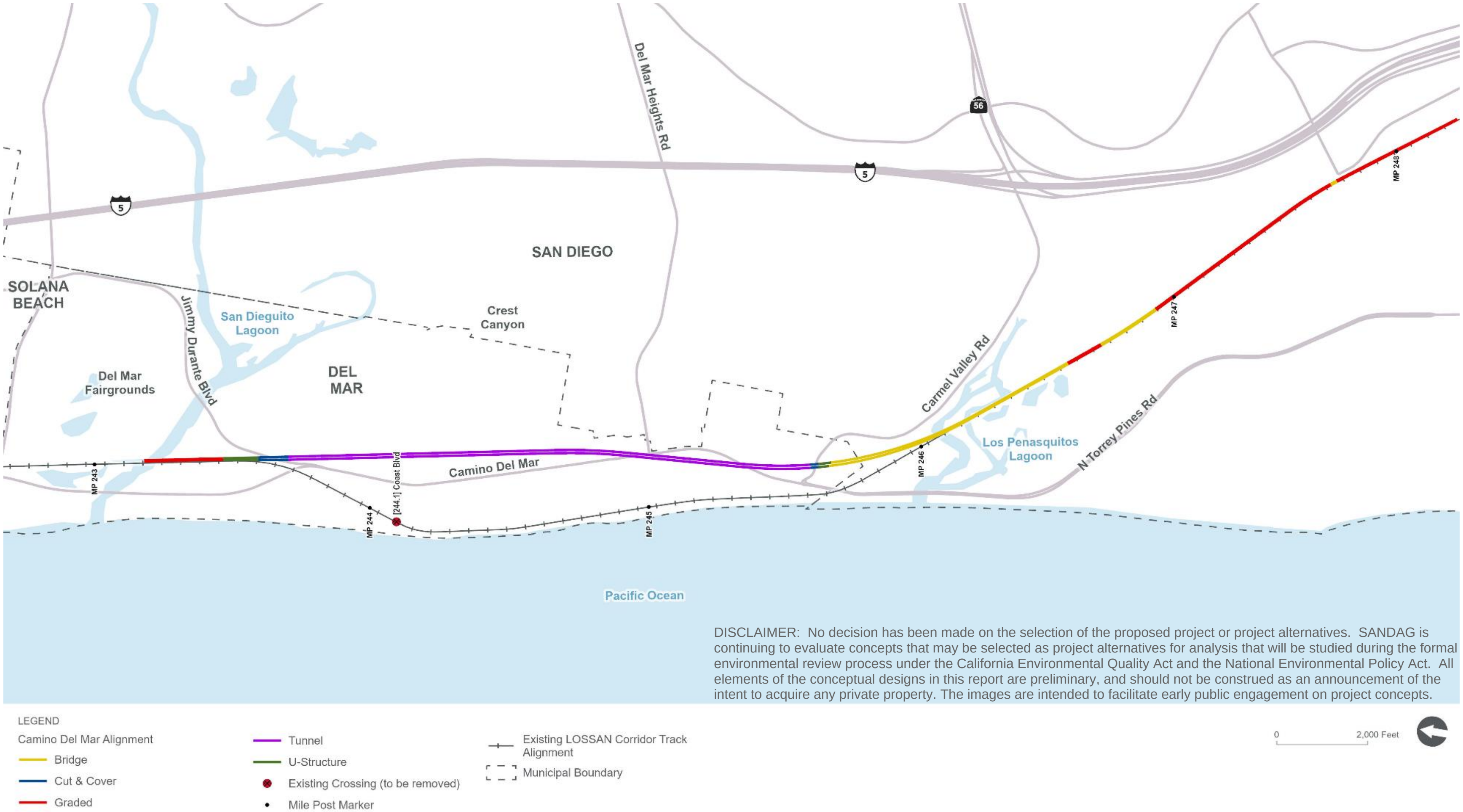
Figure 2. Crest Canyon Higher Speed Alternative



2.2 Camino Del Mar

As shown on Figure 3, the Camino Del Mar Tunnel conceptual alternative would begin the same as the Crest Canyon Higher Speed alternative at the north end. Both conceptual alternatives connect with the San Dieguito River Bridge Replacement Double Track and Special Events Platform, and the north portals are both located north of Jimmy Durante Boulevard, which would be realigned to cross over the tracks. The Camino Del Mar alternative would also consist of a twin-bored tunnel configuration with a U-structure and cut-and-cover section at the entrance. The tunnel continues underground through the residential area of Del Mar but closer to Camino Del Mar. The south portal is located between Carmel Valley Road and North Torrey Pines Road, and the alignment proceeds onto an aerial structure until it ties into the existing right-of-way. The aerial structure is similar in construction to the Crest Canyon Higher Speed Tunnel alternative. The length of the aerial structure is greater than the Crest Canyon alternative. The alignment continues within the existing right-of-way on bridge and berm structures until it ties into the existing tracks just south of Bridge 247.7

Figure 3. Camino Del Mar Alternative



2.2.1 Differences and Similarities between the two conceptual project alternatives

The Crest Canyon Higher Speed and Camino Del Mar conceptual alternatives are both identical at the north end, and both connect to the proposed San Dieguito River Bridge Replacement Double Track and Special Events Platform. The main difference between the conceptual alternatives is the location of the tunnel pathway, the location of the southern portal exit, and the length of the track extended over the Los Peñasquitos Lagoon. These differences could affect various land uses and have slightly different existing noise and vibration environments that could affect the magnitude of change in noise and vibration levels within the existing environment.

3 Fundamentals of Noise

This section provides a general background for noise and ground-borne vibration and defines the terms used to quantify transit projects. Noise and vibration descriptors and the correlation between the source-path-receiver framework provide the foundation for understanding the factors influencing noise and vibration levels.

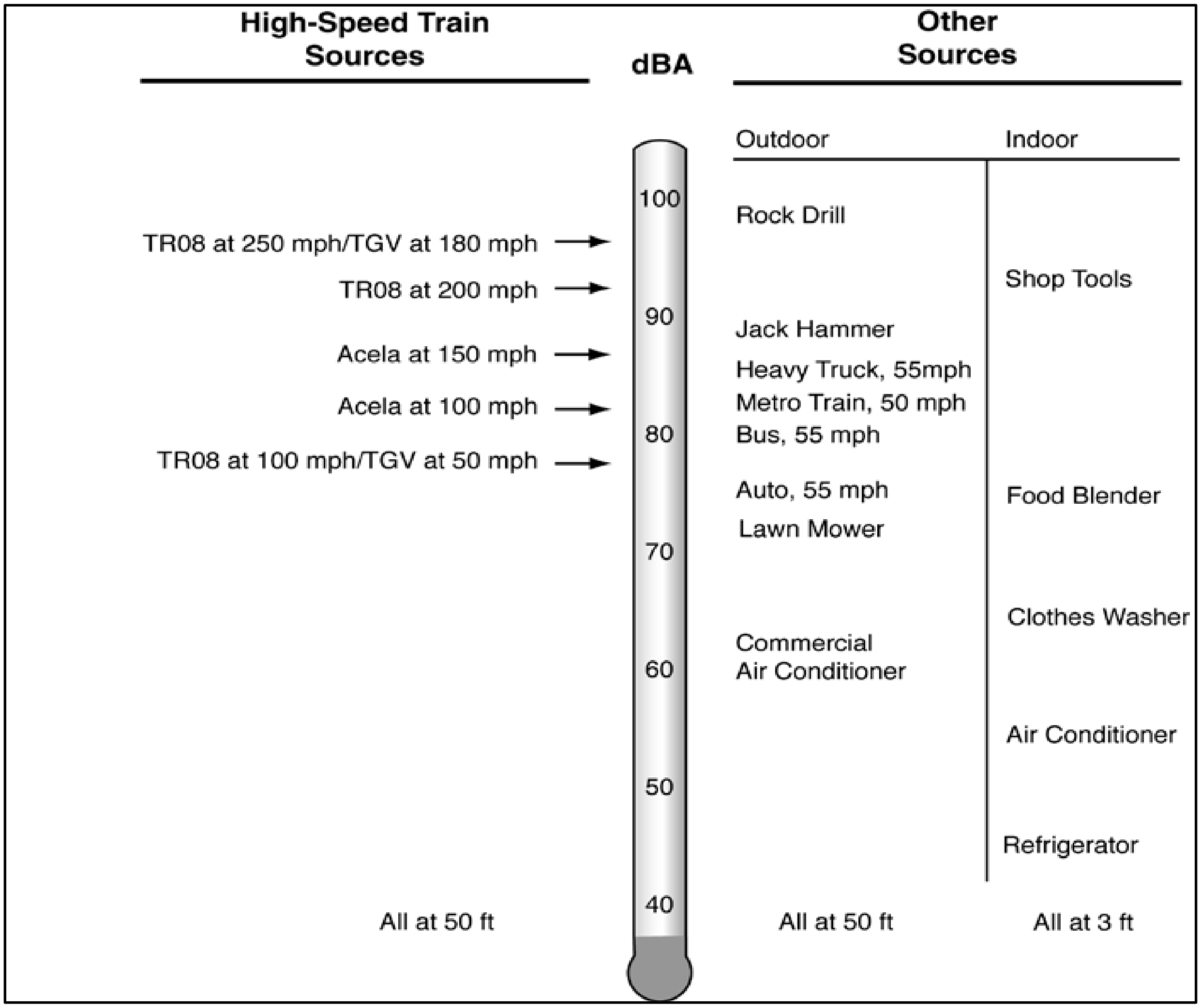
3.1 Noise

Noise can be quantified in many different manners, depending on its temporal (time), tonal (frequency), or loudness characteristics. In general, environmental noise assessment addresses relative changes in noise levels over time and relates those changes with effects on human beings. Noise magnitude is expressed in decibels (dB) units, which is a logarithmic quantity comparing fluctuating air pressure to that of a standardized reference air pressure of 20 micro-pascals (i.e., dB re 20 µPa). For this reason, the noise levels that humans hear are called sound pressure levels. Noise is expressed as a logarithmic quantity because humans are sensitive to relative changes in noise levels. To illustrate, humans can barely perceive a change in noise levels of +/- 3 dB, can easily perceive a change of +/- 5 dB, and generally perceive a change of +/- 10 dB as a doubling or halving in noise levels.

Concerning tonal qualities (frequency), a frequency-weighting adjustment has been standardized to account for human auditory response over the audible frequency range of approximately 20 hertz to 20,000 hertz. Humans are less sensitive to low-frequency noise ranges, exhibit a maximum sensitivity to tones in mid-frequency ranges, and are somewhat less sensitive at higher frequency ranges. This weighted frequency adjustment is referred to as "A-weighting," with results expressed as A-weighted decibels (dBA).

The A-weighted noise level is the basic descriptor for environmental noise. Typical A-weighted noise levels are illustrated on Figure 4.

Figure 4. Typical Outdoor and Indoor Noise Levels



Source: Federal Railroad Administration (FRA) 2012

The following single-number descriptors, all based on the A-weighted sound level as the fundamental unit, are commonly used for environmental noise measurements, computations, and assessment:

- The **sound exposure level (SEL)** describes a receiver's cumulative noise exposure from a single noise event. The total A-weighted sound energy represented during an event is normalized to a 1-second interval. For a vehicle passby, the time interval over which the SEL is evaluated includes all the acoustic energy related to the event, extending over time when the sound level is at least 10 dB below the highest sound level during the passby. SEL is the primary descriptor of rail vehicle noise emissions and an intermediate value in calculating Leq(h) and Ldn (defined below).
- The **hourly equivalent sound level [Leq(h)]** describes a receiver's cumulative noise exposure from all events over 1 hour. The underlying metric for calculating Leq(h) from single noise events during that 1 hour is SEL. Leq(h) is used to assess noise for non-residential land uses. For assessment, Leq is computed for the loudest operating hour during the hours of noise-sensitive activity.

- The **day-night sound level (Ldn)** describes a receiver's cumulative noise exposure from all events over 24 hours. The basic unit used in calculating Ldn is the Leq(h) for each hour. It may be thought of as noise exposure, totaled after increasing all nighttime A-Levels (between 10 p.m. and 7 a.m.) by 10 dB. Every noise event during the 24 hours increases this exposure, louder events more than quieter events, and events that are of longer duration more than briefer events. Ldn is used to assess noise for residential land uses. Typical community Ldns range from approximately 50–70 dBA, where 50 represents a quiet noise environment and 70 is a noisy one.

3.2 Vibration

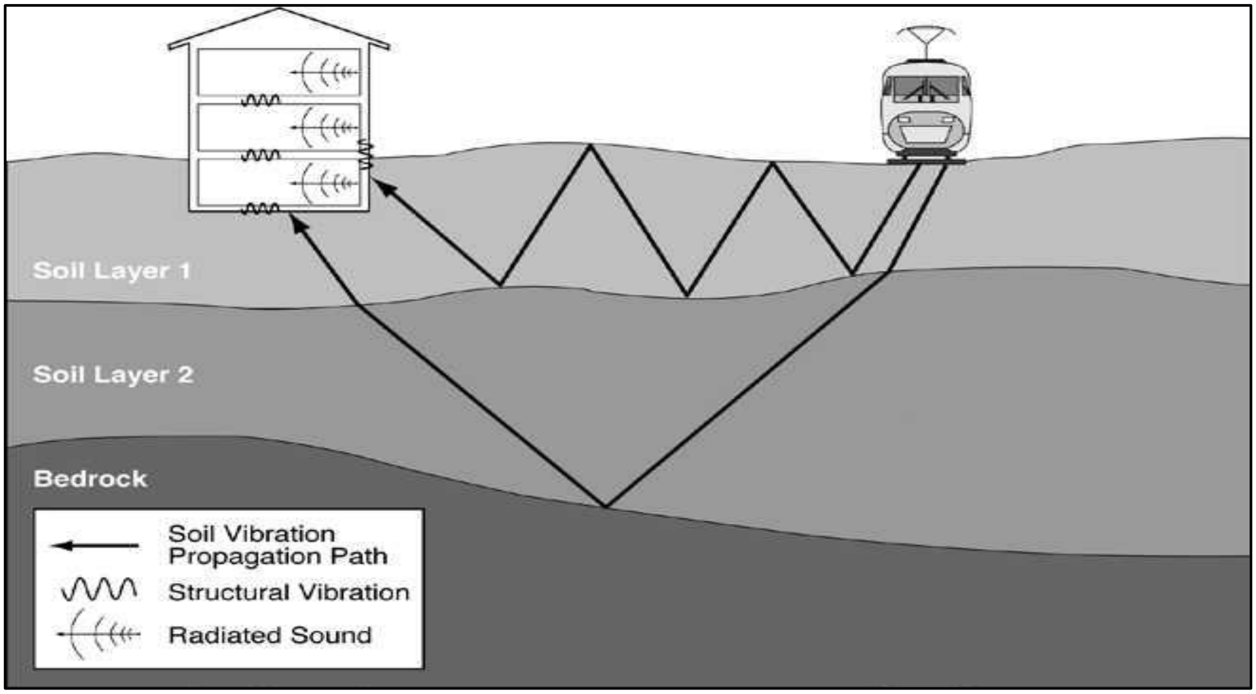
Ground-borne vibration is an oscillatory motion described in terms of the displacement, velocity, or acceleration of the motion. Each of these measures can be further described in terms of amplitude. Displacement is the easiest descriptor to understand. It is simply the distance that a vibrating point moves from its static position (i.e., its resting position when the vibration is not present). The velocity describes the instantaneous speed of the movement, and acceleration is the instantaneous rate of change of the speed.

Although displacement is fundamentally more straightforward to understand than velocity or acceleration, it is rarely used for describing ground-borne vibration for the following reasons:

- Human response to ground-borne vibration correlates more accurately with velocity or acceleration
- The effect on buildings and sensitive equipment is more accurately described using velocity or acceleration
- Most transducers used in the measurement of ground-borne vibration measure either velocity or acceleration. Therefore, for this study, velocity is the fundamental measure used to evaluate the effects of ground-borne vibration

. As with sound, vibration attenuates as a function of the distance between the source and the receptor. Vibration caused by trains moving along a transit structure, such as at-grade ballast and tie track, radiates energy into the adjacent soil. Buildings respond differently to ground vibration depending on the type of foundation, the mass of the building, and the building's interaction with the soil. Once inside the building, vibration propagates throughout the building with some attenuation with distance from the foundation and amplification due to floor resonances. The basic concepts for rail system-generated ground vibration are illustrated on Figure 5.

Figure 5. Propagation of Ground-Borne Vibration into Buildings



Source: FRA 2012

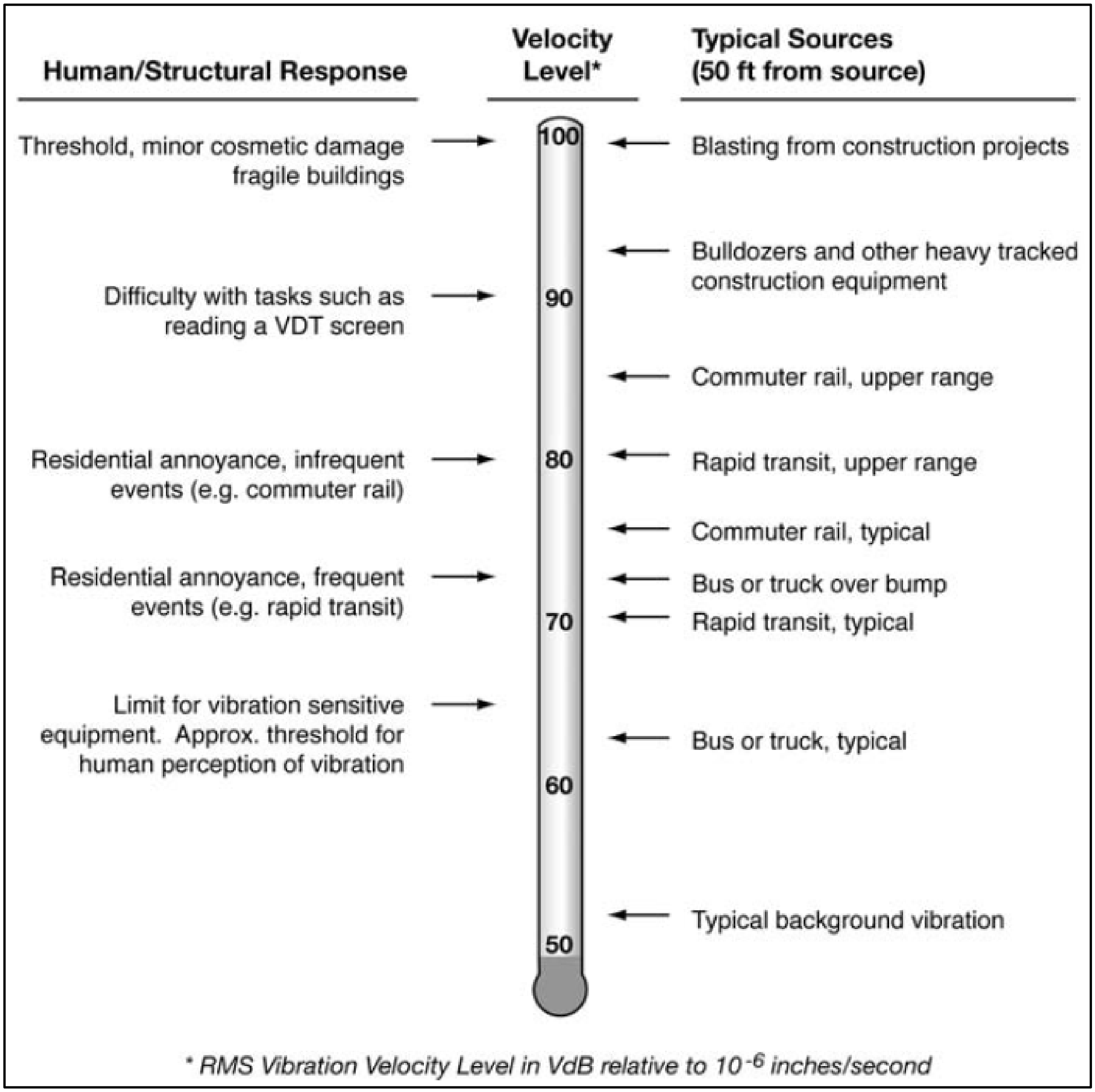
Vibration magnitude can be described using various quantities depending on the intent of the analysis and the type of sensitive receptor being evaluated. Per FRA procedures, all vibration measurements and predictions are in the form of energy-averaged root mean square levels. Root mean square is defined as the average of the squared amplitude of the vibration signal. The FRA uses a logarithmic scale to describe vibration levels; the abbreviation VdB is used for vibration decibels by the FRA. Vibration can also be expressed as the peak particle velocity, which is generally used to evaluate whether vibration has the potential to cause damage to fragile building structures. Peak particle velocity is typically expressed in inches per second. However, the FRA exclusively assesses vibration impacts using VdB.

The potential adverse effects of rail transit ground-borne vibration are as follows:

- **Perceptible Building Vibration:** This is when building occupants feel the vibration of the floor or other building surfaces. Experience has shown that the threshold of human perception is around 65 VdB and that vibration that exceeds 75–80 VdB may be intrusive and annoying to building occupants.
- **Rattle:** The building vibration can cause rattling of items on shelves or hanging on walls and various rattles and buzzing noises from windows and doors.
- **Reradiated Noise:** Ground-borne noise refers to the vibration of room surfaces that radiate sound waves that may be audible to humans. When audible ground-borne noise occurs, it sounds like a low-frequency rumble. The ground-borne noise is usually masked by the normal airborne noise radiated from the transit vehicle and the rails for surface rail systems.
- **Damage to Building Structures:** Vibration from rail systems is usually one to two orders of magnitude below the most restrictive thresholds for preventing building damage. However, fragile structures may be susceptible to damage if the tracks are in sufficient proximity to the structure.

Figure 6 shows typical root mean square vibration velocity levels from rail and other sources and the human and structure response to such levels.

Figure 6. Typical Vibration Velocity Levels



Source: FRA 2012

4 Noise and Vibration Impact Criteria

The criteria used in evaluating noise and vibration impacts from high-speed ground transportation is based on maintaining a noise environment considered acceptable for land uses where noise and vibration may have an effect. These criteria take into account the unusual characteristics of high-speed rail operations. These criteria are adapted from criteria developed by FRA for rail noise sources operating on fixed guideways or at fixed facilities. It is important to understand acceptable and unacceptable noise and vibration levels and what is considered a noticeable change.. This section presents federal and state laws and regulations applicable to noise and vibration affected by the conceptual project alternatives under evaluation.

4.1 Federal Railroad Administration

4.1.1 Noise and Vibration Impact Assessment Guidelines

The FRA guides the evaluation of noise and vibration impacts from construction and operation of high-speed trains in the *High-Speed Ground Transportation Noise and Vibration Impact Assessment* (FRA guidance manual) (FRA 2012). The manual includes prediction methods, assessment procedures, and impact criteria for noise and vibration.

The FRA noise impact criteria are based on maintaining a noise environment considered acceptable for land uses where noise may have an impact. Land-use type also factors into determining an impact. Noise criteria have been established for the various types of receptors individually because not all receptors have the same noise sensitivity. Table 1 summarizes the three land-use categories used by the FRA.

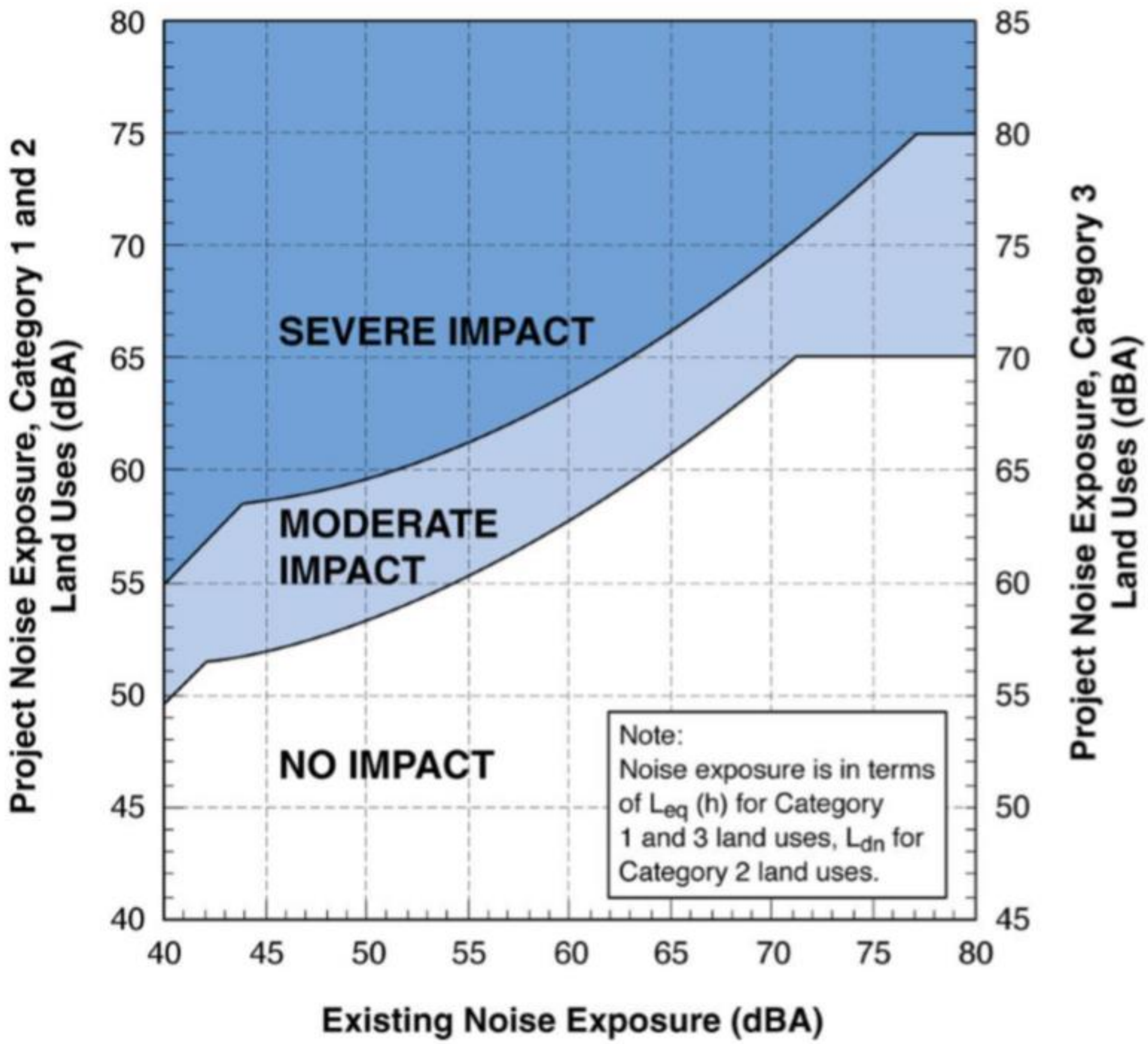
Table 1. Federal Railroad Administration Land Use Categories for Noise Exposure

Land Use Category	Descriptor	Description
1	Outdoor Leq(h)	Tracts of land where quiet is an essential element in their intended purpose. This category includes lands set aside for serenity and quiet, and such land uses as outdoor amphitheaters and concert pavilions, and national historic landmarks with significant outdoor use. Also included are recording studios and concert halls.
2	Outdoor Ldn	Residences and buildings where people normally sleep. This category includes homes, hospitals, and hotels where a nighttime sensitivity to noise is assumed to be of utmost importance.
3	Outdoor Leq(h)	Institutional land uses with primarily daytime and evening use. This category includes schools, libraries, theaters, and churches, where it is important to avoid interference with such activities as speech, meditation, and concentration on reading material. In addition, places for meditation or study associated with cemeteries, monuments, and museums can be considered in this category. Specific historical sites, parks, campgrounds, and recreational facilities are also included.

Source: FRA 2012
Notes:
Leq(h)=hourly equivalent sound level; Ldn=day-night sound level

The FRA noise impact criteria for human annoyance, presented on Figure 7, are based on comparing existing outdoor noise levels and future outdoor noise levels from the project. They incorporate both absolute criteria, which consider activity interference caused by the high-speed train project alone, and relative criteria, which consider annoyance because of the change in the noise environment caused by a project. The FRA noise impact criteria specify a comparison of future with existing noise levels because comparing a projection with an existing condition is more reflective of an impact than a comparison of build and no-build alternatives.

Figure 7. Typical Vibration Velocity Levels



Source: FRA 2012

Noise level increases are categorized as no impact, moderate impact, or severe impact. Moderate and severe impacts are defined as follows:

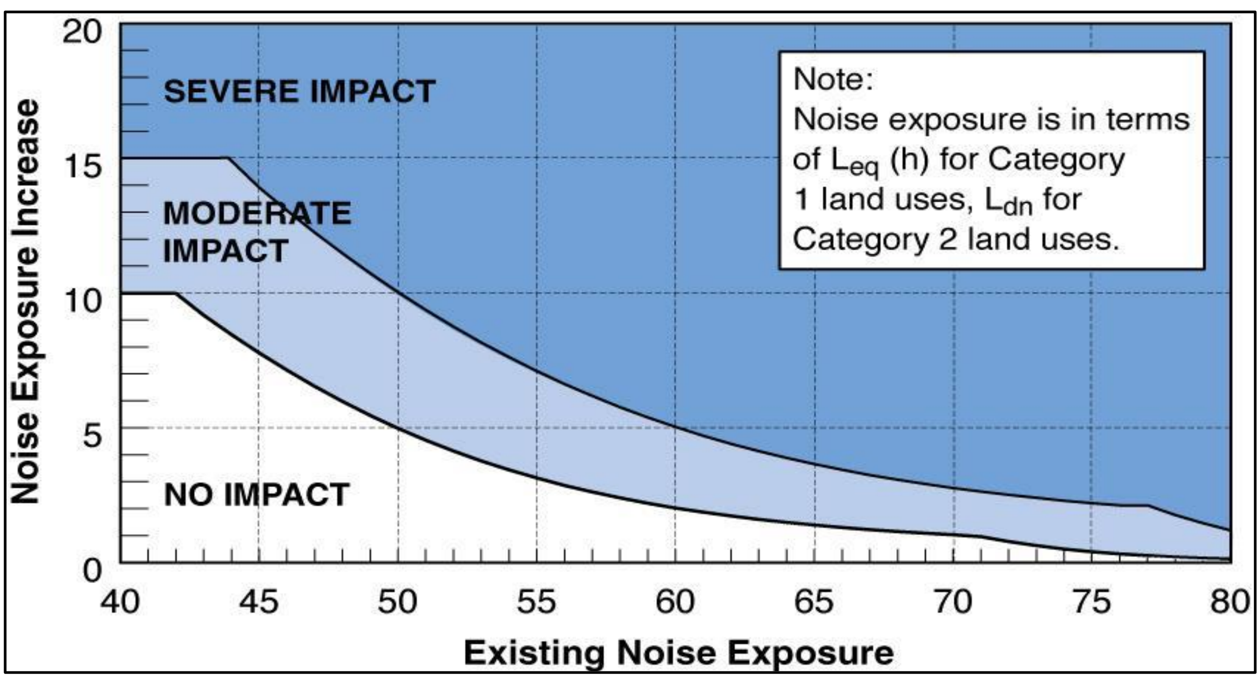
- **Moderate impact**: The change in noise level is noticeable to most people but may not be sufficient to cause strong, adverse reactions from the community. Project-specific factors would be considered to determine the magnitude of impact and the need for mitigation, including the number of affected noise-sensitive sites, the existing level of noise exposure, and the costs associated with mitigation.
- **Severe impact**: Project-generated noise in the severe impact range can be expected to cause a substantial percentage of people to be highly annoyed by the new noise levels. The FRA

policy is to implement noise mitigation for sensitive receptors experiencing severe impacts unless there are truly extenuating circumstances that prevent implementation.

In addition, FRA criteria are presented in terms of relative levels for evaluating the total future noise exposure increases, or increases in combined noise exposure, from the conceptual project alternatives. If the existing noise was dominated by a source that changed because of the project, it would be incorrect to add the project noise to the existing noise. This project proposes to alter transit operations in an existing corridor (i.e., shifts the location or profile of existing passenger or freight tracks and potential changes in vehicle technology). Therefore, the cumulative assessment method would be used for the proposed project because it allows for the relative form of the noise criteria to be applied. Figure 7 illustrates the relative form of the criteria as they apply to Category 1 and 2 land uses. These criteria are based on the increase of the existing ambient noise level associated with project operations. These criteria are applied to the outside of building locations in noise-sensitive areas.

Consider a hypothetical residential property (Category 2) with an existing noise exposure of Ldn 60 dBA. The noise exposure resulting from the project plus regional growth and other planned projects could result in a project noise level exposure of Ldn 65 dBA. Combining the project noise with the existing noise level would result in a total combined noise exposure of Ldn 66 dBA or a potential increase of 6 dBA over the existing noise level. Using Figure 8, start with the horizontal axis at 60 dBA for the existing condition to draw a vertical line, then draw a horizontal line from 6 dBA on the left-hand axis. The intersection of these two lines determines the severity of impact. In this hypothetical example, the intersection of these two lines would fall in the severe impact range.

Figure 8. Increase in Cumulative Noise Levels Allowed by Criteria (Land-Use Cat. 1 & 2)



Source: FRA 2012

The FRA criteria for environmental impact from ground-borne vibration are based on the maximum levels for a single event (i.e., one train pass-by). The criteria assume that the potential for annoyance increases as the number of events increases for any given vibration level. The criteria distinguish between projects with frequent, occasional, and infrequent events. Frequent events are defined as more than 70 events per day, occasional events are defined as 30 to 70 events per day, and infrequent events are defined as fewer than 30 events per day. The vibration criteria depend on land use type. The three primary land use categories and the associated criteria are provided in Table 2. It is noted that the guidance manual also provides additional criteria for more specific land uses such as concert halls, television studios, recording studios, auditoriums, and theaters. These criteria are not presented in this report as such land uses were not identified in the project vicinity.

Table 2. Ground-Borne Vibration and Ground-Borne Noise Impact Criteria

Land Use Category	Ground-Bourne Vibration Impact Levels (VdB re 1 µin/sec)			Ground-Bourne Noise Impact Levels (dB re 20 µPascal)		
	Frequent Events[1]	Occasional Events[2]	Infrequent Events[3]	Frequent Events[1]	Occasional Events[2]	Infrequent Events[3]
Category 1: Buildings where vibration would interfere with interior operations	65 VdB[4]	65 VdB[4]	65 VdB[4]	N/A[5]	N/A[5]	N/A[5]
Category 2: Residences and buildings where people normally sleep.	72 VdB	75 VdB	80 VdB	35 dBA	38 dBA	43 dBA
Category 3: Institutional land uses with primarily daytime use.	75 VdB	78 VdB	83 VdB	40 dBA	43 dBA	48 dBA

Source: FRA 2012

Notes:

[1] "Frequent Events" is defined as more than 70 vibration events of the same source per day. Most rapid transit projects fall into this category.

[2] "Occasional Events" is defined as between 30 and 70 vibration events of the same source per day. Most commuter trunk lines have this many operations.

[3] "Infrequent Events" is defined as fewer than 30 vibration events of the same kind per day. This category includes most commuter rail branch lines.

[4] This criterion limit is based on acceptable levels for most moderately sensitive equipment such as optical microscopes. However, vibration-sensitive manufacturing or research will require a detailed evaluation to define the acceptable vibration levels. For example, ensuring lower vibration levels in a building often requires the special design of the heating, ventilation, and air conditioning systems and stiffened floors.

[5] Vibration-sensitive equipment is generally not sensitive to ground-borne noise.

dB=decibels; VdB=vibration decibels

4.1.2 Railroad Noise Emission Compliance Regulations (49 Code of Federal Regulations Part 210)

The FRA's Railroad Noise Emission Compliance Regulations (49 Code of Federal Regulations Part 210) prescribe minimum compliance regulations for the enforcement of Noise Emission Standards for Transportation Equipment; Interstate Rail Carriers (40 Code of Federal Regulations Part 201) adopted by the United States Environmental Protection Agency. Accordingly, the selection of new locomotives for the project must meet the following noise standards at 100 feet: 70 dBA while stationary at idle throttle setting, 87 dBA while stationary at all other throttle settings, and 90 dBA while moving. In addition, rail cars must meet the following noise standards at 100 feet: 88 dBA while moving at speeds of 45 miles per hour or less and 93 dBA while moving faster than 45 miles per hour.

4.1.3 Locomotive Horn Rule (49 Code of Federal Regulations Part 222 and Part 229)

FRA regulations require that engineers sound their locomotive horns when approaching public grade crossings until the lead locomotive fully occupies the crossing. In general, the regulations require locomotive engineers to begin to sound the train horn for a minimum of 15 seconds and a maximum of 20 seconds in advance of public-grade crossings. Engineers must also sound the train horn in a standardized pattern of two long, one short, and one long blast, and the horn must continue to sound until the lead locomotive or train car occupies the grade crossing. Additionally, the minimum sound level for the locomotive horn is 96 dBA, while the maximum sound level (L_{max}) is 110 dBA, both measured at 100 feet forward of the locomotive.

The FRA allows public authorities to establish a quiet zone. A quiet zone is a segment of a rail line within one or several consecutive public road rail crossings where locomotive horns are not routinely sounded, provided sufficient safety measures are implemented at the crossing to prevent/minimize the potential for accidents to occur. Railroad authorities and railroad companies cannot establish quiet zones; only local cities and counties can establish them by applying to FRA.

At a minimum, new quiet zones must be at least 0.5 mile in length and contain at least one public grade crossing (i.e., a location where a public highway, road, or street crosses one or more railroad tracks at grade). In addition, every public grade crossing in a quiet zone must be equipped, at a minimum, with active grade-crossing warning devices consisting of flashing lights and gates. FRA provides this safety requirement as a minimum recommendation; however, additional safety requirements may include, but are not limited to, stationary audible warning signals and median barriers.

5 Methodology

The criteria used in evaluating noise impacts from the conceptual project alternatives are based on maintaining an environment considered acceptable for land uses where noise and vibration may have an effect. Therefore, it is important to understand the general land uses within the project study area and determine where noise and vibration impacts could likely occur.

An initial evaluation assesses the potential noise and vibration impacts from the non-tunnel portions of the conceptual project alternatives. As the conceptual project alternatives are considered at the early phases of development, the initial noise evaluation would identify potential impacts and determine the order of magnitude of these potential impacts on direct future detailed analysis of areas where significant impacts may occur without mitigation. The initial assessment for the project would preliminarily screen the project corridor to identify areas of potential impact for each conceptual alternative within a worst-case screening distance. All noise-sensitive receivers that are close enough to the conceptual project alternatives for noise and vibration impacts would be identified, and existing noise contours would be presented at these locations.

Both FRA and Federal Transit Administration (FTA) guidance documents are applicable as two types of trainsets will be utilizing the corridor, passenger trains and freight trains operating at a maximum speed of 110 miles per hour and 60 miles per hour, respectively. FRA and FTA guidance provides procedures for setting screening distances and evaluating noise and vibration impacts. The FRA guidance document is used when passenger train speeds are 90 miles per hour or higher. Therefore, noise from passenger trains is assessed using the FRA guidance. Freight trains are evaluated using Transit Noise and Vibration Impact Assessment (FTA 2018).

A general assessment will be performed that will include selecting source levels for the high-speed rail technology being considered, estimating existing noise exposure using a simplified procedure, determining potential noise impact based on both FTA and FRA criteria, and preparing an inventory of the potential impacts and mitigation requirements.

6 Existing Conditions

6.1 Noise and Vibration Impact Screening Contours

A majority of the land uses within Del Mar near the project alignments are residential land uses under FTA and FRA Category 2. Category 2 provides the most stringent noise and vibration criteria; therefore, only residential parcels will be identified within the FRA and FTA screening distances as locations that will undergo further evaluation to determine the order of magnitude of potential noise and vibration impacts at these locations. The following sections present FRA and FTA default screening distances for noise and vibration, discuss the adjustments made to represent the project-operating conditions, and present the worst-case screening distance used to identify residential parcels that will undergo further evaluation for impacts.

FRA and FTA provide a screening procedure for noise and vibration that narrows area land uses to those directly affected by the project. Noise default-screening distances are established based on three conditions: corridor type (railroad, highway, or new), existing environment (urban, suburban, or rural land uses with or without intervening buildings), train, and speed regime type.

Both guidance documents allow for adjustments to these default screening distances to support train traffic assumptions more representative of the conceptual project alternatives. As the project is in its early stages of design, an approximation of train traffic was first developed utilizing train traffic details from the San Elijo Lagoon Double Track Project in the City of Encinitas. This project is an adjacent project located a few miles north of this Del Mar project with a similar train fleet. Therefore, it was considered reasonable and appropriate for evaluating noise from trains on the proposed conceptual alignment alternatives. Upon reviewing more recent documents and utilizing feedback obtained through the project development process, the 2035 daily passenger train counts from SANDAG (36 Surfliner and 54 COASTER) are found to be more consistent with the service-level assumptions contained in the LOSSAN Optimization and San Diego Pathing studies. These studies are also consistent with the goals and objectives identified in both the SANDAG Regional Transit Plan as well as the 2018 California State Rail Plan.

Table 3 summarizes train traffic details based on anticipated 2035 service level assumptions from the LOSSAN Optimization Study and San Diego Pathing Study. It should be noted that these studies only identified slots, and the agencies and freight operators have the ultimate authority on schedules.

Additionally, the San Diego pathing study identified capacity for up to 16 freight slots during the daytime but did not consider nighttime slots nor identify final freight counts. BNSF has authority on deciding how many slots they use in the future. BNSF, as a participant in the Project Development Team, has not contradicted the assumption of 11 freight trains.

In consideration of the projected levels of serviced identified by the studies as well as that used in this analysis and obtained from the Environmental Noise Study for the Proposed San Elijo Lagoon Double Track Project in the City of Encinitas (Weiland Acoustics 2014) this information is considered reasonable and appropriate for evaluating noise from trains on the proposed conceptual alignment alternatives.

Schedules for future service have not yet been developed, therefore the table represents a reasonable assumption for nighttime versus daytime slots based on current schedules.

Table 3. 2035 Estimated Operational Passenger and Freight Train Details

Train Type	Total Trains Per Day	Daytime		Nighttime		Speed	Locomotives	Railcars
		Total	Trains/hour	Total	Trains/hour			
Amtrak Surfliner	36	30	2.0	6	0.7	110	1	7
BNSF Freight	11	4	0.3	7	0.8	60	5	118
Coaster	54	48	3.2	6	0.7	110	1	5

Source: Weiland Acoustics 2014; Deutche-Bahn 2022

6.1.1 Noise Screening Assessment

Noise screening distances were adjusted using the FTA General Noise Assessment spreadsheet model for freight trains. The FRA spreadsheet noise model for high-speed passenger rail trains obtains the receiver distance that would reach the lowest threshold of noise impact of 50 dBA Ldn. Table 4 summarizes the results of the adjusted screening distances and compares them with the FTA and FRA default noise screening distances.

Table 4. Comparison of Noise Screening Distances

Noise Screening Technique	Noise Screening Distance (feet)		Operational Basis[3]
	Unobstructed[1]	Obstructed[2]	
FTA			
Default Screening Distance	750	375	Passenger commuter rail mainline 66 daytime trains, 12 nighttime trains, 1 loco 6 cars per train, 55 miles per hour
Adjusted Screening Distance	2,930		Freight train 4 daytime trains, 7 nighttime trains, 5 locomotives, 118 railcars per train, 60 miles per hour
FRA			
Default Screening Distance for FRA High-Speed Passenger Rail	300	200	Railroad corridor, urban/noisy suburban, speed regime II

Table 4. Comparison of Noise Screening Distances

Noise Screening Technique	Noise Screening Distance (feet)		Operational Basis[3]
	Unobstructed[1]	Obstructed[2]	
Adjusted Screening Distance for High-Speed Passenger Rail	1,175		Passenger rail 78 daytime trains, 12 nighttime trains, 1 locomotive, 6 railcars, 110 miles per hour

Source: FTA 2018; FRA 2012

Notes:

[1] Assumes no buildings are between the source and the receiver.

[2] Assumes buildings are between the source and receiver

[3] Operational parameters for adjusted screening distance are train traffic details from the San Elijo Lagoon Double Track Project and Deutche-Bahn 2022. Default distances are from the FTA and FRA guidance documents.

FRA=Federal Railroad Administration; FTA=Federal Transit Administration

Noise screening distances in Table 4 show a wide range in distances. These ranges of distances are attributable to the operating conditions of freight trains. The FTA default screening operating parameters exclude freight train operation, whereas the adjusted screening distances account for higher freight traffic operating under the conceptual project alternatives. As a result, a greater screening distance occurs due to the long, fast freight trains and the relatively high number of nighttime freight trains. Therefore, as a conservative measure, the adjusted FTA screening distance of 2,930 feet was selected to identify affected land uses. Figure 9 and Figure 10 show the noise screening contour for the project area.

6.1.2 Vibration Screening Assessment

FTA and FRA also provide guidance for determining vibration screening distances. As shown in Table 5, the FTA default distance of 200 feet is provided for the residential land use category for conventional commuter railroad (diesel-electric locomotives). The FRA vibration default screening distance of 220 feet is provided for frequent trains (more than 70 high-speed passbys per day, and speeds between 100-200 miles per hour) under normal vibration propagation conditions. In addition, FTA and FRA guidance allows for an adjustment factor (1.5 and 2.0, respectively) to be applied to increase default screening distances to account for potential land uses that would be affected by efficient propagation of vibration. The project team reviewed available soil data in the vicinity of the proposed conceptual alignment alternatives and determined the following.

- The track in the tunneled portion of both alignments would sit atop the Tertiary-age Torrey Sandstone or Tertiary-age Delmar Formation. The Torrey Sandstone is comprised mainly of quartz and some feldspar grains, whereas the Delmar Formation is composed mostly of mudstone and siltstone with some sandstone. Both are consolidated bedrock units that could be relatively efficient at propagating ground-borne vibration.
- The track in the at-grade portions of both alignments would sit atop fill, which presumably consists of somewhat compacted unconsolidated materials. These fill materials and the unconsolidated native materials that the fill would sit upon (primarily Young Alluvium and Paralic Estuarine Deposits) are expected to be relatively inefficient at propagating ground-borne vibration.

FTA and FRA screening distances were increased by a factor of 1.5 and 2.0, respectively, for the entire length of both alignments to conservatively account for efficient propagation of ground-borne vibration. Table 5 presents the FTA and FRA adjusted vibration screening distances that account for efficient vibration propagation.

Table 5. Comparison of Ground-borne Vibration Screening Distances

Method	Distance from Railroad CL (feet)
	Land Use Category 2
FTA	
Default FTA Conventional Commuter Railroad	200
Adjusted FTA	300
FRA	
FRA Default Frequent Events	220
Adjusted FRA Frequent Events	440

Source: FTA 2018; FRA 2012
Notes:
FRA=Federal Railroad Administration; FTA=Federal Transit Administration

Comparing the FRA and FTA default and adjusted screening distances, the largest ground-borne vibration screening distance of 440 feet was selected to identify affected land uses. Screening assessment results are summarized in Table 6, which shows the number of residential parcels within the noise and vibration screening distances relative to each conceptual alignment alternative.

Table 6. Number of Residential Parcels Within Screening Distances

Conceptual Alternative Alignment	Camino Del Mar	Crest Canyon
Vibration Screening	1,011	854
Noise Screening	2,827	2,398

Figure 9 shows the noise and vibration screening contours, estimated existing noise levels, and residential parcels within the Crest Canyon alternative alignment screening distances. Figure 10 shows the same noise and vibration screening contour information for the Camino del Mar alternative alignment.

The noise and vibration screening assessment results indicate the presence of residential parcels within the non-tunnel portions of the proposed alternate alignments. Therefore, the conceptual project alternatives would undergo additional assessments to evaluate the potential for noise impacts associated with each conceptual alignment alternative.

Figure 9. Crest Canyon Higher Speed Alternative Noise and Vibration Contour Screening Distances

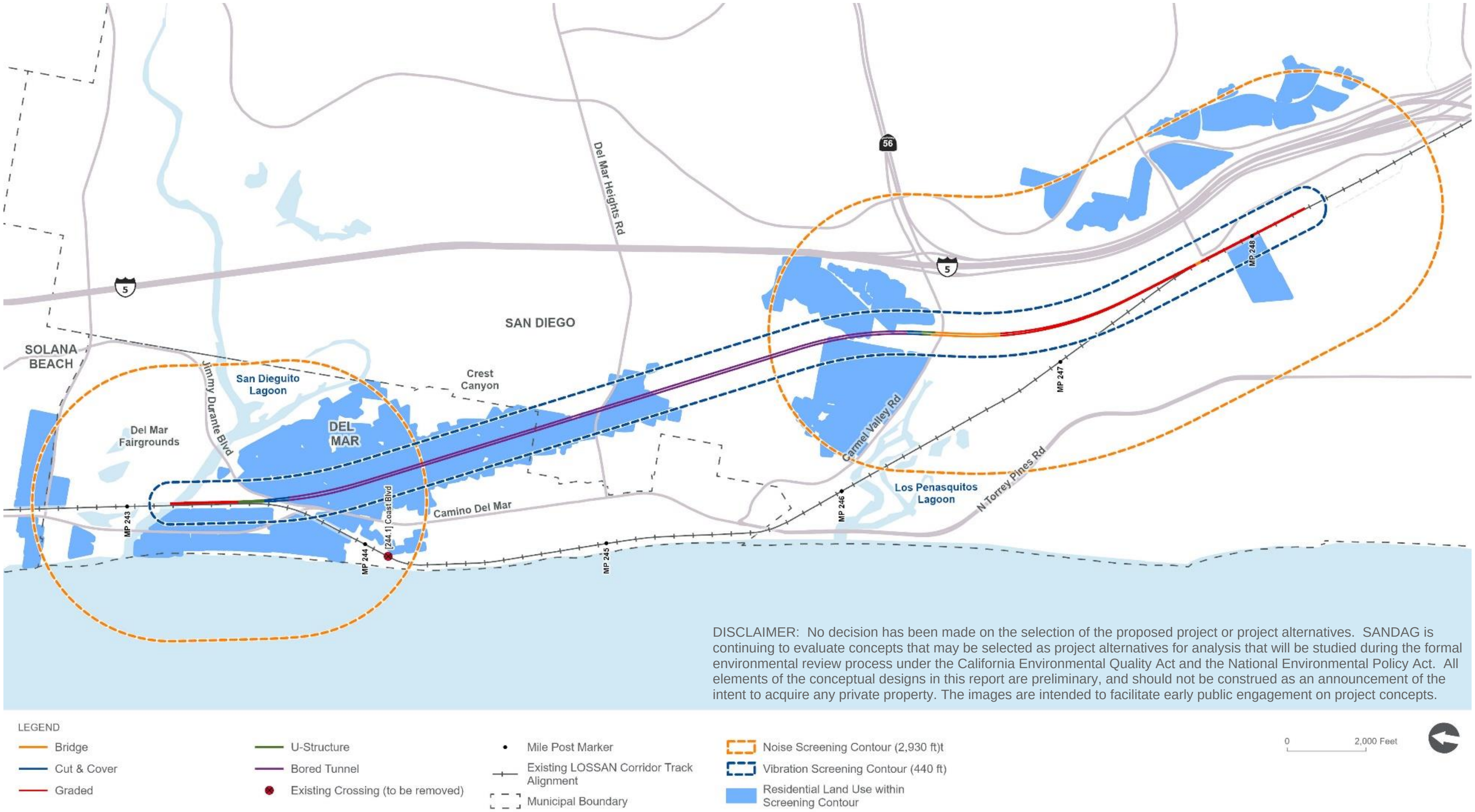
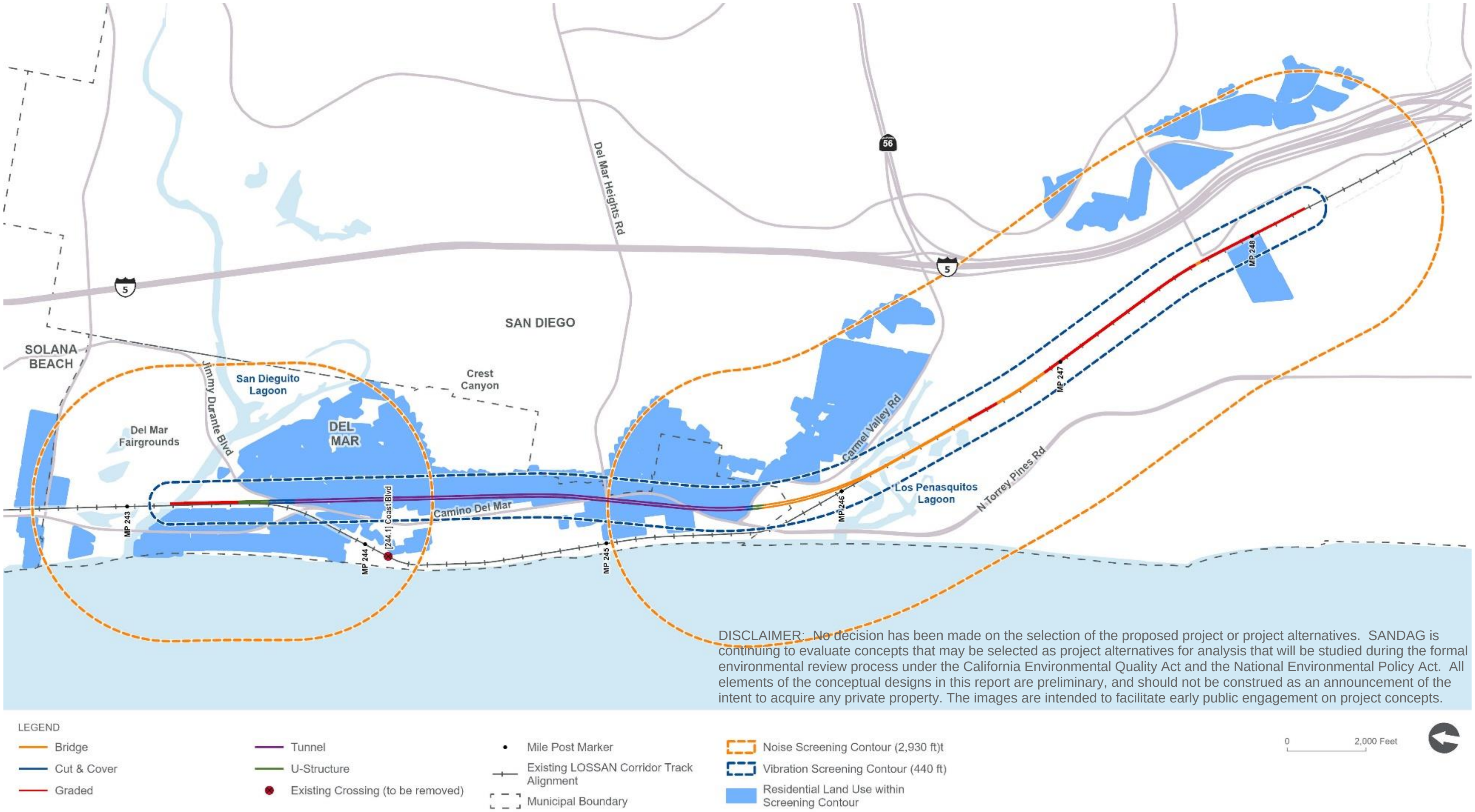


Figure 10. Camino Del Mar Alternative Noise and Vibration Contour Screening Distances



6.2 Summary of Del Mar Land Use Types

The Crest Canyon Higher Speed and the Camino Del Mar conceptual alternatives are located primarily within the City of Del Mar, with the southern portals within the City of San Diego, west of Interstate 5. The noise environment between these conceptual alternatives is common, with slight variations in affected land uses. Both conceptual alternatives begin just south of the Del Mar Fairgrounds at the San Dieguito River Bridge and continue through the City of Del Mar east of Camino Del Mar Road until 4th Street. At 4th Street, the conceptual alternatives differ in direction. The Camino Del Mar alternative moves west to connect with the existing tracks near McGonigle Road. In contrast, the Crest Canyon Road Higher Speed alternative continues east through the Torrey Pines Reserve until Carmel Valley Road. Then it enters the Los Peñasquitos Lagoon to reconnect to the existing tracks near Carmel Mountain Road.

For the two conceptual alternatives, land use types affected are similar between the San Dieguito River Bridge to 4th Street, and the area represents a residential neighborhood. Residential land uses are categorized by low-density (single-family) or medium-density mixed (multi-family) residential land uses. The area also consists of parklands, public facilities, commercial buildings, retail establishments, and hotels to accommodate tourism and general business activities. Camino Del Mar Road provides a distinct physical separation between these different land uses.

6.3 Estimated Existing Noise Levels

Existing noise levels were estimated using methods found in Table 4-5 of the Noise Levels Defining Impact for Transit Projects of the FTA guidance document (FTA 2018). Existing noise levels were overlaid inside the noise screening contours in 5 dBA increments, as shown on Figure 9 and Figure 10. FTA and FRA noise impact thresholds are a function of existing noise levels. As existing noise levels increase, the maximum allowable project related increase grows smaller. Table 7 shows a subset of the FTA/FRA noise impact thresholds for the range of estimated noise levels shown in the screening assessment figures.

Table 7. Noise Impact Thresholds for Land Use Category 2 – Residential Parcels

Existing	Noise Impact Threshold Ldn (dBA)		Maximum Allowable Increase (dBA)	
Ldn	Moderate	Severe	Moderate	Severe
50	53	60	5	10
55	55	61	3	7
60	58	63	2	5
65	61	66	1	4
70	64	69	1	3

Source: FTA 2018
Notes:
dBA=A-weighted decibels; Ldn=day-night sound level

6.4 Existing Vibration Environment

The primary sources of vibration within the Del Mar area are mainly associated with train traffic. It is unusual for vehicular traffic, such as buses and trucks, to generalize a perceptible vibration level (FRA 2012) along Camino Del Mar Road. Several vibration studies along the rail corridor have shown vibration levels at biological and residential land uses near the coast in close proximity (within 50 feet or less) of existing tracks, grade crossings, and concrete bridge abutments to range in the range of 75 to 80 VdB. Vibration levels greater than 100 feet range from 65 to 75 VdB (Entech Consulting Group 2019 and 2020).

7 Potential Project Impacts

Based on the conceptual design and preliminary analysis, the source-path-receiver framework associated with these activities has the greatest potential effect on noise and vibration noise levels. First, the noise sources from high-speed trains are influenced by a train passby and its operating characteristics (e.g., speed). The path component includes sound attenuation with increasing distance from the source, excess attenuation resulting from atmospheric absorption and ground effects, and acoustic shielding by terrain, sound barriers, or intervening buildings. Second, the receptor is the noise-sensitive land use (e.g., residence, hospital, or school [referred to as sensitive receptors]) exposed to noise from the source. Finally, the receiving land use determines noise sensitivity. These three elements are taken into account in assessing potential noise and vibration impacts.

Sources of potential noise and vibration from the conceptual project alternatives occur at several locations along the alignment. These locations include the north portal entrance where trains are traveling at grade on double tracks through a U-structure; cut-and-cover portal at the north end of the tunnel; and at the south portal when trains transfer through a similar U-structure and cut-and-cover configuration as the north portal to bring trains onto a concrete aerial structure. Both the north and south portals will also have vent shafts with vent fans to accommodate ventilation of the tunnel.

At this preliminary design level, a general assessment would be performed to quantitatively assess potential noise and vibration impacts and evaluate potential mitigation measures that can be incorporated into the design to reduce impacts.

7.1 Potential Operational Noise Impacts

7.1.1 Train Passbys Near Portals

Tracks Near Portals

The conceptual project alternatives were evaluated using the FTA and FRA general noise assessment approaches. Train speeds for Amtrak and Coaster (110 miles per hour) exceed the maximum speed regime of the FTA guidelines (<90 miles per hour); therefore, this analysis used the FRA (2012) methodology to calculate potential noise from those trains. Potential noise from BNSF freight trains was calculated using the locomotive and railcar SEL values from the CREATE (Chicago Region Environmental and Transportation Efficiency) program and the FTA’s 2018 locomotive and railcar noise emissions equations. Both sets of calculations determined the potential noise level in Ldn at 50 feet, propagated the potential noise level to fixed distances, and determined the overall calculated potential Ldn at that distance.

Train noise is partly a function of speed, and fast trains are noisier than slower trains. However, faster trains have shorter durations of passbys and therefore produce lower overall noise exposure. Longer, slower trains have longer passby durations and therefore produce higher overall noise exposure. The Ldn metric is dominated by events that happen at nighttime because, by definition, nighttime noise events receive an additional 10-decibel penalty to account for the additional nuisance associated with nighttime noise events. General noise assessment results reflect these concepts very clearly. Modeled Ldn results for fast passenger trains are lower than the modeled Ldn from the BNSF freight. Table 8 summarizes the combined potential passenger and freight train noise levels at fixed distances from the rail line.

Table 8. Calculated Potential Train Noise Levels at Fixed Distances

Ldn in dBA

Distance (feet)	Calculated Ldn in dBA			
	Amtrak	BNSF	Coaster	Overall Ldn
100	57	73	57	73
500	43	59	43	73
1000	37	53	37	53
2900	28	43	28	58

Notes:
dBA=A-weighted decibels; Ldn=day-night sound level

These calculations assumed flat ground. The ground factor was assumed to be zero for conservatism (propagation path between the rail line and the receivers) and did not account for a track on a structure. The overall potential train noise levels were calculated at each residential parcel within the noise screening distance, compared those results with the maximum allowable noise levels (FTA noise impact thresholds), and determine where moderate or severe noise impacts are projected to occur without mitigation. Noise impacts were determined using the equations in an appendix of the FTA guidance document (FTA 2018).

Table 9 summarizes the analysis of potential noise for both conceptual alignment alternatives.

Table 9. Analysis of Potential Noise

Conceptual Alternative Alignment	No Impact	Moderate Noise Impacts	Severe Noise Impacts
Camino Del Mar	0	103	2,724
Crest Canyon	6	112	2,280

A more detailed three-dimensional noise model should be developed during the environmental review and design phases of the Project that incorporates terrain and building-induced shielding, and other potential mitigation measures.

Onset Noise

The existing noise environment currently experiences noise from train passby along the single at-grade ballast track from San Dieguito Bridge near Jimmy Durante Boulevard. Both conceptual alternatives would bring passenger trains operating at maximum speeds of 110 miles per hour traveling on Class 6 tangent double tracks from the San Dieguito Bridge onto the U-structure at the north portal. This above-ground portion of the Crest Canyon Higher Speed alternative follows the existing railroad track right-of-way; therefore, noise associated with operations in Zone 1 would continue to experience train noise. FRA guidance measured data for various diesel locomotive high-speed trains indicate that SEL would range from in the low to mid-90 dBA range. Current trainsets have a similar SEL. It is anticipated that the duration of the SEL of single-train events in this area would decrease in duration due to increased train speeds. Airborne noise would range from somewhat perceptible to imperceptible, depending on the train's distance from the portal.

As trains approach and exit the north portal of the tunnel, increased train speeds can play a factor in generating onset rates (startle). Higher train speeds have the potential to generate rapid onset rates. The onset rate is the average rate of increasing sound pressure levels in dBs per second during a single noise event. Onset rates occur when high-speed trains speeds over 100 miles per hour rapidly approach receivers near the tracks. Sounds with fast onset rates are more annoying than sounds with a less rapid variation or steady noise with the same maximum noise level. When onset rates exceed approximately 30 dBs, people can be startled or surprised by the sound's sudden onset. Therefore, the onset rate of 30 dBs is the basis for establishing distances within which startle is likely to occur. FRA guidance provides a linear correlation between speed and distance where startle impacts arise. For example, commercial and residential land uses are located within 50 feet of the at-grade track at the north tunnel portal; however, a maximum speed of 110 miles per hour will not generate onset impacts greater than 30 dBs at this distance. There are no station platforms or pedestrian bridges near the proposed north or south portals; therefore, rapid onset noise impacts will not occur.

The south portal mirrors the configuration at the north portal except for an aerial structure connecting the U-structure to existing tracks in the Los Peñasquitos Lagoon. Wildlife habitat and commercial land uses are located near the south portal, which contains low-density residential and multi-family land uses, parkland land uses, and commercial property..

Noise reduction can be achieved from the concrete barriers along the tracks as part of the U-structure. The U-structure will create an attenuation effect, similar to tracks located within a trench. The concrete walls serve the same function as barrier walls in breaking the line -of -sight between source and receiver along the approximately 90 feet section of the track to the north and 20 feet of track at the south portal entrance.

7.1.2 Ventilation System

Vent fans and the ventilation shaft are potential noise sources. Therefore, the exhaust location of the vent fans could affect noise levels at residential land uses. At this preliminary design level, a specific location has not been selected for the vent shaft. Still, vent fans will be located within the shaft and at either end of the portals with no intermediate vent shafts required along the tunnel portion of the alignment. This configuration is called a transverse ventilation system that moves exhaust along the length of the tunnel to the portals. The vent shaft would be located approximately 100 feet below the ground surface. The vent fans within the vent shaft would be housed inside a room with attenuators and a louver that would reduce vent noise. Vent fans within the tunnel would operate in case of an emergency during a fire event in the tunnel. There is also potential for fans to run as needed to clear the diesel exhausts from a freight train passing through the tunnel; however, additional ventilation modeling in the next phase would be needed to determine when, and if, this is required. The vent fan design criteria utilized maximum sound power levels for frequencies ranging from 63 hertz to 4,000 hertz. The maximum vent fan noise level within the tunnel at full speed and half speed would be 89 dBA and 78 dBA, respectively. The maximum noise level at street level would not exceed 92 dBA. These potential noise levels would need to be further abated to reduce noise levels through the fans' attenuators and room enclosures. In addition, louvers would vent the exhaust. This additional noise abatement would need to reduce noise levels to meet City of Del Mar and City of San Diego noise requirements for exterior noise near residential communities.

7.2 Potential Operational Vibration Impacts

7.2.1 Potential Vibration At Grade

A general vibration assessment approach was used to estimate vibration levels for the residential land uses within the 440-foot screening contour. The FRA Generalized Ground-Borne Vibration Curve was used to obtain the vibration level at distances of 50, 100, 200, and 300 feet at a speed of 150 miles per hour. Adjustments were made to obtain the base vibration level to account for a passenger train speed of 110 miles per hour, resilient fasteners, and efficient propagation of vibration. Similarly, the FTA Base Ground Surface Vibration Curve for freight trains was used to obtain the base vibration level. Again, adjustments were made to the base vibration level to account for the freight train speed of 60 miles per hour and efficient propagation of vibration. Table 10 presents the calculated potential vibration levels at fixed distances within the screening distance contour for both passenger and freight trains without mitigation.

Table 10. Calculated Potential Ground-borne Vibration Levels at Fixed Distances

Distance (feet)	Calculated Vibration Velocity in VdB			
	Passenger Trains[1]		Freight Trains[2]	
	Vibration VdB	Impact Threshold Vdb[3]	Vibration VdB	Impact Threshold Vdb[3]
50	85	75	96	75/80
100	79	75	90	75/80
150	75	75	86	75/80
200	72	75	85	75/80
250	69	75	81	75/80
300	67	75	80	75/80

Source: FTA 2018

Notes:

[1] Calculated utilizing FRA Generalized Ground-Borne Vibration Curve for high-speed passenger trains. Base vibration adjusted to account for resilient fastener, efficient propagation of vibration, and an operating speed of 110 miles per hour utilizing the general vibration assessment approach.

[2] Calculated utilizing FTA Ground Surface Vibration Curve for freight trains. Base vibration adjusted to account for efficient propagation of vibration, and an operating speed of 60 miles per hour utilizing the general vibration assessment approach.

[3] On this project, passenger trains are considered occasional events, freight locomotives are considered occasional events, and freight railcars are considered frequent events.

FRA=Federal Railroad Administration; FTA=Federal Transit Administration; VdB=vibration decibels

Table 10 demonstrates that potential vibration levels during passenger train passbys could exceed 75 VdB within 200 feet of the at -grade tracks near the north portal. At distances greater than 200 feet, vibration levels could be perceptible but are estimated to be below the FRA vibration threshold of 75 VdB. Freight trains are predicted to produce higher vibration levels than passenger trains and could exceed FTA vibration thresholds at a maximum speed of 60 miles per hour. Ground-borne noise associated with rumbling could occur at locations where the FRA vibration levels are exceeded. A detailed vibration analysis should be developed during the final environmental review that incorporates terrain and soil characteristics and feasible mitigation options to reduce potential vibration impacts from passenger and freight trains and associated ground-borne noise impacts.

7.2.2 Potential Vibration Near Portals

As part of the north and south portal tunnel entrances, a cut-and-cover method would be used to dig a trench, build the tunnel, and return the surface to its original state. The south portal would exit the tunnel onto an aerial structure. According to the FRA guidance, the cut-and-cover and aerial structures provide a 3 dB and a 10 dB reduction in vibration levels, respectively. These reductions provide a lowered vibration level that would offset the shift in tracks near residential land uses.

7.2.3 Potential Vibration in the Tunnel

The Crest Canyon Higher Speed alternative would be operating trains underneath the ground surface, ranging in depth from approximately 30 feet at the tunnel entrance and exit to 320 feet at the maximum depth. The Camino Del Mar alternative tunnel depth ranges from 30 feet at the entrance and exit to 160 feet below the ground surface. The geological soil layer of both tunnel alignments includes a top layer of very old Paralic Deposits that consists of clay (mudstone) overlaying sandstone. The thickness of the mudstone unit ranges from approximately 4 feet to 20 feet. At depths of 100 feet below the ground surface, the soil type is Torrey Sandstone with a bottom layer of Del Mar Formation Soil (Leighton 2022). Geotechnical tests performed indicate that the mudstone is highly expansive, and the distance below the ground surface assists in reducing vibration impacts. The FRA guidance indicates that vibration levels decrease with distance. FRA cites from empirical data that for a high-speed train (steel wheel) operating at 150 miles per hour, vibration levels at a distance of 200 feet would generate vibration levels of 70 VdB. This empirical data indicates as speeds decrease, vibration levels decrease with distances as well. Therefore, it is anticipated that for high-speed trains operation 200 feet below the ground, surface vibration levels would be below 70 VdB. Based on the preliminary analysis, the soil type in the area is considered efficient in vibration propagation, which may increase potential vibration levels above FRA threshold levels at the entrance and exit of the tunnel for either conceptual alternative. Residential land uses near the alignments could experience new vibration levels that approach or exceed the FTA and FRA threshold level of 75 VdB without mitigation. Additional soil testing should be performed to assess vibration levels in the next phase. The detailed vibration assessment should also evaluate the potential for ground-borne noise impacts at vibration-sensitive land uses near the tunnel portals.

8 Summary

The Crest Canyon Higher Speed alternative and the Camino Del Mar alternative could have potential noise and vibration impacts on the surrounding land use at the portal locations; SANDAG will implement feasible noise and vibration mitigation measures to reduce long-term impacts to within acceptable limits.. Several mitigation features could be incorporated into the conceptual project alternatives to reduce potential noise and vibration levels. The at-grade sections at the entrance of each tunnel section would consist of U -structures and cut-and-cover treatments, reducing potential noise and vibration levels. Further, acoustic absorption under the trainsets, tangent track with high resilience fasteners, and a smooth track surface would reduce potential noise and vibration levels. At the tunnel portals, vents and a ventilation shaft could be a new source of noise without abatement features. Attenuators, fan enclosures, and other abatement features could be required at the proposed portal locations to lower the exiting exhaust to meet the City of Del Mar and City of San Diego daytime and nighttime exterior noise levels. Sensitive habitats could experience similar noise and vibration levels to existing conditions as these environments currently experience train passbys along at-grade tracks. The tunnel alignment could generate new sources of vibration noise; however, potential vibration levels and ground-borne noise would not be anticipated to exceed FTA and FRA threshold levels while trains are operating below the ground surface.

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Appendix G. Opinion of Probable Construction Costs for 10% Design

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Del Mar Tunnels Alternatives Analysis - Crest Canyon High Speed

CONSTRUCTION COST — **Design Level** 10%

Revised: October 2022 — **Estimated By** HDR

Item	QTY	Unit	Unit Price	Cost	Subtotal
CONSTRUCTION					
Site Civil					
Site Clearing	43	AC	$24,000	$ 1,032,000	
Unclassified Earthwork (Cut/Fill) 1	880,000	CY	$30	$ 26,400,000	
Slope Protection	17,226	CY	$200	$ 3,445,200	
Drainage 2	1	LS	$530,000	$ 530,000	
Traffic Control	1	LS	$10,800,000	$ 10,800,000	
Jimmy Durante Roadway	1	LS	$6,340,179	$ 6,340,179	
Miscellenaous Roadway Work 3	1	LS	$1,000,000	$ 1,000,000	
Subtotal					**$49,547,379**
Trackwork					
Track Removal	31,414	TF	$60	$ 1,884,840	
New Track - 136 RE CWR, Concrete Ties (Ballasted)	19,784	TF	$390	$ 7,715,760	
New Track - 136 RE CWR, Direct Fixation	30,629	TF	$600	$ 18,377,400	
Reprofile Track	800	TF	$60	$ 48,000	
Remove Existing No. 24 Turnout	2	EA	$60,000	$ 120,000	
Subballast	17000	CY	$120.00	$ 2,040,000	
Hi-Rail Access Crossing	2	EA	$140,000.00	$ 280,000	
Insulated Joints	7	PR	$15,000.00	$ 105,000	
Temporary Trackwork 4	1	LS	$6,331,000.00	$ 6,331,000	
Subtotal					**$36,902,000**
Signal					
CP Valley	1	LS	$56,930	$ 56,930	
242 Signals	1	LS	$56,930	$ 56,930	
CP Durante (Temp Phase)	1	LS	$1,707,900	$ 1,707,900	
243 Signals	1	LS	$284,650	$ 284,650	
CP Del Mar	1	LS	$113,860	$ 113,860	
Coast Blvd.	1	LS	$113,860	$ 113,860	
244 Signals	1	LS	$1,366,320	$ 1,366,320	
245 Signals (EB Only)	1	LS	$1,366,320	$ 1,366,320	
246 Signals (WB Only)	1	LS	$1,366,320	$ 1,366,320	
247 Signals	1	LS	$1,537,110	$ 1,537,110	
CP Torrey	1	LS	$113,860	$ 113,860	
CP Torrey (Temp Phase)	1	LS	$2,049,480	$ 2,049,480	
West Pedestrian Crossing	1	LS	$170,790	$ 170,790	
East Pedestrian Crossing	1	LS	$113,860	$ 113,860	
Sorrento Valley Boulevard	1	LS	$113,860	$ 113,860	
CP Sorrento	1	LS	$56,930	$ 56,930	
Fiber Duct Bank	25256	FT	$137	$ 3,460,072	
PTC/TMDS Support	1	LS	$569,300	$ 569,300	
Voice Radio	1	LS	$569,300	$ 569,300	
Subtotal					**$15,187,652**
Structures					
Remove Existing Bridge 246.1	284	TF	$1,139	$ 323,363	
Remove Existing Bridge 246.9	200	TF	$1,139	$ 227,720	
Remove Existing Bridge 247.1	88	TF	$1,139	$ 100,197	
Remove Existing Bridge 247.7	126	TF	$1,139	$ 143,464	
Los Penasquitos Lagoon Bridge #1	2,830	TF	$19,501	$ 55,187,830	
Los Penasquitos Lagoon Bridge #2 (247.7)	252	TF	$21,215	$ 5,346,180	
Retaining Walls	28,332	SF	$153	$ 4,322,682	
Floodwalls	2,000	LF	$8,600	$ 17,200,000	
Floodgates	2	EA	$1,307,000	$ 2,614,000	

Del Mar Tunnels Alternatives Analysis - Crest Canyon High Speed

CONSTRUCTION COST — **Design Level** 10%

Revised: October 2022 — **Estimated By** HDR

Item	QTY	Unit	Unit Price	Cost	Subtotal
Tunnel 5	1	LS	$477,184,527	$ 477,184,527	
North Portal 6	1	LS	$71,768,691	$ 71,768,691	
South Portal 7	1	LS	$60,294,335	$ 60,294,335	
Temporary Shoring	32,280	SF	$125	$ 4,042,941	
Subtotal					**$698,755,930**
Environmental 8					
SWPPP (Temp Erosion Control)	1	LS	$6,500,000	$ 6,500,000	
Onsite Wetlands	11.6	AC	$393,347	$ 4,562,825	
Onsite Uplands	4.6	AC	$82,457	$ 379,302	
OffSite Wetland Establishment - Berm Removal	14.2	AC	$774,005	$ 10,990,871	
Off Site Mitigation - Wetlands	27.2	AC	$774,005	$ 21,052,936	
Off Site Mitigation - Uplands	8.5	AC	$158,607	$ 1,348,160	
ESA Fencing	34,000	LF	$4	$ 136,000	
Monitors - Environmental/Biological	5,312	HR	$125	$ 664,000	
Monitors - Paleo/Archeology	6,383	HR	$125	$ 797,875	
Subtotal					**$46,431,969**
Utility Work					
Site Utilities and Relocations	1	LS	$20,000,000	$ 20,000,000	
Subtotal					**$20,000,000**
				BASE CONSTRUCTION ESTIMATE (BCE)	**$866,824,930**
CONSTRUCTION MOBIILIZATION AND CONTINGENCIES					
Construction Mobilization	11.0%	X	BCE	$ 95,350,742	
Bonds and Insurance	5.3%	X	BCE	$ 45,941,721	
Time Related Overhead (Per Caltrans Contract)	20.0%	X	BCE	$ 173,364,986	
Construction Contingency	35%	X	BCE	$ 303,388,726	
				CONSTRUCTION CONTRACT ESTIMATE (CCE)	**$1,484,871,105**

Footnotes:

[1] Includes cost for earthwork outside of tunnel limits and removal of existing track berm.

[2] Includes cost for drainage outside of tunnel limits only.

[3] Includes cost for roadway improvements such as grade crossing removals and roadway work due to damages during construction. Cost does not include proposed Jimmy Durante roadway improvements.

[4] Includes cost for temporary track work, turnouts, structures and embankment. See "Signal" for temporary signal costs.

[5] Includes cost for TBM, plant and equipment, TBM set up and disassemble, tunnel excavation and support, cross passage excavation and support, invert clean-up, cross passage final lining, guideway concrete in tunnel, ventilation and drainage, final clean-up, sump pump, muck handling and 20% tunnel contingency for the tunnel guideways

[6] Includes demolition and site work, guide wall and secant pile installation, interior excavation, cut-and-cover, u-structure, and headwall installation, ventilation and drainage, permanent facilities, site restoration and 20% tunnel contingency for the north portal

[7] Includes demolition and site work, guide wall and secant pile installation, interior excavation, cut-and-cover, u-structure, and headwall installation, ventilation and drainage, permanent facilities, site restoration and 20% tunnel contingency for the south portal

8 Costs are based on readily available information, including quantities provided by others, and therefore is subject to revision pending the completion of formal wetland delineation, selection of a build alternative alignment, and supporting engineering. Additionally, depending upon timimg, it is possible that a portion of the mitigation responsibility could be addressed through coordination and/or agreements with State Parks related to the Los Penasquitos Lagoon Enhancement Plan.

Assumptions:

1. Cost estimate is based on 2022 dollars, escalation to mid-year of construction to be done by others.

Del Mar Tunnels Alternatives Analysis - Camino Del Mar

CONSTRUCTION COST — **Design Level** 10%

Revised: October 2022 — **Estimated By** HDR

Item	QTY	Unit	Unit Price	Cost	Subtotal
CONSTRUCTION					
Site Civil					
Site Clearing	53	AC	$24,000	$ 1,272,000	
Unclassified Earthwork (Cut/Fill) 1	818,000	CY	$30.00	$ 24,540,000	
Slope Protection	25,291	CY	$200.00	$ 5,058,200	
Drainage 2	1	LS	$970,000.00	$ 970,000	
Traffic Control	1	LS	$18,500,000.00	$ 18,500,000	
Jimmy Durante Roadway	1	LS	$6,977,795.00	$ 6,977,795	
Miscellaneous Roadway Work 3	1	LS	$1,000,000.00	$ 1,000,000	
Subtotal					**$58,317,995**
Trackwork					
Track Removal	31,414	TF	$60	$ 1,884,840	
New Track - 136 RE CWR, Concrete Ties (Ballasted)	29,359	TF	$390	$ 11,450,010	
New Track - 136 RE CWR, Direct Fixation	22,359	TF	$600	$ 13,415,400	
Reprofile Track	800	TF	$60	$ 48,000	
Remove Existing No. 24 Turnout	2	EA	$60,000	$ 120,000	
Subballast	19,000	CY	$120.00	$ 2,280,000	
Hi-Rail Access Crossing	2	EA	$140,000.00	$ 280,000	
Insulated Joints	7	PR	$15,000.00	$ 105,000	
Temporary Trackwork 4	1	LS	$5,129,000.00	$ 5,129,000	
Subtotal					**$34,712,250**
Signal					
CP Valley	1	LS	$56,930	$ 56,930	
242 Signals	1	LS	$56,930	$ 56,930	
CP Durante (Temp Phase)	1	LS	$1,707,900	$ 1,707,900	
243 Signals	1	LS	$284,650	$ 284,650	
CP Del Mar	1	LS	$113,860	$ 113,860	
Coast Blvd.	1	LS	$113,860	$ 113,860	
244 Signals	1	LS	$1,537,110	$ 1,537,110	
246 Signals	1	LS	$1,707,900	$ 1,707,900	
247 Signals	1	LS	$1,537,110	$ 1,537,110	
CP Torrey	1	LS	$113,860	$ 113,860	
CP Torrey (Temp Phase)	1	LS	$2,049,480	$ 2,049,480	
West Pedestrian Crossing	1	LS	$170,790	$ 170,790	
East Pedestrian Crossing	1	LS	$113,860	$ 113,860	
Sorrento Valley Boulevard	1	LS	$113,860	$ 113,860	
CP Sorrento	1	LS	$56,930	$ 56,930	
Fiber Duct Bank	25911	FT	$137	$ 3,549,807	
PTC/TMDS Support	1	LS	$569,300	$ 569,300	
Voice Radio	1	LS	$569,300	$ 569,300	
Subtotal					**$14,423,437**
Structures					
Remove Existing Bridge 246.1	284	TF	$1,139	$ 323,363	
Remove Existing Bridge 246.9	200	TF	$1,139	$ 227,720	
Remove Existing Bridge 247.1	88	TF	$1,139	$ 100,197	
Remove Existing Bridge 247.7	126	TF	$1,139	$ 143,464	
Los Penasquitos Lagoon Bridge #1	9,570	TF	$17,013	$ 162,814,410	
Los Penasquitos Lagoon Bridge #2	2,400	TF	$18,198	$ 43,675,200	
Los Penasquitos Lagoon Bridge #3 (247.7)	252	TF	$21,215	$ 5,346,180	
Retaining Walls	31,932	SF	$153	$ 4,871,942	
Floodwalls	2,000	LF	$8,570	$ 17,140,000	
Floodgates	2	EA	$1,306,750	$ 2,613,500	

Del Mar Tunnels Alternatives Analysis - Camino Del Mar

CONSTRUCTION COST

Design Level **10%**

Revised: October 2022 **Estimated By** **HDR**

Item	QTY	Unit	Unit Price	Cost	Subtotal
Tunnel 5	1	LS	$356,848,626	$ 356,848,626	
North Portal 6	1	LS	$70,100,414	$ 70,100,414	
South Portal 7	1	LS	$42,273,941	$ 42,273,941	
Temporary Shoring	172,766	SF	$125	$ 21,638,251	
Subtotal					**$728,117,208**
Environmental 8					
SWPPP (Temp Erosion Control)	1	LS	$11,100,000	$ 11,100,000	
Onsite Wetlands - Restoration of Temporary Impacts	19.4	AC	$393,347	$ 7,630,932	
Onsite Uplands - Restoration of Temporary Impacts	6.5	AC	$82,457	$ 535,971	
Onsite Wetland Establishment - Berm Removal	8.9	AC	$393,347	$ 3,500,788	
Off Site Mitigation - for Temporal Loss of wetland	26.3	AC	$774,005	$ 20,356,332	
Off Site Mitigation - Uplands - for Permanent Impacts	4	AC	$158,607	$ 634,428	
ESA Fencing	21,000	LF	$4	$ 84,000	
Monitors - Environmental/Biological	5,507	HR	$125	$ 688,375	
Monitors - Paleo/Archeology	7,509	HR	$125	$ 938,625	
Subtotal					**$45,469,451**
Utility Work					
Site Utilities and Relocations	1	LS	$29,000,000	$ 29,000,000	
Subtotal					**$29,000,000**
				BASE CONSTRUCTION ESTIMATE (BCE)	**$910,040,341**
CONSTRUCTION MOBILIZATION AND CONTINGENCIES					
Construction Mobilization	11.0%	X	BCE	$ 100,104,438	
Bonds and Insurance	5.3%	X	BCE	$ 48,232,138	
Time Related Overhead (Per Caltrans Contract)	20.0%	X	BCE	$ 182,008,068	
Construction Contingency	35%	X	BCE	$ 318,514,119	
				CONSTRUCTION CONTRACT ESTIMATE (CCE)	**$1,558,899,104**

Footnotes:

[1] Includes cost for earthwork outside of tunnel limits and removal of existing track berm.

[2] Includes cost for drainage outside of tunnel limits only.

[3] Includes cost for roadway improvements such as grade crossing removals and roadway work due to damages during construction. Cost does not include proposed Jimmy Durante roadway improvements.

[4] Includes cost for temporary track work, turnouts, structures and embankment. See "Signal" for temporary signal costs.

[5] Includes cost for TBM, plant and equipment, TBM set up and disassemble, tunnel excavation and support, cross passage excavation and support, invert clean-up, cross passage final lining, guideway concrete in tunnel, ventilation and drainage, final clean-up, sump pump, muck handling and 20% tunnel contingency for the tunnel guideways

[6] Includes demolition and site work, guide wall and secant pile installation, interior excavation, cut-and-cover, u-structure, and headwall installation, ventilation and drainage, permanent facilities, site restoration and 20% tunnel contingency for the north portal

[7] Includes demolition and site work, guide wall and secant pile installation, interior excavation, cut-and-cover, u-structure, and headwall installation, ventilation and drainage, permanent facilities, site restoration and 20% tunnel contingency for the south portal

8 Costs are based on readily available information, including quantities provided by others, and therefore is subject to revision pending the completion of formal wetland delineation, selection of a build alternative alignment, and supporting engineering. Additionally, depending upon timing, it is possible that a portion of the mitigation responsibility could be addressed through coordination and/or agreements with State Parks related to the Los Penasquitos Lagoon Enhancement Plan.

Assumptions:

1. Cost estimate is based on 2022 dollars, escalation to mid-year of construction to done by others.

Appendix H. Estimated Operations and Maintenance Cost

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SUMMARY COSTS FOR SD-LOSSAN
Operations and Maintenance

SANDAG LOSSAN Corridor - Annual Operations and Maintenance - Del Mar Tunnels

PROJECT COST ESTIMATE | **Design Level** Conceptual

Revised: 5/28/21 | **Estimated By** JFW

SCC	Item	QTY	Unit	Unit Construction Cost	Construction Cost	Unit Cost (w/ 20% Tunnel Contingency)	Total (w/ 20% Tunnel Contingency)
	CONSTRUCTION						
	Tunnel						
	Annual O&M	1	YR	$1,675,000.00	$ 1,675,000	$2,010,000	$2,010,000
	Subtotal - Tunnel				**$1,675,000**		**$2,010,000**

Total (Pre-Contingency)	**$1,675,000**
Total (w/ 20% Tunnel Contingency)	**$2,010,000**

SAN DIEGO ASSOCIATION OF GOVERNMENTS

SD-LOSSAN - ANNUAL O&M FOR TUNNELS

SAN DIEGO CO., CA

Annual Operations and Maintenance for Tunnels

HOURS PER MAINTENANCE SHIFT	**8**	**HR**
SHIFTS PER DAY	**1**	**SHIFTS**
WORKINGS DAYS PER WEEK	**5**	**DAYS**
WORKING DAYS PER YEAR	**260**	**DAYS**
OPERATION DAYS PER YEAR	**365**	**DAYS**

OPERATIONS AND MAINTENANCE

ANNUAL O&M

LABOR				
Shifts per Year		260	SHIFTS	
Number of Shifts		**260**	**SHIFTS**	
Inspection/Maintenance Crew		$ 1,723.60		
		$ 1,723.60	PER SHIFT	
LABOR FOR ANNUAL O&M				**$ 448,137**
EQUIPMENT				
Equipment - For Inspection/Maintenance		$ 1,125.16	PER SHIFT	
		$ 1,125.16	PER SHIFT	$ 292,542
Equipment - For Daily Operation		$ 748.15	PER 8-HR PERIOD	
		$ 748.15	PER 8-HR PERIOD	$ 624,704
EQUIPMENT FOR ANNUAL O&M				**$ 917,246**
MATERIALS (Sales Tax - 7.25%)				
Allow				**$ 45,000**
SUPPLIES	10%	of Labor		**$ 44,814**
	TOTAL			**$ 1,456,000**
		Markup	15%	
Annual O&M				$ 1,675,000
TOTAL DIRECT COST				**$ 1,675,000**

Labor

Assume the Contractor will work 1- 8 hr shift per day.

O&M Crew

	Number	Hourly Rate	Hours per Shift	Rate/ 8 Hr Shift
Inspector/Maintenance	3	$ 71.82	8	$1,723.60
Total (per shift)	**3**			**$1,724**

Labor Categories Rates from General Decision Number CA20210001 Mod 4 Dated 04/09/2021

Trade	Base Hourly Rate ($/Hr.)	Fringes ($/Hr.)	Total PWR ($/Hr.)	WC (Use 12.0%) ($/Hr.)	Fixed Overhead (Use 15.45%) ($/Hr.)	Total ($/Hr.)
Inspector/Maintenance	$ 40.28	$ 20.48	$ 60.76	$ 4.83	$ 6.22	$ 71.82

Equipment

O&M Equipment (Maintenance Hours)

	Number of Equip.	Rental	Oper. Cost	Cost/8H Shift	Purchase
Pickup Truck	3	$83.97	$23.73	$323.10	
Vent Fans (Large)	8	$0.00	$86.64	$693.15	yes
UTVs	3	$10.34	$7.63	$53.92	
Misc. Power	1	$0.00	$55.00	$55.00	
Total Cost Per Shift in 2021$*				**$1,125.16**	

O&M Equipment (Non-Maintenance Hours)

	Number of Equip.	Rental	Oper. Cost	Cost/8H Shift	Purchase
Vent Fans (Large)	8	$0.00	$86.64	$693.15	yes
Power	1	$0.00	$55.00	$55.00	
Total Cost Per Shift in 2021$*				**$748.15**	

Appendix I. Other Potential Portal Locations Exhibits

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NOTES:

1. WORK AREAS AND PORTAL FACILITIES ARE NOT SHOWN

2. EXTENTS OF FLOOD WALLS ARE SHOWN TO INDICATE APPROXIMATE REACHES. HEIGHT OF FLOOD WALLS TO BE DETERMINED WITH FURTHER ENGINEERING STUDIES.

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

HORIZ 200 0 200 400

VERT 5 0 5 10

SCALE IN FEET

HDR | MOTT MACDONALD | SANDAG San Diego's Regional Planning Agency

APPROVED: DATE:

Exhibit

NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

NORTH PORTAL
FAIRGROUNDS

CONTRACT NO.	
DRAWING NO. EXHIBIT 1	
REVISION	SHEET NO. OPTION 1
SCALE AS NOTED	

NOTES:

1. WORK AREAS AND PORTAL FACILITIES ARE NOT SHOWN

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

HORIZ 200 0 200 400
VERT 5 0 5 10
SCALE IN FEET

HDR | MOTT MACDONALD | SANDAG San Diego's Regional Planning Agency

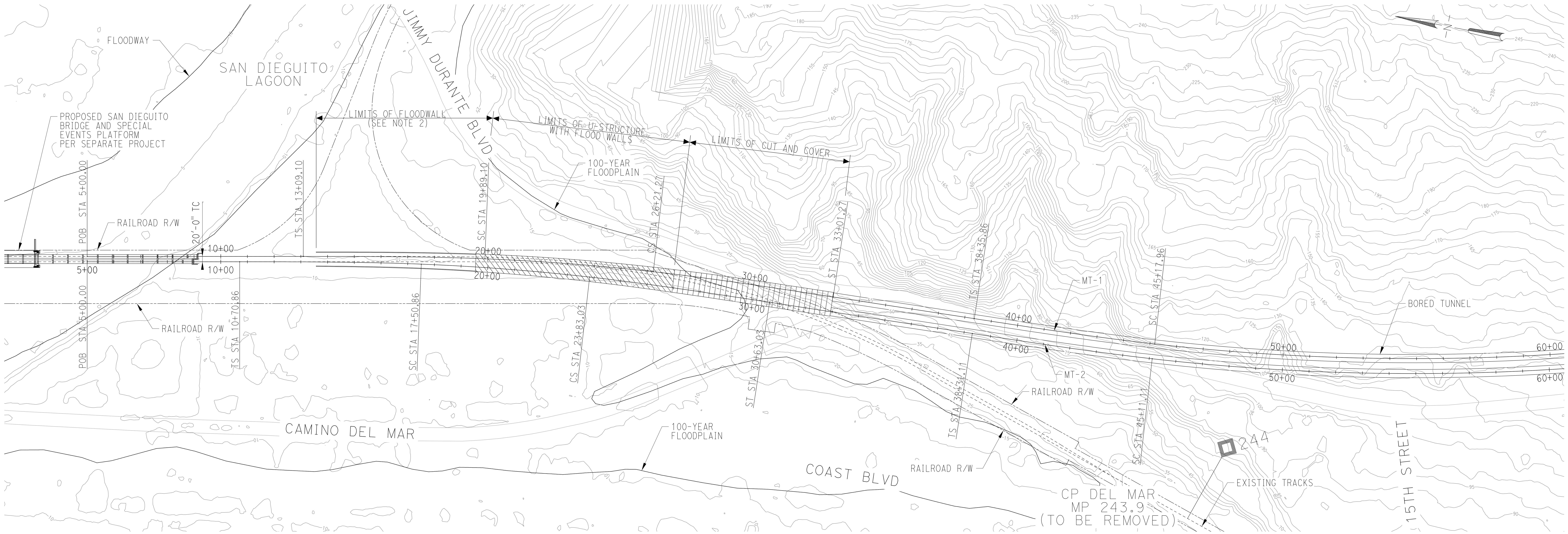
APPROVED: ____________ DATE: ______

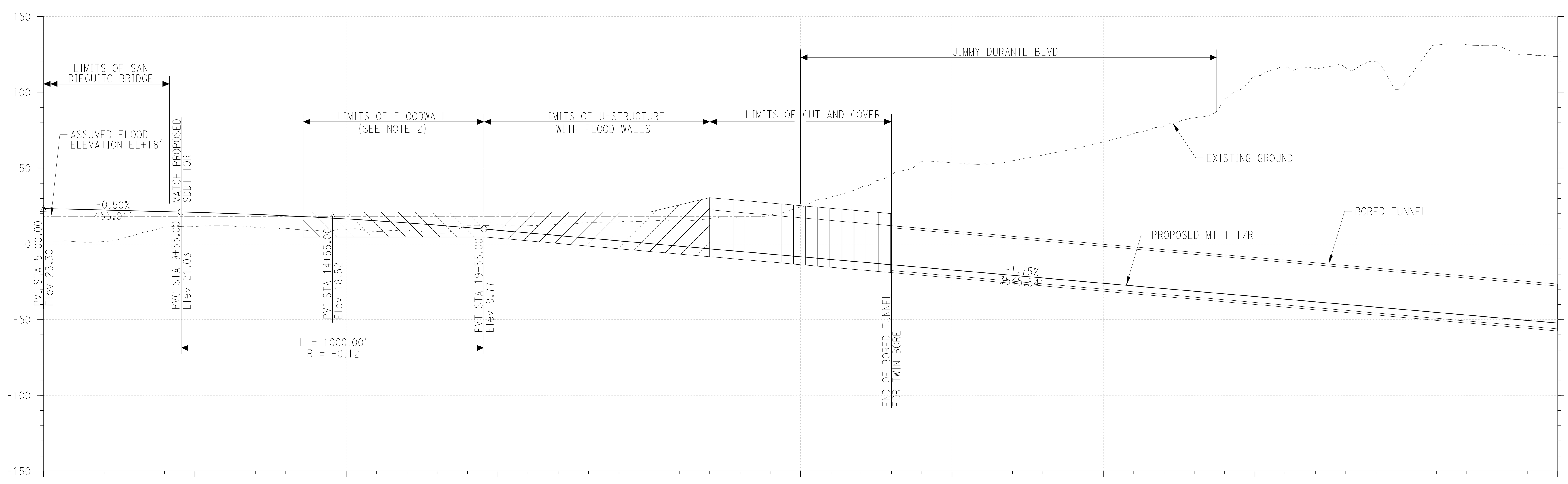
Exhibit
NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

NORTH PORTAL
OVER JIMMY DURANTE

CONTRACT NO.	
DRAWING NO.	EXHIBIT 2
REVISION	SHEET NO. OPTION 2
SCALE	AS NOTED





NOTES:

1. WORK AREAS AND PORTAL FACILITIES ARE NOT SHOWN
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HORIZ 200 0 200 400
VERT 5 0 5 10
SCALE IN FEET

HDR | MOTT MACDONALD | SANDAG San Diego's Regional Planning Agency

APPROVED: ________ DATE: ________

Exhibit

NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

NORTH PORTAL WITHIN CAMINO DEL MAR

CONTRACT NO.
DRAWING NO. EXHIBIT 3
REVISION | SHEET NO. OPTION 4
SCALE AS NOTED

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

HORIZ	200	0	200	400
VERT	5	0	5	10

SCALE IN FEET

HDR | MOTT MACDONALD | SANDAG San Diego's Regional Planning Agency

APPROVED: ______ DATE: ______

Exhibit

NOT FOR CONSTRUCTION

SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
SOUTH PORTAL
KNOLL NEAR I-5

CONTRACT NO.

DRAWING NO. EXHIBIT 4

REVISION | SHEET NO. OPTION 3

SCALE AS NOTED

Appendix J. Basis of Design Report for Track, Grade Crossings, and Signals

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SANDAG

Basis of Design Criteria

For Track, Grade Crossings and Signals in FRA Class 6

San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study (SD-LOSSAN)

December 2021

Prepared by:

HDR

Pacific Railway Enterprises, Inc.

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619) 699-1900
www.sandag.org

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ABSTRACT

TITLE:	Basis of Design Criteria for Track, Grade Crossings and Signals for FRA Class 6 Track
AUTHOR:	San Diego Association of Governments
DATE:	December 10, 2021
SOURCE OF COPIES:	San Diego Association of Governments 401 B Street, Suite 800 San Diego, CA 92101 (619) 699-1900
NUMBER OF PAGES:	18
ABSTRACT:	This report presents the technical criteria for track grade crossings, and signals for FRA Class 6 track to be followed for conceptual level engineering for the San Diego Regional Rail Corridor Alternative Alignment & Improvements Conceptual Engineering Study.

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Contents

Tables

Figures

Acronyms

ADAAG	American Disability Act Accessibility Guidelines
AREMA	American Railway Engineering and Maintenance-of-Way Association
BNSF	Burlington Northern and Santa Fe Railway
Cal/OSHA	California Department of Industrial Relations, Division of Occupational Safety and Health
CFR	Code of Federal Regulations
CPUC	California Public Utilities Commission
CTC	Centralized traffic control
CWR	Continuously Welded Rail
DIB	Design Information Bulletin
ESD	Engineering Design Standard Drawings
FRA	Federal Railroad Administration
GO	General Order
HZ	Hertz
I-ETMS	Interoperable Electronic Train Management System
KHZ	Kilohertz
LED	Light-emitting diode
LOSSAN	Los Angeles-San Diego-San Luis Obispo
MPH	Miles per hour
OCC	Operations Control Center
OTM	Other track material
NCTD	North Country Transit District
PS	Point of switch
PTC	Positive Train Control
RSI	Random Signature Island
SANDAG	San Diego Association of Governments
SCRRA	Southern California Regional Rail Authority
TO	Turnout
WCAS	Wireless Crossing Activation System
WCNSS	Wireless Crossing Nearside Stations Stop

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Executive Overview

This Basis of Design presents the technical criteria to be followed for subsequent conceptual level engineering as part of this study, the San Diego Regional Rail Corridor Alternative Alignment & Improvements Conceptual Engineering Study.

This study will assess the current corridor conditions and develop a program of improvements in order to reduce travel time for commuter and intercity passenger rail service to be competitive with the automobile travel time. This program of improvements will increase track capacity, improve resiliency, enhance safety, and support increased passenger and freight frequencies along the 60.1-mile San Diego Subdivision of the LOSSAN corridor from the Orange County/San Diego County line to downtown San Diego. The improvements to be evaluated include corridor-wide track and signal upgrades to achieve higher operating speeds (up to 110 miles per hour), including curve realignments, high-speed interlockings, grade crossing improvements, and grade separations.

The basic requirement for the railroad geometric design will be to provide safe, economical, and efficient rail passenger transportation on a shared-use corridor between NCTD, Amtrak, and BNSF. The Basis of Design for this study will build on the already established LOSSAN Design Criteria developed by SANDAG for FRA Class 5 track, with noted changes required to accommodate FRA Class 6 track. Additionally, this basis of design will follow accepted engineering practices used by AREMA and Amtrak, as well as FRA Track Safety Standards. FRA Class 6 was determined as the most viable as a result of the track speed analysis performed as part of the Operational Feasibility Analysis that was completed as part of this study. The goal of that analysis was to determine the feasibility of obtaining higher speeds throughout the corridor. Given the fleet, equipment and limitations on the corridor outside of SANDAG's purview, FRA Class 6 was determined to be the most feasible and effective alternative. FRA Class 7 was analyzed and found to offer a marginal improvement, at best, in travel time and removed from consideration.

This Basis of Design will include criteria and/or guidelines for FRA Class 6 (110 mph) track for the following disciplines:

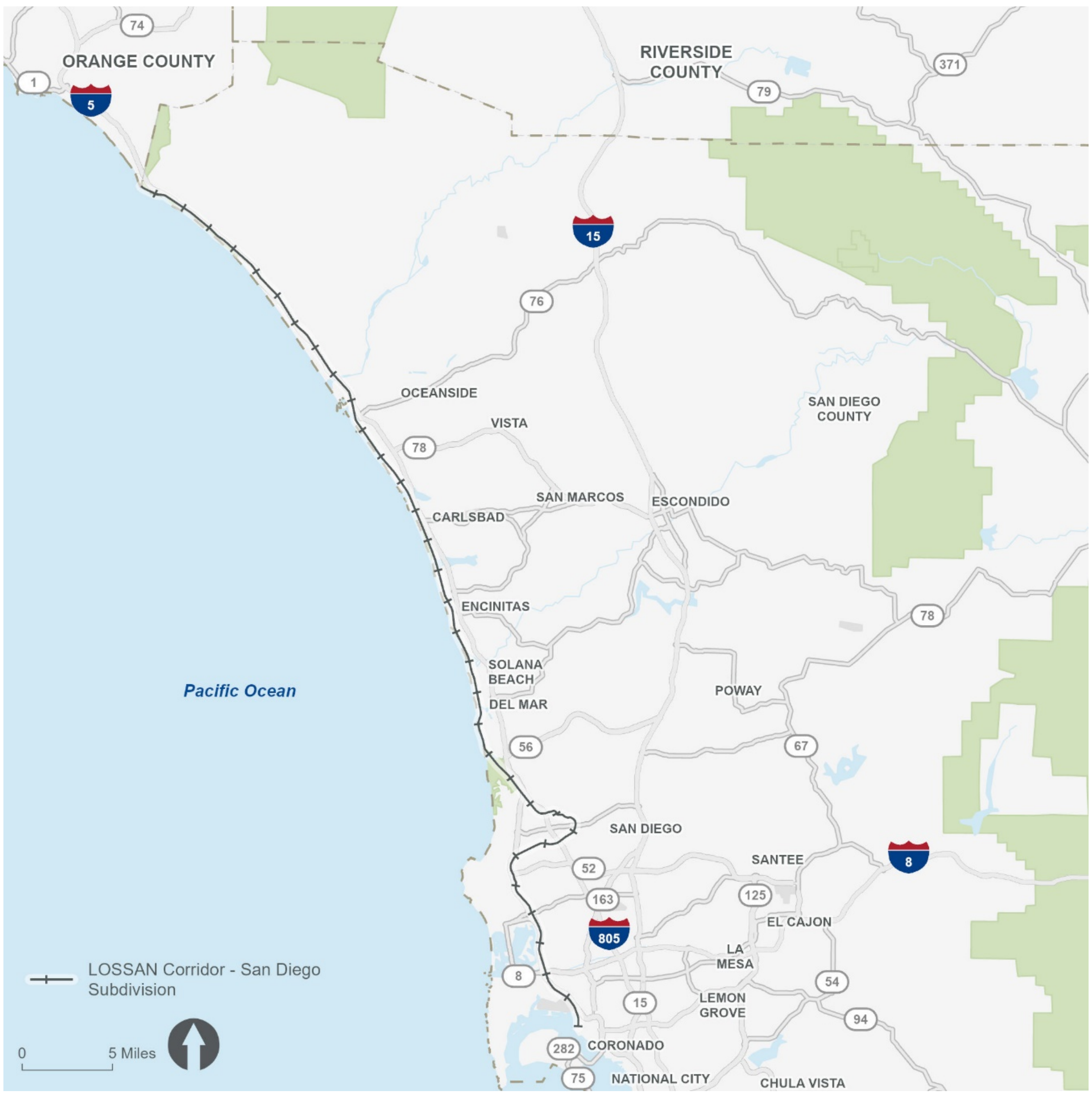
- Mainline Track
- Grade Crossings and Sealed Corridors
- Railway Signaling and Communications Train Control Signals

1 Project Overview

1.1 Project Description

The Los Angeles-San Diego-San Luis Obispo (LOSSAN) rail corridor stretches 351 miles through six southern California counties and is the nation's second busiest passenger rail corridor according to SANDAG. Nearly eight million passengers use the corridor's intercity and commuter rail services annually. As shown in Figure 1-1, the San Diego Subdivision is the southernmost 60.1 miles of the corridor, from the Orange County/San Diego County line to downtown San Diego. More than 50 trains use this segment of the corridor daily including Amtrak Pacific Surfliner intercity, Southern California Regional Rail Authority (SCRRA) Metrolink and North County Transit District (NCTD) COASTER commuter, and BNSF Railway Company (BNSF) freight trains. Corridor planning documents by the California Department of Transportation (Caltrans), the LOSSAN Rail Corridor Agency, SANDAG, and others include the goal of doubling the amount of passenger rail service along the San Diego Subdivision by 2035.

Figure 1-1. Project Location



This study will assess the current corridor conditions along the San Diego Subdivision and develop a program of improvements to reduce travel time for commuter and intercity passenger rail service and become competitive with automobile travel. This program of improvements will increase track capacity, improve resiliency, enhance safety, and support increased passenger and freight frequencies. These improvements may include curve realignments along key segments of the corridor, high-speed interlockings, grade crossing improvements, grade separations, and other enhancements. SANDAG has previously established the demand and market requirements for passenger rail service in the LOSSAN Corridor. This study addresses both current and future passenger rail service demand by increasing the corridor's competitiveness with driving the congested parallel Interstate-5 corridor.

1.2 System Design Criteria/Design Standards

The existing tracks within the project limits are currently operated on by the NCTD commuter rail service (COASTER), Amtrak intercity rail service (Pacific Surfliner), Metrolink commuter rail service, and BNSF freight trains. The basis of design for the project, as presented herein, generally follows the LOSSAN Design Criteria Manual, Volume III, Draft 4, augmented for FRA Class 6 with a compilation of other railroad system design criteria, standards, or guidelines including but not limited to the following:

- FRA Track Safety Standards, primarily Title 49 Code of Federal Regulations (CFR) Parts 213 Subpart G, 214, 234, and 236.
- CPUC General Orders (GOs)
- 2019 American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering
- Amtrak Engineering Track Design Specification – Spec No. 63 (Rev. 2020)

1.3 Technology Assumptions

Proposed higher speed services along the corridor will utilize the Siemens SC-44 Charger diesel-electric locomotives, and non-tilting passenger coaches. The limiting constraint is existing COASTER passenger coaches which can only go up to 90 mph. The assumption is future coaches will be procured that can accommodate operational speeds up to 110 mph. Refer to the Operational Feasibility Study developed as part of this project for more details regarding technology assumptions.

2 Track

2.1 Track General

2.1.1 Application of Criteria

The purpose of this section is to establish uniform and minimum standards for conceptual engineering railroad track design for the San Diego Regional Rail Corridor Alternative Alignment & Improvements Conceptual Engineering Study. This document is based on industry standards, regulatory requirements and recommended practices for commuter and Class I railroads. Conformance to the design criteria is required to achieve uniformity in the preparation of this study and the development of future projects.

2.1.2 Design Guidelines, Codes, Manuals, Standards & Specifications

The railroad design shall meet all applicable State of California laws, California Public Utilities Commission (CPUC) requirements, FRA safety requirements, and the specific project requirements. Where any conflict in criteria exists, the stricter criteria shall govern.

Unless specifically noted otherwise in these criteria, the latest edition of the standard, code, or guideline that is applicable at the time the design is initiated shall be used. If a new edition or amendment to a code, manual, regulation or standard specification is issued before the design is completed, the design may be revised or modified to conform to the new requirement(s) to the extent approved or required by the agency enforcing the standard, code, or guideline changes.

The design criteria assembled in this manual are based on industry standards, governmental regulations, local practices, and railroad guidelines and standards. The most recent editions of the following publication are documents that were used:

- LOSSAN Design Criteria Manual, Volume III, Draft 4
- LOSSAN Engineering Standards Drawings (ESD), April 2020
- FRA Track Safety Standards, primarily 49 CFR 213, Subpart G
- CPUC General Orders
- AREMA Manual for Railway Engineering, 2019 Edition
- Amtrak Engineering Track Design Specification – Spec No. 63 (Rev. 2020)

2.2 Track Geometry

2.2.1 Scope

The track design goals for this study are to enhance safety, minimize travel times, and maximize passenger comfort. An additional goal is to also minimize long-term maintenance costs based on accepted railroad industry engineering practice and the experience of operating both passenger and freight trains within a shared corridor.

2.2.2 Design Speed

Design speeds shall be increased to the extent practicable that is consistent with the corridor service objectives. Where feasible, track geometry for mainline tracks will be designed to meet up to FRA Class 6 track (110 mph) for passenger and up to FRA Class 4 (60 mph) for freight. Sections of the corridor will be evaluated in a subsequent feasibility analysis to determine where along the corridor speed may be increased to 110 mph. Potential constraints that may limit design to speed to below FRA Class 6 track may include civil constraints such as curves, right-of-way, or in locations where there is no operational advantage.

2.2.3 Gage

All new track shall be designed to the standard gage of 4 feet 8-1/2 inches, where the gage is measured between the heads of rails at 5/8" below top of rail.

2.3 Horizontal Alignment

2.3.1 Components

The horizontal track alignment is defined as a series of tangents, spirals, and circular curves. All circular curves shall be connected to tangents by transition spirals.

Connections between existing and proposed alignments shall include adequate spirals and tangent lengths to the extent practical. Where specified minimum values are not achieved, proposed deviations shall be reviewed and approved by NCTD and SANDAG as part of the design submittal-review process.

2.3.2 Curves

Horizontal curves shall be defined using the chord definition. The desired minimum length of horizontal curves shall be 100 feet or 3 times the maximum operating speed (V_{max}), whichever is greater.

Concentric Circular Curves

Where the track center spacing of the incoming and outgoing tangent of a curve are the same, circular curves shall be designed with a constant distance between the track centerlines so that the curves are concentric.

Compound Curves

Compound circular curves may be used if necessary but shall be avoided if possible. The use of such curves shall require NCTD and SANDAG approval. Should compound curves be required, transition spirals between such curves meeting the requirements of Section 2.3.3 will be used and sized to maintain the entrance superelevation transition rate. Compound curves shall also be designed to have a consistent underbalance throughout the entire length of the curve. See superelevation section below for description of underbalance.

Superelevation

Superelevation is the difference in elevation between the top of the outer (high) rail and the top of the inner (low) rail within a curve or spiral. It is provided to overcome or partially overcome the effects of curvature and speed on passenger comfort, and carbody and wheel forces.

Maximum superelevation in curves shall not exceed five inches within the limits of the curve on track where both freight and passenger trains operate. The design superelevation in curves must consider that freight and passenger trains operate at different speeds. Less superelevation should be considered on curves where freight trains also operate, and more superelevation is needed for passenger trains operating at higher speeds. A proper balance between freight and passenger service is required in order to decrease vertical forces on the low rail, potential geometry defects, and subsequent required maintenance. Curves with only passenger trains operating may have up to six inches of superelevation with the approval of NCTD and SANDAG, with the assumption that the locomotives and passenger coaches are qualified to operate on curves with up to 6 inches of superelevation and operating speeds are not expected to widely vary.

Underbalance (Eu)

Superelevation underbalance (E_u) is the difference between the equilibrium superelevation and the actual superelevation placed in the curve.

Based on the maximum operating speed and the superelevation within the limits of a curve, resulting underbalance for passenger should not exceed five inches. This maximum underbalance is predicated on acceptance of vehicle performance testing as noted in 49 CFR Part 213 Subpart G. Allowed underbalance for passenger trains of up to six inches may be considered if operating equipment is qualified, proper track conditions are verified, and FRA waivers are obtained. Operating underbalance greater than what is allowed under current operation will require coordination with the FRA and demonstrated vehicle performance testing as required by the FRA.

Resulting underbalance for freight trains should remain at two inches, as currently specified in LOSSAN Engineering Standard Drawings ESD-2003 and ESD-2004.

2.3.3 Spirals

Clothoid spirals shall be provided between all tangents and simple curves and between curve segments within a compound curve. The superelevation runoff shall be of a uniform rate, extending over the full length of the spiral.

General guidance for determining the minimum spiral length for higher speed passenger operations follow current AREMA guidelines and should be the greater length resulting from the two equations below.

Equation 1: $L_s = 82.7*E_a$

Equation 2: $L_s = 1.22\ E_u * V_{max}$

When considering superelevation runoff rate within the proposed spiral, the minimum spiral length shall be based on AREMA Chapter 17, Section 3.5.7.8, Equation 1, which indicates that for speeds up the 110 mph, at most a 3/8-inch runoff in 31 feet shall be provided. Note that the minimum spiral length resulting from this equation is greater than that specified in the current LOSSAN Engineering Standard Drawings ESD-2003 and ESD-2004, intended for FRA Class 5 track. This is primarily due to the anticipated increase in maximum authorized speed and minimizing impacts to passenger comfort at these higher speeds.

When considering vehicle dynamics through a spiral and curve, the minimum spiral length as noted in AREMA Chapter 17, Section 3.5.7.9 is based on maximum allowable change in lateral carbody acceleration (or jerk rate). Along existing corridors where physical constraints exist and where achieving minimum spiral lengths may become cost prohibitive, the maximum allowable change in carbody acceleration may be increased to 0.04g/sec per Equation 2.

Although AREMA Section 3.5.7.9 recommends carbody acceleration of 0.03g/sec, defined by the equation $L_s = 1.63\ E_u * V_{max}$, the resultant spiral length results in unnecessarily long spiral lengths within the constraints of the corridor. It will be excluded from developing spiral lengths for the project.

At no time shall the minimum spiral length be less than 40 feet. Spirals shall be rounded up to the nearest 10 feet except in the case of concentric curves. Where track centers of concentric horizontal curves are the same as the incoming and outgoing tangents, the spirals of the outside curve shall be determined such that the spiral offsets are equal to the inside curve.

2.3.4 Tangents

Tangent lengths between curves shall three times the maximum operating speed (mph).

2.4 Vertical Alignment

2.4.1 General

The profile grade represents the elevation of the top of the low rail of a track alignment.

2.4.2 Grades

Requirements for the maximum gradient, length of constant grade between vertical curves, and compensated grades shall conform to Section 6.4.2, Grades, of the LOSSAN Design Criteria Manual, Volume III, Draft 4. Although the current ruling grade, the maximum gradient over which a tonnage train can be hauled with on locomotive, along this corridor is 2.2%, grades in excess of 2% are not desirable. If local conditions warrant grades in excess of 2%, approval shall be obtained by the SANDAG Director of Rail and submitted through NCTD to BNSF for approval.

No grades are allowed within station platform limits. For platform locations located within tunnels, the minimum grade in tunnels shall take precedence.

Grades in Tunnels

Low points and very flat grades shall be avoided where possible in tunnels due to drainage issues. Should a low point be required within the tunnel, proper mitigation measures for drainage shall be required. Tunnel profiles shall be designed with a minimum grade of 0.5%. The maximum compensated grade in tunnels shall be 2%. A compensated grade is a grade that has been reduced along a curve to offset the additional resistance due to the curve. Curves shall be compensated at 0.04% per degree of curvature.

2.4.3 Vertical Curves

Vertical curve design standards for FRA Class 6 track shall conform to Section 6.4.3, Vertical Curves, of the LOSSAN Design Criteria Manual, Volume III, Draft 4; AREMA Chapter 5, Section 3.6; and AREMA Chapter 17, Section 3.5.8.

Length of tangents connecting vertical curves shall be 3 times the maximum operating speed, V_{max}.

Undulating profiles consisting of several short vertical curves and tangents shall be avoided.

2.5 Combined Horizontal and Vertical Curves

Horizontal and vertical curves may overlap. It is not desirable to place a vertical curve within any part of a spiral; however, overlaps may be used if this consideration makes the design more costly, increases the height of fill, retaining walls, or aerial structures, or causes other criteria to fall below the recommended value.

2.6 Turnouts

2.6.1 Scope

The scope of this section is to provide criteria for turnouts located within FRA Class 6 track where there is a high-speed diverging move from one mainline track to another main track. Turnouts shall conform to AREMA recommended practices and as indicated herein.

2.6.2 Speed

High-speed turnouts for this basis of design will be classified as those turnouts that can accommodate diverging speeds of 60 mph and higher. Therefore, turnout sizes to be considered shall not be less than the No. 24 turnout.

The maximum speeds through diverging moves on turnouts shall be as follows, unless otherwise directed by NCTD and SANDAG:

Table 2-1. Maximum Diverging Speed by Turnout

Frog No.	Maximum Passenger Diverging Speed (mph)	Maximum Freight Diverging Speed (mph)
No. 24	60	50
No. 32.75	80	60

2.6.3 Frogs

Movable point frogs allow for a train's wheels to traverse the frog on a continuous running surface, eliminating the drop into the flangeway at the crossing point. Use of movable point frogs are strongly encouraged for turnouts greater than No. 20 above 90 mph. Movable point frogs are required for No. 32.75 turnouts.

2.6.4 Location

Turnouts and crossovers shall be located to allow suitable placement of switch machines or switch stands and associated CPUC walkways and with consideration of the placement and visibility of control signals. Turnouts shall be located based on Table 2-1 below:

Table 2-2. Turnout (TO) Location

Turnout Location	Minimum Tangent Length
Between Point of Switches (PS) of turnouts of opposite hand	150'
Between PS of turnouts of the same hand	Greater of 150' or 3 times maximum operating speed (mph) though diverging side of turnout
Between PS and spiral/curve through diverging side of turnout	Greater of 100' or 3 times maximum operating speed (mph)
Between last long tie of TO and spiral/curve on straight side of turnout	100'
Between PS and vertical curve	10', 25' is preferred
Between PS and Grade Crossing	100' (see Section 5.2.1)
Between PS and Station Platform	100'

Turnout Location	Minimum Tangent Length
Between PS and Bridge	100’

Crossovers shall be located:

- On tracks with 15-foot minimum spacing for straight moves of 110 mph or less
- On parallel tracks only
- Without curves between opposing frogs
- On horizontal and vertical tangents

2.7 Track Construction Types

Ballasted track with continuously welded rail (CWR) shall be used for track construction.

Other types of track construction such as direct-fixation may be considered for tunnels and bridges with approval from SANDAG and NCTD. Based on tunnel type and geotechnical material, considerations for vibration may be required. Refer to Section 4.2 of the Draft Basis of Design Criteria for Noise and Vibration for more information on vibration in tunnels.

2.8 Track Materials

2.8.1 Rail

Main track rail shall be new 136 RE carbon steel rail, meeting current AREMA “Specifications for Steel Rail.”

Premium head-hardened rail shall be used for curves 1º00’ or greater and for turnouts.

2.8.2 Ties

Ties for new main track construction for FRA Class 6 track shall be concrete ties with elastic fasteners spaced at 24 inches, center to center. Refer to LOSSAN Engineering Standard Drawing ESD-2402.

Additionally, No. 32.75 turnouts shall also be constructed with concrete ties. Timber ties shall be used at grade crossings, temporary shoofly track, and for turnouts sized No. 24 and smaller. Refer to LOSSAN Engineering Standard Drawings Section ESD-2900 for additional details for special trackwork tie requirements. Standard timber ties shall be 9-feet long and 7-inches high by 9-inches wide hardwood-treated main track grade 5 and spaced 19.5 inches, center to center. Timber ties at grade crossings shall be 10-feet long and 7-inches high by 9-inches wide hardwood-treated main track grade 5 and spaced 19.5 inches, center to center.

Transition ties shall be used where track modulus changes abruptly from concrete to timber. Refer to LOSSAN Engineering Standard Drawing ESD-2351-03 for details. At timber elastic fastener turnouts, no transition ties are required. Concrete ties may abut the limits of the timber tie turnout.

2.8.3 Other Track Materials

Other track material (OTM) shall conform to current LOSSAN and AREMA standards and specifications. On main track, elastic fasteners shall be used. Cut spikes shall not be permitted on FRA Class 6 track.

2.8.4 Ballast

Ballast shall conform to Section 6.7.4, Grades, of the LOSSAN Design Criteria Manual, Volume III, Draft 4.

2.8.5 Subballast

Subballast shall conform to Section 6.7.5, Grades, of the LOSSAN Design Criteria Manual, Volume III, Draft 4.

3 Clearances

3.1 Horizontal Clearances

Per the Section 4.2.2, NCTD/BNSF Shared Use Agreement, of the LOSSAN Design Criteria Manual, Volume III, Draft 4, the minimum horizontal clearance from centerline of track is 12'. Horizontal clearance may be reduced to 10' with prior approval from SANDAG, NCTD, and BNSF. The horizontal clearance at platforms shall be 5 feet 5 inches from centerline. Refer to LOSSAN Engineering Standard Drawings ESD-2101 for details.

3.2 Vertical Clearance

Minimum vertical clearance from top of rail to closest overhead structure within the train envelope is 26 feet, as per the BNSF and NCTD joint-use agreement. Vertical clearances between 24 and 26 feet must be approved by SANDAG's Director of Rail and submitted through NCTD to BNSF for approval. The clearances are larger than the CPUC limits to accommodate future electrification per the NCTD/BNSF Shared Use Agreement.

3.3 Track Spacing

Track centers on tangent shall be 15 feet. On curves, track centers shall be increased as follows:

- Increase 1-inch for each 30 minutes of curvature
- Increase distances between track centers shall be applied in ½-inch increments.
- Where adjacent track is on the outside of a curve and its superelevation is more than on the inside track, distance between the tracks shall be increased three inches for each inch difference in superelevation.

Track spacing where two or more tracks are present at station platforms shall be wide enough to allow for an inter-track fence. This will require a minimum track spacing of 19 feet. The expanded track centers should extend a minimum of 150 feet beyond the end of the proposed platform and any foreseeable extensions, at each end of the station.

Track spacing in tunnels shall be determined by the tunnel configuration. Refer to the Tunnel Basis of Design Report.

Track spacing where tracks are designed on separate bridge structures shall have a minimum of 25 feet.

4 Grade Crossings and Sealed Corridors

4.1 General

Although grade crossings for FRA Class 6 track may be permitted, it is recommended that existing grade crossings on track segments in which the speeds will be increased be eliminated wherever possible and that the introduction of new grade crossings be avoided.

4.2 Criteria References

The California Public Utilities Commission (CPUC) regulates the safety at all Highway-Rail Grade Crossings in California. CPUC General Orders (GOs) set the minimum requirements for all crossings in the state, and rail crossing construction or modification must be authorized by CPUC.

The specific CPUC GOs that shall govern are:

- CPUC GO No. 26 - Clearances
- CPUC GO No. 72 - At-Grade Crossings
- CPUC GO No. 75 - Protection of Crossings
- CPUC GO No. 88 - Rules for Altering Public Grade Crossings

Chapter 18 Grade Crossing Warning Systems and Chapter 19 Grade Crossing Roadway Traffic Systems of the LOSSAN Design Criteria Manual, Volume III, Draft 4 apply to the grade crossing designs. They include detailed geometric criteria for horizontal and vertical approaches, traffic control devices, and traffic signal preemption, along with maintenance and operational responsibilities.

Caltrans provides guidelines for the roadway and vehicle safety designs. See the list and links below:

- Pedestrian Accessibility Guidelines (Design Information Bulletin 82-06)
- Highway Design Manual railroad crossing references
    - Section 309.1, 309.2, 309.5 - Horizontal and Vertical Clearances
    - Section 204.8, 208.9 - Underpasses and Overheads
    - Section 104.3 (frontage); Sections 403.3, 1003.5 (Angle)
- California Manual on Uniform Traffic Control Devices

Local Highway Agency Standard Drawings and Specifications will apply to the roadway approaches at each Crossing.

4.3 Sealed Corridor

A sealed Highway-Rail Grade Crossing adds protection from errant vehicles and pedestrians during train operation. Several items involved in sealing the Crossing include:

1) Fence
2) Median required as best practice for multi-lane roadways
3) Exit Gate Systems (LOSSAN Chapter 18.1.6 and 18.2)

4) Additional Warning Devices (Signs)
5) Vehicle Intrusion Detection Devices / Loops (LOSSAN Chapter 18.2.3)
6) Pedestrian Warning Devices (LOSSAN Chapter 18.1.7 and 18.3)
7) Adjacent Traffic Signal Preemption (LOSSAN Chapter 18.1.8 and 18.4)
8) Addition of Backup Traffic Signal Power (LOSSAN Chapter 18.1.10)

Site specific designs shall be based on findings of CPUC diagnostic meetings as specified in the LOSSAN Design Criteria Manual.

4.3.1 Fencing at Grade Crossings

Fence shall be installed running parallel to the track at the Railroad right of way for a distance of at least 250 feet. The fence shall be no taller than 36" and be placed to accommodate other required pedestrian features at each quadrant. This length of fence is a best practice on other National High-Speed Rail projects approved by the FRA. See LOSSAN Design Criteria Manual, Volume III, Draft 4, Chapter 19 for more information.

4.3.2 Pedestrian Crossings

Pedestrian crossings shall conform to Chapters 18 and 19 of the LOSSAN Design Criteria Manual. Pedestrian warning devices shall be standard AREMA, California Manual on Uniform Traffic Control Devices (CA MUTCD), and CPUC compliant devices and shall include flashing lights and bells and be separate from the vehicular device. Emergency swing gates and associated buffer zones shall also be included. Walking surface shall conform to current standards of the jurisdiction of each crossing location, considering industry best practices, and LOSSAN Engineering Standard Drawings.

Signal control cabinets adjacent to pedestrian crossings shall be placed in areas that will not be in conflict with the possible position of proposed pedestrian gates. The design of these signal control houses shall be large enough to accommodate additional circuitry and backup batteries required for the pedestrian gate operation.

At crossings where an existing sidewalk is located between the roadway and the roadway crossing gate device, CPUC recommends that the sidewalk be relocated behind the gate so that it will be at least 4 feet 3 inches from the center of the roadway crossing gate device, so as to provide appropriate clearance for the crossing gate counterweight when the gate is in the lowered position and accommodate the pedestrian swing gate and buffer zone configuration.

4.4 Field Diagnostic

The Field Diagnostic Meeting should be held early in the process to establish specific improvements at each crossing. See LOSSAN Design Criteria Manual, Volume III, Draft 4, Chapter 19.3 for more detailed information on attendees, meeting notes, and items to be reviewed.

4.5 Design Criteria

Designs shall incorporate the standards reflected in the LOSSAN Engineering Standard Drawings. Any variations to the standards required to conform to site conditions shall be approved by NCTD and SANDAG.

Table 4-1. Grade Crossing Location

Type of Track Item	Criteria
Location of Point of Switches (PS) of turnouts	Not closer than 100’ from end of crossing surface (See Section 5.2.1)
Exothermic Rail Welds, Insulated Joints, or bonds	Not closer than 10 feet from end of crossing surface
Turnouts and Crossovers	Not closer than 100’ from end of crossing surface (See Section 5.2.1)
Crossing Surface	Concrete Precast Panels with Timber Ties
Curved Track	Not closer than 25’ from end of crossing surface’

5 Railway Signaling and Communications Train Control Signals

5.1 Railway Signaling and Communications General

5.1.1 Application of Criteria

This section includes railroad engineering design criteria to be used in the design of railway signaling and communications systems.

The purpose of this section is to establish uniform and minimum standards for planning, engineering design, and construction. This document is based on industry standards, regulatory requirements, and recommended practices for Commuter, High-Speed Passenger, and Class I railroads. Conformance to the design criteria is required to achieve uniformity in the preparation of construction documents.

5.1.2 Design Guidelines, Codes, Manuals, Standards & Specifications

North County Transit District operates and maintains the railroad right-of-way from the Orange County line to the Santa Fe Depot in the City of San Diego.

The railroad systems design shall meet all applicable parts of the CPUC requirements, FRA safety requirements, and Federal Communications Commission (FCC) requirements.

The design criteria assembled are based on industry standards, governmental regulations, local practices, and railroad guidelines/standards. The most recent editions of the following publications and documents were used:

- FRA Title 49 Code of Federal Regulations (CFR) Parts 213, 214, 234, 235, and 236
- CPUC General Orders
- AREMA Recommended Practice
- California Manual on Uniform Traffic Control Devices (CA MUTCD)
- State of California Division of Occupational Safety and Health (Cal/OSHA) safety orders
- NCTD and BNSF Timetables

- NCTD Design Criteria
- NCTD Engineering Standards

5.2 Train Control Systems Criteria

5.2.1 General

The designer shall specify equipment and applications that are fail-safe and have proven to be reliable, durable, and effective on other Commuter, High-Speed Passenger, and Class I rail networks. The design shall incorporate features that shall aid maintenance forces in the inspection, testing, repair, and overall maintenance of the system.

Application logic software for microprocessor-based systems shall be "safe" and conform to all applicable regulatory rules and regulations but also "simple in form" so as to be easily understood by personnel responsible for the maintenance and care of the system. Where these guidelines make reference to system logic and design criteria utilizing vital relays, the same logic shall be applied to solid-state electronic interlocking application programs.

Signals shall be placed a minimum of 50' from the point of switch. Signals shall also be placed a minimum of 50' from the edge of a grade crossing to allow for the crossing island. Any deviation from this must be approved by NCTD.

5.2.2 Safe Braking

When proposing modifications to existing signal systems, it is necessary to consider the impacts of proposed signal placements relative to average track grades, distance to the adjacent signal locations, and the specific types of trains operating through the project limits.

Signal spacing shall consider all factors necessary to provide a safe and efficient operation. Where practical, the signal block length shall be between 6,000 and 9,000 feet in length. Such spacing affords passenger trains to operate with optimum headways, and utilization of "fourth aspect" (i.e. flashing yellow) signaling provides "safe braking distance" for freight trains. Considerations should be taken for the change of the existing signal rules for trains operating with Positive Train Control (PTC), removing the speeds associated with signal aspects for trains with PTC active. Without PTC active, trains will only be able to operate at 59 MPH.

As part of the design effort, the designer shall calculate the distance and average grade for each signal block, and perform necessary safe braking calculations for each type of train allowed to operate through the project limits and as allowed for by San Diego Subdivision Timetable Special Instructions.

With increased speed, the time required for Time Locking should be analyzed to determine what is safe. The use of Approach Locking should similarly be researched. Approach Locking will require extensive additional testing on the corridor. Safe braking calculations will be utilized for FRA Class 6 (110 MPH) for passenger trains, while also ensuring safe operations for freight traffic.

5.2.3 Signal Visibility

Signals shall be placed and aligned to allow optimum viewing by the locomotive engineer. Where possible, signals shall be placed adjacent to tangent track, and the locomotive engineer shall be provided an unrestricted view of the signal for a minimum of 2,000 ft. in approach to the signal, and where speeds exceed 90 MPH the distance should be raised to 2500 feet. Where the 2,000 ft. (or 2500 feet above 90 MPH) signal preview distance cannot be provided, the designer shall obtain guidance from NCTD Operations. The design may incorporate more restrictive aspects in approach to a STOP signal with restricted preview.

Where possible, block signals shall be placed to the right of the track governed. Left-hand signals shall be placed where track centers do not accommodate right-hand placement. Back-to-back ground signals shall be placed where practical to minimize construction costs. In some areas, signal bridges or cantilever signal structures may be required due to an inability to meet horizontal or vertical clearances or other issues. Where practical, signals shall be placed in full view of station platforms so that the aspect displayed can be seen by the locomotive engineer when leaving the station.

5.2.4 Microprocessor-Based Systems

Wayside signal locations shall utilize microprocessor-based systems configured for use with colorlight LED signal units. These systems shall also be capable of interfacing with existing controllers utilizing a serial or modem connection without the use of external signal converters. The utilization of vital relays shall be minimized where possible. Microprocessor-based systems shall be equipped with electronic data recorders that will record information useful in the maintenance and repair of the system.

Electronic coded track circuits shall be utilized to transmit and receive vital block signal data between wayside signal locations. Standard code configurations shall be applied wherever possible and in accordance with current NCTD systems practice.

Application logic shall be configured to provide “approach lighting” of signals. Control signals shall light on approach, when a “signal control” bit is received from the control station, and when a test clip or switch is “closed” (i.e. lamp test). Where multiple track operations are present, lighting cross checks shall not be utilized except as directed by NCTD.

Application software operating features, nomenclature, and equation configuration shall be consistent with logic currently present on NCTD systems.

5.2.5 Communications

The designer shall depict requirements to maintain the existing communications network in support of centralized traffic control (CTC) and PTC systems. A fiber optic backbone communications system is the primary train control communications data transport system and will be interfaced with the wayside signal system in order to transport critical data to and from the Operations Control Center facility located in Escondido, CA.

The Communication Systems specified by the designer shall be safe, reliable, maintainable, and compatible with existing NCTD systems, utilize current technologies, and meet the availability requirements of the system. The designer shall specify components that have been accepted for use on NCTD property.

5.2.6 Train Control Systems Materials

The designer shall specify equipment, materials, and components that are readily available and currently in use on NCTD systems. The purpose of this is to establish standard applications and maintainability for train control systems and to develop consistency for systems utilized throughout the rail corridor.

In general, equipment, material, and components for train control systems will consist of:

- Instrument enclosures

- Wayside Signal/Cantilevers/Bridge Structures
- Dual Control Switch Machines and Layouts
- Switch Circuit Controllers
- Microprocessor-Based Systems
- Coded Track Circuits
- Communications Network Components
- Relays
- Battery and Charging Equipment
- Vaults, Pull Boxes, Cable, and Wire
- Miscellaneous Products and Components

5.2.7 Positive Train Control

In order for trains to operate at FRA Class 6 speed, the PTC System will need to meet the fail-safe operation per 49 CFR §236.1007 (Additional requirements for high-speed service). The PTC system must be certified as a vital overlay. Speeds above 90 MPH cannot be operated at with a non-vital overlay. The currently deployed Interoperable Electronic Train Management System (I-ETMS) system is capable of supporting speeds above 90 MPH if deployed as a Vital System. Use of the I-ETMS is beneficial as it is already being used by all of the operators along the corridor and does not require any additional equipment for the on-board segment.

5.3 Highway-Rail Grade Crossing Warning System Criteria

5.3.1 General

The designer shall specify equipment and applications that will not only provide optimum safety but also will maximize the efficiency and reliability of the passenger and freight rail system. The design shall incorporate systems and equipment that have been proven to be reliable, durable, and effective on other rail networks and are in current use by NCTD. Introduction of new materials, which would require an inventory of spare parts and additional training, must be approved by NCTD.

The design shall incorporate features that will aid signal personnel in the inspection, testing, repair, and overall maintenance of the system. Any new test equipment or procedures required by new materials or methodologies must be identified and submitted to NCTD for consideration.

5.3.2 Existing Systems Operational Overview

Current grade crossing warning systems are typically comprised of constant warning devices which are utilized to detect incoming train movements, and solid-state crossing controllers to activate the warning system.

Active warning devices are consistent with CPUC GOs and are typically comprised of CPUC Standard No. 8, No. 9, No. 9A, or No. 9E warning devices.

5.3.3 Train Detection System

For Higher-Speed operations, a Wireless Crossing Activation System (WCAS) shall be incorporated with the use of redundant constant warning devices. The constant warning devices will operate for slower train traffic and as a redundant protection. The WCAS will utilize the I-ETMS system. The use of WCAS will limit the need for extra-long crossing approaches. Existing long approaches will also be reviewed to use shorter approaches for slower trains and utilize the WCAS for passenger trains operating at above freight train speeds. These train detection systems shall be combined with solid-state crossing controllers to ensure compliance with “lamp voltage” and “standby lamp voltage” regulations. Event recorders shall be utilized to record data useful in the maintenance, troubleshooting, and repair of the entire system. Event Recorders can be placed on the rail network with notifications provided back to the Operations Control Center (OCC). Where it is necessary to deviate from preferred grade crossing control standards, approval must be obtained from NCTD.

On multiple track where uni-directional applications are utilized, a single two-track unit should control warning for train movements on Main Track No. 1, a second unit should control warning for movements on Main Track No. 2, a third unit for Main Track No. 3, and so on.

5.3.4 Selection of Warning Time

The warning time at a grade crossing must be sufficient for vehicles and pedestrians to clear the track(s). The minimum warning time required by law for motor vehicles is 20 seconds. The design minimum for through train moves on NCTD is 30 seconds and is based upon 20 seconds minimum warning time plus 10 seconds buffer time. The actual warning time may differ from the design minimum due to variations in train speed in the approach to the crossing. The only exception to the requirement for a 20 second minimum warning time occurs when a train stops in the approach to a grade crossing. A Wireless Crossing Nearside Stations Stop (WCNSS) can be utilized where stations are near grade crossings to prevent the pre-ring of the crossing when the train will make a station stop.

Guidelines for vehicular warning time are spelled out in the AREMA Manual, as well as the requirements in Part 234 of CFR Title 49; however, there are no comparable guidelines for pedestrians.

There are existing warning time guidelines for Light Rail Systems under MUTCD Part 10, as well as standards for pedestrian crossings for roadways under MUTCD Part 4. These standards derive timing based on a walking speed of 4 feet per second. American Disability Act Accessibility Guidelines (ADAAG), however, recommends a walking speed of 3 feet per second to allow for the elderly.

5.3.5 Frequency Selection

For the constant warning devices all systems shall be applied in accordance with the manufacturer’s recommendations. The preferred application is bi-directional, but uni-directional applications shall be utilized to provide adequate frequency separation, where following train movements may occur, and where insulated joints must be maintained in the vicinity of crossings to support wayside signal systems.

Remote applications should be used where insulated joints exist within the approach limits to the crossing. Tuned joint couplers may be used only when applied in accordance with the manufacturer’s recommendations. Additional systems may be required to accommodate special applications and unique train operations. When a grade crossing adjoins a Control Point, the designer must carefully analyze moves towards the grade crossing and determine whether special circuits are required to mitigate a potential momentary loss of detection as the train diverges from the track on which detection is active.

The preferred constant warning device frequencies to be utilized are 86, 114, 156, 211, 285, 348, 430, 525, 645, 790, and 970 Hertz for the primary system. Utilization of the 348 HZ system shall be confined to areas where 60 HZ interference is not likely and electrified transit systems do not parallel the tracks. The frequency selected shall be dependent upon the required approach distance and ballast conditions. A 4 Ohm/1000 ft. distributed ballast resistance value shall be utilized in comparing frequency to required “look” distance. “Six wire” applications shall be avoided where possible. High impedance termination shunts, such as the NBS-2, should be used.

Signal circuitry island frequencies shall be 10.0 KHZ, 11.5 KHZ, 13.2 KHZ, and 15.2 KHZ. Random Signature Island (RSI) modules are acceptable for use on NCTD property. Careful evaluation of existing frequencies and equipment shall be made prior to selecting island frequencies.

5.3.6 Grade Crossing Warning Systems Materials

The designer shall specify equipment, materials, and components that are readily available and currently in use on NCTD systems. The purpose of this is to establish standard applications and maintainability for highway-rail grade crossing warning systems and to develop consistency for systems utilized throughout the rail corridor.

In general, equipment, material and components for train control systems will consist of;

- Instrument enclosures
- CPUC Standard Warning Devices
- Constant Warning Devices
- Solid-state Crossing Controllers
- Data Recorders
- Relays
- Battery and Charging Equipment
- Vaults, Pull Boxes, Cable, and Wire
- Miscellaneous Products and Components

Appendix K. Basis of Design Report for Tunneling

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SANDAG

Tunnel Basis of Design Report

San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study (SD-LOSSAN)

January 12, 2022

Prepared by:

HDR

MOTT MACDONALD

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619) 699-1900
www.sandag.org

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ABSTRACT

TITLE:	Tunnel Basis of Design Report
AUTHOR:	San Diego Association of Governments
DATE:	January 12, 2022
SOURCE OF COPIES:	San Diego Association of Governments 401 B Street, Suite 800 San Diego, CA 92101 (619) 699-1900
NUMBER OF PAGES:	52
ABSTRACT:	This report presents the tunnel basis of design to be followed for the Del Mar and Miramar tunnel alternatives analysis and conceptual engineering studies

The tunnel space proofing and resulting tunnel diameters for twin and single bores shown in this report do not reflect the tunnel optimization effort. Refer to Appendix M Tunnel Optimization Memo for updated tunnel diameters.

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Acronyms

ASCE	American Society of Civil Engineers
CBC	California Building Standard Codes
CCR	California Code of Regulations
CFD	Computational Fluid Dynamics
CPUC	California Public Utilities Commission
EPB	Earth Pressure Balance
FEMA	Federal Emergency Management Agency
F_{PGA}	Site Coefficient Peak Ground Acceleration
HDPE	High Density Polyurethane
LOSSAN	Los Angeles-San Diego-San Luis Obispo Rail Corridor Agency
MCE_g	Maximum Considered Earthquake
MCE_G	Maximum Considered Earthquake Geometric Mean
MSL	Mean sea level
Mya	Million Years Ago
NFPA	National Fire Protection Association
PGA	Peak Ground Acceleration
PGA_M	Peak Ground Acceleration Mean
PVC	Polyvinyl Chloride
RCFZ	Rose Canyon Fault Zone
SCRRA	Southern California Regional Rail Authority
SEM	Sequential Excavation Method
Su	Undrained shear strength
TBM	Tunnel Boring Machine
UCS	Unconfined compressive strength

1 Introduction

1.1 Project Overview

The Los Angeles-San Diego-San Luis Obispo Rail Corridor (LOSSAN Corridor) stretches 351 miles through six southern California counties. The market for passenger rail service is well established in the LOSSAN Corridor, which is the nation's second busiest passenger rail corridor. Nearly eight million passengers use the corridor's intercity and commuter rail services annually. The San Diego Subdivision is the southernmost 60.1 miles of the corridor, from the Orange County/San Diego County line to downtown San Diego. More than fifty trains use this segment of the corridor daily, including Amtrak Pacific Surfliner intercity, Southern California Regional Rail Authority (SCRRA) Metrolink and North County Transit District (NCTD) COASTER commuter, and BNSF Railway freight services. LOSSAN Corridor planning documents include the goal of doubling the number of rail services along the San Diego Subdivision by 2035.

This conceptual engineering study assesses the current conditions in the corridor along the San Diego Subdivision (project area) and develops a program of improvements to address both current and future demand by increasing the corridor's competitiveness with driving the congested parallel Interstate 5 corridor. This program of improvements will increase track capacity, improve resiliency, enhance safety, and support increased passenger and freight frequencies. Improvements include alternative alignments along key segments of the corridor, grade separations, and other enhancements.

1.2 Purpose and Scope of this Report

This Basis of Design Report provides design criteria for proposed alternative alignments that include the Del Mar Tunnel options (approximately 5 miles long) and the Miramar Tunnel options (approximately 3 miles long). The tunnels are proposed to facilitate improved service on the LOSSAN Corridor along the San Diego Subdivision. The tunnel alignments are shown in the Del Mar Alternatives Analysis Report and Miramar Alternatives Analysis Report.

The objective of this report is to describe the underground portion of the corridor, underground project components, and their function. The report summarizes the baseline description, functions, constraints, and assumptions and contains a preliminary list of codes and standards that have been used in developing the alignment options.

The report discusses the following:

- The alternative tunnel alignments
- Geotechnical information for the project area
- Ground risks
- Tunnel design criteria and concepts for the tunnels
- Station design criteria and concepts for the station
- Fire life safety considerations
- Shared use criteria for passenger and freight operating in the tunnels
- Tunnel systems

- Tunnel operation and maintenance
- Construction staging and constructability
- Construction impacts
- Project risks
- Current trends in tunnel technology

This report provides a uniform design basis for underground structures. The criteria serve as guidelines for design professionals. They do not substitute for sound engineering judgment or compliance with applicable codes and local approvals for fire life safety and security. Facilities and systems should be designed to relevant codes and standards for safe operation.

Tunnel design and construction shall be integrated and coordinated with other project requirements including track alignment, vehicle dynamic envelope, tunnel opening size relative to clearance envelopes, space proofing for systems infrastructure, cost-effective maintenance and safe operations. Constructability, durability and cost-effectiveness are key considerations for tunnel design. Means and methods of tunnel construction shall be consistent with project requirements.

- Design internal dimensions to accommodate vehicles, track, super elevations, emergency egress, walkways, ventilation, maintenance and systems infrastructure.
- Incorporate spatial, clearance and tolerance requirements for services and equipment.
- Demonstrate that design accounts for geology, stability during excavation, ground support to maintain worker health and safety, variability in ground conditions, rock and soil geotechnical properties, rock mass strength and behavior, ground movements, groundwater inflow, insitu stress, earthquake resistance, support and lining durability, and mitigation of foreseeable risks.
- Design to maintain structural integrity of existing utility infrastructure and third party facilities, and maintain ground movement to acceptable limits.
- Design to provide a safe working environment, maintain stability during tunnel excavation, and minimize ground movements and detrimental impacts of groundwater flow.
- Demonstrate mitigation of potential settlement and damage to existing infrastructure with 2D or 3D numerical modelling of the ground with finite element or finite difference software.
- Prepare design reports, drawings, specifications and other supporting documents to demonstrate that design of tunnels meets requirements over the design life."

1.3 Applicable Codes and Standards

The following list of applicable codes and standards is provided for guidance. Additional codes and standards will be identified in future design stages.

- Design Criteria. LOSSAN Corridor in San Diego County, Volume III
- AASHTO
  - LFRD Road Tunnel Design and Construction Guide Specifications
  - LFRD Bridge Design Specifications
- American Railway Engineering and Maintenance-of-Way Association (AREMA)

  - Manual for Railway Engineering
  - Improved Seismic Design Criteria for California Bridges: Provisional Recommendations.

- American Concrete Institute (ACI)
  - ACI 318, Building Code Requirements for Structural Concrete
  - ACI 365.1R - Service Life Prediction
  - ACI 506R - Guide to Shotcrete
  - ACI 506.1R - Guide to Fiber Reinforced Shotcrete
  - ACI 506.2 - Specification for Shotcrete
  - ACI 533.5R - Guide for Precast Concrete Tunnel Segments
  - ACI 544.7R - Design and Construction of Fiber-Reinforced Precast Concrete Tunnel Segments
  - ACI 544.8R - Indirect Method to Obtain Stress-Strain Response of Fiber-Reinforced Concrete

- American Institute of Steel Construction (AISC)
  - Seismic Design Manual
  - Steel Construction Manual

- American Society of Civil Engineering (ASCE)
  - Minimum Design Loads for Buildings and Associated Criteria for Buildings and Other Structures
  - Geotechnical Baseline Reports for Underground Construction, ASCE Research Council

- California Department of Transportation (Caltrans)
  - Highway Design Manual
  - Bridge Design Manuals
  - Standards, Specifications, and Plans
  - Trenching and Shoring Manual

- California Code of Regulations (CCR)
  - Title 8, Industrial Relations
  - Title 24, Building Standards, Part 2, California Building Code (CBC)
  - Title 24, Building Standards, Part 4, California Mechanical Code (CMC)
  - Title 24, Building Standards, Part 5, California Plumbing Code (CPC)
  - Title 24, Building Standards, Part 6, California Energy Code

- National Fire Protection Association (NFPA)
  - NFPA 10, Standard for Portable Fire Extinguishers
  - NFPA 13, Standard for the Installation of Sprinkler Systems
  - NFPA 14, Standard for the Installation of Standpipe and Hose Systems
  - NFPA 15, Standard for Water Spray Fixed Systems for Fire Protection
  - NFPA 24, Standard for the Installation of Private Fire Service Mains and their Appurtenances
  - NFPA 70, National Electrical Code
  - NFPA 72, National Fire Alarm Code
  - NFPA 92, Standard for Smoke Control Systems
  - NFPA 101, Life Safety Code
  - NFPA 130, Standard for Fixed Guideway Transit and Passenger Rail Systems
  - NFPA 220, Types of Building Construction.

  - NFPA 2001, Clean Agent Fire Extinguishing Systems.

- Occupational Safety and Health Administration, Department of Labor
  - Occupational Safety and Health Standards, 29 CFR Part 1910

- Sheet Metal and Air Conditioning Contractors National Association (SMACNA)

1.4 Previous Studies

Several previous studies looked at potential alignments for both the Del Mar and Miramar tunnel alignments. The following relevant studies have been reviewed:

- Conceptual Engineering and Environmental Constraints for Double Track Alignment Alternatives between Del Mar Fairgrounds and Sorrento Valley, dated December 29, 2017, prepared for SANDAG by HNTB.

- Miramar Tunnel Feasibility Study for LOSSAN Corridor, dated November 9, 2018, prepared for Caltrans by Parsons Transportation Group

2 Alternative Alignments

The current conceptual engineering study evaluates multiple proposed alignment alternatives for the Del Mar Tunnel and Miramar Tunnel segments. For descriptions of the alignment alternatives for the Del Mar Tunnel, including the Camino Del Mar and Crest Canyon High Speed alignments, see the Del Mar Alternatives Analysis Report. For descriptions of the alignment alternatives for the Miramar Tunnel, including the UTC (University Town Center) alignment and Torrey Pines alignment, see the Miramar Alternatives Analysis Report.

3 Design Constraints

The following assumptions have been established for this study and only pertain to civil constraints for the tunnel. For tunnel portal design criteria, refer to Section 7.1. For other design constraints and criteria such as FLS, Ventilation, Systems, Communications, etc refer to their respective BOD reports and/or AA reports:

- Headways for the alignment are based on the forecasts in the California State Rail Plan.

- The preliminary tunnel cross sections are based on current SANDAG standards and fleet of diesel locomotives.

- Passenger loads are based on a maximum 10-car consist.

- Freight usage includes general merchandise, automobiles, and military/project cargo. Further studies are required to determine any explosive or flammable cargo that will be transported through the tunnels. Appropriate explosive and flammable material constraints will be considered following those studies.

- Train speeds within the tunnels are based on the alignment constraints.

- The tunnels will be used by both passenger and freight trains. Any differences in operational requirements will be considered. For a list of operational constraints for each fleet type, refer to the Basis of Design Criteria for Track, Grade Crossing and Signals for FRA Class 6 Track.

Where possible in determining the optimum tunnel alignment the following underground criteria will be used:

- Minimize tunnel length to reduce underground construction costs, note that track speed requirements will dictate tunnel length and that the shortest tunnel was one of slower travel times.
- Minimize deep alignments below the groundwater table
- Avoid poor ground conditions
- Minimize dewatering during tunneling where possible, note this assumption will be re-examined in further studies.
- Minimize private land takes, note some areas may not be avoidable due to track speed/travel time requirements.
- Maintain a minimum ground cover of one tunnel diameter as an initial criterion to generate preliminary alignments
- Maintain a ratio of 25D for the minimum horizontal curve, where D is the excavated diameter of the tunnel

Note that the above points are assumptions and design constraints to be considered, not formal criteria. Expanded details for design constraints and other track criteria located throughout the report can be found in the Basis of Design Criteria for Track, Grade Crossing and Signals for FRA Class 6 Track.

4 Geotechnical Information

4.1 Regional Geologic Setting

The project study area is situated in the western portion of the Peninsular Ranges geomorphic province of southern California. This geomorphic province encompasses an area that roughly extends from the Transverse Ranges and the Los Angeles Basin, south to the Mexican border, and beyond another approximately 800 miles to the tip of Baja California (Norris and Webb 1990; Harden 1998). The geomorphic province varies in width from approximately 30 to 100 miles, most of which is characterized by northwest trending mountain ranges separated by subparallel fault zones. In general, the Peninsular Ranges are underlain by Jurassic-age metavolcanic and metasedimentary rocks and by Cretaceous-age igneous rocks of the southern California batholith. Geologic cover in the westernmost portion of the province in San Diego County generally consists of Upper Cretaceous, Tertiary, and Quaternary-age sedimentary rocks and includes the Eocene-age Scripps Formation, Ardath Shale and La Jolla Group. Structurally, the Peninsular Ranges are traversed by several major active faults. The Whittier-Elsinore, San Jacinto, and the San Andreas faults are major active fault systems located northeast of the site and the Rose Canyon, Coronado Bank, San Diego Trough, and

San Clemente faults are major active faults located to the west-southwest. Major tectonic activity associated with these and other faults within this regional tectonic framework is generally right-lateral strike-slip movement. These faults, as well as other faults in the region, have the potential for generating strong ground motions in the project area.

4.2 Site Setting

The Del Mar Tunnel alternative alignments traverse the Del Mar Mesa between the San Dieguito River valley in the North and Peñasquitos Lagoon and Soledad River valley in the south. The Del Mar Mesa is underlain by gently dipping marine sedimentary rocks of the Eocene-age La Jolla Group. This consists of several formations including the Delmar Formation and Torrey Sandstone, which will be the principal materials encountered on the alignments.

The Miramar Tunnel alignment will traverse the Miramar Mesa between the Sorrento Valley in the north and Rose Canyon in the south. The mesa has been incised by drainage features near the center and along its margin, resulting in the formation of a system of interconnected gullies and canyons that can be up to several hundred feet deep. Several drainages are present along the alignment, many of which have been infilled during previous grading operations. The tunnel alignment is located north of and above the Rose Canyon drainage. This drainage occupies a low-lying fault-controlled valley that developed by uplift of Mount Soledad, which was associated with fault displacement within the Rose Canyon Fault Zone. This zone is comprised of numerous active and potentially active faults that trend mostly northwest/southeast within Rose Canyon, generally to the west of the south portal.

4.3 Drainage

The flood hazard potential along the alignment was evaluated based on flood hazard maps available through the Federal Emergency Management Agency (FEMA) Map Service Center website. These identify special flood hazard areas subject to inundation by the 1% annual chance flood (100-year flood). The conceptual rail alignment design will be developed for the 0.2% annual chance flood (500-year flood).

Flood hazard maps are included in Appendix A for the Del Mar Tunnel northern and southern portal locations and the Miramar Tunnel northern and southern portal locations.

While the alternative tunnel alignments will typically be outside of the flood hazard zones, flood hazard zones are a consideration for the tunnel approaches. The alternative alignments, with regard to the flood maps, will either:

- Avoid the flood plain area where possible
- Provide portal protection in the flood plain to ensure the tunnel is not inundated

Low points within the tunnels will require low point sump pump stations, which will be located within cross passages for the twin bore tunnel option or in-tunnel for the single bore option. Portals must be protected from surface run-off as the tunnel drainage is designed to handle fire standpipe water only.

4.4 Geotechnical Units

The extract of the Geologic Map of the San Diego 30' x 60' Quadrangle (Kennedy and Tan 2008), covering the area between La Jolla and Solano Beach including the Del Mar and Miramar corridors, is included in Appendix B. The geologic units present in the Del Mar and Miramar corridors are described in sections 4.4.1 and 4.4.2, respectively.

4.4.1 Del Mar Tunnel Alignments

The Del Mar Tunnel alignments are within gently dipping weak marine sedimentary rock of the Eocene-age La Jolla Group. These sedimentary rock underlay much of the coastal area of western San Diego County, and they consist of several different formations. Two of these, the Delmar Formation and the Torrey Sandstone should be encountered along the tunnel alignments. In addition to these formations, old paralic estuarine deposits, younger alluvial deposits, landslide deposits, and artificial fill will also be present along the alignment. A brief description of each of these geological units is as follows:

af Artificial fill
Deposits of fill resulting from human construction, mining, or quarrying activities; includes compacted engineered and non-compacted, non-engineered fill. Some large deposits are mapped, but in some areas no deposits are shown. Artificial fill, possibly derived from earthworks for construction of the Interstate 5, are indicated on the Geologic Map at a potential south portal site south of Carmel Valley Road.

Qpe Paralic estuarine deposits
Unconsolidated estuarine deposits. Composed mostly of fine-grained sand and unconsolidated clay and silt. Recent estuarine deposits exist north of Del Mar Mesa, within the San Dieguito River and Lagoon and south of Del Mar Mesa, within Peñasquitos Lagoon.

Qya Young alluvial flood-plain deposits
Poorly consolidated, poorly sorted, permeable flood-plain deposits of sandy, silty or clay-bearing alluvium. Qya occurs north of Del Mar Mesa, within the San Dieguito River and Lagoon and south of Del Mar Mesa, within Peñasquitos Lagoon.

Qls Landslip deposits
Two landslides have been mapped by Ninyo and Moore on the northern portion of the project site adjacent to Racetrack View Drive (Ninyo and Moore 2014). These landslides are apparently large blocks of formational materials that have moved to the north along relatively low-angle clay-lined rupture surfaces.

Qop_6 Old paralic deposits -Unit 6
The old paralic deposits are interfingered strand line marine and non-marine sediments consisting of loose to medium-dense, unconsolidated silty to clayey sand and sandy clay. These deposits rest on the Nestor terrace, near sea level to approximately 80 feet to 120 feet above mean sea level.

Tt Torrey sandstone
Medium to coarse-grained, moderately well indurated, massive and broadly cross-bedded, arkosic sandstone. Relatively thin claystone beds exist near the base of the unit, where it is conformably interbedded with the Delmar Formation. Bedding is essentially flat to dipping roughly 3 to 5 degrees to the east, northeast, and southeast.

Td Delmar Formation
The Delmar Formation conformably underlies and is interbedded with the Torrey Sandstone. It consists of sandy claystone interbedded with medium-gray, coarse-grained sandstone and clayey sandstone. The claystone is typically highly expansive. Bedding is essentially flat to

dipping roughly 3 to 5 degrees to the east, northeast, and southeast. Numerous coastal bluff failures have occurred in the Del Mar Formation.

4.4.2 Miramar Tunnel Alignments

At the southern end of the Miramar Tunnel alignments, the alluvial deposits consist of recent alluvium, colluvial deposits and stream terrace deposits. From these unconsolidated deposits, the alignments rise to a mesa consisting of consolidated Eocene and Pleistocene-age sandstone and siltstone. The top of the mesa is capped by the Lindavista Formation, which is an older marine terrace platform deposits. As the alignments move north from Rose Canyon, the southerly portion of the alignments is underlain mostly by the Scripps Formation, which is composed of marine sandstone that overlies the deeper marine Ardath Shale.

Moving north, the alignments are expected to transition into the Ardath Shale within the tunnel segment. Previous geotechnical investigations (Parsons and Kleinfelder 2018) found the Scripps Sandstone/Ardath Shale to be transitional with lenses of claystone, siltstone and sandstone. At their northern ends, the alignments move out of the mesa. In this area, outcrops of the Ardath Shale are separated by infilled areas of artificial fill, placed in drainage channels during the construction of the existing building developments.

af Artificial fill

Deposits of fill resulting from human construction, mining, or quarrying activities; includes compacted engineered and non-compacted, non-engineered fill. Some large deposits are mapped, but in some areas no deposits are shown.

As the alignment progresses northerly and out of the mesa, outcrops of the Ardath Shale are separated by infilled areas of artificial fill, placed in drainage channels during construction of the existing building developments. These drainages have a shallow accumulation of alluvium that overlies the Ardath Shale. The alluvium and Ardath Shale are locally mantled with artificial fill placed during the various construction projects. The fill may consist of either clay to silty clay, sandy lean clay, clayey sand, or silty sand. Silts, sands, and gravels were encountered in relatively small quantities. Most of the fill was likely placed during the original construction of the existing commercial and residential developments. Fill may have been derived from local sources cut from both the Ardath Shale and the Scripps Formation.

Qya Young alluvial flood-plain deposits; stream terrace deposits

Poorly consolidated, poorly sorted, permeable flood-plain deposits of sandy, silty or clay-bearing alluvium. Qya is present in Sorrento Valley drainage where it consists of sandy lean clay and poorly graded gravel with silt and sand.

Qoa Old alluvial flood-plain deposits

Fluvial sediments deposited on canyon floors. Consists of moderately well consolidated, poorly sorted, permeable, commonly slightly dissected gravel, sand, silt, and clay-bearing alluvium. Qoa is present on the terrace on the north side of Rose Canyon where the alternative sites for the southern portal are located. The unit ranges from a few feet to approximately 15-ft thick. It consists mostly of lean clay to fat clay and poorly graded gravel. The clay is typically secondary, being derived from pedogenic soil development processes. The material is typically hard to very hard, medium to highly plastic, moist, and organic.

Qln Lindavista Formation

This is a Pleistocene age marine terrace platform deposit, several tens of feet in thickness.

Tsc Scripps Formation

Mostly pale-yellowish-brown, medium-grained sandstone containing occasional cobble-conglomerate, gravel, siltstone and claystone interbeds. The Scripps Formation conformably overlies the Ardath Shale and the contact between the two formations interfingers. The Scripps Formation is thinly to massively bedded and the bedding was observed to dip on the order of up to 5 degrees to the northwest based on the Geologic Map. Structural data from borings and observations of outcrops by Kleinfelder (Parsons and Kleinfelder, 2018) show bedding to be folded with dips up to 10 to 25 degrees. The upper 10 feet of the Scripps Formation is intensely weathered and grades to moderately weathered at depth.

Limited geotechnical parameters for the Scripps Formation are given in Table 4.1 and are based on the Geotechnical and Tunneling Technical Study for Miramar Tunnel Feasibility Study (Parsons and Kleinfelder 2018).

Table 4.1. Scripps Formation - Geotechnical Parameters

Property	Range	Average
Undrained Shear Strength (Su)	6,430 psf to 10,850 psf	8,730 psf
Unconfined Compressive Strength (UCS)	1,000 psf to 3,400 psf (7 psi to 23.6 psi)	2,100 psf (14.6 psi)
Dry unit weight	Approximately 90 to 110 pcf	
Clogging Potential (Hollomann & Thewes 2013)	little clogging potential to fines dispersing	
Tunnelman's Ground Classification (Heuer 1974)	Running to flowing ground	
Face Stability – Tunnelman's Classification derived from Stability Number N (Broms & Bennemark 1967)	Fast raveling.	
Corrosivity	Contains corrosive materials	

Ta Ardath Shale

The Ardath Shale consists of mostly uniform, weakly fissile olive gray silty shale. The upper part contains thin beds of medium grained sandstone and concretionary beds with molluscan fossils. The Ardath Shale is typically soft to moderately soft rock condition with some subunits in a moderately hard condition. It ranges from moderately to slightly weathered and moderately fractured, as observed in local outcrops. The Ardath Shale contains highly cemented zones known as concretions. The concretions can occur as tabular bedded zones that extend for several tens of feet and up to 2-ft thick, or rounded blocks or boulders that can range up to 4 ft in diameter. According to Kleinfelder, local experience reveals that the distribution of the concretions is not uniform and is unpredictable (Parsons and Kleinfelder 2018). Concentrations of concretions may be high in some areas and nonexistent in others. Concretions were encountered locally in borings. The bedding of the Ardath Shale was observed, by Kleinfelder, to range from laminated to thickly bedded and dips up to 5 degrees to the northwest based on data from the Geologic Map and bedding attitudes measured by Kleinfelder in the site vicinity. Locally and near the faults, the bedding was observed to be folded with dips in excess of 10 degrees. In the tunnel alignment alternatives, the fine-grained materials of the Ardath Shale could contain potentially expansive clay.

Limited geotechnical parameters for the Ardath Shale are given in Table 4.2 and are based on Geotechnical and Tunneling Technical Study for Miramar Tunnel Feasibility Study (Parsons and Kleinfelder 2018). In the next round of study these Geotechnical Parameters will be re-examined and the ground conditions as required will be recharacterized.

Table 4.2 Ardath Shale - Geotechnical Parameters

Property	Range	Average
Undrained Shear Strength (Su)	4.50 ksf to 26.30 ksf	12.16 ksf
Unconfined Compressive Strength (UCS)	617 psf to 24,000 psf (4.3 psi to 166.7psi)	7,000 psf, (48.6 psi)
Dry unit weight	Approximately 90 to 110 pcf	
Clogging Potential (Hollomann & Thewes 2013)	Medium	
Tunnelman's Ground Classification (Heuer 1974)	Firm to fast raveling with localized squeezing ground.	
Face Stability – Tunnelman's Classification derived from Stability Number N (Broms & Bennemark 1967)	Fast raveling	
Corrosivity	Contains corrosive materials	

Parsons and Kleinfelder conclude:

> Although the "bedrock" that the tunnel alignment would traverse is Cretaceous-age sandstone and siltstone (Scripps Formation and the Ardath Shale), the process of lithification (hardening and cementation) has not resulted in hard rock along the proposed tunnel excavation. With respect to tunneling these formations are classified as soft rock or stiff soil and can be excavated easily with conventional grading equipment or even a hand shovel. They are not expected to behave like jointed rock but are more like a homogeneous stiff soil. The soft soil results in ease of mechanical excavation for tunneling but it also results in less stable ground for tunneling and will need early support to prevent tunnel collapse during construction.

4.5 Structural Geology

City of San Diego Seismic Safety Study Grid Tiles (City of San Diego 2008) show zones with geologic hazards and faults. These include zones with liquefaction potential, landform instability, and potentially active faults. Grid Tiles covering the tunnel alignments are included as Appendix B, Preliminary Geologic Maps and Appendix C, Seismic Hazard Maps. For both the Del Mar Tunnel and the Miramar tunnel, additional fault studies will be required in subsequent phases of the project.

4.5.1 Del Mar Tunnel

North-northeast trending faults are shown on City of San Diego Seismic Study Grid Tile 38. These are considered to be pre-Pleistocene (33 million years ago) in age and classified as "potentially active."

The alternative alignments cross the "potentially active" Carmel Valley Fault in the Soledad Valley. This fault is located outside of the tunnel alignments. The potential south portal sites are north of the fault.

The Crest Canyon High Speed alignment, crosses an un-named concealed "potentially active" fault south of Del Mar Heights Road, according to Grid Tile 38. The CGS Quaternary-Younger Faults ArcGIS layer (CGS 2005) records a north-northeast trending, Quaternary age "potentially active" fault crossing the Crest Canyon High Speed alignment to the west of the Red Ridge Loop Trail. This is not shown on Grid Tile 38. Surface reconnaissance performed as part of this study did not confirm the surface expression of a fault at this location. It is anticipated that at tunnel horizon, the faults will consist of clay gouge and breccia with associated sheared and folded rock.

4.5.2 Miramar Tunnel

A series of north-northeast trending faults, considered to be predominantly pre-Pleistocene (33 million years ago) in age, cross the tunnel alignments. These include the Salk and Torrey Pines fault systems. These faults are shown on Grid Tiles 30 and 34 and classified as potentially active. Potentially active faults are faults that have undergone movement during the Pleistocene epoch but ceased their activity sometime prior to the beginning of the Holocene epoch. They are commonly referred to as "pre-Holocene" faults, which correspond to an activity period of from 1.6 million years to about 11,000 years ago. Because of the long period of non-activity, fault rupture hazard risk near the alignment is considered low to negligible.

Four faults are shown crossing the tunnel alignment on Tile 34. The southern-most of these was mapped as a "potentially active" fault graben at the intersection of Genesee Boulevard and Eastgate Mall Road by Kleinfelder (Parsons and Kleinfelder 2018). At tunnel depth, the faults are likely to consist of clay gouge and brecciated material with associated shearing and folding in the proximal rock-mass.

4.6 Groundwater

4.6.1 Del Mar Tunnel

Groundwater is not anticipated to be encountered in the upper elevations of the Del Mar Mesa. Static groundwater is expected to be encountered along the lower elevations of the project area and should be further evaluated by subsurface evaluation.

Perched groundwater conditions exist in several areas along the contact between the Delmar Formation and the overlying Torrey Sandstone. Perched conditions are indicated by numerous springs and seeps exposed in the coastal bluffs and margins of the major drainages. This condition typically results from the presence of permeable sandstone underlain by less permeable claystone beds.

Groundwater levels are expected to fluctuate due to tidal variations, seasonal variations, groundwater withdrawal or injection, or other factors. Artesian conditions may exist in some areas where claystone and sandstone lenses interfinger. Water table elevations may also vary near the faults or fractures.

Given the likely tunnel alignment is above the static groundwater table, groundwater is not anticipated to be an issue on the Del Mar alternative alignments.

Leighton Associates interpret groundwater at Elevation 0' MSL along the Del Mar alternative alignments as shown on Plate 2 of the Geologic Reconnaissance Study Report enclosed as Appendix E of the Del Mar Alternatives Analysis Report. This is below the current vertical alignments of the tunnel but is based on limited data. The groundwater surface and perched water will be assessed by future sub-surface investigations.

4.6.2 Miramar Tunnel

Some groundwater inflows during Miramar Tunnel construction are anticipated and would be especially associated with potentially active faults, open discontinuities and occasional more permeable sand beds. Such inflows could cause increased stability problems locally. Kleinfelder assessed groundwater levels based on limited subsurface data and topographic considerations (Parsons and Kleinfelder 2018), which are summarized in Table 4.3.

Table 4.3. Provisional Groundwater Parameters for Miramar Tunnel Scripps Formation and Ardath Shales

Formation	Scripps Formation	Ardath Shale
Hydraulic Conductivity (k)	10^{-3} to 10^{-5} cm/sec	10^{-4} to 10^{-7} cm/sec
Hydrostatic Head	34.7 psi / 2.4 bar	31.7 to 41.2 psi / 2.7 to 2.8 bar

4.7 Local Seismicity

Although no active faulting is known to exist along the proposed alignments, local seismicity attributed to nearby active faults is expected. The Rose Canyon Fault Zone (RCFZ) was recently evaluated by SANDAG (2014) and, because it is near the alignments, could produce a seismic event within the design life of the tunnels. Except for the occurrence of a local event in 1862, there has been no significant (greater than magnitude 5 earthquake) local seismic activity during the recorded history of San Diego attributed to the RCFZ. Until the late 1980s, the RCFZ was thought to be a pre-Holocene structure and was mostly thought to be an inactive or potentially active fault. However, over the past 30 years, fault studies have shown a substantial potential for local seismic activity on certain fault segments within the RCFZ. The most important of these studies was in the southern Rose Creek drainage (east of Mount Soledad, near La Jolla), which uncovered an active strand of the Mount Soledad fault.

Evidence has been presented that indicates that the RCFZ may be structurally connected to the Newport Inglewood Fault Zone on the north and the San Miguel–Vallecitos fault or the offshore Descanso fault on the south, all of which are active faults. Based on this data, the RFCZ is likely part of a larger active fault system that stretches more than 150 miles.

Ninyo and Moore assessed seismic hazards for the Del Mar Tunnel alignments including principal faults in the proximity of the alignments (Ninyo and Moore 2014), which are shown in Table 4.4. The same faults are assessed for the Miramar alignments.

Table 4.4. Principal Faults and Proximity to Del Mar and Miramar Tunnel Alignments

Fault	Distance from Del Mar Alignments (miles)	Moment Magnitude
Rose Canyon	2.0	7.2
Coronado Bank	16.2	7.6
Newport Inglewood	16.7	7.1
Elsinore (Julian Segment)	30	7.1
Elsinore (Temecula Segment)	30	6.8
Fault	Distance from Miramar Alignments (miles)	Moment Magnitude
Rose Canyon	0.71	7.2
Coronado Bank	13.2	7.6
Newport Inglewood	23.3	7.1
Elsinore (Julian Segment)	30	7.1
Elsinore (Temecula Segment)	30	6.8

4.8 Site Investigations

As part of the Mid-Coast Corridor Transit Project, Kleinfelder drilled several bore holes in the Miramar Hill area, close to the proposed alignments. The borings ranged in depth from 4 feet to 151 feet in depth, most of these were advanced to bedrock (Parsons and Kleinfelder 2018). Additional borings have been also conducted in the project area as part of other studies; these include:

- Caltrans, 1966
- Geocon, 1986, 1989a, 1989b, 1998 and 2006
- Geotek Insite, 1999
- Ninyo and Moore, 1999
- Allied Geotechnical Engineers, 2006

As part of the current conceptual design work, additional site investigations have also been undertaken, and reports have been prepared by both EMI and Leighton, and are provided as part of the Del Mar Alternatives Analysis Report and Miramar Alternatives Analysis Report.

Additional site investigations should be undertaken to supplement this existing information.

5 Geotechnical Risks

Refer to Del Mar Alternatives Analysis Report and Miramar Alternatives Analysis Report.

6 Tunnel Design

6.1 Configurations and Space Proofing

Twin-bore, single-bore, and triple-bore tunnel cross sections are being considered in this study. Cross sections of each of the following configurations are presented in Appendix D. For the twin bore tunnel option, a tunnel with an internal diameter of 33 feet has been selected. For the single bore option, an internal tunnel diameter of 57 feet has been selected. The tunnel diameters are based on the Southern California Regional Rail Authority (SCRRA) standard clearance of structures and incorporates a 2-ft 6-in. walkway in accordance with National Fire Protection Association (NFPA) 130. A second twin bore configuration will be examined at a later stage using the smaller California Public Utilities Commission (CPUC) clearance envelope. The revised diameter of this tunnel will be further explored at a later stage. The current assumption for the study is to accommodate the SCRRA standard clearance of structures. Further optimization of the minimum tunnel diameter should be performed at a later design stage. It should be noted that a 57-ft internal diameter tunnel constructed using a tunnel boring machine (TBM) has yet to be built. However, the possibility of using a TBM to construct a 57-ft tunnel is discussed in more detail in Section 12. Due to the alignment constraints and limited cover at the portals it was deemed that a single bore would be difficult to construct. In subsequent phases of the design, measures such as jet grouted blocks, concrete pads at the launch site, slurry walls will be studied to prevent excessive ground deformation due to tunneling. It should be noted that these mitigation measures need to be assessed in relation to their proximity to residential properties which are typically adjacent to the portal locations. Any single bore construction would require deeper alignments than are currently shown in the 10% Design Drawings. The assumptions made in this discussion between twin-bore and single-bore options will be re-investigated in the next round of study, with the assumption that the considered vehicle envelope will change.

A triple bore option is also considered in this study consisting of the same 33 foot internal diameter running tunnels as the twin bore option with a center bore between the two with an internal diameter of 20 feet. The center bore would be used for egress and maintenance access. Due to the increased costs, increased easement areas, and marginal benefits in terms of maintenance access this option was deemed less desirable than the twin bore option.

In addition to the rail tunnels, smaller cross-passage tunnels will need to be constructed between the twin bore tunnels to provide emergency access between the two tunnels as shown in Figure 6.1. These cross passages can also be used to accommodate communication, mechanical, electrical, and drainage equipment. Cross passage tunnels are discussed in more detail in Section 8.5.4, Tunnel Cross-Passages and Exits. Conceptual cross passage cross sections are included in the 10% Design Drawings.

Figure 6.1. View of a Cross-Passage Access from a Tunnel

6.2 Excavation Methods and Support

The depth, size, length, and ground conditions along the proposed tunnel alignments will dictate what type of tunnel excavation methods can be used.

For both the Del Mar Tunnel and Miramar Tunnel options, the tunnels are expected to be constructed in weak rock or soils. Where the ground conditions consist of weak rock or soil, or both, two excavation method are typically used: TBM and Sequential Excavation Method (SEM). Use of a partial face excavation machine (PFEM) might also be considered if the tunnel alignment is in homogenous ground above the water table.

For road and rail tunnel projects where large-diameter tunnels (in excess of 20 feet) are required, TBMs are typically used, especially when the tunnels are over a mile in length. If the tunnels are relatively short, then using a TBM is less economical, given the high procurement costs, which can be in excess of $40 million.

For shorter tunnels, SEM may be more economical and is commonly used. SEM is also used for tunnels with irregular cross sections.

Given the poor quality of the rock along the alignment, it is unlikely that drill and blast excavation techniques will be required.

6.3 TBM Tunnels

Several types of TBMs can be used to construct a bored tunnel in soft ground; these include Earth Pressure Balance Machines (EPBs), Slurry TBMs, Variable Density TBMs and Mixed Shield Machines.

Previous studies for the corridor (HNTB 2017 and Parsons 2018) established that the preferred machine types are a pressurized face TBMs.

HNTB concluded that a pressurized face TBM would be required for the Del Mar Tunnels, which would likely be excavated above the static ground water table, primarily in the Delmar Formation claystone interbedded with, clayey sandstone and coarse-grained sandstone. HNTB recommended that the TBM be equipped with a short, tapered shield to provide ground support through soft ground with expansive claystone.

Parsons determined that EPBs were appropriate for excavating the Miramar Tunnels which would be constructed primarily in Ardath Shale consisting of claystone interbedded with sandy siltstone and silty sandstone.

Determination of the optimum type of tunneling method will be performed at later stages of design based on site specific geotechnical information.

6.3.1 Earth Pressure Balance TBMs

EPBs can apply a pressure to the excavation in front of the cutterhead of the machine. This helps to maintain the stability of the tunnel excavation and reduce ground movements around the tunnel. There are two modes of operation for EPB TBMs: closed and open. In closed mode, the excavated spoil in the excavation chamber is pressurized. In open mode, the spoil in the chamber is not pressurized. The open mode can be used where active support of the face is not required, such as with hard rock and dry competent soils.

EPB TBMs regulate the chamber pressure by controlling the rate of removal of spoil from the excavation chamber. Pressurization of spoil in the excavation chamber is maintained during excavation by synchronizing the TBM advance rate with the excavated spoil removal rate. This is achieved by controlling the rate of spoil entering the excavation chamber, which is a function of the cutterhead rotation speed and TBM thrust, and the rate of spoil exiting the chamber, which in turn is a function of the screw conveyor rotation speed and length. The process of regulating the chamber pressure is assisted greatly by the injection and mixing of soil conditioners with the spoil. The screw conveyor then discharges the muck or spoil onto either a conveyor or into muck skips behind the TBM for transportation to the surface

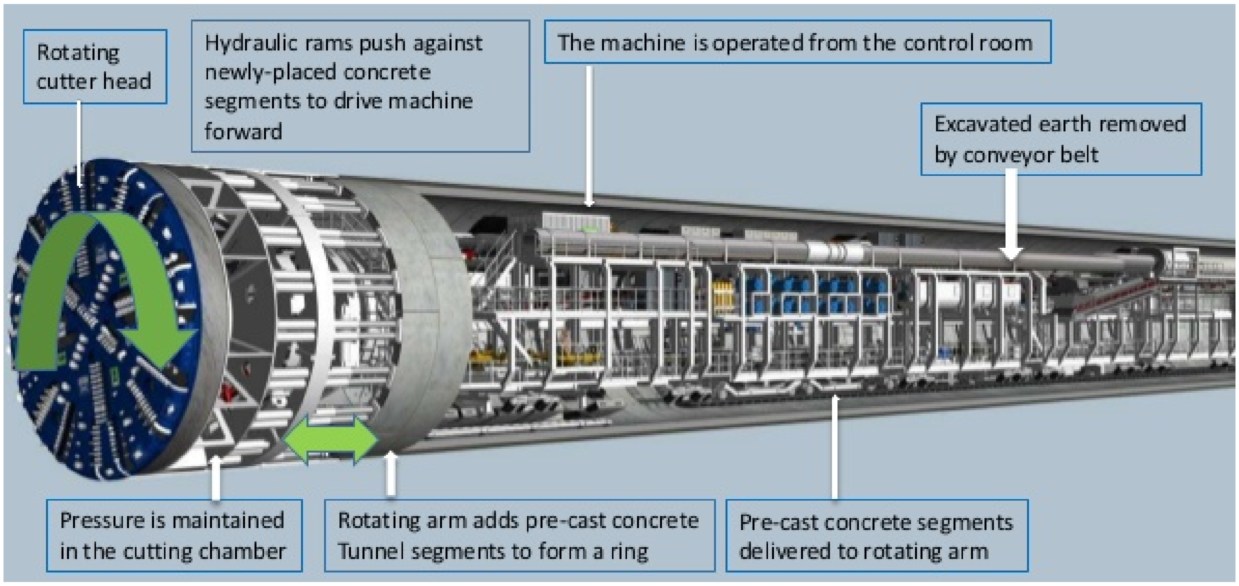

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Figure 6.2. General Arrangement of EPB TBM

Figure 6.3. EPB TBM (left TBM, right TBM with full backup)

6.3.2 Slurry Tunnel Boring Machines

Slurry tunnelling machines (STMs) balance external pressures by mixing the excavated material in the excavation chamber with a bentonite suspension (or slurry) and pressurizing the contents of the chamber to create a support medium for the tunnel face. The slurry mixed with excavated soil is then pumped through a pipe system to a separation plant, where the excavated material is separated out of the slurry, and the slurry is recirculated through the slurry circuit. The contents of the excavation chamber are pressurized by regulating the pressure in the slurry circuit and, in some cases, by means of an air bubble in a special chamber behind the excavation chamber. STMs perform best in clean sands and gravels because separating fines from slurry is a complex, costly and time-consuming method. A fines content exceeding 20% to 30% creates difficulty for the slurry separation system.

Although the tunnel alignments pass through the Scripps Formation (composed primarily of marine sandstone) and weak marine sedimentary rock which underlay much of the coastal area of Western San Diego County, some consideration to fines content and soils testing must be done prior to selecting STMs. As described in sections 4.4.1 and 4.4.2, lenses of claystone and siltstone can be found throughout the region as well as artificial fills containing clays and silts among other possibly clayey and silty deposits.

6.3.3 Bored Tunnel Linings

The TBM would install a single pass lining system consisting of concrete segmental rings. With TBM tunnels, the final lining is installed behind the TBM as it moves forward, and therefore temporary ground support is not needed. The tunnel lining segments provide both temporary and final support.

The tunnel lining is built inside the shield at the back of the TBM. The tunnel lining segments are bolted together to form a complete ring. The TBM pushes off the completed ring to move forward. It is possible to install multiple rings per day, allowing production rates of 50 feet or more per day. Figure 6.4 shows the lining being installed behind the TBM.

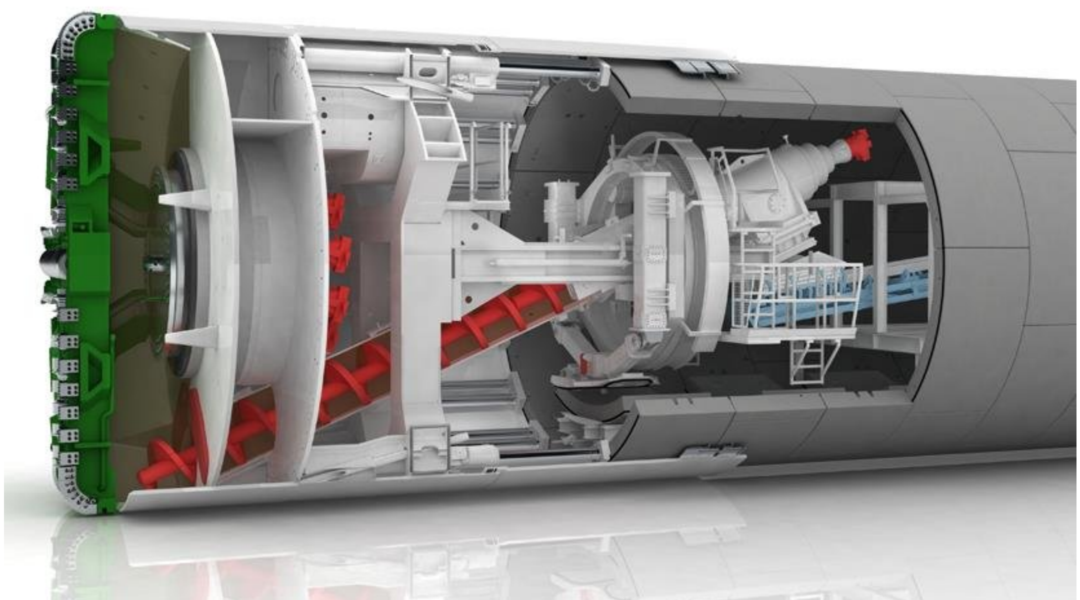

Figure 6.4. Tunnel Segment Installed behind Tunnel Boring Machine

The precast tunnel lining will need to consider the following elements:

- Ground and hydrostatic loads including those developed by expansive claystone.
- Lining geometry (including segment shape, thickness, ring length, taper, and joint geometry)
- Combination of lining reinforcement (conventional reinforcing bar and/or steel/synthetic fibers;
- The alignments do not cross any known active faults. If site investigations and fault studies indicate that any of the potential active faults are redefined as active, then linings for active faults will

consider either post-tensioning, steel linings, or concrete steel hybrid linings. Inactive faults generally do not require special lining.

- Segment connectors
- Segment lifting methods
- Lining grouting provisions
- Types of lining grout
- Joint packer types
- Segment gaskets
- Foam strips
- Provisions for cross passage opening if twin bored are selected

A graphic of an erected precast tunnel lining is shown in Figure 6.5.

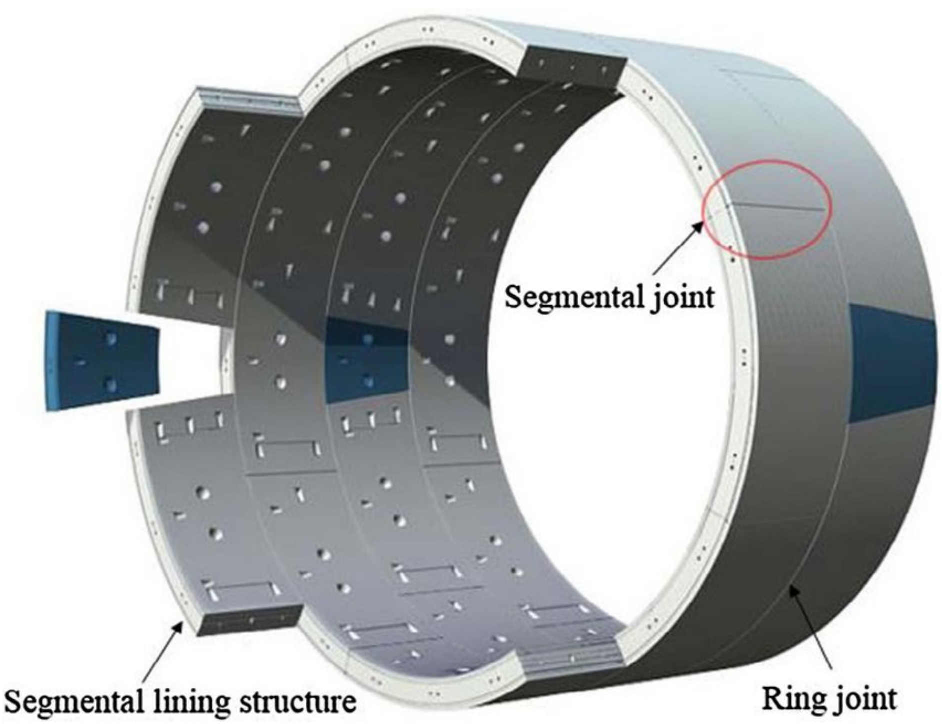

Figure 6.5. Typical Precast Tunnel Lining Tunnel

6.4 SEM Tunnels

Sequential Excavation Method (SEM) is a viable method of construction in soils that are above the static groundwater table or where groundwater can be managed through dewatering, depressurization or ground treatment.

SEM involves subdividing the full-face cross section of the tunnel into multiple “pockets” or headings of excavation. Each heading is excavated either by hand or using excavation equipment. A temporary layer of shotcrete is then applied to support the ground. An example of this method is shown in Figure 6.6.

SEM uses the inherent geological strength in the surrounding soil/rock mass to stabilize the tunnel. SEM integrates the principles of rock mass behavior under load and requires monitoring performance during construction. SEM provides a toolbox of ground support elements that are used as needed as the excavation progresses, providing optimized ground support in response to observed ground conditions and behavior. SEM construction may be more cost-effective than TBM for tunnels of less than one mile long. SEM is also more appropriate where conditions are not suitable for the use of a full-face TBM, such as irregular and complex underground geometry that include chambers, cross passages, bifurcations, crossovers, and stations.

Ground support for the SEM tunnels could be completed with a combination of the following materials:

- Grouted pipe canopy over the arch of the tunnel
- Grouted spiles
- Reinforced shotcrete
- Lattice girders

Figure 6.6. SEM Tunnel Showing Pocket Excavation, Regional Connector, Los Angeles

Figure 6.7 shows a typical section for a station-sized opening of a large SEM tunnel. Fewer openings would be required for smaller tunnels, such as cross passages, in good ground. Figure 6.8 shows a typical SEM tunnel construction sequence and support.

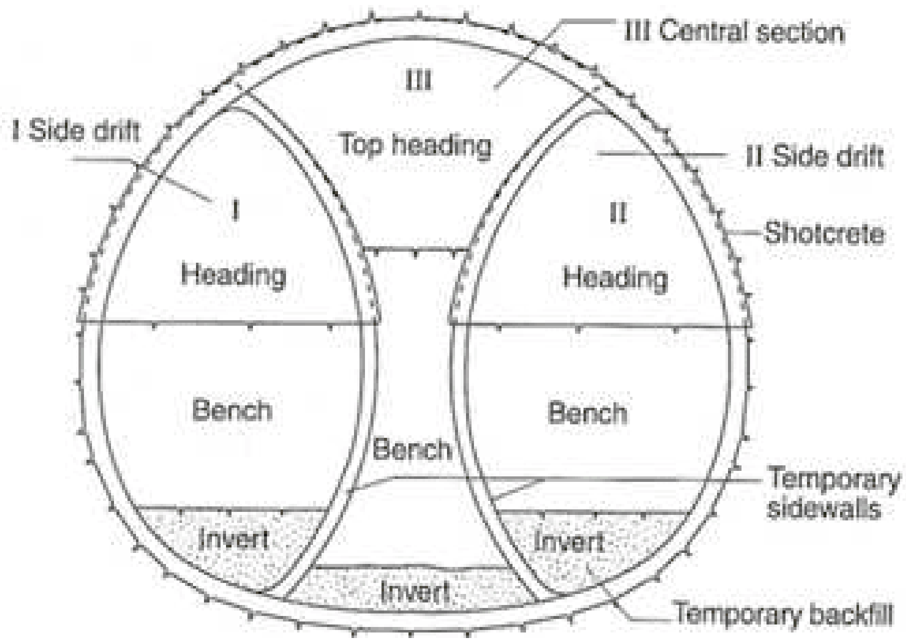

Figure 6.7. Typical SEM Tunnel Construction Sequence and Support

Figure 6.8. Typical SEM Tunnel Construction Sequence and Support

Once the initial lining has been constructed, a waterproof barrier is installed. Refer to Section 6.7 for more information. After the initial lining and waterproofing is installed, a secondary permanent concrete lining is installed. This lining will be designed to accommodate long-term hydrostatic and earth pressure loads. The permanent lining could be reinforced with either conventional reinforcement or steel fibers.

SEM techniques will be used to construct the smaller cross-passage tunnels between the main rail tunnels. SEM techniques are also being proposed for the station and associated tunnels; refer to Section 1, .

6.4.1 SEM Linings

The SEM permanent lining will be a conventionally reinforced concrete lining. At this stage, it is assumed that these linings will be constructed using cast-in-place techniques installed over the initial sprayed concrete linings.

6.5 Tunnel Design Life

The design life of the tunnel linings will be 100 years, in accordance with industry practice. Structural material must be able to resist any foreseeable loading conditions, vibration, and exposure to chemical compositions in the soil and water during the lifetime of the structure. The concrete cover shall be designed so the tunnel lining’s durability against chloride exposure and the temperature cycle will match the design life.

The design life of an element within a structure is defined as the time it takes for the extent of deterioration to exceed a given level of acceptability. Usually, “unacceptable deterioration” is defined as a level of deterioration that affects the intended functionality, appearance, or capacity of the element and signifies when repairs are required. Exceedance of the design life does not mean that the structure needs to be replaced, but rather that a period of enhanced maintenance may be required to restore the structure to its original level of functionality.

As the design progresses, durability plans shall be produced for each major structure. The durability plans shall, at a minimum, cover the following items.

- A summary of the design criteria and the service life requirements for each of the elements and components within the structure
- Identification of the corresponding environmental exposure conditions for each structural component
- Applicable degradation mechanisms for the structural elements and materials under consideration
- A description of the service life prediction models used in the analysis
- The predicted service life for each structure based on the proposed materials, exposure conditions, relevant degradation mechanisms and anticipated or proposed protective measures
- An inspection and maintenance plan for components of each structure that may be affected by corrosion or other long-term deterioration mechanisms

6.5.1 Strategies to Extend Design Life

The following strategies to extend the design life of the tunnels shall be explored during subsequent design phases.

- Oversizing of the bored tunnel to allow for a non-structural lining to be installed in case of gasket deterioration in the precast segmentally lined tunnel.

- Use of steel fibers and macrosynethic fibers instead of or in combination with reinforcement bars in the tunnel linings. Structural demands on the lining may dictate that reinforcement bars will be required but use of fibers, which offer superior corrosion performance over reinforcement bars, will also be investigated. Other considerations for enhancing lining durability include design concrete mix for reduced permeability and increasing concrete cover over reinforcement bars.

6.6 Fire Performance Requirements

The tunnel linings, precast or cast-in-place, shall be designed to not collapse for the duration of a fire under any event that involves the burning of the passenger rail and/or freight cars.

Tunnel structural integrity shall not be compromised during a fire event. Explosive spalling risk may be mitigated by inclusion of micro polypropylene fibers in the concrete and/or shotcrete mix, or by continuous fire-resistant interior lining.

Design criteria for tunnel lining structural fire resistance shall incorporate:

- Protection of structural steel and rebar reinforcement with appropriate minimum cover
- Protection of structural steel from direct fire exposure based on fire intensity and appropriate fire duration
- Selection of materials, accessories and gaskets in the lining that do not compromise fire life safety
- Design of lining so that a fire event does not cause a failure, lining collapse, or unstable condition, ensuring compliance to ASTM E119 time temperature curve as part of the fire resistance test.

6.7 Groundwater Inflows & Waterproofing

The tunnel linings shall be designed so that the ability of the system to prevent water ingress does not significantly deteriorate during the design life of the structure. In addition, visible water ingress at the station or at the interfaces between the running tunnels and other underground structures will not be acceptable.

One of the common industry standards for acceptable levels of water ingress in underground structures was devised by Dr. A. Haack in 1991. Haack developed five classes of acceptable water ingress for various types of underground facilities. The definitions of the five water tightness classes are shown in Table 6.1. Underground Water Tightness Criteria.

For the running tunnels, a Class 3 water tightness shall be achieved, and for the stations, a Class 2 water tightness shall be achieved. Stations typically require a higher level of water tightness because of the nature of sensitive equipment (mechanical and electrical) that are at these locations.

Table 6.1. Underground Water Tightness Criteria

Tightness Class	Moisture Characteristics	Intended Use	Definition
1	Completely dry	Storerooms and workrooms, restrooms	The wall of the lining must be so tight that no moisture patches are detectable on the inside.
2	Substantially dry	Frost-endangered sections of traffic tunnels; station	The wall of the lining must be so tight that only slight, isolated patches of moisture can

Tightness Class	Moisture Characteristics	Intended Use	Definition
		tunnels, cross passages	be detected on the inside (e.g. as a result of discoloration). After touching such slightly moist patches with a dry hand, no traces of water should be detectable on it. If a piece of blotting paper or newspaper is placed upon a patch, it must on no account become discolored as result of absorbing moisture.
3	Capillary wetting	Route sections of traffic tunnels for which Tightness Class 2 is not required	The wall of the lining must be so tight that only isolated, locally restricted patches of moisture occur. Restricted patches of moisture reveal that the wall is wet, leading to a discoloration of a piece of blotting paper or newspaper if placed upon it – but no trickling water is evident.
4	Weak trickling water	Utility tunnels	Trickling water permitted at isolated spots and locally.
5	Trickling water	Sewer tunnels	

. If limited groundwater inflows are expected, then a single gasket system could be used. The decision for double or single gaskets will be made in consultation with the client and once the actual ground water levels are determined.

Groundwater inflow to tunnels can severely affect design life and maintenance costs. The design of the tunnel lining should prevent water movement across internal surfaces from affecting the safety, durability, structural integrity, and function of facilities and systems. The full perimeter of SEM-mined tunnels should be protected with a sheet waterproofing membrane system installed between the initial and final SEM lining. Waterproofing membranes shall be installed in accordance with the manufacturer's specifications, classified as self-extinguishing, compatible with waterstops used, and certified that no components will leach out over the structure's design life and deleteriously affect the durability of any component. Membranes shall be protected from damage at all times. A protective layer can be provided over the membrane after proof testing to prevent rupture or damage during final lining construction.

For an SEM tunnel, a PVC (polyvinyl chloride) or HDPE (high density polyethylene) waterproof lining will be installed after installation of the initial lining. This waterproof layer will be installed in sheets and thermo welded together to provide a watertight seal. Waterbarrier (blue strips) in Figure 6.9 are installed along the length of the tunnel to compartmentalize the waterproofing system. Secondary grout ports and regroutable grout hoses are also installed to provide a secondary line of defense against water ingress. Another option for waterproofing could be the use of a sprayed waterproof membrane. All three options will be considered during subsequent phases of the design. The components of a typical SEM waterproofing system are shown in Figure 6.10

Figure 6.9. Installed PVC Waterproofing

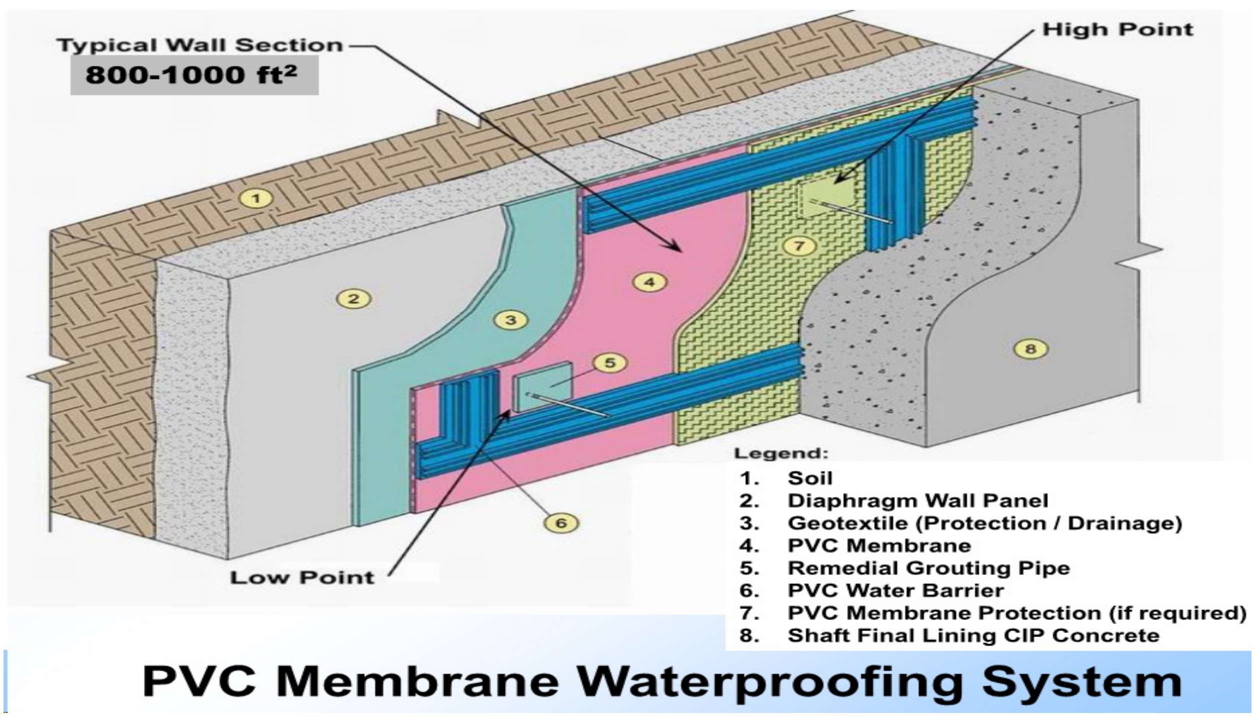

Figure 6.10. PVC Membrane Waterproofing Components

PVC and HDPE have different mechanical and physical properties as well as field applicability. However, both have been widely used in many tunnel waterproofing applications. The following are some of the main differences between the two materials:

- PVC geomembranes are flexible and relatively easy to handle, while HDPE geomembranes are tough and not very flexible, making them difficult to install in small spaces and where there is an irregular geometry, i.e., in tunnel cross passages.
- HDPE geomembranes tend to exhibit a sharp peak in their stress-strain curve, and therefore tend to undergo relatively abrupt failure whereas PVC undergoes a very large amount of elongation before failure.
- PVC and HDPE perform differently when exposed to gases or hydrocarbons, so this needs to be considered.

Alternatives to waterproofing membranes could include the use of either:

- spray on membranes, that are applied directly to the initial shotcrete
- watertight concrete for the final lining
- a drainage system installed in between the initial shotcrete lining and final concrete lining

These are less commonly used but can have schedule benefits given that they can be installed in less time than a traditional membrane. These alternatives can be considered during the development of the final lining design.

6.8 Loads and Load Combinations

Although this Basis of Design is at the alternatives analysis stage of project development, it is worth discussing some of the loads and load combinations that will need to be assessed throughout design development:

- Dead loads—includes in situ stresses from ground and self-weight of construction materials as ground stress redistribution.
- Water pressure and buoyancy—water pressure on the tunnel linings. Lower cover with high water tables may also make the tunnels susceptible to buoyant forces.
- Grout pressure—due to grouting of the annular space around the precast tunnel lining and proof grouting behind the tail shield.
- Live loads on the lining, such as train loads or maintenance.
- Longitudinal, Derailment and Centrifugal forces
- Loads on the tunnel lining from ventilation fans and other attachments
- Ground surcharge, such as bearing pressure due to buildings or construction surcharge loads.
- Seismic loads—loading due to earthquake motion. The Maximum Design Earthquake (MDE), Operating Design Earthquake (ODE) parameters and associated Return Periods will be developed in a subsequent version of the BODR.
- Temperature load due to a fire event inside the tunnel.

7 Tunnel Portal Design

7.1 Design Criteria

Design criteria for tunnel portals are as follows. For other tunnel design constraints, refer to Section 3:

- Preference is to locate portals/alignments along public or existing owner held Right-of-Way (ROW)
- Minimize visual impacts and blend in with the natural surroundings and environment
- Engage architects to improve portal visual impact and aesthetics
- Limit disturbance to existing conditions including groundwater and surface water drainage
- Design to mitigate risks to public safety during construction and operation
- Provide stable permanent cut and fill slopes with erosion protection above portals
- Maintain slope stability and prevent water from flowing into the tunnel throughout construction
- Prevent tunnel flooding from existing surface water runoff, drainage courses and streams
- Provide drainage measures to prevent surface water ponding, slope instability and flooding
- Direct drainage away from the tunnel entrance into appropriate drainage facilities
- Mitigate raveling, landslip and rockfall including during a seismic event
- Provide landslip/rockfall containment below slopes such as catch ditches or rockfall fences
- Provide sufficient space for maintenance access to slope toes for rockfall cleanout
- Provide high fences or other control measures to prevent public access to track.
- Portals, where possible, should be located in areas with rapidly rising topography so that tunnel cover can be quickly established thus mitigating the need for construction specifically designed to reduce the risk of tunnel settlement due to low cover
- Design portals considering topography, geotechnical conditions, method of tunnel construction, geologic hazards, slope stability, seismic hazards, space for construction and operation, right-of-way, easements, access roads, environmental constraints and local regulatory requirements.

7.2 Configurations and Space Proofing

For the tunnel portals, and in order to provide realistic space proofing for tunnel systems including MEP and FLS related rooms, a minimum of width of 185 feet will be used during the alternatives analysis to allow for construction of the tunnel portals. A minimum width of 135 feet will be used for the final portal structure. These widths have been generated from the twin bore tunnel configuration and are comprised of the elements presented in Table 7.1.

Table 7.1. Width Required for Tunnel Portals for Alternatives Analysis

Type	Size
Twin bore tunnels	2 x 35 ft = 70 ft
Separation between the tunnels	1 x 35 ft = 35 ft

Temporary support of excavation	2 x 10 ft = 20 ft
Permanent structure	2 x 5 ft = 10 ft
Allowance for construction equipment	2 x 25 ft = 50 ft
Total	185 ft

8 Fire Life Safety

8.1 Strategy

Fire protection strategies and designs focus on prevention, retardation of fire growth, detection, suppression, containment, and evacuation, in order of precedence.

Fire life safety (FLS) provisions for the Miramar and Del Mar tunnels use as the primary reference the National Fire Protection Association's (NFPA) 130 *Standard for Fixed Guideway Transit and Passenger Rail Systems,* with provisions added for the specific characteristics of passenger and freight train operations.

The Office of the State Fire Marshal is most likely the FLS Authority Having Jurisdiction (AHJ), as is the case with the California High Speed Rail Project (California High-Speed Rail Authority 2012), although the AHJ has not yet been established. During detailed design a FLS committee should be set up to get input from the AHJ and other stakeholders regarding key FLS issues.

8.2 Implications of Combined Freight and Passenger Use

Freight and passenger rail operators need to be trained on procedures for tunnel fire and life safety emergencies. The design of the station and guideway ventilation and fire protection system will address the hazards associated with both freight and passenger rail use. The freight train arrangements will be reviewed in the design process to ensure that a safe egress path is provided for freight train personnel.

8.3 Fire Scenarios

The tunnel ventilation system must be capable of providing a tenable environment for patrons along a safe egress route during the time required for emergency evacuation from the tunnel. Therefore, the ventilation rate in the tunnel must be sufficient to achieve the required critical velocity for a design fire size of:

- Passenger Trains - Modern fire hardened rail cars: 8 - 15 Megawatts.
- Freight Trains - Not an emergency ventilation design criterion because only personnel trained in emergency operational procedures will be allowed in the tunnel. Risk of fire and risk of operator exposure are reduced.

The fire scenario must be specified based on train data. This model fire has implications for design criteria such as structural, fire suppression, ventilation, fire detection, lighting, exits, and communications.

8.4 Rail Tunnel Hazards

The main FLS incidents that could disrupt normal operations of the LOSSAN Corridor tunnels are:

- Collision
- Derailment
- Fire
- Release of hazardous materials

In terms of the consequences, three types of rail incidents—cold incidents, hot incidents, and prolonged stop— are discussed in the following sections.

8.4.1 Cold Incidents

A collision or derailment is considered a “cold incident.” Relevant mitigation measures concentrate on access to and egress from facilities to support evacuation and the intervention of rescue forces. Cold incidents will not be addressed in this report but must be handled separately in a hazard analysis.

8.4.2 Hot Incidents

A fire or explosion is considered a “hot incident.” Fire could start in a passenger train and could become fully developed within 15 to 20 minutes after ignition. Thus, in contrast to the cold incident, the hot scenario involves a time constraint for evacuation, due to the presence of a hostile environment created by fire. Whenever possible, the train will leave the tunnel or stop in the station. Any freight vehicle identified as having an onboard fire incident will continue out of the portal. Figure 8.1 describes the risks that can cause cold and hot incidents in mainline railway systems.

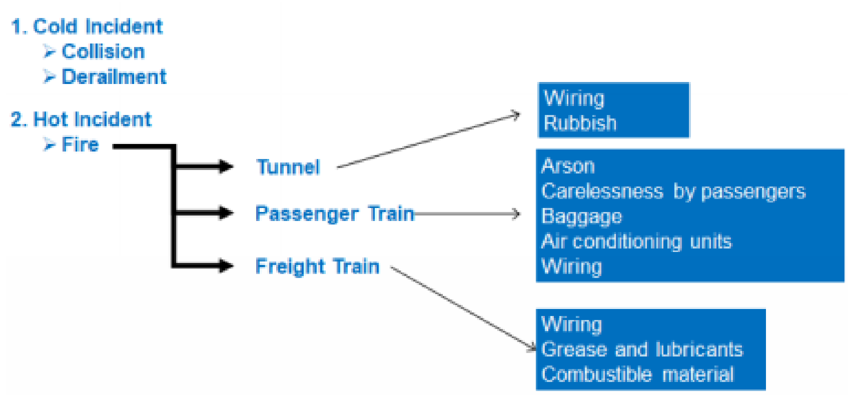

Figure 8.1. Main Incident Risks in Mainline Railway Systems

8.4.3 Prolonged Stop

A prolonged stop is an unplanned stop in a tunnel. There is no fire on board the train, and dwell time exceeds 10 to 15 minutes. A prolonged stop is not, by itself, a threat to passengers and staff. However, it may lead to panic and to spontaneous, uncontrolled tunnel evacuation that exposes people to dangers present in a tunnel environment.

8.5 Tunnel Fire Life Safety

8.5.1 Occupant Protection

Fire life safety provisions are designed to protect occupants who are not close to the initial fire development and to allow enough time for occupants to reach a Point of Safety as defined by NFPA 130 Section 3.3.40 " A point of safety is one of the following: (1) an enclosed exit that leads to a public way or safe location outside the station, trainway, or vehicle; (2) an at-grade point beyond the vehicle, enclosing station, or trainway; (3) any other approved location." (a location where patrons are no longer in danger and away from the fire).

The following measures are considered:

- Prevention
- Mitigation of the impact of accidents (hot or cold incidents)
- Facilitation of self-rescue
- Facilitation of intervention by First Responders Tunnel

Typical components of a tunnel FLS system are:

- Tunnel ventilation system
- Fire suppression systems, including sprinklers, standpipes, and deluge and gaseous suppression (see Figure 8.2)
- Automatic fire detection and alarm system
- Means of escape, including exit routes, emergency walkways, travel distance, occupant numbers, exit widths, and fire protection of exit routes
- Fire resistance and fire hazard properties of materials
- Firefighting access and facilities
- Manual firefighting equipment
- Wayfinding (emergency lighting and signage)
- Integration with FLS elements of rail systems, rolling stock, and operations.

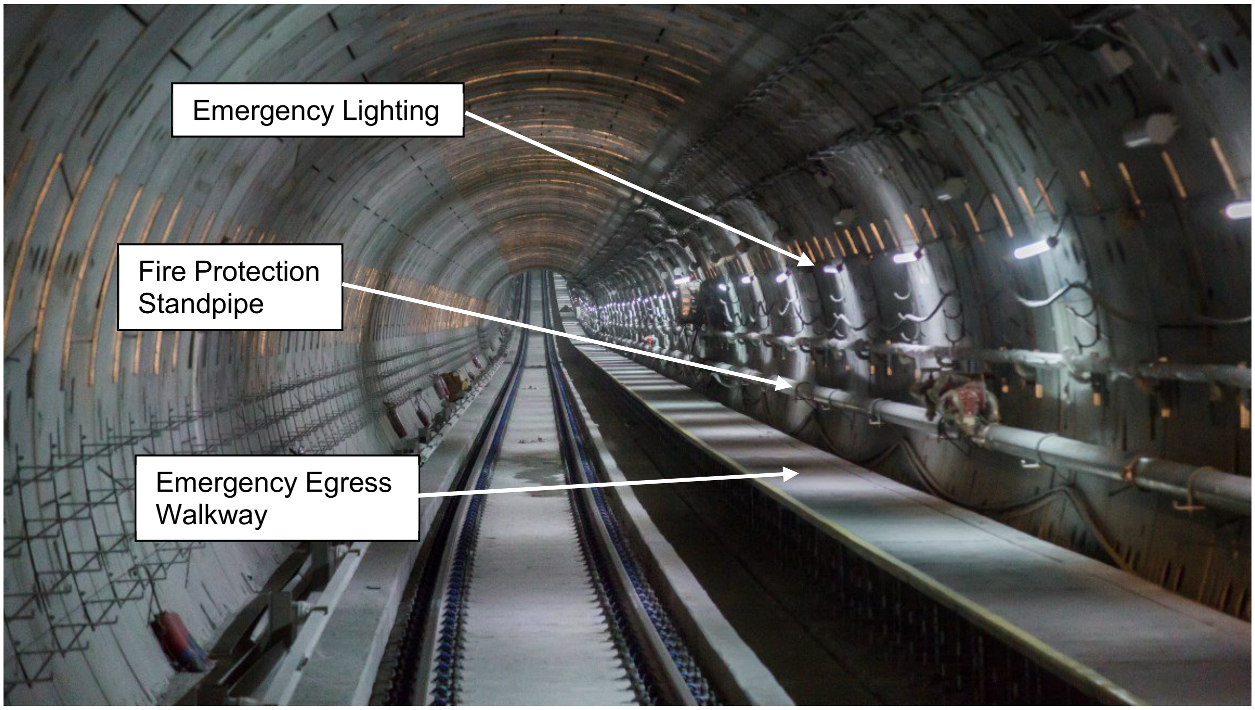

Figure 8.2. Bored Tunnel with FLS Elements Shown

Most of the smaller FLS components fit within tunnels in the spaces already created by other needs, such as clearances for dynamic envelopes, raised walkways, and traction power.

Mechanical ventilation systems for smoke control are installed in tunnels greater than 1,000 feet in length. Ventilation systems are designed to provide a tenable environment for a duration sufficient to allow passenger egress to a Point of Safety. Based on the tunnel parameters, mechanical ventilation will be required for the Del Mar and Miramar tunnels.

8.5.2 Tunnel Emergency Egress/Access

The evacuation of large numbers of people from tunnels is challenging, time-consuming, and difficult. In an emergency, such as a fire on a train, the policy is to avoid detraining passengers between stations wherever possible, and for the train to proceed to the station or out of the tunnel through the portal. Referred to as a “drive through” strategy, this is beneficial in a fire scenario because the station has a platform designed for detraining passengers and is in a location where firefighting and rescue activities can be easily conducted.

If it is not possible to move the train, a second train may be brought to the scene (if safely practicable) to facilitate the evacuation of passengers from the tunnel.

In the event that operation of a second train is not possible, passengers will be afforded a safe and efficient means of egress by self-evacuation from the trainway. Passengers will be guided to the nearest emergency egress point (e.g. a station or tunnel portal entrance at grade level), cross passage, or protected exit staircase. Passenger notification will be via the Public Address announcement, signage inside the train, and directional exit signage along the tunnels.

NFPA 130 provides requirements for access and egress for passenger rail systems, including criteria for the design of walkways, crosswalks, handrails, lighting, cross passageways, and other access/egress support infrastructure.

California Code of Regulations, Title 24, Part 9 California Fire Code 2010, Appendix D has requirements for the design of access roads to support operation of fire apparatus. Access roads designed to the California Fire Code will support the needs of emergency access/egress operations and routine operations and maintenance activities.

8.5.3 Tunnel Walkways

The tunnels will be equipped with a continuous walkway to allow passengers to evacuate a train to a station or Point of Safety. Walkways are located adjacent to each track and may be located between and shared by two adjacent tracks. Walkways have uniform, slip-resistant surface. The walkway envelope is 2 feet wide at the walkway surface, widening to 2.5 feet at 5.5 feet in height and tapering to 1.5 feet at 6.5 feet above the walkway surface and must be free of obstructions. Crosswalks, connecting the two sides of the trainway with a walking surface uniform to the top of rail, will be located at access/egress points, special trackwork (switches and crossovers), and cross passageways. Raised walkways are provided with a handrail at a height of 3 feet along the side opposite the trainway.

Walkways are located along the tunnel walls on the same side as the access/egress points or cross passageways where possible. Walkways are illuminated to provide safe passage in the event of an evacuation, in accordance with the requirements of NFPA 130.

8.5.4 Tunnel Cross-Passages and Exits

Egress from a twin-bore, single-track tunnel is provided by passage to a place of safety at 800-ft intervals as shown in Figure 8.3. Cross passageways are equipped with fire-rated doors with self-closing mechanisms at each opening. A tenable environment will be maintained in the portion of the trainway that is not involved in the emergency and is being used for evacuation.

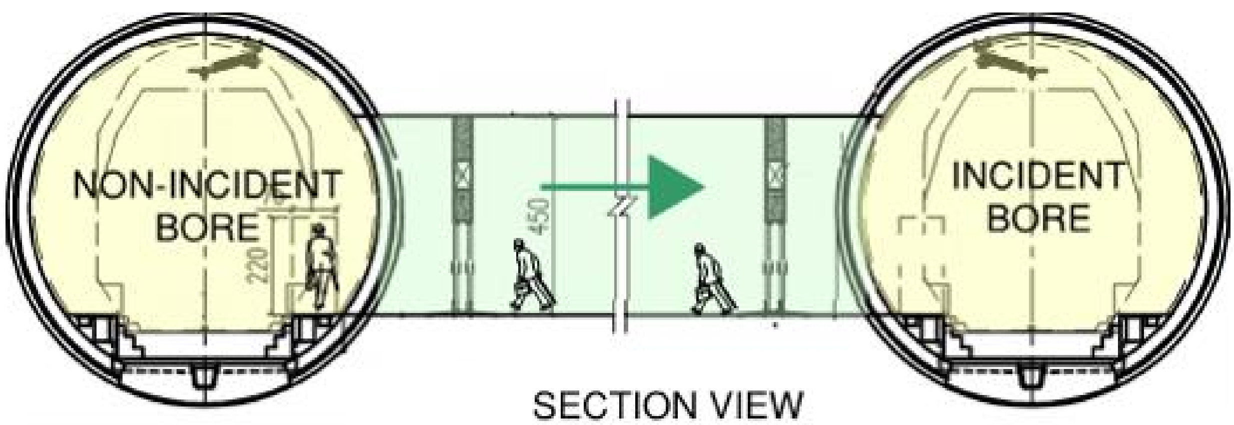

Figure 8.3. Twin Bore, Single Track Tunnel with Cross Passages. Green arrow represents ventilation airflow direction. (Not to Scale)

Egress from a single-bore, double-track tunnel equipped with center walls is provided by fire-rated sliding doors opening to the opposite track as shown in Appendix D. Doors are 1.5-hour fire-rated and equipped with self-closing mechanisms.

Egress from a single bore, two-track tunnel that is not equipped with a center wall is provided by lateral or vertical exits to the surface every 2,500 ft. Including a dividing wall is preferable from a safety perspective, to avoid oncoming train collisions and provide a non-incident tunnel as a point of safety for emergency egress.

8.6 Tunnel Portals

Tunnel portals are primary locations for emergency response operations during an incident and require FLS infrastructure. The tunnel portal is a destination of evacuees from the tunnel, emergency responders attempting to access the tunnel to implement FLS operations, and smoke discharge from the tunnel fire. All three incident response elements are considered when designing tunnel portal areas. Additionally, the portals will house the tunnel ventilation fans and mechanical rooms and power for FLS infrastructure.

FLS infrastructure at tunnel portals for passenger egress includes paved emergency egress walkways from the tunnels, designated crosswalks at top-of-rail height, area lighting, and a rescue area/passenger assembly area. Infrastructure for emergency response operations includes an access road, emergency vehicle assembly and turnaround area, fire hydrants and emergency water supply, an emergency power supply, cross track emergency vehicle access, emergency communication facilities for hard-wired and radio communication systems, and an incident command post. Figure 8.4 shows a tunnel portal emergency assembly area.

Figure 8.4. Tunnel Portal Showing Emergency Assembly Area, Trackway Crosswalks

8.7 Stations Fire Life Safety

NFPA 130 and the California Building Code provide guidance for access/egress at station facilities. Emergency responders must be provided access to all areas in and around the station in the event of an incident requiring emergency response. Fire lanes are to be designated in the vehicle roadways and parking areas surrounding the station.

Evacuation routes must be available to evacuate all passengers from the affected station in four minutes or less and from the most remote point on the platform to a Point of Safety in six minutes or less. At least two means of egress, remote from each other, are to be provided from each station platform. Additional protective measures, such as platform enclosures, will also be evaluated during station design.

An automatic sprinkler protection system is to be provided in areas of stations used by passengers, for concessions, storage areas, and trash rooms and in the steel truss areas of all escalators and other similar areas with combustible loadings, except trainways. Equipment rooms with electrical and electronic equipment, such as communications and ATC (automatic train control) equipment rooms, must have special fire protection provisions that are defined in discipline-specific technical memoranda. Fire protection standpipe and hose systems are to be installed in accordance with NFPA 14 and as modified by NFPA 130. Fire hose cabinets and portable fire extinguishers will be provided, as required by NFPA 130.

Backup electrical power will be supplied by a redundant power feeder or emergency standby generator. Emergency power is provided for select electrical loads including fire protection systems, ventilation systems, emergency lights and signage, communication systems, train control systems, and low-voltage direct-current battery supply systems to support emergency lighting and communications.

8.8 Tunnel Ventilation

8.8.1 Ventilation System Operating Modes

Demand for ventilation, natural or mechanical, is derived by the conditions that are encountered during the five operating modes. The operating modes for railway tunnels are:

- Normal Operations: Trains transport passengers through the system on routine operating schedule.
- Purging Operations: Removal of diesel emissions. The emergency ventilation system provided for passenger trains will also be used to purge the tunnels of diesel exhaust after the passage of a freight train.
- Congested Operations: Trains are stopped for more than a few minutes in the tunnels for non-routine, non-emergency reasons.
- Emergency Operations: A fire or incident occurs somewhere in the system and revenue train operations are stopped.
- Maintenance Operations: The tunnel ventilation system is used to dilute welding/cutting fumes, rail grinding, or diesel emissions produced by maintenance equipment, for maintenance workers to carry out their duties safely.

Typically, ventilation systems sized for emergency operations will satisfy the capacity requirements for normal, purging, congested, and maintenance operations

8.8.2 Emergency Operations

Mechanical ventilation systems are needed in the event of a fire or smoke incident to protect evacuating passengers and emergency responders from smoke. The ventilation systems are used to move the smoke in one direction in order to provide a tenable escape route for passengers and access by first responders.

The minimum ventilation requirements are specified in NFPA 130, which requires that the emergency ventilation system be designed to do the following:

- Provide a tenable environment along the path of egress from a fire incident in a station or tunnel
- Provide sufficient airflow rates to control smoke
- Provide ventilation system fans capable of satisfying the emergency ventilation requirements to move air in either direction as required to provide the needed ventilation response
- Accommodate the maximum number of trains that could be between ventilation shafts during an emergency
- Maintain the required airflow rates for a minimum of 1 hour but not less than the required time of tenability

- Operate fans to avoid smoke ingress to the non-incident tunnels/stations to avoid contamination.
- Operate emergency ventilation fans, motors, and all related components, exposed to exhaust airflow at the fan inlet hot temperature, based on the design fire load, for a minimum of 1 hour
- Keep the maximum air velocity in areas accessible to the public under 2,200 fpm

An engineering analysis will be needed to evaluate whether a proposed mechanical ventilation system has sufficient capacity to provide the minimum ventilation airflows required to control smoke during a tunnel fire. The ventilation system is part of the design.

8.8.3 Emergency Response

Emergency procedures are developed for each tunnel, specific to each emergency scenario. The procedures must require agency participation in emergency response and training to be performed for each of the emergency scenarios. The responding agencies may vary depending upon where the incident happens within the tunnel, different alignment options, and different station location options.

8.8.4 Rail Tunnel Ventilation Methods

Ventilation is provided in rail tunnels using the following methods:

- Natural ventilation
- Jet fans
- Nozzles
- Mid-tunnel ventilation shafts
- Tunnel length plenums: transverse, semi-transverse, single point extraction
- Combinations of the above

Typical ventilation systems for railroad tunnels include jet fans positioned at regular intervals within the tunnel or reversible fans located in ventilation buildings at portals or at remote ventilation structures connected to the train tunnels, as shown in Figure 8.5.

All the ventilation methods considered produce a longitudinal airflow through the tunnel. The magnitude of airflow is governed by providing sufficient airflow to overcome the effects of the fire, tunnel friction, and other aerodynamic influences.

Figure 8.5. Typical Tunnel Ventilation Fan with Duct Transitions and Attenuators

Natural Ventilation

Natural ventilation relies on air being moved through the tunnel without mechanical aid. Sources for natural ventilation power include differences in temperature and atmospheric pressure between the tunnel ends, wind, and movement of trains through the tunnel. Although the Miramar and Del Mar tunnels will require mechanical ventilation, the effects of natural ventilation are included in the design requirements for ventilation system performance calculations.

The piston-action of trains moving through confined tunnel spaces is something of a natural resource for tunnel ventilation under normal conditions. The benefit of the piston-generated airflow is the continual turnover of warm tunnel airflow in favor of relatively cool ambient airflow. The general reduction in the tunnel air temperature benefits both vehicle air-conditioning systems and station air-conditioning systems, in particular, and passenger comfort. The key parameter in determining the effectiveness of the piston-action is the blockage ratio or the vehicle cross sectional area divided by the tunnel cross sectional area. The greater the blockage area, the greater the piston-effect.

Jet Fans

The jet fan method includes a series of small axial fans installed tight against the tunnel roof. A system schematic for jet fans (indicated as blue rectangles) is shown in Figure 8.6. The green arrows represent fresh air, and the red arrows represent air that has passed over the fire.

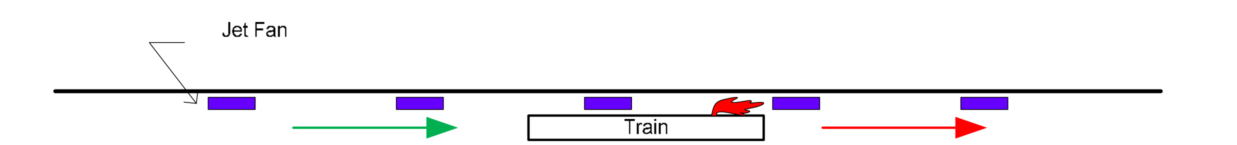

Figure 8.6.Jet Fan Schematic, Tunnel Section View

Examples of jet fans are shown in Figure 8.7. Jet fans direct smoke and hot gases toward one portal and induce outdoor air into the tunnel through the opposite portal. The resulting airflow provides a supply of fresh air in the path of passengers being evacuated toward the safety of a portal or cross passage. The fan motors are reversible, allowing the direction of airflow to coincide with the direction of train evacuation to keep passengers in fresh air. In addition to the jet fan, a structure is required to house electrical switchgear, control systems, and motor control centers.

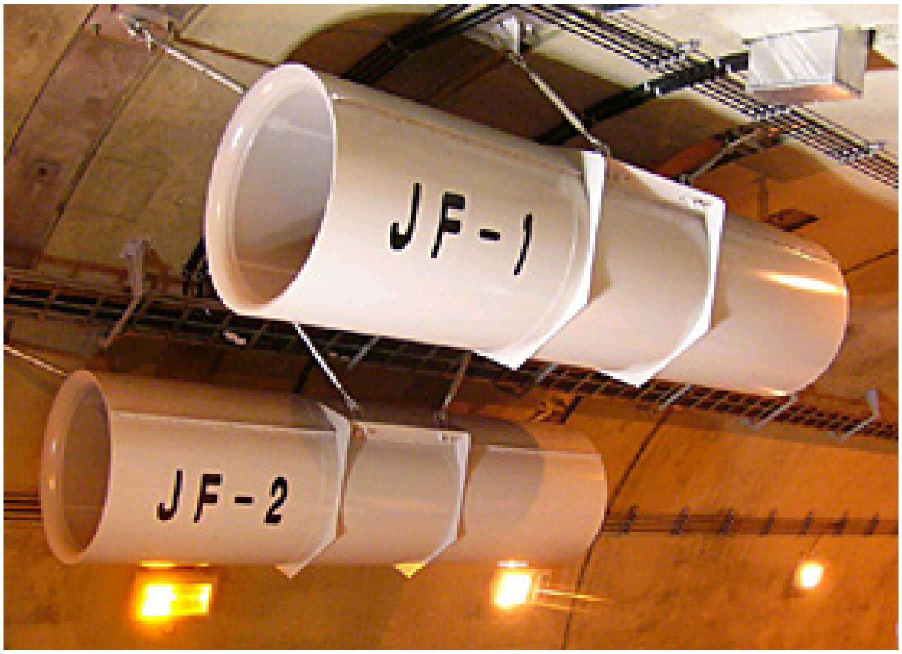

Figure 8.7. Example of Jet Fans

Maintainability is an issue with jet fans that hang in the tunnel as maintenance or repair requires closure of the tunnel.

Portal Fan Plants with Nozzles

The tunnel ventilation fans and nozzles are located in portal ventilation buildings at each end of the tunnel. Air is delivered to the tunnel through a nozzle in the ceiling or in the tunnel walls, as shown in Figure 8.8. Air is fed into the tunnel with sufficient force and velocity to generate a longitudinal airflow in the tunnel. The fan plant at the opposite end of the tunnel is used to pressurize the non-incident bore to prevent smoke from migrating into it by way of open cross-passage doors or the portals. Two nozzles are installed in each bore to provide the ability to move air in either direction, as shown in Figure 8.9. Given the tunnel arrangement and local geography, and based on previous design experience on heavy rail projects, this is the preferred alternative at this conceptual stage of the project.

Figure 8.8. Photo of Nozzle in Tunnel

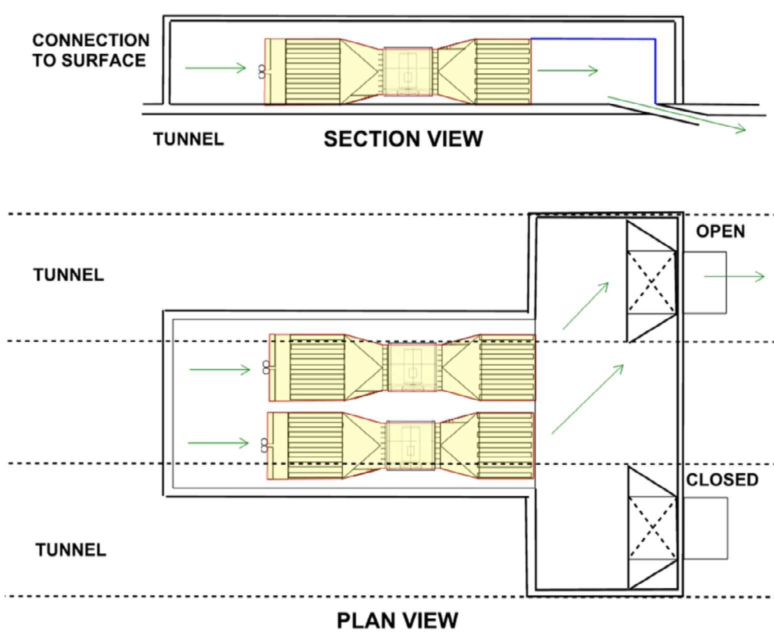

Figure 8.9. Sketch of Fan Plant and Nozzle

The nozzle injects a high-velocity air jet into the tunnel at the smallest possible angle to the longitudinal axis of the tunnel and induces airflow in the tunnel. The momentum exchange between the nozzle airflow and the slower moving tunnel air results in a static pressure rise across the nozzle. The effectiveness of the nozzle depends on the nozzle discharge velocity, the momentum exchange coefficient and the injection angle. Figure 8.10 shows the concept and flow situations of nozzles used in the development of a longitudinal ventilation system for the train tunnel.

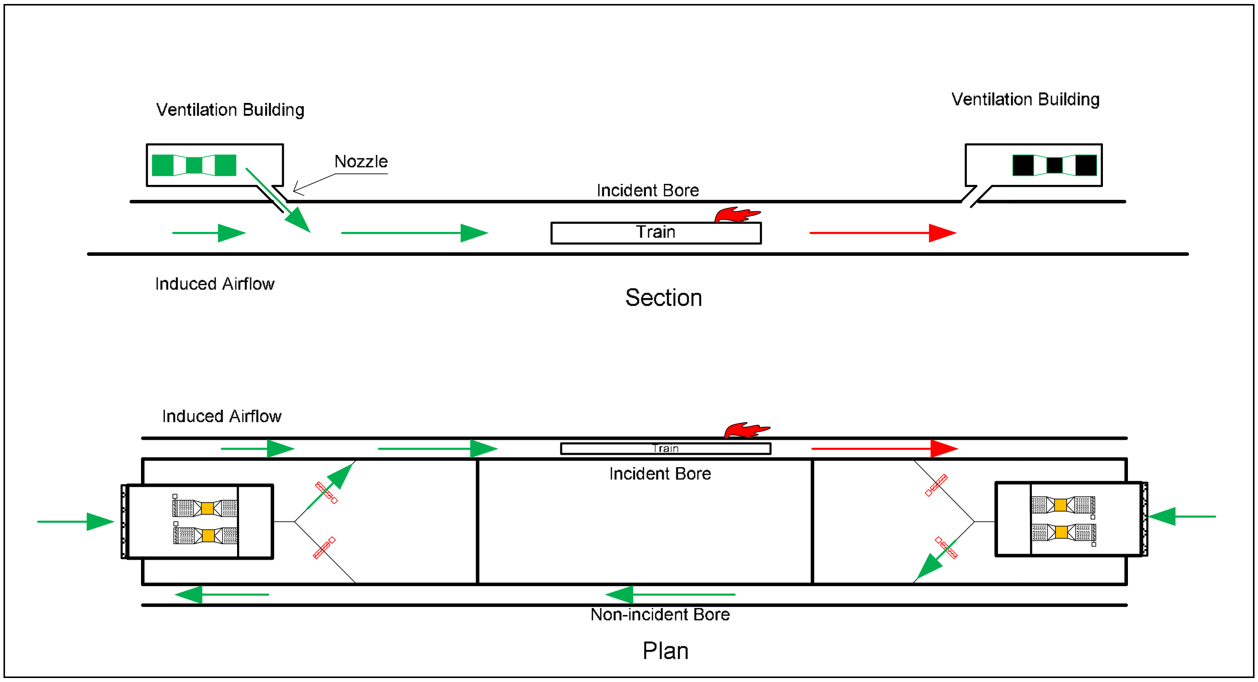

Figure 8.10. Nozzle Schematic

The portal fan plants will include vertically oriented fan discharge at ventilation facilities to facilitate dispersion of emissions from locomotives and avoid violation of air quality regulations during normal or purging operations.

Mid Tunnel Shaft Method

The mid-tunnel ventilation method requires the installation of fans in ventilation buildings atop ventilation shafts at or near the tunnel mid-point. The ventilation buildings are connected to the tunnels by ducts or shafts. A mid-tunnel vent shaft may be required to allow multiple trains in a tunnel bore at the same time.

During emergency operations, fans operate in supply or exhaust. Air is either supplied or exhausted from the tunnel depending on the direction of egress, as shown in Figure 8.11.

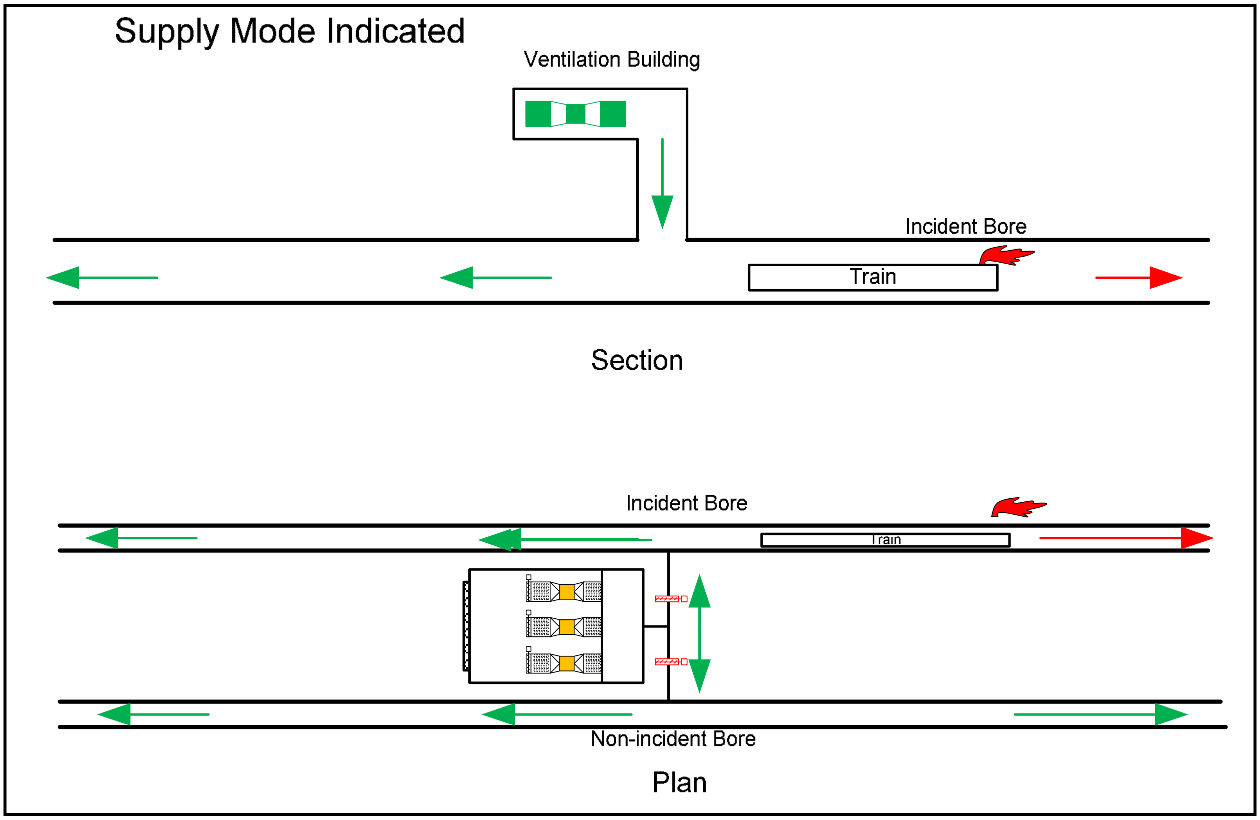

Figure 8.11. Mid-Tunnel Ventilation Shafts

Ventilation equipment includes louvers, sound attenuators, fan isolation dampers, track isolation dampers, ductwork, electrical switchgear, a control system, and motor control centers.

Mid-tunnel vent shafts would require additional land and access at the surface.

Tunnel Length Plenums

The following concepts use a plenum running the length of the tunnel to supply and/or exhaust air.

Transverse/Semi-Transverse

Transverse and semi-transverse methods of tunnel ventilation supply and/or exhaust air evenly along the tunnels' length using opening sizes calculated to balance air flow. A fully transverse system consists of two plenums along the tunnel. One supplies fresh air while the other exhausts air from the tunnel. A semi-transverse system only has one plenum. If the fans are running in supply mode the displaced tunnel air is pushed out of both portals, in exhaust mode fresh air comes in from the portals and is drawn into the plenum. These ventilation methods are commonly seen in highway tunnels and are often used to maintain air quality in normal operations.

Single Point Extraction

The single point extraction ventilation method is similar to the semi-transverse system in that it consists of a single tunnel-length plenum operating in exhaust mode; however, the openings are controlled by dampers, allowing the operator to open only one air path into the plenum. This allows the emergency ventilation system to provide critical velocity past the fire location and remove the smoke from the tunnel. This method is very flexible but makes for a much more complicated controls scenario. Figure 8.12 shows the single point extraction method.

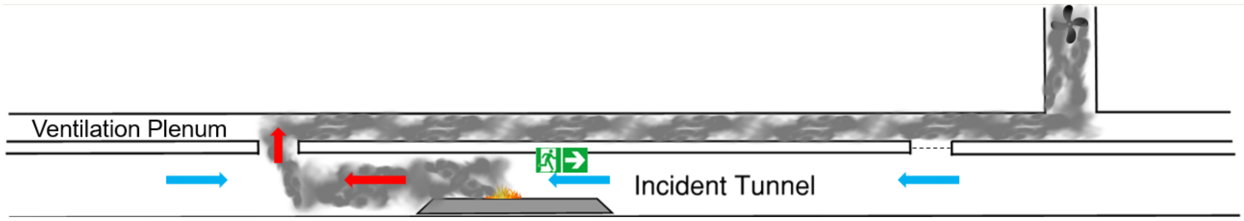

Figure 8.12. Single Point Extraction Ventilation Method

8.9 Tunnel Ventilation System Design Life

Electronic components shall have a design life of 15 years. All other tunnel ventilation equipment shall have a design life of 40 years.

9 Tunnel Systems

The tunnel systems design will minimize signal infrastructure, such as wayside signals, to accommodate clearances, space requirements, and access for maintenance. If wayside signal infrastructure cannot be avoided within a tunnel, the systems and tunnel designs will be coordinated to ensure that space is provided for the wayside signals. This includes visibility and space available to accommodate the signal cases or bungalows.

Systems such as Tunnel Radio's Tunnel Voice FD4 and Tunnel Link for PTC (Positive Train Control) will be analyzed for use within the tunnel to ensure continuous voice and PTC communications.

The railroad systems along the entire corridor are addressed in the Track & Signal Basis of Design.

10 Construction Staging

Refer to Del Mar Alternatives Analysis Report and Miramar Alternatives Analysis Report.

11 Construction Impacts

Refer to Del Mar Alternatives Analysis Report and Miramar Alternatives Analysis Report.

12 Tunnel Technology Trends

In the last few decades, significant advances have been made in both TBM technology and our understanding of how TBMs operate. Advances include the development of larger TBMs that are more productive, can minimize ground movements, and can be operated more efficiently and safely than in the past.

12.1 Advances in TBM Technology

12.1.1 TBM Size

The largest TBM tunnels built to date include the Tuen Mun–Chek Lap Kok Link Tunnel in Hong Kong and the Alaskan Way viaduct replacement tunnel in Seattle, with excavated diameters of 57.73 feet and 57.33 feet, respectively. A single bore tunnel for the Del Mar alignments or Miramar alignments would have an excavated diameter of approximately 63 feet including lining thickness and an allowance for overcut.

While a 63-ft TBM represents a significant challenge for tunnel contractors, manufacturers, and designers, the tunnel may not be built until 2035 by which time the use of a 63-ft TBM may have become more common. Figure 12.1 shows the sizes of TBMs manufactured by Herrenknecht, a leading supplier and designer of TBMs, over a span of 45 years. In 2011, Herrenknecht received an order for a 61.17-ft TBM for the Orlovski Tunnel in St. Petersburg, Russia, after the evaluation of tenders and award of the project to a construction consortium. Although the project was cancelled by the City of St. Petersburg due to lack of funding, the project established the feasibility of using a TBM greater than 60 feet.

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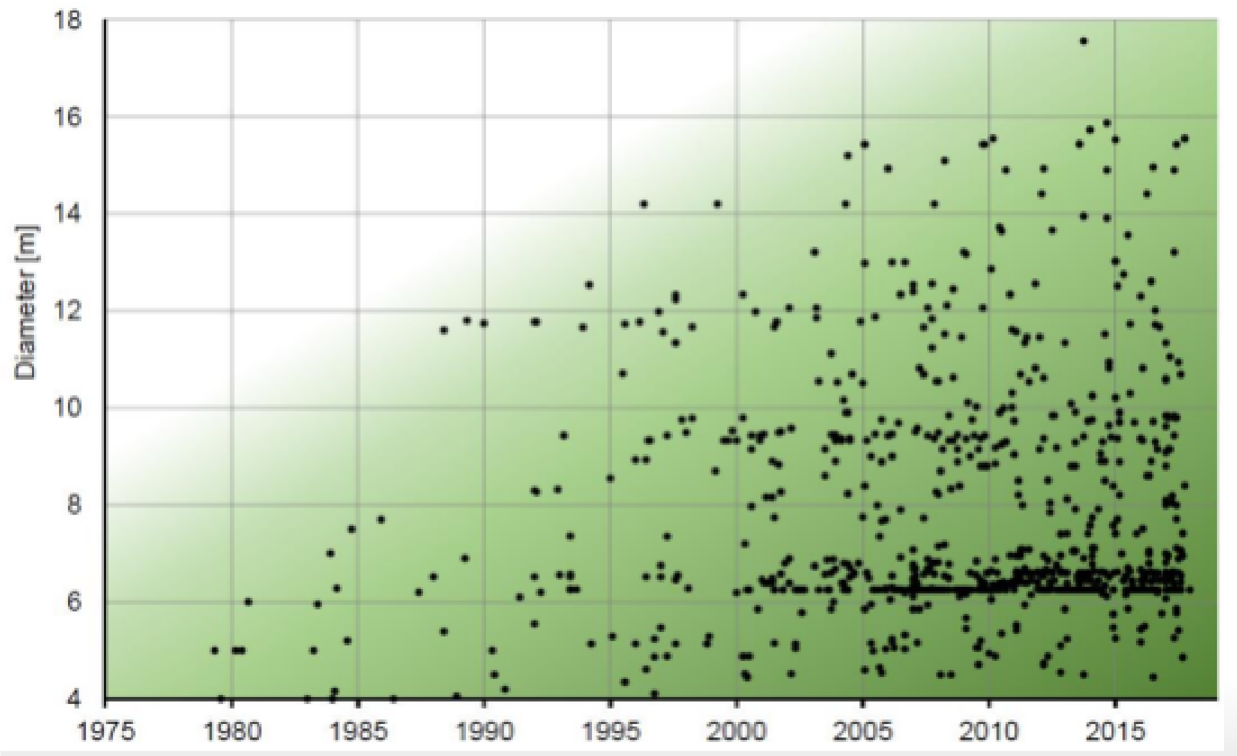

Figure 12.1. Evolution of Herrenknecht Large Bore TBMs

12.1.2 TBM Cutterhead and Excavation

Recent advances in the design of TBM cutterheads now allow tool selection to be better optimized for the specific ground conditions, and advances in how cutter wear is monitored have resulted in less down time and damage to the TBMs.

The overall strength and reliability of TBMs have also been improved, with more powerful, efficient, and reliable equipment, such as pumps and motors.

Regarding EPB TBMS, advancements have also been made in how face pressure is calculated and how it can be monitored and maintained both during tunneling and TBM stoppages.

12.1.3 TBM Excavation, Grouting and Spoil Measurement

The design and manufacture of the TBM screw conveyor—a vital part of the TBM—have been improved over the years. This has helped to optimize its performance and reliability and has been key to improving the overall operation and reliability of the TBM.

Another key area of improvement has been the technology used to weigh and calculate the volume of spoil excavated. The over-excavation of soil can result in surface settlement. Recent advances in how the spoil is weighed on the belt and how its volume is measured have helped reduce the potential for over-excavation during tunneling.

Advances have also been made to both the type of grouts used and how they are injected around and behind the tail skin of the TBM. This has been important in helping to reduce the overall settlement associated with the use of TBMs.

12.2 Tunnel Segment and Lining Design

Tunnel segment design also continues to change. Traditional empirical and analytical design techniques are being replaced by more advanced numerical modeling of the segments. In recent years, the use of fiber-reinforced concrete for the segments has increased, and improvements have been made to the waterproofing gaskets that are used. In addition, concrete mix designs and additives have improved, and options such as waterproof concrete with high corrosion resistance are starting to be used.

12.3 Advances in Geotechnical Investigations

Innovations in drilling and sampling techniques are helping to obtain better quality samples for the design of tunnels. This has included the increased of use geophysical techniques to better understand the geology along the alignment.

Advances have also been made in how geological information can be obtained in front of the TBM during mining. Various geophysical techniques are being developed to get a better understanding of both the ground conditions and the presence of potential obstructions in front of the TBM.

12.4 Application of BIM Technology

Building Information Management (BIM) is a highly collaborative process that allows owners, engineers, and contractors to plan, design, and construct large infrastructure or building projects within a single 3D CAD model.

These models can also be used in the operation and maintenance of the structures, allowing owners to make informed decisions and better manage their assets.

12.5 Alternative Approach to Contract Procurement

Up until the last 10 to 15 years, traditional contract procurement methods for tunnel projects in the United States remained unchanged, with either design-bid-build or design-build approaches being commonly used. However, in recent years, alternative contracting approaches have been used to help reduce cost and risk for both owners and contractors.

Other examples of Contract Procurement include Progressive Design-Build, CM/GC or Construction Manager/General Contractor and Alliancing, whereby all parties involved work together as a single team with contractually defined risk-reward provisions to meet or exceed defined target costs. As the project progresses the optimal contract procurement strategery will be developed amongst the various project stakeholders.

12.6 Fire Life Safety and Ventilation

The design and testing of fire life safety systems continue to develop, and in recent years advances have been made in fire and tunnel ventilation modeling and simulation using CFD (computational fluid dynamics) technology. How tunnel structures are affected during a fire, such as how, when, and why the concrete linings are damaged during a fire, is better understood, which ultimately allows the tunnel design team to design safer tunnels.

Fire suppression both within tunnels and onboard trains is an emerging technology in the fire life safety field.

12.7 Mixed Use for Tunnels

Increases in tunnel sizes have allowed for mixed-use tunnels. Examples include both road and rail in the same tunnel or tunnels for utilities, such as power or water, to be included with road or rail projects. The Smart Tunnel in Malaysia is an example where a road tunnel (above) was combined with a storm drain tunnel as shown in Figure 12.2.

Figure 12.2. The SMART Tunnel in Malaysia: Road tunnel combined with storm drain

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Appendix A – Flood Hazard Maps

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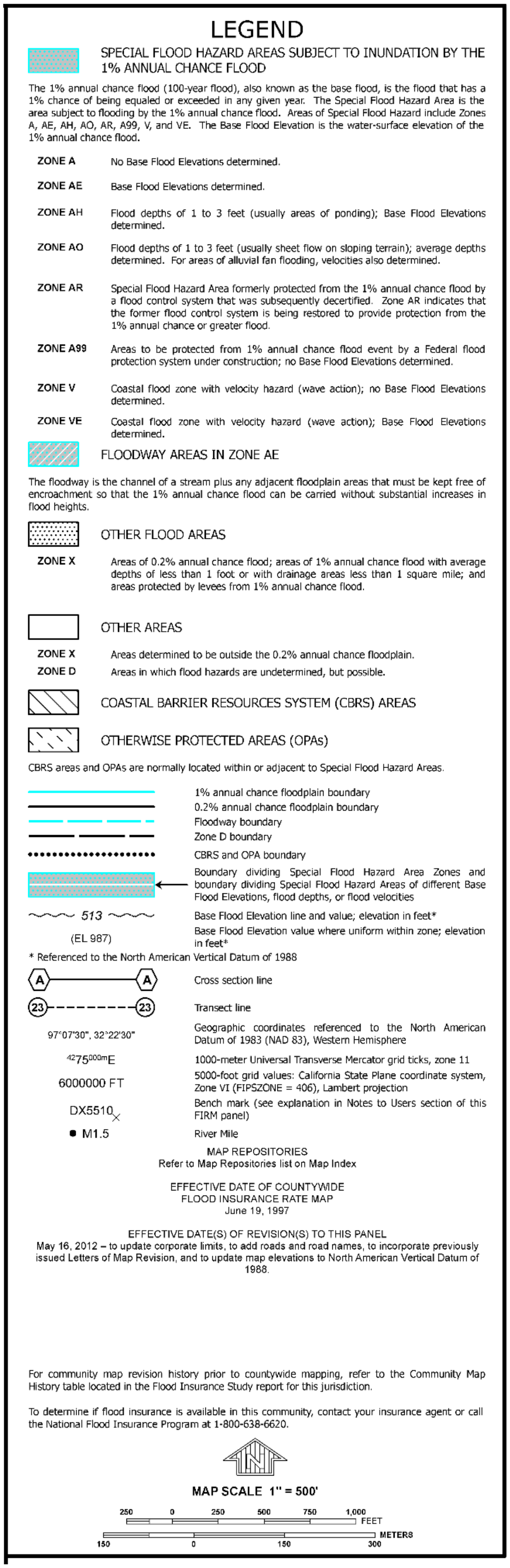
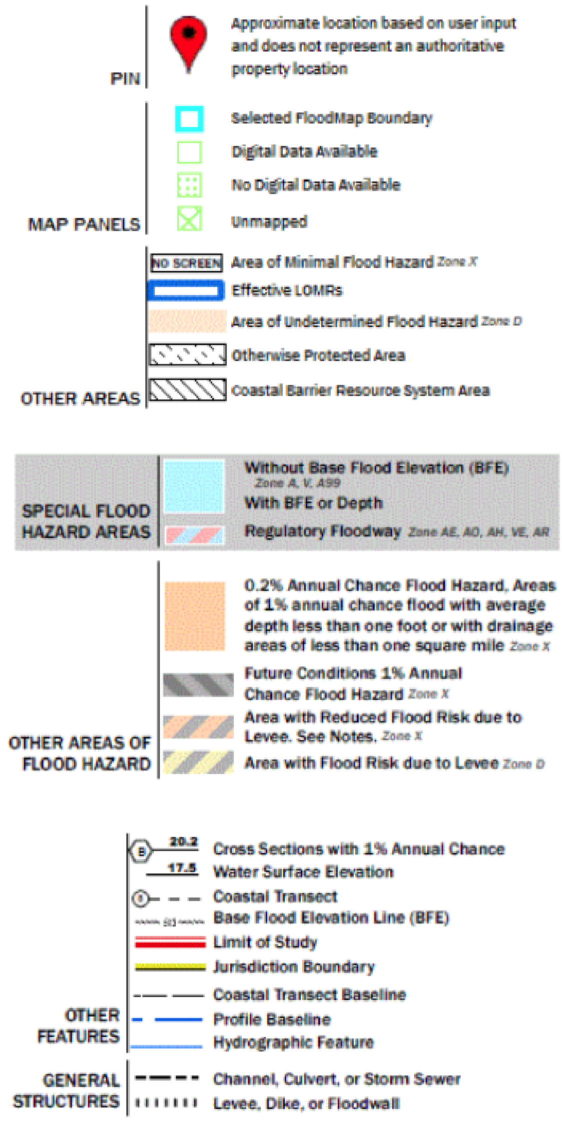
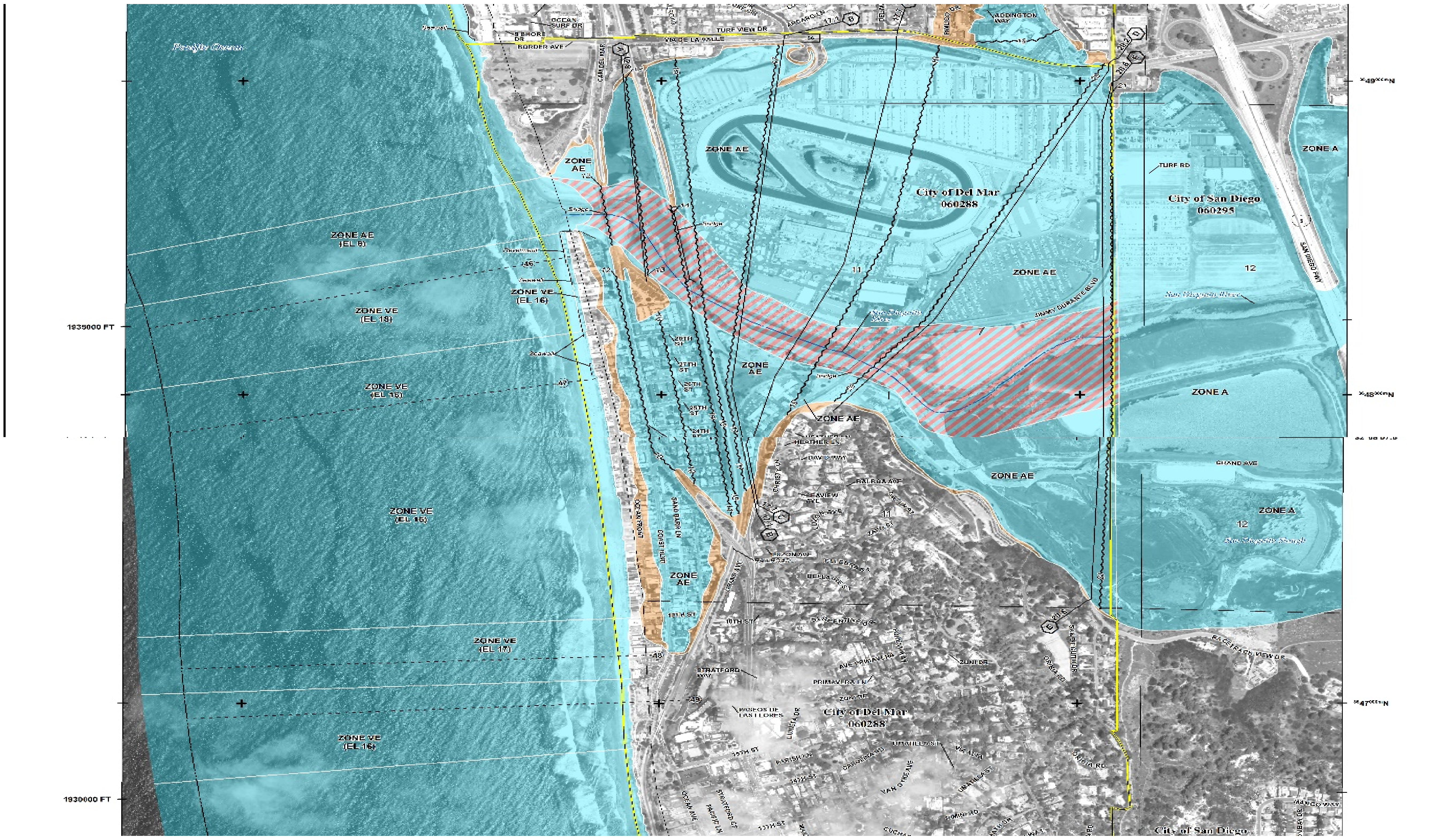
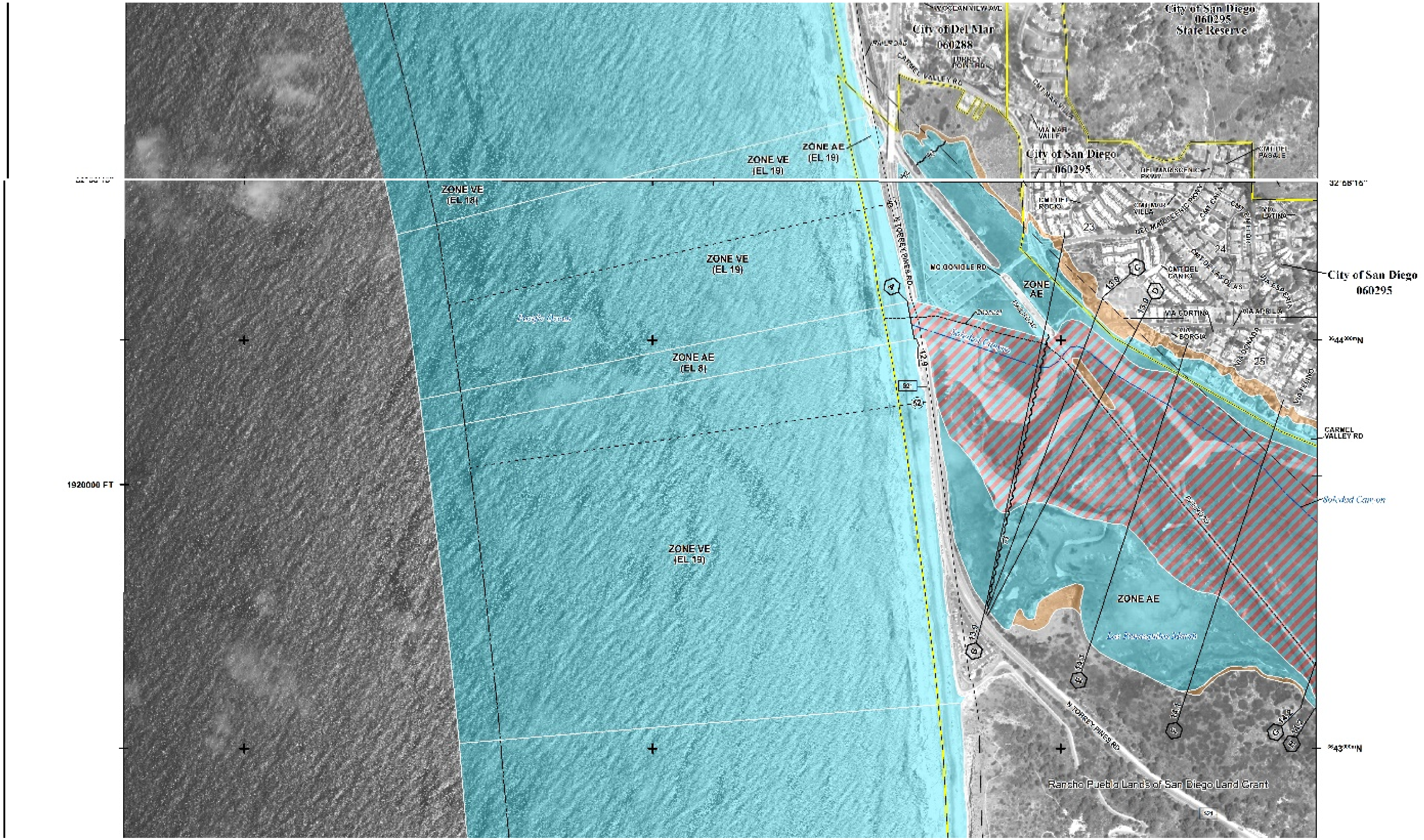




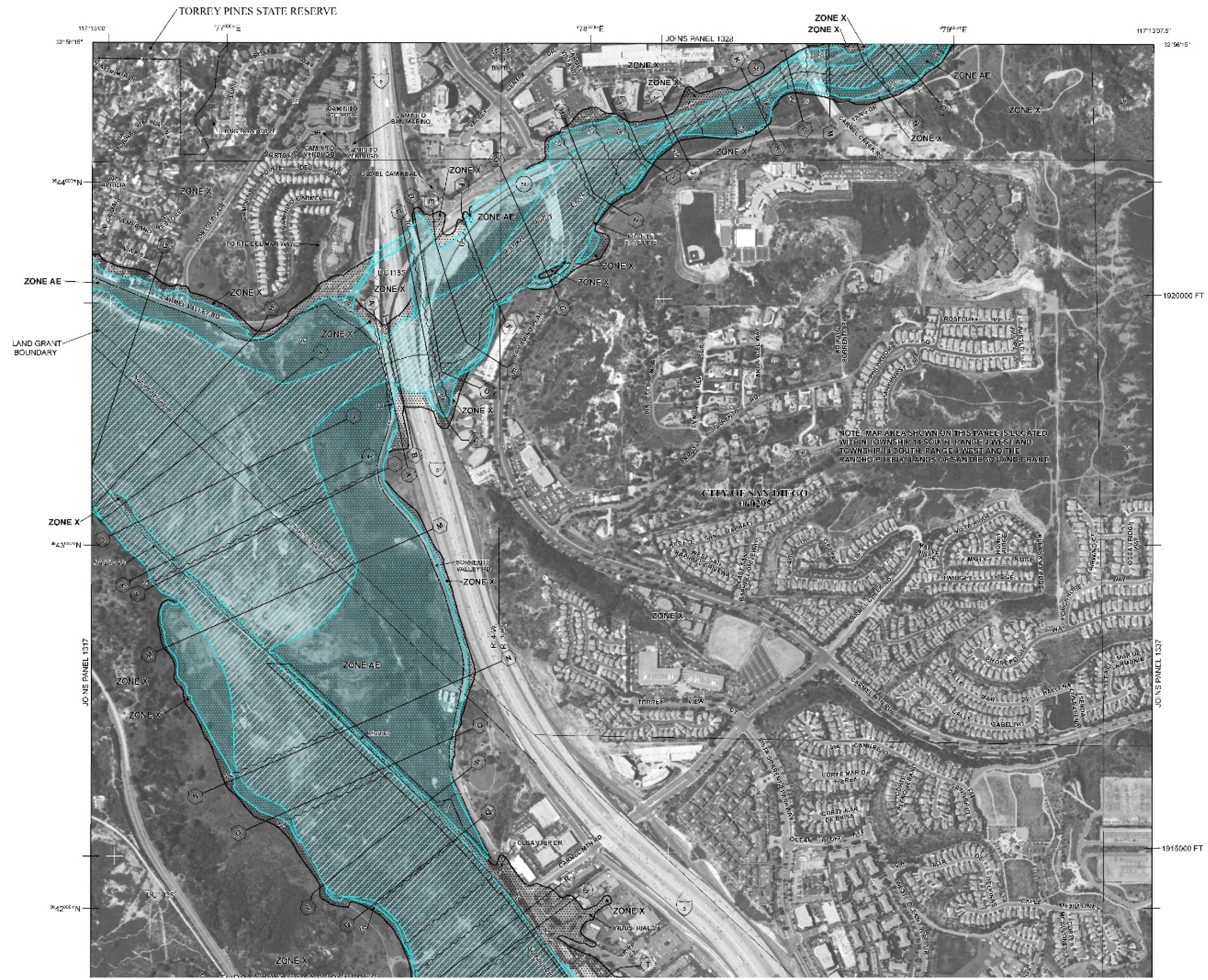
FEMA Flood Hazard Map Legend



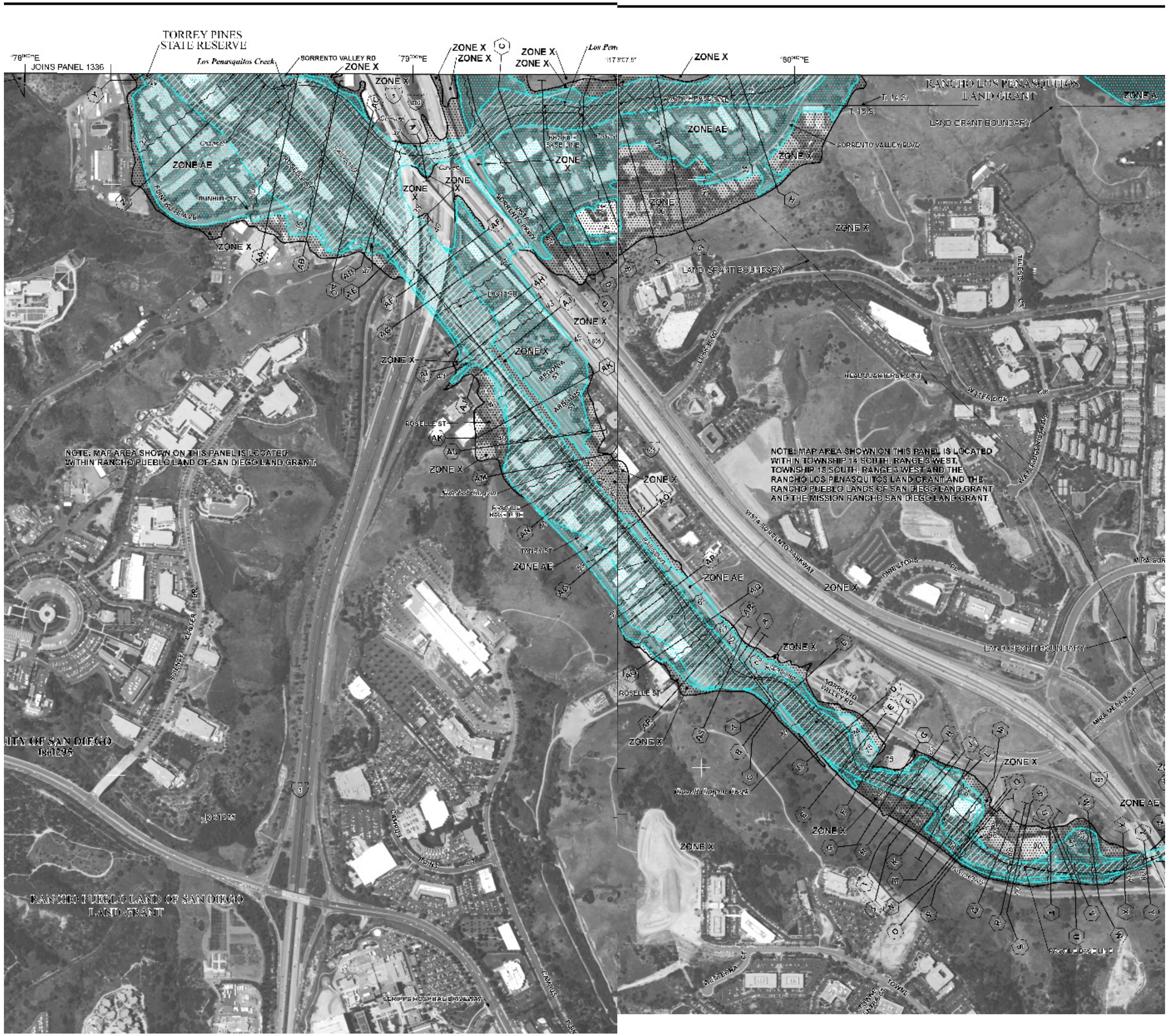
Del Mar Tunnel North Portal (FEMA 06073C1307H & 06073C1309H)



Del Mar Tunnel- Camino Del Mar alignment South Portal (FEMA 06073C1317H)



Del Mar Tunnel South Portal - Crest Canyon High Speed alternative (FEMA 06073C1336G



Miramar Tunnel North Portal (FEMA 06073C1338G & 06073C1339G)

Miramar Tunnel South Portal (FEMA 06037C1601G & 06073C1602G)

Appendix B – Preliminary Geologic Maps

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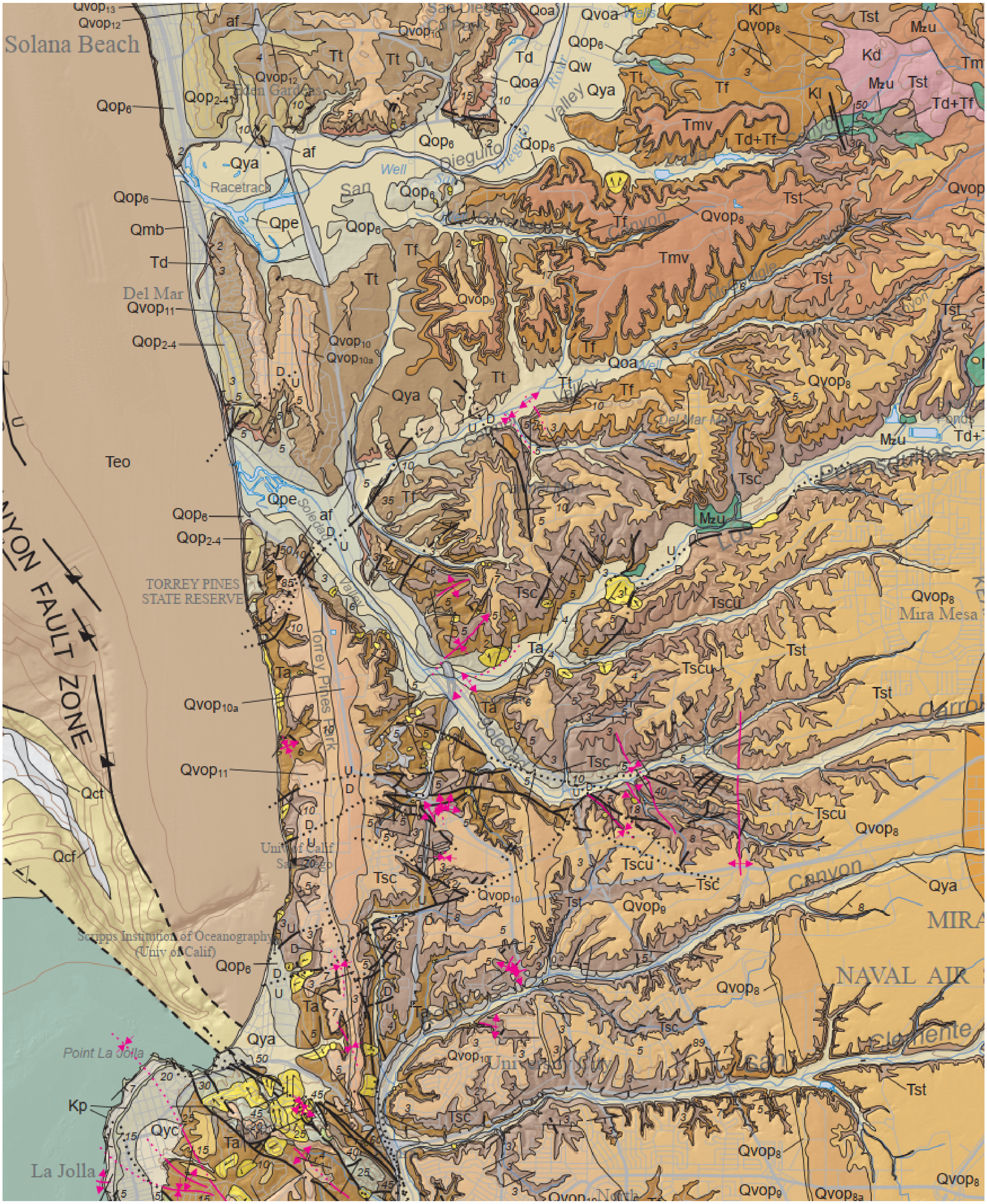

Extract from Geologic Map of the San Diego 30' x 60' Quadrangle, California (Kennedy and Tan, 2008)

Appendix C - Seismic Hazard Maps

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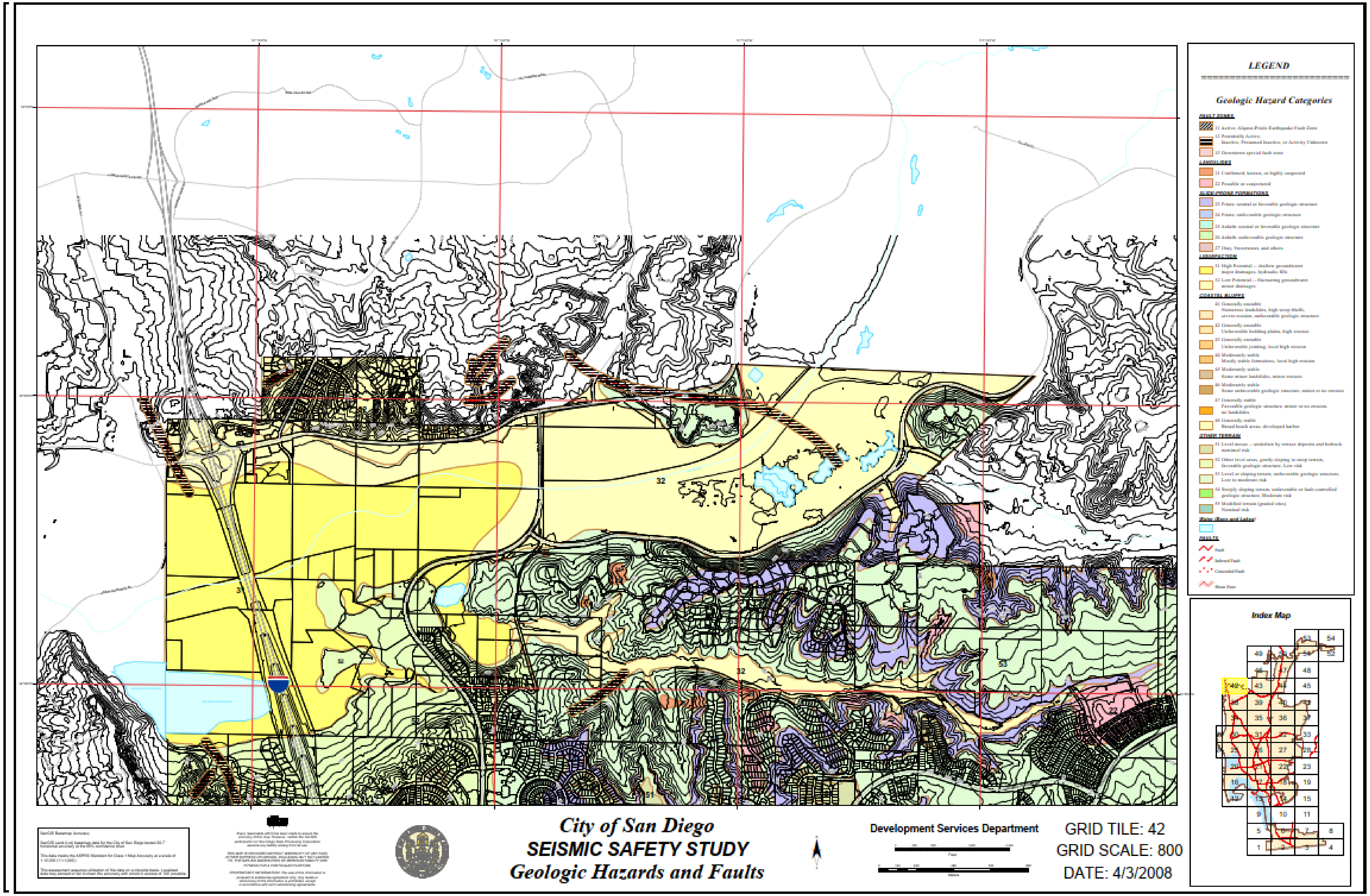
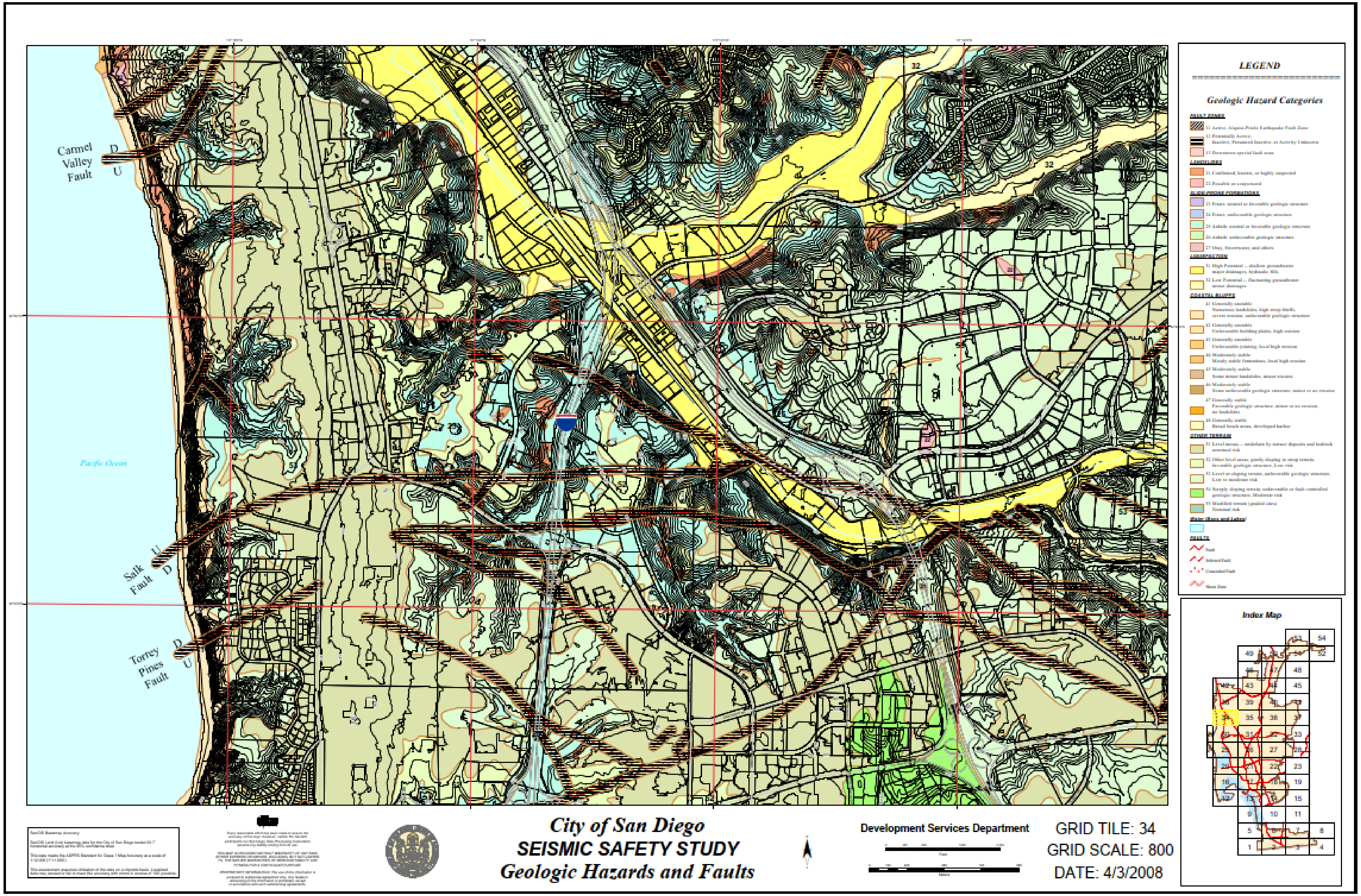
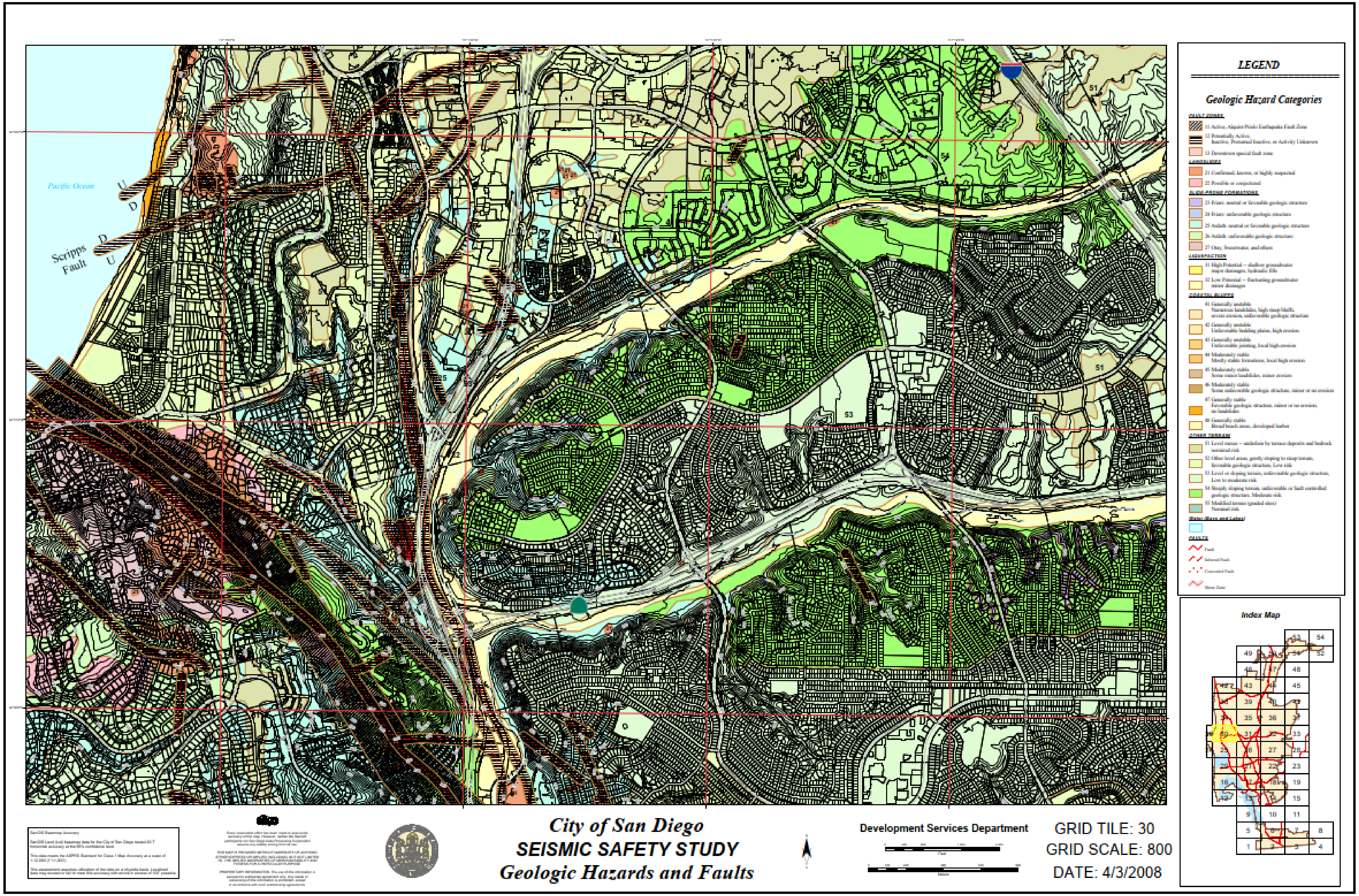


Geologic Hazards and Faults - Del Mar Tunnel (North)Geologic Hazards and Faults - Del Mar Tunnel (South)



Geologic Hazards and Faults - Miramar Tunnel (North)



Geologic Hazards and Faults - Miramar Tunnel (South)

Appendix D –Tunnel Cross-Section Drawings

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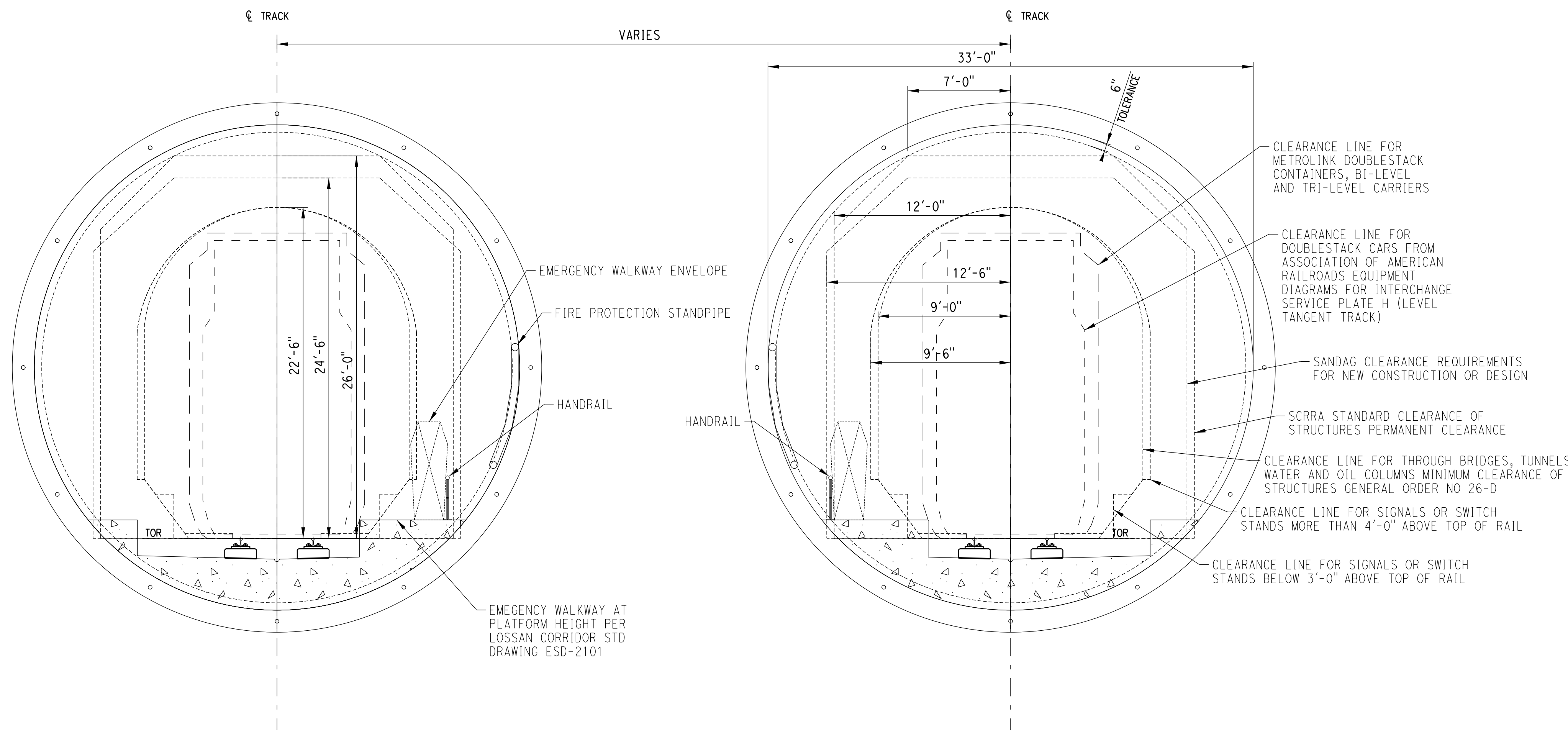

TYPICAL TWIN BORE TUNNEL
NOT TO SCALE

The tunnel space proofing and resulting tunnel diameters for twin and single bores shown in this report do not reflect the tunnel optimization effort. Refer to Appendix M Tunnel Optimization Memo for updated tunnel diameters.

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DRAWN BY C. JURKIEWICZ
CHECKED BY D. FIELD
APPROVED BY S. LO GRASSO
DATE 5/28/2021

MOTT MACDONALD
750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

TWIN BORE TUNNELS
TYPICAL SECTIONS

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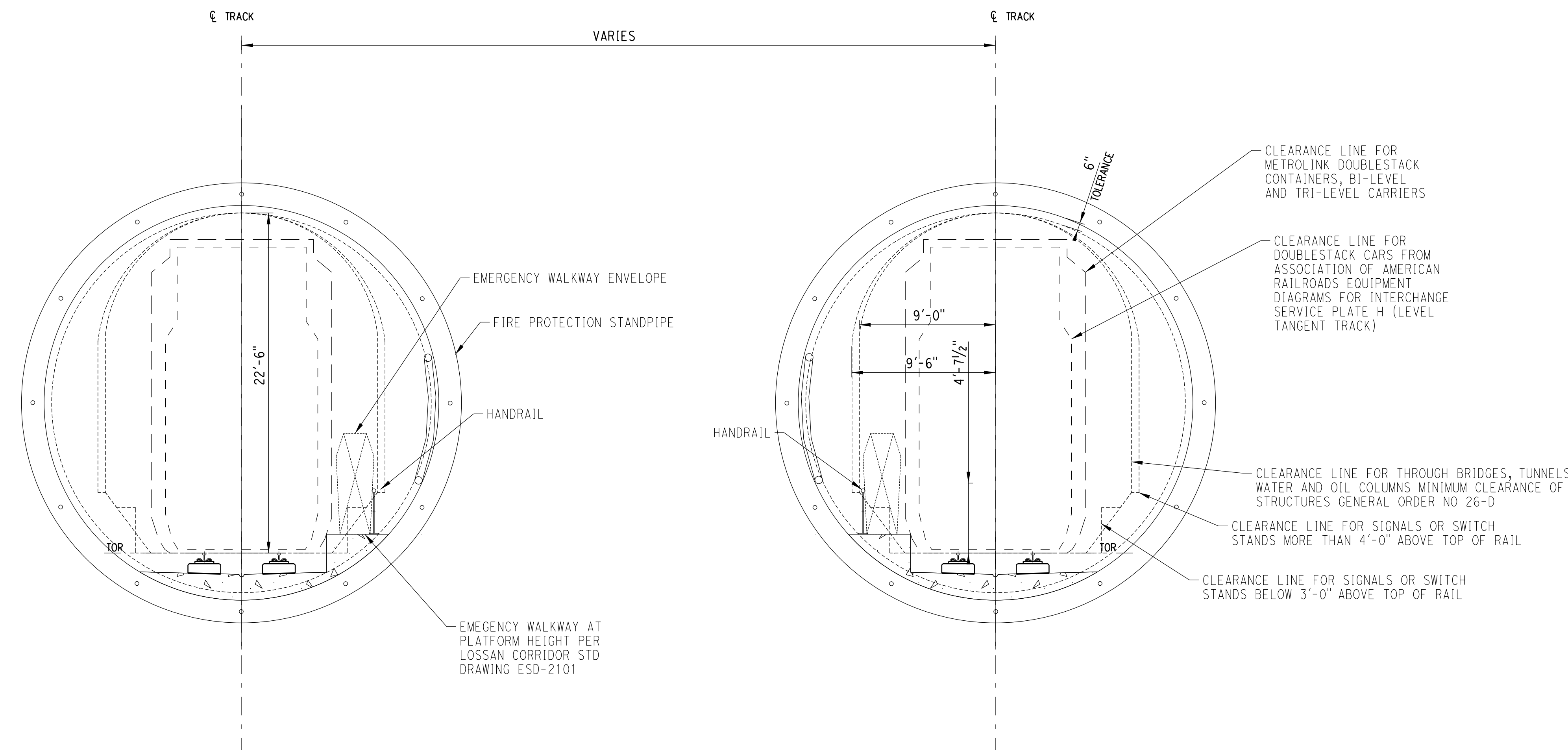

TYPICAL TWIN BORE TUNNEL - WITH CPUC CLEARANCE LINES

NOT TO SCALE

NOTES

1. TUNNELS HAVE NOT BEEN SPACEPROOFED.

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TWIN BORE TUNNELS TYPICAL SECTIONS WITH CPUC CLEARANCE LINES

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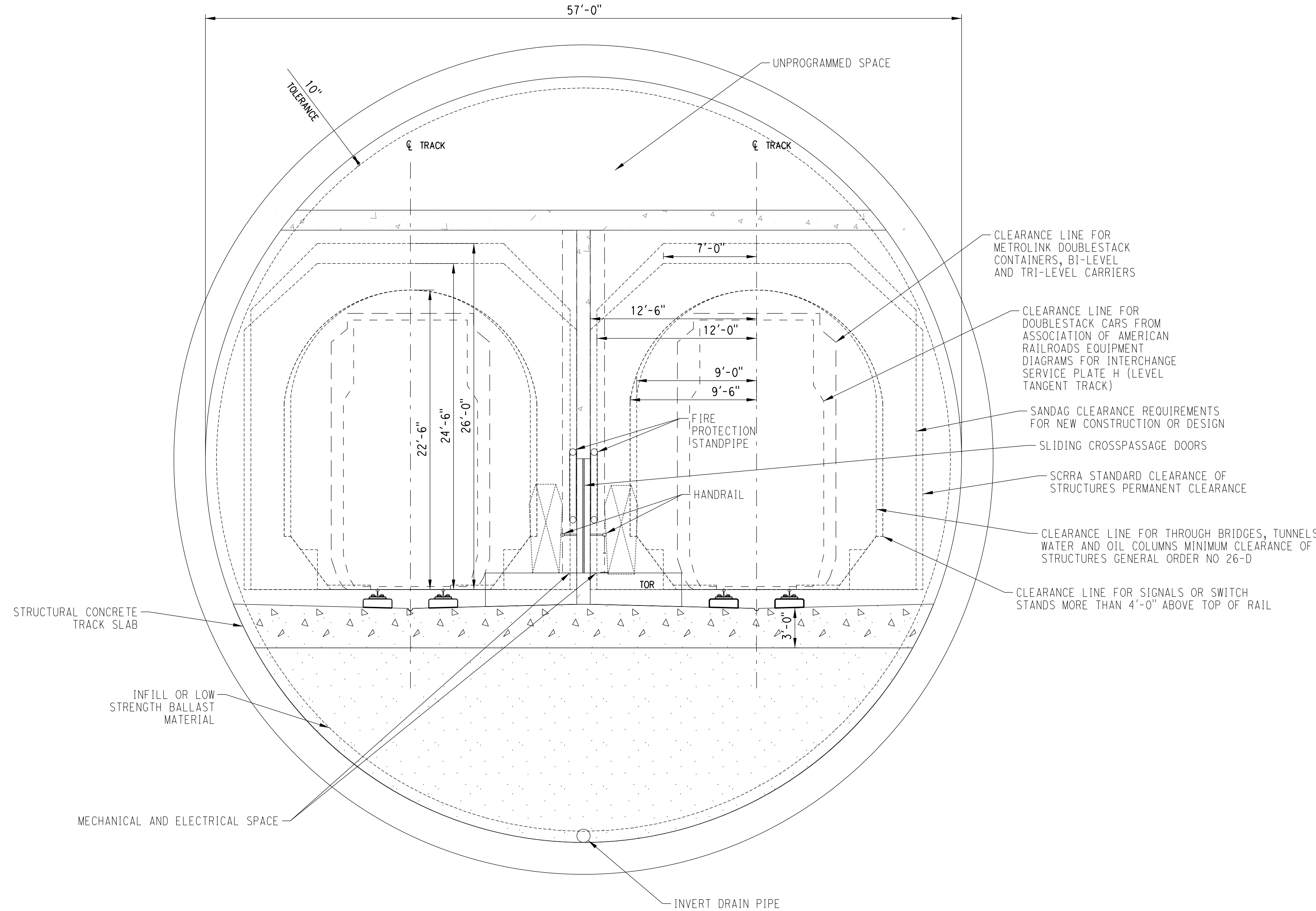

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TYPICAL SINGLE BORE TUNNEL

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750 B Street, Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

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DEL MAR ALTERNATIVES ANALYSIS

SINGLE BORE TUNNEL
TYPICAL SECTION

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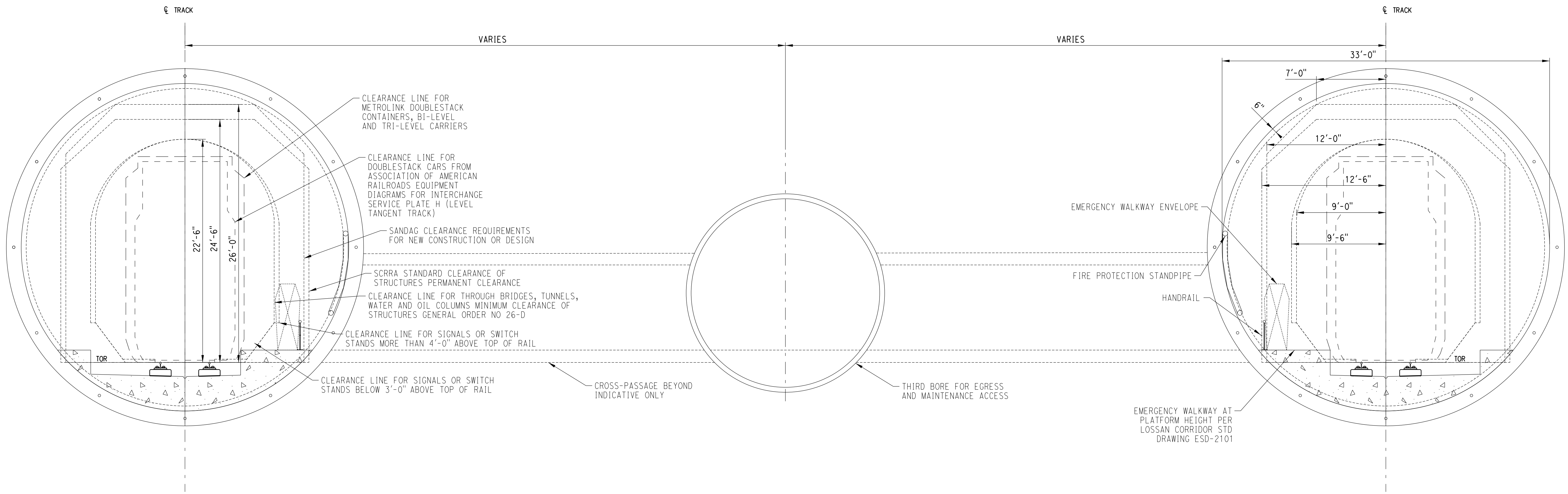

TYPICAL TRIPLE BORE TUNNEL

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DATE 5/28/2021

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Appendix L. Basis of Design Report for Noise and Vibration

SANDAG

Basis of Design Criteria

For Noise & Vibration

San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study (SD-LOSSAN)

December 2021

Prepared by:

HDR

Entech Consulting Group
Planning • Environmental • Construction Management

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619) 699-1900
www.sandag.org

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ABSTRACT

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AUTHOR:	San Diego Association of Governments
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ABSTRACT:	This noise and vibration design criteria report was prepared to supplement the conceptual engineering study. The conceptual engineering study will implement multiple improvements to facilitate increased speeds and service frequency that carry the potential to alter the wayside noise and vibration environment experienced by sensitive receivers adjacent to the alignments within the corridor subject to further evaluation. The noise and vibration design criteria and guidelines presented for the track and signal conceptual engineering study are based on the *High-Speed Ground Transportation Noise and Vibration Impact Assessment* guidance provided by the Federal Railroad Administration (FRA 2012). This noise and vibration design criteria report provides criteria for inclusion in track and signal design that takes into consideration the design elements that influence the source-path and receiver framework and applicable federal, state, and local regulations.

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Acronyms

CFR	Code of Federal Regulations
dB	Decibels
dBA	A-weighted decibels
DNL	Day Night Noise Level
FRA	Federal Railroad Administration
LOSSAN	Los Angeles-San Diego-San Luis Obispo Rail Corridor
FTA	Federal Transit Administration
HSGT	High Speed Ground Transportation
Lmax	Maximum Sound Level
Leq (h)	A-weighted equivalent hourly sound level
Ldn	Day-night average sound level with a 10-decibel penalty applied to nighttime hours (between 10pm to 7am)
PPV	Peak Particle Velocity
RMS	Root Mean Square
SANDAG	San Diego Association of Governments
SEL	Sound Exposure Level
USEPA	United States Environmental Protection Agency
VdB	Vibration velocity

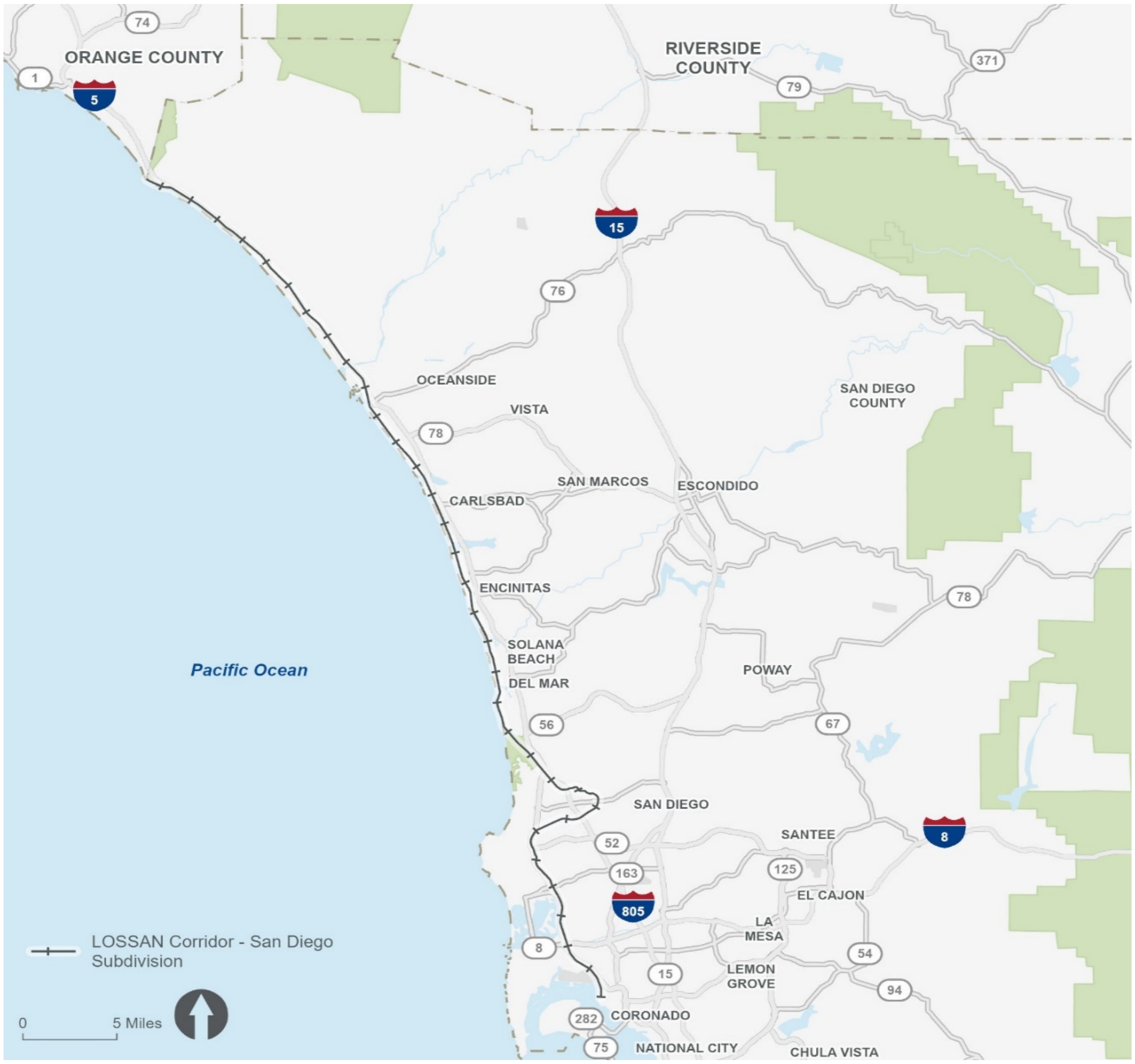
1 Introduction

This noise and vibration basis of design report was prepared by the San Diego Association of Governments (SANDAG) to supplement the conceptual engineering study, which will develop a program of improvements along the San Diego Subdivision of the Los Angeles-San Diego-San Luis Obispo (LOSSAN) Rail Corridor to reduce travel time for commuter and intercity passenger rail service to be competitive with the automobile travel time. This program of improvements will increase track capacity, improve resiliency, enhance safety, and support increased passenger and freight frequencies along the 60.1-mile San Diego Subdivision of the LOSSAN corridor from the Orange County/San Diego County line to downtown San Diego, supporting the goal of doubling the amount of passenger rail service within the corridor by 2035. Figure 1-1 shows the segment of the LOSSAN Rail Corridor under evaluation. At this stage, these improvements remain conceptual and this report outlines key noise and vibration criteria that should be considered in conjunction with the conceptual engineering of these improvements.

Currently, the corridor has constraints that limit increases in train speeds due to horizontal curves, track class, train type, special trackwork, and crossovers. The conceptual engineering study will propose design changes that will facilitate increased speeds and train frequency that in turn could alter the wayside noise and vibration environment experienced by sensitive receivers adjacent to the alignments within the project corridor. Many of the improvements will occur within the existing, in-service rail right-of-way, while other locations may result in a deviation from the existing alignment. Shifts in track location, special trackwork along the existing alignment, or the addition of new track with increased train speeds may cause increases or decreases in noise and vibration compared to existing levels.

The track and signal design process will focus on implementing higher speed track classes on the shared use LOSSAN corridor. The Federal Railroad Administration's (FRA) manual on *High-Speed Ground Transportation Noise and Vibration Impact Assessment* is specifically designed to address train speeds above 90 mph; therefore, the noise and vibration design criteria and guidelines presented for the track and signal conceptual engineering study are based on FRA guidance (FRA 2012). Trains traveling at high speeds have the greatest influence on wayside noise and vibration. Therefore, successful noise and vibration control require consideration of both the track and the vehicle as a system, because the interaction of the wheel and the rail is responsible for the bulk of wayside noise and vibration impacts. This noise and vibration design criteria report will provide criteria for inclusion in track and signal design that takes into consideration the design elements that influence the source-path-receiver framework and applicable federal, state, and local regulations.

Figure 1-1. Project Study Area



2 Fundamentals

This section provides general background on noise and ground-borne vibration and defines the terms used to quantify levels for transit projects. Noise and vibration descriptors and the correlation between the source-path-receiver framework provides the foundation for understanding the factors that influence noise and vibration levels.

2.1 Noise

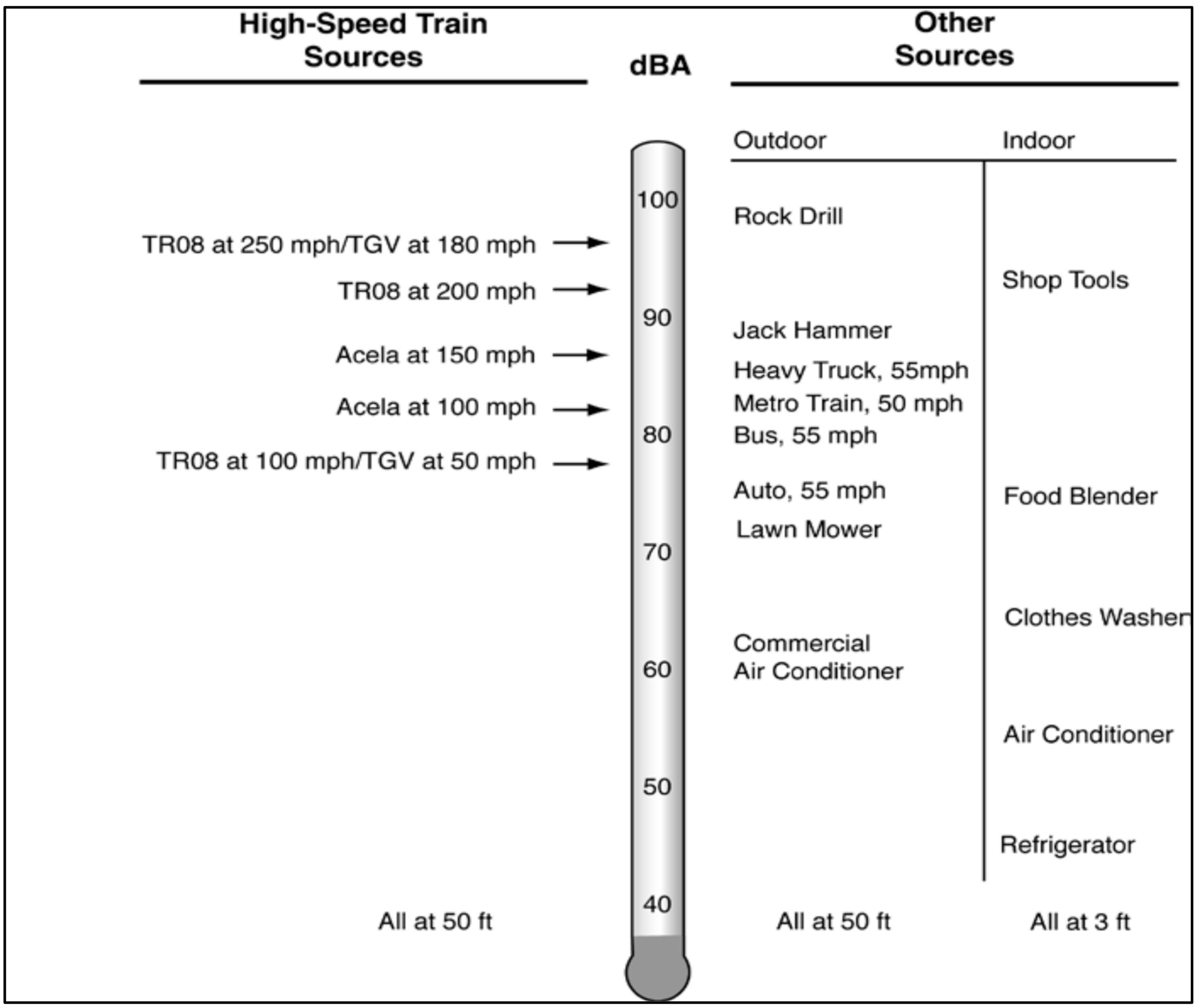
Noise can be quantified in many different manners, depending on its temporal (time), tonal (frequency), or loudness characteristics. In general, environmental noise assessment addresses relative changes in noise levels over time and relates those changes with effects on human beings.

Noise magnitude is expressed in units of decibels (dB), which is a logarithmic quantity comparing fluctuating air pressure to that of a standardized reference air pressure of 20 micro-pascals (i.e., dB re 20 μPa). For this reason, the noise levels that humans hear are called sound pressure levels. Noise is expressed as a logarithmic quantity because humans are sensitive to relative changes in noise levels. To illustrate, humans can just barely perceive a change in noise levels of +/- 3 dB, can easily perceive a change of +/- 5 dB, and will generally perceive a change of +/- 10 dB as a doubling or halving in noise levels.

With respect to tonal qualities (frequency), a frequency weighting adjustment has been standardized to account for human auditory response over the audible frequency range of approximately 20 Hz to 20,000 Hz. Humans respond less sensitively to low frequency noise ranges, exhibiting a maximum sensitivity to tones in mid-frequency ranges and being somewhat less sensitive at higher frequency ranges. This weighted frequency adjustment is referred to as "A-weighting", with results expressed as A-weighted decibels, or dBA.

The A-weighted noise level is the basic descriptor for environmental noise. Typical A-weighted noise levels are illustrated in Figure 2-1.

Figure 2-1. Typical Outdoor and Indoor Noise Levels



Source: FRA, 2012

The following single-number descriptors, all based on the A-weighted sound level as the fundamental unit, are commonly used for environmental noise measurements, computations, and assessment:

- The **Sound Exposure Level (SEL)** describes a receiver's cumulative noise exposure from a single noise event. It is represented by the total A-weighted sound energy during the event, normalized to a 1-second interval. For a vehicle passby, the time interval over which the SEL is evaluated includes all the acoustic energy related to the event, extending over the points in time when the sound level is at least 10 dB below the highest sound level during the passby. SEL is the primary descriptor of rail vehicle noise emissions and an intermediate value in the calculation of both Leq(h) and Ldn (defined below).
- The **Hourly Equivalent Sound Level [Leq(h)]** describes a receiver's cumulative noise exposure from all events over a 1-hour period. The underlying metric for calculating Leq(h) from single noise events during a 1-hour period is SEL. Leq(h) is used to assess noise impacts on non-residential land uses. For assessment, Leq(h) is computed for the loudest operating hour during the hours of noise-sensitive activity.
- The **Day-Night Sound Level (Ldn or DNL)** describes a receiver's cumulative noise exposure from all events over a 24-hour period. The basic unit used in calculating Ldn is the Leq(h) for each 1-hour period. It may be thought of as noise exposure, totaled after increasing all nighttime A-Levels (between 10 p.m. and 7 a.m.) by 10 dB. Every noise event during the 24-hour period increases this exposure, louder events more than quieter events, and events that are of longer duration more than briefer events. Ldn is used to assess noise impacts on residential land uses. Typical community Ldns range from approximately 50–70 dBA, where 50 represents a quiet noise environment, and 70 is a noisy one.

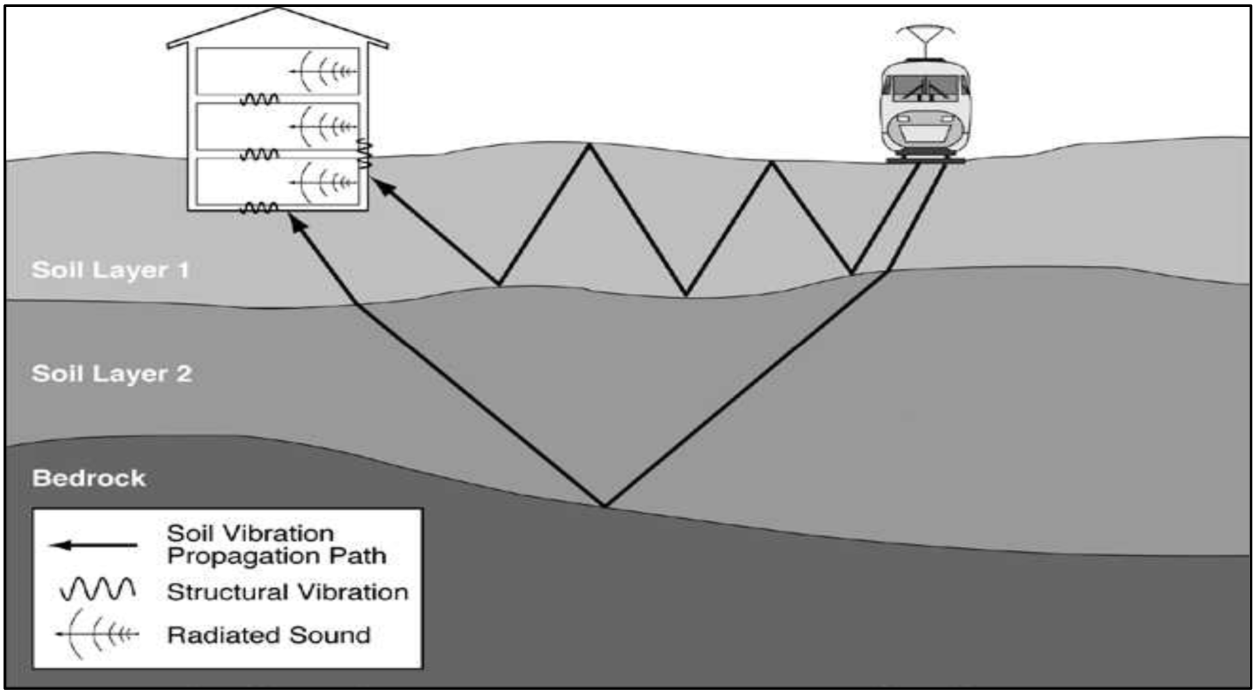
2.2 Vibration

Ground-borne vibration is an oscillatory motion that can be described in terms of the displacement, velocity, or acceleration of the motion. Each of these measures can be further described in terms of amplitude. Displacement is the easiest descriptor to understand. It is simply the distance that a vibrating point moves from its static position (i.e., its resting position when the vibration is not present). The velocity describes the instantaneous speed of the movement, and acceleration is the instantaneous rate of change of the speed.

Although displacement is fundamentally easier to understand than velocity or acceleration, it is rarely used for describing ground-borne vibration, the following reasons: 1) human response to ground-borne vibration correlates more accurately with velocity or acceleration; 2) the effect on buildings and sensitive equipment is more accurately described using velocity or acceleration; and 3) most transducers used in the measurement of ground-borne vibration measure either velocity or acceleration. For this study, velocity is the fundamental measure used to evaluate the effects of ground-borne vibration.

One potential effect of the project alternatives is an increase in vibration that is transmitted from the tracks through the ground into adjacent residential buildings. As with sound, vibration attenuates as a function of the distance between the source and the receptor. Vibration caused by trains moving along a transit structure, such as at-grade ballast and tie track, radiates energy into the adjacent soil. Buildings respond differently to ground vibration depending on the type of foundation, the mass of the building, and the building interaction with the soil. Once inside the building, vibration propagates throughout the building with some attenuation with distance from the foundation, but often with amplification due to floor resonances. The basic concepts for rail system-generated ground vibration are illustrated in Figure 2-2.

Figure 2-2. Propagation of Ground-Borne Vibration into Buildings



Source: FRA, 2012

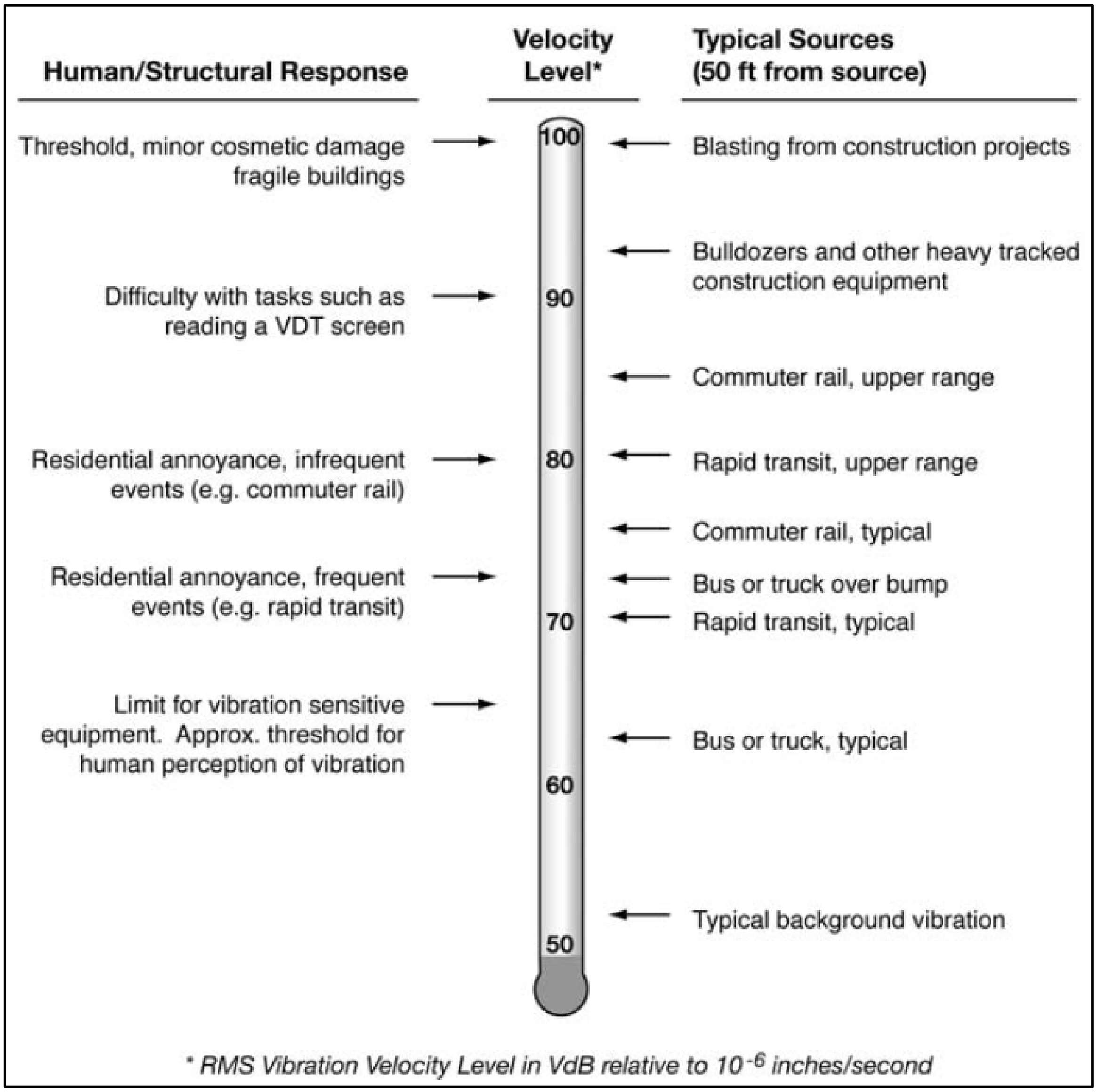
Root mean square (RMS) is defined as the square root of the average of the squared amplitude of the vibration signal. FRA uses a logarithmic scale is to describe vibration levels; the abbreviation VdB is used for vibration decibels by the FRA. Vibration can also be expressed as the peak particle velocity (PPV), which is generally used to evaluate whether vibration has the potential to cause damage to fragile building structures. PPV is normally expressed in inches per second. However, FRA exclusively assesses vibration impacts using VdB.

The potential adverse effects of rail transit ground-borne vibration are as follows:

- Perceptible Building Vibration: This is when building occupants feel the vibration of the floor or other building surfaces. Experience has shown that the threshold of human perception is around 65 VdB and that vibration that exceeds 75 to 80 VdB may be intrusive and annoying to building occupants.
- Rattle: The building vibration can cause rattling of items on shelves and hanging on walls, and various rattle and buzzing noises from windows and doors.
- Reradiated Noise: The vibration of room surfaces radiates sound waves that may be audible to humans. This is referred to as ground-borne noise. When audible ground-borne noise occurs, it sounds like a low-frequency rumble. For surface rail systems, the ground-borne noise is usually masked by the normal airborne noise radiated from the transit vehicle and the rails.
- Damage to Building Structures: Vibration from rail systems is usually one to two orders of magnitude below the most restrictive thresholds for preventing building damage. However, fragile structures may be susceptible to damage if the tracks are in sufficient proximity to the structure.

Figure 2-3 shows typical RMS vibration velocity levels from rail and other sources, as well as the human and structure response to such levels.

Figure 2-3. Typical Vibration Velocity Levels



Source: FRA, 2012

Vibration magnitude can be described using various quantities depending on the intent of the analysis and type of sensitive receptor being evaluated. In accordance with FRA procedures, all vibration measurements and predictions are in the form of energy-averaged RMS levels. RMS represents a mathematically averaged level, which is proportional to the energy-of-motion generated by a vibrating surface. The RMS vibration velocity level has been shown to correlate better with the human body's sensitivity to vibration when computed with a one-second response time (i.e., RMS ‘slow’). Train passby vibration events are typically expressed in VdB levels using the maximum RMS levels within each frequency band.

A related vibration metric would be the PPV, which is a measure of the vibration signal’s highest absolute instantaneous magnitude. Being a measure of vibration velocity, the PPV is expressed in linear units of inches per second. Human annoyance is generally not a function of instantaneous PPV levels; however, potential damage to buildings and structures can be, so an analysis of PPV levels is only used to assess potential cosmetic or major damages to structures. For example, PPV levels are used to describe potential building damages from impact sources such as construction.

3 Noise & Vibration Impact Criteria

The criteria used in evaluating noise and vibration impacts from high-speed ground transportation are based on maintaining a noise environment considered acceptable for land uses where noise and vibration may have an effect. These criteria take into account the unusual characteristics of high-speed rail operations. These criteria are adapted from criteria developed by FRA for rail noise sources operating on fixed guideways or at fixed facilities. It is important to understand what is considered acceptable and unacceptable noise and vibration levels and what is considered a noticeable change for community reaction above existing levels. This section presents federal and state laws and regulations applicable to noise and vibration affected by the project alternatives under evaluation.

3.1 Federal Railroad Administration

3.1.1 Noise and Vibration Impact Assessment Guidelines

FRA provides guidance regarding the evaluation of noise and vibration impacts from construction and operation of high-speed trains in the *High-Speed Ground Transportation Noise and Vibration Impact Assessment* (FRA guidance manual) (FRA 2012). The manual includes prediction methods, assessment procedures, and impact criteria for noise and vibration.

The FRA noise impact criteria are based on maintaining a noise environment considered acceptable for land uses where noise may have an impact. Land use also factors into determining an impact; while impacts on industrial uses are not considered, places where people sleep or where quiet is an integral component of the land use require evaluation to determine if noise impacts would occur and if mitigation is appropriate. Noise criteria have been established for the various types of receptors individually because not all receptors have the same noise sensitivity. Table 3-1 summarizes the three land-use categories used by the FRA.

Table 3-1. Federal Railroad Administration Land Use Categories for Noise Exposure

Land Use Category	Descriptor	Description
1	Outdoor Leq(h)	Tracts of land where quiet is an essential element in their intended purpose. This category includes lands set aside for serenity and quiet, and such land uses as outdoor amphitheaters and concert pavilions, as well as national historic landmarks with significant outdoor use. Also included are recording studios and concert halls.
2	Outdoor Ldn	Residences and buildings where people normally sleep. This category includes homes, hospitals, and hotels where a nighttime sensitivity to noise is assumed to be of utmost importance.
3	Outdoor Leq(h)	Institutional land uses with primarily daytime and evening use. This category includes schools, libraries, theaters, and churches, where it is important to avoid interference with such activities as speech, meditation, and concentration on reading material. Places for meditation or study associated with cemeteries, monuments, and museums can be considered to be in this category. Certain historical sites, parks, campgrounds, and recreational facilities are also included.

Source: FRA 2012

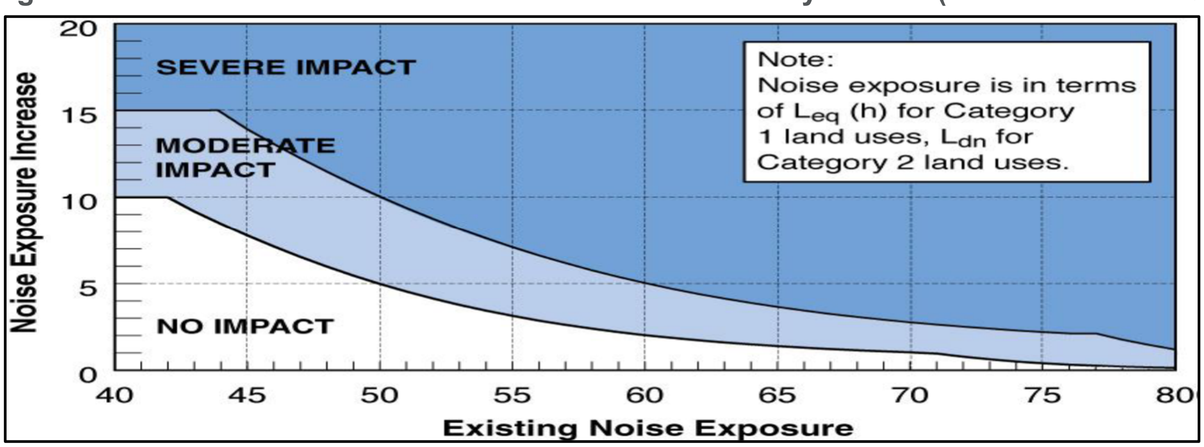
FRA noise impact criteria for human annoyance are based on the comparison of existing outdoor noise levels and future outdoor noise levels from the project. The FRA noise impact criteria specify a comparison of future with existing noise levels because comparison of a projection with an existing condition is more reflective of an impact than a comparison of Build and No-Build Alternatives. Noise level increases are categorized as no impact, moderate impact, or severe impact. Moderate and severe impacts are defined as follows:

- **Moderate impact**—The change in noise level is noticeable to most people but may not be sufficient to cause strong, adverse reactions from the community. Project-specific factors would be considered to determine the magnitude of impact and the need for mitigation, including the number of affected noise-sensitive sites, the existing level of noise exposure, and the costs associated with mitigation.
- **Severe impact**—Project-generated noise in the severe impact range can be expected to cause a substantial percentage of people to be highly annoyed by the new noise levels. It is FRA policy that noise mitigation be implemented for sensitive receptors experiencing severe impacts unless there are truly extenuating circumstances that prevent implementation.

In addition, the FRA criteria are presented in terms of relative levels for evaluating the total future noise exposure increases, or increases in combined noise exposure, from the project alternatives. If the existing noise were dominated by a source that changed because of the project, it would be incorrect to add the project noise to the existing noise. This project proposes to alter transit operations in an existing corridor (i.e. shifts the location or profile of existing passenger or freight tracks and changes the vehicle technology. Therefore, the cumulative assessment method is used for the proposed project because it allows for the relative form of the noise criteria to be used. Figure 3-1 illustrates the relative form of the criteria as they apply to Category 1 and 2 land uses. These criteria are based on the increase in the existing ambient noise level associated with project operations. These criteria are applied to the outside of building locations at noise-sensitive areas.

Consider a hypothetical residential property (Category 2) that has an existing noise exposure of Ldn 60 dBA. The noise exposure resulting from the project plus regional growth and other planned projects could result in a project noise level exposure of Ldn 65 dBA. Combining the project noise with the existing noise level would result in a total combined noise exposure of Ldn 66 dBA or a potential increase of 6 dBA over the existing noise level. Using Figure 3-1, one would start with the horizontal axis at 60 dBA for the existing condition to draw a vertical line, then draw a horizontal line from 6 dBA on the left-hand axis. The intersection of these two lines determines the severity of impact. In this hypothetical example, the intersection of these two lines would fall in the severe impact range.

Figure 3-1. Increase in Cumulative Noise Levels Allowed by Criteria (Land-Use Cat. 1 & 2)



Source: FRA 2012

The FRA criteria for environmental impact from ground-borne vibration are based on the maximum levels for a single event (i.e., one train passby). The criteria also assume that, for any given vibration level, the potential for annoyance increases as the number of events increases. To account for this, the criteria distinguishes between projects with frequent, occasional, and infrequent events, where frequent events are defined as more than 70 events per day, occasional events are defined as 30 to 70 events per day, and infrequent events are defined as fewer than 30 events per day. The vibration criteria depend on land use type. For the three primarily land use categories, the associated criteria are provided in Table 3-2. It is noted that the guidance manual also provides additional criteria for more specific such as concert halls, TV studios, recording studios, auditoriums, and theaters; these criteria are not presented in this report as such land uses were not identified in the project vicinity.

Table 3-2. Ground-Borne Vibration and Ground-Borne Noise Impact Criteria

Land Use Category	GBV Impact Levels (VdB re 1 µin/sec)			GBN Impact Levels (dB re 20 µPascal)		
	Frequent Events[1]	Occasional Events[2]	Infrequent Events[3]	Frequent Events[10]	Occasional Events[2]	Infrequent Events[3]
Category 1: Buildings where vibration would interfere with interior operations	65 VdB[4]	65 VdB[4]	65 VdB[4]	N/A[5]	N/A[5]	N/A[5]
Category 2: Residences and buildings where people normally sleep.	72 VdB	75 VdB	80 VdB	35 dBA	38 dBA	43 dBA
Category 3: Institutional land uses with primarily daytime use.	75 VdB	78 VdB	83 VdB	40 dBA	43 dBA	48 dBA

Source: FRA 2012
Notes:
GBN = Ground-Borne Noise, GBV = Ground-Borne Vibration
1. "Frequent Events" is defined as more than 70 vibration events of the same source per day. Most rapid transit projects fall into this category.
2. “Occasional Events” is defined as between 30 and 70 vibration events of the same source per day. Most commuter trunk lines have this many operations.
3. "Infrequent Events" is defined as fewer than 30 vibration events of the same kind per day. This category includes most commuter rail branch lines.
4. This criterion limit is based on levels that are acceptable for most moderately sensitive equipment such as optical microscopes. Vibration-sensitive manufacturing or research will require a detailed evaluation to define the acceptable vibration levels. Ensuring lower vibration levels in a building often requires special design of the heating, ventilation, and air conditioning (HVAC) systems and stiffened floors.
5. Vibration-sensitive equipment is generally not sensitive to ground-borne noise.

3.1.2 Railroad Noise Emission Compliance Regulations (49 C.F.R. Part 210)

FRA’s *Railroad Noise Emission Compliance Regulations* (49 C.F.R. Part 210) prescribe minimum compliance regulations for the enforcement of *Noise Emission Standards for Transportation Equipment; Interstate Rail Carriers* (40 C.F.R. Part 201) adopted by the U.S. Environmental Protection Agency (USEPA). Selection of new locomotives for the project must meet the following noise standards: 70 dBA at 100 feet while stationary at idle throttle setting, 87 dBA at 100 feet while stationary at all other throttle settings, and 90 dBA at 100 feet while moving. Rail cars must meet the following noise standards: 88 dBA while moving at speeds of 45 mph or less and 93 dBA at 100 feet while moving at speeds faster than 45 mph.

3.1.3 Locomotive Horn Rule (49 C.F.R. Part 222 and Part 229)

FRA regulations require that engineers sound their locomotive horns while approaching public grade crossings until the lead locomotive fully occupies the crossing. In general, the regulations require locomotive engineers to begin to sound the train horn a minimum of 15 seconds and a maximum of 20 seconds in advance of public grade crossings. Engineers must also sound the train horn in a standardized pattern of two long, one short, and one long blast, and the horn must continue to sound until the lead locomotive or train car occupies the grade crossing. Additionally, the minimum sound level for the locomotive horn is 96 dBA, while the maximum sound level (L_{max}) is 110 dBA, both measured at 100 feet forward of the locomotive.

FRA allows public authorities to establish a Quiet Zone, which is a segment of a rail line, within which is situated one or more consecutive public highway-rail grade crossings at which locomotive horns are not routinely sounded, provided sufficient Supplemental Safety Measures are implemented at the crossing to minimize the potential for accidents to occur. Railroad authorities and railroad companies cannot establish Quiet Zones; only local cities and counties can establish them by applying to FRA.

At a minimum, new quiet zones must be at least 0.5 miles in length and contain at least one public grade crossing (i.e., a location where a public highway, road, or street crosses one or more railroad tracks at grade). Every public grade crossing in a Quiet Zone must be equipped at a minimum with active grade crossing warning devices consisting of flashing lights and gates. FRA provides this safety requirement as a minimum recommendation; however, additional safety requirements may include but are not limited to stationary audible warning signals and median barriers.

3.1.4 Summary of Noise and Vibration Criteria

FRA has individual standards that regulate the main noise sources from railroads (operation of train equipment and horn blowing activities) which can cause human annoyance. The locomotive and rail car noise emission standards assist equipment manufacturers in incorporating noise-damping features within the trainset design to reduce noise levels. Due to safety risks near the at-grade crossing, horns are required to sound, which can cause frequent and annoying levels of noise. However, FRA affords public authorities the opportunity to establish “Quiet Zones” and provides design criteria for their application.

FRA also establishes noise impact thresholds at sensitive receiving properties that are based on existing outdoor noise levels and the future outdoor noise levels from a proposed high-speed rail project. They incorporate both absolute criteria, which consider activity interference caused by the high-speed rail project alone, and relative criteria, which consider annoyance resulting from the change in the noise environment caused by the project. Further, the criteria account for heightened community annoyance caused by late-night or early-morning train operations and the varying sensitivities of communities to projects under different background noise conditions. These standards limit the magnitude of increase in noise levels in areas that have high existing noise levels. Therefore, the proposed project alternatives are limited by the allowable incremental increase, which encourages mitigation to reduce project noise levels.

FRA vibration impact thresholds are based on the number of “events” or train passbys. A typical train passby’s duration is short (less than 10s); however, as the number of events increases, vibration levels are expected to increase. FRA has established standards for each event frequency category (infrequent, occasional, and frequent) by land-use type.

4 Basis for Assessing Noise & Vibration Impacts

The FRA *High-Speed Ground Transportation Noise and Vibration Impact Assessment* manual provides guidance and procedures for the assessment of potential noise and vibration impacts resulting from proposed high-speed ground transportation (HSGT) projects based on a "source-path-receiver" framework. The sources of noise from high-speed trains are influenced by a train passby and its operating characteristics (e.g., speed). The path component, which includes aspects such as sound attenuation with increasing distance from the source, excess attenuation as a result of atmospheric absorption and ground effects, and acoustic shielding by terrain, sound barriers, or intervening buildings, influences the level of noise carried to the receiver. The receptor is the noise-sensitive land use (e.g., residence, hospital, or school, referred to as sensitive receptors) exposed to noise from the source. Noise sensitivity is determined by the receiving land use. These three elements will be taken into account in developing criteria that influence noise and vibration levels.

4.1 Noise

4.1.1 Factors Related to Noise Source

One of the main dominating noise sources from high-speed transit projects are trainsets. FRA has determined that the amount of noise generated from trainset is a function of its travel speed. The FRA guidance has grouped train types into three categories: steel-wheeled electric-powered, steel-wheeled fossil fuel-powered, or maglev. Speed regimes have been developed for each type. For each train type and speed regime, a source reference level (SEL) for a typical train passby is given under reference operating conditions. This SEL differs depending on the type and speed of the high-speed vehicle chosen for the project.

Once the SEL has been selected for a particular train type based on speed, further adjustments are made for project-specific operating conditions, as listed below.

- number of train passbys during daytime hours (defined as 7 a.m.–10 p.m.) and nighttime hours (defined as 10 p.m.–7 a.m.),
- maximum number of train passbys (usually the peak-hour train volume),
- number and unit length of locomotives (power cars) and passenger coaches per train,
- speed (maximum expected)
- guideway configuration

After adjusting the SEL level to the project's operating condition, noise exposure is expressed in terms of Ldn or Leq(h).

4.1.2 Factors Related to Noise Pathway

The resultant noise exposure level is further adjusted if there are any vertical terrain effects, such as embankments and trenches or shielding attenuation from rows of buildings between the tracks and the receiver.

4.1.3 Factors Related to Noise Receiver

When evaluating what factors will influence noise levels, receivers must be identified to understand the relationship between the noise source, the pathway to the receiver location, the type of receiver affected, and its location relative to the noise source. Further, the existing noise environment at the receiver locations determines the allowable increment of increase in noise levels that is permissible after project noise levels are calculated. FRA has established screening distances for land use types that would be affected by train passbys. The purpose of establishing screening distances is to define the area where land uses would experience the greatest noise impact. These screening distances identify land uses that experience a direct effect from changes in noise levels from the project. Table 4-1 list the screening distances for noise for train speeds for the project alternatives.

Table 4-1. Recommended Screening Distances for High-Speed Rail Noise Impacts

Corridor Type	Existing Noise Environment	Screening Distance for Project Type and Speed Regime (feet from centerline)2
		90 to 170 miles per hour
Railroad	Urban/noisy suburban—unobstructed	300
	Urban/noisy suburban—intervening buildings	200
	Quiet suburban	500
New Rail	Urban/noisy suburban—unobstructed	350
	Urban/noisy suburban—intervening buildings	250
	Quiet suburban	600

Source: FRA 2012

4.2 Vibration

Vibration levels from high-speed trains are also affected by the source-path-receiver framework; however, the parameters differ from noise. FRA provides a summary of key parameters that have the greatest influence on ground-borne vibration levels. The important physical parameters can be divided into the following four categories: operational and vehicle factors, guideway, geology, and the receiving building.

4.2.1 Factors Related to Vibration Source

The sources of vibration from high-speed trains are influenced by operational and guideway conditions. Table 4-2 provides specific source parameters that influence ground-borne vibration levels. Operational and vehicle factors include all of the parameters that relate to rail vehicles and the operation of trains. Factors such as high speed, stiff primary suspensions on the vehicle, and flat or worn wheels will increase the possibility of ground-borne vibration problems. The type and condition of the rails, the type of guideway, the rail support system, and the mass and stiffness of the guideway structure can also influence the level of ground-borne vibration. Worn rail and wheel impacts at special track work can substantially increase ground-borne vibration. A high-speed rail system guideway will be in a tunnel, in an open trench, at-grade, or on an aerial viaduct. Directly radiated airborne noise is usually the dominant problem from guideways at-grade or in cut, although ground-borne noise can sometimes be a problem. The ground-borne vibration from trains in tunnels tends to be of a higher frequency than the vibration from at-grade track, and higher frequencies make the ground-borne noise from tunnels more noticeable in nearby buildings.

Table 4-2. Factors Related to Vibration Source

Factors	Influence
Operational	
Vehicle Suspension	If the suspension is stiff in the vertical direction, the effective vibration forces will be higher. On transit cars, only the primary suspension affects the vibration levels, the secondary suspension that supports the car body has no apparent effect. Similar effects are likely to occur with high-speed trainsets.
Wheel Condition	Wheel roughness and flat spots are the major cause of vibration from steel-wheel/steel-rail train systems.
Speed	As intuitively expected, higher speeds result in higher vibration levels. Doubling speed usually results in vibration levels 4–6 dB higher.
Guideway	
Track Surface	Rough track is often the cause of vibration problems. Maintaining a smooth track surface will reduce vibration levels.
Track Support System	On rail systems, the track support system is one of the major components in determining the levels of ground-borne vibration. The highest vibration levels are created by a track that is rigidly attached to a concrete trackbed. The vibration levels are much lower when special vibration control track systems such as resilient fasteners, ballast mats, and floating slabs are used.
Track Structure	The general rule of thumb is that the heavier the track structure, the lower the vibration levels. The vibration levels from a lightweight bored tunnel will usually be higher than from a poured concrete box tunnel.
Depth of Vibration Source	There are significant differences in the vibration characteristics when the source is underground compared to at the ground surface.

Source: FRA 2012

4.2.2 Factors Related to Vibration Pathway

Ground-borne vibration is influenced by soil elasticity and stiffness. Ground-borne vibrations are associated with different types of elastic waves propagating through the ground**.** Ground-borne vibration consists of surface waves, mostly Rayleigh waves, bulk longitudinal waves, and transverse waves (or shear waves) propagating into the ground depth. Table 4-3 lists five types of soil factors that influence the vibration propagation path. Among the most important factors that influence vibration through the soil are the stiffness and internal damping of the soil and the depth to bedrock. Experience has shown that vibration propagation is more efficient in stiff clay soils as well as areas with shallow bedrock; the latter condition seems to channel or concentrate the vibration energy close to the surface, resulting in ground-borne vibration problems at large distances from the track. Factors such as layering of the soil and depth to the water table can also have significant effects on the propagation of ground-borne vibration.

Table 4-3. Factors Related to Vibration Path

Factors	Influence
Soil Type	It is generally expected that vibration levels will be higher in stiff clay type soils than in loose sandy soils. Vibration waves propagate efficiently in sandy soils, but evidence to date cannot be expressed with a definite relationship.
Rock Layers	Vibration levels often seem to be high near at-grade track when the depth to bedrock is 30 ft or less.
Soil Layering	Soil layering will have a substantial but unpredictable effect on the vibration levels because each stratum can have significantly different dynamic characteristics.
Depth-to-Water Table	The presence of the water table is often expected to have a significant effect on ground-borne vibration, but evidence to date cannot be expressed with a definite relationship.
Frost Depth	There is some indication that vibration propagation is less efficient when the ground is frozen because water does not distribute vibration evenly.

4.2.3 Factors Related to Vibration Receiver

The FRA has established screening distances for land use types that would be affected by train passbys. Table 4-4 presents these screening distances by land-use categories considered to be vibration-sensitive by FRA.

Table 4-4. Recommended Screening Distances for Vibration Assessments

Land-use Type	Train Frequency	Screening Distance (feet from centerline) Train Speed	
		Less than 100 mph	100 to 200 mph
Residential	Frequent	120	220
	Infrequent	60	100
Institutional	Frequent	100	160
	Infrequent	20	70

Source: FRA 2012

Ground-borne vibration problems occur almost exclusively inside buildings. Therefore, the characteristics of the receiving building are a key component in the evaluation of ground-borne vibration. The train vibration may be perceptible to people who are outdoors, but it is very rare for outdoor vibration to cause complaints. Table 4-5 provides the three parameters that influence how vibration affects a building façade. The vibration levels inside a building depend on the vibration energy that reaches the building foundation, the coupling of the building foundation to the soil, and the propagation of the vibration through the building structure. The general guideline is that the more mass a building has, the lower its response to incident vibration energy in the ground.

Table 4-5. Factors Related to Vibration Receiver

Factors	Influence
Foundation Type	The general rule of thumb is that the heavier the building foundation, the greater the coupling loss as the vibration propagates from the ground into the building.
Building Construction	Because ground-borne vibration and noise almost always are evaluated in terms of indoor receivers, the propagation of the vibration through the building must be considered. Each building has different characteristics relative to structure-borne vibration, although the general rule of thumb is that the more massive a building is, the lower the levels of ground-borne vibration will be.
Acoustical Absorption	The amount of acoustical absorption in the receiver room affects the levels of ground-borne noise.

5 Noise & Vibration Design Criteria Considerations

The conceptual engineering study will consider the following significant changes to the project corridor: curve straightening, new track work (FRA Class 6 track), special trackwork and crossovers, vehicle selection of high-speed trains operating at greater than 90 mph, and increased capacity, which may result in increases in the number of trains operating. These changes were reviewed at a cursory level utilizing the source-path-receiver factors discussed in section 4.0 to determine potential effects on noise and vibration levels.

5.1 Cursory Existing Conditions Review

Reviewing the existing corridor and previous studies provides insights on how train passbys affect existing noise and vibration levels at sensitive land uses. The train passbys have the greatest effects on land uses within 300 feet, as defined by FRA screening distance parameters. The majority of the land uses have an unobstructed view of train passbys. Therefore, the existing environment does not provide significant reductions in noise and vibration through pathway shielding or attenuation effects. Land uses along the corridor reside primarily in an urban setting with some natural habitat. Existing noise levels are dominated by current train passbys and freeway and local roadway traffic. Based on previous studies conducted within the corridor, Ldn noise levels range from the mid-60s to the low 70s (Entech Consulting Group 2019, 2020). These existing noise levels indicate that the allowable incremental increase in noise is small. Further, vibration levels tend to trend higher near crossovers and within sections of the rail corridor where land uses reside along narrow sections of the right of way.

Pending the selection of alternatives for consideration as part of the conceptual design, the adjacent noise sensitive receivers will be identified consistent with FRA's Guidance. This may include the collection of ambient noise and/or vibration information.

5.2 Track and Signal Design Improvements that Increase Noise & Vibration Levels

Improvements proposed within the corridor that facilitate higher operating speeds will have the greatest influence on noise and vibration generation. Table 5-1 provides a summary of the potential design changes that would occur along the corridor and the effect these changes would have on existing noise and vibration levels.

Table 5-1. Influence of Potential Design Changes on Noise and Vibration Levels

Potential Design Changes Along the Corridor	Influence on Noise & Ground-borne Vibration Levels
Curve Straightening	Tracks might be brought closer to sensitive receivers resulting in higher noise, and vibration levels at land uses within the FRA screening distances.
Special trackwork/crossovers	Wheel impacts at special trackwork with standard frogs will increase vibration levels.
High-Speed Vehicle Selection	Resilient wheels increase ground-borne vibration at frequencies greater than 80 Hz.
Class 6 track	Installation of class 6 tracks allow for higher train speeds. Increased train speeds result in higher noise and vibration levels.
Operational Conditions	Increases in capacity for the rail corridor will increase the number of trains operating at higher speeds.

Noise and vibration levels will have the potential to increase based on the following reasons:

- Class 6 tracks will increase speeds and capacity. Higher speeds and an increased number of train passbys will increase noise and vibration levels proportionally. Areas along the corridor that currently have high noise levels that are at or near FRA noise criteria may require abatement, either on the trainset or the track treatment, the path between the receiver and the track, or a combination thereof.
- Special trackwork and crossovers with standard frogs will increase vibration levels.
- Curve straightening may bring tracks closer to receivers that already experience high noise levels. Several locations along the corridor have been evaluated for double-track improvements, and noise levels are currently within the moderate impact level (Ldn low 70's) at distances less than 200 feet from the nearest track. Depending on the placement of the track, receivers may experience an increase in noise and vibration levels.
- Pathways between the receiver and the noise source may experience changes in terrain effects (embankments, trenches, etc.) that can increase or decrease noise and vibration levels depending upon elevation and type of terrain feature.
- New at-grade crossings and increased service frequency may result in additional horn noise near sensitive land uses.

5.3 Possible Mitigation Measures to Incorporate into the Design

This noise and vibration basis of design report provides the opportunity to incorporate the most cost-effective solutions for both the track and vehicle as a system, because the interaction between the wheel and the rail is responsible for a bulk of wayside noise and vibration impacts. The following areas should be considered in reducing noise and vibration generation as part of the project design:

- Vehicle treatments of trainsets that have noise mitigation built into the design can assist with reducing noise levels. For example, vehicle body design can provide shielding and absorption of the noise generated by the vehicle components. Acoustical absorption under the car has been demonstrated to provide up to 5 dB of mitigation for wheel-rail noise and propulsion-system noise on rapid transit trains (Hanson 1983).
- Selection of trainsets that contain resilient or damped wheels, vehicle shirts, under-car absorption, and a shroud will reduce wayside noise; turning radii greater than 1,000 feet will assist in reducing squeal noise. A typical reduction is 2 dB on tangent track. This treatment is more effective in eliminating wheel squeal in tight curves; reductions of 10–20 dB for high-frequency squeal noise is typical.
- Current locations along the corridor have or will be implementing Quiet Zones. If additional at-grade crossings are needed, it is recommended to work with local cities to implement Quiet Zones. FRA recommends that Quiet Zones are at least 0.5 miles in length and contain at least one public grade crossing.
- Locate track work that has the potential to generate high levels of noise and vibration in geology that has a rock layer greater than 50 feet. This can help reduce vibration energy into the rock. Propagation through rock usually results in a lower vibration than propagation through the soil.
- Incorporate special trackwork such as movable point frogs, ballast mats, and high resilience fasteners to reduce noise and vibration levels along the track.

6 References

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——— 2020. *Eastbrook to Shell Double Track Noise & Vibration Study*.

Federal Railroad Administration (FRA). 2012. *High-Speed Ground Transportation Noise and Vibration Impact Assessment*.

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Appendix M. Tunnel Optimization Memo

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San Dieguito to Sorrento Valley Double Track

Del Mar Tunnels Alternatives Analysis Tunnel Optimization Technical Memorandum

San Diego Regional Rail Corridor Alternative Alignment and Improvements Conceptual Engineering Study

November 2022

Prepared by:

HDR

MOTT MACDONALD

Prepared for:

SANDAG

San Diego Association of Governments
401 B Street Suite 800
San Diego, CA 92101
(619) 699-1900
www.sandag.org

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Appendices

1 Introduction

A joint HDR/Mott MacDonald team is providing alternatives analysis and engineering studies for the proposed Del Mar Tunnels to facilitate improved service on the Los Angeles-San Diego-San Luis Obispo (LOSSAN) Rail Corridor. Under a previous task, the team produced a draft Tunnel Basis of Design Report (TBDR), dated May 2021, which outlined twin bore and single bore tunnel section configurations for the Del Mar Tunnels. The configurations outlined in the TBDR were developed to accommodate the clearance requirements in the reserved freight rail service easement in the Shared Use Agreement (SUA) between the BNSF, North County Transit District (NCTD), and Metropolitan Transit Systems (MTS), and the current San Diego Association of Governments (SANDAG) design criteria. This resulted in an internal diameter of 33 feet for each of the twin bores and an internal diameter of 57 feet for the single bore tunnel.

A single bore internal diameter of 57 feet would require a tunnel boring machine (TBM) of approximately 61 feet in diameter, well in excess of the largest TBM manufactured and successfully implemented to-date. The single bore option was, therefore, not further developed in the draft Del Mar Alternatives Analysis Report dated June 2021.

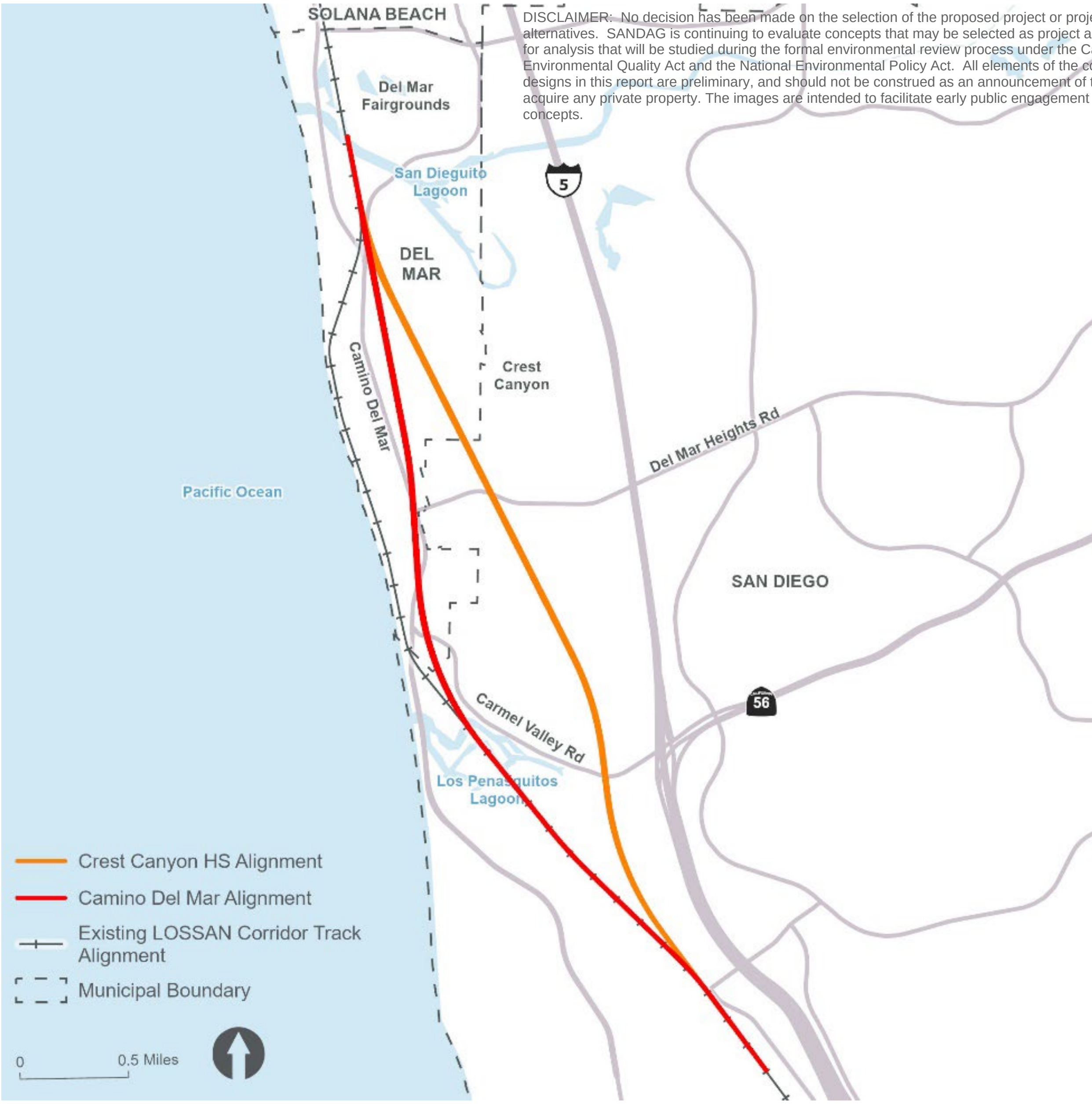
The 33-foot diameter twin bore tunnels would require portal locations and configurations that could result in significant potential impacts to private properties next to the proposed north portal.

After early engagement with the project stakeholders, SANDAG tasked the HDR/Mott MacDonald team with optimizing the tunnel configuration and track profile to minimize the impacts to the community. As part of the optimization, two constraints were removed from the project design: (a) the design team was allowed to deviate from the clearance requirements of the SUA to reduce the tunnel diameter, and (b) the proposed track bed was allowed to be below the level of peak storm events by requiring engineering solutions to protect all new infrastructure.

This memorandum focuses on the benefits of reducing the tunnel diameter and incorporating the refined track profile when locating portal infrastructure.

Figure 1-1 is a schematic of the two alignments selected for further development by the Project Development Team. The yellow line represents the Crest Canyon Higher Speed (HS) alignment, and the red represents the Camino Del Mar alignment.

Figure 1-1. Del Mar Tunnels Conceptual Alignments



1.1 Optimization Study

The purpose of the tunnel optimization study is to:

- Evaluate ways in which the internal diameters of the twin bore and single bore tunnels could be optimized
- Evaluate ways in which the horizontal and vertical tunnel profiles could be optimized to reduce the project footprint and associated concept design for the portal locations and associated facilities
- Provide an indicative concept design for the portal structures and facilities at the north and south end of the tunnels

- Adjust the construction staging area requirements and evaluate other construction considerations
- Based on the conceptual design, preliminarily assess the potential impacts to private properties associated with both the optimized twin bore and single bore concepts
- Compare the preliminary estimate of the cost of the optimized single and twin bore options against the preliminary estimate of the cost of the original twin bore option

This memorandum summarizes the results of the study for the purpose of determining the tunnel configurations to be carried forward into the next phase of the project. Conceptual level plans showing portal arrangements, construction staging and cross sections for the single bore and twin bore tunnel configurations have been included as Appendix A in this memo.

1.2 Tunnel Optimization Benefits

Optimizing the track profiles and reducing the tunnel diameters will benefit the project in several ways:

- Reducing the tunnel diameters would result in a reduction in schedule, cost, and project impacts.
- Reducing the diameter of the single bore TBM would bring the single bore option into the realm of proven TBM diameter sizes.
- Reducing the tunnel diameters for both configurations would result in a smaller carbon footprint for the tunneling component of the project. The reduced carbon footprint is due to the reduction of tunnel spoils to be hauled off site and the reduction of concrete and steel needed for the construction of permanent structure of the tunnels.
- Reducing the tunnel diameters for both configurations would result in a smaller width, depth, and length of portal structures.
- Reducing the tunnel diameter and optimizing the tunnel alignment would reduce potential impacts to private properties.

2 Reduced Tunnel Diameter

This section provides details regarding the original clearance envelope assumptions (May 2021), the current clearance assumptions, and the resulting reduced tunnel diameters for the single- and twin bore configurations.

2.1 Original Clearance Assumptions

Four clearance envelopes were used to space-proof the internal diameters for the twin bore and single bore tunnels in the TBDR. These were:

- Reserved Freight Rail Service Easement from the SUA & LOSSAN Engineering Standard Drawing ESD-2101 (SUA & LOSSAN Criteria)
- NFPA emergency walkway clearance envelope
- Composite car equipment envelope from Metrolink Standard Drawing ES2013
- CPUC G.O. 26-D Clearance Line for Tunnels

The SUA & LOSSAN Criteria envelope (represented by the red envelope line on) accounts for the Reserved Freight Rail Service Easement required as part of the Grant Deed in the SUA and the clearance requirements provided in the LOSSAN Engineering Standard Drawings. The vertical clearance of 26 feet is required to accommodate future electrification of the corridor.

The National Fire Protection Association (NFPA) Emergency Walkway envelope (represented by the pink envelope line on) accounts for the emergency egress corridor.

The composite car equipment envelope (represented by the green envelope line on) accounts for a composite of the various equipment dynamic envelopes and AAR plates anticipated on the corridor, including double-stack containers and bi-level and tri-level carriers.

The California Public Utilities Commission (CPUC) General Order (G.O.) 26-D Clearance Line for Tunnels (represented by the orange envelope line on) is a California State requirement for train tunnels.

The clearance envelope for the tunnel diameter governed by SUA & LOSSAN Criteria. This resulted in a minimum required internal diameter of 33 feet for each of the twin bore tunnels and 57 feet for the single bore tunnel. Figure 2-1 shows these clearance envelopes for the twin bore configuration, and Figure 2-2 shows the clearance envelopes for the single bore configuration.

Figure 2-1. Clearance Envelopes for the May 2021 Twin Bore Configuration

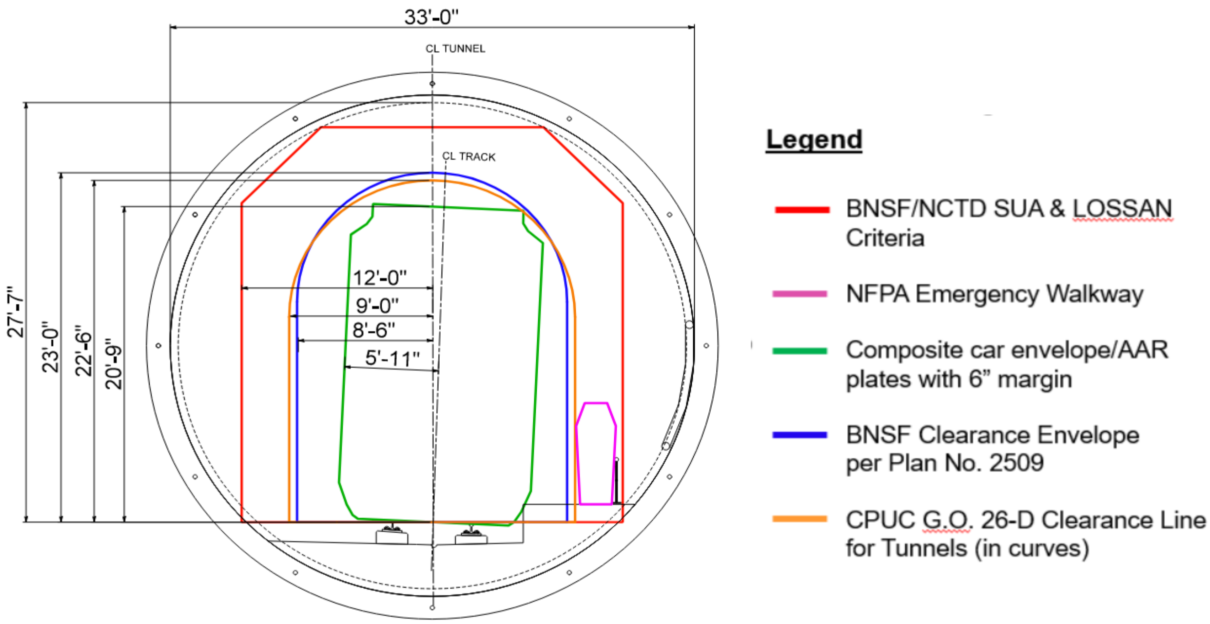
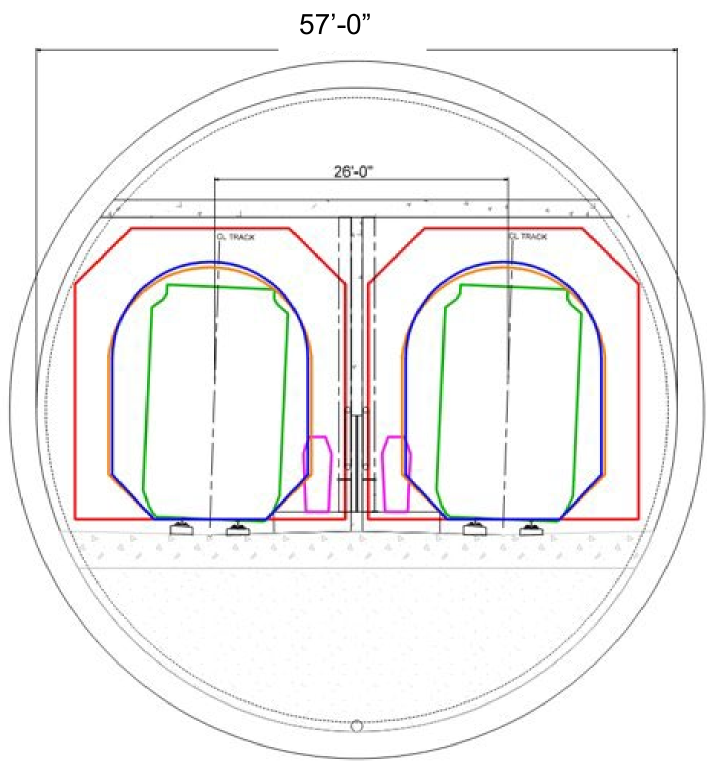


Figure 2-2. Clearance Envelopes for the May 2021 Single Bore Configuration



This single bore configuration (with side-by-side tracks, as depicted in Figure 2-2), with an internal diameter of 57 feet, would require a TBM that is approximately 4 feet larger in diameter than the largest TBM manufactured and used to excavate a tunnel worldwide, which adds a significant risk to the project. The increase in diameter results in only a 7% increase in the current largest TBM size but a 15% increase in the tunnel cross-sectional area, which would significantly increase the torque and power requirements for the TBM. The additional torque and power requirements along with the TBM

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being the largest ever manufactured would unnecessarily increase the risk to the project. In addition, the single bore configuration would result in a 44% greater volume of excavated material compared to the twin bore configuration, generate a significant amount of unused space, and require portal structures that could result in significantly larger impacts to private properties. Due to these factors, the 57-foot single bore configuration was deemed to be infeasible and was not advanced in the June 2021 alternatives analysis report.

Other single bore track configurations that could be considered include stacking one track above the other track or configuring the tracks diagonally within the tunnel. However, neither of these configurations would result in a smaller-diameter tunnel and both would create challenging and lengthy track transitions, making both of these configurations infeasible.

2.2 Current Clearance Assumptions

On August 25, 2022, a workshop was held with SANDAG, NCTD, BNSF, the Railpros program management consultant and the HDR/Mott MacDonald design consultant to discuss how the tunnel diameter size could be reduced to better align with current tunnel design best practices for sizing and to realize the benefits that reducing the tunnel diameter would provide. The design team presented reduced tunnel diameters for both the single bore and twin bore configurations that are based on the greater of the BNSF Clearance Envelope per Plan No. 2509, the CPUC GO 26-D Clearance Line for Tunnels, the composite car envelope and the NFPA emergency walkway envelope (see). It was reiterated in the meeting that overhead electrification of the trains is not planned for the corridor and space proofing of the tunnels should assume use of battery- or hydrogen-operated trains in the future. Any necessary charging infrastructure could be placed at other locations and therefore, should not be considered within the tunnel configuration.

Following the workshop, BNSF indicated that the BNSF clearance envelope should be used as a minimum requirement; therefore, it was determined that the reduced tunnel diameter as presented at the workshop was acceptable to use as a best practice. Because the SUA & LOSSAN Criteria would no longer govern the clearance envelope, the SUA and subsequently the LOSSAN criteria would need to be amended to serve as formal approval of the modified clearances.

The reduced twin bore diameter is controlled by the vertical requirement for the BNSF clearance envelope. Two tunnel cross sections were developed to account for different locations along the alignment, including at tangent track and at 2.75 inches superelevation. This enabled the diameter of each bore to be reduced by 5 feet, to 28 feet, as shown in Figure 2-3. This refined criteria also enabled the single bore tunnel diameter to be reduced by 10 feet, to 47 feet, as shown in Figure 2-4.

In both configurations, the diameter accommodates clearance of the governing envelope plus a 6-inch construction tolerance on the radius of the tunnel lining for the twin bore tunnel and a 10-inch construction tolerance on the radius of the tunnel lining for the single bore tunnel.

The construction tolerances account for:

- TBM steering
- Lining deformation (commonly referred to as ovalization)
- Lipping (minor offset differences in precast ring construction)
- Track construction

Figure 2-3. Clearance Envelopes for the Optimized Twin Bore Configuration

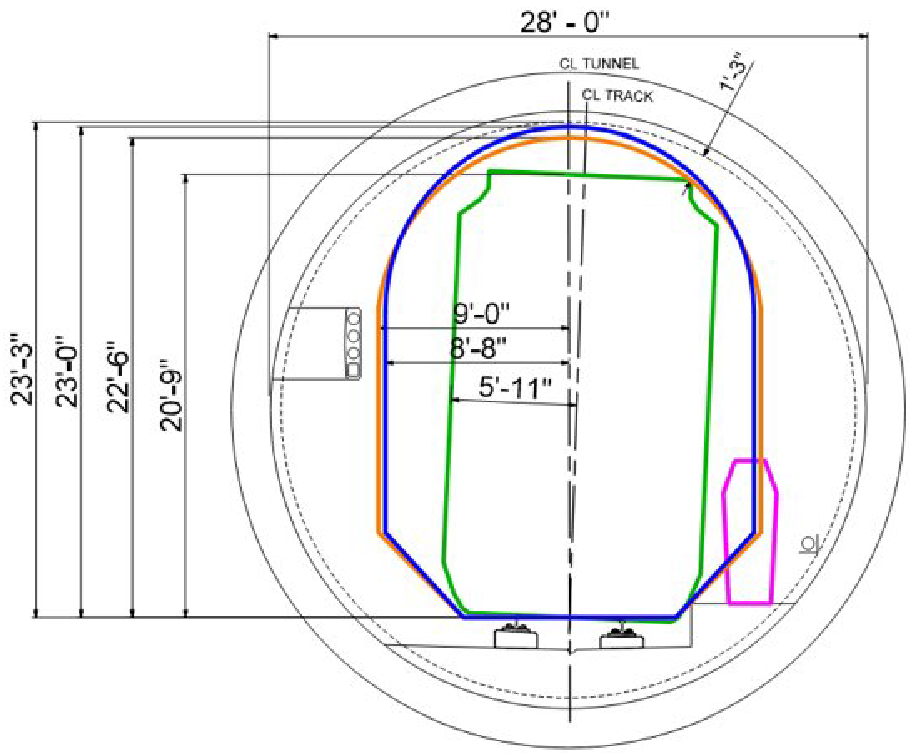
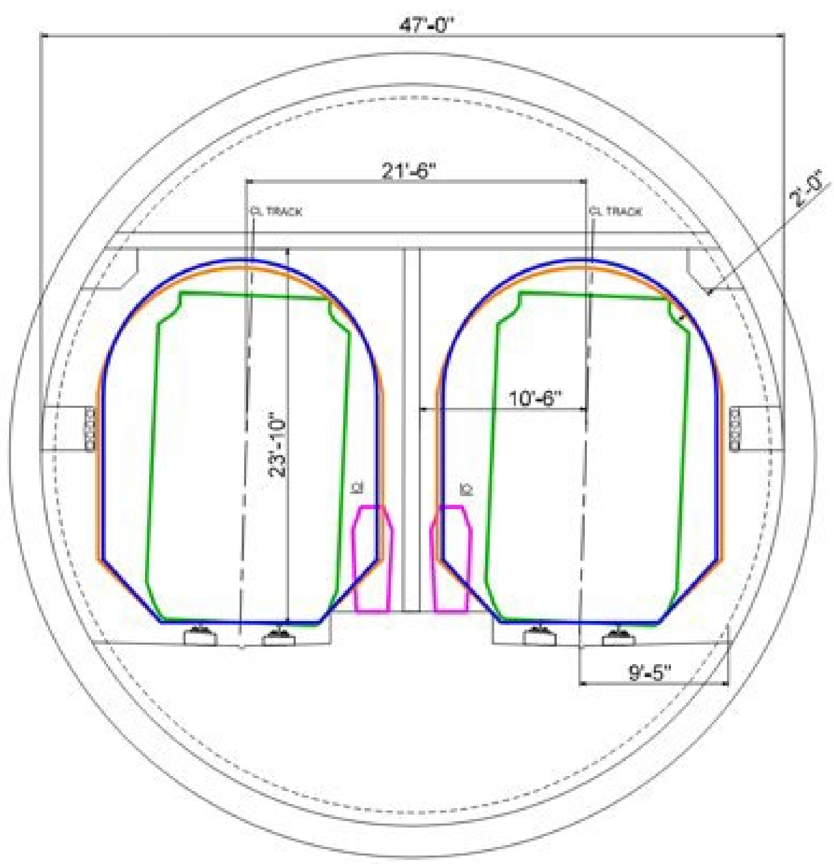


Figure 2-4. Clearance Envelopes for the Optimized Single Bore Configuration



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3 Horizontal and Vertical Profiles

New track alignments and profiles were developed for the single bore configuration, and the track alignments and profiles for the twin bore configuration were updated for both the Crest Canyon HS and Camino Del Mar alternatives. In addition to updating the alignments based on the reduced tunnel diameters, criteria regarding the location of the track subgrade in relation to the 100-year flood event with accounting for sea level rise were also revised. Table 3-1 summarizes the revised key track design criteria and assumptions resulting from the optimization efforts.

Table 3-1. Revised Key Track Design Criteria and Assumptions

Initial Design Criteria/Assumption	Revised Design Criteria/Assumption
Track spacing shall be minimum 70-ft track centers for twin bored tunnels.	Track spacing shall be minimum 56-ft track centers for twin bored tunnels. Track spacing shall be minimum 21.5-ft track centers inside single bored tunnel.
Track subgrade (or track slab invert for direct fixation) elevation within sea-level-rise area of influence shall be above 100-year flood profile accounting for up to 7.1-ft of sea-level-rise.	Along portal structures where the alignment is within the sea-level-rise area of influence, the tracks shall be protected against the 100-year flood profile accounting for up to 7.1 ft of sea-level-rise. The tunnels shall be protected through use of flood walls along u-structures and at portals. These would be configured to provide this protection and would require detailed studies to determine the heights, extents, and any secondary impacts to the areas in which they are located.
Track subgrade at portals shall be above the FEMA 500-year flood elevation.	Protect against the FEMA 500-year flood elevation through the use of floodgates or other mitigation measures.

The revised track subgrade criteria allow the track profiles to be lowered after the alignment crosses the proposed San Dieguito River Bridge. Lowering the track profile and reducing the track centers would provide several benefits described in the following sections.

4 Emergency Operations – Fire Life Safety Considerations

Emergency operations and fire and life safety considerations and concepts remain would largely unchanged from those outlined in the TBDR. The optimization of the twin bore and single bore tunnel diameters would still allow for a “push-pull” ventilation system with fans located at both the north and south portals irrespective of the tunnel configuration finally adopted.

Emergency egress in the twin bore tunnels would be through connecting cross passages at 800-foot spacing that would provide access to the non-incident tunnel. The cross passages would be excavated using small equipment and require local ground treatment, including spiling bars, canopy tubes, or both, in combination with grouting. Ground treatment for the cross-passage excavation could be performed from within the tunnels to avoid construction impacts at the surface.

Emergency egress from the single bore tunnel would require sliding cross-passage doors built into a concrete wall separating the two tracks at 800-foot spacing, to allow for egress to the non-incident guideway in the case of an emergency.

5 Portal Structures

The reduced tunnel diameters would provide an opportunity to reduce the width of the tunnel portal structures. The twin bore tunnels would not only be smaller than those previously considered in the TBDR but would also be closer together, resulting in an overall reduction in the footprint of 13%. Similarly, the reduced single bore tunnel diameter would result in a 13% reduced footprint at the portals.

The revised horizontal alignments and vertical profiles would allow modifications at both ends of the bored tunnels that reduce impacts to private properties. At the current conceptual level of design development and with minimal geotechnical information available, a depth of cover equal to one tunnel diameter was used to determine the depth of the tunnel at portal locations. Therefore, the reduced tunnel diameters would require proportionally less cover at these locations and can be placed farther downslope, thereby further reducing impacts to private properties.

5.1 North Portal Structures

Optimizations, common to both the twin bore and single bore tunnels for both the Crest Canyon HS and Camino Del Mar alignments, would result in the following conditions at the north portal:

i. The vertical profile near the north portal is governed by the constraint to tie into the top-of-rail elevation of the proposed San Dieguito River Bridge. The revised alignments and extents of the cut and cover structure could now allow the raising of Jimmy Durante Boulevard to be a re-grading operation with retaining walls as opposed to the viaduct scheme that would have been required for the previous portal configuration. The cut and cover structure would now extend beyond Jimmy Durante Boulevard with associated raising of Jimmy Durante Blvd above the cut and cover structure, eliminating the potential impacts to private properties and the community that are associated with the viaducts.

ii. Many of the permanent facilities needed for the operation of trains within the tunnels, including tunnel ventilation fans and associated electrical rooms, could be located between the cut and cover box structures or housed above the cut and cover structures. During design development, a detailed plan for access and removal of the equipment placed within the structures should be reviewed with operations and maintenance staff. Ducting of the tunnel ventilation exhaust and supply should be routed in a way to minimize potential impacts to the community. Any permanent facilities that cannot be accommodated in this manner could be housed in a building in an area north of Jimmy Durante Boulevard that incorporates architectural treatments to blend within the community and minimize potential impacts. Other permanent facilities located within this property would include maintenance and emergency vehicle parking and an assembly area.

iii. The area above the cut and cover structure south of Jimmy Durante Boulevard could be restored to match the local native landscape.

iv. Flood walls would be extended near the new San Dieguito River Bridge crossing.

5.2 South Portal Structures

The permanent facilities at the south portal locations were reconfigured to maximize the use of space between and above the cut and cover box structures and minimize the potential impacts to the community. Similar to the plan for the north portal structures, a detailed plan for access and removal of the equipment placed within the structure should be reviewed with operations and maintenance staff during design development.

The south portal configurations for the Crest Canyon HS and Camino Del Mar alignments could make use of the local topography (common to both the twin bore and single bore tunnels), and result in the following improvements:

i. The reconfigured facilities would provide the opportunity to place the permanent facilities, except for maintenance and emergency parking and assembly areas, underground (between and immediately above the cut and cover structure).

ii. Accommodating permanent parking areas and an assembly area would result in minimal impacts.

iii. The potential to restore the area to the existing condition by backfilling the permanent structures and relandscaping would be improved.

6 Construction Staging Areas

At the north end of the tunnel alignment, the construction staging areas for both alternatives could be reduced in size to minimize potential impacts to private properties.

7 Other Construction Considerations

The reduced single bore tunnel diameter could now make the single bore a viable tunnel configuration, based on current TBM technology and recent successful projects.

However, the single bore configuration would produce more tunnel spoils than the twin bore option. Further study is necessary to develop the means to remove the spoils in a manner that would minimize potential impacts to the community and private properties, and to develop concepts for the beneficial reuse of the spoils.

With the reduction of the respective tunnel diameters, the anticipated carbon footprint for each option would be reduced. This is mainly due to the reduced volume of tunnel spoils to be removed from the project and the reduced quantities of steel and concrete needed for the tunnel lining and all associated permanent structures. For example, the reduction of the single bore tunnel diameter would reduce the net volume of excavation by roughly 30 cubic yards per foot of alignment, and the reduction of the twin bore tunnel diameter would reduce the net volume of excavation by 20 cubic yards per foot of alignment. These volumes represent the neat line excavated volumes and would result in a reduction of approximately 68,000 truckloads for the single bore and approximately 42,000 truckloads for the twin bore tunnel, over the approximately two-mile length of tunnel.

8 Tunnel Easements and Property Impacts

Based on the conceptual design and preliminary analysis, the refinements to the twin bore configuration could result in an approximately 77% reduction in potential impacts to private properties for Crest Canyon HS and approximately 70% reduction in potential impacts to private properties for Camino Del Mar. The refinements to the single bore configuration could result in an approximately 81% reduction in potential impacts to private properties for Crest Canyon HS and approximately 45% reduction in potential impacts to private properties for Camino Del Mar.

9 Construction Cost and Schedule

Table 9-1 shows the likely construction costs for the optimized single bore and twin bore tunnels and portal structures as well as the costs for the twin bore configuration prior to the optimization study. The costs and schedules shown are for the tunneling portions of the project only. The relative impacts on the overall project schedule should consider other components of the works.

Table 9-1. Comparison of Construction Costs and Schedule for Tunnels and Portal Structures (in $millions)

Description	Cost (Optimized) *	Cost (Pre-optimization)	Tunneling Schedule**
Crest Canyon HS			
Twin Bore	1,044	1,201	43 – 55 months
Single Bore	1,162	N/A	54 - 66 months
Camino Del Mar			
Twin Bore	804	1,053	37 - 49 months
Single Bore	876	N/A	48 - 60 months

Notes:

*Tunnel costs are in 2022 dollars and include a 20% tunneling contingency, 11% Construction Mobilization, 5.3% for Bonds and Insurance, 20% Time Related Overhead (Per Caltrans) and the 35% Construction Contingency . These costs should be used to compare the single- and twin bore tunnel costs only.

** Schedule is for construction of tunnels and portal structures from NTP for construction to hand over from tunnel contractor to corridor-wide trackwork and rail systems contractor. Early contractor involvement and early TBM design and procurement would allow the lower end of the ranges indicated to be realized. Schedule includes all tunnel work, related facilities, ventilation, and Fire & Life Safety Systems.

10 Evaluation of the Optimized Single bore and Twin bore Tunnel Configurations

In addition to construction cost and schedule, other factors and ways to score a qualitative assessment should be considered when comparing alternatives. For the purposes of this study, Table 10-1 provides a high-level (positive, neutral, negative) evaluation of some of the key factors to consider when comparing the single bore and twin bore options.

Table 10-1. Qualitative Evaluation of Optimized Single Bore and Twin Bore Tunnel Configurations

Single Bore	Feature	Twin Bore
Neutral	Construction Feasibility	Neutral
Negative	Risk Profile	Positive
Neutral	Construction Schedule	Neutral
Neutral	TBM Technology	Positive
Neutral	FLS / Egress	Neutral
Positive	Redundancy / Resilience	Positive
Neutral	Operations & Maintenance (incl. portal facilities)	Neutral
Negative	Spoils Removal	Positive
Positive	Subsurface Easements	Negative
Negative	Properties Affected	Positive
Neutral	Project Delivery Approach	Neutral

Positive
Neutral
Negative

Although Table 10-1 shows the twin bore option with more positives and fewer negatives than the single bore option, there is no discernable difference between the two options on either the Crest Canyon HS or Camino Del Mar alignments at this level of design development to warrant definitively selecting one option over the other, for the following reasons:

i. The size of the TBM for the single bore tunnel could now make it a viable option from a tunneling perspective.

ii. TBM technologies will continue to advance along with TBM availability and contractor experience with this size of TBM tunnel prior to award of a construction contract for the Del Mar Tunnels.

iii. Contracting and delivery methods have not been factored into the evaluation.

iv. The elimination of 18 cross passages needed for a twin bore tunnel configuration.

v. Operational considerations have not been fully factored into the evaluation.

11 Summary

Both the optimized twin bore and single bore tunnels are viable from a tunneling technology and construction viewpoint, and there appears to be no discernable difference between the twin bore and single bore options on either the Crest Canyon HS or Camino Del Mar alignments to definitively select one option over the other at this level of design development.

As the project moves forward into the next phase, the tunnel configuration should not be constrained to either a single bore configuration or a twin bore configuration. This approach will strengthen the environmental assessment process and will not affect the schedule for the delivery of the environmental document. During the next phase, the project delivery team should reevaluate this decision and develop the options to the appropriate level of detail.

At this early stage of the project, a number of factors need to be finalized before the final configuration can be determined; these include construction contracting methodology, final alignment route, potential impacts to private properties, and geotechnical conditions. Even when these factors are known, it may be beneficial to allow the design and contracting community to determine the final tunnel configuration, as long it fulfills all environmental, funding, and technical requirements and demonstrates that potential impacts to the Del Mar communities are minimized to the extent possible.

12 References

Mott MacDonald. May 2021. Tunnel Basis of Design Report. Prepared for San Diego Association of Governments. San Diego, CA.

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Appendix A. Conceptual Exhibits

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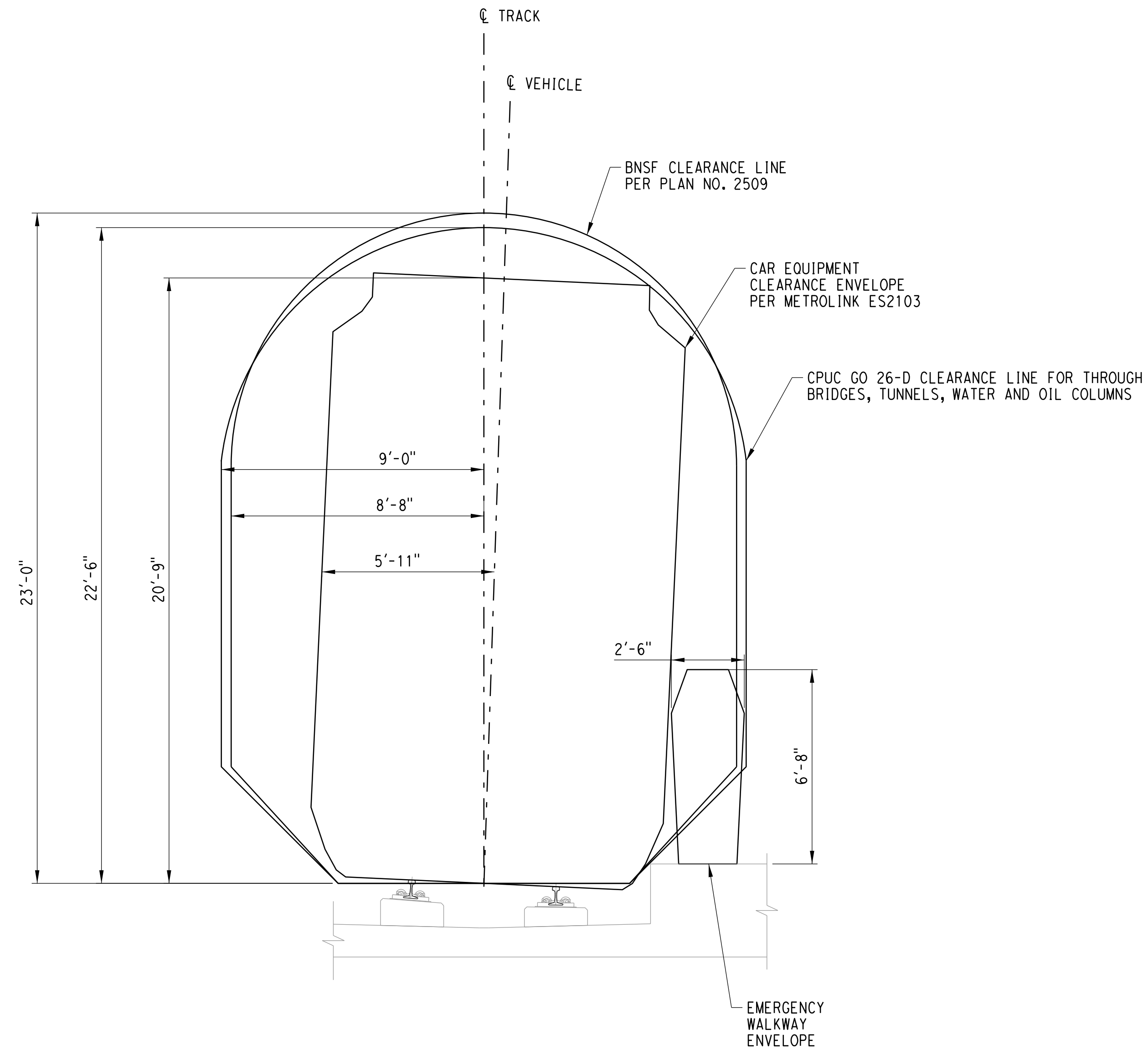

BNSF, CPUC AND VEHICLE CLEARANCE ENVELOPES (WITH 2.75" SUPERELEVATION)

SCALE: 1" = 6'-0"

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DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street,
Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

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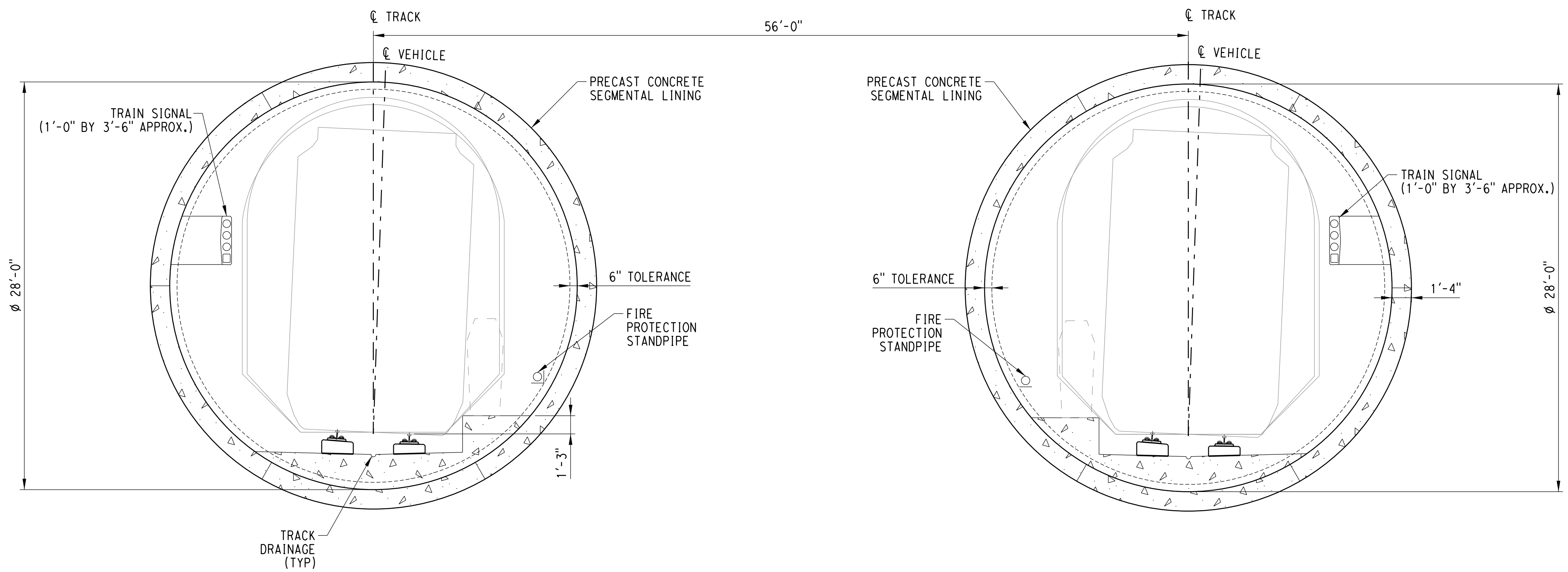
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
BNSF, CPUC AND VEHICLE CLEARANCE ENVELOPES

CONTRACT NO.
DRAWING NO. SS001
REVISION
SHEET NO.
SCALE AS NOTED



TYPICAL TWIN BORE TUNNEL

SCALE 1"=10'-0"

NOTE

1. TUNNEL SPACING AT PORTALS TO BE FURTHER REFINED.
2. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001

5 0 5 10

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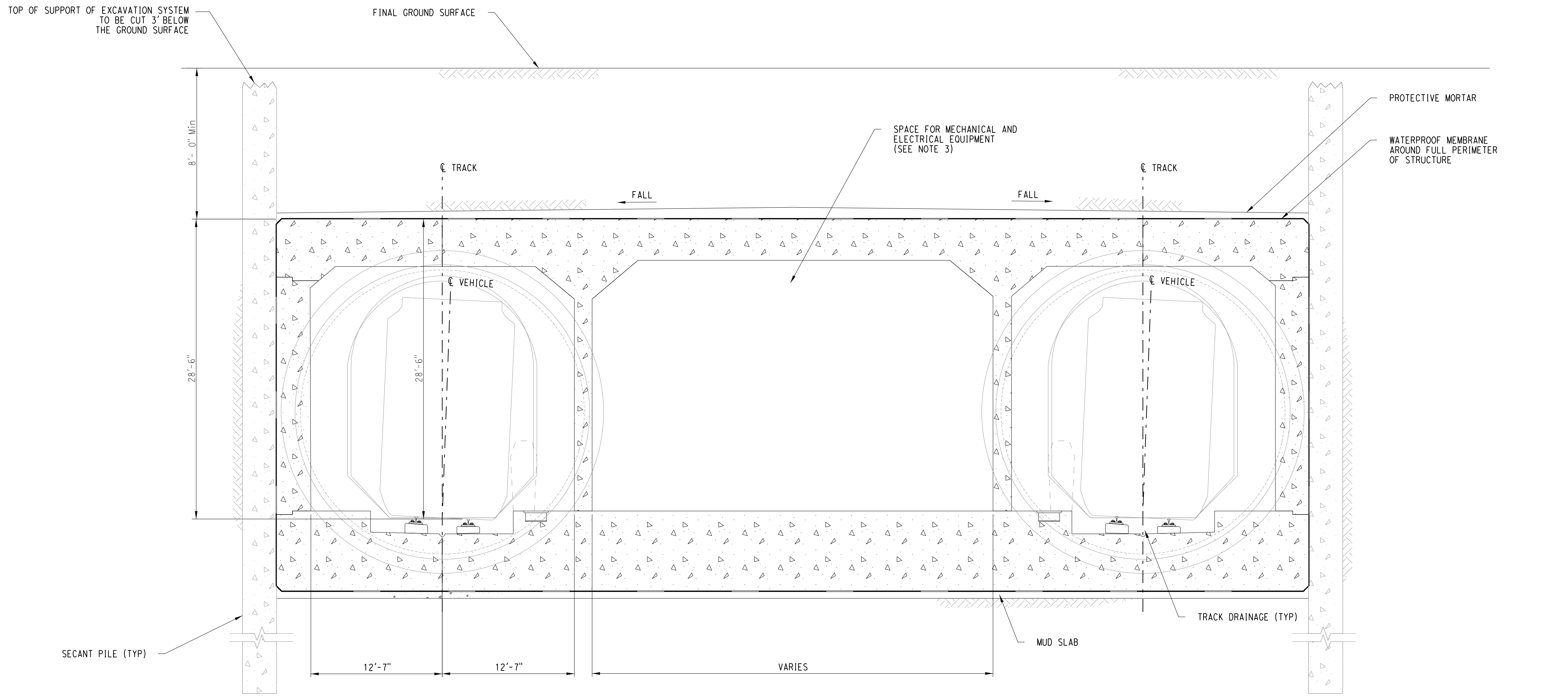
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS

TWIN BORE TUNNELS
TYPICAL SECTION

CONTRACT NO.
DRAWING NO. SS002
REVISION
SHEET NO.
SCALE AS NOTED



CUT & COVER SECTION AT END OF BORED TUNNEL

SCALE: 1" = 10'-0"

NOTES:

1. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001.
2. DISTANCE BETWEEN TRACK CENTERLINE VARIES.

5 0 5 10

SCALE IN FEET

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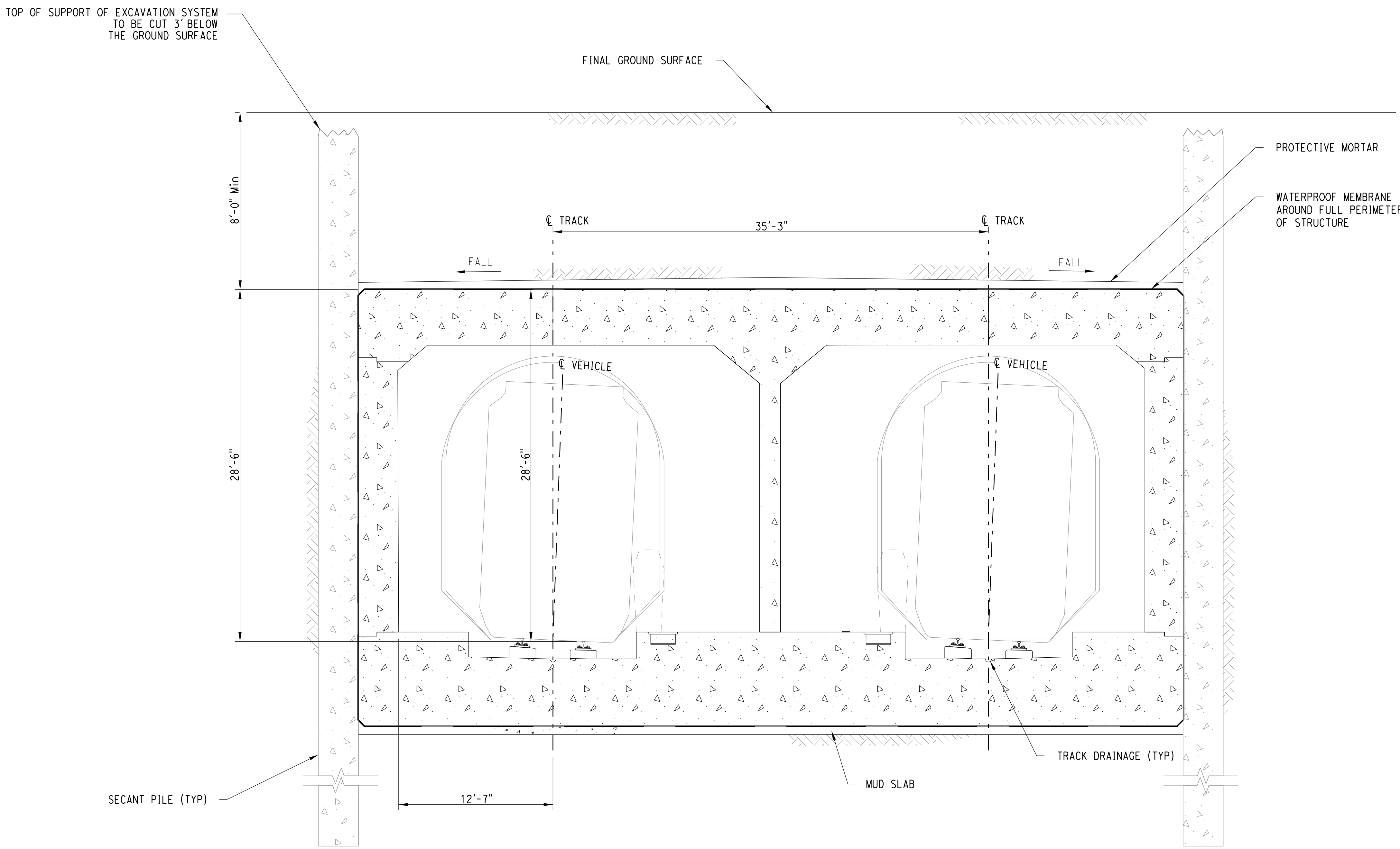
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CUT AND COVER TYPICAL SECTION
SHEET 1 OF 2

CONTRACT NO.
DRAWING NO. SS003
REVISION
SHEET NO.
SCALE AS NOTED

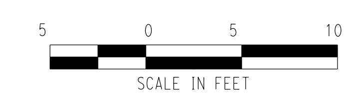


CUT & COVER SECTION AT U-WALL INTERFACE

SCALE: 1" = 10'-0"

NOTES:

1. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001.
2. DISTANCE BETWEEN TRACK CENTERLINE VARIES.



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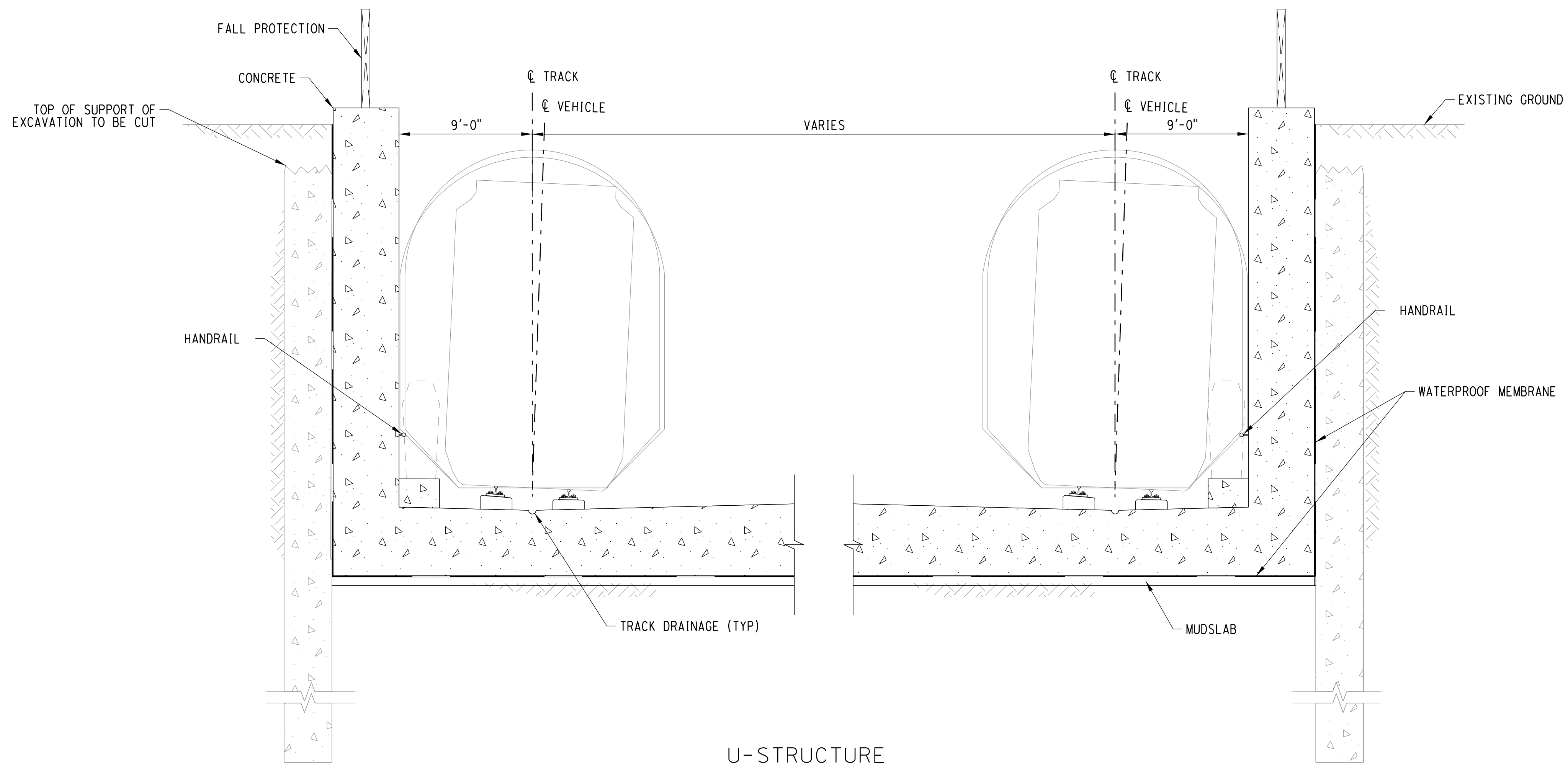
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CUT AND COVER TYPICAL SECTION SHEET 2 OF 2

CONTRACT NO.
DRAWING NO. SS004
REVISION | SHEET NO.
SCALE AS NOTED



U-STRUCTURE

SCALE: 1"=10'-0"

NOTE:

1. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001.

5 0 5 10

SCALE IN FEET

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U-STRUCTURE
TYPICAL SECTION

CONTRACT NO.
DRAWING NO. SS005
REVISION
SHEET NO.
SCALE AS NOTED

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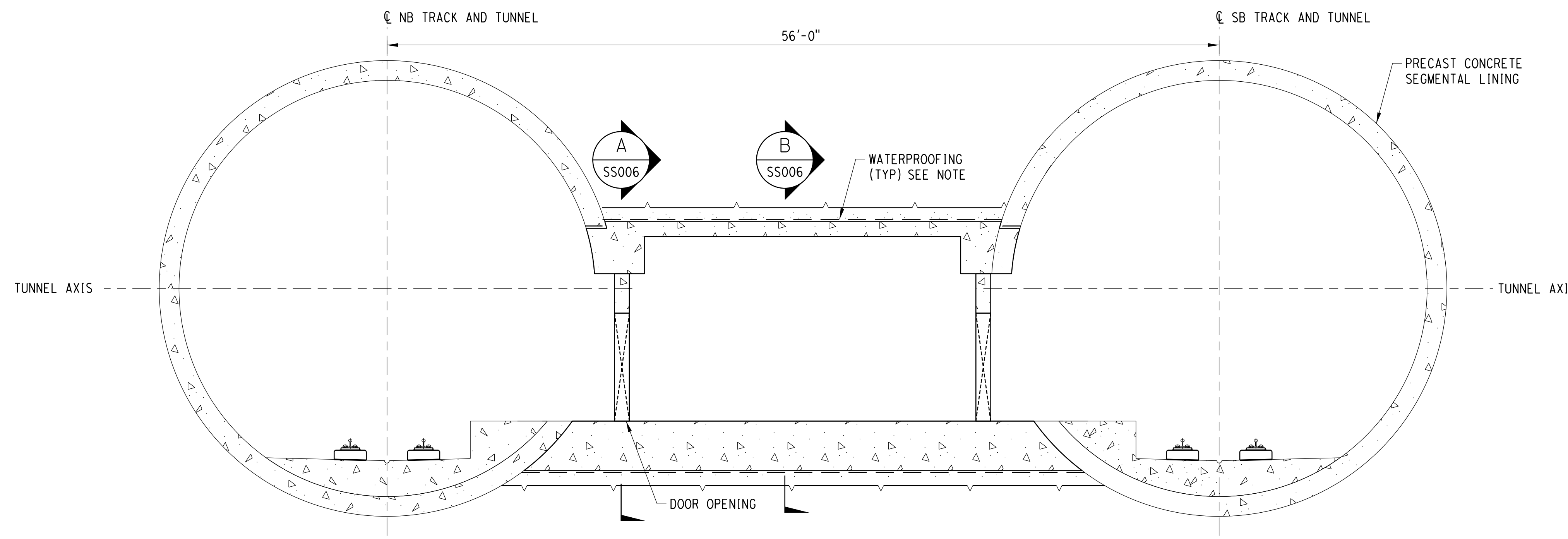
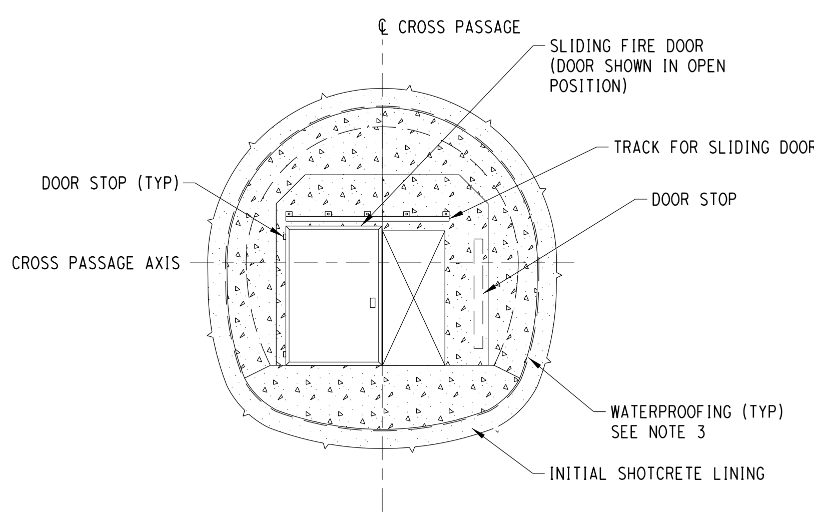


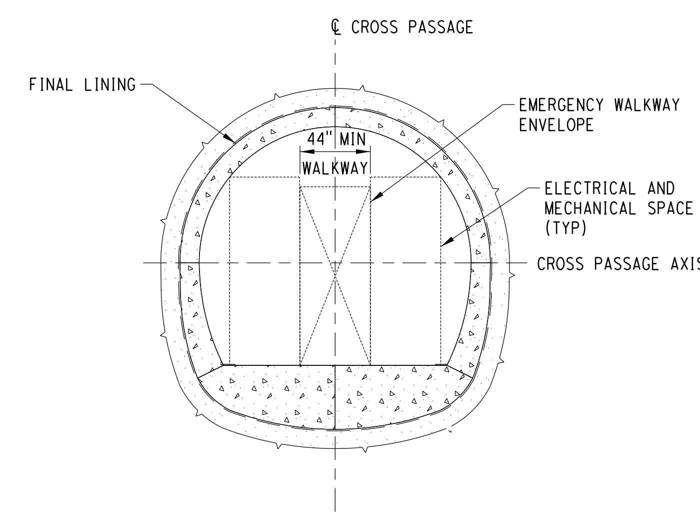
LONGITUDINAL SECTION

SCALE: 1"=10'-0"



A SS006 SECTION

SCALE: 1"=10'-0"

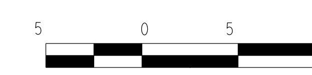


B SS006 SECTION

SCALE: 1"=10'-0"

NOTES:

1. TUNNEL SUPPORT SHOWN IS PRELIMINARY CONCEPT ONLY, ACTUAL REINFORCEMENT AND LINING DESIGN TBD.
2. CROSS PASSAGE SECTION TO BE HORSESHOE SHAPE FOR LINING SYSTEM SHOWN.
3. WATERPROOF MEMBRANE TYPE TO BE CONFIRMED. MEMBRANE TO BE EITHER HYDRO-CARBON RESISTENT (HCR) OR POLYVINYL CHLORIDE (PVC).
4. IN ORDER TO FACILITATE CROSS PASSAGE CONSTRUCTION GROUND IMPROVEMENT MAY BE REQUIRED.



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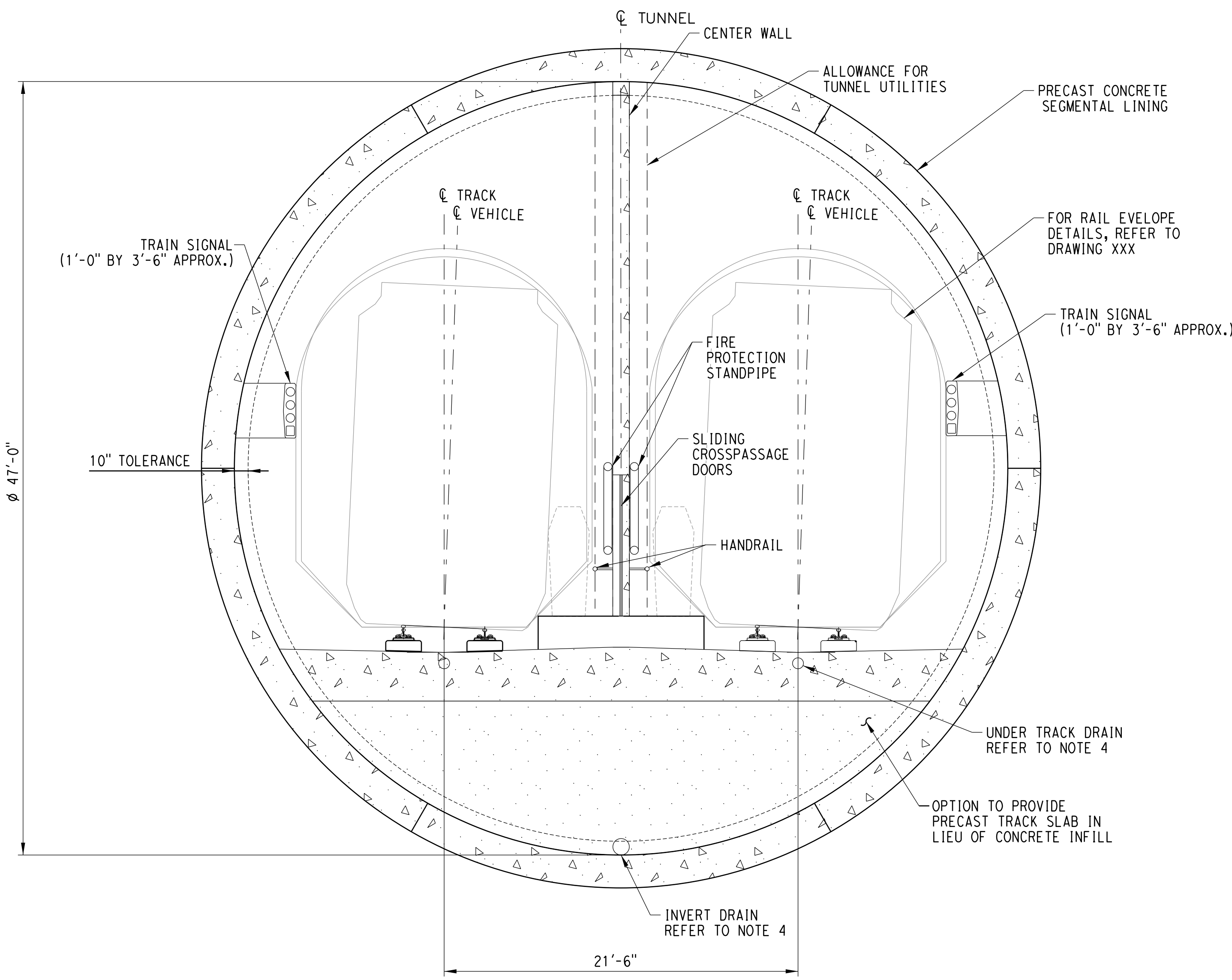
DEL MAR ALTERNATIVES ANALYSIS

CROSS PASSAGE TYPICAL SECTIONS

CONTRACT NO.
DRAWING NO. SS006
REVISION | SHEET NO.
SCALE AS NOTED

10/28/2022



SINGLE BORE TUNNEL

SCALE: 1"=10'-0"

NOTES:

1. FOR RAIL ENVELOPE DETAILS, REFER TO DRAWING SS001
2. UNDER TRACK DRAIN WILL PROVIDE DRAINAGE FOR TRACK BED. INVERT DRAIN WILL PROVIDE DRAINAGE FOR ANY WATER INGRESS AROUND INVERT CONCRETE FILL.

5 0 5 10

SCALE IN FEET

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
SINGLE BORE TUNNEL
TYPICAL SECTION

CONTRACT NO.
DRAWING NO. SS007
REVISION
SHEET NO.
SCALE AS NOTED

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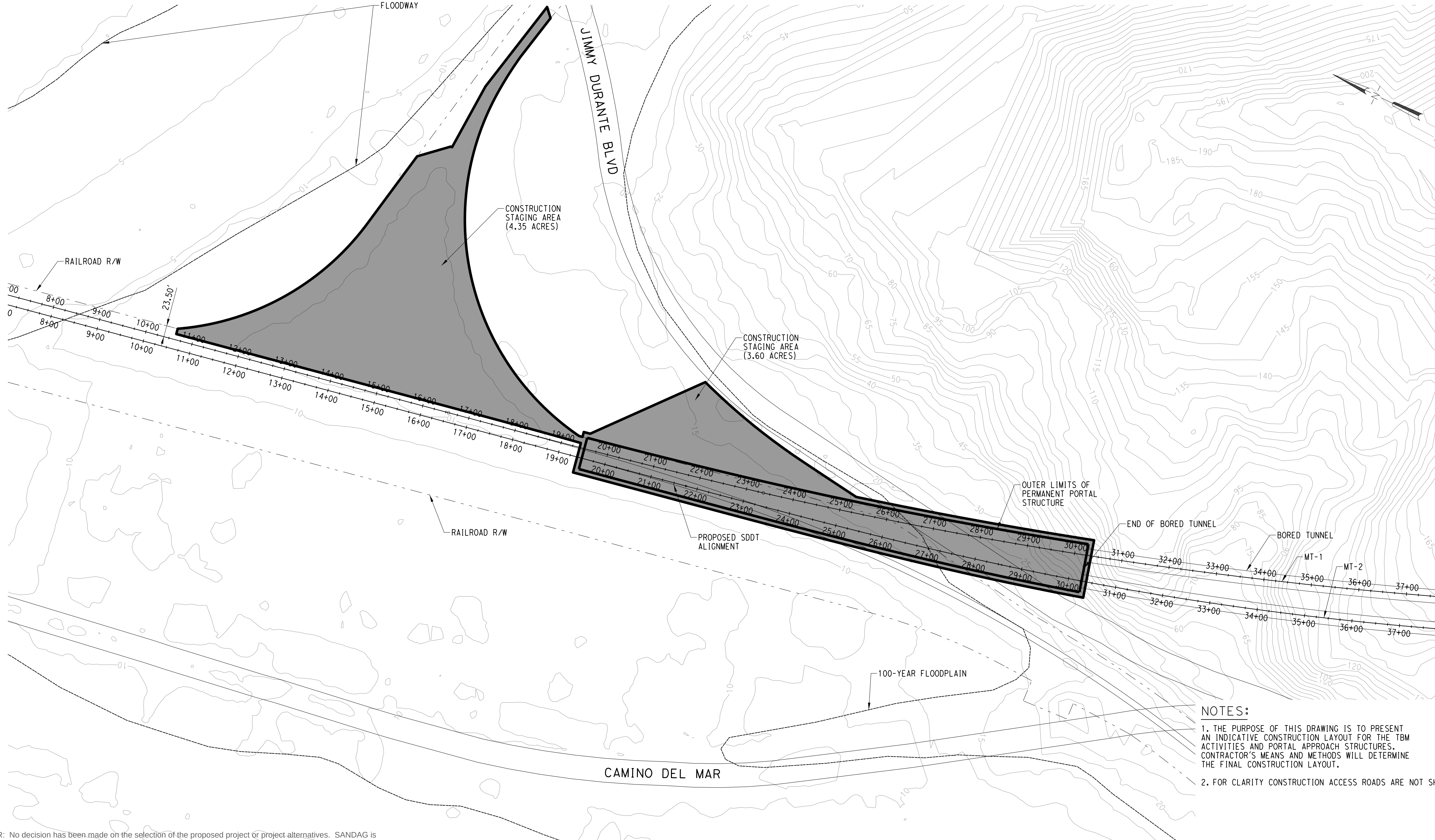

NOTES:

1. THE PURPOSE OF THIS DRAWING IS TO PRESENT AN INDICATIVE CONSTRUCTION LAYOUT FOR THE TBM ACTIVITIES AND PORTAL APPROACH STRUCTURES. CONTRACTOR'S MEANS AND METHODS WILL DETERMINE THE FINAL CONSTRUCTION LAYOUT.
2. FOR CLARITY CONSTRUCTION ACCESS ROADS ARE NOT SHOWN.

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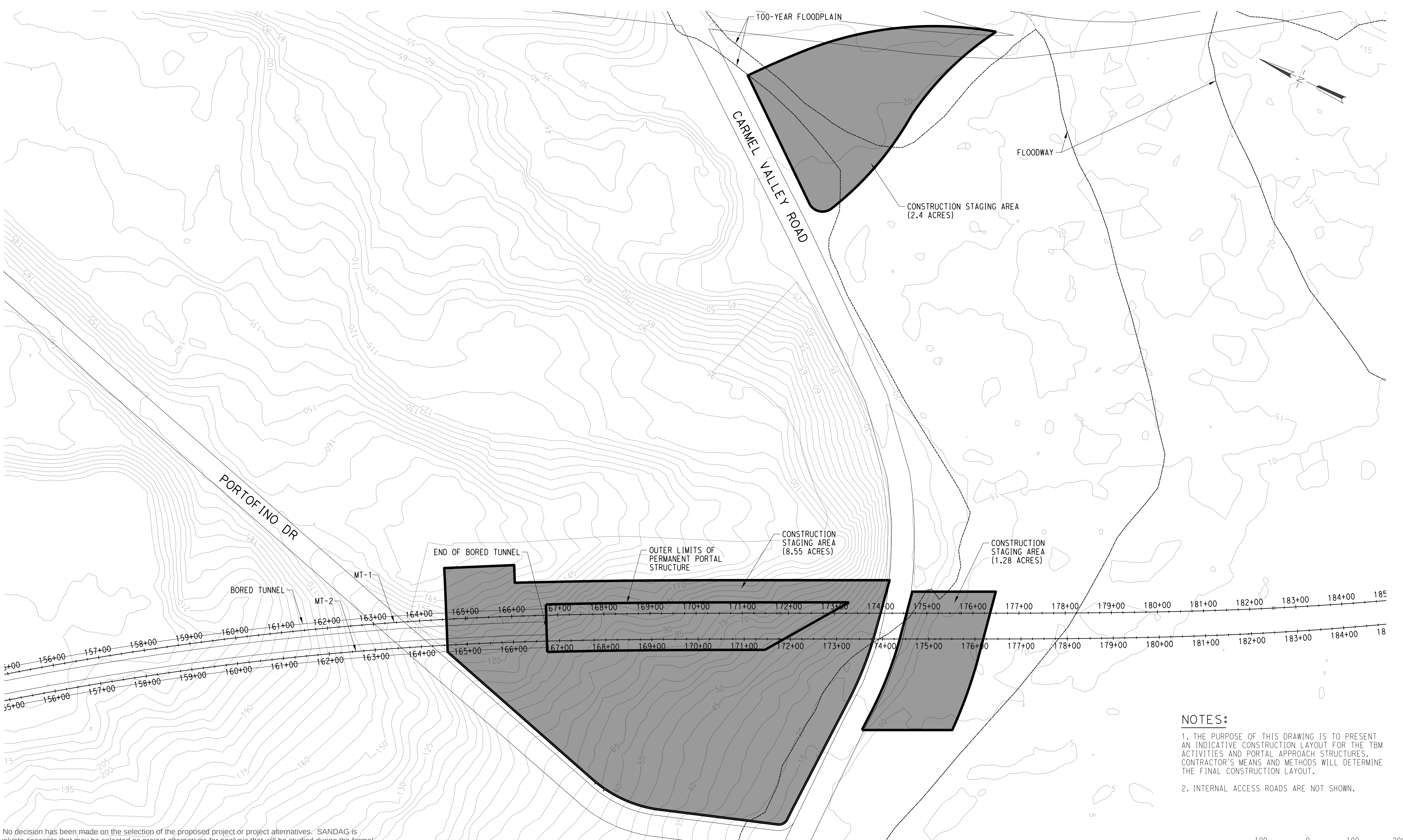
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE NORTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA101
REVISION
SHEET NO.
SCALE HORIZ: 1"=200'



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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE SOUTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA102
REVISION
SHEET NO.
SCALE HORIZ: 1"=200'

10/28/2022
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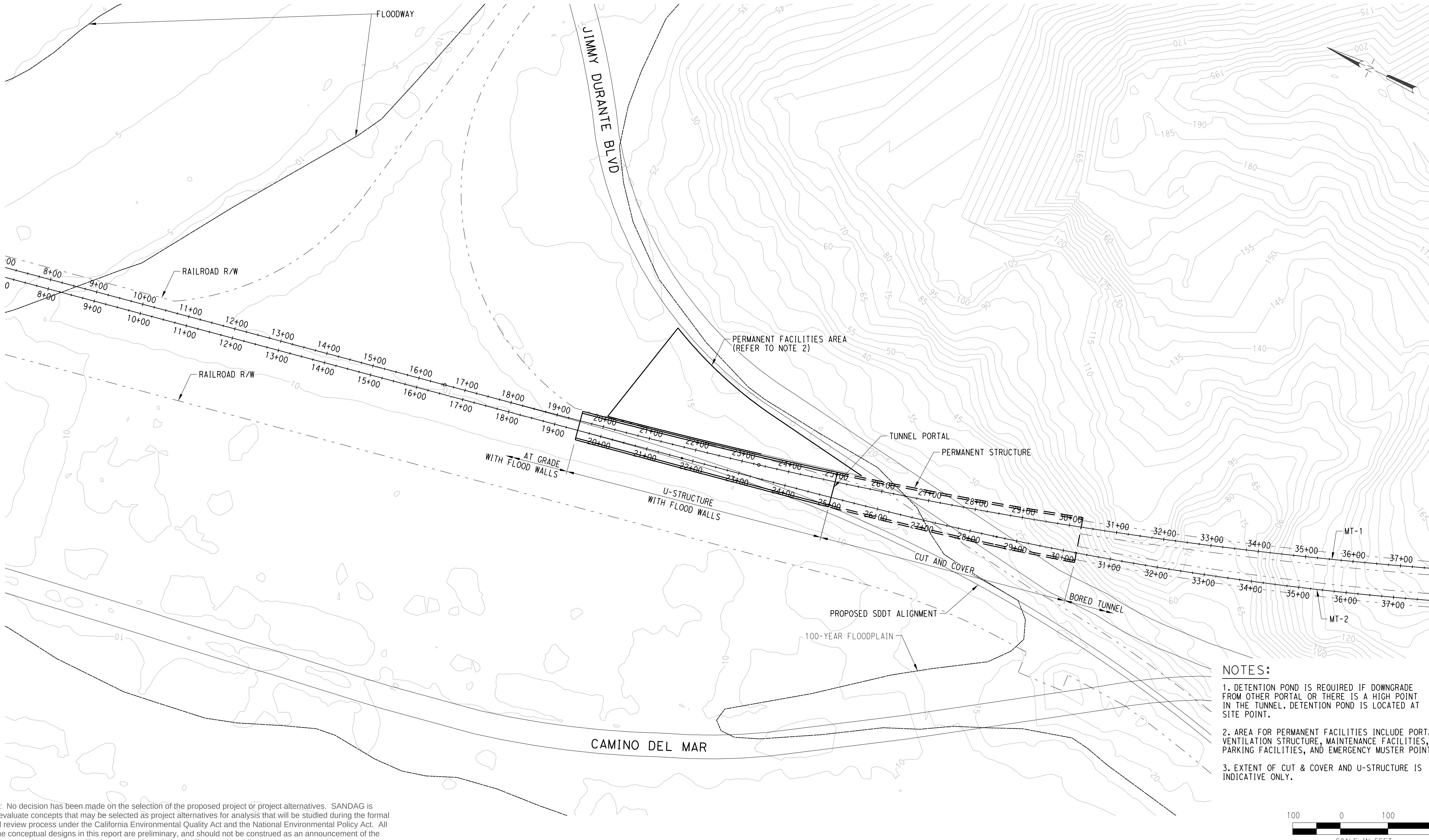

NOTES:

1. DETENTION POND IS REQUIRED IF DOWNGRADE FROM OTHER PORTAL OR THERE IS A HIGH POINT IN THE TUNNEL. DETENTION POND IS LOCATED AT SITE POINT.

2. AREA FOR PERMANENT FACILITIES INCLUDE PORTAL VENTILATION STRUCTURE, MAINTENANCE FACILITIES, PARKING FACILITIES, AND EMERGENCY MUSTER POINTS.

3. EXTENT OF CUT & COVER AND U-STRUCTURE IS INDICATIVE ONLY.

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE NORTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA103
REVISION | SHEET NO.
SCALE HORIZ: 1"=200'

10/28/2022
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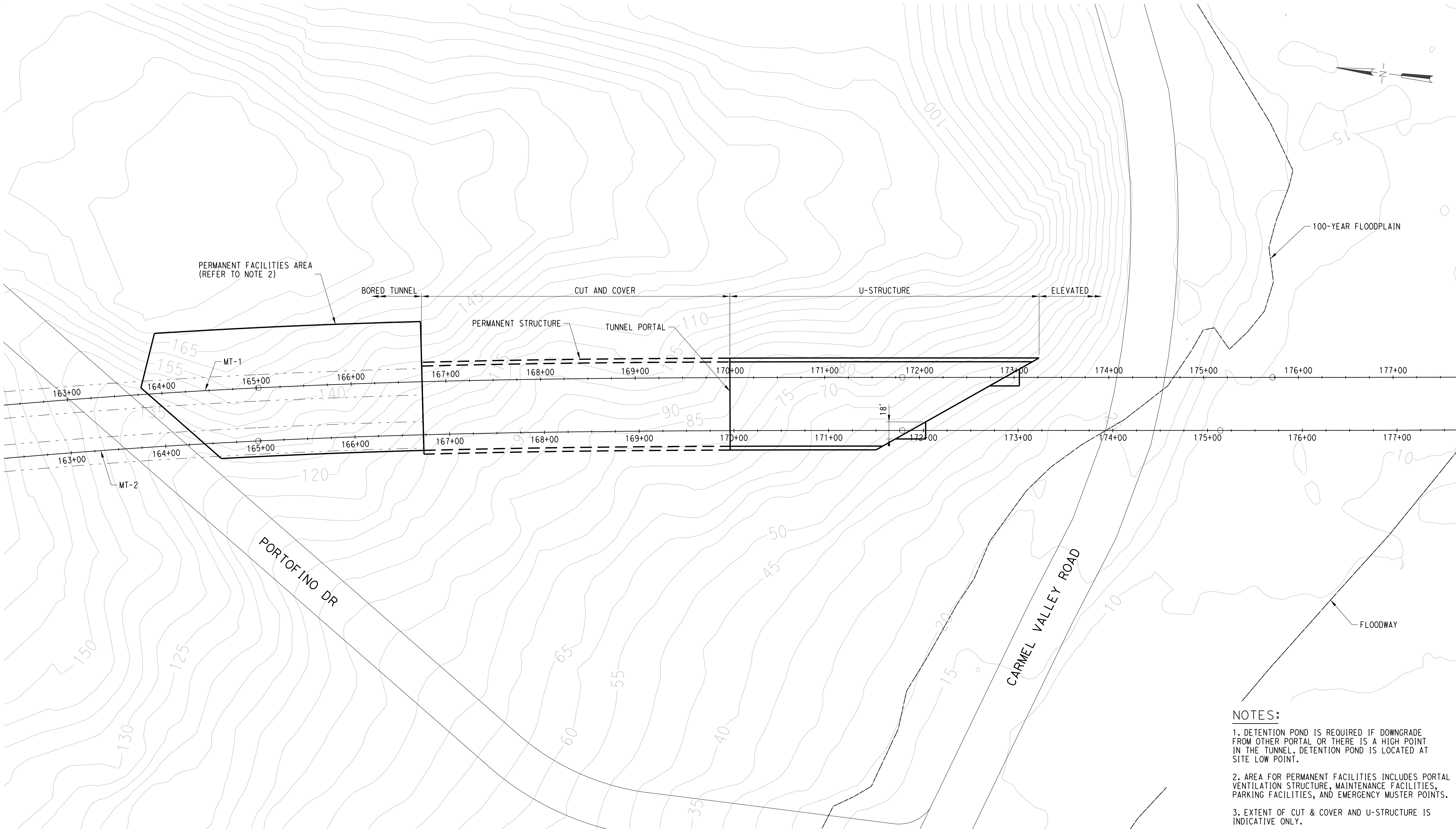

NOTES:

1. DETENTION POND IS REQUIRED IF DOWNGRADE FROM OTHER PORTAL OR THERE IS A HIGH POINT IN THE TUNNEL. DETENTION POND IS LOCATED AT SITE LOW POINT.

2. AREA FOR PERMANENT FACILITIES INCLUDES PORTAL VENTILATION STRUCTURE, MAINTENANCE FACILITIES, PARKING FACILITIES, AND EMERGENCY MUSTER POINTS.

3. EXTENT OF CUT & COVER AND U-STRUCTURE IS INDICATIVE ONLY.

50 0 50 100
SCALE IN FEET

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - TWIN BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA104
REVISION | SHEET NO.
SCALE HORIZ: 1"=100'

10/28/2022
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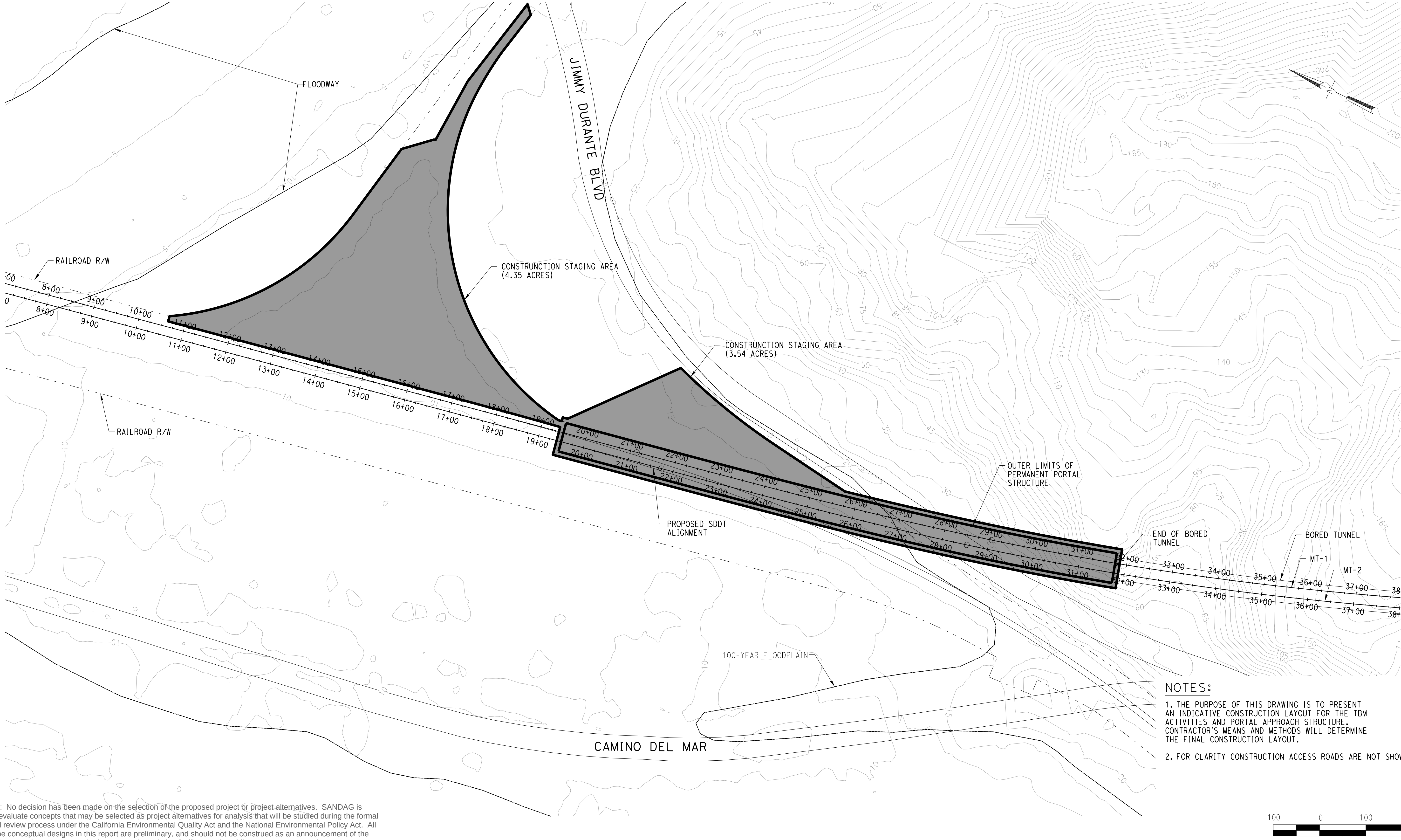

NOTES:

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DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street,
Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

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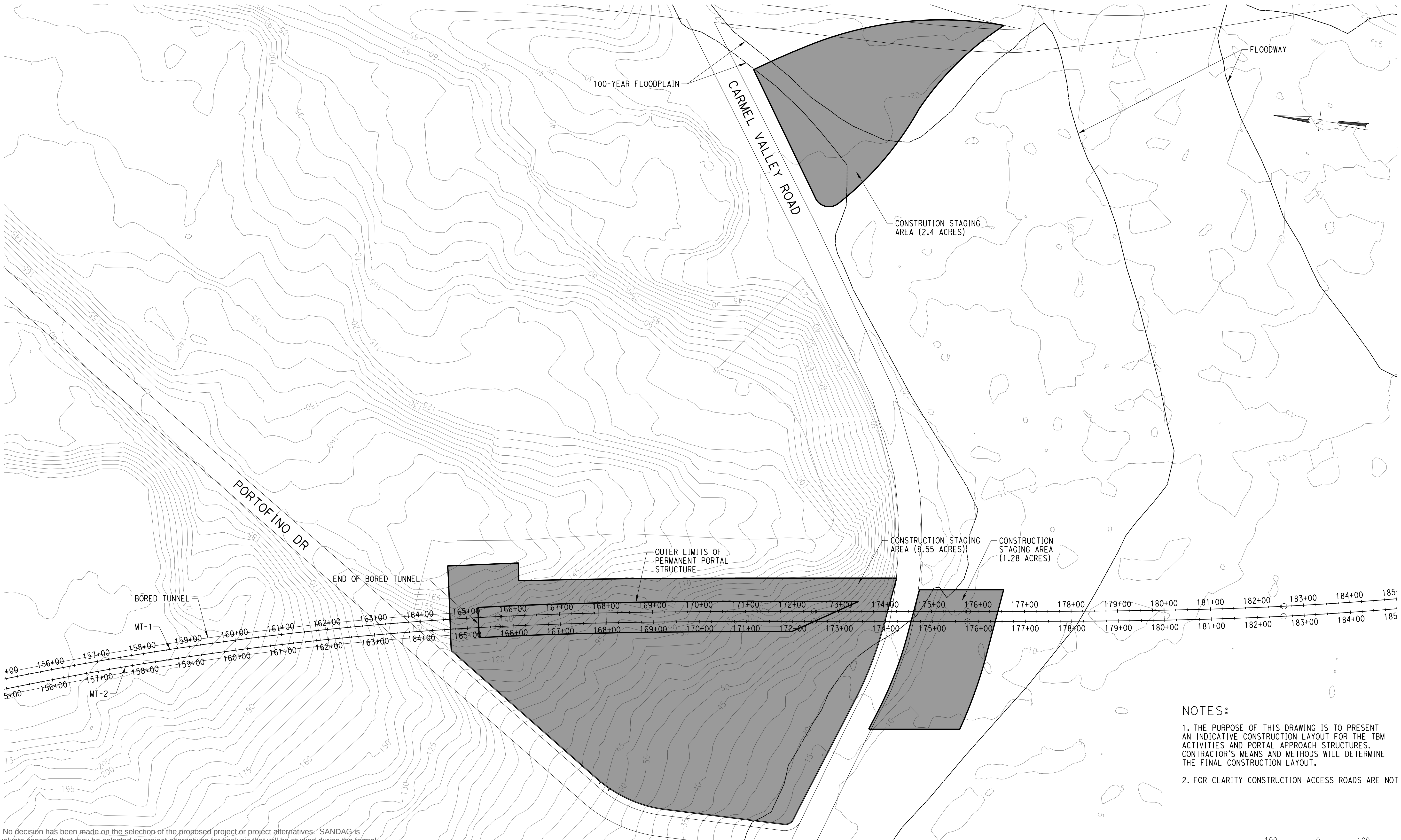
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON HS - SINGLE BORE NORTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA105
REVISION | SHEET NO.
SCALE HORIZ: 1"=200'



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CREST CANYON HS - SINGLE BORE SOUTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO.: SA106
REVISION | SHEET NO.
SCALE: HORIZ: 1"=200'

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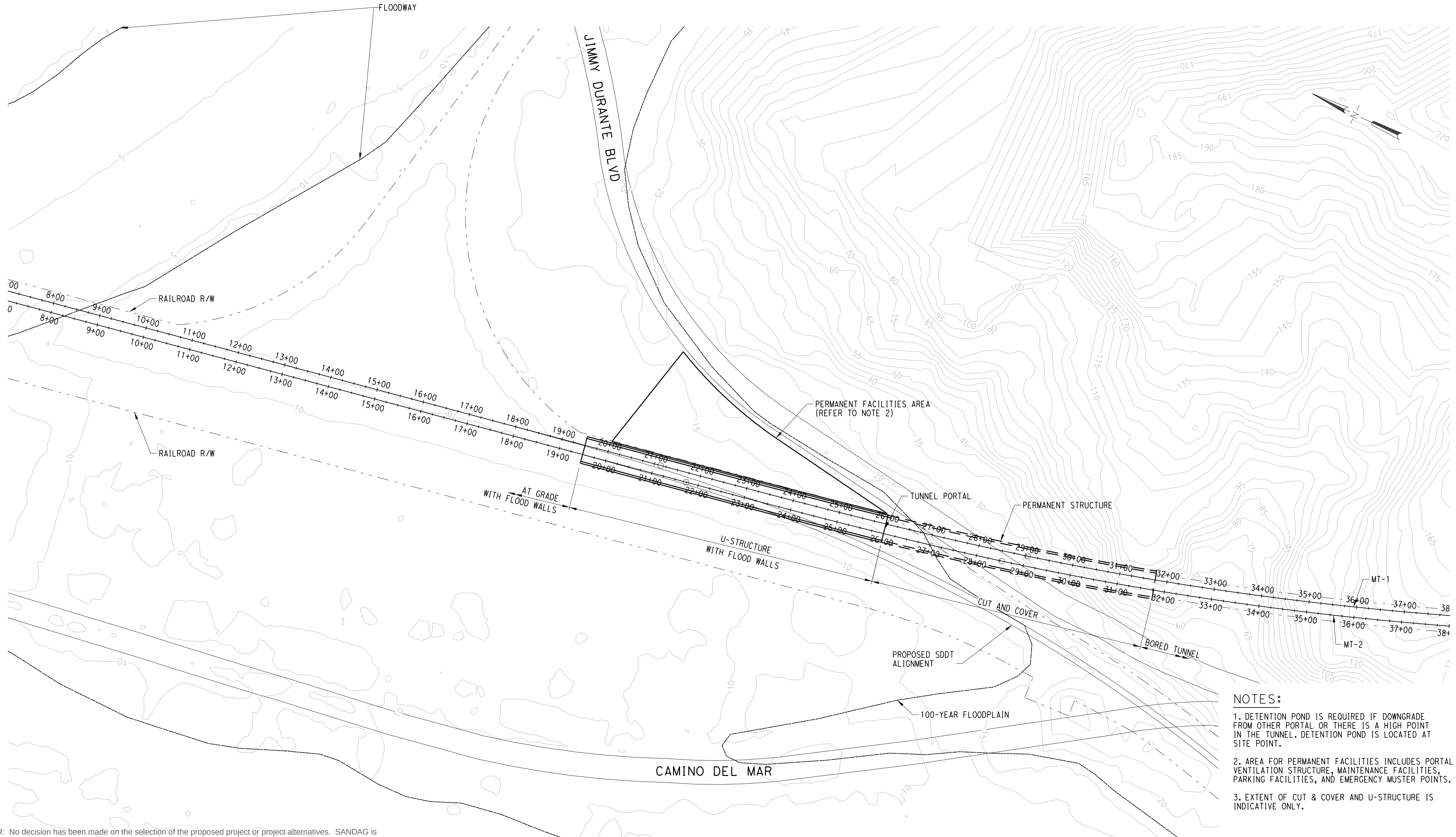

NOTES:

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CONTRACT NO.
DRAWING NO. SA107
REVISION | SHEET NO.
SCALE: HORIZ: 1"=200'

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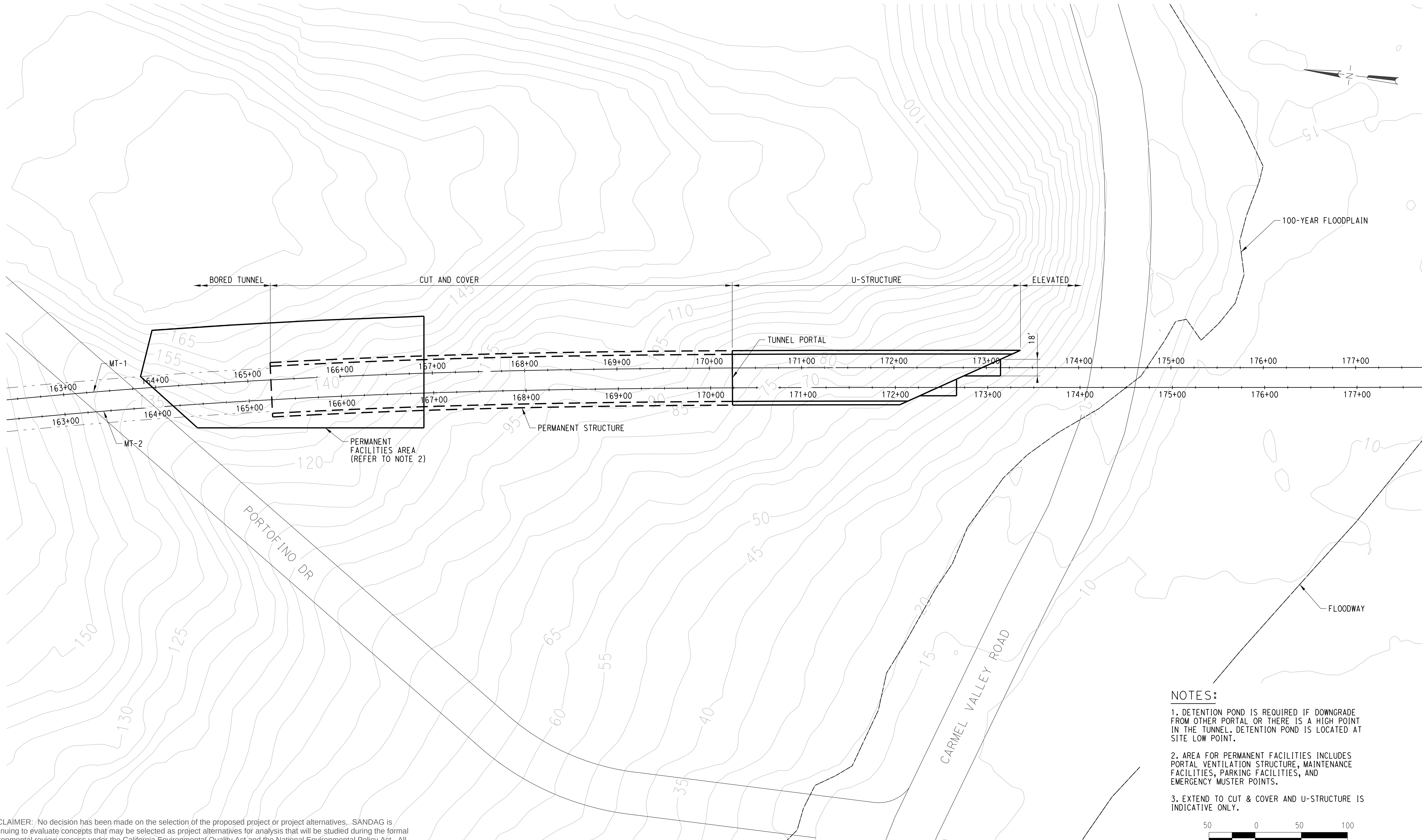

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SCALE IN FEET

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CREST CANYON HS - SINGLE BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA108
REVISION
SHEET NO.
SCALE HORIZ: 1"=100'

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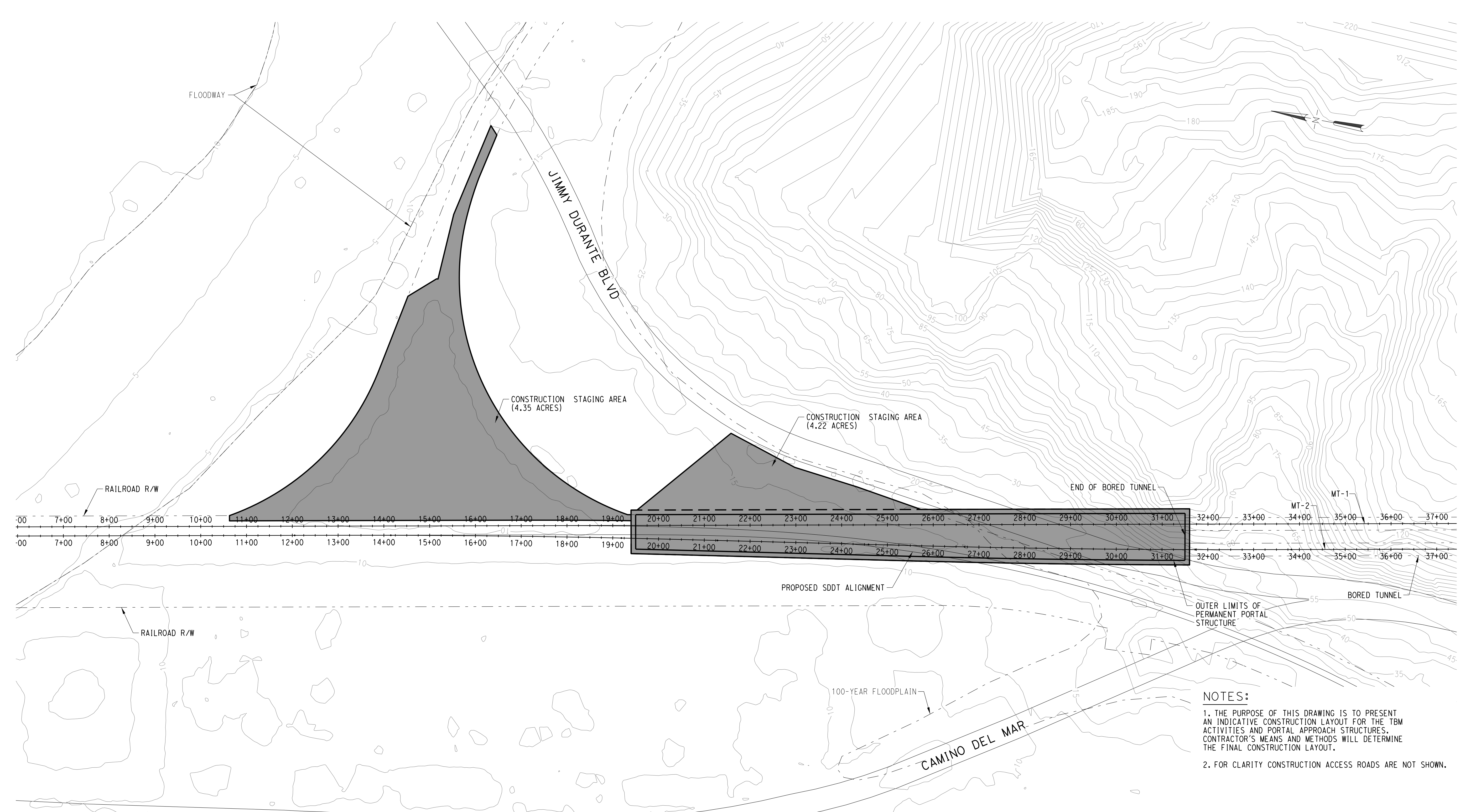

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DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR - TWIN BORE NORTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA201
REVISION
SHEET NO.
SCALE HORIZ: 1"=200'

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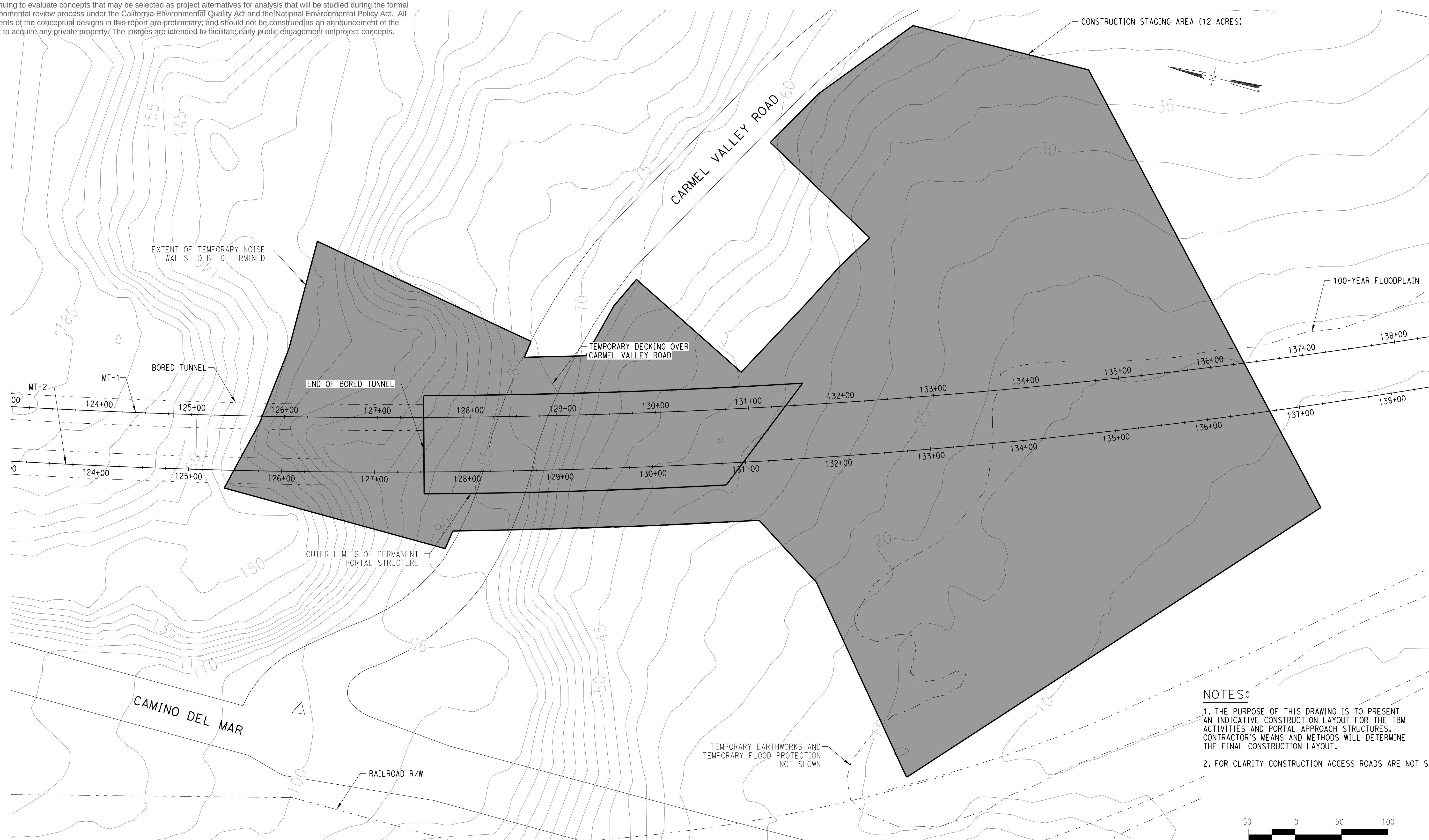

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CAMINO DEL MAR - TWIN BORE SOUTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA202
REVISION
SHEET NO.
SCALE HORIZ: 1"=100'

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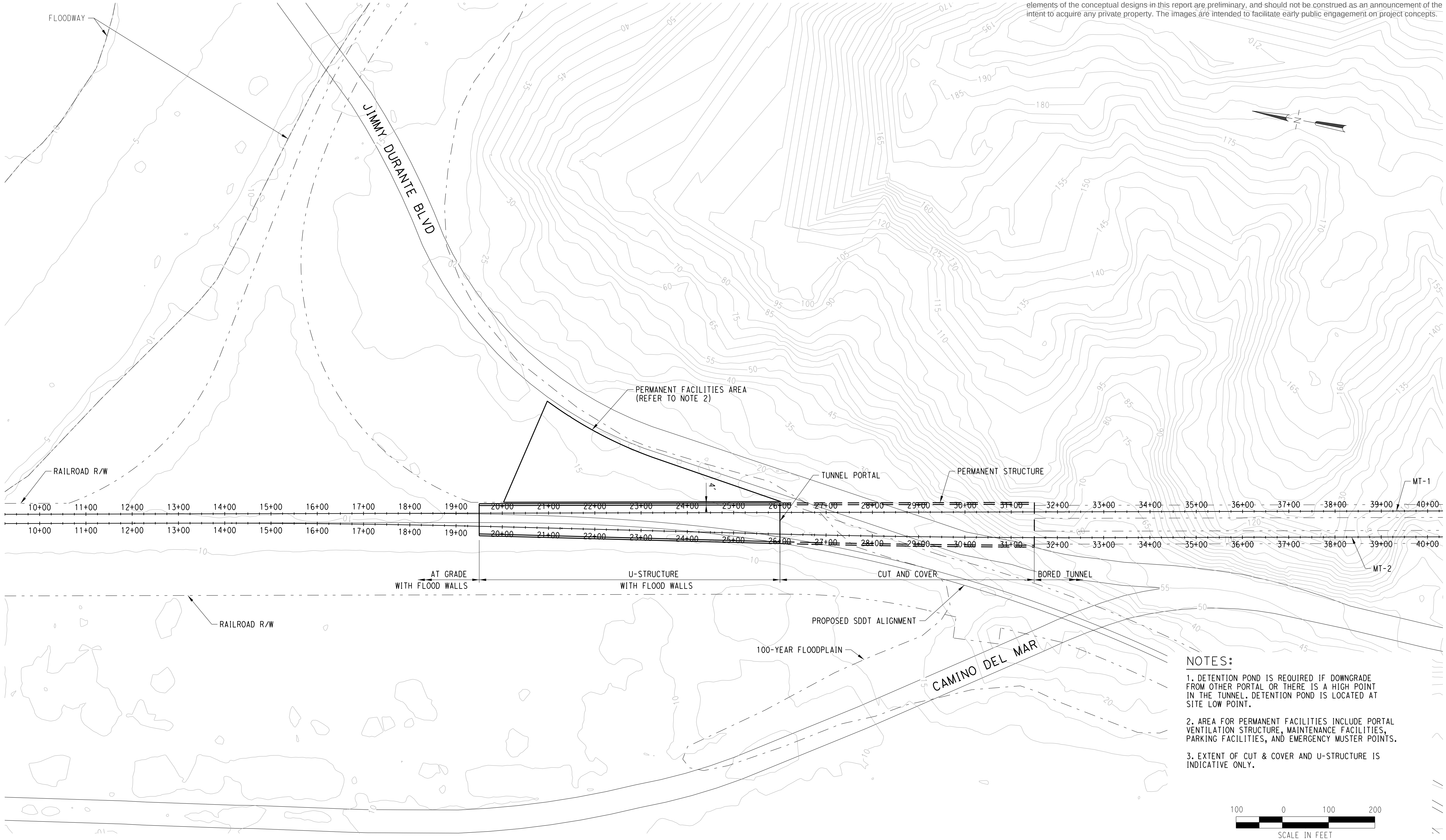

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR TWIN BORE NORTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA203
REVISION
SHEET NO.
SCALE HORIZ: 1"=200'

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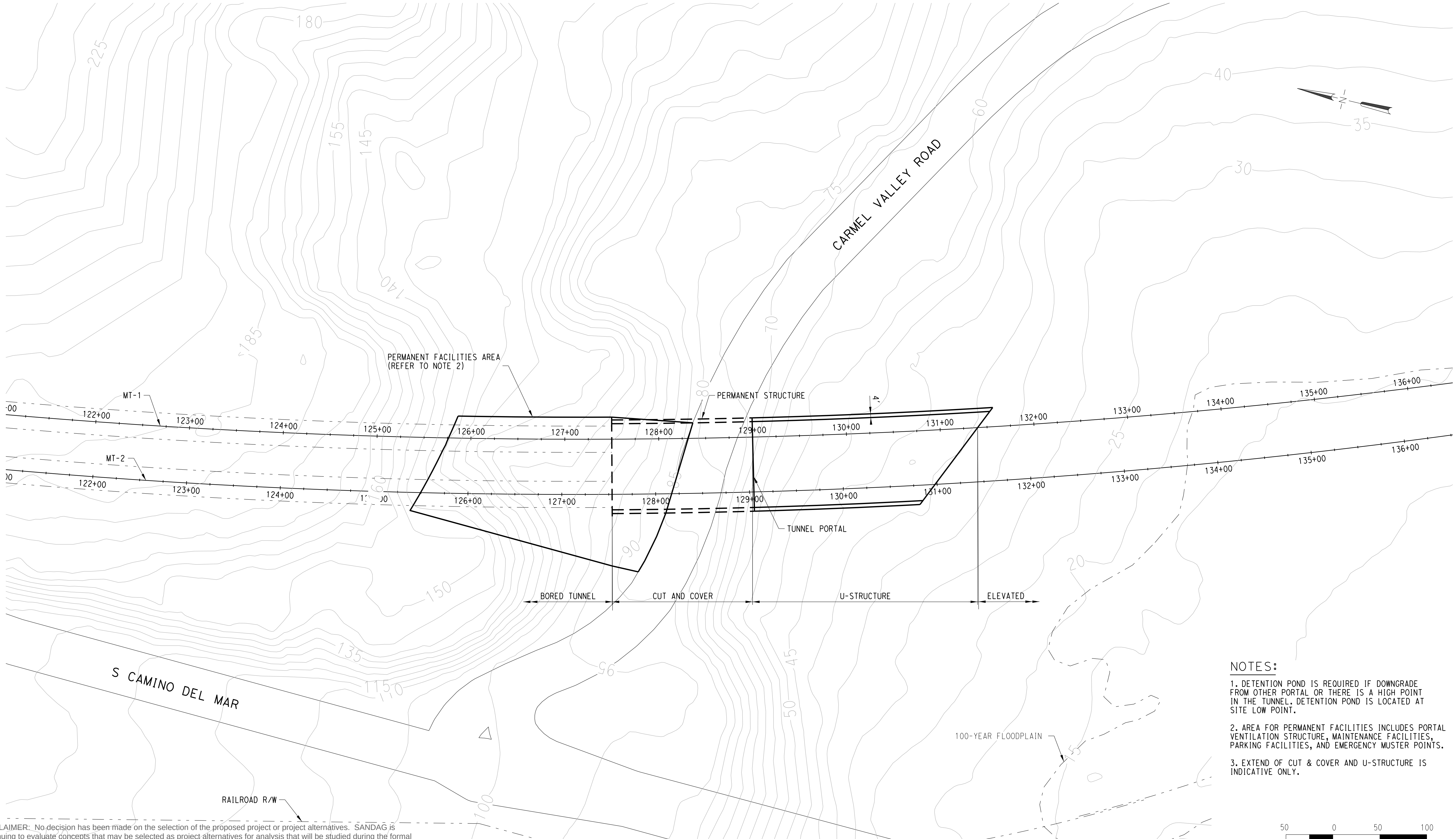

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR TWIN BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA204
REVISION
SHEET NO.
SCALE HORIZ: 1"=100'

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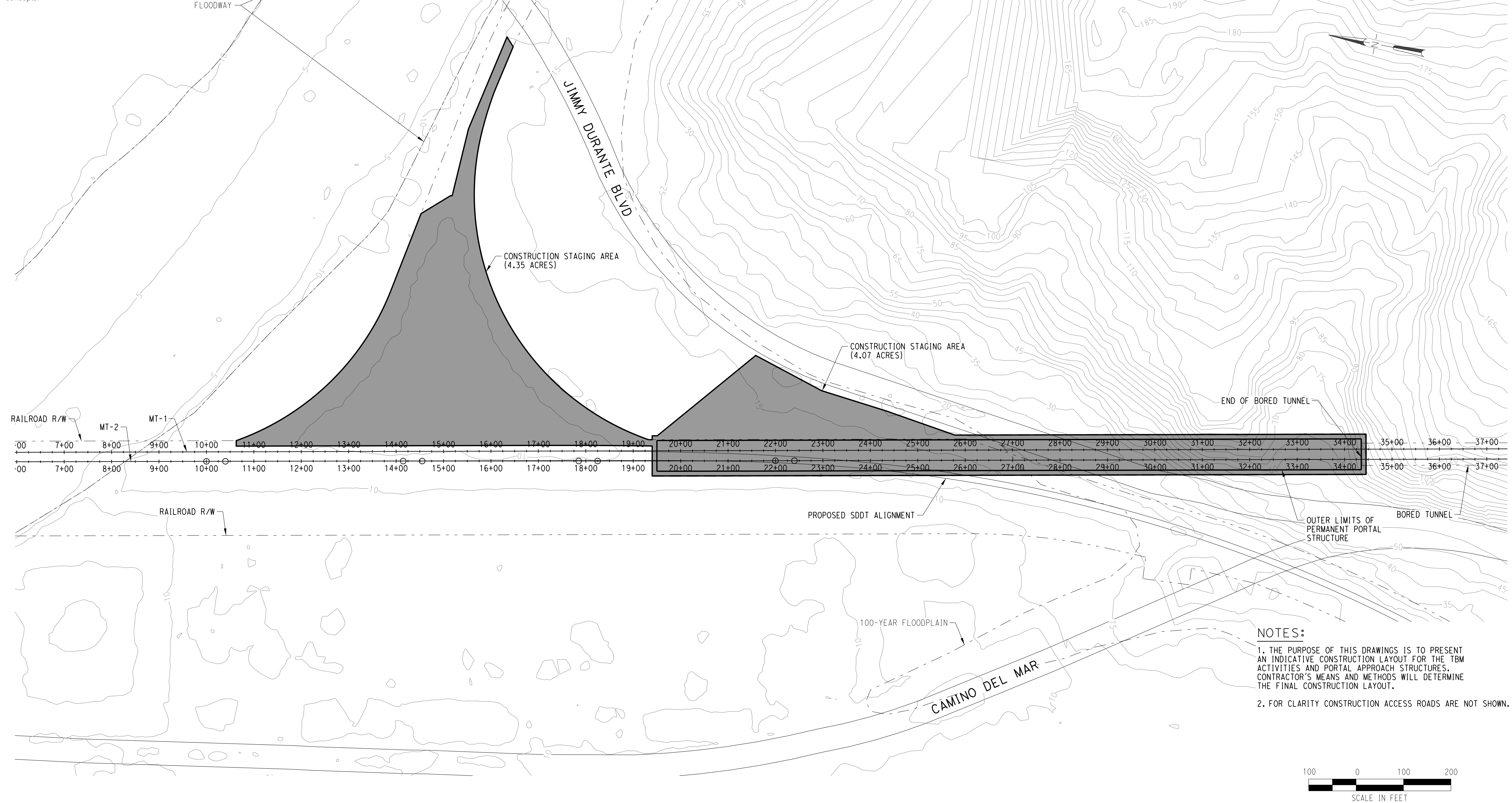

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CAMINO DEL MAR - SINGLE BORE NORTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA205
REVISION | SHEET NO.
SCALE: HORIZ: 1"=200'

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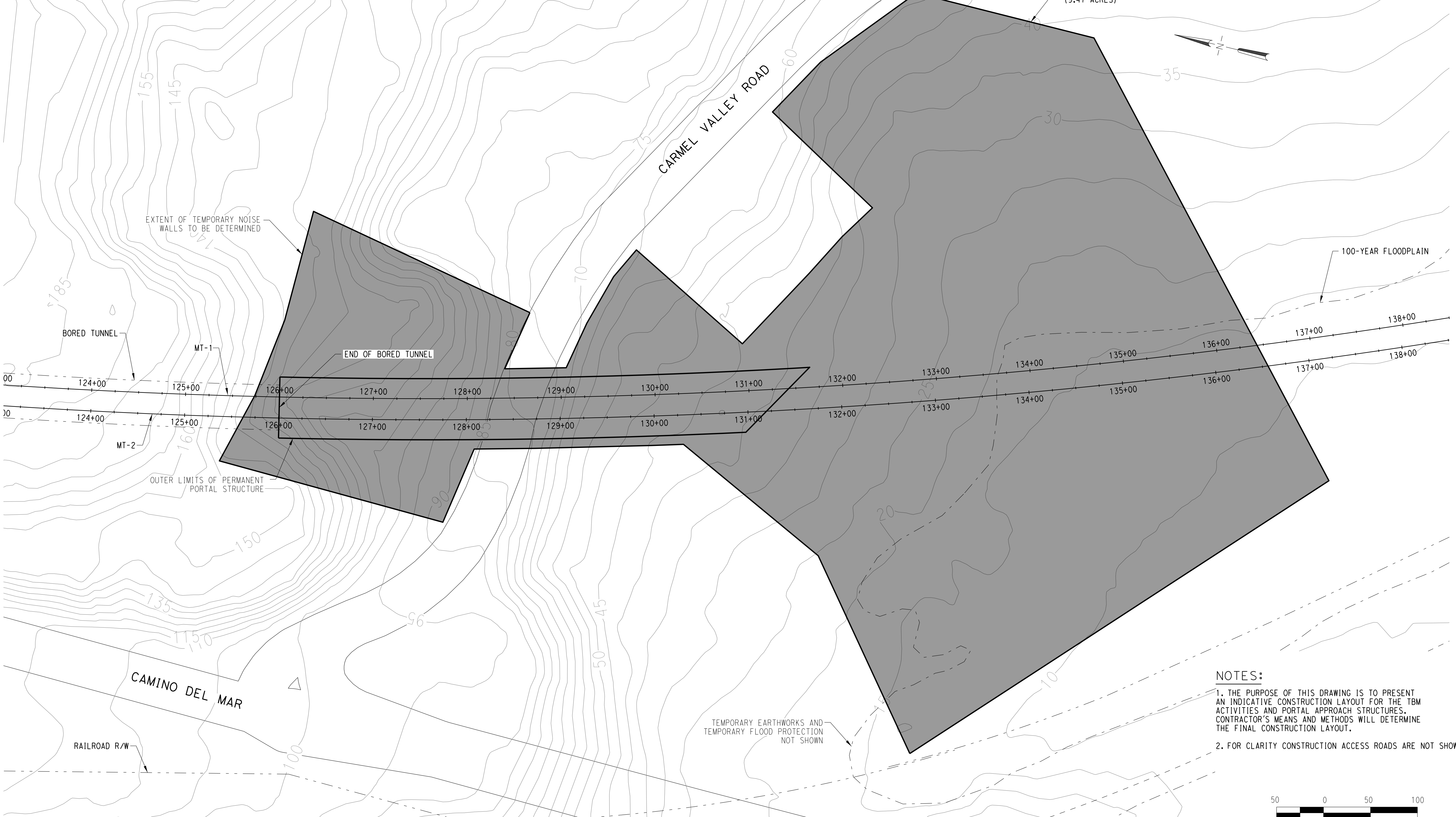

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CAMINO DEL MAR - SINGLE BORE SOUTH PORTAL CONSTRUCTION STAGING AREA

CONTRACT NO.
DRAWING NO. SA206
REVISION
SHEET NO.
SCALE HORIZ: 1"=100'

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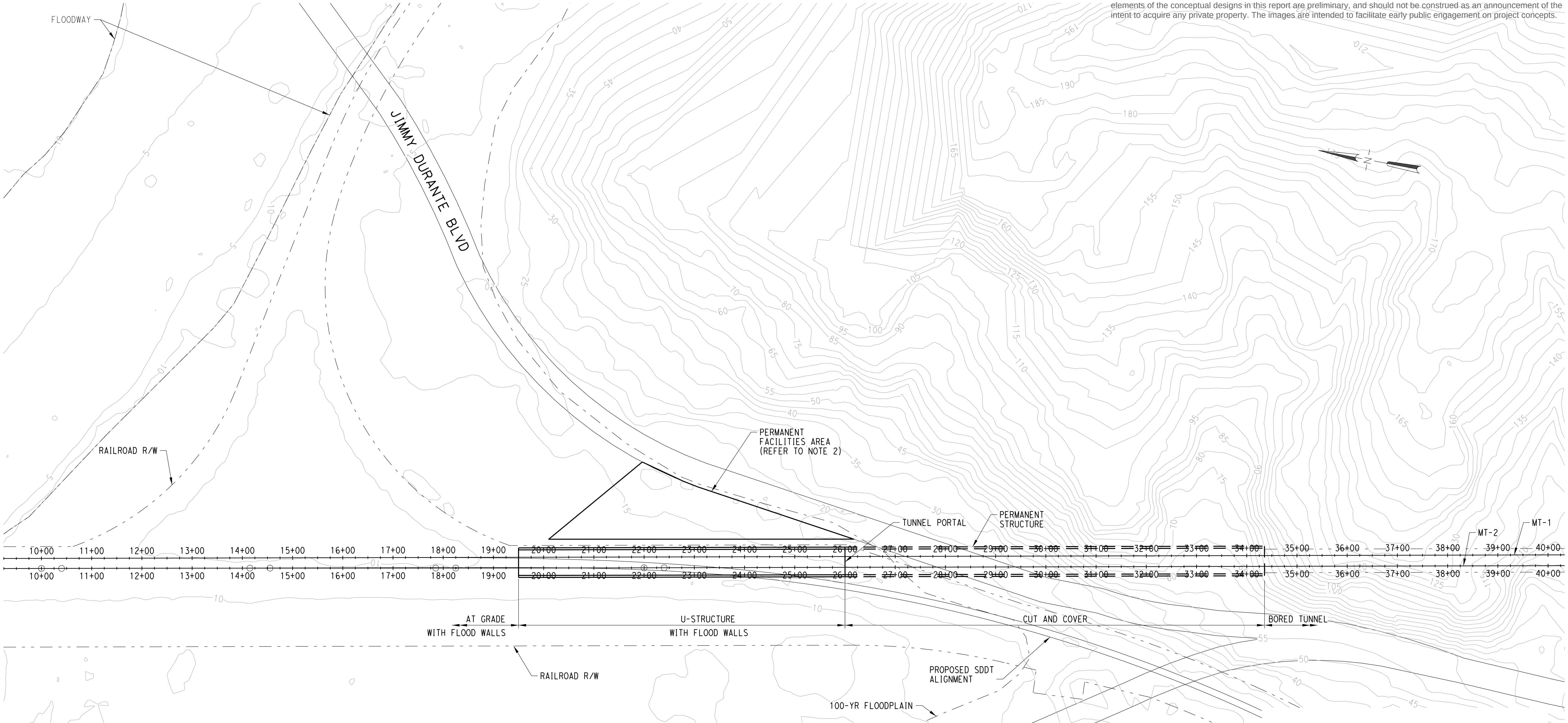

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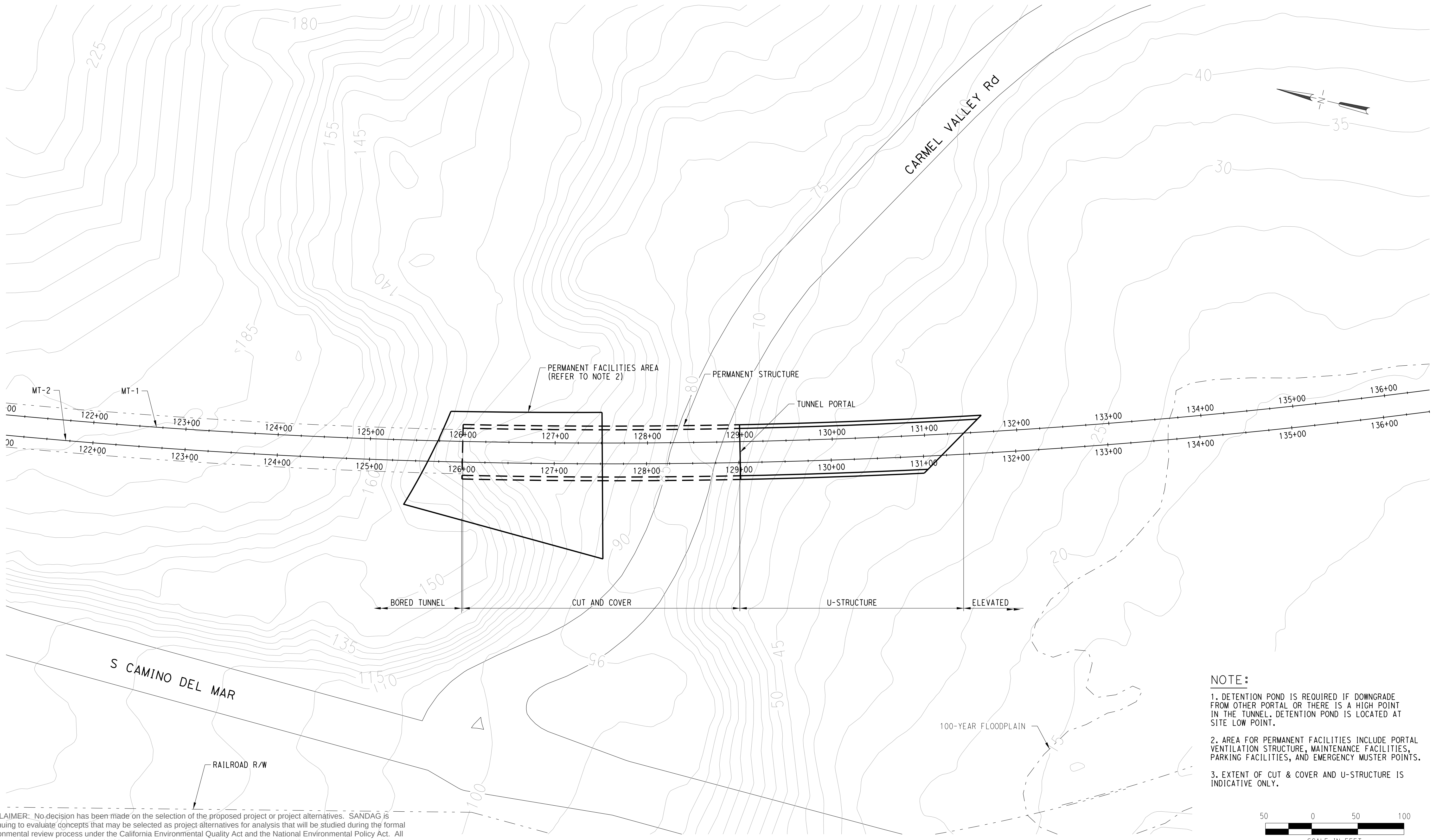
CAMINO DEL MAR SINGLE BORE NORTH PORTAL STRUCTURES AND PERMANENT FACILITIES

CONTRACT NO.
DRAWING NO. SA207
REVISION | SHEET NO.
SCALE HORIZ: 1"=200'

10/28/2022



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DESIGNED BY S. LO GRASSO
DRAWN BY A. RODRIGUEZ
CHECKED BY F. NOURBAKHSH
APPROVED BY M. RAMSEY
DATE OCTOBER 2022

MOTT MACDONALD
750 B Street,
Suite 2880
San Diego, CA 92101
Tel: 619-881-0400

SANDAG
San Diego's Regional Planning Agency

APPROVED: ____________ DATE: ________

Exhibit

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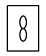
SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR SINGLE BORE SOUTH PORTAL STRUCTURES AND PERMANENT FACILITIES

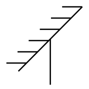
CONTRACT NO.
DRAWING NO. SA208
REVISION
SHEET NO.
SCALE HORIZ: 1"=100'

LEGEND

MOTORIZED DAMPER

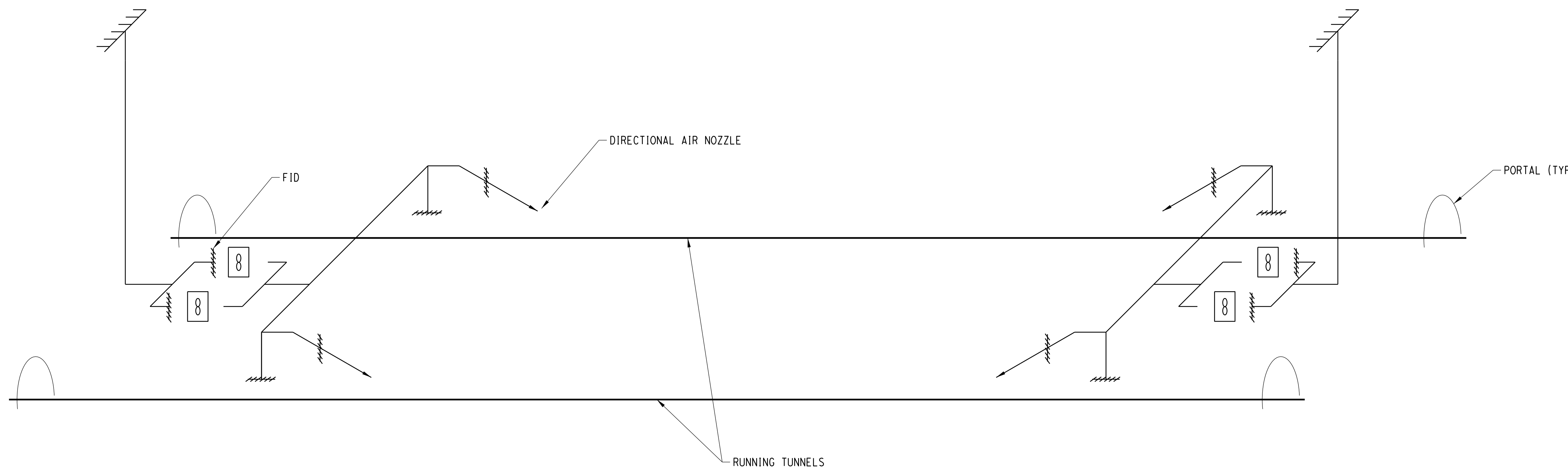

SOUND ATTENUATOR


TUNNEL VENTILATION FAN


OUTSIDE AIR LOUVER

ABBREVIATIONS

OAL	OUTSIDE AIR LOUVER
SA	SOUND ATTENUATOR
FID	FAN ISOLATION DAMPER
TVF	TUNNEL VENTILATION FAN



TWIN BORE AIRFLOW DIAGRAM
NTS

NOTE:
1. CONCEPT CAN BE ADAPTED TO SINGLE BORE ARRANGEMENTS.

DISCLAIMER: No decision has been made on the selection of the proposed project or project alternatives. SANDAG is continuing to evaluate concepts that may be selected as project alternatives for analysis that will be studied during the formal environmental review process under the California Environmental Quality Act and the National Environmental Policy Act. All elements of the conceptual designs in this report are preliminary, and should not be construed as an announcement of the intent to acquire any private property. The images are intended to facilitate early public engagement on project concepts.

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DEL MAR ALTERNATIVES ANALYSIS
TWIN BORE TUNNELS EMERGENCY VENTILATION SCHEMATIC

CONTRACT NO.
DRAWING NO. SU001
REVISION
SHEET NO.
SCALE AS NOTED

10/28/2022
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C:\Program Files (x86)\Common Files\InterPlot\IPLOT\bin\iplotdrvm.plt

Appendix N. Railroad Signal Schematics

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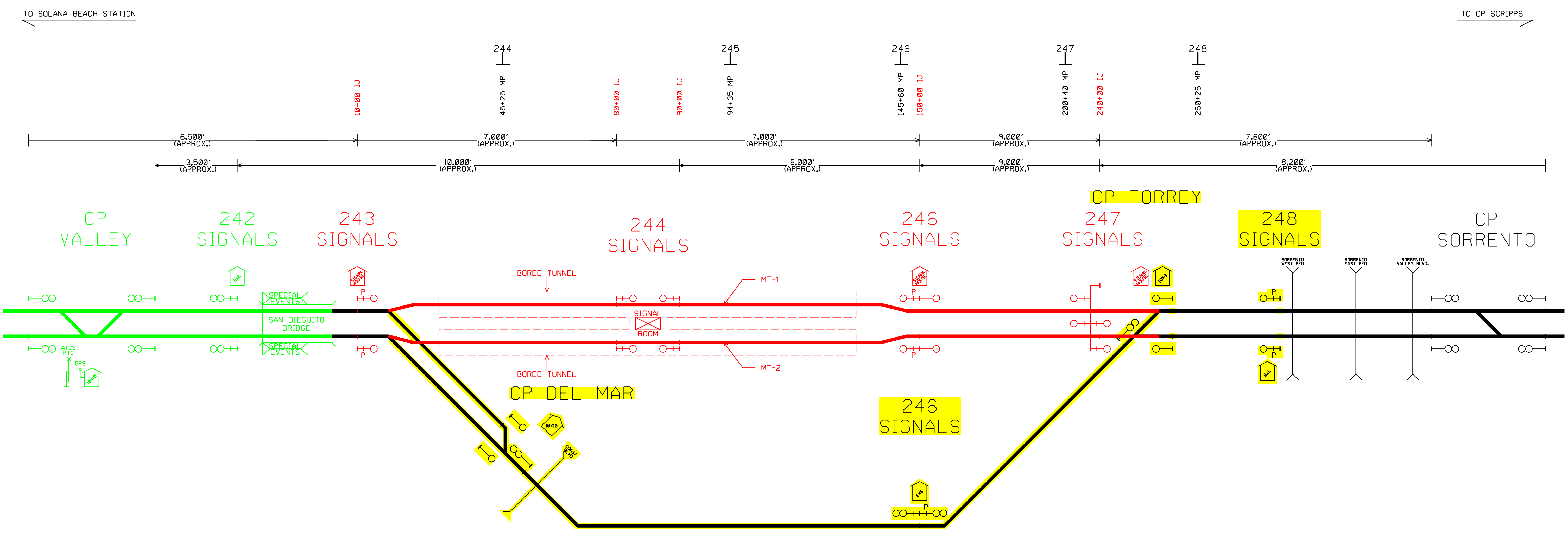
CAMINO DEL MAR TUNNEL ALTERNATIVE



PROJECT GENERAL NOTES:

1. NEW SIGNAL INSTRUMENT ENCLOSURES SIZE TO BE COORDINATED IN FUTURE ENGINEERING PHASES.
2. NEW 243 EASTBOUND SIGNALS AND NEW 246 WESTBOUND SIGNALS WILL BE INTERCONNECTED TO FIRE/LIFE/SAFETY SYSTEMS TO ALERT TRAIN CREW OF ANY POTENTIAL HAZARDOUS CONDITION PRIOR TO ENTERING THE TUNNEL(S).
3. EXISTING CP TORREY SIGNAL HOUSE WILL BE RETROFITTED TO OPERATE NEW 247 BI-DIRECTIONAL CANTILEVER SIGNALS.
4. CIRCUIT MODIFICATIONS WILL BE REQUIRED UPON REMOVAL OF 248 SIGNALS TO MAINTAIN IN-SERVICE GRADE CROSSING WARNING SYSTEMS AT SORRENTO VALLEY WEST PED CROSSING, SORRENTO VALLEY EAST PED XING, AND SORRENTO VALLEY BLVD.
5. A NEW FIBER DUCT BANK WILL BE REQUIRED TO OPERATE POSITIVE TRAIN CONTROL COMMUNICATIONS, RAIL SYSTEMS NETWORK, AND INFORMATION TECHNOLOGY AND FIRE/LIFE/SAFETY COMMUNICATIONS.
6. POSTIVE TRAIN CONTROL SYSTEM WILL USE DEAD RECKONING (AXLE TURNS) TO DETERMINE TRAIN LOCATION WITHIN THE TUNNEL(S).

SAN DIEGUITO DOUBLE TRACK PROJECT
GREEN = IN BROWN = OUT

CAMINO DEL MAR ALTERNATIVE
RED = IN YELLOW = OUT

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DESIGNED BY J. GONZALEZ
DRAWN BY J. GONZALEZ
CHECKED BY R. MARTINEZ
APPROVED BY R. KLOVSKY
DATE OCTOBER 25, 2022

PRE PACIFIC RAILWAY ENTERPRISES, INC. 3560 University Avenue, Suite F Riverside, California 92501

SANDAG San Diego's Regional Planning Agency

APPROVED: ________ DATE: ________

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CAMINO DEL MAR
RAILWAY SYSTEMS OVERVIEW
SH. 1 OF 2

CONTRACT NO.
DRAWING NO.
REVISION
SHEET NO.
SCALE
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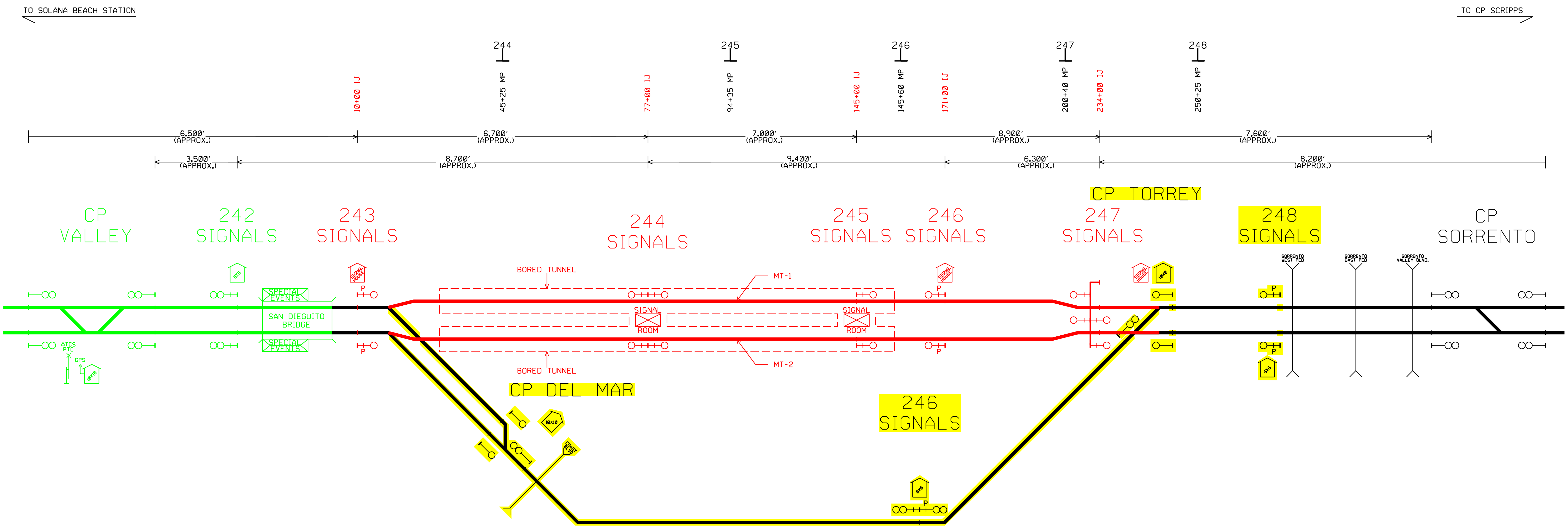
CREST CANYON TUNNEL ALTERNATIVE



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SAN DIEGUITO DOUBLE TRACK PROJECT
GREEN = IN BROWN = OUT

CREST CANYON ALTERNATIVE
RED = IN YELLOW = OUT

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Riverside, California 92501

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SD-LOSSAN
DEL MAR ALTERNATIVES ANALYSIS
CREST CANYON
RAILWAY SYSTEMS OVERVIEW
SH. 2 OF 2

CONTRACT NO.
DRAWING NO.
REVISION
SHEET NO.
SCALE
NOT TO SCALE

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